QTS Data Center – DC2 and DC3

Geotechnical Engineering Report

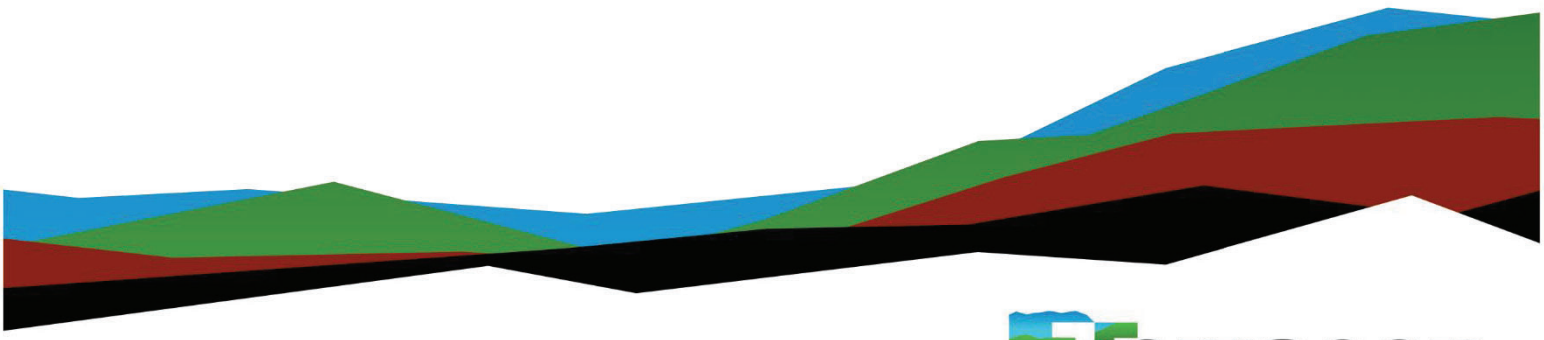
1120 N Gun Club Road

Aurora, Colorado

October 4, 2024 | Terracon Project No. 25245150

Prepared for:

Quality Technology Services, LLC
6431 Longhorn Drive
Irving, Texas 75063



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Re: Geotechnical Engineering Report
QTS Data Center – DC2 and DC3
1120 N Gun Club Road
Aurora, Colorado
Terracon Project No. 25245150

Mr. Hayes:

We have completed the scope of Geotechnical Engineering services for the above referenced project in general accordance with our proposal dated July 3, 2024. This report presents the findings of the subsurface exploration and provides geotechnical recommendations concerning earthwork and the design and construction of foundations, floor slabs, and pavements for the proposed project.

We appreciate the opportunity to be of service to you on this project. If you have any questions concerning this report or if we may be of further service, please contact us.

Sincerely,

Terracon



Trevor Meinholz, E.I.T.
Staff Engineer

Scott B. Myers, P.E.
Regional Senior Consultant

Table of Contents

Report Summary i
Introduction 1
Project Description 1
Site Conditions 3
Geotechnical Characterization 4
Subsurface Profile 4
Field Electrical Resistivity Test Results 5
Groundwater Conditions 5
Corrosivity 7
Thermal Resistivity Tests 9
Geotechnical Overview 9
Existing Fill Materials 9
Expansive Soils and Bedrock 10
Shallow Bedrock 10
Difficult Excavations 10
Earthwork 10
Site Preparation 11
Settlement of Deep Engineered Fill Zones 11
Material Types 12
Compaction Requirements 14
Excavation 15
Grading and Drainage 16
Earthwork Construction Considerations 17
Construction Observation and Testing 17
Utility Recommendations 18
Thrust Block Design Recommendations 19
Foundation Recommendations – Data Center Buildings 19
Foundation Recommendations – Ancillary Equipment 22
Drilled Pier Recommendations 23
Spread Footing Foundation Recommendations 23
Seismic Considerations 24
Interior Floors 25
Structurally Supported Floor Recommendations 25
Floor Slabs 25
Exterior Flatwork 27
Lateral Earth Pressures 28
Design Parameters 28
Pavements 30
Design Traffic 30
Subgrade Soils 31
Recommended Minimum Pavement Sections and Materials 31

Drainage Adjacent to Pavements 32
Pavement Maintenance 32
Pavement Construction Considerations 33
General Comments 33

Figures

GeoModel

Attachments

Exploration and Testing Procedures
Site Location and Exploration Plans
Exploration and Laboratory Results
Field Electrical Resistivity Test Results
Previous Geotechnical Data (Terracon Project No. 25215031)
Supporting Information

Note: This report was originally delivered in a web-based format. **Blue Bold** text in the report indicates a referenced section heading. The PDF version also includes hyperlinks which direct the reader to that section and clicking on the Terracon logo will bring you back to this page. For more interactive features, please view your project online at **client.terracon.com**.

Refer to each individual Attachment for a listing of contents.

Report Summary

Topic [1]	Overview Statement [2]
Project Description	We understand the proposed project consists of the design and construction of two new 244,876 square foot data center buildings (DC2 and DC3) with associated service yards and parking/driveway areas. We anticipate the proposed data center buildings will have two interior levels with a roof top mechanical platform, no basement areas, and have a steel frame with either concrete tilt-panel or precast concrete perimeter walls. We also understand that each data center will have an exterior equipment yard.
Geotechnical Characterization	Subsurface conditions encountered in the borings generally consisted of approximately 3 inches of asphalt or no surficial materials underlain by native soils consisting of clay with varying amounts of sand and gravel to depths of about 0 to 17 feet below the ground surface (bgs). Existing fill was encountered in Boring Nos. 1, 5, 6, 10, 11, 12, 13, 14, 15, 17, 18, 19, 20, 21, and P1 to varying depths of about 1 to 2 feet bgs. Below the native clay soils, the subsurface conditions consisted of sandstone and claystone bedrock to the maximum depths explored of about 50 feet bgs. Groundwater was encountered in Boring Nos. 4 and 8 at depths ranging from about 24 to 35 feet bgs at the time of our exploration. Groundwater was not encountered in the remaining borings to the maximum depths explored of about 10 to 50 feet bgs at the time of our exploration.
Earthwork	Based upon the results of our field exploration and laboratory testing, it is our opinion the existing fill should not be used to support foundations, interior floors, exterior slabs-on-grade, or pavement construction without complete removal and modification. If the owner is willing to accept a higher risk of movement for pavements and exterior slabs, consideration could be given to overexcavating a portion of the existing fill materials below these elements, then processing, moisture conditioning, and compacting the materials back to subgrade elevation.
Foundation Recommendations	Due to the presence of expansive clay and claystone materials, we recommend the proposed data center buildings and any

Topic [1]	Overview Statement [2]
	movement sensitive equipment, be constructed on drilled pier foundation systems socketed into the underlying claystone and sandstone bedrock. If the owner is willing to accept a higher risk of movement, the proposed equipment that is not sensitive to movement may be constructed on shallow spread footings or mat foundations over a zone of new engineered fill.
Interior Floors	Due to the presence of expansive soils and bedrock, we recommend the interior floor system of the data center buildings consist of structural floors.
Pavements	New pavements should be constructed on a zone of engineered fill to improve performance.
General Comments	This section contains important information about the limitations of this geotechnical engineering report.

1. If the reader is reviewing this report as a pdf, the topics above can be used to access the appropriate section of the report by simply clicking on the topic itself.
2. This summary is for convenience only. It should be used in conjunction with the entire report for design purposes.

Introduction

This report presents the results of our subsurface exploration and Geotechnical Engineering services performed for the proposed data centers to be located at 1120 N Gun Club Road in Aurora, Colorado. The purpose of these services was to provide information and geotechnical engineering recommendations relative to:

- Subsurface conditions
- Groundwater conditions
- Earthwork
- Grading and drainage
- Foundation design and construction
- Floor slab design and construction
- Seismic site classification
- Lateral earth pressures
- Pavement design and construction

The geotechnical engineering Scope of Services for this project included the advancement of 23 test borings (designated as Boring Nos. 1 to 22 and P1) to depths ranging from approximately 10 to 50 feet below existing site grades.

Previously, Terracon prepared a geotechnical report at this project site (Terracon project No. 25215031, dated September 7, 2021). The geotechnical study included the advancement of 56 test borings (designated as Boring Nos. 01 to 42, F1 to F6, and S1 to S8) to depths ranging from approximately 30 to 50 feet below existing site grades. The relevant information obtained from this study has been incorporated into our geotechnical analysis and design for the project and relevant data is included in the **Previous Geotechnical Data** section.

Plans showing the site location and boring locations are shown on the **Site Location** and **Exploration Plan**, respectively. The results of the laboratory testing performed on soil and bedrock samples obtained from the site during our field exploration are included on the boring logs and/or as separate graphs in the **Exploration Results** section.

Project Description

Our understanding of the project conditions is as follows:

Item	Description
Information Provided	We were provided the following documents: ▪ QTS Geotech RFP.pdf ▪ 2B_LOWSEISFINAL.pdf ▪ DEN1 DC2 DC3 Phasing Plan.pdf ▪ DEN1 DC2&3 – Grading and Utilities Plans.pdf ▪ 2023-1214_QTS DEN 1_Telecom Alignment_rev.pdf
Project Description	Based on information provided to us, we understand the proposed project consists of the design and construction of two new 244,876 square foot data center buildings (DC2 and DC3) with associated service yards and parking/driveway areas. We anticipate the proposed data center buildings will have two interior levels with a roof top mechanical platform, no basement areas, and have a steel frame with either concrete tilt-panel or precast concrete perimeter walls. We also understand that each data center will have an exterior equipment yard.
Finished Floor Elevation	Based on the provided grading plans, we understand the finished floor elevations for the buildings are anticipated to be about 5,545.27 feet and 5,548.27 feet for DC2 and about 5,546.60 feet and 5,549.60 feet for DC3.
Maximum Loads (Provided)	Anticipated structural loads were provided for single-story data center structures and equipment pads: ▪ Building Columns: 1500 to 1800 kips with a slab-on-grade floor; 2600 to 2800 kips with a structural floor ▪ Perimeter Walls: 6 to 10 kips per linear foot (klf) ▪ Floor slabs: 300 pounds per square foot (psf) ▪ Mat Foundations: 800 to 1000 psf ▪ Service Yard Equipment: 150 to 200 kips
Grading/Slopes	Based on our review of the provided grading plan the maximum cut/fill for the site will be about +/- 8 feet.
Excavation Depth	3 feet for frost depth foundations for equipment that is not movement sensitive.
Below-Grade Structures	None indicated.
Free-Standing Retaining Walls	None indicated.

Item	Description
Pavements	New pavements will likely consist of flexible asphalt and rigid concrete pavement. QTS provided the following traffic loading for light and heavy duty pavements: ▪ Light Duty – 7,500 Equvalent Single Axle Loads (ESALs) ▪ Heavy Duty – 75,000 ESALs ▪ Terminal Serviceability Index – 2.0 We understand the pavement design period is 20 years.

Terracon should be notified if any of the above information is inconsistent with the planned construction as revised and/or additional geotechnical recommendations may be required.

Site Conditions

The following description of site conditions is derived from our site visit in association with the field exploration.

Item	Description
Parcel Information	The project is located at 1120 N Gun Club Road in Aurora, Colorado. Latitude/Longitude (approximate) 39.7332° N, -104.7123° W See **Site Location**
Previous Studies	Terracon previously performed a geotechnical exploration and prepared a geotechnical engineering report (Terracon Project No. 25215031, dated September 7, 2023) for the DC1 building, which involved additional exploration across the site in the area of the proposed new DC2 and DC3 buildings. The relevant data from our previous study at the site has been used in our engineering analysis and pertinent data from our previous exploration is presented in this report.
Existing Improvements	The project site currently consists of a vacant lot. The existing data center building (DC1) is adjacent east of the vacant lot. The project site for DC2 and DC3 is currently being graded.
Current Ground Cover	The site is currently covered by moderate vegetation consisting of grass, weeds. Portions of the site consist of barren ground barren land.
Existing Topography	Topography at the project site slopes gently down to the north with an elevation difference on the order of about 25 to 30 feet.

Geotechnical Characterization

We have developed a general characterization of the subsurface conditions based upon our review of the subsurface exploration, laboratory data, and geologic setting. This characterization, termed GeoModel, forms the basis of our geotechnical calculations and evaluation of the site. Conditions observed at each exploration point are indicated on the individual logs. The individual logs can be found in the **Exploration Results** and the GeoModel can be found in the **Figures** attachment of this report. As noted in **General Comments**, the characterization is based upon widely spaced exploration points across the site, and variations are likely.

Subsurface Profile

As part of our analyses, we identified the following model layers within the subsurface profile. For a more detailed view of the model layer depths at each boring location, refer to the GeoModel.

Model Layer	Layer Name	General Description
1	**Asphalt**	Asphalt; about 3 inches
2	**Fill**	Fill material consisting of lean to fat clay with varying amounts of sand, silt and gravel; various stiffnesses
3	**Native Clay**	Native lean to fat clay with varying amounts of sand and silt; medium stiff to very stiff
4	**Native Sand**	Native sand with varying amounts of clay and silt; medium dense
5	**Bedrock**	Bedrock consisting of claystone and sandstone; soft to hard

Stratification boundaries on the boring logs represent the approximate location of changes in soil and material types; in situ, the transition between materials may be gradual. Further details of the borings can be found on the boring logs in the **Exploration Results**.

Based on the results of the laboratory testing and our experience in the area, the clay fill material, native clay soils, and claystone bedrock have low to high expansive potential, while the native sand soils and sandstone bedrock have nil to low expansive potential. A summary of laboratory test results is included in the **Exploration Results**.

Field Electrical Resistivity Test Results

Field measurements of soil electrical resistivity were performed by Terracon on September 3, 2024. The Wenner arrangement (equal electrode spacing) was used with "a" spacings of 2, 5, 10, 20, 30, 40, 50, 60, 70, 80, 100, 150, 200, and 250 feet at two locations (designated locations ER-1 and ER-2) within the DC2 and DC3 data center footprints. The "a" spacing is generally considered to be the depth of influence of the test. The testing was performed in both a northwest-southeast and a northeast-southwest orientation at the location ER-1 and performed in both a north-south and an east-west orientation at the location ER-2. The approximate location of the center of the in-situ electrical resistivity lines are shown in the **Site Location and Exploration Plans**. Results of the soil electrical resistivity measurements are presented in the **Field Electrical Resistivity Test Results**.

It should be noted that the resistivity values measured in the field may vary by material type, moisture content, surface temperature, groundwater depth, and other climatic conditions. During the site visit, our field representative indicated the ground surface cover consisted of graded clay and silt soils. The weather conditions during the site visit are indicated on the field data sheets.

Groundwater Conditions

The borings were observed while drilling and upon completion of drilling for the presence and level of groundwater. The water levels encountered in the boreholes can be found on the boring logs in **Exploration Results** and are summarized below.

Boring No.	Shallowest depth to groundwater encountered while or upon completion of drilling [1,2]	Elevation of the shallowest groundwater encountered while or upon completion of drilling [1,2]
1	Not encountered	Not encountered
2	Not encountered	Not encountered
3	Not encountered	Not encountered
4	24	5525.5
5	Not encountered	Not encountered
6	Not encountered	Not encountered
7	Not encountered	Not encountered
8	34	5515.7
9	Not encountered	Not encountered
10	Not encountered	Not encountered

Boring No.	Shallowest depth to groundwater encountered while or upon completion of drilling [1,2]	Elevation of the shallowest groundwater encountered while or upon completion of drilling [1,2]
11	Not encountered	Not encountered
12	Not encountered	Not encountered
13	Not encountered	Not encountered
14	Not encountered	Not encountered
15	Not encountered	Not encountered
16	Not encountered	Not encountered
17	Not encountered	Not encountered
18	Not encountered	Not encountered
19	Not encountered	Not encountered
20	Not encountered	Not encountered
21	Not encountered	Not encountered
22	Not encountered	Not encountered
P1	Not encountered	Not encountered
22 (25215031)	Not encountered	Not encountered
23 (25215031)	Not encountered	Not encountered
24 (25215031)	Not encountered	Not encountered
25 (25215031)	Not encountered	Not encountered
26 (25215031)	Not encountered	Not encountered
27 (25215031)	Not encountered	Not encountered
28 (25215031)	Not encountered	Not encountered
29 (25215031)	Not encountered	Not encountered
30 (25215031)	Not encountered	Not encountered
31 (25215031)	Not encountered	Not encountered
32 (25215031)	Not encountered	Not encountered
33 (25215031)	Not encountered	Not encountered
34 (25215031)	Not encountered	Not encountered
35 (25215031)	Not encountered	Not encountered
36 (25215031)	Not encountered	Not encountered
37 (25215031)	Not encountered	Not encountered
38 (25215031)	Not encountered	Not encountered

Boring No.	Shallowest depth to groundwater encountered while or upon completion of drilling [1,2]	Elevation of the shallowest groundwater encountered while or upon completion of drilling [1,2]
39 (25215031)	Not encountered	Not encountered
40 (25215031)	Not encountered	Not encountered
41 (25215031)	Not encountered	Not encountered
42 (25215031)	Not encountered	Not encountered
F1 (25215031)	Not encountered	Not encountered
F2 (25215031)	Not encountered	Not encountered
F3 (25215031)	Not encountered	Not encountered
F4 (25215031)	Not encountered	Not encountered
F5 (25215031)	Not encountered	Not encountered
F6 (25215031)	Not encountered	Not encountered

1. Borings were backfilled immediately after completion. Therefore, subsequent groundwater measurements were not obtained.
2. To the maximum explored depths of about 40 to 50 feet for Boring Nos. 1 through 22, and to the maximum explored depth of about 10 feet for Boring No. P1.

These observations represent groundwater conditions at the time of the field exploration and may not be indicative of other times or at other locations.

Zones of perched and/or trapped groundwater may also occur at times in the subsurface soils overlying bedrock, on top of the bedrock surface or within permeable fractures in the bedrock materials. The location and amount of perched water is dependent upon several factors, including hydrologic conditions, type of site development, irrigation demands on or adjacent to the site, fluctuations in water features, seasonal and weather conditions.

Groundwater level fluctuations occur due to seasonal variations in the amount of rainfall, runoff, and other factors not evident at the time the borings were performed. Groundwater levels during construction or at other times in the life of the structure may be higher or lower than the levels indicated on the boring logs. The possibility of groundwater level fluctuations should be considered when developing the design and construction plans for the project.

Corrosivity

The table below lists the results of laboratory soluble sulfate, soluble chloride, sulfides, Red-Ox potential, electrical resistivity, pH, and total salts testing. The values may be

used to estimate potential corrosive characteristics of the on-site soils with respect to contact with the various underground materials which will be used for project construction.

Corrosivity Test Results Summary

Boring No.	Sample Depth (feet)	Soluble Sulfate (%)	Soluble Chloride (%)	Sulfides	Red-Ox (mV)	Electrical Resistivity (Ω-cm)	pH	Total Salts (mg/kg)
1	0 - 5	<0.10	21	Negative	+364	780	8.22	1046
4	0 - 5	<0.10	67	Trace	+326	650	7.82	2448
6	0 - 5	<0.10	18	Trace	+398	500	7.86	3430
10	0 - 5	<0.10	330	Trace	+323	340	7.78	7552
11	0 - 5	<0.10	11	Present	+359	430	7.87	4474
13	0 - 5	<0.10	22	Trace	+317	455	7.77	3200
14	0 - 5	<0.10	191	Present	+303	430	7.93	2384
16	0 - 5	<0.10	85	Trace	+313	390	7.84	4992
17	0 - 5	<0.10	28	Trace	+314	430	7.88	3693
19	0 - 5	<0.10	99	Trace	+316	340	7.87	1600
20	0 - 5	<0.10	107	Negative	+307	505	7.86	4022

Results of water-soluble sulfate testing indicate that samples of the on-site soils have an exposure class of S0 when classified in accordance with of the American Concrete Institute (ACI) Design Manual. The results of the testing indicate ASTM Type I portland cement is suitable for project concrete in contact with on-site soils. However, if there is no (or minimal) cost differential, use of ASTM Type II portland cement is recommended for additional sulfate resistance of construction concrete. Concrete should be designed in accordance with the provisions of the ACI Design Manual.

Imported fill materials may have significantly different properties than the site materials noted above and should be evaluated if expected to be in contact with materials used for construction. We recommend a National Association of Corrosion Engineers (NACE) certified corrosion professional be engaged to determine the need for corrosion protection and to design appropriate protective measures.

Thermal Resistivity Tests

Laboratory thermal resistivity testing was conducted by GeoTherm on bulk samples obtained from depths between about 0 to 5 feet at two (2) boring locations. The results of the thermal resistivity testing are presented in the following table:

Boring No.	Depth (ft.)	Soil Description	Thermal Resistivity (°C-cm/W)		Moisture Content (%)	Dry Density (lb/ft³)
			Wet	Dry		
7	0 – 5	Lean Clay	67	173	18	102
18	0 – 5	Fat Clay	73	182	19	99

Geotechnical Overview

Based on subsurface conditions encountered in the borings, the site appears suitable for the proposed construction from a geotechnical point of view provided certain precautions and design and construction recommendations outlined in this report are followed. We have identified geotechnical conditions that could impact design and construction of the proposed data center buildings and other site improvements.

Existing Fill Materials

Up to about 2 feet of fill materials were encountered in portions of the data center footprints and in the pavement boring (Boring No. P1). It should be noted that fill depths presented in the boring logs are approximate and the depth, lateral extents, and composition of fill should be expected to vary. We do not possess any information regarding whether the fill was placed under the observation of a geotechnical engineer.

Based upon the results of our field exploration and laboratory testing, it is our opinion the existing fill should not be used to support foundations, interior slabs, exterior slabs-on-grade, or pavement construction without complete removal and modification.

If the owner is willing to accept a higher risk of movement for pavements and exterior slabs, consideration could be given to overexcavating a portion of the existing fill materials below these elements, then processing, moisture conditioning and compacting the materials back to subgrade elevation.

It should be noted that there exists the potential for construction debris and/or domestic trash to be encountered within the fill on some portions of the site. Based on the results of our field exploration, it is our opinion the potential for encountering construction debris and/or domestic trash is low. The fill materials should be observed for the presence of debris and trash during site grading and construction.

The existing fill can be reused as engineered fill below foundations, slabs-on-grade, and pavements, provided the any deleterious materials are removed. Some removal and replacement may be required if unsuitable or soft materials are exposed.

Expansive Soils and Bedrock

Based on the results of the laboratory testing and our experience in the area, the clay fill material, native clay soils, and claystone bedrock have low to high expansive potential, while the native sand soils and sandstone bedrock have nil to low expansive potential.

This report provides recommendations to help mitigate the effects of soil and bedrock shrinkage and expansion. However, even if these procedures are followed, some movement and cracking in the buildings, pavements, and flatwork should be anticipated. The severity of cracking and other damage such as uneven slabs-on-grade will likely increase if any modification of the site results in excessive wetting or drying of the expansive soils and bedrock. Eliminating the risk of movement and distress is generally not feasible, but it may be possible to further reduce the risk of movement if significantly more expensive measures are used during construction. It is imperative the recommendations outlined in the **Grading and Drainage** subsection of **Earthwork** in this report be followed to reduce movement.

Shallow Bedrock

Shallow bedrock was encountered in borings within the entire footprint for the DC3 building and along the southern edges of the footprint for the DC2 building. Bedrock generally consisted of expansive claystone interbedded lenses of sandstone bedrock. The presence of shallow bedrock below portions of the proposed buildings may result in difficult excavations in these areas.

Difficult Excavations

It is anticipated that excavations for the proposed improvements will require heavy-duty equipment due to shallow claystone or sandstone bedrock. Consideration should be given to obtaining a unit price for difficult excavation in the contract documents for the project.

Earthwork

The following sections present recommendations for site preparation, excavation, subgrade preparation, and placement of engineered fills on the project. All earthwork on the project should be observed and evaluated by Terracon.

Site Preparation

Strip and remove existing pavements, vegetation, organics, and other deleterious materials from proposed building and pavement areas. All exposed surfaces should be free of mounds and depressions that could prevent uniform compaction.

Stripped materials consisting of vegetation, unsuitable fills, and organic materials should be wasted from the site or used to revegetate landscaped areas or exposed slopes after completion of grading operations.

Where possible, the site should be initially graded to create a relatively level surface to receive fill and to provide for a relatively uniform thickness of fill beneath the proposed building and improvement areas. All exposed areas that will receive fill, once properly cleared, should be scarified to a minimum depth of 12 inches, conditioned to near optimum moisture content, and compacted. It is imperative the moisture content of prepared materials be protected from moisture loss.

In areas receiving engineered fill to meet proposed grades, all existing fill materials must be removed, moisture conditioned, and compacted prior to placement of new engineered fill.

Although evidence of underground facilities was not observed during our exploration, such features could be encountered during construction. If unexpected fills or underground facilities are encountered, such features should be removed and the excavation thoroughly cleaned prior to backfill placement and/or construction.

The stability of subgrade soils may be affected by precipitation, repetitive construction traffic, or other factors. If unstable conditions are encountered or develop during construction, workability may be improved by over-excavation of wet zones and mixing these soils with crushed gravel. Use of geotextiles could also be considered as a stabilization technique. Lightweight excavation equipment may be required to reduce subgrade pumping.

Settlement of Deep Engineered Fill Zones

Settlement will occur in new engineered fills due to the self-weight of the newly placed engineered fill materials. The magnitude of settlement of the fill zones will be directly related to the type of fill material used, the degree of compaction, and the thickness of the fill zone. The following table presents various fill thicknesses and the estimated settlement due to the self-weight of properly moisture conditioned and compacted fill:

Maximum Fill Height	Estimated Settlement of New Engineered Fill [1]
About 0 to 5 feet	Less than 1 inch
About 5 to 10 feet	About 1 inch

Maximum Fill Height	Estimated Settlement of New Engineered Fill [1]

1. Assumes on-site lean or fat clays or processed claystone bedrock are used as new engineered fill as recommended in the **Earthwork** section of this report.

Use of granular materials or controlled low-strength material (CLSM) can be used to reduce the amount of settlement of engineered fills. However, due to the expansive nature of the native soils and claystone bedrock, we do not recommend the use of granular fill materials, unless a drain is constructed at the base of these materials. If CLSM is used as fill, the material should be designed to have a low permeability.

We have not included fill specifications for granular fill, due to the expansive nature of the on-site soils and bedrock. If the design team would like to pursue the use of granular fill for this project, we should be consulted with in order to provide proper drainage recommendations for these materials.

Material Types

Fill for this project should consist of engineered fill. Engineered fill is fill that meets the criteria presented in this report and has been properly documented.

Engineered fill should meet the following material property requirements:

Fill Type [1,2]	USCS Classification	Acceptable location for placement
On-site clay soils	CL, CH	On-site clay soils are considered suitable for reuse as fill below foundations, slabs, and pavements and as general fill for this project.
On-site sand soils [3,4]	SC	On-site sand soils mixed with clay soils are considered suitable for reuse as fill below foundations, slabs, and pavements and as general fill for this project.
Processed claystone bedrock [5]	N/A	Processed onsite claystone bedrock may be used at depths greater than 10 feet below bottom of foundation elements and interior slabs-on-grade, or 5 feet below the bottom of pavements and exterior slabs-on-grade.
Lime treated on-site soils [6]	N/A	On-site clay soils treated with lime are considered suitable for use as fill below foundations, slabs, and pavements and as general fill for this project.

Fill Type [1,2]	USCS Classification	Acceptable location for placement
CDOT Class 1 [4]	Varies	CDOT Class 1 Structural Fill (or an approved equivalent) is considered acceptable for use as engineered fill beneath slabs and pavements.
Imported soils	Varies	Imported soils meeting the gradation outlined herein can be considered acceptable for use as engineered fill beneath foundations, slabs and pavements.

1. Engineered fill should consist of approved materials that are free of organic matter and debris. Frozen material should not be used, and fill should not be placed on a frozen subgrade. A sample of each material type should be submitted to the Geotechnical Engineer for evaluation.
2. Care should be taken during the fill placement process to avoid zones of dis-similar fill. Improvements constructed over varying fill types are at a higher risk of differential movement compared to improvements over a uniform fill zone.
3. On-site sand soils should be mixed with on-site clay soils to meet the gradation requirements outlined below for imported soils.
4. Granular soils should not be used in the overexcavation zone below the buildings, slabs, or pavement areas unless they are drained.
5. On-site claystone bedrock materials should be staged separately from excavated soils. The bedrock materials must be processed to meet the gradation requirements for imported soils.
6. A formal mix design should be prepared for lime treated soils. For estimating purposes, 5% lime can be assumed.

Imported soils for engineered fill (if required) should meet the following material property requirements:

Gradation	Percent finer by weight (ASTM C136)
1”	100
¾”	50-100
No. 4 Sieve	50-100
No. 200 Sieve	>50

- Liquid Limit..... 30 (max)
- Plasticity Index..... 15 (max)
- Maximum Expansive Potential (%)..... 1.0*

  *Measured on a sample compacted to approximately 95 percent of the ASTM D698 maximum dry density at optimum water content. The sample is confined under a 200-psf surcharge and submerged.

Lime Treated Soils/Processed Claystone

As previously stated, expansive soils and bedrock are present below this site and present a risk for potential heave and associated distress to grade supported structures. One option to reduce risk for potential heave when using on-site soils or processed claystone bedrock as engineered fill is to mix lime with these soils. The addition of lime to the on-site soils or processed claystone significantly reduces the expansive potential of these materials and increases the strength of these materials after placement due to the chemical reaction between the materials.

Additional laboratory testing on samples to be used as fill and engineering analysis will be necessary to determine the appropriate lime mix-design criteria. The lime should meet the requirements of ASTM C207, Type N and be mixed as outlined in ASTM C977. The lime shall be spread by a method approved by the engineer at the rate shown on the plan or as directed by the engineer. Lime shall not be applied when wind conditions, in the opinion of the engineer, are such that blowing lime becomes objectionable to traffic or adjacent property owners. Water should be added (as needed) to the lime-soil mixture during the mixing process until a uniform, homogeneous mixture free from clods or lumps is achieved at 1 to 4 percent above optimum moisture content for the mixture.

Following initial mixing with the lime, we recommend allowing the lime-soil mixture to mellow for a period of 5 to 7 days. The mellowing time needed will depend on weather conditions, continuous moistening of the lime-soil mixture and soil properties. Terracon will assist the project team with determining the appropriate mellowing time as the process approaches 5 days. Fill soils planned for lime treatment should be treated outside of the fill placement areas and should be stockpiled prior to placement.

After the mellowing period has completed (to allow for the chemical reaction to occur), the treated materials can be placed as engineered fill in the designated areas. If used, the treated material should be placed and compacted in accordance with the recommendations presented in the **Compaction Requirements** subsection of **Earthwork**.

The treated soils can also be used in critical areas where structures may be sensitive to movement or areas where compaction may be difficult. These treated materials can also be used to repair local unstable areas.

Compaction Requirements

Engineered fill should be placed and compacted in horizontal lifts, using equipment and procedures that will produce recommended moisture contents and densities throughout the lift.

Item	Description
Fill lift thickness	8-inches or less in loose thickness when heavy, self-propelled compaction equipment is used 4 to 6-inches in loose thickness when hand-guided equipment (i.e. jumping jack, plate compactor) is used
Compaction requirements [1,2]	Minimum of 98% of the material's standard Proctor maximum dry density (ASTM D698) for fills below a depth of 10 feet. Fills at or less than 10 feet in depth may be compacted to a minimum of 95% of the material's standard Proctor maximum dry density (ASTM D698) for clay soils. Minimum of 98% of the material's standard Proctor maximum dry density of CDOT Class 1 Structural Fill.
Moisture content cohesive soils (clay and lime treated soils) [3]	+1 to +4% of the optimum moisture content.
Moisture content CDOT Class 1	-2 to +2% of the optimum moisture content.

1. We recommend that engineered fill be tested for water content and compaction during placement. Should the results of the in-place density tests indicate the specified water or compaction limits have not been met, the area represented by the test should be reworked and retested as required until the specified water and compaction requirements are achieved.
2. Water levels should be maintained low enough to allow for satisfactory compaction to be achieved without the compacted fill material pumping when proofrolled.
3. Moisture conditioned clay soils should not be allowed to dry out. A loss of moisture within these materials could result in an increase in the materials expansive potential. Subsequent wetting of these materials could result in undesirable movement.

Excavation

Excavations into the subsurface soils and bedrock will encounter a variety of conditions. The individual contractor(s) is responsible for designing and constructing stable, temporary excavations as required to maintain stability of both the excavation sides and bottom. All excavations should be sloped or shored in the interest of safety following local and federal regulations, including current Occupational Safety and Health Administration (OSHA) excavation and trench safety standards.

Soils and bedrock penetrated by the proposed excavations may vary significantly across the site. The soil and bedrock classifications are based solely on the materials encountered in the exploratory borings. The contractor should verify that similar conditions exist throughout the proposed area of excavation. If different subsurface conditions are encountered at the time of construction, the actual conditions should be evaluated to determine any excavation modifications necessary to maintain safe conditions.

Grading and Drainage

All grades must be adjusted to provide positive drainage away from the buildings during construction and maintained throughout the life of the proposed project. Infiltration of water into utility or foundation excavations must be prevented during construction. Landscaped irrigation adjacent to the foundation systems should be minimized or eliminated. Water permitted to pond near or adjacent to the perimeter of the structures (either during or post-construction) can result in significantly higher soil movements than those discussed in this report. As a result, any estimations of potential movement described in this report cannot be relied upon if positive drainage is not obtained and maintained, and water is allowed to infiltrate the fill and/or subgrade.

Permanent grades should be sloped at a minimum of 10 percent grade for at least 10 feet beyond the perimeter of the buildings. Asphalt pavement or concrete flatwork should be sloped at a minimum of 2 percent beyond the building perimeters for the life of the buildings. Where Americans with Disabilities Act (ADA) or other requirements or existing site features limit the gradient, slopes on the order of ½ to 1 percent minimum may be necessary to comply with the ADA but do increase the risk of unanticipated movement. Backfill against footings, exterior walls, and in utility and sprinkler line trenches should be compacted in accordance with recommendations in this report and free of all construction debris to reduce the possibility of water infiltration. After building construction and prior to project completion, we recommend that verification of final grading be performed to document that positive drainage, as described above, has been achieved.

Where paving or flatwork abuts the structures, care should be taken that joints are properly sealed and maintained to prevent the infiltration of surface water.

Landscape or xeriscape areas within 10 feet of the foundation systems shall not be hindered by landscape edging, grade variations, or vegetation. In addition, consideration should be given to snow removal practices that will minimize the stockpiling of snow in planter and landscaped areas adjacent to structural improvements.

Planters located adjacent to the structures should be watertight. Sprinkler mains and spray heads should be located a minimum of 10 feet away from the building lines. Where drip line irrigation is located near the buildings, we recommend that drip line

irrigation systems be located at least 5 feet from the outside edge of the foundations. Roof drains should discharge on pavements or be extended away from the structures a minimum of 10 feet through the use of splash blocks or downspout extensions.

Trees or other vegetation whose root systems have the ability to remove excessive moisture from the subgrade and foundation soils should not be planted next to the buildings. Trees and shrubbery should be kept away from the exterior edges of foundations, a distance at least equal to their expected mature height.

Earthwork Construction Considerations

Upon completion of grading operations, care should be taken to maintain the moisture content of the subgrade prior to construction of slabs-on-grade, pavements, etc. Construction traffic over prepared subgrade should be minimized and avoided to the extent practical. Construction traffic over processed clay subgrade will eventually reduce the moisture content and increase the density of the subgrade. Subsequent wetting of these materials will result in undesirable movement.

The site should also be graded to prevent ponding of surface water on prepared subgrade or in excavations. In areas where water is allowed to pond over a period of time, the affected area should be removed and allowed to dry out; however, allowing the clay soils to dry out below the optimum moisture content is not recommended. If constraints do not allow for moisture conditioning of affected clays as recommended in this report, the affected area should be overexcavated and replaced with engineered fill.

The Geotechnical Engineer should be retained during the construction phase of the project to observe earthwork and to perform necessary tests and observations during overexcavation operations, excavations, subgrade preparation; proof-rolling; placement and compaction of controlled compacted fills; backfilling of excavations into the completed subgrade, and just prior to construction of slabs-on-grade.

Construction Observation and Testing

The earthwork efforts should be monitored under the direction of the Geotechnical Engineer. Monitoring should include documentation of adequate removal of existing structures, vegetation and topsoil, proofrolling, and mitigation of areas delineated by the proofroll to require mitigation.

Each lift of compacted fill should be tested, evaluated, and reworked, as necessary, until approved by the Geotechnical Engineer prior to placement of additional lifts. Each lift of fill should be tested for density and water content.

In areas of foundation excavations, the subgrade should be evaluated under the direction of the Geotechnical Engineer. If unanticipated conditions are encountered, the Geotechnical Engineer should prescribe mitigation options.

In addition to the documentation of the essential parameters necessary for construction, the continuation of the Geotechnical Engineer into the construction phase of the project provides the continuity to maintain the Geotechnical Engineer's evaluation of subsurface conditions, including assessing variations and associated design changes.

Utility Recommendations

Based on the presence of expansive materials at this site, we do not recommend the use of clean, granular bedding materials below utilities. If clean bedding materials are required by the utility installers or civil engineer, there is an increased risk of movement associated with the use of these materials. Regardless of the bedding material used below utilities, all utilities should be sloped away from the proposed buildings.

All underground piping within 10 feet of the proposed buildings should be designed with flexible couplings, so minor deviations in alignment do not result in breakage or distress. Utility knockouts in foundation walls should be oversized to accommodate differential movements on the order of 3 to 5 inches. If utility trenches are backfilled with relatively clean granular material, they should be capped with at least 18 inches of on-site clay soil to reduce the infiltration and conveyance of surface water through the trench backfill.

Utility trenches are a common source of water infiltration and migration. All utility trenches that penetrate beneath the data center buildings should be effectively sealed to restrict water intrusion and flow through the trenches that could migrate below the buildings. We recommend constructing an effective clay "trench plug" that completely surrounds the utility line to the extents of the trench and extends at least 10 feet out from the building exteriors. The clay plug material should consist of on-site clay compacted in accordance with the recommendations presented in the **Compaction Requirements** subsection of **Earthwork**. The clay fill should completely surround the utility line and be properly placed and compacted in accordance with recommendations in this report.

Utilities entering the building through the structural floor should be sleeved to allow for potential vertical movement of these utilities.

The potential movement of utilities is primarily influenced by water infiltration into the utility trenches. The recommendations presented in this section are intended to reduce the qualitative risk of movement of utilities by reducing the risk of water infiltration into the utility trenches. Additional considerations for dry utilities is the routing of the utility

below landscaped areas and the proximity of the dry utility to a wet utility. Dry utilities below irrigated areas, in close proximity to wet utilities, or where a dry utility crosses below a wet utility will have a qualitative higher risk of movement, then a dry utility placed under hardscape. An additional consideration to further reduce the risk of water infiltration into utility trenches is the use of a low permeable controlled low-strength material (CLSM) or lime-treated on-site soils or claystone bedrock as trench backfill.

Thrust Block Design Recommendations

Thrust blocks bearing on and against undisturbed native soils, claystone, or properly compacted fill/backfill can be evaluated and designed using the following parameters. Stabilization of thrust block excavations may be required where low strength soils are encountered or develop during construction.

Description	Design Parameter
Bearing material	Native soils or new engineered fill
Allowable soil bearing pressure [1]	2,000 psf
Ultimate passive pressure [2]	185 psf/ft
Ultimate Coefficient of sliding friction [2]	0.3

1. Unstable soils, if encountered, should be removed and replaced with engineered fill or stabilized by other suitable methods. Terracon should be contacted to provide site-specific recommendations if unstable soils are encountered in thrust block excavations.
2. For evaluating resistance to lateral movement (or sliding) of thrust blocks, the sides of the excavation for thrust blocks must be nearly vertical and concrete should be placed neat against these vertical faces or backfill must be compacted to at least 95 percent of the standard Proctor maximum dry density for the passive earth pressure values to be valid. Passive pressure requires movement to generate the resistance and should only be used when movement is tolerable and the soil is well compacted and will not be removed. The passive resistance and friction factor are ultimate values. As such, appropriate factors of safety should be applied.

Foundation Recommendations – Data Center Buildings

Based on the subsurface conditions encountered in the exploratory borings, we recommend the data center buildings will be constructed on drilled piers socketed into the underlying claystone bedrock. Design recommendations for drilled piers are presented in the following tables and paragraphs:

Description	Value
Minimum Pier Length [1]	35 feet
Minimum Bedrock Embedment [2,3]	15 feet
Maximum Allowable End-Bearing Pressure [4]	25,000 psf
Maximum Allowable Skin Friction [5,6,7]	2,000 psf
Uplift Force (Tension due to Soil/Bedrock Uplift, kips)	85 x pier diameter (feet)
Minimum Pier Diameter	18 inches or length/diameter < 30
Shear Rings	3 inches high by 3 inches deep at 16 inches on center within bottom 15 feet of shaft
Void Thickness [8]	12 inches

1. The required minimum length is from the bottom of the grade beams or pier caps.
2. The required minimum bedrock embedment considers the uplift force from soil/bedrock uplift only.
3. Drilled shafts should be embedded into firm or harder bedrock materials.
4. In accordance with IBC Section 1806.1, if design of the pier foundations is completed using allowable stress design (ASD) methods, the vertical bearing resistance may be increased by 1/3 where used with the alternative basic load combinations that include wind or seismic loads that are described in IBC Section 1605.3.2. This 1/3 increase does not apply if design is completed using load and resistance factor design (LRFD) methods. Increases also do not apply to the resistance of uplift loads, nor to the soil properties provided in this report for modeling the pier response to lateral loads, nor the resulting lateral pier deflection vs. applied load graph.
5. The portion of the drilled piers that can resist the uplift force must be (a) embedded in firm or harder bedrock and (b) be at least 20 feet below the top of the drilled pier.
6. Skin friction should not be applied in fill or overburden soils, or the upper 5 feet of the drilled pier, whichever is deepest.
7. Skin friction can be used to resist compressive and uplift loads.
8. The void space below grade beams (between the piers) and pier caps should be periodically maintained and re-established, as necessary.
9. Movement on the order of less than ½ inch should be anticipated for the drilled piers, provided the piers and properly designed and constructed.

Piers should be considered to work in group action if the center-to-center horizontal spacing is less than three pier diameters. A minimum practical center-to-center horizontal spacing between piers of at least three diameters should be maintained, and adjacent piers should bottom at the same elevation. The capacity of individual piers must be reduced when considering the effects of group action. Capacity reduction is a

function of pier spacing and the number of piers within a group. The following table presents capacity reductions for closely spaced piers.

Description	Value		
Drilled Pier Spacing (Center-to-Center)	>3 diameters	>2 to 3 diameters	1 to 2 diameters
Pier Capacity Reduction	None	30 percent	50 percent

1. End bearing values do not need to be reduced for closely spaced piers if bottom of piers are at the same elevation.

To satisfy forces in the horizontal direction using the computer program LPile©, piers may be designed for the following lateral load criteria:

Soil Type	Curve Model (p-y)	Unit Weight (pcf)	Undrained Shear Strength (psf)	Angle of Internal Friction, Φ (degrees)	Coeff. of Subgrade Reaction, k (pci)	Strain, ε_{50} (%)
Clay	Stiff clay w/out free water	110	1,000	--	Allow LPile program to choose values based on the other parameters in this table.	
Sand	Reese	110	--	30		
Claystone / Sandstone	Stiff clay w/out free water	120	2,000	--		

Lateral analysis should account for the center-to-center spacing and P-Y multiplier values per the following table:

Pier Center-to-Center Spacing (In Direction of Loading)	P-multiplier, P_M Row 1	P-multiplier, P_M Row 2	P-multiplier, P_M Row 3 and Higher
3 x diameter	0.8	0.4	0.3
5 x diameter	1.0	0.85	0.7

The structural engineer should determine the reinforcement necessary for the piers. At a minimum, all piers should be reinforced full depth for the applied axial, lateral, and uplift stresses imposed. We recommend a minimum reinforcement of at least 1 percent of the cross-sectional area of the pier.

Drilling to design depth should be possible with conventional single-flight power augers; however, very hard and potentially cemented bedrock layers could require the use of heavy-duty equipment.

Pier casing will likely be required where groundwater, loose soils, or caving soils are encountered. Casing should be withdrawn in a slow continuous manner maintaining a sufficient head of concrete to prevent infiltration of water or caving soils or the creation of voids in pier concrete. Pier concrete should have a relatively high fluidity when placed in cased pier holes or through a tremie. Pier concrete with slump in the range of 5 to 7 inches is recommended for uncased piers. For cased piers, a slump in the range of 7 to 9 inches is recommended.

Groundwater (if encountered) should be removed from each pier hole prior to concrete placement. Pier concrete should be placed immediately after completion of drilling and cleaning. If pier concrete cannot be placed in dry conditions, a tremie should be used for concrete placement. Free-fall concrete placement in piers will only be acceptable if provisions are taken to avoid striking the concrete on the sides of the hole or reinforcing steel. The use of a bottom-dump hopper, or an elephant's trunk discharging near the bottom of the hole where concrete segregation will be reduced, is recommended.

Due to potential sloughing and raveling, foundation concrete quantities may exceed calculated geometric volumes. Pier-bearing surfaces must be cleaned prior to concrete placement. A Terracon representative should observe the bearing surface and shaft configuration.

The top of the piers should be cylindrical in shape. Forms may be necessary at the top of the piers in order to minimize the disturbance of the soils and to maintain a cylindrical shape. Failure to provide this shape (i.e., allowing mushrooming of pier tops) may result in additional uplift forces and unanticipated movement.

Foundation Recommendations – Ancillary Equipment

Based upon the results of the field exploration and laboratory testing program for this exploration, the following foundation systems were evaluated for use for the proposed ancillary equipment pads:

- Straight-shaft, cast-in-place piers constructed in bedrock.
- Spread footing foundations on engineered fill.

Due to the presence of expansive soils and bedrock, we recommend movement sensitive ancillary equipment be constructed on a drilled pier foundation systems.

If the owner is willing to accept a higher risk of movement, non-movement sensitive ancillary equipment could be constructed on spread footings over a zone of new engineered fill.

Drilled Pier Recommendations

Drilled piers for movement sensitive ancillary equipment may be designed using the recommendations presented in the **Foundation Recommendations – Data Center Buildings** section.

Spread Footing Foundation Recommendations

Spread footing foundation systems may be considered for non-movement sensitive ancillary equipment when constructed on a zone of new engineered fill, provided all existing fill materials are removed and modified or replaced and the potential for movement can be tolerated. New fill materials beneath foundations should be placed and compacted as outlined in the **Earthwork** section of this report.

Design recommendations and predicted performance for spread footing foundation systems are presented in the following table and paragraphs.

Description	Value
Thickness of Zone of New Engineered Fill	Minimum of 10 feet below bottom of lowest foundation element
Lateral Extents of New Engineered Fill	15 feet
Supporting Stratum	New engineered fill
Maximum Allowable Bearing Pressure [1]	3,000 psf
Minimum Dead Load Pressure	800 psf or what is practical
Void Thickness, If Needed	4 inches
Coefficient of Friction (Sliding)	0.3
Minimum Footing Dimensions	Isolated footings: 24 inches Continuous footings: 18 inches
Minimum Embedment Below Finished Grade for Frost Protection [2]	3 feet
Approximate Total Movement [3]	Up to about 1½ inches
Estimated Differential Movement [3,4,5]	About ½ to ¾ inches of total movement

1. The maximum allowable soil bearing pressure can be increased by 1/3 for transient loading conditions.
2. For footings that will be exposed to freezing conditions.

Description	Value

3. Foundation movement will depend upon the variations within the subsurface soil profile, the structural loading conditions, the embedment depth of the footings, the thickness of engineered fill, and the quality of the earthwork operations and footing construction.
4. Differential movement is considered over a distance of about 40 feet.
5. Footings should be proportioned on the basis of equal total dead load pressure to reduce differential movement between adjacent footings.

Additional foundation movements could occur if water from any source infiltrates the foundation soils; therefore, proper drainage should be provided in the final design and during construction and throughout the life of the structure. Failure to maintain the proper drainage as recommended in the **Grading and Drainage** section of **Earthwork** will nullify the movement estimates provided above.

Unstable subgrade conditions should be observed by the Geotechnical Engineer to assess the subgrade and provide suitable alternatives for stabilization. Stabilized areas should be proofrolled prior to continuing construction to assess the stability of the subgrade.

The base of all foundation excavations should be free of water and loose soil prior to concrete placement. Concrete should be placed soon after excavating to reduce bearing soil disturbance. Should the soils at bearing level become excessively dry, disturbed or saturated, or frozen, the affected soil should be removed prior to placing concrete.

Footings, foundations, and masonry walls should be detailed and reinforced as necessary to reduce the potential for distress caused by differential foundation movement. The use of joints at openings or other discontinuities in masonry walls is recommended.

Seismic Considerations

The following table presents the seismic site classification based on the 2018 International Building Code (IBC) and the subsurface conditions encountered within the borings:

Code Used	Site Classification
2018 International Building Code (IBC) [1,2]	D

1. In general accordance with the 2018 International Building Code.
2. The 2018 International Building Code (IBC) requires a site subsurface profile determination extending a depth of 100 feet for seismic site classification. The current scope requested does not include the required 100-foot subsurface profile determination. The deepest borings of this exploration extended to a maximum depth of about 40 feet and this seismic site class definition considers that

Interior Floors

Due to the presence of expansive clay soils and claystone bedrock, we recommend the interior floors of the data centers to be constructed as structural floor systems, structurally supported independent of the subgrade soils as a positive means of reducing the potentially detrimental effects of floor movement.

If the owner is willing to accept some movement, slab-on-grade floors may be utilized for the interior floor systems provided the building slabs are constructed on a zone of new engineered fill as described in the **Floor Slabs** subsection.

Structurally Supported Floor Recommendations

A minimum 12-inch crawlspace or void space between the low point for any element of the floor system and the ground surface should be used for structural floor construction. Carton type void forms, such as SureVoid or an approved equivalent, may be used to provide the minimum void space of 12 inches. The void form manufacturer's installation recommendations must be followed during construction of the structurally supported floor.

Larger void space/crawlspace areas may be considered to allow for safe access, inspection, and maintenance. Trenches excavated below slabs-on-void may act as conduits for water transmission and should be avoided. If used, they must be sloped to a drain, sump pit, or other appropriate outfall and must be periodically inspected and maintained.

Utilities below structural floors should be limited to crawlspace areas, should be independent of the subgrade, and should maintain the minimum 12-inch space between the utility and subgrade soils. Where possible, utilities should not be constructed below slabs-on-void.

The design team should account for control of humidity within crawlspaces. We recommend a firm specializing in the design and construction of air quality control systems should consulted.

Floor Slabs

If the owner is willing to accept some movement, slab-on-grade floors may be utilized for the interior floor system for the data center buildings provided the building slabs are constructed on a zone of new engineered fill, lime treated engineered fill, or granular fill mixed with onsite soils as recommended in the following table.

Description	Value		
Supporting Stratum [1,2]	New engineered fill with the top foot lime treated	5 feet of CDOT Class 1 structural fill over 10 feet of engineered fill	New Engineered fill with the top 5 feet lime treated
Depth of Overexcavation	Minimum of 15 feet below slab-on-grade		
Lateral extent of overexcavation zone [3]	15 feet outside building footprints		
Anticipated Movement	Up to 1 to 1½ inches	About 1 inch	About 1 inch

1. The engineered fill should meet the criteria presented in the **Material Types** subsection of **Earthwork**.
2. CDOT Class 1 Structural Fill (or equivalent) may only be used in the overexcavation zone below slab-on-grade areas if a drain is constructed at the base of these materials
3. Lateral extent of the overexcavation zone indicates a zone that received the full depth of overexcavation treatment. The lateral extent zone does not include the required slope layback area. If the overexcavation zone cannot be extended laterally outside of the perimeter footing, there is an increased risk of the overburden soils and underlying claystone becoming wetted.

For structural design of concrete slabs-on-grade, a modulus of subgrade reaction of 100 pounds per cubic inch (pci) may be used for point or limited area loads for floors supported on lime treated engineered fill or CDOT Class 1 structural fill.

Additional floor slab design and construction recommendations are as follows:

- Positive separations and/or isolation joints should be provided between slabs and all foundations, columns, or utility lines to allow independent movement.
- Control joints should be provided in slabs to control the location and extent of cracking.
- A minimum 3-inch void space should be constructed above or below non-bearing partition walls placed on the floor slab. Special framing details should be provided at doorjambs and frames within partition walls to avoid potential distortion. Partition walls should be isolated from suspended ceilings.
- Interior trench backfill placed beneath slabs should be compacted in accordance with recommended specifications described previously.
- The use of a vapor retarder should be considered beneath concrete slabs on grade that will be covered with wood, tile, carpet, or other moisture sensitive or impervious coverings, or when the slab will support equipment sensitive to moisture. When conditions warrant the use of a vapor retarder, the slab designer

and slab contractor should refer to ACI 302 for procedures and cautions regarding the use and placement of a vapor retarder.
- Floor slabs should not be constructed on frozen subgrade.
- Other design and construction considerations, as outlined in Section 302.1R of the ACI Design Manual, are recommended.

Movements of slab-on-grades using the above outlined technique will likely be reduced and tend to be more uniform. The estimates outlined previously assume that the other recommendations in this report are followed. Additional movement could occur should the subsurface soils become wetted to significant depths, which could result in potential excessive movement causing uneven floor slabs and severe cracking. This could be due to over watering of landscaping, poor drainage, improperly functioning drain systems, and/or broken utility lines. Therefore, it is imperative that the recommendations outlined in this section and in the **Grading and Drainage** subsection of **Earthwork** be followed.

Exterior Flatwork

Exterior flatwork can be expected to perform similarly to interior slabs-on-grade, provided the recommendations provided in the **Floor Slabs** section of this report are followed for exterior flatwork. However, if the owner is willing to accept a higher risk of movement for exterior flatwork, consideration could be given to constructing exterior slabs-on-grade on a minimum of 4 feet of new engineered fill or lime treated soils. Flatwork constructed on lime treated soils will experience better performance than when constructed on untreated soils. New fill materials beneath exterior flatwork should be placed and compacted as outlined in the **Earthwork** section of this report.

For structural design of exterior concrete slabs-on-grade, a modulus of subgrade reaction of 90 pci may be used for point or limited area loads for exterior slabs-on-grade at this site. If a modulus of subgrade reaction of 100 pci is desired, the upper 1 foot of new engineered fill must be lime treated.

Additional slab design and construction recommendations are as follows:

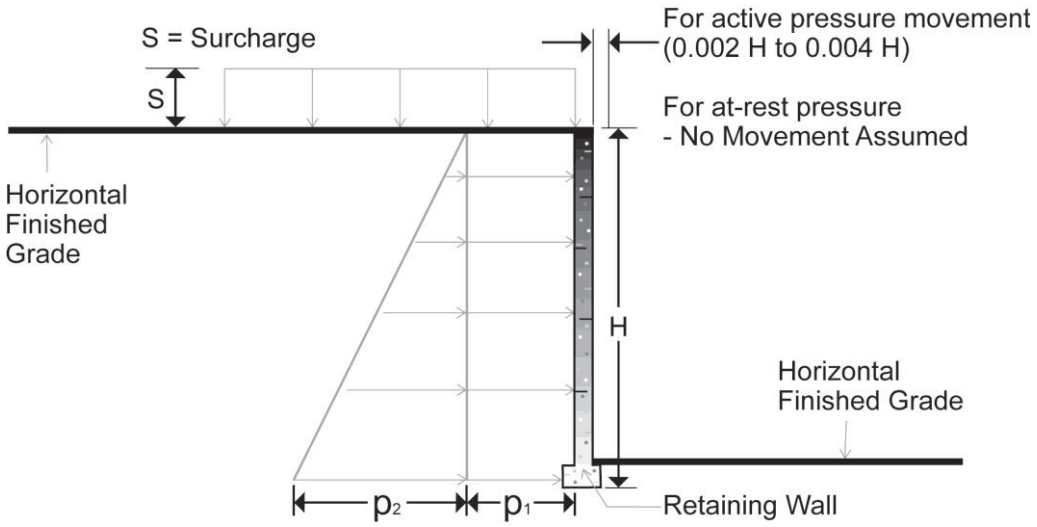
- Minimizing moisture increases in the backfill.
- Controlling moisture-density during placement of backfill.
- Positive separations and/or isolation joints should be provided between exterior slabs and the buildings to allow independent movement.
- Control joints should be provided in slabs to control the location and extent of cracking.
- Exterior slabs should not be constructed on frozen subgrade
- Other design and construction considerations, as outlined in Section 302.1R of the ACI Design Manual, are recommended.

Movements of exterior slabs-on-grade using the above technique will likely be reduced and tend to be more uniform. Additional movement could occur should the subsurface soils and bedrock become wetted to significant depths, which could result in potential excessive movement causing uneven exterior slabs and severe cracking. This could be due to over watering of landscaping, poor drainage, and/or broken utility lines. Therefore, it is imperative that the recommendations outlined in the **Grading and Drainage** subsection of **Earthwork** be followed.

Lateral Earth Pressures

Design Parameters

Reinforced concrete walls with unbalanced backfill levels on opposite sides should be designed for earth pressures at least equal to those indicated in the following table. Earth pressures will be influenced by structural design of the walls, conditions of wall restraint, methods of construction and/or compaction and the strength of the materials being restrained. Two wall restraint conditions are shown. Active earth pressure is commonly used for design of free-standing cantilever retaining walls and assumes wall movement. The "at-rest" condition assumes no wall movement. The recommended design lateral earth pressures do not include a factor of safety and do not provide for possible hydrostatic pressure on the walls.



Earth Pressure Conditions	Lateral Earth Pressure Coefficient	Equivalent Fluid Density (pcf)	Surcharge Pressure, p_1 (psf)	Earth Pressure, p_2 (psf)
Active (Ka)	0.47	60	(0.47)S	(60)H
At-Rest (Ko)	0.64	80	(0.64)S	(80)H

Earth Pressure Conditions	Lateral Earth Pressure Coefficient	Equivalent Fluid Density (pcf)	Surcharge Pressure, p_1 (psf)	Earth Pressure, p_2 (psf)
Passive (Kp)	2.1	250	---	---

Applicable conditions to the above include:

- For active earth pressure, wall must rotate about base, with top lateral movements of about 0.002 H to 0.004 H, where H is wall height
- For passive earth pressure to develop, wall must move horizontally to mobilize resistance.
- Uniform surcharge, where S is surcharge pressure
- In-situ soil backfill weight a maximum of 120 pcf
- Horizontal backfill, compacted to at least 95 percent of standard Proctor maximum dry density
- Loading from heavy compaction equipment not included
- No hydrostatic pressures acting on wall
- No dynamic loading
- No safety factor included in soil parameters

To control hydrostatic pressure behind earth-retaining walls we recommend that a drain be installed below the foundation of the wall with a collection pipe leading to a reliable discharge.

The preceding data are applicable only to cast-in-place concrete or modular block walls up to 5 feet in height. **If taller single walls, tiered walls, or Mechanically Stabilized Earth (MSE) walls will be included in the proposed development, additional site-specific studies and laboratory testing will be required.** In addition, the wall designer should perform standard wall design practices including analysis for overturning, sliding, bearing capacity, and global stability, and results of these analyses should be provided for our review. Additional sampling, laboratory testing and document review associated with retaining walls is beyond the original scope of work but can be performed as a separate scope, for a separate fee.

Retaining Wall Drainage

To reduce hydrostatic loading on retaining walls, a subsurface drain system should be placed behind the walls. The drain system should consist of free-draining granular soils containing less than 5 percent fines (by weight) passing a No. 200 sieve placed adjacent to the wall. The free-draining granular material should be graded to prevent the intrusion of fines or encapsulated in a suitable filter fabric. A drainage system consisting of either weep holes or perforated drain lines (placed near the base of the wall) should be used to intercept and discharge water which would tend to saturate the backfill.

Where used, drain lines should be embedded in a uniformly graded filter material and provided with adequate clean-outs for periodic maintenance.

Crawl Space Drainage

In order to reduce the risk of exterior surface water infiltrating into the crawl space if the structural floor option is selected, we recommend a drain be constructed along the perimeter of the crawl space. Perimeter and subfloor drains are not required if a slab-on-void floor is used for the interior floor of the buildings and the void space is 2 feet or less in height.

The drainage system should consist of a minimum 4-inch diameter perforated or slotted rigid pipe that is embedded in free-draining gravel around the perimeter of the crawl space. Gravel should extend 3 inches beneath the bottom of the pipe and at least 6 inches above top of the pipe. The gravel should be covered with filter fabric (such as a Mirafi 140N geotextile or approved equivalent). The base of the overexcavation should be crowned at the midpoint of the overexcavation and sloped to the drain at the perimeter at a minimum 2% slope.

The performance of the drain will be dependent primarily on maintaining proper slope to an appropriate outfall. We recommend the drain be sloped and maintained at a slope of at least 1/8-inch per foot of wall. The drain should be evaluated periodically using a camera to verify that no "bells" or "humps" are present that would disrupt drainage. In addition, the outfall and/or sump pump should also be observed and tested to confirm that any water that the drain collects is properly discharged away from all improvements and is not allowed to accumulate. The drain should be repaired promptly if any deficiencies are identified.

Groundwater measured during this exploration is not anticipated to enter the crawl space drain. Due to unpredictability of surface water entering the excavation drain, the amount of water and frequency of discharge from the crawl space drainage system is indeterminate.

Pavements

New pavements will likely consist of flexible asphalt and rigid concrete pavement. Design of pavements has been based on the 1993 Guideline for Design of Pavement Structures by the American Association of State Highway and Transportation Officials (AASHTO) (hereafter referred to as the AASHTO Specifications).

Design Traffic

The following traffic loads were provided by QTS for design of the light and heavy duty pavements for this project.

Pavement Area	Approximate 20-year ESAL Value
Light Duty	7,500
Heavy Duty	75,000

We should be contacted to confirm and/or modify the recommendations contained herein if actual traffic volumes differ from the assumed values shown above.

Subgrade Soils

Based on the laboratory test results, the subgrade soils beneath the proposed pavement areas are anticipated to classify as A-7-6 soils in accordance with American Association of State and Highway and Transportation Officials (AASHTO). A California Bearing Ratio (CBR) Test was performed on a composite sample prepared from samples from Boring No. P1 which resulted in a value of 3.8.

Based on the subsurface conditions, the CBR value, and to account for seasonal fluctuations, a resilient modulus (MR) of about 3,000 psi and a modulus of subgrade reaction of 90 pci were considered for the pavement design.

Recommended Minimum Pavement Sections and Materials

Recommended alternatives for flexible and rigid pavements are summarized for each traffic area as follows:

Traffic Area	Alternative	Pavement Thickness (Inches)			
		Asphalt Concrete Surface	Aggregate Base Course	Portland Cement Concrete	Total
Light Duty	A	6	--	--	6
	B	5	8	--	13
	C [1]	--	--	5	5
Heavy Duty	A	9	--	--	9
	B	6	11	--	17
	C [1]	--	--	6	6

1. The minimum pavement section thickness per ACI

Each alternative should be investigated with respect to current material availability and economic conditions. A minimum 7-inch thickness of rigid reinforced concrete pavement

is recommended at the location of dumpsters where trash trucks park and load, and in areas of tight turning radius.

Concrete pavement joint spacing and reinforcement should be in accordance with specifications in ACI 330.

For analysis of pavement costs, the following specifications should be considered for each pavement component:

Pavement Component	Colorado Department of Transportation Criteria
Asphalt Concrete Surface	Grading S or SX
Aggregate Base Course	Class 5 or 6
Portland Cement Concrete	Class P

Drainage Adjacent to Pavements

Clay subgrade materials will expand and/or lose stability with increases in moisture content. To reduce pavement distress due to wetting of the subgrade in areas of water intensive landscaping or other nearby water sources (or if aggregate base course is used) located adjacent to pavements, edge drains should be considered.

Pavement Maintenance

Future performance of pavements constructed at this site will be dependent upon several factors, including:

- Maintaining stable moisture content of the subgrade soils both before and after pavement construction.
- Providing for a planned program of preventative maintenance.

The performance of all pavements can be enhanced by minimizing excess moisture, which can reach the subgrade soils. The following recommendations should be implemented:

- Site grading at a minimum 2 percent grade onto or away from the pavements.
- Water should not be allowed to pond behind curbs.
- Compaction of any utility trenches for landscaped areas to the same criteria as the pavement subgrade.
- Sealing all landscaped areas in or adjacent to pavements or providing drains to reduce the risk of moisture migration to subgrade soils.
- Placing compacted backfill against the exterior side of curb and gutter.

- Placing curb, gutter, and/or sidewalk directly on subgrade soils without the use of base course materials.

Preventative maintenance should be planned and provided for an ongoing pavement management program in order to enhance future pavement performance. Preventative maintenance activities are intended to slow the rate of pavement deterioration.

Preventative maintenance consists of both localized maintenance (e.g. crack sealing and patching) and global maintenance (e.g. surface sealing). Preventative maintenance is usually the first priority when implementing a planned pavement maintenance program.

Pavement Construction Considerations

Site grading is generally accomplished early in the construction phase. However, as construction proceeds, the subgrade may be disturbed due to utility excavations, construction traffic, desiccation, or rainfall. As a result, the pavement subgrade may not be suitable for pavement construction and corrective action will be required. The subgrade should be carefully evaluated at the time of pavement construction for signs of disturbance or excessive rutting. If disturbance has occurred, pavement subgrade areas should be reworked, moisture conditioned, and properly compacted to the recommendations in this report immediately prior to paving.

We recommend the pavement areas be rough graded and then thoroughly proofrolled with a loaded tandem axle dump truck prior to final grading and paving. Particular attention should be paid to high traffic areas that were rutted and disturbed earlier and to areas where backfilled trenches are located. Areas where unsuitable conditions are located should be repaired by removing and replacing the materials with properly compacted fills. All pavement areas should be moisture conditioned and properly compacted to the recommendations in this report immediately prior to paving.

The placement of a partial pavement thickness for use during construction is not recommended without a detailed pavement analysis incorporating construction traffic. In addition, if the actual traffic varies from the assumptions outlined above, we should be contacted to confirm and/or modify the pavement thickness recommendations outlined above.

General Comments

Our analysis and opinions are based upon our understanding of the project, the geotechnical conditions in the area, and the data obtained from our site exploration. Variations will occur between exploration point locations or due to the modifying effects of construction or weather. The nature and extent of such variations may not become evident until during or after construction. Terracon should be retained as the

Geotechnical Engineer, where noted in this report, to provide observation and testing services during pertinent construction phases. If variations appear, we can provide further evaluation and supplemental recommendations. If variations are noted in the absence of our observation and testing services on-site, we should be immediately notified so that we can provide evaluation and supplemental recommendations.

Our Scope of Services does not include either specifically or by implication any environmental or biological (e.g., mold, fungi, bacteria) assessment of the site or identification or prevention of pollutants, hazardous materials or conditions. If the owner is concerned about the potential for such contamination or pollution, other studies should be undertaken.

Our services and any correspondence are intended for the sole benefit and exclusive use of our client for specific application to the project discussed and are accomplished in accordance with generally accepted geotechnical engineering practices with no third-party beneficiaries intended. Any third-party access to services or correspondence is solely for information purposes to support the services provided by Terracon to our client. Reliance upon the services and any work product is limited to our client and is not intended for third parties. Any use or reliance of the provided information by third parties is done solely at their own risk. No warranties, either express or implied, are intended or made.

Site characteristics as provided are for design purposes and not to estimate excavation cost. Any use of our report in that regard is done at the sole risk of the excavating cost estimator as there may be variations on the site that are not apparent in the data that could significantly affect excavation cost. Any parties charged with estimating excavation costs should seek their own site characterization for specific purposes to obtain the specific level of detail necessary for costing. Site safety and cost estimating including excavation support and dewatering requirements/design are the responsibility of others. Construction and site development have the potential to affect adjacent properties. Such impacts can include damages due to vibration, modification of groundwater/surface water flow during construction, foundation movement due to undermining or subsidence from excavation, as well as noise or air quality concerns. Evaluation of these items on nearby properties are commonly associated with contractor means and methods and are not addressed in this report. The owner and contractor should consider a preconstruction/precondition survey of surrounding development. If changes in the nature, design, or location of the project are planned, our conclusions and recommendations shall not be considered valid unless we review the changes and either verify or modify our conclusions in writing.

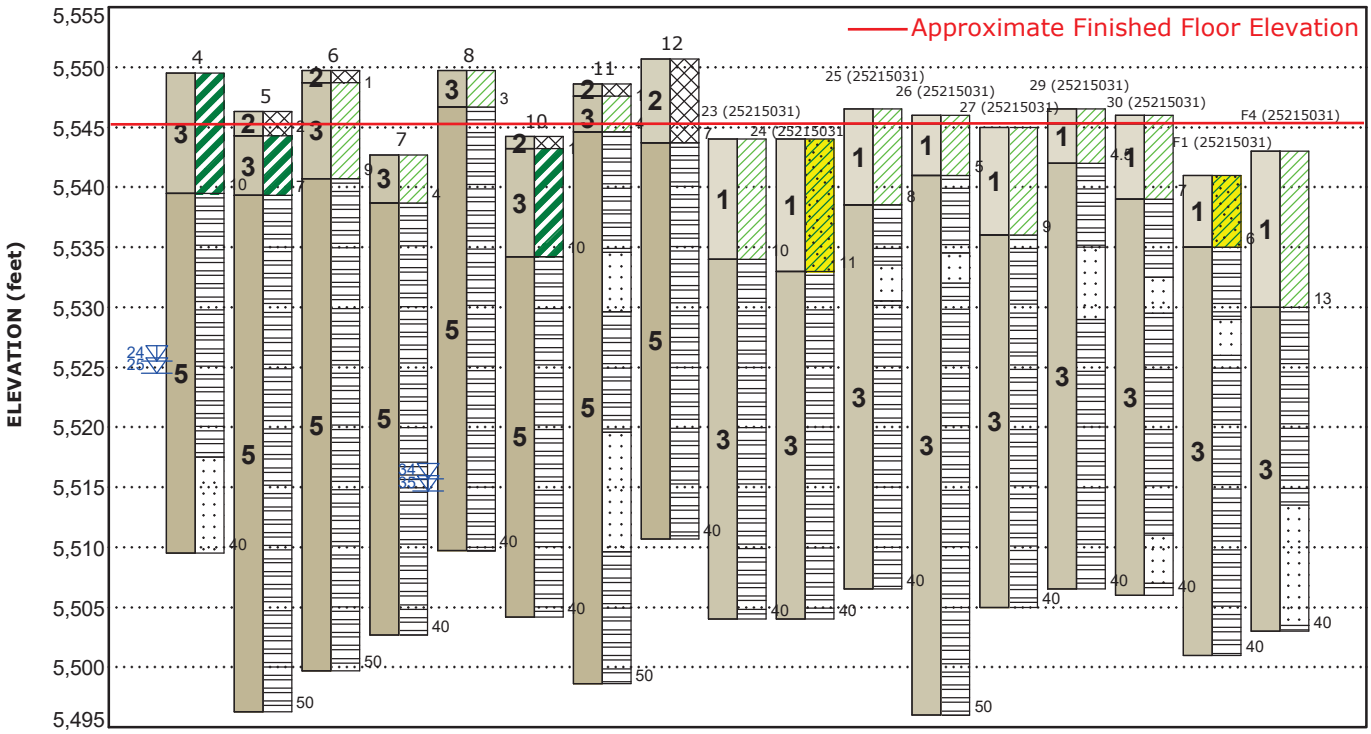
Figures

Contents:

GeoModel (4 pages)

Terracon
10625 W I70 Frontage Rd N
Ste 3
Wheat Ridge, CO

GeoModel - DC2



This is not a cross section. This is intended to display the Geotechnical Model only. See individual logs for more detailed conditions.

Model Layer	Layer Name	General Description
1	Asphalt	Asphalt; about 3 inches
2	Fill	Fill material consisting of lean to fat clay with varying amounts of sand, silt and gravel; various stiffnesses
3	Native Clay	Native lean to fat clay with varying amounts of sand and silt; medium stiff to very stiff
4	Native Sand	Native sand with varying amounts of clay and silt; medium dense
5	Bedrock	Bedrock consisting of claystone to sandstone; soft to very hard

Legend

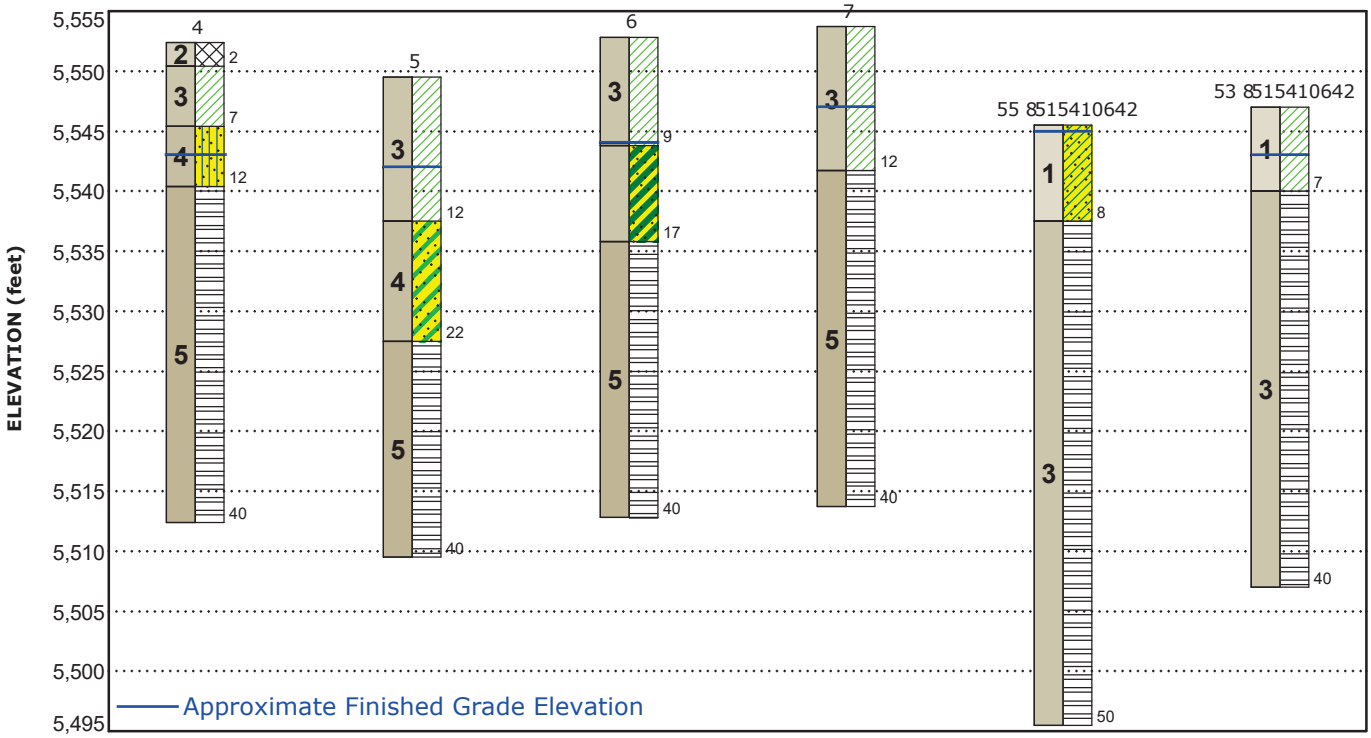
- Fat Clay
- Claystone
- Sandstone
- Fill
- Lean Clay
- Sandy Lean Clay

First Water Observation
Second Water Observation

Groundwater levels are temporal. The levels shown are representative of the date and time of our exploration. Significant changes are possible over time.
Water levels shown are as measured during and/or after drilling. In some cases, boring advancement methods mask the presence/absence of groundwater. See individual logs for details.

NOTES:
Layering shown on this figure has been developed by the geotechnical engineer for purposes of modeling the subsurface conditions as required for the subsequent geotechnical engineering for this project.
Numbers adjacent to soil column indicate depth below ground surface.

GeoModel - DC2 Site



()9F 9F Thi s nchFF Ftni9hTa()9F 9F 9TitTctc ih c9Fres. i)t dthitn)T9nsephctehTe al tt 9Tc9/9tGseehMF Shov hot ctis9tc nhTc99hTFa

Model Layer	Layer Name	General Description
1	Asphalt	CFr)sei; sf hGi 6 9Tn)tF
2	Fill	I9ee v sito9senhTF9Fi9TMhSetsT ih Ssi nes. w9) yso. 9TM sv hGTiF hSFsTcbF9e sTc Mosyte, ysoehGF Fi9SStFFtF
3	Native Clay	usi9yt etsT ih Ssi nes. w9) yso. 9TMsv hGTiF hSFsTc sTc F9e; v tc9Gv Fi9SSih yto. Fi9SS
4	Native Sand	usi9yt FsTc w9) yso. 9TMsv hGTiF hSnes. sTc F9e; v tc9Gv ctTFt
5	Bedrock	Btcohnk nhTF9Fi9TMhSnes. FihTt ih FsTcFihTt; FhSi ih yto.)soc

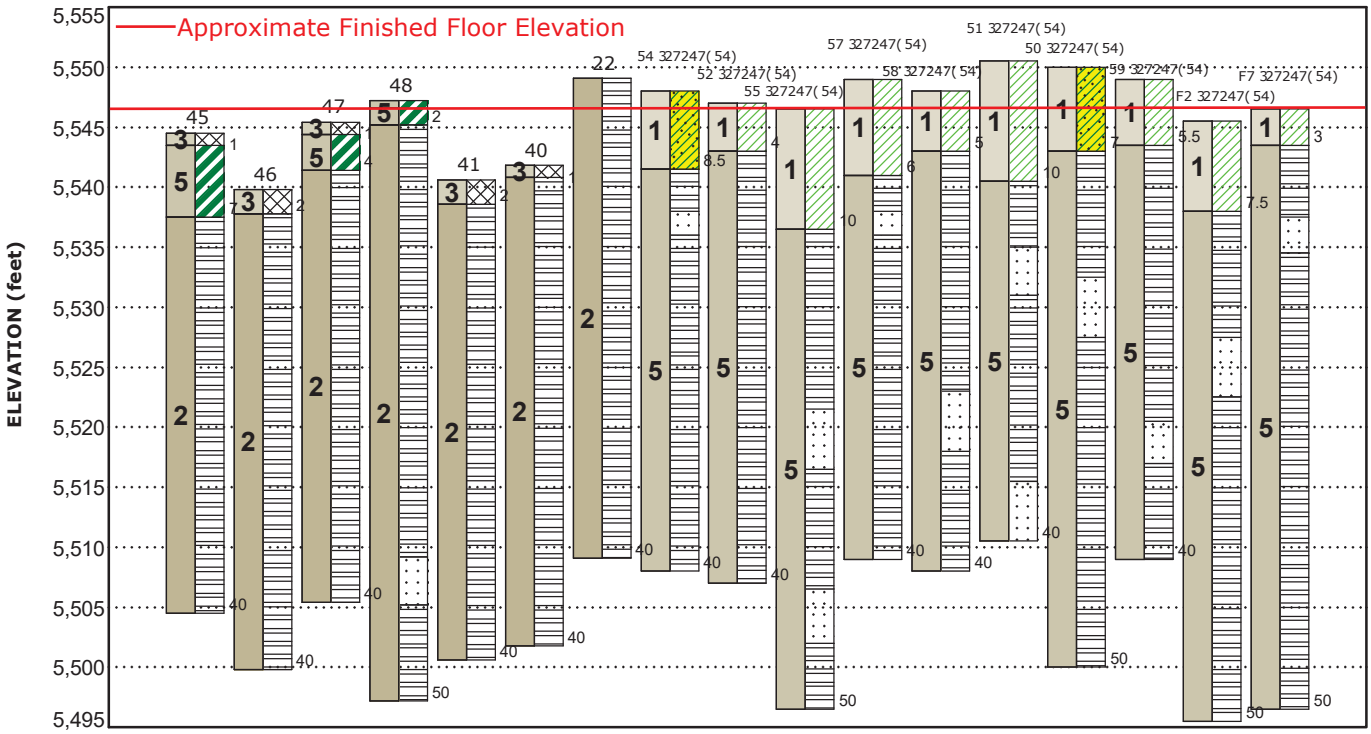
Legend	
I9ee	LtsT ges.
l 9e. l sTc	ges. FihTt
ges. t. l sTc	l sTc. Isi ges.
l sTc. LtsT ges.	

I9eFi D sito Of Ftoysi9hT
l tnhTc D sito Of Ftoysi9hT

dohGTcwsitoetyteF sot itv rhosea()t etyteF F)hwT sot otrotFtTisi9yt hSi)t csit sTc i9v t hShGotxrehosi9hTal 9MT99hsTi n)sTMtF sot rhFF9fet hytoi9v ta
D sitoetyteF F)hwT sot sF v tsFGotc cGo9TMsTc/hosStocoee9TMa-T Fhv t nsFtFbfho9TMscysTntv tTi v ti)hcF v sFk i)t rotFtTnt/sf FtTnt hSMohGTcwsitoalt t 9Tc9/9tGseehMF Shoctis9eFa

NOTES:
Layering shown on this figure has been developed by the geotechnical engineer for purposes of modeling the subsurface conditions as required for the subsequent geotechnical engineering for this project.
Numbers adjacent to soil column indicate depth below ground surface.

GeoModel - DC3



This is not a cross section. This is intended to display the Geotechnical Model only. See individual logs for more detailed conditions.

Model Layer	Layer Name	General Description
1	**Asphalt**	Asphalt; about 5 inches
2	**Fill**	Fill material consisting of lean to fat clay with varying amounts of sand, silt and gravel; various stiffnesses
3	**Native Clay**	Native lean to fat clay with varying amounts of sand and silt; medium stiff to very stiff
4	**Native Sand**	Native sand with varying amounts of clay and silt; medium dense
5	**Bedrock**	Bedrock consisting of claystone to sandstone; soft to very hard

Legend	
Fill	Fat Clay
Claystone	Sandstone
Sandy Lean Clay	Lean Clay

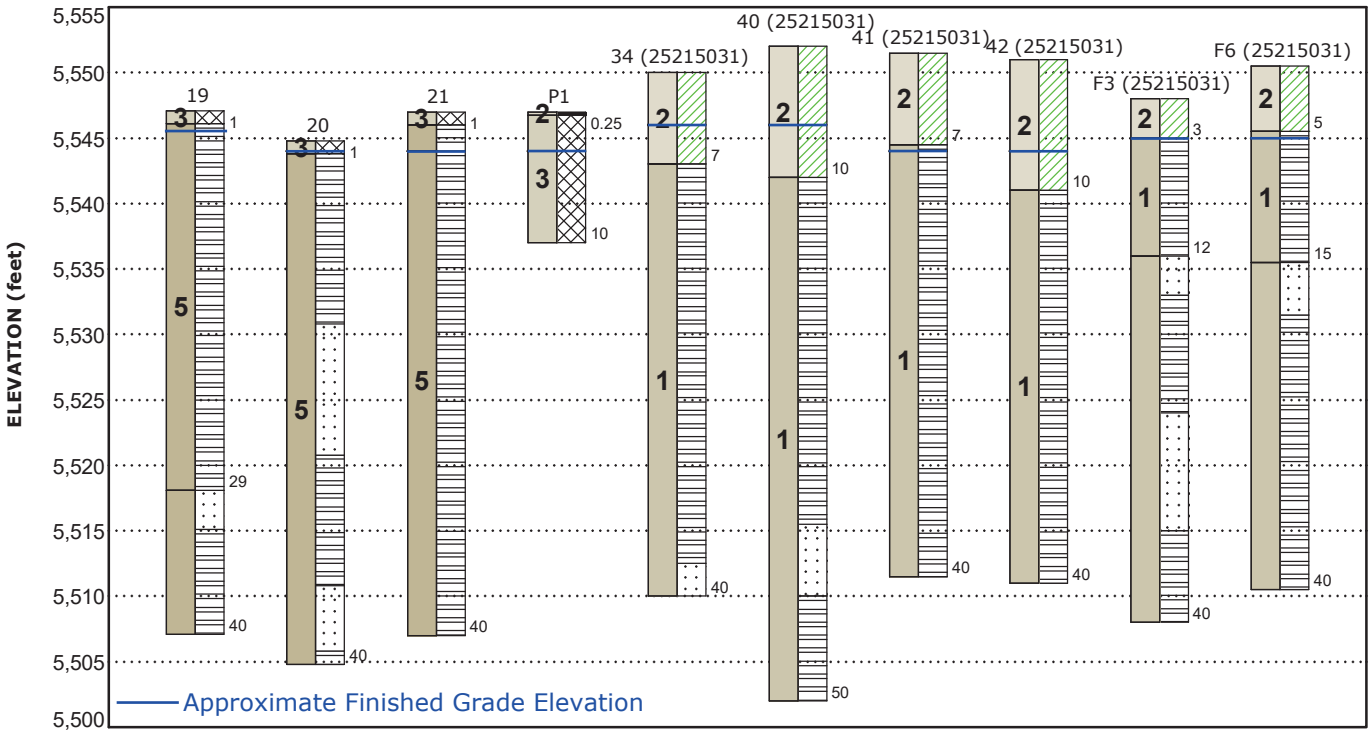
▽ First Water Observation
▽ Second Water Observation

Groundwater levels are temporal. The levels shown are representative of the date and time of our exploration. Significant changes are possible over time.
Water levels shown are as measured during and/or after drilling. In some cases, boring advancement methods mask the presence/absence of groundwater. See individual logs for details.

NOTES:
Layering shown on this figure has been developed by the geotechnical engineer for purposes of modeling the subsurface conditions as required for the subsequent geotechnical engineering for this project.
Numbers adjacent to soil column indicate depth below ground surface.

Terracon
10625 W I70 Frontage Rd N
Ste 3
Wheat Ridge, CO

GeoModel - DC3 Site



This is not a cross section. This is intended to display the Geotechnical Model only. See individual logs for more detailed conditions.

Model Layer	Layer Name	General Description
1	**Asphalt**	Asphalt; about 3 inches
2	**Fill**	Fill material consisting of lean to fat clay with varying amounts of sand, silt and gravel; various stiffnesses
3	**Native Clay**	Native lean to fat clay with varying amounts of sand and silt; medium stiff to very stiff
4	**Native Sand**	Native sand with varying amounts of clay and silt; medium dense
5	**Bedrock**	Bedrock consisting of claystone to sandstone; soft to very hard

Legend

Fill
Claystone
Sandstone
Asphalt
Lean Clay

First Water Observation
Second Water Observation

Groundwater levels are temporal. The levels shown are representative of the date and time of our exploration. Significant changes are possible over time.
Water levels shown are as measured during and/or after drilling. In some cases, boring advancement methods mask the presence/absence of groundwater. See individual logs for details.

NOTES:
Layering shown on this figure has been developed by the geotechnical engineer for purposes of modeling the subsurface conditions as required for the subsequent geotechnical engineering for this project.
Numbers adjacent to soil column indicate depth below ground surface.

Attachments

Exploration and Testing Procedures

Field Exploration

Boring Layout and Elevations: The locations of the borings are presented in the **Site Location and Exploration Plans**. The borings locations were obtained using a handheld, recreational-grade GPS unit. The accuracy of the latitude and longitude values is typically about +/- 25 feet when obtaining the values using this method. The relative elevations at the boring locations were obtained using a level and using an electric utility cover (approximately 39.7321° N, 104.7116° W) as a temporary benchmark with an estimated elevation of 5,549 feet based on the provided topographic survey of the site. The accuracy of the boring locations and elevations should only be assumed to the level implied by the methods used.

Subsurface Exploration Procedures: The borings were drilled with a Mobile B-57 and CME-55 truck-mounted rotary drill rigs with solid-stem augers. During the drilling operations, lithologic logs of the borings were recorded by the field engineer. Relatively undisturbed samples were obtained at selected intervals utilizing a 2½-inch outside diameter modified California barrel sampler. Bulk samples were obtained from auger cuttings. Penetration resistance values were recorded in a manner similar to the standard penetration test (SPT). This test consists of driving the sampler into the ground with a 140-pound hammer free falling through a distance of 30 inches. The number of blows required to advance the barrel sampler 12 inches (18 inches for standard split-spoon samplers, final 12 inches are recorded) or the interval indicated is recorded and can be correlated to the standard penetration resistance value (N-value). The blow count values are indicated on the boring logs at the respective sample depths, barrel sampler blow counts are not considered N-values.

An automatic hammer was used to advance the samplers in the borings performed on this site. A greater efficiency is typically achieved with the automatic hammer compared to the conventional safety hammer operated with a cathead and rope. Published correlations between the SPT values and soil properties are based on the lower efficiency cathead and rope method. This higher efficiency affects the standard penetration resistance blow count value by increasing the penetration per hammer blow over what would be obtained using the cathead and rope method. The effect of the automatic hammer's efficiency has been considered in the interpretation and analysis of the subsurface information for this report.

The standard penetration test provides a reasonable indication of the in-place density of sandy type materials, but only provides an indication of the relative stiffness of cohesive materials since the blow count in these soils may be affected by the soil moisture content. In addition, considerable care should be exercised in interpreting the N-values in gravelly soils, particularly where the size of the gravel particle exceeds the inside diameter of the sampler.

Groundwater measurements were obtained in the borings at the time of drilling. Due to safety concerns, the borings were backfilled with auger cuttings after drilling. Some settlement of the backfill may occur and should be repaired as soon as possible.

Laboratory Testing

Samples retrieved during the field exploration were returned to the laboratory for observation by the Geotechnical Engineer and were classified in general accordance with the Unified Soil Classification System presented in the **Supporting Information**.

At this time, an applicable laboratory-testing program was formulated to determine engineering properties of the subsurface materials. Following the completion of the laboratory testing, the field descriptions were confirmed or modified as necessary, and the boring logs were prepared. The boring logs are included in the **Exploration Results**.

Laboratory test results are included in the **Exploration Results**. These results were used for the geotechnical engineering analyses and the development of foundation, earthwork, and pavement recommendations. All laboratory tests were performed in general accordance with the applicable local or other accepted standards.

Selected soil and bedrock samples were tested for the following engineering properties:

- Water content
- Dry density
- Grain size distribution
- Atterberg limits
- Swell/consolidation
- Remolded swell/consolidation
- Water-soluble sulfate content
- Sulfide
- Chloride
- Red-Ox
- pH
- Electrical resistivity
- Thermal Resistivity
- Moisture/density relationship (Proctor)
- California Bearing Ratio

Site Location and Exploration Plans

Contents:

Site Location Plan
Exploration Plan with Aerial Image
Exploration Plan with Project Overlay
Electrical Resistivity Testing Plan

Note: All attachments are one page unless noted above.

Site Location



DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

MAP PROVIDED BY MICROSOFT BING MAPS

Permit #25-2535171-LT
RSN 1903242

Exploration Plan with Aerial Image

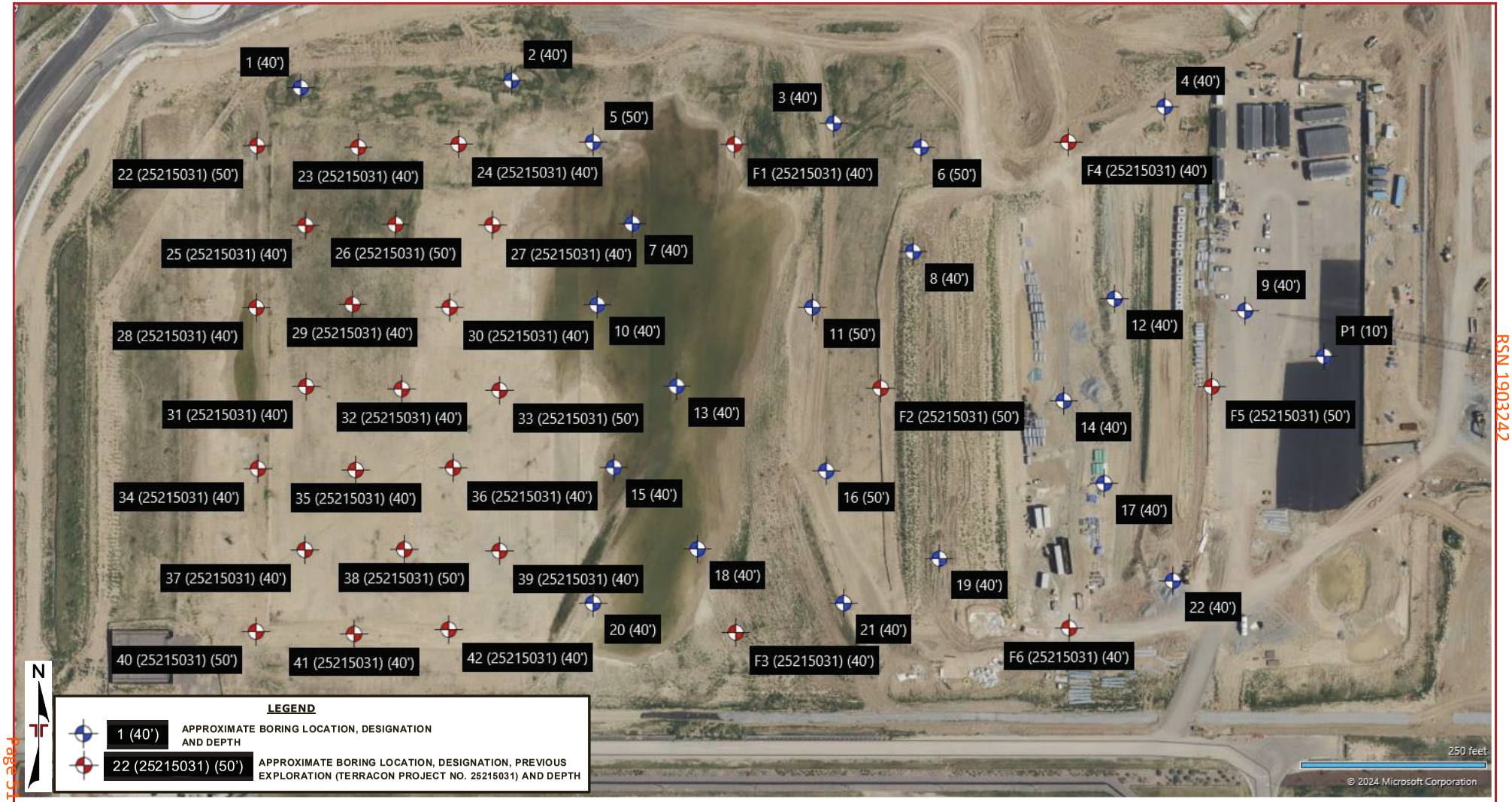


DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

MAP PROVIDED BY MICROSOFT BING MAPS

Permit #25-2535171-LT
RSN 1903242

Exploration Plan with Project Overlay

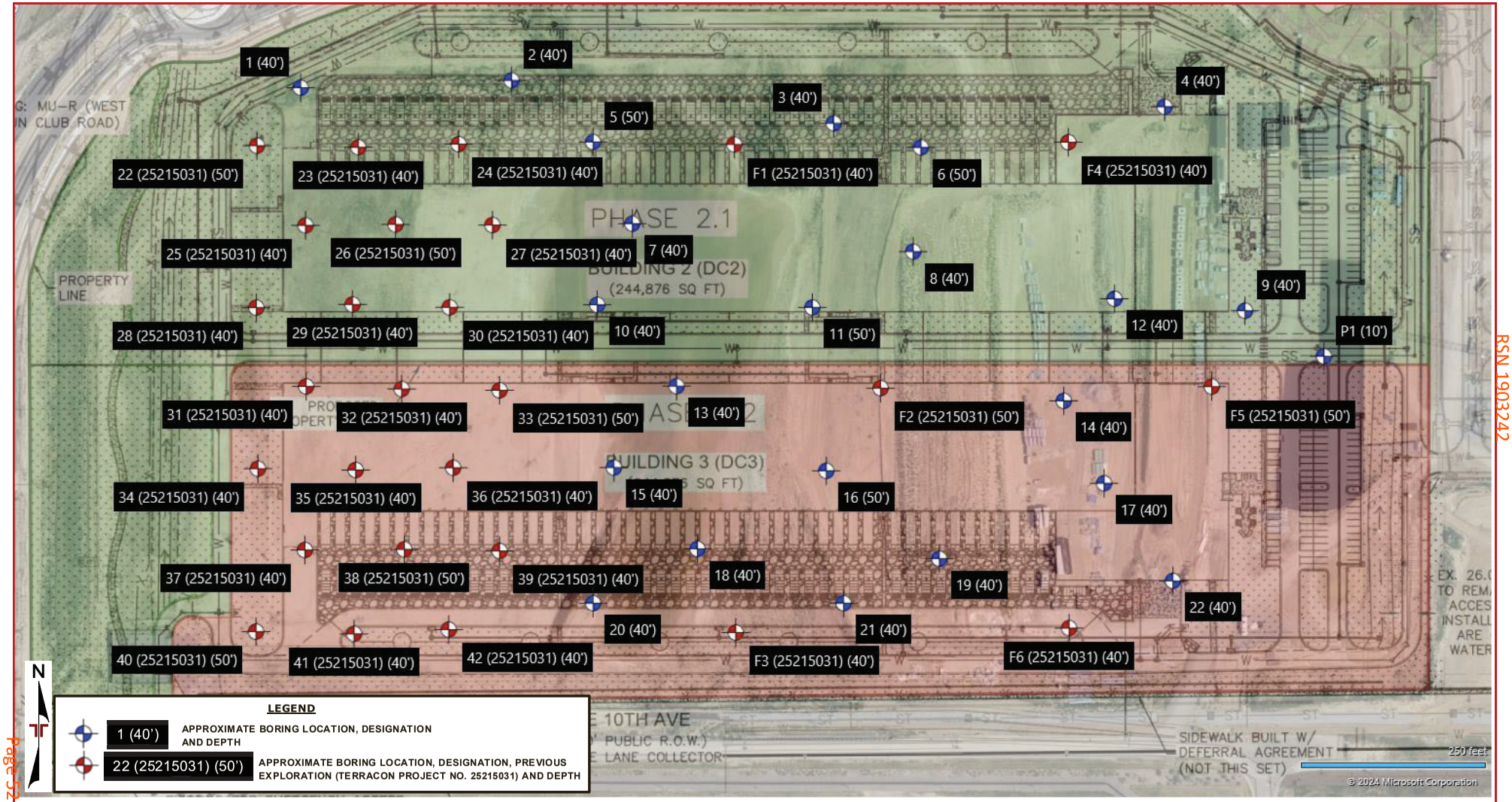
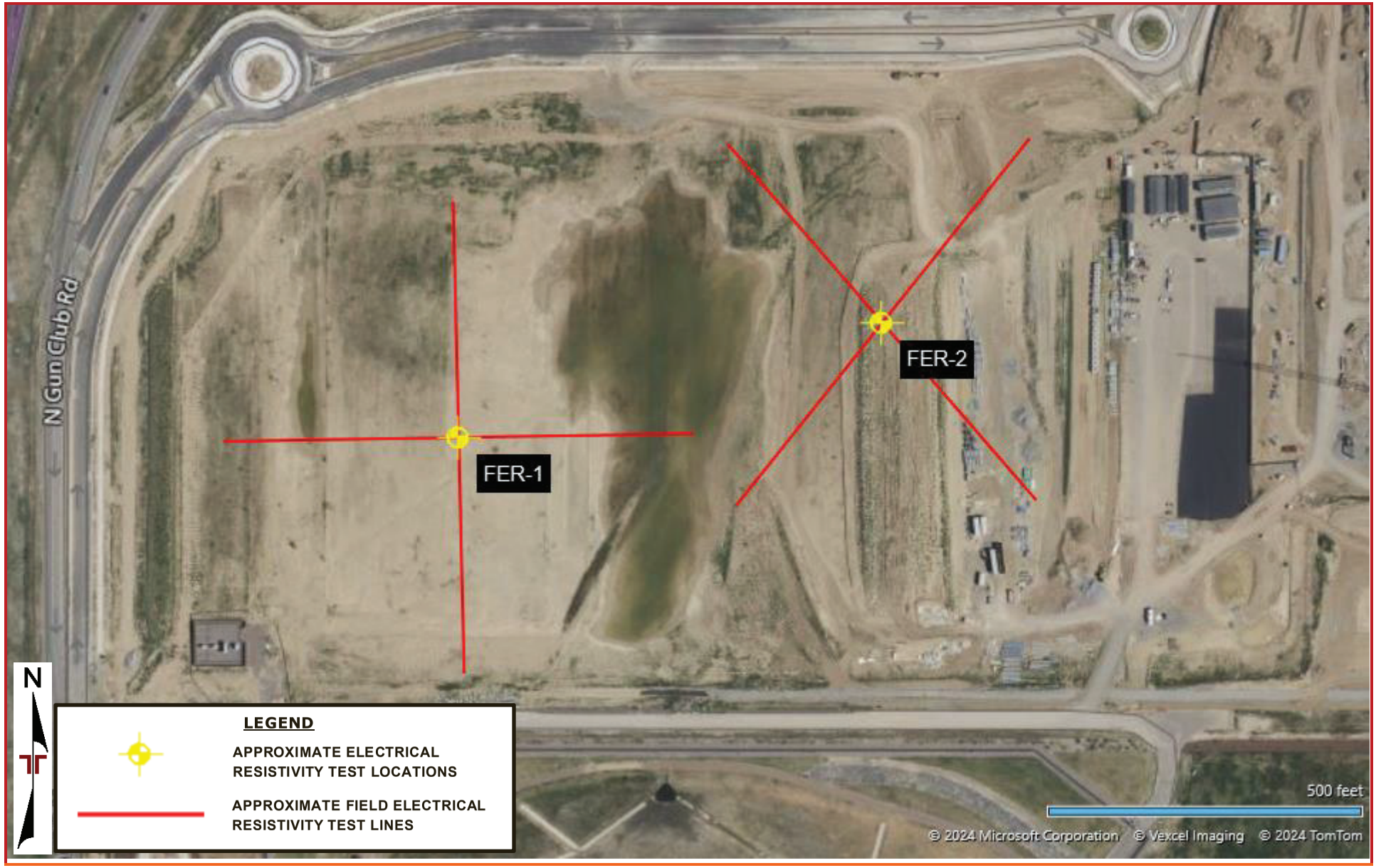


DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

MAP PROVIDED BY MICROSOFT BING MAPS

Permit #25-2535171-LT
RSN 1903242

Electrical Resistivity Testing Plan



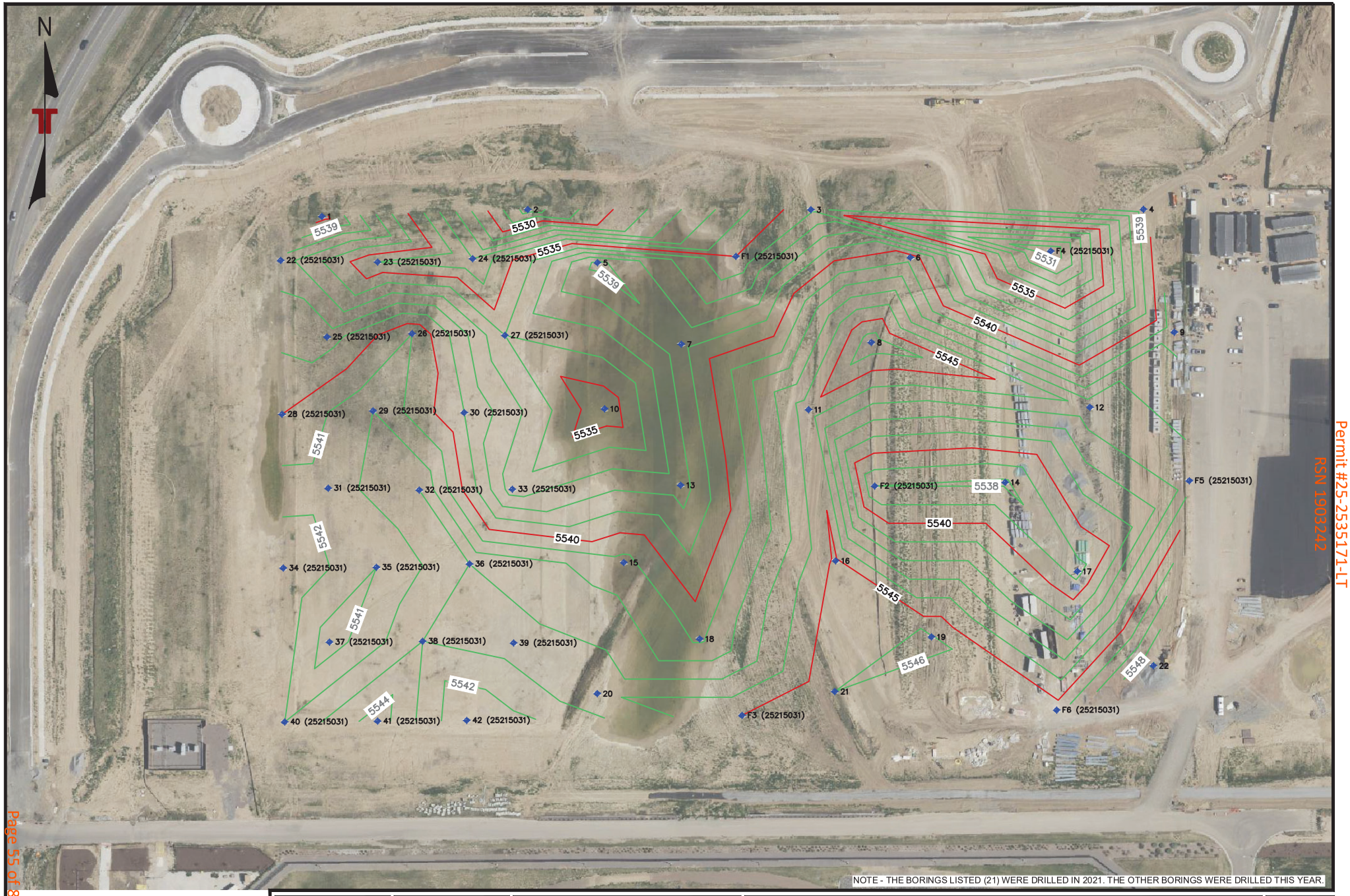
Permit #25-2535171-LT
RSN 1903242

Exploration and Laboratory Results

Contents:

Bedrock Contour Map
Boring Logs (Boring Nos. 1 to 22, and P1)
One-Dimensional Swell or Collapse (37 pages)
Grain Size Distribution (9 pages)
Moisture Density Relationship (4 pages)
California Bearing Ratio (CBR)
Thermal Resistivity (3 pages)
Corrosivity (3 pages)
Summary of Laboratory Test Results (5 pages)

Note: All attachments are one page unless noted above.



NOTE - THE BORINGS LISTED (21) WERE DRILLED IN 2021. THE OTHER BORINGS WERE DRILLED THIS YEAR.

180 0 45 90 180

SCALE IN FEET

Project Mngr:	TM
Drawn By:	AP
Checked By:	TM
Approved By:	TM

Project No.	25245150
Scale:	AS SHOWN
File No.	Exhibit 1
Date:	August 2024

Terracon
Explore with us
21905 64TH AVENUE W, SUITE 100 MOUNTLAKE TERRACE, WA 98043
PH. (425) 771-3304 FAX. (425) 771-3549

BEDROCK CONTOUR MAP

QTS Data Center - DC2 AND DC3
1120 N Gun Club Road
Aurora, Colorado

EXHIBIT 1

Permit #25-2535171-LT
RSN 1903242

Boring Log No. 1

Location: See Exploration Plan

Latitude: 39.7342° Longitude: -104.7144°

Depth (Ft.) | Elevation: 5552.4 (Ft.) +/-

Model Layer	Graphic Log	Depth (Ft.)	Water Level Observations	Sample Type	Field Test Results	Swell (%)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL-PL-PI	Percent Fines
2		**FILL - LEAN CLAY (CL)**, with sand, brown 2.0 — 5550.4								
3		**LEAN CLAY (CL)**, with sand, brown, very stiff			8-12		9.0		44-16-28	80
					7-12		10.1	97		
		7.0 — 5545.4								
4		**SILTY SAND (SM)**, fine to coarse grained, brown to gray, medium dense 9.4 — 5543								
		Approximate Finished Grade Elevation: 5543 feet			18-25		7.7	126	NP	19
		12.0 — 5540.4								
5		**CLAYSTONE**, brown to gray, soft to firm			8-15	+3.9 @ 500 psf	27.4	95		
					11-22		23.2	108		
					11-22		23.9	103		
					10-20		25.0	101		
					12-23		20.0			
		40.0 — 5512.4			13-22		29.4	93		
		Boring Terminated at 40 Feet								

Depth (Ft.): 5, 10, 15, 20, 25, 30, 35, 40

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Water Level Observations
None encountered while drilling

Drill Rig
CME-55

Hammer Type
Automatic

Driller
Terracon

Logged by
JL

Boring Started
07-19-2024

Boring Completed
07-19-2024

Notes
Elevation was obtained using a level and the top of the manhole cover southeast of Boring No. 21 (39.7321° N 104.7116° W) with a known elevation of 5549 feet.

Advancement Method
4-inch diameter, solid-stem, continous-flight power auger

Abandonment Method
Boring backfilled with auger cuttings upon completion.

Boring Log No. 2

Location: See Exploration Plan

Latitude: 39.7342° Longitude: -104.7134°

Depth (Ft.) Elevation: 5549.5 (Ft.) +/-

Model Layer	Graphic Log	Depth (Ft.)	Water Level Observations	Sample Type	Field Test Results	Swell (%)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL-PL-PI	Percent Fines
3		**LEAN CLAY (CL)**, with sand, brown, medium stiff to very stiff			7-11	+3.8 @ 500 psf	13.6	110	42-14-28	78
					8-17		10.5	104		
		7.5 5542								
		Approximate Finished Grade Elevation: 5542 feet			4-6	+0.4 @ 500 psf	19.3	108		
		12.0 5537.5								
4		**CLAYEY SAND (SC)**, fine to coarse grained, brown, medium dense			10-19		19.1	102	37-20-17	34
					14-26		15.5	115		
		22.0 5527.5								
5		**CLAYSTONE**, brown to gray, firm to medium hard			12-20		25.1	94		
					10-16		23.5	101		
					15-25		23.7	97		
		40.0 5509.5			17-28		23.8	104		
		Boring Terminated at 40 Feet								

Depth (Ft.): 5, 10, 15, 20, 25, 30, 35, 40

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Water Level Observations
None encountered while drilling

Drill Rig
CME-55

Hammer Type
Automatic

Driller
Terracon

Logged by
TPS

Boring Started
07-25-2024

Boring Completed
07-25-2024

Notes
Elevation was obtained using a level and the top of the manhole cover southeast of Boring No. 21 (39.7321° N 104.7116° W) with a known elevation of 5549 feet.

Advancement Method
4-inch diameter, solid-stem, continous-flight power auger

Abandonment Method
Boring backfilled with auger cuttings upon completion.

Boring Log No. 3

Location: See Exploration Plan

Latitude: 39.7341° Longitude: -104.7118°

Depth (Ft.) Elevation: 5552.8 (Ft.) +/-

Model Layer	Graphic Log	Depth (Ft.)	Water Level Observations	Sample Type	Field Test Results	Swell (%)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL-PL-PI	Percent Fines
3		**LEAN CLAY (CL)**, with sand, brown, medium stiff to stiff			4-5	+3.1 @ 500 psf	15.4	99	49-14-35	83
					4-7		12.1	109		
		8.8 5544								
		9.0 Approximate Finished Grade Elevation: 5544 feet 5543.8								
		SANDY FAT CLAY (CH), brown, stiff to very stiff			5-6		20.2	100		
					8-15	+1.2 @ 500 psf	19.5	108	50-18-32	50
		17.0 5535.8								
5		**CLAYSTONE**, trace sand, gray to brown, firm to medium hard			12-20		19.6	103		
					13-26		22.3	108		
					9-13		23.3	99		
					20-32		22.3	105		
		40.0 5512.8			13-21		21.4	97		
		Boring Terminated at 40 Feet								

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Water Level Observations
None encountered while drilling

Drill Rig
CME-55

Hammer Type
Automatic

Driller
Terracon

Logged by
TPS

Boring Started
07-25-2024

Boring Completed
07-25-2024

Notes
Elevation was obtained using a level and the top of the manhole cover southeast of Boring No. 21 (39.7321° N 104.7116° W) with a known elevation of 5549 feet.

Advancement Method
4-inch diameter, solid-stem, continous-flight power auger

Abandonment Method
Boring backfilled with auger cuttings upon completion.

Boring Log No. 4

Location: See Exploration Plan

Latitude: 39.7341° Longitude: -104.7103°

Depth (Ft.) Elevation: 5549.5 (Ft.) +/-

Model Layer	Graphic Log	Depth (Ft.)	Water Level Observations	Sample Type	Field Test Results	Swell (%)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL-PL-PI	Percent Fines
		FAT CLAY (CH), with sand, brown to gray, stiff to very stiff			7-14	+1.9 @ 500 psf	13.5	102	57-16-41	82
		4.2 5545.3								
3		Approximate Finished Floor Elevation: 5545.3 feet			7-8		14.7	113		
		10.0 5539.5			5-8		18.7	103		
5		**CLAYSTONE**, brown to gray, soft to firm			9-17		23.2	102		
					10-16		20.5	101	48-20-28	77
			▽		11-22		23.7	103		
					7-11		27.3	93		
		32.0 5517.5								
		CLAYEY SANDSTONE, fine to coarse grained, brown to gray, firm to hard			27-44		16.1	115	37-21-16	38
		40.0 5509.5			12-15		20.9	96		
		Boring Terminated at 40 Feet								

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Water Level Observations
▽ 25 feet while drilling
▽ 24 feet upon completion of drilling

Drill Rig
CME-55

Hammer Type
Automatic

Driller
Terracon

Logged by
TPS

Boring Started
07-16-2024

Boring Completed
07-16-2024

Notes
Elevation was obtained using a level and the top of the manhole cover southeast of Boring No. 21 (39.7321° N 104.7116° W) with a known elevation of 5549 feet.

Advancement Method
4-inch diameter, solid-stem, continous-flight power auger

Abandonment Method
Boring backfilled with auger cuttings upon completion.

Boring Log No. 5

Location: See Exploration Plan

Latitude: 39.7340° Longitude: -104.7130°

Depth (Ft.) | Elevation: 5546.3 (Ft.) +/-

Model Layer	Graphic Log	Depth (Ft.)	Description	Elevation (Ft.)	Depth (Ft.)	Water Level Observations	Sample Type	Field Test Results	Swell (%)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL-PL-PI	Percent Fines
2		1.0	**FILL - SILTY CLAY (CL-ML)**, trace sand, brown	5545.3									
		2.0	Approximate Finished Floor Elevation: 5545.3 feet	5544.3									
3			**FAT CLAY (CH)**, with sand, brown to gray, medium stiff					4-5		11.9			
								4-5	+0.9 @ 500 psf	5.6	119	63-18-45	72
		7.0		5539.3	5								
5			**CLAYSTONE**, brown to gray, soft to medium hard					9-14	+3.4 @ 500 psf	22.9	105		
					10								
								10-15		26.5	100		
					15								
								12-25		23.8	104		
					20								
								13-20		22.5	106		
					25								
								13-24		22.5	103		
					30								
								15-26		20.4	112		
					35								
								18-26		28.1	97		
					40								
								15-28		20.2	110		
					45								
		50.0		5496.3				17-30		18.9	111		
			Boring Terminated at 50 Feet		50								

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Water Level Observations
None encountered while drilling

Drill Rig
CME-55

Hammer Type
Automatic

Driller
Terracon

Logged by
JL

Boring Started
07-19-2024

Boring Completed
07-19-2024

Notes
Elevation was obtained using a level and the top of the manhole cover southeast of Boring No. 21 (39.7321° N 104.7116° W) with a known elevation of 5549 feet.

Advancement Method
4-inch diameter, solid-stem, continous-flight power auger

Abandonment Method
Boring backfilled with auger cuttings upon completion.

Boring Log No. 6

Location: See Exploration Plan

Latitude: 39.7340° Longitude: -104.7114°

Depth (Ft.) Elevation: 5549.7 (Ft.) +/-

Model Layer	Graphic Log	Depth (Ft.)	Description	Elevation (Ft.)	Depth (Ft.)	Water Level Observations	Sample Type	Field Test Results	Swell (%)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL-PL-PI	Percent Fines
2		1.0	**FILL - LEAN CLAY (CL)**, with sand, brown	5548.7									
			LEAN CLAY (CL), with sand, brown, stiff					6-8		10.0	118	41-17-24	84
		4.4		5545.3				5-10		12.5	107	49-14-35	85
3			Approximate Finished Floor Elevation: 5545.3 feet		5								
		9.0		5540.7									
5			**CLAYSTONE**, gray, firm to hard		10			10-17	+3.7 @ 500 psf	20.7	100	59-19-40	95
					15			50/9"		14.4	122		
					20			14-22		25.6	99		
					25			15-23		24.3	92		
					30			18-31		23.8	105		
					35			15-33		23.6	101		
					40			16-31		18.9	108		
					45			15-25		27.5	96		
		50.0		5499.7	50			50/10"		17.2	115		
			Boring Terminated at 50 Feet										

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Water Level Observations
None encountered while drilling

Drill Rig
CME-55

Hammer Type
Automatic

Driller
Terracon

Logged by
BA

Boring Started
07-25-2024

Boring Completed
07-25-2024

Notes
Elevation was obtained using a level and the top of the manhole cover southeast of Boring No. 21 (39.7321° N 104.7116° W) with a known elevation of 5549 feet.

Advancement Method
4-inch diameter, solid-stem, continous-flight power auger

Abandonment Method
Boring backfilled with auger cuttings upon completion.

Boring Log No. 7

Location: See Exploration Plan

Latitude: 39.7337° Longitude: -104.7128°

Approximate Finished Floor Elevation: 5545.3 Ft.

Depth (Ft.) Elevation: 5542.7 (Ft.) +/-

Model Layer	Graphic Log	Depth (Ft.)	Water Level Observations	Sample Type	Field Test Results	Swell (%)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL-PL-PI	Percent Fines
3		**LEAN CLAY (CL)**, with sand, grayish brown, stiff								
		4.0 5538.7			5-8	+2.3 @ 500 psf	24.6	97	46-15-31	73
		CLAYSTONE, brown to gray, firm to hard			16-50/4"		9.8	122	30-20-10	32
		5			16-21		24.8	100		
		10			11-26	+3.8 @ 500 psf	23.0	98	72-23-49	97
		15			12-18		21.4	102		
5		20			12-19		20.9	108		
		25			19-32		20.0	100		
		30			22-42		26.2	98		
		35			18-30		15.0	100		
		40.0 5502.7								
		Boring Terminated at 40 Feet								

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Water Level Observations
None encountered while drilling

Drill Rig
CME-55

Hammer Type
Automatic

Driller
Terracon

Logged by
TPS

Boring Started
07-25-2024

Boring Completed
07-25-2024

Notes
Elevation was obtained using a level and the top of the manhole cover southeast of Boring No. 21 (39.7321° N 104.7116° W) with a known elevation of 5549 feet.

Advancement Method
4-inch diameter, solid-stem, continous-flight power auger

Abandonment Method
Boring backfilled with auger cuttings upon completion.

Boring Log No. 8

Location: See Exploration Plan

Latitude: 39.7336° Longitude: -104.7115°

Depth (Ft.) | Elevation: 5549.7 (Ft.) +/-

Model Layer	Graphic Log	Depth (Ft.)	Water Level Observations	Sample Type	Field Test Results	Swell (%)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL-PL-PI	Percent Fines
3		**LEAN CLAY (CL)**, with sand, brown, very stiff								
		3.0 — 5546.7			9-12	+2.9 @ 500 psf	16.9	112	44-16-28	84
		4.4 **CLAYSTONE**, trace sand, gray to brown, firm to hard — 5545.3								
		Approximate Finished Floor Elevation: 5545.3 feet			10-16		20.1	109		
5					12-23	+1.9 @ 500 psf	20.0	112		
					14-28		23.7	105		
					50/11"		23.5	103	72-18-54	88
					50/11"		24.3	102		
					50/9"		18.1	114		
			▽							
			▽		50/12"		24.2	103		
		40.0 — 5509.7			50/9"					
		Boring Terminated at 40 Feet								

Depth (Ft.): 5, 10, 15, 20, 25, 30, 35, 40

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Water Level Observations
▽ 34 feet while drilling
▽ 35 feet upon completion of drilling

Drill Rig
CME-55

Hammer Type
Automatic

Driller
Terracon

Logged by
AH

Boring Started
07-26-2024

Boring Completed
07-26-2024

Notes
Elevation was obtained using a level and the top of the manhole cover southeast of Boring No. 21 (39.7321° N 104.7116° W) with a known elevation of 5549 feet.

Advancement Method
4-inch diameter, solid-stem, continous-flight power auger

Abandonment Method
Boring backfilled with auger cuttings upon completion.

Boring Log No. 9

Location: See Exploration Plan

Latitude: 39.7334° Longitude: -104.7099°

Depth (Ft.) | Elevation: 5553.7 (Ft.) +/-

Model Layer	Graphic Log	Depth (Ft.)	Water Level Observations	Sample Type	Field Test Results	Swell (%)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL-PL-PI	Percent Fines
3		**LEAN CLAY (CL)**, with sand, trace silt, brown, medium stiff to stiff								
					6-9		15.6	102	45-14-31	83
									48-14-34	86
					8-8		12.2	95		
		6.7 5547								
		Approximate Finished Grade Elevation: 5547 feet								
					4-5	-0.1 @ 500 psf	19.2	98	37-20-17	55
		12.0 5541.7								
5		**CLAYSTONE**, brown to gray, soft to medium hard								
					7-12		21.7	106		
					18-30		22.7	104		
					18-30	+3.3 @ 500 psf	20.1	111		
					15-34		23.1	100		
					18-42		20.4	110		
		40.0 5513.7								
		Boring Terminated at 40 Feet			17-21		23.8	98		

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Water Level Observations
None encountered while drilling

Drill Rig
CME-55

Hammer Type
Automatic

Driller
Terracon

Logged by
TPS

Boring Started
07-16-2024

Boring Completed
07-16-2024

Notes
Elevation was obtained using a level and the top of the manhole cover southeast of Boring No. 21 (39.7321° N 104.7116° W) with a known elevation of 5549 feet.

Advancement Method
4-inch diameter, solid-stem, continous-flight power auger

Abandonment Method
Boring backfilled with auger cuttings upon completion.

Boring Log No. 10

Location: See Exploration Plan

Latitude: 39.7334° Longitude: -104.7130°

Approximate Finished Floor Elevation: 5545.3 Ft.

Depth (Ft.) Elevation: 5544.2 (Ft.) +/-

Model Layer	Graphic Log	Depth (Ft.)	Water Level Observations	Sample Type	Field Test Results	Swell (%)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL-PL-PI	Percent Fines
2		1.0 **FILL - FAT CLAY (CH)**, trace sand, brown 5543.2								
3		**FAT CLAY (CH)**, trace sand, brown to gray, stiff to very stiff			6-11	+2.4 @ 500 psf	20.9	106	63-19-44	89
					7-12		24.3	98		
		10.0 5534.2			8-12	+2.6 @ 500 psf	22.3	103		
5		**CLAYSTONE**, brown to gray, firm to medium hard			10-19		24.0	99		
					10-21		21.0	107		
					15-21		21.6	100		
					17-42		17.4	113		
					20-30		26.2	95		
		40.0 5504.2			15-20		15.6	99		
		Boring Terminated at 40 Feet								

Depth (Ft.): 5, 10, 15, 20, 25, 30, 35, 40

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Water Level Observations
None encountered while drilling

Drill Rig
CME-55

Hammer Type
Automatic

Driller
Terracon

Logged by
BA

Boring Started
07-18-2024

Boring Completed
07-18-2024

Notes
Elevation was obtained using a level and the top of the manhole cover southeast of Boring No. 21 (39.7321° N 104.7116° W) with a known elevation of 5549 feet.

Advancement Method
4-inch diameter, solid-stem, continous-flight power auger

Abandonment Method
Boring backfilled with auger cuttings upon completion.

Boring Log No. 11

Location: See Exploration Plan

Latitude: 39.7334° Longitude: -104.7120°

Depth (Ft.) | Elevation: 5548.6 (Ft.) +/-

Model Layer	Graphic Log	Depth (Ft.)	Description	Elevation	Water Level Observations	Sample Type	Field Test Results	Swell (%)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL-PL-PI	Percent Fines
2		1.0	**FILL - LEAN CLAY (CL)**, with sand, brown	5547.6								
3		3.3	**LEAN CLAY (CL)**, with sand, brown, stiff	5545.3			5-8		23.4	101	49-15-34	75
		4.0	Approximate Finished Floor Elevation: 5545.3 feet	5544.6								
							8-11		20.9	100		
		14.0	**CLAYSTONE**, gray to brown, soft to medium hard	5534.6			15-23	+2.4 @ 500 psf	19.6	109		
		19.0	**CLAYEY SANDSTONE**, fine to medium grained, brown, firm	5529.6			11-16		15.4	104	41-18-23	38
5		29.0	**CLAYSTONE**, gray to brown, medium hard	5519.6			17-27		23.9	96		
							13-25		23.2	106		
		39.0	**CLAYEY SANDSTONE**, fine to medium grained, gray, medium hard	5509.6			18-35		19.4	106	44-19-25	45
							14-30		18.9	103		
			CLAYSTONE, gray to brown, medium hard				50/10"		23.9	103		
							50/11"		23.9	104		
		50.0		5498.6			50/10"		20.9	107		
			Boring Terminated at 50 Feet									

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Water Level Observations
None encountered while drilling

Drill Rig
CME-550

Hammer Type
Automatic

Driller
Terracon

Logged by
BA

Boring Started
07-25-2024

Boring Completed
07-25-2024

Notes
Elevation was obtained using a level and the top of the manhole cover southeast of Boring No. 21 (39.7321° N 104.7116° W) with a known elevation of 5549 feet.

Advancement Method
4-inch diameter, solid-stem, continous-flight power auger

Abandonment Method
Boring backfilled with auger cuttings upon completion.

Boring Log No. 12

Location: See Exploration Plan

Latitude: 39.7334° Longitude: -104.7105°

Depth (Ft.) Elevation: 5550.7 (Ft.) +/-

Model Layer	Graphic Log	Depth (Ft.)	Water Level Observations	Sample Type	Field Test Results	Swell (%)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL-PL-PI	Percent Fines
2		**FILL - SANDY LEAN CLAY (CL)**, trace gravel, brown, medium stiff								
					4-6		15.1	112	42-14-28	60
					3-4	+1.7 @ 500 psf	16.4	108		
		5.4 5545.3								
		Approximate Finished Floor Elevation: 5545.3 feet								
		7.0 5543.7								
5		**CLAYSTONE**, gray to brown, firm to very hard								
					11-21	+4.4 @ 500 psf	23.9	100	67-19-48	86
					50/12"		21.0	106		
					50/9"		19.9	109		
					50/11"		22.4	106		
					50/2"					
					50/9"		22.1	102		
		40.0 5510.7			50/3"		11.3	107		
		Boring Terminated at 40 Feet								

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Water Level Observations
None encountered while drilling

Drill Rig
CME-550

Hammer Type
Automatic

Driller
Terracon

Logged by
TPS

Boring Started
07-26-2024

Boring Completed
07-26-2024

Notes
Elevation was obtained using a level and the top of the manhole cover southeast of Boring No. 21 (39.7321° N 104.7116° W) with a known elevation of 5549 feet.

Advancement Method
4-inch diameter, solid-stem, continous-flight power auger

Abandonment Method
Boring backfilled with auger cuttings upon completion.

Boring Log No. 13

Location: See Exploration Plan

Latitude: 39.7331° Longitude: -104.7126°

Approximate Finished Floor Elevation: 5546.6 Ft.

Elevation: 5544.5 (Ft.) +/-

Model Layer	Depth (Ft.)	Description	Elevation	Sample Type	Field Test Results	Swell (%)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL-PL-PI	Percent Fines
2	1.0	**FILL - FAT CLAY (CH)**, trace sand, brown	5543.5							
3		**FAT CLAY (CH)**, trace sand, gray to brown, stiff			5-8		20.7	102	66-19-47	95
					5-6	+1.4 @ 500 psf	21.7	101		
	7.0		5537.5							
5		**CLAYSTONE**, trace sand, brown to gray, soft to medium hard			8-15		23.4	104		
					16-21		18.5	106		
					10-17		21.0	100		
					17-34		20.4	105		
					13-23		21.0	104		
					14-24		22.8	89		
	40.0		5504.5		15-30		28.0	94		
		Boring Terminated at 40 Feet								

Depth (Ft.) 5 10 15 20 25 30 35 40

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Water Level Observations
None encountered while drilling

Drill Rig
CME-55

Hammer Type
Automatic

Driller
Terracon

Logged by
BA

Boring Started
07-18-2024

Boring Completed
07-18-2024

Notes
Elevation was obtained using a level and the top of the manhole cover southeast of Boring No. 21 (39.7321° N 104.7116° W) with a known elevation of 5549 feet.

Advancement Method
4-inch diameter, solid-stem, continous-flight power auger

Abandonment Method
Boring backfilled with auger cuttings upon completion.

Boring Log No. 14

Location: See Exploration Plan

Latitude: 39.7330° Longitude: -104.7107°

Approximate Finished Floor Elevation: 5546.6 Ft.

Depth (Ft.) Elevation: 5539.8 (Ft.) +/-

Model Layer	Graphic Log	Depth (Ft.)	Water Level Observations	Sample Type	Field Test Results	Swell (%)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL-PL-PI	Percent Fines
2		**FILL - FAT CLAY (CH)**, trace sand, grayish brown								
		2.0 5537.8								
5		**CLAYSTONE**, gray to brown, soft to very hard			7-12	+4.2 @ 500 psf	21.2	104	70-19-51	99
					9-18		21.6	106		
					14-26		22.2	107		
					15-30	+5.4 @ 500 psf	9.5	104		
					50/12"		21.8	101		
					50/6"		9.5	125	34-17-17	60
					15-32		22.5	105		
					11-31		21.9	104		
		40.0 5499.8			50/2"		16.3	105		
		Boring Terminated at 40 Feet								

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Water Level Observations
None encountered while drilling

Drill Rig
CME-55

Hammer Type
Automatic

Driller
Terracon

Logged by
JL

Boring Started
07-29-2024

Boring Completed
07-29-2024

Notes
Elevation was obtained using a level and the top of the manhole cover southeast of Boring No. 21 (39.7321° N 104.7116° W) with a known elevation of 5549 feet.

Advancement Method
4-inch diameter, solid-stem, continous-flight power auger

Abandonment Method
Boring backfilled with auger cuttings upon completion.

Boring Log No. 15

Location: See Exploration Plan

Latitude: 39.7328° Longitude: -104.7129°

Approximate Finished Floor Elevation: 5546.6 Ft.

Depth (Ft.) | Elevation: 5545.4 (Ft.) +/-

Model Layer	Graphic Log	Depth (Ft.)	Description	Elevation	Depth (Ft.)	Water Level Observations	Sample Type	Field Test Results	Swell (%)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL-PL-PI	Percent Fines
2		1.0	**FILL - SANDY FAT CLAY (CH)**, brown	5544.4									
3			**FAT CLAY (CH)**, brown to gray, stiff					5-6		22.4	104	66-18-48	99
		4.0		5541.4				7-11	+0.6 @ 500 psf	17.6	112		
5			**CLAYSTONE**, trace sand, brown to gray, soft to medium hard		5								
					10			10-16		16.9	103		
					15			12-22		21.0			
					20			12-22		21.1	100		
					25			12-19		20.2	109		
					30			11-20		20.8	109		
					35			20-35		21.3	105		
		40.0		5505.4	40			17-32		16.4	107		
			Boring Terminated at 40 Feet										

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Water Level Observations
None encountered while drilling

Drill Rig
CME-55

Hammer Type
Automatic

Driller
Terracon

Logged by
BA

Boring Started
07-18-2024

Boring Completed
07-18-2024

Notes
Elevation was obtained using a level and the top of the manhole cover southeast of Boring No. 21 (39.7321° N 104.7116° W) with a known elevation of 5549 feet.

Advancement Method
4-inch diameter, solid-stem, continous-flight power auger

Abandonment Method
Boring backfilled with auger cuttings upon completion.

Boring Log No. 16

Location: See Exploration Plan

Latitude: 39.7328° Longitude: -104.7119°

Depth (Ft.) | Elevation: 5547.2 (Ft.) +/-

Model Layer	Graphic Log	Depth (Ft.)	Elevation	Water Level Observations	Sample Type	Field Test Results	Swell (%)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL-PL-PI	Percent Fines
3		0.6 **FAT CLAY (CH)**, brown	5546.6								
		2.0 Approximate Finished Floor Elevation: 5546.6 feet	5545.2								
5		**CLAYSTONE**, gray to brown, soft to very hard				4-12		21.2	104	51-16-35	86
						14-27	+11.1 @ 500 psf	17.7	110	66-20-46	98
						50/11"		15.7	117		
						50/10"	+5.0 @ 500 psf	16.0	106		
						50/8"		17.1	109		
						50/8"		21.5	107		
						50/9"		20.2	106		
						50/1"					
		38.0	5509.2								
		CLAYEY SANDSTONE, fine to medium grained, grayish brown, hard				50/9"		19.4	108	44-16-28	47
		42.0	5505.2								
		CLAYSTONE, gray to brown, medium hard				50/12"		22.6	106		
		50.0	5497.2			20-27		20.3	102		
		Boring Terminated at 50 Feet									

Depth (Ft.): 5, 10, 15, 20, 25, 30, 35, 40, 45, 50

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Water Level Observations
None encountered while drilling

Drill Rig
CME-550

Hammer Type
Automatic

Driller
Terracon

Logged by
AH

Boring Started
07-26-2024

Boring Completed
07-26-2024

Notes
Elevation was obtained using a level and the top of the manhole cover southeast of Boring No. 21 (39.7321° N 104.7116° W) with a known elevation of 5549 feet.

Advancement Method
4-inch diameter, solid-stem, continous-flight power auger

Abandonment Method
Boring backfilled with auger cuttings upon completion.

Boring Log No. 17

Location: See Exploration Plan

Latitude: 39.7327° Longitude: -104.7106°

Approximate Finished Floor Elevation: 5546.6 Ft.

Depth (Ft.) Elevation: 5540.6 (Ft.) +/-

Model Layer	Graphic Log	Depth (Ft.)	Water Level Observations	Sample Type	Field Test Results	Swell (%)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL-PL-PI	Percent Fines
2		**FILL - SILTY CLAY (CL-ML)**, trace sand, grayish brown 2.0 5538.6								
5		**CLAYSTONE**, brown to gray, soft to hard			7-15	+1.4 @ 500 psf	16.6	117	42-15-27	65
					10-18		20.5	110		
					13-25	+6.2 @ 500 psf	22.0	105		
					50/11"		19.4	110		
					50/8"		19.1	112		
					50/7"		15.7	112	44-18-26	69
					50/9"		19.7	108		
					50/9"		19.5	112		
		40.0 5500.6			50/12"		21.4	106		
		Boring Terminated at 40 Feet								

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Water Level Observations
None encountered while drilling

Drill Rig
CME-55

Hammer Type
Automatic

Driller
Terracon

Logged by
JL

Boring Started
07-29-2024

Boring Completed
07-29-2024

Notes
Elevation was obtained using a level and the top of the manhole cover southeast of Boring No. 21 (39.7321° N 104.7116° W) with a known elevation of 5549 feet.

Advancement Method
4-inch diameter, solid-stem, continous-flight power auger

Abandonment Method
Boring backfilled with auger cuttings upon completion.

Boring Log No. 18

Location: See Exploration Plan

Latitude: 39.7325° Longitude: -104.7125°

Approximate Finished Floor Elevation: 5546.6 Ft.

Depth (Ft.) Elevation: 5541.8 (Ft.) +/-

Model Layer	Graphic Log	Depth (Ft.)	Water Level Observations	Sample Type	Field Test Results	Swell (%)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL-PL-PI	Percent Fines
2		1.0 **FILL - FAT CLAY (CH)**, trace sand, brown 5540.8								
5		**CLAYSTONE**, brown to gray, soft to hard			6-11		18.6	107	53-17-36	87
									50-16-34	83
					7-13	+0.5 @ 500 psf	18.2	111		
					10-17					
					10-18		23.6	103		
					10-20		24.1	102		
					12-25		21.1	108		
					12-30		19.3			
					20-45		19.3	113		
		40.0 5501.8			20-35		34.1	84		
		Boring Terminated at 40 Feet								

5 10 15 20 25 30 35 40

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Water Level Observations
None encountered while drilling

Drill Rig
CME-55

Hammer Type
Automatic

Driller
Terracon

Logged by
BA

Boring Started
07-17-2024

Boring Completed
07-17-2024

Notes
Elevation was obtained using a level and the top of the manhole cover southeast of Boring No. 21 (39.7321° N 104.7116° W) with a known elevation of 5549 feet.

Advancement Method
4-inch diameter, solid-stem, continous-flight power auger

Abandonment Method
Boring backfilled with auger cuttings upon completion.

Boring Log No. 19

Location: See Exploration Plan

Latitude: 39.7325° Longitude: -104.7113°

Depth (Ft.) Elevation: 5547.1 (Ft.) +/-

Model Layer	Graphic Log	Depth (Ft.)	Water Level Observations	Sample Type	Field Test Results	Swell (%)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL-PL-PI	Percent Fines
2		1.0 **FILL - LEAN CLAY (CL)**, with sand, brown 5546.1								
		1.6 **CLAYSTONE**, gray to brown, firm to medium hard 5545.5								
		Approximate Finished Grade Elevation: 5545.5 feet								
					10-20		20.6	101	49-17-32	79
					10-21		20.4	99		
		5								
					12-43	+7.6 @ 500 psf	14.5	115		
		10								
5					17-36		18.0	100		
		15								
					17-41		21.4	106		
		20								
					18-42		17.0	116		
		25								
		29.0 5518.1								
		CLAYEY SANDSTONE, fine to medium grained, gray, medium hard			15-32		16.5	98	43-17-26	43
		30								
		32.0 5515.1								
		CLAYSTONE, gray, medium hard to hard								
					22-50		22.2	106		
		35								
		40.0 5507.1			22-34		21.4	93		
		40								
		Boring Terminated at 40 Feet								

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Water Level Observations
None encountered while drilling

Drill Rig
CME-55

Hammer Type
Automatic

Driller
Terracon

Logged by
TPS

Boring Started
07-24-2024

Boring Completed
07-24-2024

Notes
Elevation was obtained using a level and the top of the manhole cover southeast of Boring No. 21 (39.7321° N 104.7116° W) with a known elevation of 5549 feet.

Advancement Method
4-inch diameter, solid-stem, continous-flight power auger

Abandonment Method
Boring backfilled with auger cuttings upon completion.

Boring Log No. 20

Location: See Exploration Plan

Latitude: 39.7323° Longitude: -104.7130°

Depth (Ft.) Elevation: 5544.8 (Ft.) +/-

Model Layer	Graphic Log	Depth (Ft.)	Water Level Observations	Sample Type	Field Test Results	Swell (%)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL-PL-PI	Percent Fines
2		0.8 **FILL - FAT CLAY (CH)**, with sand, brown 5544								
		1.0 Approximate Finished Grade Elevation: 5544 feet 5543.8								
		CLAYSTONE, brown to gray, soft to firm			10-13		20.3	114	62-17-45	85
					8-14	+5.7 @ 500 psf	21.9	104		
					12-22		19.3	111		
		14.0 5530.8								
		CLAYEY SANDSTONE, fine to medium grained, brown, medium hard to very hard			10-29		16.7	119		
5					50/2"		10.8		35-17-18	20
		24.0 5520.8								
		CLAYSTONE, gray, medium hard			20-38		21.0	100		
					16-31		21.1	107		
		34.0 5510.8								
		SANDSTONE, fine to coarse grained, light brown, very hard			50/6"		11.6	107		
		39.0 5505.8								
		40.0 **CLAYSTONE**, brown to gray, hard 5504.8			41-46		17.4			
		Boring Terminated at 40 Feet								

Depth (Ft.): 5, 10, 15, 20, 25, 30, 35, 40

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Water Level Observations
None encountered while drilling

Drill Rig
CME-55

Hammer Type
Automatic

Driller
Terracon

Logged by
BA

Boring Started
07-17-2024

Boring Completed
07-17-2024

Notes
Elevation was obtained using a level and the top of the manhole cover southeast of Boring No. 21 (39.7321° N 104.7116° W) with a known elevation of 5549 feet.

Advancement Method
4-inch diameter, solid-stem, continous-flight power auger

Abandonment Method
Boring backfilled with auger cuttings upon completion.

Boring Log No. 21

Location: See Exploration Plan

Latitude: 39.7323° Longitude: -104.7118°

Depth (Ft.) Elevation: 5547.0 (Ft.) +/-

Model Layer	Graphic Log	Depth (Ft.)	Description	Elevation	Depth (Ft.)	Water Level Observations	Sample Type	Field Test Results	Swell (%)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL-PL-PI	Percent Fines
2		1.0	**FILL - SANDY FAT CLAY (CH)**, brown	5546									
		3.0	**CLAYSTONE**, trace sand and silt, gray to brown, soft to hard	5544									
			Approximate Finished Grade Elevation: 5544 feet										
					5			6-12		22.1	103		
								8-14		23.4	104		
					10			8-20	+5.7 @ 500 psf	20.1	110	71-21-50	98
					15			15-34	+8.5 @ 500 psf	18.4	109		
5					20			15-25		19.1	104		
					25			22-50/2"		18.6			
					30			18-50/5"		18.3	111		
					35			20-50/5"		20.4			
		40.0		5507	40			12-25		22.2	105		
			Boring Terminated at 40 Feet										

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Water Level Observations
None encountered while drilling

Drill Rig
CME-55

Hammer Type
Automatic

Driller
Terracon

Logged by
JL

Boring Started
07-17-2024

Boring Completed
07-17-2024

Notes
Elevation was obtained using a level and the top of the manhole cover southeast of Boring No. 21 (39.7321° N 104.7116° W) with a known elevation of 5549 feet.

Advancement Method
4-inch diameter, solid-stem, continous-flight power auger

Abandonment Method
Boring backfilled with auger cuttings upon completion.

Boring Log No. 22

Location: See Exploration Plan

Latitude: 39.7324° Longitude: -104.7102°

Depth (Ft.) Elevation: 5549.1 (Ft.) +/-

Model Layer	Graphic Log	Depth (Ft.)	Water Level Observations	Sample Type	Field Test Results	Swell (%)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL-PL-PI	Percent Fines
		CLAYSTONE, trace sand, brown to gray, firm to hard								
		2.5 5546.6								
		Approximate Finished Floor Elevation: 5546.6 feet								
					10-17		15.7	118	51-16-35	66
					12-19	+2.3 @ 500 psf	15.8	119		
					12-25		15.1	109		
					22-40		17.7	115		
5					15-28		18.9	111		
					15-30		20.8	106		
					17-50/5"		17.1	115		
					20-37		18.6	111		
		40.0 5509.1			17-41		22.1	107		
		Boring Terminated at 40 Feet								

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Water Level Observations
None encountered while drilling

Drill Rig
CME-55

Hammer Type
Automatic

Driller
Terracon

Logged by
TPS

Boring Started
07-16-2024

Boring Completed
07-16-2024

Notes
Elevation was obtained using a level and the top of the manhole cover southeast of Boring No. 21 (39.7321° N 104.7116° W) with a known elevation of 5549 feet.

Advancement Method
4-inch diameter, solid-stem, continous-flight power auger

Abandonment Method
Boring backfilled with auger cuttings upon completion.

Boring Log No. P1

Location: See Exploration Plan

Latitude: 39.7332° Longitude: -104.7095°

Depth (Ft.) Elevation: 5547.0 (Ft.) +/-

Model Layer	Graphic Log	Depth (Ft.)	Water Level Observations	Sample Type	Field Test Results	Swell (%)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL-PL-PI	Percent Fines
1		0.3 **ASPHALT**, about 3 inches 5546.75								
		FILL - SANDY LEAN CLAY (CL), brown, stiff to very stiff			5-7		16.4	107	41-17-24	67
		3.0 5544								
		Approximate Finished Grade Elevation: 5544 feet								
2					5-10		14.9	103		
		5								
		10.0 5537			9-10		13.9	106		
		10								
		Boring Terminated at 10 Feet								

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Water Level Observations
None encountered while drilling

Drill Rig
CME-55

Hammer Type
Automatic

Driller
Terracon

Logged by
TPS

Boring Started
07-16-2024

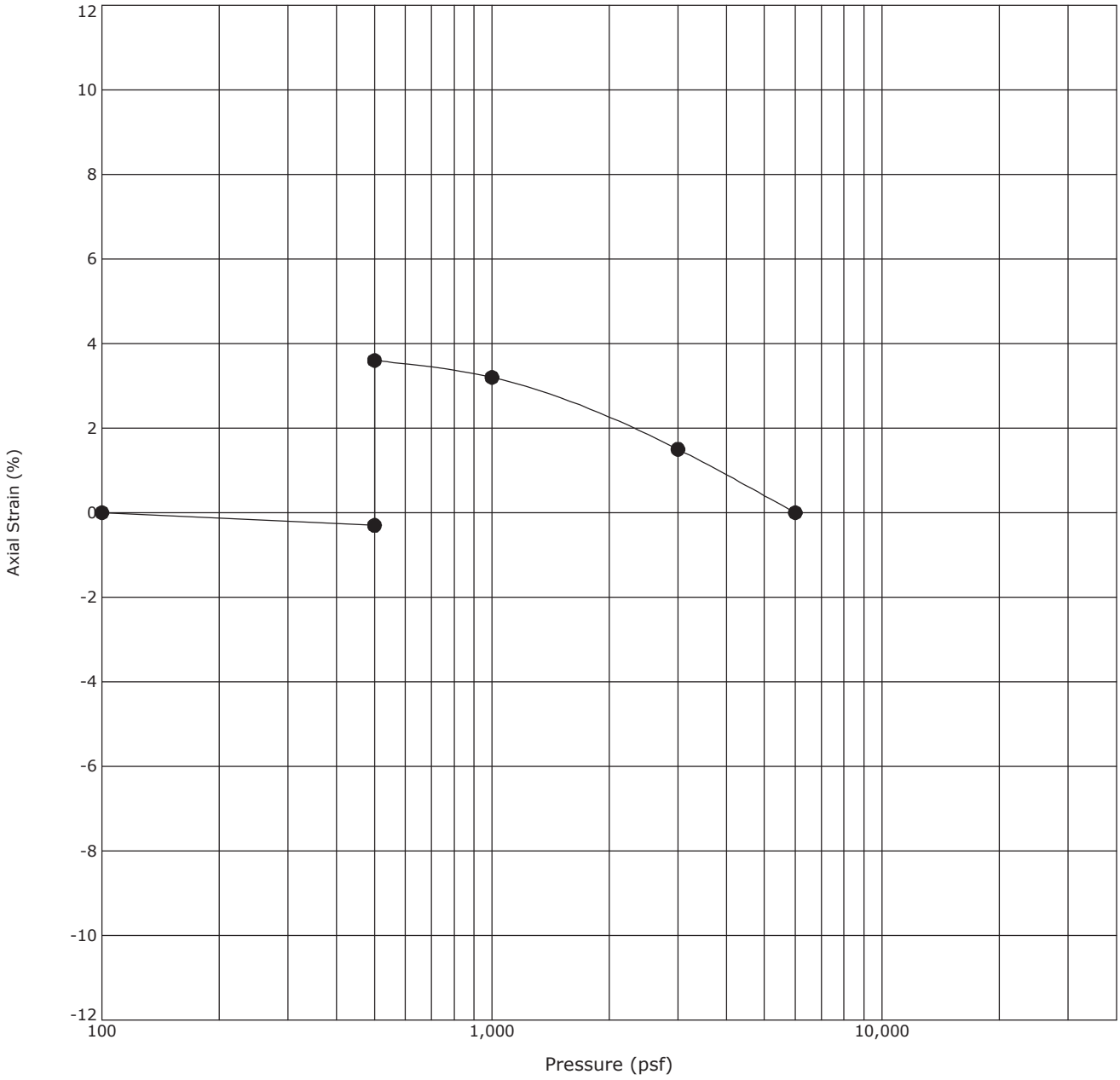
Boring Completed
07-16-2024

Notes
Elevation was obtained using a level and the top of the manhole cover southeast of Boring No. 21 (39.7321° N 104.7116° W) with a known elevation of 5549 feet.

Advancement Method
4-inch diameter, solid-stem, continous-flight power auger

Abandonment Method
Boring backfilled with auger cuttings upon completion.

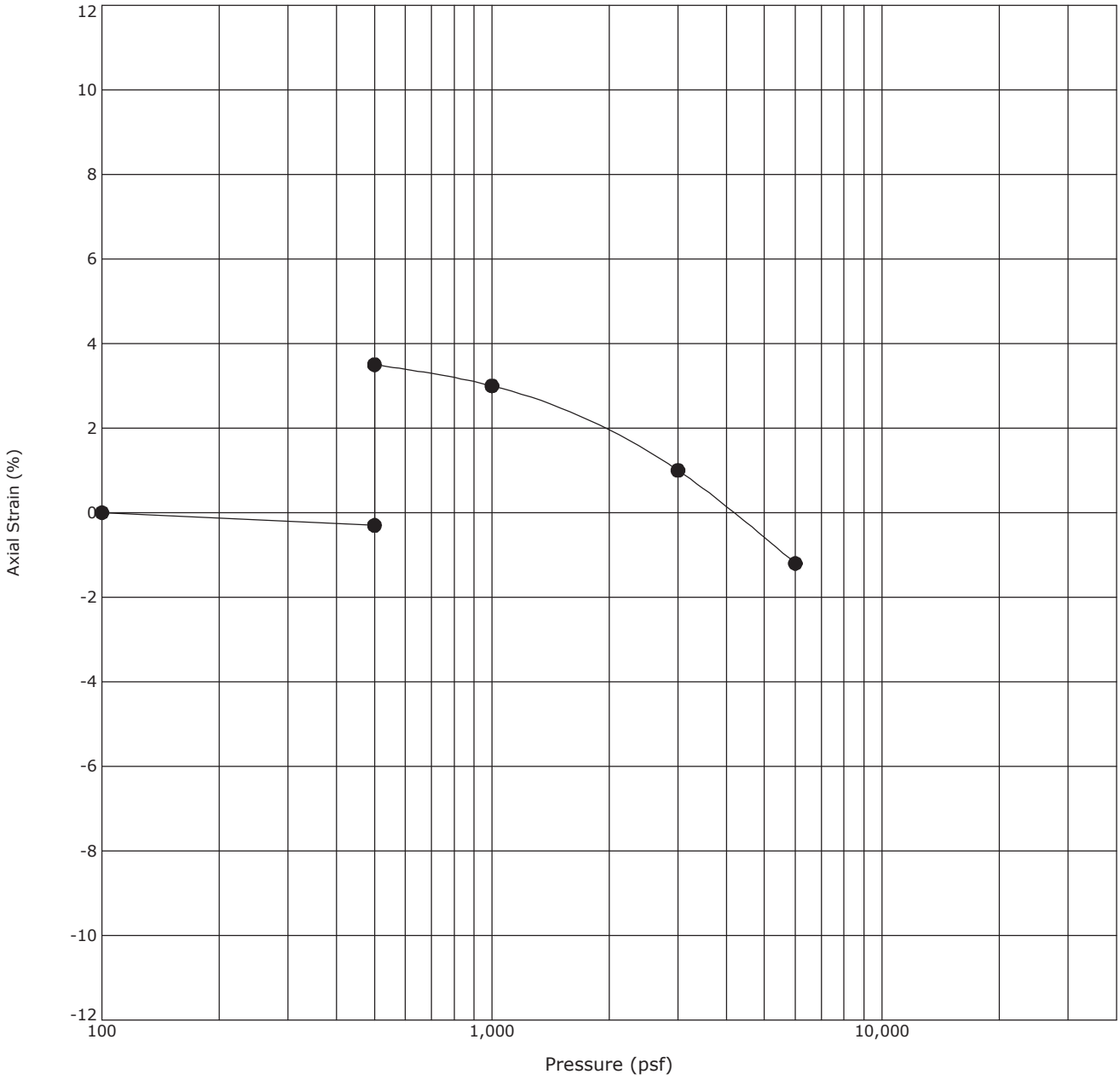
One-Dimensional Swell or Collapse



Boring ID	Depth (Ft)	Description	USCS	γ_d(pcf)	WC (%)
● 1	14 - 15	CLAYSTONE		95	27.4

Notes: Water was added at 500 psf.

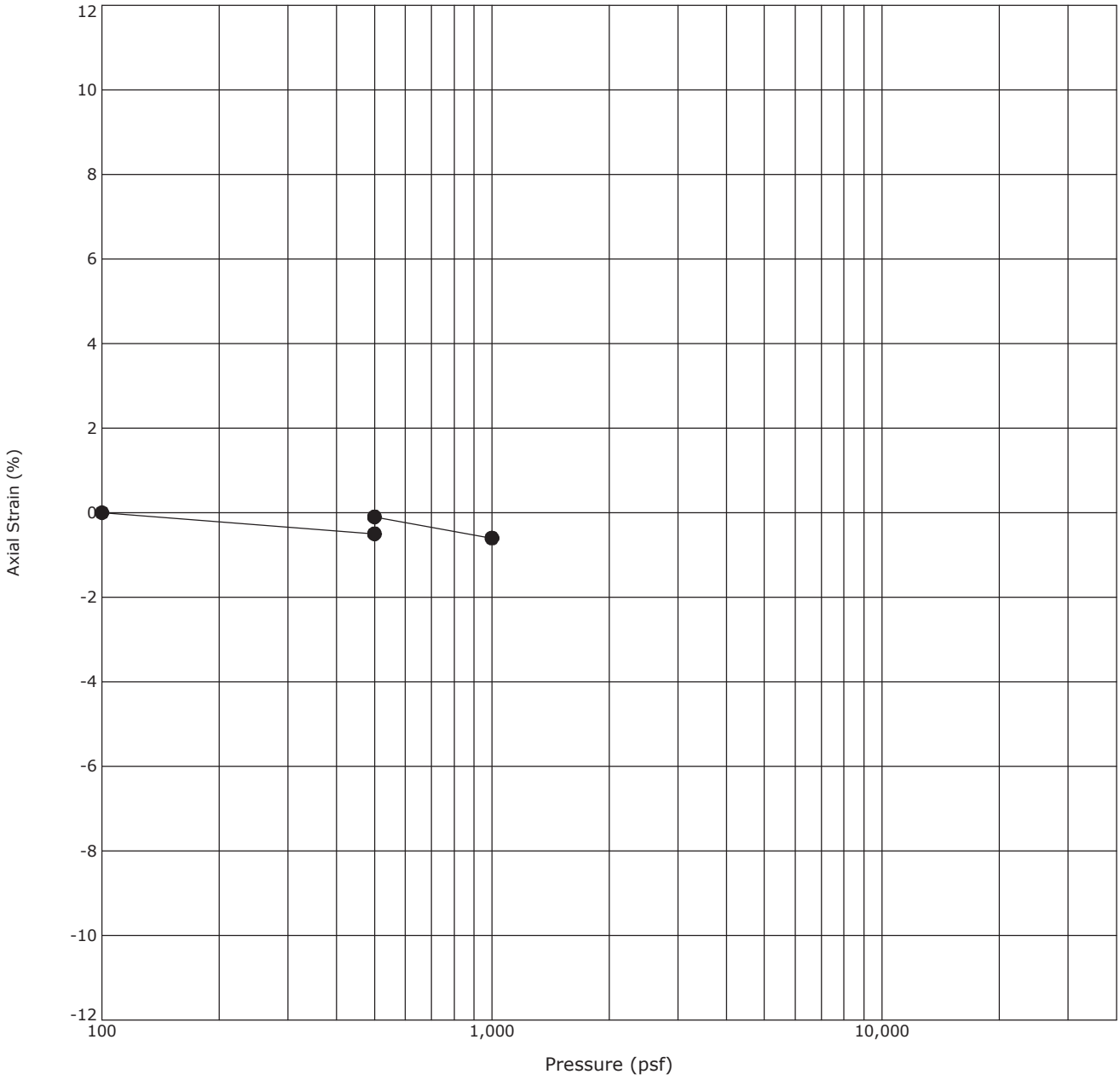
One-Dimensional Swell or Collapse



Boring ID	Depth (Ft)	Description	USCS	γ_d(pcf)	WC (%)
● 2	2 - 3	LEAN CLAY with SAND	CL	110	13.6

Notes: Water was added at 500 psf.

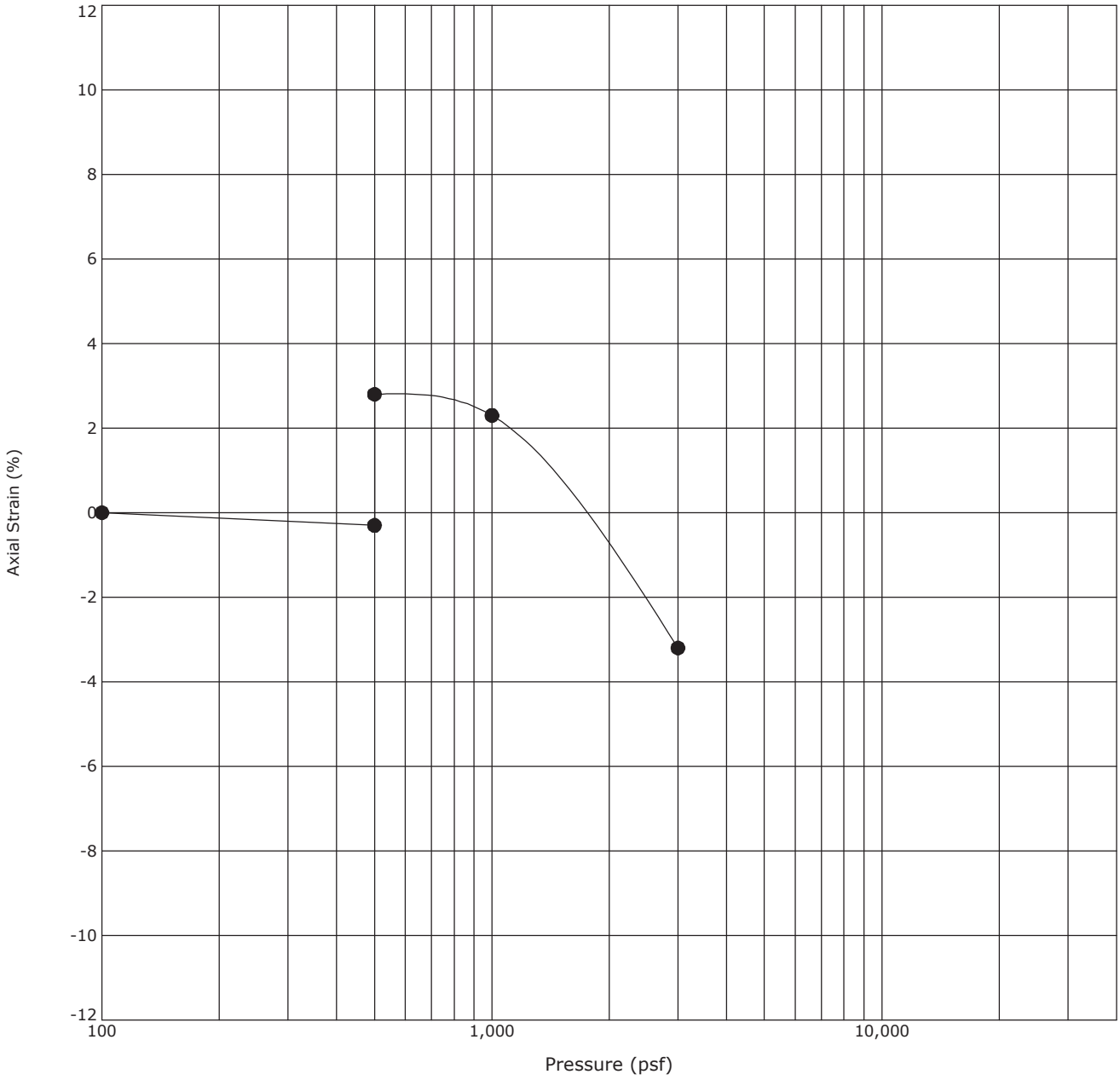
One-Dimensional Swell or Collapse



Boring ID	Depth (Ft)	Description	USCS	γ_d(pcf)	WC (%)
● 2	9 - 10	LEAN CLAY with SAND	CL	108	19.3

Notes: Water was added at 500 psf.

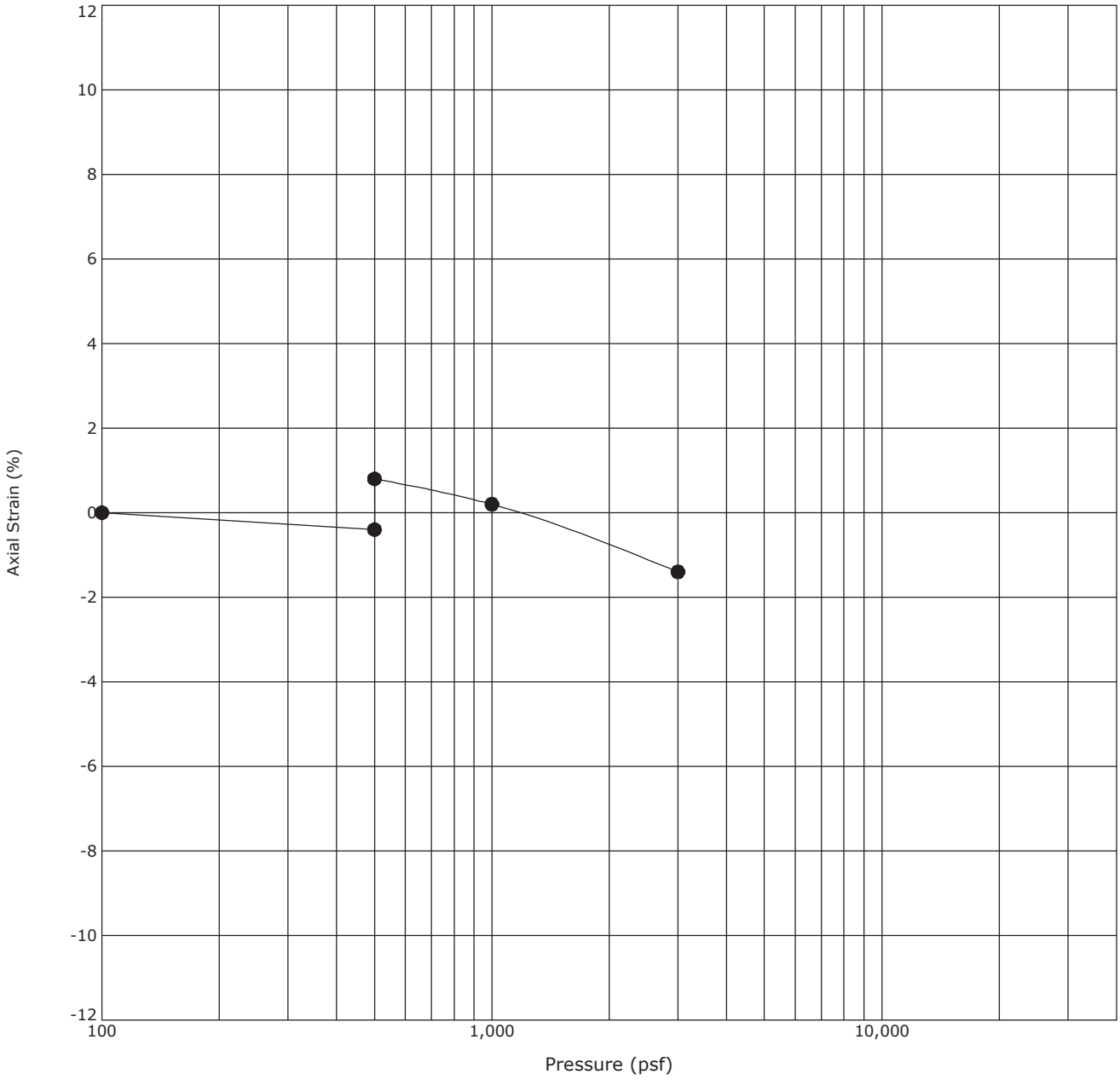
One-Dimensional Swell or Collapse



Boring ID	Depth (Ft)	Description	USCS	γ_d(pcf)	WC (%)
● 3	2 - 3	LEAN CLAY with SAND	CL	99	15.4

Notes: Water was added at 500 psf.

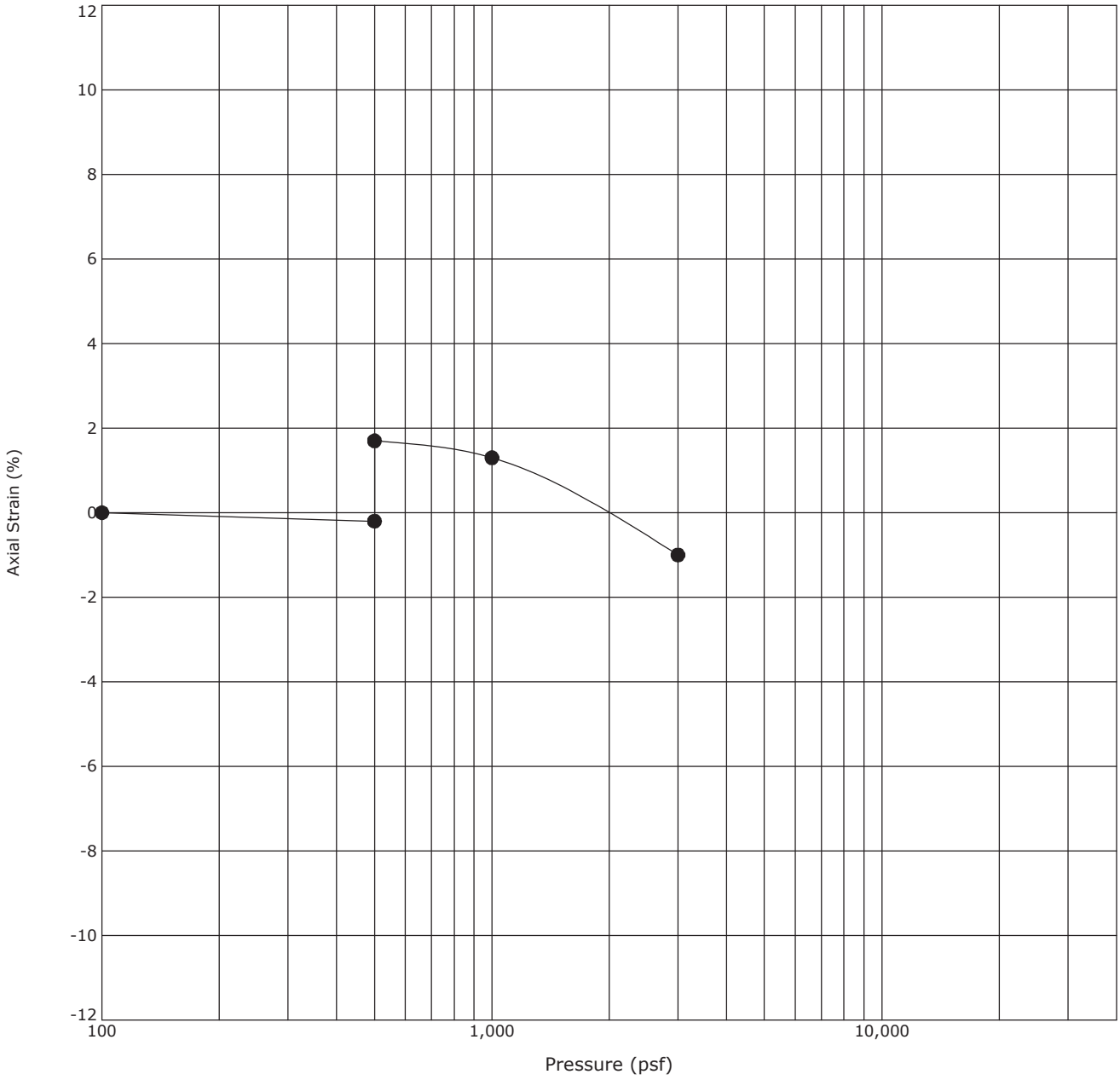
One-Dimensional Swell or Collapse



Boring ID	Depth (Ft)	Description	USCS	γ_d(pcf)	WC (%)
● 3	14 - 15	SANDY FAT CLAY	CH	108	19.5

Notes: Water was added at 500 psf.

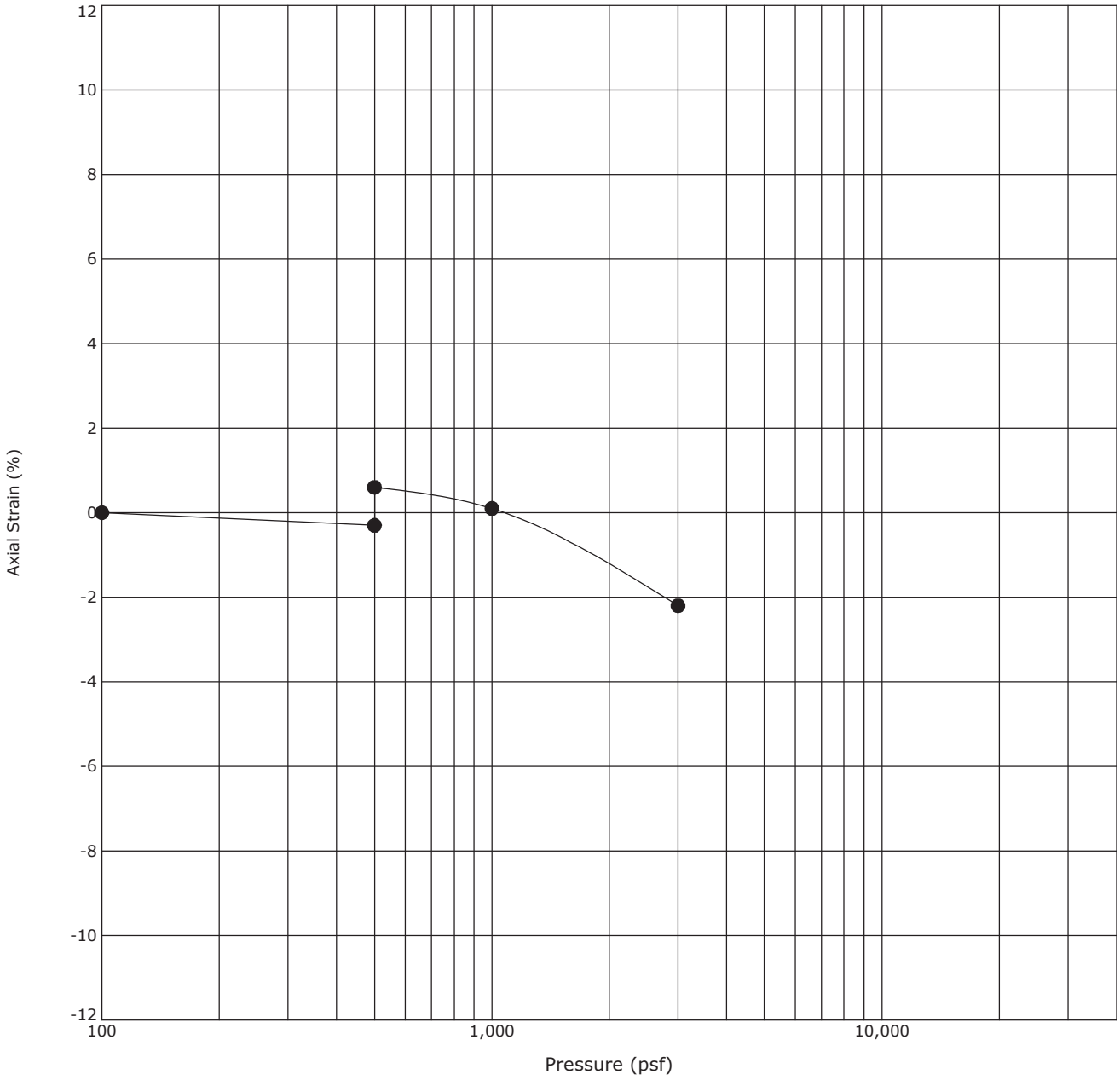
One-Dimensional Swell or Collapse



Boring ID	Depth (Ft)	Description	USCS	γ_d(pcf)	WC (%)
● 4	2 - 3	FAT CLAY with SAND	CH	102	13.5

Notes: Water was added at 500 psf.

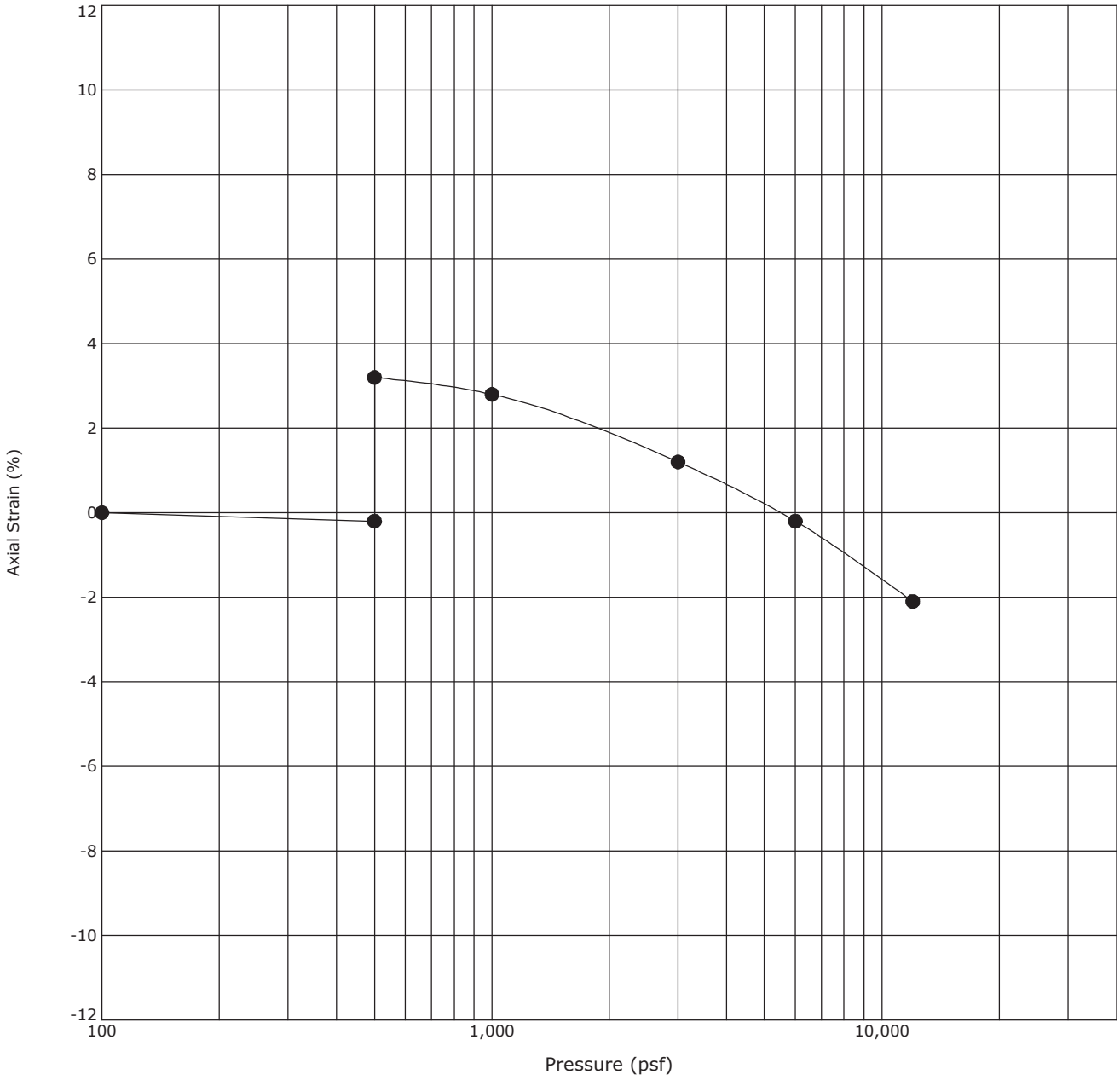
One-Dimensional Swell or Collapse



Boring ID	Depth (Ft)	Description	USCS	γ_d(pcf)	WC (%)
● 5	4 - 5	FAT CLAY with SAND	CH	119	5.6

Notes: Water was added at 500 psf.

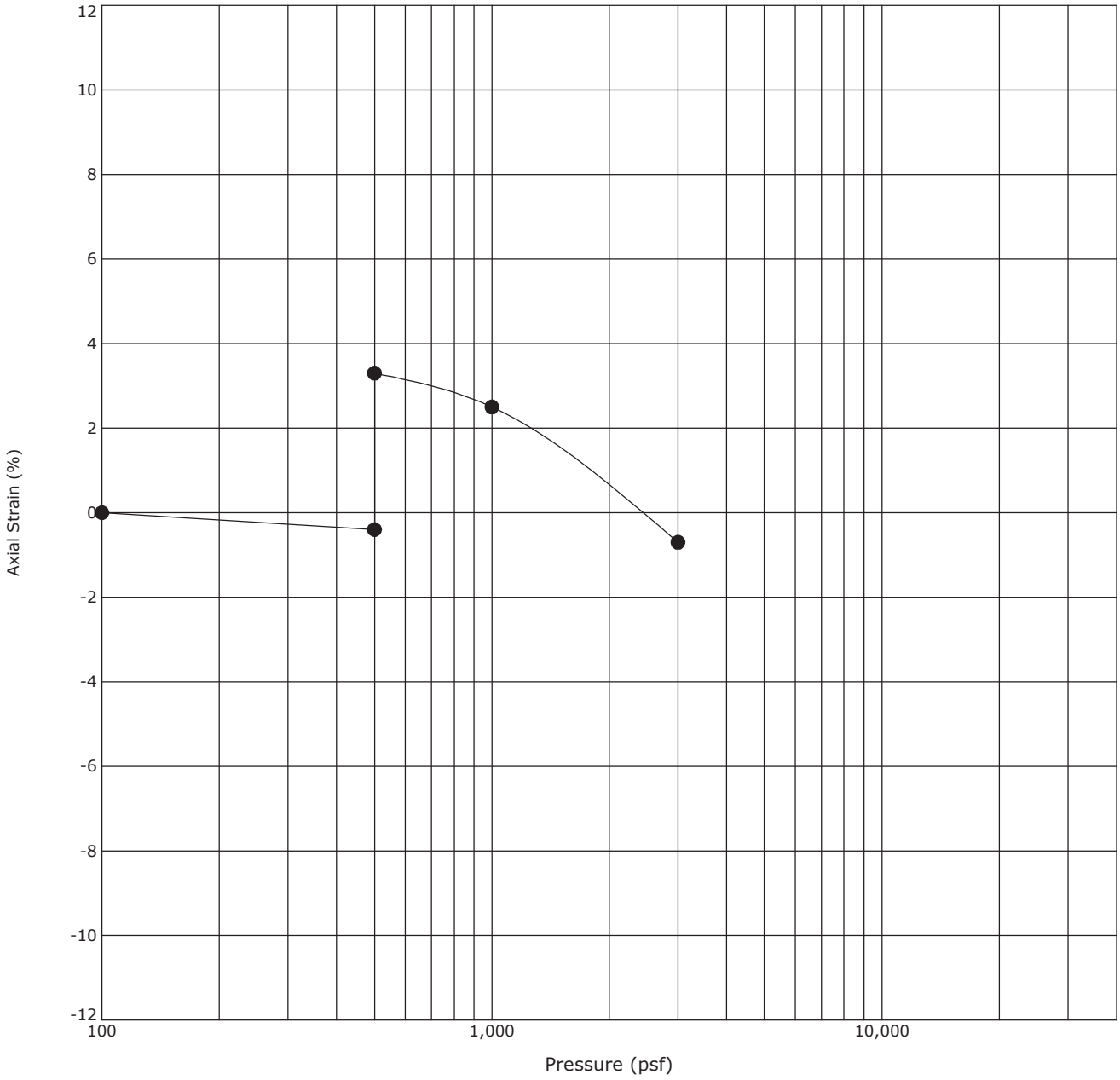
One-Dimensional Swell or Collapse



Boring ID		Depth (Ft)	Description	USCS	γ_d(pcf)	WC (%)
●	5	9 - 10	CLAYSTONE		105	22.9

Notes: Water was added at 500 psf.

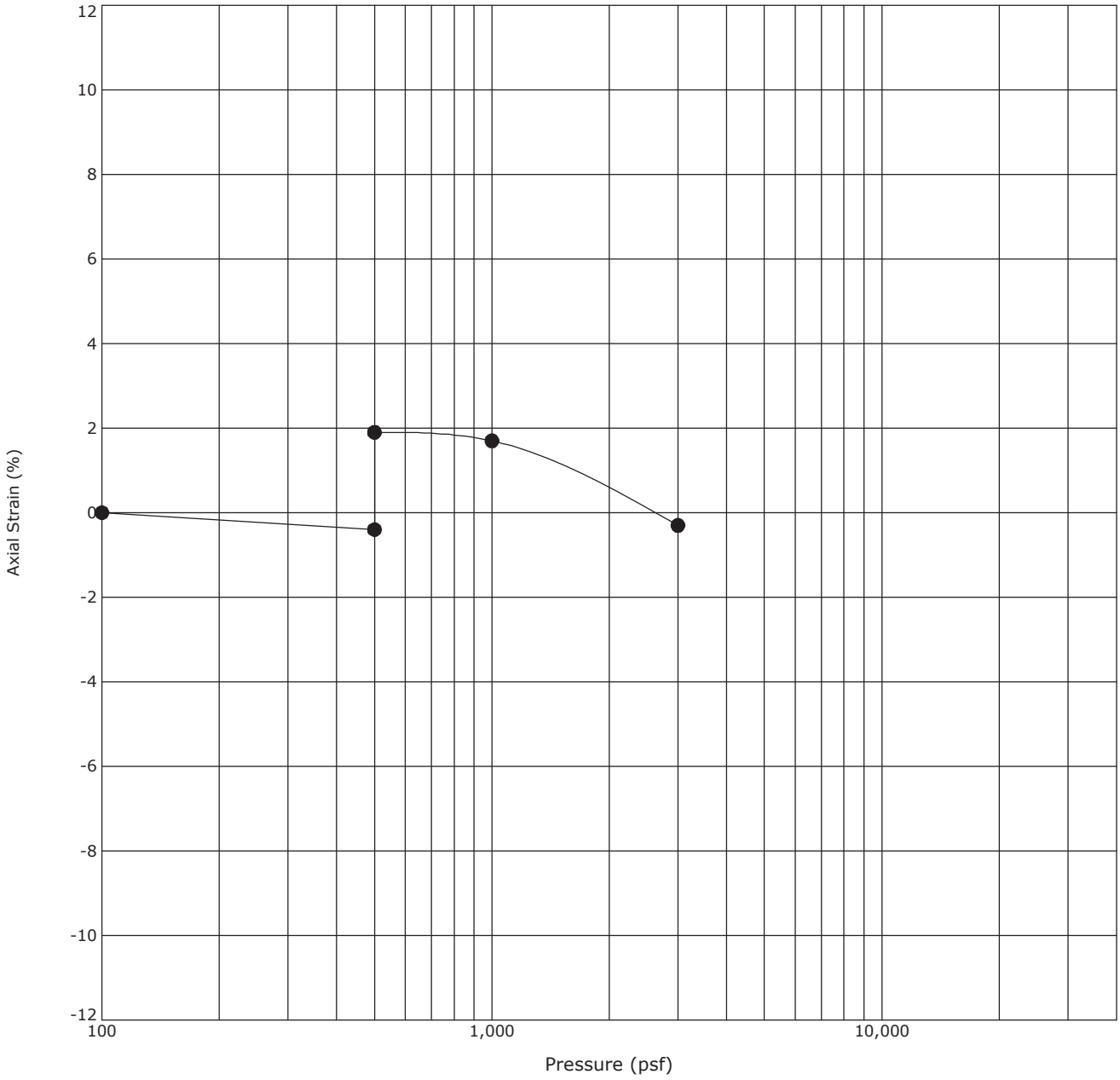
One-Dimensional Swell or Collapse



Boring ID		Depth (Ft)	Description	USCS	γ_d(pcf)	WC (%)
●	6	9 - 10	CLAYSTONE		100	20.7

Notes: Water was added at 500 psf.

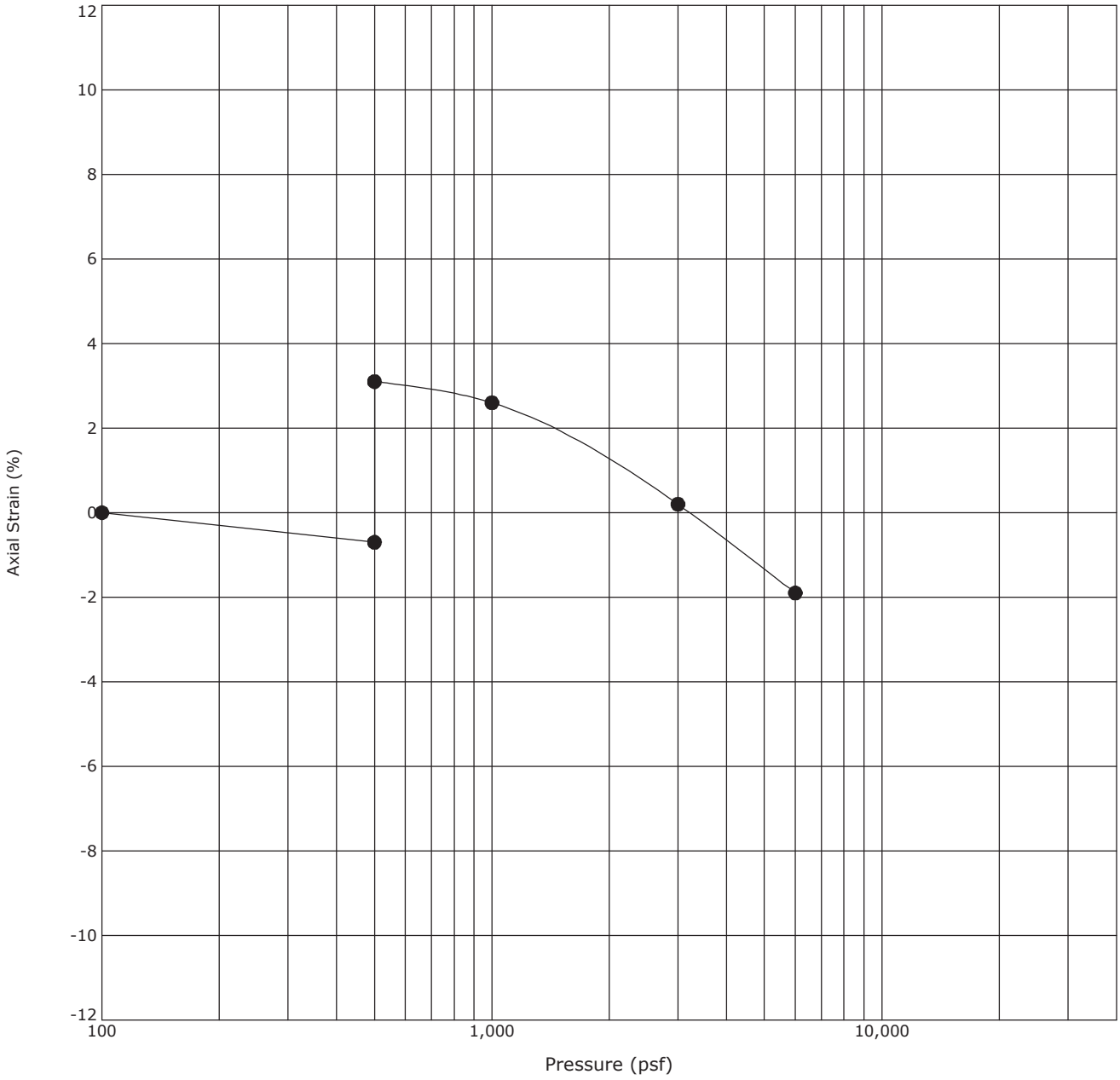
One-Dimensional Swell or Collapse



Boring ID	Depth (Ft)	Description	USCS	γ_d(pcf)	WC (%)
● 7	2 - 3	LEAN CLAY with SAND	CL	97	24.6

Notes: Water was added at 500 psf.

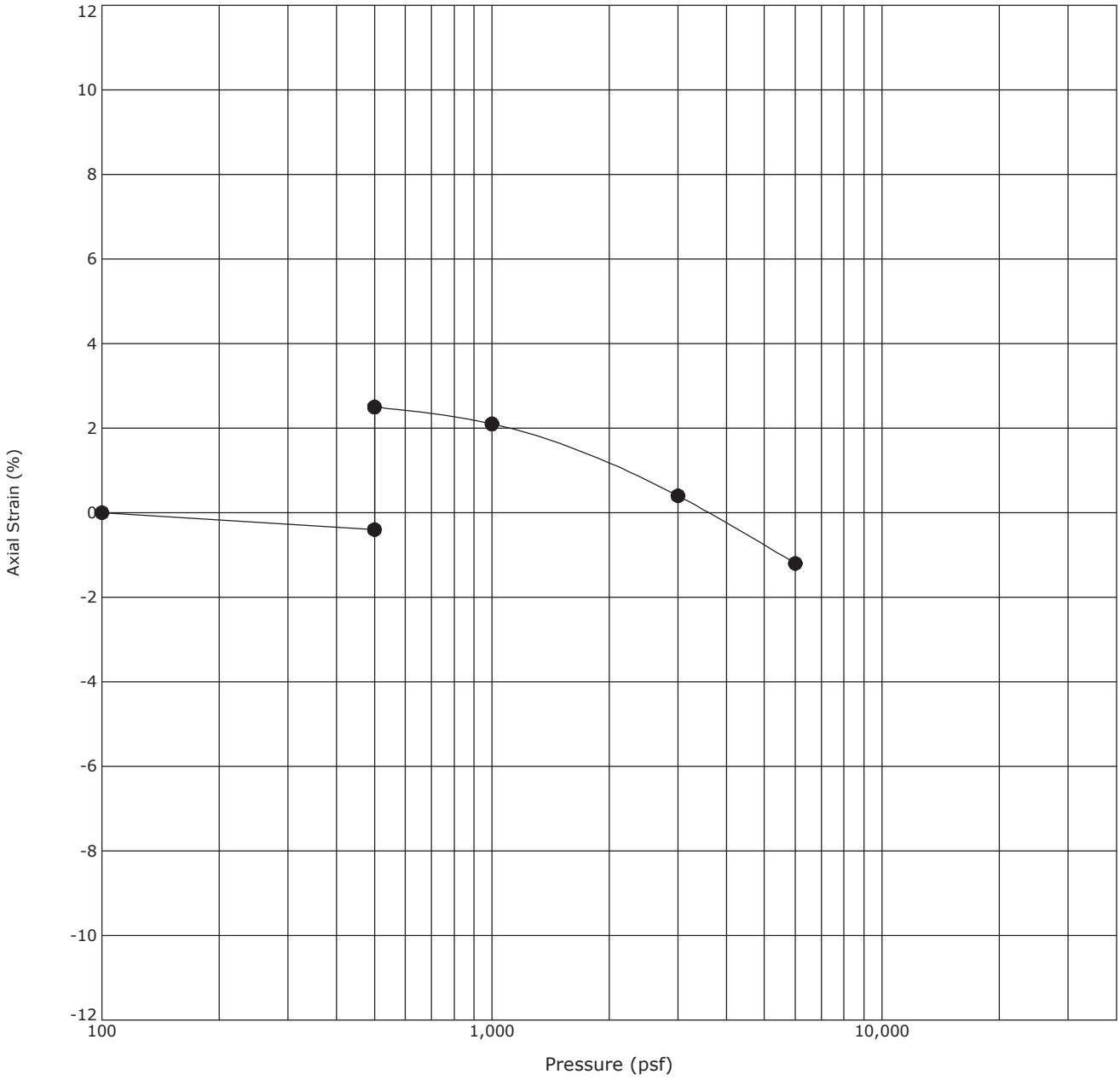
One-Dimensional Swell or Collapse



Boring ID	Depth (Ft)	Description	USCS	γ_d(pcf)	WC (%)
● 7	14 - 15	CLAYEY SANDSTONE		98	23.0

Notes: Water was added at 500 psf.

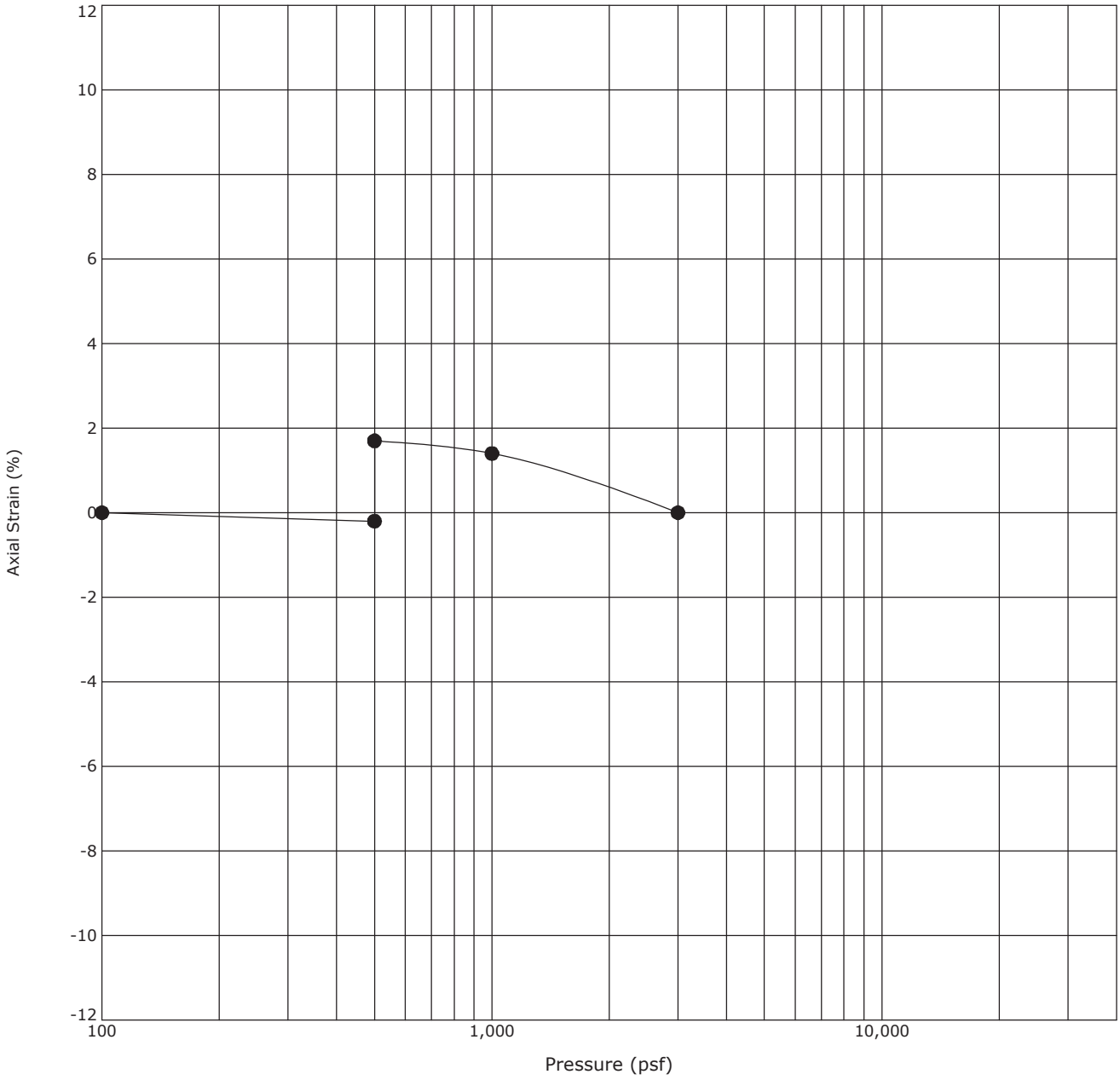
One-Dimensional Swell or Collapse



Boring ID	Depth (Ft)	Description	USCS	γ_d(pcf)	WC (%)
● 8	2 - 3	LEAN CLAY with SAND	CL	112	16.9

Notes: Water was added at 500 psf.

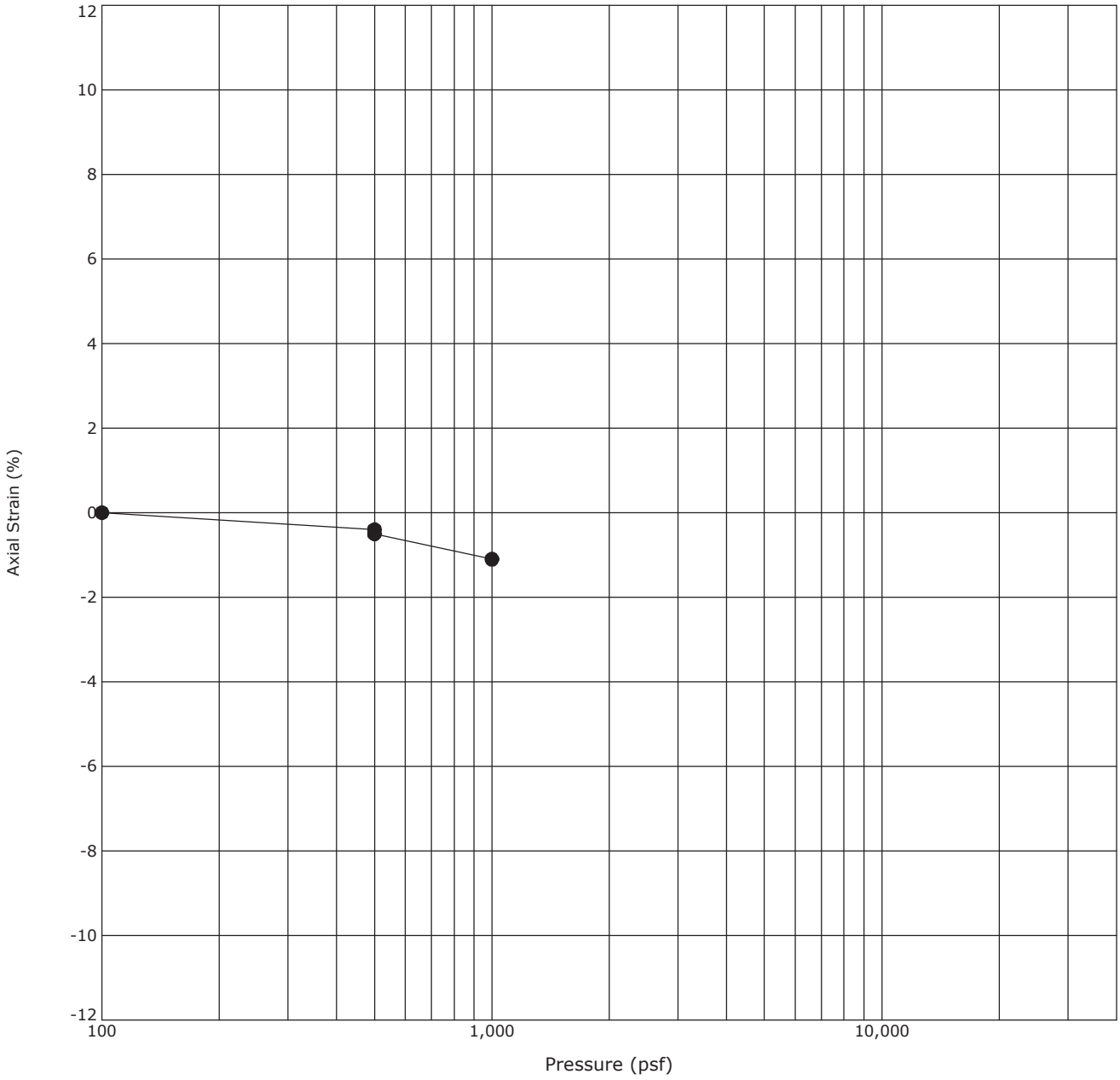
One-Dimensional Swell or Collapse



Boring ID	Depth (Ft)	Description	USCS	γ_d(pcf)	WC (%)
● 8	9 - 10	CLAYSTONE		112	20.0

Notes: Water was added at 500 psf.

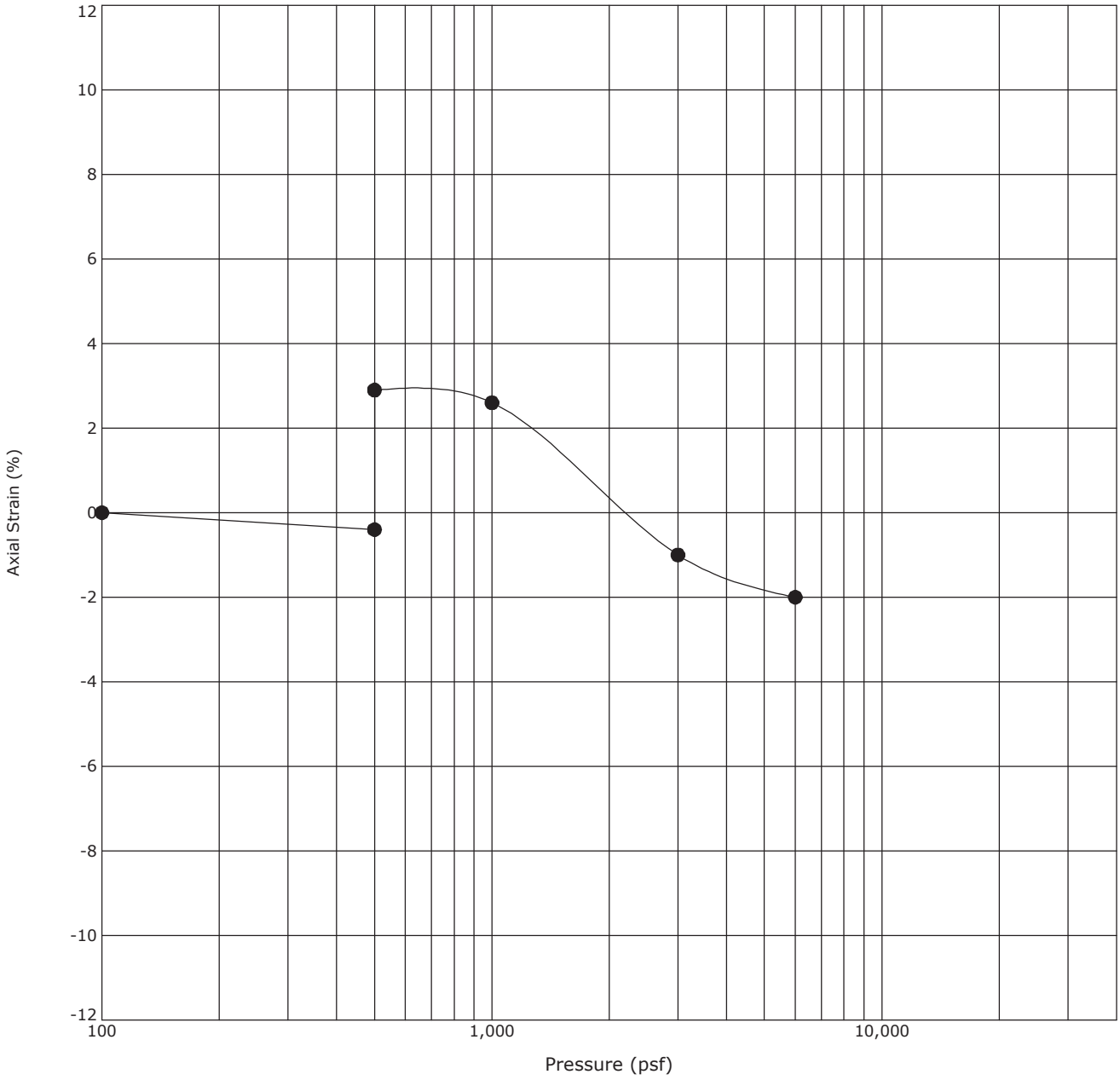
One-Dimensional Swell or Collapse



Boring ID	Depth (Ft)	Description	USCS	γ_d(pcf)	WC (%)
● 9	9 - 10	LEAN CLAY with SAND	CL	98	19.2

Notes: Water was added at 500 psf.

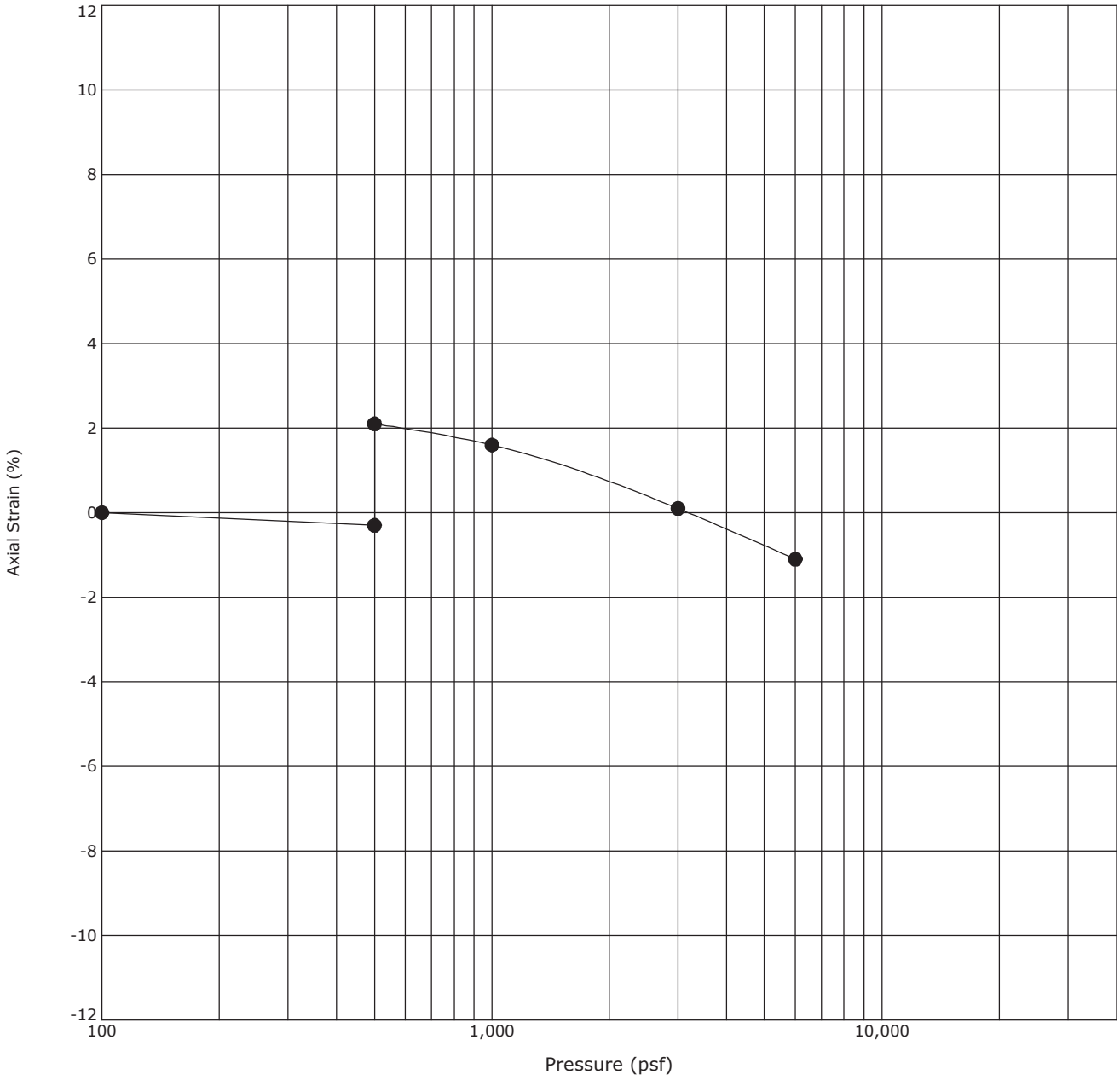
One-Dimensional Swell or Collapse



Boring ID	Depth (Ft)	Description	USCS	γ_d(pcf)	WC (%)
● 9	24 - 25	CLAYSTONE		111	20.1

Notes: Water was added at 500 psf.

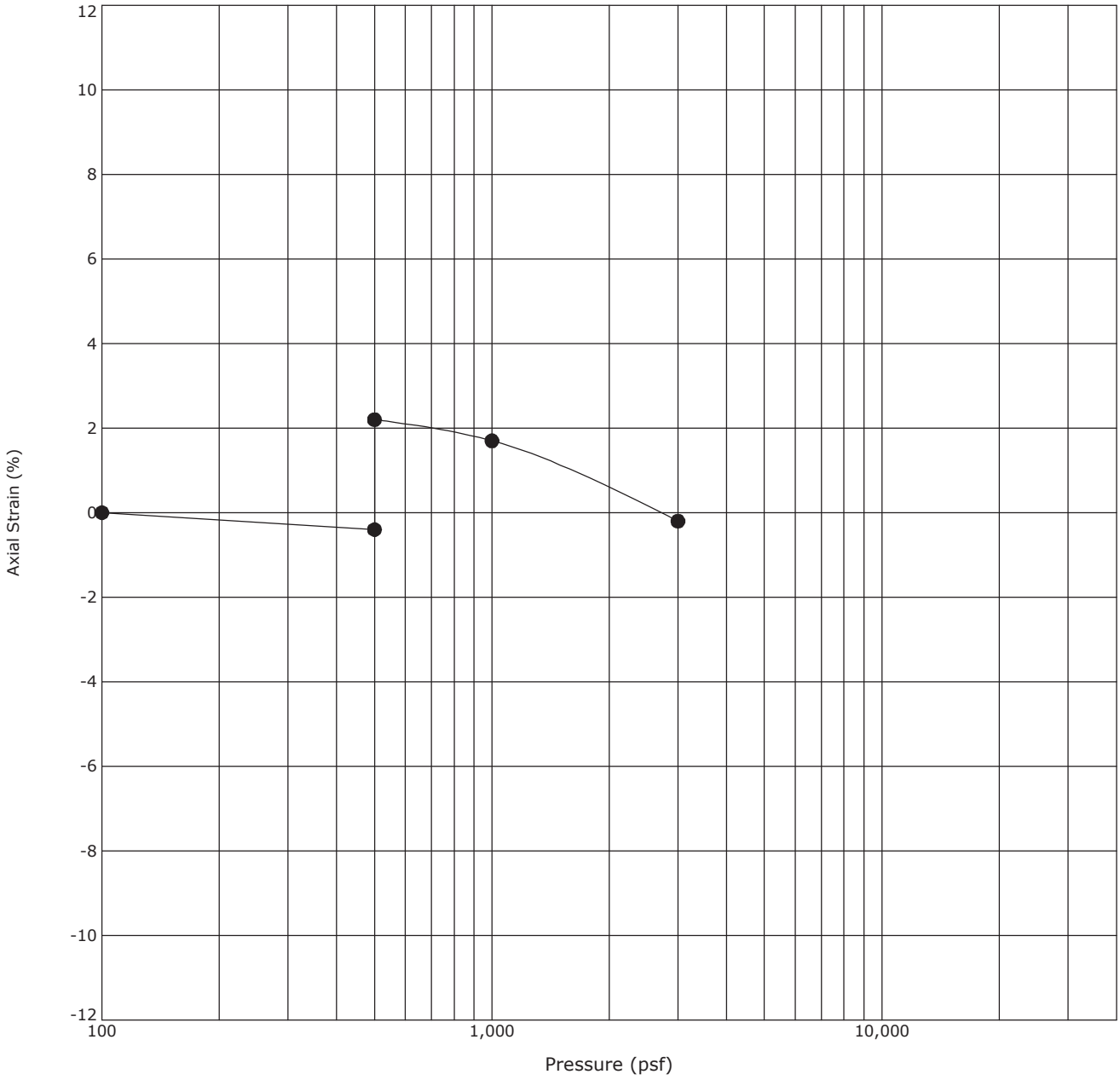
One-Dimensional Swell or Collapse



Boring ID	Depth (Ft)	Description	USCS	γ_d(pcf)	WC (%)
● 10	2 - 3	FAT CLAY	CH	106	20.9

Notes: Water was added at 500 psf.

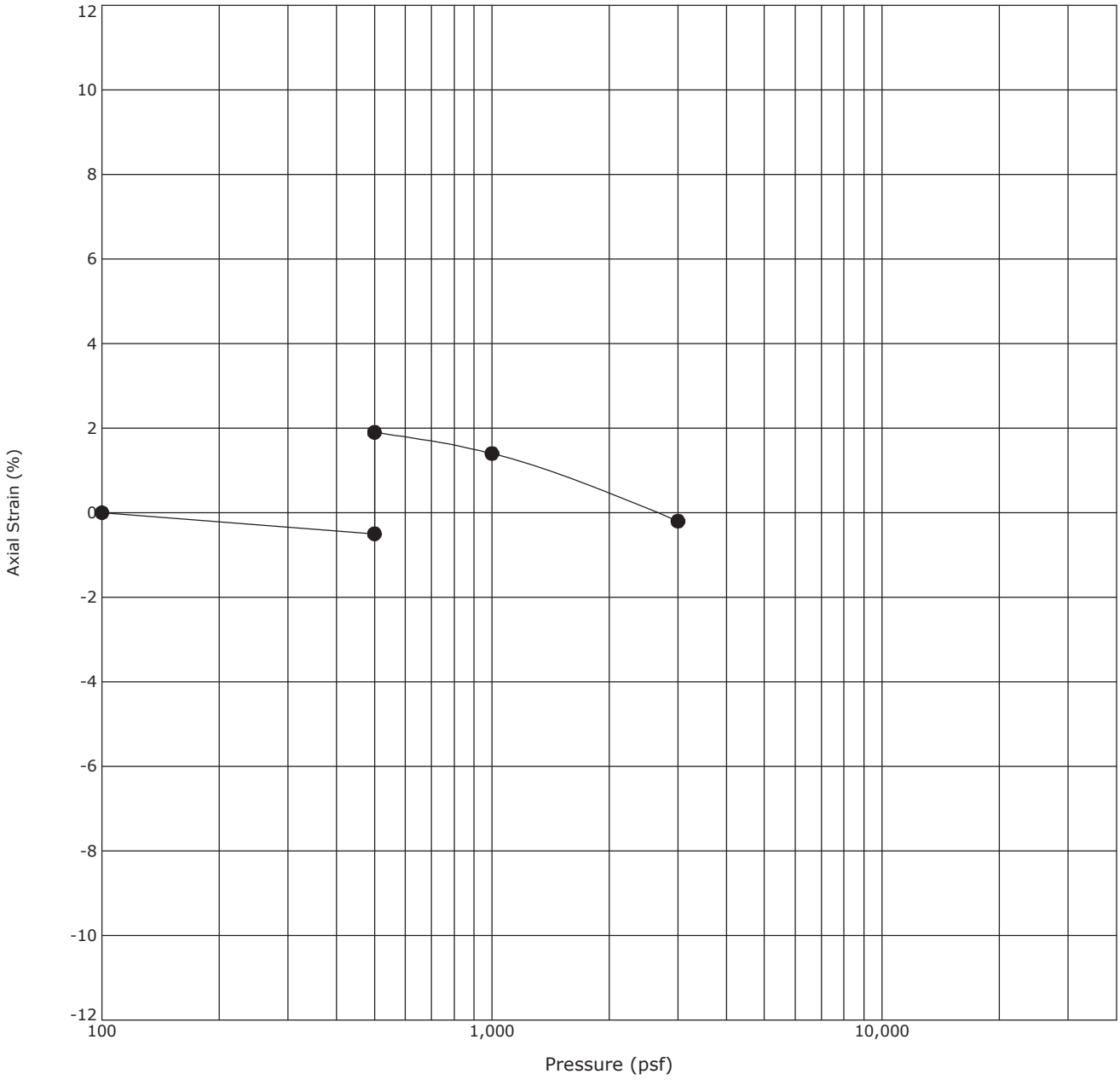
One-Dimensional Swell or Collapse



Boring ID	Depth (Ft)	Description	USCS	γ_d(pcf)	WC (%)
● 10	9 - 10	FAT CLAY	CH	103	22.3

Notes: Water was added at 500 psf.

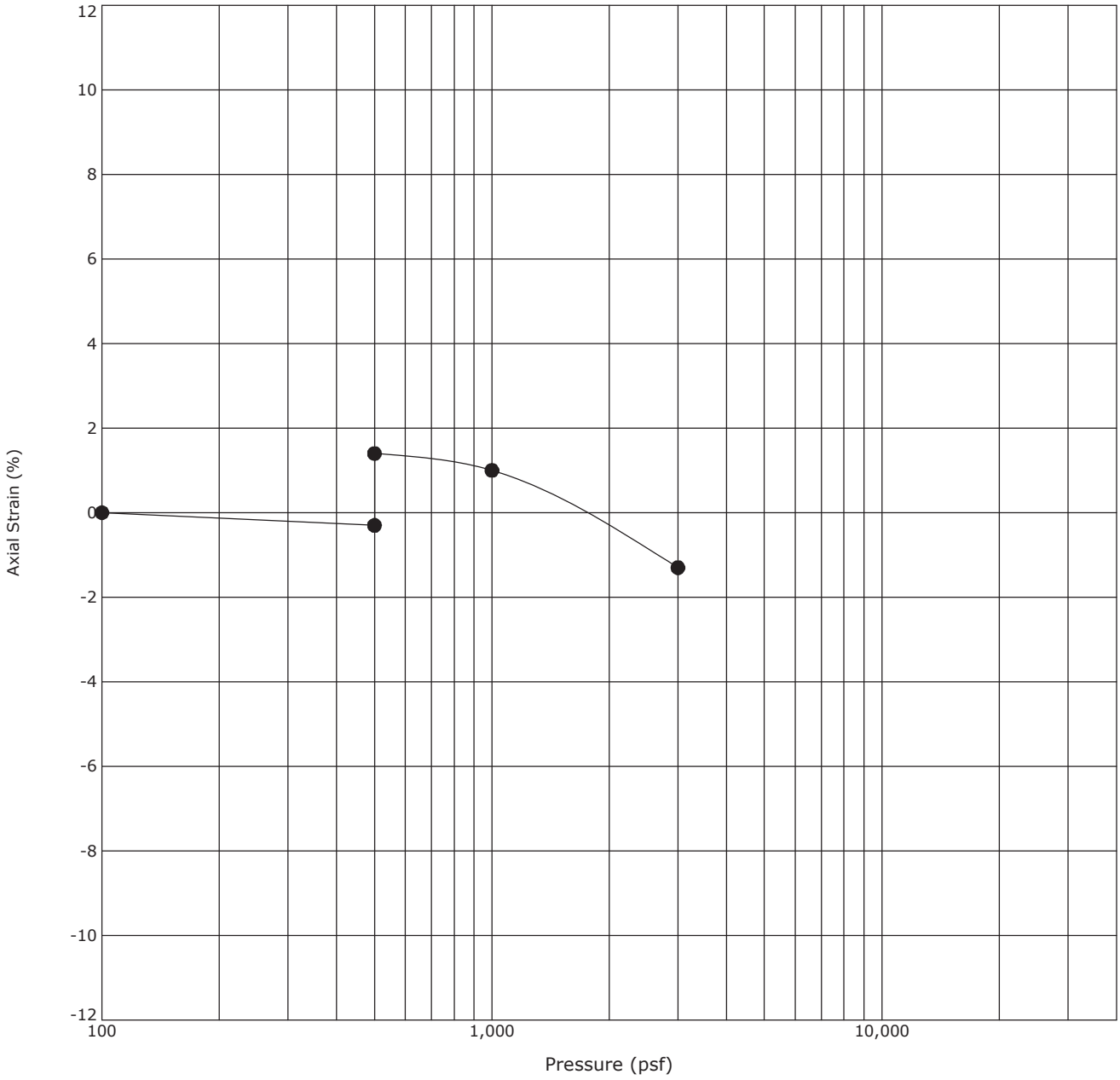
One-Dimensional Swell or Collapse



Boring ID		Depth (Ft)	Description	USCS	γ_d(pcf)	WC (%)
●	11	9 - 10	CLAYSTONE		109	19.6

Notes: Water was added at 500 psf.

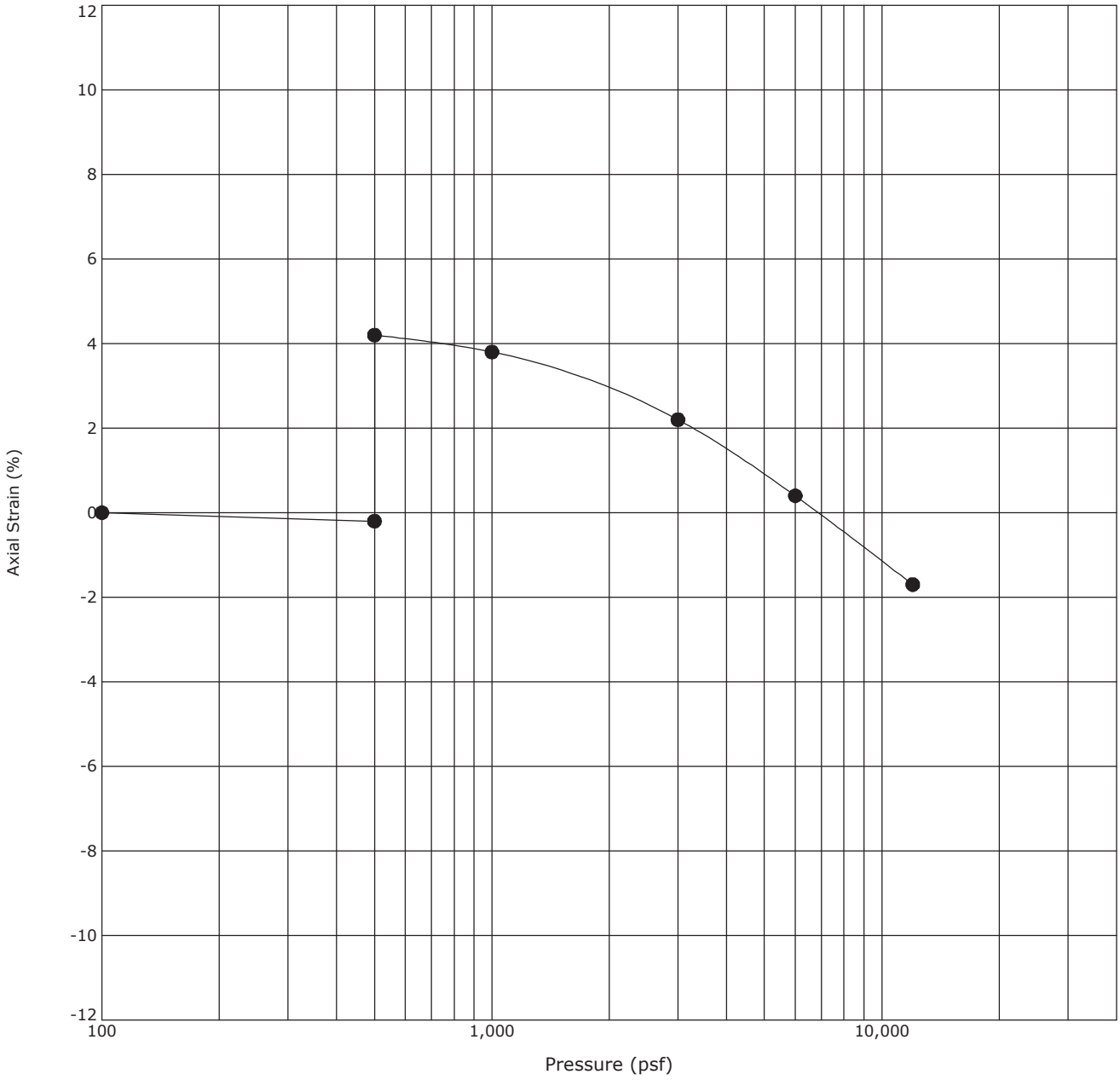
One-Dimensional Swell or Collapse



Boring ID	Depth (Ft)	Description	USCS	γ_d(pcf)	WC (%)
● 12	4 - 5	FILL - SANDY LEAN CLAY	CL	108	16.4

Notes: Water was added at 500 psf.

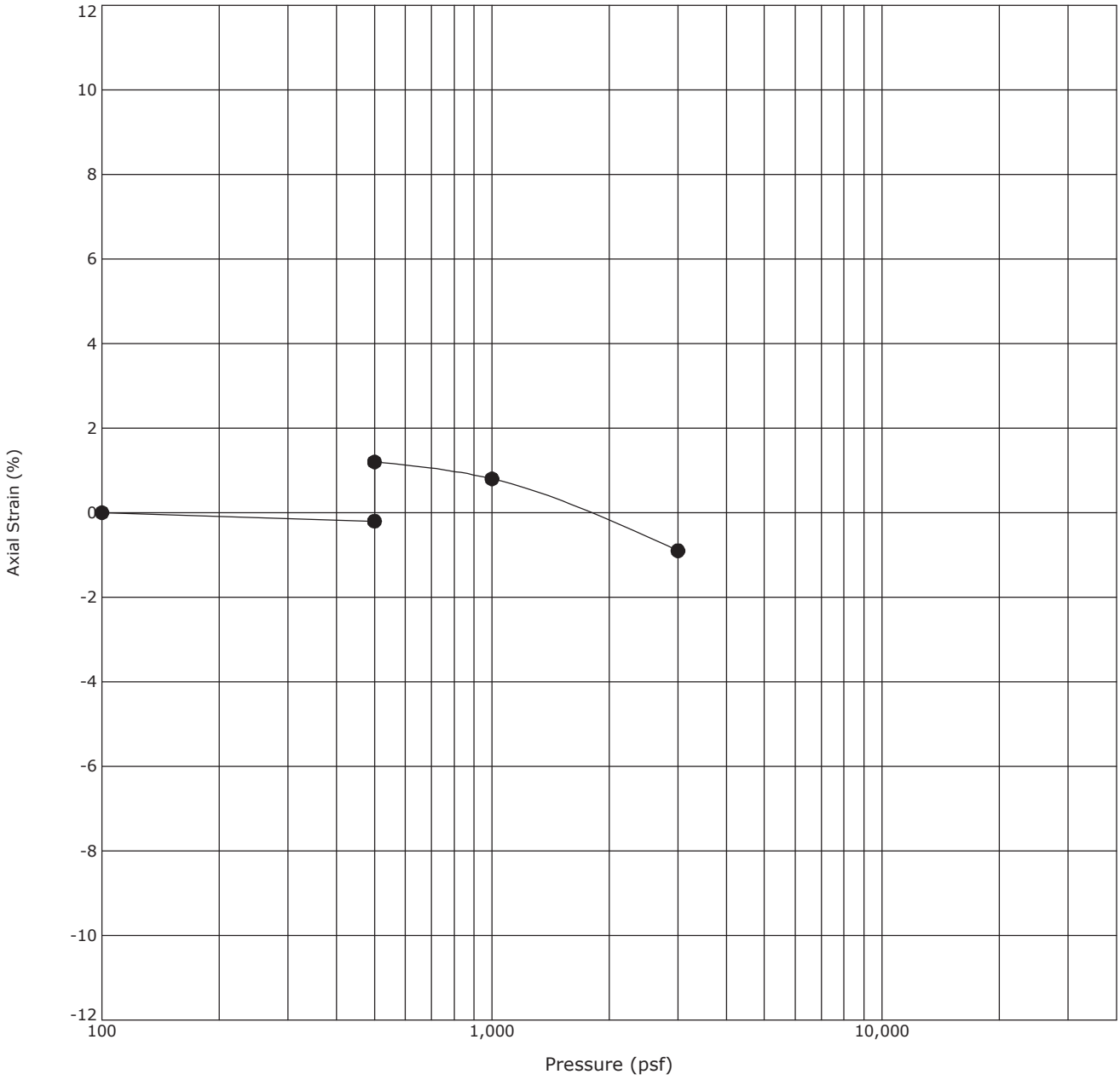
One-Dimensional Swell or Collapse



Boring ID	Depth (Ft)	Description	USCS	γ_d(pcf)	WC (%)
● 12	9 - 10	CLAYSTONE		100	23.9

Notes: Water was added at 500 psf.

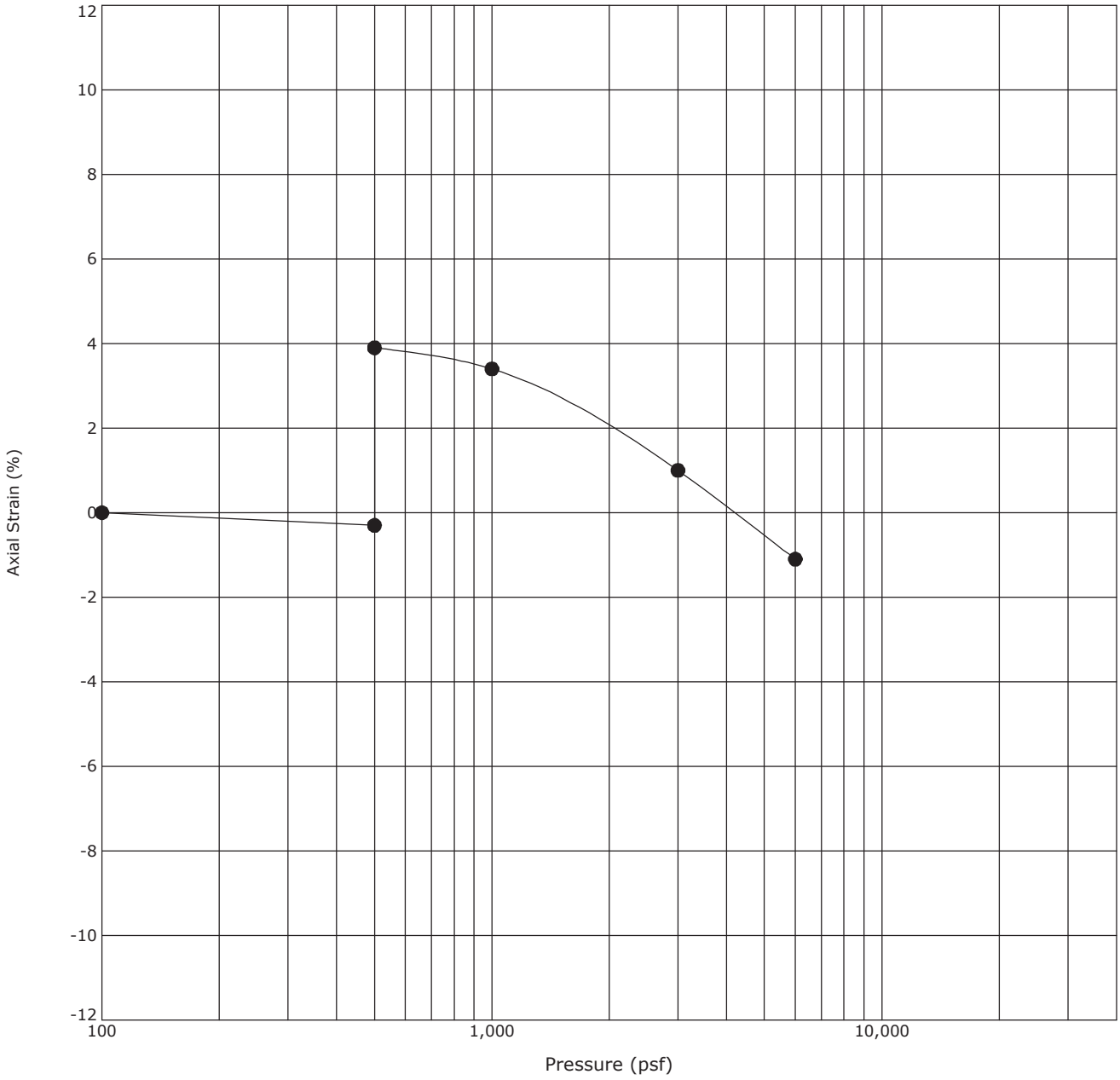
One-Dimensional Swell or Collapse



Boring ID	Depth (Ft)	Description	USCS	γ_d(pcf)	WC (%)
● 13	4 - 5	FAT CLAY	CH	101	21.7

Notes: Water was added at 500 psf.

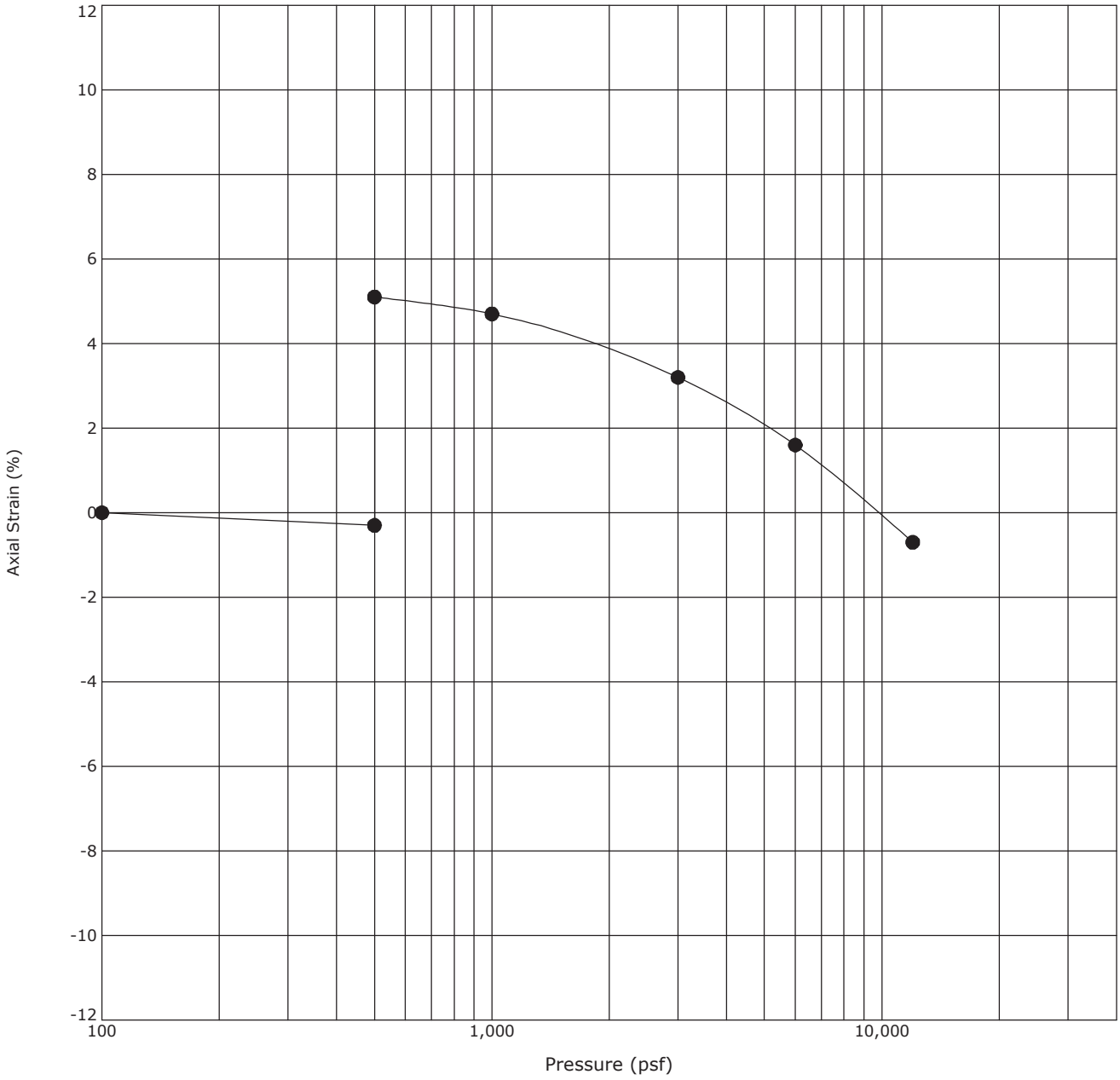
One-Dimensional Swell or Collapse



Boring ID	Depth (Ft)	Description	USCS	γ_d(pcf)	WC (%)
● 14	2 - 3	CLAYSTONE		104	21.2

Notes: Water was added at 500 psf.

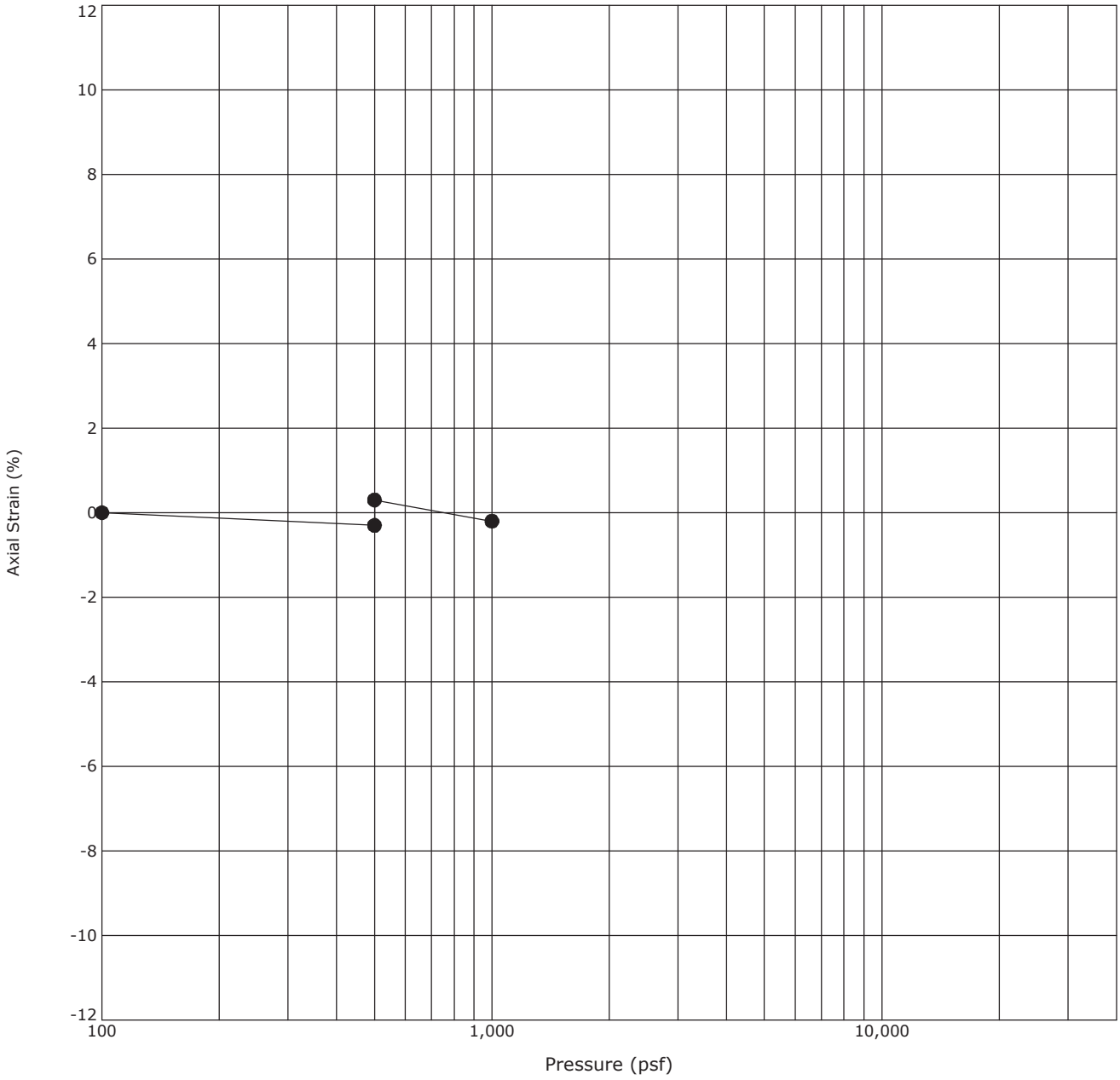
One-Dimensional Swell or Collapse



Boring ID	Depth (Ft)	Description	USCS	γ_d(pcf)	WC (%)
● 14	14 - 15	CLAYSTONE		104	22.5

Notes: Water was added at 500 psf.

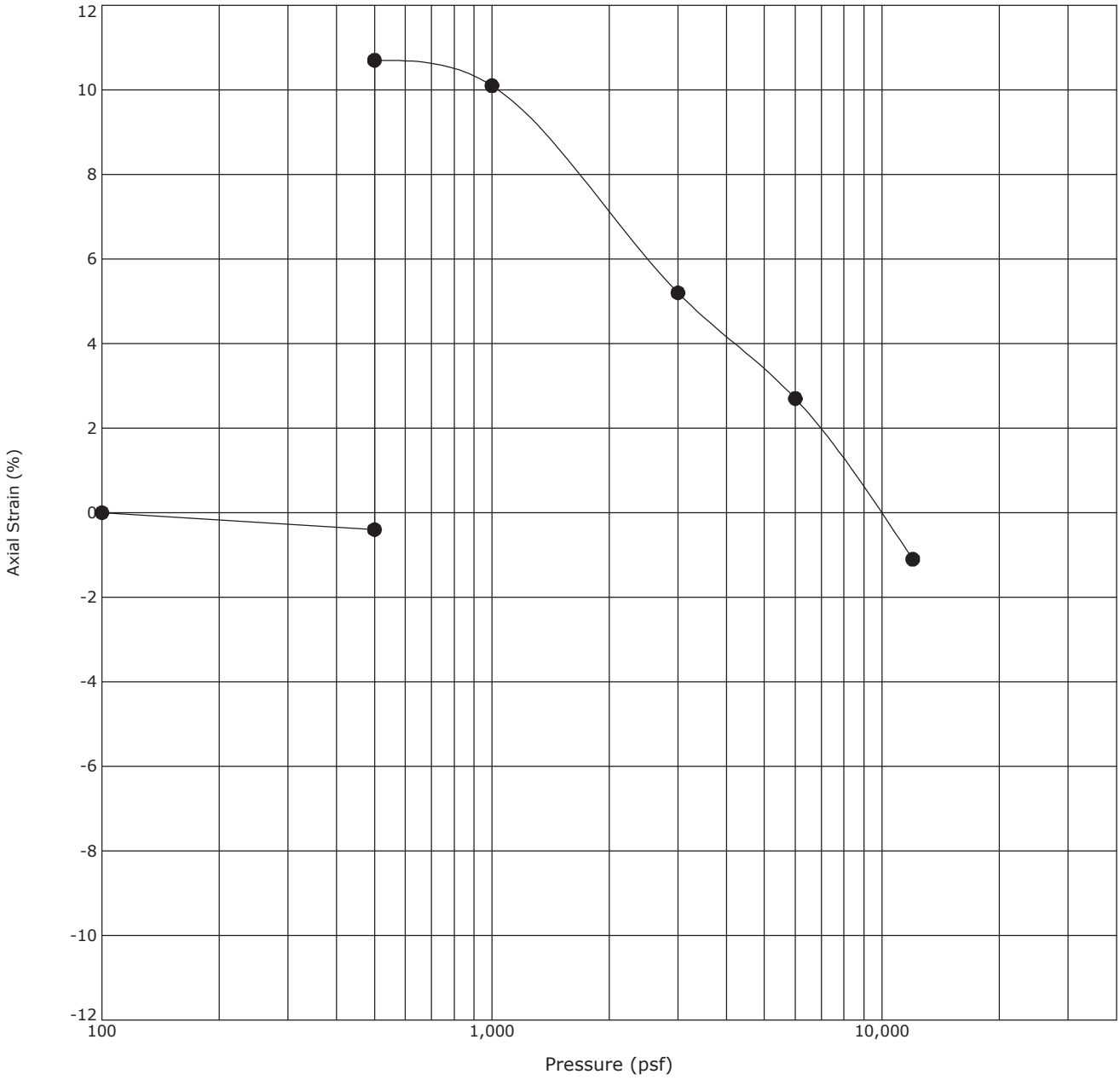
One-Dimensional Swell or Collapse



Boring ID	Depth (Ft)	Description	USCS	γ_d(pcf)	WC (%)
● 15	4 - 5	CLAYSTONE		112	17.6

Notes: Water was added at 500 psf.

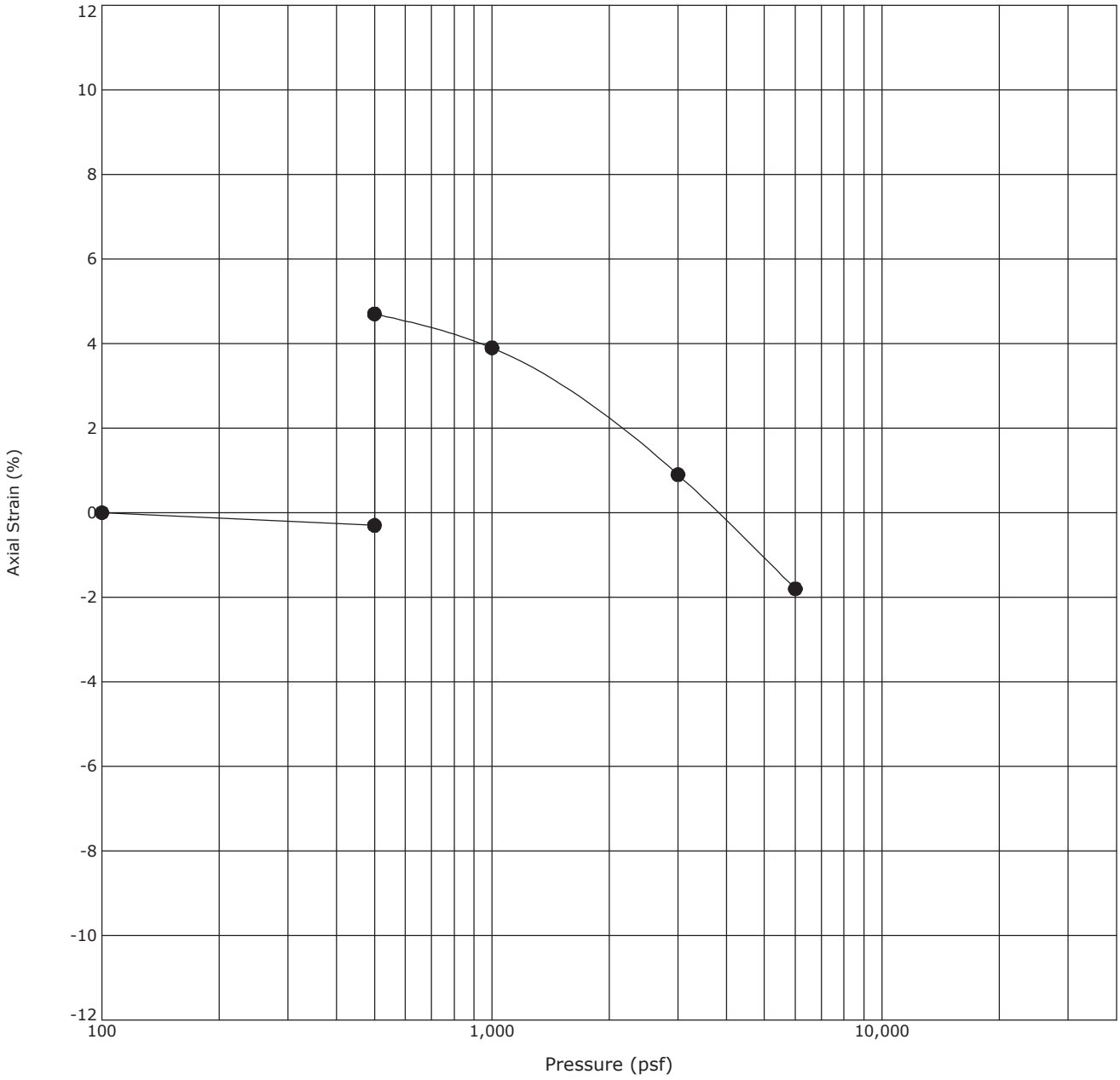
One-Dimensional Swell or Collapse



Boring ID		Depth (Ft)	Description	USCS	γ_d(pcf)	WC (%)
●	16	4 - 5	CLAYSTONE		110	17.7

Notes: Water was added at 500 psf.

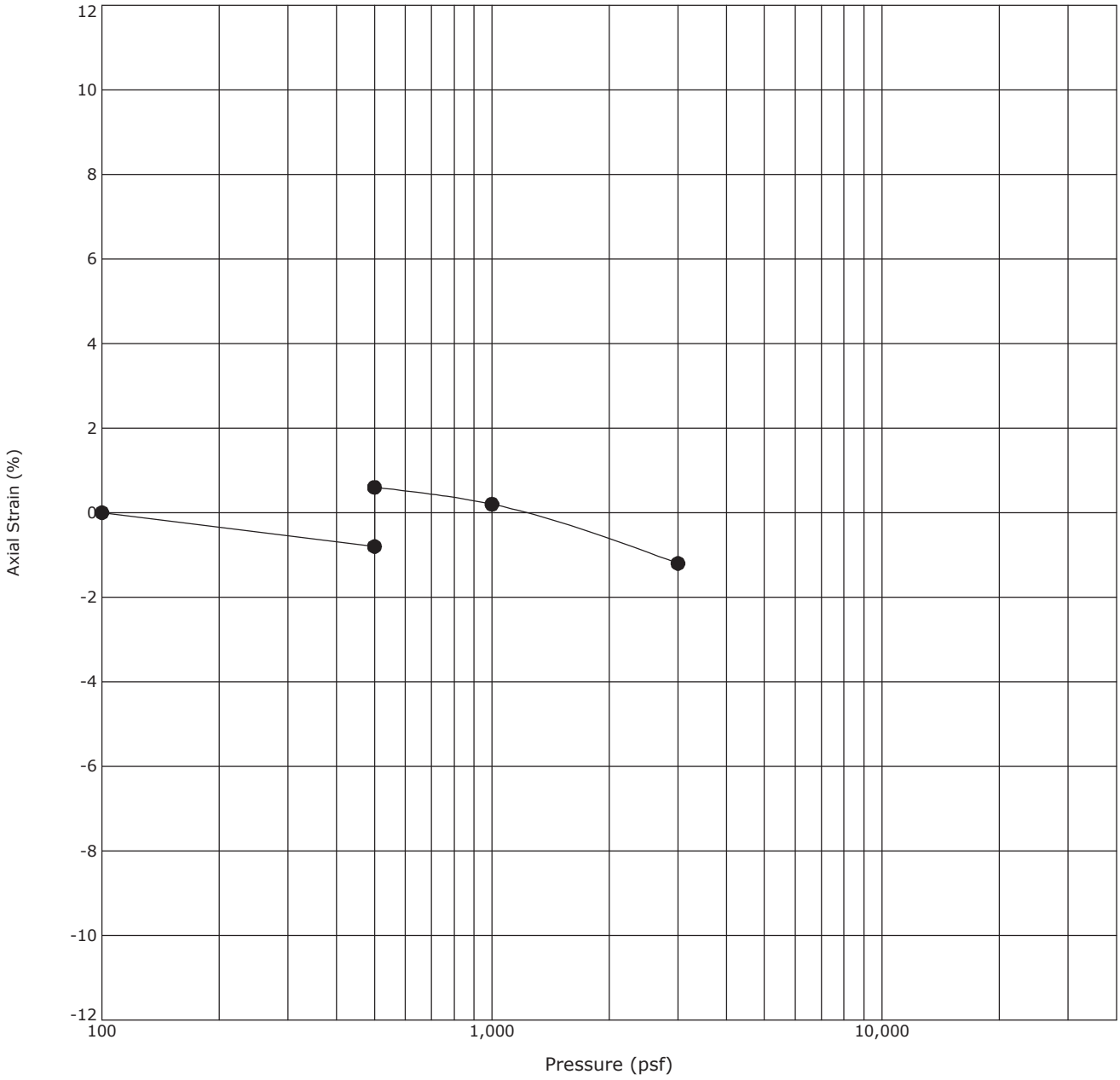
One-Dimensional Swell or Collapse



Boring ID		Depth (Ft)	Description	USCS	γ_d(pcf)	WC (%)
●	16	14 - 15	CLAYSTONE		106	16.0

Notes: Water was added at 500 psf.

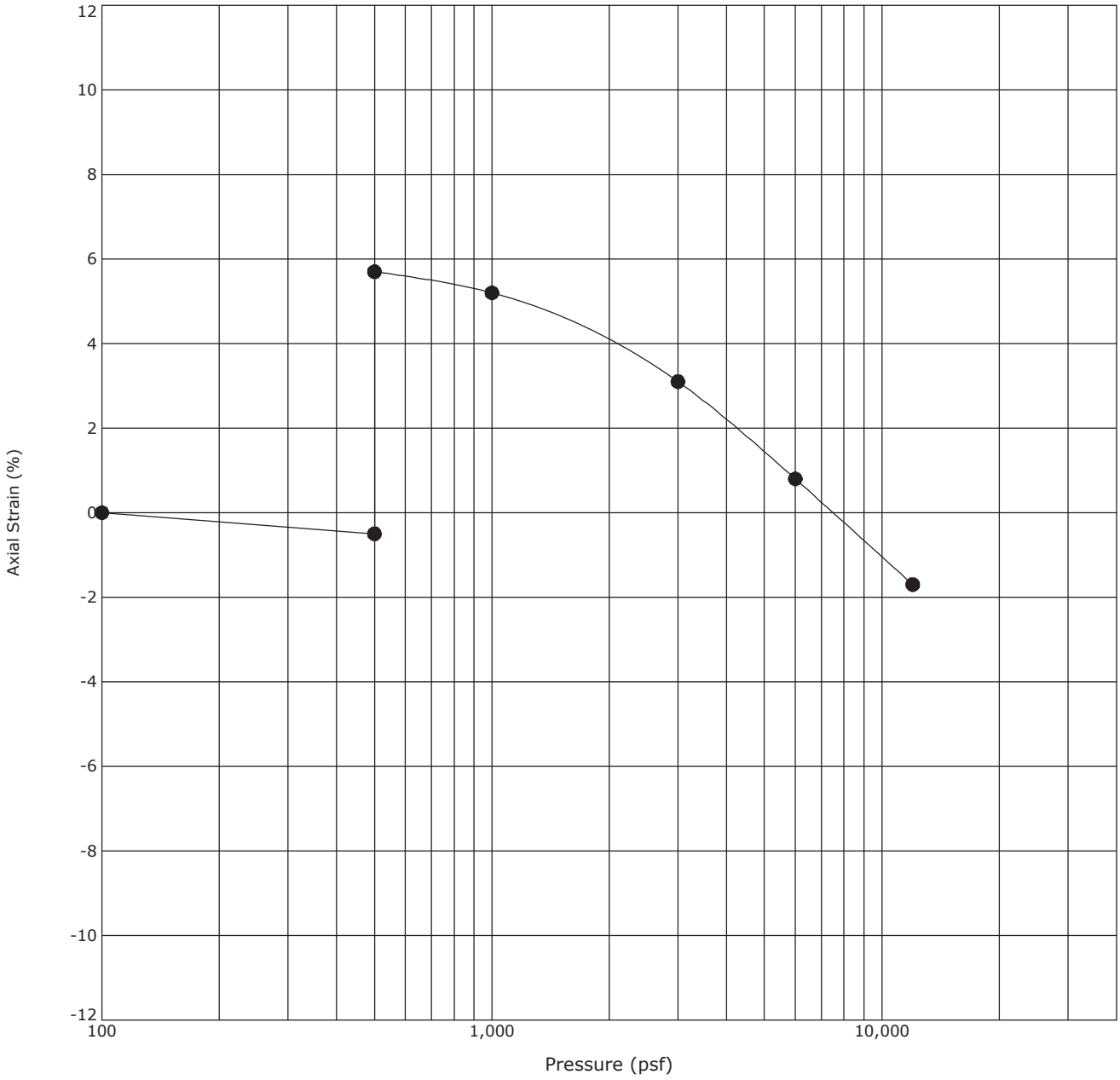
One-Dimensional Swell or Collapse



Boring ID	Depth (Ft)	Description	USCS	γ_d(pcf)	WC (%)
● 17	2 - 3	CLAYSTONE		117	16.6

Notes: Water was added at 500 psf.

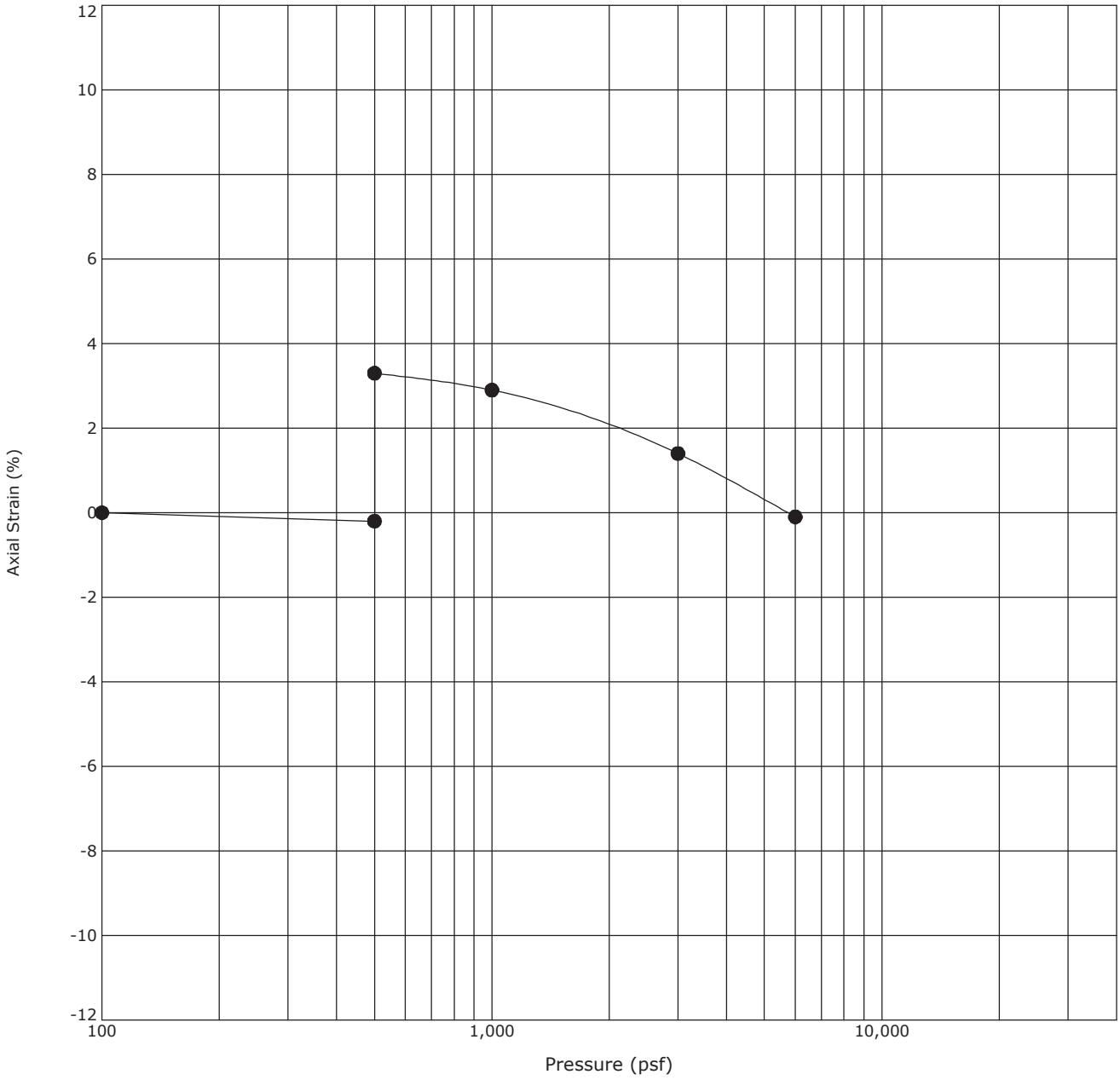
One-Dimensional Swell or Collapse



Boring ID		Depth (Ft)	Description	USCS	γ_d(pcf)	WC (%)
●	17	9 - 10	CLAYSTONE		105	22.0

Notes: Water was added at 500 psf.

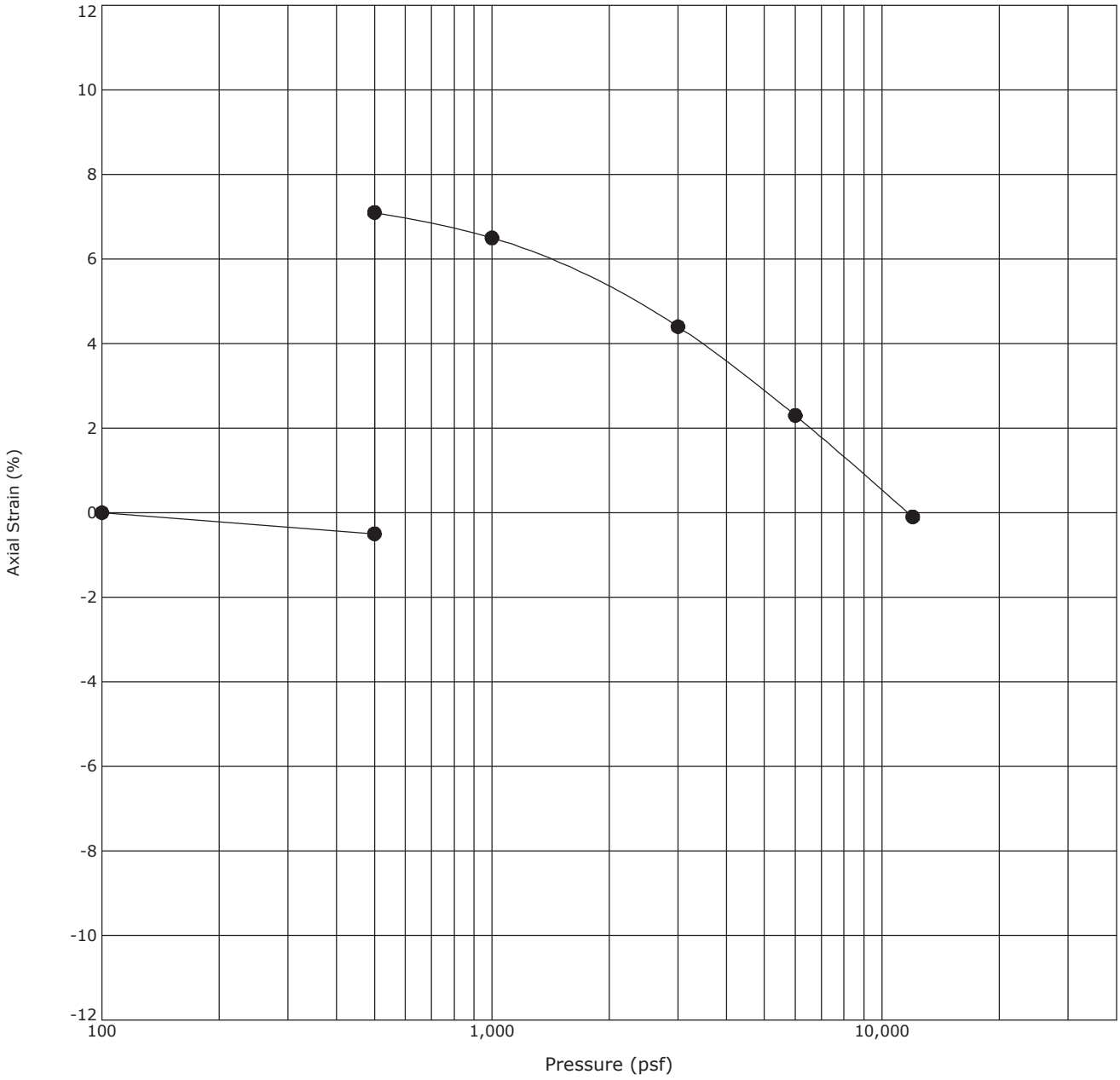
One-Dimensional Swell or Collapse



Boring ID		Depth (Ft)	Description	USCS	γ_d(pcf)	WC (%)
●	18	4 - 5	CLAYSTONE		107	20.3

Notes: Water was added at 500 psf.

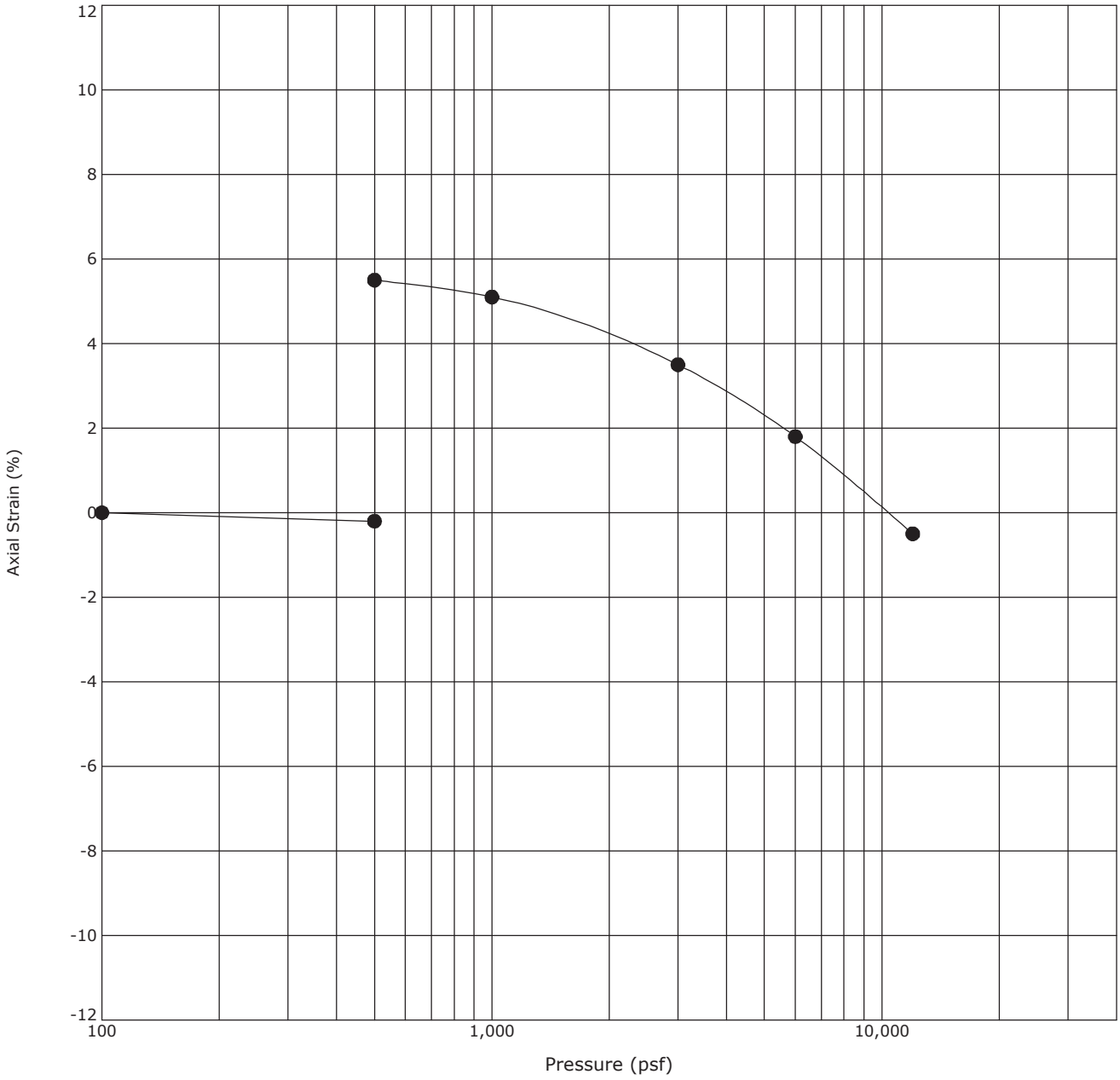
One-Dimensional Swell or Collapse



Boring ID		Depth (Ft)	Description	USCS	γ_d(pcf)	WC (%)
●	19	9 - 10	CLAYSTONE		115	14.5

Notes: Water was added at 500 psf.

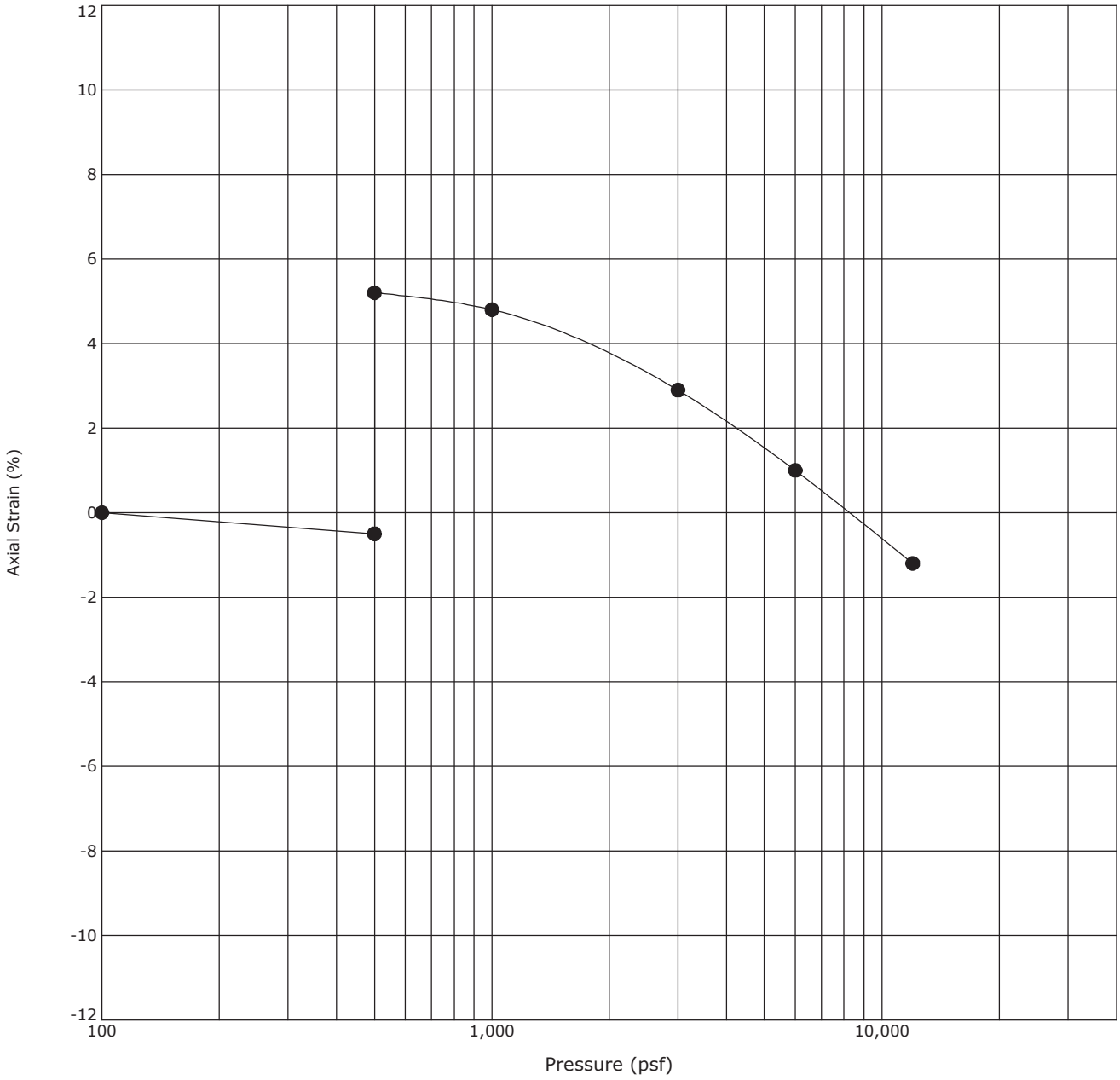
One-Dimensional Swell or Collapse



Boring ID		Depth (Ft)	Description	USCS	γ_d(pcf)	WC (%)
●	20	4 - 5	CLAYSTONE		104	21.9

Notes: Water was added at 500 psf.

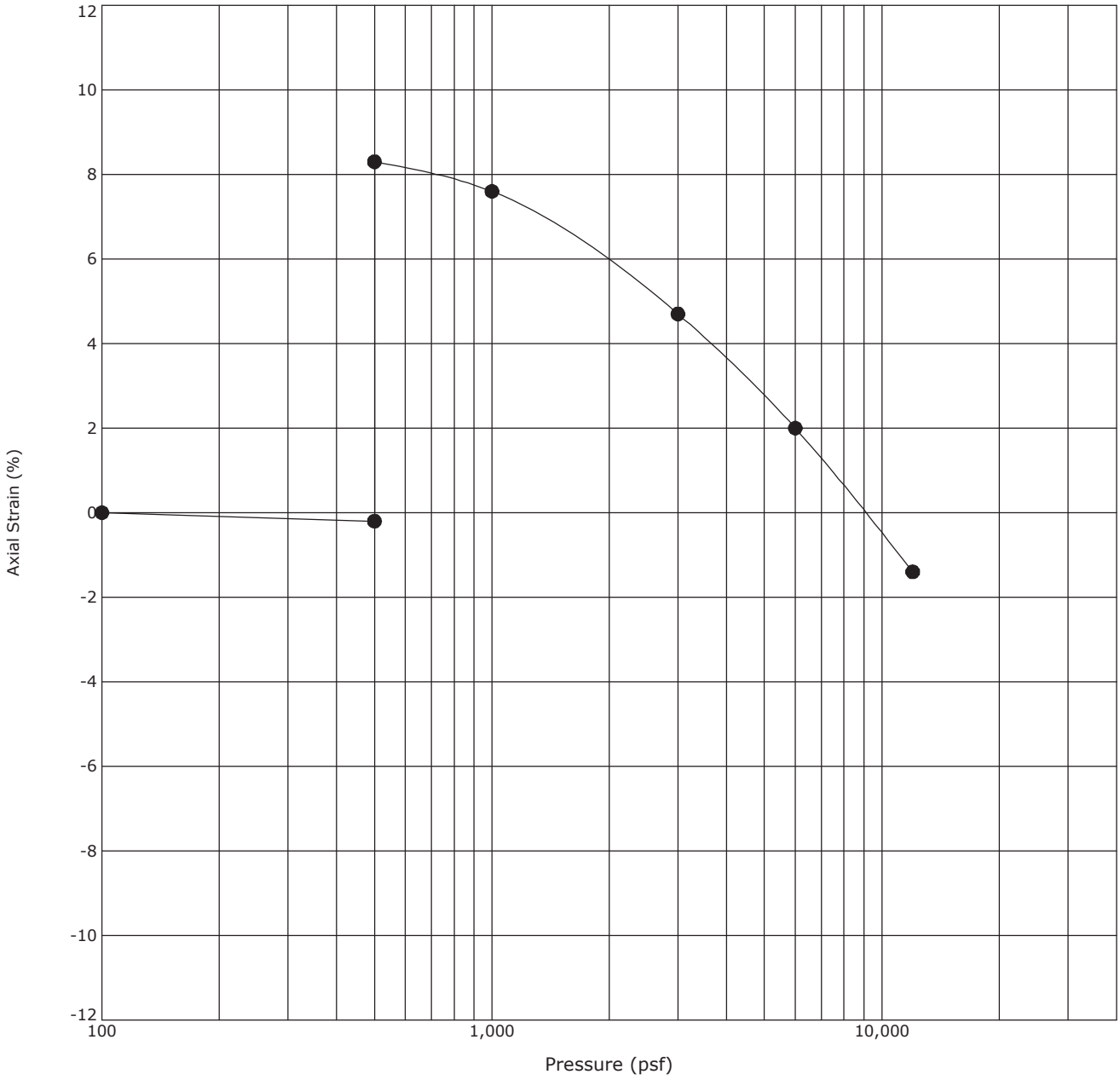
One-Dimensional Swell or Collapse



Boring ID	Depth (Ft)	Description	USCS	γ_d(pcf)	WC (%)
● 21	9 - 10	CLAYSTONE		110	20.1

Notes: Water was added at 500 psf.

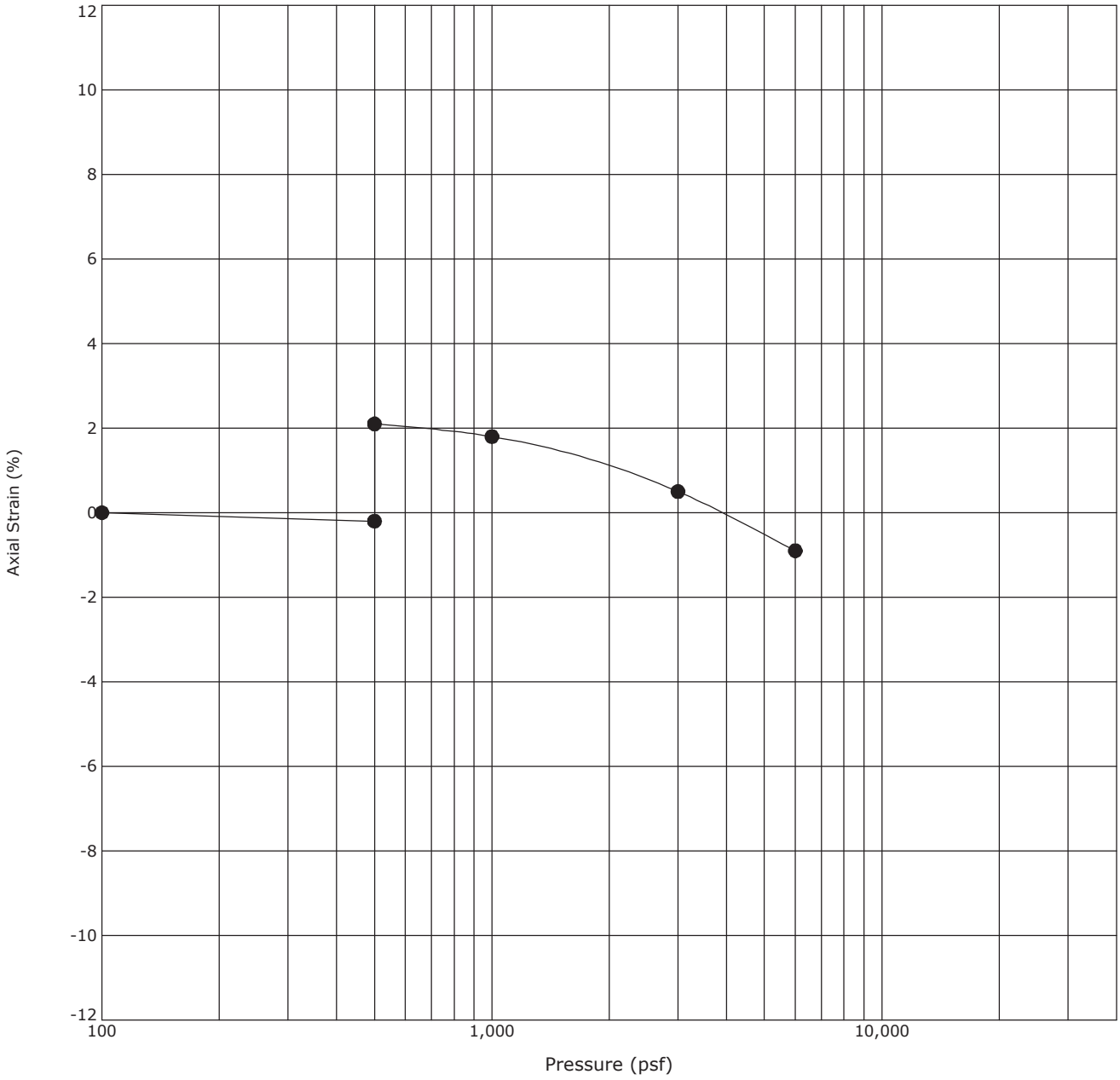
One-Dimensional Swell or Collapse



Boring ID	Depth (Ft)	Description	USCS	γ_d(pcf)	WC (%)
● 21	14 - 15	CLAYSTONE		109	18.4

Notes: Water was added at 500 psf.

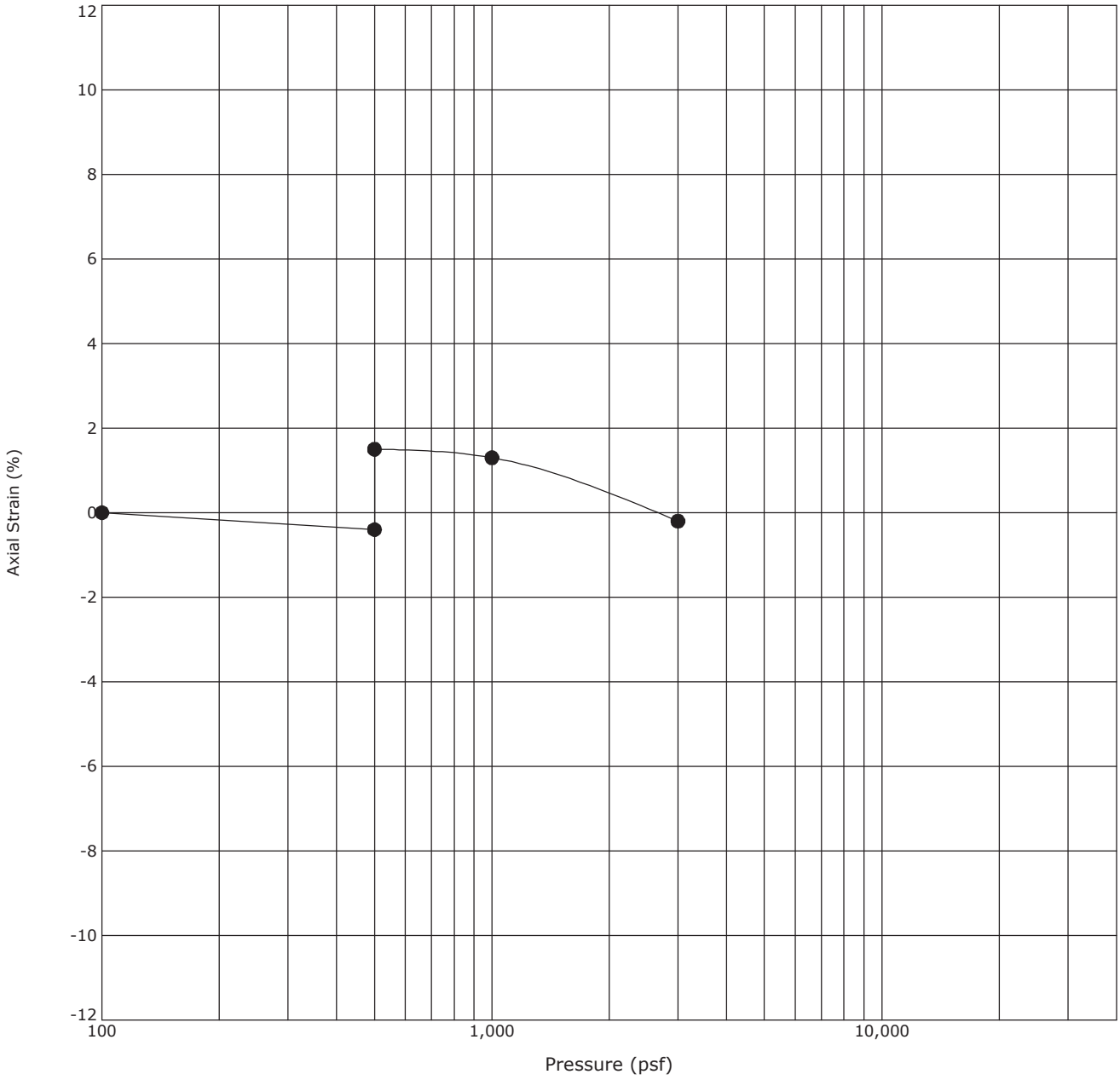
One-Dimensional Swell or Collapse



Boring ID	Depth (Ft)	Description	USCS	γ_d(pcf)	WC (%)
● 22	4 - 5	CLAYSTONE		119	15.8

Notes: Water was added at 500 psf.

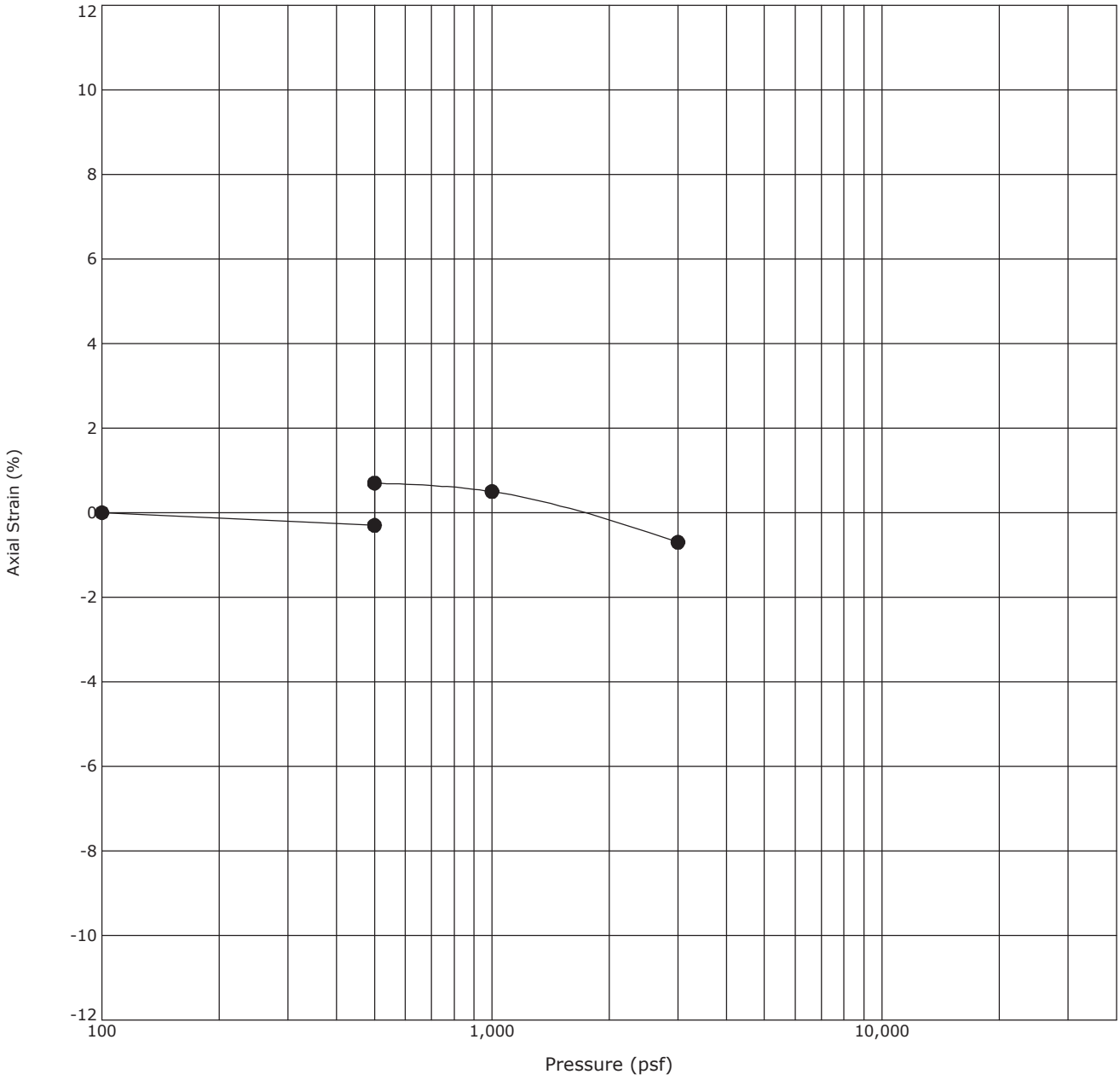
One-Dimensional Swell or Collapse



Boring ID	Depth (Ft)	Description	USCS	γ_d(pcf)	WC (%)
● 9	0 - 5	LEAN CLAY with SAND	CL	101	17.9

Notes: Water was added at 500 psf. Sample remolded to about 95% of materials maximum dry density and at about 1% below the optimum moisture content.

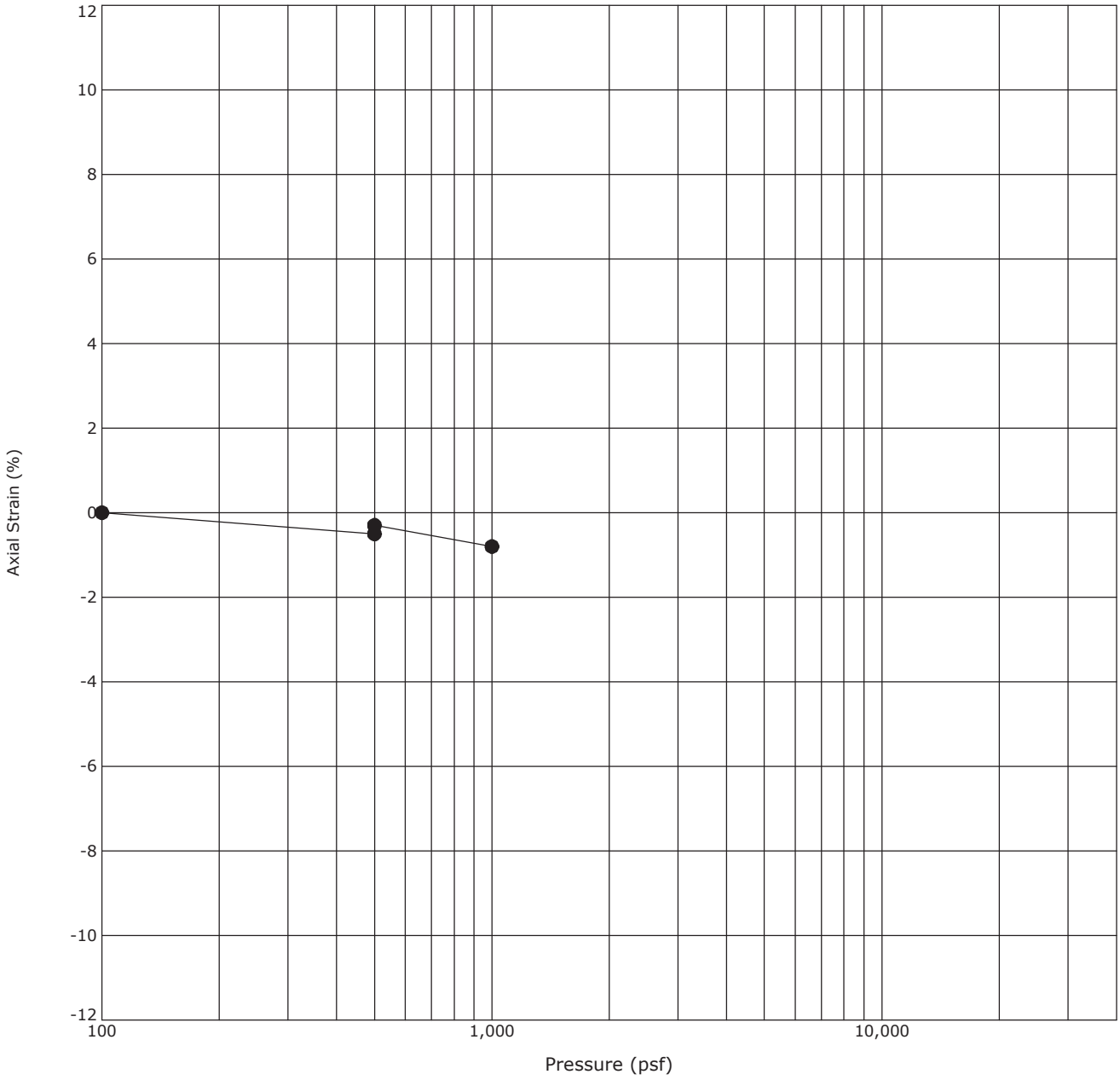
One-Dimensional Swell or Collapse



Boring ID	Depth (Ft)	Description	USCS	γ_d(pcf)	WC (%)
● 9	0 - 5	LEAN CLAY with SAND	CL	102	19.5

Notes: Water was added at 500 psf. Sample remolded to about 95% of materials maximum dry density and at about 0.6% above the optimum moisture content.

One-Dimensional Swell or Collapse

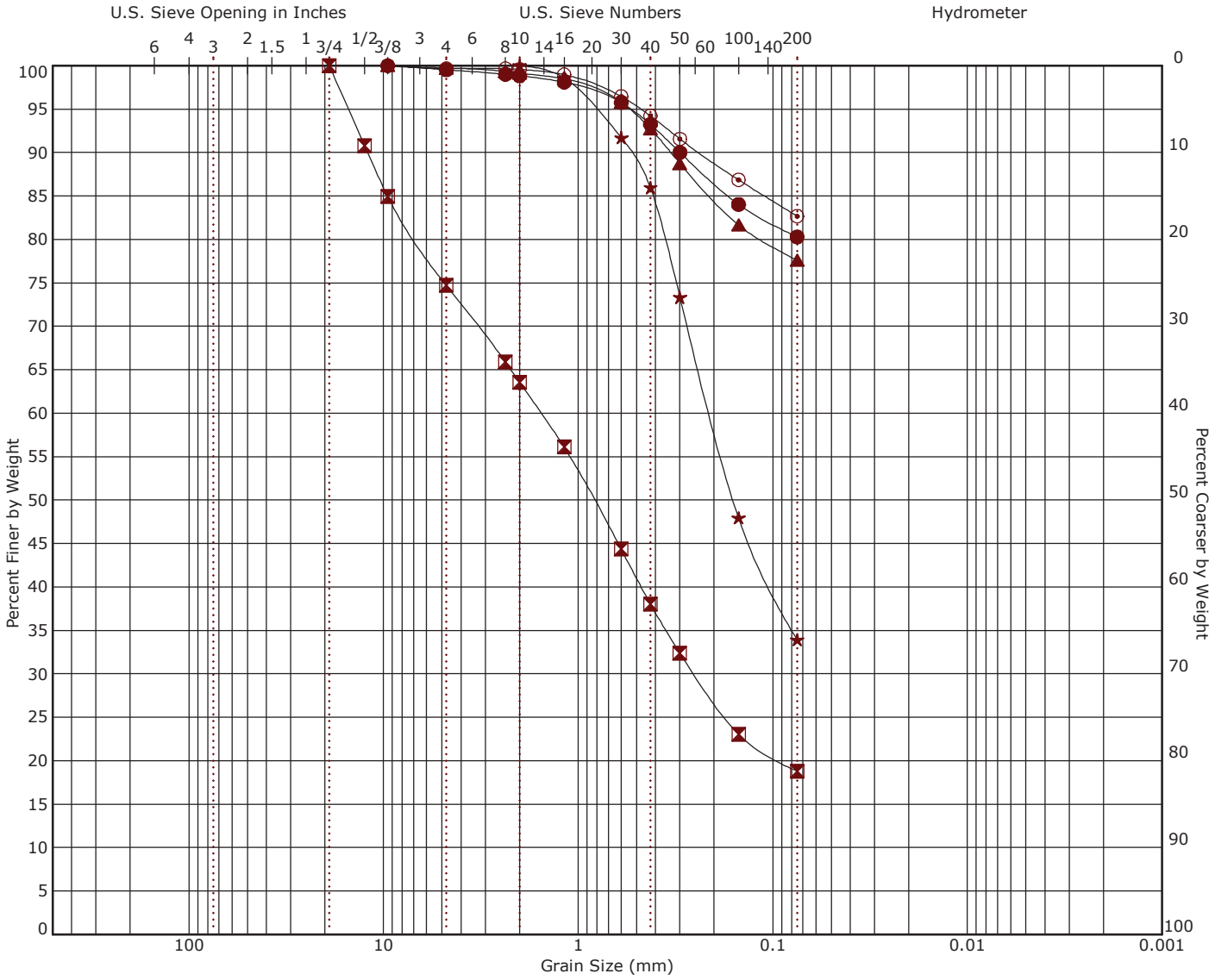


Boring ID	Depth (Ft)	Description	USCS	γ_d(pcf)	WC (%)
● 9	0 - 5	LEAN CLAY with SAND	CL	102	21.1

Notes: Water was added at 500 psf. Sample remolded to about 95% of materials maximum dry density and at about 2% above the optimum moisture content.

Grain Size Distribution

ASTM D422 / ASTM C136



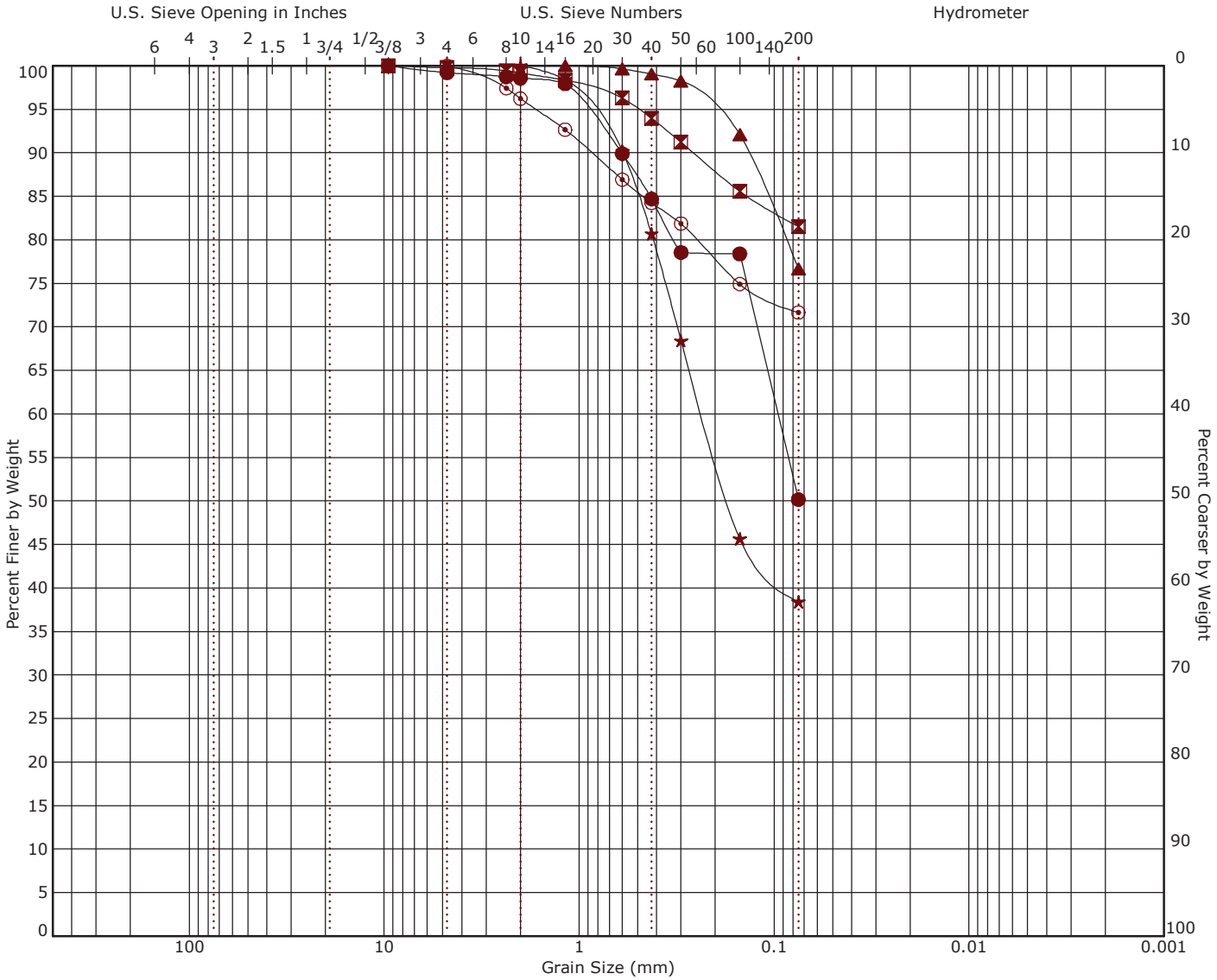
Cobbles	Gravel		Sand			Silt or Clay
	coarse	fine	coarse	medium	fine	

Boring ID		Depth (Ft)	USCS Classification	USCS	AASHTO	LL	PL	PI	Cc	Cu
●	1	0 - 5	LEAN CLAY with SAND	CL	A-7-6 (22)	44	16	28		
⊠	1	9 - 10	SILTY SAND with GRAVEL	SM	A-1-b (0)	NP	NP	NP		
▲	2	0 - 5	LEAN CLAY with SAND	CL	A-7-6 (20)	42	14	28		
★	2	14 - 15	CLAYEY SAND	SC	A-2-6 (1)	37	20	17		
⊙	3	0 - 5	LEAN CLAY with SAND	CL	A-7-6 (29)	49	14	35		

Boring ID		Depth (Ft)	D_{100}	D_{60}	D_{30}	D_{10}	%Cobbles	%Gravel	%Sand	%Fines	%Silt	%Clay
●	1	0 - 5	9.5				0.0	0.5	19.2	80.3		
⊠	1	9 - 10	19	1.555	0.251		0.0	25.3	55.9	18.8		
▲	2	0 - 5	9.5				0.0	0.3	22.1	77.6		
★	2	14 - 15	2	0.208			0.0	0.0	66.1	33.9		
⊙	3	0 - 5	9.5				0.0	0.3	17.0	82.7		

Grain Size Distribution

ASTM D422 / ASTM C136



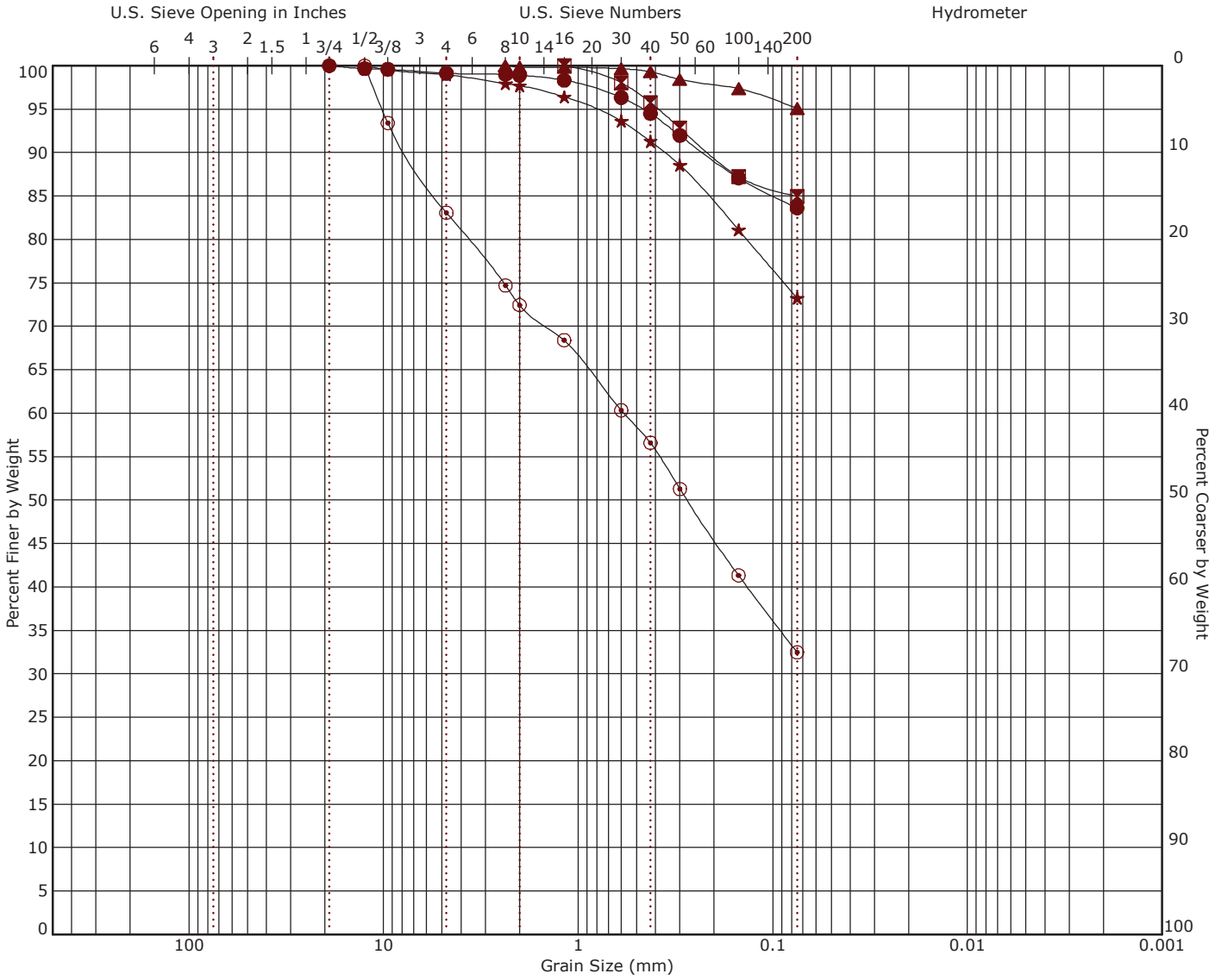
Cobbles	Gravel		Sand			Silt or Clay
	coarse	fine	coarse	medium	fine	

Boring ID		Depth (Ft)	USCS Classification	USCS	AASHTO	LL	PL	PI	Cc	Cu
●	3	14 - 15	SANDY FAT CLAY	CH	A-7-6 (11)	50	18	32		
⊠	4	0 - 5	FAT CLAY with SAND	CH	A-7-6 (34)	57	16	41		
▲	4	19 - 20	LEAN CLAY with SAND	CL	A-7-6 (21)	48	20	28		
★	4	34 - 35	CLAYEY SAND	SC	A-6 (2)	37	21	16		
⊙	5	4 - 5	FAT CLAY with SAND	CH	A-7-6 (32)	63	18	45		

Boring ID		Depth (Ft)	D_{100}	D_{60}	D_{30}	D_{10}	%Cobbles	%Gravel	%Sand	%Fines	%Silt	%Clay
●	3	14 - 15	9.5	0.095			0.0	0.8	49.1	50.2		
⊠	4	0 - 5	9.5				0.0	0.2	18.2	81.5		
▲	4	19 - 20	1.18				0.0	0.0	23.3	76.7		
★	4	34 - 35	2	0.232			0.0	0.0	61.6	38.4		
⊙	5	4 - 5	4.75				0.0	0.0	28.3	71.7		

Grain Size Distribution

ASTM D422 / ASTM C136



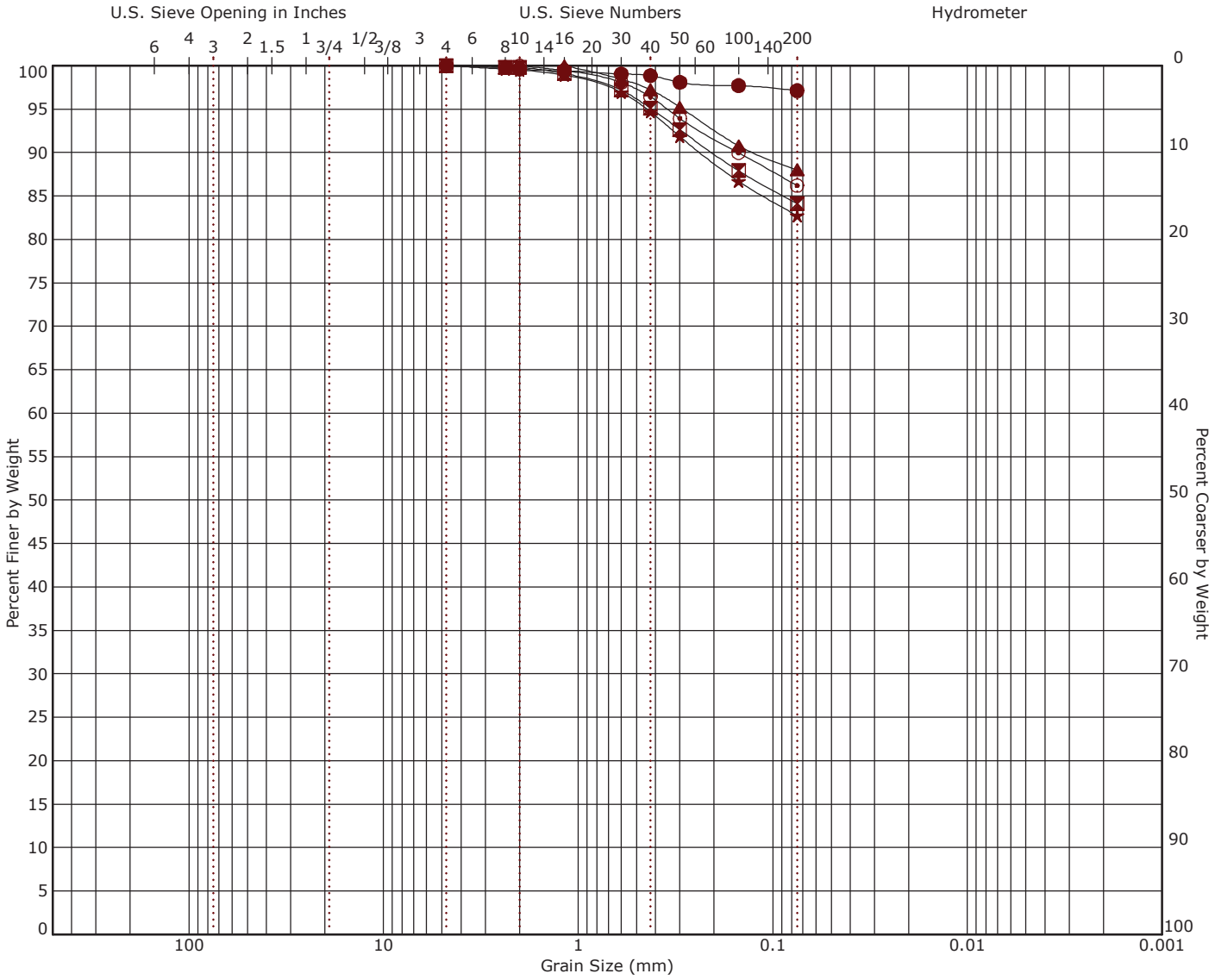
Cobbles	Gravel		Sand			Silt or Clay
	coarse	fine	coarse	medium	fine	

Boring ID		Depth (Ft)	USCS Classification	USCS	AASHTO	LL	PL	PI	Cc	Cu
●	6	0 - 5	LEAN CLAY with SAND	CL	A-7-6 (20)	41	17	24		
⊠	6	4 - 5	LEAN CLAY with SAND	CL	A-7-6 (30)	49	14	35		
▲	6	9 - 10	FAT CLAY	CH	A-7-6 (42)	59	19	40		
★	7	0 - 5	LEAN CLAY with SAND	CL	A-7-6 (21)	46	15	31		
⊙	7	4 - 5	CLAYEY SAND with GRAVEL	SC	A-2-4 (0)	30	20	10		

Boring ID		Depth (Ft)	D_{100}	D_{60}	D_{30}	D_{10}	%Cobbles	%Gravel	%Sand	%Fines	%Silt	%Clay
●	6	0 - 5	19				0.0	0.9	15.5	83.6		
⊠	6	4 - 5	1.18				0.0	0.0	15.0	85.0		
▲	6	9 - 10	2.36				0.0	0.0	4.9	95.1		
★	7	0 - 5	12.5				0.0	1.0	25.7	73.3		
⊙	7	4 - 5	12.5	0.582			0.0	16.9	50.6	32.5		

Grain Size Distribution

ASTM D422 / ASTM C136



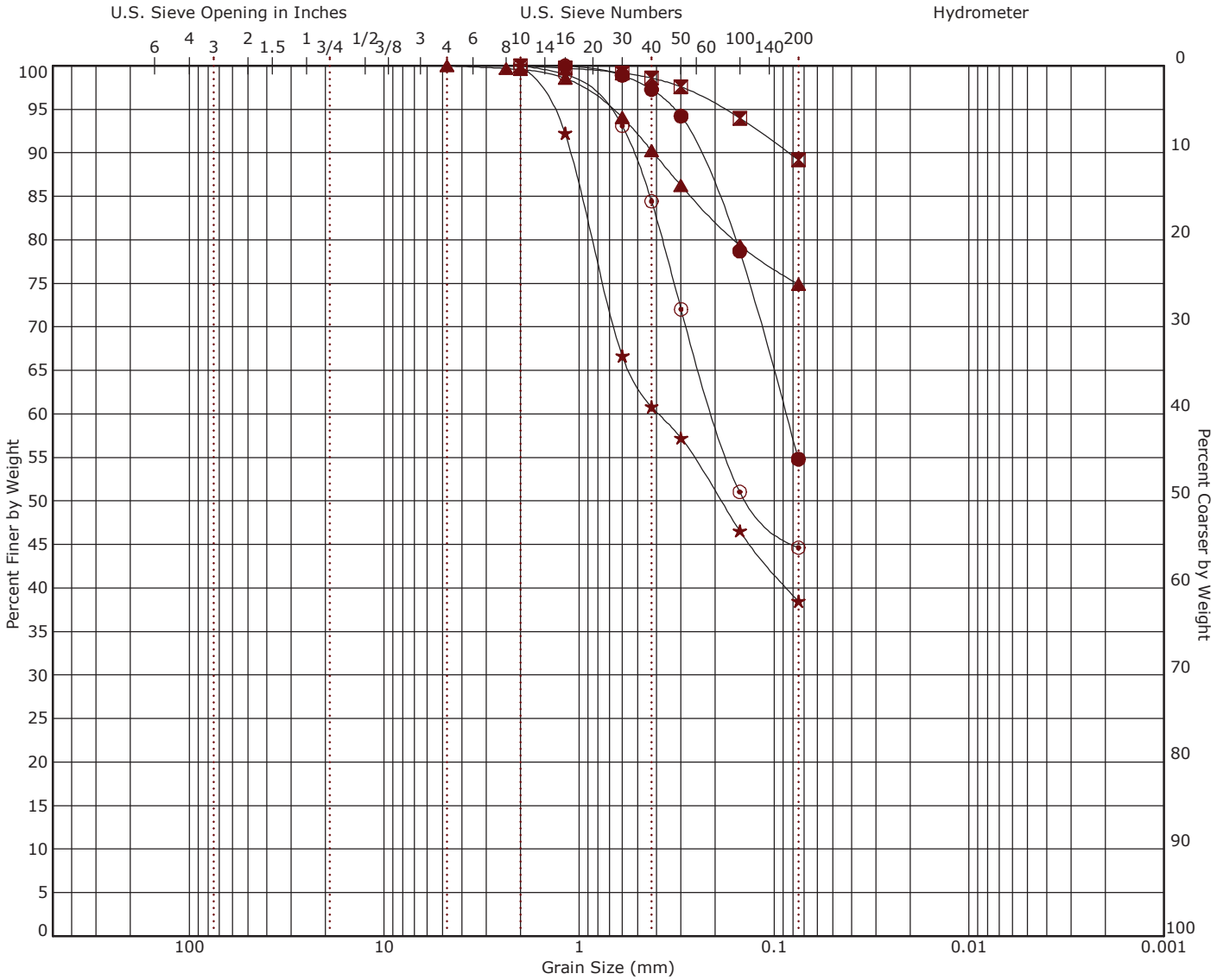
Cobbles	Gravel		Sand			Silt or Clay
	coarse	fine	coarse	medium	fine	

Boring ID		Depth (Ft)	USCS Classification	USCS	AASHTO	LL	PL	PI	Cc	Cu
●	7	14 - 15	FAT CLAY	CH	A-7-6 (54)	72	23	49		
⊠	8	0 - 5	LEAN CLAY with SAND	CL	A-7-6 (23)	44	16	28		
▲	8	19 - 20	FAT CLAY	CH	A-7-6 (51)	72	18	54		
★	9	0 - 5	LEAN CLAY with SAND	CL	A-7-6 (25)	45	14	31		
⊙	9	2 - 3	LEAN CLAY	CL	A-7-6 (29)	48	14	34		

Boring ID		Depth (Ft)	D_{100}	D_{60}	D_{30}	D_{10}	%Cobbles	%Gravel	%Sand	%Fines	%Silt	%Clay
●	7	14 - 15	4.75				0.0	0.0	2.9	97.1		
⊠	8	0 - 5	4.75				0.0	0.0	15.9	84.1		
▲	8	19 - 20	1.18				0.0	0.0	12.0	88.0		
★	9	0 - 5	4.75				0.0	0.0	17.2	82.8		
⊙	9	2 - 3	2				0.0	0.0	13.8	86.2		

Grain Size Distribution

ASTM D422 / ASTM C136



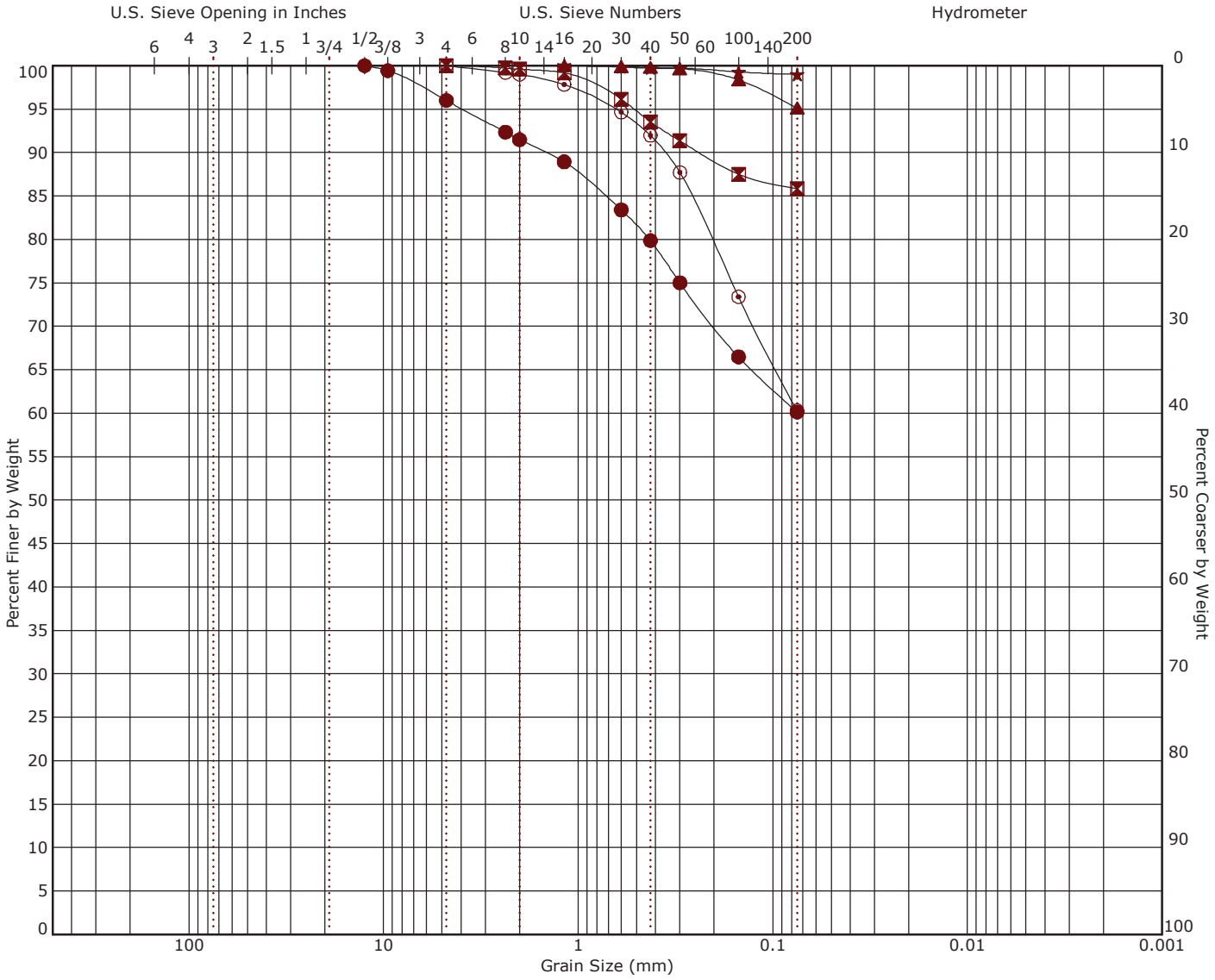
Cobbles	Gravel		Sand			Silt or Clay
	coarse	fine	coarse	medium	fine	

Boring ID		Depth (Ft)	USCS Classification	USCS	AASHTO	LL	PL	PI	Cc	Cu
●	9	9 - 10	SANDY LEAN CLAY	CL	A-6 (7)	37	20	17		
⊠	10	0 - 5	FAT CLAY	CH	A-7-6 (42)	63	19	44		
▲	11	0 - 5	LEAN CLAY with SAND	CL	A-7-6 (24)	49	15	34		
★	11	14 - 15	CLAYEY SAND	SC	A-7-6 (4)	41	18	23		
⊙	11	29 - 30	CLAYEY SAND	SC	A-7-6 (7)	44	19	25		

Boring ID		Depth (Ft)	D_{100}	D_{60}	D_{30}	D_{10}	%Cobbles	%Gravel	%Sand	%Fines	%Silt	%Clay
●	9	9 - 10	1.18	0.087			0.0	0.0	45.2	54.8		
⊠	10	0 - 5	2				0.0	0.0	10.8	89.2		
▲	11	0 - 5	4.75				0.0	0.0	25.1	74.9		
★	11	14 - 15	2	0.392			0.0	0.0	61.5	38.5		
⊙	11	29 - 30	2	0.202			0.0	0.0	55.4	44.6		

Grain Size Distribution

ASTM D422 / ASTM C136



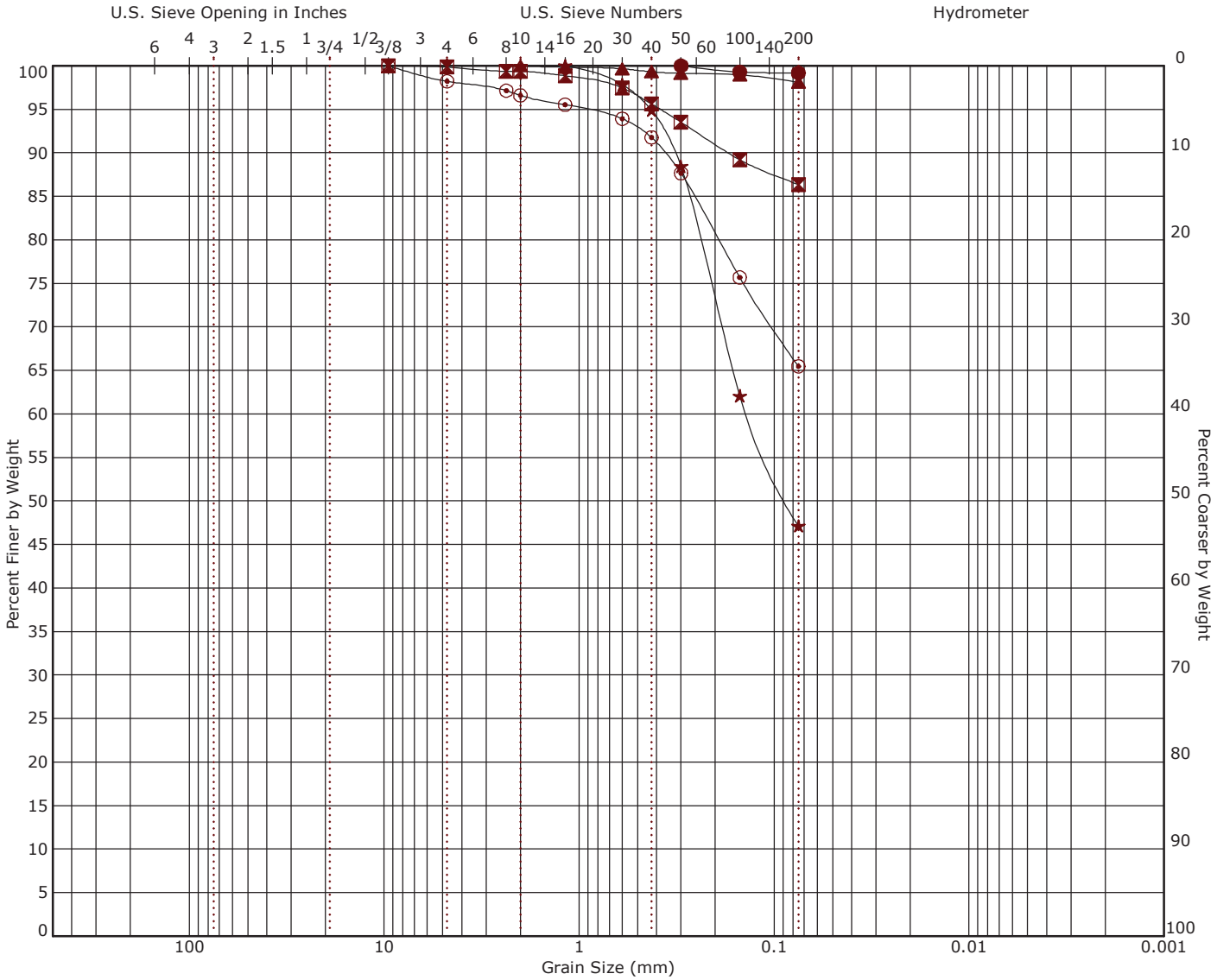
Cobbles	Gravel		Sand			Silt or Clay
	coarse	fine	coarse	medium	fine	

Boring ID		Depth (Ft)	USCS Classification	USCS	AASHTO	LL	PL	PI	Cc	Cu
●	12	0 - 5	SANDY LEAN CLAY	CL	A-7-6 (13)	42	14	28		
⊠	12	9 - 10	FAT CLAY	CH	A-7-6 (44)	67	19	48		
▲	13	0 - 5	FAT CLAY	CH	A-7-6 (49)	66	19	47		
★	14	2 - 3	FAT CLAY	CH	A-7-6 (57)	70	19	51		
⊙	14	24 - 25	SANDY LEAN CLAY	CL	A-6 (7)	34	17	17		

Boring ID		Depth (Ft)	D_{100}	D_{60}	D_{30}	D_{10}	%Cobbles	%Gravel	%Sand	%Fines	%Silt	%Clay
●	12	0 - 5	12.5				0.0	4.0	35.9	60.1		
⊠	12	9 - 10	4.75				0.0	0.0	14.1	85.9		
▲	13	0 - 5	1.18				0.0	0.0	4.9	95.1		
★	14	2 - 3	1.18				0.0	0.0	1.0	99.0		
⊙	14	24 - 25	4.75				0.0	0.0	39.7	60.3		

Grain Size Distribution

ASTM D422 / ASTM C136



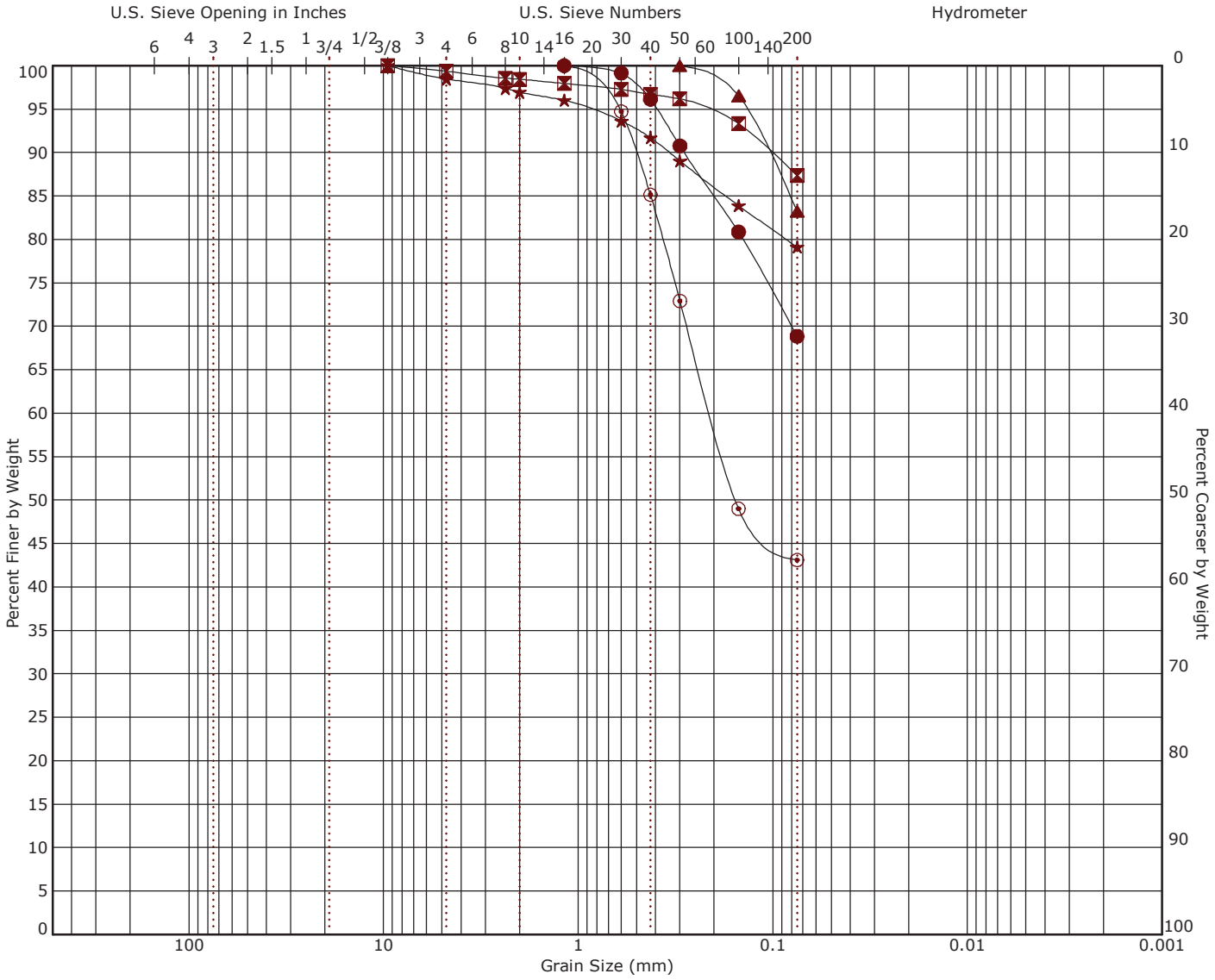
Cobbles	Gravel		Sand			Silt or Clay
	coarse	fine	coarse	medium	fine	

Boring ID		Depth (Ft)	USCS Classification	USCS	AASHTO	LL	PL	PI	Cc	Cu
●	15	2 - 3	FAT CLAY	CH	A-7-6 (53)	66	18	48		
⊠	16	0 - 5	FAT CLAY	CH	A-7-6 (31)	51	16	35		
▲	16	4 - 5	FAT CLAY	CH	A-7-6 (51)	66	20	46		
★	16	39 - 40	CLAYEY SAND	SC	A-7-6 (8)	44	16	28		
⊙	17	2 - 3	SANDY LEAN CLAY	CL	A-7-6 (15)	42	15	27		

Boring ID		Depth (Ft)	D_{100}	D_{60}	D_{30}	D_{10}	%Cobbles	%Gravel	%Sand	%Fines	%Silt	%Clay
●	15	2 - 3	0.3				0.0	0.0	0.8	99.2		
⊠	16	0 - 5	9.5				0.0	0.2	13.5	86.4		
▲	16	4 - 5	2				0.0	0.0	1.9	98.1		
★	16	39 - 40	1.18	0.136			0.0	0.0	52.9	47.1		
⊙	17	2 - 3	9.5				0.0	1.8	32.7	65.5		

Grain Size Distribution

ASTM D422 / ASTM C136



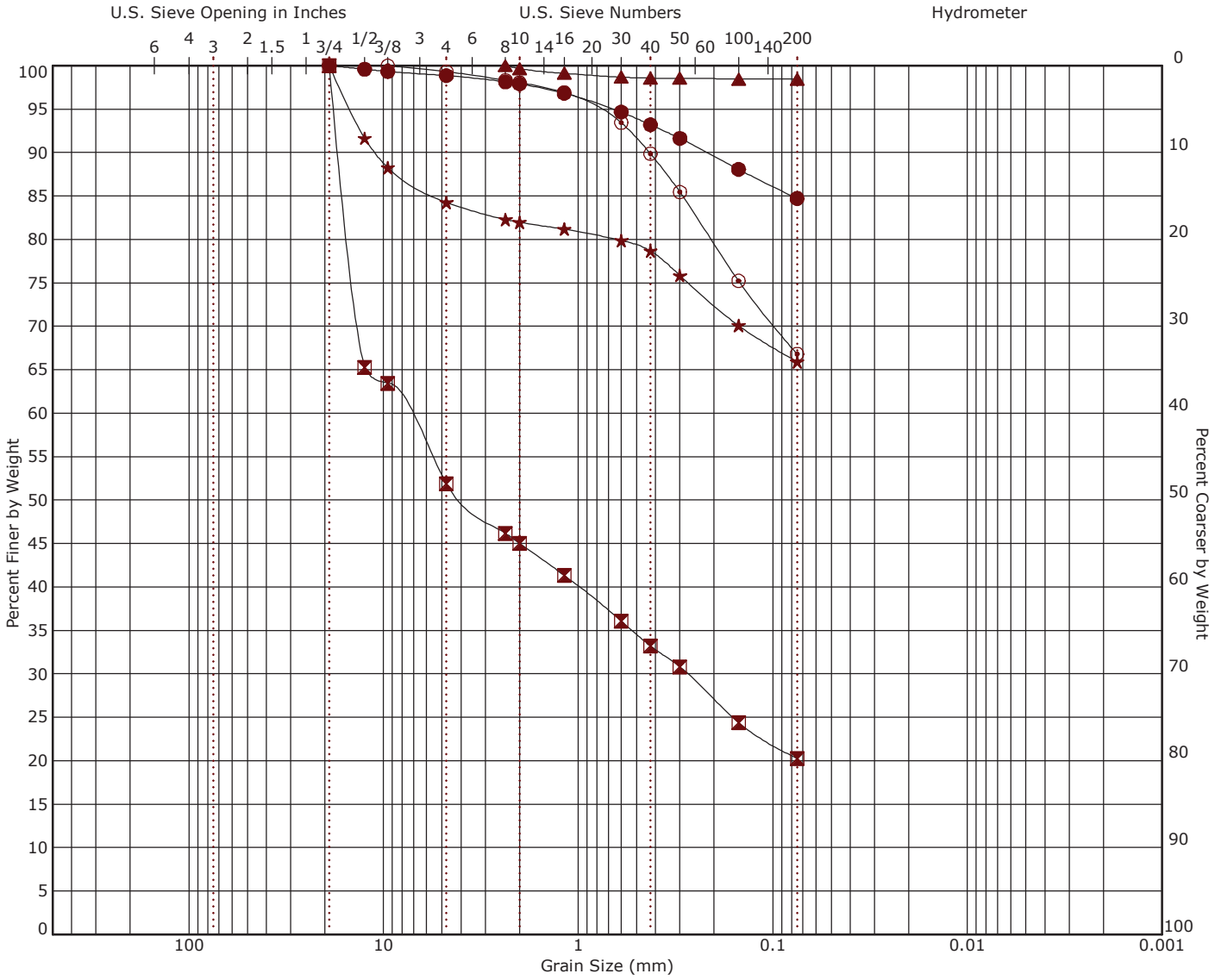
Cobbles	Gravel		Sand			Silt or Clay
	coarse	fine	coarse	medium	fine	

Boring ID		Depth (Ft)	USCS Classification	USCS	AASHTO	LL	PL	PI	Cc	Cu
●	17	24 - 25	SANDY LEAN CLAY	CL	A-7-6 (16)	44	18	26		
⊠	18	0 - 5	FAT CLAY	CH	A-7-6 (33)	53	17	36		
▲	18	2 - 3	FAT CLAY with SAND	CH	A-7-6 (28)	50	16	34		
★	19	0 - 5	LEAN CLAY with SAND	CL	A-7-6 (25)	49	17	32		
⊙	19	29 - 30	CLAYEY SAND	SC	A-7-6 (6)	43	17	26		

Boring ID		Depth (Ft)	D_{100}	D_{60}	D_{30}	D_{10}	%Cobbles	%Gravel	%Sand	%Fines	%Silt	%Clay
●	17	24 - 25	1.18				0.0	0.0	31.1	68.9		
⊠	18	0 - 5	9.5				0.0	0.6	12.0	87.3		
▲	18	2 - 3	0.3				0.0	0.0	16.8	83.2		
★	19	0 - 5	9.5				0.0	1.5	19.3	79.1		
⊙	19	29 - 30	1.18	0.206			0.0	0.0	56.9	43.1		

Grain Size Distribution

ASTM D422 / ASTM C136



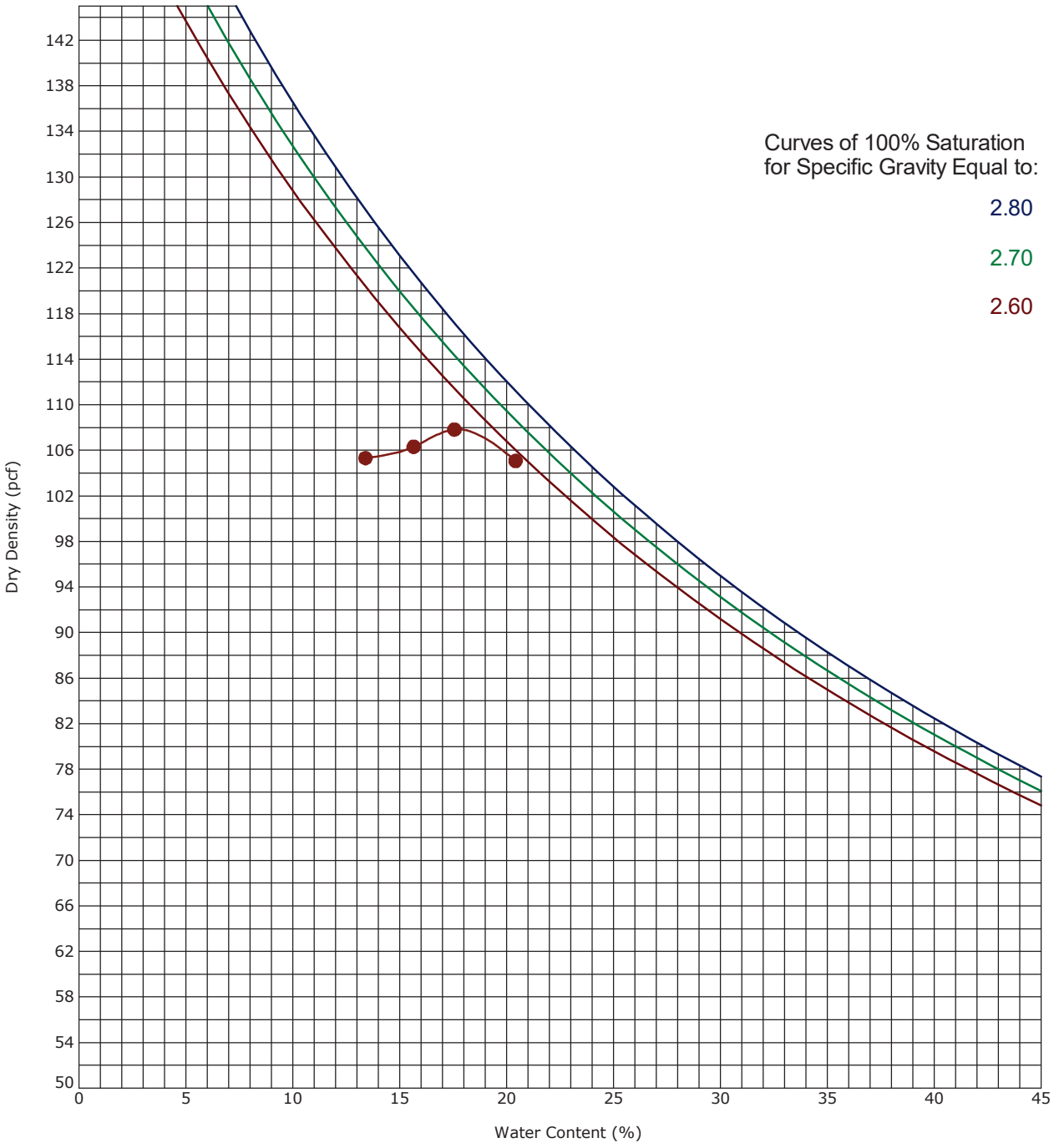
Cobbles	Gravel		Sand			Silt or Clay
	coarse	fine	coarse	medium	fine	

Boring ID		Depth (Ft)	USCS Classification	USCS	AASHTO	LL	PL	PI	Cc	Cu
●	20	0 - 5	FAT CLAY with SAND	CH	A-7-6 (40)	62	17	45		
⊠	20	19 - 20	CLAYEY GRAVEL with SAND	GC	A-2-6 (0)	35	17	18		
▲	21	9 - 10	FAT CLAY	CH	A-7-6 (56)	71	21	50		
★	22	0 - 5	SANDY FAT CLAY with GRAVEL	CH	A-7-6 (21)	51	16	35		
⊙	P1	0 - 5	SANDY LEAN CLAY	CL	A-7-6 (14)	41	17	24		

Boring ID		Depth (Ft)	D_{100}	D_{60}	D_{30}	D_{10}	%Cobbles	%Gravel	%Sand	%Fines	%Silt	%Clay
●	20	0 - 5	19				0.0	1.1	14.1	84.7		
⊠	20	19 - 20	19	7.722	0.275		0.0	48.1	31.6	20.3		
▲	21	9 - 10	2.36				0.0	0.0	1.5	98.5		
★	22	0 - 5	19				0.0	15.7	18.3	65.9		
⊙	P1	0 - 5	9.5				0.0	0.7	32.5	66.8		

Moisture-Density Relationship

ASTM D698-Method B

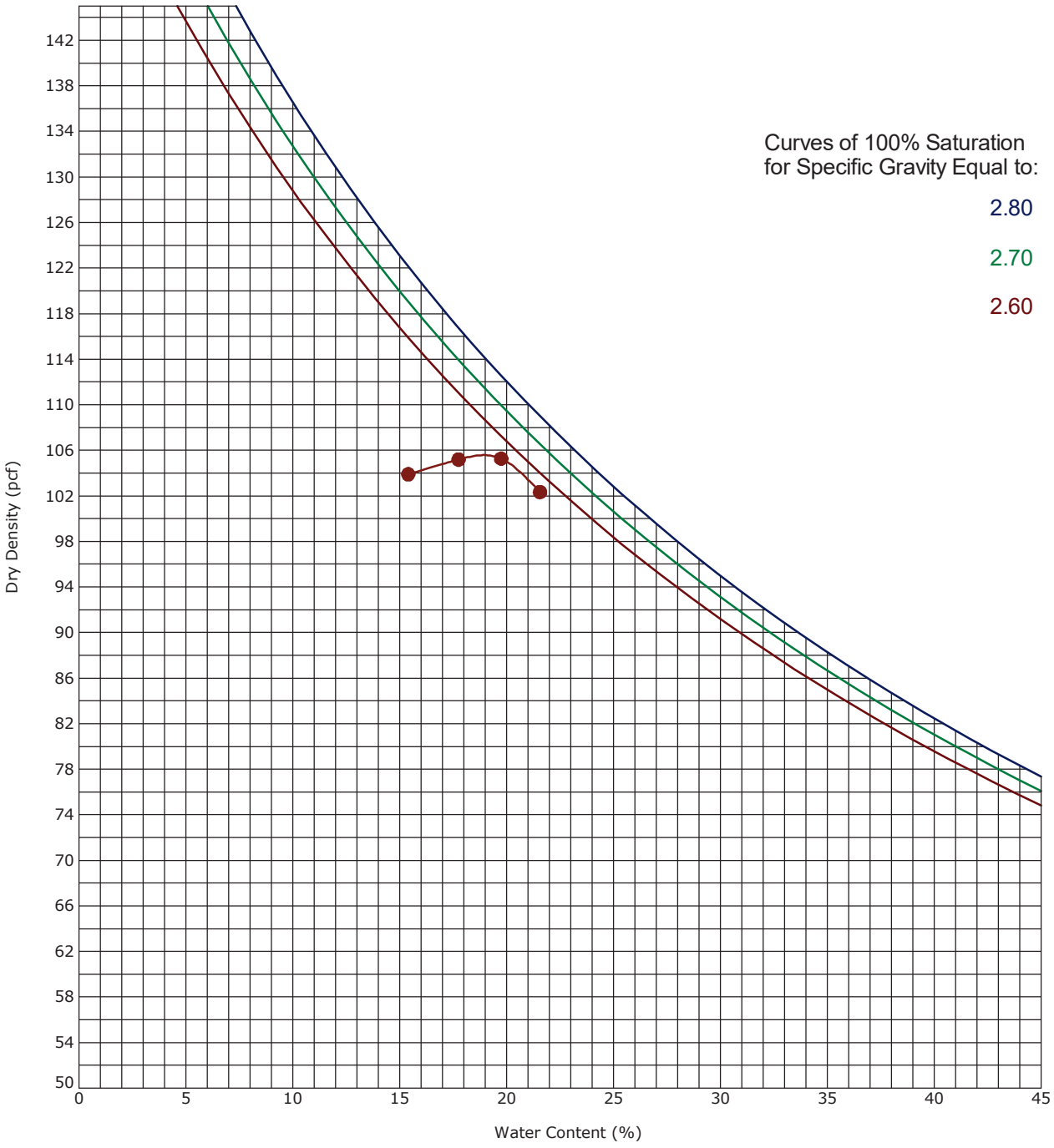


Boring ID	Depth (Ft)	Description of Materials
7	0 - 5	LEAN CLAY with SAND(CL)

Fines (%)	Fraction > mm size	LL	PL	PI	Test Method	Maximum Dry Density (pcf)	Optimum Water Content (%)
73	0.0	46	15	31	ASTM D698-Method B	107.8	17.7

Moisture-Density Relationship

ASTM D698-Method B

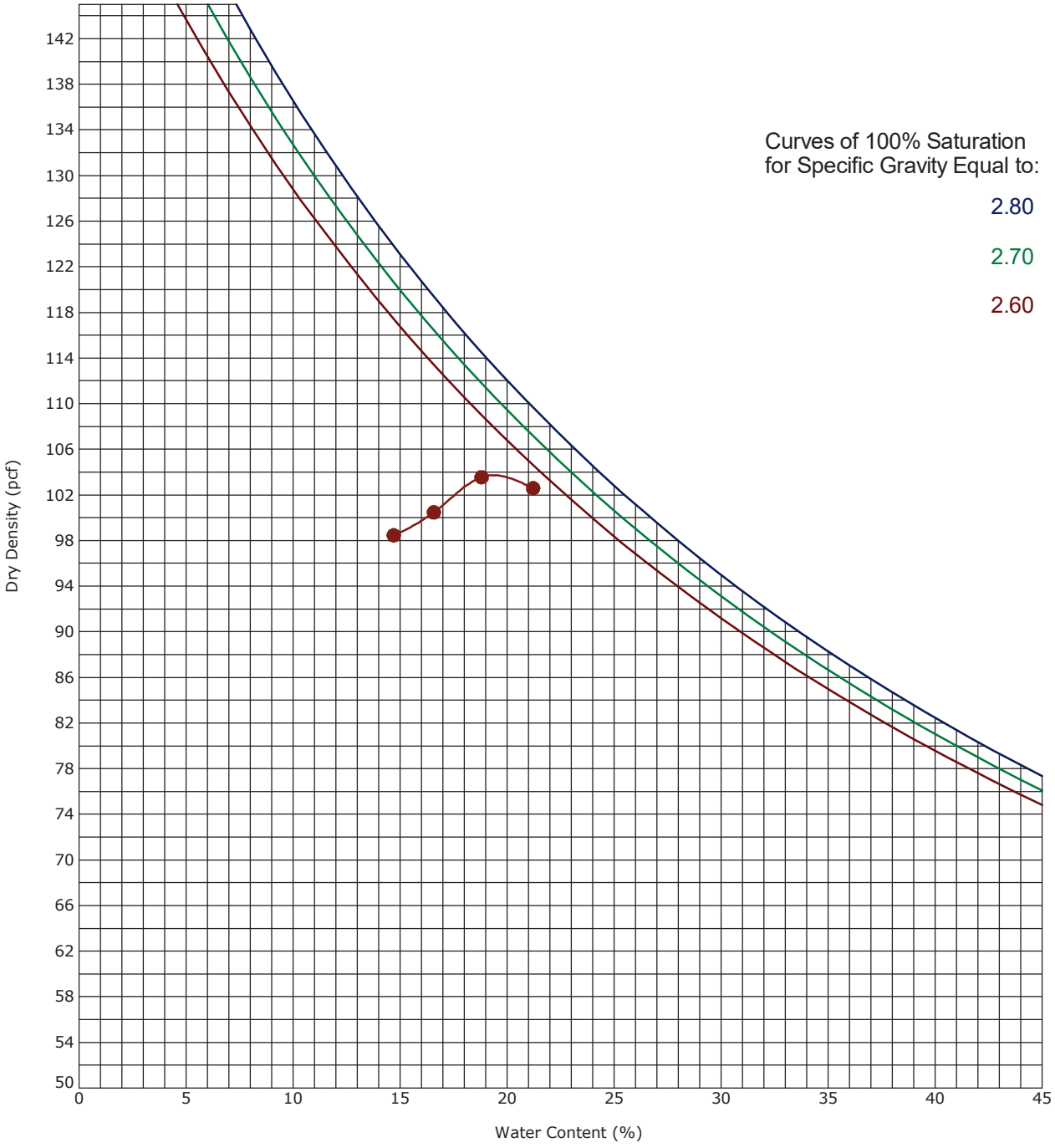


Boring ID	Depth (Ft)	Description of Materials
9	0 - 5	LEAN CLAY with SAND(CL)

Fines (%)	Fraction > mm size	LL	PL	PI	Test Method	Maximum Dry Density (pcf)	Optimum Water Content (%)
83	0.0	45	14	31	ASTM D698-Method B	105.6	18.9

Moisture-Density Relationship

ASTM D698-Method B

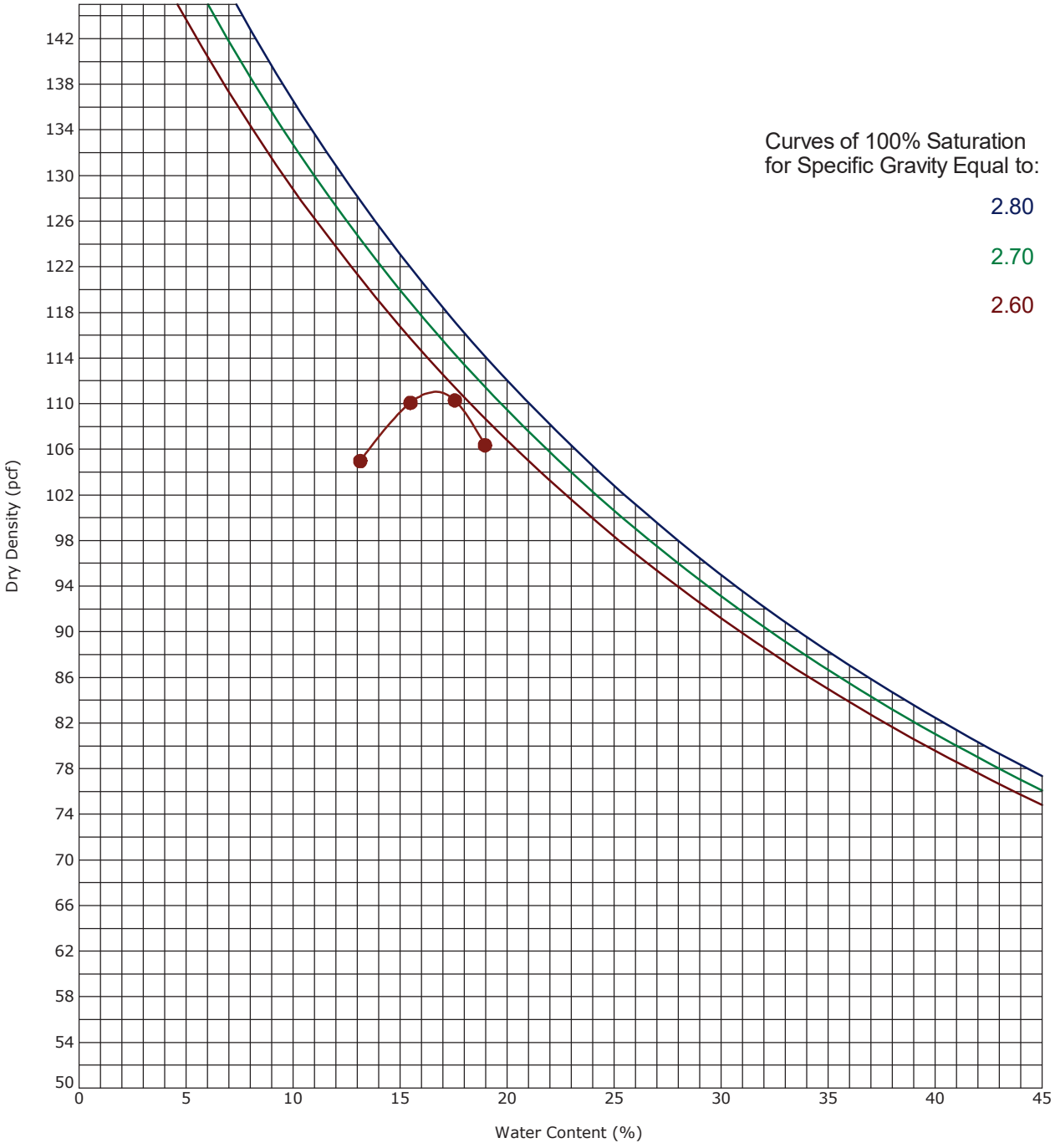


Boring ID	Depth (Ft)	Description of Materials
18	0 - 5	FAT CLAY(CH)

Fines (%)	Fraction > mm size	LL	PL	PI	Test Method	Maximum Dry Density (pcf)	Optimum Water Content (%)
87	0.0	53	17	36	ASTM D698-Method B	103.7	19.3

Moisture-Density Relationship

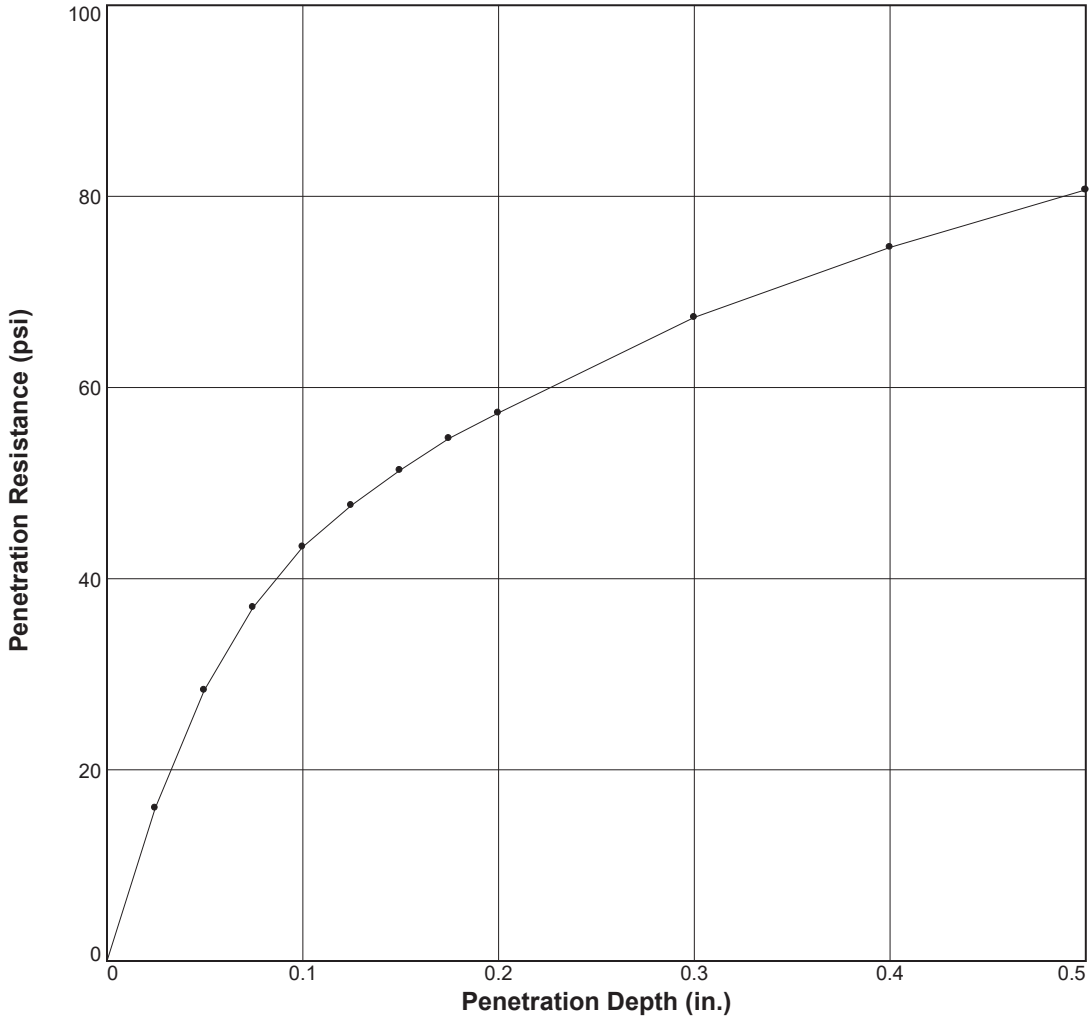
ASTM D698-Method B



Boring ID	Depth (Ft)	Description of Materials
P1	0 - 5	SANDY LEAN CLAY(CL)

Fines (%)	Fraction > mm size	LL	PL	PI	Test Method	Maximum Dry Density (pcf)	Optimum Water Content (%)
67	0.0	41	17	24	ASTM D698-Method B	111.0	16.7

BEARING RATIO TEST REPORT
ASTM D1883-16



	Molded			Soaked			CBR (%)		Linearity Correction (in.)	Surcharge (lbs.)	Max. Swell (%)
	Density (pcf)	Percent of Max. Dens.	Moisture (%)	Density (pcf)	Percent of Max. Dens.	Moisture (%)	0.10 in.	0.20 in.			
1 ○	105.5	95	16.7	103.3	93.1	21.0	4.3	3.8	0.000	10	2.1
2 △											
3 □											

Material Description	USCS	Max. Dens. (pcf)	Optimum Moisture (%)	LL	PI
CL, Sandy Lean Clay	CL	111.0	16.7	41	24

Project No: 25245150
Project: QTS Data Center - DC1 and DC3
Location: P1
Depth: 0' - 5'

Test Description/Remarks:

Prepared according to D698 compaction efforts.

BEARING RATIO TEST REPORT

Terracon Consultants, Inc.

Geotherm

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Cypress, TX 77433
Office: 291-985-9344
info@geothermusa.com
http://www.geothermusa.com

August 27, 2024

Terracon
10625 W I70 Frontage Rd N., Ste 3
Wheat Ridge, CO 80033
Attn: Trevor Meinholz

Re: Thermal Analysis of Native Soil Samples
QTS Data Center-DCI and DC3 – Aurora, CO (Project No. 25245150)

The following is the report of thermal dryout characterization tests conducted on two (2) bulk samples of native soil from the referenced project sent to our laboratory.

Thermal Resistivity Tests: The samples were tested at the 'optimum' moisture content at 95% of the standard Proctor dry density ***provided by Terracon***. The tests were conducted in accordance with the **IEEE standard 442-2017**. The results are tabulated below and the thermal dryout curves are presented in **Figures 1 and 2.**

Sample ID, Description, Thermal Resistivity, Moisture Content and Density

Sample ID	Depth (ft)	Effort (%)	Description (Terracon)	Thermal Resistivity (°C-cm/W)		Moisture Content (%)	Dry Density (lb/ft^3)
				Wet	Dry		
7	0 – 5	95	Lean Clay	67	173	18	102
18	0 – 5	95	Fat Clay	73	182	19	99

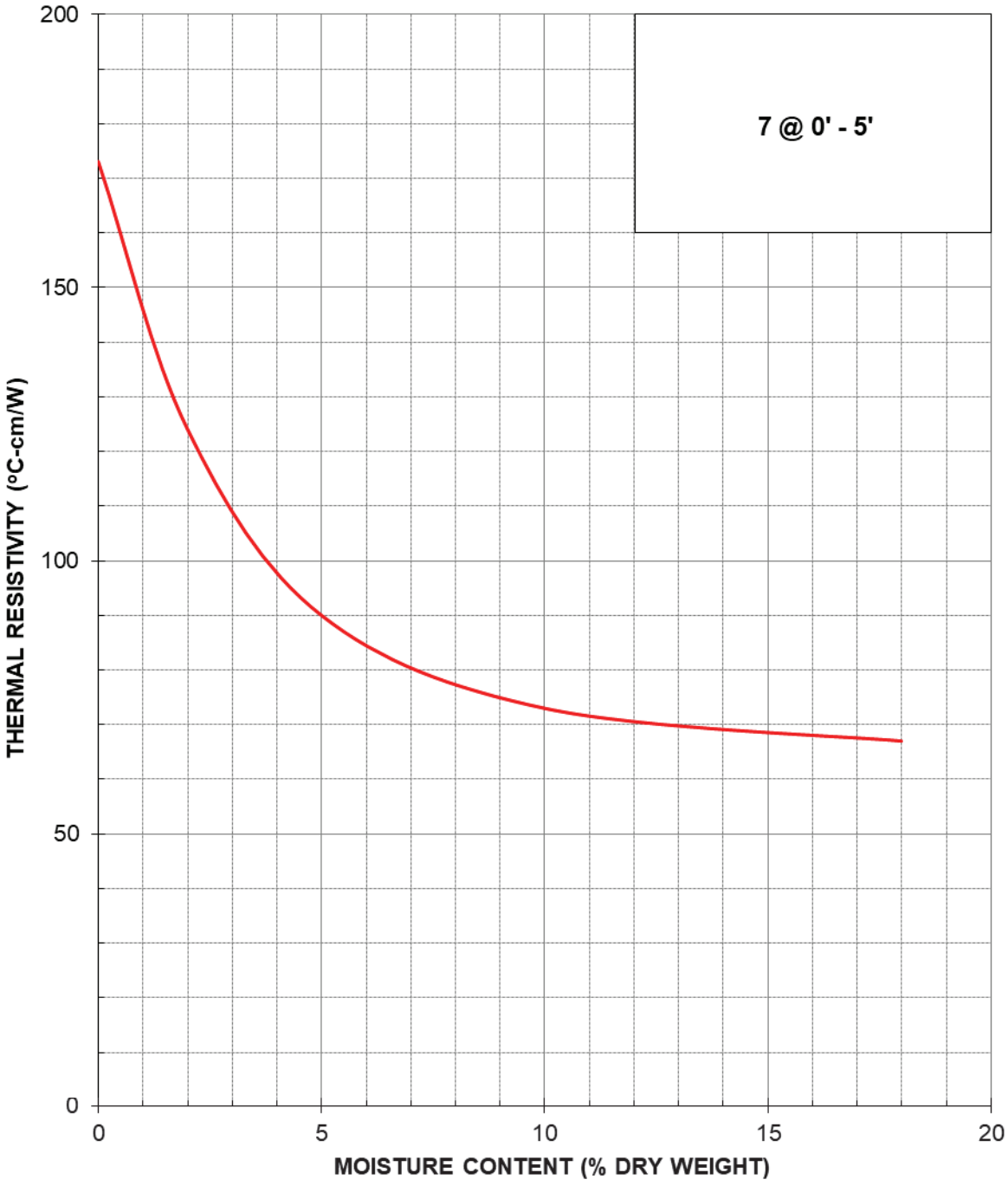
Comments: The thermal characteristic depicted in the dryout curves apply for the soils at their respective test dry density.

Please contact us if you have any questions or if we can be of further assistance.

Geotherm USA
Nimesh Patel

Geotherm

THERMAL DRYOUT CURVE



Terracon (Project No. 25245150)

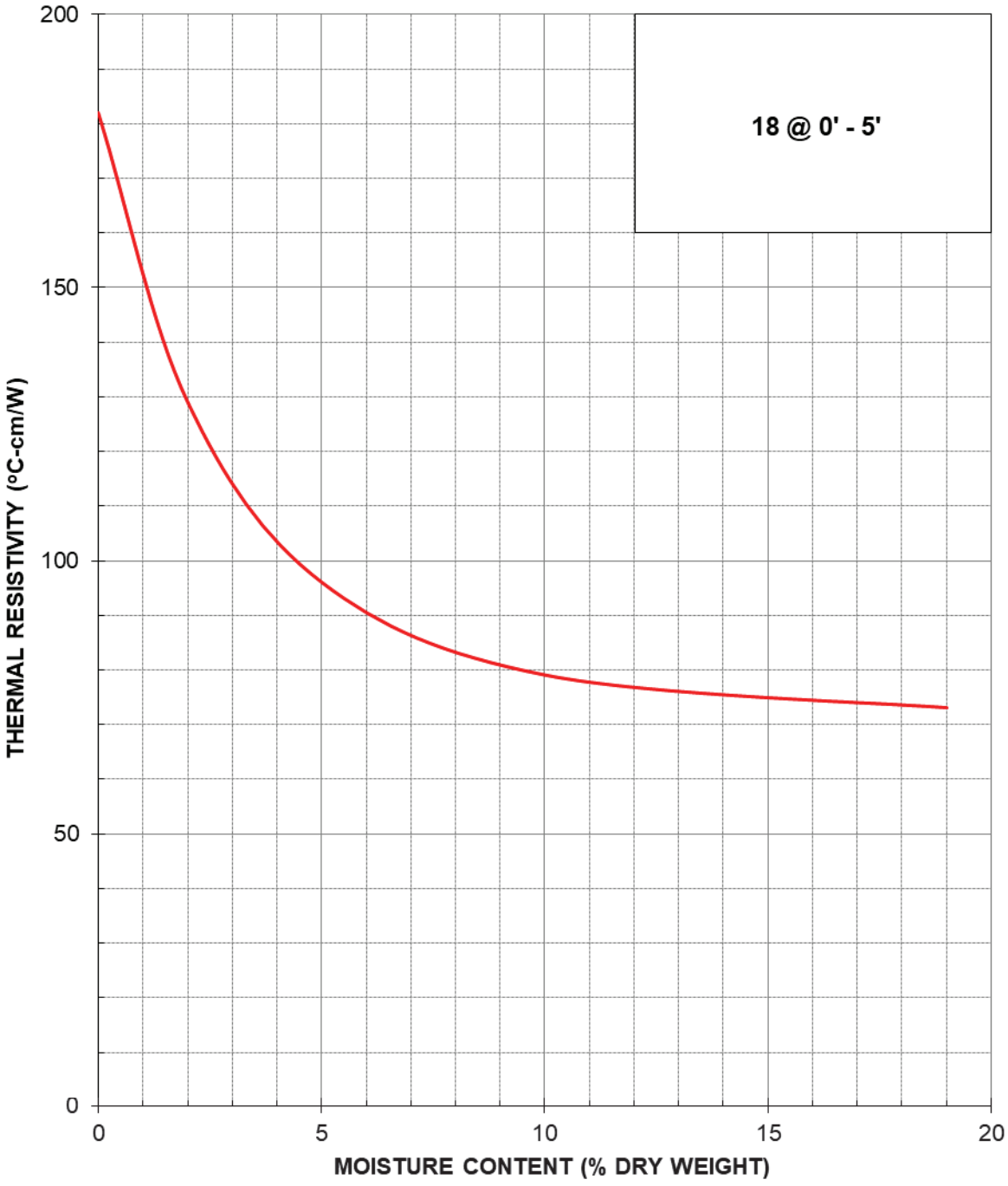
QTS Data Center-DCI and DC3 – Aurora, CO

Thermal Analysis of Native Soil Samples

August 2024 **Figure 1**

Geotherm

THERMAL DRYOUT CURVE



Terracon (Project No. 25245150)

QTS Data Center-DCI and DC3 – Aurora, CO

Thermal Analysis of Native Soil Samples

August 2024 **Figure 2**

Client
Quality Technology Services, LLC

Project
QTS Data Center - DC1 and DC3
25245150

Results from Corrosion Testing

Sample Location	1	4	6	10
Sample Depth (ft.)	0 - 5	0 - 5	0 - 5	0 - 5
pH Analysis, ASTM D4972	8.22	7.82	7.86	7.78
Water Soluble Sulfate (SO4), AASHTO T290, (%)	<0.10	<0.10	<0.10	<0.10
Sulfides, ASTM A674 - X1.6, (Negative/Trace/ Present)	Negative	Trace	Trace	Trace
Chlorides, AASHTO T291, (mg/kg)	21	67	18	330
Red-Ox, ASTM G200, (mV)	+364	+326	+398	+323
Resistivity (Saturated), ASTM G57, (ohm-cm)	780	650	500	340
Total Salts (mg/Kg)	1046	2448	3430	7552

Analyzed By: Daryl Lee
Laboratory Supervisor

The tests were performed in general accordance with applicable ASTM and AASHTO test methods. This report is exclusively for the use of the client indicated above and shall not be reproduced except in full without the written consent of our company. Test results transmitted herein are only applicable to the actual samples tested at the location(s) referenced and are not necessarily indicative of the properties of other apparently similar or identical materials.

Client
Quality Technology Services, LLC

Project
QTS Data Center - DC1 and DC3
25245150

Results from Corrosion Testing

Sample Location	11	13	14	16
Sample Depth (ft.)	0 - 5	0 - 5	0 - 5	0 - 5
pH Analysis, ASTM D4972	7.87	7.77	7.93	7.84
Water Soluble Sulfate (SO4), AASHTO T290, (%)	<0.10	<0.10	<0.10	<0.10
Sulfides, ASTM A674 - X1.6, (Negative/Trace/Present)	Present	Trace	Present	Trace
Chlorides, AASHTO T291, (mg/kg)	11	22	191	85
Red-Ox, ASTM G200, (mV)	+359	+317	+303	+313
Resistivity (Saturated), ASTM G57, (ohm-cm)	430	455	430	390
Total Salts (mg/Kg)	4474	3200	2384	4992

Analyzed By: Daryl Lee
Laboratory Supervisor

The tests were performed in general accordance with applicable ASTM and AASHTO test methods. This report is exclusively for the use of the client indicated above and shall not be reproduced except in full without the written consent of our company. Test results transmitted herein are only applicable to the actual samples tested at the location(s) referenced and are not necessarily indicative of the properties of other apparently similar or identical materials.

Client
Quality Technology Services, LLC

Project
QTS Data Center - DC1 and DC3
25245150

Results from Corrosion Testing

Sample Location	17	19	20
Sample Depth (ft.)	0 - 5	0 - 5	0 - 5
pH Analysis, ASTM D4972	7.88	7.87	7.86
Water Soluble Sulfate (SO4), AASHTO T290, (%)	<0.10	<0.10	<0.10
Sulfides, ASTM A674 - X1.6, (Negative/Trace/Present)	Trace	Trace	Negative
Chlorides, AASHTO T291, (mg/kg)	28	99	107
Red-Ox, ASTM G200, (mV)	+314	+316	+307
Resistivity (Saturated), ASTM G57, (ohm-cm)	430	340	505
Total Salts (mg/Kg)	3693	1600	4022

Analyzed By: Daryl Lee
Laboratory Supervisor

The tests were performed in general accordance with applicable ASTM and AASHTO test methods. This report is exclusively for the use of the client indicated above and shall not be reproduced except in full without the written consent of our company. Test results transmitted herein are only applicable to the actual samples tested at the location(s) referenced and are not necessarily indicative of the properties of other apparently similar or identical materials.

SUMMARY OF LABORATORY TEST RESULTS

QTS Data Center - DC1 and DC3 - Aurora, Colorado
Terracon Project No. 25245150

Boring No.	Depth (ft)	USCS Class.	Initial Water Content (%)	Initial Dry Density (pcf)	Swell/Consolidation		Moisture-Density Relationship			CBR (%)		Particle Size Distribution, Percent Passing by Weight					Atterberg Limits		Water Soluble Sulfates (%)	Chlorides (mg/kg)	pH	Sulfides (mg/kg)	Red-Ox (mV)	Resistivity (ohm-cm)	Total Salts (mg/kg)	Remarks
					Surcharge (ksf)	Swell (%)	Maximum Dry Density (pcf)	Optimum Moisture Density (%)	Test Method	0.10 in.	0.20 in.	3/4"	#4	#10	#40	#200	LL	PI								
1	0 - 5	CL										100	100	99	93	80	44	28	<0.10	21	8.22	Negative	+364	780	1046	
1	2	CL	9.0																							4
1	4	CL	10.1	97																						4
1	9	SM	7.7	126								100	75	64	38	19	NV	NP								4
1	14		27.4	95	0.5	+3.9																				3,4
1	19		23.2	108																						4
1	24		23.9	103																						4
1	29		25.0	101																						4
1	34		20.0																							4
1	39		29.4	93																						4
2	0 - 5	CL										100	100	99	93	78	42	28								
2	2	CL	13.6	110	0.5	+3.8																				3,4
2	4	CL	10.5	104																						4
2	9	CL	19.3	108	0.5	+0.4																				3,4
2	14	SC	19.1	102								100	100	100	86	34	37	17								4
2	19	SC	15.5	115																						4
2	24		25.1	94																						4
2	29		23.5	101																						4
2	34		23.7	97																						4
2	39		23.8	104																						4
3	0 - 5	CL										100	100	100	94	83	49	35								
3	2	CL	15.4	99	0.5	+3.1																				3,4
3	4	CL	12.1	109																						4
3	9	CH	20.2	100																						4
3	14	CH	19.5	108	0.5	+1.2						100	99	99	85	50	50	32								3,4
3	19		19.6	103																						4
3	24		22.3	108																						4
3	29		23.3	99																						4
3	34		22.3	105																						4
3	39		21.4	97																						4
4	0 - 5	CH										100	100	99	94	82	57	41	<0.10	67	7.82	Trace	+326	650	2448	
4	2	CH	13.5	102	0.5	+1.9																				3,4
4	4	CH	14.7	113																						4
4	9	CH	18.7	103																						4
4	14		23.2	102																						4
4	19		20.5	101								100	100	100	99	77	48	28								4
4	24		23.7	103																						4
4	29		27.3	93																						4
4	34		16.1	115								100	100	100	81	38	37	16								4
4	39		20.9	96																						4
5	2	CH	11.9																							4
5	4	CH	5.6	119	0.5	+0.9						100	100	96	84	72	63	45								3,4
5	9		22.9	105	0.5	+3.4																				3,4
5	14		26.5	100																						4
5	19		23.8	104																						4
5	24		22.5	106																						4
5	29		22.5	103																						4
5	34		20.4	112																						4
5	39		28.1	97																						4
5	44		20.2	110																						4
5	49		18.9	111																						4
6	0 - 5	CL										100	99	99	94	84	41	24	<0.10	18	7.86	Trace	+398	500	3430	
6	2	CL	10.0	118																						4

Notes:
Initial Dry Density and Initial Water Content are in-situ values unless otherwise noted.
* = Partially disturbed sample
- = Compression/settlement
NV = no value
NP = non-plastic

Remarks:
1 Remolded Compacted density (about 95% of ASTM D698 maximum density near optimum moisture content)
2 Remolded Compacted density (about 95% of ASTM D1557 maximum density near optimum moisture content)
3 Water added to sample
4 Dry density and/or moisture content determined from one ring of a multi-ring sample
5 Minus #200 Only
6 Moisture-Density Relationship Test Method ASTM D698/AASHTO T99
7 Moisture-Density Relationship Test Method ASTM D1557/AASHTO T180

Terracon
Explore with us

Permit #25-2535171-LT
RSN 1903242

SUMMARY OF LABORATORY TEST RESULTS

QTS Data Center - DC1 and DC3 - Aurora, Colorado
Terracon Project No. 25245150

Boring No.	Depth (ft)	USCS Class.	Initial Water Content (%)	Initial Dry Density (pcf)	Swell/Consolidation		Moisture-Density Relationship			CBR (%)		Particle Size Distribution, Percent Passing by Weight					Atterberg Limits		Water Soluble Sulfates (%)	Chlorides (mg/kg)	pH	Sulfides (mg/kg)	Red-Ox (mV)	Resistivity (ohm-cm)	Total Salts (mg/kg)	Remarks
					Surcharge (ksf)	Swell (%)	Maximum Dry Density (pcf)	Optimum Moisture Density (%)	Test Method	0.10 in.	0.20 in.	3/4"	#4	#10	#40	#200	LL	PI								
6	4	CL	12.5	107								100	100	100	96	85	49	35								4
6	9		20.7	100	0.5	+3.7						100	100	100	99	95	59	40								3,4
6	14		14.4	122																						4
6	19		25.6	99																						4
6	24		24.3	92																						4
6	29		23.8	105																						4
6	34		23.6	101																						4
6	39		18.9	108																						4
6	44		27.5	96																						4
6	49		17.2	115																						4
7	0 - 5	CL					107.8	17.7	D698B			100	99	98	91	73	46	31								
7	2	CL	24.6	97	0.5	+2.3																				3,4
7	4		9.8	122								100	83	72	57	32	30	10								4
7	9		24.8	100																						4
7	14		23.0	98	0.5	+3.8						100	100	100	99	97	72	49								3,4
7	19		21.4	102																						4
7	24		20.9	108																						4
7	29		20.0	100																						4
7	34		26.2	98																						4
7	39		15.0	100																						4
8	0 - 5	CL										100	100	100	95	84	44	28								
8	2	CL	16.9	112	0.5	+2.9																				3,4
8	4		20.1	109																						4
8	9		20.0	112	0.5	+1.9																				3,4
8	14		23.7	105																						4
8	19		23.5	103								100	100	100	97	88	72	54								4
8	24		24.3	102																						4
8	29		18.1	114																						4
8	34		24.2	103																						4
9	0 - 5	CL	17.9	101	0.5	+1.9	105.6	18.9	D698B			100	100	100	95	83	45	31								1,3,6
9	0 - 5	CL	19.5	102	0.5	+1.0	105.6	18.9	D698B			100	100	100	95	83	45	31								1,3,6
9	0 - 5	CL	21.1	102	0.5	+0.2	105.6	18.9	D698B			100	100	100	95	83	45	31								1,3,6
9	2	CL	15.6	102								100	100	100	96	86	48	34								4
9	4	CL	12.2	95																						4
9	9	CL	19.2	98	0.5	-0.1						100	100	100	97	55	37	17								3,4
9	14		21.7	106																						4
9	19		22.7	104																						4
9	24		20.1	111	0.5	+3.3																				3,4
9	29		23.1	100																						4
9	34		20.4	110																						4
9	39		23.8	98																						4
10	0 - 5	CH										100	100	100	99	89	63	44	<0.10	330	7.78	Trace	+323	340	7552	
10	2	CH	20.9	106	0.5	+2.4																				3,4
10	4	CH	24.3	98																						4
10	9	CH	22.3	103	0.5	+2.6																				3,4
10	14		24.0	99																						4
10	19		21.0	107																						4
10	24		21.6	100																						4
10	29		17.4	113																						4
10	34		26.2	95																						4
10	39		15.6	99																						4
11	0 - 5	CL										100	100	100	90	75	49	34	<0.10	11	7.87	Present	+359	430	4474	
11	2	CL	23.4	101																						4

Notes:
Initial Dry Density and Initial Water Content are in-situ values unless otherwise noted.
* = Partially disturbed sample
- = Compression/settlement
NV = no value
NP = non-plastic

Remarks:
1 Remolded Compacted density (about 95% of ASTM D698 maximum density near optimum moisture content)
2 Remolded Compacted density (about 95% of ASTM D1557 maximum density near optimum moisture content)
3 Water added to sample
4 Dry density and/or moisture content determined from one ring of a multi-ring sample
5 Minus #200 Only
6 Moisture-Density Relationship Test Method ASTM D698/AASHTO T99
7 Moisture-Density Relationship Test Method ASTM D1557/AASHTO T180

Terracon
Explore with us

Permit #25-2535171-LT
RSN 1903242

SUMMARY OF LABORATORY TEST RESULTS

QTS Data Center - DC1 and DC3 - Aurora, Colorado
Terracon Project No. 25245150

Boring No.	Depth (ft)	USCS Class.	Initial Water Content (%)	Initial Dry Density (pcf)	Swell/Consolidation		Moisture-Density Relationship			CBR (%)		Particle Size Distribution, Percent Passing by Weight					Atterberg Limits		Water Soluble Sulfates (%)	Chlorides (mg/kg)	pH	Sulfides (mg/kg)	Red-Ox (mV)	Resistivity (ohm-cm)	Total Salts (mg/kg)	Remarks
					Surcharge (ksf)	Swell (%)	Maximum Dry Density (pcf)	Optimum Moisture Density (%)	Test Method	0.10 in.	0.20 in.	3/4"	#4	#10	#40	#200	LL	PI								
11	4		20.9	100																						4
11	9		19.6	109	0.5	+2.4																				3,4
11	14		15.4	104								100	100	100	61	38	41	23								4
11	19		23.9	96																						4
11	24		23.2	106																						4
11	29		19.4	106								100	100	100	84	45	44	25								4
11	34		18.9	103																						4
11	39		23.9	103																						4
11	44		23.9	104																						4
11	49		20.9	107																						4
12	0 - 5	CL										100	96	91	80	60	42	28								
12	2	CL	15.1	112																						4
12	4	CL	16.4	108	0.5	+1.7																				3,4
12	9		23.9	100	0.5	+4.4						100	100	100	94	86	67	48								3,4
12	14		21.0	106																						4
12	19		19.9	109																						4
12	24		22.4	106																						4
12	34		22.1	102																						4
12	39		11.3	107																						4
13	0 - 5	CH										100	100	100	100	95	66	47	<0.10	22	7.77	Trace	+317	455	3200	
13	2	CH	20.7	102																						4
13	4	CH	21.7	101	0.5	+1.4																				3,4
13	9		23.4	104																						4
13	14		18.5	106																						4
13	19		21.0	100																						4
13	24		20.4	105																						4
13	29		21.0	104																						4
13	34		22.8	89																						4
13	39		28.0	94																						4
14	0 - 5	CH																	<0.10	191	7.93	Present	+303	430	2384	
14	2		21.2	104	0.5	+4.2						100	100	100	100	99	70	51								3,4
14	4		21.6	106								100	100	99	92	60	34	17								4
14	9		22.2	107																						4
14	14		9.5	104	0.5	+5.4																				3,4
14	19		21.8	101																						4
14	24		9.5	125								100	100	99	92	60	34	17								4
14	29		22.5	105																						4
14	34		21.9	104																						4
14	39		16.3	105																						4
15	2	CH	22.4	104								100	100	100	100	99	66	48								4
15	4		17.6	112	0.5	+0.6																				3,4
15	9		16.9	103																						4
15	14		21.0																							4
15	19		21.1	100																						4
15	24		20.2	109																						4
15	29		20.8	109																						4
15	34		21.3	105																						4
15	39		16.4	107																						4
16	0 - 5	CH										100	100	99	96	86	51	35	<0.10	85	7.84	Trace	+313	390	4992	
16	2		21.2	104																						4
16	4		17.7	110	0.5	+11.1						100	100	100	99	98	66	46								3,4
16	9		15.7	117																						4
16	14		16.0	106	0.5	+5.0																				3,4

Notes:
Initial Dry Density and Initial Water Content are in-situ values unless otherwise noted.
* = Partially disturbed sample
- = Compression/settlement
NV = no value
NP = non-plastic

Remarks:
1 Remolded Compacted density (about 95% of ASTM D698 maximum density near optimum moisture content)
2 Remolded Compacted density (about 95% of ASTM D1557 maximum density near optimum moisture content)
3 Water added to sample
4 Dry density and/or moisture content determined from one ring of a multi-ring sample
5 Minus #200 Only
6 Moisture-Density Relationship Test Method ASTM D698/AASHTO T99
7 Moisture-Density Relationship Test Method ASTM D1557/AASHTO T180

Terracon Explore with us

Permit #25-2535171-LT
RSN 1903242

SUMMARY OF LABORATORY TEST RESULTS

QTS Data Center - DC1 and DC3 - Aurora, Colorado
Terracon Project No. 25245150

Boring No.	Depth (ft)	USCS Class.	Initial Water Content (%)	Initial Dry Density (pcf)	Swell/Consolidation		Moisture-Density Relationship			CBR (%)		Particle Size Distribution, Percent Passing by Weight					Atterberg Limits		Water Soluble Sulfates (%)	Chlorides (mg/kg)	pH	Sulfides (mg/kg)	Red-Ox (mV)	Resistivity (ohm-cm)	Total Salts (mg/kg)	Remarks
					Surcharge (ksf)	Swell (%)	Maximum Dry Density (pcf)	Optimum Moisture Density (%)	Test Method	0.10 in.	0.20 in.	3/4"	#4	#10	#40	#200	LL	PI								
16	19		17.1	109																						4
16	24		21.5	107																						4
16	29		20.2	106																						4
16	39		19.4	108								100	100	100	95	47	44	28								4
16	44		22.6	106																						4
16	49		20.3	102																						4
17	0 - 5	CL-ML																	<0.10	28	7.88	Trace	+314	430	3693	
17	2		16.6	117	0.5	+1.4						100	98	97	92	65	42	27								3,4
17	4		20.5	110																						4
17	9		22.0	105	0.5	+6.2																				3,4
17	14		19.4	110																						4
17	19		19.1	112																						4
17	24		15.7	112								100	100	100	96	69	44	26								4
17	29		19.7	108																						4
17	34		19.5	112																						4
17	39		21.4	106																						4
18	0 - 5	CH					103.7	19.3	D698B			100	99	98	97	87	53	36								
18	2		18.6	107								100	100	100	100	83	50	34								4
18	4		18.2	111	0.5	+0.5																				3,4
18	14		23.6	103																						4
18	19		24.1	102																						4
18	24		21.1	108																						4
18	29		19.3																							4
18	34		19.3	113																						4
18	39		34.1	84																						4
19	0 - 5	CL										100	98	97	92	79	49	32	<0.10	99	7.87	Trace	+316	340	1600	
19	2		20.6	101																						4
19	4		20.4	99																						4
19	9		14.5	115	0.5	+7.6																				3,4
19	14		18.0	100																						4
19	19		21.4	106																						4
19	24		17.0	116																						4
19	29		16.5	98								100	100	100	85	43	43	26								4
19	34		22.2	106																						4
19	39		21.4	93																						4
20	0 - 5	CH										100	99	98	93	85	62	45	<0.10	107	7.86	Negative	+307	505	4022	
20	2		20.3	114																						4
20	4		21.9	104	0.5	+5.7																				3,4
20	9		19.3	111																						4
20	14		16.7	119																						4
20	19		10.8									100	52	45	33	20	35	18								4
20	24		21.0	100																						4
20	29		21.1	107																						4
20	34		11.6	107																						4
20	39		17.4																							4
21	2		22.1	103																						4
21	4		23.4	104																						4
21	9		20.1	110	0.5	+5.7						100	100	100	99	98	71	50								3,4
21	14		18.4	109	0.5	+8.5																				3,4
21	19		19.1	104																						4
21	24		18.6																							4
21	29		18.3	111																						4
21	34		20.4																							4

Notes:
Initial Dry Density and Initial Water Content are in-situ values unless otherwise noted.
* = Partially disturbed sample
- = Compression/settlement
NV = no value
NP = non-plastic

Remarks:
1 Remolded Compacted density (about 95% of ASTM D698 maximum density near optimum moisture content)
2 Remolded Compacted density (about 95% of ASTM D1557 maximum density near optimum moisture content)
3 Water added to sample
4 Dry density and/or moisture content determined from one ring of a multi-ring sample
5 Minus #200 Only
6 Moisture-Density Relationship Test Method ASTM D698/AASHTO T99
7 Moisture-Density Relationship Test Method ASTM D1557/AASHTO T180

Terracon
Explore with us

Permit #25-2535171-LT
RSN 1903242

SUMMARY OF LABORATORY TEST RESULTS

QTS Data Center - DC1 and DC3 - Aurora, Colorado
Terracon Project No. 25245150

Boring No.	Depth (ft)	USCS Class.	Initial Water Content (%)	Initial Dry Density (pcf)	Swell/Consolidation		Moisture-Density Relationship			CBR (%)		Particle Size Distribution, Percent Passing by Weight					Atterberg Limits		Water Soluble Sulfates (%)	Chlorides (mg/kg)	pH	Sulfides (mg/kg)	Red-Ox (mV)	Resistivity (ohm-cm)	Total Salts (mg/kg)	Remarks
					Surcharge (ksf)	Swell (%)	Maximum Dry Density (pcf)	Optimum Moisture Density (%)	Test Method	0.10 in.	0.20 in.	3/4"	#4	#10	#40	#200	LL	PI								
21	39		22.2	105																						4
22	0 - 5											100	84	82	79	66	51	35								
22	2		15.7	118																						4
22	4		15.8	119	0.5	+2.3																				3,4
22	9		15.1	109																						4
22	14		17.7	115																						4
22	19		18.9	111																						4
22	24		20.8	106																						4
22	29		17.1	115																						4
22	34		18.6	111																						4
22	39		22.1	107																						4
P1	0 - 5	CL					111.0	16.7	D698B	4.3	3.8	100	99	98	90	67	41	24								
P1	2	CL	16.4	107																						4
P1	4	CL	14.9	103																						4
P1	9	CL	13.9	106																						4

Notes:
Initial Dry Density and Initial Water Content are in-situ values unless otherwise noted.
* = Partially disturbed sample
- = Compression/settlement
NV = no value
NP = non-plastic

Remarks:
1 Remolded Compacted density (about 95% of ASTM D698 maximum density near optimum moisture content)
2 Remolded Compacted density (about 95% of ASTM D1557 maximum density near optimum moisture content)
3 Water added to sample
4 Dry density and/or moisture content determined from one ring of a multi-ring sample
5 Minus #200 Only
6 Moisture-Density Relationship Test Method ASTM D698/AASHTO T99
7 Moisture-Density Relationship Test Method ASTM D1557/AASHTO T180

Terracon
Explore with us

Permit #25-2535171-LT
RSN 1903242

Field Electrical Resistivity Test Results

Contents:

Field Electrical Resistivity Test Data (2 pages)

Note: All attachments are one page unless noted above.

FIELD ELECTRICAL RESISTIVITY TEST DATA

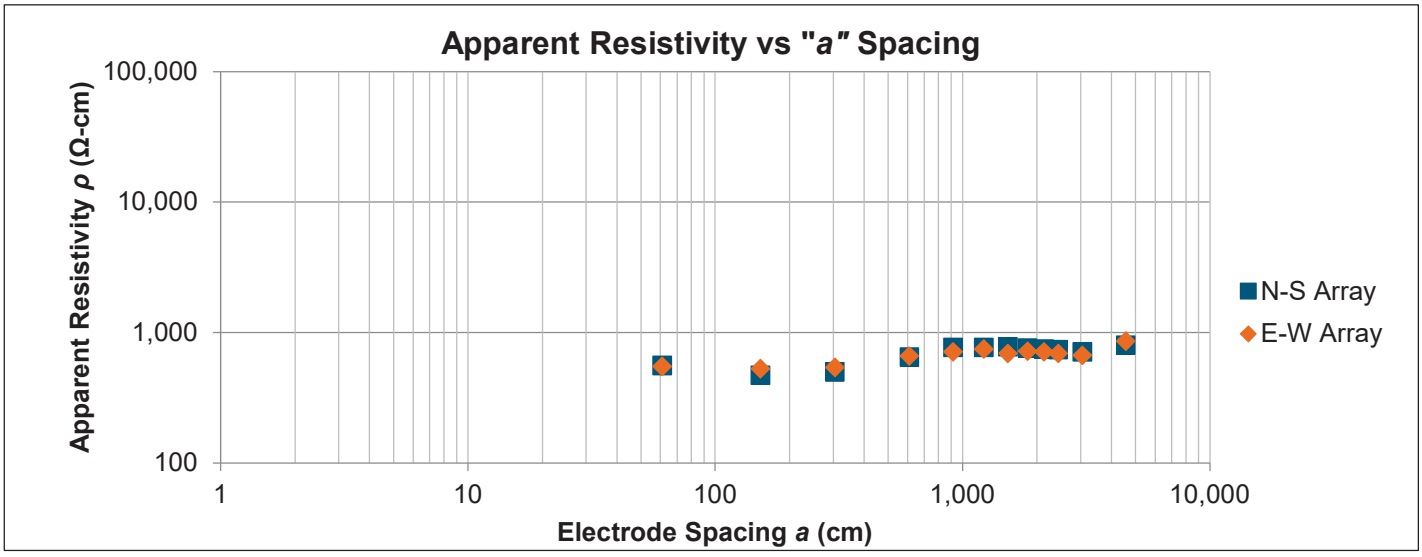
Quality Technology Services, LLC ■ Aurora, Colorado
September 6, 2024 ■ Terracon Project No. 25245150

Array Loc.	FER-1 Center at 39.7330º, -104.7138º		
Instrument	L&R Instruments MiniRes	**Weather**	86ºF, Clear
Serial #	SN-344	**Ground Cond.**	Dry/Clay/Silt
Cal. Check	9/3/204	**Tested By**	WRL
Test Date	September 3, 2024	**Method**	Wenner 4-pin (ASTM G57-06 (2020); IEEE 81-2012)
Notes & Conflicts			

Apparent resistivity ρ is calculated as :

$$\rho = \frac{4\pi a R}{1 + \frac{2a}{\sqrt{a^2 + 4b^2}} - \frac{a}{\sqrt{a^2 + b^2}}}$$

Electrode Spacing *a*		Electrode Depth *b*		N-S Test		E-W Test	
(feet)	(centimeters)	(inches)	(centimeters)	Measured Resistance *R*	Apparent Resistivity ρ	Measured Resistance *R*	Apparent Resistivity ρ
				Ω	(Ω-cm)	Ω	(Ω-cm)
2	61	4	10	1.401	560	1.373	550
5	152	4	10	0.492	470	0.549	530
10	305	12	30	0.255	500	0.277	540
20	610	12	30	0.169	650	0.171	660
30	914	12	30	0.133	770	0.124	710
40	1,219	12	30	0.101	770	0.098	750
50	1,524	12	30	0.081	780	0.072	690
60	1,829	12	30	0.066	760	0.063	720
70	2,134	12	30	0.056	750	0.053	710
80	2,438	12	30	0.048	740	0.045	690
100	3,048	12	30	0.037	710	0.035	670
150	4,572	12	30	0.028	800	0.030	860
200	6,096	12	30	0.017	650	0.019	730
250	7,620	12	30	0.014	670	0.015	720



FIELD ELECTRICAL RESISTIVITY TEST DATA

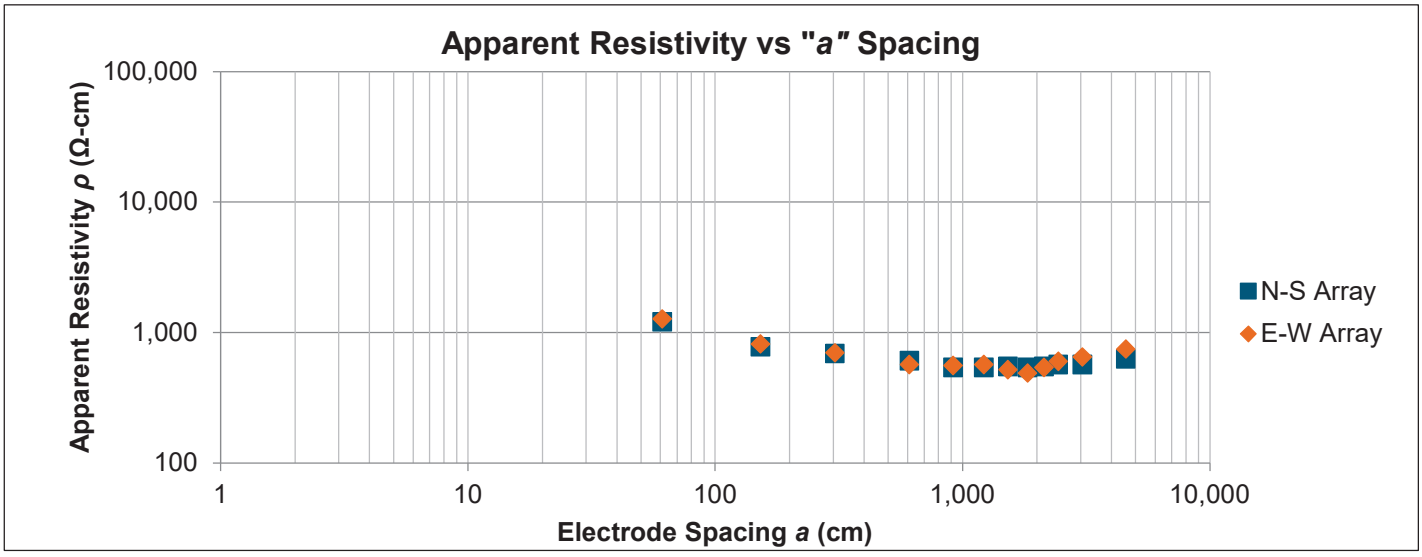
Quality Technology Services, LLC ■ Aurora, Colorado
September 6, 2024 ■ Terracon Project No. 25245150

Array Loc.	FER-2 Center at 39.7335º, -104.7114º		
Instrument	L&R Instruments MiniRes	**Weather**	90ºF, Clear
Serial #	SN-344	**Ground Cond.**	Dry/Clay/Silt
Cal. Check	9/3/204	**Tested By**	WRL
Test Date	September 3, 2024	**Method**	Wenner 4-pin (ASTM G57-06 (2020); IEEE 81-2012)
Notes & Conflicts	Rotated survey orientation due to spoil piles and construction traffic		

Apparent resistivity ρ is calculated as :

$$\rho = \frac{4\pi a R}{1 + \frac{2a}{\sqrt{a^2 + 4b^2}} - \frac{a}{\sqrt{a^2 + b^2}}}$$

Electrode Spacing *a*		Electrode Depth *b*		NW-SE Test		NE-SW Test	
(feet)	(centimeters)	(inches)	(centimeters)	Measured Resistance *R*	Apparent Resistivity ρ	Measured Resistance *R*	Apparent Resistivity ρ
				Ω	(Ω-cm)	Ω	(Ω-cm)
2	61	4	10	3.030	1,210	3.181	1,270
5	152	4	10	0.809	780	0.855	820
10	305	12	30	0.352	690	0.359	700
20	610	12	30	0.158	610	0.149	570
30	914	12	30	0.093	540	0.097	560
40	1,219	12	30	0.071	540	0.074	570
50	1,524	12	30	0.057	550	0.054	520
60	1,829	12	30	0.047	540	0.043	490
70	2,134	12	30	0.041	550	0.040	540
80	2,438	12	30	0.037	570	0.039	600
100	3,048	12	30	0.030	570	0.034	650
150	4,572	12	30	0.022	630	0.026	750
200	6,096	12	30	0.017	650	0.018	690
250	7,620	12	30	0.014	670	0.016	770



Previous Geotechnical Data (Terracon Project No. 25215031)

Contents:

Boring Logs (Boring Nos. 22 to 42 and F1 to F6)
One-Dimensional Swell or Collapse (25 pages)
Remolded Swell Test (3 pages)
Grain Size Distribution (7 pages)
Unconfined Compressive Strength (2 pages)
Moisture Density Relationship (8 pages)
California Bearing Ratio (CBR) (2 pages)
Thermal Resistivity (7 pages)
Corrosivity (7 pages)
Summary of Laboratory Test Results (6 pages)

Note: All attachments are one page unless noted above.

BORING LOG NO. 22

PROJECT: QTS Aurora

CLIENT: Quality Technology Services, Inc.

SITE: Gun Club Road and Colfax Avenue
Aurora, Colorado

LOCATION See Exploration Plan

Latitude: 39.7340° Longitude: -104.7146°

Approximate Surface Elev.: 5545.5 (Ft.) +/-

DEPTH	ELEVATION (Ft.)	MODEL LAYER	GRAPHIC LOG	DESCRIPTION
0	5545.5+/-	1		**SANDY LEAN CLAY**, brown, medium stiff to stiff
8.0	5537.5+/-	3		**CLAYSTONE**, gray to brown, firm to very hard
50.0	5495.5+/-			***Boring Terminated at 50 Feet***

DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	SWELL (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
0								37-16-21	65
5			3-6						
			6-10			22.9	102		
			10-17	+4.3 @ 500 psf		23.7	102		
10			10-18			25.2			
			15-26						
15			20-29			16.8	112		
20			15-22		10650	26.0	98		
25			18-30						
30			17-29			23.4	103		
35			50/6"						
40			50/10"			28.9	92		
45			50/6"						
50			50/2"						

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
4-inch diameter solid stem continuous flight power auger

Abandonment Method:
Boring backfilled with auger cuttings upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Notes:

Elevations were interpolated from the ALTA/NSPS Land Title Survey by Martin/Martin (Job No. 19.1522.C.86, revised March 29, 2021) and provided to us by QTS.

WATER LEVEL OBSERVATIONS

None encountered while drilling

Terracon
10625 W I70 Frontage Rd N Ste 3
Wheat Ridge, CO

Boring Started: 06-23-2021 | Boring Completed: 06-23-2021

Drill Rig: CME-550 | Driller: Terracon

Project No.: 25215031

THIS BORING LOG IS NOT VALID IF SEPARATED FROM ORIGINAL REPORT. GEO SMART LOG-NO WELL 25215031 QTS AURORA_USE THIS ONE.GPJ TERRACON_DATATEMPLATE.GDT 8/17/21

BORING LOG NO. 23

PROJECT: QTS Aurora

CLIENT: Quality Technology Services, Inc.

**SITE: Gun Club Road and Colfax Avenue
Aurora, Colorado**

LOCATION See Exploration Plan

Latitude: 39.7340° Longitude: -104.7141°

Approximate Surface Elev.: 5544 (Ft.) +/-

MODEL LAYER	GRAPHIC LOG	DEPTH	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	SWELL (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
1			**LEAN CLAY (CL)**, brown, medium stiff to stiff				2-5			15.2	115		
				5			6-10	+1.1 @ 500 psf		15.7	114		
							6-12			20.6	108		
		10.0	5534+/-	10			50/6"						
3			**CLAYSTONE**, gray to brown, soft to very hard				50/12"			22.8	103		
				15			50/9"						
				20			50/11"			24.9	101		
				25			50/11"						
				30			50/9"			23.6	103		
							50/10"						
		40.0	5504+/-	35									
			Boring Terminated at 40 Feet	40			50/8"						

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
4-inch diameter solid stem continuous flight power auger

Abandonment Method:
Boring backfilled with auger cuttings upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Notes:

Elevations were interpolated from the ALTA/NSPS Land Title Survey by Martin/Martin (Job No. 19.1522.C.86, revised March 29, 2021) and provided to us by QTS.

WATER LEVEL OBSERVATIONS

None encountered while drilling

Terracon
10625 W I70 Frontage Rd N Ste 3
Wheat Ridge, CO

Boring Started: 06-23-2021	Boring Completed: 06-23-2021
Drill Rig: CME-75	Driller: Terracon
Project No.: 25215031	

THIS BORING LOG IS NOT VALID IF SEPARATED FROM ORIGINAL REPORT. GEO SMART LOG-NO WELL 25215031 QTS AURORA_USE THIS ONE.GPJ TERRACON_DATATEMPLATE.GDT 8/17/21

BORING LOG NO. 24

PROJECT: QTS Aurora

CLIENT: Quality Technology Services, Inc.

**SITE: Gun Club Road and Colfax Avenue
Aurora, Colorado**

LOCATION See Exploration Plan

Latitude: 39.7340° Longitude: -104.7136°

Approximate Surface Elev.: 5544 (Ft.) +/-

MODEL LAYER	GRAPHIC LOG	DEPTH	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	SWELL (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
1		0.0	5544+/-									37-16-21	66
			SANDY LEAN CLAY (CL), brown, medium stiff to very stiff				3-3						
				5			5-8						
							8-10			16.3	112		
		11.0	5533+/-	10			8-14						
3			**CLAYSTONE**, gray to brown, firm to medium hard				14-20						
				15			15-25			22.3	106		
				20			50/12"						
				25			50/12"			21.1	106		
				30			50/10"						
				35			50/10"			23.1	104		
		40.0	5504+/-	40			50/10"						
			Boring Terminated at 40 Feet										

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
4-inch diameter solid stem continuous flight power auger

Abandonment Method:
Boring backfilled with auger cuttings upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Notes:

Elevations were interpolated from the ALTA/NSPS Land Title Survey by Martin/Martin (Job No. 19.1522.C.86, revised March 29, 2021) and provided to us by QTS.

WATER LEVEL OBSERVATIONS

None encountered while drilling

Terracon
10625 W I70 Frontage Rd N Ste 3
Wheat Ridge, CO

Boring Started: 06-25-2021	Boring Completed: 06-25-2021
Drill Rig: CME-75	Driller: Terracon
Project No.: 25215031	

THIS BORING LOG IS NOT VALID IF SEPARATED FROM ORIGINAL REPORT. GEO SMART LOG-NO WELL 25215031 QTS AURORA_USE THIS ONE.GPJ TERRACON_DATATEMPLATE.GDT 8/17/21

BORING LOG NO. 25

PROJECT: QTS Aurora

CLIENT: Quality Technology Services, Inc.

**SITE: Gun Club Road and Colfax Avenue
Aurora, Colorado**

LOCATION See Exploration Plan

Latitude: 39.7337° Longitude: -104.7144°

Approximate Surface Elev.: 5546.5 (Ft.) +/-

MODEL LAYER	GRAPHIC LOG	DEPTH	ELEVATION (Ft.)	DESCRIPTION
1		0.0	5546.5+/-	**LEAN CLAY (CL)**, with sand, brown, medium stiff to stiff
		8.0	5538.5+/-	**CLAYSTONE**, gray to brown, medium hard
		13.0	5533.5+/-	**SANDSTONE**, fine to coarse grained, tan, medium hard, weak cementation
3		16.0	5530.5+/-	**CLAYSTONE**, gray to brown, medium hard to hard
		40.0	5506.5+/-	***Boring Terminated at 40 Feet***

DEPTH (Ft.)	SAMPLE TYPE	FIELD TEST RESULTS	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
0					38-17-21	75
5		11-12				
		7-9	20.3	106		
		11-15				
10		17-27	19.1	117		
		50/10"				
15		50/11"	21.7	109		
		50/9"				
20						
25		50/11"	26.1	100		
30		50/11"				
35		50/8"	21.9	111		
40		50/10"				

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
4-inch diameter solid stem continuous flight power auger

Abandonment Method:
Boring backfilled with auger cuttings upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Notes:

Elevations were interpolated from the ALTA/NSPS Land Title Survey by Martin/Martin (Job No. 19.1522.C.86, revised March 29, 2021) and provided to us by QTS.

WATER LEVEL OBSERVATIONS

None encountered while drilling

Terracon
10625 W I70 Frontage Rd N Ste 3
Wheat Ridge, CO

Boring Started: 06-25-2021	Boring Completed: 06-25-2021
Drill Rig: CME-75	Driller: Terracon
Project No.: 25215031	

THIS BORING LOG IS NOT VALID IF SEPARATED FROM ORIGINAL REPORT. GEO SMART LOG-NO WELL 25215031 QTS AURORA_USE THIS ONE.GPJ TERRACON_DATATEMPLATE.GDT 8/17/21

BORING LOG NO. 26

PROJECT: QTS Aurora

CLIENT: Quality Technology Services, Inc.

SITE: Gun Club Road and Colfax Avenue
Aurora, Colorado

LOCATION See Exploration Plan

Latitude: 39.7337° Longitude: -104.7139°

Approximate Surface Elev.: 5546 (Ft.) +/-

MODEL LAYER	GRAPHIC LOG	DEPTH	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	SWELL (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
1			**LEAN CLAY (CL)**, with sand, brown, stiff				7-9			19.4	109	36-16-20	82
		5.0	5541+/-	5			16-28						
3			**CLAYSTONE**, gray to brown, medium hard				15-27	+10.7 @ 500 psf		19.9	108		
		11.5	5534.5+/-	10			50/11"						
		14.0	**SANDSTONE**, fine to coarse grained, tan, hard, moderate cementation 5532+/-				50/8"			7.0			
			CLAYSTONE, gray to brown, hard to very hard	15			50/11"						
				20			50/10"			20.4	108		
				25			15-30						
				30			50/6"			18.3	110		
				35			16-26						
				40			50/10"			21.8	106		
				45			50/11"						
		50.0	5496+/-	50			50/8"						
			Boring Terminated at 50 Feet										

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
4-inch diameter solid stem continuous flight power auger

Abandonment Method:
Boring backfilled with auger cuttings upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Notes:

Elevations were interpolated from the ALTA/NSPS Land Title Survey by Martin/Martin (Job No. 19.1522.C.86, revised March 29, 2021) and provided to us by QTS.

WATER LEVEL OBSERVATIONS

None encountered while drilling

Terracon
10625 W I70 Frontage Rd N Ste 3
Wheat Ridge, CO

Boring Started: 06-22-2021 | Boring Completed: 06-22-2021

Drill Rig: CME-55 | Driller: Terracon

Project No.: 25215031

THIS BORING LOG IS NOT VALID IF SEPARATED FROM ORIGINAL REPORT. GEO SMART LOG-NO WELL 25215031 QTS AURORA_USE THIS ONE.GPJ TERRACON_DATATEMPLATE.GDT 8/17/21

BORING LOG NO. 27

PROJECT: QTS Aurora

CLIENT: Quality Technology Services, Inc.

**SITE: Gun Club Road and Colfax Avenue
Aurora, Colorado**

LOCATION See Exploration Plan

Latitude: 39.7337° Longitude: -104.7135°

Approximate Surface Elev.: 5545 (Ft.) +/-

MODEL LAYER	GRAPHIC LOG	DEPTH	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	SWELL (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
1			**LEAN CLAY (CL)**, trace sand, brown, very stiff									46-16-30	91
							7-13			18.4	112		
				5			9-17			14.1	117		
							8-14	+1.7 @ 500 psf		18.7	109		
		9.0	5536+/-	10			11-20			21.4	107		
3			**CLAYSTONE**, gray to brown, firm to hard				14-26	+3.0 @ 500 psf		21.4	106		
				15			19-24			22.1	105		
							50/12"			21.8	108		
				20									
				25			50/10"			19.2	111		
							50/12"			20.4	107		
				30									
							50/9"			22.7	103		
				35									
		40.0	5505+/-	40			50/8"						
			Boring Terminated at 40 Feet										

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
4-inch diameter solid stem continuous flight power auger

Abandonment Method:
Boring backfilled with auger cuttings upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Notes:

Elevations were interpolated from the ALTA/NSPS Land Title Survey by Martin/Martin (Job No. 19.1522.C.86, revised March 29, 2021) and provided to us by QTS.

WATER LEVEL OBSERVATIONS

None encountered while drilling

Terracon
10625 W I70 Frontage Rd N Ste 3
Wheat Ridge, CO

Boring Started: 06-03-2021	Boring Completed: 06-03-2021
Drill Rig: CME-550	Driller: Terracon
Project No.: 25215031	

THIS BORING LOG IS NOT VALID IF SEPARATED FROM ORIGINAL REPORT. GEO SMART LOG-NO WELL 25215031 QTS AURORA_USE THIS ONE.GPJ TERRACON_DATATEMPLATE.GDT 8/17/21

BORING LOG NO. 28

PROJECT: QTS Aurora

CLIENT: Quality Technology Services, Inc.

SITE: Gun Club Road and Colfax Avenue
Aurora, Colorado

LOCATION See Exploration Plan

Latitude: 39.7334° Longitude: -104.7146°

Approximate Surface Elev.: 5547 (Ft.) +/-

MODEL LAYER	GRAPHIC LOG	DEPTH	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	SWELL (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
1			**LEAN CLAY (CL)**, with sand, brown, stiff				8-10					41-17-24	80
				5			7-7	+4.2 @ 500 psf		18.2	106		
							12-19						
				10			17-27			18.4	110		
							50/11"						
				15			15-25			18.6			
							50/11"						
3				20									
				25			50/9"		39860	18.8	109		
							50/11"						
				30									
							50/11"			18.7	111		
		7.0	5540+/-	35									
			CLAYSTONE, gray to brown, firm to hard				50/10"						
		40.0	5507+/-	40									
			Boring Terminated at 40 Feet										

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
4-inch diameter solid stem continuous flight power auger

Abandonment Method:
Boring backfilled with auger cuttings upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Notes:

Elevations were interpolated from the ALTA/NSPS Land Title Survey by Martin/Martin (Job No. 19.1522.C.86, revised March 29, 2021) and provided to us by QTS.

WATER LEVEL OBSERVATIONS

None encountered while drilling

Terracon
10625 W I70 Frontage Rd N Ste 3
Wheat Ridge, CO

Boring Started: 06-22-2021	Boring Completed: 06-22-2021
Drill Rig: CME-55	Driller: Terracon
Project No.: 25215031	

THIS BORING LOG IS NOT VALID IF SEPARATED FROM ORIGINAL REPORT. GEO SMART LOG-NO WELL 25215031 QTS AURORA_USE THIS ONE.GPJ TERRACON_DATATEMPLATE.GDT 8/17/21

BORING LOG NO. 29

PROJECT: QTS Aurora

CLIENT: Quality Technology Services, Inc.

**SITE: Gun Club Road and Colfax Avenue
Aurora, Colorado**

LOCATION See Exploration Plan

Latitude: 39.7334° Longitude: -104.7142°

Approximate Surface Elev.: 5546.5 (Ft.) +/-

MODEL LAYER	GRAPHIC LOG	DEPTH	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	SWELL (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
1		0											
			LEAN CLAY (CL), brown, very stiff				11-21						
		4.5	5542+/-	5			12-27	+7.3 @ 500 psf		16.4	115		
			CLAYSTONE, gray to brown, medium hard				16-27						
		11.5	5535+/-	10			50/12"						
			SANDSTONE, fine to coarse grained, tan, medium hard to very hard, moderate to strong cementation				50/5"			9.5			
		17.5	5529+/-	15			50/12"						
3			**CLAYSTONE**, gray to brown, medium hard to hard	20			50/12"			20.5	105		
				25			50/10"						
				30			50/9"			17.8	110		
				35			50/9"						
		40.0	5506.5+/-	40			50/10"						
			Boring Terminated at 40 Feet										

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
4-inch diameter solid stem continuous flight power auger

Abandonment Method:
Boring backfilled with auger cuttings upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Notes:

Elevations were interpolated from the ALTA/NSPS Land Title Survey by Martin/Martin (Job No. 19.1522.C.86, revised March 29, 2021) and provided to us by QTS.

WATER LEVEL OBSERVATIONS

None encountered while drilling

Terracon
10625 W I70 Frontage Rd N Ste 3
Wheat Ridge, CO

Boring Started: 06-22-2021	Boring Completed: 06-22-2021
Drill Rig: CME-55	Driller: Terracon
Project No.: 25215031	

THIS BORING LOG IS NOT VALID IF SEPARATED FROM ORIGINAL REPORT. GEO SMART LOG-NO WELL 25215031 QTS AURORA_USE THIS ONE.GPJ TERRACON_DATATEMPLATE.GDT 8/17/21

BORING LOG NO. 30

PROJECT: QTS Aurora

CLIENT: Quality Technology Services, Inc.

**SITE: Gun Club Road and Colfax Avenue
Aurora, Colorado**

LOCATION See Exploration Plan

Latitude: 39.7334° Longitude: -104.7137°

Approximate Surface Elev.: 5546 (Ft.) +/-

MODEL LAYER	GRAPHIC LOG	DEPTH	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	SWELL (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
1		0.0	5546+/-									32-16-16	89
			LEAN CLAY (CL), trace sand, brown, stiff to very stiff				5-10						
				5			7-16	+2.1 @ 500 psf		17.5	114		
							15-28						
				10			50/11"			18.6	108		
		7.0	5539+/-				50/6"						
3			**CLAYSTONE**, gray to brown, soft to medium hard	15			50/10"			18.7	112		
		13.5	5532.5+/-										
			SANDSTONE, fine to coarse grained, tan, medium hard, moderate cementation	20			50/8"						
		16.5	5529.5+/-										
			CLAYSTONE, gray to brown, hard to very hard	25			50/9"			20.9	106		
				30			50/9"						
		35.0	5511+/-										
			SANDSTONE, fine to coarse grained, tan, very hard, strong cementation	35			50/3"			19.9	108		
		39.0	5507+/-										
		40.0	5506+/- **CLAYSTONE**, gray to brown, very hard	40			50/3"						
			Boring Terminated at 40 Feet										

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
4-inch diameter solid stem continuous flight power auger

Abandonment Method:
Boring backfilled with auger cuttings upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Notes:

Elevations were interpolated from the ALTA/NSPS Land Title Survey by Martin/Martin (Job No. 19.1522.C.86, revised March 29, 2021) and provided to us by QTS.

WATER LEVEL OBSERVATIONS

None encountered while drilling

Terracon
10625 W I70 Frontage Rd N Ste 3
Wheat Ridge, CO

Boring Started: 06-03-2021	Boring Completed: 06-03-2021
Drill Rig: CME-550	Driller: Terracon
Project No.: 25215031	

THIS BORING LOG IS NOT VALID IF SEPARATED FROM ORIGINAL REPORT. GEO SMART LOG-NO WELL 25215031 QTS AURORA_USE THIS ONE.GPJ TERRACON_DATATEMPLATE.GDT 8/17/21

BORING LOG NO. 31

PROJECT: QTS Aurora

CLIENT: Quality Technology Services, Inc.

**SITE: Gun Club Road and Colfax Avenue
Aurora, Colorado**

LOCATION See Exploration Plan

Latitude: 39.7331° Longitude: -104.7144°

Approximate Surface Elev.: 5548 (Ft.) +/-

MODEL LAYER	GRAPHIC LOG	DEPTH	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	SWELL (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
1			**SANDY LEAN CLAY (CL)**, brown, stiff				8-8			18.8	105	30-15-15	63
										20.9	106		
										17.2	104		
				5			5-9			16.1	114		
		6.5	5541.5+/-										
3			**CLAYSTONE**, brown, firm				10-22	+5.9 @ 500 psf		20.6	106		
		10.0	5538+/-	10			13-24						
		12.0	**SANDSTONE**, fine to coarse grained, tan, medium hard 5536+/-										
			CLAYSTONE, gray, medium hard to very hard				15-28			23.6	103		
				15			14-24						
				20			50/2"			24.5	102		
				25			50/11"						
				30			50/8"						
				35			50/9"						
		40.0	5508+/-	40			50/6"						
			Boring Terminated at 40 Feet										

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
4-inch diameter solid stem continuous flight power auger

Abandonment Method:
Boring backfilled with auger cuttings upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Notes:

Elevations were interpolated from the ALTA/NSPS Land Title Survey by Martin/Martin (Job No. 19.1522.C.86, revised March 29, 2021) and provided to us by QTS.

WATER LEVEL OBSERVATIONS

None encountered while drilling

Terracon
10625 W I70 Frontage Rd N Ste 3
Wheat Ridge, CO

Boring Started: 06-24-2021	Boring Completed: 06-24-2021
Drill Rig: CME-550	Driller: Terracon
Project No.: 25215031	

THIS BORING LOG IS NOT VALID IF SEPARATED FROM ORIGINAL REPORT. GEO SMART LOG-NO WELL 25215031 QTS AURORA_USE THIS ONE.GPJ TERRACON_DATATEMPLATE.GDT 8/17/21

BORING LOG NO. 32

PROJECT: QTS Aurora

CLIENT: Quality Technology Services, Inc.

SITE: Gun Club Road and Colfax Avenue
Aurora, Colorado

LOCATION See Exploration Plan

Latitude: 39.7331° Longitude: -104.7139°

Approximate Surface Elev.: 5547 (Ft.) +/-

MODEL LAYER	GRAPHIC LOG	DEPTH	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	SWELL (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
1		0.0	5547+/-										
		LEAN CLAY (CL), with sand, brown, very stiff											
		4.0	5543+/-				11-20					32-14-18	75
3		**CLAYSTONE**, gray to brown, firm to hard		5			10-14	+7.1 @ 500 psf		18.5	108		
							50/12"						
				10			50/8"			14.8	119		
							50/7"						
				15			50/9"			17.1	113		
				20			50/9"						
				25			50/9"			20.7			
				30			50/10"						
				35			50/7"			20.1	108		
		40.0	5507+/-	40			50/8"						
		Boring Terminated at 40 Feet											

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
4-inch diameter solid stem continuous flight power auger

Abandonment Method:
Boring backfilled with auger cuttings upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Notes:

Elevations were interpolated from the ALTA/NSPS Land Title Survey by Martin/Martin (Job No. 19.1522.C.86, revised March 29, 2021) and provided to us by QTS.

WATER LEVEL OBSERVATIONS

None encountered while drilling

Terracon
10625 W I70 Frontage Rd N Ste 3
Wheat Ridge, CO

Boring Started: 06-24-2021	Boring Completed: 06-24-2021
Drill Rig: CME-550	Driller: Terracon
Project No.: 25215031	

THIS BORING LOG IS NOT VALID IF SEPARATED FROM ORIGINAL REPORT. GEO SMART LOG-NO WELL 25215031 QTS AURORA_USE THIS ONE.GPJ TERRACON_DATATEMPLATE.GDT 8/17/21

BORING LOG NO. 33

PROJECT: QTS Aurora

CLIENT: Quality Technology Services, Inc.

**SITE: Gun Club Road and Colfax Avenue
Aurora, Colorado**

LOCATION See Exploration Plan

Latitude: 39.7331° Longitude: -104.7134°

Approximate Surface Elev.: 5546.5 (Ft.) +/-

MODEL LAYER	GRAPHIC LOG	DEPTH	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	SWELL (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
1			**LEAN CLAY (CL)**, with sand, brown, stiff				6-9			20.4	107	39-16-23	81
				5			7-10						
							8-15	+2.7 @ 500 psf		19.8	106		
		10.0	5536.5+/-	10			17-25						
			CLAYSTONE, gray to brown, soft to very hard				16-30			22.0	105		
				15			50/12"						
							20-26			21.8	106		
		25.0	5521.5+/-	20									
			SANDSTONE, fine to coarse grained, tan, very hard, strong cementation				50/4"						
3		30.0	5516.5+/-	25									
			CLAYSTONE, gray to brown, medium hard to very hard				50/3"			9.2	131		
				30									
							50/10"						
				35									
		40.0	5506.5+/-				50/7"			17.4			
			SANDSTONE, fine to coarse grained, tan, very hard, strong cementation	40									
		44.5	5502+/-				50/6"						
			CLAYSTONE, gray to brown, very hard	45									
		50.0	5496.5+/-				50/6"						
			Boring Terminated at 50 Feet	50									

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
4-inch diameter solid stem continuous flight power auger

Abandonment Method:
Boring backfilled with auger cuttings upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Notes:

Elevations were interpolated from the ALTA/NSPS Land Title Survey by Martin/Martin (Job No. 19.1522.C.86, revised March 29, 2021) and provided to us by QTS.

WATER LEVEL OBSERVATIONS

None encountered while drilling

Terracon
10625 W I70 Frontage Rd N Ste 3
Wheat Ridge, CO

Boring Started: 06-04-2021	Boring Completed: 06-04-2021
Drill Rig: CME-550	Driller: Terracon
Project No.: 25215031	

THIS BORING LOG IS NOT VALID IF SEPARATED FROM ORIGINAL REPORT. GEO SMART LOG-NO WELL 25215031 QTS AURORA_USE THIS ONE.GPJ TERRACON_DATATEMPLATE.GDT 8/17/21

BORING LOG NO. 34

PROJECT: QTS Aurora

CLIENT: Quality Technology Services, Inc.

**SITE: Gun Club Road and Colfax Avenue
Aurora, Colorado**

LOCATION See Exploration Plan

Latitude: 39.7328° Longitude: -104.7146°

Approximate Surface Elev.: 5550 (Ft.) +/-

MODEL LAYER	GRAPHIC LOG	DEPTH	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	SWELL (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
1			**LEAN CLAY (CL)**, with sand, brown, stiff to very stiff				8-12					36-15-21	77
		7.0	5543+/-	5			6-12	+2.3 @ 500 psf		19.7	108		
3			**CLAYSTONE**, gray to brown, firm to hard				11-22						
				10			21-24			22.7	105		
							50/9"						
				15			50/7"			16.8	115		
				20			50/12"						
				25			50/7"			16.7	111		
							50/8"						
				30			50/9"			21.6	107		
		37.5	5512.5+/-	35									
		40.0	**SANDSTONE**, fine to coarse grained, tan, hard, moderate cementation 5510+/-	40			50/7"						
			Boring Terminated at 40 Feet										

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
4-inch diameter solid stem continuous flight power auger

Abandonment Method:
Boring backfilled with auger cuttings upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Notes:

Elevations were interpolated from the ALTA/NSPS Land Title Survey by Martin/Martin (Job No. 19.1522.C.86, revised March 29, 2021) and provided to us by QTS.

WATER LEVEL OBSERVATIONS

None encountered while drilling

Terracon
10625 W I70 Frontage Rd N Ste 3
Wheat Ridge, CO

Boring Started: 06-24-2021	Boring Completed: 06-24-2021
Drill Rig: CME-550	Driller: Terracon
Project No.: 25215031	

THIS BORING LOG IS NOT VALID IF SEPARATED FROM ORIGINAL REPORT. GEO SMART LOG-NO WELL 25215031 QTS AURORA_USE THIS ONE.GPJ TERRACON_DATATEMPLATE.GDT 8/17/21

BORING LOG NO. 35

PROJECT: QTS Aurora

CLIENT: Quality Technology Services, Inc.

SITE: Gun Club Road and Colfax Avenue
Aurora, Colorado

LOCATION See Exploration Plan

Latitude: 39.7328° Longitude: -104.7141°

Approximate Surface Elev.: 5549 (Ft.) +/-

MODEL LAYER	GRAPHIC LOG	DEPTH	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	SWELL (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
1			**LEAN CLAY (CL)**, trace sand, brown, stiff to very stiff				7-11			16.6	113	40-18-22	87
				5			9-12						
		8.0	5541+/-				10-19	+6.2 @ 500 psf		18.6	112		
			CLAYSTONE, gray to brown, hard	10			50/7"						
		11.0	5538+/-										
			SANDSTONE, fine to coarse grained, tan, medium hard, moderate cementation				50/12"			20.1	109		
		13.0	5536+/-	15			14-24						
3			**CLAYSTONE**, gray to brown, medium hard to very hard	20			50/12"			19.6	113		
				25			50/7"						
				30			50/6"			20.3	108		
				35			50/5"						
		40.0	5509+/-	40			50/4"			15.6	110		
			Boring Terminated at 40 Feet										

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
4-inch diameter solid stem continuous flight power auger

Abandonment Method:
Boring backfilled with auger cuttings upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Notes:

Elevations were interpolated from the ALTA/NSPS Land Title Survey by Martin/Martin (Job No. 19.1522.C.86, revised March 29, 2021) and provided to us by QTS.

WATER LEVEL OBSERVATIONS

None encountered while drilling

Terracon
10625 W I70 Frontage Rd N Ste 3
Wheat Ridge, CO

Boring Started: 06-23-2021 | Boring Completed: 06-23-2021

Drill Rig: CME-75 | Driller: Terracon

Project No.: 25215031

THIS BORING LOG IS NOT VALID IF SEPARATED FROM ORIGINAL REPORT. GEO SMART LOG-NO WELL 25215031 QTS AURORA_USE THIS ONE.GPJ TERRACON_DATATEMPLATE.GDT 8/17/21

BORING LOG NO. 36

PROJECT: QTS Aurora

CLIENT: Quality Technology Services, Inc.

**SITE: Gun Club Road and Colfax Avenue
Aurora, Colorado**

LOCATION See Exploration Plan

Latitude: 39.7328° Longitude: -104.7137°

Approximate Surface Elev.: 5548 (Ft.) +/-

MODEL LAYER	GRAPHIC LOG	DEPTH	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	SWELL (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
1			**LEAN CLAY (CL)**, with sand, brown, very stiff				7-13					40-17-23	77
		5.0	5543+/-	5			12-22	+6.0 @ 500 psf		15.2	117		
			CLAYSTONE, gray to brown, firm to very hard				12-19			18.7	110		
				10			15-27	+6.2 @ 500 psf		21.7	105		
							17-27						
				15			50/8"						
3				20			50/10"			17.8	116		
		25.0	5523+/-	25			50/6"						
			SANDSTONE, fine to coarse grained, tan, very hard, strong cementation										
		30.0	5518+/-	30			50/6"			22.6	104		
			CLAYSTONE, gray to brown, hard										
				35			50/7"						
		40.0	5508+/-	40			50/8"			20.6	107		
			Boring Terminated at 40 Feet										

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
4-inch diameter solid stem continuous flight power auger

Abandonment Method:
Boring backfilled with auger cuttings upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Notes:

Elevations were interpolated from the ALTA/NSPS Land Title Survey by Martin/Martin (Job No. 19.1522.C.86, revised March 29, 2021) and provided to us by QTS.

WATER LEVEL OBSERVATIONS

None encountered while drilling

Terracon
10625 W I70 Frontage Rd N Ste 3
Wheat Ridge, CO

Boring Started: 06-03-2021	Boring Completed: 06-03-2021
Drill Rig: CME-75	Driller: Terracon
Project No.: 25215031	

THIS BORING LOG IS NOT VALID IF SEPARATED FROM ORIGINAL REPORT. GEO SMART LOG-NO WELL 25215031 QTS AURORA_USE THIS ONE.GPJ TERRACON_DATATEMPLATE.GDT 8/17/21

BORING LOG NO. 37

PROJECT: QTS Aurora

CLIENT: Quality Technology Services, Inc.

**SITE: Gun Club Road and Colfax Avenue
Aurora, Colorado**

LOCATION See Exploration Plan

Latitude: 39.7325° Longitude: -104.7144°

Approximate Surface Elev.: 5550.5 (Ft.) +/-

MODEL LAYER	GRAPHIC LOG	DEPTH	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	SWELL (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
1		0.0	5550.5+/-									38-17-21	88
			LEAN CLAY (CL), trace sand, brown, very stiff				8-12						
				5			10-12			23.0	101		
							10-17						
		10.0	5540.5+/-	10			15-20	+3.6 @ 500 psf		19.0	110		
			CLAYSTONE, tan to light brown, soft to medium hard				17-22						
		15.5	5535+/-	15			50/7"			22.7	104		
			SANDSTONE, fine to coarse grained, brown, hard, moderate cementation										
		19.5	5531+/-	20			50/6"						
3			**CLAYSTONE**, tan to gray, hard to very hard										
				25			50/9"			20.8			
				30			50/6"						
		35.0	5515.5+/-	35			50/6"			18.5	117		
			SANDSTONE, fine to coarse grained, tan to gray, very hard, strong cementation										
		40.0	5510.5+/-	40			50/4"						
			Boring Terminated at 40 Feet										

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
4-inch diameter solid stem continuous flight power auger

Abandonment Method:
Boring backfilled with auger cuttings upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Notes:

Elevations were interpolated from the ALTA/NSPS Land Title Survey by Martin/Martin (Job No. 19.1522.C.86, revised March 29, 2021) and provided to us by QTS.

WATER LEVEL OBSERVATIONS

None encountered while drilling

Terracon
10625 W I70 Frontage Rd N Ste 3
Wheat Ridge, CO

Boring Started: 06-23-2021 | Boring Completed: 06-23-2021

Drill Rig: CME-75 | Driller: Terracon

Project No.: 25215031

THIS BORING LOG IS NOT VALID IF SEPARATED FROM ORIGINAL REPORT. GEO SMART LOG-NO WELL 25215031 QTS AURORA_USE THIS ONE.GPJ TERRACON_DATATEMPLATE.GDT 8/17/21

BORING LOG NO. 38

PROJECT: QTS Aurora

CLIENT: Quality Technology Services, Inc.

**SITE: Gun Club Road and Colfax Avenue
Aurora, Colorado**

LOCATION See Exploration Plan

Latitude: 39.7325° Longitude: -104.7139°

Approximate Surface Elev.: 5550 (Ft.) +/-

MODEL LAYER	GRAPHIC LOG	DEPTH	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	SWELL (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
1		0.0	5550+/-										
			SANDY LEAN CLAY (CL), brown, stiff to very stiff				6-11					32-15-17	70
		7.0	5543+/-	5			13-13			16.4			
			CLAYSTONE, gray to brown, firm to medium hard				12-23			17.6			
				10			13-23			18.0			
							16-23						
				15			14-20			19.6			
		17.5	5532.5+/-										
			SANDSTONE, fine to coarse grained, tan, medium hard, weak cementation				15-30						
		22.5	5527.5+/-	20									
3			**CLAYSTONE**, gray to brown, hard to medium hard	25			50/12"			16.4			
							15-27						
				30									
				35			50/11"			21.5			
							50/11"						
				40									
				45			50/11"			26.7	96		
		50.0	5500+/-										
			Boring Terminated at 50 Feet	50			50/8"						

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
4-inch diameter solid stem continuous flight power auger

Abandonment Method:
Boring backfilled with auger cuttings upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Notes:

Elevations were interpolated from the ALTA/NSPS Land Title Survey by Martin/Martin (Job No. 19.1522.C.86, revised March 29, 2021) and provided to us by QTS.

WATER LEVEL OBSERVATIONS

None encountered while drilling

Terracon
10625 W I70 Frontage Rd N Ste 3
Wheat Ridge, CO

Boring Started: 06-24-2021	Boring Completed: 06-24-2021
Drill Rig: CME-550	Driller: Terracon
Project No.: 25215031	

THIS BORING LOG IS NOT VALID IF SEPARATED FROM ORIGINAL REPORT. GEO SMART LOG-NO WELL 25215031 QTS AURORA_USE THIS ONE.GPJ TERRACON_DATATEMPLATE.GDT 8/17/21

BORING LOG NO. 39

PROJECT: QTS Aurora

CLIENT: Quality Technology Services, Inc.

**SITE: Gun Club Road and Colfax Avenue
Aurora, Colorado**

LOCATION See Exploration Plan

Latitude: 39.7325° Longitude: -104.7134°

Approximate Surface Elev.: 5549 (Ft.) +/-

MODEL LAYER	GRAPHIC LOG	DEPTH	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	SWELL (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
1			**LEAN CLAY (CL)**, trace sand, brown, stiff				5-10			18.0	114	41-20-21	88
		5.5	5543.5+/-	5			10-24	+7.1 @ 500 psf		14.5	117		
3			**CLAYSTONE**, gray to brown, firm to medium hard				10-21			18.7	111		
				10			13-27						
							50/11"			14.7	125		
				15			50/10"						
				20			50/11"			18.0	111		
				25			14-18						
		28.5	5520.5+/-										
			SANDSTONE, fine to coarse grained, tan, very hard, strong cementation	30			50/2"			15.0	115		
		32.0	5517+/-										
			CLAYSTONE, gray to brown, hard to medium hard	35			50/11"						
		40.0	5509+/-	40			50/8"			15.3	114		
			Boring Terminated at 40 Feet										

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
4-inch diameter solid stem continuous flight power auger

Abandonment Method:
Boring backfilled with auger cuttings upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Notes:

Elevations were interpolated from the ALTA/NSPS Land Title Survey by Martin/Martin (Job No. 19.1522.C.86, revised March 29, 2021) and provided to us by QTS.

WATER LEVEL OBSERVATIONS

None encountered while drilling

Terracon
10625 W I70 Frontage Rd N Ste 3
Wheat Ridge, CO

Boring Started: 06-04-2021	Boring Completed: 06-04-2021
Drill Rig: CME-55	Driller: Terracon
Project No.: 25215031	

THIS BORING LOG IS NOT VALID IF SEPARATED FROM ORIGINAL REPORT. GEO SMART LOG-NO WELL 25215031 QTS AURORA_USE THIS ONE.GPJ TERRACON_DATATEMPLATE.GDT 8/17/21

BORING LOG NO. 40

PROJECT: QTS Aurora

CLIENT: Quality Technology Services, Inc.

**SITE: Gun Club Road and Colfax Avenue
Aurora, Colorado**

LOCATION See Exploration Plan

Latitude: 39.7322° Longitude: -104.7146°

Approximate Surface Elev.: 5552 (Ft.) +/-

MODEL LAYER	GRAPHIC LOG	DEPTH	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	SWELL (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
1			**LEAN CLAY (CL)**, with sand, brown, stiff to very stiff				11-12					31-18-13	81
				5			12-16						
							3-9			16.1	113		
		10.0	5542+/-	10			50/11"	+5.2 @ 500 psf		15.1	119		
			CLAYSTONE, gray to brown, medium hard to very hard				50/7"						
				15			50/9"						
				20			50/7"			19.1	111		
							50/6"						
3				25									
				30			50/8"			18.2	108		
		36.5	5515.5+/-										
			SANDSTONE, fine to coarse grained, tan, very hard, strong cementation	35			50/7"						
				40			50/6"			18.4	109		
		42.0	5510+/-										
			CLAYSTONE, gray to brown, medium hard to hard	45			50/12"						
		50.0	5502+/-	50			50/7"			18.2	110		
			Boring Terminated at 50 Feet										

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
4-inch diameter solid stem continuous flight power auger

Abandonment Method:
Boring backfilled with auger cuttings upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Notes:

Elevations were interpolated from the ALTA/NSPS Land Title Survey by Martin/Martin (Job No. 19.1522.C.86, revised March 29, 2021) and provided to us by QTS.

WATER LEVEL OBSERVATIONS

None encountered while drilling

Terracon
10625 W I70 Frontage Rd N Ste 3
Wheat Ridge, CO

Boring Started: 06-30-2021	Boring Completed: 06-30-2021
Drill Rig: CME-550	Driller: Terracon
Project No.: 25215031	

THIS BORING LOG IS NOT VALID IF SEPARATED FROM ORIGINAL REPORT. GEO SMART LOG-NO WELL 25215031 QTS AURORA_USE THIS ONE.GPJ TERRACON_DATATEMPLATE.GDT 8/17/21

BORING LOG NO. 41

PROJECT: QTS Aurora

CLIENT: Quality Technology Services, Inc.

**SITE: Gun Club Road and Colfax Avenue
Aurora, Colorado**

LOCATION See Exploration Plan

Latitude: 39.7322° Longitude: -104.7141°

Approximate Surface Elev.: 5551.5 (Ft.) +/-

MODEL LAYER	GRAPHIC LOG	DEPTH	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	SWELL (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
1			**LEAN CLAY (CL)**, with sand, brown, stiff to very stiff				7-11					31-16-15	79
				5			9-12			12.5	98		
							50/10"						
				10			50/12"			18.8			
							50/11"						
				15			50/10"						
							50/11"			19.3	107		
3				20									
				25			50/11"						
							50/11"			22.0	103		
				30									
							50/11"						
				35									
		40.0	5511.5+/-				50/11"						
			Boring Terminated at 40 Feet	40									

7.0 5544.5+/-

CLAYSTONE, gray to brown, medium hard to hard

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
4-inch diameter solid stem continuous flight power auger

Abandonment Method:
Boring backfilled with auger cuttings upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Notes:

Elevations were interpolated from the ALTA/NSPS Land Title Survey by Martin/Martin (Job No. 19.1522.C.86, revised March 29, 2021) and provided to us by QTS.

WATER LEVEL OBSERVATIONS

None encountered while drilling

Terracon
10625 W I70 Frontage Rd N Ste 3
Wheat Ridge, CO

Boring Started: 06-29-2021	Boring Completed: 06-29-2021
Drill Rig: CME-550	Driller: Terracon
Project No.: 25215031	

THIS BORING LOG IS NOT VALID IF SEPARATED FROM ORIGINAL REPORT. GEO SMART LOG-NO WELL 25215031 QTS AURORA_USE THIS ONE.GPJ TERRACON_DATATEMPLATE.GDT 8/17/21

BORING LOG NO. 42

PROJECT: QTS Aurora

CLIENT: Quality Technology Services, Inc.

**SITE: Gun Club Road and Colfax Avenue
Aurora, Colorado**

LOCATION See Exploration Plan

Latitude: 39.7322° Longitude: -104.7137°

Approximate Surface Elev.: 5551 (Ft.) +/-

MODEL LAYER	GRAPHIC LOG	DEPTH	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	SWELL (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
1			**LEAN CLAY (CL)**, with sand, brown, stiff				7-10					39-19-20	83
				5			7-8			20.6	106		
							4-10						
		10.0	5541+/-	10			12-20			19.9	111		
3			**CLAYSTONE**, gray to brown, firm to medium hard				16-27	+10.4 @ 500 psf		22.8	105		
				15			18-30						
				20			50/12"			18.6	111		
				25			50/11"						
				30			50/12"			23.7	103		
				35			50/8"						
		40.0	5511+/-	40			50/9"			18.3	114		
			Boring Terminated at 40 Feet										

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
4-inch diameter solid stem continuous flight power auger

Abandonment Method:
Boring backfilled with auger cuttings upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Notes:

Elevations were interpolated from the ALTA/NSPS Land Title Survey by Martin/Martin (Job No. 19.1522.C.86, revised March 29, 2021) and provided to us by QTS.

WATER LEVEL OBSERVATIONS

None encountered while drilling

Terracon
10625 W I70 Frontage Rd N Ste 3
Wheat Ridge, CO

Boring Started: 06-30-2021	Boring Completed: 06-30-2021
Drill Rig: CME-550	Driller: Terracon
Project No.: 25215031	

THIS BORING LOG IS NOT VALID IF SEPARATED FROM ORIGINAL REPORT. GEO SMART LOG-NO WELL 25215031 QTS AURORA_USE THIS ONE.GPJ TERRACON_DATATEMPLATE.GDT 8/17/21

BORING LOG NO. F1

PROJECT: QTS Aurora

CLIENT: Quality Technology Services, Inc.

**SITE: Gun Club Road and Colfax Avenue
Aurora, Colorado**

LOCATION See Exploration Plan

Latitude: 39.7340° Longitude: -104.7123°

Approximate Surface Elev.: 5541 (Ft.) +/-

MODEL LAYER	GRAPHIC LOG	DEPTH	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	SWELL (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
1		0.0										26-14-12	63
			SANDY LEAN CLAY (CL), brown, very stiff to hard				7-13						
		6.0	5535+/-	5			50/10"			10.4	130		
			CLAYSTONE, brown, medium hard to very hard				50/11"						
				10			50/5"	+6.0 @ 500 psf		10.7	126		
		12.0	5529+/-										
			SANDSTONE, fine to coarse grained, brown, medium hard, weak to hard cementation				16-26						
		15.0	5526+/-	15			50/12"			11.5			
3			**CLAYSTONE**, gray to light brown, medium hard to very hard										
				20			50/11"			19.3	108		
				25			50/12"						
				30			50/10"			23.3	102		
				35			50/3"						
		40.0	5501+/-	40			50/9"						
			Boring Terminated at 40 Feet										

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
4-inch diameter solid stem continuous flight power auger

Abandonment Method:
Boring backfilled with auger cuttings upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Notes:

Elevations were interpolated from the ALTA/NSPS Land Title Survey by Martin/Martin (Job No. 19.1522.C.86, revised March 29, 2021) and provided to us by QTS.

WATER LEVEL OBSERVATIONS

None encountered while drilling

Terracon
10625 W I70 Frontage Rd N Ste 3
Wheat Ridge, CO

Boring Started: 06-04-2021	Boring Completed: 06-04-2021
Drill Rig: CME-550	Driller: Terracon
Project No.: 25215031	

THIS BORING LOG IS NOT VALID IF SEPARATED FROM ORIGINAL REPORT. GEO SMART LOG-NO WELL 25215031 QTS AURORA_USE THIS ONE.GPJ TERRACON_DATATEMPLATE.GDT 8/17/21

BORING LOG NO. F2

PROJECT: QTS Aurora

CLIENT: Quality Technology Services, Inc.

**SITE: Gun Club Road and Colfax Avenue
Aurora, Colorado**

LOCATION See Exploration Plan

Latitude: 39.7331° Longitude: -104.7116°

Approximate Surface Elev.: 5545.5 (Ft.) +/-

MODEL LAYER	GRAPHIC LOG	DEPTH	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	SWELL (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
1			**LEAN CLAY (CL)**, with organics, trace sand, calcareous, light brown to brown, stiff				5-9					37-15-22	86
				5			6-11			20.8	104		
		7.5	5538+/-				14-22						
			CLAYSTONE, brown to brownish gray, medium hard to hard	10			13-23	+5.2 @ 500 psf		21.1	106		
							50/11"						
				15			50/9"			15.1	114		
		18.0	5527.5+/-										
			SANDSTONE, fine grained, orangish brown, hard, moderate cementation	20			50/8"			12.9	121		
		23.0	5522.5+/-										
3			**CLAYSTONE**, grayish brown to brown, medium hard to hard	25			50/9"						
				30			50/10"			23.8	104		
				35			50/10"						
				40			50/9"						
				45			50/8"						
		50.0	5495.5+/-	50			50/11"						
			Boring Terminated at 50 Feet										

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
4-inch diameter solid stem continuous flight power auger

Abandonment Method:
Boring backfilled with auger cuttings upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Notes:

Elevations were interpolated from the ALTA/NSPS Land Title Survey by Martin/Martin (Job No. 19.1522.C.86, revised March 29, 2021) and provided to us by QTS.

WATER LEVEL OBSERVATIONS

None encountered while drilling

Terracon
10625 W I70 Frontage Rd N Ste 3
Wheat Ridge, CO

Boring Started: 05-31-2021	Boring Completed: 06-02-2021
Drill Rig: CME-55	Driller: Terracon
Project No.: 25215031	

THIS BORING LOG IS NOT VALID IF SEPARATED FROM ORIGINAL REPORT. GEO SMART LOG-NO WELL 25215031 QTS AURORA_USE THIS ONE.GPJ TERRACON_DATATEMPLATE.GDT 8/17/21

BORING LOG NO. F3

PROJECT: QTS Aurora

CLIENT: Quality Technology Services, Inc.

**SITE: Gun Club Road and Colfax Avenue
Aurora, Colorado**

LOCATION See Exploration Plan

Latitude: 39.7322° Longitude: -104.7123°

Approximate Surface Elev.: 5548 (Ft.) +/-

MODEL LAYER	GRAPHIC LOG	DEPTH	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	SWELL (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
1												52-18-34	85
		3.0	5545+/-				7-13						
			CLAYSTONE, orangish brown to gray, firm	5			11-17	+3.5 @ 500 psf		19.4	110		
							12-17						
				10			12-17			23.5	102		
		12.0	5536+/-										
			SANDSTONE, fine to medium grained, reddish brown, medium hard, weak cementation				18-21						
		15.0	5533+/-	15			13-23			23.3	102		
			CLAYSTONE, gray to brownish gray, medium hard										
3				20			18-27			24.0	101		
		24.0	5524+/-										
			SANDSTONE, fine to medium grained, gray, firm to very hard, weak to strong cementation	25			13-19						
				30			50/1"			20.9	105		
		33.0	5515+/-										
			CLAYSTONE, gray, medium hard	35			50/11"						
		40.0	5508+/-	40			50/10"						
			Boring Terminated at 40 Feet										

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
4-inch diameter solid stem continuous flight power auger

Abandonment Method:
Boring backfilled with auger cuttings upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Notes:

Elevations were interpolated from the ALTA/NSPS Land Title Survey by Martin/Martin (Job No. 19.1522.C.86, revised March 29, 2021) and provided to us by QTS.

WATER LEVEL OBSERVATIONS

None encountered while drilling

Terracon
10625 W I70 Frontage Rd N Ste 3
Wheat Ridge, CO

Boring Started: 05-31-2021	Boring Completed: 06-02-2021
Drill Rig: CME-55	Driller: Terracon
Project No.: 25215031	

THIS BORING LOG IS NOT VALID IF SEPARATED FROM ORIGINAL REPORT. GEO SMART LOG-NO WELL 25215031 QTS AURORA_USE THIS ONE.GPJ TERRACON_DATATEMPLATE.GDT 8/17/21

BORING LOG NO. F4

PROJECT: QTS Aurora

CLIENT: Quality Technology Services, Inc.

SITE: Gun Club Road and Colfax Avenue
Aurora, Colorado

LOCATION See Exploration Plan

Latitude: 39.7340° Longitude: -104.7107°

Approximate Surface Elev.: 5543 (Ft.) +/-

MODEL LAYER	GRAPHIC LOG	DEPTH	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	SWELL (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
1		0					7-10					35-14-21	74
				5			5-7			17.4	108		
							4-5	+0.1 @ 500 psf		16.3	109		
				10			13-17			20.4	105		
		13.0	5530+/-				13-27						
3				15			50/12"			25.6	98		
				20			50/9"			25.4	97		
							50/10"						
		29.5	5513.5+/-	25									
				30			50/9"			22.2	103		
				35			50/6"						
		39.5	5503.5+/-										
		40.0	5503+/-	40			50/6"						

LEAN CLAY (CL), with sand, brown, medium stiff to very stiff

CLAYSTONE, light brown to gray, medium hard to hard

SANDSTONE, fine to coarse grained, light brown to tan, medium hard to very hard, moderate to strong cementation

CLAYSTONE, light brown to gray, very hard

Boring Terminated at 40 Feet

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
4-inch diameter solid stem continuous flight power auger

Abandonment Method:
Boring backfilled with auger cuttings upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Notes:

Elevations were interpolated from the ALTA/NSPS Land Title Survey by Martin/Martin (Job No. 19.1522.C.86, revised March 29, 2021) and provided to us by QTS.

WATER LEVEL OBSERVATIONS

None encountered while drilling

Terracon
10625 W I70 Frontage Rd N Ste 3
Wheat Ridge, CO

Boring Started: 05-31-2021	Boring Completed: 06-02-2021
Drill Rig: CME-550	Driller: Terracon
Project No.: 25215031	

THIS BORING LOG IS NOT VALID IF SEPARATED FROM ORIGINAL REPORT. GEO SMART LOG-NO WELL 25215031 QTS AURORA_USE THIS ONE.GPJ TERRACON_DATATEMPLATE.GDT 8/17/21

BORING LOG NO. F5

PROJECT: QTS Aurora

CLIENT: Quality Technology Services, Inc.

SITE: Gun Club Road and Colfax Avenue
Aurora, Colorado

LOCATION See Exploration Plan

Latitude: 39.7331° Longitude: -104.7100°

Approximate Surface Elev.: 5546.5 (Ft.) +/-

MODEL LAYER	GRAPHIC LOG	DEPTH	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	SWELL (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
1			**LEAN CLAY (CL)**, brown, very stiff									30-18-12	85
		3.0	5543.5+/-				8-18						
			CLAYSTONE, brown, firm to medium hard	5			11-20			16.1	112		
							9-36						
		9.0	5537.5+/-										
			CLAYEY SANDSTONE, fine to coarse grained, white to tan, hard, moderate cementation	10			50/7"	+3.4 @ 500 psf		6.6	133		
		12.0	5534.5+/-				9/20						
			CLAYSTONE, gray to reddish brown, firm to hard	15			14-26			19.1	108		
				20			50/11"			21.8	105		
3				25			50/7"						
				30			15-35			19.3	107		
				35			50/11"						
				40			50/9"						
				45			50/8"						
		50.0	5496.5+/-	50			50/8"						
			Boring Terminated at 50 Feet										

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
4-inch diameter solid stem continuous flight power auger

Abandonment Method:
Boring backfilled with auger cuttings upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Notes:

Elevations were interpolated from the ALTA/NSPS Land Title Survey by Martin/Martin (Job No. 19.1522.C.86, revised March 29, 2021) and provided to us by QTS.

WATER LEVEL OBSERVATIONS

None encountered while drilling

Terracon
10625 W I70 Frontage Rd N Ste 3
Wheat Ridge, CO

Boring Started: 05-31-2021	Boring Completed: 06-02-2021
Drill Rig: CME-55	Driller: Terracon
Project No.: 25215031	

THIS BORING LOG IS NOT VALID IF SEPARATED FROM ORIGINAL REPORT. GEO SMART LOG-NO WELL 25215031 QTS AURORA_USE THIS ONE.GPJ TERRACON_DATATEMPLATE.GDT 8/17/21

BORING LOG NO. F6

PROJECT: QTS Aurora

CLIENT: Quality Technology Services, Inc.

SITE: Gun Club Road and Colfax Avenue
Aurora, Colorado

LOCATION See Exploration Plan

Latitude: 39.7322° Longitude: -104.7107°

Approximate Surface Elev.: 5550.5 (Ft.) +/-

MODEL LAYER	GRAPHIC LOG	DEPTH	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	SWELL (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
1		0.0	**LEAN CLAY (CL)**, trace sand, trace organics, calcareous, brown to orangish brown, stiff				6-9					44-17-27	86
		5.0	5545.5+/-	5			8-12	+2.9 @ 500 psf		24.6	100		
			CLAYSTONE, grayish brown, soft to medium hard				14-10						
				10			14-23			19.1	112		
							17-28						
		15.0	5535.5+/-	15			10-30			14.6	116		
		19.0	**SANDSTONE**, fine to medium grained, orangish brown, medium hard, weak cementation 5531.5+/-										
3			**CLAYSTONE**, grayish brown to gray, hard	20			50/12"			24.4	101		
				25			50/8"						
				30			50/10"			22.1	103		
				35			50/8"						
		40.0	5510.5+/-	40			50/9"						
			Boring Terminated at 40 Feet										

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
4-inch diameter solid stem continuous flight power auger

Abandonment Method:
Boring backfilled with auger cuttings upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Notes:

Elevations were interpolated from the ALTA/NSPS Land Title Survey by Martin/Martin (Job No. 19.1522.C.86, revised March 29, 2021) and provided to us by QTS.

WATER LEVEL OBSERVATIONS

None encountered while drilling

Terracon
10625 W I70 Frontage Rd N Ste 3
Wheat Ridge, CO

Boring Started: 05-31-2021 | Boring Completed: 06-02-2021

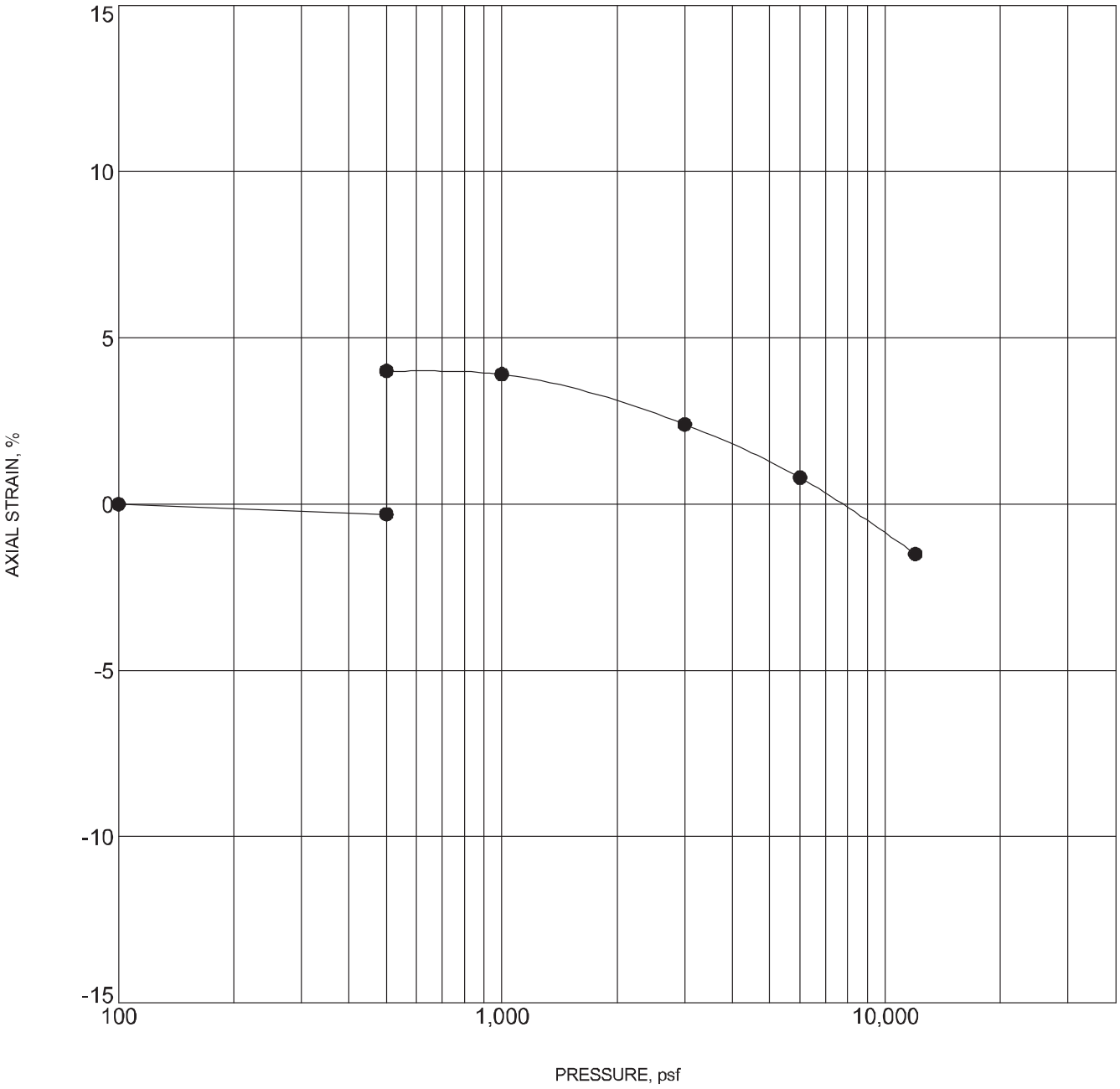
Drill Rig: CME-55 | Driller: Terracon

Project No.: 25215031

THIS BORING LOG IS NOT VALID IF SEPARATED FROM ORIGINAL REPORT. GEO SMART LOG-NO WELL 25215031 QTS AURORA_USE THIS ONE.GPJ TERRACON_DATATEMPLATE.GDT 8/17/21

Permit #25-2535171-LT
RSN 1903242

SWELL CONSOLIDATION TEST



Specimen Identification			Classification	γ_d, pcf	WC, %
●	22	7.5 - 8.5 ft	SANDY LEAN CLAY (CL)	102	23.7

NOTES: Water was added at 500 psf.

PROJECT: QTS Aurora

SITE: Gun Club Road and Colfax Avenue
Aurora, Colorado

Terracon
10625 W I70 Frontage Rd N Ste 3
Wheat Ridge, CO

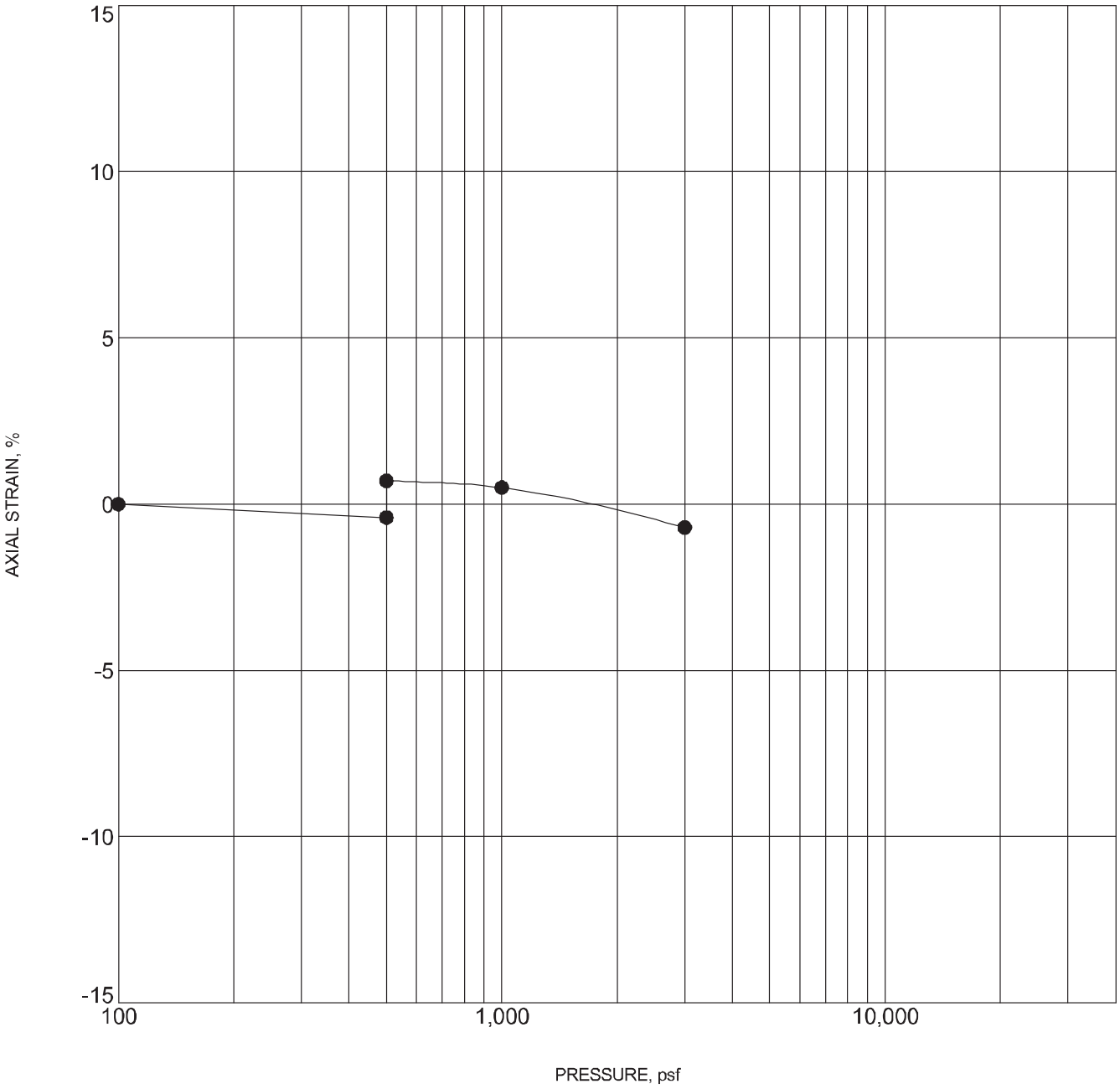
PROJECT NUMBER: 25215031

CLIENT: Quality Technology Services, Inc.

LABORATORY TESTS ARE NOT VALID IF SEPARATED FROM ORIGINAL REPORT. TC_CONSOL_STRAIN-USCS-NO ASTM 25215031 QTS AURORA_USE THIS ONE.GPJ TERRACON_DATATEMPLATE.GDT 8/17/21

Permit #25-2535171-LT
RSN 1903242

SWELL CONSOLIDATION TEST



Specimen Identification			Classification	γ_d, pcf	WC, %
●	23	5 - 6 ft	LEAN CLAY (CL)	114	15.7

NOTES: Water was added at 500 psf.

PROJECT: QTS Aurora

SITE: Gun Club Road and Colfax Avenue
Aurora, Colorado

Terracon
10625 W I70 Frontage Rd N Ste 3
Wheat Ridge, CO

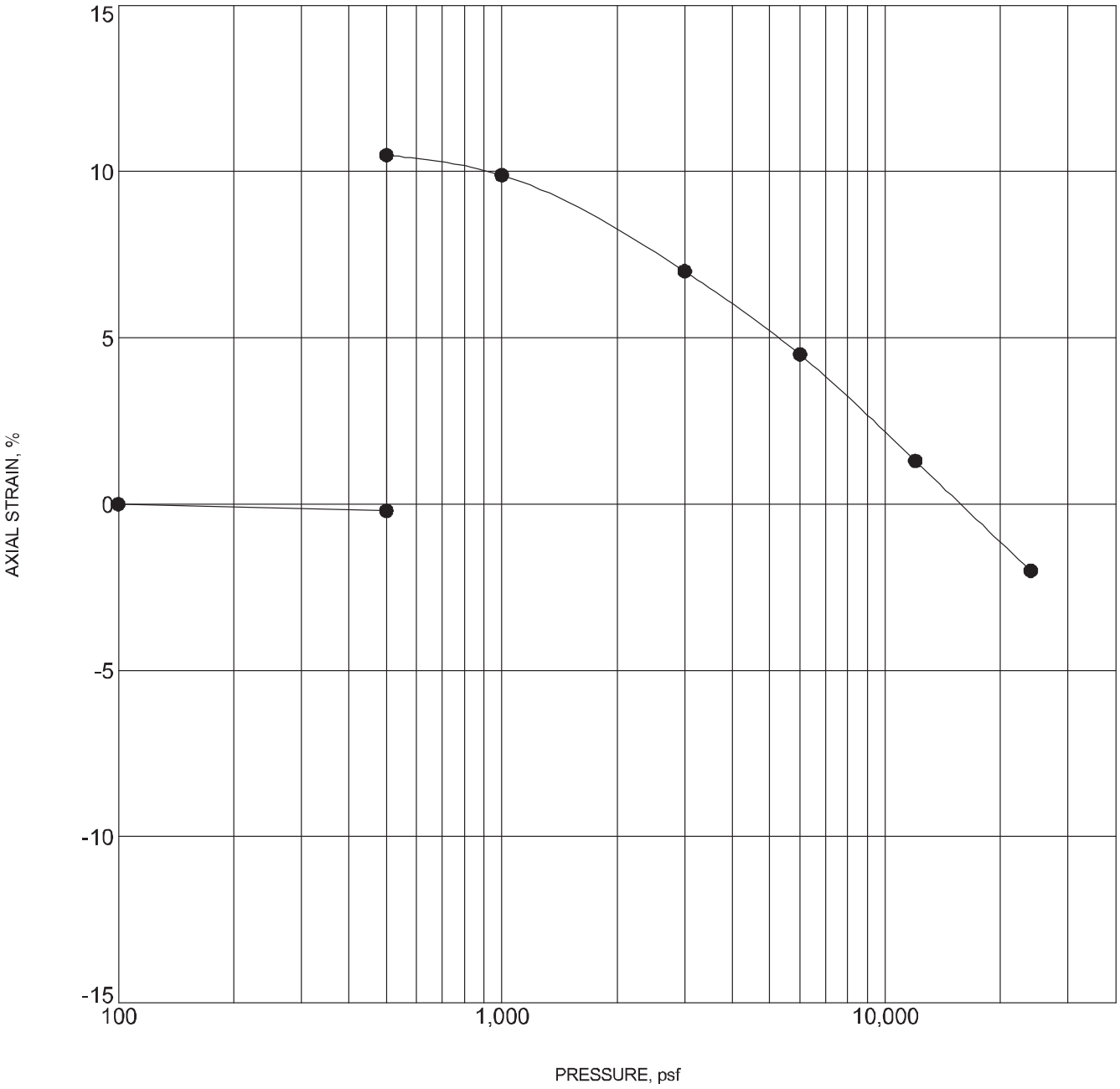
PROJECT NUMBER: 25215031

CLIENT: Quality Technology Services, Inc.

LABORATORY TESTS ARE NOT VALID IF SEPARATED FROM ORIGINAL REPORT. TC_CONSOL_STRAIN-USCS-NO ASTM 25215031 QTS AURORA_USE THIS ONE.GPJ TERRACON_DATATEMPLATE.GDT 8/17/21

Permit #25-2535171-LT
RSN 1903242

SWELL CONSOLIDATION TEST



Specimen Identification			Classification	γ_d, pcf	WC, %
●	26	7.5 - 8.5 ft	CLAYSTONE	108	19.9

NOTES: Water was added at 500 psf.

LABORATORY TESTS ARE NOT VALID IF SEPARATED FROM ORIGINAL REPORT. TC_CONSOL_STRAIN-USCS-NO ASTM 25215031 QTS AURORA_USE THIS ONE.GPJ TERRACON_DATATEMPLATE.GDT 8/17/21

PROJECT: QTS Aurora

PROJECT NUMBER: 25215031

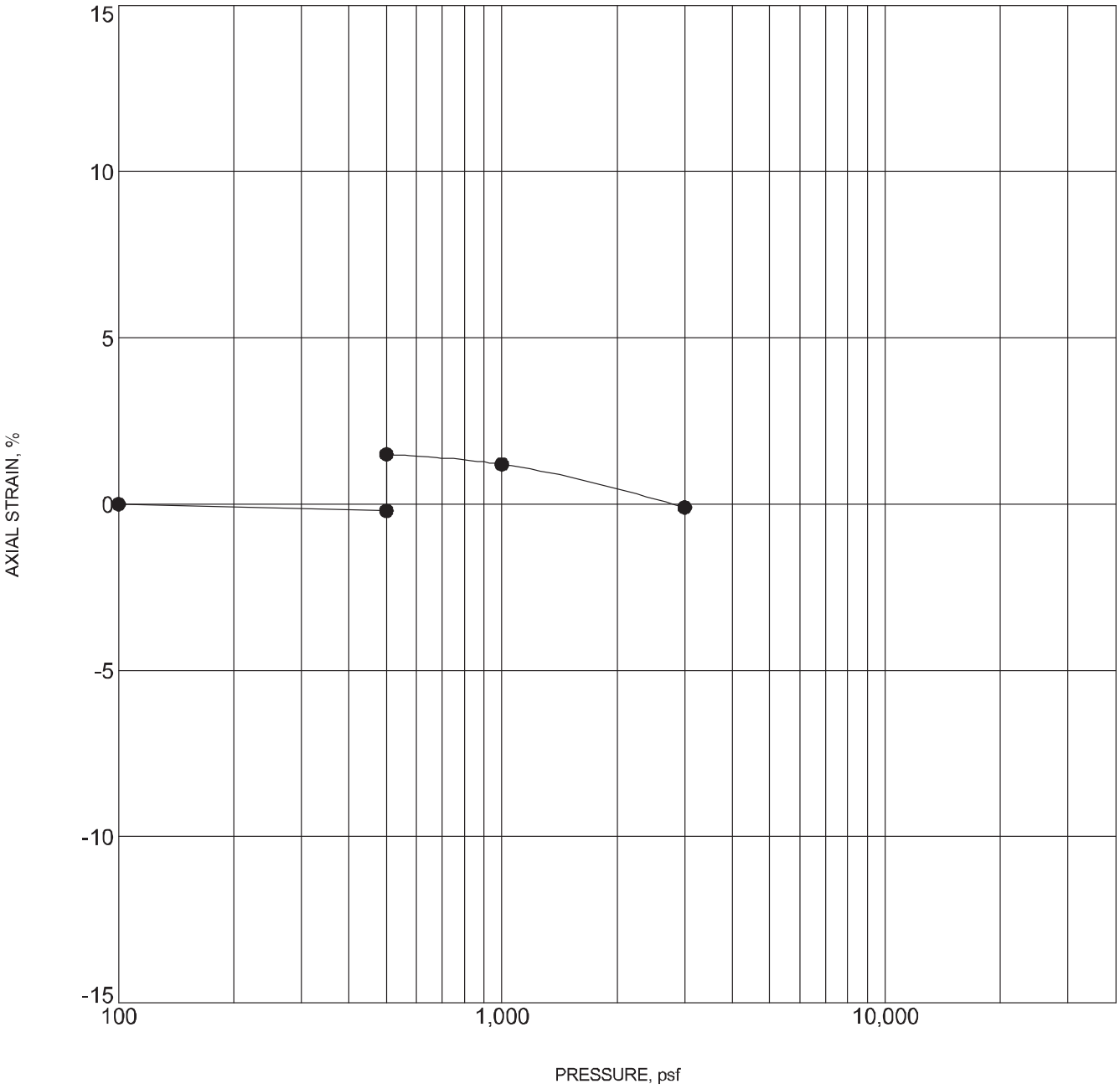
SITE: Gun Club Road and Colfax Avenue
Aurora, Colorado

Terracon
10625 W I70 Frontage Rd N Ste 3
Wheat Ridge, CO

CLIENT: Quality Technology Services, Inc.

Permit #25-2535171-LT
RSN 1903242

SWELL CONSOLIDATION TEST



Specimen Identification			Classification	γ_d, pcf	WC, %
●	27	7.5 - 8.5 ft	LEAN CLAY (CL)	109	18.7

NOTES: Water was added at 500 psf.

LABORATORY TESTS ARE NOT VALID IF SEPARATED FROM ORIGINAL REPORT. TC_CONSOL_STRAIN-USCS-NO ASTM 25215031 QTS AURORA_USE THIS ONE.GPJ TERRACON_DATATEMPLATE.GDT 8/17/21

PROJECT: QTS Aurora

SITE: Gun Club Road and Colfax Avenue
Aurora, Colorado

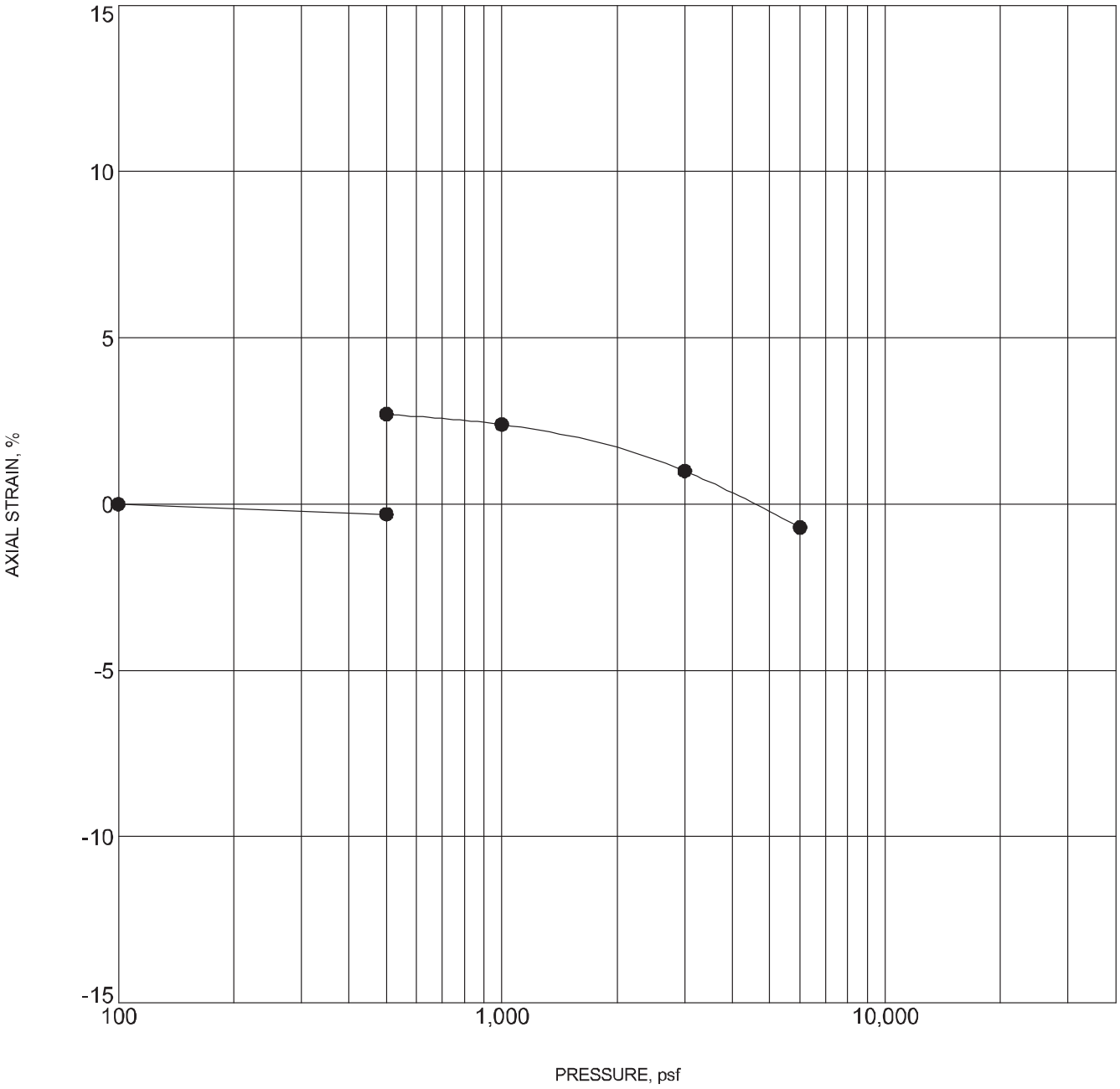
Terracon
10625 W I70 Frontage Rd N Ste 3
Wheat Ridge, CO

PROJECT NUMBER: 25215031

CLIENT: Quality Technology Services, Inc.

Permit #25-2535171-LT
RSN 1903242

SWELL CONSOLIDATION TEST



Specimen Identification			Classification	γ_d, pcf	WC, %
●	27	12.5 - 13.5 ft	CLAYSTONE	106	21.4

NOTES: Water was added at 500 psf.

LABORATORY TESTS ARE NOT VALID IF SEPARATED FROM ORIGINAL REPORT. TC_CONSOL_STRAIN-USCS-NO ASTM 25215031 QTS AURORA_USE THIS ONE.GPJ TERRACON_DATATEMPLATE.GDT 8/17/21

PROJECT: QTS Aurora

PROJECT NUMBER: 25215031

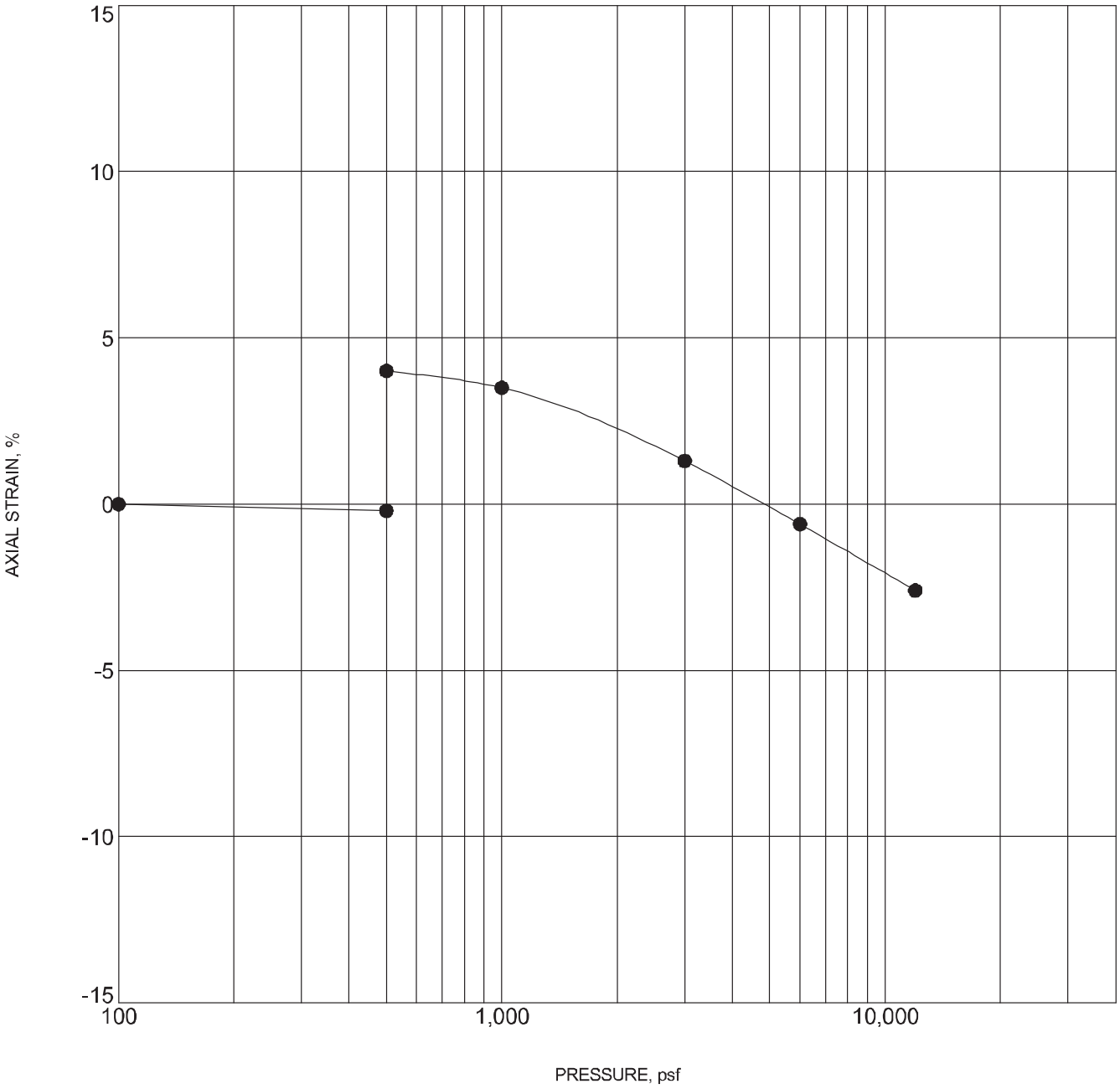
SITE: Gun Club Road and Colfax Avenue
Aurora, Colorado

Terracon
10625 W I70 Frontage Rd N Ste 3
Wheat Ridge, CO

CLIENT: Quality Technology Services, Inc.

Permit #25-2535171-LT
RSN 1903242

SWELL CONSOLIDATION TEST



Specimen Identification			Classification	γ_d, pcf	WC, %
●	28	5 - 6 ft	LEAN CLAY with SAND (CL)	106	18.2

NOTES: Water was added at 500 psf.

LABORATORY TESTS ARE NOT VALID IF SEPARATED FROM ORIGINAL REPORT. TC_CONSOL_STRAIN-USCS-NO ASTM 25215031 QTS AURORA_USE THIS ONE.GPJ TERRACON_DATATEMPLATE.GDT 8/17/21

PROJECT: QTS Aurora

SITE: Gun Club Road and Colfax Avenue
Aurora, Colorado

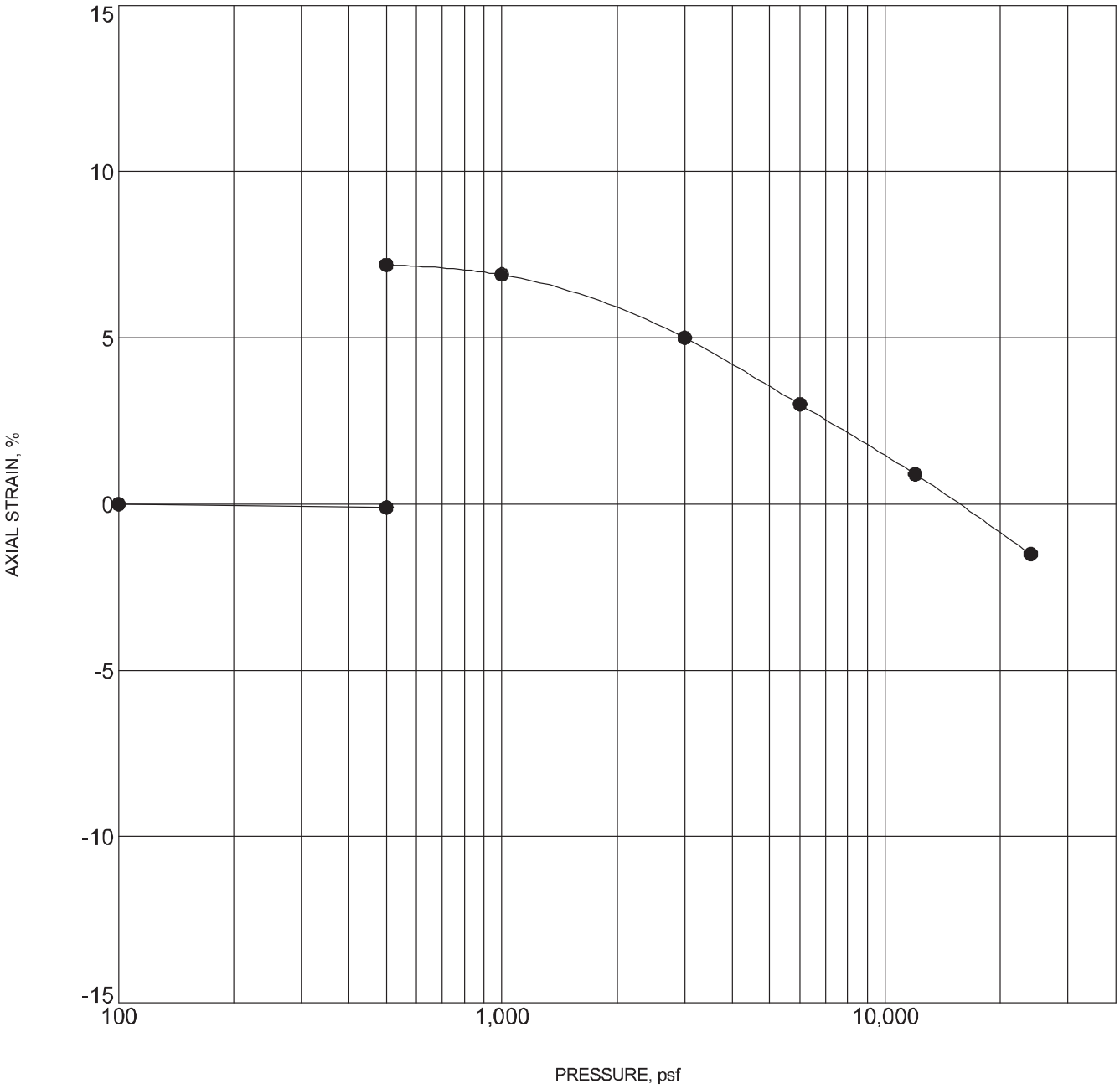
Terracon
10625 W I70 Frontage Rd N Ste 3
Wheat Ridge, CO

PROJECT NUMBER: 25215031

CLIENT: Quality Technology Services, Inc.

Permit #25-2535171-LT
RSN 1903242

SWELL CONSOLIDATION TEST



Specimen Identification			Classification	γ_d, pcf	WC, %
●	29	5 - 6 ft	CLAYSTONE	115	16.4

NOTES: Water was added at 500 psf.

PROJECT: QTS Aurora

SITE: Gun Club Road and Colfax Avenue
Aurora, Colorado

Terracon
10625 W I70 Frontage Rd N Ste 3
Wheat Ridge, CO

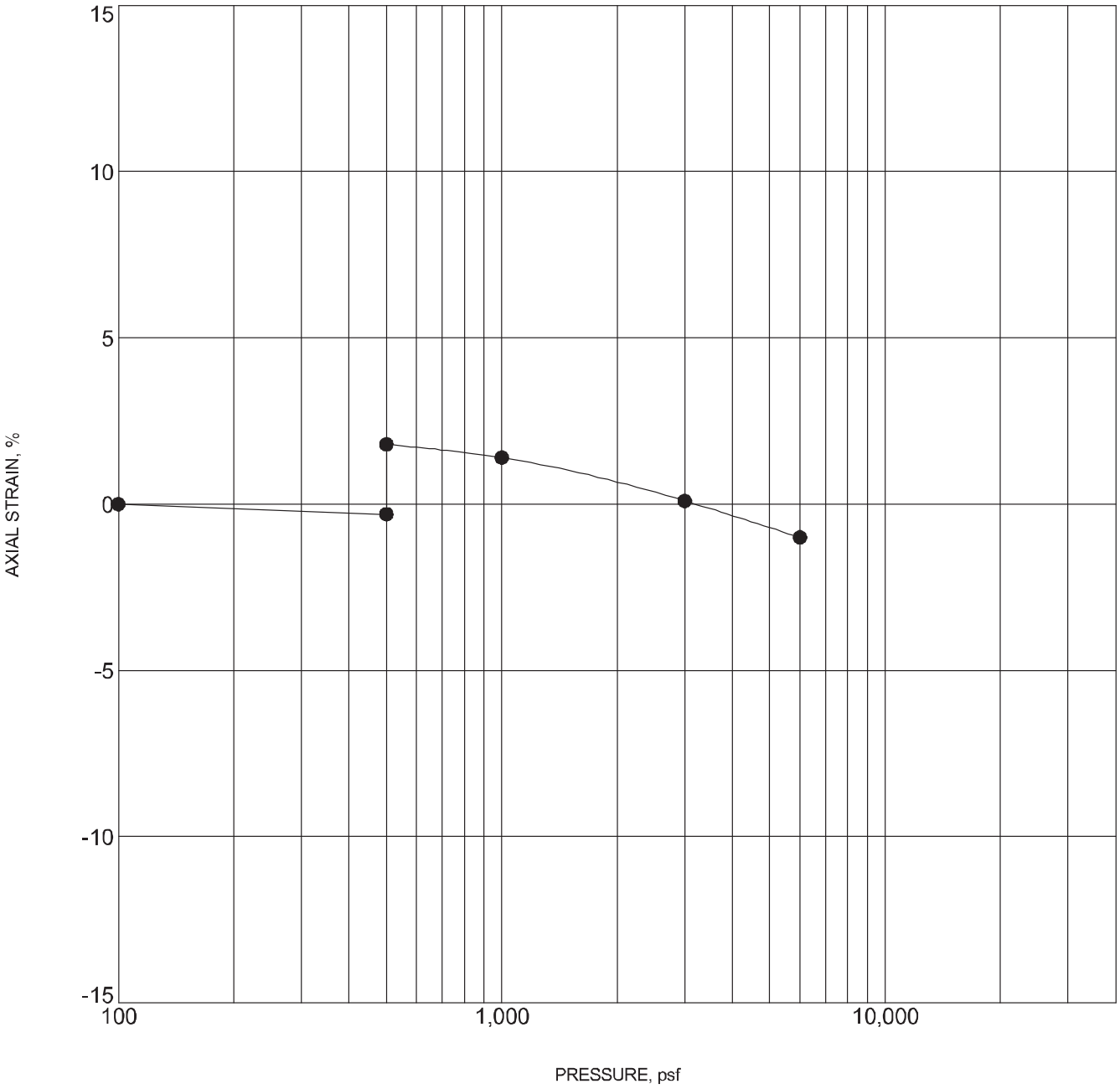
PROJECT NUMBER: 25215031

CLIENT: Quality Technology Services, Inc.

LABORATORY TESTS ARE NOT VALID IF SEPARATED FROM ORIGINAL REPORT. TC_CONSOL_STRAIN-USCS-NO ASTM 25215031 QTS AURORA_USE THIS ONE.GPJ TERRACON_DATATEMPLATE.GDT 8/17/21

Permit #25-2535171-LT
RSN 1903242

SWELL CONSOLIDATION TEST



Specimen Identification			Classification	γ_d, pcf	WC, %
●	30	5 - 6 ft	LEAN CLAY (CL)	114	17.5

NOTES: Water was added at 500 psf.

LABORATORY TESTS ARE NOT VALID IF SEPARATED FROM ORIGINAL REPORT. TC_CONSOL_STRAIN-USCS-NO ASTM 25215031 QTS AURORA_USE THIS ONE.GPJ TERRACON_DATATEMPLATE.GDT 8/17/21

PROJECT: QTS Aurora

SITE: Gun Club Road and Colfax Avenue
Aurora, Colorado

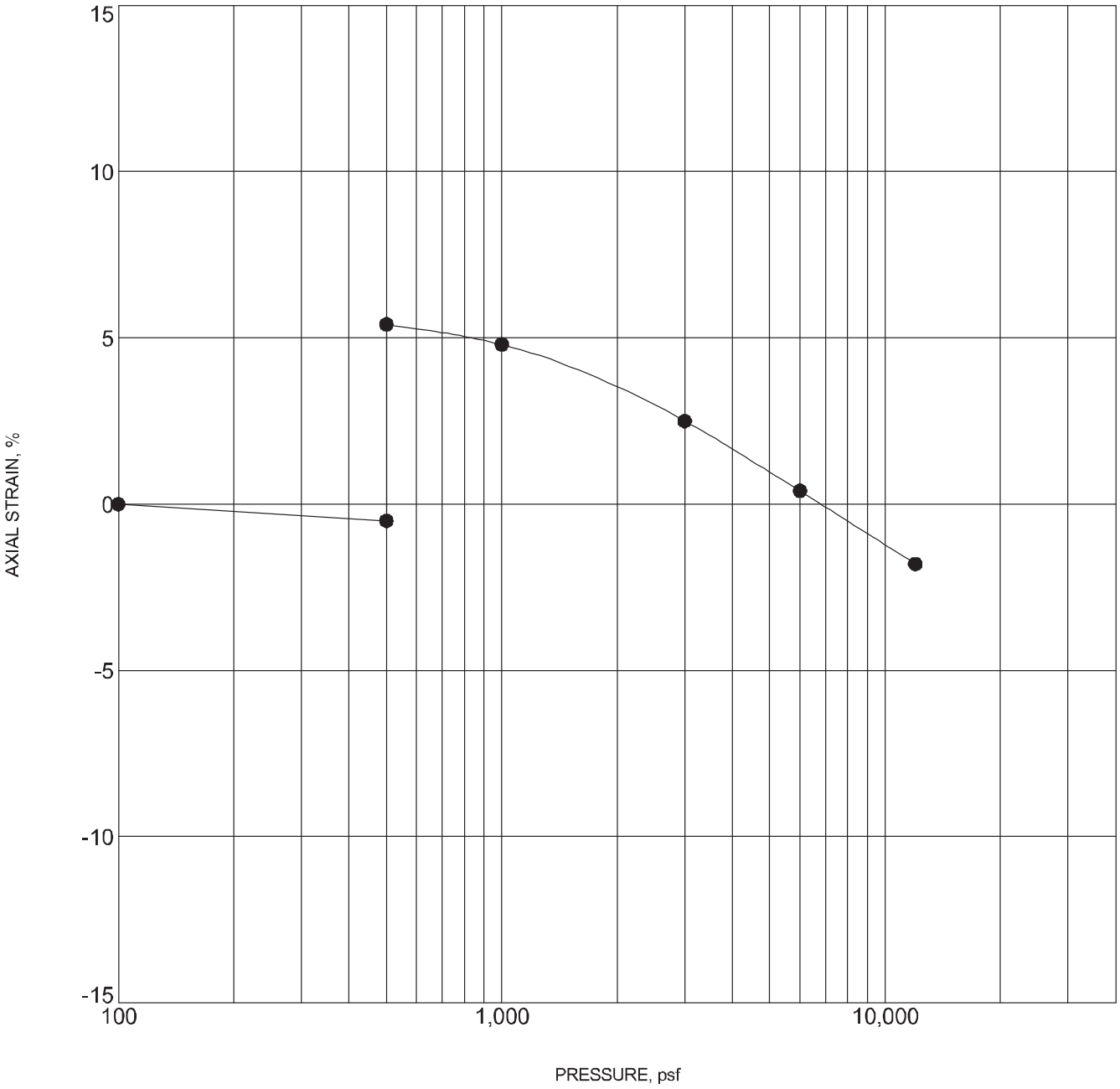
Terracon
10625 W I70 Frontage Rd N Ste 3
Wheat Ridge, CO

PROJECT NUMBER: 25215031

CLIENT: Quality Technology Services, Inc.

Permit #25-2535171-LT
RSN 1903242

SWELL CONSOLIDATION TEST



Specimen Identification			Classification	γ_d, pcf	WC, %
●	31	7.5 - 8.5 ft	CLAYSTONE	106	20.6

NOTES: Water was added at 500 psf.

LABORATORY TESTS ARE NOT VALID IF SEPARATED FROM ORIGINAL REPORT. TC_CONSOL_STRAIN-USCS-NO ASTM 25215031 QTS AURORA_USE THIS ONE.GPJ TERRACON_DATATEMPLATE.GDT 8/17/21

PROJECT: QTS Aurora

PROJECT NUMBER: 25215031

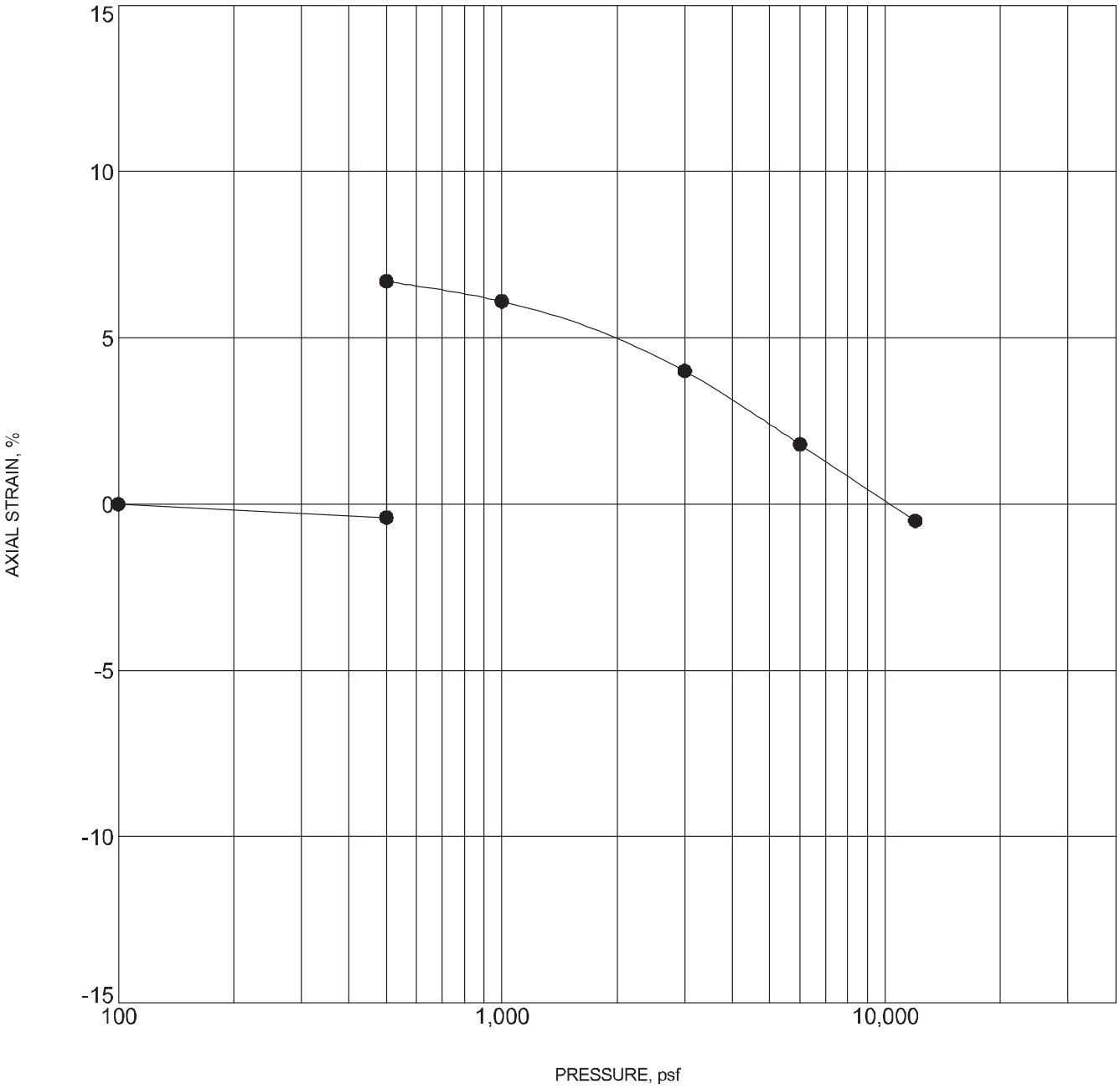
SITE: Gun Club Road and Colfax Avenue
Aurora, Colorado

Terracon
10625 W I70 Frontage Rd N Ste 3
Wheat Ridge, CO

CLIENT: Quality Technology Services, Inc.

Permit #25-2535171-LT
RSN 1903242

SWELL CONSOLIDATION TEST



Specimen Identification			Classification	γ_d, pcf	WC, %
●	32	5 - 6 ft	CLAYSTONE	108	18.5

NOTES: Water was added at 500 psf.

LABORATORY TESTS ARE NOT VALID IF SEPARATED FROM ORIGINAL REPORT. TC_CONSOL_STRAIN-USCS-NO ASTM 25215031 QTS AURORA_USE THIS ONE.GPJ TERRACON_DATATEMPLATE.GDT 8/17/21

PROJECT: QTS Aurora

SITE: Gun Club Road and Colfax Avenue
Aurora, Colorado

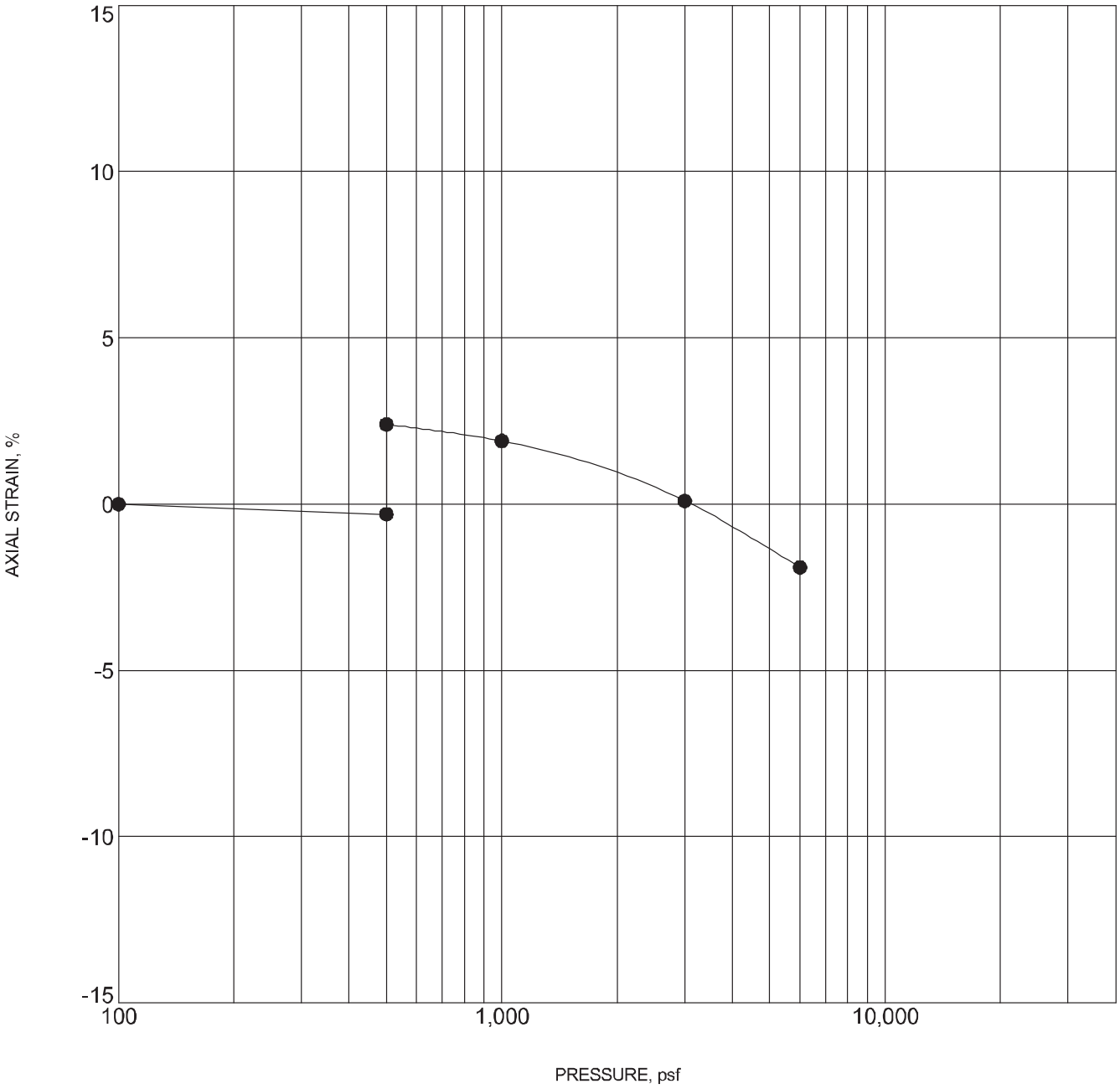
Terracon
10625 W I70 Frontage Rd N Ste 3
Wheat Ridge, CO

PROJECT NUMBER: 25215031

CLIENT: Quality Technology Services, Inc.

Permit #25-2535171-LT
RSN 1903242

SWELL CONSOLIDATION TEST



Specimen Identification			Classification	γ_d, pcf	WC, %
●	33	7.5 - 8.5 ft	LEAN CLAY with SAND (CL)	106	19.8

NOTES: Water was added at 500 psf.

LABORATORY TESTS ARE NOT VALID IF SEPARATED FROM ORIGINAL REPORT. TC_CONSOL_STRAIN-USCS-NO ASTM 25215031 QTS AURORA_USE THIS ONE.GPJ TERRACON_DATATEMPLATE.GDT 8/17/21

PROJECT: QTS Aurora

SITE: Gun Club Road and Colfax Avenue
Aurora, Colorado

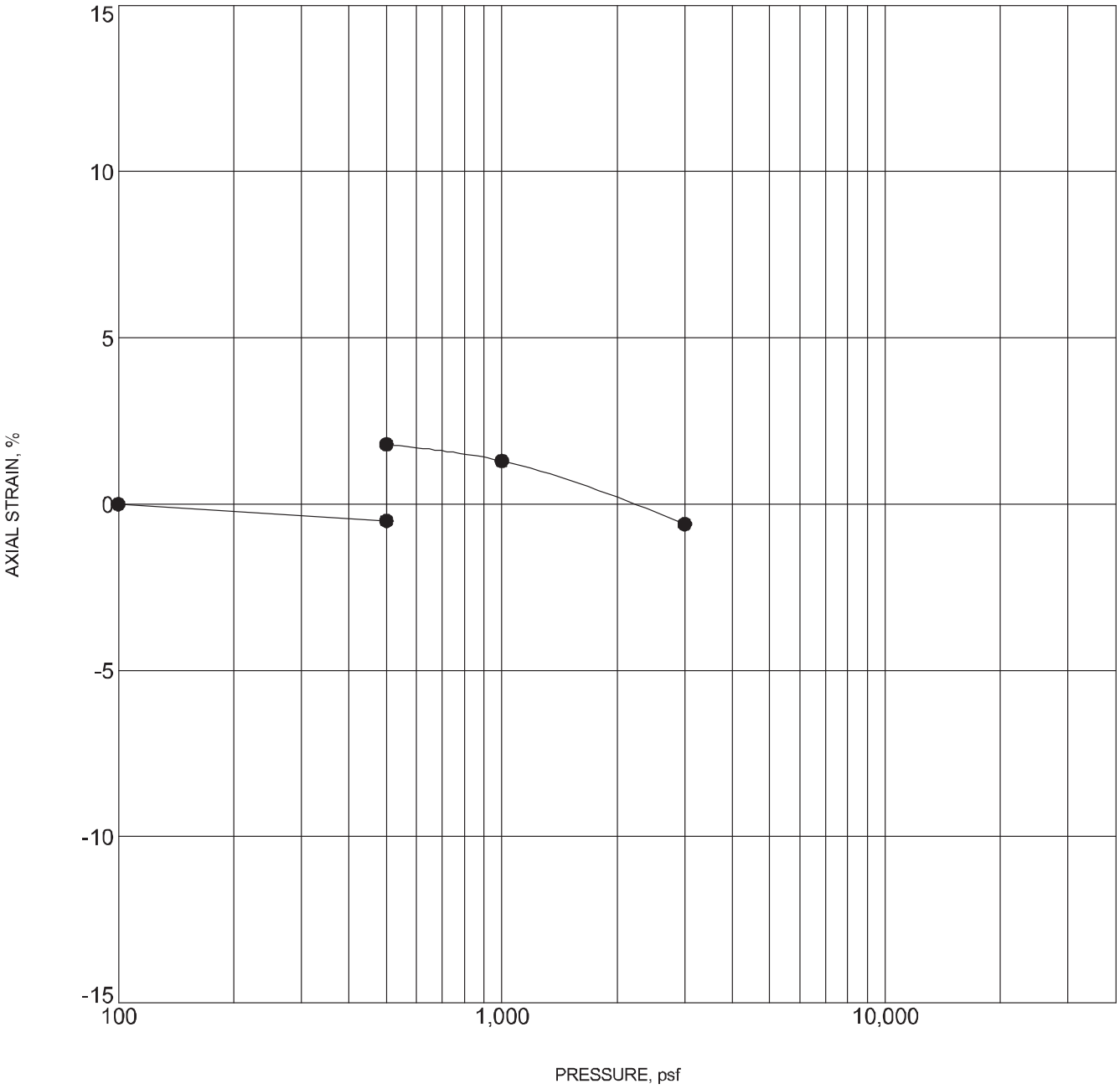
Terracon
10625 W I70 Frontage Rd N Ste 3
Wheat Ridge, CO

PROJECT NUMBER: 25215031

CLIENT: Quality Technology Services, Inc.

Permit #25-2535171-LT
RSN 1903242

SWELL CONSOLIDATION TEST



Specimen Identification			Classification	γ_d, pcf	WC, %
●	34	5 - 6 ft	LEAN CLAY with SAND (CL)	108	19.7

NOTES: Water was added at 500 psf.

PROJECT: QTS Aurora

SITE: Gun Club Road and Colfax Avenue
Aurora, Colorado

Terracon
10625 W I70 Frontage Rd N Ste 3
Wheat Ridge, CO

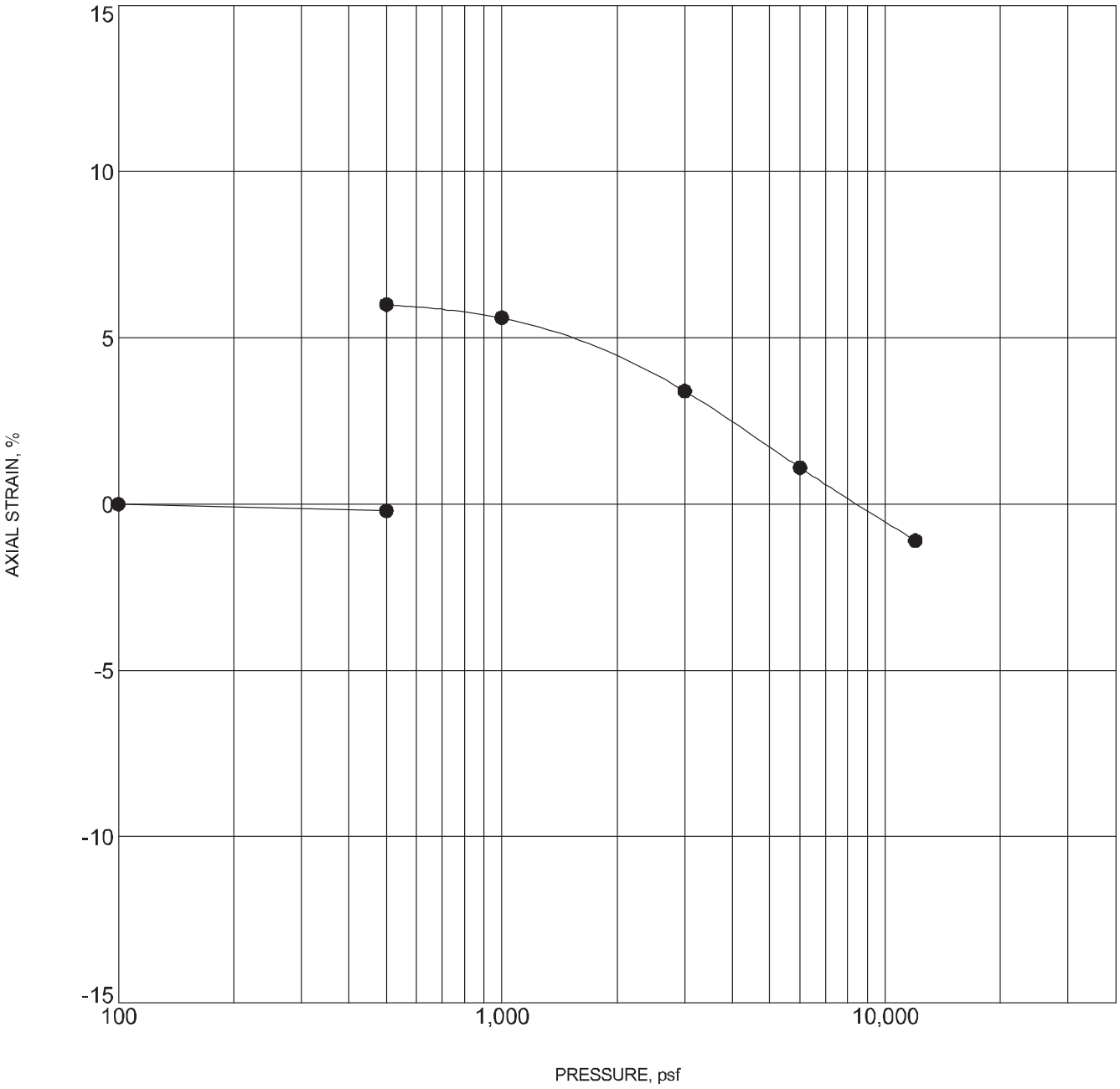
PROJECT NUMBER: 25215031

CLIENT: Quality Technology Services, Inc.

LABORATORY TESTS ARE NOT VALID IF SEPARATED FROM ORIGINAL REPORT. TC_CONSOL_STRAIN-USCS-NO ASTM 25215031 QTS AURORA_USE THIS ONE.GPJ TERRACON_DATATEMPLATE.GDT 8/17/21

Permit #25-2535171-LT
RSN 1903242

SWELL CONSOLIDATION TEST



Specimen Identification			Classification	γ_d, pcf	WC, %
●	35	7.5 - 8.5 ft	CLAYSTONE	112	18.6

NOTES: Water was added at 500 psf.

LABORATORY TESTS ARE NOT VALID IF SEPARATED FROM ORIGINAL REPORT. TC_CONSOL_STRAIN-USCS-NO ASTM 25215031 QTS AURORA_USE THIS ONE.GPJ TERRACON_DATATEMPLATE.GDT 8/17/21

PROJECT: QTS Aurora

SITE: Gun Club Road and Colfax Avenue
Aurora, Colorado

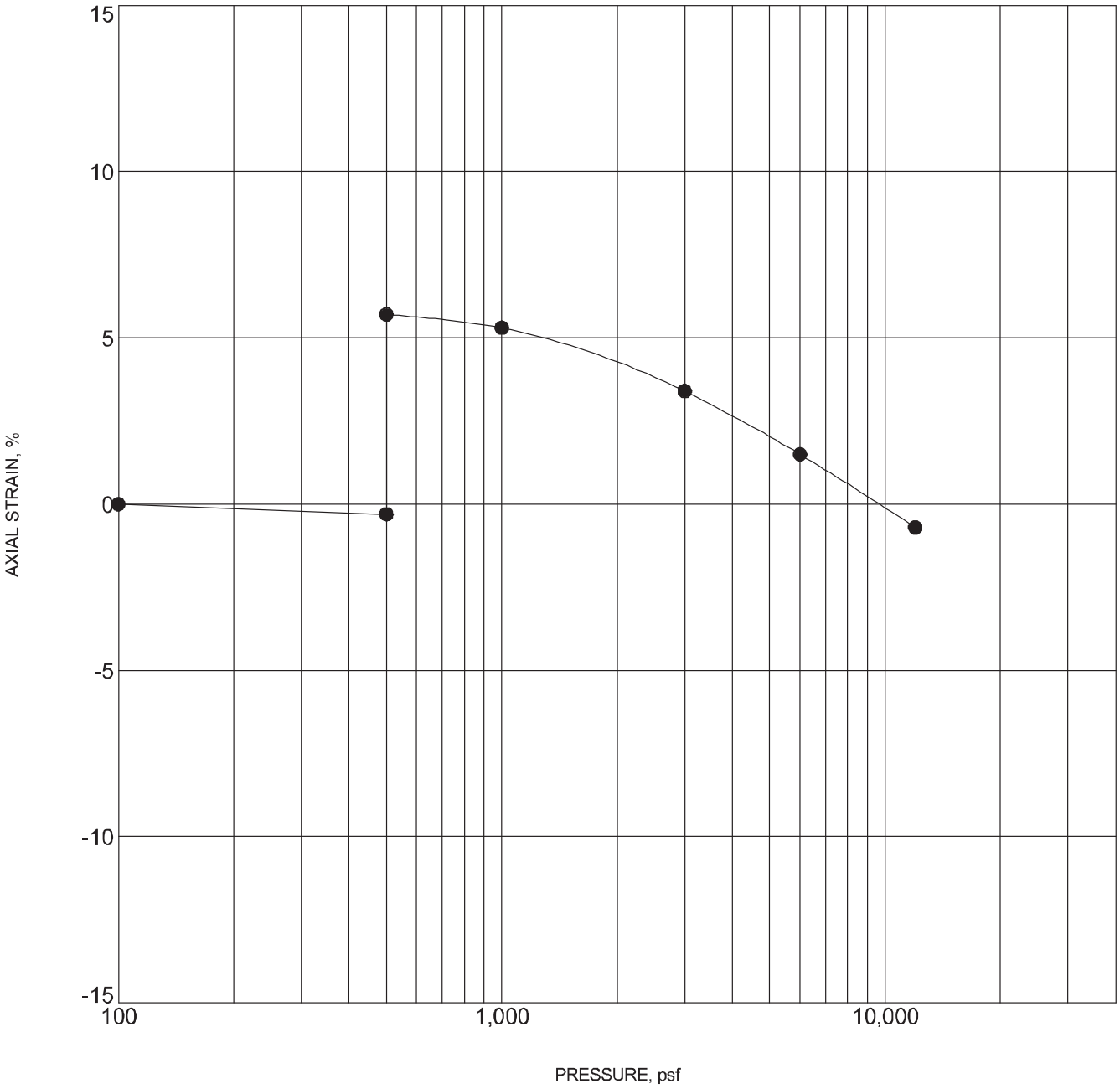
Terracon
10625 W I70 Frontage Rd N Ste 3
Wheat Ridge, CO

PROJECT NUMBER: 25215031

CLIENT: Quality Technology Services, Inc.

Permit #25-2535171-LT
RSN 1903242

SWELL CONSOLIDATION TEST



Specimen Identification			Classification	γ_d, pcf	WC, %
●	36	5 - 6 ft	CLAYSTONE	117	15.2

NOTES: Water was added at 500 psf.

LABORATORY TESTS ARE NOT VALID IF SEPARATED FROM ORIGINAL REPORT. TC_CONSOL_STRAIN-USCS-NO ASTM 25215031 QTS AURORA_USE THIS ONE.GPJ TERRACON_DATATEMPLATE.GDT 8/17/21

PROJECT: QTS Aurora

SITE: Gun Club Road and Colfax Avenue
Aurora, Colorado

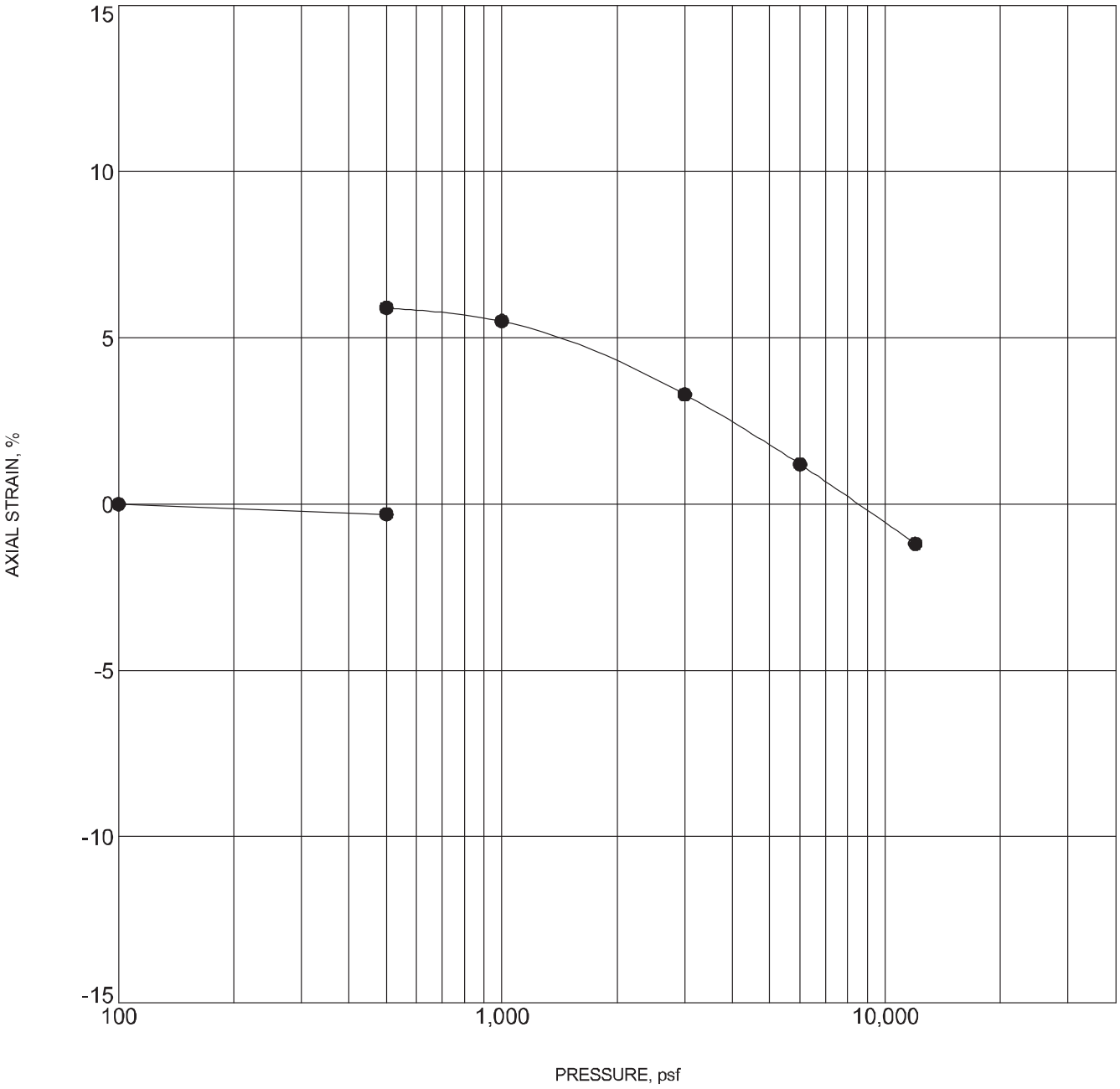
Terracon
10625 W I70 Frontage Rd N Ste 3
Wheat Ridge, CO

PROJECT NUMBER: 25215031

CLIENT: Quality Technology Services, Inc.

Permit #25-2535171-LT
RSN 1903242

SWELL CONSOLIDATION TEST



Specimen Identification			Classification	γ_d, pcf	WC, %
●	36	7.5 - 8.5 ft	CLAYSTONE	110	18.7

NOTES: Water was added at 500 psf.

PROJECT: QTS Aurora

SITE: Gun Club Road and Colfax Avenue
Aurora, Colorado

Terracon
10625 W I70 Frontage Rd N Ste 3
Wheat Ridge, CO

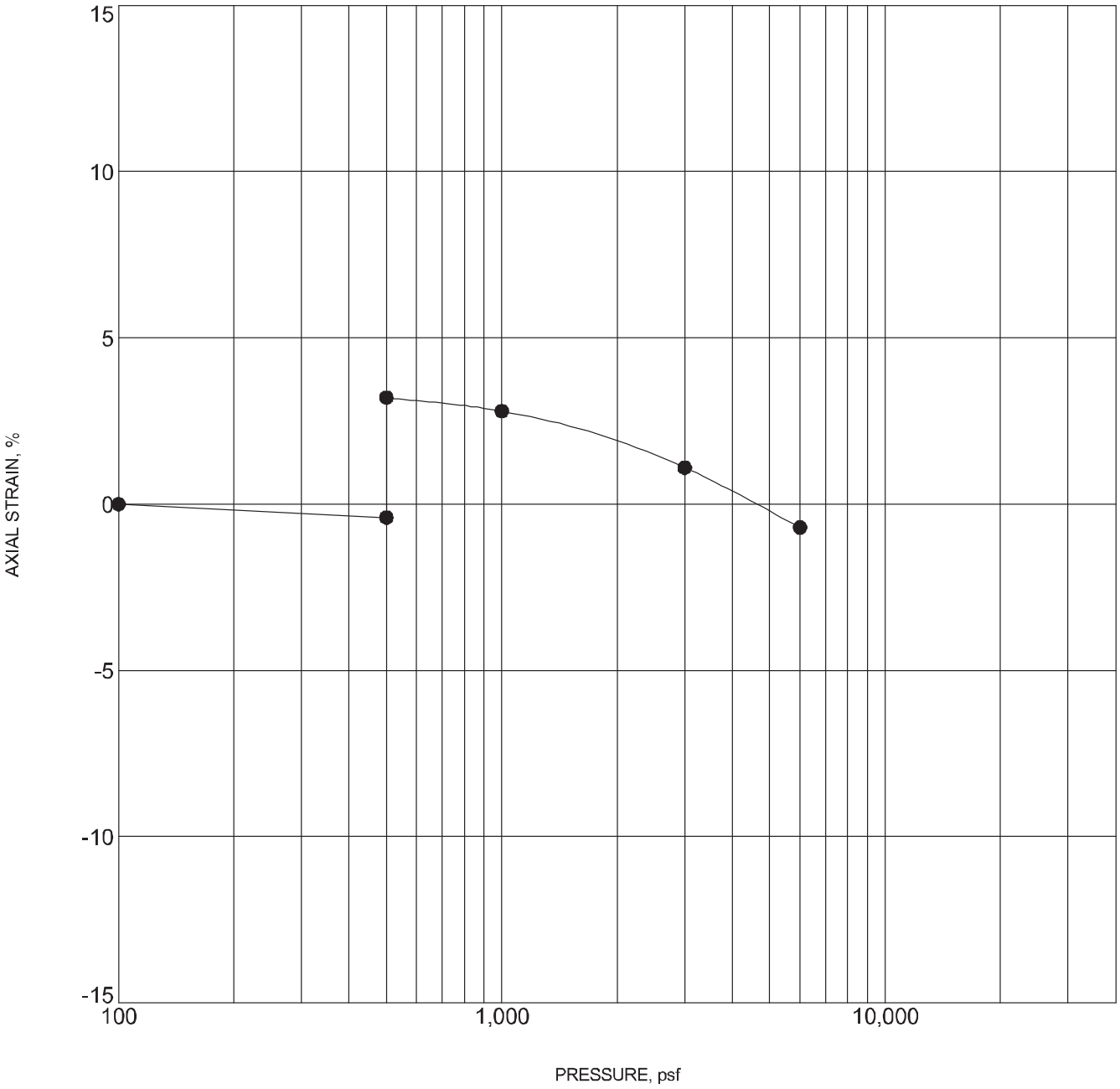
PROJECT NUMBER: 25215031

CLIENT: Quality Technology Services, Inc.

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Permit #25-2535171-LT
RSN 1903242

SWELL CONSOLIDATION TEST



Specimen Identification			Classification	γ_d, pcf	WC, %
●	37	10 - 11 ft	CLAYSTONE	110	19.0

NOTES: Water was added at 500 psf.

PROJECT: QTS Aurora

SITE: Gun Club Road and Colfax Avenue
Aurora, Colorado

Terracon
10625 W I70 Frontage Rd N Ste 3
Wheat Ridge, CO

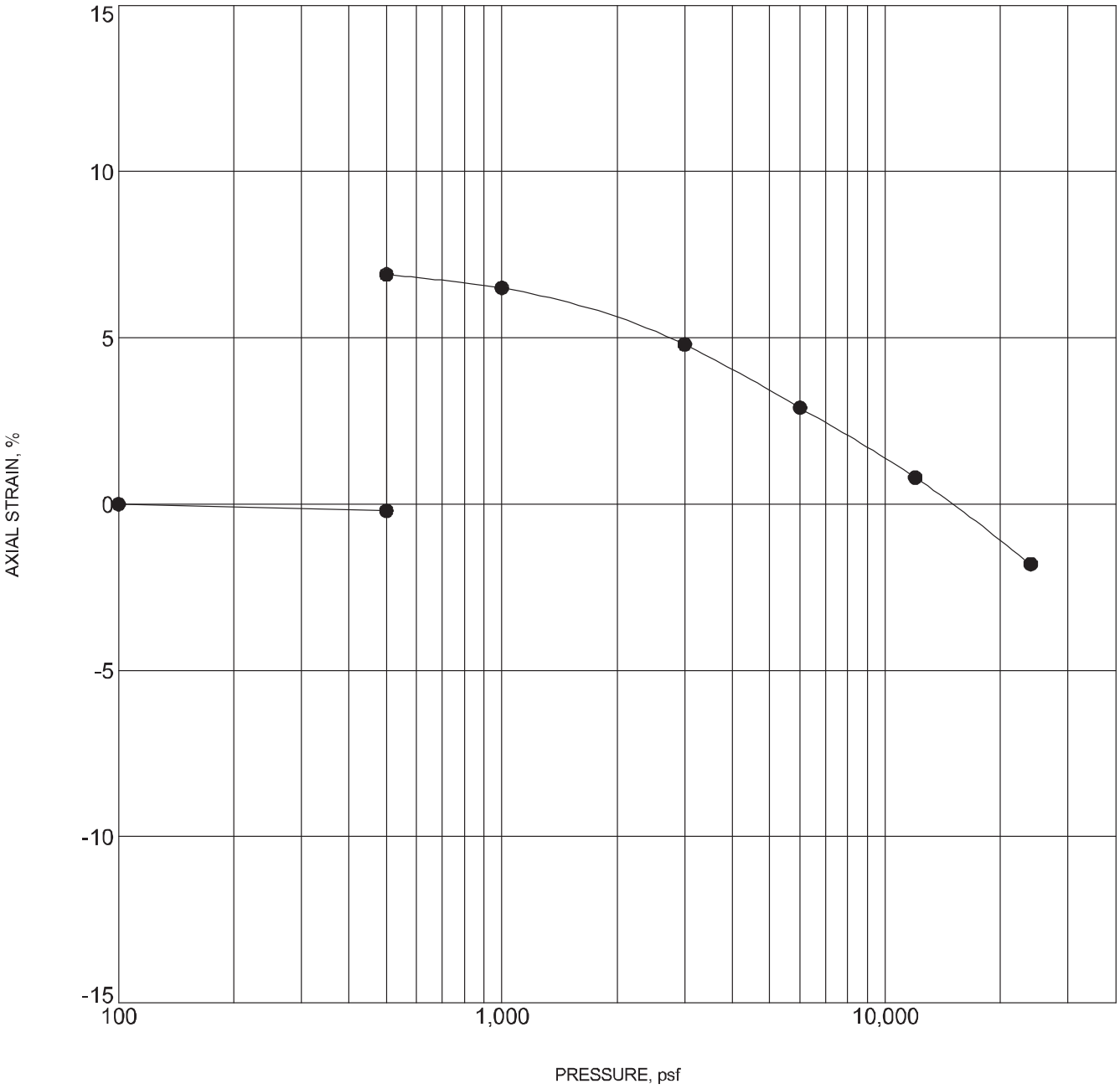
PROJECT NUMBER: 25215031

CLIENT: Quality Technology Services, Inc.

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Permit #25-2535171-LT
RSN 1903242

SWELL CONSOLIDATION TEST



Specimen Identification			Classification	γ_d, pcf	WC, %
●	39	5 - 6 ft	CLAYSTONE	117	14.5

NOTES: Water was added at 500 psf.

LABORATORY TESTS ARE NOT VALID IF SEPARATED FROM ORIGINAL REPORT. TC_CONSOL_STRAIN-USCS-NO ASTM 25215031 QTS AURORA_USE THIS ONE.GPJ TERRACON_DATATEMPLATE.GDT 8/17/21

PROJECT: QTS Aurora

SITE: Gun Club Road and Colfax Avenue
Aurora, Colorado

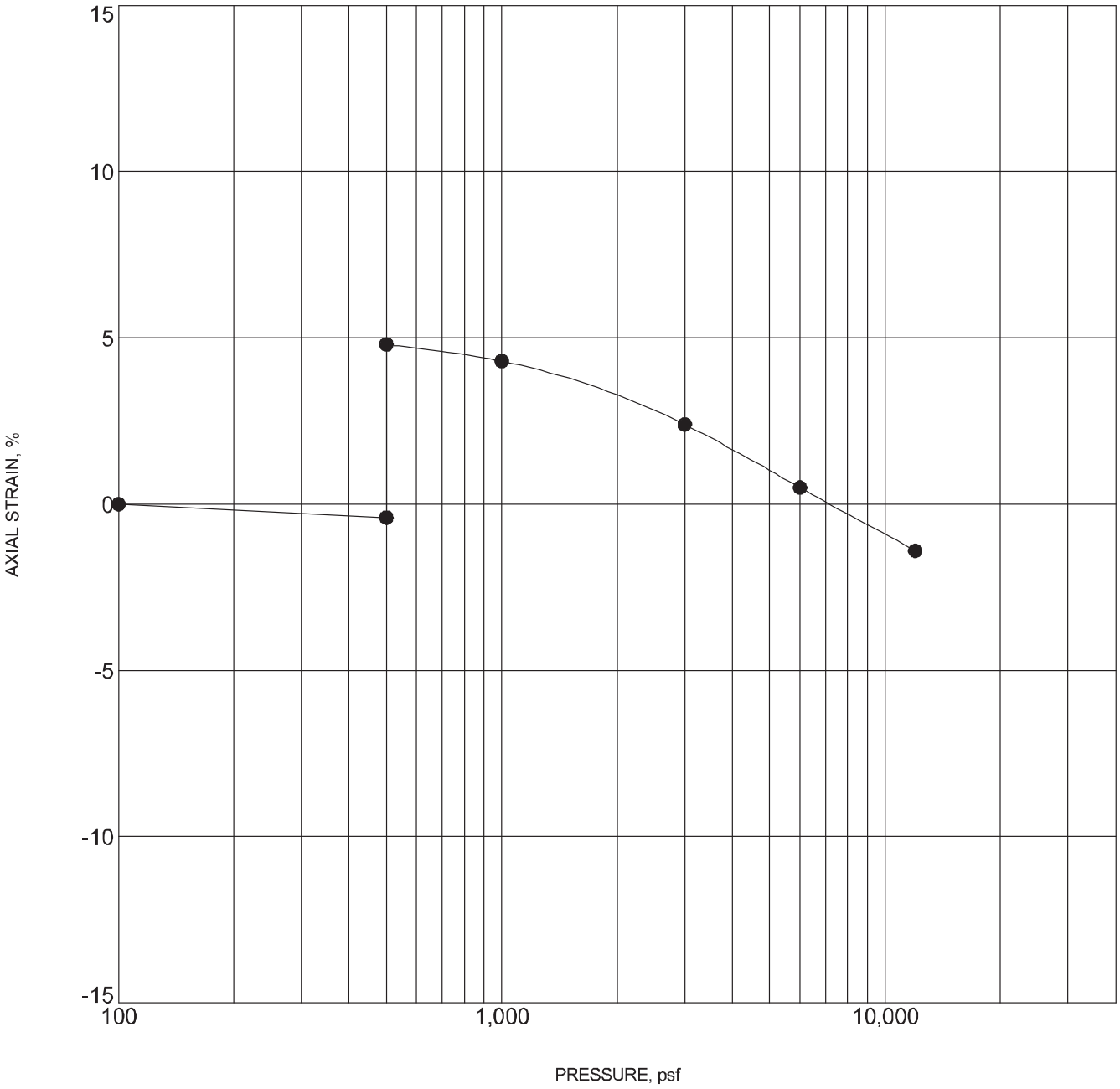
Terracon
10625 W I70 Frontage Rd N Ste 3
Wheat Ridge, CO

PROJECT NUMBER: 25215031

CLIENT: Quality Technology Services, Inc.

Permit #25-2535171-LT
RSN 1903242

SWELL CONSOLIDATION TEST



Specimen Identification			Classification	γ_d, pcf	WC, %
●	40	10 - 11 ft	CLAYSTONE	119	15.1

NOTES: Water added at 500 psf.

PROJECT: QTS Aurora

SITE: Gun Club Road and Colfax Avenue
Aurora, Colorado

Terracon
10625 W I70 Frontage Rd N Ste 3
Wheat Ridge, CO

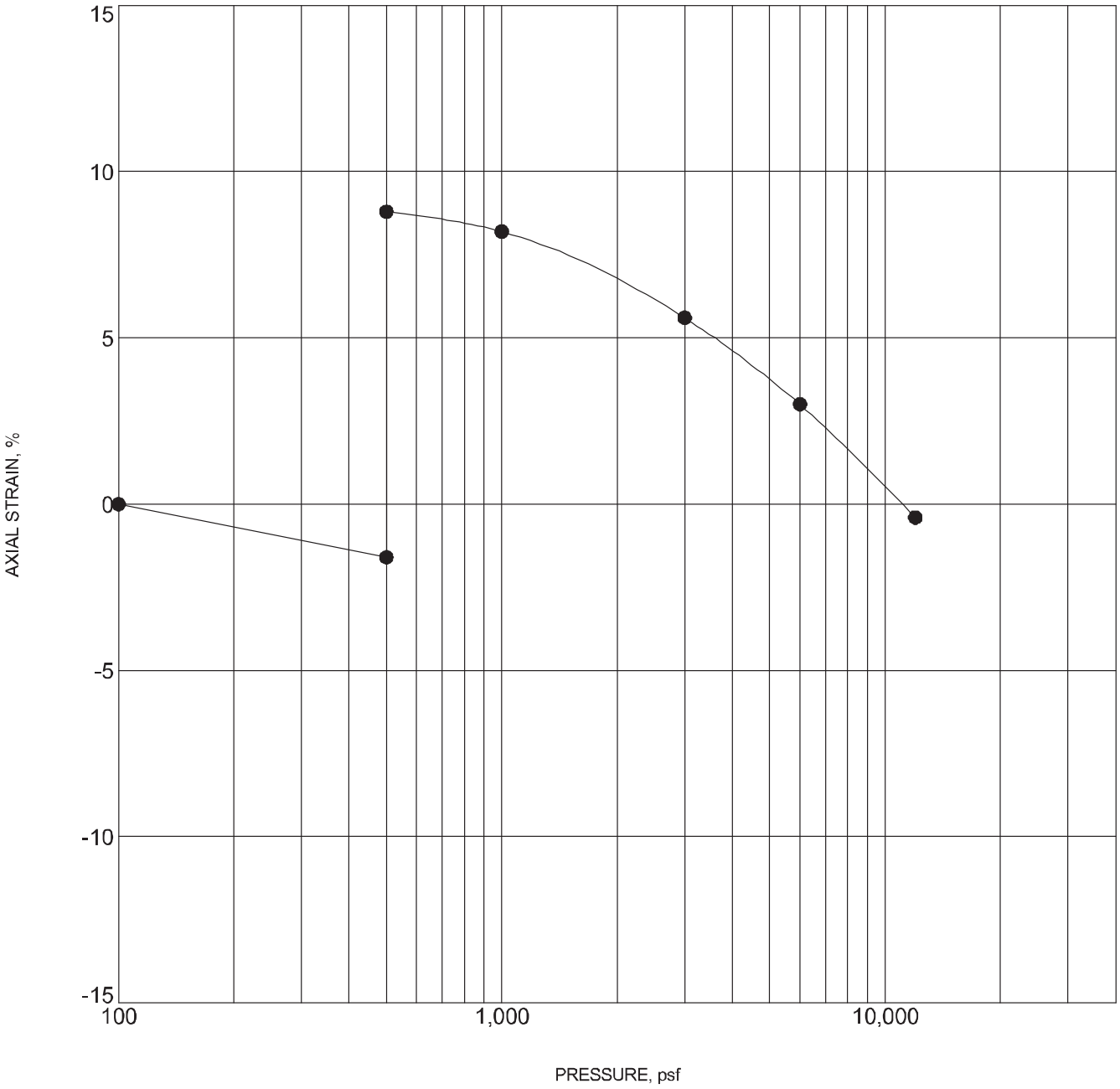
PROJECT NUMBER: 25215031

CLIENT: Quality Technology Services, Inc.

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Permit #25-2535171-LT
RSN 1903242

SWELL CONSOLIDATION TEST



Specimen Identification			Classification	γ_d, pcf	WC, %
●	42	12.5 - 13.5 ft	CLAYSTONE	105	22.8

NOTES: Water was added at 500 psf.

LABORATORY TESTS ARE NOT VALID IF SEPARATED FROM ORIGINAL REPORT. TC_CONSOL_STRAIN-USCS-NO ASTM 25215031 QTS AURORA_USE THIS ONE.GPJ TERRACON_DATATEMPLATE.GDT 8/17/21

PROJECT: QTS Aurora

PROJECT NUMBER: 25215031

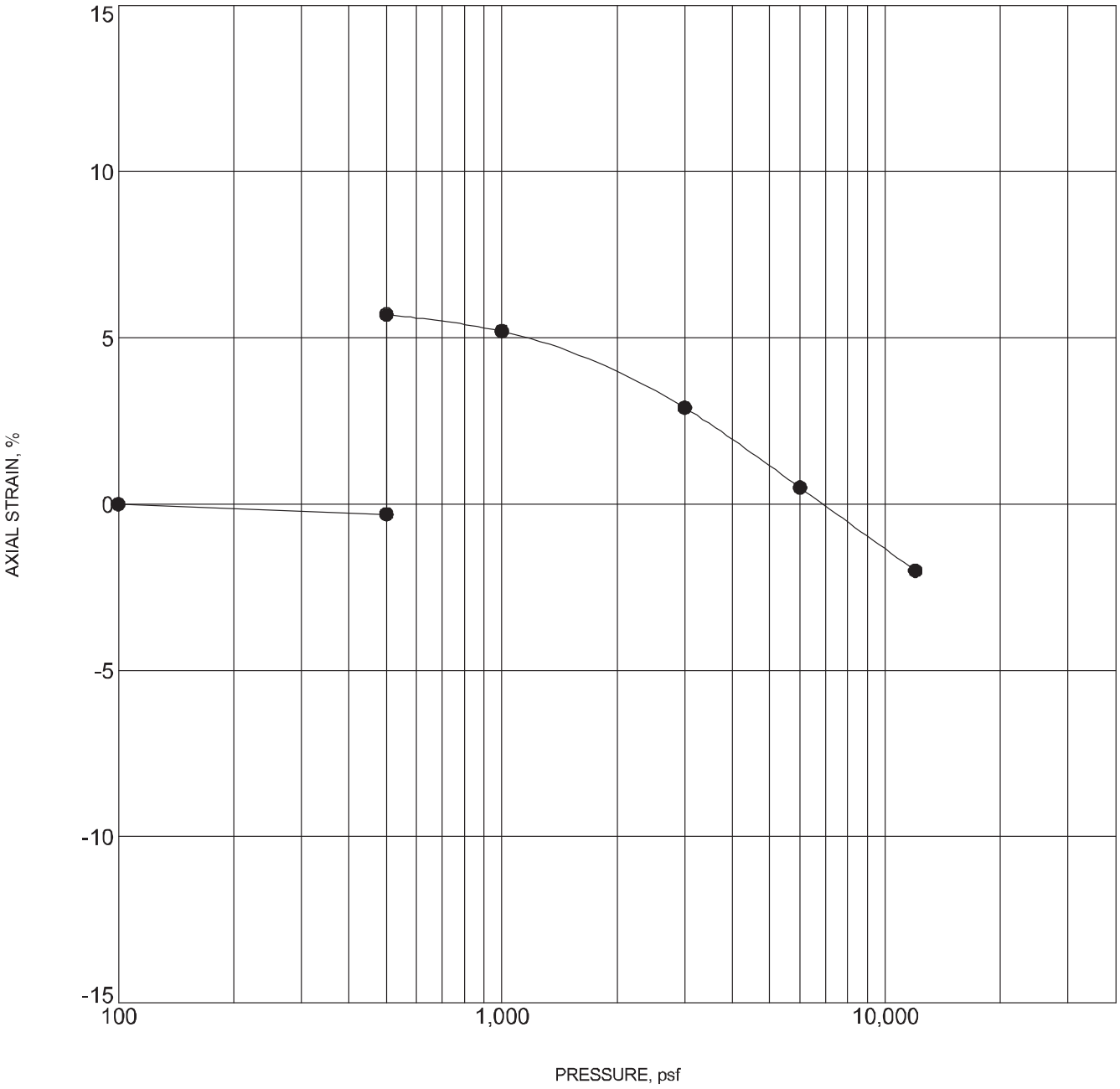
SITE: Gun Club Road and Colfax Avenue
Aurora, Colorado

Terracon
10625 W I70 Frontage Rd N Ste 3
Wheat Ridge, CO

CLIENT: Quality Technology Services, Inc.

Permit #25-2535171-LT
RSN 1903242

SWELL CONSOLIDATION TEST



Specimen Identification			Classification	γ_d, pcf	WC, %
●	F1	10 - 11 ft	CLAYSTONE	126	10.7

NOTES: Water was added at 500 psf.

LABORATORY TESTS ARE NOT VALID IF SEPARATED FROM ORIGINAL REPORT. TC_CONSOL_STRAIN-USCS-NO ASTM 25215031 QTS AURORA_USE THIS ONE.GPJ TERRACON_DATATEMPLATE.GDT 8/17/21

PROJECT: QTS Aurora

SITE: Gun Club Road and Colfax Avenue
Aurora, Colorado

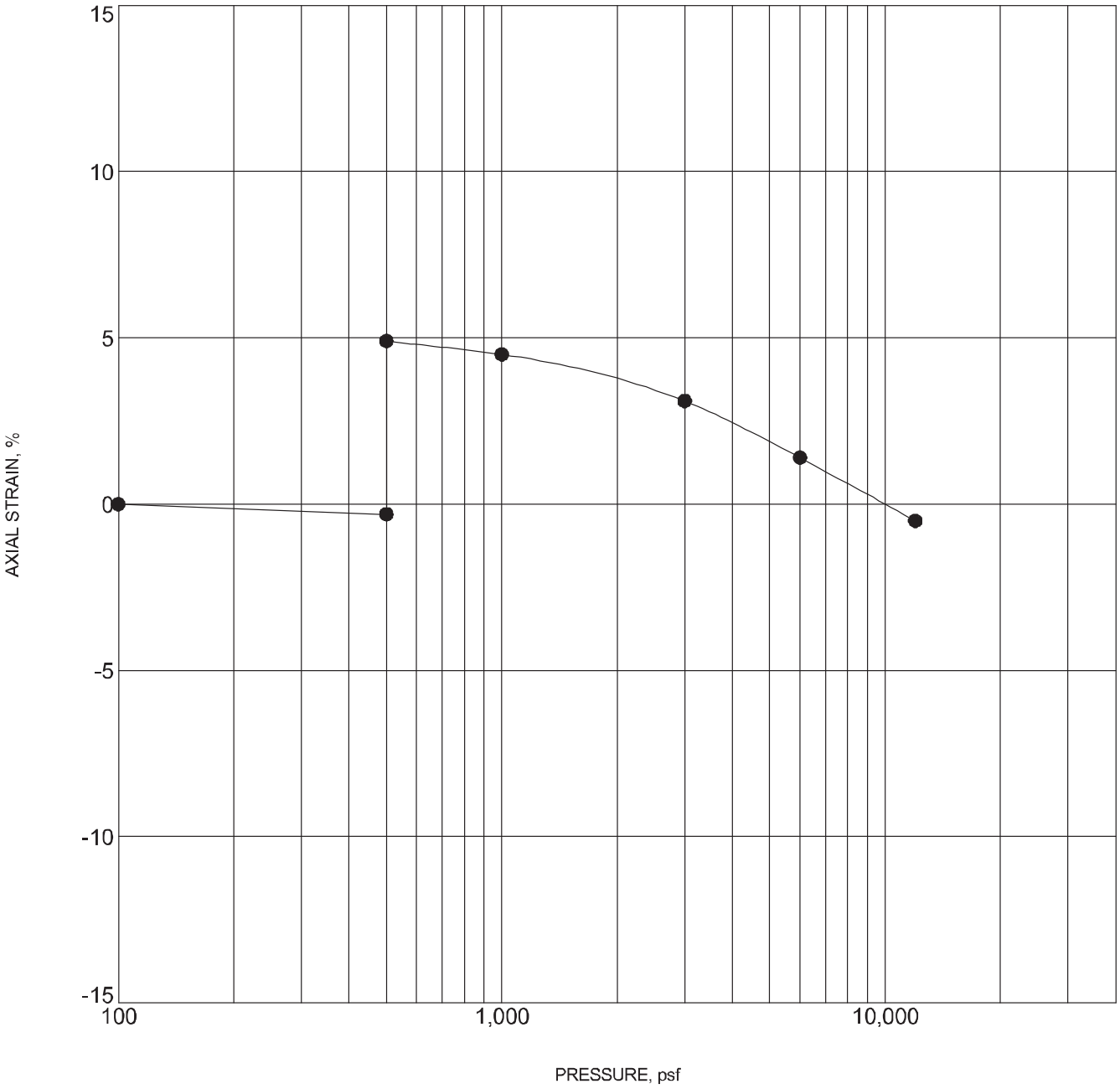
Terracon
10625 W I70 Frontage Rd N Ste 3
Wheat Ridge, CO

PROJECT NUMBER: 25215031

CLIENT: Quality Technology Services, Inc.

Permit #25-2535171-LT
RSN 1903242

SWELL CONSOLIDATION TEST



Specimen Identification			Classification	γ_d, pcf	WC, %
●	F2	10 - 11 ft	CLAYSTONE	106	21.1

NOTES: Water was added at 500 psf.

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PROJECT: QTS Aurora

SITE: Gun Club Road and Colfax Avenue
Aurora, Colorado

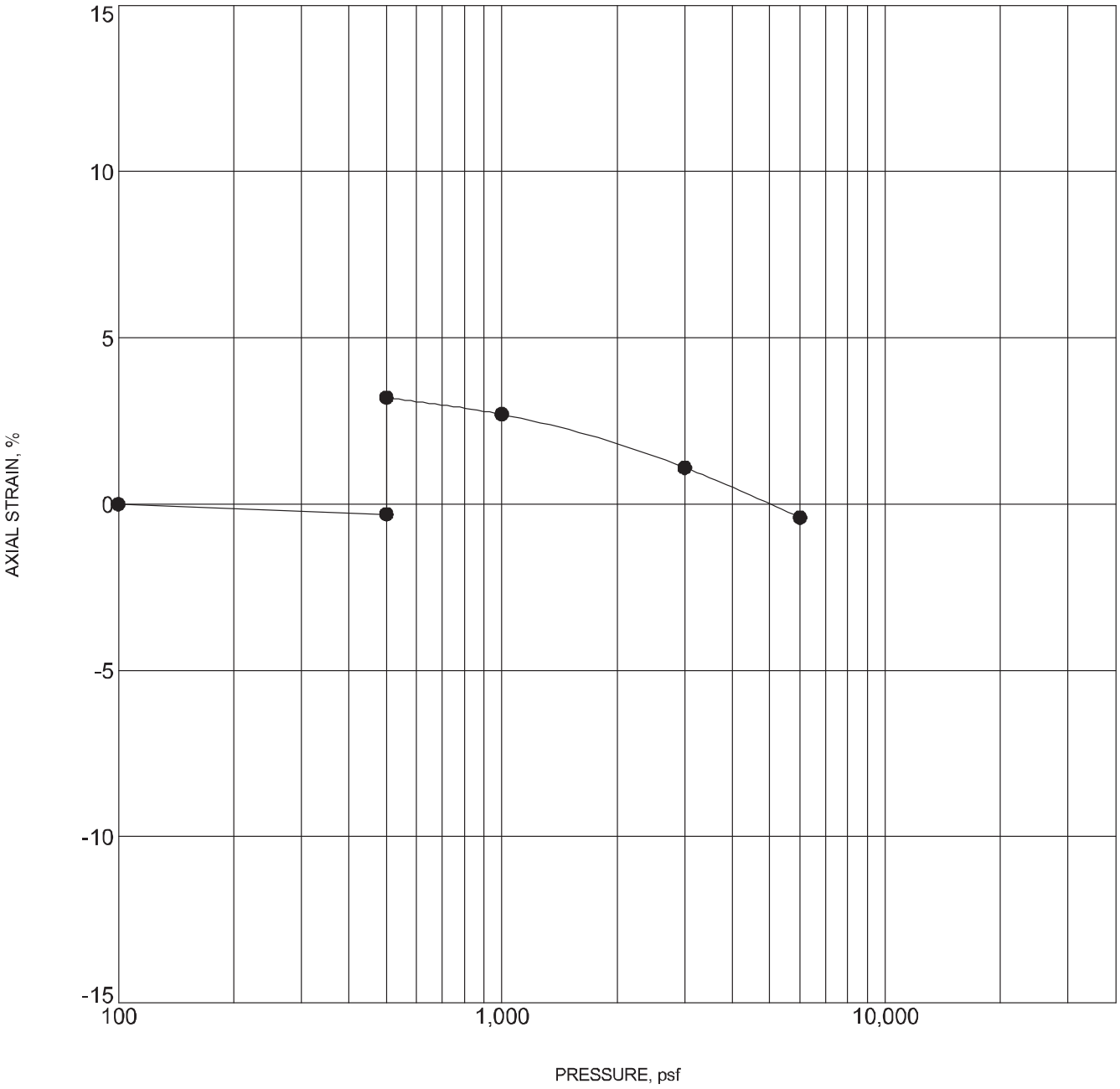
Terracon
10625 W I70 Frontage Rd N Ste 3
Wheat Ridge, CO

PROJECT NUMBER: 25215031

CLIENT: Quality Technology Services, Inc.

Permit #25-2535171-LT
RSN 1903242

SWELL CONSOLIDATION TEST



Specimen Identification			Classification	γ_d, pcf	WC, %
●	F3	5 - 6 ft	CLAYSTONE	110	19.4

NOTES: Water was added at 500 psf.

PROJECT: QTS Aurora

SITE: Gun Club Road and Colfax Avenue
Aurora, Colorado

Terracon
10625 W I70 Frontage Rd N Ste 3
Wheat Ridge, CO

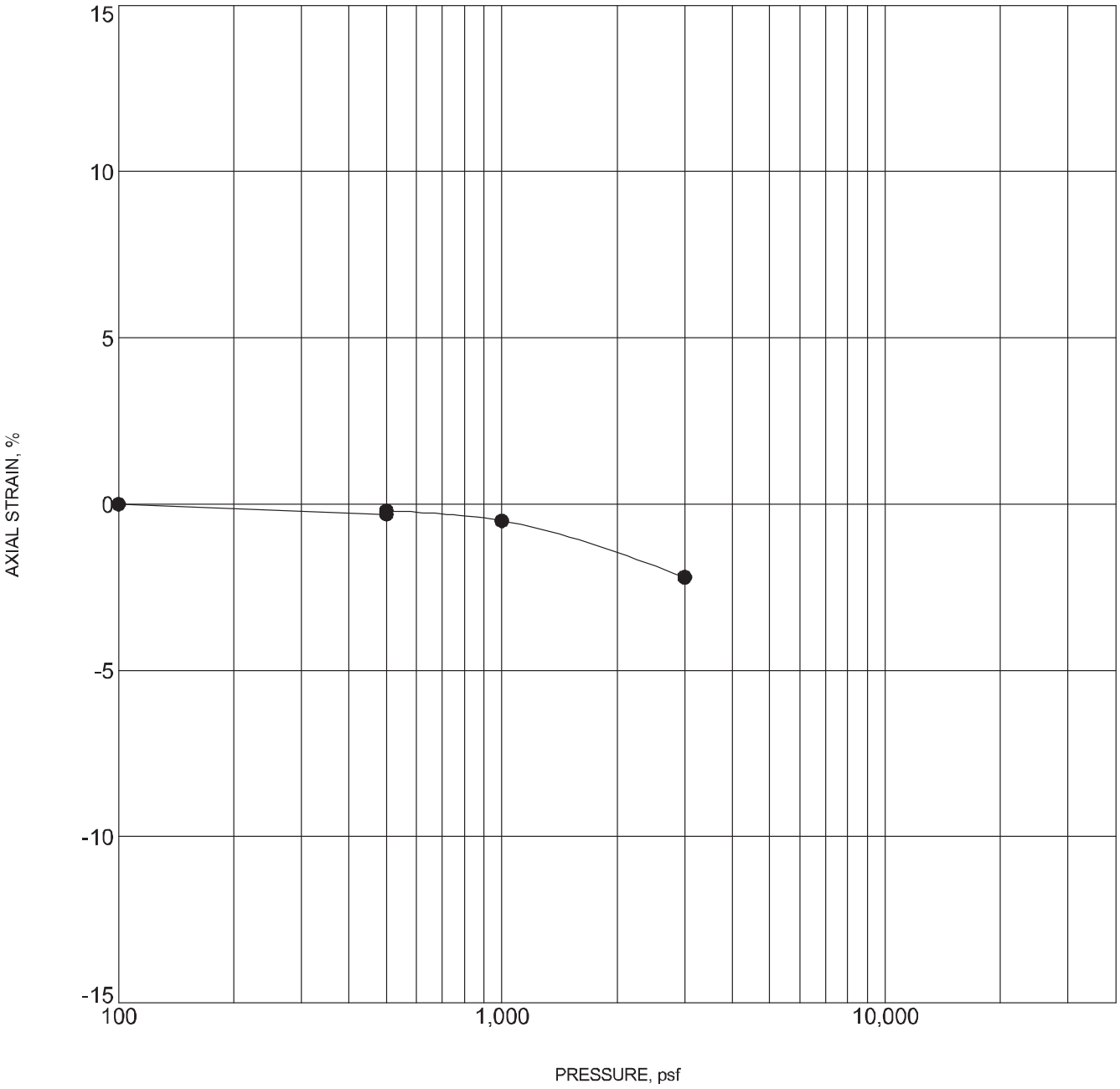
PROJECT NUMBER: 25215031

CLIENT: Quality Technology Services, Inc.

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Permit #25-2535171-LT
RSN 1903242

SWELL CONSOLIDATION TEST



Specimen Identification			Classification	γ_d, pcf	WC, %
●	F4	7.5 - 8.5 ft	LEAN CLAY with SAND (CL)	109	16.3

NOTES: Water was added at 500 psf.

LABORATORY TESTS ARE NOT VALID IF SEPARATED FROM ORIGINAL REPORT. TC_CONSOL_STRAIN-USCS-NO ASTM 25215031 QTS AURORA_USE THIS ONE.GPJ TERRACON_DATATEMPLATE.GDT 8/17/21

PROJECT: QTS Aurora

PROJECT NUMBER: 25215031

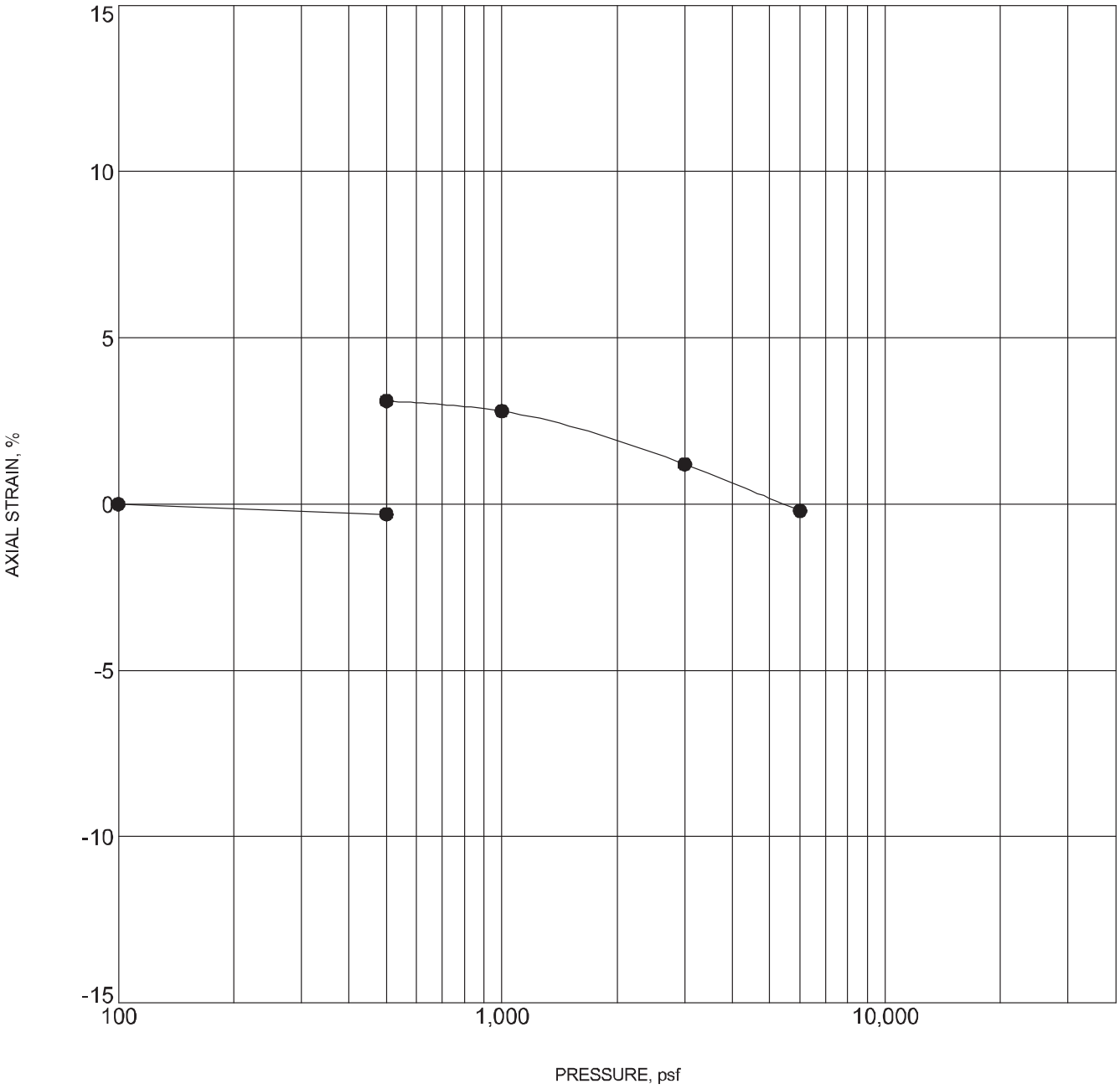
SITE: Gun Club Road and Colfax Avenue
Aurora, Colorado

Terracon
10625 W I70 Frontage Rd N Ste 3
Wheat Ridge, CO

CLIENT: Quality Technology Services, Inc.

Permit #25-2535171-LT
RSN 1903242

SWELL CONSOLIDATION TEST



Specimen Identification			Classification	γ_d, pcf	WC, %
●	F5	10 - 11 ft	CLAYEY SANDSTONE	133	6.6

NOTES: Water was added at 500 psf.

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PROJECT: QTS Aurora

SITE: Gun Club Road and Colfax Avenue
Aurora, Colorado

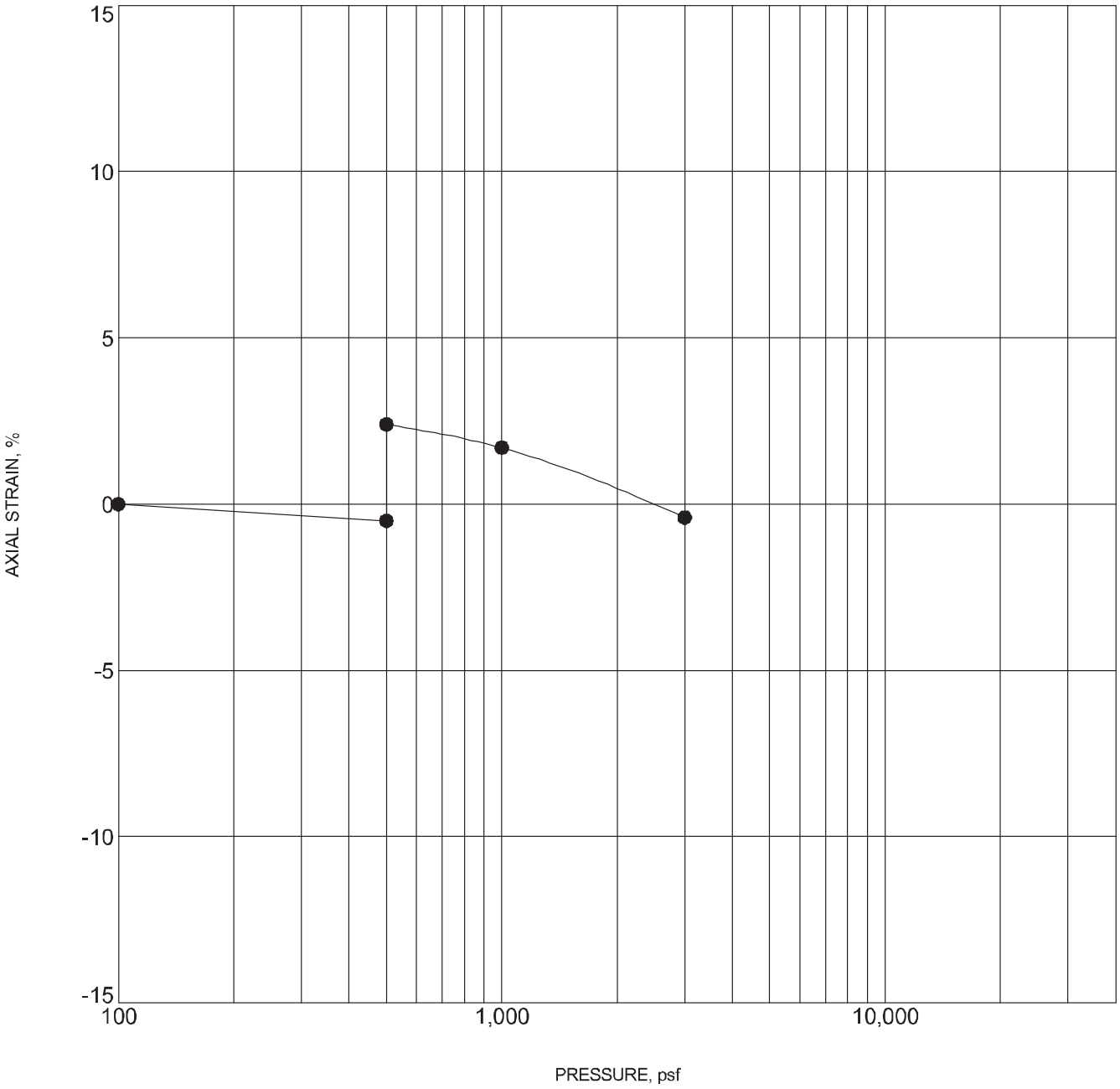
Terracon
10625 W I70 Frontage Rd N Ste 3
Wheat Ridge, CO

PROJECT NUMBER: 25215031

CLIENT: Quality Technology Services, Inc.

Permit #25-2535171-LT
RSN 1903242

SWELL CONSOLIDATION TEST



Specimen Identification			Classification	γ_d, pcf	WC, %
●	F6	5 - 6 ft	CLAYSTONE	100	24.6

NOTES: Water was added at 500 psf.

PROJECT: QTS Aurora

SITE: Gun Club Road and Colfax Avenue
Aurora, Colorado

Terracon
10625 W I70 Frontage Rd N Ste 3
Wheat Ridge, CO

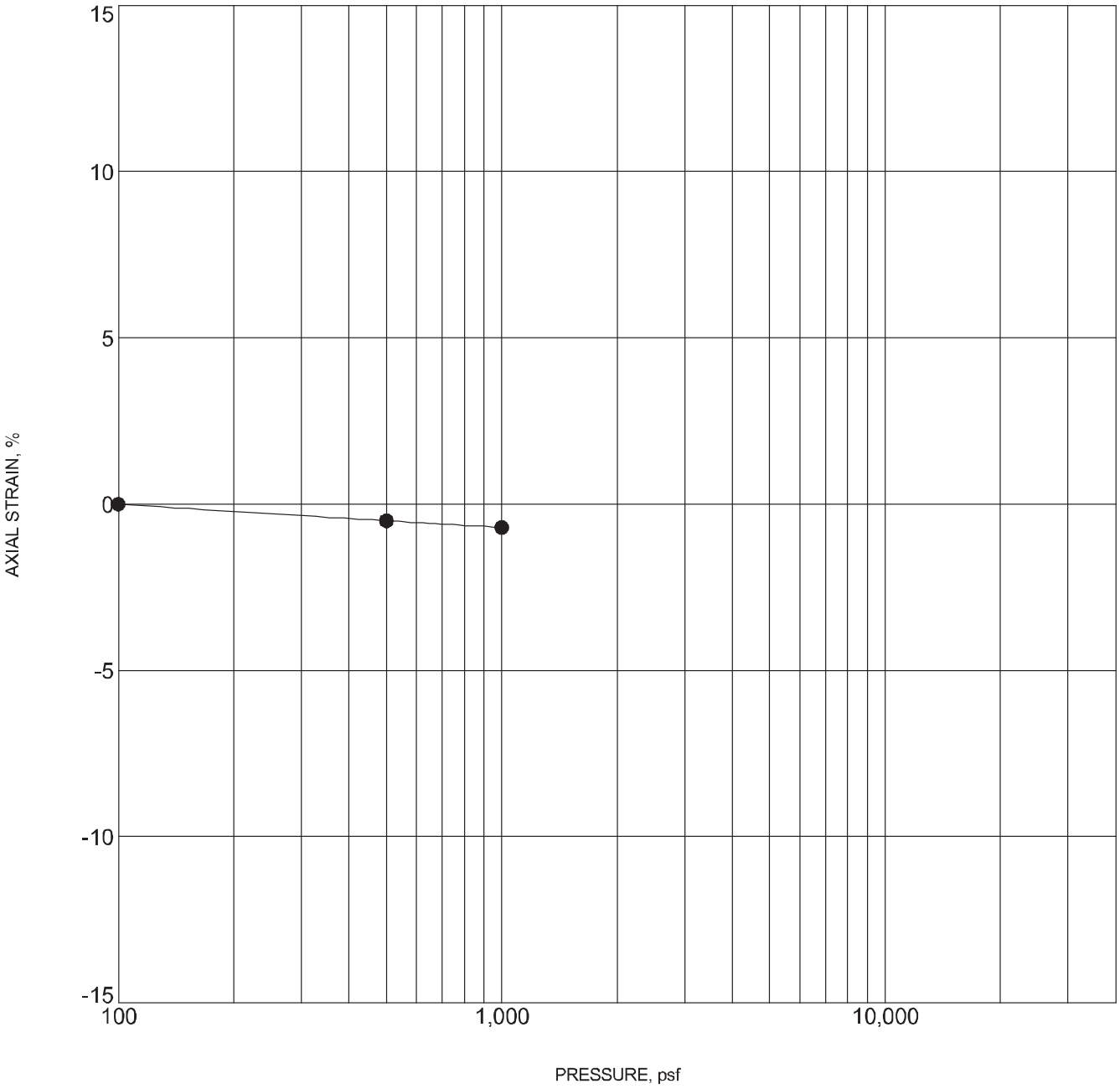
PROJECT NUMBER: 25215031

CLIENT: Quality Technology Services, Inc.

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Permit #25-2535171-LT
RSN 1903242

SWELL CONSOLIDATION TEST



Specimen Identification			Classification	γ_d, pcf	WC, %
●	31	0 - 4 ft	SANDY LEAN CLAY (CL)	104	17.2

NOTES: Water was added at 500 psf. Sample remolded to 95% of Standard Proctor Maximum Dry Density at the Optimum Moisture Content.

PROJECT: QTS Aurora

SITE: Gun Club Road and Colfax Avenue
Aurora, Colorado

Terracon
10625 W I70 Frontage Rd N Ste 3
Wheat Ridge, CO

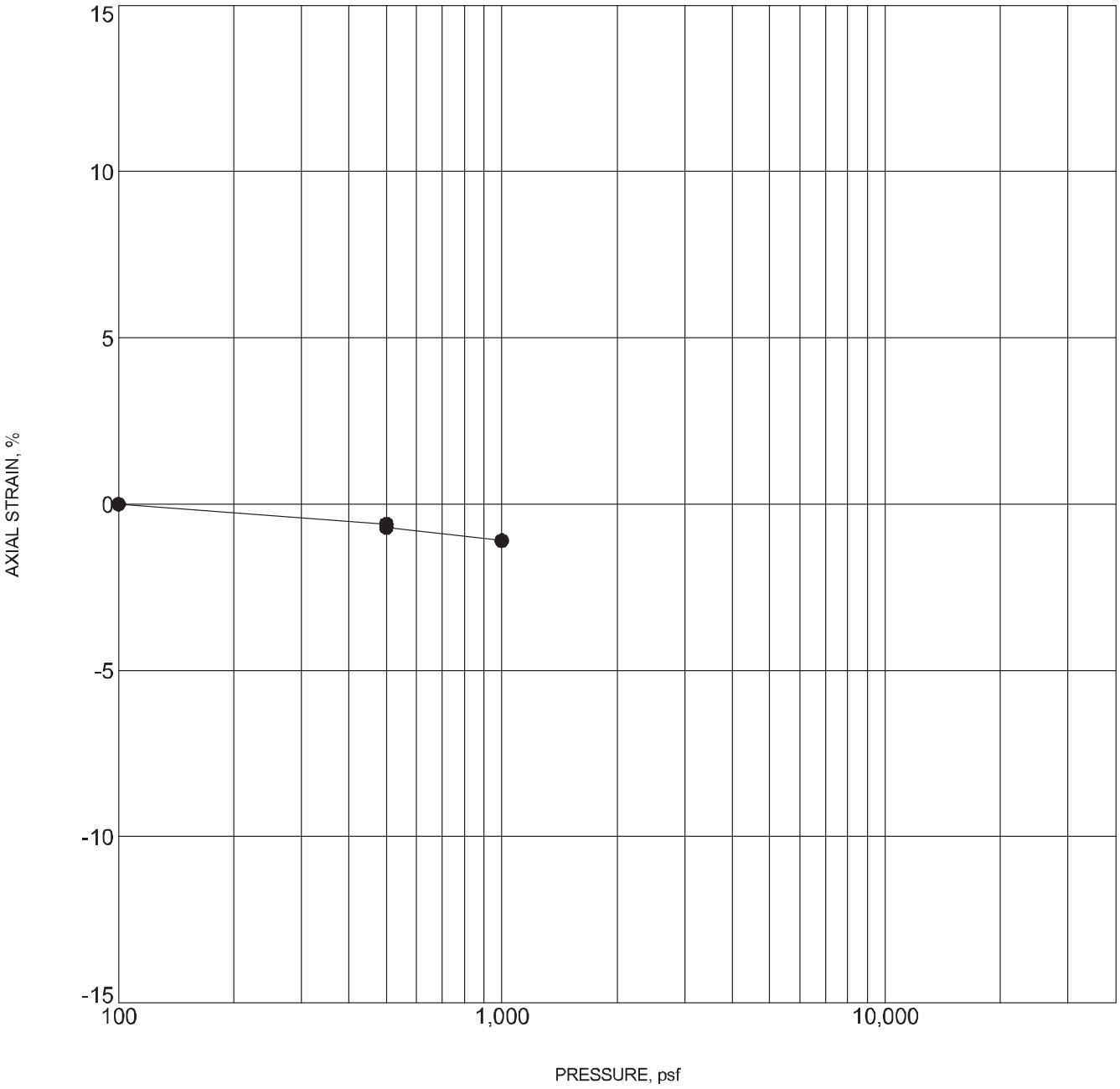
PROJECT NUMBER: 25215031

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Permit #25-2535171-LT
RSN 1903242

SWELL CONSOLIDATION TEST



Specimen Identification			Classification	γ_d, pcf	WC, %
●	31	0 - 4 ft	SANDY LEAN CLAY (CL)	105	18.8

NOTES: Water was added at 500 psf. Sample remolded to 95% of Standard Proctor Maximum Dry Density at +2% of Optimum Moisture Content.

PROJECT: QTS Aurora

SITE: Gun Club Road and Colfax Avenue
Aurora, Colorado

Terracon
10625 W I70 Frontage Rd N Ste 3
Wheat Ridge, CO

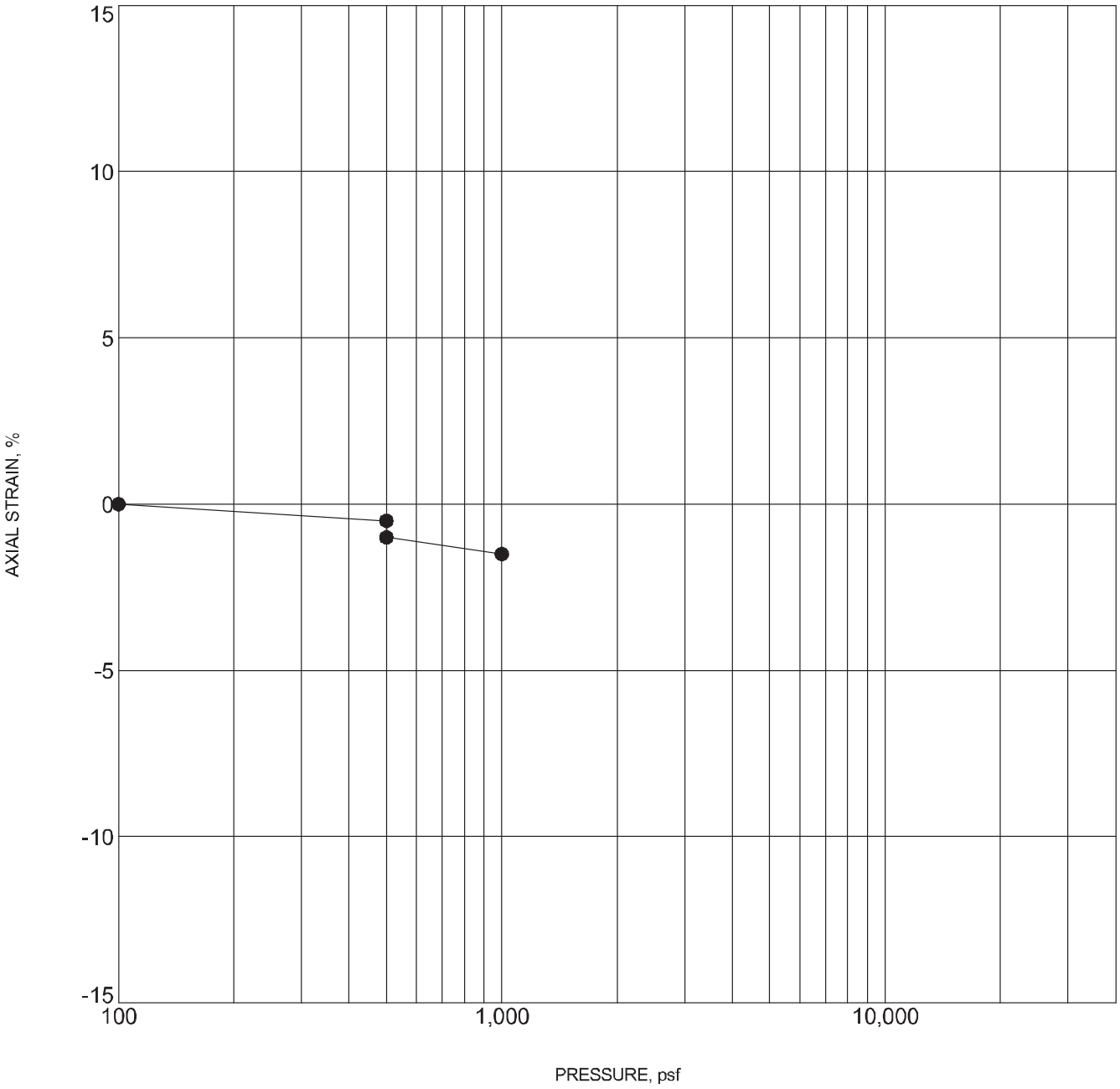
PROJECT NUMBER: 25215031

CLIENT: Quality Technology Services, Inc.

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Permit #25-2535171-LT
RSN 1903242

SWELL CONSOLIDATION TEST



Specimen Identification			Classification	γ_d, pcf	WC, %
●	31	0 - 4 ft	SANDY LEAN CLAY (CL)	106	20.9

NOTES: Water was added at 500 psf. Sample remolded to 95% of Standard Proctor Maximum Dry Density at +4% of Optimum Moisture Content.

PROJECT: QTS Aurora

SITE: Gun Club Road and Colfax Avenue
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Terracon
10625 W I70 Frontage Rd N Ste 3
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PROJECT NUMBER: 25215031

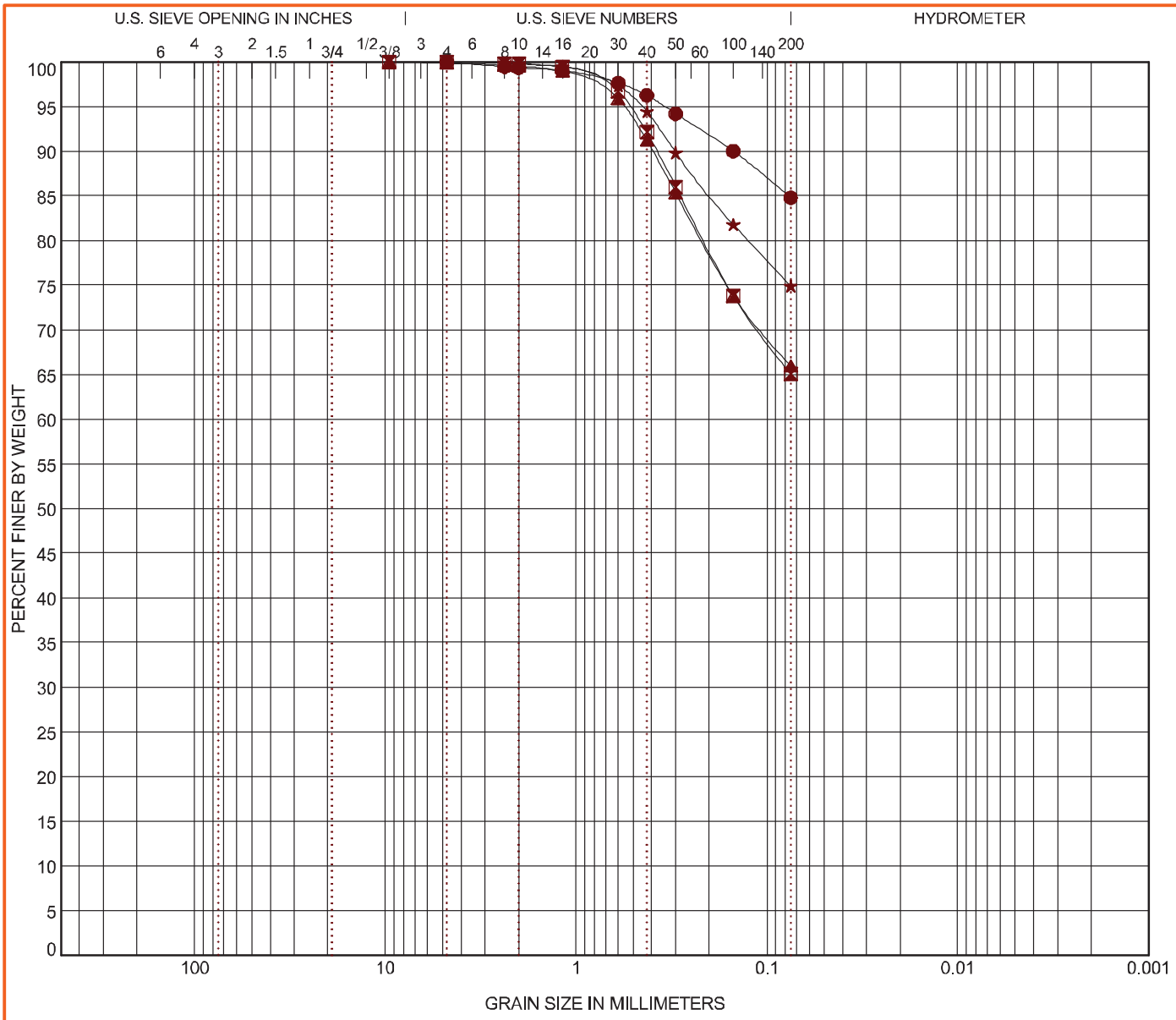
CLIENT: Quality Technology Services, Inc.

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Permit #25-3535171-LT
RSN 1903242

GRAIN SIZE DISTRIBUTION

ASTM D422 / ASTM C136



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Boring ID	Depth	USCS Classification	AASHTO Classification	WC (%)	LL	PL	PI	Cc	Cu
●	21	0 - 4	LEAN CLAY with SAND (CL)	A-6 (12)		33	17	16		
⊠	22	0 - 4	SANDY LEAN CLAY (CL)	A-6 (11)		37	16	21		
▲	24	0 - 4	SANDY LEAN CLAY (CL)	A-6 (11)		37	16	21		
★	25	0 - 4	LEAN CLAY with SAND (CL)	A-6 (14)		38	17	21		

	Boring ID	Depth	D_{100}	D_{60}	D_{30}	D_{10}	%Gravel	%Sand	%Silt	%Fines	%Clay
●	21	0 - 4	4.75				0.0	15.2		84.8	
⊠	22	0 - 4	9.5				0.0	34.9		65.1	
▲	24	0 - 4	9.5				0.2	33.8		66.0	
★	25	0 - 4	4.75				0.0	25.1		74.9	

PROJECT: QTS Aurora

SITE: Gun Club Road and Colfax Avenue
Aurora, Colorado

Terracon
10625 W I70 Frontage Rd N Ste 3
Wheat Ridge, CO

PROJECT NUMBER: 25215031

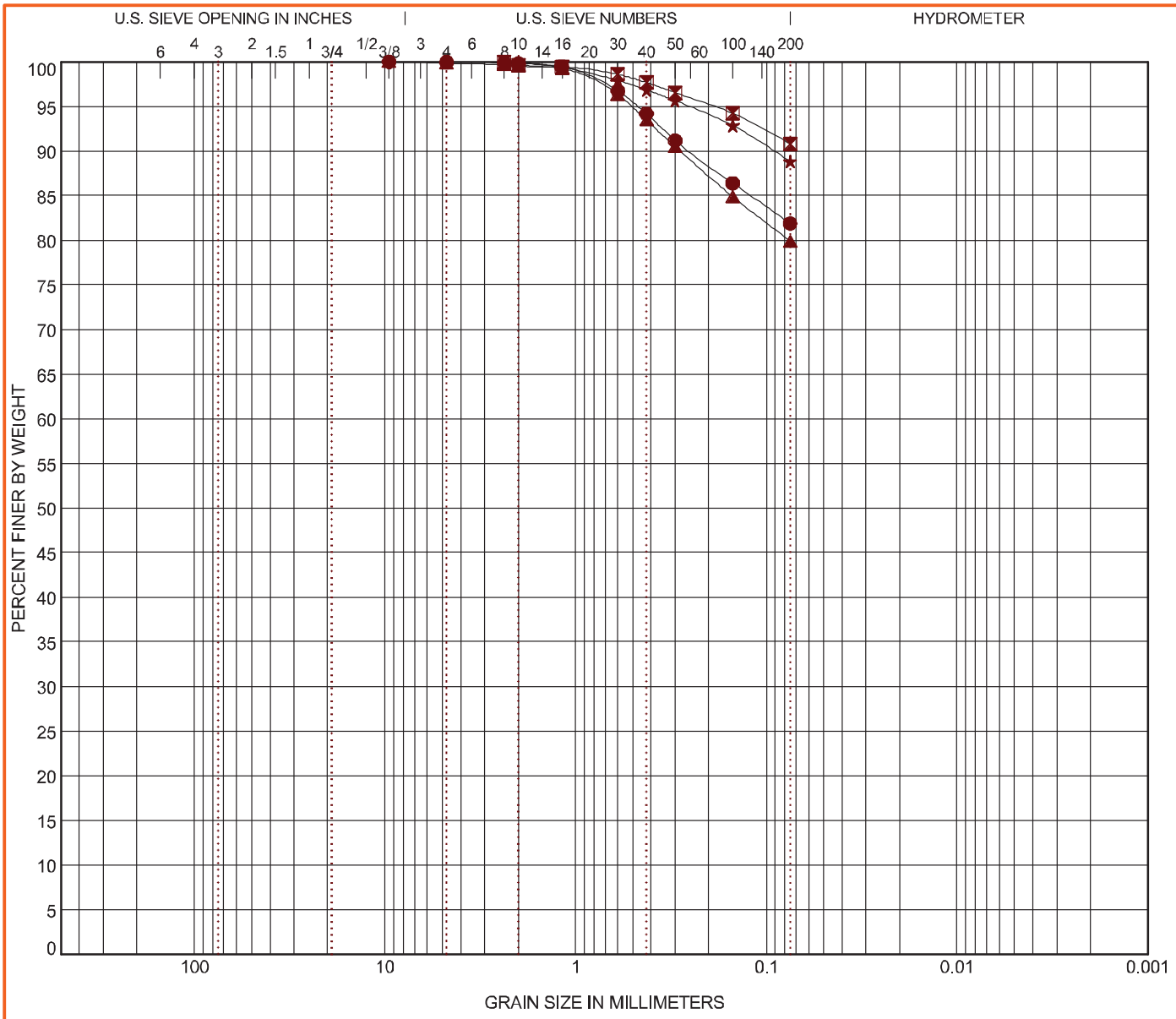
CLIENT: Quality Technology Services, Inc.

LABORATORY TESTS ARE NOT VALID IF SEPARATED FROM ORIGINAL REPORT. GRAIN SIZE: USCS & AASHTO DESC COMBINED 25215031 QTS AURORA_USE THIS ONE.GPJ TERRACON_DATATEMPLATE.GDT 8/12/21

Permit #25-3535171 LT
RSN 1903242

GRAIN SIZE DISTRIBUTION

ASTM D422 / ASTM C136



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Boring ID	Depth	USCS Classification	AASHTO Classification	WC (%)	LL	PL	PI	Cc	Cu
●	26	0 - 4	LEAN CLAY with SAND (CL)	A-6 (15)		36	16	20		
⊠	27	0 - 4	LEAN CLAY (CL)	A-7-6 (28)		46	16	30		
▲	28	0 - 4	LEAN CLAY with SAND (CL)	A-7-6 (18)		41	17	24		
★	30	0 - 4	LEAN CLAY (CL)	A-6 (13)		32	16	16		

	Boring ID	Depth	D_{100}	D_{60}	D_{30}	D_{10}	%Gravel	%Sand	%Silt	%Fines	%Clay
●	26	0 - 4	9.5				0.0	18.1		81.9	
⊠	27	0 - 4	2.36				0.0	9.2		90.8	
▲	28	0 - 4	9.5				0.1	19.9		79.9	
★	30	0 - 4	2				0.0	11.2		88.8	

PROJECT: QTS Aurora

SITE: Gun Club Road and Colfax Avenue
Aurora, Colorado

Terracon
10625 W I70 Frontage Rd N Ste 3
Wheat Ridge, CO

PROJECT NUMBER: 25215031

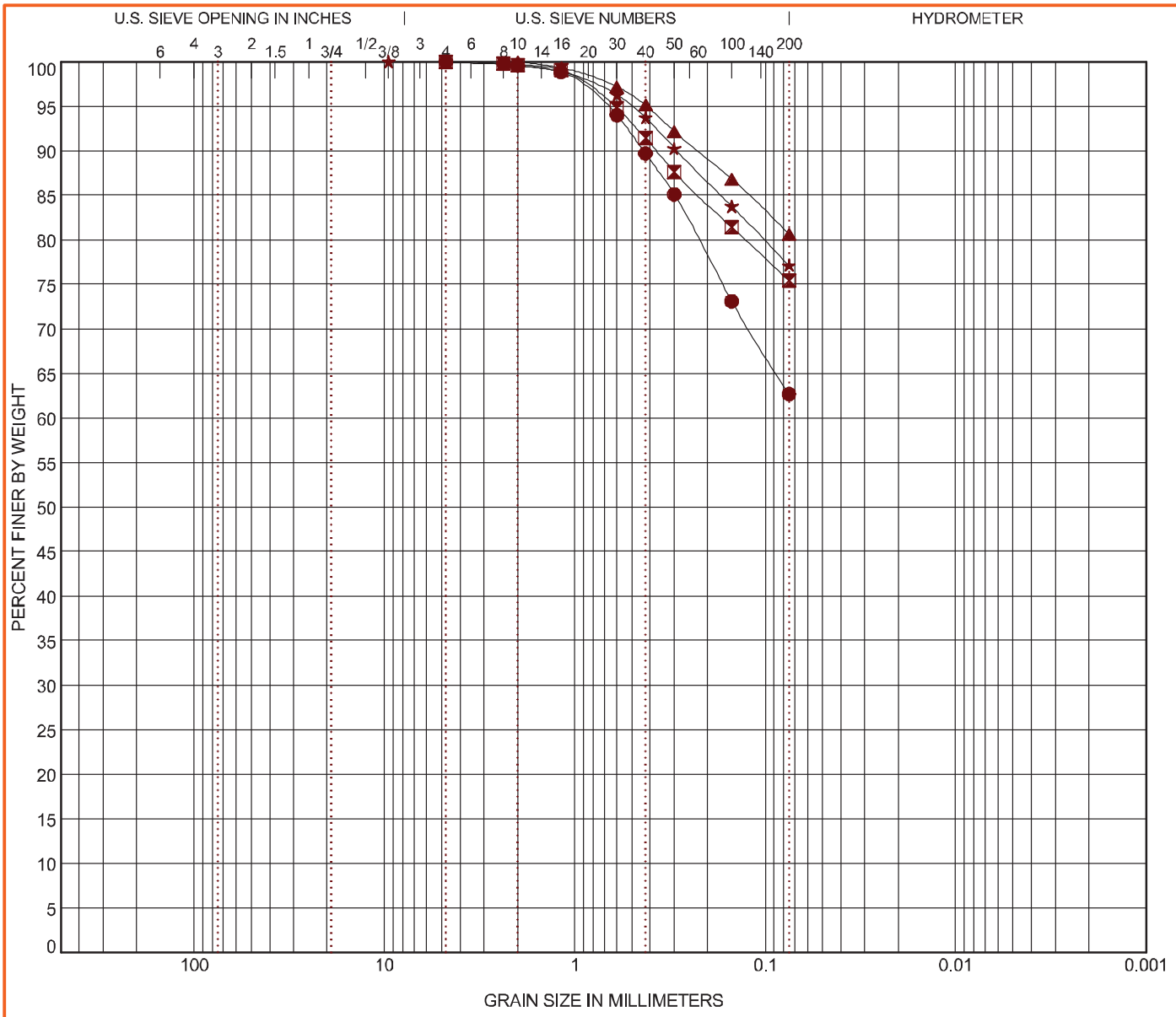
CLIENT: Quality Technology Services, Inc.

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Permit #25-3535171 LT
RSN 1903242

GRAIN SIZE DISTRIBUTION

ASTM D422 / ASTM C136



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Boring ID	Depth	USCS Classification	AASHTO Classification	WC (%)	LL	PL	PI	Cc	Cu
●	31	0 - 4	SANDY LEAN CLAY (CL)	A-6 (7)		30	15	15		
⊠	32	0 - 4	LEAN CLAY with SAND (CL)	A-6 (11)		32	14	18		
▲	33	0 - 4	LEAN CLAY with SAND (CL)	A-6 (18)		39	16	23		
★	34	0 - 4	LEAN CLAY with SAND (CL)	A-6 (14)		36	15	21		

	Boring ID	Depth	D_{100}	D_{60}	D_{30}	D_{10}	%Gravel	%Sand	%Silt	%Fines	%Clay
●	31	0 - 4	4.75				0.0	37.3		62.7	
⊠	32	0 - 4	4.75				0.0	24.6		75.4	
▲	33	0 - 4	2				0.0	19.3		80.7	
★	34	0 - 4	9.5				0.1	22.8		77.1	

PROJECT: QTS Aurora

SITE: Gun Club Road and Colfax Avenue
Aurora, Colorado

Terracon
10625 W I70 Frontage Rd N Ste 3
Wheat Ridge, CO

PROJECT NUMBER: 25215031

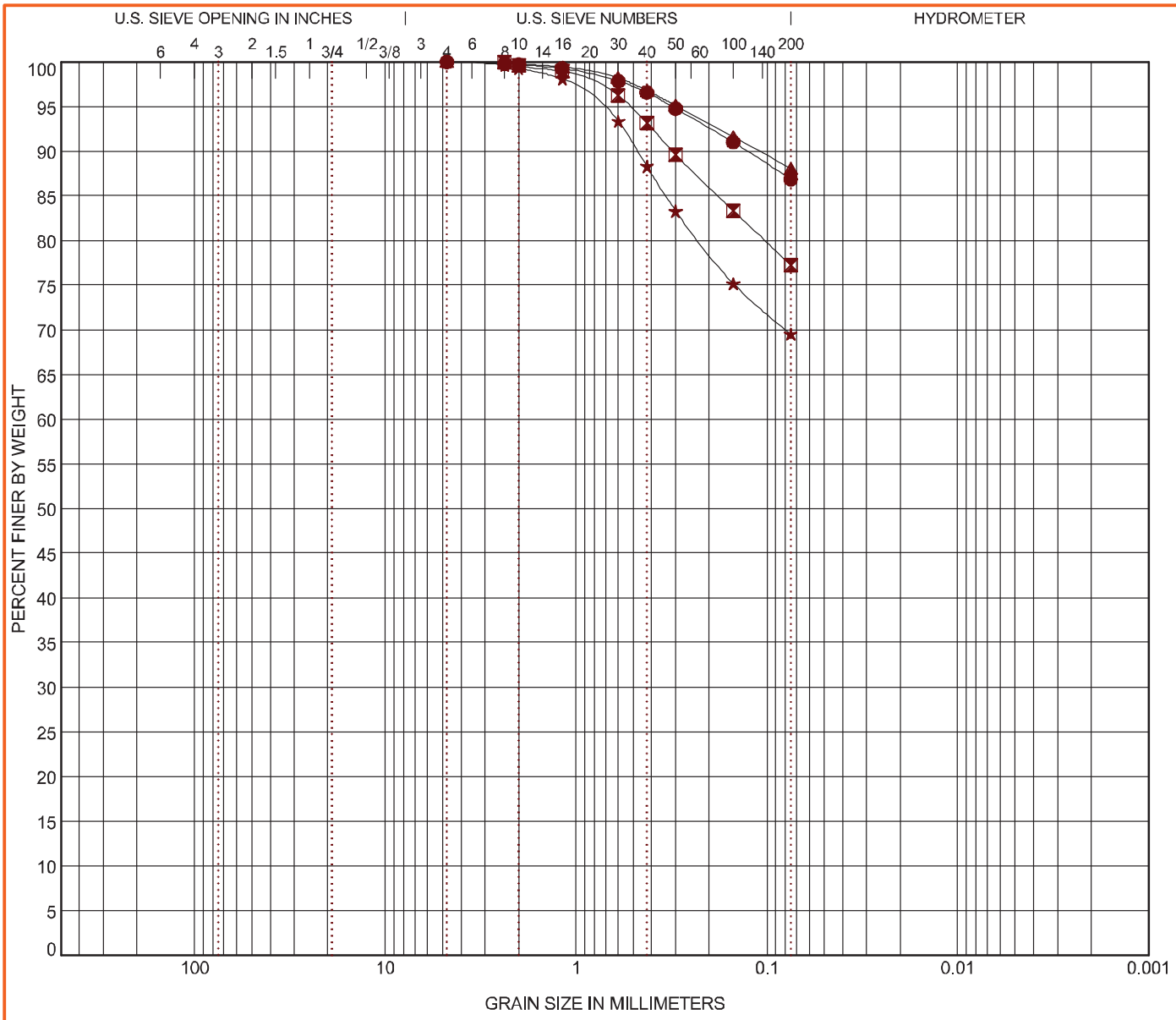
CLIENT: Quality Technology Services, Inc.

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Permit #25-3535171-LT
RSN 1903242

GRAIN SIZE DISTRIBUTION

ASTM D422 / ASTM C136



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Boring ID	Depth	USCS Classification	AASHTO Classification	WC (%)	LL	PL	PI	Cc	Cu
●	35	0 - 4	LEAN CLAY (CL)	A-6 (19)		40	18	22		
⊠	36	0 - 4	LEAN CLAY with SAND (CL)	A-6 (16)		40	17	23		
▲	37	0 - 4	LEAN CLAY (CL)	A-6 (18)		38	17	21		
★	38	0 - 4	SANDY LEAN CLAY (CL)	A-6 (9)		32	15	17		

	Boring ID	Depth	D_{100}	D_{60}	D_{30}	D_{10}	%Gravel	%Sand	%Silt	%Fines	%Clay
●	35	0 - 4	4.75				0.0	13.1		86.9	
⊠	36	0 - 4	2.36				0.0	22.8		77.2	
▲	37	0 - 4	4.75				0.0	11.9		88.1	
★	38	0 - 4	4.75				0.0	30.5		69.5	

PROJECT: QTS Aurora

SITE: Gun Club Road and Colfax Avenue
Aurora, Colorado

Terracon
10625 W I70 Frontage Rd N Ste 3
Wheat Ridge, CO

PROJECT NUMBER: 25215031

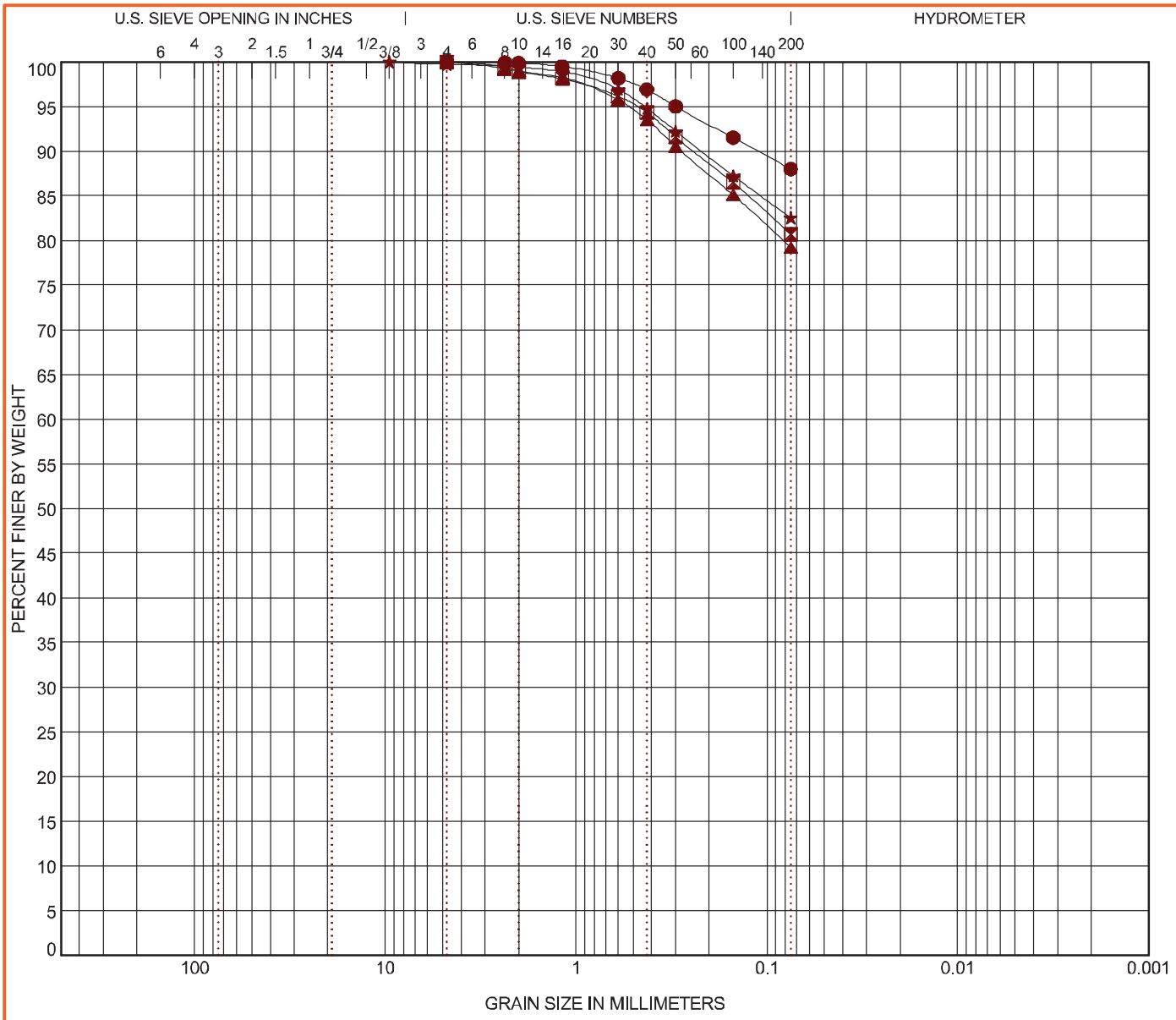
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Permit #25-3535171-LT
RSN 1903242

GRAIN SIZE DISTRIBUTION

ASTM D422 / ASTM C136



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Boring ID	Depth	USCS Classification	AASHTO Classification	WC (%)	LL	PL	PI	Cc	Cu
●	39	0 - 4	LEAN CLAY (CL)	A-7-6 (19)		41	20	21		
⊠	40	0 - 4	LEAN CLAY with SAND (CL)	A-6 (9)		31	18	13		
▲	41	0 - 4	LEAN CLAY with SAND (CL)	A-6 (10)		31	16	15		
★	42	0 - 4	LEAN CLAY with SAND (CL)	A-6 (16)		39	19	20		

	Boring ID	Depth	D_{100}	D_{60}	D_{30}	D_{10}	%Gravel	%Sand	%Silt	%Fines	%Clay
●	39	0 - 4	4.75				0.0	12.0		88.0	
⊠	40	0 - 4	4.75				0.0	19.3		80.7	
▲	41	0 - 4	4.75				0.0	20.8		79.2	
★	42	0 - 4	9.5				0.3	17.2		82.5	

PROJECT: QTS Aurora

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10625 W I70 Frontage Rd N Ste 3
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PROJECT NUMBER: 25215031

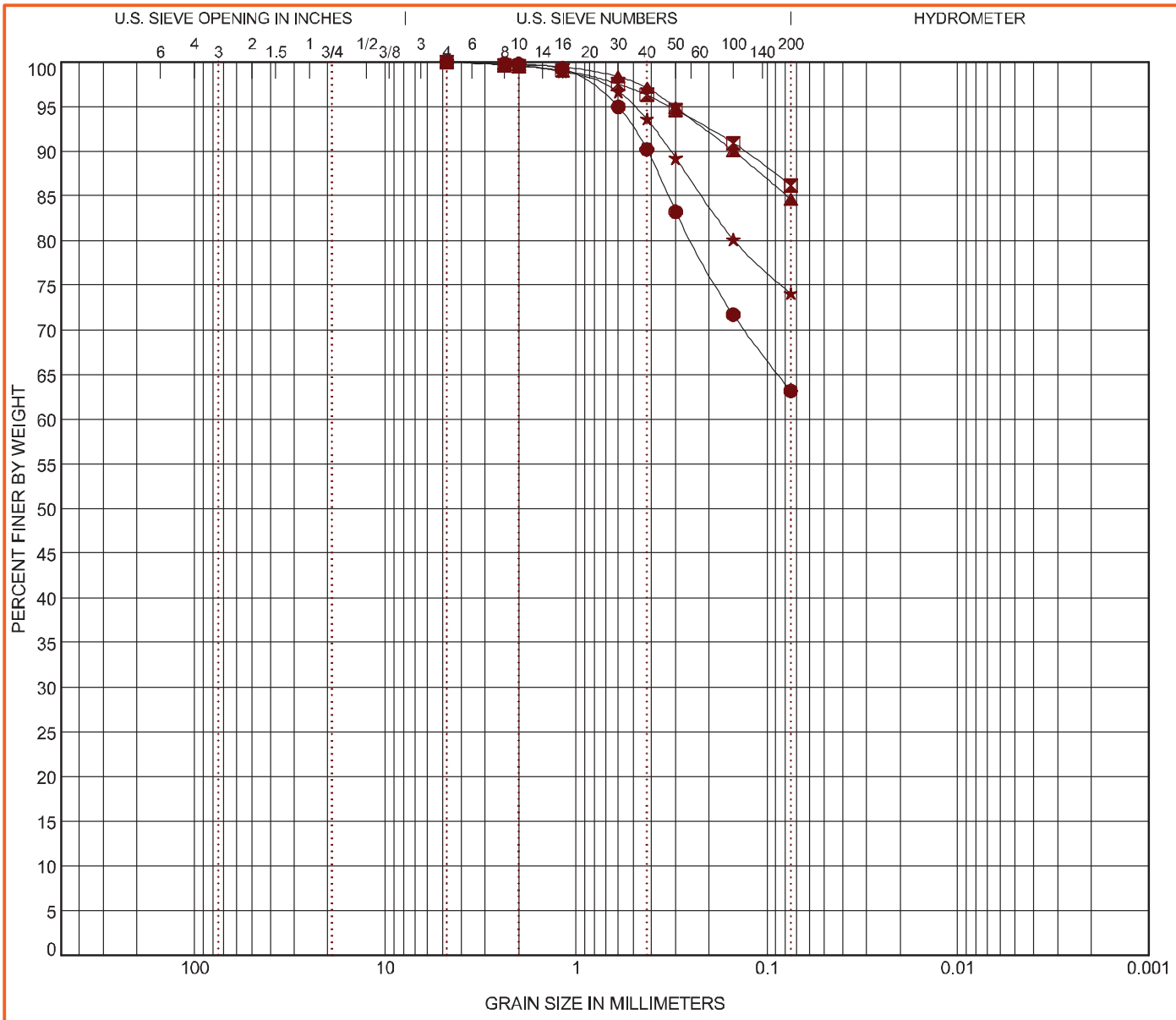
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Permit #25-3535171-LT
RSN 1903242

GRAIN SIZE DISTRIBUTION

ASTM D422 / ASTM C136



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Boring ID	Depth	USCS Classification	AASHTO Classification	WC (%)	LL	PL	PI	Cc	Cu
●	F1	0 - 4	SANDY LEAN CLAY (CL)	A-6 (5)		26	14	12		
⊠	F2	0 - 4	LEAN CLAY (CL)	A-6 (18)		37	15	22		
▲	F3	0 - 4	FAT CLAY with SAND (CH)	A-7-6 (30)		52	18	34		
★	F4	0 - 4	LEAN CLAY with SAND (CL)	A-6 (13)		35	14	21		

	Boring ID	Depth	D_{100}	D_{60}	D_{30}	D_{10}	%Gravel	%Sand	%Silt	%Fines	%Clay
●	F1	0 - 4	4.75				0.0	36.8		63.2	
⊠	F2	0 - 4	4.75				0.0	13.8		86.2	
▲	F3	0 - 4	4.75				0.0	15.4		84.6	
★	F4	0 - 4	4.75				0.0	25.9		74.1	

PROJECT: QTS Aurora

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PROJECT NUMBER: 25215031

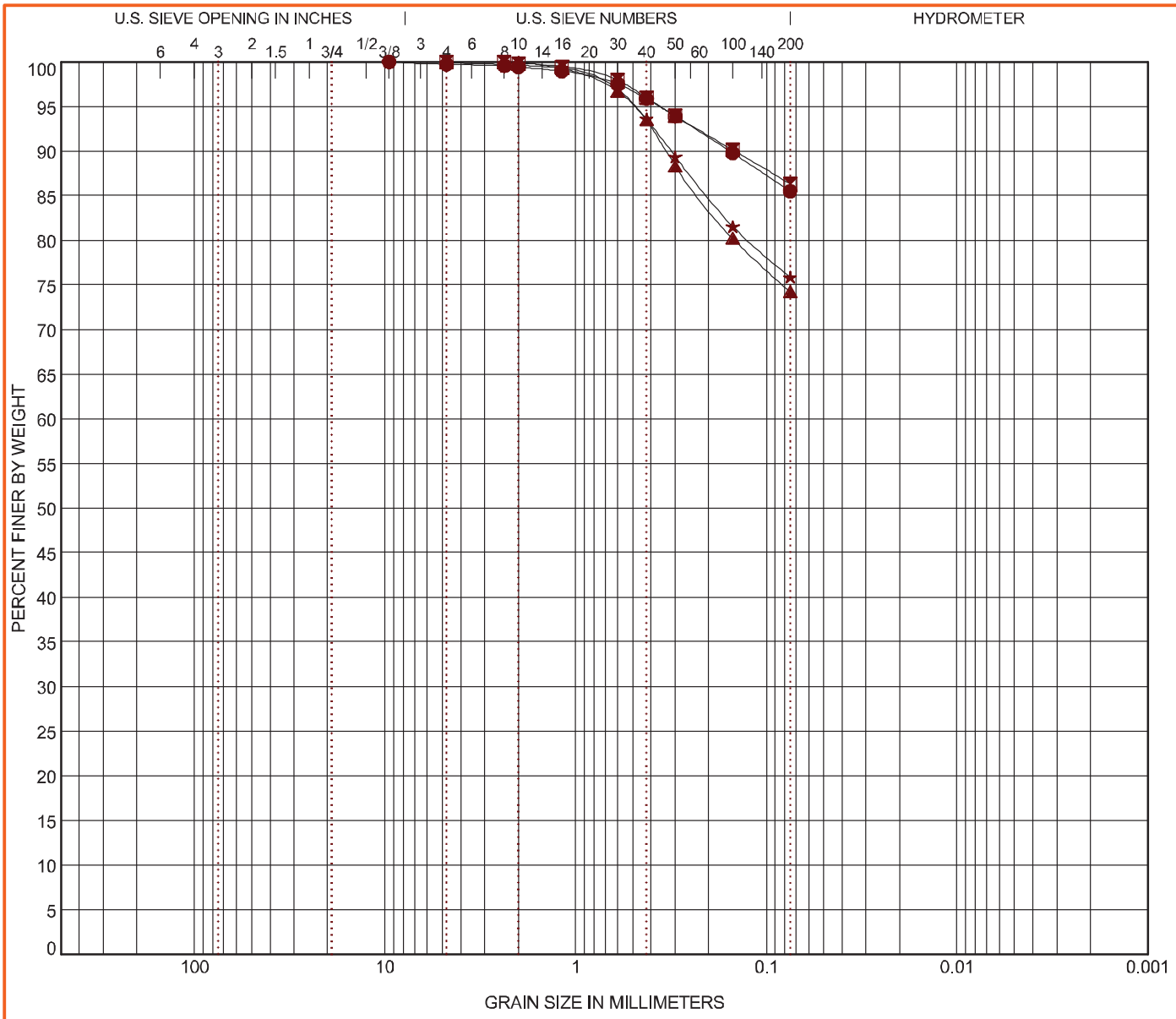
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Permit #25-3535171-LT
RSN 1903242

GRAIN SIZE DISTRIBUTION

ASTM D422 / ASTM C136



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Boring ID	Depth	USCS Classification	AASHTO Classification	WC (%)	LL	PL	PI	Cc	Cu
●	F5	0 - 4	LEAN CLAY (CL)	A-6 (9)		30	18	12		
⊠	F6	0 - 4	LEAN CLAY (CL)	A-7-6 (23)		44	17	27		
▲	S3	0 - 4	LEAN CLAY with SAND (CL)	A-6 (11)		34	16	18		
★	S4	0 - 4	LEAN CLAY with SAND (CL)	A-6 (13)		35	15	20		

	Boring ID	Depth	D_{100}	D_{60}	D_{30}	D_{10}	%Gravel	%Sand	%Silt	%Fines	%Clay
●	F5	0 - 4	9.5				0.3	14.2		85.5	
⊠	F6	0 - 4	4.75				0.0	13.7		86.3	
▲	S3	0 - 4	2				0.0	25.8		74.2	
★	S4	0 - 4	9.5				0.1	24.0		75.9	

PROJECT: QTS Aurora

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Terracon
10625 W I70 Frontage Rd N Ste 3
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PROJECT NUMBER: 25215031

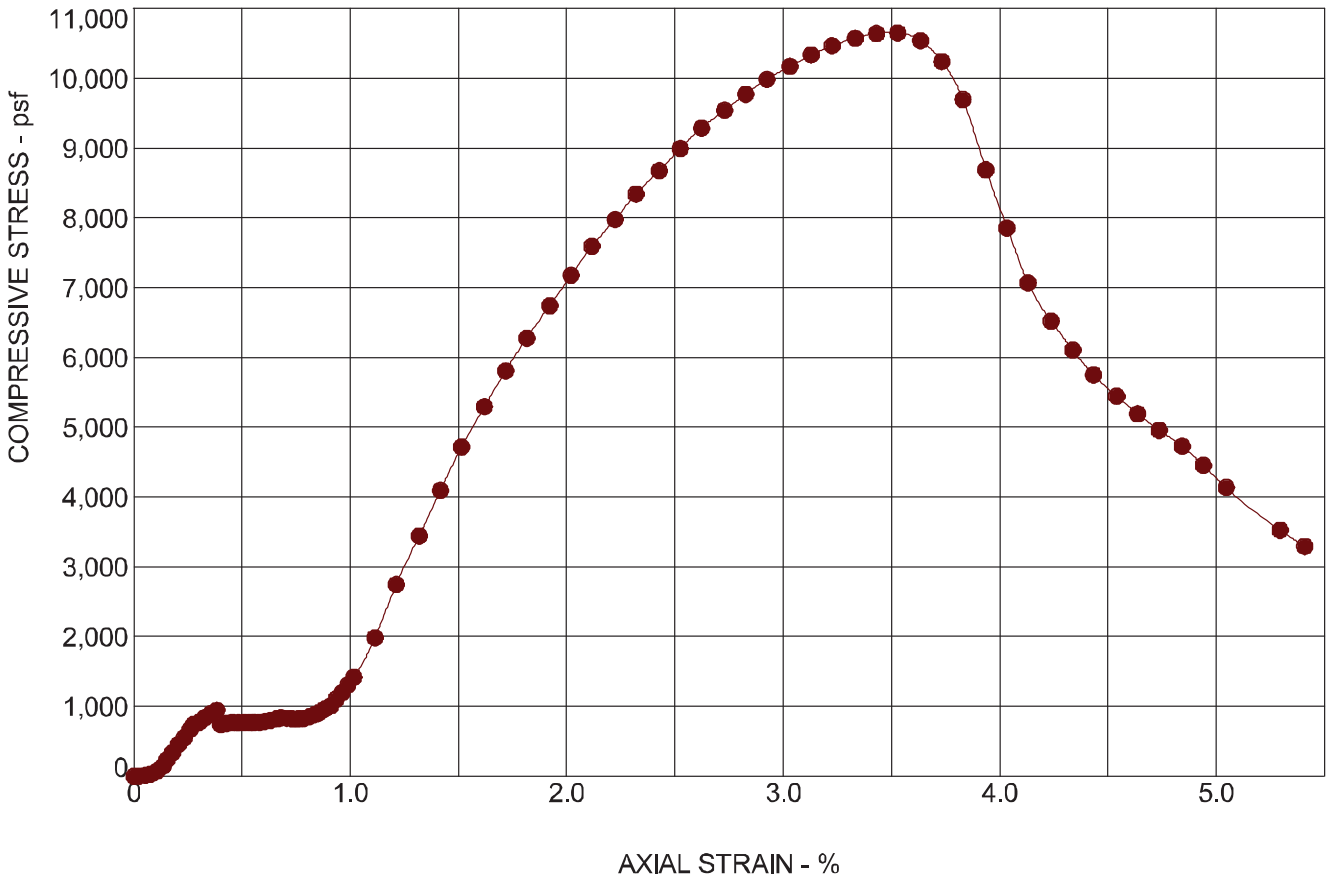
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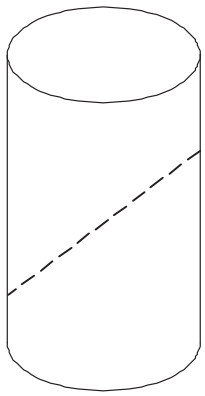
Permit #25-2535171-LT
RGN 2005242

UNCONFINED COMPRESSION TEST

ASTM D2166



SPECIMEN FAILURE MODE



Failure Mode: Shear (dashed)

SPECIMEN TEST DATA

Moisture Content:	%	26.0
Dry Density:	pcf	98
Diameter:	in.	1.93
Height:	in.	3.80
Height / Diameter Ratio:		1.97
Calculated Saturation:	%	98.51
Calculated Void Ratio:		0.71
Assumed Specific Gravity:		2.7
Failure Strain:	%	3.53
Unconfined Compressive Strength	(psf)	10647
Undrained Shear Strength:	(psf)	5323
Strain Rate:	in/min	0.0381
Remarks:		

SAMPLE TYPE: CARS

SAMPLE LOCATION: 22 @ 19 - 20 feet

DESCRIPTION: CLAYSTONE

LL	PL	PI	Percent < #200 Sieve

PROJECT: QTS Aurora

PROJECT NUMBER: 25215031

SITE: Gun Club Road and Colfax Avenue
Aurora, Colorado

Terracon
10625 W I70 Frontage Rd N Ste 3
Wheat Ridge, CO

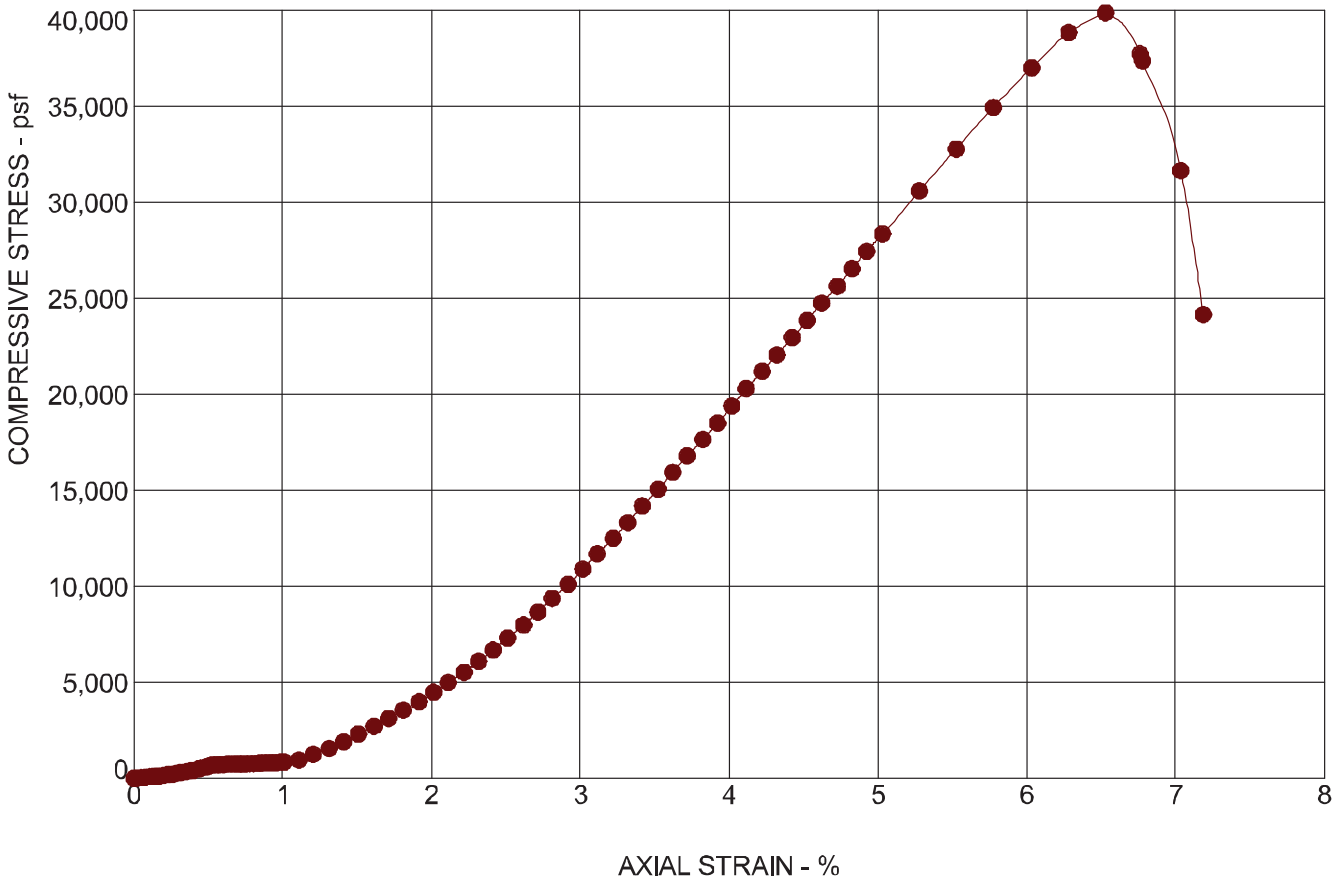
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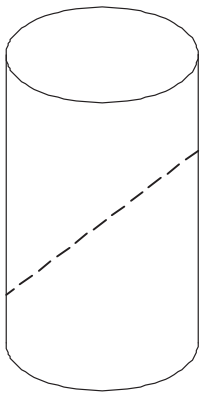
Permit #25-2535171-LT
RSN 2003242

UNCONFINED COMPRESSION TEST

ASTM D2166



SPECIMEN FAILURE MODE



Failure Mode: Shear (dashed)

SPECIMEN TEST DATA

Moisture Content:	%	18.8
Dry Density:	pcf	108
Diameter:	in.	1.95
Height:	in.	3.91
Height / Diameter Ratio:		2.00
Calculated Saturation:	%	91.92
Calculated Void Ratio:		0.55
Assumed Specific Gravity:		2.7
Failure Strain:	%	6.53
Unconfined Compressive Strength	(psf)	39860
Undrained Shear Strength:	(psf)	19930
Strain Rate:	in/min	0.0390
Remarks:		

SAMPLE TYPE: CARS

SAMPLE LOCATION: 28 @ 24 - 25 feet

DESCRIPTION: CLAYSTONE

LL	PL	PI	Percent < #200 Sieve

PROJECT: QTS Aurora

PROJECT NUMBER: 25215031

SITE: Gun Club Road and Colfax Avenue
Aurora, Colorado

Terracon
10625 W I70 Frontage Rd N Ste 3
Wheat Ridge, CO

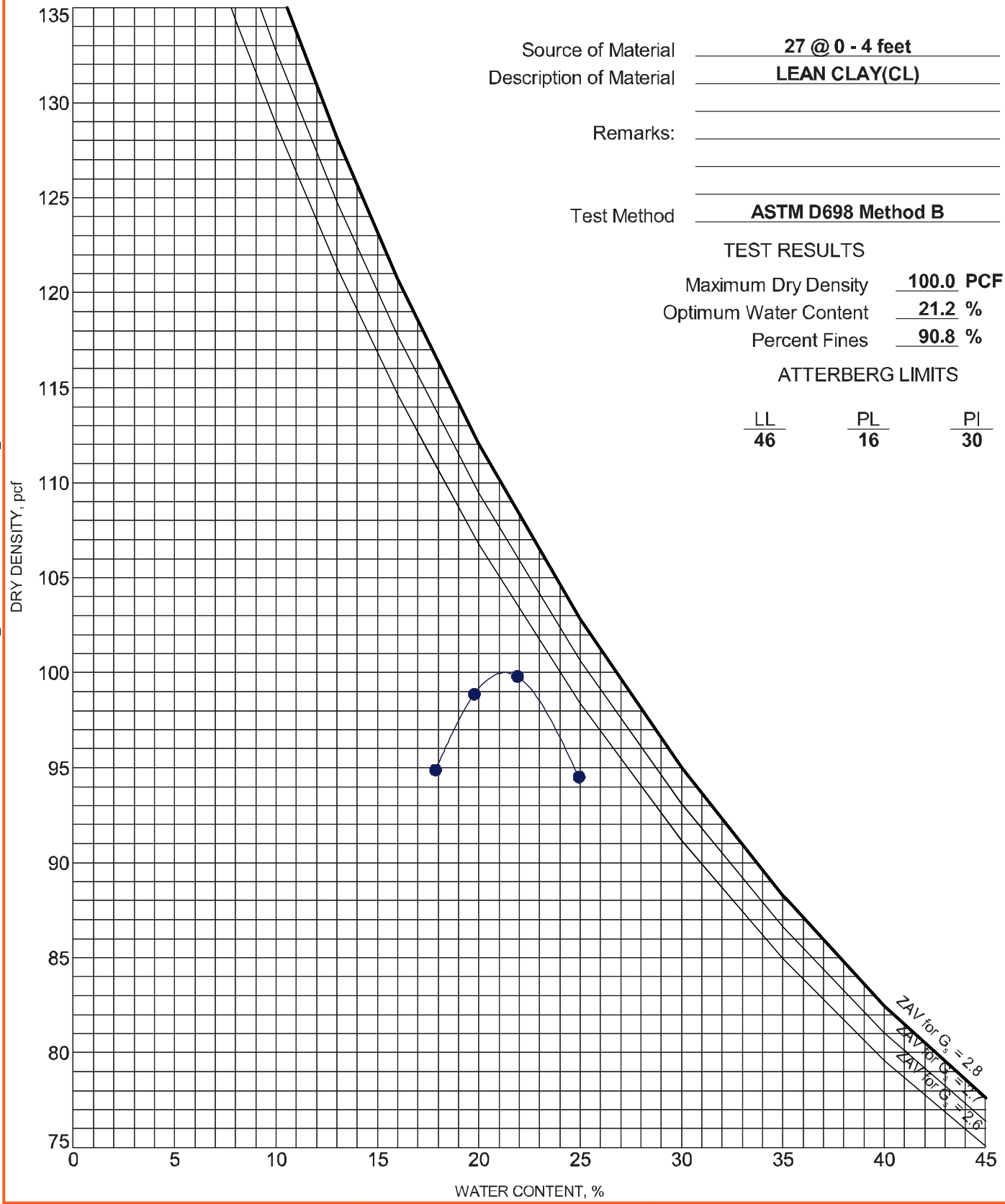
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Permit #25-2535171-LT
RSN 19032422

MOISTURE-DENSITY RELATIONSHIP

ASTM D698/D1557



Source of Material: **27 @ 0 - 4 feet**

Description of Material: **LEAN CLAY(CL)**

Remarks:

Test Method: **ASTM D698 Method B**

TEST RESULTS

Maximum Dry Density	**100.0**	**PCF**
Optimum Water Content	**21.2**	**%**
Percent Fines	**90.8**	**%**

ATTERBERG LIMITS

LL	PL	PI
46	**16**	**30**

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PROJECT: QTS Aurora

SITE: Gun Club Road and Colfax Avenue
Aurora, Colorado

Terracon
10625 W I70 Frontage Rd N Ste 3
Wheat Ridge, CO

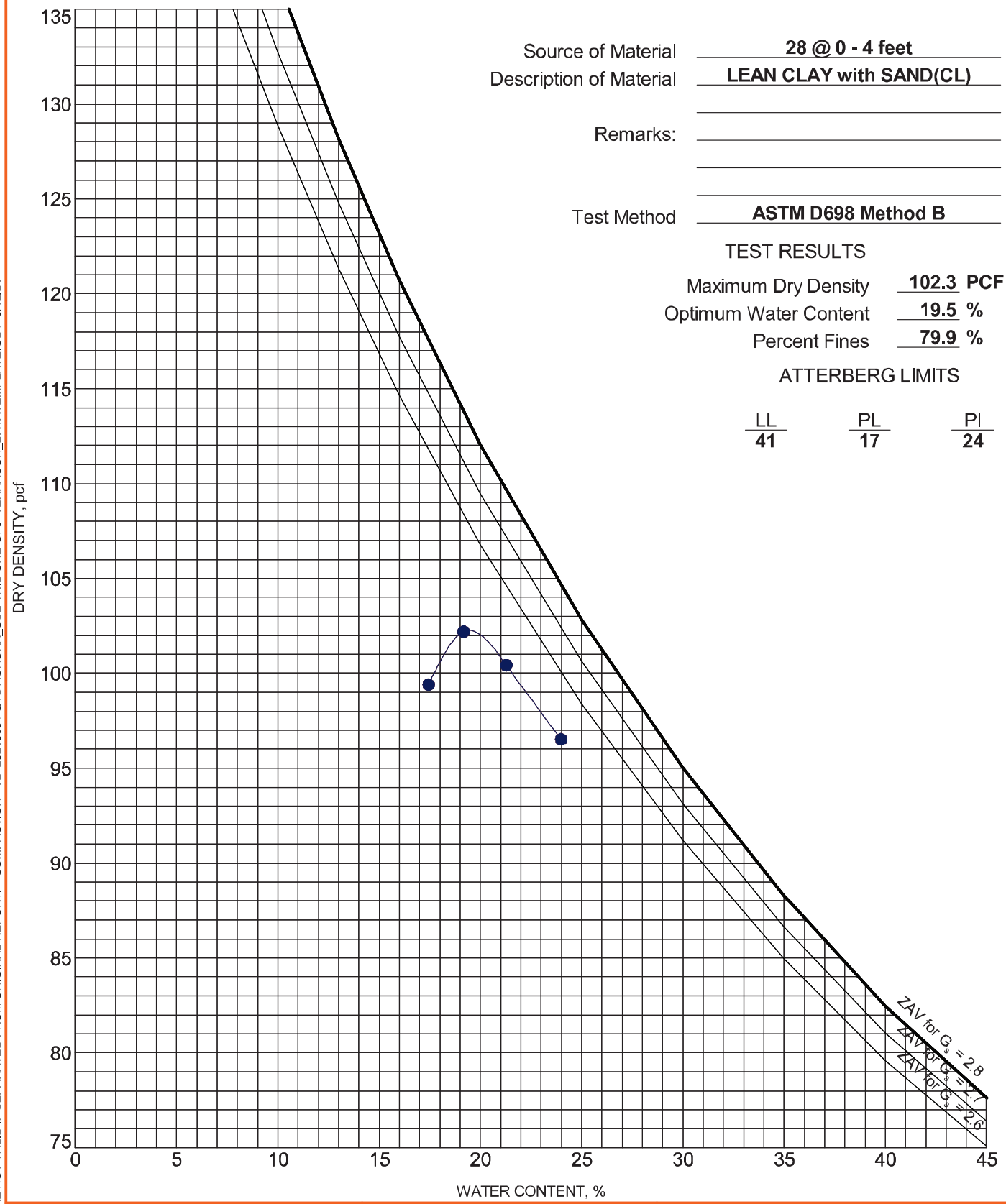
PROJECT NUMBER: 25215031

CLIENT: Quality Technology Services, Inc.

Permit #25-2535171-LT
RSN 19032422

MOISTURE-DENSITY RELATIONSHIP

ASTM D698/D1557



Source of Material	28 @ 0 - 4 feet
Description of Material	LEAN CLAY with SAND(CL)
Remarks:	
Test Method	ASTM D698 Method B

TEST RESULTS

Maximum Dry Density	102.3 PCF
Optimum Water Content	19.5 %
Percent Fines	79.9 %

ATTERBERG LIMITS

LL	PL	PI
41	17	24

PROJECT: QTS Aurora

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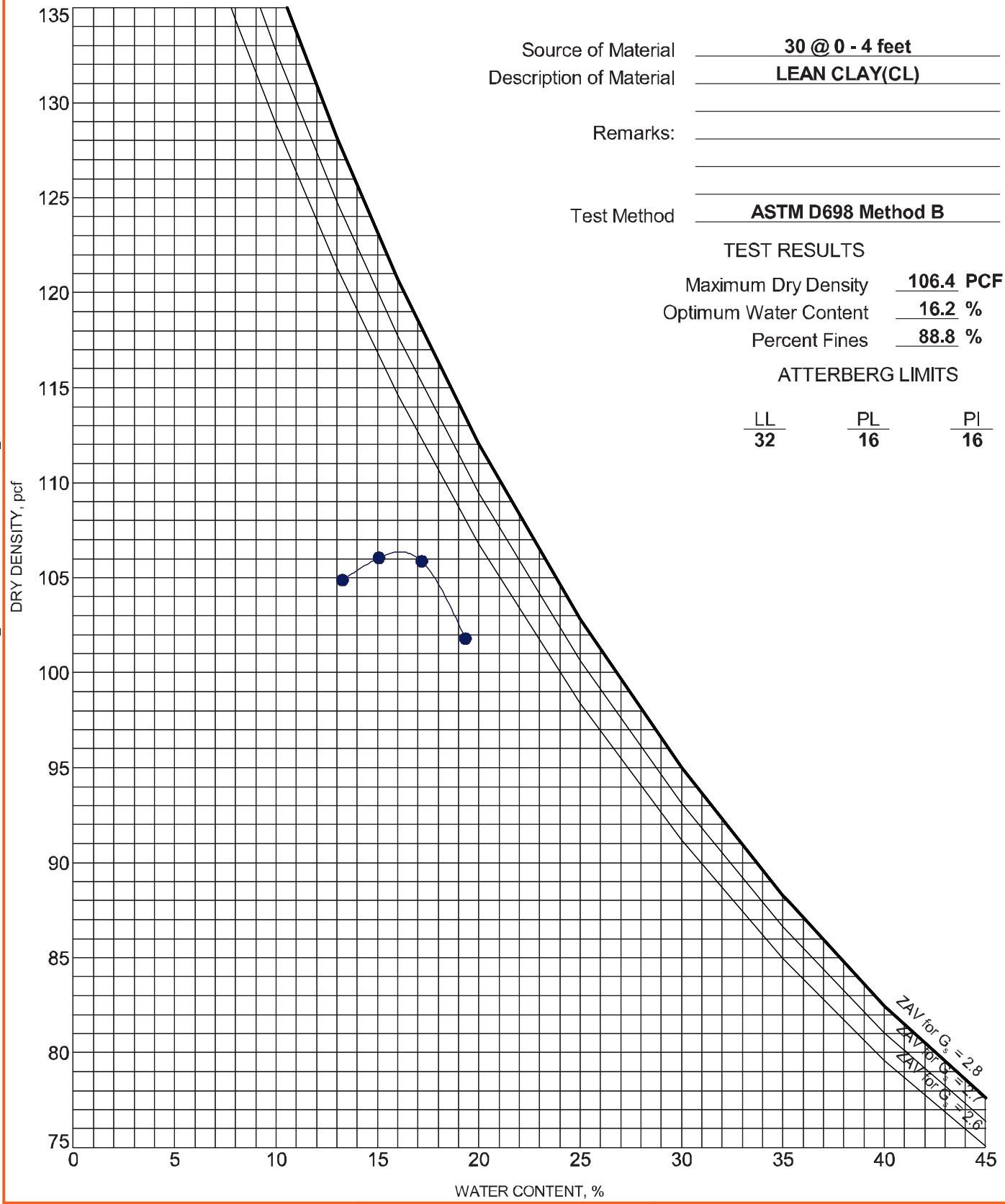
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Permit #25-2535171-LT
RSN 19032422

MOISTURE-DENSITY RELATIONSHIP

ASTM D698/D1557



Source of Material: **30 @ 0 - 4 feet**

Description of Material: **LEAN CLAY(CL)**

Remarks:

Test Method: **ASTM D698 Method B**

TEST RESULTS

Maximum Dry Density	106.4	PCF
Optimum Water Content	16.2	%
Percent Fines	88.8	%

ATTERBERG LIMITS

LL	PL	PI
32	16	16

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PROJECT: QTS Aurora

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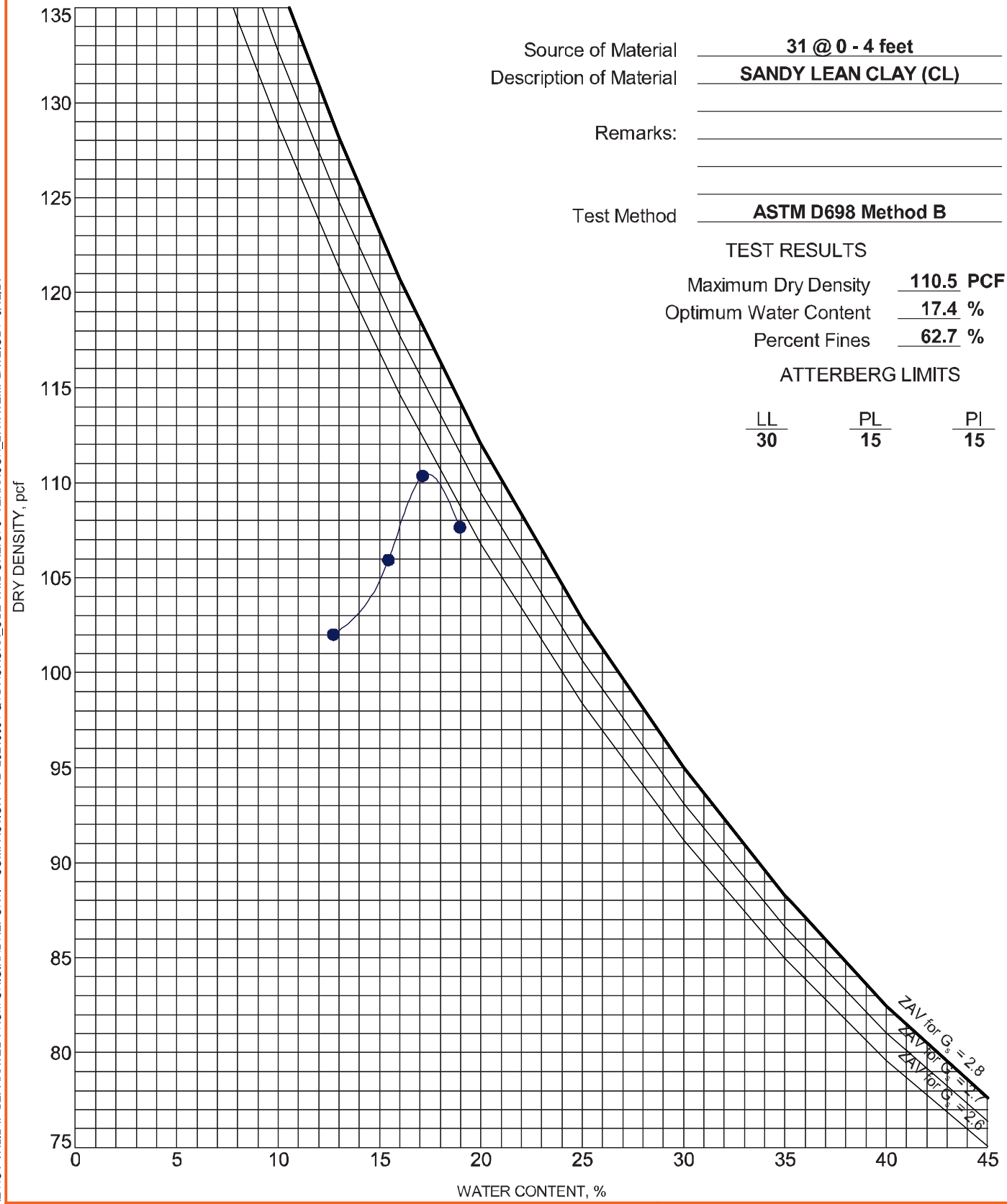
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MOISTURE-DENSITY RELATIONSHIP

ASTM D698/D1557



Source of Material: **31 @ 0 - 4 feet**

Description of Material: **SANDY LEAN CLAY (CL)**

Remarks:

Test Method: **ASTM D698 Method B**

TEST RESULTS

Maximum Dry Density	**110.5 PCF**
Optimum Water Content	**17.4 %**
Percent Fines	**62.7 %**

ATTERBERG LIMITS

LL	PL	PI
30	**15**	**15**

PROJECT: QTS Aurora

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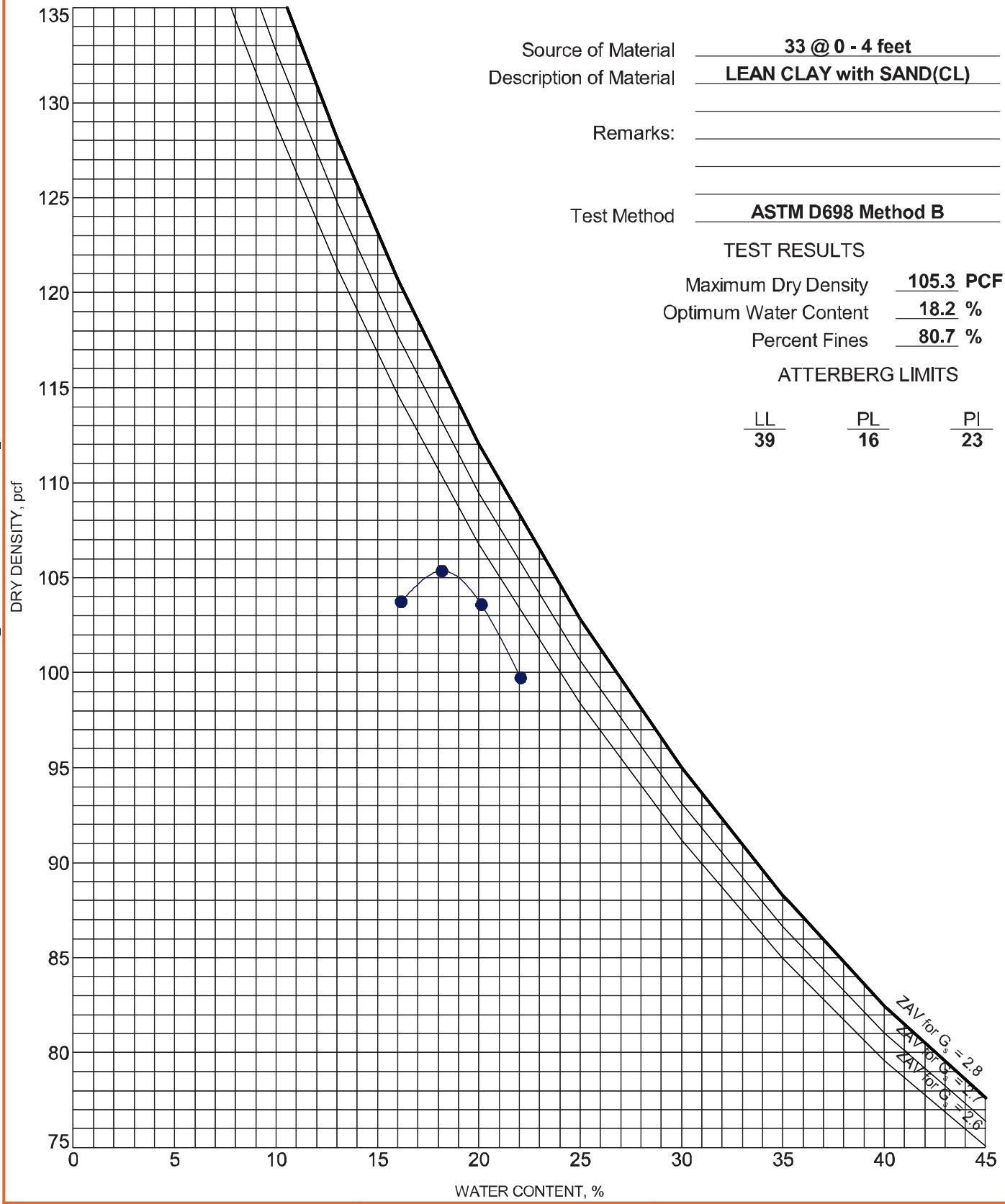
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RSN 19032422

MOISTURE-DENSITY RELATIONSHIP

ASTM D698/D1557



Source of Material	**33 @ 0 - 4 feet**
Description of Material	**LEAN CLAY with SAND(CL)**
Remarks:	
Test Method	**ASTM D698 Method B**

TEST RESULTS

Maximum Dry Density	**105.3**	**PCF**
Optimum Water Content	**18.2**	**%**
Percent Fines	**80.7**	**%**

ATTERBERG LIMITS

LL	PL	PI
39	**16**	**23**

PROJECT: QTS Aurora

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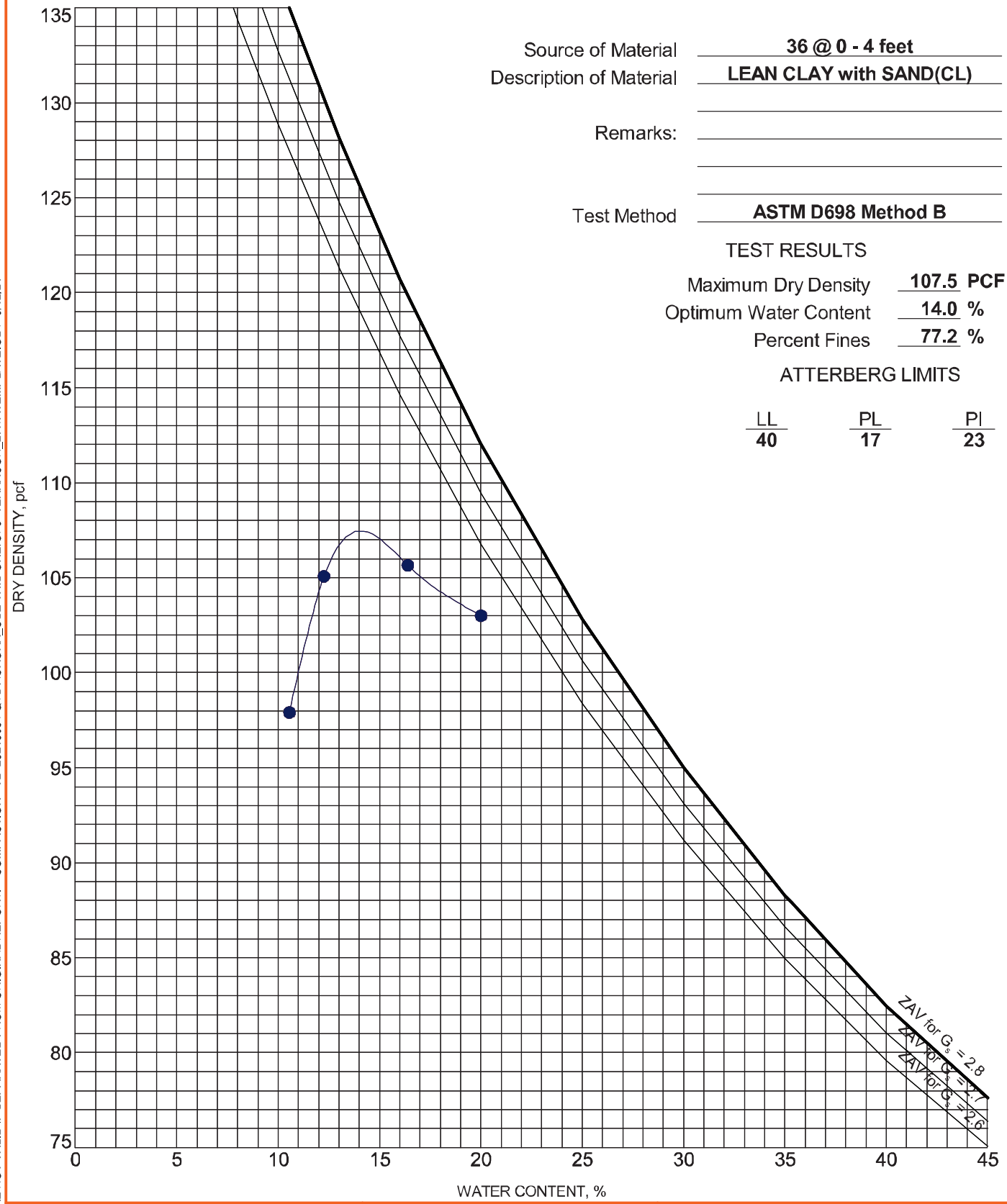
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Permit #25-2535171-LT
RSN 19032422

MOISTURE-DENSITY RELATIONSHIP

ASTM D698/D1557



Source of Material	**36 @ 0 - 4 feet**
Description of Material	**LEAN CLAY with SAND(CL)**
Remarks:	
Test Method	**ASTM D698 Method B**

TEST RESULTS

Maximum Dry Density	**107.5**	**PCF**
Optimum Water Content	**14.0**	**%**
Percent Fines	**77.2**	**%**

ATTERBERG LIMITS

LL	PL	PI
40	**17**	**23**

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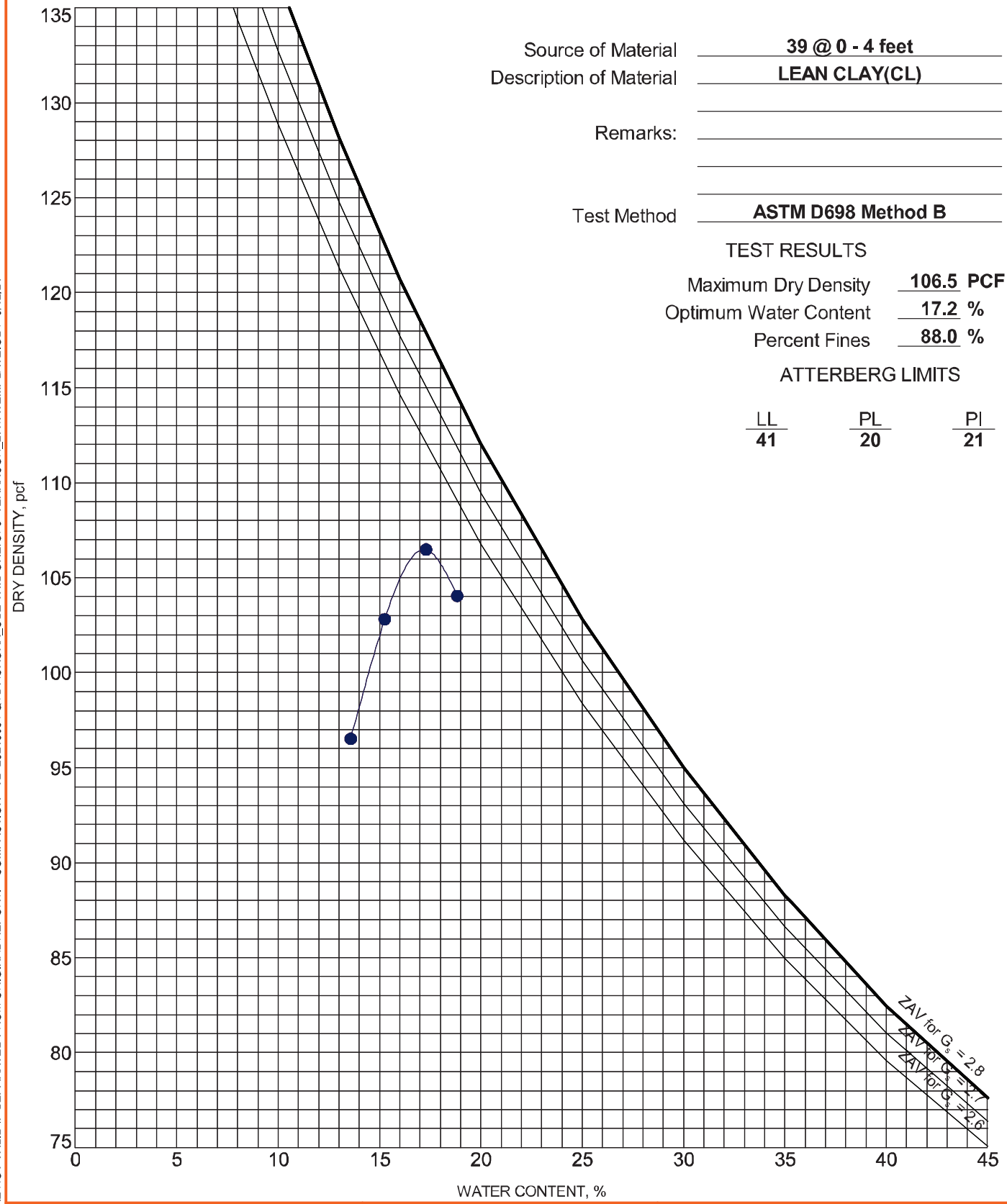
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Permit #25-2535171-LT
RSN 19032422

MOISTURE-DENSITY RELATIONSHIP

ASTM D698/D1557



Source of Material: **39 @ 0 - 4 feet**

Description of Material: **LEAN CLAY(CL)**

Remarks:

Test Method: **ASTM D698 Method B**

TEST RESULTS

Maximum Dry Density	106.5	PCF
Optimum Water Content	17.2	%
Percent Fines	88.0	%

ATTERBERG LIMITS

LL	PL	PI
41	20	21

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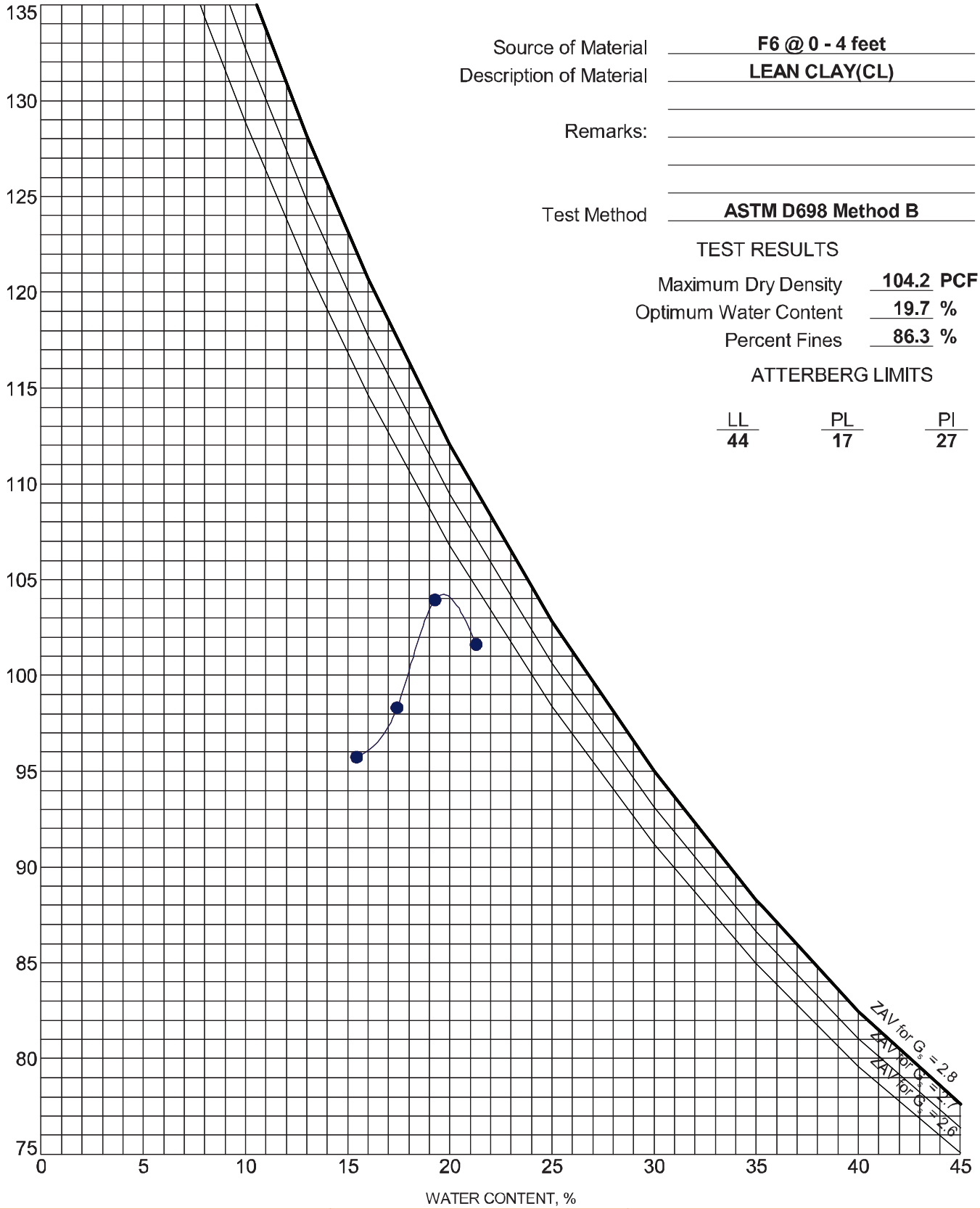
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MOISTURE-DENSITY RELATIONSHIP

ASTM D698/D1557



Source of Material: **F6 @ 0 - 4 feet**

Description of Material: **LEAN CLAY(CL)**

Remarks:

Test Method: **ASTM D698 Method B**

TEST RESULTS

Maximum Dry Density	104.2	PCF
Optimum Water Content	19.7	%
Percent Fines	86.3	%

ATTERBERG LIMITS

LL	PL	PI
44	17	27

PROJECT: QTS Aurora

PROJECT NUMBER: 25215031

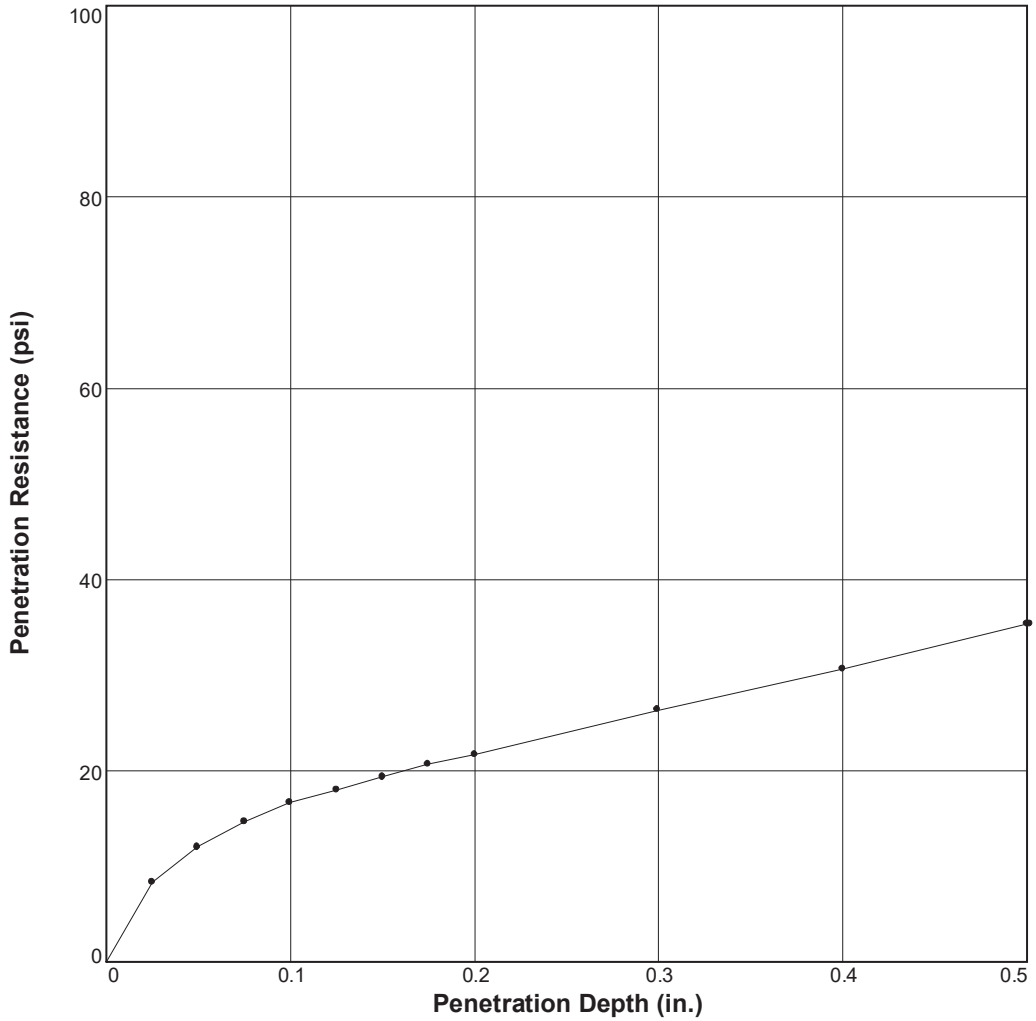
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BEARING RATIO TEST REPORT
ASTM D1883-16



	Molded			Soaked			CBR (%)		Linearity Correction (in.)	Surcharge (lbs.)	Max. Swell (%)
	Density (pcf)	Percent of Max. Dens.	Moisture (%)	Density (pcf)	Percent of Max. Dens.	Moisture (%)	0.10 in.	0.20 in.			
1 ○	97.5	95.3	19.2	95.0	92.9	23.6	1.7	1.4	0.000	10	2.6
2 △											
3 □											

Material Description	USCS	Max. Dens. (pcf)	Optimum Moisture (%)	LL	PI
Lean Clay with Sand	CL	102.3	19.5	44	24

Project No: 25215031
Project: QTS Aurora
Source of Sample: 28 **Depth:** 0-4'

Date: 7/19/2021

BEARING RATIO TEST REPORT

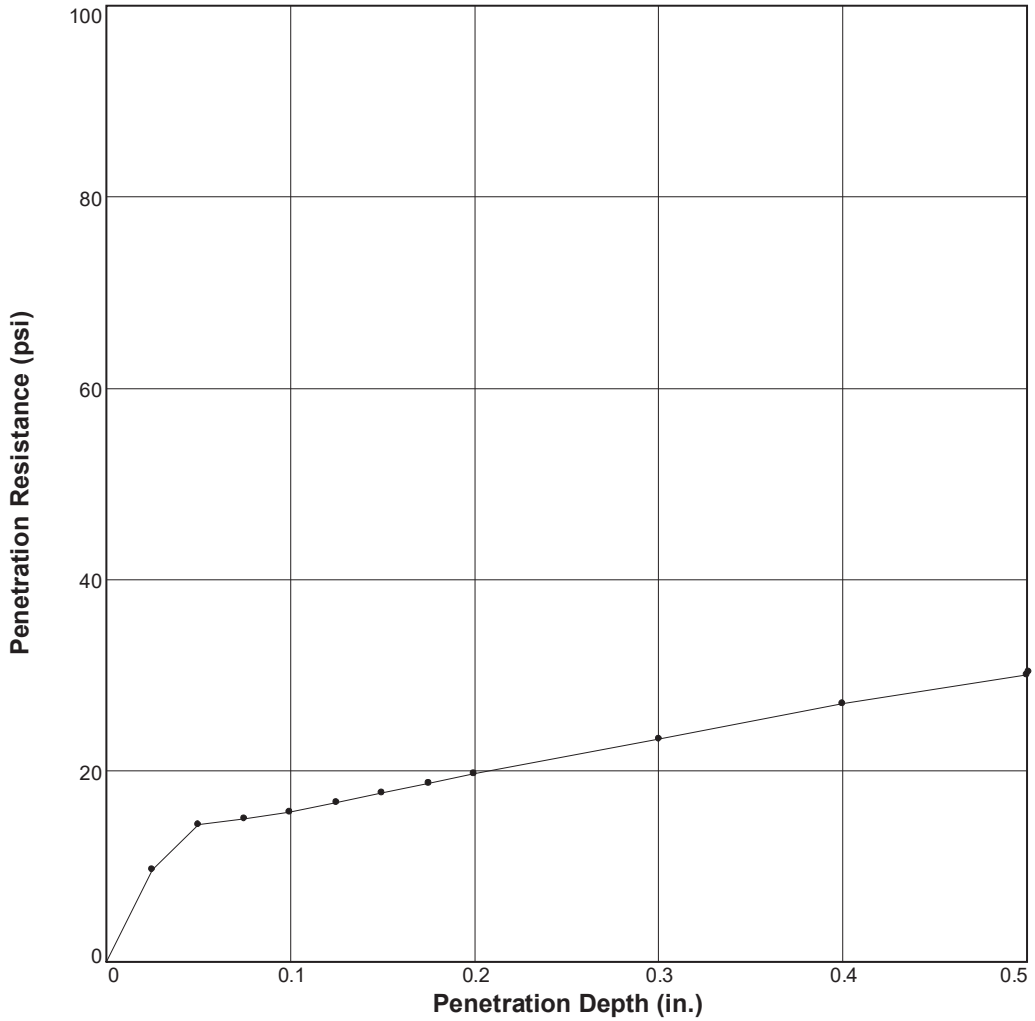
Terracon Consultants, Inc.

Test Description/Remarks:

Compaction based on D698 efforts.

Figure _____

BEARING RATIO TEST REPORT
ASTM D1883-16



	Molded			Soaked			CBR (%)		Linearity Correction (in.)	Surcharge (lbs.)	Max. Swell (%)
	Density (pcf)	Percent of Max. Dens.	Moisture (%)	Density (pcf)	Percent of Max. Dens.	Moisture (%)	0.10 in.	0.20 in.			
1 ○	99.3	95.3	19.4	96.1	92.3	23.6	1.6	1.3	0.000	10	3.3
2 △											
3 □											

Material Description	USCS	Max. Dens. (pcf)	Optimum Moisture (%)	LL	PI
Lean Clay	CL	104.2	19.7	44	27

Project No: 25215031
Project: QTS Aurora
Source of Sample: F6 **Depth:** 0-4'

Date: 7/19/2021

BEARING RATIO TEST REPORT

Terracon Consultants, Inc.

Test Description/Remarks:

Compaction based on D698 efforts.

Figure _____

21239 FM529 Rd., Bldg. F
Cypress, TX 77433
Tel: 281-985-9344
Fax: 832-427-1752
info@geothermusa.com
http://www.geothermusa.com

July 13, 2021

Terracon
10625 W. I-70 Frontage Rd N, Ste 3 5
Wheat Ridge, CO 80033
Attn: John T. Taylor

Re: Thermal Analysis of Native Soil Samples
QTS Aurora – Aurora, CO (Project No. 25215031)

The following is the report of thermal dryout characterization tests conducted on twelve (12) native soil samples from the referenced project sent to our laboratory.

Thermal Resistivity Tests: The samples were tested their 'optimum' moisture content and 95% of maximum dry density ***provided by Terracon***. The tests were conducted in accordance with the IEEE standard 442-2017. The results are tabulated below and the thermal dryout curves are presented in **Figures 1 to 12.**

Sample ID, Description, Thermal Resistivity, Moisture Content and Density

Sample ID	Depth (ft)	Description (Terracon)	Thermal Resistivity (°C-cm/W)		Moisture Content (%)	Dry Density (lb/ft^3)
			Wet	Dry		
06	0-4	Lean clay with sand (CL)	79	172	15	104
09	0-4	Lean clay (CL)	61	165	16	105
12	0-4	Lean clay with sand (CL)	79	174	17	100
15	0-4	Lean clay (CL)	76	162	17	103
18	0-4	Lean clay with sand (CL)	85	193	17	99
27	0-4	Lean clay (CL)	92	198	21	95
30	0-4	Lean clay (CL)	77	181	16	101
33	0-4	Lean clay with sand (CL)	75	173	18	100

Sample ID, Description, Thermal Resistivity, Moisture Content and Density

Sample ID	Depth (ft)	Description (Terracon)	Thermal Resistivity (°C-cm/W)		Moisture Content (%)	Dry Density (lb/ft^3)
			Wet	Dry		
36	0-4	Lean clay with sand (CL)	86	175	14	102
39	0-4	Lean clay (CL)	91	184	17	101
53	0-4	Lean clay with sand (CL)	84	174	16	102
56	0-4	Lean clay with sand (CL)	88	193	19	98

Comments: The thermal characteristic depicted in the dryout curves apply for the soils at their respective test dry density.

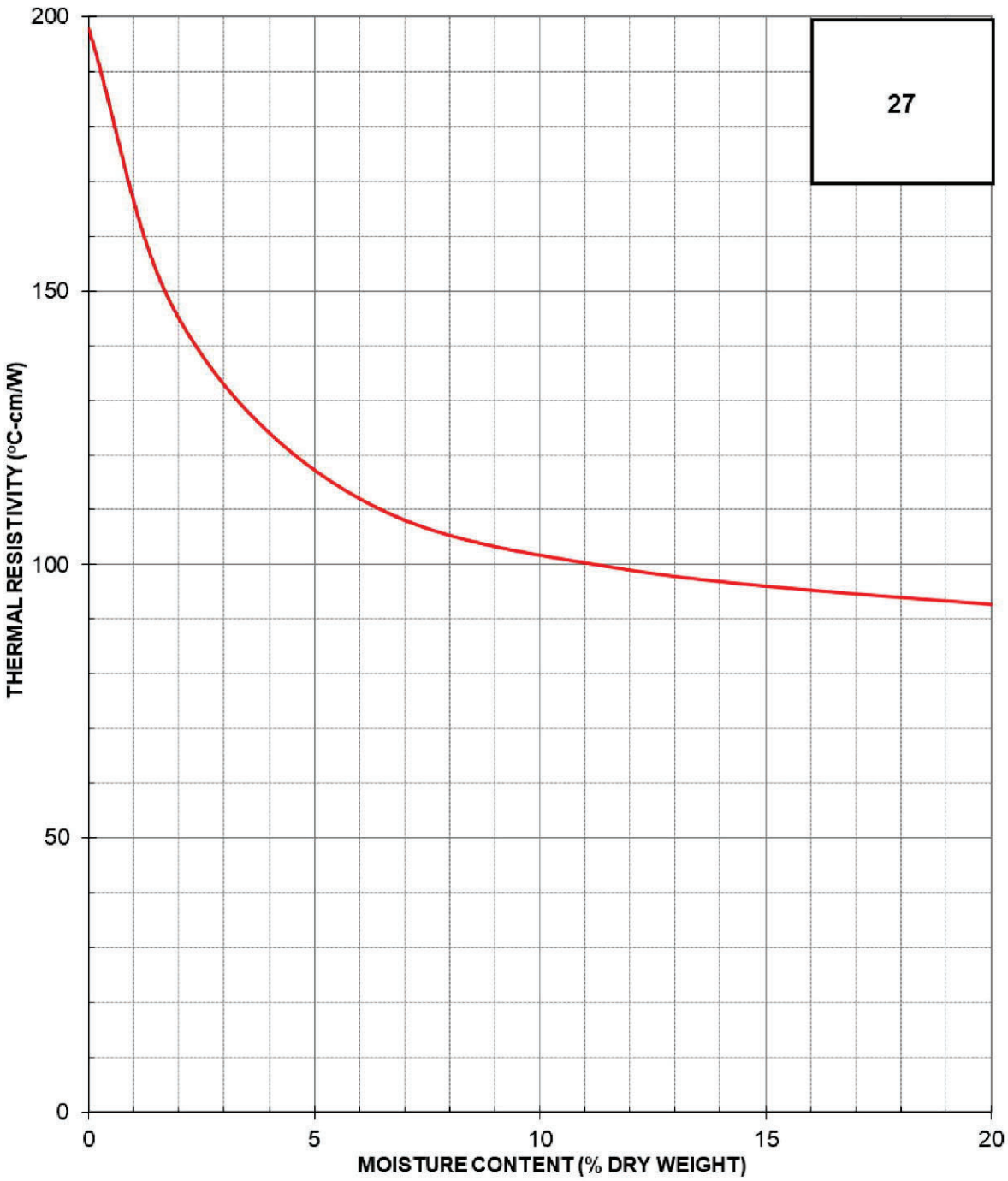
Please contact us if you have any questions or if we can be of further assistance.

Geotherm USA

Deepak Parmar

Geotherm USA

THERMAL DRYOUT CURVE



Terracon (Project No. 25215031)

Thermal Analysis of Native Soil Samples

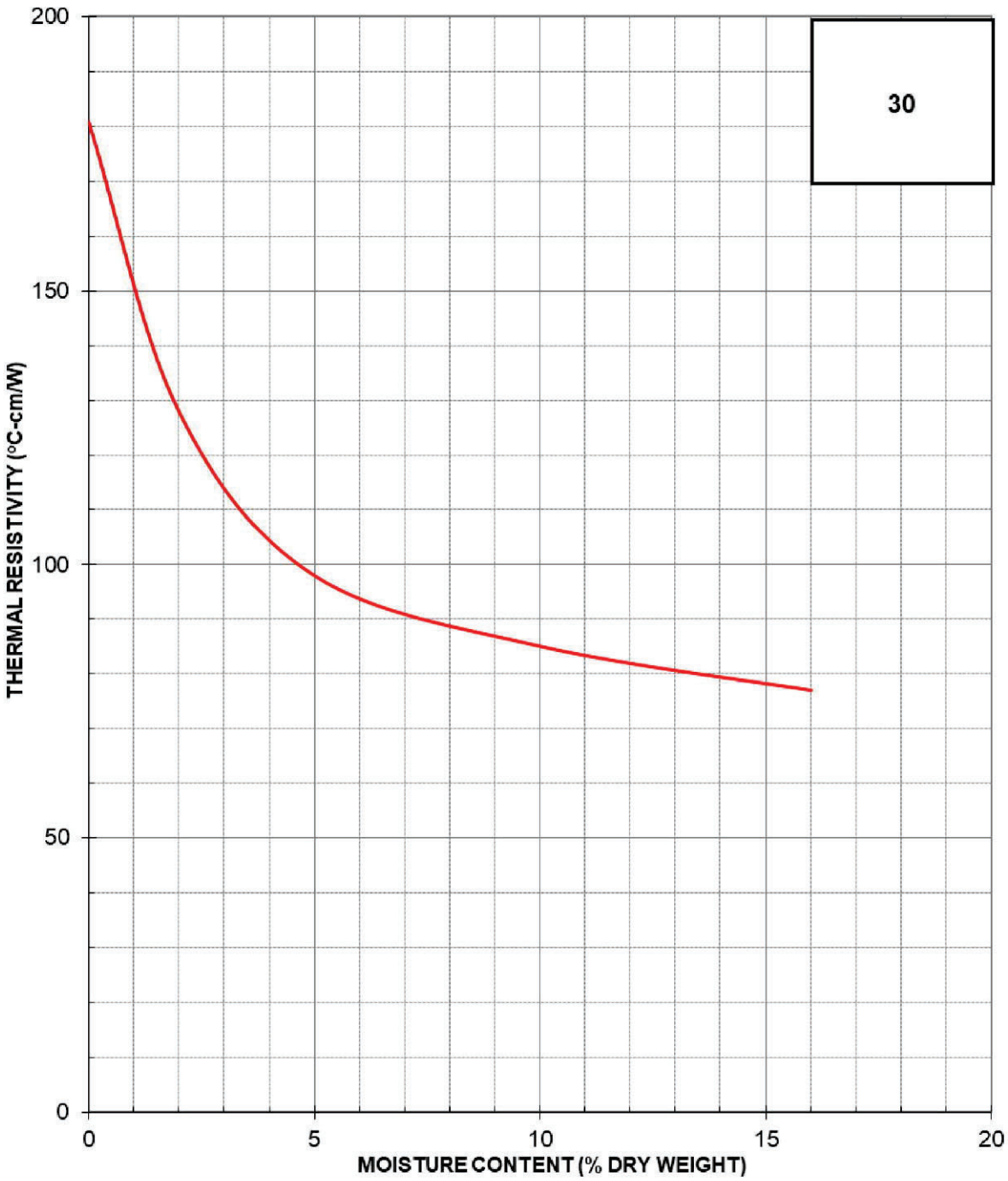
QTS Aurora – Aurora, CO

July 2021

Figure 6

Geotherm USA

THERMAL DRYOUT CURVE



Terracon (Project No. 25215031)

Thermal Analysis of Native Soil Samples

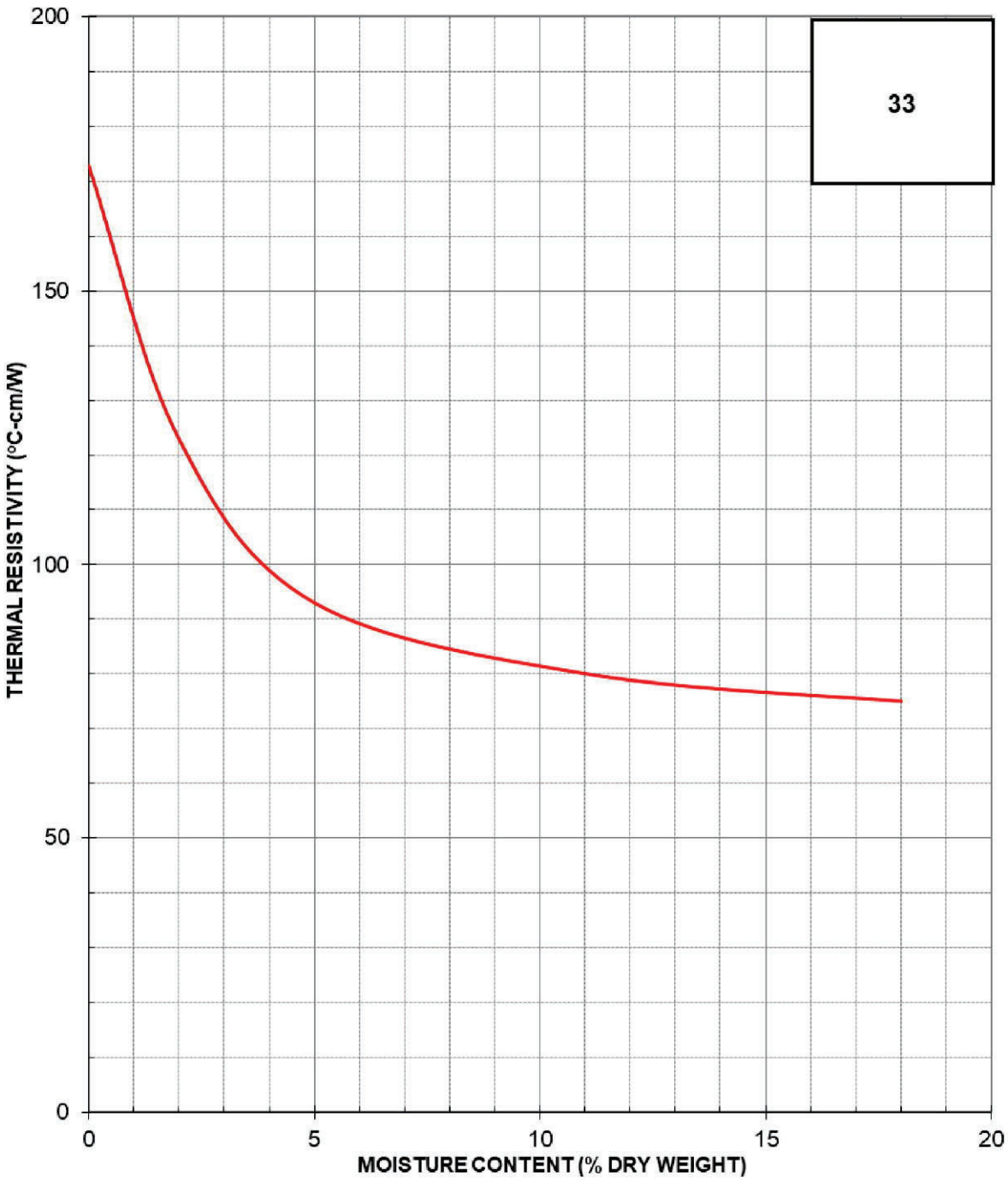
QTS Aurora – Aurora, CO

July 2021

Figure 7

Geotherm USA

THERMAL DRYOUT CURVE



Terracon (Project No. 25215031)

Thermal Analysis of Native Soil Samples

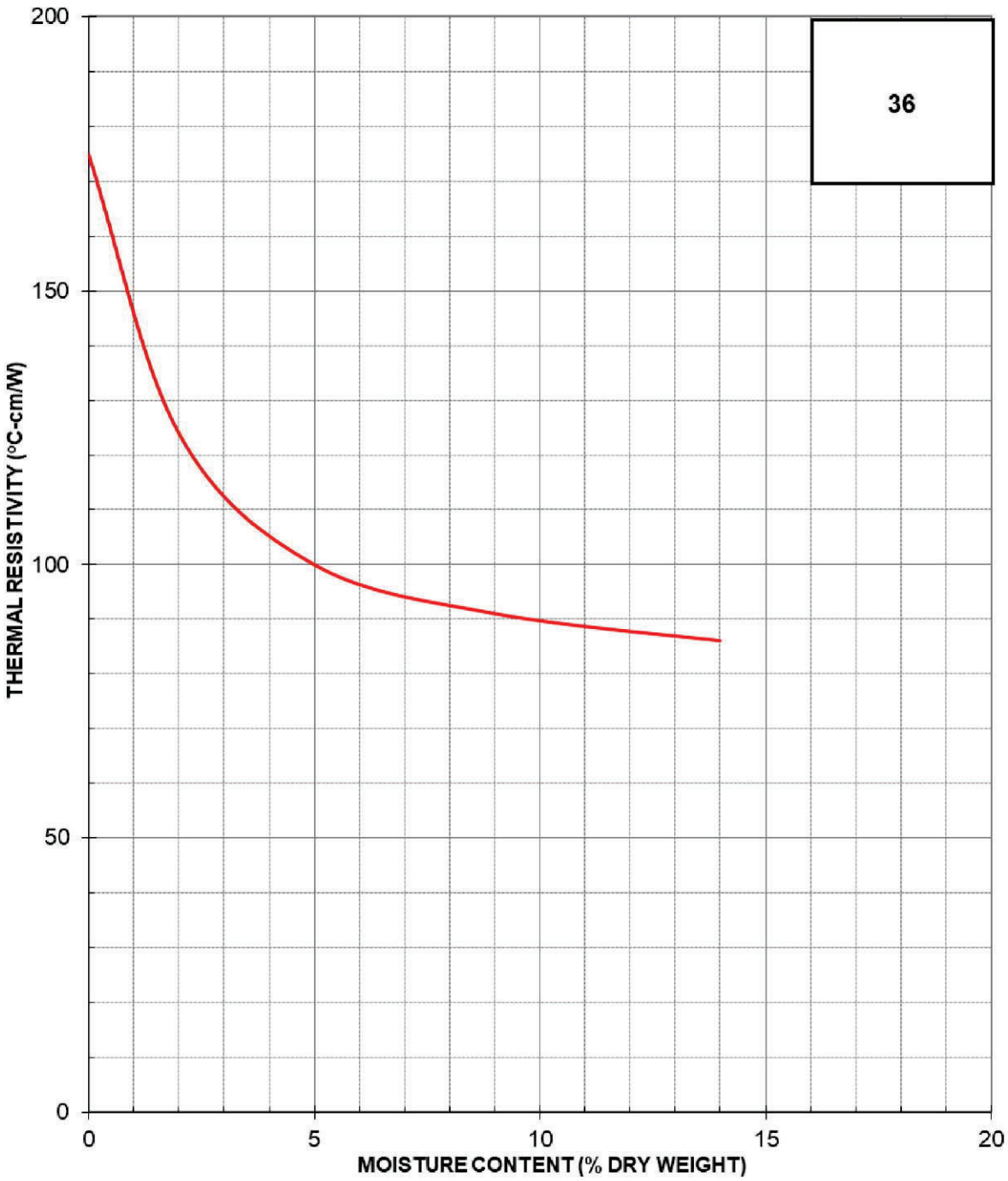
QTS Aurora – Aurora, CO

July 2021

Figure 8

Geotherm USA

THERMAL DRYOUT CURVE



Terracon (Project No. 25215031)

Thermal Analysis of Native Soil Samples

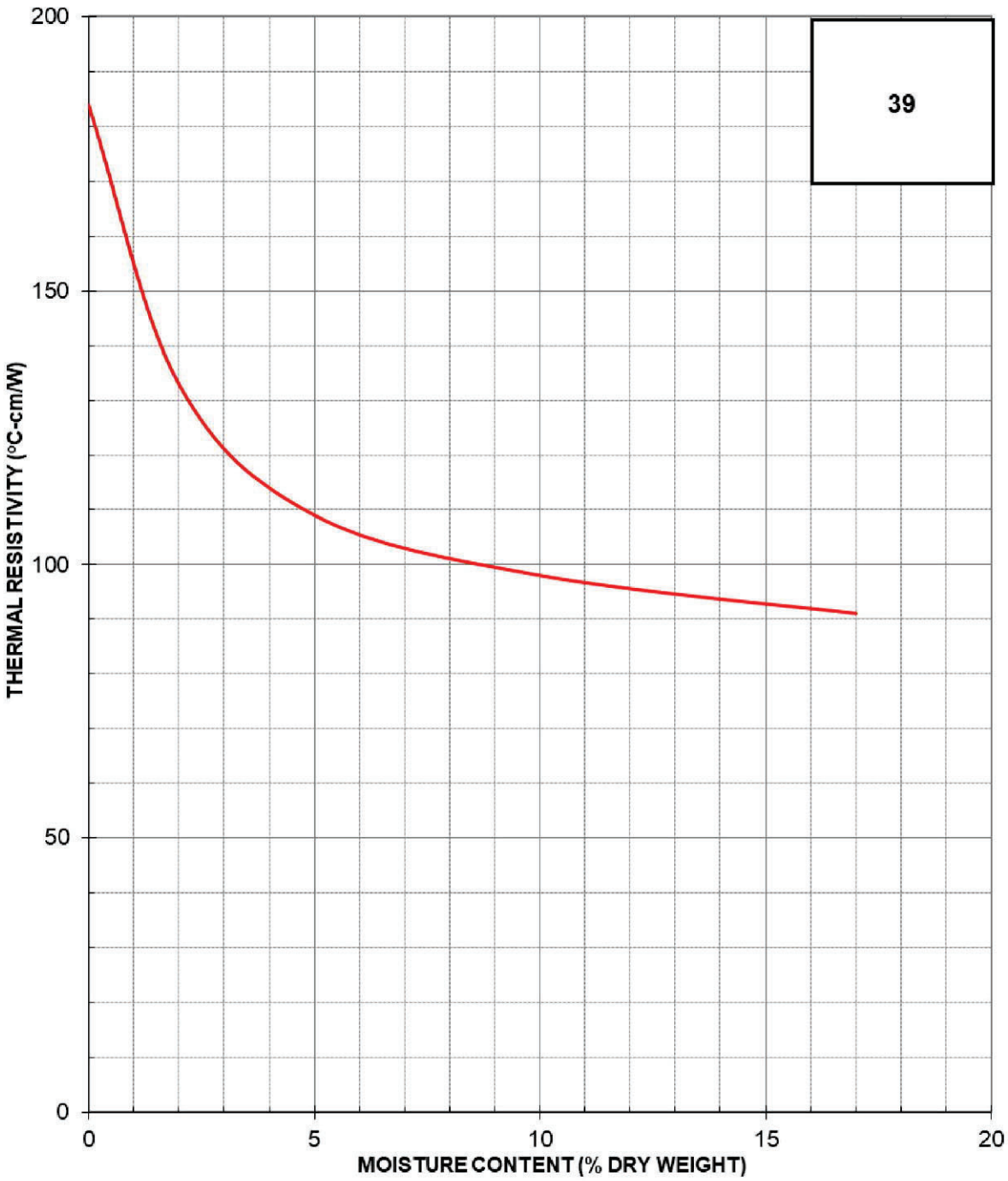
QTS Aurora – Aurora, CO

July 2021

Figure 9

Geotherm USA

THERMAL DRYOUT CURVE



Terracon (Project No. 25215031)

Thermal Analysis of Native Soil Samples

QTS Aurora – Aurora, CO

July 2021

Figure 10

Colorado Analytical LABORATORIES, INC.

Analytical Results

TASK NO: 210618111

Report To: John Taylor
Company: Terracon, Inc. - Wheat Ridge
10625 W. I-70 Frontage Road
Wheat Ridge CO 80033

Bill To: John Taylor
Company: Terracon, Inc. - Accounts Payable
18001 W. 106th St
Suite 300
Olathe KS 66061

Task No.: 210618111
Client PO:
Client Project: QTS Aurora 25215031

Date Received: 6/18/21
Date Reported: 6/25/21
Matrix: Soil - Geotech

Customer Sample ID 21@0-4ft
Lab Number: 210618111-07

Test	Result	Method
Chloride - Water Soluble	**0.0050 %**	AASHTO T291-91/ ASTM D4327
pH	**7.2 units**	AASHTO T289-91
Redox Potential	**348.3 mv**	ASTM D1498
Resistivity	**664 ohm.cm**	AASHTO T288-91
Sulfate - Water Soluble	**0.124 %**	AASHTO T290-91/ ASTM D4327

Abbreviations/ References:

AASHTO - American Association of State Highway and Transportation Officials.
ASTM - American Society for Testing and Materials.
ASA - American Society of Agronomy.
DIPRA - Ductile Iron Pipe Research Association Handbook of Ductile Iron Pipe.

DATA APPROVED FOR RELEASE BY

Colorado Analytical LABORATORIES, INC.

Analytical Results

TASK NO: 210630112

Report To: John Taylor
Company: Terracon, Inc. - Wheat Ridge
10625 W. I-70 Frontage Road
Wheat Ridge CO 80033

Bill To: John Taylor
Company: Terracon, Inc. - Accounts Payable
18001 W. 106th St
Suite 300
Olathe KS 66061

Task No.: 210630112
Client PO:
Client Project: QTS Aurora 25215031

Date Received: 6/30/21
Date Reported: 7/8/21
Matrix: Soil - Geotech

Customer Sample ID 22 @ 0-5
Lab Number: 210630112-01

Test	Result	Method
pH	**7.7 units**	AASHTO T289-91
Redox Potential	**463.4 mv**	ASTM D1498
Resistivity	**726 ohm.cm**	AASHTO T288-91
Sulfate - Water Soluble	**0.038 %**	AASHTO T290-91/ ASTM D4327

Customer Sample ID 24 @ 0-5
Lab Number: 210630112-02

Test	Result	Method
pH	**6.8 units**	AASHTO T289-91
Redox Potential	**461.0 mv**	ASTM D1498
Resistivity	**1532 ohm.cm**	AASHTO T288-91
Sulfate - Water Soluble	**< 0.001 %**	AASHTO T290-91/ ASTM D4327

Customer Sample ID 26 @ 0-5
Lab Number: 210630112-03

Test	Result	Method
pH	**6.6 units**	AASHTO T289-91
Redox Potential	**443.3 mv**	ASTM D1498
Resistivity	**521 ohm.cm**	AASHTO T288-91
Sulfate - Water Soluble	**0.133 %**	AASHTO T290-91/ ASTM D4327

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ASA - American Society of Agronomy.
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Analytical Results

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10625 W. I-70 Frontage Road
Wheat Ridge CO 80033

Bill To: John Taylor
Company: Terracon, Inc. - Accounts Payable
18001 W. 106th St
Suite 300
Olathe KS 66061

Task No.: 210630112
Client PO:
Client Project: QTS Aurora 25215031

Date Received: 6/30/21
Date Reported: 7/8/21
Matrix: Soil - Geotech

Customer Sample ID 32 @ 0-5
Lab Number: 210630112-04

Test	Result	Method
pH	**7.0 units**	AASHTO T289-91
Redox Potential	**428.6 mv**	ASTM D1498
Resistivity	**475 ohm.cm**	AASHTO T288-91
Sulfate - Water Soluble	**0.324 %**	AASHTO T290-91/ ASTM D4327

Abbreviations/ References:

AASHTO - American Association of State Highway and Transportation Officials.
ASTM - American Society for Testing and Materials.
ASA - American Society of Agronomy.
DIPRA - Ductile Iron Pipe Research Association Handbook of Ductile Iron Pipe.

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Colorado Analytical LABORATORIES, INC.

Analytical Results

TASK NO: 210721171

Report To: John Taylor
Company: Terracon, Inc. - Wheat Ridge
10625 W. I-70 Frontage Road
Wheat Ridge CO 80033

Bill To: John Taylor
Company: Terracon, Inc. - Accounts Payable
18001 W. 106th St
Suite 300
Olathe KS 66061

Task No.: 210721171
Client PO:
Client Project: QTS Aurora 25215031

Date Received: 7/21/21
Date Reported: 7/28/21
Matrix: Soil - Geotech

Customer Sample ID B34 @ 0-5 Ft
Lab Number: 210721171-01

Test	Result	Method
pH	**7.5 units**	AASHTO T289-91
Redox Potential	**425.8 mv**	ASTM D1498
Resistivity	**605 ohm.cm**	AASHTO T288-91
Sulfate - Water Soluble	**0.484 %**	AASHTO T290-91/ ASTM D4327

Customer Sample ID B38 @ 0-5 Ft
Lab Number: 210721171-02

Test	Result	Method
pH	**7.5 units**	AASHTO T289-91
Redox Potential	**381.8 mv**	ASTM D1498
Resistivity	**650 ohm.cm**	AASHTO T288-91
Sulfate - Water Soluble	**0.169 %**	AASHTO T290-91/ ASTM D4327

Customer Sample ID B40 @ 0-5 Ft
Lab Number: 210721171-03

Test	Result	Method
pH	**7.6 units**	AASHTO T289-91
Redox Potential	**391.5 mv**	ASTM D1498
Resistivity	**720 ohm.cm**	AASHTO T288-91
Sulfate - Water Soluble	**0.145 %**	AASHTO T290-91/ ASTM D4327

Abbreviations/ References:

AASHTO - American Association of State Highway and Transportation Officials.
ASTM - American Society for Testing and Materials.
ASA - American Society of Agronomy.
DIPRA - Ductile Iron Pipe Research Association Handbook of Ductile Iron Pipe.

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Colorado Analytical LABORATORIES, INC.

Analytical Results

TASK NO: 210721171

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10625 W. I-70 Frontage Road
Wheat Ridge CO 80033

Bill To: John Taylor
Company: Terracon, Inc. - Accounts Payable
18001 W. 106th St
Suite 300
Olathe KS 66061

Task No.: 210721171
Client PO:
Client Project: QTS Aurora 25215031

Date Received: 7/21/21
Date Reported: 7/28/21
Matrix: Soil - Geotech

Customer Sample ID B42 @ 0-5 Ft
Lab Number: 210721171-04

Test	Result	Method
pH	**7.6 units**	AASHTO T289-91
Redox Potential	**366.7 mv**	ASTM D1498
Resistivity	**1186 ohm.cm**	AASHTO T288-91
Sulfate - Water Soluble	**0.004 %**	AASHTO T290-91/ ASTM D4327

Abbreviations/ References:

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ASTM - American Society for Testing and Materials.
ASA - American Society of Agronomy.
DIPRA - Ductile Iron Pipe Research Association Handbook of Ductile Iron Pipe.

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Colorado Analytical LABORATORIES, INC.

Analytical Results

TASK NO: 210630112

Report To: John Taylor
Company: Terracon, Inc. - Wheat Ridge
10625 W. I-70 Frontage Road
Wheat Ridge CO 80033

Bill To: John Taylor
Company: Terracon, Inc. - Accounts Payable
18001 W. 106th St
Suite 300
Olathe KS 66061

Task No.: 210630112
Client PO:
Client Project: QTS Aurora 25215031

Date Received: 6/30/21
Date Reported: 8/18/21
Matrix: Soil - Geotech

Customer Sample ID 22 @ 0-5
Lab Number: 210630112-01

Test	Result	Method
Chloride - Water Soluble	**0.0004 %**	AASHTO T291-91/ ASTM D4327

Customer Sample ID 24 @ 0-5
Lab Number: 210630112-02

Test	Result	Method
Chloride - Water Soluble	**0.0003 %**	AASHTO T291-91/ ASTM D4327

Customer Sample ID 26 @ 0-5
Lab Number: 210630112-03

Test	Result	Method
Chloride - Water Soluble	**0.0027 %**	AASHTO T291-91/ ASTM D4327

Customer Sample ID 32 @ 0-5
Lab Number: 210630112-04

Test	Result	Method
Chloride - Water Soluble	**0.0129 %**	AASHTO T291-91/ ASTM D4327

Abbreviations/ References:

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ASTM - American Society for Testing and Materials.
ASA - American Society of Agronomy.
DIPRA - Ductile Iron Pipe Research Association Handbook of Ductile Iron Pipe.

DATA APPROVED FOR RELEASE BY

Colorado Analytical LABORATORIES, INC.

Analytical Results

TASK NO: 210721171

Report To: John Taylor
Company: Terracon, Inc. - Wheat Ridge
10625 W. I-70 Frontage Road
Wheat Ridge CO 80033

Bill To: John Taylor
Company: Terracon, Inc. - Accounts Payable
18001 W. 106th St
Suite 300
Olathe KS 66061

Task No.: 210721171
Client PO:
Client Project: QTS Aurora 25215031

Date Received: 7/21/21
Date Reported: 8/18/21
Matrix: Soil - Geotech

Customer Sample ID B34 @ 0-5 Ft
Lab Number: 210721171-01

Test	Result	Method
Chloride - Water Soluble	**0.0052 %**	AASHTO T291-91/ ASTM D4327

Customer Sample ID B38 @ 0-5 Ft
Lab Number: 210721171-02

Test	Result	Method
Chloride - Water Soluble	**0.0067 %**	AASHTO T291-91/ ASTM D4327

Customer Sample ID B40 @ 0-5 Ft
Lab Number: 210721171-03

Test	Result	Method
Chloride - Water Soluble	**0.0008 %**	AASHTO T291-91/ ASTM D4327

Customer Sample ID B42 @ 0-5 Ft
Lab Number: 210721171-04

Test	Result	Method
Chloride - Water Soluble	**0.0006 %**	AASHTO T291-91/ ASTM D4327

Abbreviations/ References:

AASHTO - American Association of State Highway and Transportation Officials.
ASTM - American Society for Testing and Materials.
ASA - American Society of Agronomy.
DIPRA - Ductile Iron Pipe Research Association Handbook of Ductile Iron Pipe.

DATA APPROVED FOR RELEASE BY

SUMMARY OF LABORATORY TEST RESULTS

QTS Aurora - Aurora, Colorado
Terracon Project No. 25215013

Boring No.	Depth (ft)	USCS Class.	Initial Water Content (%)	Initial Dry Density (pcf)	Swell/Consolidation		Moisture-Density Relationship			Particle Size Distribution, Percent Passing by Weight				Atterberg Limits		Unconfined Compressive Strength (psf)	Water Soluble Sulfates (%)	Water Soluble Chlorides (%)	pH	Redox (mv)	Resistivity (ohm.cm)	Remarks
					Surcharge (ksf)	Swell (%)	Maximum Dry Density (pcf)	Optimum Water Content (%)	Test Method	3/4"	#4	#40	#200	LL	PI							
19	12.5		21.4	105																		4
19	29		27.4	92																		4
19	39		24.5	101																		4
20	0 - 4	CL								100	100	95	83	29	14							
20	5		11.7	122																		4
20	7.5		12.3	123	0.5	+9.2																3,4
20	12.5		5.0																			
20	19		18.0																			
20	29		26.5	96																		4
21	0 - 4	CL								100	100	96	85	33	16		0.124	0.0050	7.2	348.3	664	
21	2.5	CL	14.8	109																		4
21	7.5		19.7	111	0.5	+9.7																3,4
21	12.5		12.9	118																		4
21	24		21.8	104												23614						4
21	34		21.5																			
22	0 - 4	CL								100	100	92	65	37	21		0.038	0.0004	7.7	463.4	726	
22	5	CL	22.9	102																		4
22	7.5		23.7	102	0.5	+4.3																3,4
22	12.5		25.2																			4
22	15		16.8	112																		4
22	19		26.0	98												16050						4
22	29		23.4	103																		4
22	39		28.9	92																		4
23	2.5	CL	15.2	115																		4
23	5	CL	15.7	114	0.5	+1.1																3,4
23	7.5	CL	20.6	108																		4
23	12.5		22.8	103																		4
23	19		24.9	101																		4
23	29		23.6	103																		4
24	0 - 4	CL								100	100	91	66	37	21		<0.001	0.0003	6.8	461.0	1532	
24	7.5	CL	16.3	112																		4
24	15		22.3	106																		4
24	24		21.1	106																		4
24	34		23.1	104																		4

Notes:
Initial Dry Density and Initial Water Content are in-situ values unless otherwise noted.
* = Partially disturbed sample
- = Compression/settlement
NV = no value
NP = non-plastic

Remarks:
1 Remolded Compacted density (about 95% of ASTM D698 maximum density near optimum moisture content)
2 Remolded Compacted density (about 95% of ASTM D1557 maximum density near optimum moisture content)
3 Water added to sample
4 Dry density and/or moisture content determined from one ring of a multi-ring sample
5 Minus #200 Only
6 Moisture-Density Relationship Test Method ASTM D698/AASHTO T99
7 Moisture-Density Relationship Test Method ASTM D1557/AASHTO T180

Terracon GeoReport

Permit #25-2535171-LT
RSN 1903242

SUMMARY OF LABORATORY TEST RESULTS

QTS Aurora - Aurora, Colorado
Terracon Project No. 25215013

Boring No.	Depth (ft)	USCS Class.	Initial Water Content (%)	Initial Dry Density (pcf)	Swell/Consolidation		Moisture-Density Relationship			Particle Size Distribution, Percent Passing by Weight				Atterberg Limits		Unconfined Compressive Strength (psf)	Water Soluble Sulfates (%)	Water Soluble Chlorides (%)	pH	Redox (mv)	Resistivity (ohm.cm)	Remarks
					Surcharge (ksf)	Swell (%)	Maximum Dry Density (pcf)	Optimum Water Content (%)	Test Method	3/4"	#4	#40	#200	LL	PI							
25	0 - 4	CL								100	100	95	75	38	21							
25	5	CL	20.3	106																		4
25	10		19.1	117																		4
25	15		21.7	109																		4
25	24		26.1	100																		4
25	34		21.9	111																		4
26	0 - 4	CL								100	100	94	82	36	20		0.133	0.0027	6.6	443.3	521	
26	2.5	CL	19.4	109																		4
26	7.5		19.9	108	0.5	+10.7																3,4
26	12.5		7.0																			4
26	19		20.4	108																		4
26	29		18.3	110																		4
26	39		21.8	106																		4
27	0 - 4	CL					100.0	21.2	D698B	100	100	98	91	46	30							
27	2.5	CL	18.4	112																		4
27	5	CL	14.1	117																		4
27	7.5	CL	18.7	110	0.5	+1.7																3,4
27	10		21.4	107																		4
27	12.5		21.4	106	0.5	+3.0																3,4
27	15		22.1	105																		4
27	19		21.8	108																		4
27	24		19.2	111																		4
27	29		20.4	107																		4
27	34		22.7	103																		6
28	0 - 4	CL					102.3	19.5	D698B	100	100	93	80	41	24							
28	5	CL	18.2	106	0.5	+4.2																3,4
28	10		18.4	110																		4
28	15		18.6																			4
28	24		18.8	109												39860						4
28	34		18.7	111																		4
29	5		16.4	115	0.5	+7.3																3,4
29	12.5		9.5																			4
29	19		20.5	105																		4
29	29		17.8	110																		4

Notes:
Initial Dry Density and Initial Water Content are in-situ values unless otherwise noted.
* = Partially disturbed sample
- = Compression/settlement
NV = no value
NP = non-plastic

Remarks:
1 Remolded Compacted density (about 95% of ASTM D698 maximum density near optimum moisture content)
2 Remolded Compacted density (about 95% of ASTM D1557 maximum density near optimum moisture content)
3 Water added to sample
4 Dry density and/or moisture content determined from one ring of a multi-ring sample
5 Minus #200 Only
6 Moisture-Density Relationship Test Method ASTM D698/AASHTO T99
7 Moisture-Density Relationship Test Method ASTM D1557/AASHTO T180

Terracon GeoReport

Permit #25-2535171-LT
RSN 1903242

SUMMARY OF LABORATORY TEST RESULTS

QTS Aurora - Aurora, Colorado
Terracon Project No. 25215013

Boring No.	Depth (ft)	USCS Class.	Initial Water Content (%)	Initial Dry Density (pcf)	Swell/Consolidation		Moisture-Density Relationship			Particle Size Distribution, Percent Passing by Weight				Atterberg Limits		Unconfined Compressive Strength (psf)	Water Soluble Sulfates (%)	Water Soluble Chlorides (%)	pH	Redox (mv)	Resistivity (ohm.cm)	Remarks
					Surcharge (ksf)	Swell (%)	Maximum Dry Density (pcf)	Optimum Water Content (%)	Test Method	3/4"	#4	#40	#200	LL	PI							
30	0 - 4	CL					106.4	16.2	D698B	100	100	97	89	32	16							6
30	5	CL	17.5	114	0.5	+2.1																3,4
30	10		18.6	108																		4
30	15		18.7	112																		4
30	24		20.9	106																		4
30	34		19.9	108																		4
31	0 - 4	CL	17.2	104	0.5	0.0	110.5	17.4	D698B	100	100	90	63	30	15							1,3,6
31	0 - 4	CL	18.8	105	0.5	-0.1	110.5	17.4	D698B	100	100	90	63	30	15							1,3,6
31	0 - 4	CL	20.9	106	0.5	-0.5	110.5	17.4	D698B	100	100	90	63	30	15							1,3,6
31	2.5	CL	16.1	114																		4
31	7.5		20.6	106	0.5	+5.9																3,4
31	12.5		23.6	103																		4
31	19		24.5	102																		4
32	0 - 4	CL								100	100	91	75	32	18		0.324	0.0129	7.0	428.6	475	
32	5		18.5	108	0.5	+7.1																3,4
32	10		14.8	119																		4
32	15		17.1	113																		4
32	24		20.7																			4
32	34		20.1	108																		4
33	0 - 4	CL					105.3	18.2	D698B	100	100	95	81	39	23							6
33	2.5	CL	20.4	107																		4
33	7.5	CL	19.8	106	0.5	+2.7																3,4
33	12.5		22.0	105																		4
33	19		21.8	106																		4
33	29		9.2	131																		4
33	39		17.4																			4
34	0 - 4	CL								100	100	94	77	36	21		0.484	0.0052	7.5	425.8	605	
34	5	CL	19.7	108	0.5	+2.3																3,4
34	10		22.7	105																		4
34	15		16.8	115																		4
34	24		16.7	111																		4
34	34		21.6	107																		4
35	0 - 4	CL								100	100	97	87	40	22							
35	2.5	CL	16.6	113																		4

Notes:
Initial Dry Density and Initial Water Content are in-situ values unless otherwise noted.
* = Partially disturbed sample
- = Compression/settlement
NV = no value
NP = non-plastic

Remarks:
1 Remolded Compacted density (about 95% of ASTM D698 maximum density near optimum moisture content)
2 Remolded Compacted density (about 95% of ASTM D1557 maximum density near optimum moisture content)
3 Water added to sample
4 Dry density and/or moisture content determined from one ring of a multi-ring sample
5 Minus #200 Only
6 Moisture-Density Relationship Test Method ASTM D698/AASHTO T99
7 Moisture-Density Relationship Test Method ASTM D1557/AASHTO T180

Terracon GeoReport

Permit #25-2535171-LT
RSN 1903242

SUMMARY OF LABORATORY TEST RESULTS

QTS Aurora - Aurora, Colorado
Terracon Project No. 25215013

Boring No.	Depth (ft)	USCS Class.	Initial Water Content (%)	Initial Dry Density (pcf)	Swell/Consolidation		Moisture-Density Relationship			Particle Size Distribution, Percent Passing by Weight				Atterberg Limits		Unconfined Compressive Strength (psf)	Water Soluble Sulfates (%)	Water Soluble Chlorides (%)	pH	Redox (mv)	Resistivity (ohm.cm)	Remarks
					Surcharge (ksf)	Swell (%)	Maximum Dry Density (pcf)	Optimum Water Content (%)	Test Method	3/4"	#4	#40	#200	LL	PI							
35	7.5		18.6	112	0.5	+6.2																3,4
35	12.5		20.1	109																		4
35	19		19.6	113																		4
35	29		20.3	108																		4
35	39		15.6	110																		4
36	0 - 4	CL					107.5	14.0	D698B	100	100	93	77	40	23							6
36	5		15.2	117	0.5	+6.0																3,4
36	7.5		18.7	110	0.5	+6.2																3,4
36	10		21.7	105																		4
36	19		17.8	116																		4
36	29		22.6	104																		4
36	39		20.6	107																		4
37	0 - 4	CL								100	100	97	88	38	21							
37	5	CL	23.0	101																		4
37	10		19.0	110	0.5	+3.6																3,4
37	15		22.7	104																		4
37	24		20.8																			4
37	34		18.5	117																		4
38	0 - 4	CL								100	100	88	70	32	17		0.169	0.0067	7.5	381.8	650	
38	5	CL	16.4																			4
38	7.5		17.6																			4
38	10		18.0																			4
38	15		19.6																			4
38	24		16.4																			4
38	34		21.5																			4
38	44		26.7	96																		4
39	0 - 4	CL					106.5	17.2	D698B	100	100	97	88	41	21							
39	2.5	CL	18.0	114																		4
39	5		14.5	117	0.5	+7.1																3,4
39	7.5		18.7	111																		4
39	12.5		14.7	125																		4
39	19		18.0	111																		4
39	29		15.0	115																		4
39	39		15.3	114																		4

Notes:
Initial Dry Density and Initial Water Content are in-situ values unless otherwise noted.
* = Partially disturbed sample
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NV = no value
NP = non-plastic

Remarks:
1 Remolded Compacted density (about 95% of ASTM D698 maximum density near optimum moisture content)
2 Remolded Compacted density (about 95% of ASTM D1557 maximum density near optimum moisture content)
3 Water added to sample
4 Dry density and/or moisture content determined from one ring of a multi-ring sample
5 Minus #200 Only
6 Moisture-Density Relationship Test Method ASTM D698/AASHTO T99
7 Moisture-Density Relationship Test Method ASTM D1557/AASHTO T180

Terracon GeoReport

Permit #25-2535171-LT
RSN 1903242

SUMMARY OF LABORATORY TEST RESULTS

QTS Aurora - Aurora, Colorado
Terracon Project No. 25215013

Boring No.	Depth (ft)	USCS Class.	Initial Water Content (%)	Initial Dry Density (pcf)	Swell/Consolidation		Moisture-Density Relationship			Particle Size Distribution, Percent Passing by Weight				Atterberg Limits		Unconfined Compressive Strength (psf)	Water Soluble Sulfates (%)	Water Soluble Chlorides (%)	pH	Redox (mv)	Resistivity (ohm.cm)	Remarks
					Surcharge (ksf)	Swell (%)	Maximum Dry Density (pcf)	Optimum Water Content (%)	Test Method	3/4"	#4	#40	#200	LL	PI							
40	0 - 4	CL								100	100	94	81	31	13		0.145	0.0008	7.6	391.5	720	
40	7.5	CL	16.1	113																		4
40	10		15.1	119	0.5	+5.2																3,4
40	19		19.1	111																		4
40	29		18.2	108																		4
40	39		18.4	109																		4
40	49		18.2	110																		4
41	0 - 4	CL								100	100	94	79	31	15							
41	5	CL	12.5	98																		4
41	10		18.8																			4
41	19		19.3	107																		4
41	29		22.0	103																		4
42	0 - 4	CL								100	100	95	83	39	20		0.004	0.0006	7.6	366.7	1186	
42	5	CL	20.6	106																		4
42	10		19.9	111																		4
42	12.5		22.8	105	0.5	+10.4																3,4
42	19		18.6	111																		4
42	29		23.7	103																		4
42	39		18.3	114																		4
F1	0 - 4	CL								100	100	90	63	26	12							
F1	5	CL	10.4	130																		4
F1	10		10.7	126	0.5	+6.0																3,4
F1	15		11.5																			4
F1	19		19.3	108																		4
F1	29		23.3	102																		4
F2	0 - 4	CL								100	100	96	86	37	22							
F2	5	CL	20.8	104																		4
F2	10		21.1	106	0.5	+5.2																3,4
F2	15		15.1	114																		4
F2	19		12.9	121																		4
F2	29		23.8	104																		4
F3	0 - 4	CL								100	100	97	85	52	34							
F3	5		19.4	110	0.5	+3.5																3,4
F3	10		23.5	102																		4

Notes:
Initial Dry Density and Initial Water Content are in-situ values unless otherwise noted.
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Remarks:
1 Remolded Compacted density (about 95% of ASTM D698 maximum density near optimum moisture content)
2 Remolded Compacted density (about 95% of ASTM D1557 maximum density near optimum moisture content)
3 Water added to sample
4 Dry density and/or moisture content determined from one ring of a multi-ring sample
5 Minus #200 Only
6 Moisture-Density Relationship Test Method ASTM D698/AASHTO T99
7 Moisture-Density Relationship Test Method ASTM D1557/AASHTO T180

Terracon GeoReport

Permit #25-2535171-LT
RSN 1903242

SUMMARY OF LABORATORY TEST RESULTS

QTS Aurora - Aurora, Colorado
Terracon Project No. 25215013

Boring No.	Depth (ft)	USCS Class.	Initial Water Content (%)	Initial Dry Density (pcf)	Swell/Consolidation		Moisture-Density Relationship			Particle Size Distribution, Percent Passing by Weight				Atterberg Limits		Unconfined Compressive Strength (psf)	Water Soluble Sulfates (%)	Water Soluble Chlorides (%)	pH	Redox (mv)	Resistivity (ohm.cm)	Remarks
					Surcharge (ksf)	Swell (%)	Maximum Dry Density (pcf)	Optimum Water Content (%)	Test Method	3/4"	#4	#40	#200	LL	PI							
F3	15		23.3	102																		4
F3	19		24.0	101																		4
F3	29		20.9	105																		4
F4	0 - 4	CL								100	100	94	74	35	21							
F4	5	CL	17.4	108																		4
F4	7.5	CL	16.3	109	0.5	+0.1																3,4
F4	10	CL	20.4	105																		4
F4	15		25.6	98																		4
F4	19		25.4	97																		4
F4	29		22.2	103																		4
F5	0 - 4	CL								100	100	96	85	30	12							
F5	5		16.1	112																		4
F5	10		6.6	133	0.5	+3.4																3,4
F5	15		19.1	108																		4
F5	19		21.8	105																		4
F5	29		19.3	107																		4
F6	0 - 4	CL					104.2	19.7	D698B	100	100	96	86	44	27							
F6	5		24.6	100	0.5	+2.9																3,4
F6	10		19.1	112																		4
F6	15		14.6	116																		4
F6	19		24.4	101																		4
F6	29		22.1	103																		4
S1	0 - 4	CL												34	20							
S1	5	CL	24.7	96																		4
S1	7.5	CL	18.0	111	0.5	+2.3																3,4
S1	12.5		25.0	100																		4
S1	19		22.3	105																		4
S1	29		18.3	109																		4
S1	39		23.4	103																		4
S2	0 - 4	CL												34	22							
S2	2.5	CL	17.2	113																		4
S2	7.5		20.8	107	0.5	+4.1																3,4
S2	10		23.3	101																		4
S2	15		24.3	103																		4

Notes:
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Terracon GeoReport

Permit #25-2535171-LT
RSN 1903242

Supporting Information

Contents:

General Notes
Unified Soil Classification System

Note: All attachments are one page unless noted above.

Terracon
10625 W I70 Frontage Rd N
Ste 3
Wheat Ridge, CO

General Notes

Sampling



Water Level



Water Initially Encountered

Water Level After a Specified Period of Time

Water Level After a Specified Period of Time

Cave In Encountered

Water levels indicated on the soil boring logs are the levels measured in the borehole at the times indicated. Groundwater level variations will occur over time. In low permeability soils, accurate determination of groundwater levels is not possible with short term water level observations.

Field Tests

N	Standard Penetration Test Resistance (Blows/Ft.)
(HP)	Hand Penetrometer
(T)	Torvane
(DCP)	Dynamic Cone Penetrometer
UC	Unconfined Compressive Strength
(PID)	Photo-Ionization Detector
(OVA)	Organic Vapor Analyzer

Descriptive Soil Classification

Soil classification as noted on the soil boring logs is based Unified Soil Classification System. Where sufficient laboratory data exist to classify the soils consistent with ASTM D2487 "Classification of Soils for Engineering Purposes" this procedure is used. ASTM D2488 "Description and Identification of Soils (Visual-Manual Procedure)" is also used to classify the soils, particularly where insufficient laboratory data exist to classify the soils in accordance with ASTM D2487. In addition to USCS classification, coarse grained soils are classified on the basis of their in-place relative density, and fine-grained soils are classified on the basis of their consistency. See "Strength Terms" table below for details. The ASTM standards noted above are for reference to methodology in general. In some cases, variations to methods are applied as a result of local practice or professional judgment.

Location And Elevation Notes

Exploration point locations as shown on the Exploration Plan and as noted on the soil boring logs in the form of Latitude and Longitude are approximate. See Exploration and Testing Procedures in the report for the methods used to locate the exploration points for this project. Surface elevation data annotated with +/- indicates that no actual topographical survey was conducted to confirm the surface elevation. Instead, the surface elevation was approximately determined from topographic maps of the area.

Strength Terms

Relative Density of Coarse-Grained Soils
(More than 50% retained on No. 200 sieve.)
Density determined by Standard Penetration Resistance

Relative Density	Standard Penetration or N-Value (Blows/Ft.)	Ring Sampler (Blows/Ft.)
Very Loose	0 - 3	0 - 5
Loose	4 - 9	6 - 14
Medium Dense	10 - 29	15 - 46
Dense	30 - 50	47 - 79
Very Dense	> 50	≥ 80

Consistency of Fine-Grained Soils
(50% or more passing the No. 200 sieve.)
Consistency determined by laboratory shear strength testing, field visual-manual procedures or standard penetration resistance

Consistency	Unconfined Compressive Strength Qu (psf)	Standard Penetration or N-Value (Blows/Ft.)	Ring Sampler (Blows/Ft.)
Very Soft	less than 500	0 - 1	< 3
Soft	500 to 1,000	2 - 4	3 - 5
Medium Stiff	1,000 to 2,000	4 - 8	6- 10
Stiff	2,000 to 4,000	8 - 15	11 - 18
Very Stiff	4,000 to 8,000	15 - 30	19 - 36
Hard	> 8,000	> 30	> 36

Bedrock

Consistency	Standard Penetration or N-Value (Blows/Ft.)	Ring Sampler (Blows/Ft.)
Weathered	< 20	< 24
Firm	20 - 29	24 - 35
Medium Hard	30 - 49	36 - 60
Hard	50 - 79	61 - 96
Very Hard	>79	> 96

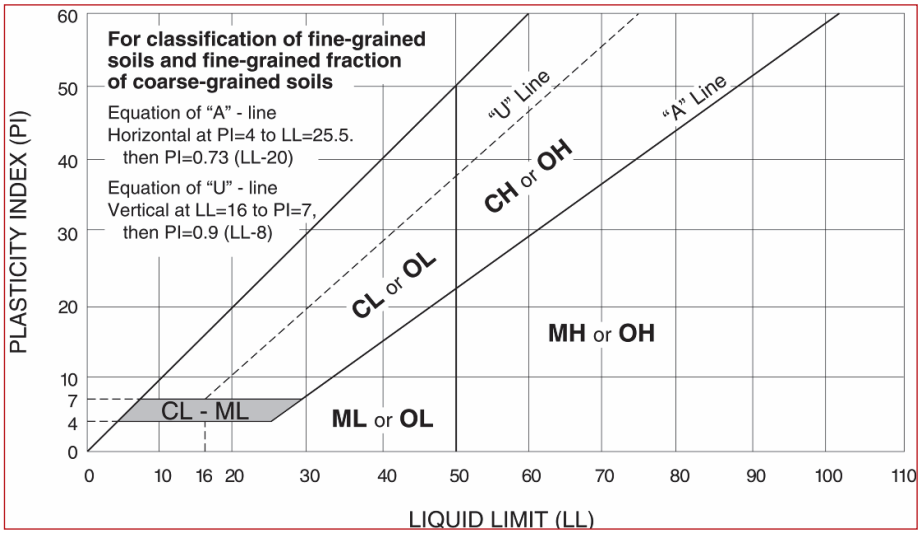
Relevance of Exploration and Laboratory Test Results

Exploration/field results and/or laboratory test data contained within this document are intended for application to the project as described in this document. Use of such exploration/field results and/or laboratory test data should not be used independently of this document.

Unified Soil Classification System

Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests [A]				Soil Classification	
				Group Symbol	Group Name [B]
Coarse-Grained Soils: More than 50% retained on No. 200 sieve	**Gravels:** More than 50% of coarse fraction retained on No. 4 sieve	**Clean Gravels:** Less than 5% fines [C]	Cu≥4 and 1≤Cc≤3 [E]	GW	Well-graded gravel [F]
			Cu<4 and/or [Cc<1 or Cc>3.0] [E]	GP	Poorly graded gravel [F]
		Gravels with Fines: More than 12% fines [C]	Fines classify as ML or MH	GM	Silty gravel [F, G, H]
			Fines classify as CL or CH	GC	Clayey gravel [F, G, H]
	Sands: 50% or more of coarse fraction passes No. 4 sieve	**Clean Sands:** Less than 5% fines [D]	Cu≥6 and 1≤Cc≤3 [E]	SW	Well-graded sand [I]
			Cu<6 and/or [Cc<1 or Cc>3.0] [E]	SP	Poorly graded sand [I]
		Sands with Fines: More than 12% fines [D]	Fines classify as ML or MH	SM	Silty sand [G, H, I]
			Fines classify as CL or CH	SC	Clayey sand [G, H, I]
Fine-Grained Soils: 50% or more passes the No. 200 sieve	**Silts and Clays:** Liquid limit less than 50	**Inorganic:**	PI > 7 and plots above "A" line [J]	CL	Lean clay [K, L, M]
			PI < 4 or plots below "A" line [J]	ML	Silt [K, L, M]
		Organic:	$\frac{LL\ oven\ dried}{LL\ not\ dried} < 0.75$	OL	Organic clay [K, L, M, N]
					Organic silt [K, L, M, O]
	Silts and Clays: Liquid limit 50 or more	**Inorganic:**	PI plots on or above "A" line	CH	Fat clay [K, L, M]
			PI plots below "A" line	MH	Elastic silt [K, L, M]
		Organic:	$\frac{LL\ oven\ dried}{LL\ not\ dried} < 0.75$	OH	Organic clay [K, L, M, P]
					Organic silt [K, L, M, Q]
Highly organic soils:	Primarily organic matter, dark in color, and organic odor			PT	Peat

[A] Based on the material passing the 3-inch (75-mm) sieve.
[B] If field sample contained cobbles or boulders, or both, add "with cobbles or boulders, or both" to group name.
[C] Gravels with 5 to 12% fines require dual symbols: GW-GM well-graded gravel with silt, GW-GC well-graded gravel with clay, GP-GM poorly graded gravel with silt, GP-GC poorly graded gravel with clay.
[D] Sands with 5 to 12% fines require dual symbols: SW-SM well-graded sand with silt, SW-SC well-graded sand with clay, SP-SM poorly graded sand with silt, SP-SC poorly graded sand with clay.
[E] $Cu = D_{60}/D_{10}$ $Cc = \frac{(D_{30})^2}{D_{10} \times D_{60}}$
[F] If soil contains ≥ 15% sand, add "with sand" to group name.
[G] If fines classify as CL-ML, use dual symbol GC-GM, or SC-SM.
[H] If fines are organic, add "with organic fines" to group name.
[I] If soil contains ≥ 15% gravel, add "with gravel" to group name.
[J] If Atterberg limits plot in shaded area, soil is a CL-ML, silty clay.
[K] If soil contains 15 to 29% plus No. 200, add "with sand" or "with gravel," whichever is predominant.
[L] If soil contains ≥ 30% plus No. 200 predominantly sand, add "sandy" to group name.
[M] If soil contains ≥ 30% plus No. 200, predominantly gravel, add "gravelly" to group name.
[N] PI ≥ 4 and plots on or above "A" line.
[O] PI < 4 or plots below "A" line.
[P] PI plots on or above "A" line.
[Q] PI plots below "A" line.



PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

STRUCTURAL FOUNDATION CALCULATIONS

for

QTS DEN1-DC2
1160 N. Gun Club Road,
Aurora, CO 80018

Prepared by

AG&E Associates, PLLC
9 Greenway Plaza, Suite 2025
Houston, Texas 77046
Project # 24053.00000

Prepared for

City of Aurora

May 30, 2025
Calculations for IFP

Structural plan review is limited to a general survey for code compliance. No review is implied nor was undertaken to verify structural adequacy.



05.25.2025

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

TABLE OF CONTENTS

1.0 STRUCTURAL SCOPE OF WORK 3
2.0 GRAVITY LOAD PATH: DATA HALL ANALYSIS 5
2.1 FLOORS PLANS, ASSUMPTIONS AND LOADINGS 6
3.0 TSD MODEL RESULTS: DATA HALL 30
4.0 SAMPLE CALCULATIONS / ANALYSIS 34
5.0 FOUNDATION CALCULATIONS 38
6.0 STEEL COLUMN BASE PLATES & ANCHORAGE 116

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

1.0 STRUCTURAL SCOPE OF WORK

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

Structural Scope of Work

a) Analysis of Data Hall's & Office

- Second floor composite slab and framing.
- Roof deck and framings.
- Checking the floors and roof beams.
- Gravity and Brace Foundations check.
- Base Plate check.
- Lateral design of braced frames, diaphragms, chords, and collectors.

b) Software Used

- Tekla Structural Designer (TSD) Version 2024.
- Tekla Tedds
- Visual Foundations
- Hilti
- L-Pile

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

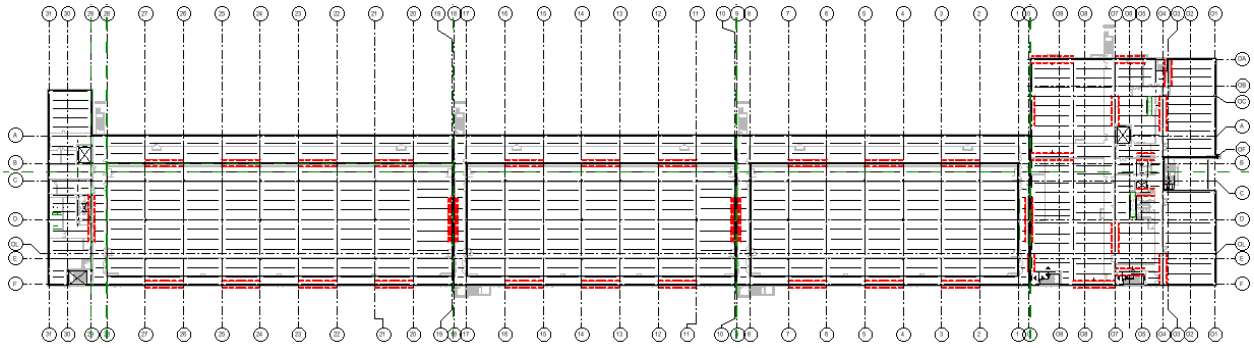
2.0 GRAVITY LOAD PATH: DATA HALL ANALYSIS

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

2.1 FLOORS PLANS, ASSUMPTIONS AND LOADINGS

Second Floor Plan

a) Overall, Second Floor Plan - REVIT

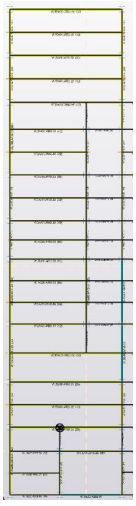


b) Data Hall (DH), Second Floor Plan – TSD

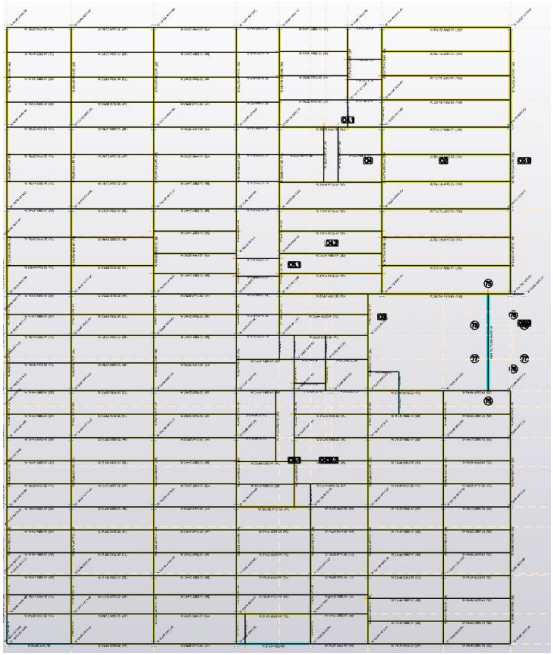


PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

c) Data Hall – Back of the House (BOH), Second Floor Plan – TSD



d) Office, Second Floor Plan – TSD



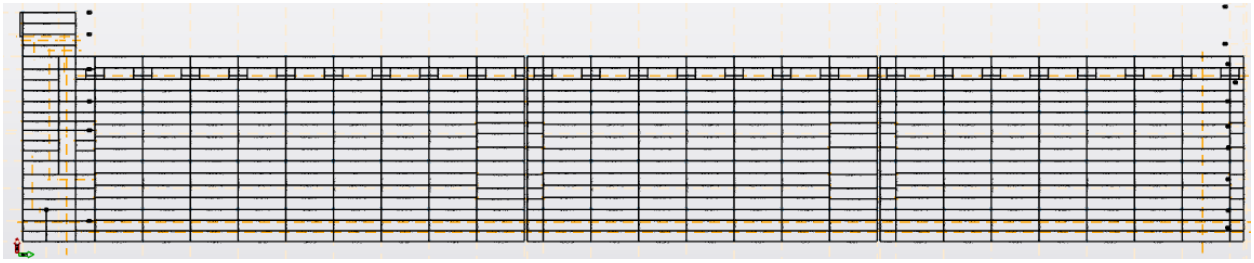
PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

Roof Plan

a) Overall, Roof Plan - REVIT

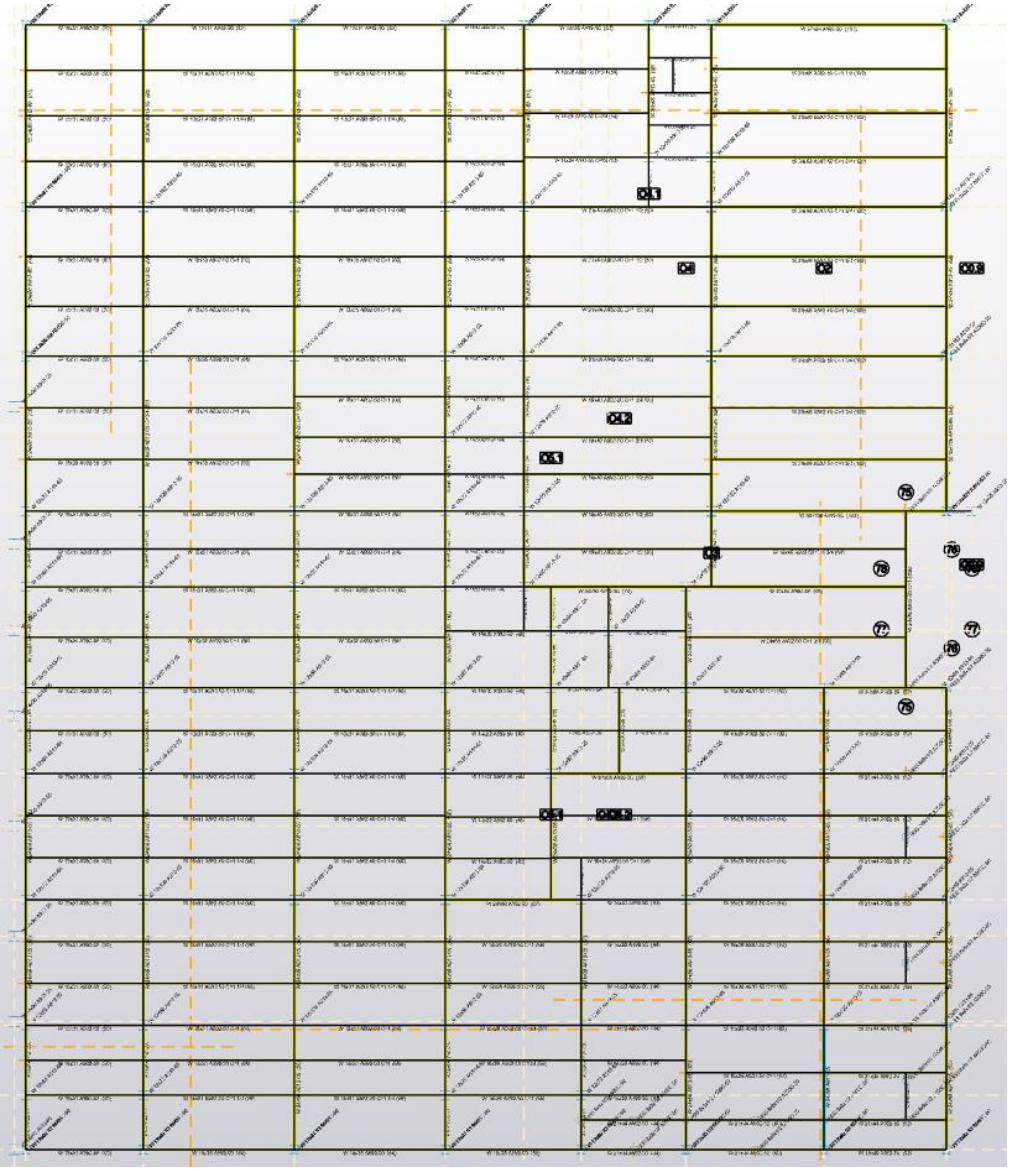


b) Data Hall (DH), Roof Plan – TSD



PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

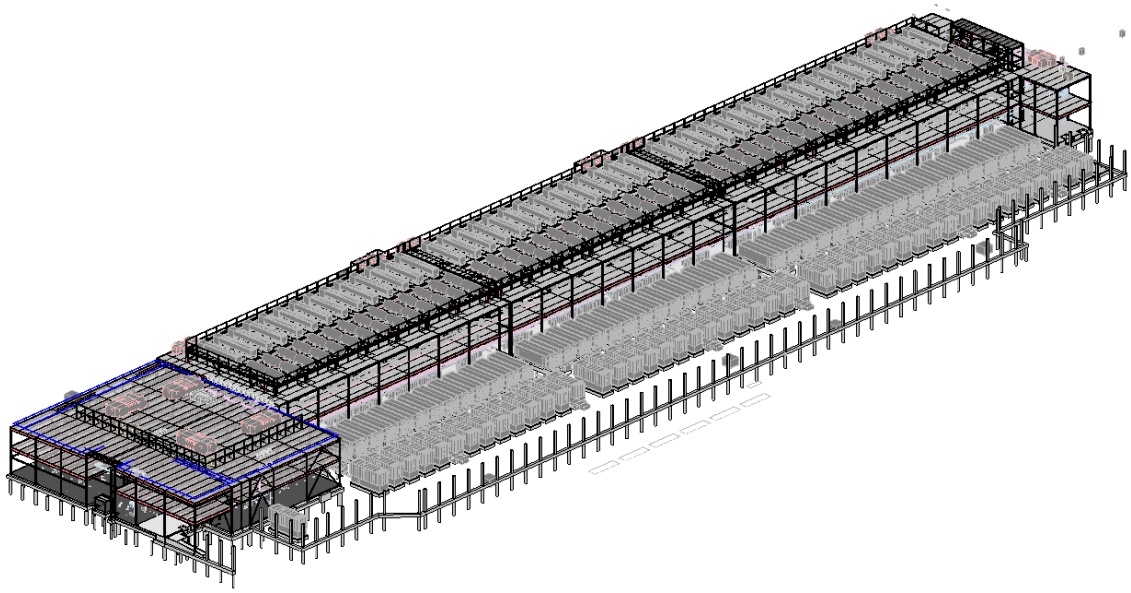
c) Office, Roof Plan



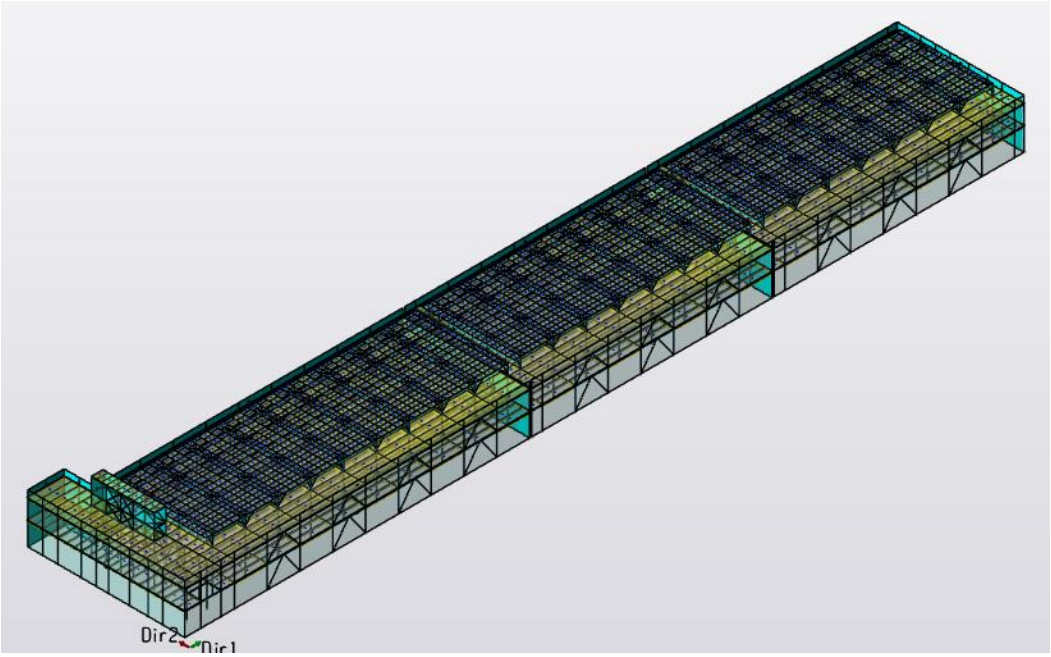
PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

REVIT AND TSD 3-D MODEL

a) Overall – 3D View – REVIT

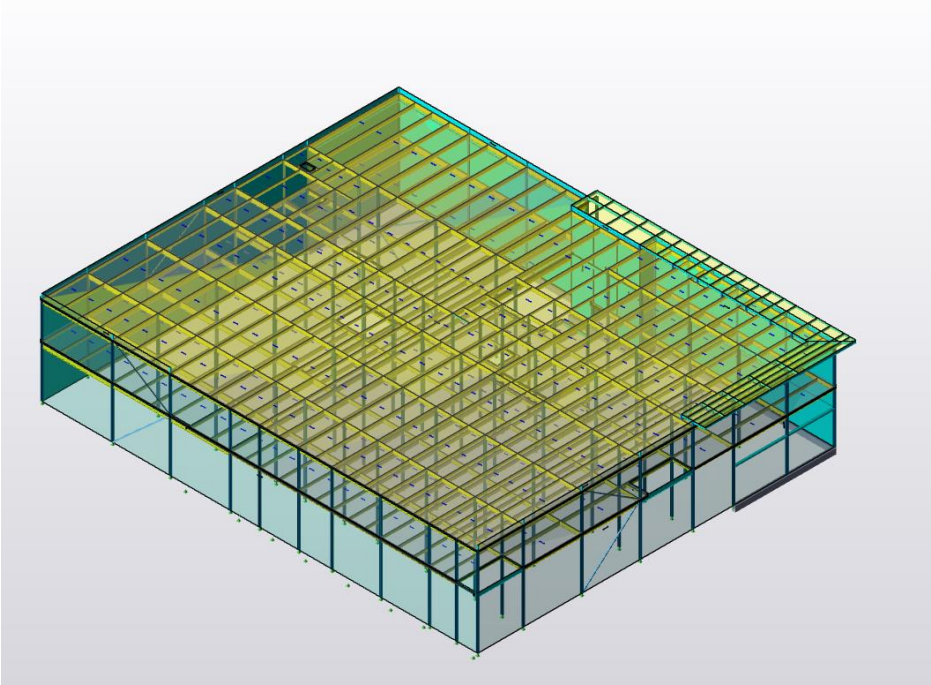


b) Data Hall (DH) – 3D View – TSD



PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

c) Office – 3D View



PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

Loading Summary

a) Second Floor Loading Summary (Data Hall)

- Gravity Load
  - Self -weight
  - Floor, Dead Load = D_1 = 15 psf.
  - Concrete Ponding, Dead Load = D_2 = 5 psf.
  - Hung Below Floor Framing Members, Dead Load = D_3= 100 psf.
    - ⇨ Total Distributed Dead Load of Second Floor (Data Hall) → $D = D_1+D_2+D_3$= 120 psf.
  - Cabinets anchored to Data Hall floor (Dead Load = D_4 (Unreducible) = 350 psf.

b) Second Floor Loading Summary (Office)

- Gravity Load
  - Self -weight
  - Floor, Dead Load = D_1 = 16 psf.
  - Concrete Ponding, Dead Load = D_2= 5 psf.
  - Fitness Area = D_3 = 100 psf.
    - ⇨ Total Distributed Dead Load of Second Floor (Office) =$D = D_1+D_2$ = 21psf.
    - ⇨ Total Distributed Dead Load of Second Floor (Office/Fitness) =$D = D_1+D_2+D_3$ = 121psf.
  - Live Load = LL (Unreducible) = 250 psf.

c) Roof Loading Summary (Data Hall)

- Gravity Load
  - Self-weight
  - Collateral Dead Load = D_1 = 10 psf.
  - Concrete Ponding, Dead Load = D_2 = 5 psf.
  - Hung Below Floor Framing Members, Dead Load = D_3 = 100 psf.
    - ⇨ Total Distributed Dead Load of Second Floor → $D = D_1 + D_2 + D_3$ = 115 psf.
  - Roof Live Load (Unreducible) = Lr = 20 psf.

d) Roof Loading Summary (Office)

- Gravity Load
  - Self -weight
  - Collateral Dead Load = D_1 = 18 psf.
  - Concrete Ponding, Dead Load = D_2 = 5 psf.
    - ⇨ Total Distributed Dead Load of Second Floor → $D = D_1 + D_2$ = 23 psf.
  - Roof Live Load (Unreducible) = Lr = 20 psf.
- Load Combinations (ASCE 7-16)
  - 1.4D
  - 1.2D+1.6L
  - 1.2D+1.6L+0.5Lr
  - 1.2D+L+1.6Lr

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

Wind Summary (Data Hall - 1)

Structure

Wind Data

General

Method
ASCE/SEI 7-16 : Directional Procedure

Site Details

Site Ground Level	0″	ft, in
Orientation of Principal Axes relative to Global axes	0.0000	Â°
Mean Roof Height, h (overridden)	52′ 0″	ft, in
Level of Highest Opening in Building (Use Mean Roof	52′ 0″	ft, in
Overall Building X' Dimension (overridden)	441′ 7″	ft, in
Overall Building Y' Dimension (overridden)	211′ 2 3/4″	ft, in
Torsion Design Pressure Factor	75.00	%
Torsion Eccentricity	15.00	%
Wind Loading		
Basic Wind Speed, V	125.00	mph
Directionality Factor, K_d	0.850	
Ground Elevation above Sea Level, z_g	0	ft
Ground Elevation Factor, K_e	1.000	
Enclosure Classification	Enclosed	
Gust Effect factor, G	0.850	
Principal Axis	+X'	
Exposure Category	C	
Topographic Feature	None	
Principal Axis	+Y'	
Exposure Category	C	
Topographic Feature	None	
Principal Axis	-X'	
Exposure Category	C	
Topographic Feature	None	
Principal Axis	-Y'	
Exposure Category	C	
Topographic Feature	None	

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

Wind Summary (Data Hall - 2)

Structure

Wind Data

General

Method
ASCE/SEI 7-16 : Directional Procedure Part 1

Site Details

Site Ground Level	0″	ft, in
Orientation of Principal Axes relative to Global axes	0.0000	Â°
Mean Roof Height, h (overridden)	52′ 0″	ft, in
Level of Highest Opening in Building (Use Mean Roof Height)	52′ 0″	ft, in
Overall Building X' Dimension (overridden)	308′ 0″	ft, in
Overall Building Y' Dimension (overridden)	162′ 4″	ft, in
Torsion Design Pressure Factor	75.00	%
Torsion Eccentricity	15.00	%
Wind Loading		
Basic Wind Speed, V	125.00	mph
Directionality Factor, K_d	0.850	
Ground Elevation above Sea Level, z_g	0	ft
Ground Elevation Factor, K_e	1.000	
Enclosure Classification	Enclosed	
Gust Effect factor, G	0.850	
Principal Axis	+X'	
Exposure Category	C	
Topographic Feature	None	
Principal Axis	+Y'	
Exposure Category	C	
Topographic Feature	None	
Principal Axis	-X'	
Exposure Category	C	
Topographic Feature	None	
Principal Axis	-Y'	
Exposure Category	C	
Topographic Feature	None	

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

Wind Summary (Data Hall - 3)

Structure

Wind Data

General

Method
ASCE/SEI 7-16 : Directional Procedure

Site Details

Site Ground Level	0″	ft, in
Orientation of Principal Axes relative to Global axes	0.0000	Â°
Mean Roof Height, h (overridden)	52′ 0″	ft, in
Level of Highest Opening in Building (Use Mean Roof	52′ 0″	ft, in
Overall Building X' Dimension (overridden)	320′ 0″	ft, in
Overall Building Y' Dimension (overridden)	162′ 4″	ft, in
Torsion Design Pressure Factor	75.00	%
Torsion Eccentricity	15.00	%
Wind Loading		
Basic Wind Speed, V	125.00	mph
Directionality Factor, K_d	0.850	
Ground Elevation above Sea Level, z_g	0	ft
Ground Elevation Factor, K_e	1.000	
Enclosure Classification	Enclosed	
Gust Effect factor, G	0.850	
Principal Axis	+X'	
Exposure Category	C	
Topographic Feature	None	
Principal Axis	+Y'	
Exposure Category	C	
Topographic Feature	None	
Principal Axis	-X'	
Exposure Category	C	
Topographic Feature	None	
Principal Axis	-Y'	
Exposure Category	C	
Topographic Feature	None	

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

Snow Summary (Data Hall - 1)

Structure

Snow Data

General

Method ASCE7

Basic data

Ground Snow Load, p_g	40.000	psf	ASCE7-16 Clause 7.2
Terrain Category	B (urban, suburban, wooded)		
Exposure	Sheltered		
Exposure Factor, C_e	1.200		ASCE7-16 Clause 7.3.1
Thermal Condition	All except below		
Thermal Factor, C_t	1.000		ASCE7-16 Clause 7.3.2
Risk Category	IV		
Snow Importance Factor, I_s	1.200		
Flat Roof Snow Load, p_f	40.320	psf	ASCE7-16 Clause 7.3

Snow load cases

Minimum Snow Load	No
Balanced Snow Load	Yes
Unbalanced Snow Load	No
Drift Snow Load	Yes
Rain on Snow Surcharge	No

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

Snow Summary (Data Hall - 2)

Structure

Snow Data

General

Method ASCE7

Basic data

Ground Snow Load, p_g	40.000	psf	ASCE7-16 Clause 7.2
Terrain Category	B (urban, suburban, wooded)		
Exposure	Sheltered		
Exposure Factor, C_e	1.200		ASCE7-16 Clause 7.3.1
Thermal Condition	All except below		
Thermal Factor, C_t	1.000		ASCE7-16 Clause 7.3.2
Risk Category	IV		
Snow Importance Factor, I_s	1.200		
Flat Roof Snow Load, p_f	40.320	psf	ASCE7-16 Clause 7.3

Snow load cases

Minimum Snow Load	No
Balanced Snow Load	Yes
Unbalanced Snow Load	No
Drift Snow Load	Yes
Rain on Snow Surcharge	No

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

Snow Summary (Data Hall - 3)

Structure

Snow Data

General

Method ASCE7

Basic data

Ground Snow Load, p_g	40.000	psf	ASCE7-16 Clause 7.2
Terrain Category	B (urban, suburban, wooded)		
Exposure	Sheltered		
Exposure Factor, C_e	1.200		ASCE7-16 Clause 7.3.1
Thermal Condition	All except below		
Thermal Factor, C_t	1.000		ASCE7-16 Clause 7.3.2
Risk Category	IV		
Snow Importance Factor, I_s	1.200		
Flat Roof Snow Load, p_f	40.320	psf	ASCE7-16 Clause 7.3

Snow load cases

Minimum Snow Load	No
Balanced Snow Load	Yes
Unbalanced Snow Load	No
Drift Snow Load	Yes
Rain on Snow Surcharge	No

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

Seismic Summary (Data Hall - 1)

General			
Method	ASCE7		
Structure details			
Height to highest level	67' 6"	ft, in	
Ignore seismic in floor (and below)	St. 1 (1)		
Number of storeys	3		
Two-Period Spectrum			
Max earthquake spectral response			
S_S short (0.2 s) period (mapped)	18.60	% g	
S_1 1.0 s period (mapped)	5.40	% g	
Site Class	D - Stiff soil		ASCE7-16 Table 20.3-1
Risk Category	IV		ASCE7-16 Table 1.5-1
Design spectral response acceleration			
S_{DS} short (0.2 s) period	19.84	% g	ASCE7-16 Clause 11.4-3
S_{D1} 1.0 s period	8.64	% g	ASCE7-16 Clause 11.4-4
Seismic Importance Factor, I_e	1.500		ASCE7-16 Table 1.5-2
Seismic Design Category	C		ASCE7-16 Clause 11.6
T_L long period transition period	4.000	sec	
T_S (S_{D1}/S_{DS})	0.435	sec	
Effective Seismic Weight, W	48117.372	kip	ASCE7-16 Clause 12.7.2
Structure Type			ASCE7-16 Table 12.8-2
Direction Dir1	All other structural systems		
Direction Dir2	All other structural systems		
Basic seismic force resisting system			ASCE7-16 Table 12.2-1
Direction Dir1	H. Steel Systems not Specifically detailed for Seismic		
Direction Dir2	H. Steel Systems not Specifically detailed for Seismic		
	Direction Dir1	**Direction Dir2**	
Response Modification Factor, R	3.000	3.000	
System Over-Strength Factor, Ω_0	3.000	3.000	
Deflection Amplification Factor, Cd	3.000	3.000	
Redundancy Factor, ρ	1.000	1.000	ASCE7-16 Clause 12.3.4.2
Approximate fundamental period, T_a [sec]	0.471	0.471	ASCE7-16 Clause 12.8.2.1
Exponent related to structural period k	1.151	1.151	
Structure fundamental period [sec]	0.801	0.801	
Seismic response coefficient, C_S	0.054	0.054	ASCE7-16 Clause 12.8.1.1
Seismic base shear, V [kip]	**2595.094**	**2595.094**	
Scaling Factor for Forces (1.00 V/V_t)	-	-	ASCE7-16 Clause 12.9.1.4.1
Scaling Factor for Drifts	-	-	ASCE7-16 Clause 12.9.1.4.2

Equivalent Lateral Force Procedure is permitted (ASCE7-16 12.8).

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

Seismic Summary (Data Hall - 2)

General			
Method	ASCE7		
Structure details			
Height to highest level	67′ 6″	ft, in	
Ignore seismic in floor (and below)	St. 1 (1)		
Number of storeys	3		
Two-Period Spectrum			
Max earthquake spectral response			
S_S short (0.2 s) period (mapped)	18.60	% g	
S_1 1.0 s period (mapped)	5.40	% g	
Site Class	D - Stiff soil		ASCE7-16 Table 20.3-1
Risk Category	IV		ASCE7-16 Table 1.5-1
Design spectral response acceleration			
S_{DS} short (0.2 s) period	19.84	% g	ASCE7-16 Clause 11.4-3
S_{D1} 1.0 s period	8.64	% g	ASCE7-16 Clause 11.4-4
Seismic Importance Factor, I_e	1.500		ASCE7-16 Table 1.5-2
Seismic Design Category	C		ASCE7-16 Clause 11.6
T_L long period transition period	4.000	sec	
T_S (S_{D1}/S_{DS})	0.435	sec	
Effective Seismic Weight, W	32833.998	kip	ASCE7-16 Clause 12.7.2
Structure Type			ASCE7-16 Table 12.8-2
Direction Dir1	All other structural systems		
Direction Dir2	All other structural systems		
Basic seismic force resisting system			ASCE7-16 Table 12.2-1
Direction Dir1	H. Steel Systems not Specifically detailed for Seismic		
Direction Dir2	H. Steel Systems not Specifically detailed for Seismic		

	Direction Dir1	**Direction Dir2**	
Response Modification Factor, R	3.000	3.000	
System Over-Strength Factor, Ω_0	3.000	3.000	
Deflection Amplification Factor, Cd	3.000	3.000	
Redundancy Factor, ρ	1.000	1.000	ASCE7-16 Clause 12.3.4.2
Approximate fundamental period, T_a [sec]	0.471	0.471	ASCE7-16 Clause 12.8.2.1
Exponent related to structural period k	1.151	1.151	
Structure fundamental period [sec]	0.801	0.801	
Seismic response coefficient, C_S	0.054	0.054	ASCE7-16 Clause 12.8.1.1
Seismic base shear, V [kip]	**1770.822**	**1770.822**	
Scaling Factor for Forces (1.00 V/V_t)	-	-	ASCE7-16 Clause 12.9.1.4.1
Scaling Factor for Drifts	-	-	ASCE7-16 Clause 12.9.1.4.2

Equivalent Lateral Force Procedure is permitted (ASCE7-16 12.8).

Reference	Level [ft, in]	Weight [kip]	Direction Dir1			Direction Dir2		
			C_V	F [kip]	Ecc [ft, in]	C_V	F [kip]	Ecc [ft, in]
St. 8 (4.0 - PLATFORM FRAMING PLAN)	66′ 6″	5521.227	0.291	516.035	8′ 1 3/8″	0.291	516.035	15′ 4 3/4″
St. 6.1 (3.1 - PARAPET HT)	61′ 0″	3.721	0.000	0.315	8′ 1 3/8″	0.000	0.315	15′ 4 3/4″
	55′ 0″	62.415	0.003	4.706	8′ 1 3/8″	0.003	4.706	15′ 4 3/4″
St. 6 (3.0 - ROOF FRAMING PLAN)	52′ 0″	9973.019	0.399	705.725	8′ 1 3/8″	0.399	705.725	15′ 4 3/4″
St. 3 (2.0 - SECOND FLOOR FRAMING	27′ 4 1/2″	4860.570	0.095	167.618	8′ 1 3/8″	0.095	167.618	15′ 4 3/4″
St. 2.2 (2.2 - LOW SLAB)	24′ 4 1/2″	12413.046	0.213	376.424	6′ 7 3/8″	0.213	376.424	15′ 4 3/4″

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

Seismic Summary (Data Hall - 1)

General			
Method	ASCE7		
Structure details			
Height to highest level	67' 6"	ft, in	
Ignore seismic in floor (and below)	St. 1 (1)		
Number of storeys	3		
Two-Period Spectrum			
Max earthquake spectral response			
S_S short (0.2 s) period (mapped)	18.60	% g	
S_1 1.0 s period (mapped)	5.40	% g	
Site Class	D - Stiff soil		ASCE7-16 Table 20.3-1
Risk Category	IV		ASCE7-16 Table 1.5-1
Design spectral response acceleration			
S_{DS} short (0.2 s) period	19.84	% g	ASCE7-16 Clause 11.4-3
S_{D1} 1.0 s period	8.64	% g	ASCE7-16 Clause 11.4-4
Seismic Importance Factor, I_e	1.500		ASCE7-16 Table 1.5-2
Seismic Design Category	C		ASCE7-16 Clause 11.6
T_L long period transition period	4.000	sec	
T_S (S_{D1}/S_{DS})	0.435	sec	
Effective Seismic Weight, W	48117.372	kip	ASCE7-16 Clause 12.7.2
Structure Type			ASCE7-16 Table 12.8-2
Direction Dir1	All other structural systems		
Direction Dir2	All other structural systems		
Basic seismic force resisting system			ASCE7-16 Table 12.2-1
Direction Dir1	H. Steel Systems not Specifically detailed for Seismic		
Direction Dir2	H. Steel Systems not Specifically detailed for Seismic		
	Direction Dir1	**Direction Dir2**	
Response Modification Factor, R	3.000	3.000	
System Over-Strength Factor, Ω_0	3.000	3.000	
Deflection Amplification Factor, Cd	3.000	3.000	
Redundancy Factor, ρ	1.000	1.000	ASCE7-16 Clause 12.3.4.2
Approximate fundamental period, T_a [sec]	0.471	0.471	ASCE7-16 Clause 12.8.2.1
Exponent related to structural period k	1.151	1.151	
Structure fundamental period [sec]	0.801	0.801	
Seismic response coefficient, C_S	0.054	0.054	ASCE7-16 Clause 12.8.1.1
Seismic base shear, V [kip]	**2595.094**	**2595.094**	
Scaling Factor for Forces (1.00 V/V_t)	-	-	ASCE7-16 Clause 12.9.1.4.1
Scaling Factor for Drifts	-	-	ASCE7-16 Clause 12.9.1.4.2

Equivalent Lateral Force Procedure is permitted (ASCE7-16 12.8).

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

Wind Summary (Office)

Structure
Wind Data
General

Method
ASCE/SEI 7-16 : Directional Procedure Part 1

Site Details

Site Ground Level	0”	ft, in
Orientation of Principal Axes relative to Global axes	0.0000	Â°
Mean Roof Height, h (overridden)	52’ 0”	ft, in
Level of Highest Opening in Building (Use Mean	52’ 0”	ft, in
Overall Building X' Dimension (overridden)	203’ 4	ft, in
Overall Building Y' Dimension (overridden)	245’ 9”	ft, in
Torsion Design Pressure Factor	75.00	%
Torsion Eccentricity	15.00	%
Wind Loading		
Basic Wind Speed, V	125.00	mph
Directionality Factor, K_d	0.850	
Ground Elevation above Sea Level, z_g	0	ft
Ground Elevation Factor, K_e	1.000	
Enclosure Classification	Enclosed	
Gust Effect factor, G	0.850	
Principal Axis	+X'	
Exposure Category	C	
Topographic Feature	None	
Principal Axis	+Y'	
Exposure Category	C	
Topographic Feature	None	
Principal Axis	-X'	
Exposure Category	C	
Topographic Feature	None	
Principal Axis	-Y'	
Exposure Category	C	
Topographic Feature	None	

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

Snow Summary (Office)

Structure

Snow Data

General

Method ASCE7

Basic data

Ground Snow Load, p_g	40.000	psf	ASCE7-16 Clause 7.2
Terrain Category	B (urban, suburban, wooded)		
Exposure	Sheltered		
Exposure Factor, C_e	1.200		ASCE7-16 Clause 7.3.1
Thermal Condition	All except below		
Thermal Factor, C_t	1.000		ASCE7-16 Clause 7.3.2
Risk Category	IV		
Snow Importance Factor, I_s	1.200		
Flat Roof Snow Load, p_f	40.320	psf	ASCE7-16 Clause 7.3

Snow load cases

Minimum Snow Load	No
Balanced Snow Load	Yes
Unbalanced Snow Load	No
Drift Snow Load	Yes
Rain on Snow Surcharge	No

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

Seismic Summary (Office)

Structure

Seismic Loading Summary

General			
Method	ASCE7		
Structure details			
Height to highest level	50′ 7″	ft, in	
Ignore seismic in floor (and below)	St. 1 (1)		
Number of storeys	2		
Two-Period Spectrum			
Max earthquake spectral response			
S_S short (0.2 s) period (mapped)	18.60	% g	
S_1 1.0 s period (mapped)	5.40	% g	
Site Class	D - Stiff soil		ASCE7-16 Table 20.3-1
Risk Category	IV		ASCE7-16 Table 1.5-1
Design spectral response acceleration			
S_{DS} short (0.2 s) period	19.84	% g	ASCE7-16 Clause 11.4-3
S_{D1} 1.0 s period	8.64	% g	ASCE7-16 Clause 11.4-4
Seismic Importance Factor, I_e	1.500		ASCE7-16 Table 1.5-2
Seismic Design Category	C		ASCE7-16 Clause 11.6
T_L long period transition period	4.000	sec	
T_S (S_{D1}/S_{DS})	0.435	sec	
Effective Seismic Weight, W	18035.346	kip	ASCE7-16 Clause 12.7.2
Structure Type			ASCE7-16 Table 12.8-2
Direction Dir1	All other structural systems		
Direction Dir2	All other structural systems		
Basic seismic force resisting system			ASCE7-16 Table 12.2-1
Direction Dir1	H. Steel Systems not Specifically detailed for Seismic		
Direction Dir2	H. Steel Systems not Specifically detailed for Seismic		

	Direction Dir1	**Direction Dir2**	
Response Modification Factor, R	3.000	3.000	
System Over-Strength Factor, Ω_0	3.000	3.000	
Deflection Amplification Factor, Cd	3.000	3.000	
Redundancy Factor, ρ	1.000	1.000	ASCE7-16 Clause 12.3.4.2
Approximate fundamental period, T_a [sec]	0.380	0.380	ASCE7-16 Clause 12.8.2.1
Exponent related to structural period k	1.073	1.073	
Structure fundamental period [sec]	0.645	0.645	
Seismic response coefficient, C_S	0.067	0.067	ASCE7-16 Clause 12.8.1.1
Seismic base shear, V [kip]	**1207.949**	**1207.949**	
Scaling Factor for Forces (1.00 V/V_t)	-	-	ASCE7-16 Clause 12.9.1.4.1
Scaling Factor for Drifts	-	-	ASCE7-16 Clause 12.9.1.4.2

Equivalent Lateral Force Procedure is permitted (ASCE7-16 12.8).

Reference	Level [ft, in]	Weight [kip]	Direction Dir1			Direction Dir2		
			C_V	F [kip]	Ecc [ft, in]	C_V	F [kip]	Ecc [ft, in]
St. 7 (T O SCREEN WALL)	65′ 2″	15.683	0.002	1.844	12′ 3 1/2″	0.002	1.844	10′ 4″
St. 6.2 (6.2)	57′ 10″	22.718	0.002	2.355	12′ 3 1/2″	0.002	2.355	10′ 4″

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

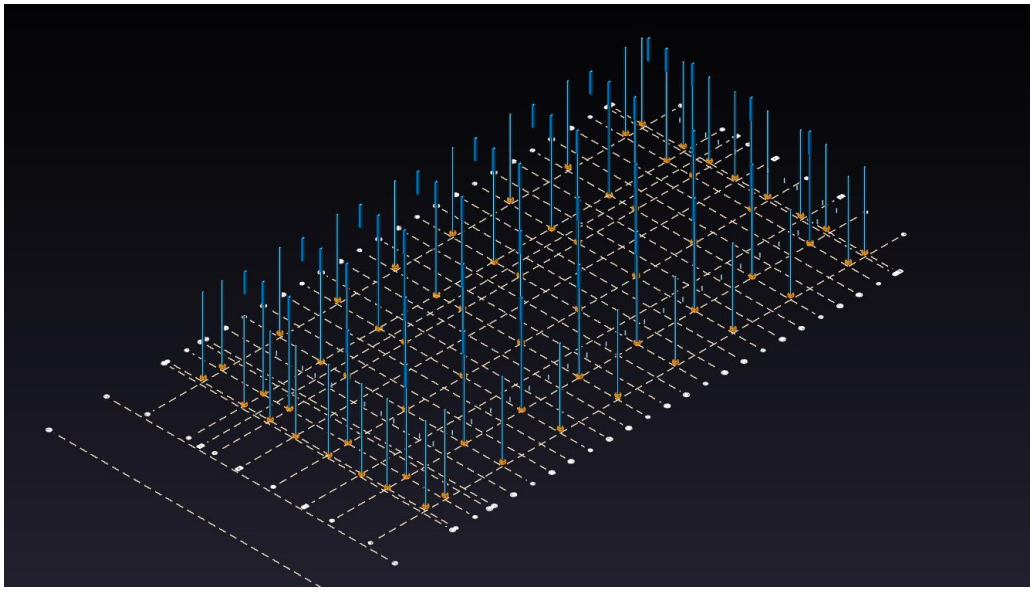
Reference	Level [ft, in]	Weight [kip]	Direction Dir1			Direction Dir2		
			Cv	F [kip]	Ecc [ft, in]	Cv	F [kip]	Ecc [ft, in]
St. 6.1 (3.1 - PARAPET HT)	53′ 7″	21.084	0.002	2.017	12′ 3 1/2″	0.002	2.017	10′ 4″
St. 5 (5)	51′ 7″	2.456	0.000	0.226	12′ 3 1/2″	0.000	0.226	10′ 4″
St. 4.42 (OFFICE ROOF TSD)	50′ 7″	8196.562	0.611	737.876	12′ 3 1/2″	0.611	737.876	10′ 4″
	44′ 0″	11.464	0.001	0.891	12′ 3 1/2″	0.001	0.891	10′ 4″
	43′ 10″	5.392	0.000	0.418	12′ 3 1/2″	0.000	0.418	10′ 4″
St. 3.2 (3.2)	34′ 0″	3.022	0.000	0.179	12′ 3 1/2″	0.000	0.179	10′ 4″
St. 3 (2.0 - SECOND FLOOR FRAMING	27′ 4 1/2″	9733.089	0.382	461.540	12′ 3 1/2″	0.382	461.540	10′ 4″
St. 2.1 (2.1)	17′ 1/4″	5.617	0.000	0.164	12′ 3 1/2″	0.000	0.164	10′ 4″
	17′ 0″	11.406	0.000	0.332	12′ 3 1/2″	0.000	0.332	10′ 4″
St. 2 (VESTIBULE FRAMING PLAN)	9′ 2 1/2″	4.216	0.000	0.067	12′ 3 1/2″	0.000	0.067	10′ 4″
St. 1.2 (1.2)	9′ 0″	2.637	0.000	0.041	12′ 3 1/2″	0.000	0.041	10′ 4″

Analysis procedure to be used:

Equivalent Lateral Force Procedure

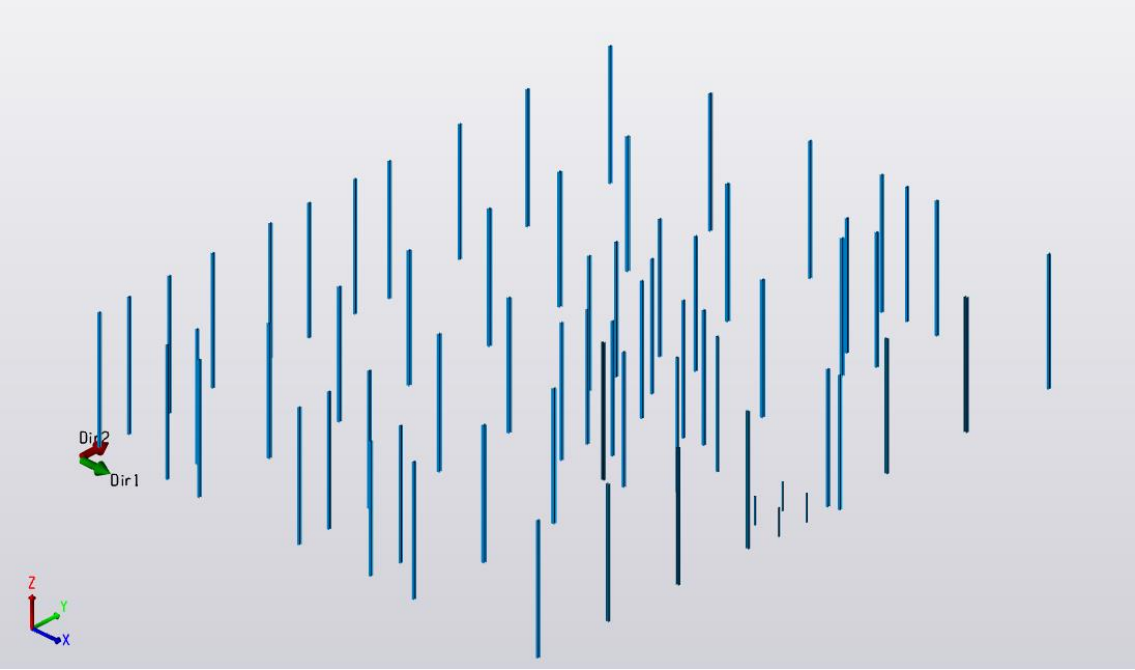
Column Locations

a) Data Hall (DH2)



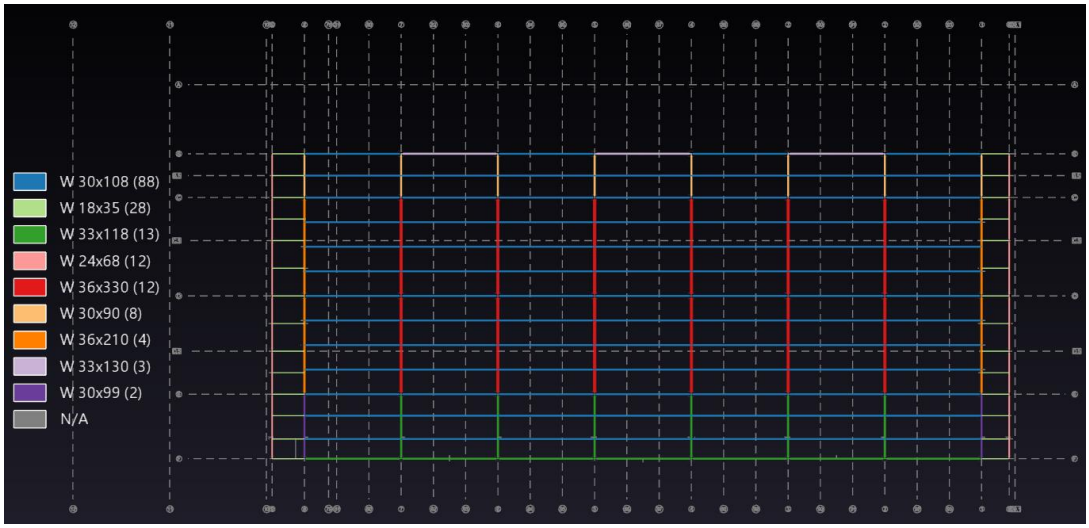
b) Office

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND



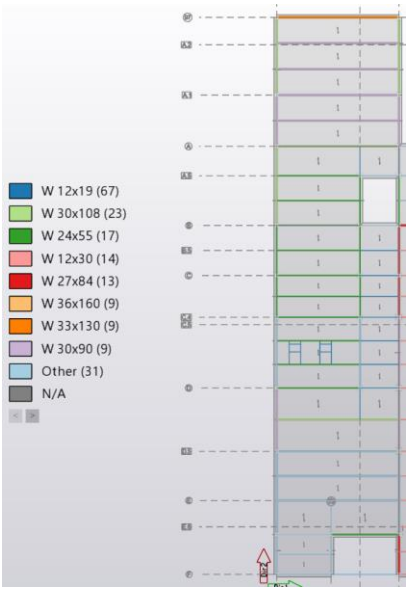
Second Floor Beam Sizes

a) Data Hall (DH2)

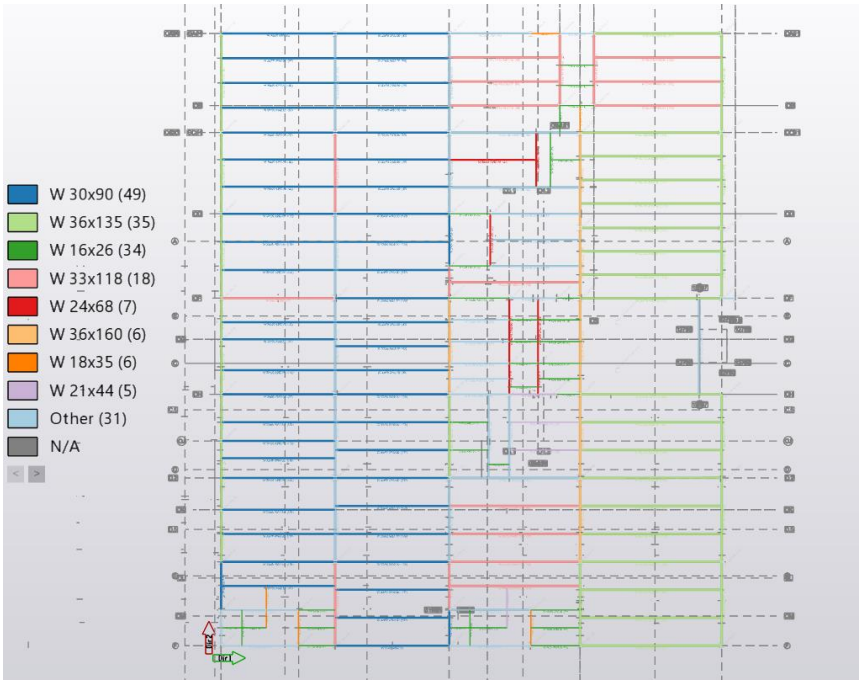


PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

b) Data Hall – (BOH)



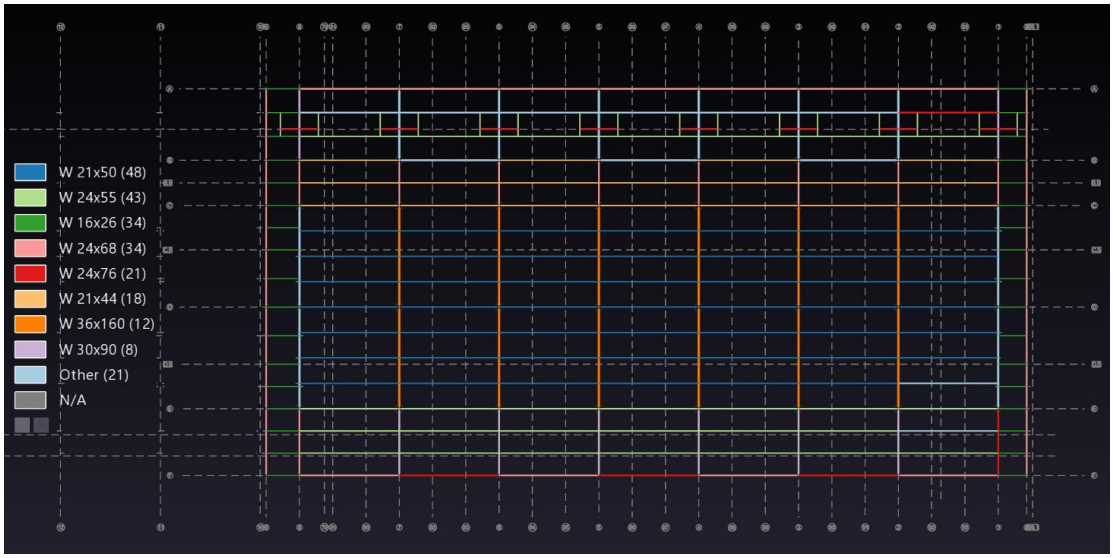
c) Office



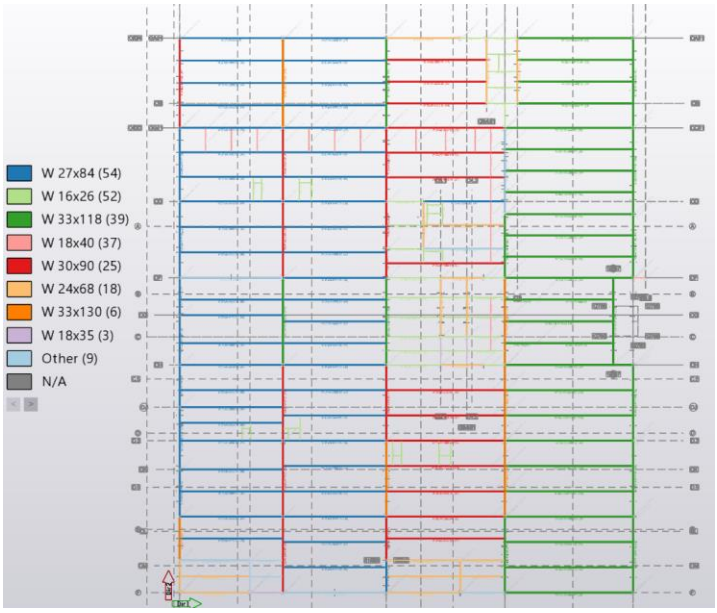
PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

Roof Floor Beam Sizes

a) Data Hall (DH2)



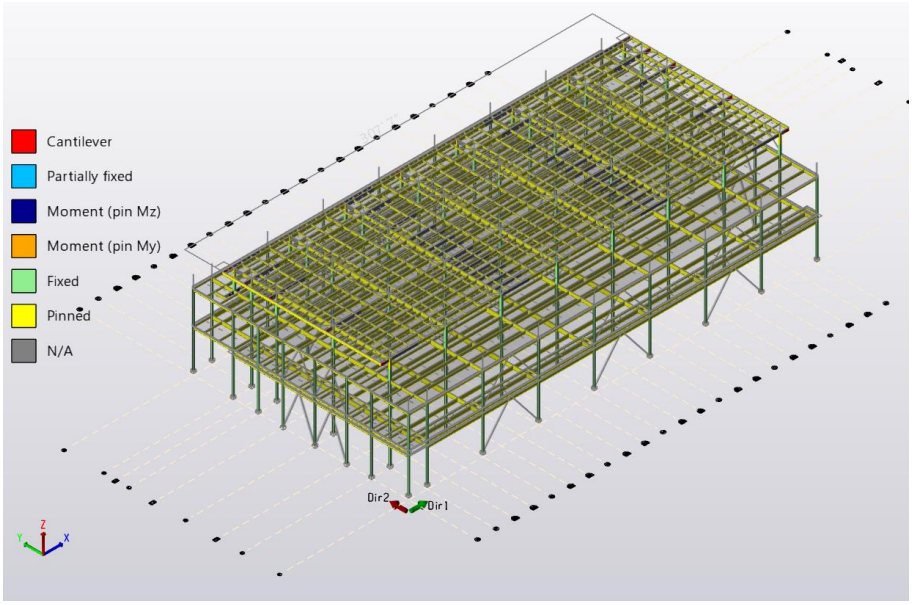
b) Office



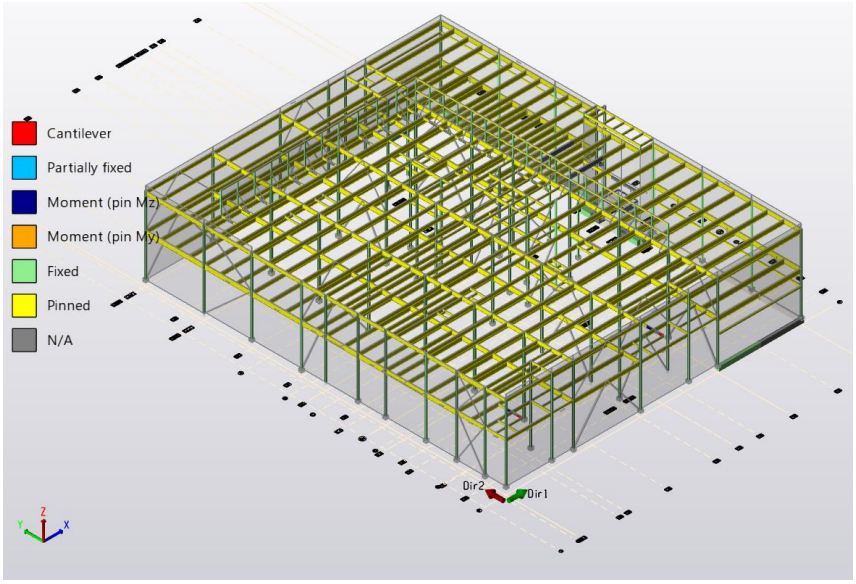
PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

End Beams-Columns Fixities

a) Data Hall (DH2)



b) Office



PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

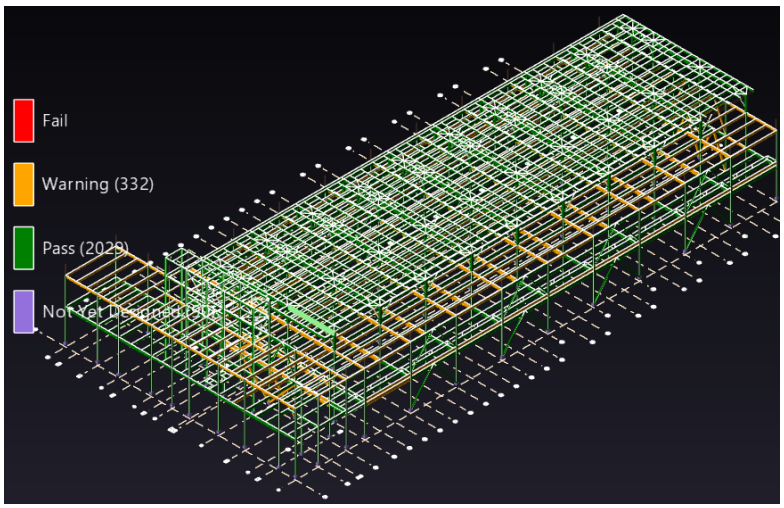
3.0 TSD MODEL RESULTS: DATA HALL

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

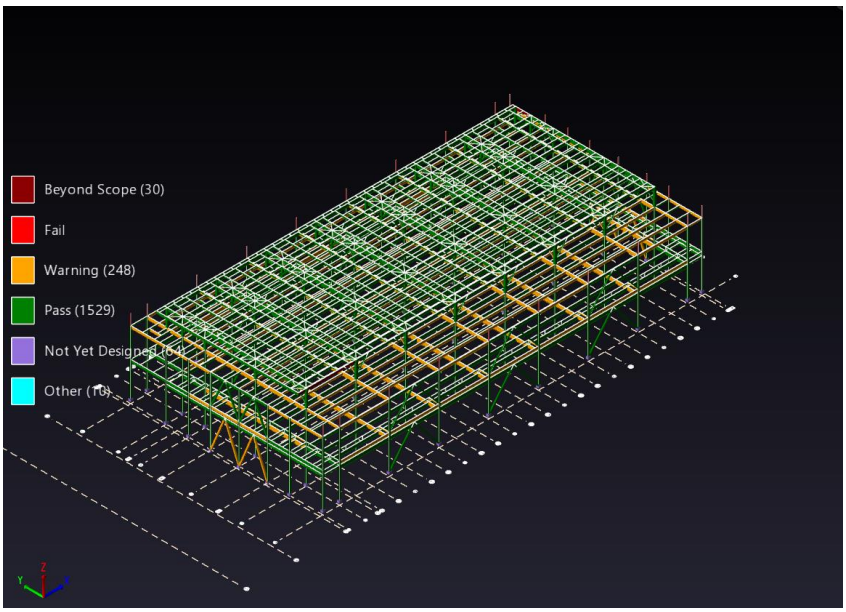
Building Status and Utilization Ratio

Members Passed the Design Criteria, and the Utility Ratios Less Than 1, Hence Ok.

a) Building Status
   a. Data Hall 1



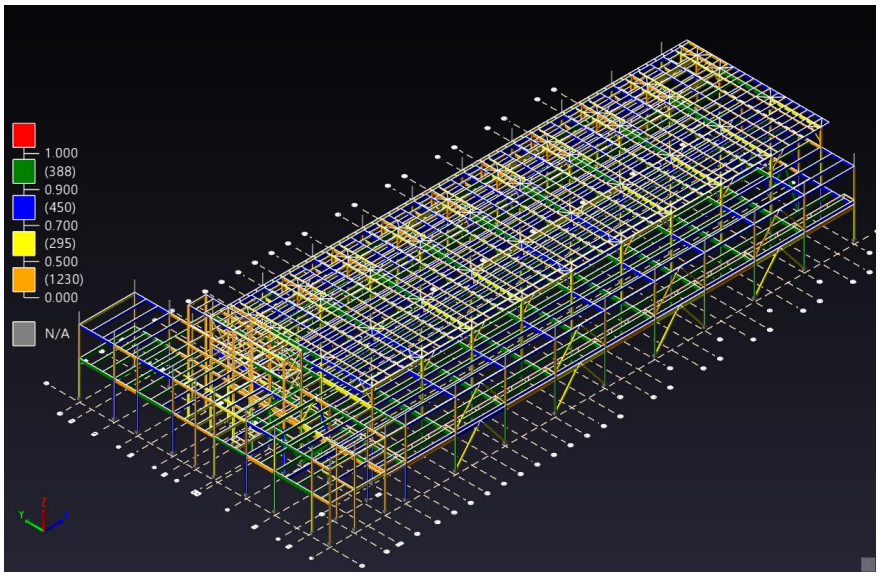
   b. Data Hall 3



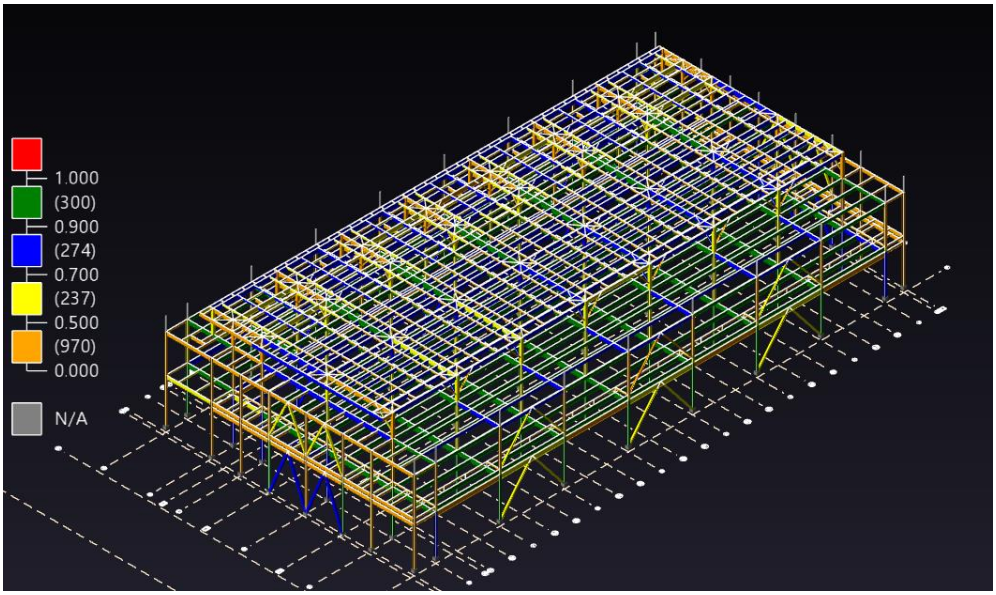
PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

b) Utility Ratio

a. Data Hall 1



b. Data Hall 3

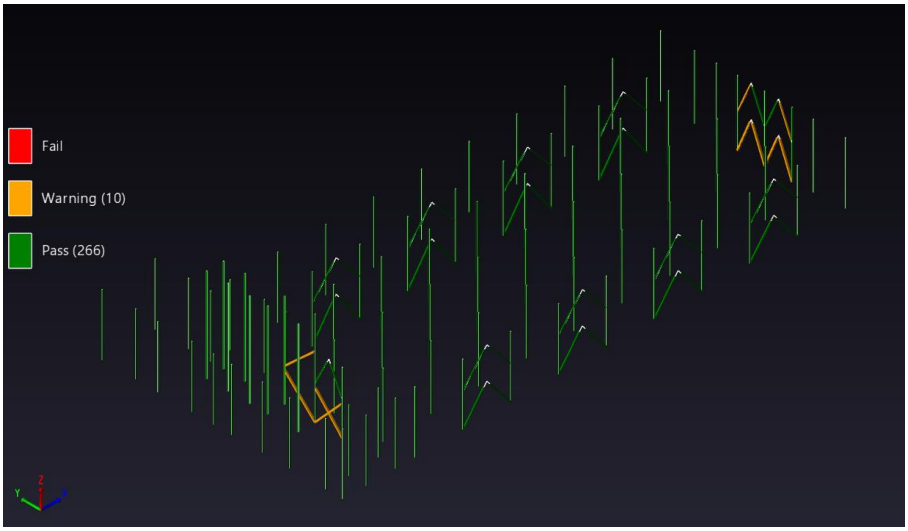


PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

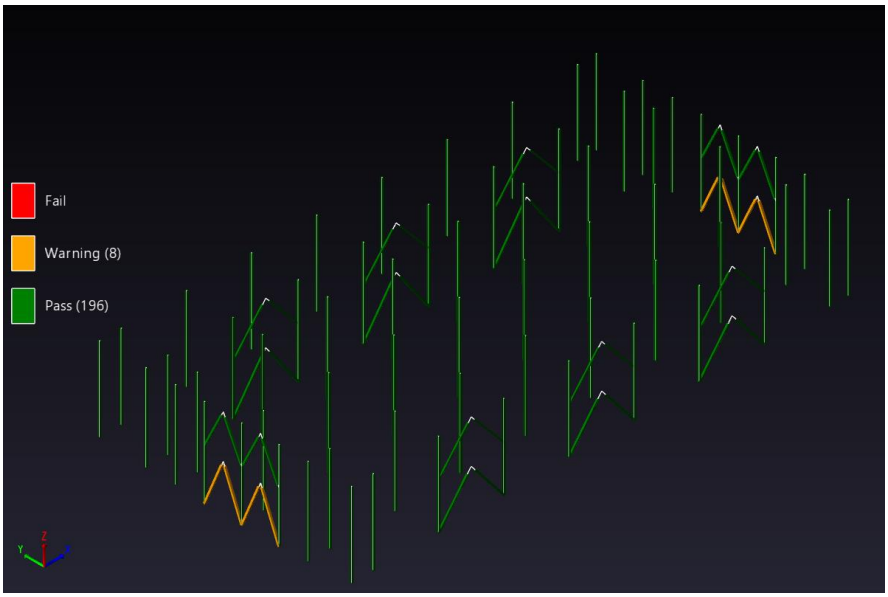
Columns Forces Analysis

Columns Passed the Design Criteria, and the Utility Ratios Less Than 1, Hence Ok.

a) Columns Status- Pass
   a. Data Hall 1



   b. Data Hall 3



PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

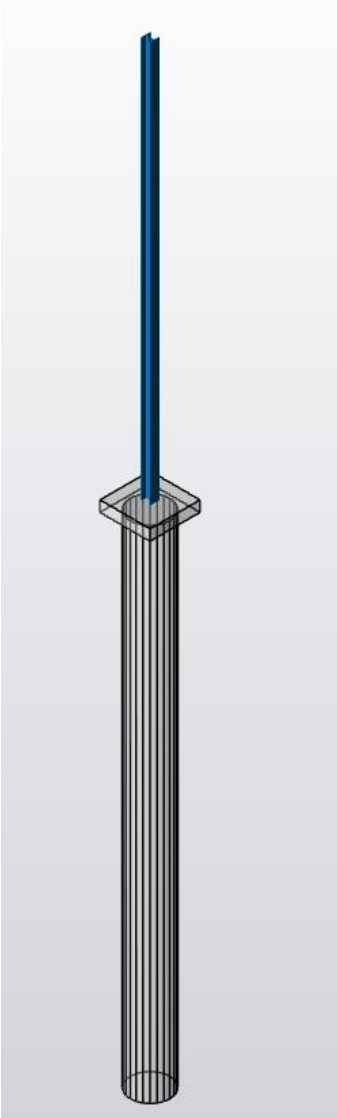
4.0 SAMPLE CALCULATIONS / ANALYSIS

NOTE: Sample calculations provided for review; additional supporting calculations are available upon request.

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

GRAVITY FOUNDATION ANALYSIS

- **Design Report Example for the following Gravity Foundation at Column - (D/12)**



PC 1

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

Applied Loads Summary

	Strength	**Service**
Analysis	3D Building Analysis	3D Building Analysis
Combination	270 Roof uplift deflection	85 (Final) $LRFD_{8.47}$-
P	0.000 kip	1509.047 kip
$F_{x,sup}$	0.000 kip	-0.328 kip
$F_{y,sup}$	0.000 kip	-0.301 kip
$M_{x,sup}$	0.000 kip ft	0.000 kip ft
$M_{y,sup}$	0.000 kip ft	0.000 kip ft
	Added Loads	
F_{swt}	11.025 kip	
F_{soil}	0.000 kip	
$F_{dl,sur}$	0.000 kip	
$F_{ll,sur}$	0.000 kip	

Foundation Details

Foundation Type	Pile Cap
Concrete Class	4000 psi
Size	7′ 0″ × 7′ 0″
Overall Depth	18.0000 in
Top Cover	1.5000 in
Bottom Cover	1.5000 in
Side Cover	1.5000 in
Depth From Surface	0.0000 in
Number of Piles	1
Pile Shape and Size	Circular 66 in
Pile Installation Type	Bored
Pile Length	70′ 0″ ft, in
Pile Edge Distance	9.0000 in
Pile Embedment	4.0000 in

Pile Axial Capacity

Summary

	Compression	**Tension**
Analysis	3D Building Analysis	3D Building Analysis
Combination	85 (Final) $LRFD_{8.47}$-	270 Roof uplift deflection
$P_{n,max}$	1520.072 kip	-85.132 kip
R	1782.000 kip	277.000 kip
Ratio	**0.853**	**0.307**

Pile Lateral Capacity

Summary

PROJECT QTS DEN1-DC2
DESCRIPTION
PROJECT # 24053.00000 DATE 05/30/2025
PREPARED BY LB CHECKED BY ND

	Lateral Resistance
Analysis	3D Building Analysis
Combination	220 (Final) LRFD$_{11.5}$-
H_F	6.971 kip
H_{Rd}	233.660 kip
Ratio	**0.030**

Bending Capacity - Critical

Summary

Direction	X	Y
Analysis	3D Building Analysis	3D Building Analysis
Combination	270 Roof uplift deflection	270 Roof uplift deflection
m_u	0.000 kip-ft/ft	0.000 kip-ft/ft
d	15.1875 in	16.0625 in
R_n / R_{nt}	**0.000**	**0.000**
$A_{s,reqd,bot}$	0.00 in^2/ft	0.00 in^2/ft
$A_{s,prov,bot}$	0.40 in^2/ft	0.40 in^2/ft
$A_{s,min,bot}$	0.39 in^2/ft	0.39 in^2/ft
Ratio	**0.000**	**0.000**
Top	-	-
Bottom	Deformed # 7 @ 18.0000-	Deformed # 7 @ 18.0000-

Shear Capacity

Summary

Direction	X	Y
Analysis	-	-
Combination	-	-
v_{su}	-	-
$\varphi \times v_c$	-	-
Ratio	Not required	Not required

Column Punching Shear

Summary

Analysis	Combination	Perimeter	b_s / b_o [in]	v_u [ksi]	v_n [ksi]	Ratio	Status
3D Building	1 Construction Stage	Loaded	-	-	-	-	Not required
3D Building	1 Construction Stage	Critical	-	-	-	-	Not required

Pile Punching Shear

Summary

b_o [in]	v_u [ksi]	v_n [ksi]	Ratio	Status
-	-	-	-	Not required

Pile One-Way Shear

Check Not Required

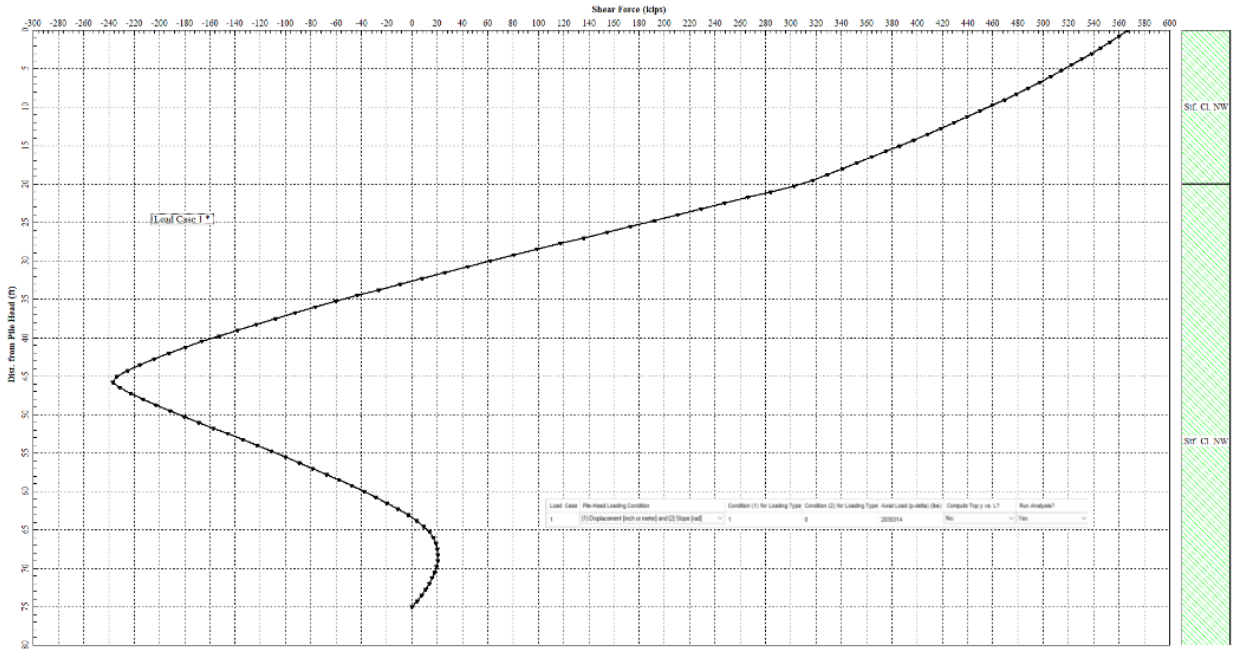
Check not required for any pile

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

5.0 FOUNDATION CALCULATIONS

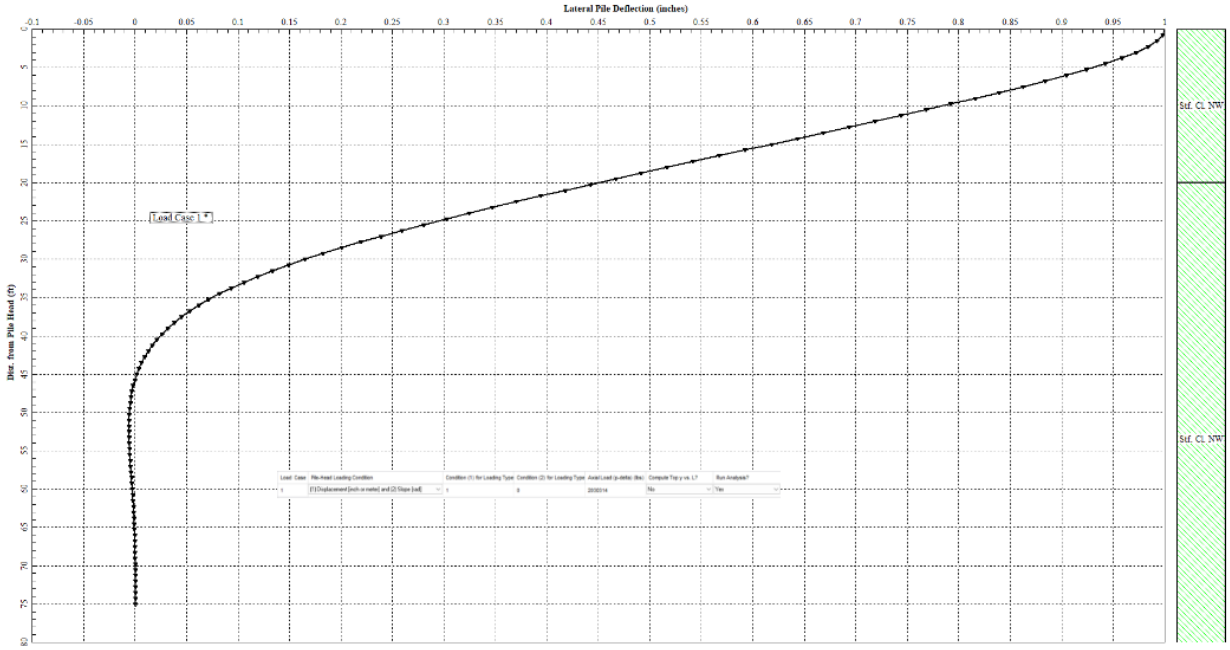
PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

L-Pile Result
P72



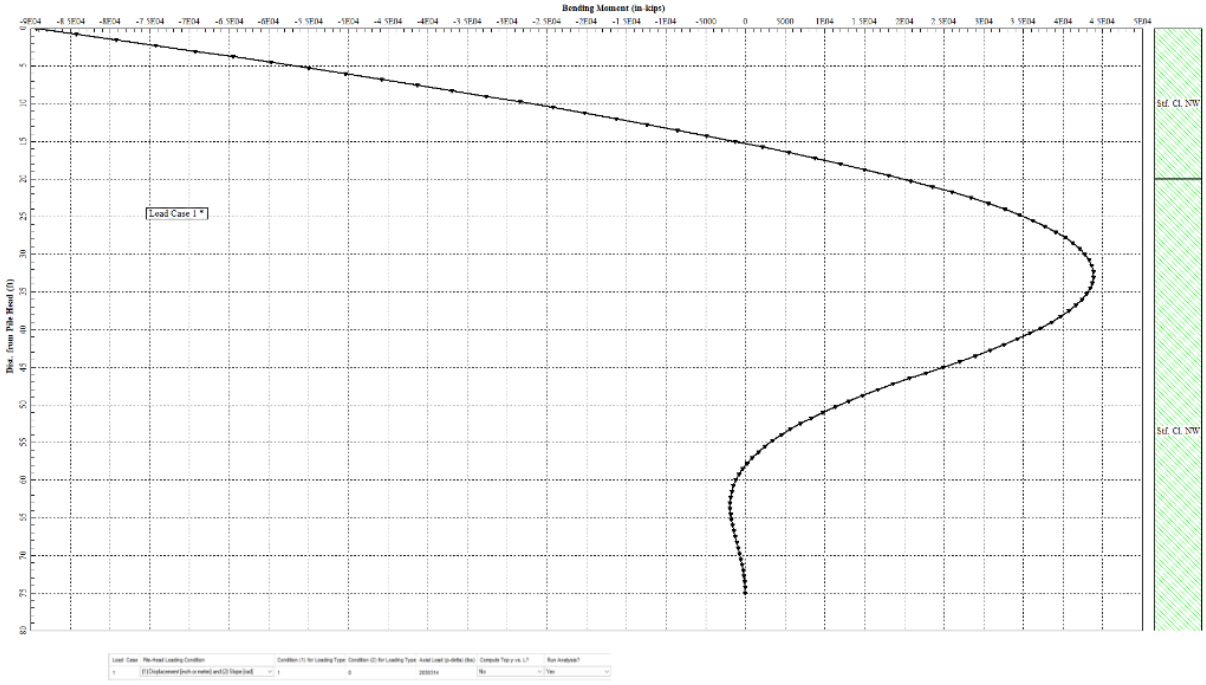
Shear vs. Depth Graph

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND



Deflection vs. Depth Graph

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND



Moment Capacity vs. Depth Graph

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

Pier Capacity

JOB: QTS DEN1-DC2

(SELF WEIGHT OF THE PIER IS NOT INCLUDED)

Depth to Bearing Strata =	20	ft.	
End Bearing Capacity =	25	ksf	
Skin Friction =	2	ksf	
Skin Friction min. depth =	15	ft.	Depth which is neglected in skin friction design.
Uplift Pressure =	0	ksf	
Uplift Resistance =	2	ksf	
Depth of uplift zone =	30	ft.	
Concrete Strength (f'c) =	4	ksi	
Reinforcing Yield (Fy) =	60	ksi	

Concrete Cost/CY =	$ 120.00
Soil Excavation Cost/CY =	$ 50.00
Rock Ecavation Cost/CY =	$ 55.00
Reinforcing Cost/LB =	$ 0.45

Minimum penetration required to offset soil uplift without any applied dead load = 15 ft.

PIER DIAMETER	24			30			36			42			48		
Vertical Bar Size	8			8			8			9			9		
No. of Vert. Bars	10			12			14			14			20		
Tie Size	3			3			3			3			3		
Tie Spacing	12			12			12			12			12		
Casing Cost	$7.00			$8.00			$9.00			$10.00			$11.00		
Penetration (ft.)	Capacity	Net Uplift	Pier Cost	Capacity	Net Uplift	Pier Cost	Capacity	Net Uplift	Pier Cost	Capacity	Net Uplift	Pier Cost	Capacity	Net Uplift	Pier Cost
15	79	170	$ 1,295	123	213	$ 1,803	177	255	$ 2,398	240	298	$ 3,157	314	340	$ 4,165
20	141	107	$ 1,462	201	134	$ 2,040	271	161	$ 2,718	350	188	$ 3,585	440	214	$ 4,735
25	204	44	$ 1,628	280	56	$ 2,277	365	67	$ 3,039	460	78	$ 4,012	565	89	$ 5,305
30	267	-18	$ 1,795	358	-15	$ 2,514	459	-10	$ 3,359	570	-32	$ 4,440	691	-37	$ 5,875
32	292	-20	$ 1,862	390	-15	$ 2,609	497	-10	$ 3,488	614	-39	$ 4,611	741	-87	$ 6,103
34	317	-20	$ 1,928	421	-15	$ 2,704	535	-10	$ 3,616	658	-39	$ 4,782	791	-137	$ 6,331
35	330	-20	$ 1,962	437	-15	$ 2,752	553	-10	$ 3,680	680	-39	$ 4,867	816	-140	$ 6,445
37	355	-20	$ 2,028	468	-15	$ 2,846	591	-10	$ 3,808	724	-39	$ 5,038	867	-140	$ 6,674
39	380	-20	$ 2,095	499	-15	$ 2,941	629	-10	$ 3,936	768	-39	$ 5,210	917	-140	$ 6,902
40	393	-20	$ 2,128	515	-15	$ 2,989	648	-10	$ 4,000	790	-39	$ 5,295	942	-140	$ 7,016
45	455	-20	$ 2,295	594	-15	$ 3,226	742	-10	$ 4,321	900	-39	$ 5,723	1068	-140	$ 7,586
47	480	-20	$ 2,362	625	-15	$ 3,321	780	-10	$ 4,449	944	-39	$ 5,894	1118	-140	$ 7,814
49	506	-20	$ 2,428	656	-15	$ 3,416	817	-10	$ 4,577	988	-39	$ 6,065	1168	-140	$ 8,042
50	518	-20	$ 2,462	672	-15	$ 3,463	836	-10	$ 4,642	1010	-39	$ 6,150	1193	-140	$ 8,156
52	543	-20	$ 2,528	704	-15	$ 3,558	874	-10	$ 4,770	1054	-39	$ 6,321	1243	-140	$ 8,384
55	581	-20	$ 2,628	751	-15	$ 3,701	930	-10	$ 4,962	1120	-39	$ 6,578	1319	-140	$ 8,726
57	606	-20	$ 2,695	782	-15	$ 3,795	968	-10	$ 5,090	1164	-39	$ 6,749	1369	-140	$ 8,954
59	631	-20	$ 2,762	813	-15	$ 3,890	1006	-10	$ 5,219	1208	-39	$ 6,920	1419	-140	$ 9,182
61	656	-20	$ 2,828	845	-15	$ 3,985	1043	-10	$ 5,347	1251	-39	$ 7,091	1470	-140	$ 9,410
65	707	-20	$ 2,962	908	-15	$ 4,175	1119	-10	$ 5,603	1339	-39	$ 7,433	1570	-140	$ 9,867
Pier Squash Load	723			1064			1473			1981			2626		
Pier Tensile Capacity		-190			-228			-265			-336			-480	
Max. Soil Induced Uplift		-170.0			-212.5			-255.0			-297.5			-340.0	

Note: A positive value in the "Net Uplift" column indicates the amount of dead load required on the pier to prevent the pier from pulling out of the bearing strata or to prevent exceeding the tensile capacity of the pier based on it's reinforcing - whichever controls.

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

Rebar Lookup		
Bar Size	Area	Weight
3	0.11	0.376
4	0.2	0.688
5	0.31	1.043
6	0.44	1.502
7	0.6	2.044
8	0.79	2.67
9	1	3.4
10	1.27	4.303
11	1.41	5.313
14	2.25	7.65
18	4	13.6

54		
10		
20		
3		
12		
$12.00		
Capacity	Net Uplift	Pier Cost
397	383	$ 5,223
539	241	$ 5,943
680	100	$ 6,663
821	-41	$ 7,384
878	-98	$ 7,672
934	-154	$ 7,960
963	-183	$ 8,104
1019	-227	$ 8,392
1076	-227	$ 8,680
1104	-227	$ 8,824
1245	-227	$ 9,545
1302	-227	$ 9,833
1358	-227	$ 10,121
1387	-227	$ 10,265
1443	-227	$ 10,553
1528	-227	$ 10,985
1584	-227	$ 11,273
1641	-227	$ 11,561
1697	-227	$ 11,849
1810	-227	$ 12,426
3326		
	-610	
	-382.5	

60		
10		
24		
3		
12		
$13.00		
Capacity	Net Uplift	Pier Cost
491	425	$ 6,355
648	268	$ 7,237
805	111	$ 8,118
962	-46	$ 8,999
1024	-109	$ 9,351
1087	-172	$ 9,704
1119	-203	$ 9,880
1181	-266	$ 10,233
1244	-307	$ 10,585
1276	-307	$ 10,761
1433	-307	$ 11,642
1495	-307	$ 11,995
1558	-307	$ 12,347
1590	-307	$ 12,524
1652	-307	$ 12,876
1747	-307	$ 13,405
1809	-307	$ 13,757
1872	-307	$ 14,110
1935	-307	$ 14,462
2061	-307	$ 15,167
4086		
	-732	
	-425.0	

66		
11		
22		
3		
12		
$14.00		
Capacity	Net Uplift	Pier Cost
594	468	$ 7,519
766	295	$ 8,565
939	122	$ 9,612
1112	-51	$ 10,659
1181	-120	$ 11,077
1250	-189	$ 11,496
1284	-223	$ 11,705
1354	-277	$ 12,124
1423	-277	$ 12,543
1457	-277	$ 12,752
1630	-277	$ 13,799
1699	-277	$ 14,218
1768	-277	$ 14,636
1803	-277	$ 14,846
1872	-277	$ 15,264
1975	-277	$ 15,892
2044	-277	$ 16,311
2113	-277	$ 16,730
2182	-277	$ 17,148
2321	-277	$ 17,986
4811		
	-744	
	-467.5	

72		
11		
26		
3		
12		
$15.00		
Capacity	Net Uplift	Pier Cost
707	510	$ 8,890
895	322	$ 10,132
1083	133	$ 11,374
1272	-55	$ 12,616
1347	-131	$ 13,113
1422	-206	$ 13,610
1460	-244	$ 13,858
1535	-319	$ 14,355
1611	-370	$ 14,852
1649	-370	$ 15,101
1837	-370	$ 16,343
1912	-370	$ 16,839
1988	-370	$ 17,336
2025	-370	$ 17,585
2214	-370	$ 18,827
2289	-370	$ 19,324
2364	-370	$ 19,821
2440	-370	$ 20,317
2591	-370	$ 21,311
5719		
	-880	
	-510.0	

72A		
14		
36		
3		
12		
$15.00		
Capacity	Net Uplift	Pier Cost
707	510	$ 11,052
895	322	$ 12,603
1083	133	$ 14,154
1272	-55	$ 15,705
1347	-131	$ 16,325
1422	-206	$ 16,945
1460	-244	$ 17,256
1535	-319	$ 17,876
1611	-394	$ 18,496
1649	-432	$ 18,807
1837	-620	$ 20,358
1912	-696	$ 20,978
1988	-771	$ 21,598
2025	-809	$ 21,908
2101	-884	$ 22,529
2214	-997	$ 23,459
2289	-1073	$ 24,080
2364	-1148	$ 24,700
2440	-1223	$ 25,321
2591	-1374	$ 26,561
6730		
	-1944	
	-510.0	

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

Project: QTS DEN1 DC2				Job Ref.: 24053.00000	
Section: GB1				Sheet no./rev.: 1	
Calc. by: LB	Date: 4/1/2025	Chk'd by:	Date:	App'd by:	Date:

RC BEAM ANALYSIS & DESIGN (ACI318-2019 (22))

In accordance with ACI318-2019 (22)

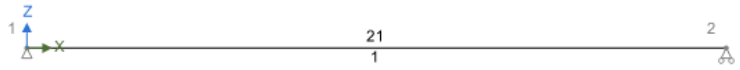
Tedds calculation version 3.3.21

ANALYSIS

Tedds calculation version 1.0.38

Geometry

Geometry (ft) - Concrete (ACI normal) - R 36x36



Span	Length (ft)	Section	Start Support	End Support
1	21	R 36x36	Pinned	Roller Pin X

R 36x36: Area 1296 in^2, Inertia Major 139968 in^4, Inertia Minor 139968 in^4, Shear area parallel to Minor 1080 in^2, Shear area parallel to Major 1080 in^2

Concrete (ACI normal): Density 150 lbm/ft^3, Youngs 4200 ksi, Shear 1750 ksi, Thermal 0.00001 $°C^{-1}$

Loading

Self weight included

Dead - Loading (kips/ft)



Load combination factors

Load combination	Self Weight	Dead	Live
1.2D + 1.6L (Strength)	1.20	1.20	1.60
1.0D + 1.0L (Service)	1.00	1.00	1.00
1.4D (Strength)	1.40	1.40	
1.0D (Service)	1.00	1.00	

Member Loads

Member	Load case	Load Type	Orientation	Description
Beam	Dead	UDL	GlobalZ	6.86 kips/ft

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

Project: QTS DEN1 DC2				Job Ref.: 24053.00000	
Section: GB1				Sheet no./rev.: 2	
Calc. by: LB	Date: 4/1/2025	Chk'd by:	Date:	App'd by:	Date:

Results

Reactions

Load case: Self Weight

Node	Force Fx (kips)	Force Fz (kips)	Moment My (kip_ft)
1	0	14.175	0
2	0	14.175	0

Load case: Dead

Node	Force Fx (kips)	Force Fz (kips)	Moment My (kip_ft)
1	0	72.056	0
2	0	72.056	0

Load case: Live

Node	Force Fx (kips)	Force Fz (kips)	Moment My (kip_ft)
1	0	0	0
2	0	0	0

Load combination: 1.2D + 1.6L (Strength)

Node	Force Fx (kips)	Force Fz (kips)	Moment My (kip_ft)
1	0	103.478	0
2	0	103.478	0

Load combination: 1.0D + 1.0L (Service)

Node	Force Fx (kips)	Force Fz (kips)	Moment My (kip_ft)
1	0	86.231	0
2	0	86.231	0

Load combination: 1.4D (Strength)

Node	Force Fx (kips)	Force Fz (kips)	Moment My (kip_ft)
1	0	120.724	0
2	0	120.724	0

Load combination: 1.0D (Service)

Node	Force Fx (kips)	Force Fz (kips)	Moment My (kip_ft)
1	0	86.231	0

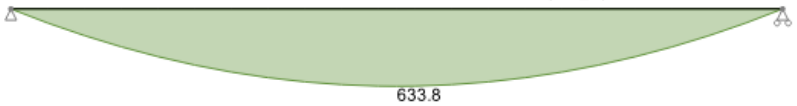
PROJECT	QTS DEN1-DC2			
DESCRIPTION				
PROJECT #	24053.00000	DATE	05/30/2025	
PREPARED BY	LB	CHECKED BY	ND	

Project: QTS DEN1 DC2					Job Ref.: 24053.00000	
Section: GB1					Sheet no./rev.: 3	
Calc. by: LB	Date: 4/1/2025	Chk'd by:	Date:	App'd by:	Date:	

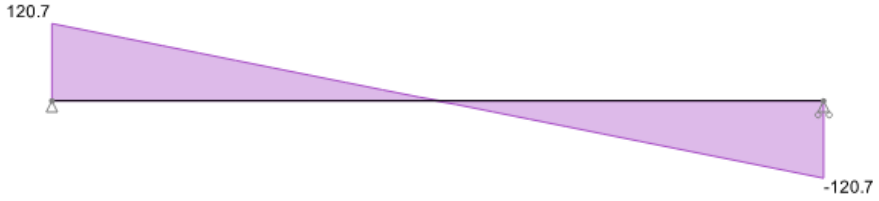
Node	Force		Moment
	Fx (kips)	Fz (kips)	My (kip_ft)
2	0	86.231	0

Forces

All combinations - Moment envelope (kip_ft)



All combinations - Shear envelope (kips)



Member results

Envelope - All combinations

Member	Position (ft)	Shear force (kips)		Moment (kip_ft)	
Beam	0	120.724 (max abs)	86.231	0 (min)	
	10.5	0		633.8 (max)	452.714
	21	-86.231	-120.724	0 (min)	

Concrete details

Compressive strength of concrete	f'_c = **4000** psi
Density of reinforced concrete	w_c = **150** lb / ft^3
Concrete type	Normal weight
Modulus of elasticity of concrete (cl.19.2.2.1)	$E = (w_c / 1\ lb/ft^3)^{1.5} \times 33\ psi \times (f'_c / 1\ psi)^{0.5}$ = **3834254** psi
Strength reduction factor for shear	ϕ_s = **0.75**

Reinforcement details

Yield strength of reinforcement	f_y = **60000** psi
Compression-controlled strain limit (cl.21.2.2.1)	ε_{ty} = **0.00200**

Nominal cover to reinforcement

Cover to top reinforcement	c_{nom_t} = **1.5** in
Cover to bottom reinforcement	c_{nom_b} = **3** in
Cover to side reinforcement	c_{nom_s} = **3** in

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

Project: QTS DEN1 DC2				Job Ref.: 24053.00000	
Section: GB1				Sheet no./rev.: 4	
Calc. by: LB	Date: 4/1/2025	Chk'd by:	Date:	App'd by:	Date:

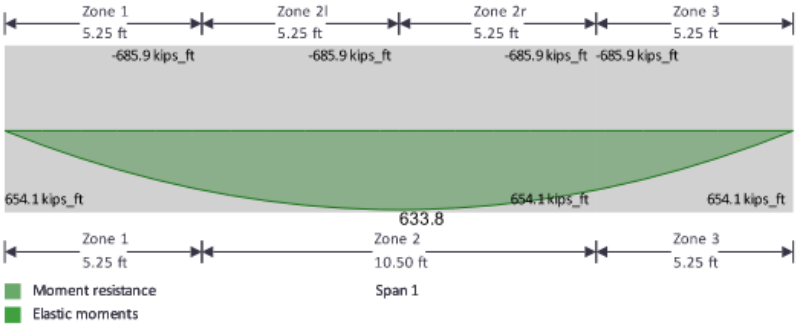
Beam - Span 1

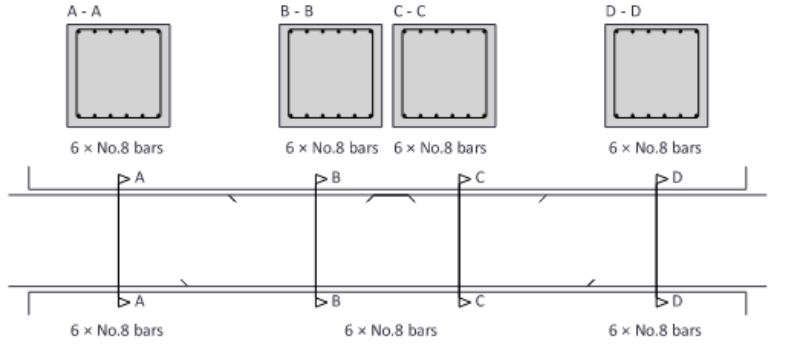
Rectangular section details

Section width — b = **36** in

Section depth — h = **36** in

Moment design





Zone 1 - Positive moment. Rectangular section in flexure (Section 9.5.2)

Factored bending moment at section	M_u = abs($M_{m1\_s1\_z1\_max\_red}$) = **475.350** kip_ft
Effective depth to tension reinforcement	d = **32** in
Tension reinforcement provided	6 × No.8 bars
Area of tension reinforcement provided	$A_{s,prov}$ = **4.712** in²
Minimum area of reinforcement (cl.9.6.1.2)	$A_{s,min}$ = max(3 psi × √(f'_c / 1 psi), 200 psi) × b × d / f_y = **3.840** in²
	PASS - Area of reinforcement provided is greater than minimum area of reinforcement required
Stress block depth factor (cl.22.2.2.4.3)	β_1 = min(max(0.85 - 0.05 × (f'_c - 4 ksi) / 1 ksi, 0.65), 0.85) = **0.85**
Depth of equivalent rectangular stress block	a = $A_{s,prov}$ × f_y / (0.85 × f'_c × b) = **2.31** in
Depth to neutral axis	c = a / β_1 = **2.718** in
Net tensile strain in extreme tension fibers	ε_t = 0.003 × (d_o - c) / max(c, 0.001 in) = **0.03232**
	Net tensile strain in tension controlled zone
Strength reduction factor (cl.21.2.1)	ϕ_t = min(max(0.65 + 0.25 × (ε_t - ε_{ty}) / (0.003), 0.65), 0.9) = **0.90**

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

Project: QTS DEN1 DC2				Job Ref.: 24053.00000	
Section: GB1				Sheet no./rev.: 5	
Calc. by: LB	Date: 4/1/2025	Chk'd by:	Date:	App'd by:	Date:

Nominal moment strength	$M_n = A_{s,prov} \times f_y \times (d - a / 2) =$ **726.768** kip_ft
Design moment strength	$\phi M_n = M_n \times \phi_f =$ **654.091** kip_ft
	PASS - Required moment strength is less than design moment strength

Flexural cracking

Max. center to center spacing of tension reinf.	$s_{b,max} = s_{bot} + \phi_{m1\_s1\_z1\_b\_L1} =$ **5.600** in
Service load stress in reinforcement (cl.24.3.2)	$f_s = 2/3 \times f_y =$ **40000** psi
Clear cover of reinforcement	$c_c = c_{nom\_b} + \phi_v =$ **3.500** in
Maximum allowable bot bar spacing (Table 24.3.2)	$s_{max} = \min(15in \times 40000psi / f_s - 2.5 \times c_c, 12in \times 40000psi / f_s) =$ **6.250** in
	PASS - Maximum allowable tension reinforcement spacing exceeds actual spacing

Spacing limits for reinforcement

Top bar clear spacing	$s_{top} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z1\_v}) + \phi_{m1\_s1\_z1\_t\_L1} \times N_{m1\_s1\_z1\_t\_L1} + \phi_{m1\_s1\_z2l\_t\_L1} \times N_{m1\_s1\_z2l\_t\_L1})) / ((N_{m1\_s1\_z1\_t\_L1} + N_{m1\_s1\_z2l\_t\_L1}) - 1) =$ **4.600** in
Min. allowable top bar clear spacing (cl.25.2.1)	$s_{top,min} =$ **1.000** in
Bottom bar clear spacing	$s_{bot} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z1\_v}) + \phi_{m1\_s1\_z1\_b\_L1} \times N_{m1\_s1\_z1\_b\_L1})) / (N_{m1\_s1\_z1\_b\_L1} - 1) =$ **4.600** in
Min. allowable bottom bar clear spacing (cl.25.2.1)	$s_{bot,min} =$ **1.000** in
	PASS - Actual bar spacing exceeds minimum allowable

Zone 2 - Positive moment. Rectangular section in flexure (Section 9.5.2)

Factored bending moment at section	$M_u = abs(M_{m1\_s1\_z2\_max\_red}) =$ **633.800** kip_ft
Effective depth to tension reinforcement	$d =$ **32** in
Tension reinforcement provided	6 × No.8 bars
Area of tension reinforcement provided	$A_{s,prov} =$ **4.712** in^2
Minimum area of reinforcement (cl.9.6.1.2)	$A_{s,min} = \max(3 psi \times \sqrt{(f'_c / 1 psi)}, 200 psi) \times b \times d / f_y =$ **3.840** in^2
	PASS - Area of reinforcement provided is greater than minimum area of reinforcement required
Stress block depth factor (cl.22.2.2.4.3)	$\beta_1 = \min(\max(0.85 - 0.05 \times (f'_c - 4 ksi) / 1 ksi, 0.65), 0.85) =$ **0.85**
Depth of equivalent rectangular stress block	$a = A_{s,prov} \times f_y / (0.85 \times f'_c \times b) =$ **2.31** in
Depth to neutral axis	$c = a / \beta_1 =$ **2.718** in
Net tensile strain in extreme tension fibers	$\varepsilon_t = 0.003 \times (d_o - c) / \max(c, 0.001 in) =$ **0.03232**
	Net tensile strain in tension controlled zone
Strength reduction factor (cl.21.2.1)	$\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.003), 0.65), 0.9) =$ **0.90**
Nominal moment strength	$M_n = A_{s,prov} \times f_y \times (d - a / 2) =$ **726.768** kip_ft
Design moment strength	$\phi M_n = M_n \times \phi_f =$ **654.091** kip_ft
	PASS - Required moment strength is less than design moment strength

Flexural cracking

Max. center to center spacing of tension reinf.	$s_{b,max} = s_{bot} + \phi_{m1\_s1\_z2\_b\_L1} =$ **5.600** in
Service load stress in reinforcement (cl.24.3.2)	$f_s = 2/3 \times f_y =$ **40000** psi
Clear cover of reinforcement	$c_c = c_{nom\_b} + \phi_v =$ **3.500** in
Maximum allowable bot bar spacing (Table 24.3.2)	$s_{max} = \min(15in \times 40000psi / f_s - 2.5 \times c_c, 12in \times 40000psi / f_s) =$ **6.250** in
	PASS - Maximum allowable tension reinforcement spacing exceeds actual spacing

Control of deflections (Section 24.2)

Concrete density factor	$K_w =$ **1.00**
Reinforcement yield strength factor	$K_f = 0.4 + f_y / 100000 psi =$ **1.00**

PROJECT QTS DEN1-DC2
DESCRIPTION
PROJECT # 24053.00000 DATE 05/30/2025
PREPARED BY LB CHECKED BY ND

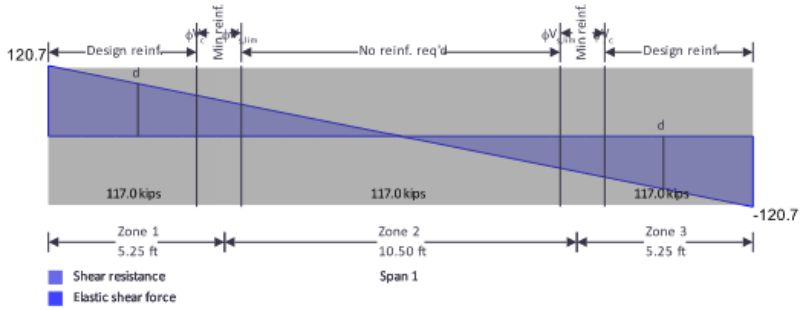
Project				Job Ref.	
QTS DEN1 DC2				24053.00000	
Section				Sheet no./rev.	
GB1				6	
Calc. by	Date	Chk'd by	Date	App'd by	Date
LB	4/1/2025				

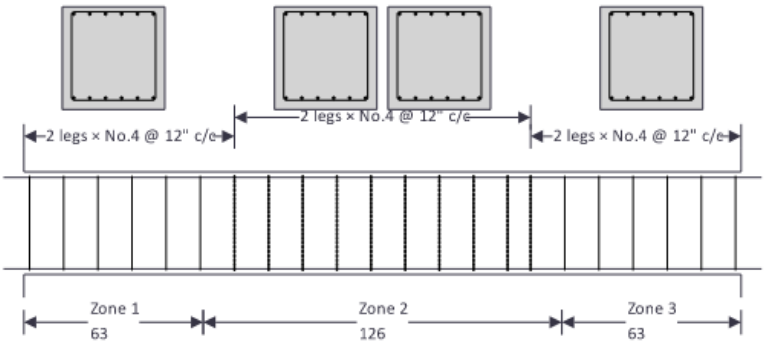
Minimum thickness of beam (Table 9.3.1.1)	$h_{min} = (L_{m1\_s1} / 16.0) \times K_w \times K_f =$ **15.75** in
	PASS - Thickness of beam exceeds minimum thickness
Spacing limits for reinforcement	
Top bar clear spacing	$s_{top} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z2\_v}) + \phi_{m1\_s1\_z2r\_t\_L1} \times N_{m1\_s1\_z2r\_t\_L1})) / (N_{m1\_s1\_z2r\_t\_L1} - 1) =$ **4.600** in
Min. allowable top bar clear spacing (cl.25.2.1)	$s_{top,min} =$ **1.000** in
Bottom bar clear spacing	$s_{bot} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z2\_v}) + \phi_{m1\_s1\_z2\_b\_L1} \times N_{m1\_s1\_z2\_b\_L1} + \phi_{m1\_s1\_z1\_b\_L1} \times N_{m1\_s1\_z1\_b\_L1})) / ((N_{m1\_s1\_z2\_b\_L1} + N_{m1\_s1\_z1\_b\_L1}) - 1) =$ **4.600** in
Min. allowable bottom bar clear spacing (cl.25.2.1)	$s_{bot,min} =$ **1.000** in
	PASS - Actual bar spacing exceeds minimum allowable
Zone 3 - Positive moment. Rectangular section in flexure (Section 9.5.2)	
Factored bending moment at section	$M_u = abs(M_{m1\_s1\_z3\_max\_red}) =$ **475.350** kip_ft
Effective depth to tension reinforcement	$d =$ **32** in
Tension reinforcement provided	6 × No.8 bars
Area of tension reinforcement provided	$A_{s,prov} =$ **4.712** in^2
Minimum area of reinforcement (cl.9.6.1.2)	$A_{s,min} = max(3\ psi \times \sqrt{f'_c} / 1\ psi, 200\ psi) \times b \times d / f_y =$ **3.840** in^2
	PASS - Area of reinforcement provided is greater than minimum area of reinforcement required
Stress block depth factor (cl.22.2.2.4.3)	$\beta_1 = min(max(0.85 - 0.05 \times (f'_c - 4\ ksi) / 1\ ksi, 0.65), 0.85) =$ **0.85**
Depth of equivalent rectangular stress block	$a = A_{s,prov} \times f_y / (0.85 \times f'_c \times b) =$ **2.31** in
Depth to neutral axis	$c = a / \beta_1 =$ **2.718** in
Net tensile strain in extreme tension fibers	$\varepsilon_t = 0.003 \times (d_o - c) / max(c, 0.001\ in) =$ **0.03232**
	Net tensile strain in tension controlled zone
Strength reduction factor (cl.21.2.1)	$\phi_f = min(max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.003), 0.65), 0.9) =$ **0.90**
Nominal moment strength	$M_n = A_{s,prov} \times f_y \times (d - a / 2) =$ **726.768** kip_ft
Design moment strength	$\phi M_n = M_n \times \phi_f =$ **654.091** kip_ft
	PASS - Required moment strength is less than design moment strength
Flexural cracking	
Max. center to center spacing of tension reinf.	$s_{b,max} = s_{bot} + \phi_{m1\_s1\_z3\_b\_L1} =$ **5.600** in
Service load stress in reinforcement (cl.24.3.2)	$f_s = 2/3 \times f_y =$ **40000** psi
Clear cover of reinforcement	$c_c = c_{nom\_b} + \phi_v =$ **3.500** in
Maximum allowable bot bar spacing (Table 24.3.2)	$s_{max} = min(15in \times 40000psi / f_s - 2.5 \times c_c, 12in \times 40000psi / f_s) =$ **6.250** in
	PASS - Maximum allowable tension reinforcement spacing exceeds actual spacing
Spacing limits for reinforcement	
Top bar clear spacing	$s_{top} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z3\_v}) + \phi_{m1\_s1\_z3\_t\_L1} \times N_{m1\_s1\_z3\_t\_L1} + \phi_{m1\_s1\_z2r\_t\_L1} \times N_{m1\_s1\_z2r\_t\_L1})) / ((N_{m1\_s1\_z3\_t\_L1} + N_{m1\_s1\_z2r\_t\_L1}) - 1) =$ **4.600** in
Min. allowable top bar clear spacing (cl.25.2.1)	$s_{top,min} =$ **1.000** in
Bottom bar clear spacing	$s_{bot} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z3\_v}) + \phi_{m1\_s1\_z3\_b\_L1} \times N_{m1\_s1\_z3\_b\_L1})) / (N_{m1\_s1\_z3\_b\_L1} - 1) =$ **4.600** in
Min. allowable bottom bar clear spacing (cl.25.2.1)	$s_{bot,min} =$ **1.000** in
	PASS - Actual bar spacing exceeds minimum allowable

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

Project: QTS DEN1 DC2				Job Ref.: 24053.00000	
Section: GB1				Sheet no./rev.: 7	
Calc. by: LB	Date: 4/1/2025	Chk'd by:	Date:	App'd by:	Date:

Shear design





Rectangular section in shear

Concrete weight modification factor	λ = **1.00**
Location where min. reinf. is req'd (V_u less ϕV_c)	Between 4.42 ft and 16.58 ft
Location where no reinf. is req'd (V_u less $\phi V_{s,lim}$)	Between 5.75 ft and 15.25 ft
Maximum reinforcement shear strength	$\phi V_{s,max} = \phi_s \times 8$ psi $\times \sqrt{(\min(f'_c, 10000\text{psi}) / 1\text{ psi})} \times b \times d$ = **437.153** kips
Minimum area of shear reinf. (Table 9.6.3.4)	$A_{sv,min} = \max(50\text{ psi}, 0.75\text{ psi} \times \sqrt{(f'_c / 1\text{ psi})}) \times b / \min(f_y, 60000\text{ psi})$ = **0.360** in²/ft
No reinforcement shear strength limit	$\phi V_{s,lim} = \phi_s \times \lambda \times 1\text{psi} \times \sqrt{(\min(f'_c, 10000\text{psi}) / 1\text{ psi})} \times b \times d$ = **54.644** kips

Zone 1

Effective depth of long. reinf. used for shear zone	d = **32** in
Size effect factor (cl. 22.5.5.1.3)	$\lambda_s = \min(\sqrt{(2 / (1 + (d / 1\text{ in}) / 10))}, 1.0)$ = **0.69**
Area of longitudinal reinforcement	A_{st} = **4.712** in²
Ratio of longitudinal reinforcement	$\rho_w = A_{st} / (b \times d)$ = **0.004**
Concrete shear strength (Table 22.5.5.1)	$\phi V_c = \phi_s \times \min(8 \times \lambda \times (\rho_w)^{1/3}, 5 \times \lambda) \times \sqrt{(f'_c \times 1\text{ psi})} \times b \times d$ = **69.914** kips
Design shear force at 32.0in from support	V_u = **90.064** kips
Reinf. shear strength required (eqn. 22.5.1.1)	$\phi V_s = \max(V_u - \phi V_c, 0\text{ kips})$ = **20.150** kips
Area of design shear reinf. req'd (eqn. 22.5.8.5.3)	$A_{sv,des} = \phi V_s / (\phi_s \times \min(f_y, 60000\text{ psi}) \times d)$ = **0.168** in²/ft

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

Project: QTS DEN1 DC2				Job Ref.: 24053.00000	
Section: GB1				Sheet no./rev.: 8	
Calc. by: LB	Date: 4/1/2025	Chk'd by:	Date:	App'd by:	Date:

Area of shear reinforcement required	$A_{sv,req} = \max(A_{sv,min}, A_{sv,des}) =$ **0.360** in²/ft
Shear reinforcement provided	2 legs × No.4 @ 12" c/c
Area of shear reinforcement provided	$A_{sv,prov} =$ **0.393** in²/ft
	PASS - Area of shear reinforcement provided exceeds area of shear reinforcement required
Maximum longitudinal spacing (Table 9.7.6.2.2)	$s_{vl,max} = d / 2 =$ **16** in
	PASS - longitudinal spacing of stirrups is less than the maximum allowable

Zone 2

Effective depth of long. reinf. used for shear zone	d = **32** in
Size effect factor (cl. 22.5.5.1.3)	$\lambda_s = \min(\sqrt{(2 / (1 + (d / 1\text{ in}) / 10))}, 1.0) =$ **0.69**
Area of longitudinal reinforcement	$A_{st} =$ **4.712** in²
Ratio of longitudinal reinforcement	$\rho_w = A_{st} / (b \times d) =$ **0.004**
Concrete shear strength (Table 22.5.5.1)	$\phi V_c = \phi_s \times \min(8 \times \lambda \times (\rho_w)^{1/3}, 5 \times \lambda) \times \sqrt{(f'_c \times 1\text{ psi})} \times b \times d =$ **69.914** kips
Design shear force within zone	$V_u =$ **60.362** kips
Reinf. shear strength required (eqn. 22.5.1.1)	$\phi V_s = \max(V_u - \phi V_c, 0\text{ kips}) =$ **0.000** kips
Area of design shear reinf. req'd (eqn. 22.5.8.5.3)	$A_{sv,des} = \phi V_s / (\phi_s \times \min(f_y, 60000\text{ psi}) \times d) =$ **0.000** in²/ft
Area of shear reinforcement required	$A_{sv,req} = A_{sv,min} =$ **0.360** in²/ft
Shear reinforcement provided	2 legs × No.4 @ 12" c/c
Area of shear reinforcement provided	$A_{sv,prov} =$ **0.393** in²/ft
	PASS - Area of shear reinforcement provided exceeds area of shear reinforcement required
Maximum longitudinal spacing (Table 9.7.6.2.2)	$s_{vl,max} = \min(d / 2, 24\text{ in}) =$ **16** in
	PASS - longitudinal spacing of stirrups is less than the maximum allowable

Zone 3

Effective depth of long. reinf. used for shear zone	d = **32** in
Size effect factor (cl. 22.5.5.1.3)	$\lambda_s = \min(\sqrt{(2 / (1 + (d / 1\text{ in}) / 10))}, 1.0) =$ **0.69**
Area of longitudinal reinforcement	$A_{st} =$ **4.712** in²
Ratio of longitudinal reinforcement	$\rho_w = A_{st} / (b \times d) =$ **0.004**
Concrete shear strength (Table 22.5.5.1)	$\phi V_c = \phi_s \times \min(8 \times \lambda \times (\rho_w)^{1/3}, 5 \times \lambda) \times \sqrt{(f'_c \times 1\text{ psi})} \times b \times d =$ **69.914** kips
Design shear force at 32.0in from support	$V_u =$ **90.064** kips
Reinf. shear strength required (eqn. 22.5.1.1)	$\phi V_s = \max(V_u - \phi V_c, 0\text{ kips}) =$ **20.150** kips
Area of design shear reinf. req'd (eqn. 22.5.8.5.3)	$A_{sv,des} = \phi V_s / (\phi_s \times \min(f_y, 60000\text{ psi}) \times d) =$ **0.168** in²/ft
Area of shear reinforcement required	$A_{sv,req} = \max(A_{sv,min}, A_{sv,des}) =$ **0.360** in²/ft
Shear reinforcement provided	2 legs × No.4 @ 12" c/c
Area of shear reinforcement provided	$A_{sv,prov} =$ **0.393** in²/ft
	PASS - Area of shear reinforcement provided exceeds area of shear reinforcement required
Maximum longitudinal spacing (Table 9.7.6.2.2)	$s_{vl,max} = d / 2 =$ **16** in
	PASS - longitudinal spacing of stirrups is less than the maximum allowable

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

Project: QTS DEN1 DC2				Job Ref.: 24053.00000	
Section: GB2				Sheet no./rev.: 1	
Calc. by: LB	Date: 4/1/2025	Chk'd by:	Date:	App'd by:	Date:

RC BEAM ANALYSIS & DESIGN (ACI318-2019 (22))

In accordance with ACI318-2019 (22)

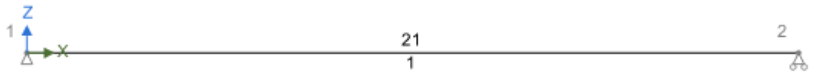
Tedds calculation version 3.3.21

ANALYSIS

Tedds calculation version 1.0.38

Geometry

Geometry (ft) - Concrete (ACI normal) - R 36x42



Span	Length (ft)	Section	Start Support	End Support
1	21	R 36x42	Pinned	Roller Pin X
R 36x42: Area 1512 in^2, Inertia Major 222264 in^4, Inertia Minor 163296 in^4, Shear area parallel to Minor 1260 in^2, Shear area parallel to Major 1260 in^2				
Concrete (ACI normal): Density 150 lbm/ft^3, Youngs 4200 ksi, Shear 1750 ksi, Thermal 0.00001 $°C^{-1}$				

Loading

Self weight included

Dead - Loading (kips/ft)



Load combination factors

Load combination	Self Weight	Dead	Live
1.2D + 1.6L (Strength)	1.20	1.20	1.60
1.0D + 1.0L (Service)	1.00	1.00	1.00
1.4D (Strength)	1.40	1.40	
1.0D (Service)	1.00	1.00	

Member Loads

Member	Load case	Load Type	Orientation	Description
Beam	Dead	UDL	GlobalZ	6.86 kips/ft

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

Project: QTS DEN1 DC2				Job Ref.: 24053.00000	
Section: GB2				Sheet no./rev.: 2	
Calc. by: LB	Date: 4/1/2025	Chk'd by:	Date:	App'd by:	Date:

Results

Reactions

Load case: Self Weight

Node	Force Fx (kips)	Force Fz (kips)	Moment My (kip_ft)
1	0	16.538	0
2	0	16.538	0

Load case: Dead

Node	Force Fx (kips)	Force Fz (kips)	Moment My (kip_ft)
1	0	72.056	0
2	0	72.056	0

Load case: Live

Node	Force Fx (kips)	Force Fz (kips)	Moment My (kip_ft)
1	0	0	0
2	0	0	0

Load combination: 1.2D + 1.6L (Strength)

Node	Force Fx (kips)	Force Fz (kips)	Moment My (kip_ft)
1	0	106.313	0
2	0	106.313	0

Load combination: 1.0D + 1.0L (Service)

Node	Force Fx (kips)	Force Fz (kips)	Moment My (kip_ft)
1	0	88.594	0
2	0	88.594	0

Load combination: 1.4D (Strength)

Node	Force Fx (kips)	Force Fz (kips)	Moment My (kip_ft)
1	0	124.031	0
2	0	124.031	0

Load combination: 1.0D (Service)

Node	Force Fx (kips)	Force Fz (kips)	Moment My (kip_ft)
1	0	88.594	0

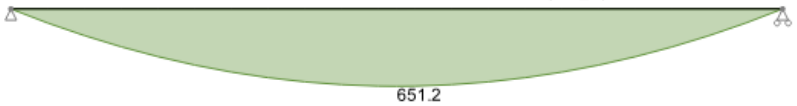
PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

Project: QTS DEN1 DC2				Job Ref.: 24053.00000	
Section: GB2				Sheet no./rev.: 3	
Calc. by: LB	Date: 4/1/2025	Chk'd by:	Date:	App'd by:	Date:

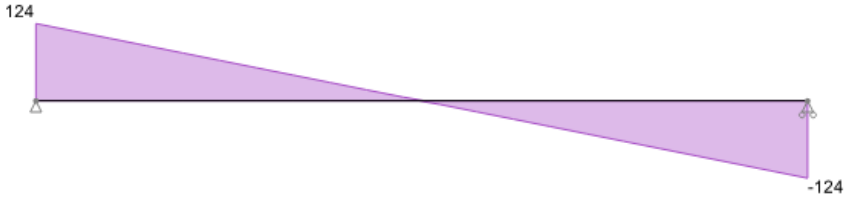
Node	Force		Moment
	Fx (kips)	Fz (kips)	My (kip_ft)
2	0	88.594	0

Forces

All combinations - Moment envelope (kip_ft)



All combinations - Shear envelope (kips)



Member results

Envelope - All combinations

Member	Position (ft)	Shear force (kips)		Moment (kip_ft)	
Beam	0	124.031 (max abs)	88.594	0	
	10.5	0		651.164 (max)	465.117
	21	-88.594	-124.031	0	

Concrete details

Compressive strength of concrete	f'_c = **4000** psi
Density of reinforced concrete	w_c = **150** lb / ft^3
Concrete type	Normal weight
Modulus of elasticity of concrete (cl.19.2.2.1)	$E = (w_c / 1\ lb/ft^3)^{1.5} \times 33\ psi \times (f'_c / 1\ psi)^{0.5}$ = **3834254** psi
Strength reduction factor for shear	ϕ_s = **0.75**

Reinforcement details

Yield strength of reinforcement	f_y = **60000** psi
Compression-controlled strain limit (cl.21.2.2.1)	ε_{ty} = **0.00200**

Nominal cover to reinforcement

Cover to top reinforcement	c_{nom_t} = **1.5** in
Cover to bottom reinforcement	c_{nom_b} = **3** in
Cover to side reinforcement	c_{nom_s} = **3** in

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

Project: QTS DEN1 DC2				Job Ref.: 24053.00000	
Section: GB2				Sheet no./rev.: 4	
Calc. by: LB	Date: 4/1/2025	Chk'd by:	Date:	App'd by:	Date:

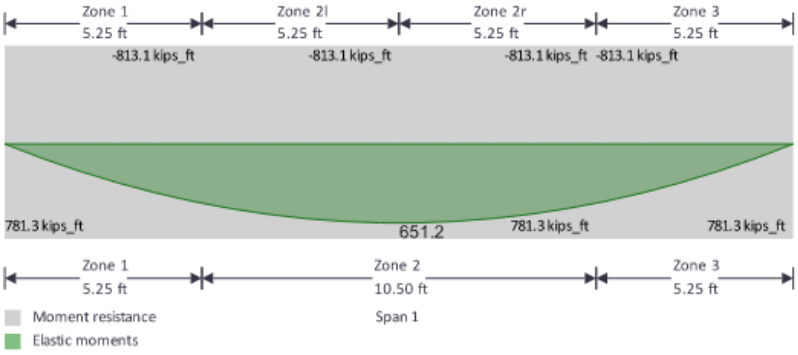
Beam - Span 1

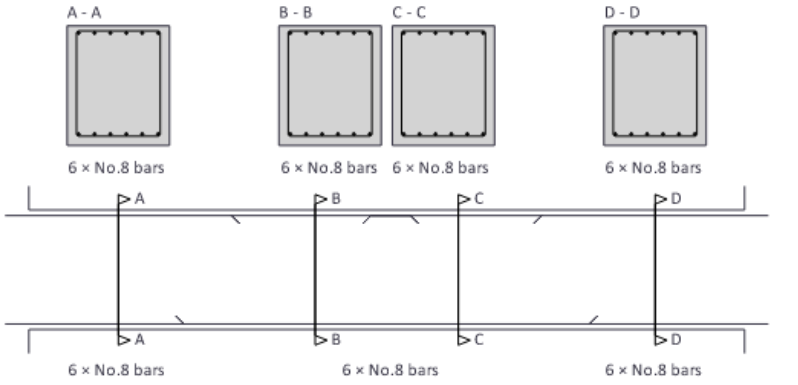
Rectangular section details

Section width — b = **36** in

Section depth — h = **42** in

Moment design





Zone 1 - Positive moment. Rectangular section in flexure (Section 9.5.2)

Factored bending moment at section	M_u = abs($M_{m1\_s1\_z1\_max\_red}$) = **488.373** kip_ft
Effective depth to tension reinforcement	d = **38** in
Tension reinforcement provided	6 × No.8 bars
Area of tension reinforcement provided	$A_{s,prov}$ = **4.712** in²
Minimum area of reinforcement (cl.9.6.1.2)	$A_{s,min}$ = max(3 psi × √(f'_c / 1 psi), 200 psi) × b × d / f_y = **4.560** in²

PASS - Area of reinforcement provided is greater than minimum area of reinforcement required

Stress block depth factor (cl.22.2.2.4.3)	β_1 = min(max(0.85 - 0.05 × (f'_c - 4 ksi) / 1 ksi, 0.65), 0.85) = **0.85**
Depth of equivalent rectangular stress block	a = $A_{s,prov}$ × f_y / (0.85 × f'_c × b) = **2.31** in
Depth to neutral axis	c = a / β_1 = **2.718** in

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

Project				Job Ref.	
QTS DEN1 DC2				24053.00000	
Section				Sheet no./rev.	
GB2				5	
Calc. by	Date	Chk'd by	Date	App'd by	Date
LB	4/1/2025				

Net tensile strain in extreme tension fibers	$\varepsilon_t = 0.003 \times (d_o - c) / \max(c, 0.001\text{ in}) =$ **0.03895**
	Net tensile strain in tension controlled zone
Strength reduction factor (cl.21.2.1)	$\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.003), 0.65), 0.9) =$ **0.90**
Nominal moment strength	$M_n = A_{s,prov} \times f_y \times (d - a / 2) =$ **868.140** kip_ft
Design moment strength	$\phi M_n = M_n \times \phi_f =$ **781.326** kip_ft
	PASS - Required moment strength is less than design moment strength
Flexural cracking	
Max. center to center spacing of tension reinf.	$s_{b,max} = s_{bot} + \phi_{m1\_s1\_z1\_b\_L1} =$ **5.600** in
Service load stress in reinforcement (cl.24.3.2)	$f_s = 2/3 \times f_y =$ **40000** psi
Clear cover of reinforcement	$c_c = c_{nom\_b} + \phi_v =$ **3.500** in
Maximum allowable bot bar spacing (Table 24.3.2)	$s_{max} = \min(15\text{in} \times 40000\text{psi} / f_s - 2.5 \times c_c, 12\text{in} \times 40000\text{psi} / f_s) =$ **6.250** in
	PASS - Maximum allowable tension reinforcement spacing exceeds actual spacing
Spacing limits for reinforcement	
Top bar clear spacing	$s_{top} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z1\_v}) + \phi_{m1\_s1\_z1\_t\_L1} \times N_{m1\_s1\_z1\_t\_L1} + \phi_{m1\_s1\_z2l\_t\_L1} \times N_{m1\_s1\_z2l\_t\_L1})) / ((N_{m1\_s1\_z1\_t\_L1} + N_{m1\_s1\_z2l\_t\_L1}) - 1) =$ **4.600** in
Min. allowable top bar clear spacing (cl.25.2.1)	$s_{top,min} =$ **1.000** in
Bottom bar clear spacing	$s_{bot} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z1\_v}) + \phi_{m1\_s1\_z1\_b\_L1} \times N_{m1\_s1\_z1\_b\_L1})) / (N_{m1\_s1\_z1\_b\_L1} - 1) =$ **4.600** in
Min. allowable bottom bar clear spacing (cl.25.2.1)	$s_{bot,min} =$ **1.000** in
	PASS - Actual bar spacing exceeds minimum allowable
Zone 2 - Positive moment. Rectangular section in flexure (Section 9.5.2)	
Factored bending moment at section	$M_u = \text{abs}(M_{m1\_s1\_z2\_max\_red}) =$ **651.164** kip_ft
Effective depth to tension reinforcement	d = **38** in
Tension reinforcement provided	6 × No.8 bars
Area of tension reinforcement provided	$A_{s,prov} =$ **4.712** in^2
Minimum area of reinforcement (cl.9.6.1.2)	$A_{s,min} = \max(3\text{ psi} \times \sqrt{(f'_c / 1\text{ psi})}, 200\text{ psi}) \times b \times d / f_y =$ **4.560** in^2
	PASS - Area of reinforcement provided is greater than minimum area of reinforcement required
Stress block depth factor (cl.22.2.2.4.3)	$\beta_1 = \min(\max(0.85 - 0.05 \times (f'_c - 4\text{ ksi}) / 1\text{ ksi}, 0.65), 0.85) =$ **0.85**
Depth of equivalent rectangular stress block	$a = A_{s,prov} \times f_y / (0.85 \times f'_c \times b) =$ **2.31** in
Depth to neutral axis	$c = a / \beta_1 =$ **2.718** in
Net tensile strain in extreme tension fibers	$\varepsilon_t = 0.003 \times (d_o - c) / \max(c, 0.001\text{ in}) =$ **0.03895**
	Net tensile strain in tension controlled zone
Strength reduction factor (cl.21.2.1)	$\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.003), 0.65), 0.9) =$ **0.90**
Nominal moment strength	$M_n = A_{s,prov} \times f_y \times (d - a / 2) =$ **868.140** kip_ft
Design moment strength	$\phi M_n = M_n \times \phi_f =$ **781.326** kip_ft
	PASS - Required moment strength is less than design moment strength
Flexural cracking	
Max. center to center spacing of tension reinf.	$s_{b,max} = s_{bot} + \phi_{m1\_s1\_z2\_b\_L1} =$ **5.600** in
Service load stress in reinforcement (cl.24.3.2)	$f_s = 2/3 \times f_y =$ **40000** psi
Clear cover of reinforcement	$c_c = c_{nom\_b} + \phi_v =$ **3.500** in
Maximum allowable bot bar spacing (Table 24.3.2)	$s_{max} = \min(15\text{in} \times 40000\text{psi} / f_s - 2.5 \times c_c, 12\text{in} \times 40000\text{psi} / f_s) =$ **6.250** in
	PASS - Maximum allowable tension reinforcement spacing exceeds actual spacing

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

Project: QTS DEN1 DC2				Job Ref.: 24053.00000	
Section: GB2				Sheet no./rev.: 6	
Calc. by: LB	Date: 4/1/2025	Chk'd by:	Date:	App'd by:	Date:

Control of deflections (Section 24.2)

Concrete density factor	K_w = **1.00**
Reinforcement yield strength factor	$K_f = 0.4 + f_y / 100000\ psi$ = **1.00**
Minimum thickness of beam (Table 9.3.1.1)	$h_{min} = (L_{m1\_s1} / 16.0) \times K_w \times K_f$ = **15.75** in
	PASS - Thickness of beam exceeds minimum thickness

Spacing limits for reinforcement

Top bar clear spacing	$s_{top} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z2\_v}) + \phi_{m1\_s1\_z2r\_t\_L1} \times N_{m1\_s1\_z2r\_t\_L1})) / (N_{m1\_s1\_z2r\_t\_L1} - 1)$ = **4.600** in
Min. allowable top bar clear spacing (cl.25.2.1)	$s_{top,min}$ = **1.000** in
Bottom bar clear spacing	$s_{bot} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z2\_v}) + \phi_{m1\_s1\_z2\_b\_L1} \times N_{m1\_s1\_z2\_b\_L1} + \phi_{m1\_s1\_z1\_b\_L1} \times N_{m1\_s1\_z1\_b\_L1})) / ((N_{m1\_s1\_z2\_b\_L1} + N_{m1\_s1\_z1\_b\_L1}) - 1)$ = **4.600** in
Min. allowable bottom bar clear spacing (cl.25.2.1)	$s_{bot,min}$ = **1.000** in
	PASS - Actual bar spacing exceeds minimum allowable

Zone 3 - Positive moment. Rectangular section in flexure (Section 9.5.2)

Factored bending moment at section	$M_u = abs(M_{m1\_s1\_z3\_max\_red})$ = **488.373** kip_ft
Effective depth to tension reinforcement	d = **38** in
Tension reinforcement provided	6 × No.8 bars
Area of tension reinforcement provided	$A_{s,prov}$ = **4.712** in^2
Minimum area of reinforcement (cl.9.6.1.2)	$A_{s,min} = max(3\ psi \times \sqrt{(f'_c / 1\ psi)}, 200\ psi) \times b \times d / f_y$ = **4.560** in^2
	PASS - Area of reinforcement provided is greater than minimum area of reinforcement required
Stress block depth factor (cl.22.2.2.4.3)	$\beta_1 = min(max(0.85 - 0.05 \times (f'_c - 4\ ksi) / 1\ ksi, 0.65), 0.85)$ = **0.85**
Depth of equivalent rectangular stress block	$a = A_{s,prov} \times f_y / (0.85 \times f'_c \times b)$ = **2.31** in
Depth to neutral axis	$c = a / \beta_1$ = **2.718** in
Net tensile strain in extreme tension fibers	$\varepsilon_t = 0.003 \times (d_o - c) / max(c, 0.001\ in)$ = **0.03895**
	Net tensile strain in tension controlled zone
Strength reduction factor (cl.21.2.1)	$\phi_f = min(max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.003), 0.65), 0.9)$ = **0.90**
Nominal moment strength	$M_n = A_{s,prov} \times f_y \times (d - a / 2)$ = **868.140** kip_ft
Design moment strength	$\phi M_n = M_n \times \phi_f$ = **781.326** kip_ft
	PASS - Required moment strength is less than design moment strength

Flexural cracking

Max. center to center spacing of tension reinf.	$s_{b,max} = s_{bot} + \phi_{m1\_s1\_z3\_b\_L1}$ = **5.600** in
Service load stress in reinforcement (cl.24.3.2)	$f_s = 2/3 \times f_y$ = **40000** psi
Clear cover of reinforcement	$c_c = c_{nom\_b} + \phi_v$ = **3.500** in
Maximum allowable bot bar spacing (Table 24.3.2)	$s_{max} = min(15in \times 40000psi / f_s - 2.5 \times c_c, 12in \times 40000psi / f_s)$ = **6.250** in
	PASS - Maximum allowable tension reinforcement spacing exceeds actual spacing

Spacing limits for reinforcement

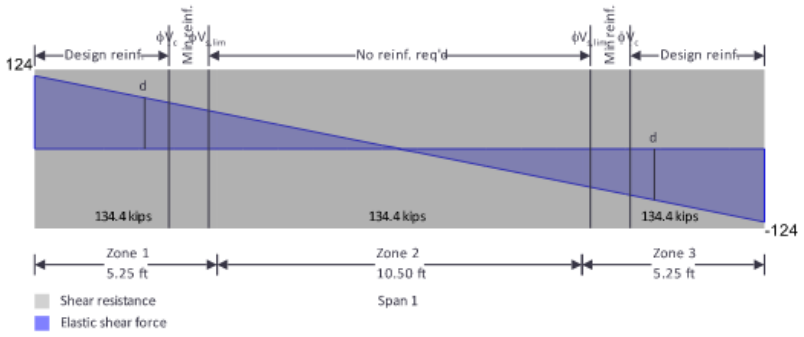
Top bar clear spacing	$s_{top} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z3\_v}) + \phi_{m1\_s1\_z3\_t\_L1} \times N_{m1\_s1\_z3\_t\_L1} + \phi_{m1\_s1\_z2r\_t\_L1} \times N_{m1\_s1\_z2r\_t\_L1})) / ((N_{m1\_s1\_z3\_t\_L1} + N_{m1\_s1\_z2r\_t\_L1}) - 1)$ = **4.600** in
Min. allowable top bar clear spacing (cl.25.2.1)	$s_{top,min}$ = **1.000** in
Bottom bar clear spacing	$s_{bot} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z3\_v}) + \phi_{m1\_s1\_z3\_b\_L1} \times N_{m1\_s1\_z3\_b\_L1})) / (N_{m1\_s1\_z3\_b\_L1} - 1)$ = **4.600** in
Min. allowable bottom bar clear spacing (cl.25.2.1)	$s_{bot,min}$ = **1.000** in

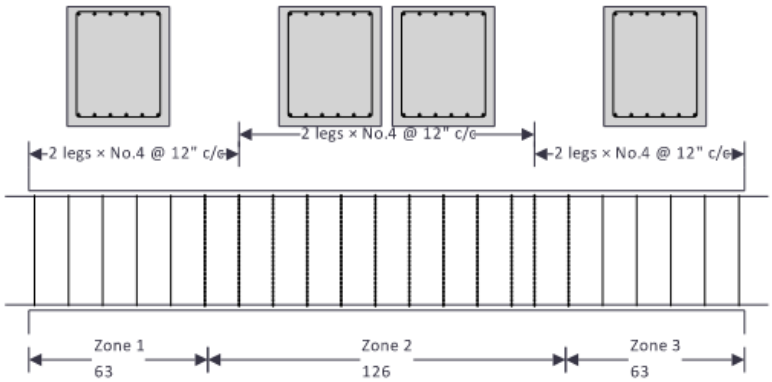
PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

Project: QTS DEN1 DC2				Job Ref.: 24053.00000	
Section: GB2				Sheet no./rev.: 7	
Calc. by: LB	Date: 4/1/2025	Chk'd by:	Date:	App'd by:	Date:

PASS - Actual bar spacing exceeds minimum allowable

Shear design





Rectangular section in shear

Concrete weight modification factor	λ = **1.00**
Location where min. reinf. is req'd (V_u less ϕV_c)	Between 3.86 ft and 17.14 ft
Location where no reinf. is req'd (V_u less $\phi V_{s,lim}$)	Between 5.01 ft and 15.99 ft
Maximum reinforcement shear strength	$\phi V_{s,max} = \phi_s \times 8$ psi $\times \sqrt{(\min(f'_c, 10000\text{psi}) / 1\text{ psi})} \times b \times d$ = **519.120** kips
Minimum area of shear reinf. (Table 9.6.3.4)	$A_{sv,min} = \max(50\text{ psi}, 0.75\text{ psi} \times \sqrt{(f'_c / 1\text{ psi})}) \times b / \min(f_y, 60000\text{ psi})$ = **0.360** in²/ft
No reinforcement shear strength limit	$\phi V_{s,lim} = \phi_s \times \lambda \times 1\text{psi} \times \sqrt{(\min(f'_c, 10000\text{psi}) / 1\text{ psi})} \times b \times d$ = **64.890** kips

Zone 1

Effective depth of long. reinf. used for shear zone	d = **38** in
Size effect factor (cl. 22.5.5.1.3)	$\lambda_s = \min(\sqrt{(2 / (1 + (d / 1\text{ in}) / 10))}, 1.0)$ = **0.65**
Area of longitudinal reinforcement	A_{st} = **4.712** in²
Ratio of longitudinal reinforcement	$\rho_w = A_{st} / (b \times d)$ = **0.003**
Concrete shear strength (Table 22.5.5.1)	$\phi V_c = \phi_s \times \min(8 \times \lambda \times (\rho_w)^{1/3}, 5 \times \lambda) \times \sqrt{(f'_c \times 1\text{ psi})} \times b \times d$ = **78.401** kips

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

Project: QTS DEN1 DC2				Job Ref.: 24053.00000	
Section: GB2				Sheet no./rev.: 8	
Calc. by: LB	Date: 4/1/2025	Chk'd by:	Date:	App'd by:	Date:

Design shear force at 38.0in from support	V_u = **86.625** kips
Reinf. shear strength required (eqn. 22.5.1.1)	$\phi V_s = \max(V_u - \phi V_c, 0 \text{ kips})$ = **8.224** kips
Area of design shear reinf. req'd (eqn. 22.5.8.5.3)	$A_{sv,des} = \phi V_s / (\phi_s \times \min(f_y, 60000 \text{ psi}) \times d)$ = **0.058** in²/ft
Area of shear reinforcement required	$A_{sv,req} = \max(A_{sv,min}, A_{sv,des})$ = **0.360** in²/ft
Shear reinforcement provided	2 legs × No.4 @ 12" c/c
Area of shear reinforcement provided	$A_{sv,prov}$ = **0.393** in²/ft
	PASS - Area of shear reinforcement provided exceeds area of shear reinforcement required
Maximum longitudinal spacing (Table 9.7.6.2.2)	$s_{vl,max} = d / 2$ = **19** in
	PASS - longitudinal spacing of stirrups is less than the maximum allowable

Zone 2

Effective depth of long. reinf. used for shear zone	d = **38** in
Size effect factor (cl. 22.5.5.1.3)	$\lambda_s = \min(\sqrt{(2 / (1 + (d / 1 \text{ in}) / 10))}, 1.0)$ = **0.65**
Area of longitudinal reinforcement	A_{st} = **4.712** in²
Ratio of longitudinal reinforcement	$\rho_w = A_{st} / (b \times d)$ = **0.003**
Concrete shear strength (Table 22.5.5.1)	$\phi V_c = \phi_s \times \min(8 \times \lambda \times (\rho_w)^{1/3}, 5 \times \lambda) \times \sqrt{(f'_c \times 1 \text{ psi})} \times b \times d$ = **78.401** kips
Design shear force within zone	V_u = **62.016** kips
Reinf. shear strength required (eqn. 22.5.1.1)	$\phi V_s = \max(V_u - \phi V_c, 0 \text{ kips})$ = **0.000** kips
Area of design shear reinf. req'd (eqn. 22.5.8.5.3)	$A_{sv,des} = \phi V_s / (\phi_s \times \min(f_y, 60000 \text{ psi}) \times d)$ = **0.000** in²/ft
Area of shear reinforcement required	$A_{sv,req}$ = 0mm²/ft = **0.000** in²/ft
Shear reinforcement provided	2 legs × No.4 @ 12" c/c
Area of shear reinforcement provided	$A_{sv,prov}$ = **0.393** in²/ft
	PASS - No shear reinforcement required ($\phi V_{s,lim}$ >= V_u)

Zone 3

Effective depth of long. reinf. used for shear zone	d = **38** in
Size effect factor (cl. 22.5.5.1.3)	$\lambda_s = \min(\sqrt{(2 / (1 + (d / 1 \text{ in}) / 10))}, 1.0)$ = **0.65**
Area of longitudinal reinforcement	A_{st} = **4.712** in²
Ratio of longitudinal reinforcement	$\rho_w = A_{st} / (b \times d)$ = **0.003**
Concrete shear strength (Table 22.5.5.1)	$\phi V_c = \phi_s \times \min(8 \times \lambda \times (\rho_w)^{1/3}, 5 \times \lambda) \times \sqrt{(f'_c \times 1 \text{ psi})} \times b \times d$ = **78.401** kips
Design shear force at 38.0in from support	V_u = **86.625** kips
Reinf. shear strength required (eqn. 22.5.1.1)	$\phi V_s = \max(V_u - \phi V_c, 0 \text{ kips})$ = **8.224** kips
Area of design shear reinf. req'd (eqn. 22.5.8.5.3)	$A_{sv,des} = \phi V_s / (\phi_s \times \min(f_y, 60000 \text{ psi}) \times d)$ = **0.058** in²/ft
Area of shear reinforcement required	$A_{sv,req} = \max(A_{sv,min}, A_{sv,des})$ = **0.360** in²/ft
Shear reinforcement provided	2 legs × No.4 @ 12" c/c
Area of shear reinforcement provided	$A_{sv,prov}$ = **0.393** in²/ft
	PASS - Area of shear reinforcement provided exceeds area of shear reinforcement required
Maximum longitudinal spacing (Table 9.7.6.2.2)	$s_{vl,max} = d / 2$ = **19** in
	PASS - longitudinal spacing of stirrups is less than the maximum allowable

PROJECT QTS DEN1-DC2
DESCRIPTION
PROJECT # 24053.00000 DATE 05/30/2025
PREPARED BY LB CHECKED BY ND

Project				Job Ref.	
QTS DEN1 DC2				24053.00000	
Section				Sheet no./rev.	
GB3				1	
Calc. by	Date	Chk'd by	Date	App'd by	Date
LB	4/1/2025				

RC BEAM ANALYSIS & DESIGN (ACI318-2019 (22))

In accordance with ACI318-2019 (22)

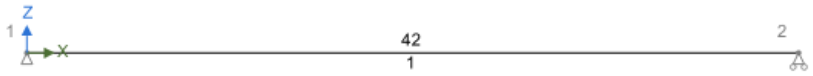
Tedds calculation version 3.3.21

ANALYSIS

Tedds calculation version 1.0.38

Geometry

Geometry (ft) - Concrete (ACI normal) - R 36x60



Span	Length (ft)	Section	Start Support	End Support
1	42	R 36x60	Pinned	Roller Pin X
R 36x60: Area 2160 in^2, Inertia Major 648000 in^4, Inertia Minor 233280 in^4, Shear area parallel to Minor 1800 in^2, Shear area parallel to Major 1800 in^2				
Concrete (ACI normal): Density 150 lbm/ft^3, Youngs 4200 ksi, Shear 1750 ksi, Thermal 0.00001 $°C^{-1}$				

Loading

Self weight included

Dead - Loading (kips/ft)



Load combination factors

Load combination	Self Weight	Dead	Live
1.2D + 1.6L (Strength)	1.20	1.20	1.60
1.0D + 1.0L (Service)	1.00	1.00	1.00
1.4D (Strength)	1.40	1.40	
1.0D (Service)	1.00	1.00	

Member Loads

Member	Load case	Load Type	Orientation	Description
Beam	Dead	UDL	GlobalZ	0.9 kips/ft

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

Project: QTS DEN1 DC2				Job Ref.: 24053.00000	
Section: GB3				Sheet no./rev.: 2	
Calc. by: LB	Date: 4/1/2025	Chk'd by:	Date:	App'd by:	Date:

Results

Reactions

Load case: Self Weight

Node	Force Fx (kips)	Force Fz (kips)	Moment My (kip_ft)
1	0	47.25	0
2	0	47.25	0

Load case: Dead

Node	Force Fx (kips)	Force Fz (kips)	Moment My (kip_ft)
1	0	18.9	0
2	0	18.9	0

Load case: Live

Node	Force Fx (kips)	Force Fz (kips)	Moment My (kip_ft)
1	0	0	0
2	0	0	0

Load combination: 1.2D + 1.6L (Strength)

Node	Force Fx (kips)	Force Fz (kips)	Moment My (kip_ft)
1	0	79.38	0
2	0	79.38	0

Load combination: 1.0D + 1.0L (Service)

Node	Force Fx (kips)	Force Fz (kips)	Moment My (kip_ft)
1	0	66.15	0
2	0	66.15	0

Load combination: 1.4D (Strength)

Node	Force Fx (kips)	Force Fz (kips)	Moment My (kip_ft)
1	0	92.61	0
2	0	92.61	0

Load combination: 1.0D (Service)

Node	Force Fx (kips)	Force Fz (kips)	Moment My (kip_ft)
1	0	66.15	0

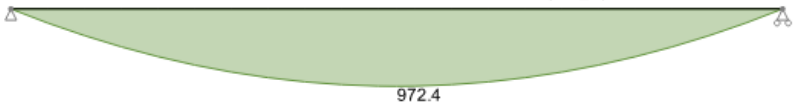
PROJECT	QTS DEN1-DC2			
DESCRIPTION				
PROJECT #	24053.00000	DATE	05/30/2025	
PREPARED BY	LB	CHECKED BY	ND	

Project: QTS DEN1 DC2					Job Ref.: 24053.00000	
Section: GB3					Sheet no./rev.: 3	
Calc. by: LB	Date: 4/1/2025	Chk'd by:	Date:	App'd by:	Date:	

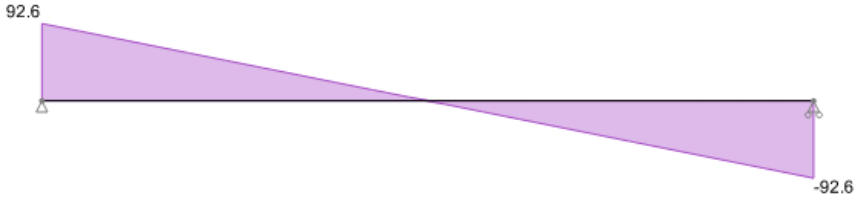
Node	Force		Moment
	Fx (kips)	Fz (kips)	My (kip_ft)
2	0	66.15	0

Forces

All combinations - Moment envelope (kip_ft)



All combinations - Shear envelope (kips)



Member results

Envelope - All combinations

Member	Position (ft)	Shear force (kips)		Moment (kip_ft)	
Beam	0	92.61 (max abs)	66.15	0 (min)	
	21	0		972.405 (max)	694.575
	42	-66.15	-92.61	0 (min)	

Concrete details

Compressive strength of concrete	f'_c = **4000** psi
Density of reinforced concrete	w_c = **150** lb / ft^3
Concrete type	Normal weight
Modulus of elasticity of concrete (cl.19.2.2.1)	$E = (w_c / 1\ lb/ft^3)^{1.5} \times 33\ psi \times (f'_c / 1\ psi)^{0.5}$ = **3834254** psi
Strength reduction factor for shear	ϕ_s = **0.75**

Reinforcement details

Yield strength of reinforcement	f_y = **60000** psi
Compression-controlled strain limit (cl.21.2.2.1)	ε_{ty} = **0.00200**

Nominal cover to reinforcement

Cover to top reinforcement	c_{nom_t} = **1.5** in
Cover to bottom reinforcement	c_{nom_b} = **3** in
Cover to side reinforcement	c_{nom_s} = **3** in

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

Project: QTS DEN1 DC2				Job Ref.: 24053.00000	
Section: GB3				Sheet no./rev.: 4	
Calc. by: LB	Date: 4/1/2025	Chk'd by:	Date:	App'd by:	Date:

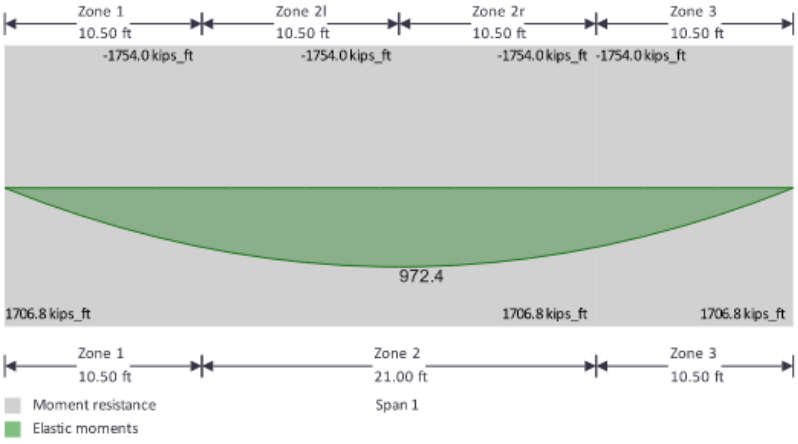
Beam - Span 1

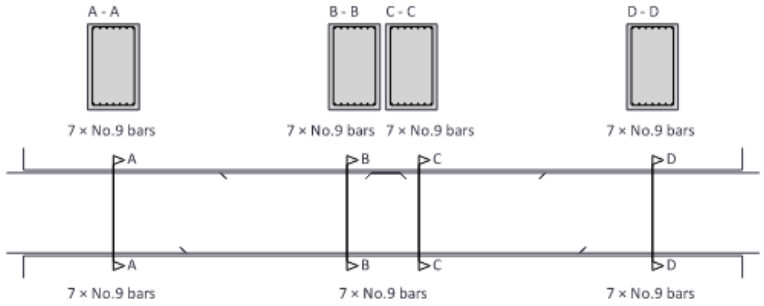
Rectangular section details

Section width — b = **36** in

Section depth — h = **60** in

Moment design





Zone 1 - Positive moment. Rectangular section in flexure (Section 9.5.2)

Factored bending moment at section — $M_u = \text{abs}(M_{m1\_s1\_z1\_max\_red})$ = **729.304** kip_ft

Effective depth to tension reinforcement — d = **55.936** in

Tension reinforcement provided — 7 × No.9 bars

Area of tension reinforcement provided — $A_{s,prov}$ = **6.995** in²

Minimum area of reinforcement (cl.9.6.1.2) — $A_{s,min} = \max(3\text{ psi} \times \sqrt{f'_c / 1\text{ psi}}, 200\text{ psi}) \times b \times d / f_y$ = **6.712** in²

PASS - Area of reinforcement provided is greater than minimum area of reinforcement required

Stress block depth factor (cl.22.2.2.4.3) — $\beta_1 = \min(\max(0.85 - 0.05 \times (f'_c - 4\text{ ksi}) / 1\text{ ksi}, 0.65), 0.85)$ = **0.85**

Depth of equivalent rectangular stress block — $a = A_{s,prov} \times f_y / (0.85 \times f'_c \times b)$ = **3.429** in

Depth to neutral axis — $c = a / \beta_1$ = **4.034** in

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

Project: QTS DEN1 DC2				Job Ref.: 24053.00000	
Section: GB3				Sheet no./rev.: 5	
Calc. by: LB	Date: 4/1/2025	Chk'd by:	Date:	App'd by:	Date:

Net tensile strain in extreme tension fibers	$\varepsilon_t = 0.003 \times (d_o - c) / \max(c, 0.001\text{ in}) =$ **0.0386**
	Net tensile strain in tension controlled zone
Strength reduction factor (cl.21.2.1)	$\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.003), 0.65), 0.9) =$ **0.90**
Nominal moment strength	$M_n = A_{s,prov} \times f_y \times (d - a / 2) =$ **1896.476** kip_ft
Design moment strength	$\phi M_n = M_n \times \phi_f =$ **1706.829** kip_ft
	PASS - Required moment strength is less than design moment strength
Flexural cracking	
Max. center to center spacing of tension reinf.	$s_{b,max} = s_{bot} + \phi_{m1\_s1\_z1\_b\_L1} =$ **4.645** in
Service load stress in reinforcement (cl.24.3.2)	$f_s = 2/3 \times f_y =$ **40000** psi
Clear cover of reinforcement	$c_c = c_{nom\_b} + \phi_v =$ **3.500** in
Maximum allowable bot bar spacing (Table 24.3.2)	$s_{max} = \min(15\text{in} \times 40000\text{psi} / f_s - 2.5 \times c_c, 12\text{in} \times 40000\text{psi} / f_s) =$ **6.250** in
	PASS - Maximum allowable tension reinforcement spacing exceeds actual spacing
Spacing limits for reinforcement	
Top bar clear spacing	$s_{top} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z1\_v}) + \phi_{m1\_s1\_z1\_t\_L1} \times N_{m1\_s1\_z1\_t\_L1} + \phi_{m1\_s1\_z2l\_t\_L1} \times N_{m1\_s1\_z2l\_t\_L1})) / ((N_{m1\_s1\_z1\_t\_L1} + N_{m1\_s1\_z2l\_t\_L1}) - 1) =$ **3.517** in
Min. allowable top bar clear spacing (cl.25.2.1)	$s_{top,min} =$ **1.000** in
Bottom bar clear spacing	$s_{bot} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z1\_v}) + \phi_{m1\_s1\_z1\_b\_L1} \times N_{m1\_s1\_z1\_b\_L1})) / (N_{m1\_s1\_z1\_b\_L1} - 1) =$ **3.517** in
Min. allowable bottom bar clear spacing (cl.25.2.1)	$s_{bot,min} =$ **1.000** in
	PASS - Actual bar spacing exceeds minimum allowable
Zone 2 - Positive moment. Rectangular section in flexure (Section 9.5.2)	
Factored bending moment at section	$M_u = \text{abs}(M_{m1\_s1\_z2\_max\_red}) =$ **972.405** kip_ft
Effective depth to tension reinforcement	$d =$ **55.936** in
Tension reinforcement provided	7 × No.9 bars
Area of tension reinforcement provided	$A_{s,prov} =$ **6.995** in^2
Minimum area of reinforcement (cl.9.6.1.2)	$A_{s,min} = \max(3\text{ psi} \times \sqrt{(f'_c / 1\text{ psi})}, 200\text{ psi}) \times b \times d / f_y =$ **6.712** in^2
	PASS - Area of reinforcement provided is greater than minimum area of reinforcement required
Stress block depth factor (cl.22.2.2.4.3)	$\beta_1 = \min(\max(0.85 - 0.05 \times (f'_c - 4\text{ ksi}) / 1\text{ ksi}, 0.65), 0.85) =$ **0.85**
Depth of equivalent rectangular stress block	$a = A_{s,prov} \times f_y / (0.85 \times f'_c \times b) =$ **3.429** in
Depth to neutral axis	$c = a / \beta_1 =$ **4.034** in
Net tensile strain in extreme tension fibers	$\varepsilon_t = 0.003 \times (d_o - c) / \max(c, 0.001\text{ in}) =$ **0.0386**
	Net tensile strain in tension controlled zone
Strength reduction factor (cl.21.2.1)	$\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.003), 0.65), 0.9) =$ **0.90**
Nominal moment strength	$M_n = A_{s,prov} \times f_y \times (d - a / 2) =$ **1896.476** kip_ft
Design moment strength	$\phi M_n = M_n \times \phi_f =$ **1706.829** kip_ft
	PASS - Required moment strength is less than design moment strength
Flexural cracking	
Max. center to center spacing of tension reinf.	$s_{b,max} = s_{bot} + \phi_{m1\_s1\_z2\_b\_L1} =$ **4.645** in
Service load stress in reinforcement (cl.24.3.2)	$f_s = 2/3 \times f_y =$ **40000** psi
Clear cover of reinforcement	$c_c = c_{nom\_b} + \phi_v =$ **3.500** in
Maximum allowable bot bar spacing (Table 24.3.2)	$s_{max} = \min(15\text{in} \times 40000\text{psi} / f_s - 2.5 \times c_c, 12\text{in} \times 40000\text{psi} / f_s) =$ **6.250** in
	PASS - Maximum allowable tension reinforcement spacing exceeds actual spacing

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

Project: QTS DEN1 DC2				Job Ref.: 24053.00000	
Section: GB3				Sheet no./rev.: 6	
Calc. by: LB	Date: 4/1/2025	Chk'd by:	Date:	App'd by:	Date:

Control of deflections (Section 24.2)

Concrete density factor	K_w = **1.00**
Reinforcement yield strength factor	$K_f = 0.4 + f_y / 100000\ psi$ = **1.00**
Minimum thickness of beam (Table 9.3.1.1)	$h_{min} = (L_{m1\_s1} / 16.0) \times K_w \times K_f$ = **31.5** in
	PASS - Thickness of beam exceeds minimum thickness

Spacing limits for reinforcement

Top bar clear spacing	$s_{top} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z2\_v}) + \phi_{m1\_s1\_z2r\_t\_L1} \times N_{m1\_s1\_z2r\_t\_L1})) / (N_{m1\_s1\_z2r\_t\_L1} - 1)$ = **3.517** in
Min. allowable top bar clear spacing (cl.25.2.1)	$s_{top,min}$ = **1.000** in
Bottom bar clear spacing	$s_{bot} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z2\_v}) + \phi_{m1\_s1\_z2\_b\_L1} \times N_{m1\_s1\_z2\_b\_L1} + \phi_{m1\_s1\_z1\_b\_L1} \times N_{m1\_s1\_z1\_b\_L1})) / ((N_{m1\_s1\_z2\_b\_L1} + N_{m1\_s1\_z1\_b\_L1}) - 1)$ = **3.517** in
Min. allowable bottom bar clear spacing (cl.25.2.1)	$s_{bot,min}$ = **1.000** in
	PASS - Actual bar spacing exceeds minimum allowable

Zone 3 - Positive moment. Rectangular section in flexure (Section 9.5.2)

Factored bending moment at section	$M_u = abs(M_{m1\_s1\_z3\_max\_red})$ = **729.304** kip_ft
Effective depth to tension reinforcement	d = **55.936** in
Tension reinforcement provided	7 × No.9 bars
Area of tension reinforcement provided	$A_{s,prov}$ = **6.995** in^2
Minimum area of reinforcement (cl.9.6.1.2)	$A_{s,min} = max(3\ psi \times \sqrt{(f'_c / 1\ psi)}, 200\ psi) \times b \times d / f_y$ = **6.712** in^2
	PASS - Area of reinforcement provided is greater than minimum area of reinforcement required
Stress block depth factor (cl.22.2.2.4.3)	$\beta_1 = min(max(0.85 - 0.05 \times (f'_c - 4\ ksi) / 1\ ksi, 0.65), 0.85)$ = **0.85**
Depth of equivalent rectangular stress block	$a = A_{s,prov} \times f_y / (0.85 \times f'_c \times b)$ = **3.429** in
Depth to neutral axis	$c = a / \beta_1$ = **4.034** in
Net tensile strain in extreme tension fibers	$\varepsilon_t = 0.003 \times (d_o - c) / max(c, 0.001\ in)$ = **0.0386**
	Net tensile strain in tension controlled zone
Strength reduction factor (cl.21.2.1)	$\phi_f = min(max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.003), 0.65), 0.9)$ = **0.90**
Nominal moment strength	$M_n = A_{s,prov} \times f_y \times (d - a / 2)$ = **1896.476** kip_ft
Design moment strength	$\phi M_n = M_n \times \phi_f$ = **1706.829** kip_ft
	PASS - Required moment strength is less than design moment strength

Flexural cracking

Max. center to center spacing of tension reinf.	$s_{b,max} = s_{bot} + \phi_{m1\_s1\_z3\_b\_L1}$ = **4.645** in
Service load stress in reinforcement (cl.24.3.2)	$f_s = 2/3 \times f_y$ = **40000** psi
Clear cover of reinforcement	$c_c = c_{nom\_b} + \phi_v$ = **3.500** in
Maximum allowable bot bar spacing (Table 24.3.2)	$s_{max} = min(15in \times 40000psi / f_s - 2.5 \times c_c, 12in \times 40000psi / f_s)$ = **6.250** in
	PASS - Maximum allowable tension reinforcement spacing exceeds actual spacing

Spacing limits for reinforcement

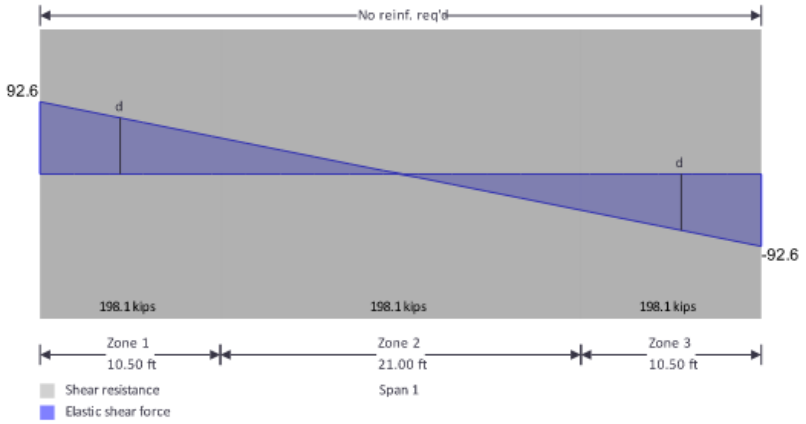
Top bar clear spacing	$s_{top} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z3\_v}) + \phi_{m1\_s1\_z3\_t\_L1} \times N_{m1\_s1\_z3\_t\_L1} + \phi_{m1\_s1\_z2r\_t\_L1} \times N_{m1\_s1\_z2r\_t\_L1})) / ((N_{m1\_s1\_z3\_t\_L1} + N_{m1\_s1\_z2r\_t\_L1}) - 1)$ = **3.517** in
Min. allowable top bar clear spacing (cl.25.2.1)	$s_{top,min}$ = **1.000** in
Bottom bar clear spacing	$s_{bot} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z3\_v}) + \phi_{m1\_s1\_z3\_b\_L1} \times N_{m1\_s1\_z3\_b\_L1})) / (N_{m1\_s1\_z3\_b\_L1} - 1)$ = **3.517** in
Min. allowable bottom bar clear spacing (cl.25.2.1)	$s_{bot,min}$ = **1.000** in

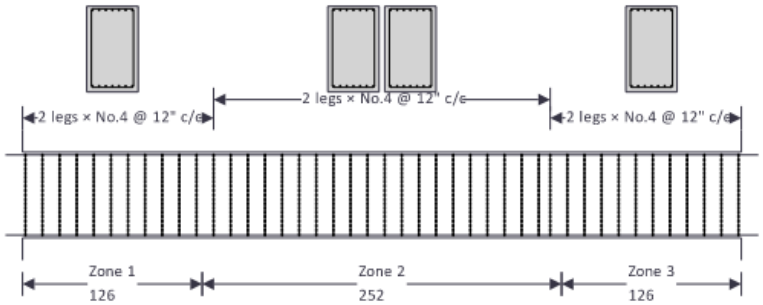
PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

Project: QTS DEN1 DC2				Job Ref.: 24053.00000	
Section: GB3				Sheet no./rev.: 7	
Calc. by: LB	Date: 4/1/2025	Chk'd by:	Date:	App'd by:	Date:

PASS - Actual bar spacing exceeds minimum allowable

Shear design





Rectangular section in shear

Concrete weight modification factor	λ = **1.00**
Location where min. reinf. is req'd (V_u less ϕV_c)	N/A no reinf. required along full length
Location where no reinf. is req'd (V_u less $\phi V_{s,lim}$)	Between 0.00 ft and 42.00 ft
Maximum reinforcement shear strength	$\phi V_{s,max} = \phi_s \times 8\ psi \times \sqrt{(\min(f'_c, 10000psi) / 1\ psi)} \times b \times d$ = **764.144** kips
Minimum area of shear reinf. (Table 9.6.3.4)	$A_{sv,min} = \max(50\ psi, 0.75\ psi \times \sqrt{(f'_c / 1\ psi)}) \times b / \min(f_y, 60000\ psi)$ = **0.360** in^2/ft
No reinforcement shear strength limit	$\phi V_{s,lim} = \phi_s \times \lambda \times 1psi \times \sqrt{(\min(f'_c, 10000psi) / 1\ psi)} \times b \times d$ = **95.518** kips

Zone 1

Effective depth of long. reinf. used for shear zone	d = **55.936** in
Size effect factor (cl. 22.5.5.1.3)	$\lambda_s = \min(\sqrt{(2 / (1 + (d / 1\ in) / 10))}, 1.0)$ = **0.55**
Area of longitudinal reinforcement	A_{st} = **6.995** in^2
Ratio of longitudinal reinforcement	$\rho_w = A_{st} / (b \times d)$ = **0.003**
Concrete shear strength (Table 22.5.5.1)	$\phi V_c = \phi_s \times \min(8 \times \lambda \times (\rho_w)^{1/3}, 5 \times \lambda) \times \sqrt{(f'_c \times 1\ psi)} \times b \times d$ = **115.730** kips

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

Project: QTS DEN1 DC2				Job Ref.: 24053.00000	
Section: GB3				Sheet no./rev.: 8	
Calc. by: LB	Date: 4/1/2025	Chk'd by:	Date:	App'd by:	Date:

Design shear force at 55.9in from support	V_u = **72.054** kips
Reinf. shear strength required (eqn. 22.5.1.1)	ϕV_s = max(V_u - ϕV_c, 0 kips) = **0.000** kips
Area of design shear reinf. req'd (eqn. 22.5.8.5.3)	$A_{sv,des}$ = ϕV_s / (ϕ_s × min(f_y, 60000 psi) × d) = **0.000** in²/ft
Area of shear reinforcement required	$A_{sv,req}$ = 0mm²/ft = **0.000** in²/ft
Shear reinforcement provided	2 legs × No.4 @ 12" c/c
Area of shear reinforcement provided	$A_{sv,prov}$ = **0.393** in²/ft
	PASS - No shear reinforcement required ($\phi V_{s,lim}$ >= V_u)

Zone 2

Effective depth of long. reinf. used for shear zone	d = **55.936** in
Size effect factor (cl. 22.5.5.1.3)	λ_s = min(√(2 / (1 + (d / 1 in) / 10)), 1.0) = **0.55**
Area of longitudinal reinforcement	A_{st} = **6.995** in²
Ratio of longitudinal reinforcement	ρ_w = A_{st} / (b × d) = **0.003**
Concrete shear strength (Table 22.5.5.1)	ϕV_c = ϕ_s × min(8 × λ × (ρ_w)^1/3, 5 × λ) × √(f'_c × 1 psi) × b × d = **115.730** kips
Design shear force within zone	V_u = **46.305** kips
Reinf. shear strength required (eqn. 22.5.1.1)	ϕV_s = max(V_u - ϕV_c, 0 kips) = **0.000** kips
Area of design shear reinf. req'd (eqn. 22.5.8.5.3)	$A_{sv,des}$ = ϕV_s / (ϕ_s × min(f_y, 60000 psi) × d) = **0.000** in²/ft
Area of shear reinforcement required	$A_{sv,req}$ = 0mm²/ft = **0.000** in²/ft
Shear reinforcement provided	2 legs × No.4 @ 12" c/c
Area of shear reinforcement provided	$A_{sv,prov}$ = **0.393** in²/ft
	PASS - No shear reinforcement required ($\phi V_{s,lim}$ >= V_u)

Zone 3

Effective depth of long. reinf. used for shear zone	d = **55.936** in
Size effect factor (cl. 22.5.5.1.3)	λ_s = min(√(2 / (1 + (d / 1 in) / 10)), 1.0) = **0.55**
Area of longitudinal reinforcement	A_{st} = **6.995** in²
Ratio of longitudinal reinforcement	ρ_w = A_{st} / (b × d) = **0.003**
Concrete shear strength (Table 22.5.5.1)	ϕV_c = ϕ_s × min(8 × λ × (ρ_w)^1/3, 5 × λ) × √(f'_c × 1 psi) × b × d = **115.730** kips
Design shear force at 55.9in from support	V_u = **72.054** kips
Reinf. shear strength required (eqn. 22.5.1.1)	ϕV_s = max(V_u - ϕV_c, 0 kips) = **0.000** kips
Area of design shear reinf. req'd (eqn. 22.5.8.5.3)	$A_{sv,des}$ = ϕV_s / (ϕ_s × min(f_y, 60000 psi) × d) = **0.000** in²/ft
Area of shear reinforcement required	$A_{sv,req}$ = 0mm²/ft = **0.000** in²/ft
Shear reinforcement provided	2 legs × No.4 @ 12" c/c
Area of shear reinforcement provided	$A_{sv,prov}$ = **0.393** in²/ft
	PASS - No shear reinforcement required ($\phi V_{s,lim}$ >= V_u)

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

Project: QTS DEN1 DC2				Job Ref.: 24053.00000	
Section: GB4				Sheet no./rev.: 1	
Calc. by: LB	Date: 4/21/2025	Chk'd by:	Date:	App'd by:	Date:

RC BEAM ANALYSIS & DESIGN (ACI318-2014)

In accordance with ACI318-2014

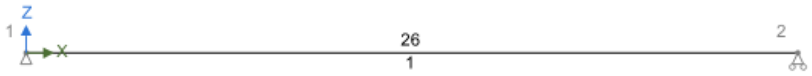
Tedds calculation version 3.3.21

ANALYSIS

Tedds calculation version 1.0.38

Geometry

Geometry (ft) - Concrete (ACI normal) - R 72x66



Span	Length (ft)	Section	Start Support	End Support
1	26	R 72x66	Pinned	Roller Pin X
R 72x66: Area 4752 in^2, Inertia Major 1724976 in^4, Inertia Minor 2052864 in^4, Shear area parallel to Minor 3960 in^2, Shear area parallel to Major 3960 in^2				
Concrete (ACI normal): Density 150 lbm/ft^3, Youngs 4200 ksi, Shear 1750 ksi, Thermal 0.00001 °C^{-1}				

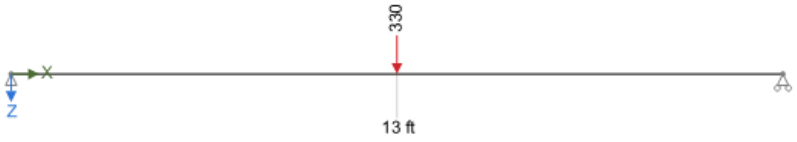
Loading

Self weight included

Dead - Loading (kips/ft)



Live - Loading (kips)



Load combination factors

Load combination	Self Weight	Dead	Live
1.2D + 1.6L (Strength)	1.20	1.20	1.60
1.0D + 1.0L (Service)	1.00	1.00	1.00
1.4D (Strength)	1.40	1.40	
1.0D (Service)	1.00	1.00	

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

Project: QTS DEN1 DC2				Job Ref.: 24053.00000	
Section: GB4				Sheet no./rev.: 2	
Calc. by: LB	Date: 4/21/2025	Chk'd by:	Date:	App'd by:	Date:

Member Loads

Member	Load case	Load Type	Orientation	Description
Beam	Dead	UDL	GlobalZ	6.86 kips/ft
Beam	Live	Point load	GlobalZ	330 kips at 13 ft

Results

Reactions

Load case: Self Weight

Node	Force Fx (kips)	Force Fz (kips)	Moment My (kip_ft)
1	0	64.35	0
2	0	64.35	0

Load case: Dead

Node	Force Fx (kips)	Force Fz (kips)	Moment My (kip_ft)
1	0	89.213	0
2	0	89.213	0

Load case: Live

Node	Force Fx (kips)	Force Fz (kips)	Moment My (kip_ft)
1	0	165	0
2	0	165	0

Load combination: 1.2D + 1.6L (Strength)

Node	Force Fx (kips)	Force Fz (kips)	Moment My (kip_ft)
1	0	448.275	0
2	0	448.275	0

Load combination: 1.0D + 1.0L (Service)

Node	Force Fx (kips)	Force Fz (kips)	Moment My (kip_ft)
1	0	318.563	0
2	0	318.563	0

Load combination: 1.4D (Strength)

Node	Force Fx (kips)	Force Fz (kips)	Moment My (kip_ft)
1	0	214.988	0
2	0	214.988	0

PROJECT	QTS DEN1-DC2			
DESCRIPTION				
PROJECT #	24053.00000	DATE	05/30/2025	
PREPARED BY	LB	CHECKED BY	ND	

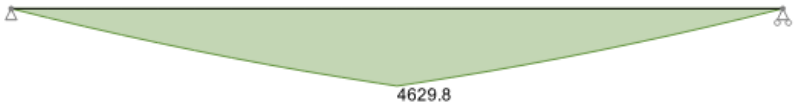
Project: QTS DEN1 DC2				Job Ref.: 24053.00000	
Section: GB4				Sheet no./rev.: 3	
Calc. by: LB	Date: 4/21/2025	Chk'd by:	Date:	App'd by:	Date:

Load combination: 1.0D (Service)

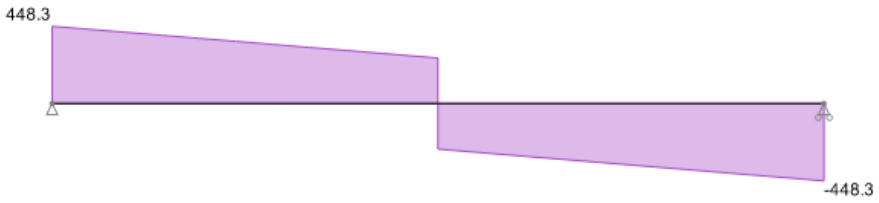
Node	Force Fx (kips)	Force Fz (kips)	Moment My (kip_ft)
1	0	153.563	0
2	0	153.563	0

Forces

All combinations - Moment envelope (kip_ft)



All combinations - Shear envelope (kips)



Member results

Envelope - All combinations

Member	Position (ft)	Shear force (kips)		Moment (kip_ft)	
Beam	0	448.275 (max abs)	153.563	0 (min)	
	13	264	-264	4629.787 (max)	998.156
	26	-153.562	-448.275	0 (min)	

Concrete details

Compressive strength of concrete	f'_c = **4000** psi
Density of reinforced concrete	w_c = **150** lb / ft^3
Concrete type	Normal weight
Modulus of elasticity of concrete (cl.19.2.2.1)	E = (w_c / 1 lb/ft^3)$^{1.5}$ × 33 psi × (f'_c / 1 psi)$^{0.5}$ = **3834254** psi
Strength reduction factor for shear	ϕ_s = **0.75**

Reinforcement details

Yield strength of reinforcement	f_y = **60000** psi
Compression-controlled strain limit (cl.21.2.2.1)	ε_{ty} = **0.00200**

Nominal cover to reinforcement

Cover to top reinforcement	c_{nom_t} = **1.5** in
Cover to bottom reinforcement	c_{nom_b} = **3** in

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

Project: QTS DEN1 DC2				Job Ref.: 24053.00000	
Section: GB4				Sheet no./rev.: 4	
Calc. by: LB	Date: 4/21/2025	Chk'd by	Date	App'd by	Date

Cover to side reinforcement c_{nom_s} = **3** in

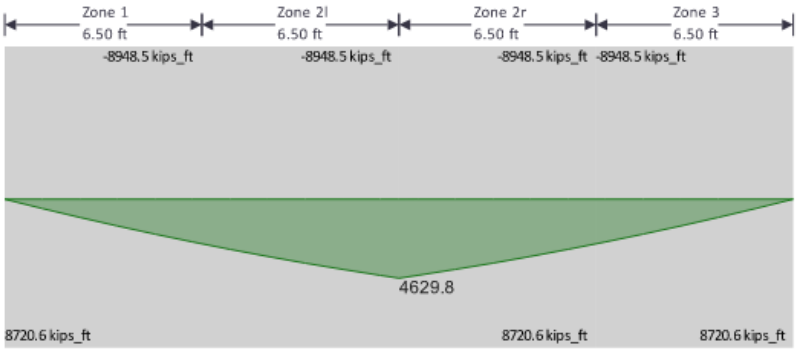
Beam - Span 1

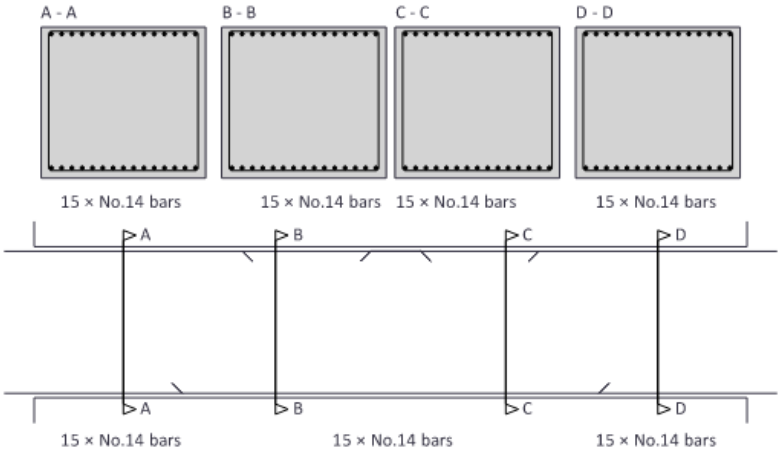
Rectangular section details

Section width b = **72** in

Section depth h = **66** in

Moment design





Zone 1 - Positive moment. Rectangular section in flexure (Section 9.5.2)

Factored bending moment at section M_u = abs($M_{m1\_s1\_z1\_max\_red}$) = **2614.341** kip_ft

Effective depth to tension reinforcement d = **61.529** in

PROJECT QTS DEN1-DC2
DESCRIPTION
PROJECT # 24053.00000 DATE 05/30/2025
PREPARED BY LB CHECKED BY ND

Project				Job Ref.	
QTS DEN1 DC2				24053.00000	
Section				Sheet no./rev.	
GB4				5	
Calc. by	Date	Chk'd by	Date	App'd by	Date
LB	4/21/2025				

Tension reinforcement provided	15 × No.14 bars
Area of tension reinforcement provided	$A_{s,prov}$ = **33.767** in^2
Minimum area of reinforcement (cl.9.6.1.2)	$A_{s,min}$ = max(3 psi × √(f'$_c$ / 1 psi), 200 psi) × b × d / f_y = **14.767** in^2
	PASS - Area of reinforcement provided is greater than minimum area of reinforcement required
Stress block depth factor (cl.22.2.2.4.3)	β_1 = min(max(0.85 - 0.05 × (f'$_c$ - 4 ksi) / 1 ksi, 0.65), 0.85) = **0.85**
Depth of equivalent rectangular stress block	a = $A_{s,prov}$ × f_y / (0.85 × f'$_c$ × b) = **8.276** in
Depth to neutral axis	c = a / β_1 = **9.737** in
Net tensile strain in extreme tension fibers	ε_t = 0.003 × (d_o - c) / max(c, 0.001 in) = **0.01596**
	Net tensile strain in tension controlled zone
Strength reduction factor (cl.21.2.1)	ϕ_f = min(max(0.65 + 0.25 × (ε_t - ε_{ty}) / (0.005 - ε_{ty}), 0.65), 0.9) = **0.90**
Nominal moment strength	M_n = $A_{s,prov}$ × f_y × (d - a / 2) = **9689.559** kip_ft
Design moment strength	ϕM_n = M_n × ϕ_f = **8720.603** kip_ft
	PASS - Required moment strength is less than design moment strength
Flexural cracking	
Max. center to center spacing of tension reinf.	$s_{b,max}$ = s_{bot} + $\phi_{m1\_s1\_z1\_b\_L1}$ = **4.504** in
Service load stress in reinforcement (cl.24.3.2)	f_s = 2/3 × f_y = **40000** psi
Clear cover of reinforcement	c_c = c_{nom_b} + ϕ_v = **3.625** in
Maximum allowable bot bar spacing (Table 24.3.2)	s_{max} = min(15in × 40000psi / f_s - 2.5 × c_c, 12in × 40000psi / f_s) = **5.938** in
	PASS - Maximum allowable tension reinforcement spacing exceeds actual spacing
Spacing limits for reinforcement	
Top bar clear spacing	s_{top} = (b - (2 × (c_{nom_s} + $\phi_{m1\_s1\_z1\_v}$) + $\phi_{m1\_s1\_z1\_t\_L1}$ × $N_{m1\_s1\_z1\_t\_L1}$ + $\phi_{m1\_s1\_z2l\_t\_L1}$ × $N_{m1\_s1\_z2l\_t\_L1}$)) / (($N_{m1\_s1\_z1\_t\_L1}$ + $N_{m1\_s1\_z2l\_t\_L1}$) - 1) = **2.811** in
Min. allowable top bar clear spacing (cl.25.2.1)	$s_{top,min}$ = **1.000** in
Bottom bar clear spacing	s_{bot} = (b - (2 × (c_{nom_s} + $\phi_{m1\_s1\_z1\_v}$) + $\phi_{m1\_s1\_z1\_b\_L1}$ × $N_{m1\_s1\_z1\_b\_L1}$)) / ($N_{m1\_s1\_z1\_b\_L1}$ - 1) = **2.811** in
Min. allowable bottom bar clear spacing (cl.25.2.1)	$s_{bot,min}$ = **1.000** in
	PASS - Actual bar spacing exceeds minimum allowable
Zone 2 - Positive moment. Rectangular section in flexure (Section 9.5.2)	
Factored bending moment at section	M_u = abs($M_{m1\_s1\_z2\_max\_red}$) = **4629.787** kip_ft
Effective depth to tension reinforcement	d = **61.529** in
Tension reinforcement provided	15 × No.14 bars
Area of tension reinforcement provided	$A_{s,prov}$ = **33.767** in^2
Minimum area of reinforcement (cl.9.6.1.2)	$A_{s,min}$ = max(3 psi × √(f'$_c$ / 1 psi), 200 psi) × b × d / f_y = **14.767** in^2
	PASS - Area of reinforcement provided is greater than minimum area of reinforcement required
Stress block depth factor (cl.22.2.2.4.3)	β_1 = min(max(0.85 - 0.05 × (f'$_c$ - 4 ksi) / 1 ksi, 0.65), 0.85) = **0.85**
Depth of equivalent rectangular stress block	a = $A_{s,prov}$ × f_y / (0.85 × f'$_c$ × b) = **8.276** in
Depth to neutral axis	c = a / β_1 = **9.737** in
Net tensile strain in extreme tension fibers	ε_t = 0.003 × (d_o - c) / max(c, 0.001 in) = **0.01596**
	Net tensile strain in tension controlled zone
Strength reduction factor (cl.21.2.1)	ϕ_f = min(max(0.65 + 0.25 × (ε_t - ε_{ty}) / (0.005 - ε_{ty}), 0.65), 0.9) = **0.90**
Nominal moment strength	M_n = $A_{s,prov}$ × f_y × (d - a / 2) = **9689.559** kip_ft
Design moment strength	ϕM_n = M_n × ϕ_f = **8720.603** kip_ft

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

Project				Job Ref.	
QTS DEN1 DC2				24053.00000	
Section				Sheet no./rev.	
GB4				6	
Calc. by	Date	Chk'd by	Date	App'd by	Date
LB	4/21/2025				

PASS - Required moment strength is less than design moment strength

Flexural cracking

Max. center to center spacing of tension reinf. $s_{b,max} = s_{bot} + \phi_{m1\_s1\_z2\_b\_L1}$ = **4.504** in

Service load stress in reinforcement (cl.24.3.2) $f_s = 2/3 \times f_y$ = **40000** psi

Clear cover of reinforcement $c_c = c_{nom\_b} + \phi_v$ = **3.625** in

Maximum allowable bot bar spacing (Table 24.3.2) $s_{max} = \min(15in \times 40000psi / f_s - 2.5 \times c_c, 12in \times 40000psi / f_s)$ = **5.938** in

PASS - Maximum allowable tension reinforcement spacing exceeds actual spacing

Control of deflections (Section 24.2)

Concrete density factor K_w = **1.00**

Reinforcement yield strength factor $K_f = 0.4 + f_y / 100000$ psi = **1.00**

Minimum thickness of beam (Table 9.3.1.1) $h_{min} = (L_{m1\_s1} / 16.0) \times K_w \times K_f$ = **19.5** in

PASS - Thickness of beam exceeds minimum thickness

Spacing limits for reinforcement

Top bar clear spacing $s_{top} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z2\_v}) + \phi_{m1\_s1\_z2r\_t\_L1} \times N_{m1\_s1\_z2r\_t\_L1})) / (N_{m1\_s1\_z2r\_t\_L1} - 1)$ = **2.811** in

Min. allowable top bar clear spacing (cl.25.2.1) $s_{top,min}$ = **1.000** in

Bottom bar clear spacing $s_{bot} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z2\_v}) + \phi_{m1\_s1\_z2\_b\_L1} \times N_{m1\_s1\_z2\_b\_L1} + \phi_{m1\_s1\_z1\_b\_L1} \times N_{m1\_s1\_z1\_b\_L1})) / ((N_{m1\_s1\_z2\_b\_L1} + N_{m1\_s1\_z1\_b\_L1}) - 1)$ = **2.811** in

Min. allowable bottom bar clear spacing (cl.25.2.1) $s_{bot,min}$ = **1.000** in

PASS - Actual bar spacing exceeds minimum allowable

Zone 3 - Positive moment. Rectangular section in flexure (Section 9.5.2)

Factored bending moment at section $M_u = abs(M_{m1\_s1\_z3\_max\_red})$ = **2614.341** kip_ft

Effective depth to tension reinforcement d = **61.529** in

Tension reinforcement provided 15 × No.14 bars

Area of tension reinforcement provided $A_{s,prov}$ = **33.767** in^2

Minimum area of reinforcement (cl.9.6.1.2) $A_{s,min} = \max(3 psi \times \sqrt{f'_c / 1 psi}, 200 psi) \times b \times d / f_y$ = **14.767** in^2

PASS - Area of reinforcement provided is greater than minimum area of reinforcement required

Stress block depth factor (cl.22.2.2.4.3) $\beta_1 = \min(\max(0.85 - 0.05 \times (f'_c - 4 ksi) / 1 ksi, 0.65), 0.85)$ = **0.85**

Depth of equivalent rectangular stress block $a = A_{s,prov} \times f_y / (0.85 \times f'_c \times b)$ = **8.276** in

Depth to neutral axis $c = a / \beta_1$ = **9.737** in

Net tensile strain in extreme tension fibers $\varepsilon_t = 0.003 \times (d_o - c) / \max(c, 0.001 in)$ = **0.01596**

Net tensile strain in tension controlled zone

Strength reduction factor (cl.21.2.1) $\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.005 - \varepsilon_{ty}), 0.65), 0.9)$ = **0.90**

Nominal moment strength $M_n = A_{s,prov} \times f_y \times (d - a / 2)$ = **9689.559** kip_ft

Design moment strength $\phi M_n = M_n \times \phi_f$ = **8720.603** kip_ft

PASS - Required moment strength is less than design moment strength

Flexural cracking

Max. center to center spacing of tension reinf. $s_{b,max} = s_{bot} + \phi_{m1\_s1\_z3\_b\_L1}$ = **4.504** in

Service load stress in reinforcement (cl.24.3.2) $f_s = 2/3 \times f_y$ = **40000** psi

Clear cover of reinforcement $c_c = c_{nom\_b} + \phi_v$ = **3.625** in

Maximum allowable bot bar spacing (Table 24.3.2) $s_{max} = \min(15in \times 40000psi / f_s - 2.5 \times c_c, 12in \times 40000psi / f_s)$ = **5.938** in

PASS - Maximum allowable tension reinforcement spacing exceeds actual spacing

PROJECT QTS DEN1-DC2
DESCRIPTION
PROJECT # 24053.00000 DATE 05/30/2025
PREPARED BY LB CHECKED BY ND

Project				Job Ref.	
QTS DEN1 DC2				24053.00000	
Section				Sheet no./rev.	
GB4				7	
Calc. by	Date	Chk'd by	Date	App'd by	Date
LB	4/21/2025				

Spacing limits for reinforcement

Top bar clear spacing
$s_{top} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z3\_v}) + \phi_{m1\_s1\_z3\_t\_L1} \times N_{m1\_s1\_z3\_t\_L1} + \phi_{m1\_s1\_z2r\_t\_L1} \times N_{m1\_s1\_z2r\_t\_L1})) / ((N_{m1\_s1\_z3\_t\_L1} + N_{m1\_s1\_z2r\_t\_L1}) - 1) =$ **2.811** in

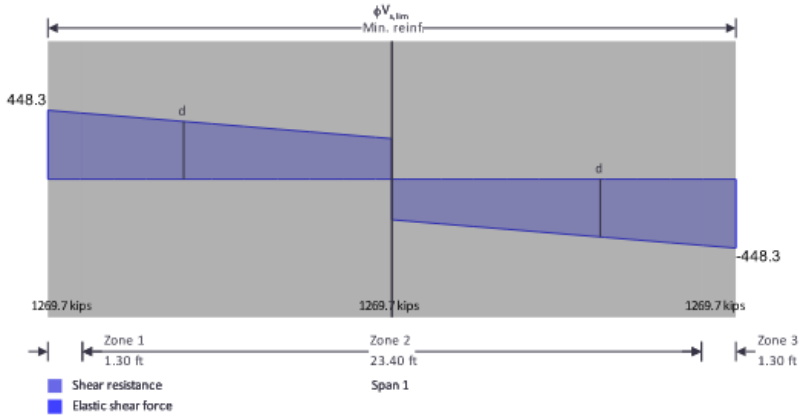
Min. allowable top bar clear spacing (cl.25.2.1) $s_{top,min} =$ **1.000** in

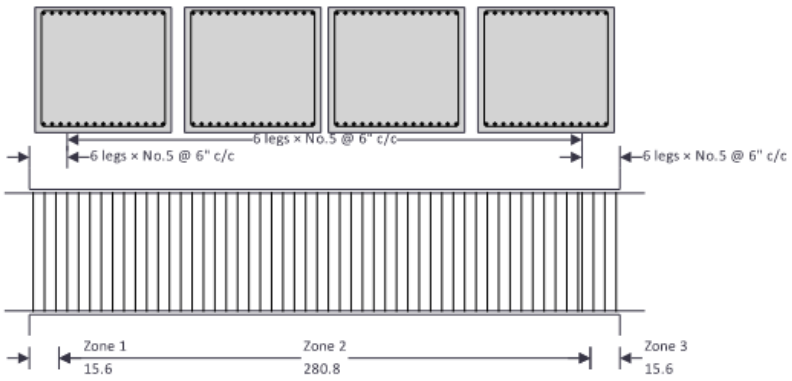
Bottom bar clear spacing
$s_{bot} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z3\_v}) + \phi_{m1\_s1\_z3\_b\_L1} \times N_{m1\_s1\_z3\_b\_L1})) / (N_{m1\_s1\_z3\_b\_L1} - 1) =$ **2.811** in

Min. allowable bottom bar clear spacing (cl.25.2.1) $s_{bot,min} =$ **1.000** in

PASS - Actual bar spacing exceeds minimum allowable

Shear design





Rectangular section in shear

Concrete weight modification factor $\lambda =$ **1.00**

Location where min. reinf. is req'd (V_u less ϕV_c) Between 0.00 ft and 26.00 ft

Location where no reinf. is req'd (V_u less $\phi V_{s,lim}$) Between 13.00 ft and 13.00 ft

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

Project: QTS DEN1 DC2				Job Ref.: 24053.00000	
Section: GB4				Sheet no./rev.: 8	
Calc. by: LB	Date: 4/21/2025	Chk'd by:	Date:	App'd by:	Date:

Maximum reinforcement shear strength	$\phi V_{s,max} = \phi_s \times 8$ psi $\times \sqrt{(\min(f'_c, 10000\text{psi}) / 1\text{ psi})} \times b \times d =$ **1681.087** kips
Minimum area of shear reinf. (Table 9.6.3.3)	$A_{sv,min} = \max(50\text{ psi}, 0.75\text{ psi} \times \sqrt{(f'_c / 1\text{ psi})}) \times b / \min(f_y, 60000\text{ psi}) =$ **0.720** in²/ft
No reinforcement shear strength limit	$\phi V_{s,lim} = \phi V_c / 2 =$ **210.136** kips
Zone 1	
Effective depth of long. reinf. used for shear zone	d = **61.529** in
Concrete shear strength (eqn. 22.5.5.1)	$\phi V_c = \phi_s \times \lambda \times 2$ psi $\times \sqrt{(\min(f'_c, 10000\text{psi}) / 1\text{ psi})} \times b \times d =$ **420.272** kips
Design shear force at 61.5in from support	$V_u =$ **375.594** kips
Reinf. shear strength required (eqn. 22.5.1.1)	$\phi V_s = \max(V_u - \phi V_c, 0\text{ kips}) =$ **0.000** kips
Area of design shear reinf. req'd (eqn. 22.5.10.5.3)	$A_{sv,des} = \phi V_s / (\phi_s \times \min(f_y, 60000\text{ psi}) \times d) =$ **0.000** in²/ft
Area of shear reinforcement required	$A_{sv,req} = A_{sv,min} =$ **0.720** in²/ft
Shear reinforcement provided	6 legs × No.5 @ 6" c/c
Area of shear reinforcement provided	$A_{sv,prov} =$ **3.682** in²/ft
	PASS - Area of shear reinforcement provided exceeds area of shear reinforcement required
Maximum longitudinal spacing (Table 9.7.6.2.2)	$s_{vl,max} = \min(d / 2, 24\text{ in}) =$ **24** in
	PASS - longitudinal spacing of stirrups is less than the maximum allowable
Zone 2	
Effective depth of long. reinf. used for shear zone	d = **61.529** in
Concrete shear strength (eqn. 22.5.5.1)	$\phi V_c = \phi_s \times \lambda \times 2$ psi $\times \sqrt{(\min(f'_c, 10000\text{psi}) / 1\text{ psi})} \times b \times d =$ **420.272** kips
Design shear force within zone	$V_u =$ **375.594** kips
Reinf. shear strength required (eqn. 22.5.1.1)	$\phi V_s = \max(V_u - \phi V_c, 0\text{ kips}) =$ **0.000** kips
Area of design shear reinf. req'd (eqn. 22.5.10.5.3)	$A_{sv,des} = \phi V_s / (\phi_s \times \min(f_y, 60000\text{ psi}) \times d) =$ **0.000** in²/ft
Area of shear reinforcement required	$A_{sv,req} = A_{sv,min} =$ **0.720** in²/ft
Shear reinforcement provided	6 legs × No.5 @ 6" c/c
Area of shear reinforcement provided	$A_{sv,prov} =$ **3.682** in²/ft
	PASS - Area of shear reinforcement provided exceeds area of shear reinforcement required
Maximum longitudinal spacing (Table 9.7.6.2.2)	$s_{vl,max} = \min(d / 2, 24\text{ in}) =$ **24** in
	PASS - longitudinal spacing of stirrups is less than the maximum allowable
Zone 3	
Effective depth of long. reinf. used for shear zone	d = **61.529** in
Concrete shear strength (eqn. 22.5.5.1)	$\phi V_c = \phi_s \times \lambda \times 2$ psi $\times \sqrt{(\min(f'_c, 10000\text{psi}) / 1\text{ psi})} \times b \times d =$ **420.272** kips
Design shear force at 61.5in from support	$V_u =$ **375.594** kips
Reinf. shear strength required (eqn. 22.5.1.1)	$\phi V_s = \max(V_u - \phi V_c, 0\text{ kips}) =$ **0.000** kips
Area of design shear reinf. req'd (eqn. 22.5.10.5.3)	$A_{sv,des} = \phi V_s / (\phi_s \times \min(f_y, 60000\text{ psi}) \times d) =$ **0.000** in²/ft
Area of shear reinforcement required	$A_{sv,req} = A_{sv,min} =$ **0.720** in²/ft
Shear reinforcement provided	6 legs × No.5 @ 6" c/c
Area of shear reinforcement provided	$A_{sv,prov} =$ **3.682** in²/ft
	PASS - Area of shear reinforcement provided exceeds area of shear reinforcement required
Maximum longitudinal spacing (Table 9.7.6.2.2)	$s_{vl,max} = \min(d / 2, 24\text{ in}) =$ **24** in
	PASS - longitudinal spacing of stirrups is less than the maximum allowable

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

Project: QTS DEN1 DC2				Job Ref.: 24053.00000	
Section: GB5				Sheet no./rev.: 1	
Calc. by: LB	Date: 4/1/2025	Chk'd by:	Date:	App'd by:	Date:

RC BEAM ANALYSIS & DESIGN (ACI318-2019 (22))

In accordance with ACI318-2019 (22)

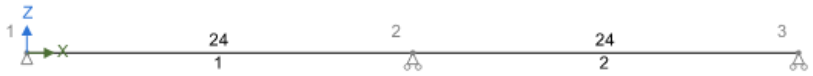
Tedds calculation version 3.3.21

ANALYSIS

Tedds calculation version 1.0.38

Geometry

Geometry (ft) - Concrete (ACI normal) - R 78x48

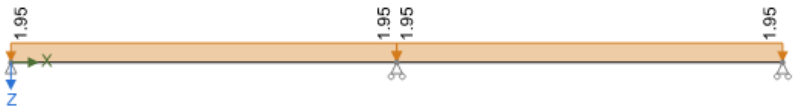


Span	Length (ft)	Section	Start Support	End Support
1	24	R 78x48	Pinned	Roller Pin X
2	24	R 78x48	Roller Pin X	Roller Pin X
R 78x48: Area 3744 in^2, Inertia Major 718848 in^4, Inertia Minor 1898208 in^4, Shear area parallel to Minor 3120 in^2, Shear area parallel to Major 3120 in^2				
Concrete (ACI normal): Density 150 lbm/ft^3, Youngs 4200 ksi, Shear 1750 ksi, Thermal 0.00001 $°C^{-1}$				

Loading

Self weight included

Dead - Loading (kips/ft)



Load combination factors

Load combination	Self Weight	Dead	Live
1.2D + 1.6L (Strength)	1.20	1.20	1.60
1.0D + 1.0L (Service)	1.00	1.00	1.00
1.4D (Strength)	1.40	1.40	
1.0D (Service)	1.00	1.00	

Member Loads

Member	Load case	Load Type	Orientation	Description
Beam	Dead	UDL	GlobalZ	1.95 kips/ft

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

Project: QTS DEN1 DC2				Job Ref.: 24053.00000	
Section: GB5				Sheet no./rev.: 2	
Calc. by: LB	Date: 4/1/2025	Chk'd by:	Date:	App'd by:	Date:

Results

Reactions

Load case: Self Weight

Node	Force Fx (kips)	Force Fz (kips)	Moment My (kip_ft)
1	0	35.329	0
2	0	116.541	0
3	0	35.329	0

Load case: Dead

Node	Force Fx (kips)	Force Fz (kips)	Moment My (kip_ft)
1	0	17.665	0
2	0	58.271	0
3	0	17.665	0

Load case: Live

Node	Force Fx (kips)	Force Fz (kips)	Moment My (kip_ft)
1	0	0	0
2	0	0	0
3	0	0	0

Load combination: 1.2D + 1.6L (Strength)

Node	Force Fx (kips)	Force Fz (kips)	Moment My (kip_ft)
1	0	63.593	0
2	0	209.774	0
3	0	63.593	0

Load combination: 1.0D + 1.0L (Service)

Node	Force Fx (kips)	Force Fz (kips)	Moment My (kip_ft)
1	0	52.994	0
2	0	174.812	0
3	0	52.994	0

Load combination: 1.4D (Strength)

Node	Force Fx (kips)	Force Fz (kips)	Moment My (kip_ft)
1	0	74.192	0
2	0	244.736	0

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

Project: QTS DEN1 DC2				Job Ref.: 24053.00000	
Section: GB5				Sheet no./rev.: 3	
Calc. by: LB	Date: 4/1/2025	Chk'd by:	Date:	App'd by:	Date:

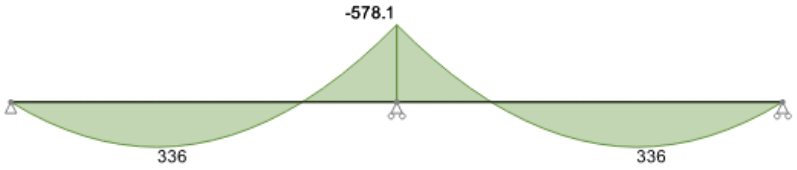
Node	Force Fx (kips)	Force Fz (kips)	Moment My (kip_ft)
3	0	74.192	0

Load combination: 1.0D (Service)

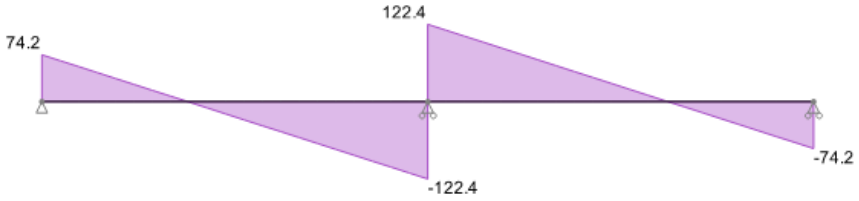
Node	Force Fx (kips)	Force Fz (kips)	Moment My (kip_ft)
1	0	52.994	0
2	0	174.812	0
3	0	52.994	0

Forces

All combinations - Moment envelope (kip_ft)



All combinations - Shear envelope (kips)



Member results

Envelope - All combinations

Member	Position (ft)	Shear force (kips)		Moment (kip_ft)	
Beam	9.06	0		336.045 (max)	240.032
	24	122.368 (max abs)	-122.368	-412.941	-578.118 (min)

Concrete details

Compressive strength of concrete	f'_c = **4000** psi
Density of reinforced concrete	w_c = **150** lb / ft^3
Concrete type	Normal weight
Modulus of elasticity of concrete (cl.19.2.2.1)	$E = (w_c / 1\ \text{lb/ft}^3)^{1.5} \times 33\ \text{psi} \times (f'_c / 1\ \text{psi})^{0.5}$ = **3834254** psi
Strength reduction factor for shear	ϕ_s = **0.75**

PROJECT QTS DEN1-DC2
DESCRIPTION
PROJECT # 24053.00000 DATE 05/30/2025
PREPARED BY LB CHECKED BY ND

Project				Job Ref.	
QTS DEN1 DC2				24053.00000	
Section				Sheet no./rev.	
GB5				4	
Calc. by	Date	Chk'd by	Date	App'd by	Date
LB	4/1/2025				

Reinforcement details

Yield strength of reinforcement	f_y = **60000** psi
Compression-controlled strain limit (cl.21.2.2.1)	ε_{ty} = **0.00200**

Nominal cover to reinforcement

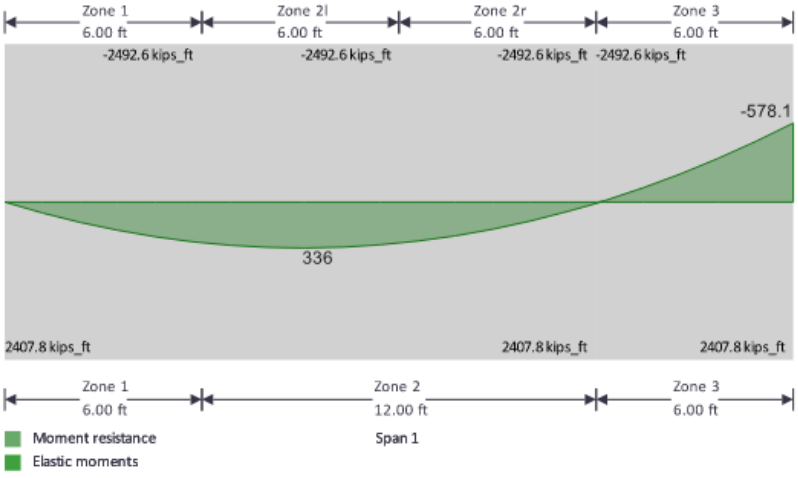
Cover to top reinforcement	c_{nom_t} = **1.5** in
Cover to bottom reinforcement	c_{nom_b} = **3** in
Cover to side reinforcement	c_{nom_s} = **3** in

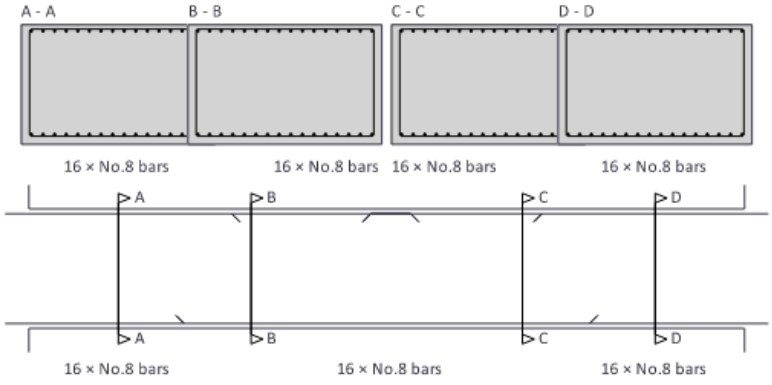
Beam - Span 1

Rectangular section details

Section width	b = **78** in
Section depth	h = **48** in

Moment design





PROJECT QTS DEN1-DC2
DESCRIPTION
PROJECT # 24053.00000 DATE 05/30/2025
PREPARED BY LB CHECKED BY ND

Project				Job Ref.	
QTS DEN1 DC2				24053.00000	
Section				Sheet no./rev.	
GB5				5	
Calc. by	Date	Chk'd by	Date	App'd by	Date
LB	4/1/2025				

Zone 1 - Positive moment. Rectangular section in flexure (Section 9.5.2)

Factored bending moment at section	$M_u = abs(M_{m1\_s1\_z1\_max\_red})$ = **297.731** kip_ft
Effective depth to tension reinforcement	d = **44** in
Tension reinforcement provided	16 × No.8 bars
Area of tension reinforcement provided	$A_{s,prov}$ = **12.566** in^2
Minimum area of reinforcement (cl.9.6.1.2)	$A_{s,min} = max(3\ psi \times \sqrt{(f'_c / 1\ psi)}, 200\ psi) \times b \times d / f_y$ = **11.440** in^2
	PASS - Area of reinforcement provided is greater than minimum area of reinforcement required
Stress block depth factor (cl.22.2.2.4.3)	$\beta_1 = min(max(0.85 - 0.05 \times (f'_c - 4\ ksi) / 1\ ksi, 0.65), 0.85)$ = **0.85**
Depth of equivalent rectangular stress block	$a = A_{s,prov} \times f_y / (0.85 \times f'_c \times b)$ = **2.843** in
Depth to neutral axis	$c = a / \beta_1$ = **3.345** in
Net tensile strain in extreme tension fibers	$\varepsilon_t = 0.003 \times (d_o - c) / max(c, 0.001\ in)$ = **0.03646**
	Net tensile strain in tension controlled zone
Strength reduction factor (cl.21.2.1)	$\phi_f = min(max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.003), 0.65), 0.9)$ = **0.90**
Nominal moment strength	$M_n = A_{s,prov} \times f_y \times (d - a / 2)$ = **2675.284** kip_ft
Design moment strength	$\phi M_n = M_n \times \phi_f$ = **2407.755** kip_ft
	PASS - Required moment strength is less than design moment strength

Flexural cracking

Max. center to center spacing of tension reinf.	$s_{b,max} = s_{bot} + \phi_{m1\_s1\_z1\_b\_L1}$ = **4.667** in
Service load stress in reinforcement (cl.24.3.2)	$f_s = 2/3 \times f_y$ = **40000** psi
Clear cover of reinforcement	$c_c = c_{nom\_b} + \phi_v$ = **3.500** in
Maximum allowable bot bar spacing (Table 24.3.2)	$s_{max} = min(15in \times 40000psi / f_s - 2.5 \times c_c, 12in \times 40000psi / f_s)$ = **6.250** in
	PASS - Maximum allowable tension reinforcement spacing exceeds actual spacing

Spacing limits for reinforcement

Top bar clear spacing	$s_{top} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z1\_v}) + \phi_{m1\_s1\_z1\_t\_L1} \times N_{m1\_s1\_z1\_t\_L1} + \phi_{m1\_s1\_z2l\_t\_L1} \times N_{m1\_s1\_z2l\_t\_L1})) / ((N_{m1\_s1\_z1\_t\_L1} + N_{m1\_s1\_z2l\_t\_L1}) - 1)$ = **3.667** in
Min. allowable top bar clear spacing (cl.25.2.1)	$s_{top,min}$ = **1.000** in
Bottom bar clear spacing	$s_{bot} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z1\_v}) + \phi_{m1\_s1\_z1\_b\_L1} \times N_{m1\_s1\_z1\_b\_L1})) / (N_{m1\_s1\_z1\_b\_L1} - 1)$ = **3.667** in
Min. allowable bottom bar clear spacing (cl.25.2.1)	$s_{bot,min}$ = **1.000** in
	PASS - Actual bar spacing exceeds minimum allowable

Zone 2 - Positive moment. Rectangular section in flexure (Section 9.5.2)

Factored bending moment at section	$M_u = abs(M_{m1\_s1\_z2\_max\_red})$ = **336.045** kip_ft
Effective depth to tension reinforcement	d = **44** in
Tension reinforcement provided	16 × No.8 bars
Area of tension reinforcement provided	$A_{s,prov}$ = **12.566** in^2
Minimum area of reinforcement (cl.9.6.1.2)	$A_{s,min} = max(3\ psi \times \sqrt{(f'_c / 1\ psi)}, 200\ psi) \times b \times d / f_y$ = **11.440** in^2
	PASS - Area of reinforcement provided is greater than minimum area of reinforcement required
Stress block depth factor (cl.22.2.2.4.3)	$\beta_1 = min(max(0.85 - 0.05 \times (f'_c - 4\ ksi) / 1\ ksi, 0.65), 0.85)$ = **0.85**
Depth of equivalent rectangular stress block	$a = A_{s,prov} \times f_y / (0.85 \times f'_c \times b)$ = **2.843** in
Depth to neutral axis	$c = a / \beta_1$ = **3.345** in
Net tensile strain in extreme tension fibers	$\varepsilon_t = 0.003 \times (d_o - c) / max(c, 0.001\ in)$ = **0.03646**

PROJECT QTS DEN1-DC2
DESCRIPTION
PROJECT # 24053.00000 DATE 05/30/2025
PREPARED BY LB CHECKED BY ND

Project				Job Ref.	
QTS DEN1 DC2				24053.00000	
Section				Sheet no./rev.	
GB5				6	
Calc. by	Date	Chk'd by	Date	App'd by	Date
LB	4/1/2025				

Net tensile strain in tension controlled zone

Strength reduction factor (cl.21.2.1)	$\phi_t = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.003), 0.65), 0.9) =$ **0.90**
Nominal moment strength	$M_n = A_{s,prov} \times f_y \times (d - a / 2) =$ **2675.284** kip_ft
Design moment strength	$\phi M_n = M_n \times \phi_t =$ **2407.755** kip_ft

PASS - Required moment strength is less than design moment strength

Flexural cracking

Max. center to center spacing of tension reinf.	$s_{b,max} = s_{bot} + \phi_{m1\_s1\_z2\_b\_L1} =$ **4.667** in
Service load stress in reinforcement (cl.24.3.2)	$f_s = 2/3 \times f_y =$ **40000** psi
Clear cover of reinforcement	$c_c = c_{nom\_b} + \phi_v =$ **3.500** in
Maximum allowable bot bar spacing (Table 24.3.2)	$s_{max} = \min(15in \times 40000psi / f_s - 2.5 \times c_c, 12in \times 40000psi / f_s) =$ **6.250** in

PASS - Maximum allowable tension reinforcement spacing exceeds actual spacing

Control of deflections (Section 24.2)

Concrete density factor	$K_w =$ **1.00**
Reinforcement yield strength factor	$K_f = 0.4 + f_y / 100000$ psi = **1.00**
Minimum thickness of beam (Table 9.3.1.1)	$h_{min} = (L_{m1\_s1} / 18.5) \times K_w \times K_f =$ **15.568** in

PASS - Thickness of beam exceeds minimum thickness

Spacing limits for reinforcement

Top bar clear spacing	$s_{top} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z2\_v}) + \phi_{m1\_s1\_z2r\_t\_L1} \times N_{m1\_s1\_z2r\_t\_L1})) / (N_{m1\_s1\_z2r\_t\_L1} - 1) =$ **3.667** in
Min. allowable top bar clear spacing (cl.25.2.1)	$s_{top,min} =$ **1.000** in
Bottom bar clear spacing	$s_{bot} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z2\_v}) + \phi_{m1\_s1\_z2\_b\_L1} \times N_{m1\_s1\_z2\_b\_L1} + \phi_{m1\_s1\_z1\_b\_L1} \times N_{m1\_s1\_z1\_b\_L1})) / ((N_{m1\_s1\_z2\_b\_L1} + N_{m1\_s1\_z1\_b\_L1}) - 1) =$ **3.667** in
Min. allowable bottom bar clear spacing (cl.25.2.1)	$s_{bot,min} =$ **1.000** in

PASS - Actual bar spacing exceeds minimum allowable

Zone 3 - Positive moment. Rectangular section in flexure (Section 9.5.2)

Factored bending moment at section	$M_u = \text{abs}(M_{m1\_s1\_z3\_max\_red}) =$ **8.672** kip_ft
Effective depth to tension reinforcement	$d =$ **44** in
Tension reinforcement provided	16 × No.8 bars
Area of tension reinforcement provided	$A_{s,prov} =$ **12.566** in^2
Minimum area of reinforcement (cl.9.6.1.2)	$A_{s,min} = \max(3 \text{ psi} \times \sqrt{(f'_c / 1 \text{ psi})}, 200 \text{ psi}) \times b \times d / f_y =$ **11.440** in^2

PASS - Area of reinforcement provided is greater than minimum area of reinforcement required

Stress block depth factor (cl.22.2.2.4.3)	$\beta_1 = \min(\max(0.85 - 0.05 \times (f'_c - 4 \text{ ksi}) / 1 \text{ ksi}, 0.65), 0.85) =$ **0.85**
Depth of equivalent rectangular stress block	$a = A_{s,prov} \times f_y / (0.85 \times f'_c \times b) =$ **2.843** in
Depth to neutral axis	$c = a / \beta_1 =$ **3.345** in
Net tensile strain in extreme tension fibers	$\varepsilon_t = 0.003 \times (d_o - c) / \max(c, 0.001 \text{ in}) =$ **0.03646**

Net tensile strain in tension controlled zone

Strength reduction factor (cl.21.2.1)	$\phi_t = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.003), 0.65), 0.9) =$ **0.90**
Nominal moment strength	$M_n = A_{s,prov} \times f_y \times (d - a / 2) =$ **2675.284** kip_ft
Design moment strength	$\phi M_n = M_n \times \phi_t =$ **2407.755** kip_ft

PASS - Required moment strength is less than design moment strength

Flexural cracking

Max. center to center spacing of tension reinf.	$s_{b,max} = s_{bot} + \phi_{m1\_s1\_z3\_b\_L1} =$ **4.667** in

PROJECT QTS DEN1-DC2
DESCRIPTION
PROJECT # 24053.00000 DATE 05/30/2025
PREPARED BY LB CHECKED BY ND

Project				Job Ref.	
QTS DEN1 DC2				24053.00000	
Section				Sheet no./rev.	
GB5				7	
Calc. by	Date	Chk'd by	Date	App'd by	Date
LB	4/1/2025				

Service load stress in reinforcement (cl.24.3.2) $f_s = 2/3 \times f_y =$ **40000** psi

Clear cover of reinforcement $c_c = c_{nom\_b} + \phi_v =$ **3.500** in

Maximum allowable bot bar spacing (Table 24.3.2) $s_{max} = \min(15in \times 40000psi / f_s - 2.5 \times c_c, 12in \times 40000psi / f_s) =$ **6.250** in

PASS - Maximum allowable tension reinforcement spacing exceeds actual spacing

Zone 3 - Negative moment. Rectangular section in flexure (Section 9.5.2)

Factored bending moment at section $M_u = abs(M_{m1\_s1\_z3\_min\_red}) =$ **578.118** kip_ft

Effective depth to tension reinforcement d = **45.5** in

Tension reinforcement provided 16 × No.8 bars

Area of tension reinforcement provided $A_{s,prov} =$ **12.566** in²

Minimum area of reinforcement (cl.9.6.1.2) $A_{s,min} = \max(3 psi \times \sqrt{(f'_c / 1 psi)}, 200 psi) \times b \times d / f_y =$ **11.830** in²

PASS - Area of reinforcement provided is greater than minimum area of reinforcement required

Stress block depth factor (cl.22.2.2.4.3) $\beta_1 = \min(\max(0.85 - 0.05 \times (f'_c - 4 ksi) / 1 ksi, 0.65), 0.85) =$ **0.85**

Depth of equivalent rectangular stress block $a = A_{s,prov} \times f_y / (0.85 \times f'_c \times b) =$ **2.843** in

Depth to neutral axis $c = a / \beta_1 =$ **3.345** in

Net tensile strain in extreme tension fibers $\varepsilon_t = 0.003 \times (d_o - c) / \max(c, 0.001 in) =$ **0.03781**

Net tensile strain in tension controlled zone

Strength reduction factor (cl.21.2.1) $\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.003), 0.65), 0.9) =$ **0.90**

Nominal moment strength $M_n = A_{s,prov} \times f_y \times (d - a / 2) =$ **2769.532** kip_ft

Design moment strength $\phi M_n = M_n \times \phi_f =$ **2492.578** kip_ft

PASS - Required moment strength is less than design moment strength

Flexural cracking

Max. center to center spacing of tension reinf. $s_{t,max} = s_{top} + \phi_{m1\_s1\_z3\_t\_L1} =$ **4.667** in

Service load stress in reinforcement (cl.24.3.2) $f_s = 2/3 \times f_y =$ **40000** psi

Clear cover of reinforcement $c_c = c_{nom\_t} + \phi_v =$ **2.000** in

Maximum allowable top bar spacing (Table 24.3.2) $s_{max} = \min(15in \times 40000psi / f_s - 2.5 \times c_c, 12in \times 40000psi / f_s) =$ **10.000** in

PASS - Maximum allowable tension reinforcement spacing exceeds actual spacing

Spacing limits for reinforcement

Top bar clear spacing $s_{top} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z3\_v}) + \phi_{m1\_s1\_z3\_t\_L1} \times N_{m1\_s1\_z3\_t\_L1} + \phi_{m1\_s1\_z2r\_t\_L1} \times N_{m1\_s1\_z2r\_t\_L1})) / ((N_{m1\_s1\_z3\_t\_L1} + N_{m1\_s1\_z2r\_t\_L1}) - 1) =$ **3.667** in

Min. allowable top bar clear spacing (cl.25.2.1) $s_{top,min} =$ **1.000** in

Bottom bar clear spacing $s_{bot} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z3\_v}) + \phi_{m1\_s1\_z3\_b\_L1} \times N_{m1\_s1\_z3\_b\_L1})) / (N_{m1\_s1\_z3\_b\_L1} - 1) =$ **3.667** in

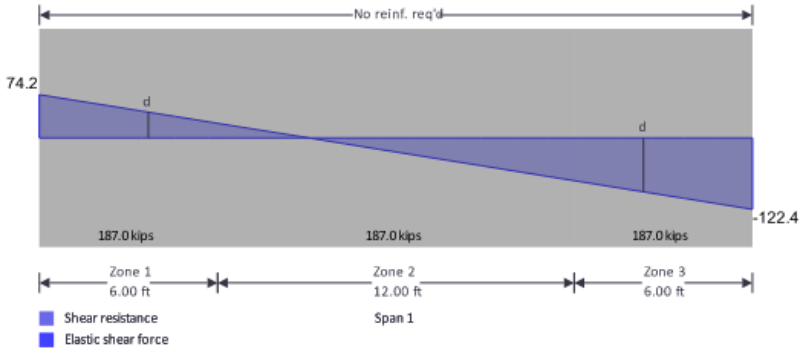
Min. allowable bottom bar clear spacing (cl.25.2.1) $s_{bot,min} =$ **1.000** in

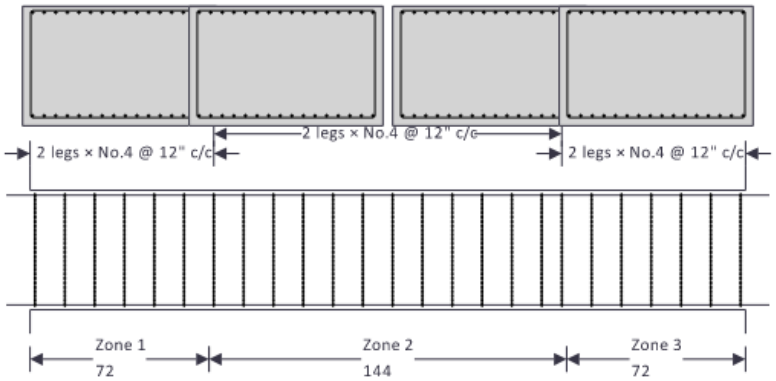
PASS - Actual bar spacing exceeds minimum allowable

PROJECT QTS DEN1-DC2
DESCRIPTION
PROJECT # 24053.00000 DATE 05/30/2025
PREPARED BY LB CHECKED BY ND

Project				Job Ref.	
QTS DEN1 DC2				24053.00000	
Section				Sheet no./rev.	
GB5				8	
Calc. by	Date	Chk'd by	Date	App'd by	Date
LB	4/1/2025				

Shear design





Rectangular section in shear

Concrete weight modification factor	λ = **1.00**
Location where min. reinf. is req'd (V_u less ϕV_c)	N/A no reinf. required along full length
Location where no reinf. is req'd (V_u less $\phi V_{s,lim}$)	Between 0.00 ft and 24.00 ft
Maximum reinforcement shear strength	$\phi V_{s,max} = \phi_s \times 8$ psi $\times \sqrt{(\min(f'_c, 10000\text{psi}) / 1\text{ psi})} \times b \times d$ = **1302.352** kips
Minimum area of shear reinf. (Table 9.6.3.4)	$A_{sv,min} = \max(50\text{ psi}, 0.75\text{ psi} \times \sqrt{(f'_c / 1\text{ psi})}) \times b / \min(f_y, 60000\text{ psi})$ = **0.780** in²/ft
No reinforcement shear strength limit	$\phi V_{s,lim} = \phi_s \times \lambda \times 1\text{psi} \times \sqrt{(\min(f'_c, 10000\text{psi}) / 1\text{ psi})} \times b \times d$ = **162.794** kips

Zone 1

Effective depth of long. reinf. used for shear zone	d = **44** in
Size effect factor (cl. 22.5.5.1.3)	$\lambda_s = \min(\sqrt{(2 / (1 + (d / 1\text{ in}) / 10))}, 1.0)$ = **0.61**
Area of longitudinal reinforcement	A_{st} = **12.566** in²
Ratio of longitudinal reinforcement	$\rho_w = A_{st} / (b \times d)$ = **0.004**

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

Project: QTS DEN1 DC2				Job Ref.: 24053.00000	
Section: GB5				Sheet no./rev.: 9	
Calc. by: LB	Date: 4/1/2025	Chk'd by:	Date:	App'd by:	Date:

Concrete shear strength (Table 22.5.5.1)	$\phi V_c = \phi_s \times \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3}, 5 \times \lambda) \times \sqrt{(f'_c \times 1\ psi)} \times b \times d =$ **122.161** kips
Design shear force at 44.0in from support	$V_u =$ **44.162** kips
Reinf. shear strength required (eqn. 22.5.1.1)	$\phi V_s = \max(V_u - \phi V_c, 0\ kips) =$ **0.000** kips
Area of design shear reinf. req'd (eqn. 22.5.8.5.3)	$A_{sv,des} = \phi V_s / (\phi_s \times \min(f_y, 60000\ psi) \times d) =$ **0.000** in^2/ft
Area of shear reinforcement required	$A_{sv,req} = 0mm^2/ft =$ **0.000** in^2/ft
Shear reinforcement provided	2 legs × No.4 @ 12" c/c
Area of shear reinforcement provided	$A_{sv,prov} =$ **0.393** in^2/ft
	PASS - No shear reinforcement required ($\phi V_{s,lim} >= V_u$)
Zone 2	
Effective depth of long. reinf. used for shear zone	d = **44** in
Size effect factor (cl. 22.5.5.1.3)	$\lambda_s = \min(\sqrt{(2 / (1 + (d / 1\ in) / 10))}, 1.0) =$ **0.61**
Area of longitudinal reinforcement	$A_{st} =$ **12.566** in^2
Ratio of longitudinal reinforcement	$\rho_w = A_{st} / (b \times d) =$ **0.004**
Concrete shear strength (Table 22.5.5.1)	$\phi V_c = \phi_s \times \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3}, 5 \times \lambda) \times \sqrt{(f'_c \times 1\ psi)} \times b \times d =$ **122.161** kips
Design shear force within zone	$V_u =$ **73.228** kips
Reinf. shear strength required (eqn. 22.5.1.1)	$\phi V_s = \max(V_u - \phi V_c, 0\ kips) =$ **0.000** kips
Area of design shear reinf. req'd (eqn. 22.5.8.5.3)	$A_{sv,des} = \phi V_s / (\phi_s \times \min(f_y, 60000\ psi) \times d) =$ **0.000** in^2/ft
Area of shear reinforcement required	$A_{sv,req} = 0mm^2/ft =$ **0.000** in^2/ft
Shear reinforcement provided	2 legs × No.4 @ 12" c/c
Area of shear reinforcement provided	$A_{sv,prov} =$ **0.393** in^2/ft
	PASS - No shear reinforcement required ($\phi V_{s,lim} >= V_u$)
Zone 3	
Effective depth of long. reinf. used for shear zone	d = **44** in
Size effect factor (cl. 22.5.5.1.3)	$\lambda_s = \min(\sqrt{(2 / (1 + (d / 1\ in) / 10))}, 1.0) =$ **0.61**
Area of longitudinal reinforcement	$A_{st} =$ **12.566** in^2
Ratio of longitudinal reinforcement	$\rho_w = A_{st} / (b \times d) =$ **0.004**
Concrete shear strength (Table 22.5.5.1)	$\phi V_c = \phi_s \times \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3}, 5 \times \lambda) \times \sqrt{(f'_c \times 1\ psi)} \times b \times d =$ **122.161** kips
Design shear force at 44.0in from support	$V_u =$ **92.338** kips
Reinf. shear strength required (eqn. 22.5.1.1)	$\phi V_s = \max(V_u - \phi V_c, 0\ kips) =$ **0.000** kips
Area of design shear reinf. req'd (eqn. 22.5.8.5.3)	$A_{sv,des} = \phi V_s / (\phi_s \times \min(f_y, 60000\ psi) \times d) =$ **0.000** in^2/ft
Area of shear reinforcement required	$A_{sv,req} = 0mm^2/ft =$ **0.000** in^2/ft
Shear reinforcement provided	2 legs × No.4 @ 12" c/c
Area of shear reinforcement provided	$A_{sv,prov} =$ **0.393** in^2/ft
	PASS - No shear reinforcement required ($\phi V_{s,lim} >= V_u$)

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

Project: QTS DEN1 DC2				Job Ref.: 24053.00000	
Section: GB5				Sheet no./rev.: 9	
Calc. by: LB	Date: 4/1/2025	Chk'd by:	Date:	App'd by:	Date:

Concrete shear strength (Table 22.5.5.1)	$\phi V_c = \phi_s \times \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3}, 5 \times \lambda) \times \sqrt{(f'_c \times 1\ psi)} \times b \times d =$ **122.161** kips
Design shear force at 44.0in from support	$V_u =$ **44.162** kips
Reinf. shear strength required (eqn. 22.5.1.1)	$\phi V_s = \max(V_u - \phi V_c, 0\ kips) =$ **0.000** kips
Area of design shear reinf. req'd (eqn. 22.5.8.5.3)	$A_{sv,des} = \phi V_s / (\phi_s \times \min(f_y, 60000\ psi) \times d) =$ **0.000** in^2/ft
Area of shear reinforcement required	$A_{sv,req} = 0mm^2/ft =$ **0.000** in^2/ft
Shear reinforcement provided	2 legs × No.4 @ 12" c/c
Area of shear reinforcement provided	$A_{sv,prov} =$ **0.393** in^2/ft
	PASS - No shear reinforcement required ($\phi V_{s,lim} >= V_u$)
Zone 2	
Effective depth of long. reinf. used for shear zone	d = **44** in
Size effect factor (cl. 22.5.5.1.3)	$\lambda_s = \min(\sqrt{(2 / (1 + (d / 1\ in) / 10))}, 1.0) =$ **0.61**
Area of longitudinal reinforcement	$A_{st} =$ **12.566** in^2
Ratio of longitudinal reinforcement	$\rho_w = A_{st} / (b \times d) =$ **0.004**
Concrete shear strength (Table 22.5.5.1)	$\phi V_c = \phi_s \times \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3}, 5 \times \lambda) \times \sqrt{(f'_c \times 1\ psi)} \times b \times d =$ **122.161** kips
Design shear force within zone	$V_u =$ **73.228** kips
Reinf. shear strength required (eqn. 22.5.1.1)	$\phi V_s = \max(V_u - \phi V_c, 0\ kips) =$ **0.000** kips
Area of design shear reinf. req'd (eqn. 22.5.8.5.3)	$A_{sv,des} = \phi V_s / (\phi_s \times \min(f_y, 60000\ psi) \times d) =$ **0.000** in^2/ft
Area of shear reinforcement required	$A_{sv,req} = 0mm^2/ft =$ **0.000** in^2/ft
Shear reinforcement provided	2 legs × No.4 @ 12" c/c
Area of shear reinforcement provided	$A_{sv,prov} =$ **0.393** in^2/ft
	PASS - No shear reinforcement required ($\phi V_{s,lim} >= V_u$)
Zone 3	
Effective depth of long. reinf. used for shear zone	d = **44** in
Size effect factor (cl. 22.5.5.1.3)	$\lambda_s = \min(\sqrt{(2 / (1 + (d / 1\ in) / 10))}, 1.0) =$ **0.61**
Area of longitudinal reinforcement	$A_{st} =$ **12.566** in^2
Ratio of longitudinal reinforcement	$\rho_w = A_{st} / (b \times d) =$ **0.004**
Concrete shear strength (Table 22.5.5.1)	$\phi V_c = \phi_s \times \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3}, 5 \times \lambda) \times \sqrt{(f'_c \times 1\ psi)} \times b \times d =$ **122.161** kips
Design shear force at 44.0in from support	$V_u =$ **92.338** kips
Reinf. shear strength required (eqn. 22.5.1.1)	$\phi V_s = \max(V_u - \phi V_c, 0\ kips) =$ **0.000** kips
Area of design shear reinf. req'd (eqn. 22.5.8.5.3)	$A_{sv,des} = \phi V_s / (\phi_s \times \min(f_y, 60000\ psi) \times d) =$ **0.000** in^2/ft
Area of shear reinforcement required	$A_{sv,req} = 0mm^2/ft =$ **0.000** in^2/ft
Shear reinforcement provided	2 legs × No.4 @ 12" c/c
Area of shear reinforcement provided	$A_{sv,prov} =$ **0.393** in^2/ft
	PASS - No shear reinforcement required ($\phi V_{s,lim} >= V_u$)

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

Project: QTS DEN1 DC2				Job Ref.: 24053.00000	
Section: GB6				Sheet no./rev.: 1	
Calc. by: LB	Date: 4/1/2025	Chk'd by:	Date:	App'd by:	Date:

RC BEAM ANALYSIS & DESIGN (ACI318-2019 (22))

In accordance with ACI318-2019 (22)

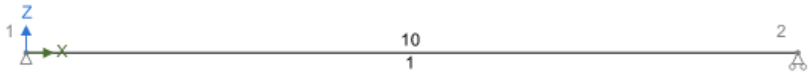
Tedds calculation version 3.3.19

ANALYSIS

Tedds calculation version 1.0.38

Geometry

Geometry (ft) - Concrete (ACI normal) - R 24x24



Span	Length (ft)	Section	Start Support	End Support
1	10	R 24x24	Pinned	Roller Pin X
R 24x24: Area 576 in^2, Inertia Major 27648 in^4, Inertia Minor 27648 in^4, Shear area parallel to Minor 480 in^2, Shear area parallel to Major 480 in^2				
Concrete (ACI normal): Density 150 lbm/ft^3, Youngs 4200 ksi, Shear 1750 ksi, Thermal 0.00001 $°C^{-1}$				

Loading

Self weight included

Dead - Loading (kips/ft)



Live - Loading (kips/ft)



Load combination factors

Load combination	Self Weight	Dead	Live
1.2D + 1.6L (Strength)	1.20	1.20	1.60
1.0D + 1.0L (Service)	1.00	1.00	1.00
1.4D (Strength)	1.40	1.40	
1.0D (Service)	1.00	1.00	

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

Project: QTS DEN1 DC2				Job Ref.: 24053.00000	
Section: GB6				Sheet no./rev.: 2	
Calc. by: LB	Date: 4/1/2025	Chk'd by:	Date:	App'd by:	Date:

Member Loads

Member	Load case	Load Type	Orientation	Description
Beam	Dead	UDL	GlobalZ	1 kips/ft
Beam	Live	UDL	GlobalZ	1 kips/ft

Results

Reactions

Load case: Self Weight

Node	Force Fx (kips)	Force Fz (kips)	Moment My (kip_ft)
1	0	3	0
2	0	3	0

Load case: Dead

Node	Force Fx (kips)	Force Fz (kips)	Moment My (kip_ft)
1	0	5	0
2	0	5	0

Load case: Live

Node	Force Fx (kips)	Force Fz (kips)	Moment My (kip_ft)
1	0	5	0
2	0	5	0

Load combination: 1.2D + 1.6L (Strength)

Node	Force Fx (kips)	Force Fz (kips)	Moment My (kip_ft)
1	0	17.6	0
2	0	17.6	0

Load combination: 1.0D + 1.0L (Service)

Node	Force Fx (kips)	Force Fz (kips)	Moment My (kip_ft)
1	0	13	0
2	0	13	0

Load combination: 1.4D (Strength)

Node	Force Fx (kips)	Force Fz (kips)	Moment My (kip_ft)
1	0	11.2	0
2	0	11.2	0

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

Project: QTS DEN1 DC2				Job Ref.: 24053.00000	
Section: GB6				Sheet no./rev.: 3	
Calc. by: LB	Date: 4/1/2025	Chk'd by:	Date:	App'd by:	Date:

Load combination: 1.0D (Service)

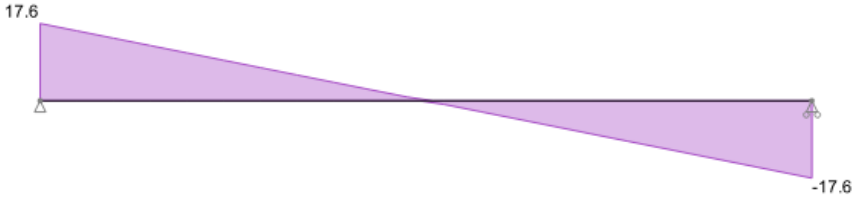
Node	Force Fx (kips)	Force Fz (kips)	Moment My (kip_ft)
1	0	8	0
2	0	8	0

Forces

All combinations - Moment envelope (kip_ft)



All combinations - Shear envelope (kips)



Member results

Envelope - All combinations

Member	Position (ft)	Shear force (kips)		Moment (kip_ft)	
Beam	0	17.6	8	0 (min)	
	5	0		44 (max)	20
	10	-8	-17.6 (max abs)	0 (min)	

Concrete details

Compressive strength of concrete	f'_c = **4000** psi
Density of reinforced concrete	w_c = **150** lb / ft^3
Concrete type	Normal weight
Modulus of elasticity of concrete (cl.19.2.2.1)	$E = (w_c / 1\ lb/ft^3)^{1.5} \times 33\ psi \times (f'_c / 1\ psi)^{0.5}$ = **3834254** psi
Strength reduction factor for shear	ϕ_s = **0.75**

Reinforcement details

Yield strength of reinforcement	f_y = **60000** psi
Compression-controlled strain limit (cl.21.2.2.1)	ε_{ty} = **0.00200**

Nominal cover to reinforcement

Cover to top reinforcement	c_{nom_t} = **1.5** in
Cover to bottom reinforcement	c_{nom_b} = **3** in

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

Project: QTS DEN1 DC2				Job Ref.: 24053.00000	
Section: GB6				Sheet no./rev.: 4	
Calc. by: LB	Date: 4/1/2025	Chk'd by	Date	App'd by	Date

Cover to side reinforcement c_{nom_s} = **3** in

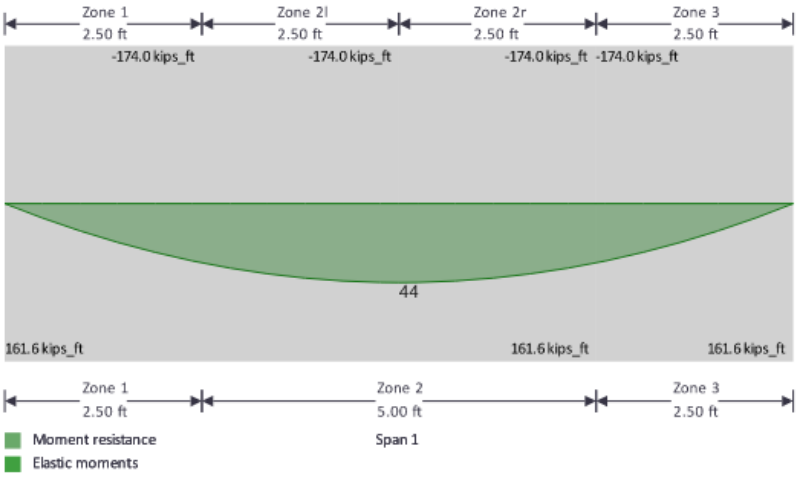
Beam - Span 1

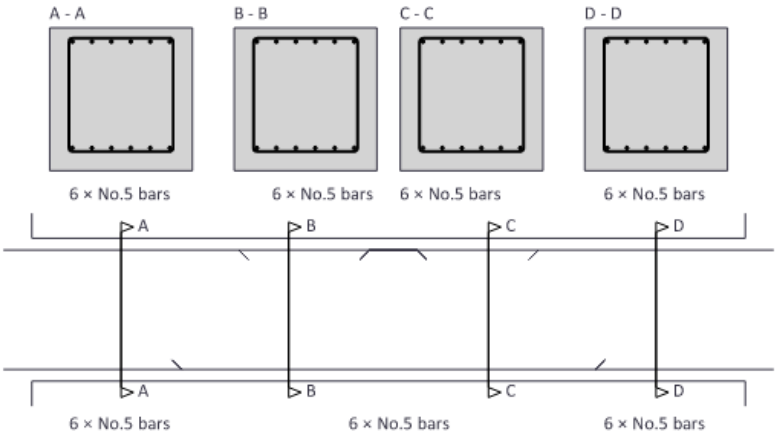
Rectangular section details

Section width b = **24** in

Section depth h = **24** in

Moment design





Zone 1 - Positive moment. Rectangular section in flexure (Section 9.5.2)

Factored bending moment at section M_u = abs($M_{m1\_s1\_z1\_max\_red}$) = **33.000** kip_ft

Effective depth to tension reinforcement d = **20.187** in

Tension reinforcement provided 6 × No.5 bars

PROJECT QTS DEN1-DC2
DESCRIPTION
PROJECT # 24053.00000 DATE 05/30/2025
PREPARED BY LB CHECKED BY ND

Project				Job Ref.	
QTS DEN1 DC2				24053.00000	
Section				Sheet no./rev.	
GB6				5	
Calc. by	Date	Chk'd by	Date	App'd by	Date
LB	4/1/2025				

Area of tension reinforcement provided	$A_{s,prov}$ = **1.841** in²
Minimum area of reinforcement (cl.9.6.1.2)	$A_{s,min}$ = max(3 psi × √(f'c / 1 psi), 200 psi) × b × d / f_y = **1.615** in²
	PASS - Area of reinforcement provided is greater than minimum area of reinforcement required
Stress block depth factor (cl.22.2.2.4.3)	β_1 = min(max(0.85 - 0.05 × (f'c - 4 ksi) / 1 ksi, 0.65), 0.85) = **0.85**
Depth of equivalent rectangular stress block	a = $A_{s,prov}$ × f_y / (0.85 × f'c × b) = **1.354** in
Depth to neutral axis	c = a / β_1 = **1.592** in
Net tensile strain in extreme tension fibers	ε_t = 0.003 × (d_o - c) / max(c, 0.001 in) = **0.03503**
	Net tensile strain in tension controlled zone
Strength reduction factor (cl.21.2.1)	ϕ_f = min(max(0.65 + 0.25 × (ε_t - ε_{ty}) / (0.003), 0.65), 0.9) = **0.90**
Nominal moment strength	M_n = $A_{s,prov}$ × f_y × (d - a / 2) = **179.575** kip_ft
Design moment strength	ϕM_n = M_n × ϕ_f = **161.617** kip_ft
	PASS - Required moment strength is less than design moment strength
Flexural cracking	
Max. center to center spacing of tension reinf.	$s_{b,max}$ = s_{bot} + $\phi_{m1\_s1\_z1\_b\_L1}$ = **3.275** in
Service load stress in reinforcement (cl.24.3.2)	f_s = 2/3 × f_y = **40000** psi
Clear cover of reinforcement	c_c = c_{nom_b} + ϕ_v = **3.500** in
Maximum allowable bot bar spacing (Table 24.3.2)	s_{max} = min(15in × 40000psi / f_s - 2.5 × c_c, 12in × 40000psi / f_s) = **6.250** in
	PASS - Maximum allowable tension reinforcement spacing exceeds actual spacing
Spacing limits for reinforcement	
Top bar clear spacing	s_{top} = (b - (2 × (c_{nom_s} + $\phi_{m1\_s1\_z1\_v}$) + $\phi_{m1\_s1\_z1\_t\_L1}$ × $N_{m1\_s1\_z1\_t\_L1}$ + $\phi_{m1\_s1\_z2l\_t\_L1}$ × $N_{m1\_s1\_z2l\_t\_L1}$)) / (($N_{m1\_s1\_z1\_t\_L1}$ + $N_{m1\_s1\_z2l\_t\_L1}$) - 1) = **2.650** in
Min. allowable top bar clear spacing (cl.25.2.1)	$s_{top,min}$ = **1.000** in
Bottom bar clear spacing	s_{bot} = (b - (2 × (c_{nom_s} + $\phi_{m1\_s1\_z1\_v}$) + $\phi_{m1\_s1\_z1\_b\_L1}$ × $N_{m1\_s1\_z1\_b\_L1}$)) / ($N_{m1\_s1\_z1\_b\_L1}$ - 1) = **2.650** in
Min. allowable bottom bar clear spacing (cl.25.2.1)	$s_{bot,min}$ = **1.000** in
	PASS - Actual bar spacing exceeds minimum allowable
Zone 2 - Positive moment. Rectangular section in flexure (Section 9.5.2)	
Factored bending moment at section	M_u = abs($M_{m1\_s1\_z2\_max\_red}$) = **44.000** kip_ft
Effective depth to tension reinforcement	d = **20.187** in
Tension reinforcement provided	6 × No.5 bars
Area of tension reinforcement provided	$A_{s,prov}$ = **1.841** in²
Minimum area of reinforcement (cl.9.6.1.2)	$A_{s,min}$ = max(3 psi × √(f'c / 1 psi), 200 psi) × b × d / f_y = **1.615** in²
	PASS - Area of reinforcement provided is greater than minimum area of reinforcement required
Stress block depth factor (cl.22.2.2.4.3)	β_1 = min(max(0.85 - 0.05 × (f'c - 4 ksi) / 1 ksi, 0.65), 0.85) = **0.85**
Depth of equivalent rectangular stress block	a = $A_{s,prov}$ × f_y / (0.85 × f'c × b) = **1.354** in
Depth to neutral axis	c = a / β_1 = **1.592** in
Net tensile strain in extreme tension fibers	ε_t = 0.003 × (d_o - c) / max(c, 0.001 in) = **0.03503**
	Net tensile strain in tension controlled zone
Strength reduction factor (cl.21.2.1)	ϕ_f = min(max(0.65 + 0.25 × (ε_t - ε_{ty}) / (0.003), 0.65), 0.9) = **0.90**
Nominal moment strength	M_n = $A_{s,prov}$ × f_y × (d - a / 2) = **179.575** kip_ft
Design moment strength	ϕM_n = M_n × ϕ_f = **161.617** kip_ft
	PASS - Required moment strength is less than design moment strength

PROJECT QTS DEN1-DC2
DESCRIPTION
PROJECT # 24053.00000 DATE 05/30/2025
PREPARED BY LB CHECKED BY ND

Project				Job Ref.	
QTS DEN1 DC2				24053.00000	
Section				Sheet no./rev.	
GB6				6	
Calc. by	Date	Chk'd by	Date	App'd by	Date
LB	4/1/2025				

Flexural cracking

Max. center to center spacing of tension reinf.	$s_{b,max} = s_{bot} + \phi_{m1\_s1\_z2\_b\_L1}$ = **3.275** in
Service load stress in reinforcement (cl.24.3.2)	$f_s = 2/3 \times f_y$ = **40000** psi
Clear cover of reinforcement	$c_c = c_{nom\_b} + \phi_v$ = **3.500** in
Maximum allowable bot bar spacing (Table 24.3.2)	s_{max} = min(15in × 40000psi / f_s - 2.5 × c_c, 12in × 40000psi / f_s) = **6.250** in

PASS - Maximum allowable tension reinforcement spacing exceeds actual spacing

Control of deflections (Section 24.2)

Concrete density factor	K_w = **1.00**
Reinforcement yield strength factor	$K_f = 0.4 + f_y / 100000$ psi = **1.00**
Minimum thickness of beam (Table 9.3.1.1)	$h_{min} = (L_{m1\_s1} / 16.0) \times K_w \times K_f$ = **7.5** in

PASS - Thickness of beam exceeds minimum thickness

Spacing limits for reinforcement

Top bar clear spacing	$s_{top} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z2\_v}) + \phi_{m1\_s1\_z2r\_t\_L1} \times N_{m1\_s1\_z2r\_t\_L1})) / (N_{m1\_s1\_z2r\_t\_L1} - 1)$ = **2.650** in
Min. allowable top bar clear spacing (cl.25.2.1)	$s_{top,min}$ = **1.000** in
Bottom bar clear spacing	$s_{bot} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z2\_v}) + \phi_{m1\_s1\_z2\_b\_L1} \times N_{m1\_s1\_z2\_b\_L1} + \phi_{m1\_s1\_z1\_b\_L1} \times N_{m1\_s1\_z1\_b\_L1})) / ((N_{m1\_s1\_z2\_b\_L1} + N_{m1\_s1\_z1\_b\_L1}) - 1)$ = **2.650** in
Min. allowable bottom bar clear spacing (cl.25.2.1)	$s_{bot,min}$ = **1.000** in

PASS - Actual bar spacing exceeds minimum allowable

Zone 3 - Positive moment. Rectangular section in flexure (Section 9.5.2)

Factored bending moment at section	$M_u = abs(M_{m1\_s1\_z3\_max\_red})$ = **33.000** kip_ft
Effective depth to tension reinforcement	d = **20.187** in
Tension reinforcement provided	6 × No.5 bars
Area of tension reinforcement provided	$A_{s,prov}$ = **1.841** in^2
Minimum area of reinforcement (cl.9.6.1.2)	$A_{s,min}$ = max(3 psi × √(f'_c / 1 psi), 200 psi) × b × d / f_y = **1.615** in^2

PASS - Area of reinforcement provided is greater than minimum area of reinforcement required

Stress block depth factor (cl.22.2.2.4.3)	β_1 = min(max(0.85 - 0.05 × (f'_c - 4 ksi) / 1 ksi, 0.65), 0.85) = **0.85**
Depth of equivalent rectangular stress block	a = $A_{s,prov} \times f_y$ / (0.85 × f'_c × b) = **1.354** in
Depth to neutral axis	c = a / β_1 = **1.592** in
Net tensile strain in extreme tension fibers	ε_t = 0.003 × (d_o - c) / max(c, 0.001 in) = **0.03503**

Net tensile strain in tension controlled zone

Strength reduction factor (cl.21.2.1)	ϕ_f = min(max(0.65 + 0.25 × (ε_t - ε_{ty}) / (0.003), 0.65), 0.9) = **0.90**
Nominal moment strength	$M_n = A_{s,prov} \times f_y \times (d - a / 2)$ = **179.575** kip_ft
Design moment strength	$\phi M_n = M_n \times \phi_f$ = **161.617** kip_ft

PASS - Required moment strength is less than design moment strength

Flexural cracking

Max. center to center spacing of tension reinf.	$s_{b,max} = s_{bot} + \phi_{m1\_s1\_z3\_b\_L1}$ = **3.275** in
Service load stress in reinforcement (cl.24.3.2)	$f_s = 2/3 \times f_y$ = **40000** psi
Clear cover of reinforcement	$c_c = c_{nom\_b} + \phi_v$ = **3.500** in
Maximum allowable bot bar spacing (Table 24.3.2)	s_{max} = min(15in × 40000psi / f_s - 2.5 × c_c, 12in × 40000psi / f_s) = **6.250** in

PASS - Maximum allowable tension reinforcement spacing exceeds actual spacing

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

Project: QTS DEN1 DC2				Job Ref.: 24053.00000	
Section: GB6				Sheet no./rev.: 7	
Calc. by: LB	Date: 4/1/2025	Chk'd by:	Date:	App'd by:	Date:

Spacing limits for reinforcement

Top bar clear spacing $s_{top} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z3\_v}) + \phi_{m1\_s1\_z3\_t\_L1} \times N_{m1\_s1\_z3\_t\_L1} + \phi_{m1\_s1\_z2r\_t\_L1} \times N_{m1\_s1\_z2r\_t\_L1})) / ((N_{m1\_s1\_z3\_t\_L1} + N_{m1\_s1\_z2r\_t\_L1}) - 1) = \mathbf{2.650}$ in

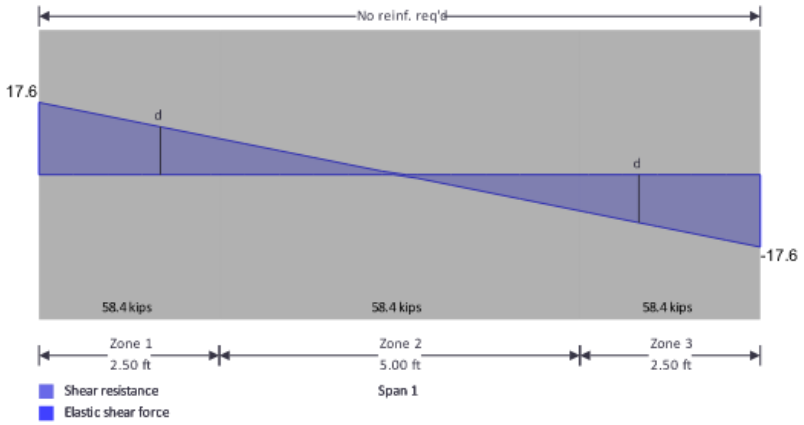
Min. allowable top bar clear spacing (cl.25.2.1) $s_{top,min} = \mathbf{1.000}$ in

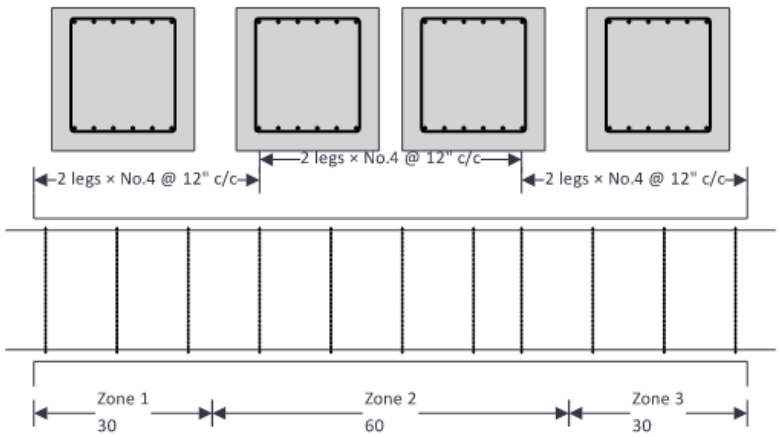
Bottom bar clear spacing $s_{bot} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z3\_v}) + \phi_{m1\_s1\_z3\_b\_L1} \times N_{m1\_s1\_z3\_b\_L1})) / (N_{m1\_s1\_z3\_b\_L1} - 1) = \mathbf{2.650}$ in

Min. allowable bottom bar clear spacing (cl.25.2.1) $s_{bot,min} = \mathbf{1.000}$ in

PASS - Actual bar spacing exceeds minimum allowable

Shear design





Rectangular section in shear

Concrete weight modification factor $\lambda = \mathbf{1.00}$

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

Project: QTS DEN1 DC2				Job Ref.: 24053.00000	
Section: GB6				Sheet no./rev.: 8	
Calc. by: LB	Date: 4/1/2025	Chk'd by	Date	App'd by	Date

Location where min. reinf. is req'd (V_u less ϕV_c)	N/A no reinf. required along full length
Location where no reinf. is req'd (V_u less $\phi V_{s,lim}$)	Between 0.00 ft and 10.00 ft
Maximum reinforcement shear strength	$\phi V_{s,max} = \phi_s \times 8$ psi $\times \sqrt{(\min(f'_c, 10000\text{psi}) / 1\text{ psi})} \times b \times d =$ **183.855** kips
Minimum area of shear reinf. (Table 9.6.3.4)	$A_{sv,min} = \max(50\text{ psi}, 0.75\text{ psi} \times \sqrt{(f'_c / 1\text{ psi})}) \times b / \min(f_y, 60000\text{ psi}) =$ **0.240** in²/ft
No reinforcement shear strength limit	$\phi V_{s,lim} = \phi_s \times \lambda \times 1\text{psi} \times \sqrt{(\min(f'_c, 10000\text{psi}) / 1\text{ psi})} \times b \times d =$ **22.982** kips
Zone 1	
Effective depth of long. reinf. used for shear zone	d = **20.187** in
Size effect factor (cl. 22.5.5.1.3)	$\lambda_s = \min(\sqrt{(2 / (1 + (d / 1\text{ in}) / 10))}, 1.0) =$ **0.81**
Area of longitudinal reinforcement	$A_{st} =$ **1.841** in²
Ratio of longitudinal reinforcement	$\rho_w = A_{st} / (b \times d) =$ **0.004**
Concrete shear strength (Table 22.5.5.1)	$\phi V_c = \phi_s \times \min(8 \times \lambda \times (\rho_w)^{1/3}, 5 \times \lambda) \times \sqrt{(f'_c \times 1\text{ psi})} \times b \times d =$ **28.689** kips
Design shear force at 20.2in from support	$V_u =$ **11.678** kips
Reinf. shear strength required (eqn. 22.5.1.1)	$\phi V_s = \max(V_u - \phi V_c, 0\text{ kips}) =$ **0.000** kips
Area of design shear reinf. req'd (eqn. 22.5.8.5.3)	$A_{sv,des} = \phi V_s / (\phi_s \times \min(f_y, 60000\text{ psi}) \times d) =$ **0.000** in²/ft
Area of shear reinforcement required	$A_{sv,req} = 0\text{mm}^2\text{/ft} =$ **0.000** in²/ft
Shear reinforcement provided	2 legs × No.4 @ 12" c/c
Area of shear reinforcement provided	$A_{sv,prov} =$ **0.393** in²/ft
	PASS - No shear reinforcement required ($\phi V_{s,lim}$ >= V_u)
Zone 2	
Effective depth of long. reinf. used for shear zone	d = **20.187** in
Size effect factor (cl. 22.5.5.1.3)	$\lambda_s = \min(\sqrt{(2 / (1 + (d / 1\text{ in}) / 10))}, 1.0) =$ **0.81**
Area of longitudinal reinforcement	$A_{st} =$ **1.841** in²
Ratio of longitudinal reinforcement	$\rho_w = A_{st} / (b \times d) =$ **0.004**
Concrete shear strength (Table 22.5.5.1)	$\phi V_c = \phi_s \times \min(8 \times \lambda \times (\rho_w)^{1/3}, 5 \times \lambda) \times \sqrt{(f'_c \times 1\text{ psi})} \times b \times d =$ **28.689** kips
Design shear force within zone	$V_u =$ **8.800** kips
Reinf. shear strength required (eqn. 22.5.1.1)	$\phi V_s = \max(V_u - \phi V_c, 0\text{ kips}) =$ **0.000** kips
Area of design shear reinf. req'd (eqn. 22.5.8.5.3)	$A_{sv,des} = \phi V_s / (\phi_s \times \min(f_y, 60000\text{ psi}) \times d) =$ **0.000** in²/ft
Area of shear reinforcement required	$A_{sv,req} = 0\text{mm}^2\text{/ft} =$ **0.000** in²/ft
Shear reinforcement provided	2 legs × No.4 @ 12" c/c
Area of shear reinforcement provided	$A_{sv,prov} =$ **0.393** in²/ft
	PASS - No shear reinforcement required ($\phi V_{s,lim}$ >= V_u)
Zone 3	
Effective depth of long. reinf. used for shear zone	d = **20.187** in
Size effect factor (cl. 22.5.5.1.3)	$\lambda_s = \min(\sqrt{(2 / (1 + (d / 1\text{ in}) / 10))}, 1.0) =$ **0.81**
Area of longitudinal reinforcement	$A_{st} =$ **1.841** in²
Ratio of longitudinal reinforcement	$\rho_w = A_{st} / (b \times d) =$ **0.004**
Concrete shear strength (Table 22.5.5.1)	$\phi V_c = \phi_s \times \min(8 \times \lambda \times (\rho_w)^{1/3}, 5 \times \lambda) \times \sqrt{(f'_c \times 1\text{ psi})} \times b \times d =$ **28.689** kips
Design shear force at 20.2in from support	$V_u =$ **11.678** kips
Reinf. shear strength required (eqn. 22.5.1.1)	$\phi V_s = \max(V_u - \phi V_c, 0\text{ kips}) =$ **0.000** kips
Area of design shear reinf. req'd (eqn. 22.5.8.5.3)	$A_{sv,des} = \phi V_s / (\phi_s \times \min(f_y, 60000\text{ psi}) \times d) =$ **0.000** in²/ft
Area of shear reinforcement required	$A_{sv,req} = 0\text{mm}^2\text{/ft} =$ **0.000** in²/ft

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

Project: QTS DEN1 DC2				Job Ref.: 24053.00000	
Section: GB6				Sheet no./rev.: 9	
Calc. by: LB	Date: 4/1/2025	Chk'd by:	Date:	App'd by:	Date:

Shear reinforcement provided	2 legs × No.4 @ 12" c/c
Area of shear reinforcement provided	$A_{sv,prov}$ = **0.393** in²/ft

PASS - No shear reinforcement required ($\phi V_{s,lim}$ >= V_u)

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

Project: QTS DEN1 DC2				Job Ref.: 24053.00000	
Section: GB7				Sheet no./rev.: 1	
Calc. by: LB	Date: 4/1/2025	Chk'd by:	Date:	App'd by:	Date:

RC BEAM ANALYSIS & DESIGN (ACI318-2019 (22))

In accordance with ACI318-2019 (22)

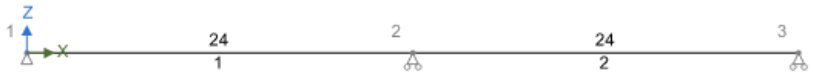
Tedds calculation version 3.3.21

ANALYSIS

Tedds calculation version 1.0.38

Geometry

Geometry (ft) - Concrete (ACI normal) - R 36x66

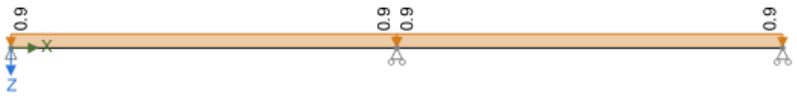


Span	Length (ft)	Section	Start Support	End Support
1	24	R 36x66	Pinned	Roller Pin X
2	24	R 36x66	Roller Pin X	Roller Pin X
R 36x66: Area 2376 in^2, Inertia Major 862488 in^4, Inertia Minor 256608 in^4, Shear area parallel to Minor 1980 in^2, Shear area parallel to Major 1980 in^2				
Concrete (ACI normal): Density 150 lbm/ft^3, Youngs 4200 ksi, Shear 1750 ksi, Thermal 0.00001 $°C^{-1}$				

Loading

Self weight included

Dead - Loading (kips/ft)



Load combination factors

Load combination	Self Weight	Dead	Live
1.2D + 1.6L (Strength)	1.20	1.20	1.60
1.0D + 1.0L (Service)	1.00	1.00	1.00
1.4D (Strength)	1.40	1.40	
1.0D (Service)	1.00	1.00	

Member Loads

Member	Load case	Load Type	Orientation	Description
Beam	Dead	UDL	GlobalZ	0.9 kips/ft

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

Project: QTS DEN1 DC2				Job Ref.: 24053.00000	
Section: GB7				Sheet no./rev.: 2	
Calc. by: LB	Date: 4/1/2025	Chk'd by:	Date:	App'd by:	Date:

Results

Reactions

Load case: Self Weight

Node	Force Fx (kips)	Force Fz (kips)	Moment My (kip_ft)
1	0	22.546	0
2	0	73.709	0
3	0	22.546	0

Load case: Dead

Node	Force Fx (kips)	Force Fz (kips)	Moment My (kip_ft)
1	0	8.198	0
2	0	26.803	0
3	0	8.198	0

Load case: Live

Node	Force Fx (kips)	Force Fz (kips)	Moment My (kip_ft)
1	0	0	0
2	0	0	0
3	0	0	0

Load combination: 1.2D + 1.6L (Strength)

Node	Force Fx (kips)	Force Fz (kips)	Moment My (kip_ft)
1	0	36.893	0
2	0	120.615	0
3	0	36.893	0

Load combination: 1.0D + 1.0L (Service)

Node	Force Fx (kips)	Force Fz (kips)	Moment My (kip_ft)
1	0	30.744	0
2	0	100.512	0
3	0	30.744	0

Load combination: 1.4D (Strength)

Node	Force Fx (kips)	Force Fz (kips)	Moment My (kip_ft)
1	0	43.041	0
2	0	140.717	0

PROJECT QTS DEN1-DC2
DESCRIPTION
PROJECT # 24053.00000 DATE 05/30/2025
PREPARED BY LB CHECKED BY ND

Project				Job Ref.	
QTS DEN1 DC2				24053.00000	
Section				Sheet no./rev.	
GB7				3	
Calc. by	Date	Chk'd by	Date	App'd by	Date
LB	4/1/2025				

Node	Force		Moment
	Fx (kips)	Fz (kips)	My (kip_ft)
3	0	43.041	0

Load combination: 1.0D (Service)

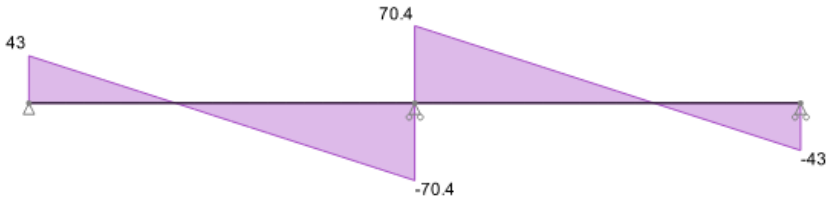
Node	Force		Moment
	Fx (kips)	Fz (kips)	My (kip_ft)
1	0	30.744	0
2	0	100.512	0
3	0	30.744	0

Forces

All combinations - Moment envelope (kip_ft)



All combinations - Shear envelope (kips)



Member results

Envelope - All combinations

Member	Position (ft)	Shear force (kips)		Moment (kip_ft)	
Beam	24	70.359 (max abs)	-70.359	-234.146	-327.805 (min)
	38.89	0		196.039 (max)	140.028

Concrete details

Compressive strength of concrete	f'_c = **4000** psi
Density of reinforced concrete	w_c = **150** lb / ft^3
Concrete type	Normal weight
Modulus of elasticity of concrete (cl.19.2.2.1)	E = (w_c / 1 lb/ft^3)$^{1.5}$ × 33 psi × (f'_c / 1 psi)$^{0.5}$ = **3834254** psi
Strength reduction factor for shear	ϕ_s = **0.75**

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

Project: QTS DEN1 DC2				Job Ref.: 24053.00000	
Section: GB7				Sheet no./rev.: 4	
Calc. by: LB	Date: 4/1/2025	Chk'd by	Date	App'd by	Date

Reinforcement details

Yield strength of reinforcement	f_y = **60000** psi
Compression-controlled strain limit (cl.21.2.2.1)	ε_{ty} = **0.00200**

Nominal cover to reinforcement

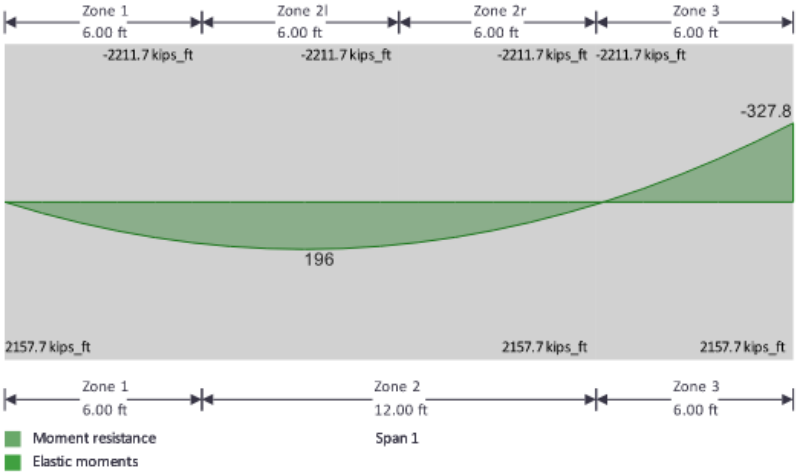
Cover to top reinforcement	c_{nom_t} = **1.5** in
Cover to bottom reinforcement	c_{nom_b} = **3** in
Cover to side reinforcement	c_{nom_s} = **3** in

Beam - Span 1

Rectangular section details

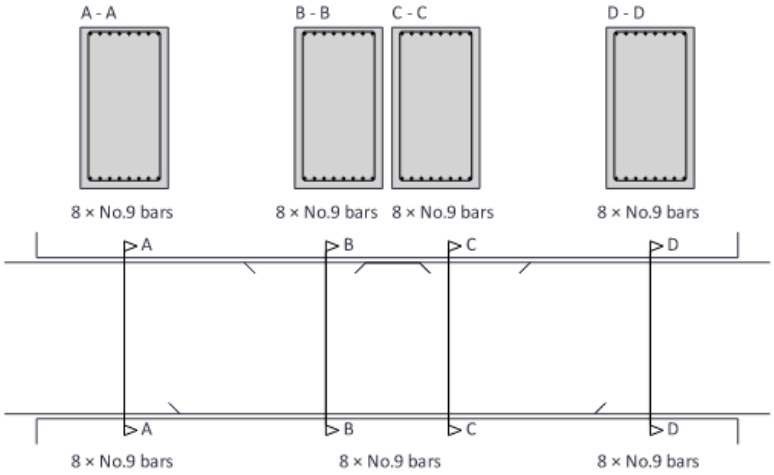
Section width	b = **36** in
Section depth	h = **66** in

Moment design



PROJECT QTS DEN1-DC2
DESCRIPTION
PROJECT # 24053.00000 DATE 05/30/2025
PREPARED BY LB CHECKED BY ND

Project				Job Ref.	
QTS DEN1 DC2				24053.00000	
Section				Sheet no./rev.	
GB7				5	
Calc. by	Date	Chk'd by	Date	App'd by	Date
LB	4/1/2025				



Zone 1 - Positive moment. Rectangular section in flexure (Section 9.5.2)

Factored bending moment at section	$M_u = abs(M_{m1\_s1\_z1\_max\_rod})$ = **173.199** kip_ft
Effective depth to tension reinforcement	d = **61.936** in
Tension reinforcement provided	8 × No.9 bars
Area of tension reinforcement provided	$A_{s,prov}$ = **7.995** in²
Minimum area of reinforcement (cl.9.6.1.2)	$A_{s,min} = max(3\ psi \times \sqrt{(f'_c / 1\ psi)}, 200\ psi) \times b \times d / f_y$ = **7.432** in²
	PASS - Area of reinforcement provided is greater than minimum area of reinforcement required
Stress block depth factor (cl.22.2.2.4.3)	$\beta_1 = min(max(0.85 - 0.05 \times (f'_c - 4\ ksi) / 1\ ksi, 0.65), 0.85)$ = **0.85**
Depth of equivalent rectangular stress block	$a = A_{s,prov} \times f_y / (0.85 \times f'_c \times b)$ = **3.919** in
Depth to neutral axis	$c = a / \beta_1$ = **4.611** in
Net tensile strain in extreme tension fibers	$\varepsilon_t = 0.003 \times (d_o - c) / max(c, 0.001\ in)$ = **0.0373**
	Net tensile strain in tension controlled zone
Strength reduction factor (cl.21.2.1)	$\phi_f = min(max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.003), 0.65), 0.9)$ = **0.90**
Nominal moment strength	$M_n = A_{s,prov} \times f_y \times (d - a / 2)$ = **2397.449** kip_ft
Design moment strength	$\phi M_n = M_n \times \phi_f$ = **2157.704** kip_ft
	PASS - Required moment strength is less than design moment strength

Flexural cracking

Max. center to center spacing of tension reinf.	$s_{b,max} = s_{bot} + \phi_{m1\_s1\_z1\_b\_L1}$ = **3.982** in
Service load stress in reinforcement (cl.24.3.2)	$f_s = 2/3 \times f_y$ = **40000** psi
Clear cover of reinforcement	$c_c = c_{nom\_b} + \phi_v$ = **3.500** in
Maximum allowable bot bar spacing (Table 24.3.2)	$s_{max} = min(15in \times 40000psi / f_s - 2.5 \times c_c, 12in \times 40000psi / f_s)$ = **6.250** in
	PASS - Maximum allowable tension reinforcement spacing exceeds actual spacing

Spacing limits for reinforcement

Top bar clear spacing	$s_{top} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z1\_v}) + \phi_{m1\_s1\_z1\_t\_L1} \times N_{m1\_s1\_z1\_t\_L1} + \phi_{m1\_s1\_z2l\_t\_L1} \times N_{m1\_s1\_z2l\_t\_L1})) / ((N_{m1\_s1\_z1\_t\_L1} + N_{m1\_s1\_z2l\_t\_L1}) - 1)$ = **2.854** in

PROJECT QTS DEN1-DC2
DESCRIPTION
PROJECT # 24053.00000 DATE 05/30/2025
PREPARED BY LB CHECKED BY ND

Project				Job Ref.	
QTS DEN1 DC2				24053.00000	
Section				Sheet no./rev.	
GB7				6	
Calc. by	Date	Chk'd by	Date	App'd by	Date
LB	4/1/2025				

Min. allowable top bar clear spacing (cl.25.2.1) $s_{top,min}$ = **1.000** in

Bottom bar clear spacing $s_{bot} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z1\_v}) + \phi_{m1\_s1\_z1\_b\_L1} \times N_{m1\_s1\_z1\_b\_L1})) / (N_{m1\_s1\_z1\_b\_L1} - 1)$ = **2.854** in

Min. allowable bottom bar clear spacing (cl.25.2.1) $s_{bot,min}$ = **1.000** in

PASS - Actual bar spacing exceeds minimum allowable

Zone 2 - Positive moment. Rectangular section in flexure (Section 9.5.2)

Factored bending moment at section $M_u = abs(M_{m1\_s1\_z2\_max\_red})$ = **196.039** kip_ft

Effective depth to tension reinforcement d = **61.936** in

Tension reinforcement provided 8 × No.9 bars

Area of tension reinforcement provided $A_{s,prov}$ = **7.995** in²

Minimum area of reinforcement (cl.9.6.1.2) $A_{s,min} = \max(3\ psi \times \sqrt{f'_c} / 1\ psi, 200\ psi) \times b \times d / f_y$ = **7.432** in²

PASS - Area of reinforcement provided is greater than minimum area of reinforcement required

Stress block depth factor (cl.22.2.2.4.3) $\beta_1 = \min(\max(0.85 - 0.05 \times (f'_c - 4\ ksi) / 1\ ksi, 0.65), 0.85)$ = **0.85**

Depth of equivalent rectangular stress block $a = A_{s,prov} \times f_y / (0.85 \times f'_c \times b)$ = **3.919** in

Depth to neutral axis $c = a / \beta_1$ = **4.611** in

Net tensile strain in extreme tension fibers $\varepsilon_t = 0.003 \times (d_o - c) / \max(c, 0.001\ in)$ = **0.0373**

Net tensile strain in tension controlled zone

Strength reduction factor (cl.21.2.1) $\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.003), 0.65), 0.9)$ = **0.90**

Nominal moment strength $M_n = A_{s,prov} \times f_y \times (d - a / 2)$ = **2397.449** kip_ft

Design moment strength $\phi M_n = M_n \times \phi_f$ = **2157.704** kip_ft

PASS - Required moment strength is less than design moment strength

Flexural cracking

Max. center to center spacing of tension reinf. $s_{b,max} = s_{bot} + \phi_{m1\_s1\_z2\_b\_L1}$ = **3.982** in

Service load stress in reinforcement (cl.24.3.2) $f_s = 2/3 \times f_y$ = **40000** psi

Clear cover of reinforcement $c_c = c_{nom\_b} + \phi_v$ = **3.500** in

Maximum allowable bot bar spacing (Table 24.3.2) $s_{max} = \min(15in \times 40000psi / f_s - 2.5 \times c_c, 12in \times 40000psi / f_s)$ = **6.250** in

PASS - Maximum allowable tension reinforcement spacing exceeds actual spacing

Control of deflections (Section 24.2)

Concrete density factor K_w = **1.00**

Reinforcement yield strength factor $K_f = 0.4 + f_y / 100000\ psi$ = **1.00**

Minimum thickness of beam (Table 9.3.1.1) $h_{min} = (L_{m1\_s1} / 18.5) \times K_w \times K_f$ = **15.568** in

PASS - Thickness of beam exceeds minimum thickness

Spacing limits for reinforcement

Top bar clear spacing $s_{top} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z2\_v}) + \phi_{m1\_s1\_z2r\_t\_L1} \times N_{m1\_s1\_z2r\_t\_L1})) / (N_{m1\_s1\_z2r\_t\_L1} - 1)$ = **2.854** in

Min. allowable top bar clear spacing (cl.25.2.1) $s_{top,min}$ = **1.000** in

Bottom bar clear spacing $s_{bot} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z2\_v}) + \phi_{m1\_s1\_z2\_b\_L1} \times N_{m1\_s1\_z2\_b\_L1} + \phi_{m1\_s1\_z1\_b\_L1} \times N_{m1\_s1\_z1\_b\_L1})) / ((N_{m1\_s1\_z2\_b\_L1} + N_{m1\_s1\_z1\_b\_L1}) - 1)$ = **2.854** in

Min. allowable bottom bar clear spacing (cl.25.2.1) $s_{bot,min}$ = **1.000** in

PASS - Actual bar spacing exceeds minimum allowable

Zone 3 - Positive moment. Rectangular section in flexure (Section 9.5.2)

Factored bending moment at section $M_u = abs(M_{m1\_s1\_z3\_max\_red})$ = **9.296** kip_ft

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

Project: QTS DEN1 DC2				Job Ref.: 24053.00000	
Section: GB7				Sheet no./rev.: 7	
Calc. by: LB	Date: 4/1/2025	Chk'd by:	Date:	App'd by:	Date:

Effective depth to tension reinforcement	d = **61.936** in
Tension reinforcement provided	8 × No.9 bars
Area of tension reinforcement provided	$A_{s,prov}$ = **7.995** in²
Minimum area of reinforcement (cl.9.6.1.2)	$A_{s,min}$ = max(3 psi × √(f'c / 1 psi), 200 psi) × b × d / f_y = **7.432** in²
	PASS - Area of reinforcement provided is greater than minimum area of reinforcement required
Stress block depth factor (cl.22.2.2.4.3)	β_1 = min(max(0.85 - 0.05 × (f'c - 4 ksi) / 1 ksi, 0.65), 0.85) = **0.85**
Depth of equivalent rectangular stress block	a = $A_{s,prov}$ × f_y / (0.85 × f'c × b) = **3.919** in
Depth to neutral axis	c = a / β_1 = **4.611** in
Net tensile strain in extreme tension fibers	ε_t = 0.003 × (d_o - c) / max(c, 0.001 in) = **0.0373**
	Net tensile strain in tension controlled zone
Strength reduction factor (cl.21.2.1)	ϕ_f = min(max(0.65 + 0.25 × (ε_t - ε_{ty}) / (0.003), 0.65), 0.9) = **0.90**
Nominal moment strength	M_n = $A_{s,prov}$ × f_y × (d - a / 2) = **2397.449** kip_ft
Design moment strength	ϕM_n = M_n × ϕ_f = **2157.704** kip_ft
	PASS - Required moment strength is less than design moment strength
Flexural cracking	
Max. center to center spacing of tension reinf.	$s_{b,max}$ = s_{bot} + $\phi_{m1\_s1\_z3\_b\_L1}$ = **3.982** in
Service load stress in reinforcement (cl.24.3.2)	f_s = 2/3 × f_y = **40000** psi
Clear cover of reinforcement	c_c = c_{nom_b} + ϕ_v = **3.500** in
Maximum allowable bot bar spacing (Table 24.3.2)	s_{max} = min(15in × 40000psi / f_s - 2.5 × c_c, 12in × 40000psi / f_s) = **6.250** in
	PASS - Maximum allowable tension reinforcement spacing exceeds actual spacing
Zone 3 - Negative moment. Rectangular section in flexure (Section 9.5.2)	
Factored bending moment at section	M_u = abs($M_{m1\_s1\_z3\_min\_red}$) = **327.805** kip_ft
Effective depth to tension reinforcement	d = **63.436** in
Tension reinforcement provided	8 × No.9 bars
Area of tension reinforcement provided	$A_{s,prov}$ = **7.995** in²
Minimum area of reinforcement (cl.9.6.1.2)	$A_{s,min}$ = max(3 psi × √(f'c / 1 psi), 200 psi) × b × d / f_y = **7.612** in²
	PASS - Area of reinforcement provided is greater than minimum area of reinforcement required
Stress block depth factor (cl.22.2.2.4.3)	β_1 = min(max(0.85 - 0.05 × (f'c - 4 ksi) / 1 ksi, 0.65), 0.85) = **0.85**
Depth of equivalent rectangular stress block	a = $A_{s,prov}$ × f_y / (0.85 × f'c × b) = **3.919** in
Depth to neutral axis	c = a / β_1 = **4.611** in
Net tensile strain in extreme tension fibers	ε_t = 0.003 × (d_o - c) / max(c, 0.001 in) = **0.03828**
	Net tensile strain in tension controlled zone
Strength reduction factor (cl.21.2.1)	ϕ_f = min(max(0.65 + 0.25 × (ε_t - ε_{ty}) / (0.003), 0.65), 0.9) = **0.90**
Nominal moment strength	M_n = $A_{s,prov}$ × f_y × (d - a / 2) = **2457.409** kip_ft
Design moment strength	ϕM_n = M_n × ϕ_f = **2211.668** kip_ft
	PASS - Required moment strength is less than design moment strength
Flexural cracking	
Max. center to center spacing of tension reinf.	$s_{t,max}$ = s_{top} + $\phi_{m1\_s1\_z3\_t\_L1}$ = **3.982** in
Service load stress in reinforcement (cl.24.3.2)	f_s = 2/3 × f_y = **40000** psi
Clear cover of reinforcement	c_c = c_{nom_t} + ϕ_v = **2.000** in
Maximum allowable top bar spacing (Table 24.3.2)	s_{max} = min(15in × 40000psi / f_s - 2.5 × c_c, 12in × 40000psi / f_s) = **10.000** in
	PASS - Maximum allowable tension reinforcement spacing exceeds actual spacing

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

Project: QTS DEN1 DC2				Job Ref.: 24053.00000	
Section: GB7				Sheet no./rev.: 8	
Calc. by: LB	Date: 4/1/2025	Chk'd by:	Date:	App'd by:	Date:

Spacing limits for reinforcement

Top bar clear spacing — $s_{top} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z3\_v}) + \phi_{m1\_s1\_z3\_t\_L1} \times N_{m1\_s1\_z3\_t\_L1} + \phi_{m1\_s1\_z2r\_t\_L1} \times N_{m1\_s1\_z2r\_t\_L1})) / ((N_{m1\_s1\_z3\_t\_L1} + N_{m1\_s1\_z2r\_t\_L1}) - 1) = \mathbf{2.854}$ in

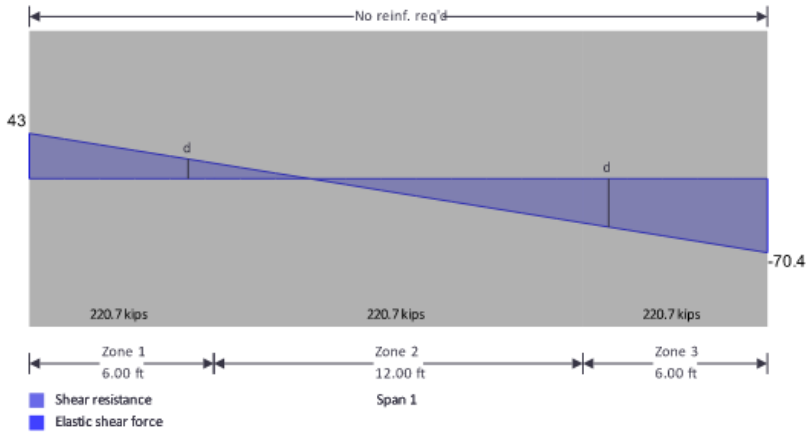
Min. allowable top bar clear spacing (cl.25.2.1) — $s_{top,min} = \mathbf{1.000}$ in

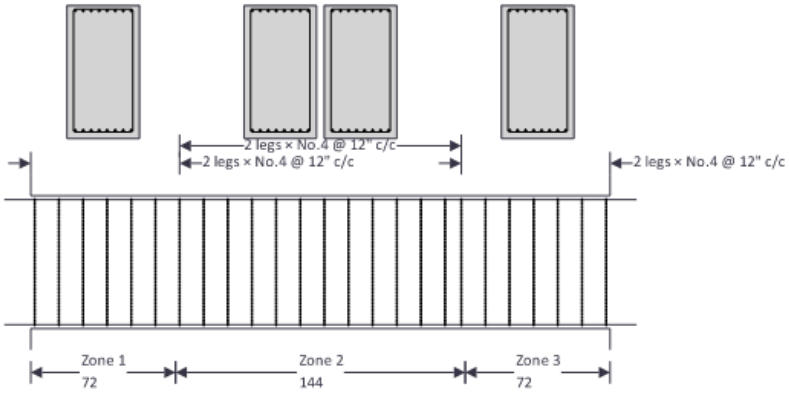
Bottom bar clear spacing — $s_{bot} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z3\_v}) + \phi_{m1\_s1\_z3\_b\_L1} \times N_{m1\_s1\_z3\_b\_L1})) / (N_{m1\_s1\_z3\_b\_L1} - 1) = \mathbf{2.854}$ in

Min. allowable bottom bar clear spacing (cl.25.2.1) — $s_{bot,min} = \mathbf{1.000}$ in

PASS - Actual bar spacing exceeds minimum allowable

Shear design





Rectangular section in shear

Concrete weight modification factor — $\lambda = \mathbf{1.00}$

Location where min. reinf. is req'd (V_u less ϕV_c) — N/A no reinf. required along full length

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

Project: QTS DEN1 DC2				Job Ref.: 24053.00000	
Section: GB7				Sheet no./rev.: 9	
Calc. by: LB	Date: 4/1/2025	Chk'd by:	Date:	App'd by:	Date:

Location where no reinf. is req'd (V_u less $\phi V_{s,lim}$)	Between 0.00 ft and 24.00 ft
Maximum reinforcement shear strength	$\phi V_{s,max} = \phi_s \times 8$ psi $\times \sqrt{(\min(f'_c, 10000\text{psi}) / 1\text{ psi})} \times b \times d$ = **846.110** kips
Minimum area of shear reinf. (Table 9.6.3.4)	$A_{sv,min} = \max(50\text{ psi}, 0.75\text{ psi} \times \sqrt{(f'_c / 1\text{ psi})}) \times b / \min(f_y, 60000\text{ psi})$ = **0.360** in²/ft
No reinforcement shear strength limit	$\phi V_{s,lim} = \phi_s \times \lambda \times 1\text{psi} \times \sqrt{(\min(f'_c, 10000\text{psi}) / 1\text{ psi})} \times b \times d$ = **105.764** kips
Zone 1	
Effective depth of long. reinf. used for shear zone	d = **61.936** in
Size effect factor (cl. 22.5.5.1.3)	$\lambda_s = \min(\sqrt{(2 / (1 + (d / 1\text{ in}) / 10))}, 1.0)$ = **0.53**
Area of longitudinal reinforcement	A_{st} = **7.995** in²
Ratio of longitudinal reinforcement	$\rho_w = A_{st} / (b \times d)$ = **0.004**
Concrete shear strength (Table 22.5.5.1)	$\phi V_c = \phi_s \times \min(8 \times \lambda \times (\rho_w)^{1/3}, 5 \times \lambda) \times \sqrt{(f'_c \times 1\text{ psi})} \times b \times d$ = **129.502** kips
Design shear force at 61.9in from support	V_u = **18.654** kips
Reinf. shear strength required (eqn. 22.5.1.1)	$\phi V_s = \max(V_u - \phi V_c, 0\text{ kips})$ = **0.000** kips
Area of design shear reinf. req'd (eqn. 22.5.8.5.3)	$A_{sv,des} = \phi V_s / (\phi_s \times \min(f_y, 60000\text{ psi}) \times d)$ = **0.000** in²/ft
Area of shear reinforcement required	$A_{sv,req}$ = 0mm²/ft = **0.000** in²/ft
Shear reinforcement provided	2 legs × No.4 @ 12" c/c
Area of shear reinforcement provided	$A_{sv,prov}$ = **0.393** in²/ft
	PASS - No shear reinforcement required ($\phi V_{s,lim} >= V_u$)
Zone 2	
Effective depth of long. reinf. used for shear zone	d = **61.936** in
Size effect factor (cl. 22.5.5.1.3)	$\lambda_s = \min(\sqrt{(2 / (1 + (d / 1\text{ in}) / 10))}, 1.0)$ = **0.53**
Area of longitudinal reinforcement	A_{st} = **7.995** in²
Ratio of longitudinal reinforcement	$\rho_w = A_{st} / (b \times d)$ = **0.004**
Concrete shear strength (Table 22.5.5.1)	$\phi V_c = \phi_s \times \min(8 \times \lambda \times (\rho_w)^{1/3}, 5 \times \lambda) \times \sqrt{(f'_c \times 1\text{ psi})} \times b \times d$ = **129.502** kips
Design shear force within zone	V_u = **42.009** kips
Reinf. shear strength required (eqn. 22.5.1.1)	$\phi V_s = \max(V_u - \phi V_c, 0\text{ kips})$ = **0.000** kips
Area of design shear reinf. req'd (eqn. 22.5.8.5.3)	$A_{sv,des} = \phi V_s / (\phi_s \times \min(f_y, 60000\text{ psi}) \times d)$ = **0.000** in²/ft
Area of shear reinforcement required	$A_{sv,req}$ = 0mm²/ft = **0.000** in²/ft
Shear reinforcement provided	2 legs × No.4 @ 12" c/c
Area of shear reinforcement provided	$A_{sv,prov}$ = **0.393** in²/ft
	PASS - No shear reinforcement required ($\phi V_{s,lim} >= V_u$)
Zone 3	
Effective depth of long. reinf. used for shear zone	d = **61.936** in
Size effect factor (cl. 22.5.5.1.3)	$\lambda_s = \min(\sqrt{(2 / (1 + (d / 1\text{ in}) / 10))}, 1.0)$ = **0.53**
Area of longitudinal reinforcement	A_{st} = **7.995** in²
Ratio of longitudinal reinforcement	$\rho_w = A_{st} / (b \times d)$ = **0.004**
Concrete shear strength (Table 22.5.5.1)	$\phi V_c = \phi_s \times \min(8 \times \lambda \times (\rho_w)^{1/3}, 5 \times \lambda) \times \sqrt{(f'_c \times 1\text{ psi})} \times b \times d$ = **129.502** kips
Design shear force at 61.9in from support	V_u = **45.971** kips
Reinf. shear strength required (eqn. 22.5.1.1)	$\phi V_s = \max(V_u - \phi V_c, 0\text{ kips})$ = **0.000** kips
Area of design shear reinf. req'd (eqn. 22.5.8.5.3)	$A_{sv,des} = \phi V_s / (\phi_s \times \min(f_y, 60000\text{ psi}) \times d)$ = **0.000** in²/ft
Area of shear reinforcement required	$A_{sv,req}$ = 0mm²/ft = **0.000** in²/ft
Shear reinforcement provided	2 legs × No.4 @ 12" c/c
Area of shear reinforcement provided	$A_{sv,prov}$ = **0.393** in²/ft

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

Project: QTS DEN1 DC2				Job Ref.: 24053.00000	
Section: GB7				Sheet no./rev.: 10	
Calc. by: LB	Date: 4/1/2025	Chk'd by:	Date:	App'd by:	Date:

PASS - No shear reinforcement required ($\phi V_{s,lim}$ >= V_u)

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

Generator Pad Slab Design

RC ONE-WAY SLAB DESIGN (ACI 318)

In accordance with ACI318-19 (22)

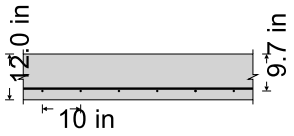
Tedds calculation version 1.2.05

Design summary

Overall design status; PASS

Overall design utilization; 0.329

Description	Unit	Capacity	Maximum	Utilization	Result
Moment	kip_ft/ft	15.651	5.156	0.329	PASS
Shear	kips/ft	6.871	1.528	0.222	PASS
Minimum thickness requirements are met					PASS



Library item - Show drawing one way slab

Slab definition

Slab type; **One-way simply supported**

Overall thickness of slab; h = **12.000** in

Span of slab; l_n = **13.50** ft

Clear cover to tension reinforcement; c_c = **2.00** in

Materials

Specified compressive strength of concrete; f'_c = **4500** psi

Specified yield strength of reinforcement; f_y = **60000** psi

Modulus of elasticity; E_{SACI} = **29000000** psi

Compression-controlled strain limit - 21.2.2.1; ε_{ty} = **0.002**

Minimum tensile strain - 7.3.3.1; $\varepsilon_{min} = \varepsilon_{ty} + 0.003$ = **0.005**

Library item:Slab definition(SAG)

Concrete modification factor; λ = **1.00**

Library item: Concrete modification factor

Maximum design moment and shear in span(per 12 in width of slab)

Maximum ultimate positive moment; M_{us} = **5.156** kip_ft/ft

Maximum ultimate shear force; V_u = **1.528** kips/ft

Library item:Max moment and shear(SAG)

Reinforcement calculation - positive moments

Tension steel provided; **No. 5 @ 12 in o.c.**

Depth to tension steel; $d = (h - c_c - D / 2)$ = **9.69** in

$\beta_1 = if(f'_c <= 4\ ksi, 0.85, max(0.65, 0.85 - 0.05 \times (f'_c - 4\ ksi) / 1\ ksi))$ = **0.83**

Stress block depth factor; $\beta_1 = 0.85 - 0.05 \times (f'_c - 4\ ksi) / 1\ ksi = 0.83$

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

Reinforcement ratio at strain of ε_{min};	$\rho_b = 0.85 \times \beta_1 \times f'_c / f_y \times (0.003 / (0.003 + \varepsilon_{min})) =$ **0.020**
Maximum reinforcement ratio;	$\rho_{max} = \rho_b =$ **0.020**
Maximum area of tension steel;	$A_{s\_max} = \rho_{max} \times d =$ **2.293** in^2/ft
Min ratio of transverse reinforcement – cl. 7.6.1.1;	$\rho_t =$**0.0018**
Min area tension steel req'd – cl. 7.6.1.1;	$A_{s\_min} = \rho_t \times h =$ **0.259** in^2/ft
Area required for bending;	$A_{s\_req} =$ **0.119** in^2/ft
Area of tension steel provided;	$A_{s\_prov} =$ **0.368** in^2/ft
	PASS - Area of steel provided - OK
Steel stress – cl. 24.3.2.1;	$f_s = 2/3 \times f_y =$ **40000** psi
Max allowable spacing – cl. 7.7.2.3 & cl. 24.3.2; 12in×(40000 psi/f_s))	$s_{max} = \min(3\times h, 18in, 15in\times(40000\ psi/f_s) – 2.5\times c_c,$
	$s_{max} =$ **12.000** in
Actual tensile bar spacing provided;	s = **12.000** in
	PASS - Spacing of bars (+ve moment steel) less than maximum allowable
Check for section - positive moments	
Depth of equivalent rectangular stress block;	$a = (A_{s\_prov} \times f_y) / (0.85 \times f'_c) =$ **0.48** in
Depth of neutral axis;	$c = a / \beta_1 =$ **0.583** in
Net tensile strain in long. steel at nominal strength;	$\varepsilon_t = 0.003 \times [(d – c) / c] =$ **0.0468**
	Tensile strain exceeds minimum required, design OK

Library item: Section check - SAGGING

Required nominal flexural strength;	$M_{us} =$ **5.156** kip_ft/ft
Strength reduction factor;	$\phi =$**0.9**
Nominal flexural strength;	$M_{ns\_prov} = A_{s\_prov} \times f_y \times (d – a / 2) =$ **17.390** kip_ft/ft
Design flexural strength;	$\phi M_{ns\_prov} = \phi \times M_{ns\_prov} =$ **15.651** kip_ft/ft
	PASS - Design flexural strength exceeds required flexural strength

Library item: Moment capacity - SAGGING

Transverse reinforcement - (for shrinkage and temperature)	
Transverse reinforcement provided;	**No. 5 @ 12 in o.c.**
Area of reinforcement provided;	$A_{t\_prov} =$ **0.307** in^2/ft
Min ratio of transverse reinforcement – cl. 7.6.1.1;	$\rho_t =$**0.0018**
Minimum area of transverse reinforcement required;	$A_{t\_req} = \rho_t \times h =$ **0.259** in^2/ft
	PASS - Area of transverse steel provided OK
Maximum allowable spacing of bars;	$s_{max\_t} = \min (5 \times h, 18 in) =$ **18.000** in
Actual transverse bar spacing provided;	$s_t =$ **12.000** in
	PASS - Spacing of transverse bars is less than allowable
Check for shear	
Required shear strength;	$V_u =$ **1.528** kips/ft
Size effect factor – cl. 22.5.5.1.3;	$\lambda_s = \min(\sqrt{(2 / (1 + (d / 1\ in) / 10))},1.0) =$ **1.00**
Ratio of longitudinal reinforcement;	$\rho_w = A_{s\_prov} / d =$ **0.003**
Shear strength provided by concrete;	$V_c = \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3}, 5 \times \lambda) \times \sqrt{(f'_c \times 1\ psi)} \times d =$ **9.162** kips/ft;
Shear strength provided by shear steel (assumed);	$V_s = 0$ kips/ft
Shear capacity of section;	$V = V_c + V_s =$ **9.162** kips/ft
Design shear capacity of section;	$\phi V = 0.75 \times V =$ **6.871** kips/ft
	PASS - One-way shear capacity

Library item: Shear check(SAG 2019)

Check of clear cover – cl. 20.5.1.1	
Permissible min nominal cover to all reinforcement;	$c_{min} = 0.75$ in

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

Clear cover to tension reinforcement (+ve mnt); $c_c = h - d - D/2 =$ **2.000** in

PASS - Cover to steel resisting positive moment exceeds allowable minimum cover

Library item: Cover check (SAGGING)

Deflection

Support condition;	**Simply supported**
Basic span-to-thickness ratio - Table 7.3.1.1;	$ratio_{basic} =$ **20**
Type of concrete;	**Normal weight**
;	;;
Concrete density factor - Table 7.3.1.1;	$f_{density} =$;**1.00**
Allowable span-to-thickness ratio;	$ratio_{allow} = ratio_{basic} / (f_{density} \times (0.4 + f_y / 100000\ psi)) =$ **20.000**
Actual span-to-thickness ratio;	$ratio_{actual} = l_n / h =$ **13.500**

PASS - The slab thickness is adequate to control deflection

Library item – Deflection check

Design summary

Slab is 12.0 in thick in 4500 psi concrete

Tension steel provided - positive moment, No. 5 @ 12 in o.c. in 60000 psi steel

Library item:Design summary-SAGGING

Transverse steel provided , No. 5 @ 12 in o.c. in 60000 psi steel

Library item:Design summary-Transverse steel

Generator Grade Beam Design

FOOTING ANALYSIS

In accordance with ACI318-

Tedds calculation version 3.3.11

Summary results

Overall design status;	PASS;
Overall design utilisation;	0.810

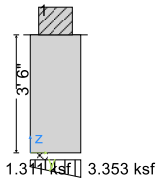
Description	Unit	Applied	Resisting	FoS	Result
Uplift verification	kips	3.5			Pass
Overturning stability, y	kip_ft	0.38	-2.62	6.85	Pass
Sliding stability, y	kips	0.1	2.0	18.459	Pass
Description	**Unit**	**Applied**	**Resisting**	**Utilization**	**Result**
Soil bearing	ksf	3.353	4.14	0.810	Pass

Strip footing details - considering a one foot strip

Length of footing;	$L_x =$ **1** ft
Width of footing;	$L_y =$ **1.5** ft
Footing area;	$A = L_x \times L_y =$ **1.5** ft^2
Depth of footing;	h = **42** in
Depth of soil over footing;	$h_{soil} =$ **0** in
Density of concrete;	$\gamma_{conc} =$ **150.0** lb/ft^3

Library item: Foundation details output

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
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Library item: Show foundation sketch

Wall no.1 details

Width of wall; l_{y1} = **12** in
position in y-axis; y_1 = **9** in

Library item: Wall details output

Soil properties

Net allowable bearing pressure; q_{allow_Net} = **4** ksf; using a soil factor of safety, FS_{soil}, of 3
Density of soil; γ_{soil} = **120.0** lb/ft^3
Angle of internal friction; ϕ_b = **30.0** deg
Design base friction angle; δ_{bb} = **30.0** deg
Coefficient of base friction; $\tan(\delta_{bb})$ = **0.577**

Footing loads

Self weight; $F_{swt} = h \times \gamma_{conc}$ = **525** psf

Wall no.1 loads per linear foot

Dead load in z; F_{Dz1} = **2.7** kips
Wind load in y; F_{Wy1} = **0.2** kips
Seismic load in y; F_{Ey1} = **0.1** kips

Footing analysis for soil and stability

Load combinations per ASCE 7-16

1.0D (0.563)
1.0D + 0.6W (0.810)
(1.0 + 0.14 × S_{DS})D + 0.7E (0.771)

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

0.6D + 0.6W (0.585)
(0.6 - 0.14 × S_{DS})D + 0.7E (0.514)

Combination 9 results: 1.0D + 0.6W

Forces on footing per linear foot

Force in y-axis; $F_{dy} = \gamma_W \times F_{Wy1}$ = **0.1** kips

Force in z-axis; $F_{dz} = \gamma_D \times A \times F_{swt} + \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})$ = **3.5** kips

Moments on footing per linear foot

Moment in y-axis, about y is 0; $M_{dy} = \gamma_D \times A \times F_{swt} \times L_y / 2 + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_W \times (F_{Wy1} \times (h))$ = **3.0** kip_ft

Uplift verification

Vertical force; F_{dz} = **3.498** kips

PASS - Footing is not subject to uplift

Stability against overturning in y direction, moment about y is L_y

Overturning moment; $M_{OTyL} = \gamma_W \times (F_{Wy1} \times (h))$ = **0.38** kip_ft

Resisting moment; $M_{RyL} = -1 \times (\gamma_D \times A \times F_{swt} \times L_y / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times (y_1 - L_y))$ = **-2.62** kip_ft

Factor of safety; $abs(M_{RyL} / M_{OTyL})$ = **6.851**

PASS - Overturning moment safety factor exceeds the minimum of 1.50

Stability against sliding

Resistance due to base friction; $F_{RFriction} = max(F_{dz}, 0\ kN) \times tan(\delta_{bb})$ = **2.019** kips

Stability against sliding in y direction

Total sliding resistance; $F_{Ry} = F_{RFriction}$ = **2.019** kips

Factor of safety; $abs(F_{RTy} / F_{dy})$ = **18.46**

PASS - Sliding factor of safety exceeds the minimum of 1.50

Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in y-axis; $e_{dy} = M_{dy} / F_{dz} - L_y / 2$ = **1.314** in

Library item: Strip eccentricity output

Strip base pressures

$q_1 = F_{dz} \times (1 - 6 \times e_{dy} / L_y) / (L_y \times 1\ ft)$ = **1.311** ksf

$q_2 = F_{dz} \times (1 + 6 \times e_{dy} / L_y) / (L_y \times 1\ ft)$ = **3.353** ksf

Minimum base pressure; $q_{min} = min(q_1,q_2)$ = **1.311** ksf

Maximum base pressure; $q_{max} = max(q_1,q_2)$ = **3.353** ksf

Library item: Strip pressure output

Allowable bearing capacity

Allowable bearing capacity; $q_{allow} = q_{allow\_Net} + ((h + h_{soil}) \times \gamma_{soil}) / FS_{soil}$ = **4.14** ksf

q_{max} / q_{allow} = **0.810**

PASS - Allowable bearing capacity exceeds design base pressure

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

Transformer Foundation Calculations

PROJECT DEN1-DC2		PROJECT NO. 24053
DESCRIPTION Transformer Pad Design.		BY
		DATE

Transformer Pad Design:-

$f'_c = 4000$ psi

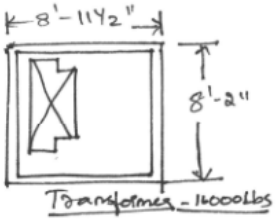
Allowable bearing pressure = 3000 psf.

Modulus of subgrade reaction = 120 pci

Transformer weight = 16,000 lbs

soil overburden = $4' \times 120$ pci = 0.48 ksf.

Transformer:- L = 7.5' (T.O.F)
H = 8'



Loads:-

$$DL = \frac{16000}{(8.96 \times 8.1)} = 0.22 \text{ ksf}$$

WL → Moment = 0.6×50 psf $\times 7.5' \times 8' \times (0.55 \times 8')$

= 7920

≃ 8000 lb-ft.

→ Moment Arm D = 0.55H

RISA Foundation used for Analysis & Design of Pad.

Results from RISA Foundation:-

Bearing pressure = 1090 psf < 3000 psf [Design Acceptable]

R/f provided = #5 @ 8" o.c EW T&B

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND

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Designer : GMurthy
Job Number :
Model Name :

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Design Rules - Mat Slab

	Label	Max Bending Chk	Max Shear Chk	Top Bar	Bottom Bar	Min Top Bar Spacing [in]	Max Top Bar Spacing [in]	Min Bot Bar Spacing [in]	Max Bot Bar Spacing [in]	Spacing Increment [in]	Top Cover [in]	Bottom Cover [in]	Side Cover [in]	Rebar Options
1	Typical	1	1	#5	#5	8	8	8	8	2	1.5	3	0	Optimize

Soil Definitions

	Label	Layers	Subgrade Modulus [k/ft³]	Allowable Bearing [ksf]	Default
1	Default	Single	207.36	2.5	Yes

Slab

	Label	Thickness [in]	Material	Local Axis Angle [deg]	Analysis Offset [in]	Passive Pressure [ksf]	Soil Overburden [ksf]	Icr Factor
1	S1	36	Conc4000NW	0	0	0	0.48	0.25

Design Strips

	Label	Rebar Angle from Plan Horizontal (deg)	No. of Design Cuts	Design Rule
1	DS1	90	50	Typical
2	DS2	0	50	Typical

Load Category

	Category	Node Loads	Distributed Loads	Area Loads
1	DL		1	1
2	WL	1		

Load Combination

	Label	Solve	Service	Category	Factor	Category	Factor
1	1DL	Yes	Yes	DL	1		
2	1DL+0.6WL	Yes	Yes	DL	1	WL	0.6
3	0.6DL+0.6WL	Yes	Yes	DL	0.6	WL	0.6
4	1.4DL	Yes		DL	1.4		
5	1.2DL+1.0WL	Yes		DL	1.2	WL	1
6	0.9DL+1.0WL	Yes		DL	0.9	WL	1

Slab Soil Pressures

	LC	Label	UC	Soil Pressure[ksf]	Allowable Bearing[ksf]	Node
1	1	S1	0.418	1.046	2.5	N1
2	2	S1	0.436	1.09	2.5	N4
3	3	S1	0.269	0.673	2.5	N4

Envelope Slab Soil Pressures

	Label	Max UC	Max LC	Soil Pressure[ksf]	Allowable Bearing[ksf]	Node
1	S1	0.436	2	1.09	2.5	N4

Strip Reinforcing

	Label	UC Top	LC Top	Bars	Gov Design Cut	UC Top	UC Bot	LC Bot	Bars/Mid Bars	Gov Design Cut	UC Bot	UC Shear	LC Shear	Gov Design Cut
1	DS1	0.009	6	#5@8in	DS1-X18	0.006	5	#5@8in	DS1-X27	0.024	6	DS1-X14		
2	DS2	0.001	6	#5@8in	DS2-X25	0	6	#5@8in	DS2-X46	0.002	5	DS2-X13		

PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
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Company : <Licensed Company>
Designer : GMurthy
Job Number :
Model Name :

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Slab Stability - Overturning

	LC	Slab	Angle[deg]	Mo-xx[k-ft]	Ms-xx[k-ft]	Mo-zz[k-ft]	Ms-zz[k-ft]	Ms-xx/Mo-xx	Ms-zz/Mo-zz
1	1	S1	0	0	311.79	0	329.49	9.99+	9.99+
2	2	S1	0	0	311.79	4.8	329.49	9.99+	9.99+
3	3	S1	0	0	187.074	4.8	197.694	9.99+	9.99+

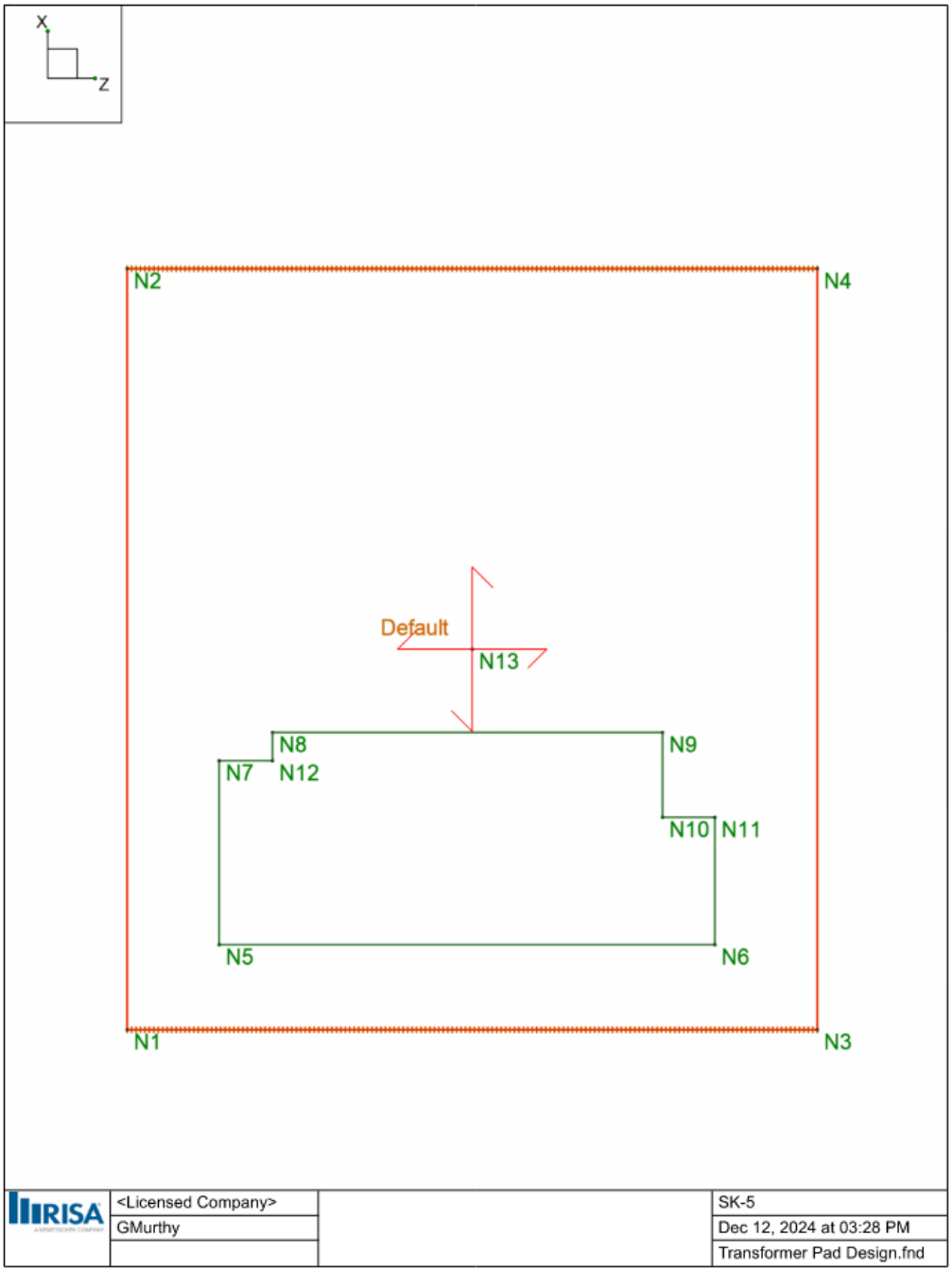
Slab Stability - Sliding

	LC	Slab	Angle[deg]	Va-xx[k]	Vr-xx[k]	Va-zz[k]	Vr-zz[k]	SR-xx	SR-zz
1	1	S1	0	0	22.978	0	22.978	9.99+	9.99+
2	2	S1	0	0	22.978	0	22.978	9.99+	9.99+
3	3	S1	0	0	13.787	0	13.787	9.99+	9.99+

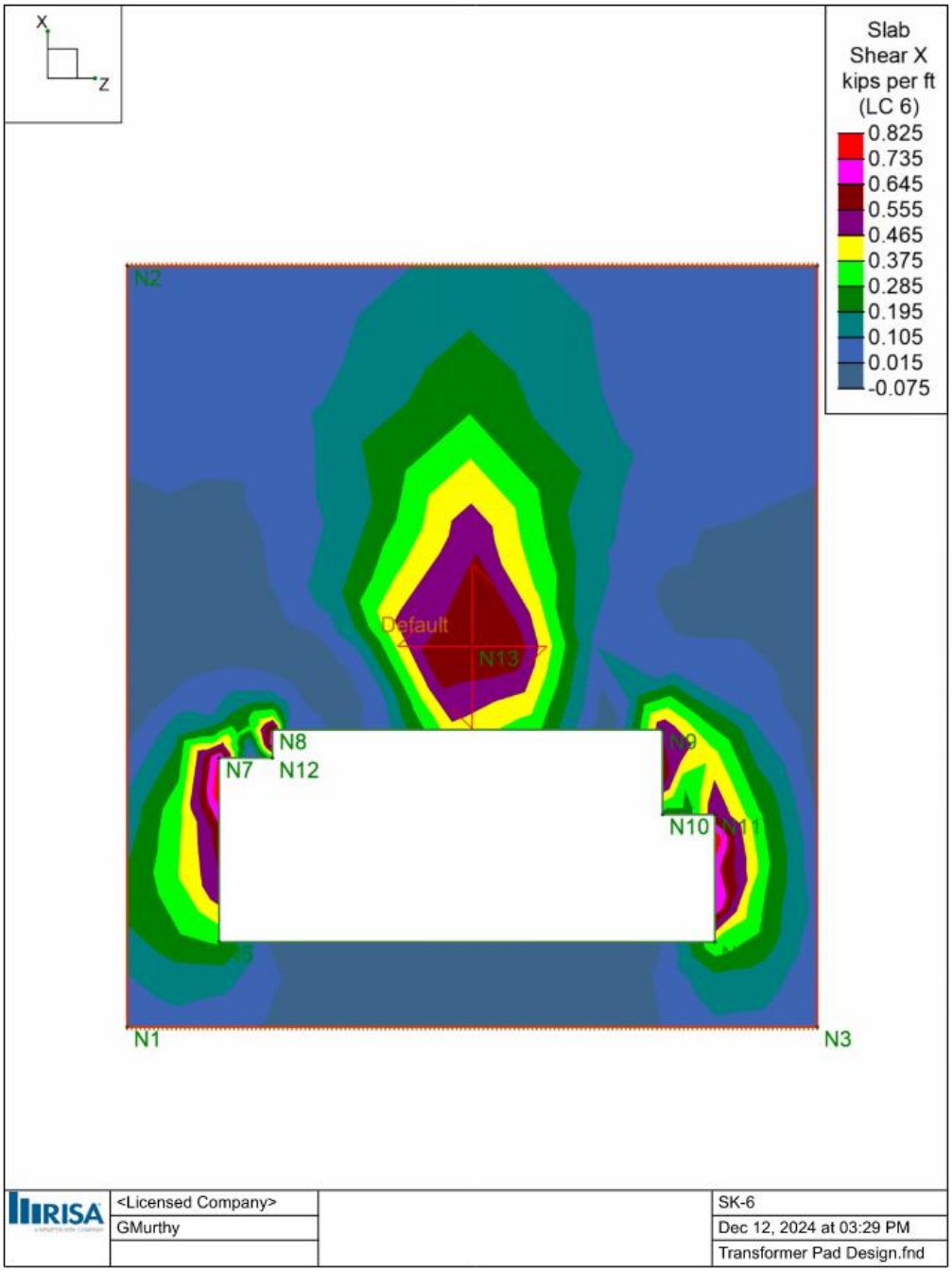
Material Take-Off

	Material	Quantity	Volume[yds^3]	Weight[K]
1	Slabs			
2	Conc4000NW	1	6.6	25.7
3				
4	Totals			
5	Conc4000NW		6.6	25.7
6	Total Concrete		6.6	25.7

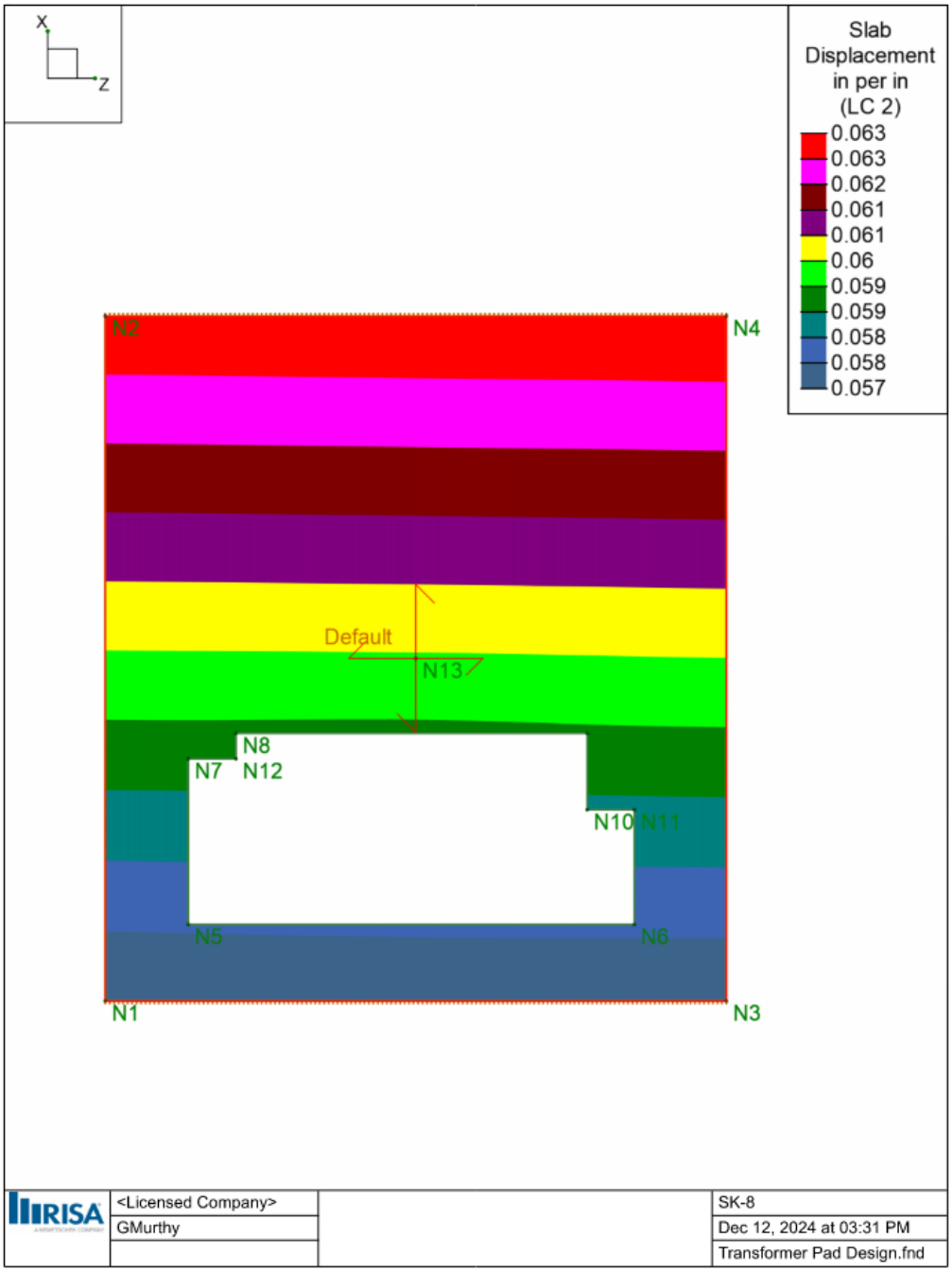
PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
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PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
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DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
PREPARED BY	LB	CHECKED BY	ND



PROJECT	QTS DEN1-DC2		
DESCRIPTION			
PROJECT #	24053.00000	DATE	05/30/2025
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6.0 STEEL COLUMN BASE PLATES & ANCHORAGE

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Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/23/2025
Fastening point:			

Specifier's comments:

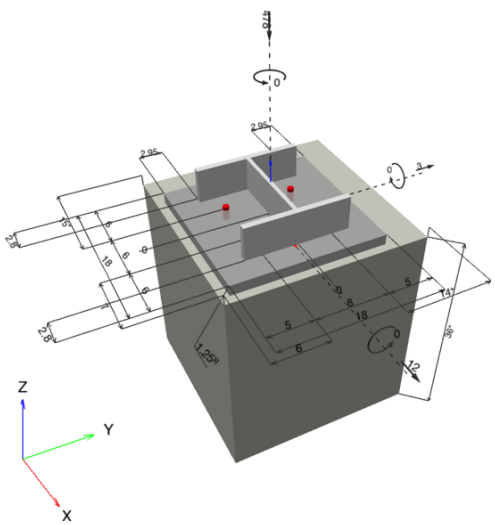
1 Input data



Anchor type and diameter:	**Heavy Hex Head ASTM F 1554 GR. 55 3/4**
Item number:	not available
Specification text:	∅ 3/4 in Heavy Hex Head ASTM F 1554 GR. 55 with 18 in nominal embedment depth per Technical data , cast in place installation per MPII,
Effective embedment depth:	h_{ef} = 18.000 in.
Material:	ASTM F 1554
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-19 / CIP
Shear edge breakout verification:	Row closest to edge (Case 3 only from ACI 318-19 Fig. R.17.7.2.1b)
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 1.250 in.
Anchor plate[R] :	l_x x l_y x t = 18.000 in. x 18.000 in. x 1.250 in.;
Profile:	W shape (AISC), W12X79; (L x W x T x FT) = 12.400 in. x 12.100 in. x 0.470 in. x 0.735 in.
Base material:	cracked concrete, Custom, f_c' = 4,500 psi; h = 36.000 in.
Reinforcement:	tension: not present, shear: not present; edge reinforcement: none or < No. 4 bar

[R] - The anchor calculation is based on a rigid anchor plate assumption.

Geometry [in.] & Loading [kip, ft.kip]



Input data and results must be checked for conformity with the existing conditions and for plausibility!
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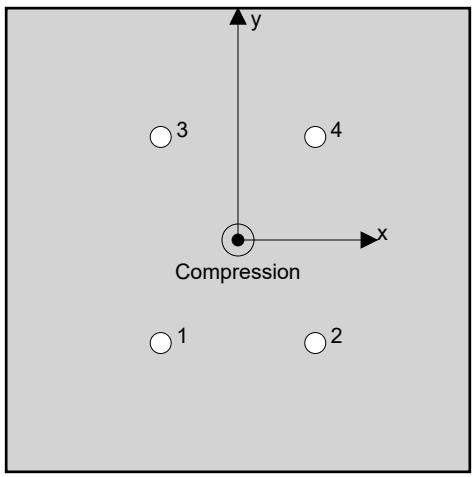
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Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/23/2025
Fastening point:			

1.1 Design results

Case	Description	Forces [kip] / Moments [ft.kip]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = -478.000; V_x = 12.000; V_y = 3.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	156

2 Load case/Resulting anchor forces



Anchor reactions [kip]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	0.000	3.092	3.000	0.750
2	0.000	3.092	3.000	0.750
3	0.000	3.092	3.000	0.750
4	0.000	3.092	3.000	0.750

Max. concrete compressive strain: 0.00 [‰]
Max. concrete compressive stress: 1,475 [psi]
Resulting tension force in (x/y)=(-/-): 0.000 [kip]
Resulting compression force in (x/y)=(0.000/0.000): 478.000 [kip]

Anchor forces are calculated based on the assumption of a rigid anchor plate.

3 Tension load

	Load N_{ua} [kip]	Capacity ϕ N_n [kip]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	N/A	N/A	N/A	N/A
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	N/A	N/A	N/A	N/A
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (anchors in tension)

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/23/2025
Fastening point:			

4 Shear load

	Load V_{ua} [kip]	Capacity ϕ V_n [kip]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	3.092	9.769	32	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	12.369	50.913	25	OK
Concrete edge failure in direction x+**	12.369	7.957	156	not recommended

* highest loaded anchor **anchor group (relevant anchors)

SEE SUPPLEMENTAL CALCULATION FOR PEDESTAL REINFORCEMENT

4.1 Steel Strength

$V_{sa} = 0.6\ A_{se,V}\ f_{uta}$ ACI 318-19 Eq. (17.7.1.2b)
$\phi\ V_{steel} \geq V_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.33	75,000

Calculations

V_{sa} [kip]
15.030

Results

V_{sa} [kip]	ϕ_{steel}	ϕ V_{sa} [kip]	V_{ua} [kip]
15.030	0.650	9.769	3.092

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	4
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/23/2025
Fastening point:			

4.2 Pryout Strength

$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \right]$ ACI 318-19 Eq. (17.7.3.1b)

$\phi V_{cpg} \geq V_{ua}$ ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9 h_{ef}^2$ ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e'_N}{3 h_{ef}}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ ACI 318-19 Eq. (17.6.2.2.1)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	10.000	0.000	0.000	6.000

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.000	-	24	1.000	4,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [kip]
784.00	900.00	1.000	1.000	0.820	1.000	50.912

Results

V_{cpg} [kip]	$\phi_{concrete}$	ϕV_{cpg} [kip]	V_{ua} [kip]
72.734	0.700	50.913	12.369

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	5
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/23/2025
Fastening point:			

4.3 Concrete edge failure in direction x+

$V_{cbg} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \Psi_{ec,V} \Psi_{ed,V} \Psi_{c,V} \Psi_{h,V} \Psi_{parallel,V} V_b$ — ACI 318-19 Eq. (17.7.2.1b)

$\phi V_{cbg} \geq V_{ua}$ — ACI 318-19 Table 17.5.2

A_{Vc} see ACI 318-19, Section 17.7.2.1, Fig. R 17.7.2.1(b)*

$A_{Vc0} = 4.5 c_{a1}^2$ — ACI 318-19 Eq. (17.7.2.1.3)

$\Psi_{ec,V} = \left(\frac{1}{1 + \frac{e'_v}{1.5c_{a1}}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.3.1)

$\Psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.4.1b)

$\Psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ — ACI 318-19 Eq. (17.7.2.6.1)

$V_b = 9 \lambda_a \sqrt{f'_c} c_{a1}^{1.5}$ — ACI 318-19 Eq. (17.7.2.2.1b)

Variables

c_{a1} [in.]	c_{a2} [in.]	e_{cV} [in.]	$\Psi_{c,V}$	h_a [in.]
7.000	6.000	0.000	1.000	36.000

l_e [in.]	λ_a	d_a [in.]	f'_c [psi]	$\Psi_{parallel,V}$
6.000	1.000	0.750	4,500	1.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{h,V}$	V_b [kip]
257.25	220.50	1.000	0.871	1.000	11.181

Results

V_{cbg} [kip]	$\phi_{concrete}$	ϕV_{cbg} [kip]	V_{ua} [kip]
11.368	0.700	7.957	12.369

*Anchor row defined by: Anchor 2, 4; Case 3 controls

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	6
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/23/2025
Fastening point:			

5 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- User is responsible for evaluating the hole bearing capacity in case of shear forces.
- The equations presented in this report are based on imperial units. When inputs are displayed in metric units, the user should be aware that the equations remain in their imperial format.
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- For additional information about ACI 318 strength design provisions, please go to https://viewer.joomag.com/profis-design-guide-us-en-summer-2021/0841849001625154758?short&/

Fastening does not meet the design criteria!

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Company:		Page:	7
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/23/2025
Fastening point:			

6 Anchor plate and concrete bearing stress check

	Load	Capacity	Utilization [%]	Status
Concentric Compression	12,765.55 [in.lb/in.]	17,578.13 [in.lb/in.]	73	OK
Concrete bearing	1,475 [psi]	2,762 [psi]	54	OK
Tension Interface	N/A	N/A	N/A	N/A
Uniaxial Moment (Strong Axis)	N/A	N/A	N/A	N/A
Uniaxial Moment (Weak Axis)	N/A	N/A	N/A	N/A

6.1 Concrete bearing check (per AISC DG1, section 3.1.1)

$A_1 = B \cdot N$

$A_2 = L \cdot W$

$f_{pu(max)} = \phi \left(0.85 f'_c \sqrt{\frac{A_2}{A_1}}\right), \sqrt{\frac{A_2}{A_1}} \leq 2$

$f_{pu} = \frac{P_u}{BN}$

$f_{pu} \leq f_{p(max)}$

Variables

B [in.]	N [in.]	L [in.]	W [in.]	f_c' [psi]	ϕ	P_u [kip]
18.000	18.000	20.000	20.000	4,500	0.650	478.000

Calculations

A_1 [in.^{2}]	A_2 [in.^{2}]
324.00	400.00

Results

f_{pu} [psi]	$f_{pu(max)}$ [psi]
1,475	2,762

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	8
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/23/2025
Fastening point:			

6.2 Plate bending, compression (per AISC DG1, sections 3.1.2, 3.1.3)

$A_1 = B \cdot N$

$A_2 = L \cdot W$

$P_p = 0.85 f'_c A_1 \sqrt{\frac{A_2}{A_1}}, \sqrt{\frac{A_2}{A_1}} \leq 2$

$X = \left\{ \frac{4 d b_f}{(d + b_f)^2} \right\} \frac{P_u}{\phi_c P_p}$

$\lambda = \frac{2\sqrt{X}}{1 + \sqrt{1 - X}} \leq 1$

$m = \frac{N - 0.95d}{2}$

$n = \frac{B - 0.80 b_f}{2}$

$\lambda n' = \lambda \frac{\sqrt{d b_f}}{4}$

$l = \max(m, n, \lambda n')$

$f_{pu} = \frac{P_u}{BN}$

$M_{pl} = \frac{f_{pu} \cdot l^2}{2}$

$\phi M_n = \phi \cdot F_y \cdot \frac{t_p^2}{4}$

$M_{pl} \leq \phi M_n$

Variables

B [in.]	N [in.]	L [in.]	W [in.]	d [in.]
18.000	18.000	20.000	20.000	12.400

b_f [in.]	F_y [psi]	ϕ	t_p [in.]	P_u' [kip]
12.100	50,000	0.900	1.250	478.000

Calculations

A_1 [in.^{2}]	A_2 [in.^{2}]	P_p [kip]	X	λ
324.00	400.00	895.050	0.821	1.000

m [in.]	n [in.]	$\lambda n'$ [in.]	l [in.]	f_{pu} [psi]
3.110	4.160	3.062	4.160	1,475

Results

M_n [in.lb/in.]	ϕ	ϕM_n [in.lb/in.]	M_{pl} [in.lb/in.]
19,531.26	0.900	17,578.13	12,765.55

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	9
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/23/2025
Fastening point:			

7 Installation data

Profile: W shape (AISC), W12X79; (L x W x T x FT) = 12.400 in. x 12.100 in. x 0.470 in. x 0.735 in.

Hole diameter in the fixture: d_f = 0.812 in.

Plate thickness (input): 1.250 in.

Anchor type and diameter: Heavy Hex Head ASTM F 1554 GR. 55 3/4

Item number: not available

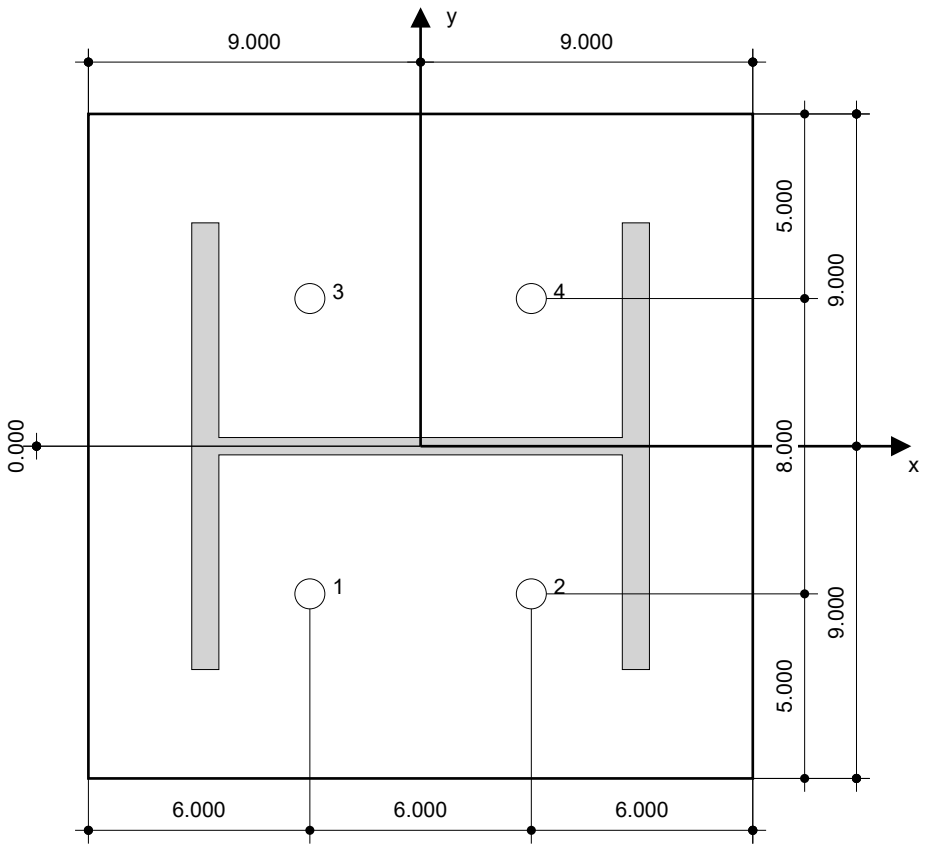
Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 18.000 in.

Minimum thickness of the base material: 19.000 in.

⌀ 3/4 in Heavy Hex Head ASTM F 1554 GR. 55 with 18 in nominal embedment depth per Technical data , cast in place installation per MPII



Coordinates Anchor in.

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-3.000	-4.000	15.000	13.000	6.000	22.000
2	3.000	-4.000	21.000	7.000	6.000	22.000
3	-3.000	4.000	15.000	13.000	14.000	14.000
4	3.000	4.000	21.000	7.000	14.000	14.000

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Company:		Page:	10
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/23/2025
Fastening point:			

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Hilti PROFIS Engineering 3.1.14

www.hilti.com

Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/23/2025
Fastening point:			

Specifier's comments:

1 Anchor Design

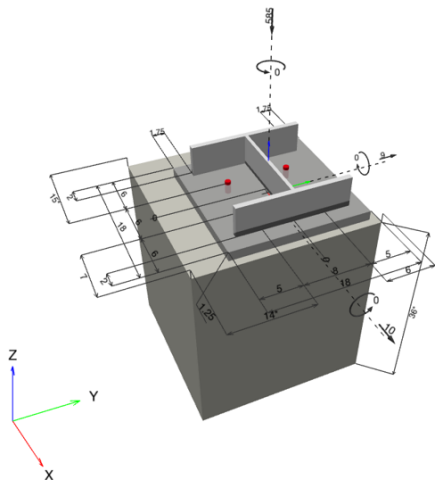
1.1 Input data



Anchor type and diameter:	**Heavy Hex Head ASTM F 1554 GR. 55 3/4**
Item number:	not available
Specification text:	∅ 3/4 in Heavy Hex Head ASTM F 1554 GR. 55 with 18 in nominal embedment depth per Technical data , cast in place installation per MPII,
Effective embedment depth:	h_{ef} = 18.000 in.
Material:	ASTM F 1554
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-19 / CIP
Shear edge breakout verification:	Row closest to edge (Case 3 only from ACI 318-19 Fig. R.17.7.2.1b)
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 1.250 in.
Anchor plate[CBFEM] :	l_x x l_y x t = 18.000 in. x 18.000 in. x 1.250 in.;
Profile:	W shape (AISC), W14X90; (L x W x T x FT) = 14.000 in. x 14.500 in. x 0.440 in. x 0.710 in.
Base material:	cracked concrete, Custom, f_c' = 4,500 psi; h = 36.000 in.
Reinforcement:	tension: not present, shear: not present; edge reinforcement: none or < No. 4 bar

[CBFEM] - The anchor calculation is based on a component-based Finite Element Method (CBFEM)

Geometry [in.] & Loading [kip, ft.kip]



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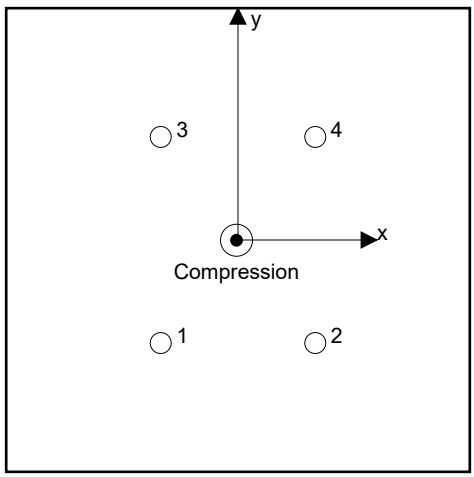
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Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/23/2025
Fastening point:			

1.1.1 Design results

Case	Description	Forces [kip] / Moments [ft.kip]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = -585.000; V_x = 10.000; V_y = 9.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	190

1.2 Load case/Resulting anchor forces



Anchor reactions [kip]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	-0.005	3.318	2.507	2.173
2	-0.005	3.307	2.491	2.175
3	-0.005	3.430	2.524	2.323
4	-0.005	3.401	2.479	2.328

Resulting tension force in (x/y)=(-/-): 0.000 [kip]
Resulting compression force in (x/y)=(-0.058/-0.010): 585.214 [kip]

Anchor forces are calculated based on a component-based Finite Element Method (CBFEM)

1.3 Tension load

	Load N_{ua} [kip]	Capacity ϕ N_n [kip]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	-0.005	18.787	1	OK
Pullout Strength*	-0.005	22.957	-0	not recommended
Concrete Breakout Failure**	N/A	N/A	N/A	N/A
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (anchors in tension)

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Company:		Page:	3
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/23/2025
Fastening point:			

1.3.1 Steel Strength

$N_{sa} = A_{se,N} f_{uta}$ ACI 318-19 Eq. (17.6.1.2)
$\phi N_{sa} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.33	75,000

Calculations

N_{sa} [kip]
25.050

Results

N_{sa} [kip]	ϕ_{steel}	ϕN_{sa} [kip]	N_{ua} [kip]
25.050	0.750	18.787	-0.005

The steel proof was done for the highest absolute force per anchor - in this case compression loading. Please be aware that buckling should be verified separately

1.3.2 Pullout Strength

$N_{pN} = \psi_{c,p} N_p$ ACI 318-19 Eq. (17.6.3.1)
$N_p = 8 A_{brg} f'_c$ ACI 318-19 Eq. (17.6.3.2.2a)
$\phi N_{pN} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$\psi_{c,p}$	A_{brg} [in.^{2}]	λ_a	f'_c [psi]
1.000	0.91	1.000	4,500

Calculations

N_p [kip]
32.796

Results

N_{pn} [kip]	$\phi_{concrete}$	ϕN_{pn} [kip]	N_{ua} [kip]
32.796	0.700	22.957	-0.005

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	4
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/23/2025
Fastening point:			

1.4 Shear load

	Load V_{ua} [kip]	Capacity ϕ V_n [kip]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	3.430	9.769	36	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	13.454	50.913	27	OK
Concrete edge failure in direction y+**	13.454	7.084	~~190~~	not recommended

* highest loaded anchor **anchor group (relevant anchors)

SEE SUPPLEMENTAL CALCULATION FOR PEDESTAL REINFORCEMENT

1.4.1 Steel Strength

$V_{sa} = 0.6\ A_{se,V}\ f_{uta}$ ACI 318-19 Eq. (17.7.1.2b)
$\phi\ V_{steel} \geq V_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.33	75,000

Calculations

V_{sa} [kip]
15.030

Results

V_{sa} [kip]	ϕ_{steel}	$\phi\ V_{sa}$ [kip]	V_{ua} [kip]
15.030	0.650	9.769	3.430

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	5
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/23/2025
Fastening point:			

1.4.2 Pryout Strength

$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \right]$ ACI 318-19 Eq. (17.7.3.1b)

$\phi V_{cpg} \geq V_{ua}$ ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9 h_{ef}^2$ ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e'_N}{3 h_{ef}}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ ACI 318-19 Eq. (17.6.2.2.1)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	10.000	0.000	0.000	6.000

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.000	-	24	1.000	4,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [kip]
784.00	900.00	1.000	1.000	0.820	1.000	50.912

Results

V_{cpg} [kip]	$\phi_{concrete}$	ϕV_{cpg} [kip]	V_{ua} [kip]
72.734	0.700	50.913	13.454

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	6
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/23/2025
Fastening point:			

1.4.3 Concrete edge failure in direction y+

$V_{cbg} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \Psi_{ec,V} \Psi_{ed,V} \Psi_{c,V} \Psi_{h,V} \Psi_{parallel,V} V_b$ — ACI 318-19 Eq. (17.7.2.1b)

$\phi V_{cbg} \geq V_{ua}$ — ACI 318-19 Table 17.5.2

A_{Vc} see ACI 318-19, Section 17.7.2.1, Fig. R 17.7.2.1(b)*

$A_{Vc0} = 4.5 c_{a1}^2$ — ACI 318-19 Eq. (17.7.2.1.3)

$\Psi_{ec,V} = \left(\frac{1}{1 + \frac{e'_v}{1.5c_{a1}}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.3.1)

$\Psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.4.1b)

$\Psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ — ACI 318-19 Eq. (17.7.2.6.1)

$V_b = 9 \lambda_a \sqrt{f'_c} c_{a1}^{1.5}$ — ACI 318-19 Eq. (17.7.2.2.1b)

Variables

c_{a1} [in.]	c_{a2} [in.]	e_{cV} [in.]	$\Psi_{c,V}$	h_a [in.]
6.000	7.000	0.002	1.000	36.000

l_e [in.]	λ_a	d_a [in.]	f'_c [psi]	$\Psi_{parallel,V}$
6.000	1.000	0.750	4,500	1.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{h,V}$	V_b [kip]
198.00	162.00	1.000	0.933	1.000	8.873

Results

V_{cbg} [kip]	$\phi_{concrete}$	ϕV_{cbg} [kip]	V_{ua} [kip]
10.120	0.700	7.084	13.454

*Anchor row defined by: Anchor 3, 4; Case 3 controls

1.5 Combined tension and shear loads, per ACI 318-19 section 17.8

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.000	1.899	1.000	159	not recommended

$\beta_{NV} = (\beta_N + \beta_V) / 1.2 <= 1$

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Company:		Page:	7
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/23/2025
Fastening point:			

1.6 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates as per current regulations (ETAG 001/Annex C, EOTA TR029, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid base plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- The equations presented in this report are based on imperial units. When inputs are displayed in metric units, the user should be aware that the equations remain in their imperial format.
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- For additional information about ACI 318 strength design provisions, please go to https://viewer.joomag.com/profis-design-guide-us-en-summer-2021/0841849001625154758?short&/
- Attention! In case of compressive anchor forces a buckling check as well as the proof of the local load transfer into and within the base material (incl. punching) has to be done separately.
- The anchor design methods in PROFIS Engineering require rigid anchor plates, as per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means that the anchor plate should be sufficiently rigid to prevent load re-distribution to the anchors due to elastic/plastic displacements. The user accepts that the anchor plate is considered close to rigid by engineering judgment."

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Company:		Page:	8
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/23/2025
Fastening point:			

1.7 Installation data

Profile: W shape (AISC), W14X90; (L x W x T x FT) = 14.000 in. x 14.500 in. x 0.440 in. x 0.710 in.

Hole diameter in the fixture: d_f = 0.812 in.

Plate thickness (input): 1.250 in.

Anchor type and diameter: Heavy Hex Head ASTM F 1554 GR. 55 3/4

Item number: not available

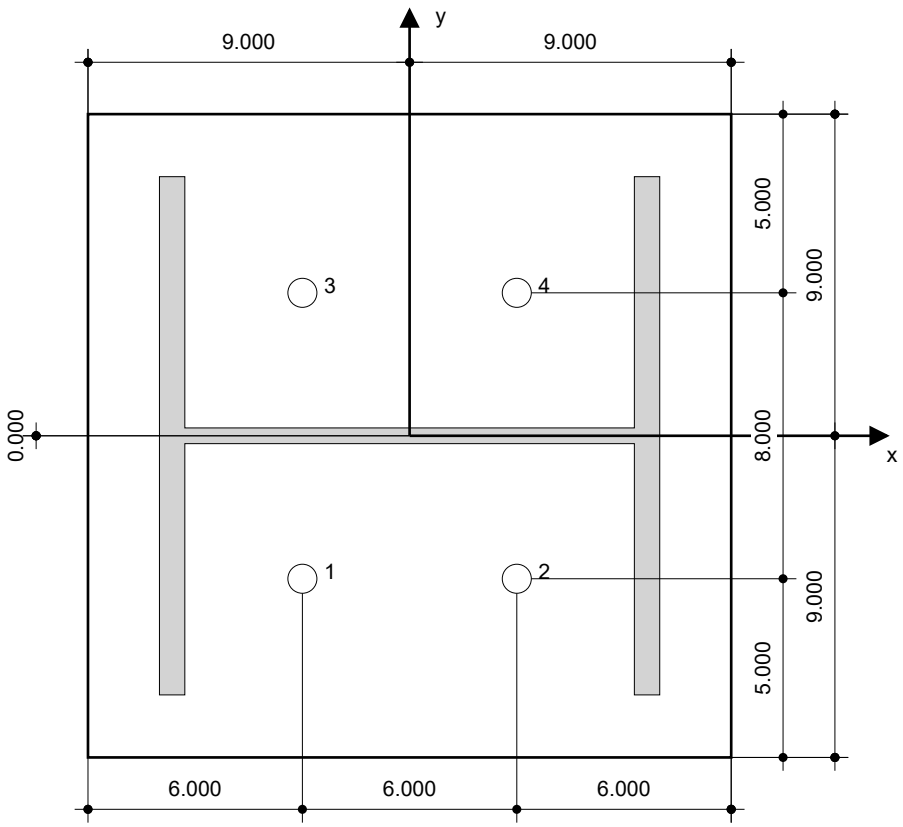
Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 18.000 in.

Minimum thickness of the base material: 19.000 in.

⌀ 3/4 in Heavy Hex Head ASTM F 1554 GR. 55 with 18 in nominal embedment depth per Technical data , cast in place installation per MPII



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-3.000	-4.000	15.000	13.000	14.000	14.000
2	3.000	-4.000	21.000	7.000	14.000	14.000
3	-3.000	4.000	15.000	13.000	22.000	6.000
4	3.000	4.000	21.000	7.000	22.000	6.000

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Company:		Page:	9
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/23/2025
Fastening point:			

2 Anchor plate design

2.1 Input data

Anchor plate:	Shape: Rectangular l_x x l_y x t = 18.000 in x 18.000 in x 1.250 in Calculation: CBFEM Material: ASTM A572 Gr.50; F_y = 50,000 psi; ε_{lim} = 5.00%
Anchor type and size:	Heavy Hex Head ASTM F 1554 GR. 55 3/4, h_{ef} = 18.000 in
Anchor stiffness:	The anchor is modeled considering stiffness values determined from load displacement curves tested in an independent laboratory. Please note that no simple replacement of the anchor is possible as the anchor stiffness has a major impact on the load distribution results.
Design method:	AISC and LRFD-based design using component-based FEM
Stand-off installation:	e_b = 0.000 in (No stand-off); t = 1.250 in
Profile:	W14X90; (L x W x T x FT) = 14.000 in x 14.500 in x 0.440 in x 0.710 in Material: ASTM A992; F_y = 50,000 psi; ε_{lim} = 5.00% Eccentricity x: 0.000 in Eccentricity y: 0.000 in
Base material:	Cracked concrete; Custom; $f_{c,cyl}$ = 4,500 psi; h = 36.000 in; E = 4,351,132 psi; G = 1,740,453 psi; v = 0.20; D = 156.07 lb/ft³
Welds (profile to anchor plate):	Type of redistribution: Plastic Material: E70xx
Mesh size:	Number of elements on edge: 8 Min. size of element: 0.394 in Max. size of element: 1.969 in

2.2 Summary

	Description	Profile		Anchor plate			Concrete [%]
		σ_{Ed} [psi]	ε_{Pl} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Hole bearing [%]	
1	Combination 1	39,083	0.00	26,116	0.00	4	71

2.3 Anchor plate classification

Results below are displayed for the decisive load combinations: Combination 1

Anchor tension forces	Equivalent rigid anchor plate (CBFEM)	Component-based Finite Element Method (CBFEM) anchor plate design
Anchor 1	-0.006 kip	-0.005 kip
Anchor 2	-0.006 kip	-0.005 kip
Anchor 3	-0.006 kip	-0.005 kip
Anchor 4	-0.006 kip	-0.005 kip

User accepted to consider the selected anchor plate as rigid by his/her engineering judgement. This means the anchor design guidelines can be applied.

2.4 Profile/Stiffeners/Plate

Profile and stiffeners are verified at the level of the steel to concrete connection. The connection design does not replace the steel design for critical cross sections, which should be performed outside of PROFIS Engineering.

2.4.1 Equivalent stress and plastic strain

Part	Load combination	Material	f_y [psi]	ε_{lim} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Status
Plate	Combination 1	ASTM A572	50,000	5.00	26,116	0.00	OK

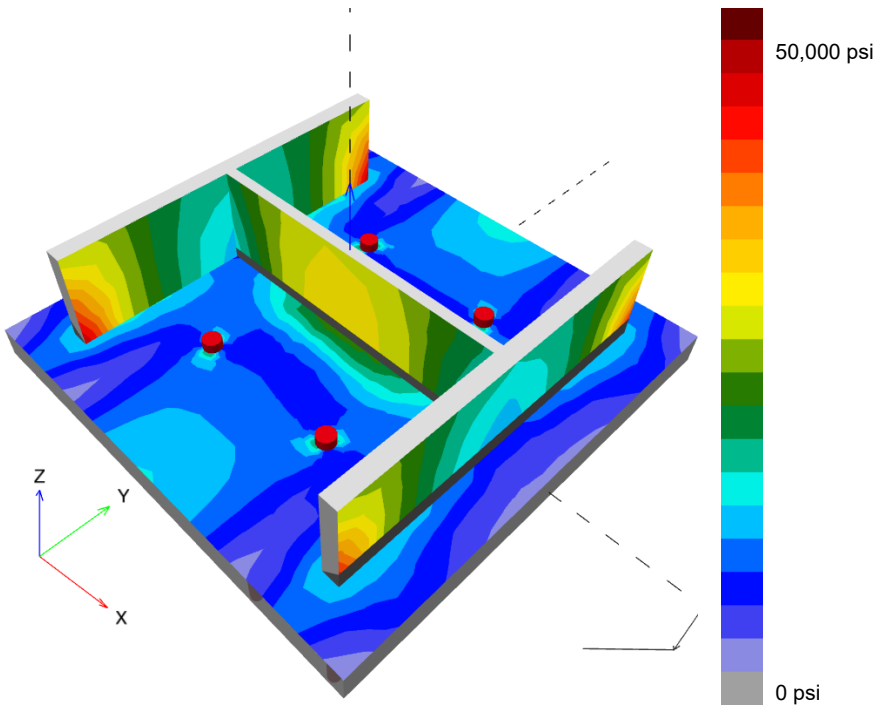
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Company:		Page:	10
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/23/2025
Fastening point:			

Part	Load combination	Material	f_y [psi]	ε_{lim} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Status
		Gr.50					
Profile	Combination 1	ASTM A992	50,000	5.00	37,786	0.00	OK
Profile	Combination 1	ASTM A992	50,000	5.00	39,083	0.00	OK
Profile	Combination 1	ASTM A992	50,000	5.00	31,717	0.00	OK

2.4.1.1 Equivalent stress

Results below are displayed for the decisive load combination: 1 - Combination 1



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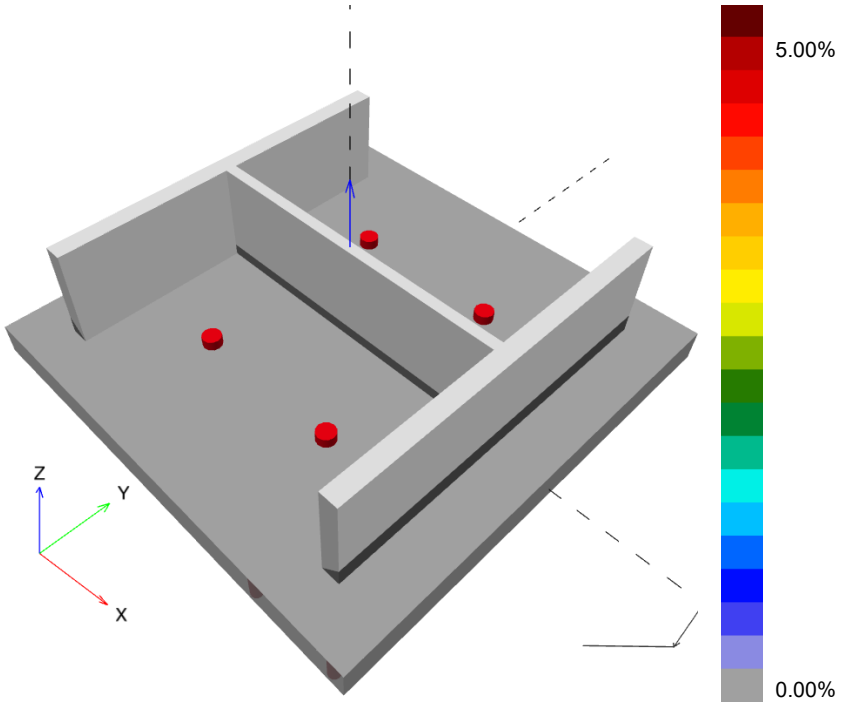
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Company:		Page:	11
Address:		Specifier:	
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2.4.1.2 Plastic strain

Results below are displayed for the decisive load combination: 1 - Combination 1



2.4.2 Plate hole bearing resistance, AISC 360-16 Section J3

Decisive load combination: 1 - Combination 1

Equations

$R_n = \min(1.2\, l_c\, t\, F_u\, ,\ 2.4\, d\, t\, F_u)$ (AISC 360-16 J3-6a, c)

$\Phi R_n = 0.75\, R_n$

$V \leq \Phi R_n$

Variables

	l_c [in]	t [in]	F_u [psi]	d [in]	R_n [kip]
Anchor 1	7.227	1.250	65,000	0.750	146.250
Anchor 2	7.195	1.250	65,000	0.750	146.250
Anchor 3	7.749	1.250	65,000	0.750	146.250
Anchor 4	16.057	1.250	65,000	0.750	146.250

Results

	V [kip]	ΦR_n [kip]	Utilization [%]	Status
Anchor 1	3.318	109.687	4	OK
Anchor 2	3.307	109.687	4	OK
Anchor 3	3.430	109.687	4	OK

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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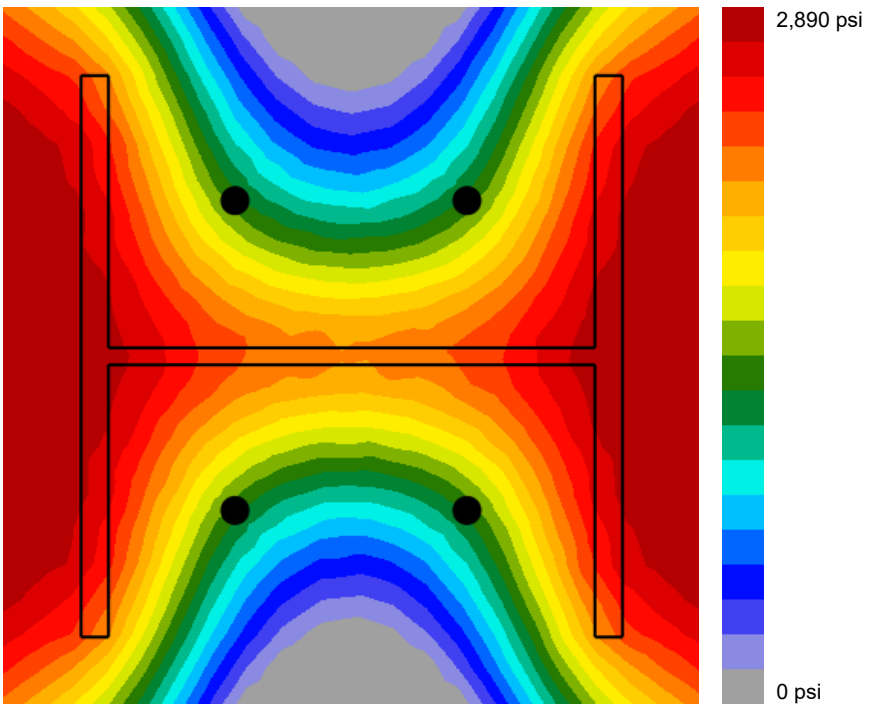
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Address:		Specifier:	
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Design:	Orange BP (~800 kips)	Date:	4/23/2025
Fastening point:			

	V [kip]	ΦR_n [kip]	Utilization [%]	Status
Anchor 4	3.401	109.687	4	OK

2.5 Concrete

Decisive load combination: 1 - Combination 1

2.5.1 Compression in concrete under the anchor plate



2.5.2 Concrete block compressive strength resistance check, AISC 360-16 Section J8

Equations

$$F_p = \Phi f_{p,max}$$

$$f_{p,max} = 0.85 f_c' \sqrt{\left(\frac{A_2}{A_1}\right)} \le 1.7 f_c; \quad \sqrt{\left(\frac{A_2}{A_1}\right)} \le 2$$

$$\sigma = \frac{N}{A_1}$$

$$\text{Utilization} = \frac{\sigma}{F_p}$$

Variables

N [kip]	f_c' [psi]	Φ	A_1 [in²]	A_2 [in²]
585.214	4,500	0.65	297.63	379.61

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Company:		Page:	13
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/23/2025
Fastening point:			

Results

Load combination	F_p [psi]	σ [psi]	Utilization [%]	Status
Combination 1	2,808	1,966	71	OK

2.6 Symbol explanation

A_1	Loaded area of concrete
A_2	Supporting area
d	Nominal diameter of the bolt
ε_{lim}	Limit plastic strain
ε_{Pl}	Plastic strain from CBFEM results
f_c	Concrete compressive strength
f_c'	Concrete compressive strength
F_u	Specified minimum tensile strength of the connected material
F_p	Concrete block design bearing strength
$f_{p,max}$	Concrete block design bearing strength maximum
f_y	Yield strength
l_c	Clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material
N	Resulting compression force
σ	Average stress in concrete
σ_{Ed}	Equivalent stress
Φ	Resistance factor
ΦR_n	Factored resistance
t	Thickness of the anchor plate
V	Resultant of shear forces Vy, Vz in bolt.

2.7 Warnings

- By using the CBFEM calculation functionality of PROFIS Engineering you may act outside the applicable design codes and your specified anchor plate may not behave rigid. Please, validate the results with a professional designer and/or structural engineer to ensure suitability and adequacy for your specific jurisdiction and project requirements.
- The anchor is modeled considering stiffness values determined from load displacement curves tested in an independent laboratory. Please note that no simple replacement of the anchor is possible as the anchor stiffness has a major impact on the load distribution results.

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	14
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/23/2025
Fastening point:			

3 Summary of results

Design of the anchor plate, anchors, welds and other elements are based on CBFEM (component based finite element method) and AISC.

	Load combination	Max. utilization	Status
Anchors	Combination 1	190%	NOT OK
Anchor plate	Combination 1	53%	OK
Concrete	Combination 1	71%	OK
Profile	Combination 1	79%	OK

Fastening does not meet the design criteria!

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	15
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/23/2025
Fastening point:			

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Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/23/2025
Fastening point:			

Specifier's comments:

1 Anchor Design

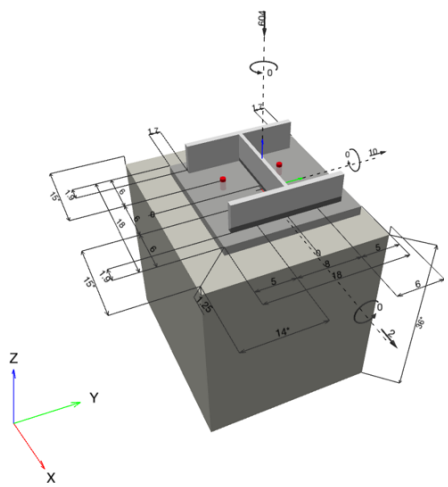
1.1 Input data



Anchor type and diameter:	**Heavy Hex Head ASTM F 1554 GR. 55 3/4**
Item number:	not available
Specification text:	∅ 3/4 in Heavy Hex Head ASTM F 1554 GR. 55 with 18 in nominal embedment depth per Technical data , cast in place installation per MPII,
Effective embedment depth:	h_{ef} = 18.000 in.
Material:	ASTM F 1554
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-19 / CIP
Shear edge breakout verification:	Row closest to edge (Case 3 only from ACI 318-19 Fig. R.17.7.2.1b)
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 1.250 in.
Anchor plate[CBFEM] :	l_x x l_y x t = 18.000 in. x 18.000 in. x 1.250 in.;
Profile:	W shape (AISC), W14X99; (L x W x T x FT) = 14.200 in. x 14.600 in. x 0.485 in. x 0.780 in.
Base material:	cracked concrete, Custom, f_c' = 4,500 psi; h = 36.000 in.
Reinforcement:	tension: not present, shear: not present; edge reinforcement: none or < No. 4 bar

[CBFEM] - The anchor calculation is based on a component-based Finite Element Method (CBFEM)

Geometry [in.] & Loading [kip, ft.kip]



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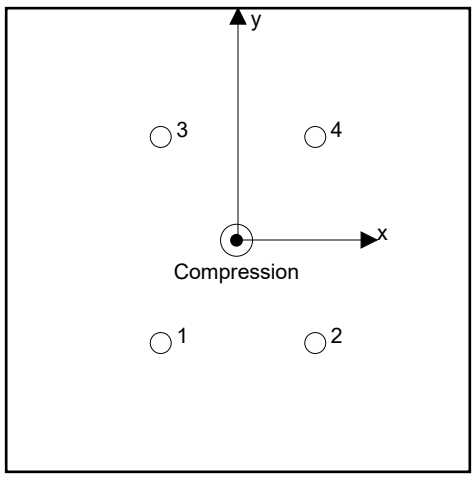
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Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/23/2025
Fastening point:			

1.1.1 Design results

Case	Description	Forces [kip] / Moments [ft.kip]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = -604.000; V_x = 2.000; V_y = 10.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	124

1.2 Load case/Resulting anchor forces



Anchor reactions [kip]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	-0.005	2.475	0.509	2.422
2	-0.005	2.471	0.491	2.422
3	-0.005	2.631	0.526	2.577
4	-0.005	2.622	0.474	2.579

Resulting tension force in (x/y)=(-/-): 0.000 [kip]
Resulting compression force in (x/y)=(-0.057/-0.010): 604.313 [kip]

Anchor forces are calculated based on a component-based Finite Element Method (CBFEM)

1.3 Tension load

	Load N_{ua} [kip]	Capacity ϕ N_n [kip]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	-0.005	18.787	1	OK
Pullout Strength*	-0.005	22.957	-0	not recommended
Concrete Breakout Failure**	N/A	N/A	N/A	N/A
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (anchors in tension)

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Company:		Page:	3
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/23/2025
Fastening point:			

1.3.1 Steel Strength

$N_{sa} = A_{se,N} f_{uta}$ ACI 318-19 Eq. (17.6.1.2)
$\phi N_{sa} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.33	75,000

Calculations

N_{sa} [kip]
25.050

Results

N_{sa} [kip]	ϕ_{steel}	ϕN_{sa} [kip]	N_{ua} [kip]
25.050	0.750	18.787	-0.005

The steel proof was done for the highest absolute force per anchor - in this case compression loading. Please be aware that buckling should be verified separately

1.3.2 Pullout Strength

$N_{pN} = \psi_{c,p} N_p$ ACI 318-19 Eq. (17.6.3.1)
$N_p = 8 A_{brg} f'_c$ ACI 318-19 Eq. (17.6.3.2.2a)
$\phi N_{pN} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$\psi_{c,p}$	A_{brg} [in.^{2}]	λ_a	f'_c [psi]
1.000	0.91	1.000	4,500

Calculations

N_p [kip]
32.796

Results

N_{pn} [kip]	$\phi_{concrete}$	ϕN_{pn} [kip]	N_{ua} [kip]
32.796	0.700	22.957	-0.005

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Company:		Page:	4
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/23/2025
Fastening point:			

1.4 Shear load

	Load V_{ua} [kip]	Capacity ϕ V_n [kip]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	2.631	9.769	27	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	10.198	65.460	16	OK
Concrete edge failure in direction y+**	10.198	8.281	~~124~~	not recommended

* highest loaded anchor **anchor group (relevant anchors)

SEE SUPPLEMENTAL CALCULATION FOR PEDESTAL REINFORCEMENT

1.4.1 Steel Strength

$V_{sa} = 0.6\ A_{se,V}\ f_{uta}$ ACI 318-19 Eq. (17.7.1.2b)
$\phi\ V_{steel} \geq V_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.33	75,000

Calculations

V_{sa} [kip]
15.030

Results

V_{sa} [kip]	ϕ_{steel}	$\phi\ V_{sa}$ [kip]	V_{ua} [kip]
15.030	0.650	9.769	2.631

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	5
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/23/2025
Fastening point:			

1.4.2 Pryout Strength

$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \right]$ ACI 318-19 Eq. (17.7.3.1b)

$\phi V_{cpg} \geq V_{ua}$ ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9 h_{ef}^2$ ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e'_N}{3 h_{ef}}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ ACI 318-19 Eq. (17.6.2.2.1)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	10.000	0.000	0.000	6.000

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.000	-	24	1.000	4,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [kip]
1,008.00	900.00	1.000	1.000	0.820	1.000	50.912

Results

V_{cpg} [kip]	$\phi_{concrete}$	ϕV_{cpg} [kip]	V_{ua} [kip]
93.515	0.700	65.460	10.198

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Company:		Page:	6
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/23/2025
Fastening point:			

1.4.3 Concrete edge failure in direction y+

$V_{cbg} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \Psi_{ec,V} \Psi_{ed,V} \Psi_{c,V} \Psi_{h,V} \Psi_{parallel,V} V_b$ — ACI 318-19 Eq. (17.7.2.1b)

$\phi V_{cbg} \geq V_{ua}$ — ACI 318-19 Table 17.5.2

A_{Vc} see ACI 318-19, Section 17.7.2.1, Fig. R 17.7.2.1(b)*

$A_{Vc0} = 4.5 c_{a1}^2$ — ACI 318-19 Eq. (17.7.2.1.3)

$\Psi_{ec,V} = \left(\frac{1}{1 + \frac{e'_v}{1.5c_{a1}}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.3.1)

$\Psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.4.1b)

$\Psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ — ACI 318-19 Eq. (17.7.2.6.1)

$V_b = 9 \lambda_a \sqrt{f'_c} c_{a1}^{1.5}$ — ACI 318-19 Eq. (17.7.2.2.1b)

Variables

c_{a1} [in.]	c_{a2} [in.]	e_{cV} [in.]	$\Psi_{c,V}$	h_a [in.]
6.000	15.000	0.000	1.000	36.000

l_e [in.]	λ_a	d_a [in.]	f'_c [psi]	$\Psi_{parallel,V}$
6.000	1.000	0.750	4,500	1.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{h,V}$	V_b [kip]
216.00	162.00	1.000	1.000	1.000	8.873

Results

V_{cbg} [kip]	$\phi_{concrete}$	ϕV_{cbg} [kip]	V_{ua} [kip]
11.831	0.700	8.281	10.198

*Anchor row defined by: Anchor 3, 4; Case 3 controls

1.5 Combined tension and shear loads, per ACI 318-19 section 17.8

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.000	1.231	1.000	103	not recommended

$\beta_{NV} = (\beta_N + \beta_V) / 1.2 <= 1$

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Company:		Page:	7
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/23/2025
Fastening point:			

1.6 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates as per current regulations (ETAG 001/Annex C, EOTA TR029, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid base plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- The equations presented in this report are based on imperial units. When inputs are displayed in metric units, the user should be aware that the equations remain in their imperial format.
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- For additional information about ACI 318 strength design provisions, please go to https://viewer.joomag.com/profis-design-guide-us-en-summer-2021/0841849001625154758?short&/
- Attention! In case of compressive anchor forces a buckling check as well as the proof of the local load transfer into and within the base material (incl. punching) has to be done separately.
- The anchor design methods in PROFIS Engineering require rigid anchor plates, as per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means that the anchor plate should be sufficiently rigid to prevent load re-distribution to the anchors due to elastic/plastic displacements. The user accepts that the anchor plate is considered close to rigid by engineering judgment."

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Company:		Page:	8
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/23/2025
Fastening point:			

1.7 Installation data

Profile: W shape (AISC), W14X99; (L x W x T x FT) = 14.200 in. x 14.600 in. x 0.485 in. x 0.780 in.

Hole diameter in the fixture: d_f = 0.812 in.

Plate thickness (input): 1.250 in.

Anchor type and diameter: Heavy Hex Head ASTM F 1554 GR. 55 3/4

Item number: not available

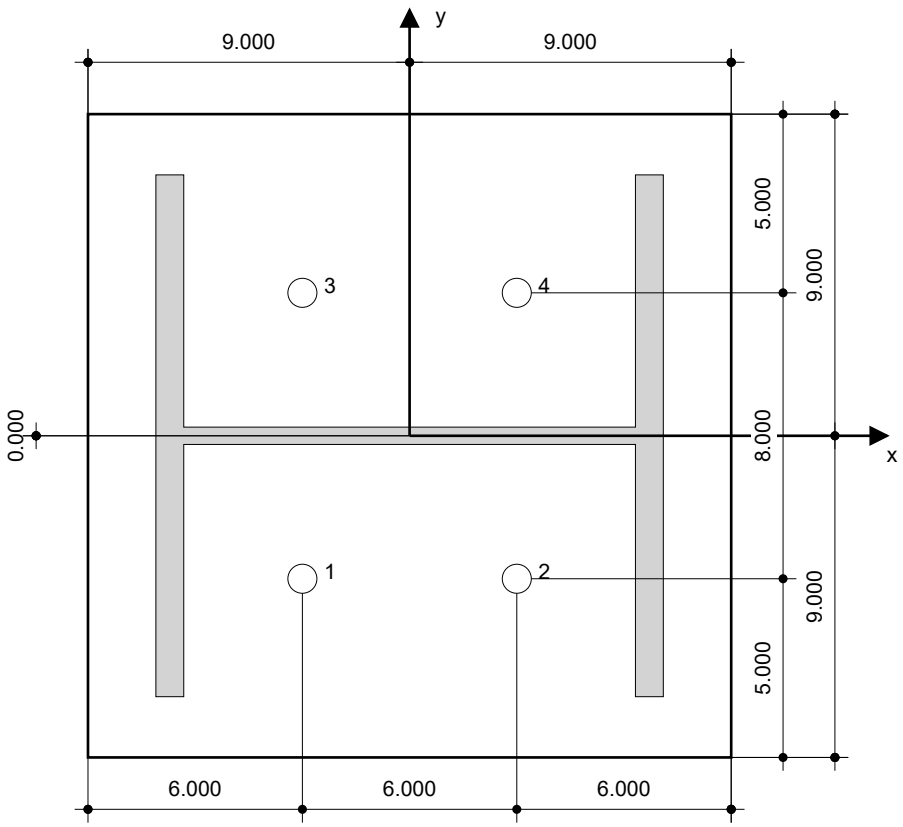
Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 18.000 in.

Minimum thickness of the base material: 19.000 in.

⌀ 3/4 in Heavy Hex Head ASTM F 1554 GR. 55 with 18 in nominal embedment depth per Technical data , cast in place installation per MPII



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-3.000	-4.000	15.000	21.000	14.000	14.000
2	3.000	-4.000	21.000	15.000	14.000	14.000
3	-3.000	4.000	15.000	21.000	22.000	6.000
4	3.000	4.000	21.000	15.000	22.000	6.000

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	9
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/23/2025
Fastening point:			

2 Anchor plate design

2.1 Input data

Anchor plate:	Shape: Rectangular l_x x l_y x t = 18.000 in x 18.000 in x 1.250 in Calculation: CBFEM Material: ASTM A572 Gr.50; F_y = 50,000 psi; ε_{lim} = 5.00%
Anchor type and size:	Heavy Hex Head ASTM F 1554 GR. 55 3/4, h_{ef} = 18.000 in
Anchor stiffness:	The anchor is modeled considering stiffness values determined from load displacement curves tested in an independent laboratory. Please note that no simple replacement of the anchor is possible as the anchor stiffness has a major impact on the load distribution results.
Design method:	AISC and LRFD-based design using component-based FEM
Stand-off installation:	e_b = 0.000 in (No stand-off); t = 1.250 in
Profile:	W14X99; (L x W x T x FT) = 14.200 in x 14.600 in x 0.485 in x 0.780 in Material: ASTM A992; F_y = 50,000 psi; ε_{lim} = 5.00% Eccentricity x: 0.000 in Eccentricity y: 0.000 in
Base material:	Cracked concrete; Custom; $f_{c,cyl}$ = 4,500 psi; h = 36.000 in; E = 4,351,132 psi; G = 1,740,453 psi; v = 0.20; D = 156.07 lb/ft³
Welds (profile to anchor plate):	Type of redistribution: Plastic Material: E70xx
Mesh size:	Number of elements on edge: 8 Min. size of element: 0.394 in Max. size of element: 1.969 in

2.2 Summary

	Description	Profile		Anchor plate			Concrete [%]
		σ_{Ed} [psi]	ε_{Pl} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Hole bearing [%]	
1	Combination 1	37,445	0.00	27,252	0.00	3	73

2.3 Anchor plate classification

Results below are displayed for the decisive load combinations: Combination 1

Anchor tension forces	Equivalent rigid anchor plate (CBFEM)	Component-based Finite Element Method (CBFEM) anchor plate design
Anchor 1	-0.006 kip	-0.005 kip
Anchor 2	-0.006 kip	-0.005 kip
Anchor 3	-0.006 kip	-0.005 kip
Anchor 4	-0.006 kip	-0.005 kip

User accepted to consider the selected anchor plate as rigid by his/her engineering judgement. This means the anchor design guidelines can be applied.

2.4 Profile/Stiffeners/Plate

Profile and stiffeners are verified at the level of the steel to concrete connection. The connection design does not replace the steel design for critical cross sections, which should be performed outside of PROFIS Engineering.

2.4.1 Equivalent stress and plastic strain

Part	Load combination	Material	f_y [psi]	ε_{lim} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Status
Plate	Combination 1	ASTM A572	50,000	5.00	27,252	0.00	OK

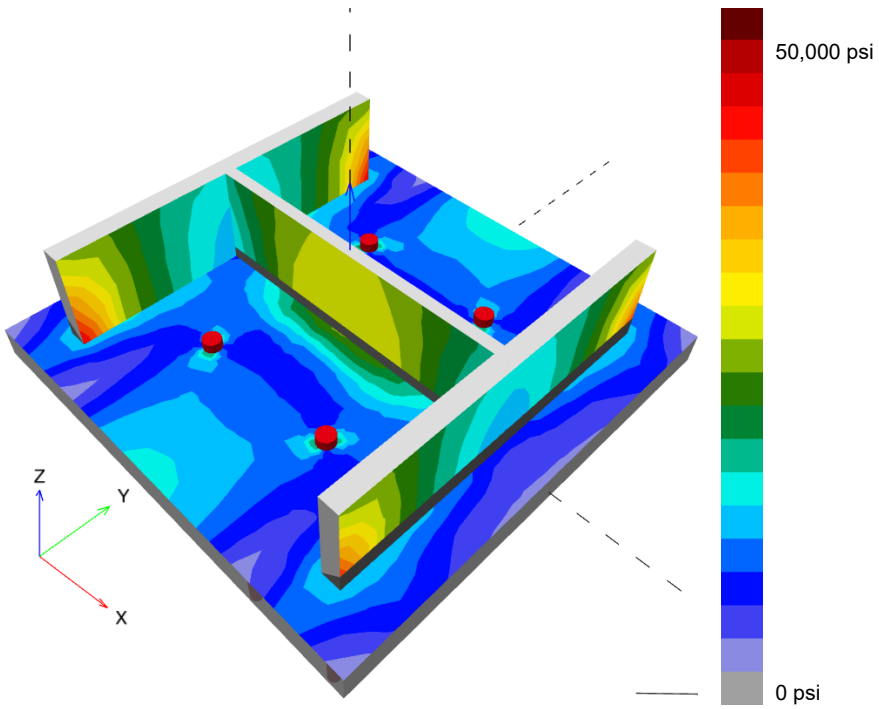
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Company:		Page:	10
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/23/2025
Fastening point:			

Part	Load combination	Material	f_y [psi]	ε_{lim} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Status
		Gr.50					
Profile	Combination 1	ASTM A992	50,000	5.00	37,177	0.00	OK
Profile	Combination 1	ASTM A992	50,000	5.00	37,445	0.00	OK
Profile	Combination 1	ASTM A992	50,000	5.00	30,044	0.00	OK

2.4.1.1 Equivalent stress

Results below are displayed for the decisive load combination: 1 - Combination 1



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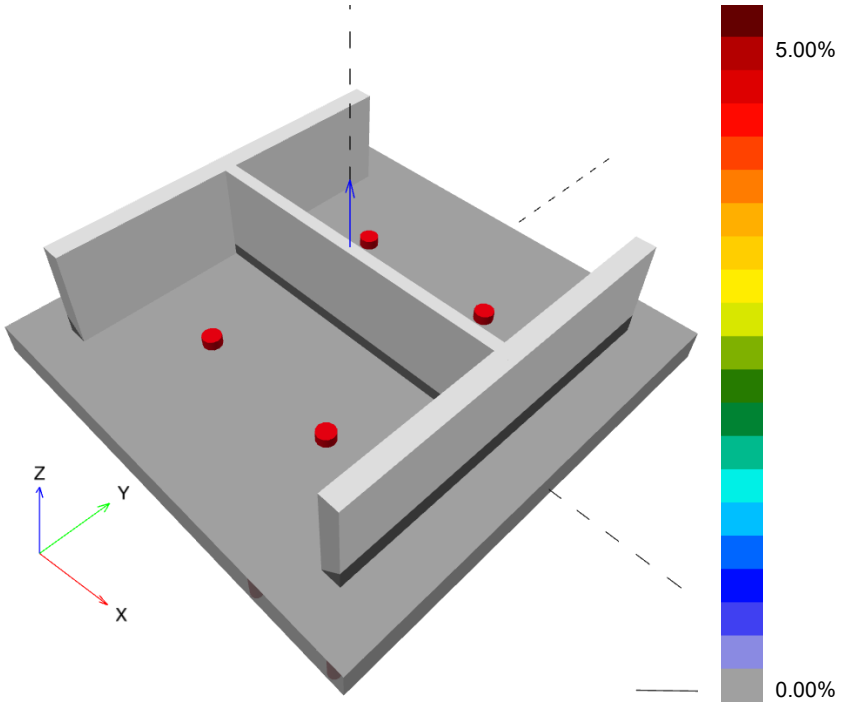
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Company:		Page:	11
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/23/2025
Fastening point:			

2.4.1.2 Plastic strain

Results below are displayed for the decisive load combination: 1 - Combination 1



2.4.2 Plate hole bearing resistance, AISC 360-16 Section J3

Decisive load combination: 1 - Combination 1

Equations

$R_n = \min(1.2\, l_c\, t\, F_u\, ,\ 2.4\, d\, t\, F_u)$ (AISC 360-16 J3-6a, c)

$\Phi R_n = 0.75\, R_n$

$V \leq \Phi R_n$

Variables

	l_c [in]	t [in]	F_u [psi]	d [in]	R_n [kip]
Anchor 1	4.703	1.250	65,000	0.750	146.250
Anchor 2	4.695	1.250	65,000	0.750	146.250
Anchor 3	12.862	1.250	65,000	0.750	146.250
Anchor 4	12.812	1.250	65,000	0.750	146.250

Results

	V [kip]	ΦR_n [kip]	Utilization [%]	Status
Anchor 1	2.475	109.687	3	OK
Anchor 2	2.471	109.687	3	OK
Anchor 3	2.631	109.687	3	OK

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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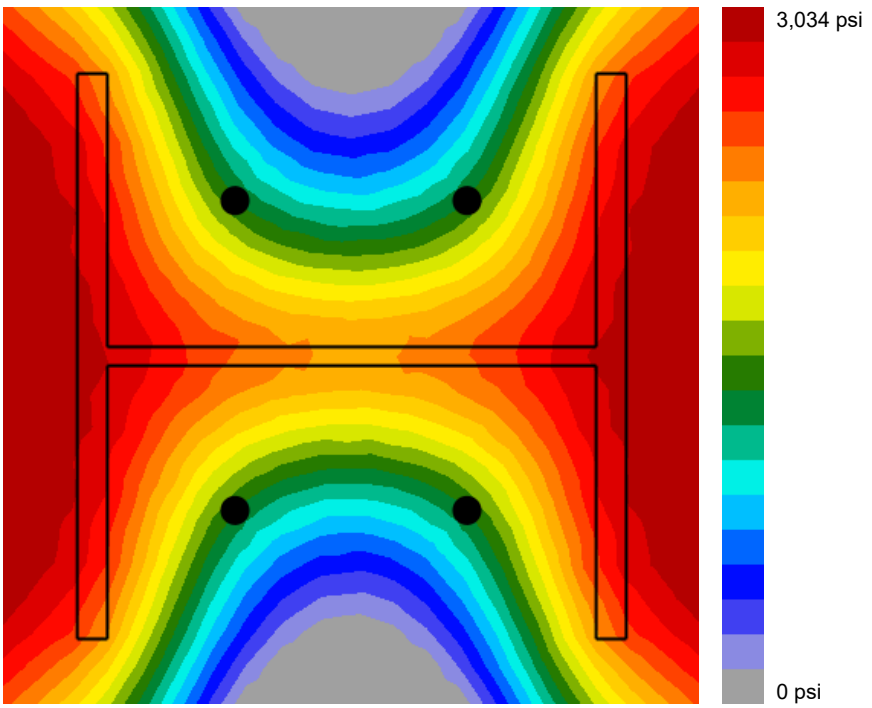
Company:		Page:	12
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/23/2025
Fastening point:			

	V [kip]	ΦR_n [kip]	Utilization [%]	Status
Anchor 4	2.622	109.687	3	OK

2.5 Concrete

Decisive load combination: 1 - Combination 1

2.5.1 Compression in concrete under the anchor plate



2.5.2 Concrete block compressive strength resistance check, AISC 360-16 Section J8

Equations

$$F_p = \Phi f_{p,max}$$

$$f_{p,max} = 0.85 f_c' \sqrt{\left(\frac{A_2}{A_1}\right)} \le 1.7 f_c; \quad \sqrt{\left(\frac{A_2}{A_1}\right)} \le 2$$

$$\sigma = \frac{N}{A_1}$$

$$\text{Utilization} = \frac{\sigma}{F_p}$$

Variables

N [kip]	f_c' [psi]	Φ	A_1 [in²]	A_2 [in²]
604.313	4,500	0.65	296.55	378.76

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Company:		Page:	13
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/23/2025
Fastening point:			

Results

Load combination	F_p [psi]	σ [psi]	Utilization [%]	Status
Combination 1	2,810	2,038	73	OK

2.6 Symbol explanation

A_1	Loaded area of concrete
A_2	Supporting area
d	Nominal diameter of the bolt
ε_{lim}	Limit plastic strain
ε_{Pl}	Plastic strain from CBFEM results
f_c	Concrete compressive strength
f_c'	Concrete compressive strength
F_u	Specified minimum tensile strength of the connected material
F_p	Concrete block design bearing strength
$f_{p,max}$	Concrete block design bearing strength maximum
f_y	Yield strength
l_c	Clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material
N	Resulting compression force
σ	Average stress in concrete
σ_{Ed}	Equivalent stress
Φ	Resistance factor
ΦR_n	Factored resistance
t	Thickness of the anchor plate
V	Resultant of shear forces Vy, Vz in bolt.

2.7 Warnings

- By using the CBFEM calculation functionality of PROFIS Engineering you may act outside the applicable design codes and your specified anchor plate may not behave rigid. Please, validate the results with a professional designer and/or structural engineer to ensure suitability and adequacy for your specific jurisdiction and project requirements.
- The anchor is modeled considering stiffness values determined from load displacement curves tested in an independent laboratory. Please note that no simple replacement of the anchor is possible as the anchor stiffness has a major impact on the load distribution results.

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Company:		Page:	14
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/23/2025
Fastening point:			

3 Summary of results

Design of the anchor plate, anchors, welds and other elements are based on CBFEM (component based finite element method) and AISC.

	Load combination	Max. utilization	Status
Anchors	Combination 1	124%	NOT OK
Anchor plate	Combination 1	55%	OK
Concrete	Combination 1	73%	OK
Profile	Combination 1	75%	OK

Fastening does not meet the design criteria!

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Company:		Page:	15
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/23/2025
Fastening point:			

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Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/23/2025
Fastening point:			

Specifier's comments:

1 Anchor Design

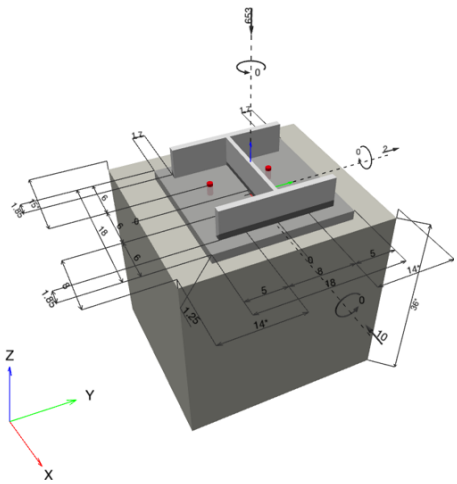
1.1 Input data



Anchor type and diameter:	**Heavy Hex Head ASTM F 1554 GR. 55 3/4**
Item number:	not available
Specification text:	∅ 3/4 in Heavy Hex Head ASTM F 1554 GR. 55 with 18 in nominal embedment depth per Technical data , cast in place installation per MPII,
Effective embedment depth:	h_{ef} = 18.000 in.
Material:	ASTM F 1554
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-19 / CIP
Shear edge breakout verification:	Row closest to edge (Case 3 only from ACI 318-19 Fig. R.17.7.2.1b)
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 1.250 in.
Anchor plate[CBFEM] :	l_x x l_y x t = 18.000 in. x 18.000 in. x 1.250 in.;
Profile:	W shape (AISC), W14X109; (L x W x T x FT) = 14.300 in. x 14.600 in. x 0.525 in. x 0.860 in.
Base material:	cracked concrete, Custom, f_c' = 4,500 psi; h = 36.000 in.
Reinforcement:	tension: not present, shear: not present; edge reinforcement: none or < No. 4 bar

[CBFEM] - The anchor calculation is based on a component-based Finite Element Method (CBFEM)

Geometry [in.] & Loading [kip, ft.kip]



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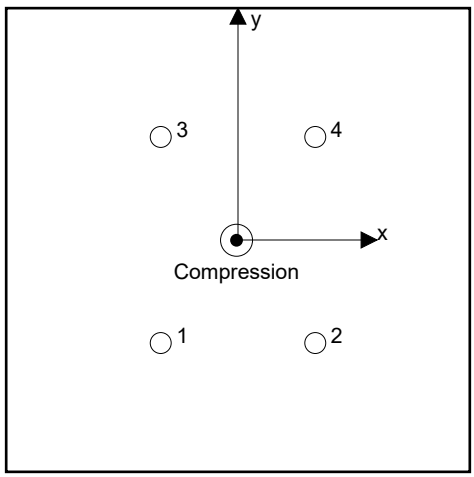
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Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/23/2025
Fastening point:			

1.1.1 Design results

Case	Description	Forces [kip] / Moments [ft.kip]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = -653.000; V_x = -10.000; V_y = 2.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	82

1.2 Load case/Resulting anchor forces



Anchor reactions [kip]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	-0.005	2.517	-2.482	0.421
2	-0.005	2.551	-2.518	0.408
3	-0.005	2.547	-2.480	0.579
4	-0.005	2.588	-2.520	0.592

Resulting tension force in (x/y)=(-/-): 0.000 [kip]
Resulting compression force in (x/y)=(-0.057/-0.010): 653.351 [kip]

Anchor forces are calculated based on a component-based Finite Element Method (CBFEM)

1.3 Tension load

	Load N_{ua} [kip]	Capacity ϕ N_n [kip]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	-0.005	18.787	1	OK
Pullout Strength*	-0.005	22.957	-0	not recommended
Concrete Breakout Failure**	N/A	N/A	N/A	N/A
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (anchors in tension)

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	3
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/23/2025
Fastening point:			

1.3.1 Steel Strength

$N_{sa} = A_{se,N} f_{uta}$ ACI 318-19 Eq. (17.6.1.2)
$\phi N_{sa} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.33	75,000

Calculations

N_{sa} [kip]
25.050

Results

N_{sa} [kip]	ϕ_{steel}	ϕN_{sa} [kip]	N_{ua} [kip]
25.050	0.750	18.787	-0.005

The steel proof was done for the highest absolute force per anchor - in this case compression loading. Please be aware that buckling should be verified separately

1.3.2 Pullout Strength

$N_{pN} = \psi_{c,p} N_p$ ACI 318-19 Eq. (17.6.3.1)
$N_p = 8 A_{brg} f'_c$ ACI 318-19 Eq. (17.6.3.2.2a)
$\phi N_{pN} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$\psi_{c,p}$	A_{brg} [in.^{2}]	λ_a	f'_c [psi]
1.000	0.91	1.000	4,500

Calculations

N_p [kip]
32.796

Results

N_{pn} [kip]	$\phi_{concrete}$	ϕN_{pn} [kip]	N_{ua} [kip]
32.796	0.700	22.957	-0.005

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	4
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/23/2025
Fastening point:			

1.4 Shear load

	Load V_{ua} [kip]	Capacity ϕ V_n [kip]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	2.588	9.769	27	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	10.198	71.105	15	OK
Concrete edge failure in direction y+**	10.198	12.447	82	OK

* highest loaded anchor **anchor group (relevant anchors)

1.4.1 Steel Strength

$V_{sa} = 0.6\ A_{se,V}\ f_{uta}$ ACI 318-19 Eq. (17.7.1.2b)
$\phi\ V_{steel} \geq V_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.33	75,000

Calculations

V_{sa} [kip]
15.030

Results

V_{sa} [kip]	ϕ_{steel}	$\phi\ V_{sa}$ [kip]	V_{ua} [kip]
15.030	0.650	9.769	2.588

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	5
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/23/2025
Fastening point:			

1.4.2 Pryout Strength

$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \Psi_{ec,N} \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_b \right]$ ACI 318-19 Eq. (17.7.3.1b)

$\phi V_{cpg} \geq V_{ua}$ ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9 h_{ef}^2$ ACI 318-19 Eq. (17.6.2.1.4)

$\Psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e'_N}{3 h_{ef}}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.3.1)

$\Psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.4.1b)

$\Psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ ACI 318-19 Eq. (17.6.2.2.1)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	10.000	0.000	0.000	8.000

$\Psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.000	-	24	1.000	4,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\Psi_{ec1,N}$	$\Psi_{ec2,N}$	$\Psi_{ed,N}$	$\Psi_{cp,N}$	N_b [kip]
1,044.00	900.00	1.000	1.000	0.860	1.000	50.912

Results

V_{cpg} [kip]	$\phi_{concrete}$	ϕV_{cpg} [kip]	V_{ua} [kip]
101.579	0.700	71.105	10.198

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Company:		Page:	6
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/23/2025
Fastening point:			

1.4.3 Concrete edge failure in direction y+

$V_{cbg} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \Psi_{ec,V} \Psi_{ed,V} \Psi_{c,V} \Psi_{h,V} \Psi_{parallel,V} V_b$ — ACI 318-19 Eq. (17.7.2.1b)

$\phi V_{cbg} \geq V_{ua}$ — ACI 318-19 Table 17.5.2

A_{Vc} see ACI 318-19, Section 17.7.2.1, Fig. R 17.7.2.1(b)*

$A_{Vc0} = 4.5 c_{a1}^2$ — ACI 318-19 Eq. (17.7.2.1.3)

$\Psi_{ec,V} = \left(\frac{1}{1 + \frac{e'_v}{1.5c_{a1}}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.3.1)

$\Psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.4.1b)

$\Psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ — ACI 318-19 Eq. (17.7.2.6.1)

$V_b = 9 \lambda_a \sqrt{f'_c} c_{a1}^{1.5}$ — ACI 318-19 Eq. (17.7.2.2.1b)

Variables

c_{a1} [in.]	c_{a2} [in.]	e_{cV} [in.]	$\Psi_{c,V}$	h_a [in.]
14.000	8.000	0.000	1.000	36.000

l_e [in.]	λ_a	d_a [in.]	f'_c [psi]	$\Psi_{parallel,V}$
6.000	1.000	0.750	4,500	1.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{h,V}$	V_b [kip]
609.00	882.00	1.000	0.814	1.000	31.626

Results

V_{cbg} [kip]	$\phi_{concrete}$	ϕV_{cbg} [kip]	V_{ua} [kip]
17.781	0.700	12.447	10.198

*Anchor row defined by: Anchor 3, 4; Case 3 controls

1.5 Combined tension and shear loads, per ACI 318-19 section 17.8

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.000	0.819	1.000	69	OK

$\beta_{NV} = (\beta_N + \beta_V) / 1.2 <= 1$

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Company:		Page:	7
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/23/2025
Fastening point:			

1.6 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates as per current regulations (ETAG 001/Annex C, EOTA TR029, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid base plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- The equations presented in this report are based on imperial units. When inputs are displayed in metric units, the user should be aware that the equations remain in their imperial format.
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- For additional information about ACI 318 strength design provisions, please go to https://viewer.joomag.com/profis-design-guide-us-en-summer-2021/0841849001625154758?short&/
- Attention! In case of compressive anchor forces a buckling check as well as the proof of the local load transfer into and within the base material (incl. punching) has to be done separately.
- The anchor design methods in PROFIS Engineering require rigid anchor plates, as per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means that the anchor plate should be sufficiently rigid to prevent load re-distribution to the anchors due to elastic/plastic displacements. The user accepts that the anchor plate is considered close to rigid by engineering judgment."

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Company:		Page:	8
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/23/2025
Fastening point:			

1.7 Installation data

Profile: W shape (AISC), W14X109; (L x W x T x FT) = 14.300 in. x 14.600 in. x 0.525 in. x 0.860 in.

Hole diameter in the fixture: d_f = 0.812 in.

Plate thickness (input): 1.250 in.

Anchor type and diameter: Heavy Hex Head ASTM F 1554 GR. 55 3/4

Item number: not available

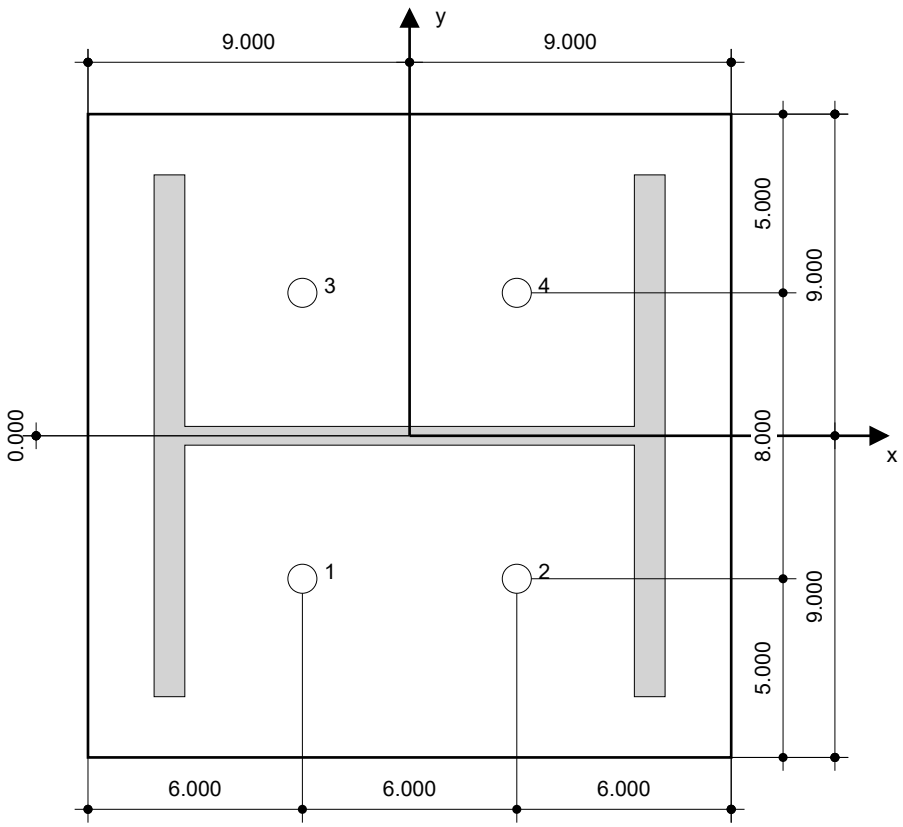
Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 18.000 in.

Minimum thickness of the base material: 19.000 in.

⌀ 3/4 in Heavy Hex Head ASTM F 1554 GR. 55 with 18 in nominal embedment depth per Technical data , cast in place installation per MPII



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-3.000	-4.000	15.000	14.000	14.000	22.000
2	3.000	-4.000	21.000	8.000	14.000	22.000
3	-3.000	4.000	15.000	14.000	22.000	14.000
4	3.000	4.000	21.000	8.000	22.000	14.000

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Company:		Page:	9
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/23/2025
Fastening point:			

2 Anchor plate design

2.1 Input data

Anchor plate:	Shape: Rectangular l_x x l_y x t = 18.000 in x 18.000 in x 1.250 in Calculation: CBFEM Material: ASTM A572 Gr.50; F_y = 50,000 psi; ε_{lim} = 5.00%
Anchor type and size:	Heavy Hex Head ASTM F 1554 GR. 55 3/4, h_{ef} = 18.000 in
Anchor stiffness:	The anchor is modeled considering stiffness values determined from load displacement curves tested in an independent laboratory. Please note that no simple replacement of the anchor is possible as the anchor stiffness has a major impact on the load distribution results.
Design method:	AISC and LRFD-based design using component-based FEM
Stand-off installation:	e_b = 0.000 in (No stand-off); t = 1.250 in
Profile:	W14X109; (L x W x T x FT) = 14.300 in x 14.600 in x 0.525 in x 0.860 in Material: ASTM A992; F_y = 50,000 psi; ε_{lim} = 5.00% Eccentricity x: 0.000 in Eccentricity y: 0.000 in
Base material:	Cracked concrete; Custom; $f_{c,cyl}$ = 4,500 psi; h = 36.000 in; E = 4,351,132 psi; G = 1,740,453 psi; v = 0.20; D = 156.07 lb/ft³
Welds (profile to anchor plate):	Type of redistribution: Plastic Material: E70xx
Mesh size:	Number of elements on edge: 8 Min. size of element: 0.394 in Max. size of element: 1.969 in

2.2 Summary

	Description	Profile		Anchor plate			Concrete [%]
		σ_{Ed} [psi]	ε_{Pl} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Hole bearing [%]	
1	Combination 1	38,804	0.00	29,299	0.00	3	71

2.3 Anchor plate classification

Results below are displayed for the decisive load combinations: Combination 1

Anchor tension forces	Equivalent rigid anchor plate (CBFEM)	Component-based Finite Element Method (CBFEM) anchor plate design
Anchor 1	-0.007 kip	-0.005 kip
Anchor 2	-0.007 kip	-0.005 kip
Anchor 3	-0.007 kip	-0.005 kip
Anchor 4	-0.007 kip	-0.005 kip

User accepted to consider the selected anchor plate as rigid by his/her engineering judgement. This means the anchor design guidelines can be applied.

2.4 Profile/Stiffeners/Plate

Profile and stiffeners are verified at the level of the steel to concrete connection. The connection design does not replace the steel design for critical cross sections, which should be performed outside of PROFIS Engineering.

2.4.1 Equivalent stress and plastic strain

Part	Load combination	Material	f_y [psi]	ε_{lim} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Status
Plate	Combination 1	ASTM A572	50,000	5.00	29,299	0.00	OK

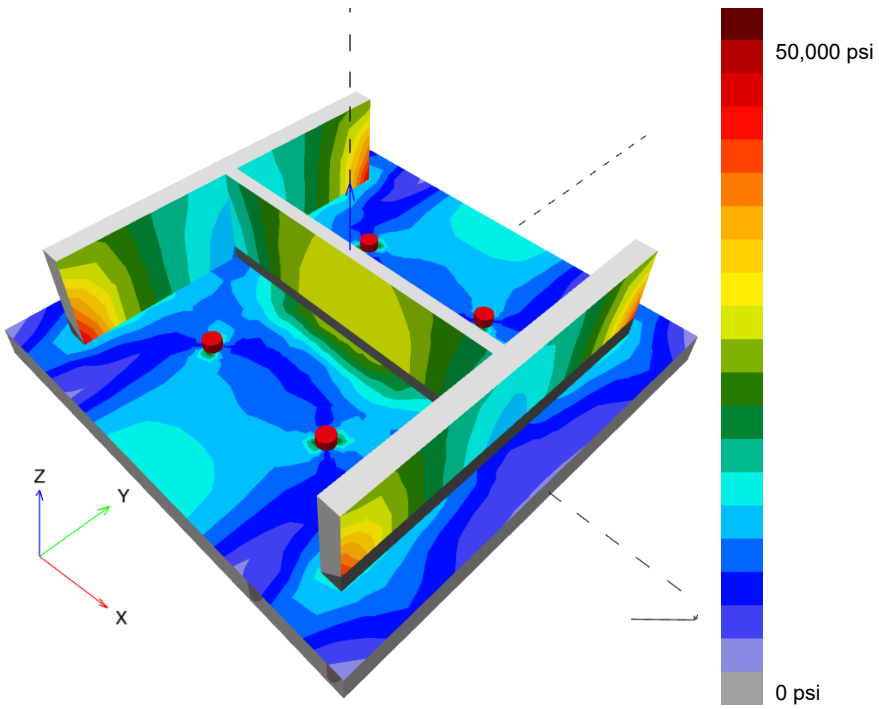
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Company:		Page:	10
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/23/2025
Fastening point:			

Part	Load combination	Material	f_y [psi]	ε_{lim} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Status
		Gr.50					
Profile	Combination 1	ASTM A992	50,000	5.00	38,804	0.00	OK
Profile	Combination 1	ASTM A992	50,000	5.00	37,652	0.00	OK
Profile	Combination 1	ASTM A992	50,000	5.00	30,225	0.00	OK

2.4.1.1 Equivalent stress

Results below are displayed for the decisive load combination: 1 - Combination 1

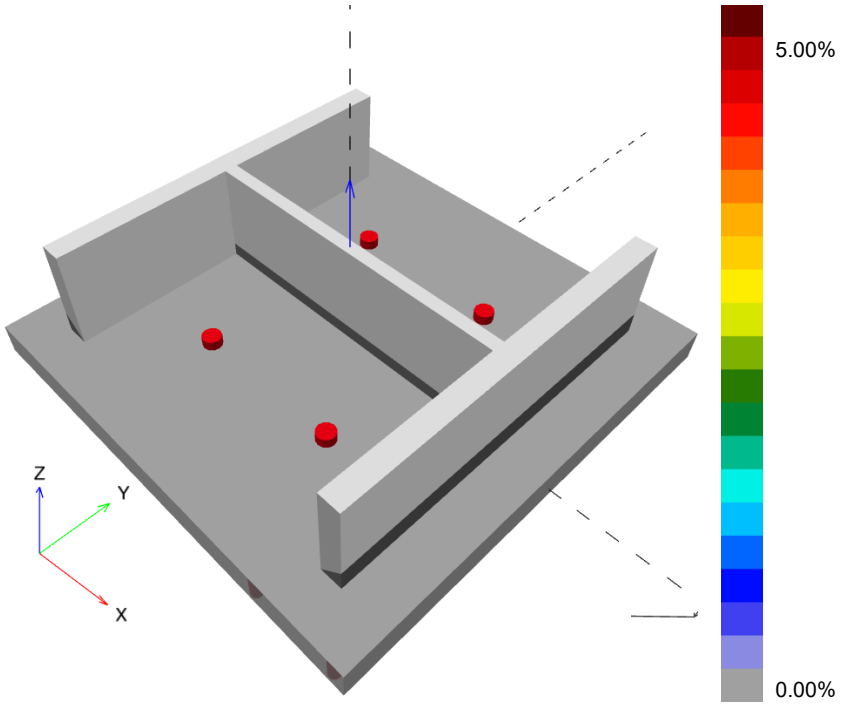


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Company:		Page:	11
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/23/2025
Fastening point:			

2.4.1.2 Plastic strain

Results below are displayed for the decisive load combination: 1 - Combination 1



2.4.2 Plate hole bearing resistance, AISC 360-16 Section J3

Decisive load combination: 1 - Combination 1

Equations

$R_n = \min(1.2\, l_c\, t\, F_u\, ,\ 2.4\, d\, t\, F_u)$ (AISC 360-16 J3-6a, c)

$\Phi R_n = 0.75\, R_n$

$V \leq \Phi R_n$

Variables

	l_c [in]	t [in]	F_u [psi]	d [in]	R_n [kip]
Anchor 1	11.765	1.250	65,000	0.750	146.250
Anchor 2	5.672	1.250	65,000	0.750	146.250
Anchor 3	11.917	1.250	65,000	0.750	146.250
Anchor 4	5.757	1.250	65,000	0.750	146.250

Results

	V [kip]	ΦR_n [kip]	Utilization [%]	Status
Anchor 1	2.517	109.687	3	OK
Anchor 2	2.551	109.687	3	OK
Anchor 3	2.547	109.687	3	OK

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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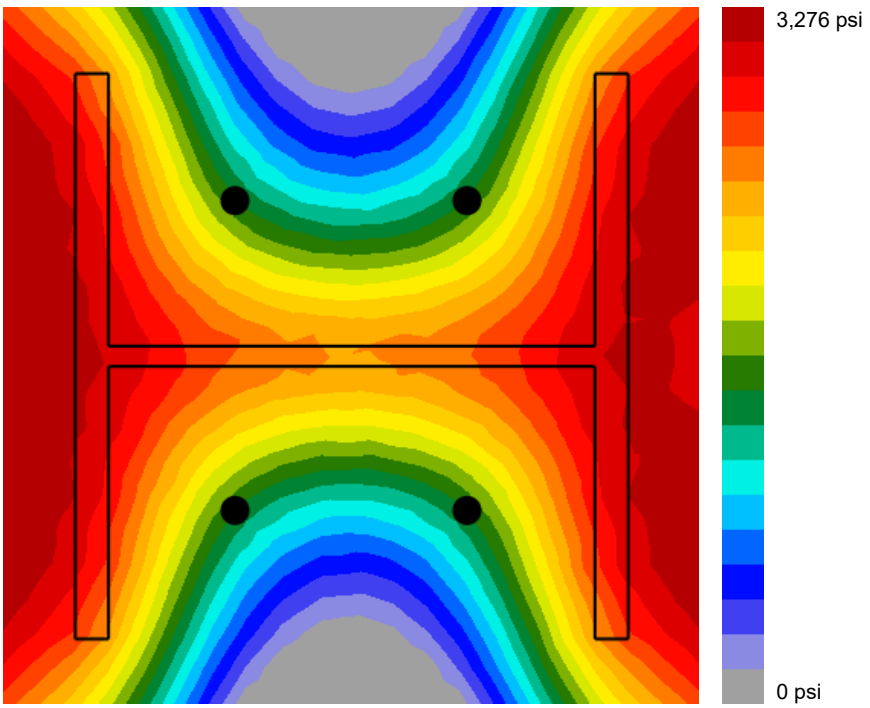
Company:		Page:	12
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/23/2025
Fastening point:			

	V [kip]	ΦR_n [kip]	Utilization [%]	Status
Anchor 4	2.588	109.687	3	OK

2.5 Concrete

Decisive load combination: 1 - Combination 1

2.5.1 Compression in concrete under the anchor plate



2.5.2 Concrete block compressive strength resistance check, AISC 360-16 Section J8

Equations

$F_p = \Phi f_{p,max}$

$f_{p,max} = 0.85 f_c' \sqrt{\left(\frac{A_2}{A_1}\right)} \le 1.7 f_c; \quad \sqrt{\left(\frac{A_2}{A_1}\right)} \le 2$

$\sigma = \frac{N}{A_1}$

$\text{Utilization} = \frac{\sigma}{F_p}$

Variables

N [kip]	f_c' [psi]	Φ	A_1 [in²]	A_2 [in²]
653.351	4,500	0.65	297.04	466.21

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Company:		Page:	13
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/23/2025
Fastening point:			

Results

Load combination	F_p [psi]	σ [psi]	Utilization [%]	Status
Combination 1	3,115	2,200	71	OK

2.6 Symbol explanation

A_1	Loaded area of concrete
A_2	Supporting area
d	Nominal diameter of the bolt
ε_{lim}	Limit plastic strain
ε_{Pl}	Plastic strain from CBFEM results
f_c	Concrete compressive strength
f_c'	Concrete compressive strength
F_u	Specified minimum tensile strength of the connected material
F_p	Concrete block design bearing strength
$f_{p,max}$	Concrete block design bearing strength maximum
f_y	Yield strength
l_c	Clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material
N	Resulting compression force
σ	Average stress in concrete
σ_{Ed}	Equivalent stress
Φ	Resistance factor
ΦR_n	Factored resistance
t	Thickness of the anchor plate
V	Resultant of shear forces Vy, Vz in bolt.

2.7 Warnings

- By using the CBFEM calculation functionality of PROFIS Engineering you may act outside the applicable design codes and your specified anchor plate may not behave rigid. Please, validate the results with a professional designer and/or structural engineer to ensure suitability and adequacy for your specific jurisdiction and project requirements.
- The anchor is modeled considering stiffness values determined from load displacement curves tested in an independent laboratory. Please note that no simple replacement of the anchor is possible as the anchor stiffness has a major impact on the load distribution results.

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	14
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/23/2025
Fastening point:			

3 Summary of results

Design of the anchor plate, anchors, welds and other elements are based on CBFEM (component based finite element method) and AISC.

	Load combination	Max. utilization	Status
Anchors	Combination 1	82%	OK
Anchor plate	Combination 1	59%	OK
Concrete	Combination 1	71%	OK
Profile	Combination 1	78%	OK

Fastening meets the design criteria!

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	15
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/23/2025
Fastening point:			

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Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/24/2025
Fastening point:			

Specifier's comments:

1 Anchor Design

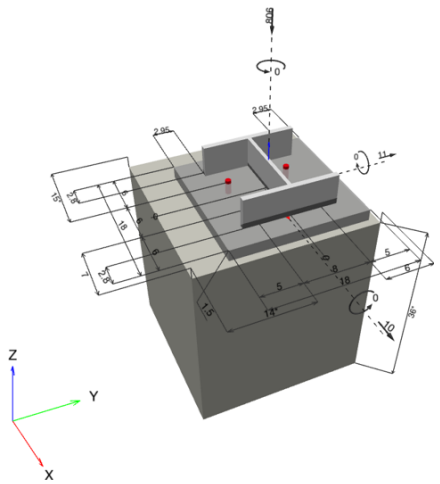
1.1 Input data



Anchor type and diameter:	**Heavy Hex Head ASTM F 1554 GR. 55 3/4**
Item number:	not available
Specification text:	∅ 3/4 in Heavy Hex Head ASTM F 1554 GR. 55 with 18 in nominal embedment depth per Technical data , cast in place installation per MPII,
Effective embedment depth:	h_{ef} = 18.000 in.
Material:	ASTM F 1554
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-19 / CIP
Shear edge breakout verification:	Row closest to edge (Case 3 only from ACI 318-19 Fig. R.17.7.2.1b)
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 1.500 in.
Anchor plate[CBFEM] :	l_x x l_y x t = 18.000 in. x 18.000 in. x 1.500 in.;
Profile:	W shape (AISC), W12X79; (L x W x T x FT) = 12.400 in. x 12.100 in. x 0.470 in. x 0.735 in.
Base material:	cracked concrete, Custom, f_c' = 4,500 psi; h = 36.000 in.
Reinforcement:	tension: not present, shear: not present; edge reinforcement: none or < No. 4 bar

[CBFEM] - The anchor calculation is based on a component-based Finite Element Method (CBFEM)

Geometry [in.] & Loading [kip, ft.kip]



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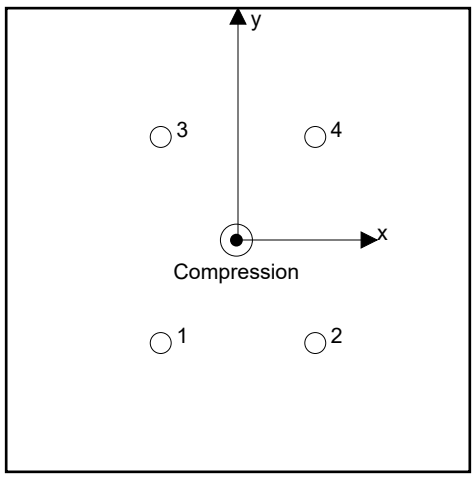
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Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/24/2025
Fastening point:			

1.1.1 Design results

Case	Description	Forces [kip] / Moments [ft.kip]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = -806.000; V_x = 10.000; V_y = 11.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	210

1.2 Load case/Resulting anchor forces



Anchor reactions [kip]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	-0.010	3.621	2.499	2.620
2	-0.009	3.620	2.499	2.619
3	-0.009	3.827	2.525	2.876
4	-0.009	3.802	2.477	2.884

Resulting tension force in (x/y)=(-/-): 0.000 [kip]
Resulting compression force in (x/y)=(-0.061/-0.005): 806.204 [kip]

Anchor forces are calculated based on a component-based Finite Element Method (CBFEM)

1.3 Tension load

	Load N_{ua} [kip]	Capacity ϕ N_n [kip]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	-0.010	18.787	1	OK
Pullout Strength*	-0.009	22.957	-0	not recommended
Concrete Breakout Failure**	N/A	N/A	N/A	N/A
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (anchors in tension)

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Company:		Page:	3
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/24/2025
Fastening point:			

1.3.1 Steel Strength

$N_{sa} = A_{se,N} f_{uta}$ ACI 318-19 Eq. (17.6.1.2)
$\phi N_{sa} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.33	75,000

Calculations

N_{sa} [kip]
25.050

Results

N_{sa} [kip]	ϕ_{steel}	ϕN_{sa} [kip]	N_{ua} [kip]
25.050	0.750	18.787	-0.010

The steel proof was done for the highest absolute force per anchor - in this case compression loading. Please be aware that buckling should be verified separately

1.3.2 Pullout Strength

$N_{pN} = \psi_{c,p} N_p$ ACI 318-19 Eq. (17.6.3.1)
$N_p = 8 A_{brg} f'_c$ ACI 318-19 Eq. (17.6.3.2.2a)
$\phi N_{pN} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$\psi_{c,p}$	A_{brg} [in.^{2}]	λ_a	f'_c [psi]
1.000	0.91	1.000	4,500

Calculations

N_p [kip]
32.796

Results

N_{pn} [kip]	$\phi_{concrete}$	ϕN_{pn} [kip]	N_{ua} [kip]
32.796	0.700	22.957	-0.009

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Company:		Page:	4
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/24/2025
Fastening point:			

1.4 Shear load

	Load V_{ua} [kip]	Capacity ϕ V_n [kip]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	3.827	9.769	40	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	14.866	50.913	30	OK
Concrete edge failure in direction y+**	14.866	7.084	~~210~~	not recommended

* highest loaded anchor **anchor group (relevant anchors)

SEE SUPPLEMENTAL CALCULATION FOR PEDESTAL REINFORCEMENT

1.4.1 Steel Strength

$V_{sa} = 0.6\ A_{se,V}\ f_{uta}$ ACI 318-19 Eq. (17.7.1.2b)
$\phi\ V_{steel} \geq V_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.33	75,000

Calculations

V_{sa} [kip]
15.030

Results

V_{sa} [kip]	ϕ_{steel}	$\phi\ V_{sa}$ [kip]	V_{ua} [kip]
15.030	0.650	9.769	3.827

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Company:		Page:	5
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/24/2025
Fastening point:			

1.4.2 Pryout Strength

$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \right]$ ACI 318-19 Eq. (17.7.3.1b)

$\phi V_{cpg} \geq V_{ua}$ ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9 h_{ef}^2$ ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e'_N}{3 h_{ef}}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ ACI 318-19 Eq. (17.6.2.2.1)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	10.000	0.000	0.000	6.000

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.000	-	24	1.000	4,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [kip]
784.00	900.00	1.000	1.000	0.820	1.000	50.912

Results

V_{cpg} [kip]	$\phi_{concrete}$	ϕV_{cpg} [kip]	V_{ua} [kip]
72.734	0.700	50.913	14.866

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Company:		Page:	6
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/24/2025
Fastening point:			

1.4.3 Concrete edge failure in direction y+

$V_{cbg} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \Psi_{ec,V} \Psi_{ed,V} \Psi_{c,V} \Psi_{h,V} \Psi_{parallel,V} V_b$ — ACI 318-19 Eq. (17.7.2.1b)

$\phi V_{cbg} \geq V_{ua}$ — ACI 318-19 Table 17.5.2

A_{Vc} see ACI 318-19, Section 17.7.2.1, Fig. R 17.7.2.1(b)*

$A_{Vc0} = 4.5 c_{a1}^2$ — ACI 318-19 Eq. (17.7.2.1.3)

$\Psi_{ec,V} = \left(\frac{1}{1 + \frac{e'_v}{1.5c_{a1}}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.3.1)

$\Psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.4.1b)

$\Psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ — ACI 318-19 Eq. (17.7.2.6.1)

$V_b = 9 \lambda_a \sqrt{f'_c} c_{a1}^{1.5}$ — ACI 318-19 Eq. (17.7.2.2.1b)

Variables

c_{a1} [in.]	c_{a2} [in.]	e_{cV} [in.]	$\Psi_{c,V}$	h_a [in.]
6.000	7.000	0.001	1.000	36.000

l_e [in.]	λ_a	d_a [in.]	f'_c [psi]	$\Psi_{parallel,V}$
6.000	1.000	0.750	4,500	1.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{h,V}$	V_b [kip]
198.00	162.00	1.000	0.933	1.000	8.873

Results

V_{cbg} [kip]	$\phi_{concrete}$	ϕV_{cbg} [kip]	V_{ua} [kip]
10.120	0.700	7.084	14.866

*Anchor row defined by: Anchor 3, 4; Case 3 controls

1.5 Combined tension and shear loads, per ACI 318-19 section 17.8

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.001	2.098	1.000	175	not recommended

$\beta_{NV} = (\beta_N + \beta_V) / 1.2 <= 1$

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Company:		Page:	7
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/24/2025
Fastening point:			

1.6 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates as per current regulations (ETAG 001/Annex C, EOTA TR029, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid base plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- The equations presented in this report are based on imperial units. When inputs are displayed in metric units, the user should be aware that the equations remain in their imperial format.
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- For additional information about ACI 318 strength design provisions, please go to https://viewer.joomag.com/profis-design-guide-us-en-summer-2021/0841849001625154758?short&/
- Attention! In case of compressive anchor forces a buckling check as well as the proof of the local load transfer into and within the base material (incl. punching) has to be done separately.
- The anchor design methods in PROFIS Engineering require rigid anchor plates, as per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means that the anchor plate should be sufficiently rigid to prevent load re-distribution to the anchors due to elastic/plastic displacements. The user accepts that the anchor plate is considered close to rigid by engineering judgment."

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Company:		Page:	8
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/24/2025
Fastening point:			

1.7 Installation data

Profile: W shape (AISC), W12X79; (L x W x T x FT) = 12.400 in. x 12.100 in. x 0.470 in. x 0.735 in.

Hole diameter in the fixture: d_f = 0.812 in.

Plate thickness (input): 1.500 in.

Anchor type and diameter: Heavy Hex Head ASTM F 1554 GR. 55 3/4

Item number: not available

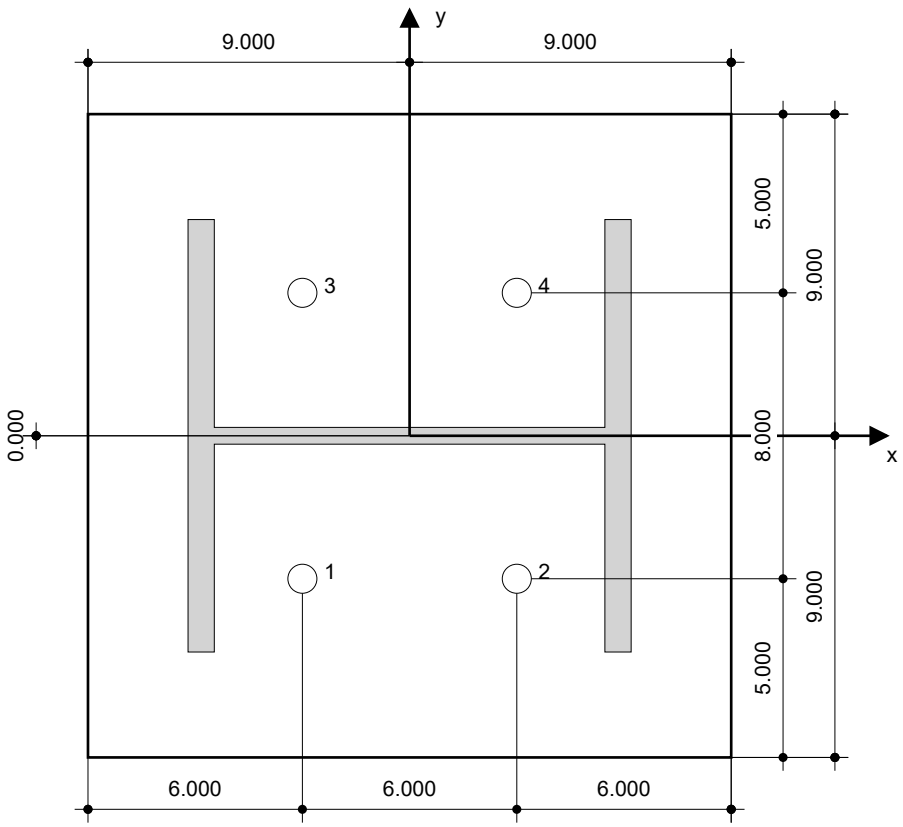
Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 18.000 in.

Minimum thickness of the base material: 19.000 in.

⌀ 3/4 in Heavy Hex Head ASTM F 1554 GR. 55 with 18 in nominal embedment depth per Technical data , cast in place installation per MPII



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-3.000	-4.000	15.000	13.000	14.000	14.000
2	3.000	-4.000	21.000	7.000	14.000	14.000
3	-3.000	4.000	15.000	13.000	22.000	6.000
4	3.000	4.000	21.000	7.000	22.000	6.000

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Company:		Page:	9
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/24/2025
Fastening point:			

2 Anchor plate design

2.1 Input data

Anchor plate:	Shape: Rectangular l_x x l_y x t = 18.000 in x 18.000 in x 1.500 in Calculation: CBFEM Material: ASTM A572 Gr.50; F_y = 50,000 psi; ε_{lim} = 5.00%
Anchor type and size:	Heavy Hex Head ASTM F 1554 GR. 55 3/4, h_{ef} = 18.000 in
Anchor stiffness:	The anchor is modeled considering stiffness values determined from load displacement curves tested in an independent laboratory. Please note that no simple replacement of the anchor is possible as the anchor stiffness has a major impact on the load distribution results.
Design method:	AISC and LRFD-based design using component-based FEM
Stand-off installation:	e_b = 0.000 in (No stand-off); t = 1.500 in
Profile:	W12X79; (L x W x T x FT) = 12.400 in x 12.100 in x 0.470 in x 0.735 in Material: ASTM A992; F_y = 50,000 psi; ε_{lim} = 5.00% Eccentricity x: 0.000 in Eccentricity y: 0.000 in
Base material:	Cracked concrete; Custom; $f_{c,cyl}$ = 4,500 psi; h = 36.000 in; E = 4,351,132 psi; G = 1,740,453 psi; v = 0.20; D = 156.07 lb/ft³
Welds (profile to anchor plate):	Type of redistribution: Plastic Material: E70xx
Mesh size:	Number of elements on edge: 8 Min. size of element: 0.394 in Max. size of element: 1.969 in

2.2 Summary

	Description	Profile		Anchor plate			Concrete [%]
		σ_{Ed} [psi]	ε_{Pl} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Hole bearing [%]	
1	Combination 1	50,064	0.22	29,960	0.00	3	94

2.3 Anchor plate classification

Results below are displayed for the decisive load combinations: Combination 1

Anchor tension forces	Equivalent rigid anchor plate (CBFEM)	Component-based Finite Element Method (CBFEM) anchor plate design
Anchor 1	-0.009 kip	-0.010 kip
Anchor 2	-0.009 kip	-0.009 kip
Anchor 3	-0.009 kip	-0.009 kip
Anchor 4	-0.009 kip	-0.009 kip

User accepted to consider the selected anchor plate as rigid by his/her engineering judgement. This means the anchor design guidelines can be applied.

2.4 Profile/Stiffeners/Plate

Profile and stiffeners are verified at the level of the steel to concrete connection. The connection design does not replace the steel design for critical cross sections, which should be performed outside of PROFIS Engineering.

2.4.1 Equivalent stress and plastic strain

Part	Load combination	Material	f_y [psi]	ε_{lim} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Status
Plate	Combination 1	ASTM A572	50,000	5.00	29,960	0.00	OK

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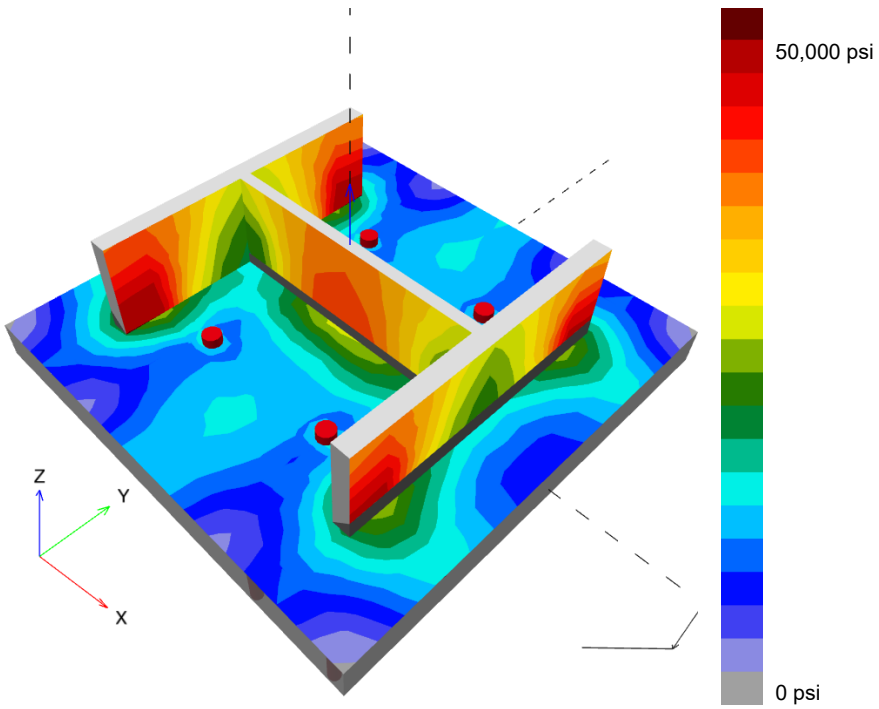
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Company:		Page:	10
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/24/2025
Fastening point:			

Part	Load combination	Material	f_y [psi]	ε_{lim} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Status
		Gr.50					
Profile	Combination 1	ASTM A992	50,000	5.00	50,061	0.21	OK
Profile	Combination 1	ASTM A992	50,000	5.00	50,064	0.22	OK
Profile	Combination 1	ASTM A992	50,000	5.00	42,408	0.00	OK

2.4.1.1 Equivalent stress

Results below are displayed for the decisive load combination: 1 - Combination 1



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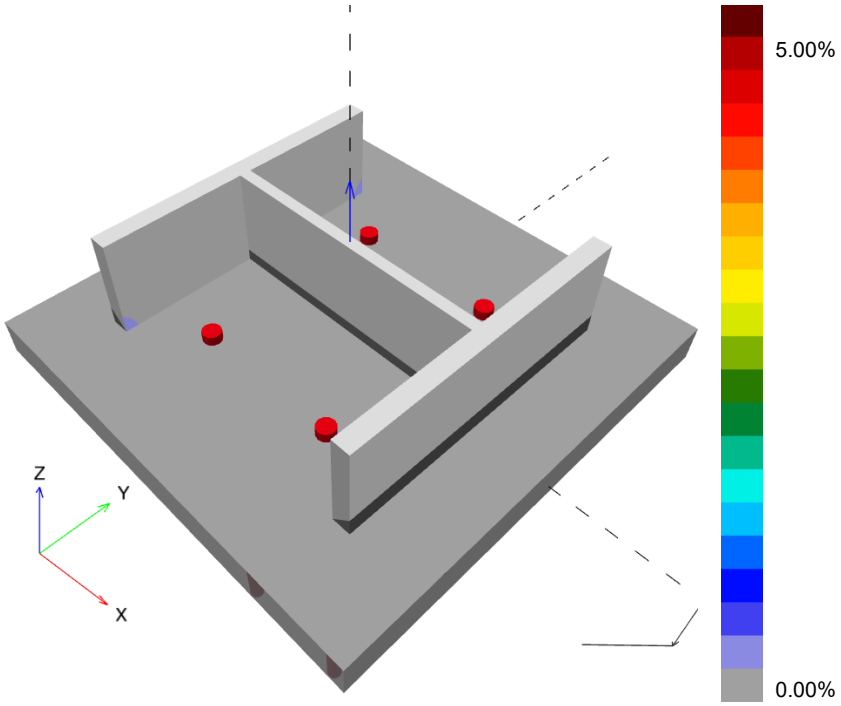
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Company:		Page:	11
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/24/2025
Fastening point:			

2.4.1.2 Plastic strain

Results below are displayed for the decisive load combination: 1 - Combination 1



2.4.2 Plate hole bearing resistance, AISC 360-16 Section J3

Decisive load combination: 1 - Combination 1

Equations

$R_n = \min(1.2\, l_c\, t\, F_u\, ,\ 2.4\, d\, t\, F_u)$ (AISC 360-16 J3-6a, c)

$\Phi R_n = 0.75\, R_n$

$V \leq \Phi R_n$

Variables

	l_c [in]	t [in]	F_u [psi]	d [in]	R_n [kip]
Anchor 1	6.503	1.500	65,000	0.750	175.500
Anchor 2	6.505	1.500	65,000	0.750	175.500
Anchor 3	8.688	1.500	65,000	0.750	175.500
Anchor 4	9.188	1.500	65,000	0.750	175.500

Results

	V [kip]	ΦR_n [kip]	Utilization [%]	Status
Anchor 1	3.621	131.625	3	OK
Anchor 2	3.620	131.625	3	OK
Anchor 3	3.827	131.625	3	OK

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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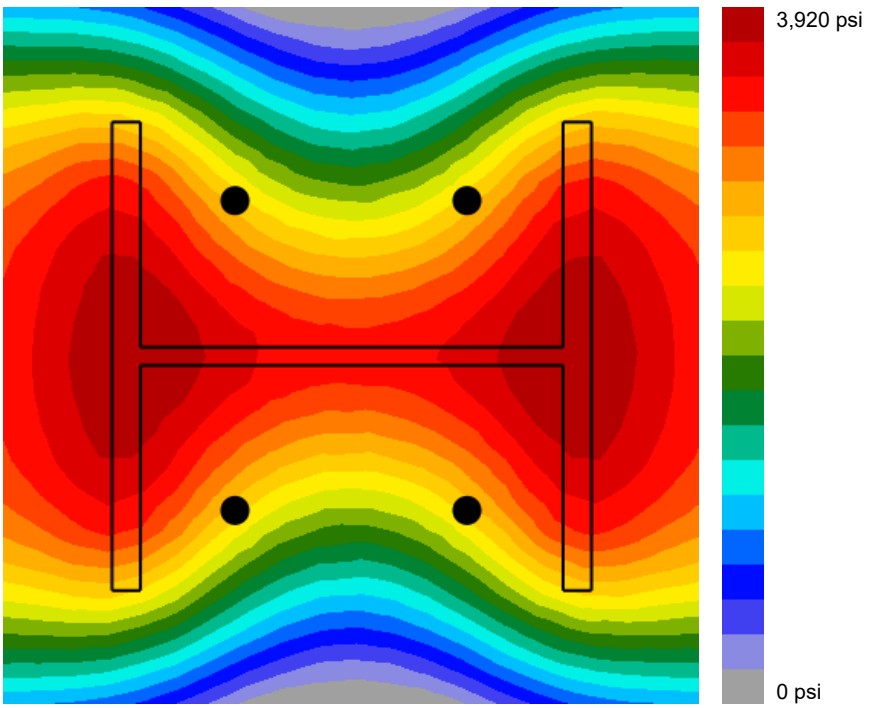
Company:		Page:	12
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/24/2025
Fastening point:			

	V [kip]	ΦR_n [kip]	Utilization [%]	Status
Anchor 4	3.802	131.625	3	OK

2.5 Concrete

Decisive load combination: 1 - Combination 1

2.5.1 Compression in concrete under the anchor plate



2.5.2 Concrete block compressive strength resistance check, AISC 360-16 Section J8

Equations

$F_p = \Phi f_{p,max}$

$f_{p,max} = 0.85 f_c' \sqrt{(\frac{A_2}{A_1})} \le 1.7 f_c;\ \sqrt{(\frac{A_2}{A_1})} \le 2$

$\sigma = \frac{N}{A_1}$

$\text{Utilization} = \frac{\sigma}{F_p}$

Variables

N [kip]	f_c' [psi]	Φ	A_1 [in²]	A_2 [in²]
806.204	4,500	0.65	311.79	389.78

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	13
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/24/2025
Fastening point:			

Results

Load combination	F_p [psi]	σ [psi]	Utilization [%]	Status
Combination 1	2,780	2,586	94	OK

2.6 Symbol explanation

A_1	Loaded area of concrete
A_2	Supporting area
d	Nominal diameter of the bolt
ε_{lim}	Limit plastic strain
ε_{Pl}	Plastic strain from CBFEM results
f_c	Concrete compressive strength
f_c'	Concrete compressive strength
F_u	Specified minimum tensile strength of the connected material
F_p	Concrete block design bearing strength
$f_{p,max}$	Concrete block design bearing strength maximum
f_y	Yield strength
l_c	Clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material
N	Resulting compression force
σ	Average stress in concrete
σ_{Ed}	Equivalent stress
Φ	Resistance factor
ΦR_n	Factored resistance
t	Thickness of the anchor plate
V	Resultant of shear forces Vy, Vz in bolt.

2.7 Warnings

- By using the CBFEM calculation functionality of PROFIS Engineering you may act outside the applicable design codes and your specified anchor plate may not behave rigid. Please, validate the results with a professional designer and/or structural engineer to ensure suitability and adequacy for your specific jurisdiction and project requirements.
- The anchor is modeled considering stiffness values determined from load displacement curves tested in an independent laboratory. Please note that no simple replacement of the anchor is possible as the anchor stiffness has a major impact on the load distribution results.

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Company:		Page:	14
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/24/2025
Fastening point:			

3 Summary of results

Design of the anchor plate, anchors, welds and other elements are based on CBFEM (component based finite element method) and AISC.

	Load combination	Max. utilization	Status
Anchors	Combination 1	210%	NOT OK
Anchor plate	Combination 1	60%	OK
Concrete	Combination 1	94%	OK
Profile	Combination 1	100%	OK

Fastening does not meet the design criteria!

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Company:		Page:	15
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/24/2025
Fastening point:			

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Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/24/2025
Fastening point:			

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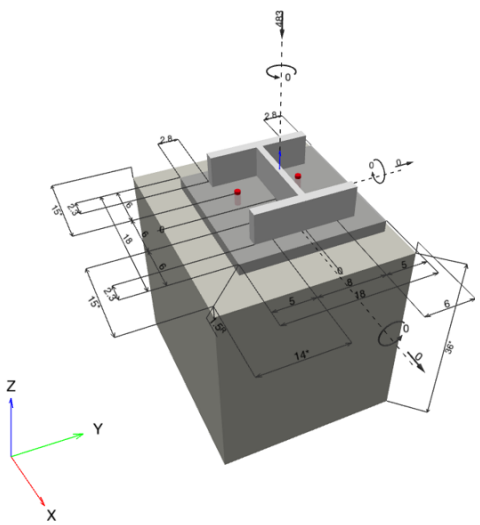
1 Input data



Anchor type and diameter:	**Heavy Hex Head ASTM F 1554 GR. 55 3/4**
Item number:	not available
Specification text:	∅ 3/4 in Heavy Hex Head ASTM F 1554 GR. 55 with 18 in nominal embedment depth per Technical data , cast in place installation per MPII,
Effective embedment depth:	h_{ef} = 18.000 in.
Material:	ASTM F 1554
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-19 / CIP
Shear edge breakout verification:	Row closest to edge (Case 3 only from ACI 318-19 Fig. R.17.7.2.1b)
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 1.500 in.
Anchor plate[R] :	l_x x l_y x t = 18.000 in. x 18.000 in. x 1.500 in.;
Profile:	W shape (AISC), W12X136; (L x W x T x FT) = 13.400 in. x 12.400 in. x 0.790 in. x 1.250 in.
Base material:	cracked concrete, Custom, f_c' = 4,500 psi; h = 36.000 in.
Reinforcement:	tension: not present, shear: not present;
	edge reinforcement: none or < No. 4 bar

[R] - The anchor calculation is based on a rigid anchor plate assumption.

Geometry [in.] & Loading [kip, ft.kip]



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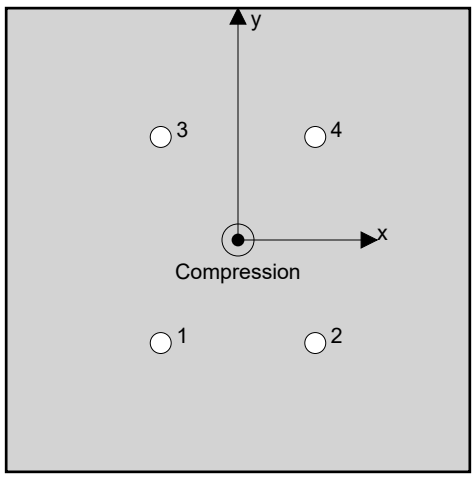
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Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/24/2025
Fastening point:			

1.1 Design results

Case	Description	Forces [kip] / Moments [ft.kip]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = -483.000; V_x = 0.000; V_y = 0.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	-

2 Load case/Resulting anchor forces



Anchor reactions [kip]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	0.000	0.000	0.000	0.000
2	0.000	0.000	0.000	0.000
3	0.000	0.000	0.000	0.000
4	0.000	0.000	0.000	0.000

Max. concrete compressive strain: 0.00 [‰]
Max. concrete compressive stress: 1,491 [psi]
Resulting tension force in (x/y)=(-/-): 0.000 [kip]
Resulting compression force in (x/y)=(0.000/0.000): 483.000 [kip]

Anchor forces are calculated based on the assumption of a rigid anchor plate.

3 Tension load

	Load N_{ua} [kip]	Capacity ϕ N_n [kip]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	N/A	N/A	N/A	N/A
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	N/A	N/A	N/A	N/A
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (anchors in tension)

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	3
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/24/2025
Fastening point:			

4 Shear load

	Load V_{ua} [kip]	Capacity ϕ V_n [kip]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	N/A	N/A	N/A	N/A
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength*	N/A	N/A	N/A	N/A
Concrete edge failure in direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (relevant anchors)

5 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Since no anchor rod forces exist, the anchor rod size can be determined based on the OSHA requirements, and practical considerations.
- The equations presented in this report are based on imperial units. When inputs are displayed in metric units, the user should be aware that the equations remain in their imperial format.
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- For additional information about ACI 318 strength design provisions, please go to https://viewer.joomag.com/profis-design-guide-us-en-summer-2021/08418490016251547 58?short&/

Fastening meets the design criteria!

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Company:		Page:	4
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/24/2025
Fastening point:			

6 Anchor plate and concrete bearing stress check

	Load	Capacity	Utilization [%]	Status
Concentric Compression	12,165.64 [in.lb/in.]	25,312.51 [in.lb/in.]	49	OK
Concrete bearing	1,491 [psi]	2,762 [psi]	54	OK
Tension Interface	N/A	N/A	N/A	N/A
Uniaxial Moment (Strong Axis)	N/A	N/A	N/A	N/A
Uniaxial Moment (Weak Axis)	N/A	N/A	N/A	N/A

6.1 Concrete bearing check (per AISC DG1, section 3.1.1)

$A_1 = B \cdot N$

$A_2 = L \cdot W$

$f_{pu(max)} = \phi \left(0.85 f'_c \sqrt{\frac{A_2}{A_1}} \right) , \sqrt{\frac{A_2}{A_1}} \leq 2$

$f_{pu} = \frac{P_u}{BN}$

$f_{pu} \leq f_{p(max)}$

Variables

B [in.]	N [in.]	L [in.]	W [in.]	f_c' [psi]	ϕ	P_u [kip]
18.000	18.000	20.000	20.000	4,500	0.650	483.000

Calculations

A_1 [in.^{2}]	A_2 [in.^{2}]
324.00	400.00

Results

f_{pu} [psi]	$f_{pu(max)}$ [psi]
1,491	2,762

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	5
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/24/2025
Fastening point:			

6.2 Plate bending, compression (per AISC DG1, sections 3.1.2, 3.1.3)

$A_1 = B \cdot N$

$A_2 = L \cdot W$

$P_p = 0.85 f'_c A_1 \sqrt{\frac{A_2}{A_1}}, \sqrt{\frac{A_2}{A_1}} \leq 2$

$X = \left\{ \frac{4 d b_f}{(d + b_f)^2} \right\} \frac{P_u}{\phi_c P_p}$

$\lambda = \frac{2\sqrt{X}}{1 + \sqrt{1 - X}} \leq 1$

$m = \frac{N - 0.95d}{2}$

$n = \frac{B - 0.80 b_f}{2}$

$\lambda n' = \lambda \frac{\sqrt{d b_f}}{4}$

$l = \max(m, n, \lambda n')$

$f_{pu} = \frac{P_u}{BN}$

$M_{pl} = \frac{f_{pu} \cdot l^2}{2}$

$\phi M_n = \phi \cdot F_y \cdot \frac{t_p^2}{4}$

$M_{pl} \leq \phi M_n$

Variables

B [in.]	N [in.]	L [in.]	W [in.]	d [in.]
18.000	18.000	20.000	20.000	13.400

b_f [in.]	F_y [psi]	ϕ	t_p [in.]	P_u' [kip]
12.400	50,000	0.900	1.500	483.000

Calculations

A_1 [in.^{2}]	A_2 [in.^{2}]	P_p [kip]	X	λ
324.00	400.00	895.050	0.829	1.000

m [in.]	n [in.]	$\lambda n'$ [in.]	l [in.]	f_{pu} [psi]
2.635	4.040	3.223	4.040	1,491

Results

M_n [in.lb/in.]	ϕ	ϕM_n [in.lb/in.]	M_{pl} [in.lb/in.]
28,125.01	0.900	25,312.51	12,165.64

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Company:		Page:	6
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/24/2025
Fastening point:			

7 Installation data

Profile: W shape (AISC), W12X136; (L x W x T x FT) = 13.400 in. x 12.400 in. x 0.790 in. x 1.250 in.

Hole diameter in the fixture: d_f = 0.812 in.

Plate thickness (input): 1.500 in.

Anchor type and diameter: Heavy Hex Head ASTM F 1554 GR. 55 3/4

Item number: not available

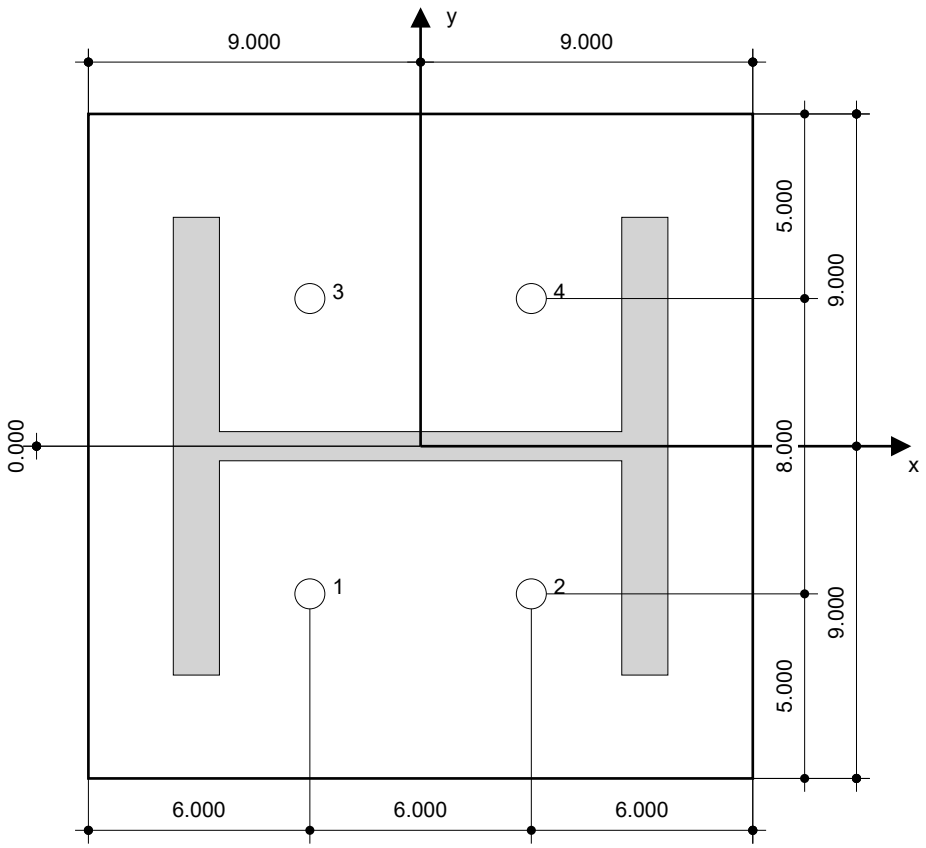
Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 18.000 in.

Minimum thickness of the base material: 19.000 in.

⌀ 3/4 in Heavy Hex Head ASTM F 1554 GR. 55 with 18 in nominal embedment depth per Technical data , cast in place installation per MPII



Coordinates Anchor in.

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-3.000	-4.000	15.000	21.000	14.000	14.000
2	3.000	-4.000	21.000	15.000	14.000	14.000
3	-3.000	4.000	15.000	21.000	22.000	6.000
4	3.000	4.000	21.000	15.000	22.000	6.000

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Company:		Page:	7
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/24/2025
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Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/24/2025
Fastening point:			

Specifier's comments:

1 Anchor Design

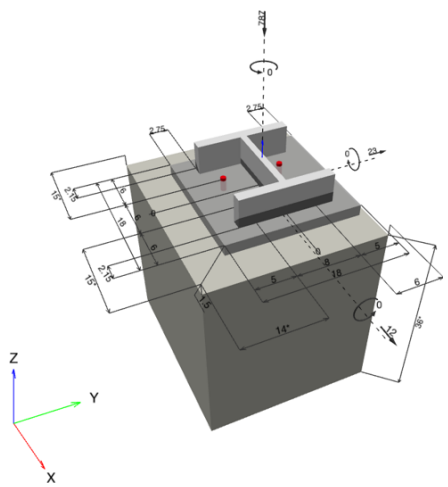
1.1 Input data



Anchor type and diameter:	**Heavy Hex Head ASTM F 1554 GR. 55 3/4**
Item number:	not available
Specification text:	∅ 3/4 in Heavy Hex Head ASTM F 1554 GR. 55 with 18 in nominal embedment depth per Technical data , cast in place installation per MPII,
Effective embedment depth:	h_{ef} = 18.000 in.
Material:	ASTM F 1554
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-19 / CIP
Shear edge breakout verification:	Row closest to edge (Case 3 only from ACI 318-19 Fig. R.17.7.2.1b)
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 1.500 in.
Anchor plate[CBFEM] :	l_x x l_y x t = 18.000 in. x 18.000 in. x 1.500 in.;
Profile:	W shape (AISC), W12X152; (L x W x T x FT) = 13.700 in. x 12.500 in. x 0.870 in. x 1.400 in.
Base material:	cracked concrete, Custom, f_c' = 4,500 psi; h = 36.000 in.
Reinforcement:	tension: not present, shear: not present; edge reinforcement: none or < No. 4 bar

[CBFEM] - The anchor calculation is based on a component-based Finite Element Method (CBFEM)

Geometry [in.] & Loading [kip, ft.kip]



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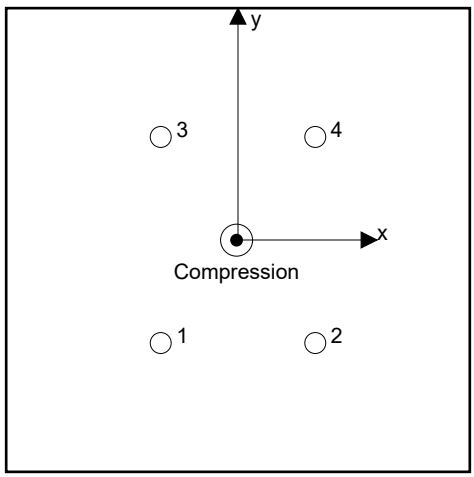
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Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/24/2025
Fastening point:			

1.1.1 Design results

Case	Description	Forces [kip] / Moments [ft.kip]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = -787.000; V_x = 12.000; V_y = 23.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	314

1.2 Load case/Resulting anchor forces



Anchor reactions [kip]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	-0.008	6.364	3.003	5.611
2	-0.008	6.359	2.995	5.610
3	-0.008	6.623	3.038	5.885
4	-0.008	6.597	2.965	5.894

Resulting tension force in (x/y)=(-/-): 0.000 [kip]
Resulting compression force in (x/y)=(-0.055/-0.007): 787.002 [kip]

Anchor forces are calculated based on a component-based Finite Element Method (CBFEM)

1.3 Tension load

	Load N_{ua} [kip]	Capacity ϕ N_n [kip]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	-0.008	18.787	1	OK
Pullout Strength*	-0.008	22.957	-0	not recommended
Concrete Breakout Failure**	N/A	N/A	N/A	N/A
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (anchors in tension)

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	3
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/24/2025
Fastening point:			

1.3.1 Steel Strength

$N_{sa} = A_{se,N} f_{uta}$ ACI 318-19 Eq. (17.6.1.2)
$\phi N_{sa} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.33	75,000

Calculations

N_{sa} [kip]
25.050

Results

N_{sa} [kip]	ϕ_{steel}	ϕN_{sa} [kip]	N_{ua} [kip]
25.050	0.750	18.787	-0.008

The steel proof was done for the highest absolute force per anchor - in this case compression loading. Please be aware that buckling should be verified separately

1.3.2 Pullout Strength

$N_{pN} = \psi_{c,p} N_p$ ACI 318-19 Eq. (17.6.3.1)
$N_p = 8 A_{brg} f'_c$ ACI 318-19 Eq. (17.6.3.2.2a)
$\phi N_{pN} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$\psi_{c,p}$	A_{brg} [in.^{2}]	λ_a	f'_c [psi]
1.000	0.91	1.000	4,500

Calculations

N_p [kip]
32.796

Results

N_{pn} [kip]	$\phi_{concrete}$	ϕN_{pn} [kip]	N_{ua} [kip]
32.796	0.700	22.957	-0.008

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Company:		Page:	4
Address:		Specifier:	
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Design:	Orange BP (~800 kips)	Date:	4/24/2025
Fastening point:			

1.4 Shear load

	Load V_{ua} [kip]	Capacity ϕ V_n [kip]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	6.623	9.769	68	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	25.942	65.460	40	OK
Concrete edge failure in direction y+**	25.942	8.281	~~314~~	not recommended

* highest loaded anchor **anchor group (relevant anchors)

SEE SUPPLEMENTAL CALCULATION FOR PEDESTAL REINFORCEMENT

1.4.1 Steel Strength

$V_{sa} = 0.6\ A_{se,V}\ f_{uta}$ ACI 318-19 Eq. (17.7.1.2b)
$\phi\ V_{steel} \geq V_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.33	75,000

Calculations

V_{sa} [kip]
15.030

Results

V_{sa} [kip]	ϕ_{steel}	$\phi\ V_{sa}$ [kip]	V_{ua} [kip]
15.030	0.650	9.769	6.623

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	5
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/24/2025
Fastening point:			

1.4.2 Pryout Strength

$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \right]$ — ACI 318-19 Eq. (17.7.3.1b)

$\phi V_{cpg} \geq V_{ua}$ — ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9 h_{ef}^2$ — ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e'_N}{3 h_{ef}}} \right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ — ACI 318-19 Eq. (17.6.2.2.1)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	10.000	0.000	0.000	6.000

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.000	-	24	1.000	4,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [kip]
1,008.00	900.00	1.000	1.000	0.820	1.000	50.912

Results

V_{cpg} [kip]	$\phi_{concrete}$	ϕV_{cpg} [kip]	V_{ua} [kip]
93.515	0.700	65.460	25.942

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Company:		Page:	6
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/24/2025
Fastening point:			

1.4.3 Concrete edge failure in direction y+

$V_{cbg} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \Psi_{ec,V} \Psi_{ed,V} \Psi_{c,V} \Psi_{h,V} \Psi_{parallel,V} V_b$ — ACI 318-19 Eq. (17.7.2.1b)

$\phi V_{cbg} \geq V_{ua}$ — ACI 318-19 Table 17.5.2

A_{Vc} see ACI 318-19, Section 17.7.2.1, Fig. R 17.7.2.1(b)*

$A_{Vc0} = 4.5 c_{a1}^2$ — ACI 318-19 Eq. (17.7.2.1.3)

$\Psi_{ec,V} = \left(\frac{1}{1 + \frac{e'_v}{1.5c_{a1}}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.3.1)

$\Psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.4.1b)

$\Psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ — ACI 318-19 Eq. (17.7.2.6.1)

$V_b = 9 \lambda_a \sqrt{f'_c} c_{a1}^{1.5}$ — ACI 318-19 Eq. (17.7.2.2.1b)

Variables

c_{a1} [in.]	c_{a2} [in.]	e_{cV} [in.]	$\Psi_{c,V}$	h_a [in.]
6.000	15.000	0.001	1.000	36.000

l_e [in.]	λ_a	d_a [in.]	f'_c [psi]	$\Psi_{parallel,V}$
6.000	1.000	0.750	4,500	1.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{h,V}$	V_b [kip]
216.00	162.00	1.000	1.000	1.000	8.873

Results

V_{cbg} [kip]	$\phi_{concrete}$	ϕV_{cbg} [kip]	V_{ua} [kip]
11.830	0.700	8.281	25.942

*Anchor row defined by: Anchor 3, 4; Case 3 controls

1.5 Combined tension and shear loads, per ACI 318-19 section 17.8

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.000	3.133	1.000	262	not recommended

$\beta_{NV} = (\beta_N + \beta_V) / 1.2 <= 1$

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Company:		Page:	7
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/24/2025
Fastening point:			

1.6 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates as per current regulations (ETAG 001/Annex C, EOTA TR029, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid base plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- The equations presented in this report are based on imperial units. When inputs are displayed in metric units, the user should be aware that the equations remain in their imperial format.
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- For additional information about ACI 318 strength design provisions, please go to https://viewer.joomag.com/profis-design-guide-us-en-summer-2021/0841849001625154758?short&/
- Attention! In case of compressive anchor forces a buckling check as well as the proof of the local load transfer into and within the base material (incl. punching) has to be done separately.
- The anchor design methods in PROFIS Engineering require rigid anchor plates, as per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means that the anchor plate should be sufficiently rigid to prevent load re-distribution to the anchors due to elastic/plastic displacements. The user accepts that the anchor plate is considered close to rigid by engineering judgment."

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Company:		Page:	8
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/24/2025
Fastening point:			

1.7 Installation data

Profile: W shape (AISC), W12X152; (L x W x T x FT) = 13.700 in. x 12.500 in. x 0.870 in. x 1.400 in.

Hole diameter in the fixture: d_f = 0.812 in.

Plate thickness (input): 1.500 in.

Anchor type and diameter: Heavy Hex Head ASTM F 1554 GR. 55 3/4

Item number: not available

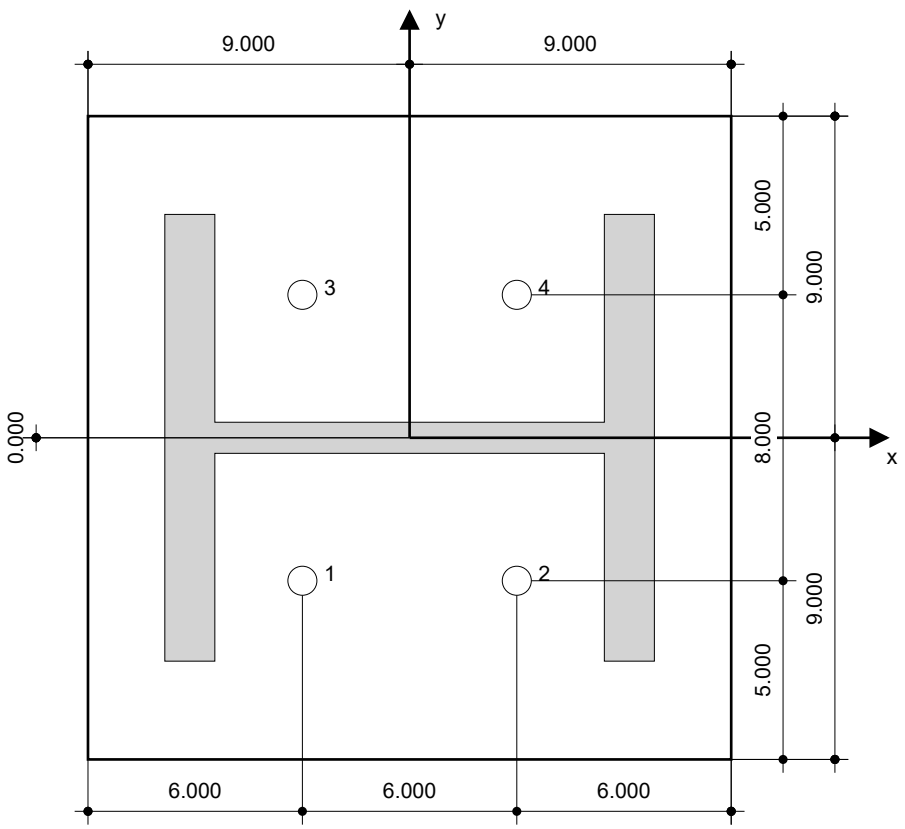
Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 18.000 in.

Minimum thickness of the base material: 19.000 in.

⌀ 3/4 in Heavy Hex Head ASTM F 1554 GR. 55 with 18 in nominal embedment depth per Technical data , cast in place installation per MPII



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-3.000	-4.000	15.000	21.000	14.000	14.000
2	3.000	-4.000	21.000	15.000	14.000	14.000
3	-3.000	4.000	15.000	21.000	22.000	6.000
4	3.000	4.000	21.000	15.000	22.000	6.000

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	9
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/24/2025
Fastening point:			

2 Anchor plate design

2.1 Input data

Anchor plate:	Shape: Rectangular l_x x l_y x t = 18.000 in x 18.000 in x 1.500 in Calculation: CBFEM Material: ASTM A572 Gr.50; F_y = 50,000 psi; ε_{lim} = 5.00%
Anchor type and size:	Heavy Hex Head ASTM F 1554 GR. 55 3/4, h_{ef} = 18.000 in
Anchor stiffness:	The anchor is modeled considering stiffness values determined from load displacement curves tested in an independent laboratory. Please note that no simple replacement of the anchor is possible as the anchor stiffness has a major impact on the load distribution results.
Design method:	AISC and LRFD-based design using component-based FEM
Stand-off installation:	e_b = 0.000 in (No stand-off); t = 1.500 in
Profile:	W12X152; (L x W x T x FT) = 13.700 in x 12.500 in x 0.870 in x 1.400 in Material: ASTM A992; F_y = 50,000 psi; ε_{lim} = 5.00% Eccentricity x: 0.000 in Eccentricity y: 0.000 in
Base material:	Cracked concrete; Custom; $f_{c,cyl}$ = 4,500 psi; h = 36.000 in; E = 4,351,132 psi; G = 1,740,453 psi; v = 0.20; D = 156.07 lb/ft³
Welds (profile to anchor plate):	Type of redistribution: Plastic Material: E70xx
Mesh size:	Number of elements on edge: 8 Min. size of element: 0.394 in Max. size of element: 1.969 in

2.2 Summary

	Description	Profile		Anchor plate			Concrete [%]
		σ_{Ed} [psi]	ε_{Pl} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Hole bearing [%]	
1	Combination 1	40,828	0.01	27,056	0.00	6	90

2.3 Anchor plate classification

Results below are displayed for the decisive load combinations: Combination 1

Anchor tension forces	Equivalent rigid anchor plate (CBFEM)	Component-based Finite Element Method (CBFEM) anchor plate design
Anchor 1	-0.009 kip	-0.008 kip
Anchor 2	-0.009 kip	-0.008 kip
Anchor 3	-0.009 kip	-0.008 kip
Anchor 4	-0.009 kip	-0.008 kip

User accepted to consider the selected anchor plate as rigid by his/her engineering judgement. This means the anchor design guidelines can be applied.

2.4 Profile/Stiffeners/Plate

Profile and stiffeners are verified at the level of the steel to concrete connection. The connection design does not replace the steel design for critical cross sections, which should be performed outside of PROFIS Engineering.

2.4.1 Equivalent stress and plastic strain

Part	Load combination	Material	f_y [psi]	ε_{lim} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Status
Plate	Combination 1	ASTM A572	50,000	5.00	27,056	0.00	OK

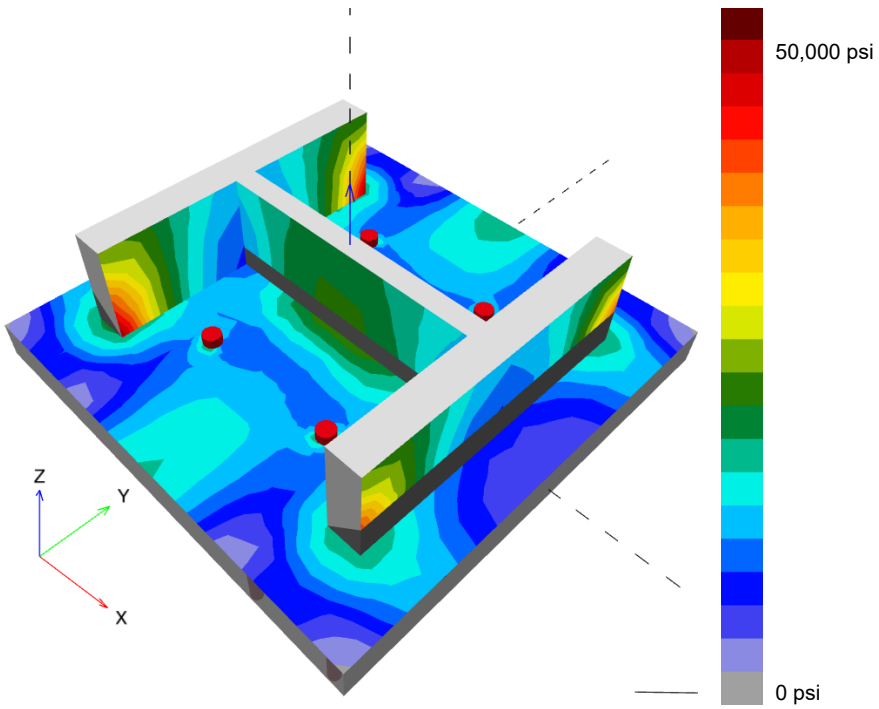
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Company:		Page:	10
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/24/2025
Fastening point:			

Part	Load combination	Material	f_y [psi]	ε_{lim} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Status
		Gr.50					
Profile	Combination 1	ASTM A992	50,000	5.00	40,235	0.00	OK
Profile	Combination 1	ASTM A992	50,000	5.00	40,828	0.01	OK
Profile	Combination 1	ASTM A992	50,000	5.00	23,758	0.00	OK

2.4.1.1 Equivalent stress

Results below are displayed for the decisive load combination: 1 - Combination 1



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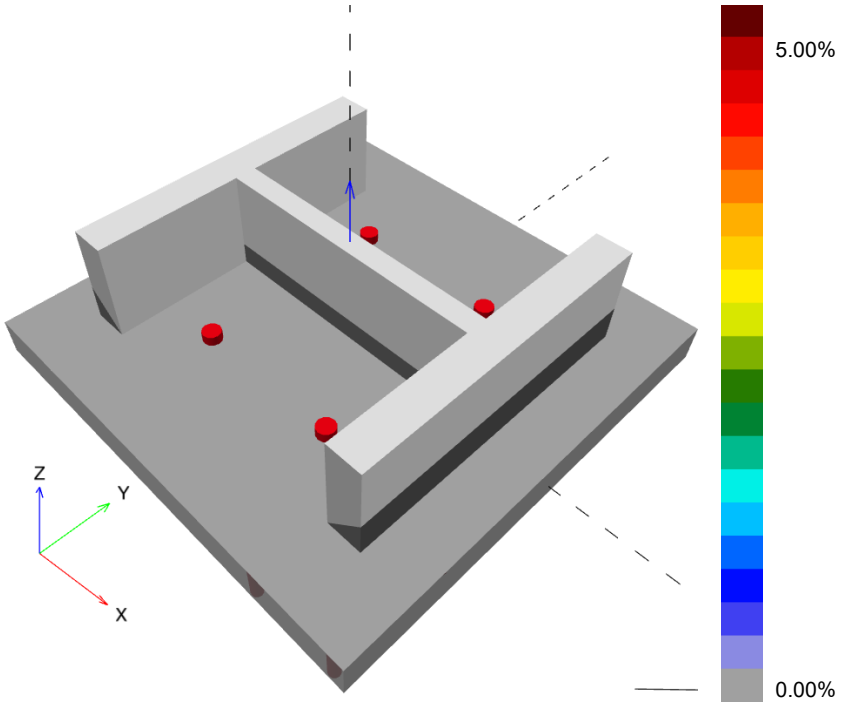
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Company:		Page:	11
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/24/2025
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2.4.1.2 Plastic strain

Results below are displayed for the decisive load combination: 1 - Combination 1



2.4.2 Plate hole bearing resistance, AISC 360-16 Section J3

Decisive load combination: 1 - Combination 1

Equations

$R_n = \min(1.2\, l_c\, t\, F_u\, ,\ 2.4\, d\, t\, F_u)$ (AISC 360-16 J3-6a, c)

$\Phi R_n = 0.75\, R_n$

$V \leq \Phi R_n$

Variables

	l_c [in]	t [in]	F_u [psi]	d [in]	R_n [kip]
Anchor 1	5.265	1.500	65,000	0.750	175.500
Anchor 2	5.261	1.500	65,000	0.750	175.500
Anchor 3	12.674	1.500	65,000	0.750	175.500
Anchor 4	14.146	1.500	65,000	0.750	175.500

Results

	V [kip]	ΦR_n [kip]	Utilization [%]	Status
Anchor 1	6.364	131.625	5	OK
Anchor 2	6.359	131.625	5	OK
Anchor 3	6.623	131.625	6	OK

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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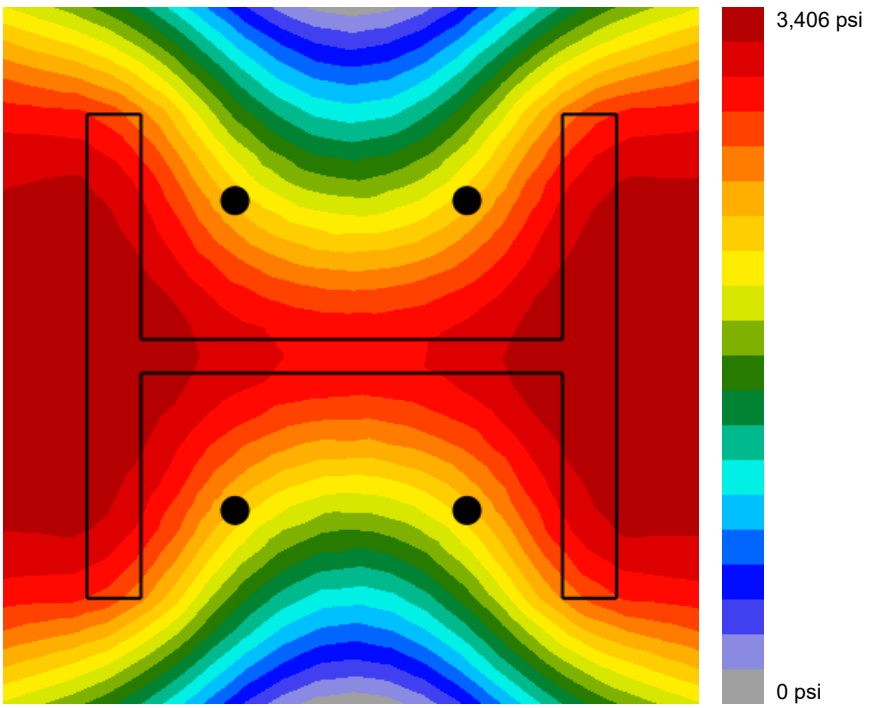
Company:		Page:	12
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/24/2025
Fastening point:			

	V [kip]	ΦR_n [kip]	Utilization [%]	Status
Anchor 4	6.597	131.625	6	OK

2.5 Concrete

Decisive load combination: 1 - Combination 1

2.5.1 Compression in concrete under the anchor plate



2.5.2 Concrete block compressive strength resistance check, AISC 360-16 Section J8

Equations

$F_p = \Phi f_{p,max}$

$f_{p,max} = 0.85 f_c' \sqrt{(\frac{A_2}{A_1})} \le 1.7 f_c; \quad \sqrt{(\frac{A_2}{A_1})} \le 2$

$\sigma = \frac{N}{A_1}$

$\text{Utilization} = \frac{\sigma}{F_p}$

Variables

N [kip]	f_c' [psi]	Φ	A_1 [in²]	A_2 [in²]
787.002	4,500	0.65	318.25	396.01

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Company:		Page:	13
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/24/2025
Fastening point:			

Results

Load combination	F_p [psi]	σ [psi]	Utilization [%]	Status
Combination 1	2,773	2,473	90	OK

2.6 Symbol explanation

A_1	Loaded area of concrete
A_2	Supporting area
d	Nominal diameter of the bolt
ε_{lim}	Limit plastic strain
ε_{Pl}	Plastic strain from CBFEM results
f_c	Concrete compressive strength
f_c'	Concrete compressive strength
F_u	Specified minimum tensile strength of the connected material
F_p	Concrete block design bearing strength
$f_{p,max}$	Concrete block design bearing strength maximum
f_y	Yield strength
l_c	Clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material
N	Resulting compression force
σ	Average stress in concrete
σ_{Ed}	Equivalent stress
Φ	Resistance factor
ΦR_n	Factored resistance
t	Thickness of the anchor plate
V	Resultant of shear forces Vy, Vz in bolt.

2.7 Warnings

- By using the CBFEM calculation functionality of PROFIS Engineering you may act outside the applicable design codes and your specified anchor plate may not behave rigid. Please, validate the results with a professional designer and/or structural engineer to ensure suitability and adequacy for your specific jurisdiction and project requirements.
- The anchor is modeled considering stiffness values determined from load displacement curves tested in an independent laboratory. Please note that no simple replacement of the anchor is possible as the anchor stiffness has a major impact on the load distribution results.

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Company:		Page:	14
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/24/2025
Fastening point:			

3 Summary of results

Design of the anchor plate, anchors, welds and other elements are based on CBFEM (component based finite element method) and AISC.

	Load combination	Max. utilization	Status
Anchors	Combination 1	314%	NOT OK
Anchor plate	Combination 1	55%	OK
Concrete	Combination 1	90%	OK
Profile	Combination 1	82%	OK

Fastening does not meet the design criteria!

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Company:		Page:	15
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/24/2025
Fastening point:			

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Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/24/2025
Fastening point:			

Specifier's comments:

1 Anchor Design

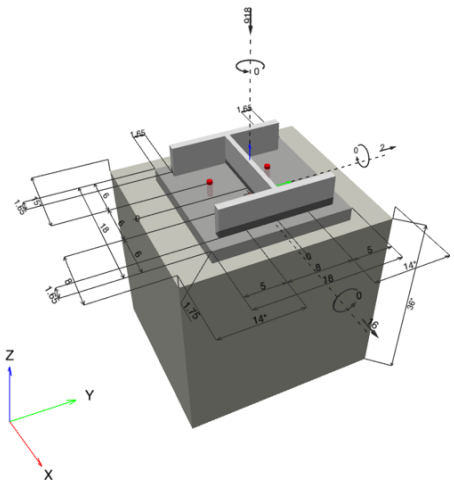
1.1 Input data



Anchor type and diameter:	**Heavy Hex Head ASTM F 1554 GR. 55 3/4**
Item number:	not available
Specification text:	∅ 3/4 in Heavy Hex Head ASTM F 1554 GR. 55 with 18 in nominal embedment depth per Technical data , cast in place installation per MPII,
Effective embedment depth:	h_{ef} = 18.000 in.
Material:	ASTM F 1554
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-19 / CIP
Shear edge breakout verification:	Row closest to edge (Case 3 only from ACI 318-19 Fig. R.17.7.2.1b)
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 1.750 in.
Anchor plate[CBFEM] :	l_x x l_y x t = 18.000 in. x 18.000 in. x 1.750 in.;
Profile:	W shape (AISC), W14X132; (L x W x T x FT) = 14.700 in. x 14.700 in. x 0.645 in. x 1.030 in.
Base material:	cracked concrete, Custom, f_c' = 4,500 psi; h = 36.000 in.
Reinforcement:	tension: not present, shear: not present; edge reinforcement: none or < No. 4 bar

[CBFEM] - The anchor calculation is based on a component-based Finite Element Method (CBFEM)

Geometry [in.] & Loading [kip, ft.kip]



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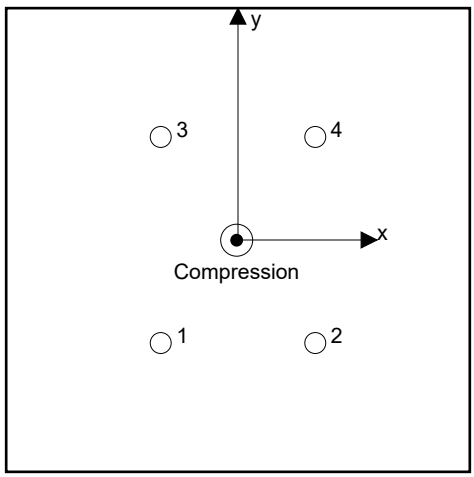
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Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/24/2025
Fastening point:			

1.1.1 Design results

Case	Description	Forces [kip] / Moments [ft.kip]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = -918.000; V_x = 16.000; V_y = 2.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	130

1.2 Load case/Resulting anchor forces



Anchor reactions [kip]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	-0.009	4.054	4.036	0.383
2	-0.009	3.986	3.965	0.403
3	-0.009	4.082	4.035	0.618
4	-0.009	4.008	3.964	0.596

Resulting tension force in (x/y)=(-/-): 0.000 [kip]
Resulting compression force in (x/y)=(-0.051/-0.008): 917.847 [kip]

Anchor forces are calculated based on a component-based Finite Element Method (CBFEM)

1.3 Tension load

	Load N_{ua} [kip]	Capacity ϕ N_n [kip]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	-0.009	18.787	1	OK
Pullout Strength*	-0.009	22.957	-0	not recommended
Concrete Breakout Failure**	N/A	N/A	N/A	N/A
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (anchors in tension)

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	3
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/24/2025
Fastening point:			

1.3.1 Steel Strength

$N_{sa} = A_{se,N} f_{uta}$ ACI 318-19 Eq. (17.6.1.2)
$\phi N_{sa} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.33	75,000

Calculations

N_{sa} [kip]
25.050

Results

N_{sa} [kip]	ϕ_{steel}	ϕN_{sa} [kip]	N_{ua} [kip]
25.050	0.750	18.787	-0.009

The steel proof was done for the highest absolute force per anchor - in this case compression loading. Please be aware that buckling should be verified separately

1.3.2 Pullout Strength

$N_{pN} = \psi_{c,p} N_p$ ACI 318-19 Eq. (17.6.3.1)
$N_p = 8 A_{brg} f'_c$ ACI 318-19 Eq. (17.6.3.2.2a)
$\phi N_{pN} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$\psi_{c,p}$	A_{brg} [in.^{2}]	λ_a	f'_c [psi]
1.000	0.91	1.000	4,500

Calculations

N_p [kip]
32.796

Results

N_{pn} [kip]	$\phi_{concrete}$	ϕN_{pn} [kip]	N_{ua} [kip]
32.796	0.700	22.957	-0.009

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	4
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/24/2025
Fastening point:			

1.4 Shear load

	Load V_{ua} [kip]	Capacity ϕ V_n [kip]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	4.082	9.769	42	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	16.125	71.105	23	OK
Concrete edge failure in direction y+**	16.125	12.447	130	not recommended

* highest loaded anchor **anchor group (relevant anchors)

SEE SUPPLEMENTAL CALCULATION FOR PEDESTAL REINFORCEMENT

1.4.1 Steel Strength

$V_{sa} = 0.6\ A_{se,V}\ f_{uta}$ ACI 318-19 Eq. (17.7.1.2b)
$\phi\ V_{steel} \geq V_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.33	75,000

Calculations

V_{sa} [kip]
15.030

Results

V_{sa} [kip]	ϕ_{steel}	$\phi\ V_{sa}$ [kip]	V_{ua} [kip]
15.030	0.650	9.769	4.082

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	5
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/24/2025
Fastening point:			

1.4.2 Pryout Strength

$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \right]$ ACI 318-19 Eq. (17.7.3.1b)

$\phi V_{cpg} \geq V_{ua}$ ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9 h_{ef}^2$ ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e'_N}{3 h_{ef}}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ ACI 318-19 Eq. (17.6.2.2.1)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	10.000	0.000	0.000	8.000

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.000	-	24	1.000	4,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [kip]
1,044.00	900.00	1.000	1.000	0.860	1.000	50.912

Results

V_{cpg} [kip]	$\phi_{concrete}$	ϕV_{cpg} [kip]	V_{ua} [kip]
101.579	0.700	71.105	16.125

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Company:		Page:	6
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/24/2025
Fastening point:			

1.4.3 Concrete edge failure in direction y+

$V_{cbg} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \Psi_{ec,V} \Psi_{ed,V} \Psi_{c,V} \Psi_{h,V} \Psi_{parallel,V} V_b$ — ACI 318-19 Eq. (17.7.2.1b)

$\phi V_{cbg} \geq V_{ua}$ — ACI 318-19 Table 17.5.2

A_{Vc} see ACI 318-19, Section 17.7.2.1, Fig. R 17.7.2.1(b)*

$A_{Vc0} = 4.5 c_{a1}^2$ — ACI 318-19 Eq. (17.7.2.1.3)

$\Psi_{ec,V} = \left(\frac{1}{1 + \frac{e'_v}{1.5c_{a1}}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.3.1)

$\Psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.4.1b)

$\Psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ — ACI 318-19 Eq. (17.7.2.6.1)

$V_b = 9 \lambda_a \sqrt{f'_c} c_{a1}^{1.5}$ — ACI 318-19 Eq. (17.7.2.2.1b)

Variables

c_{a1} [in.]	c_{a2} [in.]	e_{cV} [in.]	$\Psi_{c,V}$	h_a [in.]
14.000	8.000	0.000	1.000	36.000

l_e [in.]	λ_a	d_a [in.]	f'_c [psi]	$\Psi_{parallel,V}$
6.000	1.000	0.750	4,500	1.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{h,V}$	V_b [kip]
609.00	882.00	1.000	0.814	1.000	31.626

Results

V_{cbg} [kip]	$\phi_{concrete}$	ϕV_{cbg} [kip]	V_{ua} [kip]
17.781	0.700	12.447	16.125

*Anchor row defined by: Anchor 3, 4; Case 3 controls

1.5 Combined tension and shear loads, per ACI 318-19 section 17.8

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.000	1.295	1.000	108	not recommended

$\beta_{NV} = (\beta_N + \beta_V) / 1.2 <= 1$

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Company:		Page:	7
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/24/2025
Fastening point:			

1.6 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates as per current regulations (ETAG 001/Annex C, EOTA TR029, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid base plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- The equations presented in this report are based on imperial units. When inputs are displayed in metric units, the user should be aware that the equations remain in their imperial format.
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- For additional information about ACI 318 strength design provisions, please go to https://viewer.joomag.com/profis-design-guide-us-en-summer-2021/0841849001625154758?short&/
- Attention! In case of compressive anchor forces a buckling check as well as the proof of the local load transfer into and within the base material (incl. punching) has to be done separately.
- The anchor design methods in PROFIS Engineering require rigid anchor plates, as per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means that the anchor plate should be sufficiently rigid to prevent load re-distribution to the anchors due to elastic/plastic displacements. The user accepts that the anchor plate is considered close to rigid by engineering judgment."

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Company:		Page:	8
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/24/2025
Fastening point:			

1.7 Installation data

Profile: W shape (AISC), W14X132; (L x W x T x FT) = 14.700 in. x 14.700 in. x 0.645 in. x 1.030 in.

Hole diameter in the fixture: d_f = 0.812 in.

Plate thickness (input): 1.750 in.

Anchor type and diameter: Heavy Hex Head ASTM F 1554 GR. 55 3/4

Item number: not available

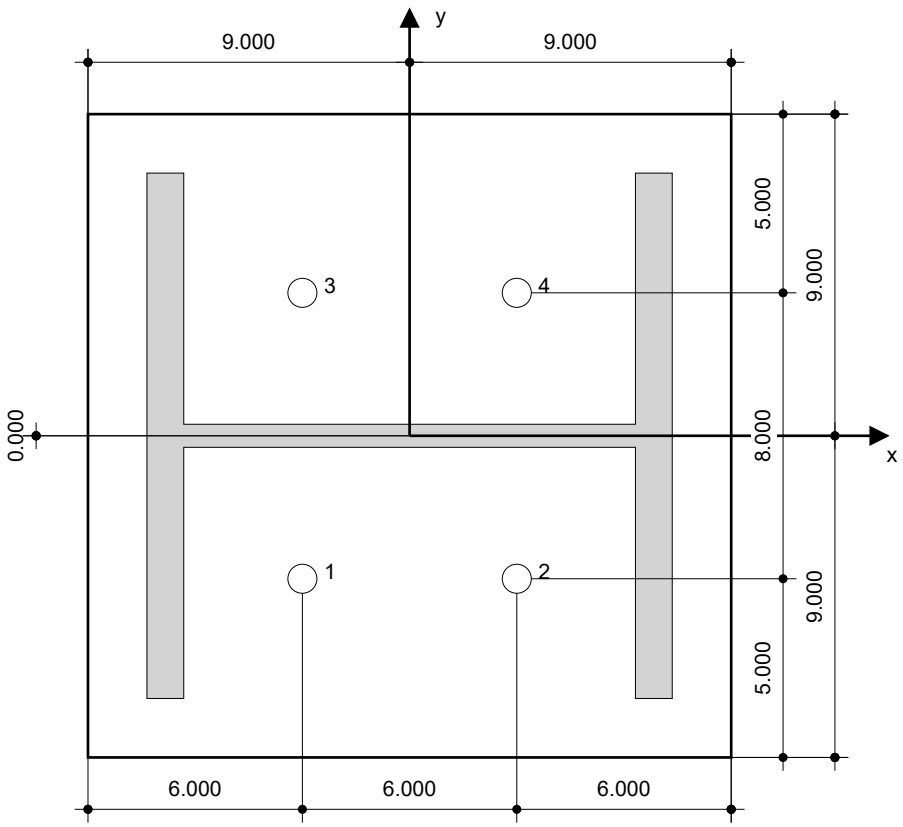
Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 18.000 in.

Minimum thickness of the base material: 19.000 in.

⌀ 3/4 in Heavy Hex Head ASTM F 1554 GR. 55 with 18 in nominal embedment depth per Technical data , cast in place installation per MPII



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-3.000	-4.000	15.000	14.000	14.000	22.000
2	3.000	-4.000	21.000	8.000	14.000	22.000
3	-3.000	4.000	15.000	14.000	22.000	14.000
4	3.000	4.000	21.000	8.000	22.000	14.000

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	9
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/24/2025
Fastening point:			

2 Anchor plate design

2.1 Input data

Anchor plate:	Shape: Rectangular l_x x l_y x t = 18.000 in x 18.000 in x 1.750 in Calculation: CBFEM Material: ASTM A572 Gr.50; F_y = 50,000 psi; ε_{lim} = 5.00%
Anchor type and size:	Heavy Hex Head ASTM F 1554 GR. 55 3/4, h_{ef} = 18.000 in
Anchor stiffness:	The anchor is modeled considering stiffness values determined from load displacement curves tested in an independent laboratory. Please note that no simple replacement of the anchor is possible as the anchor stiffness has a major impact on the load distribution results.
Design method:	AISC and LRFD-based design using component-based FEM
Stand-off installation:	e_b = 0.000 in (No stand-off); t = 1.750 in
Profile:	W14X132; (L x W x T x FT) = 14.700 in x 14.700 in x 0.645 in x 1.030 in Material: ASTM A992; F_y = 50,000 psi; ε_{lim} = 5.00% Eccentricity x: 0.000 in Eccentricity y: 0.000 in
Base material:	Cracked concrete; Custom; $f_{c,cyl}$ = 4,500 psi; h = 36.000 in; E = 4,351,132 psi; G = 1,740,453 psi; v = 0.20; D = 156.07 lb/ft³
Welds (profile to anchor plate):	Type of redistribution: Plastic Material: E70xx
Mesh size:	Number of elements on edge: 8 Min. size of element: 0.394 in Max. size of element: 1.969 in

2.2 Summary

	Description	Profile		Anchor plate			Concrete [%]
		σ_{Ed} [psi]	ε_{Pl} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Hole bearing [%]	
1	Combination 1	46,333	0.02	31,076	0.00	3	95

2.3 Anchor plate classification

Results below are displayed for the decisive load combinations: Combination 1

Anchor tension forces	Equivalent rigid anchor plate (CBFEM)	Component-based Finite Element Method (CBFEM) anchor plate design
Anchor 1	-0.011 kip	-0.009 kip
Anchor 2	-0.011 kip	-0.009 kip
Anchor 3	-0.011 kip	-0.009 kip
Anchor 4	-0.011 kip	-0.009 kip

User accepted to consider the selected anchor plate as rigid by his/her engineering judgement. This means the anchor design guidelines can be applied.

2.4 Profile/Stiffeners/Plate

Profile and stiffeners are verified at the level of the steel to concrete connection. The connection design does not replace the steel design for critical cross sections, which should be performed outside of PROFIS Engineering.

2.4.1 Equivalent stress and plastic strain

Part	Load combination	Material	f_y [psi]	ε_{lim} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Status
Plate	Combination 1	ASTM A572	50,000	5.00	31,076	0.00	OK

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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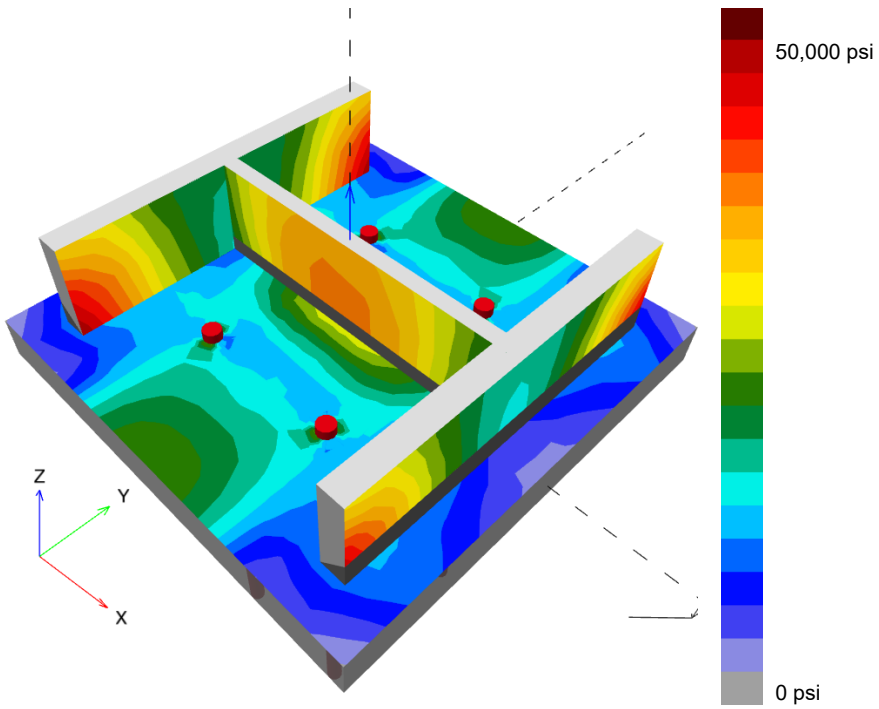
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Company:		Page:	10
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/24/2025
Fastening point:			

Part	Load combination	Material	f_y [psi]	ε_{lim} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Status
		Gr.50					
Profile	Combination 1	ASTM A992	50,000	5.00	45,365	0.02	OK
Profile	Combination 1	ASTM A992	50,000	5.00	46,333	0.02	OK
Profile	Combination 1	ASTM A992	50,000	5.00	39,026	0.00	OK

2.4.1.1 Equivalent stress

Results below are displayed for the decisive load combination: 1 - Combination 1



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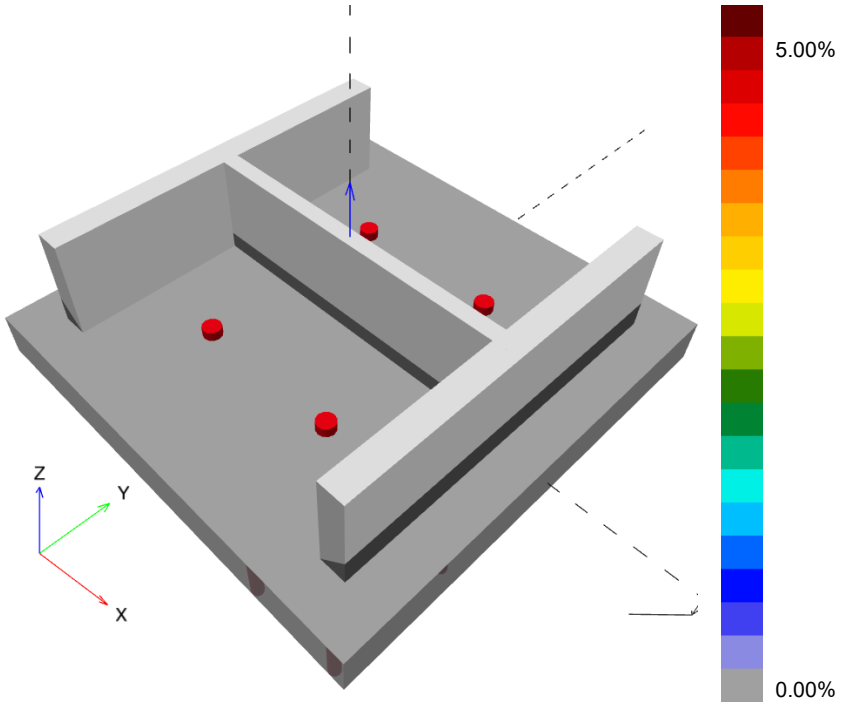
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Company:		Page:	11
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/24/2025
Fastening point:			

2.4.1.2 Plastic strain

Results below are displayed for the decisive load combination: 1 - Combination 1



2.4.2 Plate hole bearing resistance, AISC 360-16 Section J3

Decisive load combination: 1 - Combination 1

Equations

$R_n = \min(1.2\, l_c\, t\, F_u\, ,\ 2.4\, d\, t\, F_u)$ (AISC 360-16 J3-6a, c)

$\Phi R_n = 0.75\, R_n$

$V \leq \Phi R_n$

Variables

	l_c [in]	t [in]	F_u [psi]	d [in]	R_n [kip]
Anchor 1	5.621	1.750	65,000	0.750	204.750
Anchor 2	5.188	1.750	65,000	0.750	204.750
Anchor 3	5.664	1.750	65,000	0.750	204.750
Anchor 4	11.729	1.750	65,000	0.750	204.750

Results

	V [kip]	ΦR_n [kip]	Utilization [%]	Status
Anchor 1	4.054	153.562	3	OK
Anchor 2	3.986	153.562	3	OK
Anchor 3	4.083	153.562	3	OK

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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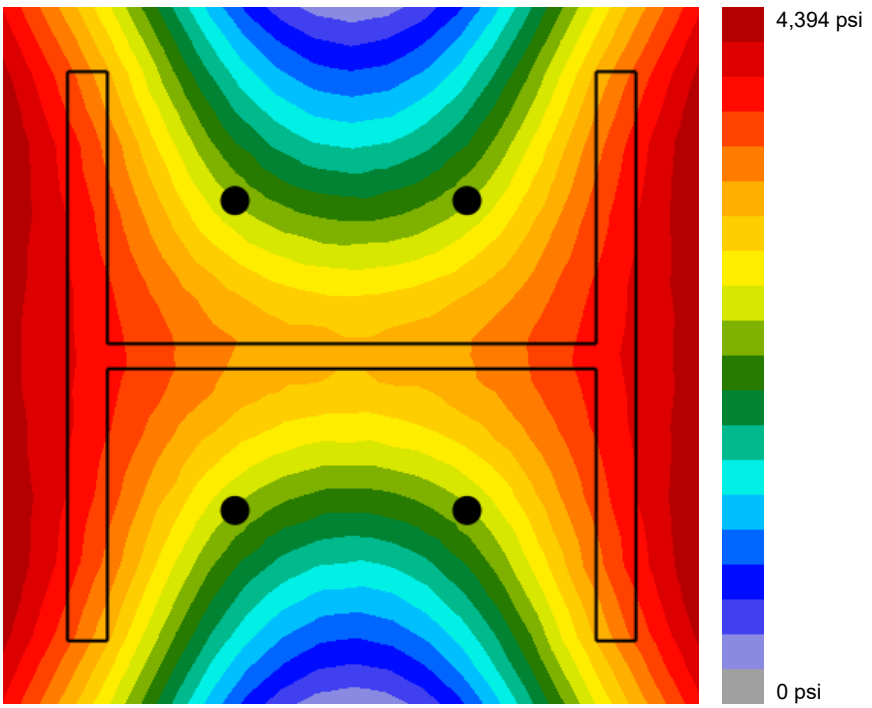
Company:		Page:	12
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/24/2025
Fastening point:			

	V [kip]	ΦR_n [kip]	Utilization [%]	Status
Anchor 4	4.008	153.562	3	OK

2.5 Concrete

Decisive load combination: 1 - Combination 1

2.5.1 Compression in concrete under the anchor plate



2.5.2 Concrete block compressive strength resistance check, AISC 360-16 Section J8

Equations

$F_p = \Phi f_{p,max}$

$f_{p,max} = 0.85 f_c' \sqrt{(\frac{A_2}{A_1})} \le 1.7 f_c;\ \sqrt{(\frac{A_2}{A_1})} \le 2$

$\sigma = \frac{N}{A_1}$

$\text{Utilization} = \frac{\sigma}{F_p}$

Variables

N [kip]	f_c' [psi]	Φ	A_1 [in²]	A_2 [in²]
917.847	4,500	0.65	320.60	480.39

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	13
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/24/2025
Fastening point:			

Results

Load combination	F_p [psi]	σ [psi]	Utilization [%]	Status
Combination 1	3,043	2,863	95	OK

2.6 Symbol explanation

A_1	Loaded area of concrete
A_2	Supporting area
d	Nominal diameter of the bolt
ε_{lim}	Limit plastic strain
ε_{Pl}	Plastic strain from CBFEM results
f_c	Concrete compressive strength
f_c'	Concrete compressive strength
F_u	Specified minimum tensile strength of the connected material
F_p	Concrete block design bearing strength
$f_{p,max}$	Concrete block design bearing strength maximum
f_y	Yield strength
l_c	Clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material
N	Resulting compression force
σ	Average stress in concrete
σ_{Ed}	Equivalent stress
Φ	Resistance factor
ΦR_n	Factored resistance
t	Thickness of the anchor plate
V	Resultant of shear forces Vy, Vz in bolt.

2.7 Warnings

- By using the CBFEM calculation functionality of PROFIS Engineering you may act outside the applicable design codes and your specified anchor plate may not behave rigid. Please, validate the results with a professional designer and/or structural engineer to ensure suitability and adequacy for your specific jurisdiction and project requirements.
- The anchor is modeled considering stiffness values determined from load displacement curves tested in an independent laboratory. Please note that no simple replacement of the anchor is possible as the anchor stiffness has a major impact on the load distribution results.

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	14
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/24/2025
Fastening point:			

3 Summary of results

Design of the anchor plate, anchors, welds and other elements are based on CBFEM (component based finite element method) and AISC.

	Load combination	Max. utilization	Status
Anchors	Combination 1	130%	NOT OK
Anchor plate	Combination 1	63%	OK
Concrete	Combination 1	95%	OK
Profile	Combination 1	93%	OK

Fastening does not meet the design criteria!

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	15
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/24/2025
Fastening point:			

4 Remarks; Your Cooperation Duties

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Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/24/2025
Fastening point:			

Specifier's comments:

1 Anchor Design

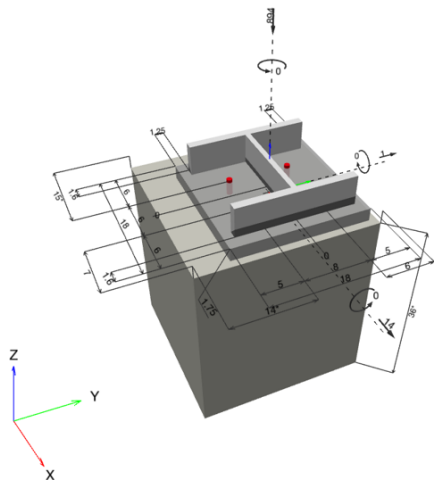
1.1 Input data



Anchor type and diameter:	**Heavy Hex Head ASTM F 1554 GR. 55 3/4**
Item number:	not available
Specification text:	∅ 3/4 in Heavy Hex Head ASTM F 1554 GR. 55 with 18 in nominal embedment depth per Technical data , cast in place installation per MPII,
Effective embedment depth:	h_{ef} = 18.000 in.
Material:	ASTM F 1554
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-19 / CIP
Shear edge breakout verification:	Row closest to edge (Case 3 only from ACI 318-19 Fig. R.17.7.2.1b)
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 1.750 in.
Anchor plate[CBFEM] :	l_x x l_y x t = 18.000 in. x 18.000 in. x 1.750 in.;
Profile:	W shape (AISC), W14X145; (L x W x T x FT) = 14.800 in. x 15.500 in. x 0.680 in. x 1.090 in.
Base material:	cracked concrete, Custom, f_c' = 4,500 psi; h = 36.000 in.
Reinforcement:	tension: not present, shear: not present; edge reinforcement: none or < No. 4 bar

[CBFEM] - The anchor calculation is based on a component-based Finite Element Method (CBFEM)

Geometry [in.] & Loading [kip, ft.kip]



Input data and results must be checked for conformity with the existing conditions and for plausibility!
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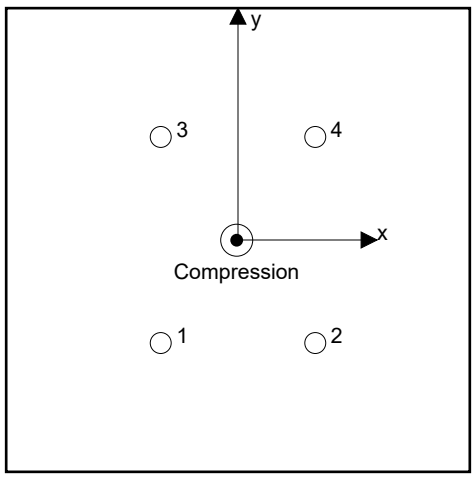
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Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/24/2025
Fastening point:			

1.1.1 Design results

Case	Description	Forces [kip] / Moments [ft.kip]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = -894.000; V_x = 14.000; V_y = 1.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	199

1.2 Load case/Resulting anchor forces



Anchor reactions [kip]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	-0.009	3.542	3.540	0.145
2	-0.009	3.465	3.461	0.164
3	-0.009	3.557	3.539	0.356
4	-0.009	3.477	3.460	0.335

Resulting tension force in (x/y)=(-/-): 0.000 [kip]
Resulting compression force in (x/y)=(-0.050/-0.008): 894.019 [kip]

Anchor forces are calculated based on a component-based Finite Element Method (CBFEM)

1.3 Tension load

	Load N_{ua} [kip]	Capacity ϕ N_n [kip]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	-0.009	18.787	1	OK
Pullout Strength*	-0.009	22.957	-0	not recommended
Concrete Breakout Failure**	N/A	N/A	N/A	N/A
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (anchors in tension)

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Company:		Page:	3
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/24/2025
Fastening point:			

1.3.1 Steel Strength

$N_{sa} = A_{se,N} f_{uta}$ ACI 318-19 Eq. (17.6.1.2)
$\phi N_{sa} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.33	75,000

Calculations

N_{sa} [kip]
25.050

Results

N_{sa} [kip]	ϕ_{steel}	ϕN_{sa} [kip]	N_{ua} [kip]
25.050	0.750	18.787	-0.009

The steel proof was done for the highest absolute force per anchor - in this case compression loading. Please be aware that buckling should be verified separately

1.3.2 Pullout Strength

$N_{pN} = \psi_{c,p} N_p$ ACI 318-19 Eq. (17.6.3.1)
$N_p = 8 A_{brg} f'_c$ ACI 318-19 Eq. (17.6.3.2.2a)
$\phi N_{pN} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$\psi_{c,p}$	A_{brg} [in.^{2}]	λ_a	f'_c [psi]
1.000	0.91	1.000	4,500

Calculations

N_p [kip]
32.796

Results

N_{pn} [kip]	$\phi_{concrete}$	ϕN_{pn} [kip]	N_{ua} [kip]
32.796	0.700	22.957	-0.009

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Company:		Page:	4
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/24/2025
Fastening point:			

1.4 Shear load

	Load V_{ua} [kip]	Capacity ϕ V_n [kip]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	3.557	9.769	37	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	14.036	50.913	28	OK
Concrete edge failure in direction y+**	14.036	7.085	~~199~~	not recommended

* highest loaded anchor **anchor group (relevant anchors)

SEE SUPPLEMENTAL CALCULATION FOR PEDESTAL REINFORCEMENT

1.4.1 Steel Strength

$V_{sa} = 0.6\ A_{se,V}\ f_{uta}$ ACI 318-19 Eq. (17.7.1.2b)
$\phi\ V_{steel} \geq V_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.33	75,000

Calculations

V_{sa} [kip]
15.030

Results

V_{sa} [kip]	ϕ_{steel}	$\phi\ V_{sa}$ [kip]	V_{ua} [kip]
15.030	0.650	9.769	3.557

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Company:		Page:	5
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/24/2025
Fastening point:			

1.4.2 Pryout Strength

$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \right]$ ACI 318-19 Eq. (17.7.3.1b)

$\phi V_{cpg} \geq V_{ua}$ ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9 h_{ef}^2$ ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e'_N}{3 h_{ef}}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ ACI 318-19 Eq. (17.6.2.2.1)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	10.000	0.000	0.000	6.000

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.000	-	24	1.000	4,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [kip]
784.00	900.00	1.000	1.000	0.820	1.000	50.912

Results

V_{cpg} [kip]	$\phi_{concrete}$	ϕV_{cpg} [kip]	V_{ua} [kip]
72.734	0.700	50.913	14.036

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Company:		Page:	6
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/24/2025
Fastening point:			

1.4.3 Concrete edge failure in direction y+

$V_{cbg} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \Psi_{ec,V} \Psi_{ed,V} \Psi_{c,V} \Psi_{h,V} \Psi_{parallel,V} V_b$ — ACI 318-19 Eq. (17.7.2.1b)

$\phi V_{cbg} \geq V_{ua}$ — ACI 318-19 Table 17.5.2

A_{Vc} see ACI 318-19, Section 17.7.2.1, Fig. R 17.7.2.1(b)*

$A_{Vc0} = 4.5 c_{a1}^2$ — ACI 318-19 Eq. (17.7.2.1.3)

$\Psi_{ec,V} = \left(\frac{1}{1 + \frac{e'_v}{1.5c_{a1}}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.3.1)

$\Psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.4.1b)

$\Psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ — ACI 318-19 Eq. (17.7.2.6.1)

$V_b = 9 \lambda_a \sqrt{f'_c} c_{a1}^{1.5}$ — ACI 318-19 Eq. (17.7.2.2.1b)

Variables

c_{a1} [in.]	c_{a2} [in.]	e_{cV} [in.]	$\Psi_{c,V}$	h_a [in.]
6.000	7.000	0.000	1.000	36.000

l_e [in.]	λ_a	d_a [in.]	f'_c [psi]	$\Psi_{parallel,V}$
6.000	1.000	0.750	4,500	1.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{h,V}$	V_b [kip]
198.00	162.00	1.000	0.933	1.000	8.873

Results

V_{cbg} [kip]	$\phi_{concrete}$	ϕV_{cbg} [kip]	V_{ua} [kip]
10.121	0.700	7.085	14.036

*Anchor row defined by: Anchor 3, 4; Case 3 controls

1.5 Combined tension and shear loads, per ACI 318-19 section 17.8

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.000	1.981	1.000	166	not recommended

$\beta_{NV} = (\beta_N + \beta_V) / 1.2 <= 1$

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Company:		Page:	7
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/24/2025
Fastening point:			

1.6 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates as per current regulations (ETAG 001/Annex C, EOTA TR029, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid base plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- The equations presented in this report are based on imperial units. When inputs are displayed in metric units, the user should be aware that the equations remain in their imperial format.
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- For additional information about ACI 318 strength design provisions, please go to https://viewer.joomag.com/profis-design-guide-us-en-summer-2021/0841849001625154758?short&/
- Attention! In case of compressive anchor forces a buckling check as well as the proof of the local load transfer into and within the base material (incl. punching) has to be done separately.
- The anchor design methods in PROFIS Engineering require rigid anchor plates, as per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means that the anchor plate should be sufficiently rigid to prevent load re-distribution to the anchors due to elastic/plastic displacements. The user accepts that the anchor plate is considered close to rigid by engineering judgment."

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Company:		Page:	8
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/24/2025
Fastening point:			

1.7 Installation data

Profile: W shape (AISC), W14X145; (L x W x T x FT) = 14.800 in. x 15.500 in. x 0.680 in. x 1.090 in.

Hole diameter in the fixture: d_f = 0.812 in.

Plate thickness (input): 1.750 in.

Anchor type and diameter: Heavy Hex Head ASTM F 1554 GR. 55 3/4

Item number: not available

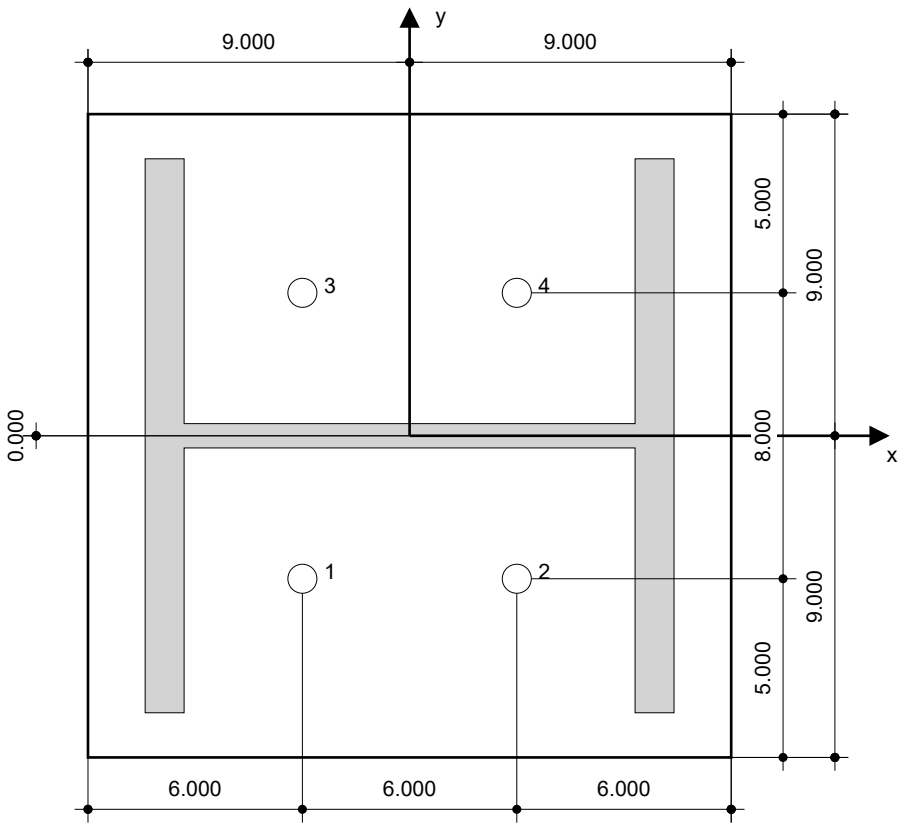
Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 18.000 in.

Minimum thickness of the base material: 19.000 in.

⌀ 3/4 in Heavy Hex Head ASTM F 1554 GR. 55 with 18 in nominal embedment depth per Technical data , cast in place installation per MPII



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-3.000	-4.000	15.000	13.000	14.000	14.000
2	3.000	-4.000	21.000	7.000	14.000	14.000
3	-3.000	4.000	15.000	13.000	22.000	6.000
4	3.000	4.000	21.000	7.000	22.000	6.000

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Company:		Page:	9
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/24/2025
Fastening point:			

2 Anchor plate design

2.1 Input data

Anchor plate:	Shape: Rectangular l_x x l_y x t = 18.000 in x 18.000 in x 1.750 in Calculation: CBFEM Material: ASTM A572 Gr.50; F_y = 50,000 psi; ε_{lim} = 5.00%
Anchor type and size:	Heavy Hex Head ASTM F 1554 GR. 55 3/4, h_{ef} = 18.000 in
Anchor stiffness:	The anchor is modeled considering stiffness values determined from load displacement curves tested in an independent laboratory. Please note that no simple replacement of the anchor is possible as the anchor stiffness has a major impact on the load distribution results.
Design method:	AISC and LRFD-based design using component-based FEM
Stand-off installation:	e_b = 0.000 in (No stand-off); t = 1.750 in
Profile:	W14X145; (L x W x T x FT) = 14.800 in x 15.500 in x 0.680 in x 1.090 in Material: ASTM A992; F_y = 50,000 psi; ε_{lim} = 5.00% Eccentricity x: 0.000 in Eccentricity y: 0.000 in
Base material:	Cracked concrete; Custom; $f_{c,cyl}$ = 4,500 psi; h = 36.000 in; E = 4,351,132 psi; G = 1,740,453 psi; v = 0.20; D = 156.07 lb/ft³
Welds (profile to anchor plate):	Type of redistribution: Plastic Material: E70xx
Mesh size:	Number of elements on edge: 8 Min. size of element: 0.394 in Max. size of element: 1.969 in

2.2 Summary

	Description	Profile		Anchor plate			Concrete [%]
		σ_{Ed} [psi]	ε_{Pl} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Hole bearing [%]	
1	Combination 1	43,609	0.00	29,402	0.00	3	101

2.3 Anchor plate classification

Results below are displayed for the decisive load combinations: Combination 1

Anchor tension forces	Equivalent rigid anchor plate (CBFEM)	Component-based Finite Element Method (CBFEM) anchor plate design
Anchor 1	-0.011 kip	-0.009 kip
Anchor 2	-0.011 kip	-0.009 kip
Anchor 3	-0.011 kip	-0.009 kip
Anchor 4	-0.011 kip	-0.009 kip

User accepted to consider the selected anchor plate as rigid by his/her engineering judgement. This means the anchor design guidelines can be applied.

2.4 Profile/Stiffeners/Plate

Profile and stiffeners are verified at the level of the steel to concrete connection. The connection design does not replace the steel design for critical cross sections, which should be performed outside of PROFIS Engineering.

2.4.1 Equivalent stress and plastic strain

Part	Load combination	Material	f_y [psi]	ε_{lim} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Status
Plate	Combination 1	ASTM A572	50,000	5.00	29,402	0.00	OK

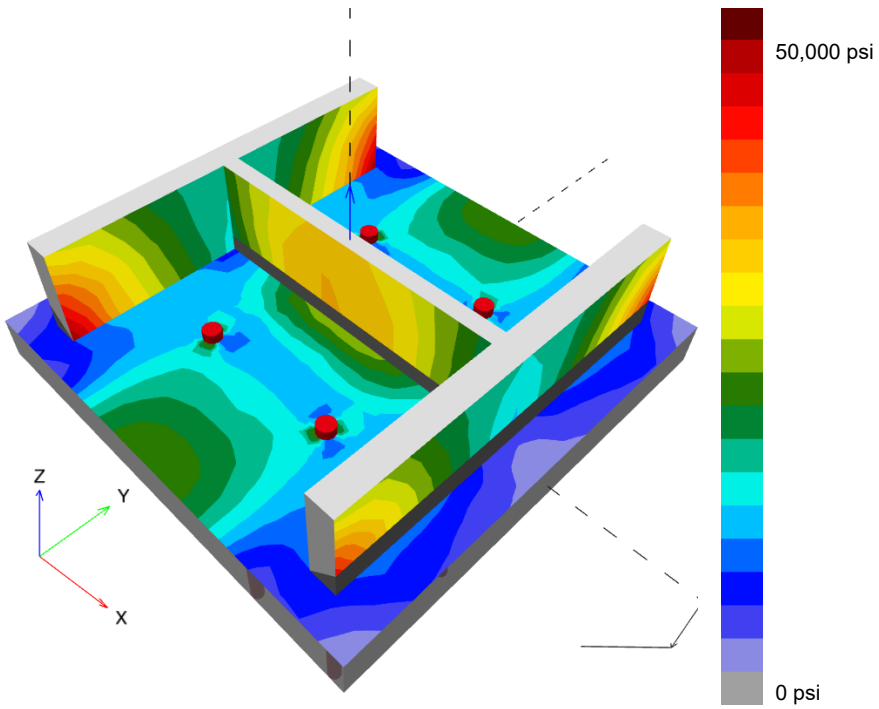
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Company:		Page:	10
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/24/2025
Fastening point:			

Part	Load combination	Material	f_y [psi]	ε_{lim} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Status
		Gr.50					
Profile	Combination 1	ASTM A992	50,000	5.00	42,286	0.00	OK
Profile	Combination 1	ASTM A992	50,000	5.00	43,609	0.00	OK
Profile	Combination 1	ASTM A992	50,000	5.00	35,741	0.00	OK

2.4.1.1 Equivalent stress

Results below are displayed for the decisive load combination: 1 - Combination 1



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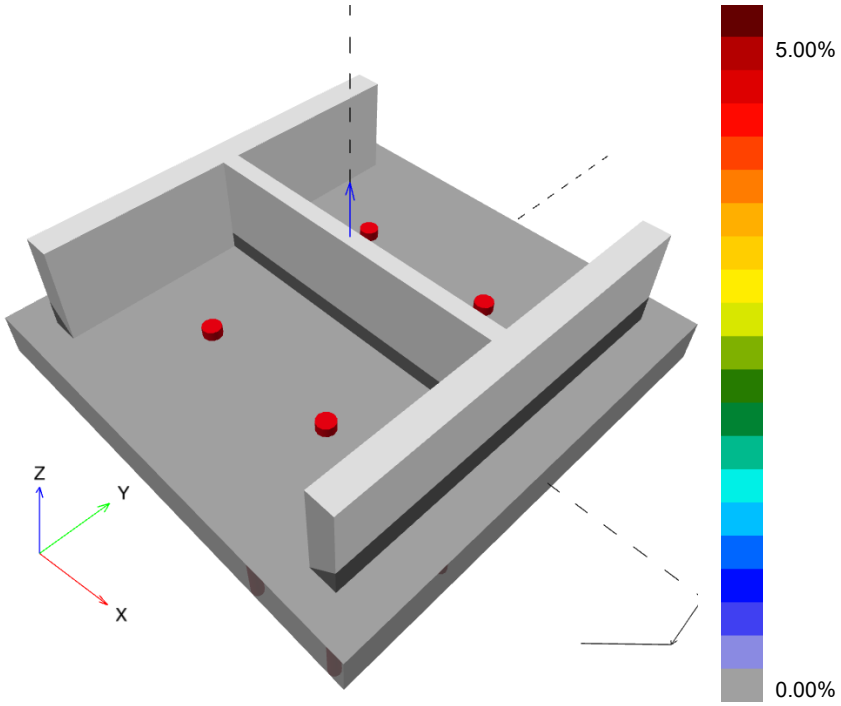
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Company:		Page:	11
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/24/2025
Fastening point:			

2.4.1.2 Plastic strain

Results below are displayed for the decisive load combination: 1 - Combination 1



2.4.2 Plate hole bearing resistance, AISC 360-16 Section J3

Decisive load combination: 1 - Combination 1

Equations

$R_n = \min(1.2\, l_c\, t\, F_u\, ,\ 2.4\, d\, t\, F_u)$ (AISC 360-16 J3-6a, c)

$\Phi R_n = 0.75\, R_n$

$V \leq \Phi R_n$

Variables

	l_c [in]	t [in]	F_u [psi]	d [in]	R_n [kip]
Anchor 1	5.599	1.750	65,000	0.750	204.750
Anchor 2	5.188	1.750	65,000	0.750	204.750
Anchor 3	5.624	1.750	65,000	0.750	204.750
Anchor 4	5.188	1.750	65,000	0.750	204.750

Results

	V [kip]	ΦR_n [kip]	Utilization [%]	Status
Anchor 1	3.543	153.562	3	OK
Anchor 2	3.465	153.562	3	OK
Anchor 3	3.557	153.562	3	OK

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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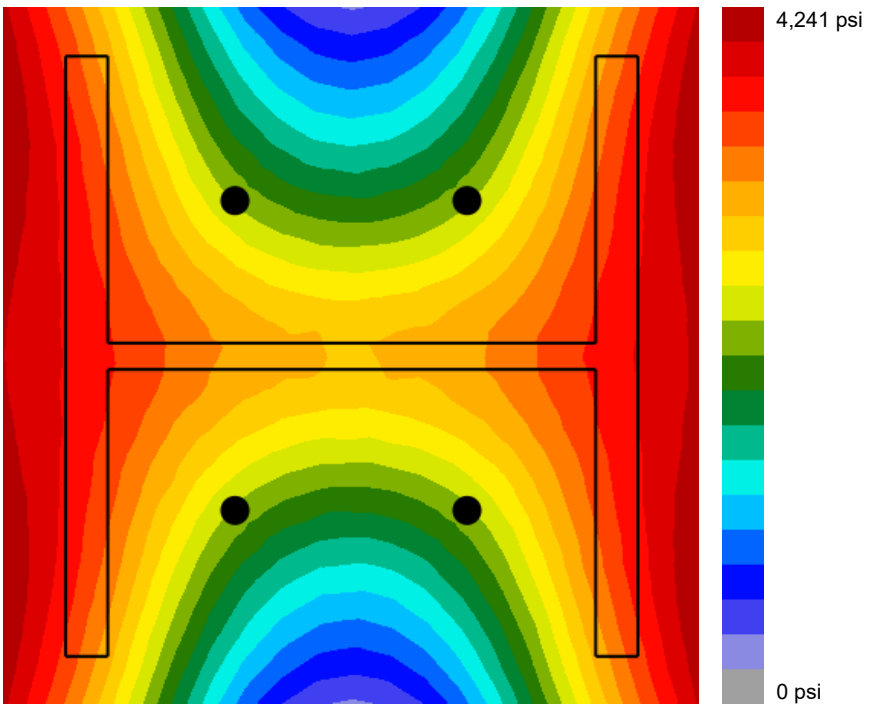
Company:		Page:	12
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/24/2025
Fastening point:			

	V [kip]	ΦR_n [kip]	Utilization [%]	Status
Anchor 4	3.476	153.562	3	OK

2.5 Concrete

Decisive load combination: 1 - Combination 1

2.5.1 Compression in concrete under the anchor plate



2.5.2 Concrete block compressive strength resistance check, AISC 360-16 Section J8

Equations

$$F_p = \Phi f_{p,max}$$

$$f_{p,max} = 0.85 f_c' \sqrt{\left(\frac{A_2}{A_1}\right)} \le 1.7 f_c; \quad \sqrt{\left(\frac{A_2}{A_1}\right)} \le 2$$

$$\sigma = \frac{N}{A_1}$$

$$\text{Utilization} = \frac{\sigma}{F_p}$$

Variables

N [kip]	f_c' [psi]	Φ	A_1 [in²]	A_2 [in²]
894.019	4,500	0.65	322.03	399.25

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	13
Address:		Specifier:	
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Design:	Orange BP (~800 kips)	Date:	4/24/2025
Fastening point:			

Results

Load combination	F_p [psi]	σ [psi]	Utilization [%]	Status
Combination 1	2,768	2,776	101	NOT OK

2.6 Symbol explanation

A_1	Loaded area of concrete
A_2	Supporting area
d	Nominal diameter of the bolt
ε_{lim}	Limit plastic strain
ε_{Pl}	Plastic strain from CBFEM results
f_c	Concrete compressive strength
f_c'	Concrete compressive strength
F_u	Specified minimum tensile strength of the connected material
F_p	Concrete block design bearing strength
$f_{p,max}$	Concrete block design bearing strength maximum
f_y	Yield strength
l_c	Clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material
N	Resulting compression force
σ	Average stress in concrete
σ_{Ed}	Equivalent stress
Φ	Resistance factor
ΦR_n	Factored resistance
t	Thickness of the anchor plate
V	Resultant of shear forces Vy, Vz in bolt.

2.7 Warnings

- By using the CBFEM calculation functionality of PROFIS Engineering you may act outside the applicable design codes and your specified anchor plate may not behave rigid. Please, validate the results with a professional designer and/or structural engineer to ensure suitability and adequacy for your specific jurisdiction and project requirements.
- The anchor is modeled considering stiffness values determined from load displacement curves tested in an independent laboratory. Please note that no simple replacement of the anchor is possible as the anchor stiffness has a major impact on the load distribution results.

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	14
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/24/2025
Fastening point:			

3 Summary of results

Design of the anchor plate, anchors, welds and other elements are based on CBFEM (component based finite element method) and AISC.

	Load combination	Max. utilization	Status
Anchors	Combination 1	199%	NOT OK
Anchor plate	Combination 1	59%	OK
Concrete	Combination 1	101%	NOT OK
Profile	Combination 1	88%	OK

Fastening does not meet the design criteria!

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Company:		Page:	15
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	4/24/2025
Fastening point:			

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Address:		Specifier:	
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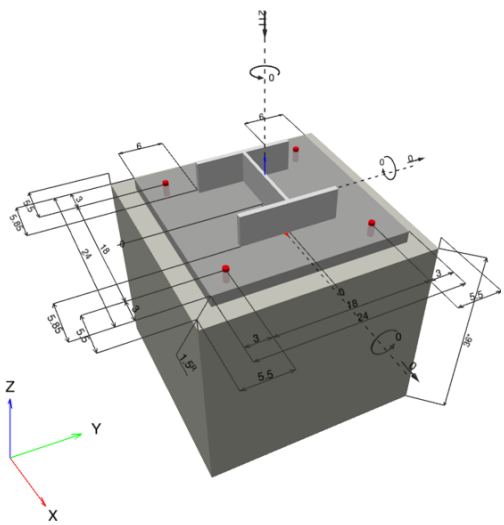
1 Input data



Anchor type and diameter:	**Heavy Hex Head ASTM F 1554 GR. 36 3/4**
Item number:	not available
Specification text:	∅ 3/4 in Heavy Hex Head ASTM F 1554 GR. 36 with 18 in nominal embedment depth per Technical data , cast in place installation per MPII,
Effective embedment depth:	h_{ef} = 18.000 in.
Material:	ASTM F 1554
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-19 / CIP
Shear edge breakout verification:	Row closest to edge (Case 3 only from ACI 318-19 Fig. R.17.7.2.1b)
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 1.500 in.
Anchor plate[R] :	l_x x l_y x t = 24.000 in. x 24.000 in. x 1.500 in.;
Profile:	W shape (AISC), W12X72; (L x W x T x FT) = 12.300 in. x 12.000 in. x 0.430 in. x 0.670 in.
Base material:	cracked concrete, Custom, f_c' = 4,500 psi; h = 36.000 in.
Reinforcement:	tension: not present, shear: not present; edge reinforcement: none or < No. 4 bar

[R] - The anchor calculation is based on a rigid anchor plate assumption.

Geometry [in.] & Loading [kip, ft.kip]



Input data and results must be checked for conformity with the existing conditions and for plausibility!
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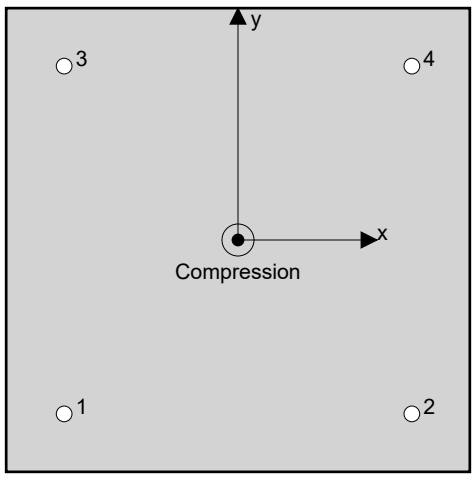
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Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP (~400 kips)	Date:	4/24/2025
Fastening point:			

1.1 Design results

Case	Description	Forces [kip] / Moments [ft.kip]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = -211.000; V_x = 0.000; V_y = 0.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	-

2 Load case/Resulting anchor forces



Anchor reactions [kip]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	0.000	0.000	0.000	0.000
2	0.000	0.000	0.000	0.000
3	0.000	0.000	0.000	0.000
4	0.000	0.000	0.000	0.000

Max. concrete compressive strain:	0.00 [‰]
Max. concrete compressive stress:	366 [psi]
Resulting tension force in (x/y)=(-/-):	0.000 [kip]
Resulting compression force in (x/y)=(0.000/-0.000):	211.000 [kip]

Anchor forces are calculated based on the assumption of a rigid anchor plate.

3 Tension load

	Load N_{ua} [kip]	Capacity ϕ N_n [kip]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	N/A	N/A	N/A	N/A
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	N/A	N/A	N/A	N/A
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (anchors in tension)

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	3
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP (~400 kips)	Date:	4/24/2025
Fastening point:			

4 Shear load

	Load V_{ua} [kip]	Capacity ϕ V_n [kip]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	N/A	N/A	N/A	N/A
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength*	N/A	N/A	N/A	N/A
Concrete edge failure in direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (relevant anchors)

5 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Since no anchor rod forces exist, the anchor rod size can be determined based on the OSHA requirements, and practical considerations.
- The equations presented in this report are based on imperial units. When inputs are displayed in metric units, the user should be aware that the equations remain in their imperial format.
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- For additional information about ACI 318 strength design provisions, please go to https://viewer.joomag.com/profis-design-guide-us-en-summer-2021/08418490016251547 58?short&/

Fastening meets the design criteria!

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Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP (~400 kips)	Date:	4/24/2025
Fastening point:			

6 Anchor plate and concrete bearing stress check

	Load	Capacity	Utilization [%]	Status
Concentric Compression	9,495.00 [in.lb/in.]	25,312.51 [in.lb/in.]	38	OK
Concrete bearing	366 [psi]	3,004 [psi]	13	OK
Tension Interface	N/A	N/A	N/A	N/A
Uniaxial Moment (Strong Axis)	N/A	N/A	N/A	N/A
Uniaxial Moment (Weak Axis)	N/A	N/A	N/A	N/A

6.1 Concrete bearing check (per AISC DG1, section 3.1.1)

$A_1 = B \cdot N$

$A_2 = L \cdot W$

$f_{pu(max)} = \phi \left(0.85 f'_c \sqrt{\frac{A_2}{A_1}} \right) , \sqrt{\frac{A_2}{A_1}} \leq 2$

$f_{pu} = \frac{P_u}{BN}$

$f_{pu} \leq f_{p(max)}$

Variables

B [in.]	N [in.]	L [in.]	W [in.]	f_c' [psi]	ϕ	P_u [kip]
24.000	24.000	29.000	29.000	4,500	0.650	211.000

Calculations

A_1 [in.^{2}]	A_2 [in.^{2}]
576.00	841.00

Results

f_{pu} [psi]	$f_{pu(max)}$ [psi]
366	3,004

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Company:		Page:	5
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP (~400 kips)	Date:	4/24/2025
Fastening point:			

6.2 Plate bending, compression (per AISC DG1, sections 3.1.2, 3.1.3)

$A_1 = B \cdot N$

$A_2 = L \cdot W$

$P_p = 0.85 f'_c A_1 \sqrt{\frac{A_2}{A_1}}, \sqrt{\frac{A_2}{A_1}} \leq 2$

$X = \left\{ \frac{4 d b_f}{(d + b_f)^2} \right\} \frac{P_u}{\phi_c P_p}$

$\lambda = \frac{2\sqrt{X}}{1 + \sqrt{1 - X}} \leq 1$

$m = \frac{N - 0.95d}{2}$

$n = \frac{B - 0.80 b_f}{2}$

$\lambda n' = \lambda \frac{\sqrt{d b_f}}{4}$

$l = \max(m, n, \lambda n')$

$f_{pu} = \frac{P_u}{BN}$

$M_{pl} = \frac{f_{pu} \cdot l^2}{2}$

$\phi M_n = \phi \cdot F_y \cdot \frac{t_p^2}{4}$

$M_{pl} \leq \phi M_n$

Variables

B [in.]	N [in.]	L [in.]	W [in.]	d [in.]
24.000	24.000	29.000	29.000	12.300

b_f [in.]	F_y [psi]	ϕ	t_p [in.]	P_u' [kip]
12.000	50,000	0.900	1.500	211.000

Calculations

A_1 [in.^{2}]	A_2 [in.^{2}]	P_p [kip]	X	λ
576.00	841.00	1,730.430	0.188	0.456

m [in.]	n [in.]	$\lambda n'$ [in.]	l [in.]	f_{pu} [psi]
6.157	7.200	1.384	7.200	366

Results

M_n [in.lb/in.]	ϕ	ϕM_n [in.lb/in.]	M_{pl} [in.lb/in.]
28,125.01	0.900	25,312.51	9,495.00

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Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
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Fastening point:			

7 Installation data

Profile: W shape (AISC), W12X72; (L x W x T x FT) = 12.300 in. x 12.000 in. x 0.430 in. x 0.670 in.

Hole diameter in the fixture: d_f = 0.812 in.

Plate thickness (input): 1.500 in.

Anchor type and diameter: Heavy Hex Head ASTM F 1554 GR. 36 3/4

Item number: not available

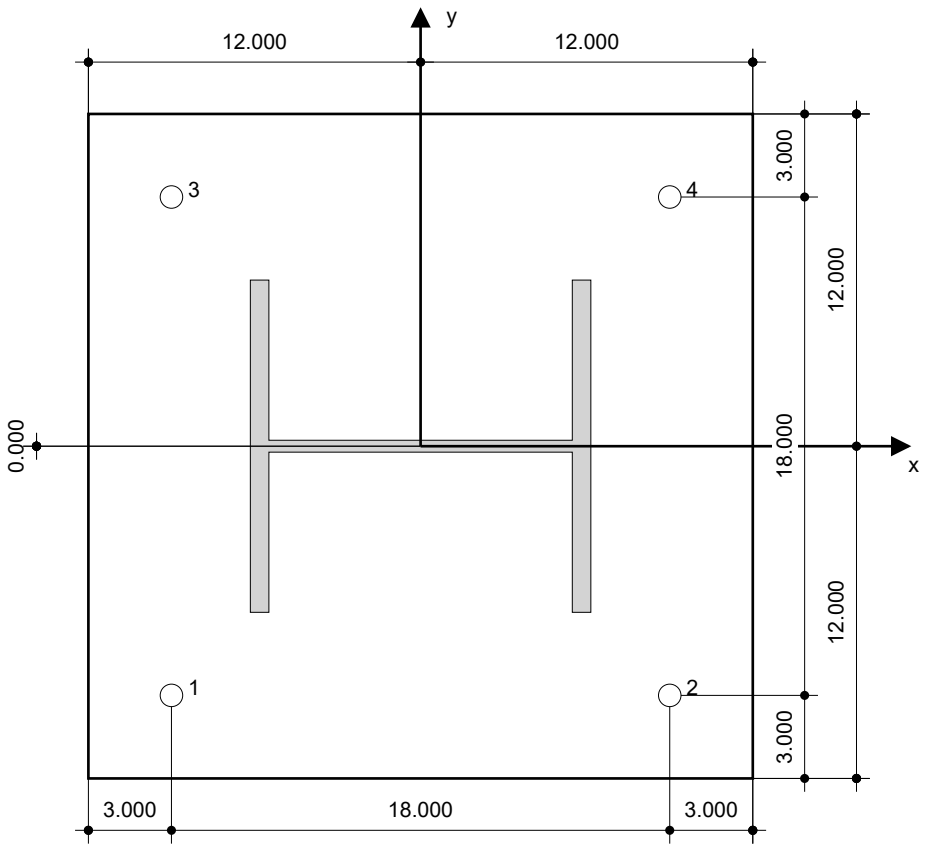
Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 18.000 in.

Minimum thickness of the base material: 19.000 in.

⌀ 3/4 in Heavy Hex Head ASTM F 1554 GR. 36 with 18 in nominal embedment depth per Technical data , cast in place installation per MPII



Coordinates Anchor in.

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-9.000	-9.000	5.500	23.500	5.500	23.500
2	9.000	-9.000	23.500	5.500	5.500	23.500
3	-9.000	9.000	5.500	23.500	23.500	5.500
4	9.000	9.000	23.500	5.500	23.500	5.500

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Company:		Page:	7
Address:		Specifier:	
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Design:	Yellow BP (~400 kips)	Date:	4/24/2025
Fastening point:			

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- You must take all necessary and reasonable steps to prevent or limit damage caused by the Software. In particular, you must arrange for the regular backup of programs and data and, if applicable, carry out the updates of the Software offered by Hilti on a regular basis. If you do not use the AutoUpdate function of the Software, you must ensure that you are using the current and thus up-to-date version of the Software in each case by carrying out manual updates via the Hilti Website. Hilti will not be liable for consequences, such as the recovery of lost or damaged data or programs, arising from a culpable breach of duty by you.

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Address:		Specifier:	
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Fastening point:			

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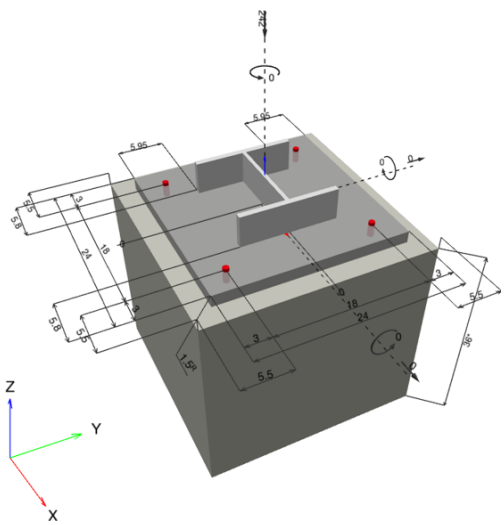
1 Input data



Anchor type and diameter:	**Heavy Hex Head ASTM F 1554 GR. 36 3/4**
Item number:	not available
Specification text:	∅ 3/4 in Heavy Hex Head ASTM F 1554 GR. 36 with 18 in nominal embedment depth per Technical data , cast in place installation per MPII,
Effective embedment depth:	h_{ef} = 18.000 in.
Material:	ASTM F 1554
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-19 / CIP
Shear edge breakout verification:	Row closest to edge (Case 3 only from ACI 318-19 Fig. R.17.7.2.1b)
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 1.500 in.
Anchor plate[R] :	l_x x l_y x t = 24.000 in. x 24.000 in. x 1.500 in.;
Profile:	W shape (AISC), W12X79; (L x W x T x FT) = 12.400 in. x 12.100 in. x 0.470 in. x 0.735 in.
Base material:	cracked concrete, Custom, f_c' = 4,500 psi; h = 36.000 in.
Reinforcement:	tension: not present, shear: not present; edge reinforcement: none or < No. 4 bar

[R] - The anchor calculation is based on a rigid anchor plate assumption.

Geometry [in.] & Loading [kip, ft.kip]



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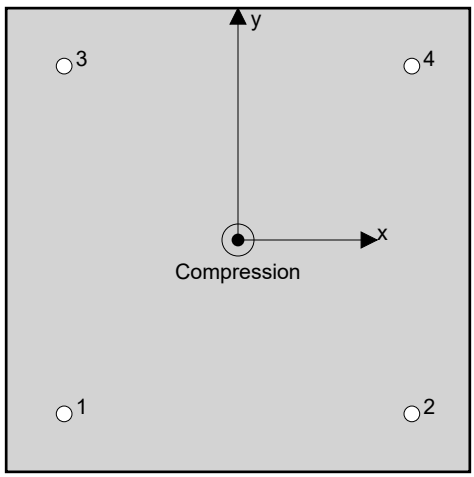
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Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP (~400 kips)	Date:	4/24/2025
Fastening point:			

1.1 Design results

Case	Description	Forces [kip] / Moments [ft.kip]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = -242.000; V_x = 0.000; V_y = 0.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	-

2 Load case/Resulting anchor forces



Anchor reactions [kip]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	0.000	0.000	0.000	0.000
2	0.000	0.000	0.000	0.000
3	0.000	0.000	0.000	0.000
4	0.000	0.000	0.000	0.000

Max. concrete compressive strain:	0.00 [‰]
Max. concrete compressive stress:	420 [psi]
Resulting tension force in (x/y)=(-/-):	0.000 [kip]
Resulting compression force in (x/y)=(0.000/-0.000):	242.000 [kip]

Anchor forces are calculated based on the assumption of a rigid anchor plate.

3 Tension load

	Load N_{ua} [kip]	Capacity ϕ N_n [kip]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	N/A	N/A	N/A	N/A
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	N/A	N/A	N/A	N/A
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (anchors in tension)

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	3
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP (~400 kips)	Date:	4/24/2025
Fastening point:			

4 Shear load

	Load V_{ua} [kip]	Capacity ϕ V_n [kip]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	N/A	N/A	N/A	N/A
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength*	N/A	N/A	N/A	N/A
Concrete edge failure in direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (relevant anchors)

5 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Since no anchor rod forces exist, the anchor rod size can be determined based on the OSHA requirements, and practical considerations.
- The equations presented in this report are based on imperial units. When inputs are displayed in metric units, the user should be aware that the equations remain in their imperial format.
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- For additional information about ACI 318 strength design provisions, please go to https://viewer.joomag.com/profis-design-guide-us-en-summer-2021/08418490016251547 58?short&/

Fastening meets the design criteria!

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Company:		Page:	4
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP (~400 kips)	Date:	4/24/2025
Fastening point:			

6 Anchor plate and concrete bearing stress check

	Load	Capacity	Utilization [%]	Status
Concentric Compression	10,769.34 [in.lb/in.]	25,312.51 [in.lb/in.]	43	OK
Concrete bearing	420 [psi]	3,004 [psi]	14	OK
Tension Interface	N/A	N/A	N/A	N/A
Uniaxial Moment (Strong Axis)	N/A	N/A	N/A	N/A
Uniaxial Moment (Weak Axis)	N/A	N/A	N/A	N/A

6.1 Concrete bearing check (per AISC DG1, section 3.1.1)

$A_1 = B \cdot N$

$A_2 = L \cdot W$

$f_{pu(max)} = \phi \left(0.85 f'_c \sqrt{\frac{A_2}{A_1}} \right) , \sqrt{\frac{A_2}{A_1}} \leq 2$

$f_{pu} = \frac{P_u}{BN}$

$f_{pu} \leq f_{p(max)}$

Variables

B [in.]	N [in.]	L [in.]	W [in.]	f_c' [psi]	ϕ	P_u [kip]
24.000	24.000	29.000	29.000	4,500	0.650	242.000

Calculations

A_1 [in.^{2}]	A_2 [in.^{2}]
576.00	841.00

Results

f_{pu} [psi]	$f_{pu(max)}$ [psi]
420	3,004

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	5
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP (~400 kips)	Date:	4/24/2025
Fastening point:			

6.2 Plate bending, compression (per AISC DG1, sections 3.1.2, 3.1.3)

$A_1 = B \cdot N$

$A_2 = L \cdot W$

$P_p = 0.85 f'_c A_1 \sqrt{\frac{A_2}{A_1}}, \sqrt{\frac{A_2}{A_1}} \leq 2$

$X = \left\{ \frac{4 d b_f}{(d + b_f)^2} \right\} \frac{P_u}{\phi_c P_p}$

$\lambda = \frac{2\sqrt{X}}{1 + \sqrt{1 - X}} \leq 1$

$m = \frac{N - 0.95d}{2}$

$n = \frac{B - 0.80 b_f}{2}$

$\lambda n' = \lambda \frac{\sqrt{d b_f}}{4}$

$l = \max(m, n, \lambda n')$

$f_{pu} = \frac{P_u}{BN}$

$M_{pl} = \frac{f_{pu} \cdot l^2}{2}$

$\phi M_n = \phi \cdot F_y \cdot \frac{t_p^2}{4}$

$M_{pl} \leq \phi M_n$

Variables

B [in.]	N [in.]	L [in.]	W [in.]	d [in.]
24.000	24.000	29.000	29.000	12.400

b_f [in.]	F_y [psi]	ϕ	t_p [in.]	P_u' [kip]
12.100	50,000	0.900	1.500	242.000

Calculations

A_1 [in.^{2}]	A_2 [in.^{2}]	P_p [kip]	X	λ
576.00	841.00	1,730.430	0.215	0.492

m [in.]	n [in.]	$\lambda n'$ [in.]	l [in.]	f_{pu} [psi]
6.110	7.160	1.506	7.160	420

Results

M_n [in.lb/in.]	ϕ	ϕM_n [in.lb/in.]	M_{pl} [in.lb/in.]
28,125.01	0.900	25,312.51	10,769.34

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	6
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP (~400 kips)	Date:	4/24/2025
Fastening point:			

7 Installation data

Profile: W shape (AISC), W12X79; (L x W x T x FT) = 12.400 in. x 12.100 in. x 0.470 in. x 0.735 in.

Hole diameter in the fixture: d_f = 0.812 in.

Plate thickness (input): 1.500 in.

Anchor type and diameter: Heavy Hex Head ASTM F 1554 GR. 36 3/4

Item number: not available

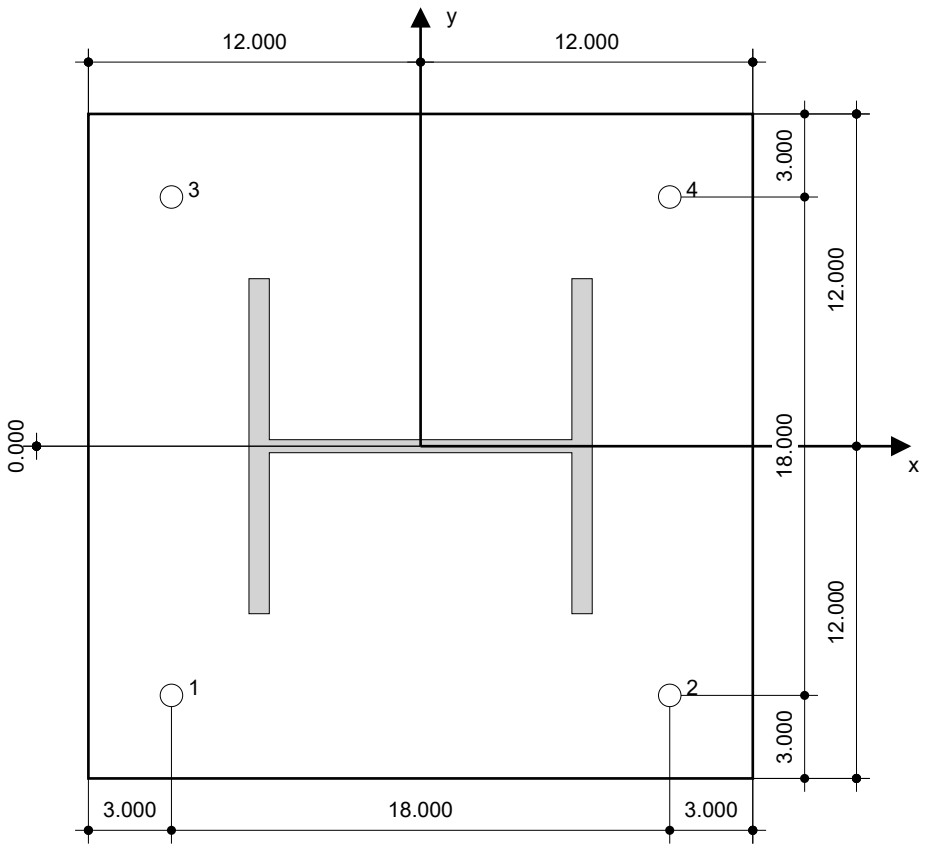
Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 18.000 in.

Minimum thickness of the base material: 19.000 in.

⌀ 3/4 in Heavy Hex Head ASTM F 1554 GR. 36 with 18 in nominal embedment depth per Technical data , cast in place installation per MPII



Coordinates Anchor in.

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-9.000	-9.000	5.500	23.500	5.500	23.500
2	9.000	-9.000	23.500	5.500	5.500	23.500
3	-9.000	9.000	5.500	23.500	23.500	5.500
4	9.000	9.000	23.500	5.500	23.500	5.500

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Company:		Page:	7
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP (~400 kips)	Date:	4/24/2025
Fastening point:			

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Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP (~400 kips)	Date:	1/28/2025
Fastening point:			

Specifier's comments:

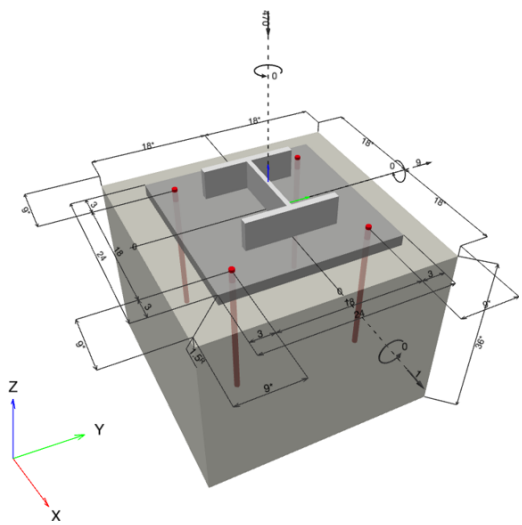
1 Input data



Anchor type and diameter:	**Heavy Hex Head ASTM F 1554 GR. 36 3/4**
Item number:	not available
Specification text:	∅ 3/4 in Heavy Hex Head ASTM F 1554 GR. 36 with 18 in nominal embedment depth per Technical data , cast in place installation per MPII
Effective embedment depth:	h_{ef} = 18.000 in.
Material:	ASTM F 1554
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-19 / CIP
Shear edge breakout verification:	Row closest to edge (Case 3 only from ACI 318-19 Fig. R.17.7.2.1b)
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 1.500 in.
Anchor plate^R :	l_x x l_y x t = 24.000 in. x 24.000 in. x 1.500 in.;
Profile:	W shape (AISC), W12X106; (L x W x T x FT) = 12.900 in. x 12.200 in. x 0.610 in. x 0.990 in.
Base material:	cracked concrete, 4000, f_c' = 4,000 psi; h = 36.000 in.
Reinforcement:	tension: not present, shear: not present;
	edge reinforcement: none or < No. 4 bar

^{R} - The anchor calculation is based on a rigid anchor plate assumption.

Geometry [in.] & Loading [kip, ft.kip]



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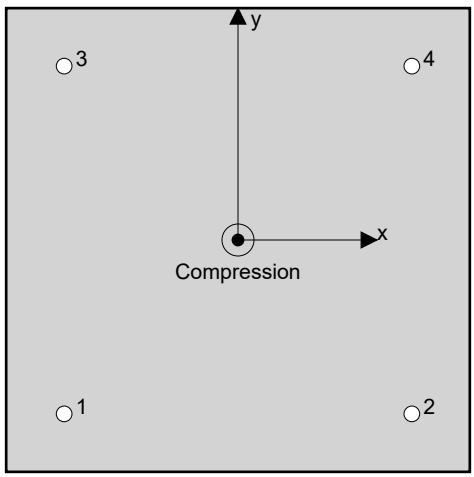
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Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP (~400 kips)	Date:	1/28/2025
Fastening point:			

1.1 Design results

Case	Description	Forces [kip] / Moments [ft.kip]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = -470.000; V_x = 1.000; V_y = 9.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	71

2 Load case/Resulting anchor forces



Anchor reactions [kip]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	0.000	2.264	0.250	2.250
2	0.000	2.264	0.250	2.250
3	0.000	2.264	0.250	2.250
4	0.000	2.264	0.250	2.250

Max. concrete compressive strain:	0.00 [‰]
Max. concrete compressive stress:	816 [psi]
Resulting tension force in (x/y)=(-/-):	0.000 [kip]
Resulting compression force in (x/y)=(0.000/-0.000):	470.000 [kip]

Anchor forces are calculated based on the assumption of a rigid anchor plate.

3 Tension load

	Load N_{ua} [kip]	Capacity ϕ N_n [kip]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	N/A	N/A	N/A	N/A
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	N/A	N/A	N/A	N/A
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (anchors in tension)

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	3
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP (~400 kips)	Date:	1/28/2025
Fastening point:			

4 Shear load

	Load V_{ua} [kip]	Capacity ϕ V_n [kip]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	2.264	7.555	30	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	9.055	124.927	8	OK
Concrete edge failure in direction y+**	9.055	12.910	71	OK

* highest loaded anchor **anchor group (relevant anchors)

4.1 Steel Strength

$V_{sa} = 0.6\ A_{se,V}\ f_{uta}$ ACI 318-19 Eq. (17.7.1.2b)
$\phi\ V_{steel} \geq V_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.33	58,000

Calculations

V_{sa} [kip]
11.623

Results

V_{sa} [kip]	ϕ_{steel}	ϕ V_{sa} [kip]	V_{ua} [kip]
11.623	0.650	7.555	2.264

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	4
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP (~400 kips)	Date:	1/28/2025
Fastening point:			

4.2 Pryout Strength

$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \right]$ ACI 318-19 Eq. (17.7.3.1b)

$\phi V_{cpg} \geq V_{ua}$ ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9 h_{ef}^2$ ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e'_N}{3 h_{ef}}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ ACI 318-19 Eq. (17.6.2.2.1)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	6.000	0.000	0.000	9.000

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.000	-	24	1.000	4,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [kip]
1,296.00	324.00	1.000	1.000	1.000	1.000	22.308

Results

V_{cpg} [kip]	$\phi_{concrete}$	ϕV_{cpg} [kip]	V_{ua} [kip]
178.467	0.700	124.927	9.055

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	5
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP (~400 kips)	Date:	1/28/2025
Fastening point:			

4.3 Concrete edge failure in direction y+

$V_{cbg} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \Psi_{ec,V} \Psi_{ed,V} \Psi_{c,V} \Psi_{h,V} \Psi_{parallel,V} V_b$ — ACI 318-19 Eq. (17.7.2.1b)

$\phi V_{cbg} \geq V_{ua}$ — ACI 318-19 Table 17.5.2

A_{Vc} see ACI 318-19, Section 17.7.2.1, Fig. R 17.7.2.1(b)*

$A_{Vc0} = 4.5 c_{a1}^2$ — ACI 318-19 Eq. (17.7.2.1.3)

$\Psi_{ec,V} = \left(\frac{1}{1 + \frac{e'_v}{1.5c_{a1}}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.3.1)

$\Psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.4.1b)

$\Psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ — ACI 318-19 Eq. (17.7.2.6.1)

$V_b = 9 \lambda_a \sqrt{f'_c} c_{a1}^{1.5}$ — ACI 318-19 Eq. (17.7.2.2.1b)

Variables

c_{a1} [in.]	c_{a2} [in.]	e_{cV} [in.]	$\Psi_{c,V}$	h_a [in.]
9.000	9.000	0.000	1.000	36.000

l_e [in.]	λ_a	d_a [in.]	f'_c [psi]	$\Psi_{parallel,V}$
6.000	1.000	0.750	4,000	1.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{h,V}$	V_b [kip]
486.00	364.50	1.000	0.900	1.000	15.369

Results

V_{cbg} [kip]	$\phi_{concrete}$	ϕV_{cbg} [kip]	V_{ua} [kip]
18.442	0.700	12.910	9.055

*Anchor row defined by: Anchor 3, 4; Case 3 controls

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	6
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP (~400 kips)	Date:	1/28/2025
Fastening point:			

5 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- User is responsible for evaluating the hole bearing capacity in case of shear forces.
- The equations presented in this report are based on imperial units. When inputs are displayed in metric units, the user should be aware that the equations remain in their imperial format.
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- For additional information about ACI 318 strength design provisions, please go to https://submittals.us.hilti.com/PROFISAnchorDesignGuide/

Fastening meets the design criteria!

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Company:		Page:	7
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP (~400 kips)	Date:	1/28/2025
Fastening point:			

6 Anchor plate and concrete bearing stress check

	Load	Capacity	Utilization [%]	Status
Concentric Compression	20,682.61 [in.lb/in.]	25,312.51 [in.lb/in.]	82	OK
Concrete bearing	816 [psi]	3,315 [psi]	25	OK
Tension Interface	N/A	N/A	N/A	N/A
Uniaxial Moment (Strong Axis)	N/A	N/A	N/A	N/A
Uniaxial Moment (Weak Axis)	N/A	N/A	N/A	N/A

6.1 Concrete bearing check (per AISC DG1, section 3.1.1)

$A_1 = B \cdot N$

$A_2 = L \cdot W$

$f_{pu(max)} = \phi \left(0.85 f'_c \sqrt{\frac{A_2}{A_1}} \right) , \sqrt{\frac{A_2}{A_1}} \leq 2$

$f_{pu} = \frac{P_u}{BN}$

$f_{pu} \leq f_{p(max)}$

Variables

B [in.]	N [in.]	L [in.]	W [in.]	f_c' [psi]	ϕ	P_u [kip]
24.000	24.000	36.000	36.000	4,000	0.650	470.000

Calculations

A_1 [in.^{2}]	A_2 [in.^{2}]
576.00	1,296.00

Results

f_{pu} [psi]	$f_{pu(max)}$ [psi]
816	3,315

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	8
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP (~400 kips)	Date:	1/28/2025
Fastening point:			

6.2 Plate bending, compression (per AISC DG1, sections 3.1.2, 3.1.3)

$A_1 = B \cdot N$

$A_2 = L \cdot W$

$P_p = 0.85 f'_c A_1 \sqrt{\frac{A_2}{A_1}}, \sqrt{\frac{A_2}{A_1}} \leq 2$

$X = \left\{ \frac{4 d b_f}{(d + b_f)^2} \right\} \frac{P_u}{\phi_c P_p}$

$\lambda = \frac{2\sqrt{X}}{1 + \sqrt{1 - X}} \leq 1$

$m = \frac{N - 0.95d}{2}$

$n = \frac{B - 0.80 b_f}{2}$

$\lambda n' = \lambda \frac{\sqrt{d b_f}}{4}$

$l = \max(m, n, \lambda n')$

$f_{pu} = \frac{P_u}{BN}$

$M_{pl} = \frac{f_{pu} \cdot l^2}{2}$

$\phi M_n = \phi \cdot F_y \cdot \frac{t_p^2}{4}$

$M_{pl} \leq \phi M_n$

Variables

B [in.]	N [in.]	L [in.]	W [in.]	d [in.]
24.000	24.000	36.000	36.000	12.900

b_f [in.]	F_y [psi]	ϕ	t_p [in.]	P_u' [kip]
12.200	50,000	0.900	1.500	470.000

Calculations

A_1 [in.^{2}]	A_2 [in.^{2}]	P_p [kip]	X	λ
576.00	1,296.00	1,909.440	0.378	0.688

m [in.]	n [in.]	$\lambda n'$ [in.]	l [in.]	f_{pu} [psi]
5.872	7.120	2.157	7.120	816

Results

M_n [in.lb/in.]	ϕ	ϕM_n [in.lb/in.]	M_{pl} [in.lb/in.]
28,125.01	0.900	25,312.51	20,682.61

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	9
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP (~400 kips)	Date:	1/28/2025
Fastening point:			

7 Installation data

Profile: W shape (AISC), W12X106; (L x W x T x FT) = 12.900 in. x 12.200 in. x 0.610 in. x 0.990 in.

Hole diameter in the fixture: d_f = 0.812 in.

Plate thickness (input): 1.500 in.

Anchor type and diameter: Heavy Hex Head ASTM F 1554 GR. 36 3/4

Item number: not available

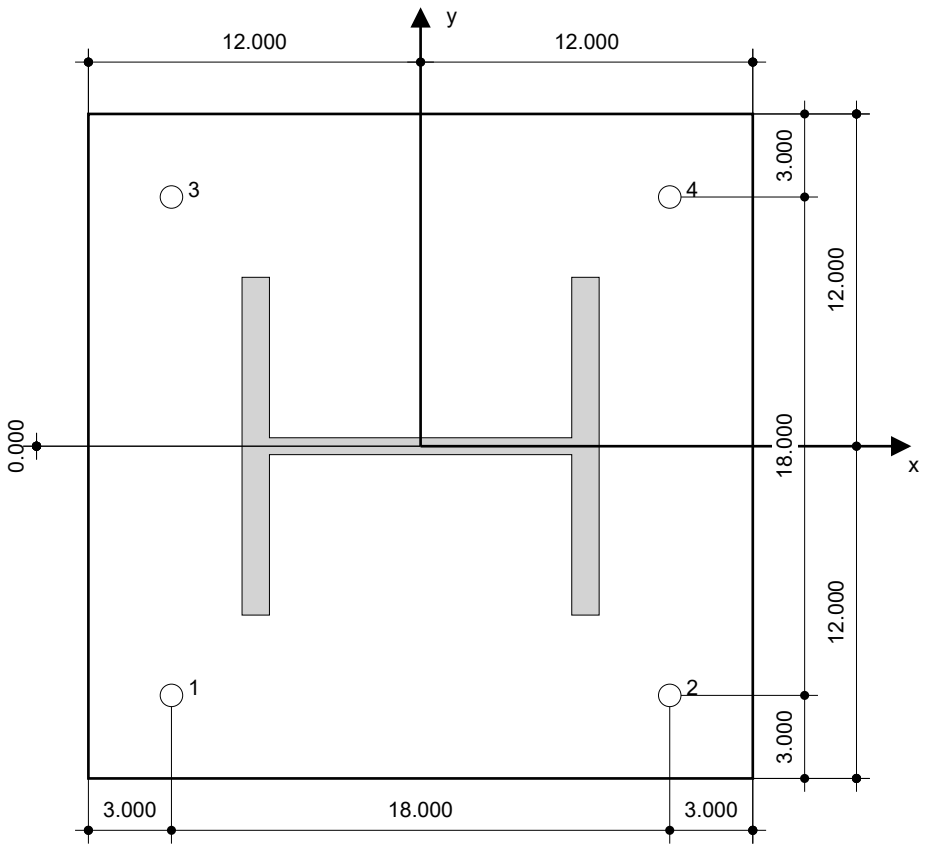
Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 18.000 in.

Minimum thickness of the base material: 19.000 in.

⌀ 3/4 in Heavy Hex Head ASTM F 1554 GR. 36 with 18 in nominal embedment depth per Technical data , cast in place installation per MPII



Coordinates Anchor in.

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-9.000	-9.000	9.000	27.000	9.000	27.000
2	9.000	-9.000	27.000	9.000	9.000	27.000
3	-9.000	9.000	9.000	27.000	27.000	9.000
4	9.000	9.000	27.000	9.000	27.000	9.000

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Company:		Page:	10
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP (~400 kips)	Date:	1/28/2025
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Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
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Fastening point:			

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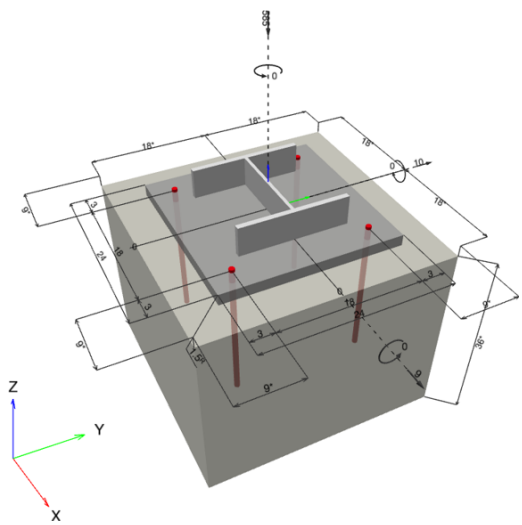
1 Input data



Anchor type and diameter:	**Heavy Hex Head ASTM F 1554 GR. 36 3/4**
Item number:	not available
Specification text:	∅ 3/4 in Heavy Hex Head ASTM F 1554 GR. 36 with 18 in nominal embedment depth per Technical data , cast in place installation per MPII
Effective embedment depth:	h_{ef} = 18.000 in.
Material:	ASTM F 1554
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-19 / CIP
Shear edge breakout verification:	Row closest to edge (Case 3 only from ACI 318-19 Fig. R.17.7.2.1b)
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 1.500 in.
Anchor plate[R] :	l_x x l_y x t = 24.000 in. x 24.000 in. x 1.500 in.;
Profile:	W shape (AISC), W14X90; (L x W x T x FT) = 14.000 in. x 14.500 in. x 0.440 in. x 0.710 in.
Base material:	cracked concrete, Custom, f_c' = 4,500 psi; h = 36.000 in.
Reinforcement:	tension: not present, shear: not present; edge reinforcement: none or < No. 4 bar

[R] - The anchor calculation is based on a rigid anchor plate assumption.

Geometry [in.] & Loading [kip, ft.kip]



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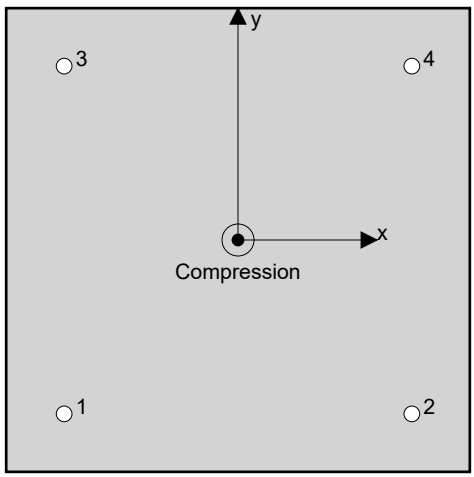
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Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP (~400 kips)	Date:	1/28/2025
Fastening point:			

1.1 Design results

Case	Description	Forces [kip] / Moments [ft.kip]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = -585.000; V_x = 9.000; V_y = 10.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	99

2 Load case/Resulting anchor forces



Anchor reactions [kip]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	0.000	3.363	2.250	2.500
2	0.000	3.363	2.250	2.500
3	0.000	3.363	2.250	2.500
4	0.000	3.363	2.250	2.500

Max. concrete compressive strain:	0.00 [‰]
Max. concrete compressive stress:	1,016 [psi]
Resulting tension force in (x/y)=(-/-):	0.000 [kip]
Resulting compression force in (x/y)=(0.000/-0.000):	585.000 [kip]

Anchor forces are calculated based on the assumption of a rigid anchor plate.

3 Tension load

	Load N_{ua} [kip]	Capacity ϕ N_n [kip]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	N/A	N/A	N/A	N/A
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	N/A	N/A	N/A	N/A
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (anchors in tension)

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Company:		Page:	3
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP (~400 kips)	Date:	1/28/2025
Fastening point:			

4 Shear load

	Load V_{ua} [kip]	Capacity ϕ V_n [kip]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	3.363	7.555	45	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	13.454	132.505	11	OK
Concrete edge failure in direction y+**	13.454	13.693	99	OK

* highest loaded anchor **anchor group (relevant anchors)

4.1 Steel Strength

$V_{sa} = 0.6\ A_{se,V}\ f_{uta}$ ACI 318-19 Eq. (17.7.1.2b)
$\phi\ V_{steel} \geq V_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.33	58,000

Calculations

V_{sa} [kip]
11.623

Results

V_{sa} [kip]	ϕ_{steel}	ϕ V_{sa} [kip]	V_{ua} [kip]
11.623	0.650	7.555	3.363

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Company:		Page:	4
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP (~400 kips)	Date:	1/28/2025
Fastening point:			

4.2 Pryout Strength

$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \right]$ ACI 318-19 Eq. (17.7.3.1b)

$\phi V_{cpg} \geq V_{ua}$ ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9 h_{ef}^2$ ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e'_N}{3 h_{ef}}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ ACI 318-19 Eq. (17.6.2.2.1)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	6.000	0.000	0.000	9.000

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.000	-	24	1.000	4,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [kip]
1,296.00	324.00	1.000	1.000	1.000	1.000	23.662

Results

V_{cpg} [kip]	$\phi_{concrete}$	ϕV_{cpg} [kip]	V_{ua} [kip]
189.293	0.700	132.505	13.454

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Company:		Page:	5
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP (~400 kips)	Date:	1/28/2025
Fastening point:			

4.3 Concrete edge failure in direction y+

$V_{cbg} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \Psi_{ec,V} \Psi_{ed,V} \Psi_{c,V} \Psi_{h,V} \Psi_{parallel,V} V_b$ — ACI 318-19 Eq. (17.7.2.1b)

$\phi V_{cbg} \geq V_{ua}$ — ACI 318-19 Table 17.5.2

A_{Vc} see ACI 318-19, Section 17.7.2.1, Fig. R 17.7.2.1(b)*

$A_{Vc0} = 4.5 c_{a1}^2$ — ACI 318-19 Eq. (17.7.2.1.3)

$\Psi_{ec,V} = \left(\frac{1}{1 + \frac{e'_v}{1.5c_{a1}}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.3.1)

$\Psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.4.1b)

$\Psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ — ACI 318-19 Eq. (17.7.2.6.1)

$V_b = 9 \lambda_a \sqrt{f'_c} c_{a1}^{1.5}$ — ACI 318-19 Eq. (17.7.2.2.1b)

Variables

c_{a1} [in.]	c_{a2} [in.]	e_{cV} [in.]	$\Psi_{c,V}$	h_a [in.]
9.000	9.000	0.000	1.000	36.000

l_e [in.]	λ_a	d_a [in.]	f'_c [psi]	$\Psi_{parallel,V}$
6.000	1.000	0.750	4,500	1.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{h,V}$	V_b [kip]
486.00	364.50	1.000	0.900	1.000	16.301

Results

V_{cbg} [kip]	$\phi_{concrete}$	ϕV_{cbg} [kip]	V_{ua} [kip]
19.561	0.700	13.693	13.454

*Anchor row defined by: Anchor 3, 4; Case 3 controls

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Company:		Page:	6
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP (~400 kips)	Date:	1/28/2025
Fastening point:			

5 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- User is responsible for evaluating the hole bearing capacity in case of shear forces.
- The equations presented in this report are based on imperial units. When inputs are displayed in metric units, the user should be aware that the equations remain in their imperial format.
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- For additional information about ACI 318 strength design provisions, please go to https://submittals.us.hilti.com/PROFISAnchorDesignGuide/

Fastening meets the design criteria!

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Company:		Page:	7
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP (~400 kips)	Date:	1/28/2025
Fastening point:			

6 Anchor plate and concrete bearing stress check

	Load	Capacity	Utilization [%]	Status
Concentric Compression	19,520.31 [in.lb/in.]	25,312.51 [in.lb/in.]	78	OK
Concrete bearing	1,016 [psi]	3,729 [psi]	28	OK
Tension Interface	N/A	N/A	N/A	N/A
Uniaxial Moment (Strong Axis)	N/A	N/A	N/A	N/A
Uniaxial Moment (Weak Axis)	N/A	N/A	N/A	N/A

6.1 Concrete bearing check (per AISC DG1, section 3.1.1)

$A_1 = B \cdot N$

$A_2 = L \cdot W$

$f_{pu(max)} = \phi \left(0.85 f'_c \sqrt{\frac{A_2}{A_1}} \right), \sqrt{\frac{A_2}{A_1}} \leq 2$

$f_{pu} = \frac{P_u}{BN}$

$f_{pu} \leq f_{p(max)}$

Variables

B [in.]	N [in.]	L [in.]	W [in.]	f_c' [psi]	ϕ	P_u [kip]
24.000	24.000	36.000	36.000	4,500	0.650	585.000

Calculations

A_1 [in.^{2}]	A_2 [in.^{2}]
576.00	1,296.00

Results

f_{pu} [psi]	$f_{pu(max)}$ [psi]
1,016	3,729

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Company:		Page:	8
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP (~400 kips)	Date:	1/28/2025
Fastening point:			

6.2 Plate bending, compression (per AISC DG1, sections 3.1.2, 3.1.3)

$A_1 = B \cdot N$

$A_2 = L \cdot W$

$P_p = 0.85 f'_c A_1 \sqrt{\frac{A_2}{A_1}}, \sqrt{\frac{A_2}{A_1}} \leq 2$

$X = \left\{ \frac{4 d b_f}{(d + b_f)^2} \right\} \frac{P_u}{\phi_c P_p}$

$\lambda = \frac{2\sqrt{X}}{1 + \sqrt{1 - X}} \leq 1$

$m = \frac{N - 0.95d}{2}$

$n = \frac{B - 0.80 b_f}{2}$

$\lambda n' = \lambda \frac{\sqrt{d b_f}}{4}$

$l = \max(m, n, \lambda n')$

$f_{pu} = \frac{P_u}{BN}$

$M_{pl} = \frac{f_{pu} \cdot l^2}{2}$

$\phi M_n = \phi \cdot F_y \cdot \frac{t_p^2}{4}$

$M_{pl} \leq \phi M_n$

Variables

B [in.]	N [in.]	L [in.]	W [in.]	d [in.]
24.000	24.000	36.000	36.000	14.000

b_f [in.]	F_y [psi]	ϕ	t_p [in.]	P_u' [kip]
14.500	50,000	0.900	1.500	585.000

Calculations

A_1 [in.^{2}]	A_2 [in.^{2}]	P_p [kip]	X	λ
576.00	1,296.00	2,148.120	0.419	0.734

m [in.]	n [in.]	$\lambda n'$ [in.]	l [in.]	f_{pu} [psi]
5.350	6.200	2.616	6.200	1,016

Results

M_n [in.lb/in.]	ϕ	ϕM_n [in.lb/in.]	M_{pl} [in.lb/in.]
28,125.01	0.900	25,312.51	19,520.31

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Company:		Page:	9
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP (~400 kips)	Date:	1/28/2025
Fastening point:			

7 Installation data

Profile: W shape (AISC), W14X90; (L x W x T x FT) = 14.000 in. x 14.500 in. x 0.440 in. x 0.710 in.

Hole diameter in the fixture: d_f = 0.812 in.

Plate thickness (input): 1.500 in.

Anchor type and diameter: Heavy Hex Head ASTM F 1554 GR. 36 3/4

Item number: not available

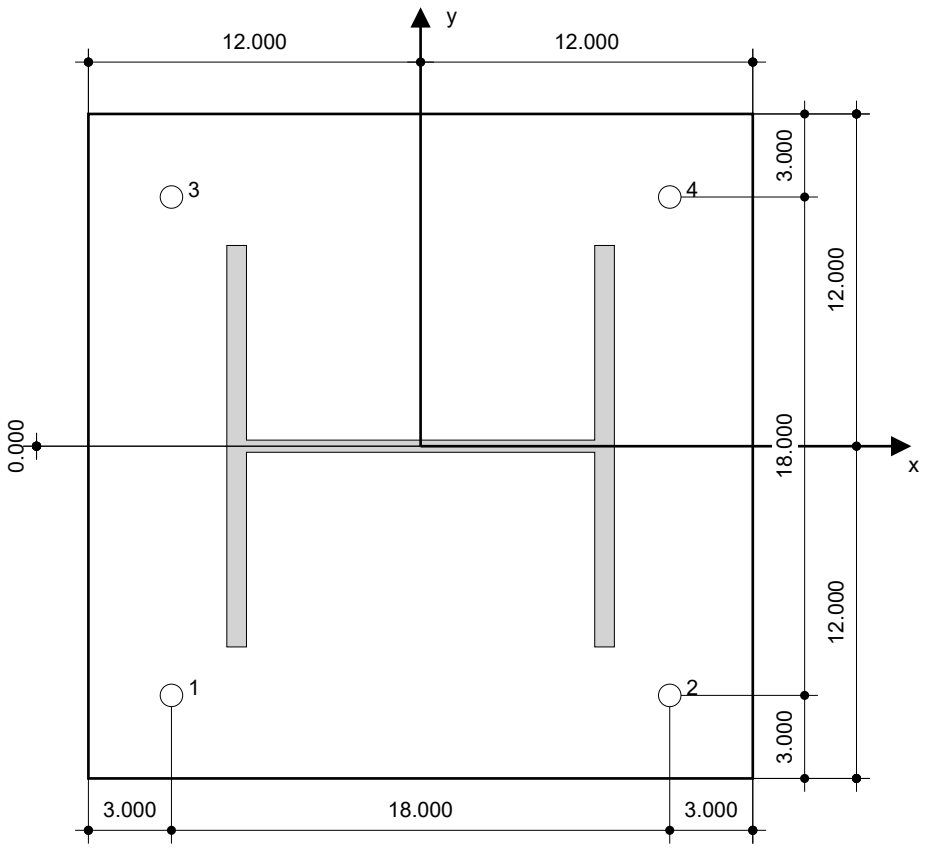
Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 18.000 in.

Minimum thickness of the base material: 19.000 in.

⌀ 3/4 in Heavy Hex Head ASTM F 1554 GR. 36 with 18 in nominal embedment depth per Technical data , cast in place installation per MPII



Coordinates Anchor in.

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-9.000	-9.000	9.000	27.000	9.000	27.000
2	9.000	-9.000	27.000	9.000	9.000	27.000
3	-9.000	9.000	9.000	27.000	27.000	9.000
4	9.000	9.000	27.000	9.000	27.000	9.000

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Company:		Page:	10
Address:		Specifier:	
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- You must take all necessary and reasonable steps to prevent or limit damage caused by the Software. In particular, you must arrange for the regular backup of programs and data and, if applicable, carry out the updates of the Software offered by Hilti on a regular basis. If you do not use the AutoUpdate function of the Software, you must ensure that you are using the current and thus up-to-date version of the Software in each case by carrying out manual updates via the Hilti Website. Hilti will not be liable for consequences, such as the recovery of lost or damaged data or programs, arising from a culpable breach of duty by you.

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Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP (~400 kips)	Date:	1/28/2025
Fastening point:			

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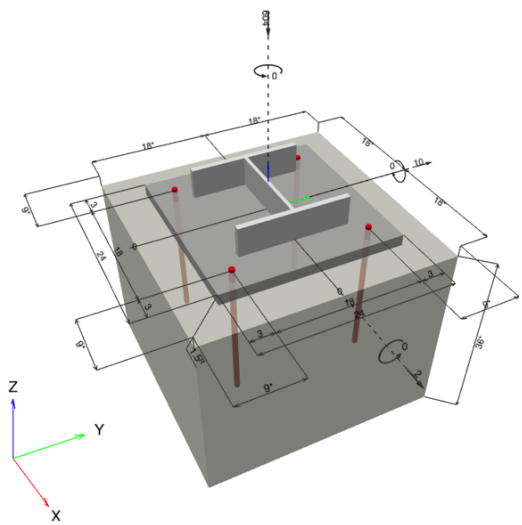
1 Input data



Anchor type and diameter:	**Heavy Hex Head ASTM F 1554 GR. 36 3/4**
Item number:	not available
Specification text:	∅ 3/4 in Heavy Hex Head ASTM F 1554 GR. 36 with 18 in nominal embedment depth per Technical data , cast in place installation per MPII
Effective embedment depth:	h_{ef} = 18.000 in.
Material:	ASTM F 1554
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-19 / CIP
Shear edge breakout verification:	Row closest to edge (Case 3 only from ACI 318-19 Fig. R.17.7.2.1b)
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 1.500 in.
Anchor plate[R] :	l_x x l_y x t = 24.000 in. x 24.000 in. x 1.500 in.;
Profile:	W shape (AISC), W14X99; (L x W x T x FT) = 14.200 in. x 14.600 in. x 0.485 in. x 0.780 in.
Base material:	cracked concrete, Custom, f_c' = 4,500 psi; h = 36.000 in.
Reinforcement:	tension: not present, shear: not present;
	edge reinforcement: none or < No. 4 bar

[R] - The anchor calculation is based on a rigid anchor plate assumption.

Geometry [in.] & Loading [kip, ft.kip]



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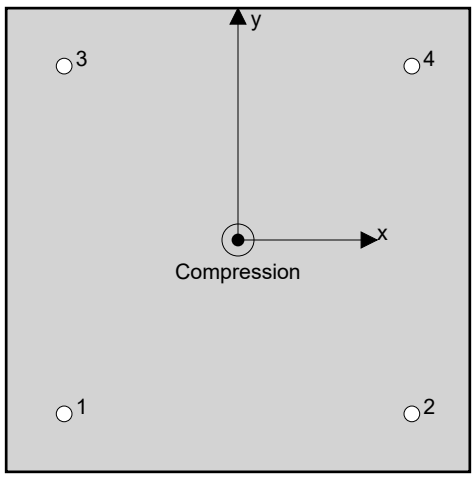
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Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP (~400 kips)	Date:	1/28/2025
Fastening point:			

1.1 Design results

Case	Description	Forces [kip] / Moments [ft.kip]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = -604.000; V_x = 2.000; V_y = 10.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	75

2 Load case/Resulting anchor forces



Anchor reactions [kip]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	0.000	2.550	0.500	2.500
2	0.000	2.550	0.500	2.500
3	0.000	2.550	0.500	2.500
4	0.000	2.550	0.500	2.500

Max. concrete compressive strain:	0.00 [‰]
Max. concrete compressive stress:	1,049 [psi]
Resulting tension force in (x/y)=(-/-):	0.000 [kip]
Resulting compression force in (x/y)=(0.000/-0.000):	604.000 [kip]

Anchor forces are calculated based on the assumption of a rigid anchor plate.

3 Tension load

	Load N_{ua} [kip]	Capacity ϕ N_n [kip]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	N/A	N/A	N/A	N/A
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	N/A	N/A	N/A	N/A
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (anchors in tension)

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	3
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP (~400 kips)	Date:	1/28/2025
Fastening point:			

4 Shear load

	Load V_{ua} [kip]	Capacity ϕ V_n [kip]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	2.550	7.555	34	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	10.198	132.505	8	OK
Concrete edge failure in direction y+**	10.198	13.693	75	OK

* highest loaded anchor **anchor group (relevant anchors)

4.1 Steel Strength

$V_{sa} = 0.6\ A_{se,V}\ f_{uta}$ ACI 318-19 Eq. (17.7.1.2b)
$\phi\ V_{steel} \geq V_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.33	58,000

Calculations

V_{sa} [kip]
11.623

Results

V_{sa} [kip]	ϕ_{steel}	ϕ V_{sa} [kip]	V_{ua} [kip]
11.623	0.650	7.555	2.550

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	4
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP (~400 kips)	Date:	1/28/2025
Fastening point:			

4.2 Pryout Strength

$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \Psi_{ec,N} \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_b \right]$ ACI 318-19 Eq. (17.7.3.1b)

$\phi V_{cpg} \geq V_{ua}$ ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9 h_{ef}^2$ ACI 318-19 Eq. (17.6.2.1.4)

$\Psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e'_N}{3 h_{ef}}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.3.1)

$\Psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.4.1b)

$\Psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ ACI 318-19 Eq. (17.6.2.2.1)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	6.000	0.000	0.000	9.000

$\Psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.000	-	24	1.000	4,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\Psi_{ec1,N}$	$\Psi_{ec2,N}$	$\Psi_{ed,N}$	$\Psi_{cp,N}$	N_b [kip]
1,296.00	324.00	1.000	1.000	1.000	1.000	23.662

Results

V_{cpg} [kip]	$\phi_{concrete}$	ϕV_{cpg} [kip]	V_{ua} [kip]
189.293	0.700	132.505	10.198

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	5
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP (~400 kips)	Date:	1/28/2025
Fastening point:			

4.3 Concrete edge failure in direction y+

$V_{cbg} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \Psi_{ec,V} \Psi_{ed,V} \Psi_{c,V} \Psi_{h,V} \Psi_{parallel,V} V_b$ — ACI 318-19 Eq. (17.7.2.1b)

$\phi V_{cbg} \geq V_{ua}$ — ACI 318-19 Table 17.5.2

A_{Vc} see ACI 318-19, Section 17.7.2.1, Fig. R 17.7.2.1(b)*

$A_{Vc0} = 4.5 c_{a1}^2$ — ACI 318-19 Eq. (17.7.2.1.3)

$\Psi_{ec,V} = \left(\frac{1}{1 + \frac{e'_v}{1.5c_{a1}}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.3.1)

$\Psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.4.1b)

$\Psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ — ACI 318-19 Eq. (17.7.2.6.1)

$V_b = 9 \lambda_a \sqrt{f'_c} c_{a1}^{1.5}$ — ACI 318-19 Eq. (17.7.2.2.1b)

Variables

c_{a1} [in.]	c_{a2} [in.]	e_{cV} [in.]	$\Psi_{c,V}$	h_a [in.]
9.000	9.000	0.000	1.000	36.000

l_e [in.]	λ_a	d_a [in.]	f'_c [psi]	$\Psi_{parallel,V}$
6.000	1.000	0.750	4,500	1.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{h,V}$	V_b [kip]
486.00	364.50	1.000	0.900	1.000	16.301

Results

V_{cbg} [kip]	$\phi_{concrete}$	ϕV_{cbg} [kip]	V_{ua} [kip]
19.561	0.700	13.693	10.198

*Anchor row defined by: Anchor 3, 4; Case 3 controls

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Company:		Page:	6
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP (~400 kips)	Date:	1/28/2025
Fastening point:			

5 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- User is responsible for evaluating the hole bearing capacity in case of shear forces.
- The equations presented in this report are based on imperial units. When inputs are displayed in metric units, the user should be aware that the equations remain in their imperial format.
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- For additional information about ACI 318 strength design provisions, please go to https://submittals.us.hilti.com/PROFISAnchorDesignGuide/

Fastening meets the design criteria!

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Company:		Page:	7
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP (~400 kips)	Date:	1/28/2025
Fastening point:			

6 Anchor plate and concrete bearing stress check

	Load	Capacity	Utilization [%]	Status
Concentric Compression	19,895.09 [in.lb/in.]	25,312.51 [in.lb/in.]	79	OK
Concrete bearing	1,049 [psi]	3,729 [psi]	29	OK
Tension Interface	N/A	N/A	N/A	N/A
Uniaxial Moment (Strong Axis)	N/A	N/A	N/A	N/A
Uniaxial Moment (Weak Axis)	N/A	N/A	N/A	N/A

6.1 Concrete bearing check (per AISC DG1, section 3.1.1)

$A_1 = B \cdot N$

$A_2 = L \cdot W$

$f_{pu(max)} = \phi \left(0.85 f'_c \sqrt{\frac{A_2}{A_1}} \right) , \sqrt{\frac{A_2}{A_1}} \leq 2$

$f_{pu} = \frac{P_u}{BN}$

$f_{pu} \leq f_{p(max)}$

Variables

B [in.]	N [in.]	L [in.]	W [in.]	f_c' [psi]	ϕ	P_u [kip]
24.000	24.000	36.000	36.000	4,500	0.650	604.000

Calculations

A_1 [in.^{2}]	A_2 [in.^{2}]
576.00	1,296.00

Results

f_{pu} [psi]	$f_{pu(max)}$ [psi]
1,049	3,729

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	8
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP (~400 kips)	Date:	1/28/2025
Fastening point:			

6.2 Plate bending, compression (per AISC DG1, sections 3.1.2, 3.1.3)

$A_1 = B \cdot N$

$A_2 = L \cdot W$

$P_p = 0.85 f'_c A_1 \sqrt{\frac{A_2}{A_1}}, \sqrt{\frac{A_2}{A_1}} \leq 2$

$X = \left\{ \frac{4 d b_f}{(d + b_f)^2} \right\} \frac{P_u}{\phi_c P_p}$

$\lambda = \frac{2\sqrt{X}}{1 + \sqrt{1 - X}} \leq 1$

$m = \frac{N - 0.95d}{2}$

$n = \frac{B - 0.80 b_f}{2}$

$\lambda n' = \lambda \frac{\sqrt{d b_f}}{4}$

$l = \max(m, n, \lambda n')$

$f_{pu} = \frac{P_u}{BN}$

$M_{pl} = \frac{f_{pu} \cdot l^2}{2}$

$\phi M_n = \phi \cdot F_y \cdot \frac{t_p^2}{4}$

$M_{pl} \leq \phi M_n$

Variables

B [in.]	N [in.]	L [in.]	W [in.]	d [in.]
24.000	24.000	36.000	36.000	14.200

b_f [in.]	F_y [psi]	ϕ	t_p [in.]	P_u' [kip]
14.600	50,000	0.900	1.500	604.000

Calculations

A_1 [in.^{2}]	A_2 [in.^{2}]	P_p [kip]	X	λ
576.00	1,296.00	2,148.120	0.432	0.750

m [in.]	n [in.]	$\lambda n'$ [in.]	l [in.]	f_{pu} [psi]
5.255	6.160	2.700	6.160	1,049

Results

M_n [in.lb/in.]	ϕ	ϕM_n [in.lb/in.]	M_{pl} [in.lb/in.]
28,125.01	0.900	25,312.51	19,895.09

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Company:		Page:	9
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP (~400 kips)	Date:	1/28/2025
Fastening point:			

7 Installation data

Profile: W shape (AISC), W14X99; (L x W x T x FT) = 14.200 in. x 14.600 in. x 0.485 in. x 0.780 in.

Hole diameter in the fixture: d_f = 0.812 in.

Plate thickness (input): 1.500 in.

Anchor type and diameter: Heavy Hex Head ASTM F 1554 GR. 36 3/4

Item number: not available

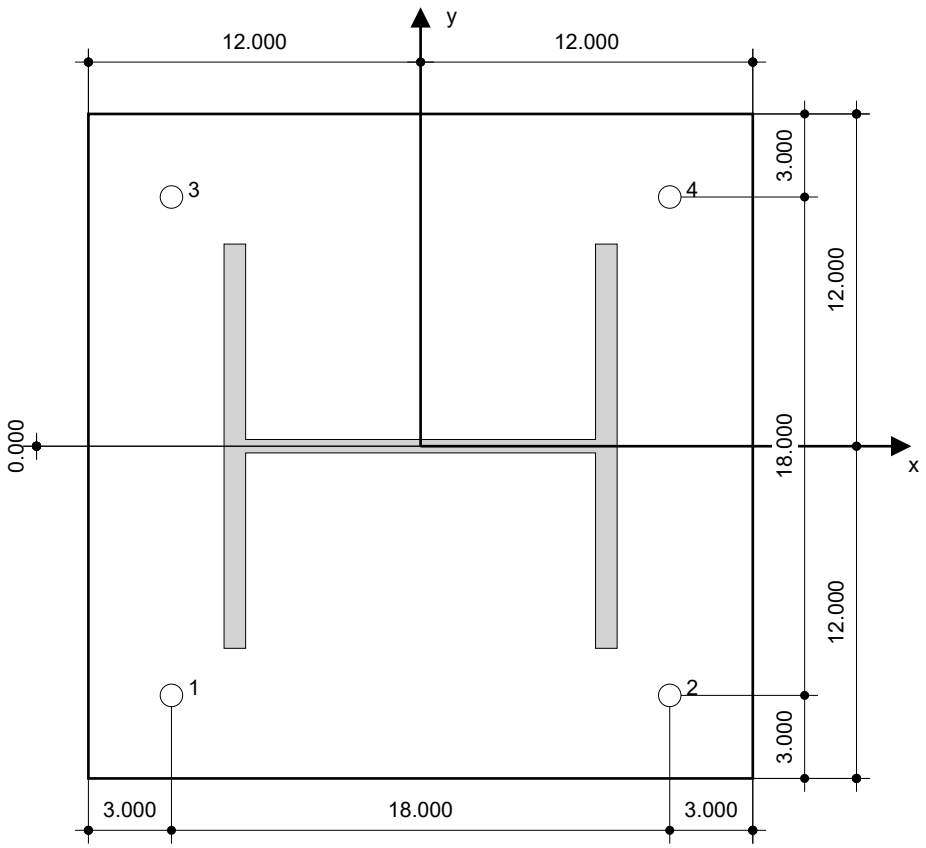
Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 18.000 in.

Minimum thickness of the base material: 19.000 in.

⌀ 3/4 in Heavy Hex Head ASTM F 1554 GR. 36 with 18 in nominal embedment depth per Technical data , cast in place installation per MPII



Coordinates Anchor in.

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-9.000	-9.000	9.000	27.000	9.000	27.000
2	9.000	-9.000	27.000	9.000	9.000	27.000
3	-9.000	9.000	9.000	27.000	27.000	9.000
4	9.000	9.000	27.000	9.000	27.000	9.000

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Company:		Page:	10
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP (~400 kips)	Date:	1/28/2025
Fastening point:			

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Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	1/28/2025
Fastening point:			

Specifier's comments:

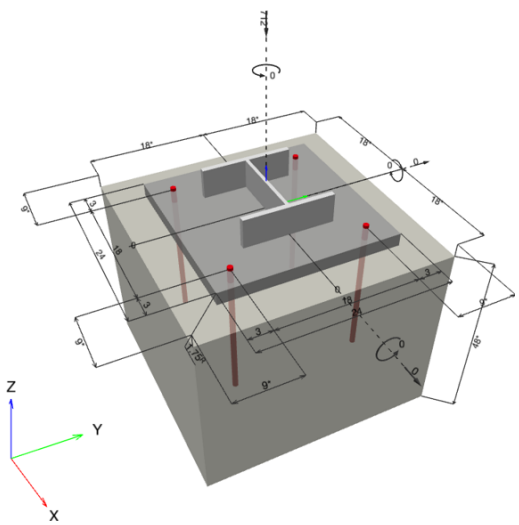
1 Input data



Anchor type and diameter:	**Heavy Hex Head ASTM F 1554 GR. 36 3/4**
Item number:	not available
Specification text:	∅ 3/4 in Heavy Hex Head ASTM F 1554 GR. 36 with 18 in nominal embedment depth per Technical data , cast in place installation per MPII
Effective embedment depth:	h_{ef} = 18.000 in.
Material:	ASTM F 1554
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-19 / CIP
Shear edge breakout verification:	Row closest to edge (Case 3 only from ACI 318-19 Fig. R.17.7.2.1b)
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 1.750 in.
Anchor plate[R] :	l_x x l_y x t = 24.000 in. x 24.000 in. x 1.750 in.;
Profile:	W shape (AISC), W12X79; (L x W x T x FT) = 12.400 in. x 12.100 in. x 0.470 in. x 0.735 in.
Base material:	cracked concrete, 4000, f_c' = 4,000 psi; h = 48.000 in.
Reinforcement:	tension: not present, shear: not present;
	edge reinforcement: none or < No. 4 bar

[R] - The anchor calculation is based on a rigid anchor plate assumption.

Geometry [in.] & Loading [kip, ft.kip]



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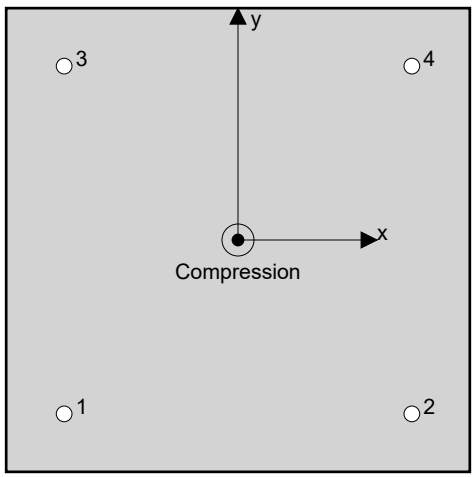
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Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	1/28/2025
Fastening point:			

1.1 Design results

Case	Description	Forces [kip] / Moments [ft.kip]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = -712.000; V_x = 0.000; V_y = 0.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	-

2 Load case/Resulting anchor forces



Anchor reactions [kip]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	0.000	0.000	0.000	0.000
2	0.000	0.000	0.000	0.000
3	0.000	0.000	0.000	0.000
4	0.000	0.000	0.000	0.000

Max. concrete compressive strain: 0.00 [‰]
Max. concrete compressive stress: 1,236 [psi]
Resulting tension force in (x/y)=(-/-): 0.000 [kip]
Resulting compression force in (x/y)=(0.000/0.000): 712.000 [kip]

Anchor forces are calculated based on the assumption of a rigid anchor plate.

3 Tension load

	Load N_{ua} [kip]	Capacity ϕ N_n [kip]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	N/A	N/A	N/A	N/A
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	N/A	N/A	N/A	N/A
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (anchors in tension)

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	3
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	1/28/2025
Fastening point:			

4 Shear load

	Load V_{ua} [kip]	Capacity ϕ V_n [kip]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	N/A	N/A	N/A	N/A
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength*	N/A	N/A	N/A	N/A
Concrete edge failure in direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (relevant anchors)

5 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Since no anchor rod forces exist, the anchor rod size can be determined based on the OSHA requirements, and practical considerations.
- The equations presented in this report are based on imperial units. When inputs are displayed in metric units, the user should be aware that the equations remain in their imperial format.
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- For additional information about ACI 318 strength design provisions, please go to https://submittals.us.hilti.com/PROFISAnchorDesignGuide/

Fastening meets the design criteria!

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Company:		Page:	4
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	1/28/2025
Fastening point:			

6 Anchor plate and concrete bearing stress check

	Load	Capacity	Utilization [%]	Status
Concentric Compression	31,684.99 [in.lb/in.]	34,453.14 [in.lb/in.]	92	OK
Concrete bearing	1,236 [psi]	3,315 [psi]	38	OK
Tension Interface	N/A	N/A	N/A	N/A
Uniaxial Moment (Strong Axis)	N/A	N/A	N/A	N/A
Uniaxial Moment (Weak Axis)	N/A	N/A	N/A	N/A

6.1 Concrete bearing check (per AISC DG1, section 3.1.1)

$A_1 = B \cdot N$

$A_2 = L \cdot W$

$f_{pu(max)} = \phi \left(0.85 f'_c \sqrt{\frac{A_2}{A_1}} \right) , \sqrt{\frac{A_2}{A_1}} \leq 2$

$f_{pu} = \frac{P_u}{BN}$

$f_{pu} \leq f_{p(max)}$

Variables

B [in.]	N [in.]	L [in.]	W [in.]	f_c' [psi]	ϕ	P_u [kip]
24.000	24.000	36.000	36.000	4,000	0.650	712.000

Calculations

A_1 [in.^{2}]	A_2 [in.^{2}]
576.00	1,296.00

Results

f_{pu} [psi]	$f_{pu(max)}$ [psi]
1,236	3,315

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Company:		Page:	5
Address:		Specifier:	
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Design:	Orange BP (~800 kips)	Date:	1/28/2025
Fastening point:			

6.2 Plate bending, compression (per AISC DG1, sections 3.1.2, 3.1.3)

$A_1 = B \cdot N$

$A_2 = L \cdot W$

$P_p = 0.85 f'_c A_1 \sqrt{\frac{A_2}{A_1}}, \sqrt{\frac{A_2}{A_1}} \leq 2$

$X = \left\{ \frac{4 d b_f}{(d + b_f)^2} \right\} \frac{P_u}{\phi_c P_p}$

$\lambda = \frac{2\sqrt{X}}{1 + \sqrt{1 - X}} \leq 1$

$m = \frac{N - 0.95d}{2}$

$n = \frac{B - 0.80 b_f}{2}$

$\lambda n' = \lambda \frac{\sqrt{d b_f}}{4}$

$l = \max(m, n, \lambda n')$

$f_{pu} = \frac{P_u}{BN}$

$M_{pl} = \frac{f_{pu} \cdot l^2}{2}$

$\phi M_n = \phi \cdot F_y \cdot \frac{t_p^2}{4}$

$M_{pl} \leq \phi M_n$

Variables

B [in.]	N [in.]	L [in.]	W [in.]	d [in.]
24.000	24.000	36.000	36.000	12.400

b_f [in.]	F_y [psi]	ϕ	t_p [in.]	P_u' [kip]
12.100	50,000	0.900	1.750	712.000

Calculations

A_1 [in.^{2}]	A_2 [in.^{2}]	P_p [kip]	X	λ
576.00	1,296.00	1,909.440	0.574	0.916

m [in.]	n [in.]	$\lambda n'$ [in.]	l [in.]	f_{pu} [psi]
6.110	7.160	2.806	7.160	1,236

Results

M_n [in.lb/in.]	ϕ	ϕM_n [in.lb/in.]	M_{pl} [in.lb/in.]
38,281.27	0.900	34,453.14	31,684.99

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Company:		Page:	6
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	1/28/2025
Fastening point:			

7 Installation data

Profile: W shape (AISC), W12X79; (L x W x T x FT) = 12.400 in. x 12.100 in. x 0.470 in. x 0.735 in.

Hole diameter in the fixture: d_f = 0.812 in.

Plate thickness (input): 1.750 in.

Anchor type and diameter: Heavy Hex Head ASTM F 1554 GR. 36 3/4

Item number: not available

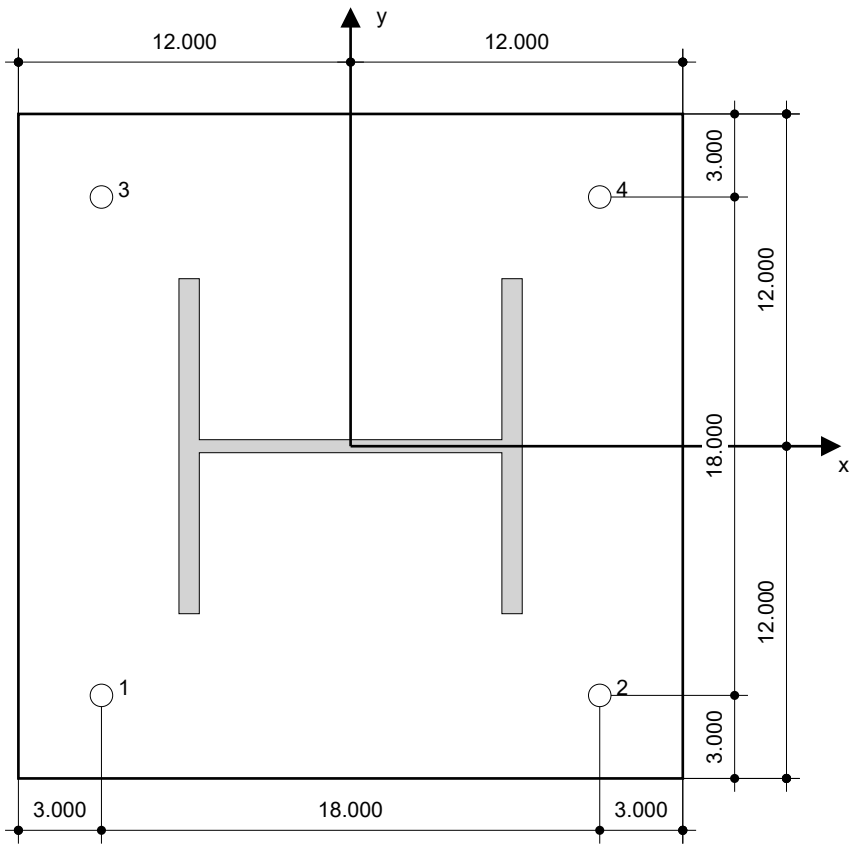
Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 18.000 in.

Minimum thickness of the base material: 19.000 in.

⌀ 3/4 in Heavy Hex Head ASTM F 1554 GR. 36 with 18 in nominal embedment depth per Technical data , cast in place installation per MPII



Coordinates Anchor in.

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-9.000	-9.000	9.000	27.000	9.000	27.000
2	9.000	-9.000	27.000	9.000	9.000	27.000
3	-9.000	9.000	9.000	27.000	27.000	9.000
4	9.000	9.000	27.000	9.000	27.000	9.000

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Company:		Page:	7
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
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- You must take all necessary and reasonable steps to prevent or limit damage caused by the Software. In particular, you must arrange for the regular backup of programs and data and, if applicable, carry out the updates of the Software offered by Hilti on a regular basis. If you do not use the AutoUpdate function of the Software, you must ensure that you are using the current and thus up-to-date version of the Software in each case by carrying out manual updates via the Hilti Website. Hilti will not be liable for consequences, such as the recovery of lost or damaged data or programs, arising from a culpable breach of duty by you.

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Company:		Page:	1
Address:		Specifier:	
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Design:	Orange BP (~800 kips)	Date:	1/28/2025
Fastening point:			

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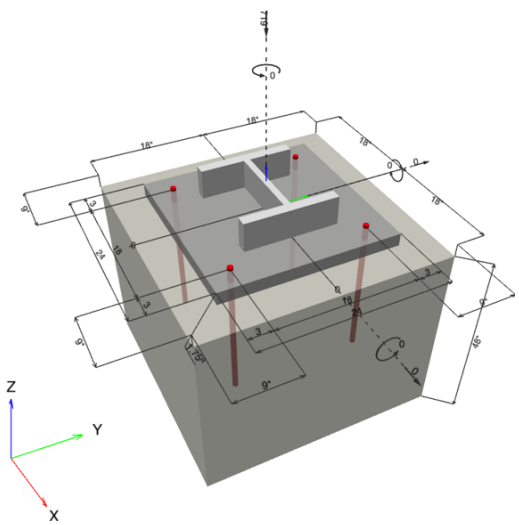
1 Input data



Anchor type and diameter:	**Heavy Hex Head ASTM F 1554 GR. 36 3/4**
Item number:	not available
Specification text:	∅ 3/4 in Heavy Hex Head ASTM F 1554 GR. 36 with 18 in nominal embedment depth per Technical data , cast in place installation per MPII
Effective embedment depth:	h_{ef} = 18.000 in.
Material:	ASTM F 1554
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-19 / CIP
Shear edge breakout verification:	Row closest to edge (Case 3 only from ACI 318-19 Fig. R.17.7.2.1b)
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 1.750 in.
Anchor plate[R] :	l_x x l_y x t = 24.000 in. x 24.000 in. x 1.750 in.;
Profile:	W shape (AISC), W12X152; (L x W x T x FT) = 13.700 in. x 12.500 in. x 0.870 in. x 1.400 in.
Base material:	cracked concrete, 4000, f_c' = 4,000 psi; h = 48.000 in.
Reinforcement:	tension: not present, shear: not present;
	edge reinforcement: none or < No. 4 bar

[R] - The anchor calculation is based on a rigid anchor plate assumption.

Geometry [in.] & Loading [kip, ft.kip]



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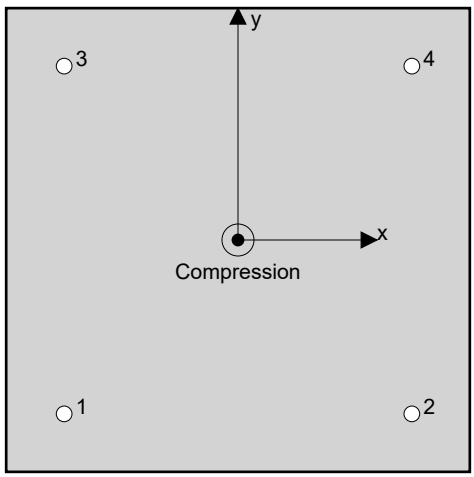
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Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	1/28/2025
Fastening point:			

1.1 Design results

Case	Description	Forces [kip] / Moments [ft.kip]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = -719.000; V_x = 0.000; V_y = 0.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	-

2 Load case/Resulting anchor forces



Anchor reactions [kip]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	0.000	0.000	0.000	0.000
2	0.000	0.000	0.000	0.000
3	0.000	0.000	0.000	0.000
4	0.000	0.000	0.000	0.000

Max. concrete compressive strain: 0.00 [‰]
Max. concrete compressive stress: 1,248 [psi]
Resulting tension force in (x/y)=(-/-): 0.000 [kip]
Resulting compression force in (x/y)=(0.000/0.000): 719.000 [kip]

Anchor forces are calculated based on the assumption of a rigid anchor plate.

3 Tension load

	Load N_{ua} [kip]	Capacity ϕ N_n [kip]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	N/A	N/A	N/A	N/A
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	N/A	N/A	N/A	N/A
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (anchors in tension)

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	3
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	1/28/2025
Fastening point:			

4 Shear load

	Load V_{ua} [kip]	Capacity ϕ V_n [kip]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	N/A	N/A	N/A	N/A
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength*	N/A	N/A	N/A	N/A
Concrete edge failure in direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (relevant anchors)

5 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Since no anchor rod forces exist, the anchor rod size can be determined based on the OSHA requirements, and practical considerations.
- The equations presented in this report are based on imperial units. When inputs are displayed in metric units, the user should be aware that the equations remain in their imperial format.
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- For additional information about ACI 318 strength design provisions, please go to https://submittals.us.hilti.com/PROFISAnchorDesignGuide/

Fastening meets the design criteria!

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Company:		Page:	4
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	1/28/2025
Fastening point:			

6 Anchor plate and concrete bearing stress check

	Load	Capacity	Utilization [%]	Status
Concentric Compression	30,582.47 [in.lb/in.]	34,453.14 [in.lb/in.]	89	OK
Concrete bearing	1,248 [psi]	3,315 [psi]	38	OK
Tension Interface	N/A	N/A	N/A	N/A
Uniaxial Moment (Strong Axis)	N/A	N/A	N/A	N/A
Uniaxial Moment (Weak Axis)	N/A	N/A	N/A	N/A

6.1 Concrete bearing check (per AISC DG1, section 3.1.1)

$A_1 = B \cdot N$

$A_2 = L \cdot W$

$f_{pu(max)} = \phi \left(0.85 f'_c \sqrt{\frac{A_2}{A_1}} \right) , \sqrt{\frac{A_2}{A_1}} \leq 2$

$f_{pu} = \frac{P_u}{BN}$

$f_{pu} \leq f_{p(max)}$

Variables

B [in.]	N [in.]	L [in.]	W [in.]	f_c' [psi]	ϕ	P_u [kip]
24.000	24.000	36.000	36.000	4,000	0.650	719.000

Calculations

A_1 [in.^{2}]	A_2 [in.^{2}]
576.00	1,296.00

Results

f_{pu} [psi]	$f_{pu(max)}$ [psi]
1,248	3,315

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	1/28/2025
Fastening point:			

6.2 Plate bending, compression (per AISC DG1, sections 3.1.2, 3.1.3)

$A_1 = B \cdot N$

$A_2 = L \cdot W$

$P_p = 0.85 f'_c A_1 \sqrt{\frac{A_2}{A_1}}, \sqrt{\frac{A_2}{A_1}} \leq 2$

$X = \left\{ \frac{4 d b_f}{(d + b_f)^2} \right\} \frac{P_u}{\phi_c P_p}$

$\lambda = \frac{2\sqrt{X}}{1 + \sqrt{1 - X}} \leq 1$

$m = \frac{N - 0.95d}{2}$

$n = \frac{B - 0.80 b_f}{2}$

$\lambda n' = \lambda \frac{\sqrt{d b_f}}{4}$

$l = \max(m, n, \lambda n')$

$f_{pu} = \frac{P_u}{BN}$

$M_{pl} = \frac{f_{pu} \cdot l^2}{2}$

$\phi M_n = \phi \cdot F_y \cdot \frac{t_p^2}{4}$

$M_{pl} \leq \phi M_n$

Variables

B [in.]	N [in.]	L [in.]	W [in.]	d [in.]
24.000	24.000	36.000	36.000	13.700

b_f [in.]	F_y [psi]	ϕ	t_p [in.]	P_u' [kip]
12.500	50,000	0.900	1.750	719.000

Calculations

A_1 [in.^{2}]	A_2 [in.^{2}]	P_p [kip]	X	λ
576.00	1,296.00	1,909.440	0.578	0.922

m [in.]	n [in.]	$\lambda n'$ [in.]	l [in.]	f_{pu} [psi]
5.492	7.000	3.016	7.000	1,248

Results

M_n [in.lb/in.]	ϕ	ϕM_n [in.lb/in.]	M_{pl} [in.lb/in.]
38,281.27	0.900	34,453.14	30,582.47

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Company:		Page:	6
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Orange BP (~800 kips)	Date:	1/28/2025
Fastening point:			

7 Installation data

Profile: W shape (AISC), W12X152; (L x W x T x FT) = 13.700 in. x 12.500 in. x 0.870 in. x 1.400 in.

Hole diameter in the fixture: d_f = 0.812 in.

Plate thickness (input): 1.750 in.

Anchor type and diameter: Heavy Hex Head ASTM F 1554 GR. 36 3/4

Item number: not available

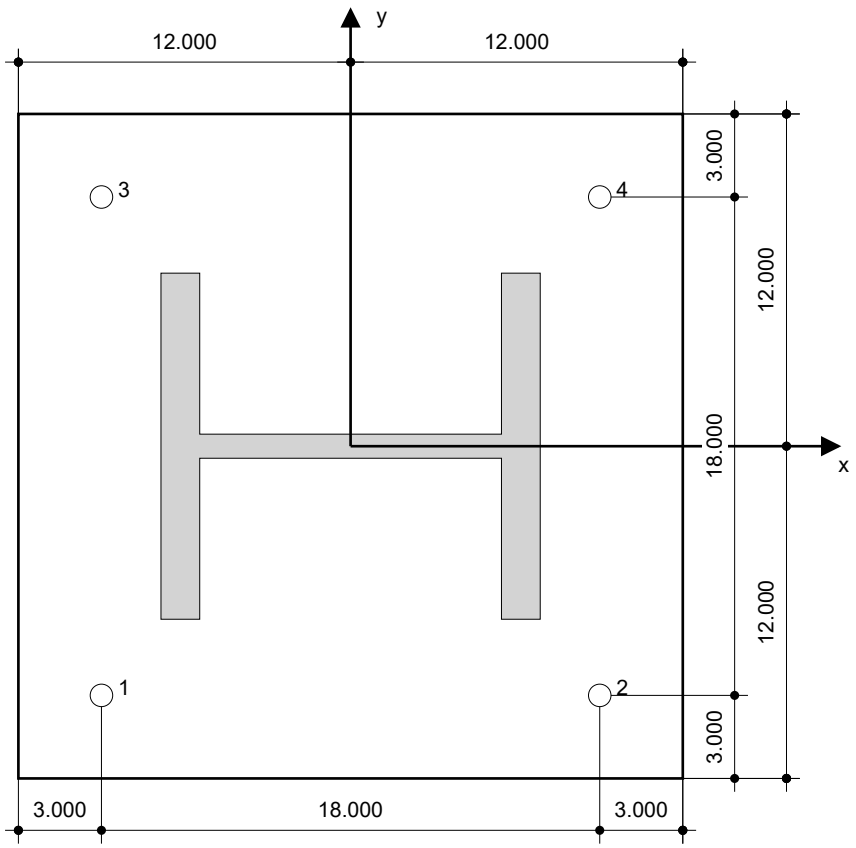
Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 18.000 in.

Minimum thickness of the base material: 19.000 in.

⌀ 3/4 in Heavy Hex Head ASTM F 1554 GR. 36 with 18 in nominal embedment depth per Technical data , cast in place installation per MPII



Coordinates Anchor in.

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-9.000	-9.000	9.000	27.000	9.000	27.000
2	9.000	-9.000	27.000	9.000	9.000	27.000
3	-9.000	9.000	9.000	27.000	27.000	9.000
4	9.000	9.000	27.000	9.000	27.000	9.000

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Company:		Page:	7
Address:		Specifier:	
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Fastening point:			

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- You must take all necessary and reasonable steps to prevent or limit damage caused by the Software. In particular, you must arrange for the regular backup of programs and data and, if applicable, carry out the updates of the Software offered by Hilti on a regular basis. If you do not use the AutoUpdate function of the Software, you must ensure that you are using the current and thus up-to-date version of the Software in each case by carrying out manual updates via the Hilti Website. Hilti will not be liable for consequences, such as the recovery of lost or damaged data or programs, arising from a culpable breach of duty by you.

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Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Blue BP (~1200 kips)	Date:	1/28/2025
Fastening point:			

Specifier's comments:

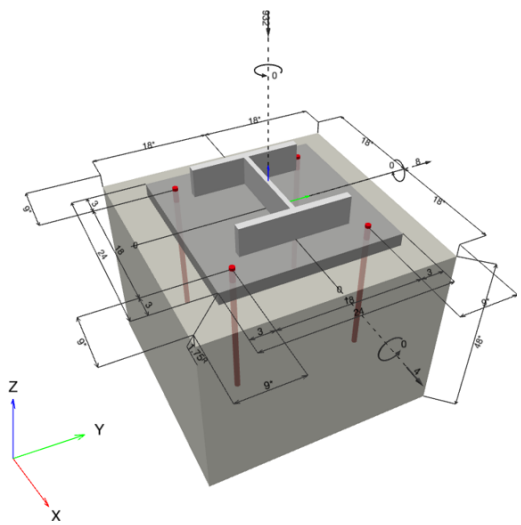
1 Input data



Anchor type and diameter:	**Heavy Hex Head ASTM F 1554 GR. 36 3/4**
Item number:	not available
Specification text:	∅ 3/4 in Heavy Hex Head ASTM F 1554 GR. 36 with 18 in nominal embedment depth per Technical data , cast in place installation per MPII
Effective embedment depth:	h_{ef} = 18.000 in.
Material:	ASTM F 1554
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-19 / CIP
Shear edge breakout verification:	Row closest to edge (Case 3 only from ACI 318-19 Fig. R.17.7.2.1b)
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 1.750 in.
Anchor plate^R :	l_x x l_y x t = 24.000 in. x 24.000 in. x 1.750 in.;
Profile:	W shape (AISC), W14X132; (L x W x T x FT) = 14.700 in. x 14.700 in. x 0.645 in. x 1.030 in.
Base material:	cracked concrete, 4000, f_c' = 4,000 psi; h = 48.000 in.
Reinforcement:	tension: not present, shear: not present;
	edge reinforcement: none or < No. 4 bar

^{R} - The anchor calculation is based on a rigid anchor plate assumption.

Geometry [in.] & Loading [kip, ft.kip]



Input data and results must be checked for conformity with the existing conditions and for plausibility!
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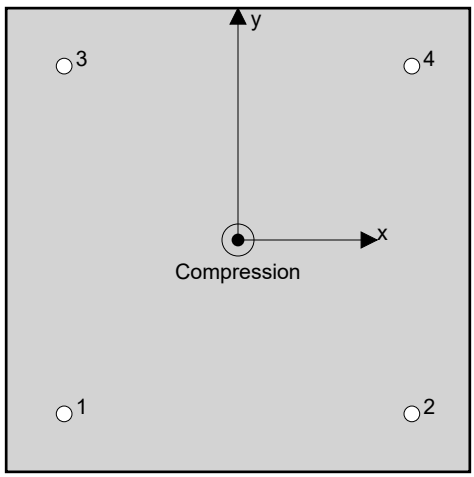
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Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Blue BP (~1200 kips)	Date:	1/28/2025
Fastening point:			

1.1 Design results

Case	Description	Forces [kip] / Moments [ft.kip]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = -932.000; V_x = 4.000; V_y = 8.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	70

2 Load case/Resulting anchor forces



Anchor reactions [kip]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	0.000	2.236	1.000	2.000
2	0.000	2.236	1.000	2.000
3	0.000	2.236	1.000	2.000
4	0.000	2.236	1.000	2.000

Max. concrete compressive strain:	0.00 [‰]
Max. concrete compressive stress:	1,618 [psi]
Resulting tension force in (x/y)=(-/-):	0.000 [kip]
Resulting compression force in (x/y)=(0.000/0.000):	932.000 [kip]

Anchor forces are calculated based on the assumption of a rigid anchor plate.

3 Tension load

	Load N_{ua} [kip]	Capacity ϕ N_n [kip]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	N/A	N/A	N/A	N/A
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	N/A	N/A	N/A	N/A
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (anchors in tension)

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	3
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Blue BP (~1200 kips)	Date:	1/28/2025
Fastening point:			

4 Shear load

	Load V_{ua} [kip]	Capacity ϕ V_n [kip]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	2.236	7.555	30	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	8.944	124.927	8	OK
Concrete edge failure in direction y+**	8.944	12.910	70	OK

* highest loaded anchor **anchor group (relevant anchors)

4.1 Steel Strength

$V_{sa} = 0.6\ A_{se,V}\ f_{uta}$ ACI 318-19 Eq. (17.7.1.2b)
$\phi\ V_{steel} \geq V_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.33	58,000

Calculations

V_{sa} [kip]
11.623

Results

V_{sa} [kip]	ϕ steel	ϕ V_{sa} [kip]	V_{ua} [kip]
11.623	0.650	7.555	2.236

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	4
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Blue BP (~1200 kips)	Date:	1/28/2025
Fastening point:			

4.2 Pryout Strength

$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \right]$ ACI 318-19 Eq. (17.7.3.1b)

$\phi V_{cpg} \geq V_{ua}$ ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9 h_{ef}^2$ ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e'_N}{3 h_{ef}}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ ACI 318-19 Eq. (17.6.2.2.1)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	6.000	0.000	0.000	9.000

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.000	-	24	1.000	4,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [kip]
1,296.00	324.00	1.000	1.000	1.000	1.000	22.308

Results

V_{cpg} [kip]	$\phi_{concrete}$	ϕV_{cpg} [kip]	V_{ua} [kip]
178.467	0.700	124.927	8.944

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Company:		Page:	5
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Blue BP (~1200 kips)	Date:	1/28/2025
Fastening point:			

4.3 Concrete edge failure in direction y+

$V_{cbg} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \psi_{ec,V}\ \psi_{ed,V}\ \psi_{c,V}\ \psi_{h,V}\ \psi_{parallel,V}\ V_b$ — ACI 318-19 Eq. (17.7.2.1b)

$\phi\ V_{cbg} \geq V_{ua}$ — ACI 318-19 Table 17.5.2

A_{Vc} see ACI 318-19, Section 17.7.2.1, Fig. R 17.7.2.1(b)*

$A_{Vc0} = 4.5\ c_{a1}^2$ — ACI 318-19 Eq. (17.7.2.1.3)

$\psi_{ec,V} = \left(\frac{1}{1 + \frac{e'_v}{1.5c_{a1}}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.3.1)

$\psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.4.1b)

$\psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ — ACI 318-19 Eq. (17.7.2.6.1)

$V_b = 9\ \lambda_a\ \sqrt{f'_c}\ c_{a1}^{1.5}$ — ACI 318-19 Eq. (17.7.2.2.1b)

Variables

c_{a1} [in.]	c_{a2} [in.]	e_{cV} [in.]	$\psi_{c,V}$	h_a [in.]
9.000	9.000	0.000	1.000	48.000

l_e [in.]	λ_a	d_a [in.]	f'_c [psi]	$\psi_{parallel,V}$
6.000	1.000	0.750	4,000	1.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\psi_{ec,V}$	$\psi_{ed,V}$	$\psi_{h,V}$	V_b [kip]
486.00	364.50	1.000	0.900	1.000	15.369

Results

V_{cbg} [kip]	$\phi_{concrete}$	$\phi\ V_{cbg}$ [kip]	V_{ua} [kip]
18.442	0.700	12.910	8.944

*Anchor row defined by: Anchor 3, 4; Case 3 controls

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Company:		Page:	6
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Blue BP (~1200 kips)	Date:	1/28/2025
Fastening point:			

5 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- User is responsible for evaluating the hole bearing capacity in case of shear forces.
- The equations presented in this report are based on imperial units. When inputs are displayed in metric units, the user should be aware that the equations remain in their imperial format.
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- For additional information about ACI 318 strength design provisions, please go to https://submittals.us.hilti.com/PROFISAnchorDesignGuide/

Fastening meets the design criteria!

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Company:		Page:	7
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Blue BP (~1200 kips)	Date:	1/28/2025
Fastening point:			

6 Anchor plate and concrete bearing stress check

	Load	Capacity	Utilization [%]	Status
Concentric Compression	30,301.65 [in.lb/in.]	34,453.14 [in.lb/in.]	88	OK
Concrete bearing	1,618 [psi]	3,315 [psi]	49	OK
Tension Interface	N/A	N/A	N/A	N/A
Uniaxial Moment (Strong Axis)	N/A	N/A	N/A	N/A
Uniaxial Moment (Weak Axis)	N/A	N/A	N/A	N/A

6.1 Concrete bearing check (per AISC DG1, section 3.1.1)

$A_1 = B \cdot N$

$A_2 = L \cdot W$

$f_{pu(max)} = \phi \left(0.85 f'_c \sqrt{\frac{A_2}{A_1}}\right), \sqrt{\frac{A_2}{A_1}} \leq 2$

$f_{pu} = \frac{P_u}{BN}$

$f_{pu} \leq f_{p(max)}$

Variables

B [in.]	N [in.]	L [in.]	W [in.]	f_c' [psi]	ϕ	P_u [kip]
24.000	24.000	36.000	36.000	4,000	0.650	932.000

Calculations

A_1 [in.^{2}]	A_2 [in.^{2}]
576.00	1,296.00

Results

f_{pu} [psi]	$f_{pu(max)}$ [psi]
1,618	3,315

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	8
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Blue BP (~1200 kips)	Date:	1/28/2025
Fastening point:			

6.2 Plate bending, compression (per AISC DG1, sections 3.1.2, 3.1.3)

$A_1 = B \cdot N$

$A_2 = L \cdot W$

$P_p = 0.85 f'_c A_1 \sqrt{\frac{A_2}{A_1}}, \sqrt{\frac{A_2}{A_1}} \leq 2$

$X = \left\{ \frac{4 d b_f}{(d + b_f)^2} \right\} \frac{P_u}{\phi_c P_p}$

$\lambda = \frac{2\sqrt{X}}{1 + \sqrt{1 - X}} \leq 1$

$m = \frac{N - 0.95d}{2}$

$n = \frac{B - 0.80 b_f}{2}$

$\lambda n' = \lambda \frac{\sqrt{d b_f}}{4}$

$l = \max(m, n, \lambda n')$

$f_{pu} = \frac{P_u}{BN}$

$M_{pl} = \frac{f_{pu} \cdot l^2}{2}$

$\phi M_n = \phi \cdot F_y \cdot \frac{t_p^2}{4}$

$M_{pl} \leq \phi M_n$

Variables

B [in.]	N [in.]	L [in.]	W [in.]	d [in.]
24.000	24.000	36.000	36.000	14.700

b_f [in.]	F_y [psi]	ϕ	t_p [in.]	P_u' [kip]
14.700	50,000	0.900	1.750	932.000

Calculations

A_1 [in.^{2}]	A_2 [in.^{2}]	P_p [kip]	X	λ
576.00	1,296.00	1,909.440	0.751	1.000

m [in.]	n [in.]	$\lambda n'$ [in.]	l [in.]	f_{pu} [psi]
5.017	6.120	3.675	6.120	1,618

Results

M_n [in.lb/in.]	ϕ	ϕM_n [in.lb/in.]	M_{pl} [in.lb/in.]
38,281.27	0.900	34,453.14	30,301.65

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Company:		Page:	9
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Blue BP (~1200 kips)	Date:	1/28/2025
Fastening point:			

7 Installation data

Profile: W shape (AISC), W14X132; (L x W x T x FT) = 14.700 in. x 14.700 in. x 0.645 in. x 1.030 in.

Hole diameter in the fixture: d_f = 0.812 in.

Plate thickness (input): 1.750 in.

Anchor type and diameter: Heavy Hex Head ASTM F 1554 GR. 36 3/4

Item number: not available

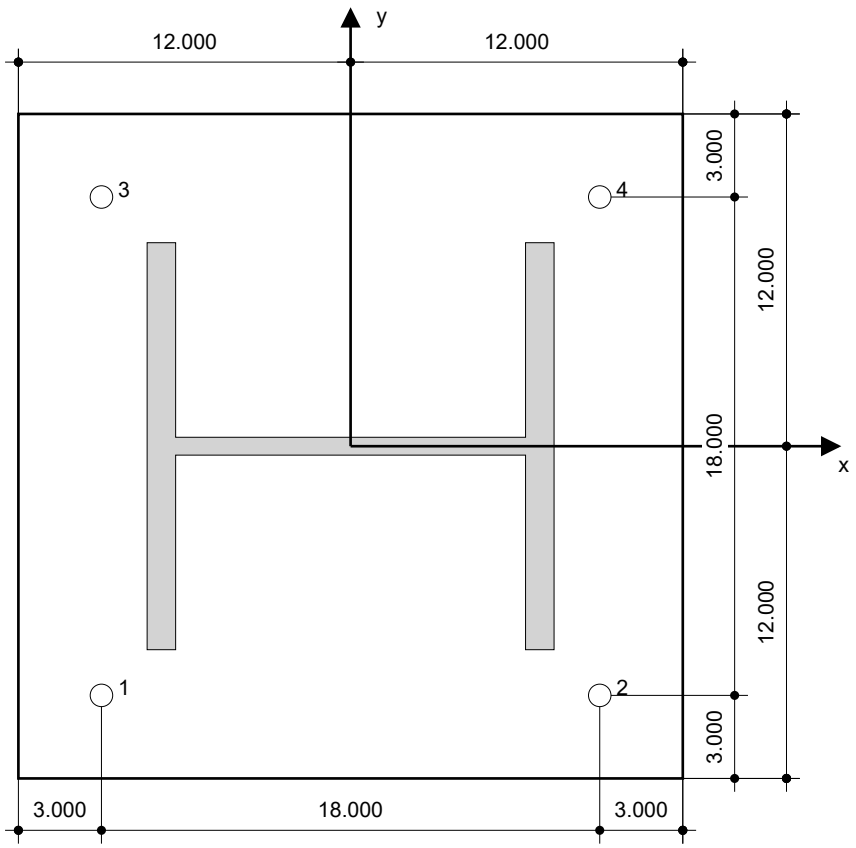
Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 18.000 in.

Minimum thickness of the base material: 19.000 in.

⌀ 3/4 in Heavy Hex Head ASTM F 1554 GR. 36 with 18 in nominal embedment depth per Technical data , cast in place installation per MPII



Coordinates Anchor in.

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-9.000	-9.000	9.000	27.000	9.000	27.000
2	9.000	-9.000	27.000	9.000	9.000	27.000
3	-9.000	9.000	9.000	27.000	27.000	9.000
4	9.000	9.000	27.000	9.000	27.000	9.000

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Company:		Page:	10
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Blue BP (~1200 kips)	Date:	1/28/2025
Fastening point:			

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Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP (~400 kips)	Date:	4/24/2025
Fastening point:			

Specifier's comments:

1 Anchor Design

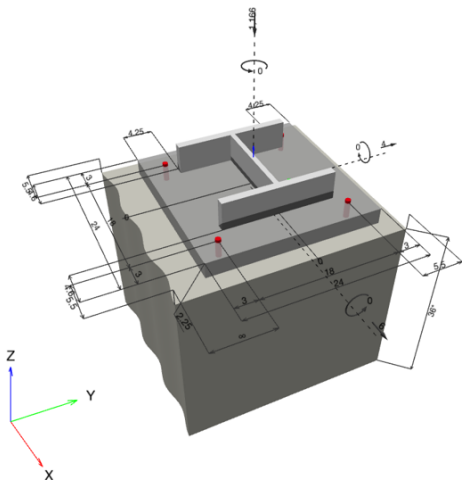
1.1 Input data



Anchor type and diameter:	**Heavy Hex Head ASTM F 1554 GR. 36 3/4**
Item number:	not available
Specification text:	∅ 3/4 in Heavy Hex Head ASTM F 1554 GR. 36 with 18 in nominal embedment depth per Technical data , cast in place installation per MPII,
Effective embedment depth:	h_{ef} = 18.000 in.
Material:	ASTM F 1554
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-19 / CIP
Shear edge breakout verification:	Row closest to edge (Case 3 only from ACI 318-19 Fig. R.17.7.2.1b)
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 2.250 in.
Anchor plate[CBFEM] :	l_x x l_y x t = 24.000 in. x 24.000 in. x 2.250 in.;
Profile:	W shape (AISC), W14X145; (L x W x T x FT) = 14.800 in. x 15.500 in. x 0.680 in. x 1.090 in.
Base material:	cracked concrete, Custom, f_c' = 4,500 psi; h = 36.000 in.
Reinforcement:	tension: not present, shear: not present; edge reinforcement: none or < No. 4 bar

[CBFEM] - The anchor calculation is based on a component-based Finite Element Method (CBFEM)

Geometry [in.] & Loading [kip, ft.kip]



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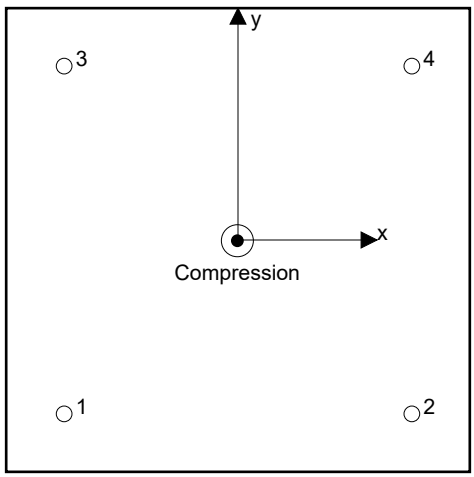
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Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP (~400 kips)	Date:	4/24/2025
Fastening point:			

1.1.1 Design results

Case	Description	Forces [kip] / Moments [ft.kip]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = -1,166.000; V_x = 6.000; V_y = 4.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	89

1.2 Load case/Resulting anchor forces



Anchor reactions [kip]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	-0.008	1.697	1.499	0.796
2	-0.008	1.690	1.502	0.775
3	-0.008	1.917	1.491	1.205
4	-0.008	1.943	1.509	1.225

Resulting tension force in (x/y)=(-/-): 0.000 [kip]
Resulting compression force in (x/y)=(-0.032/-0.038): 1,166.185 [kip]

Anchor forces are calculated based on a component-based Finite Element Method (CBFEM)

1.3 Tension load

	Load N_{ua} [kip]	Capacity ϕ N_n [kip]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	-0.008	14.529	1	OK
Pullout Strength*	-0.008	22.957	-0	not recommended
Concrete Breakout Failure**	N/A	N/A	N/A	N/A
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (anchors in tension)

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Company:		Page:	3
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP (~400 kips)	Date:	4/24/2025
Fastening point:			

1.3.1 Steel Strength

$N_{sa} = A_{se,N} f_{uta}$ ACI 318-19 Eq. (17.6.1.2)
$\phi N_{sa} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.33	58,000

Calculations

N_{sa} [kip]
19.372

Results

N_{sa} [kip]	ϕ_{steel}	ϕN_{sa} [kip]	N_{ua} [kip]
19.372	0.750	14.529	-0.008

The steel proof was done for the highest absolute force per anchor - in this case compression loading. Please be aware that buckling should be verified separately

1.3.2 Pullout Strength

$N_{pN} = \psi_{c,p} N_p$ ACI 318-19 Eq. (17.6.3.1)
$N_p = 8 A_{brg} f_c'$ ACI 318-19 Eq. (17.6.3.2.2a)
$\phi N_{pN} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$\psi_{c,p}$	A_{brg} [in.^{2}]	λ_a	f_c' [psi]
1.000	0.91	1.000	4,500

Calculations

N_p [kip]
32.796

Results

N_{pn} [kip]	$\phi_{concrete}$	ϕN_{pn} [kip]	N_{ua} [kip]
32.796	0.700	22.957	-0.008

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	4
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP (~400 kips)	Date:	4/24/2025
Fastening point:			

1.4 Shear load

	Load V_{ua} [kip]	Capacity ϕ V_n [kip]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	1.943	7.555	26	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	7.211	85.120	9	OK
Concrete edge failure in direction y+**	7.211	8.176	89	OK

* highest loaded anchor **anchor group (relevant anchors)
When the input edge distance is set to "infinity", edge breakout verification is not performed in that direction

1.4.1 Steel Strength

$V_{sa} = 0.6\ A_{se,V}\ f_{uta}$ ACI 318-19 Eq. (17.7.1.2b)
$\phi\ V_{steel} \geq V_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.33	58,000

Calculations

V_{sa} [kip]
11.623

Results

V_{sa} [kip]	ϕ_{steel}	$\phi\ V_{sa}$ [kip]	V_{ua} [kip]
11.623	0.650	7.555	1.943

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	5
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP (~400 kips)	Date:	4/24/2025
Fastening point:			

1.4.2 Pryout Strength

$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \right]$ ACI 318-19 Eq. (17.7.3.1b)

$\phi V_{cpg} \geq V_{ua}$ ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9 h_{ef}^2$ ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e'_N}{3 h_{ef}}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ ACI 318-19 Eq. (17.6.2.2.1)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	6.000	0.000	0.000	5.500

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.000	-	24	1.000	4,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [kip]
942.50	324.00	1.000	1.000	0.883	1.000	23.662

Results

V_{cpg} [kip]	$\phi_{concrete}$	ϕV_{cpg} [kip]	V_{ua} [kip]
121.600	0.700	85.120	7.211

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Company:		Page:	6
Address:		Specifier:	
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Fastening point:			

1.4.3 Concrete edge failure in direction y+

$V_{cbg} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \psi_{ec,V}\ \psi_{ed,V}\ \psi_{c,V}\ \psi_{h,V}\ \psi_{parallel,V}\ V_b$ — ACI 318-19 Eq. (17.7.2.1b)

$\phi\ V_{cbg} \geq V_{ua}$ — ACI 318-19 Table 17.5.2

A_{Vc} see ACI 318-19, Section 17.7.2.1, Fig. R 17.7.2.1(b)*

$A_{Vc0} = 4.5\ c_{a1}^2$ — ACI 318-19 Eq. (17.7.2.1.3)

$\psi_{ec,V} = \left(\frac{1}{1 + \frac{e'_v}{1.5c_{a1}}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.3.1)

$\psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.4.1b)

$\psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ — ACI 318-19 Eq. (17.7.2.6.1)

$V_b = 9\ \lambda_a\ \sqrt{f'_c}\ c_{a1}^{1.5}$ — ACI 318-19 Eq. (17.7.2.2.1b)

Variables

c_{a1} [in.]	c_{a2} [in.]	e_{cV} [in.]	$\psi_{c,V}$	h_a [in.]
5.500	5.500	0.001	1.000	36.000

l_e [in.]	λ_a	d_a [in.]	f'_c [psi]	$\psi_{parallel,V}$
6.000	1.000	0.750	4,500	1.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\psi_{ec,V}$	$\psi_{ed,V}$	$\psi_{h,V}$	V_b [kip]
226.87	136.12	1.000	0.900	1.000	7.787

Results

V_{cbg} [kip]	$\phi_{concrete}$	$\phi\ V_{cbg}$ [kip]	V_{ua} [kip]
11.680	0.700	8.176	7.211

*Anchor row defined by: Anchor 3, 4; Case 3 controls

When the input edge distance is set to "infinity", edge breakout verification is not performed in that direction

1.5 Combined tension and shear loads, per ACI 318-19 section 17.8

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.001	0.882	1.000	74	OK

$\beta_{NV} = (\beta_N + \beta_V) / 1.2 <= 1$

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Company:		Page:	7
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP (~400 kips)	Date:	4/24/2025
Fastening point:			

1.6 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates as per current regulations (ETAG 001/Annex C, EOTA TR029, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid base plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- The equations presented in this report are based on imperial units. When inputs are displayed in metric units, the user should be aware that the equations remain in their imperial format.
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- For additional information about ACI 318 strength design provisions, please go to https://viewer.joomag.com/profis-design-guide-us-en-summer-2021/0841849001625154758?short&/
- Attention! In case of compressive anchor forces a buckling check as well as the proof of the local load transfer into and within the base material (incl. punching) has to be done separately.
- The anchor design methods in PROFIS Engineering require rigid anchor plates, as per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means that the anchor plate should be sufficiently rigid to prevent load re-distribution to the anchors due to elastic/plastic displacements. The user accepts that the anchor plate is considered close to rigid by engineering judgment."

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Company:		Page:	8
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP (~400 kips)	Date:	4/24/2025
Fastening point:			

1.7 Installation data

Profile: W shape (AISC), W14X145; (L x W x T x FT) = 14.800 in. x 15.500 in. x 0.680 in. x 1.090 in.

Hole diameter in the fixture: d_f = 0.812 in.

Plate thickness (input): 2.250 in.

Anchor type and diameter: Heavy Hex Head ASTM F 1554 GR. 36 3/4

Item number: not available

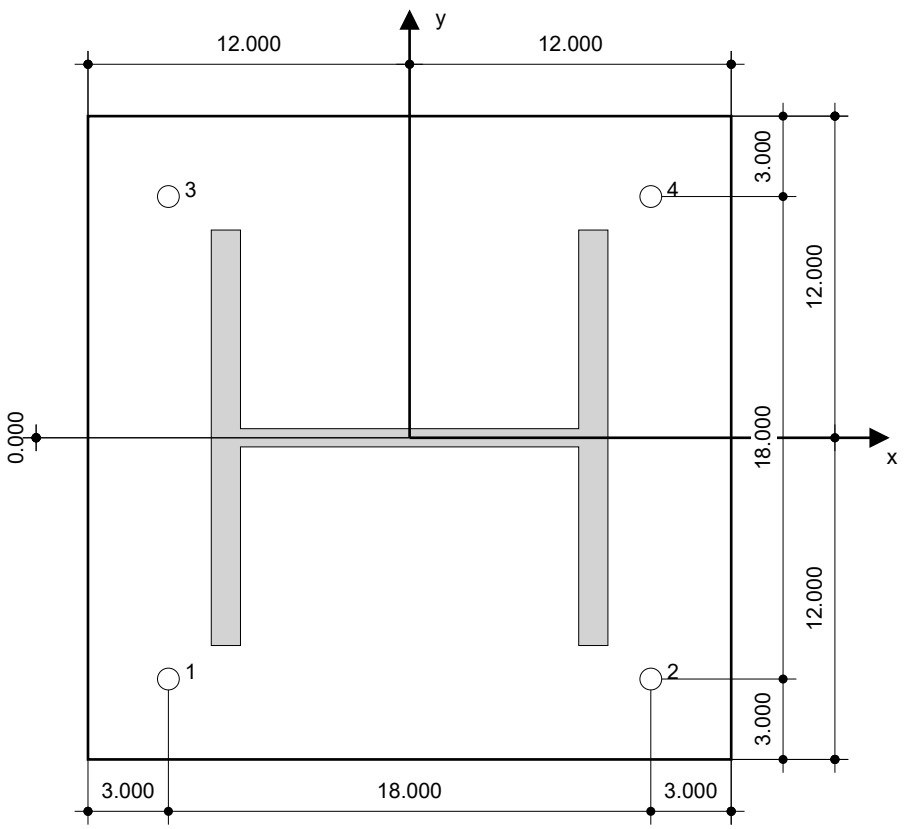
Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 18.000 in.

Minimum thickness of the base material: 19.000 in.

⌀ 3/4 in Heavy Hex Head ASTM F 1554 GR. 36 with 18 in nominal embedment depth per Technical data , cast in place installation per MPII



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-9.000	-9.000	5.500	23.500	-	23.500
2	9.000	-9.000	23.500	5.500	-	23.500
3	-9.000	9.000	5.500	23.500	-	5.500
4	9.000	9.000	23.500	5.500	-	5.500

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Company:		Page:	9
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP (~400 kips)	Date:	4/24/2025
Fastening point:			

2 Anchor plate design

2.1 Input data

Anchor plate:	Shape: Rectangular l_x x l_y x t = 24.000 in x 24.000 in x 2.250 in Calculation: CBFEM Material: ASTM A572 Gr.50; F_y = 50,000 psi; ε_{lim} = 5.00%
Anchor type and size:	Heavy Hex Head ASTM F 1554 GR. 36 3/4, h_{ef} = 18.000 in
Anchor stiffness:	The anchor is modeled considering stiffness values determined from load displacement curves tested in an independent laboratory. Please note that no simple replacement of the anchor is possible as the anchor stiffness has a major impact on the load distribution results.
Design method:	AISC and LRFD-based design using component-based FEM
Stand-off installation:	e_b = 0.000 in (No stand-off); t = 2.250 in
Profile:	W14X145; (L x W x T x FT) = 14.800 in x 15.500 in x 0.680 in x 1.090 in Material: ASTM A992; F_y = 50,000 psi; ε_{lim} = 5.00% Eccentricity x: 0.000 in Eccentricity y: -0.000 in
Base material:	Cracked concrete; Custom; $f_{c,cyl}$ = 4,500 psi; h = 36.000 in; E = 4,351,132 psi; G = 1,740,453 psi; v = 0.20; D = 156.07 lb/ft³
Welds (profile to anchor plate):	Type of redistribution: Plastic Material: E70xx
Mesh size:	Number of elements on edge: 8 Min. size of element: 0.394 in Max. size of element: 1.969 in

2.2 Summary

	Description	Profile		Anchor plate			Concrete [%]
		σ_{Ed} [psi]	ε_{Pl} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Hole bearing [%]	
1	Combination 1	48,478	0.05	21,879	0.00	1	68

2.3 Anchor plate classification

Results below are displayed for the decisive load combinations: Combination 1

Anchor tension forces	Equivalent rigid anchor plate (CBFEM)	Component-based Finite Element Method (CBFEM) anchor plate design
Anchor 1	-0.010 kip	-0.008 kip
Anchor 2	-0.010 kip	-0.008 kip
Anchor 3	-0.010 kip	-0.008 kip
Anchor 4	-0.010 kip	-0.008 kip

User accepted to consider the selected anchor plate as rigid by his/her engineering judgement. This means the anchor design guidelines can be applied.

2.4 Profile/Stiffeners/Plate

Profile and stiffeners are verified at the level of the steel to concrete connection. The connection design does not replace the steel design for critical cross sections, which should be performed outside of PROFIS Engineering.

2.4.1 Equivalent stress and plastic strain

Part	Load combination	Material	f_y [psi]	ε_{lim} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Status
Plate	Combination 1	ASTM A572	50,000	5.00	21,879	0.00	OK

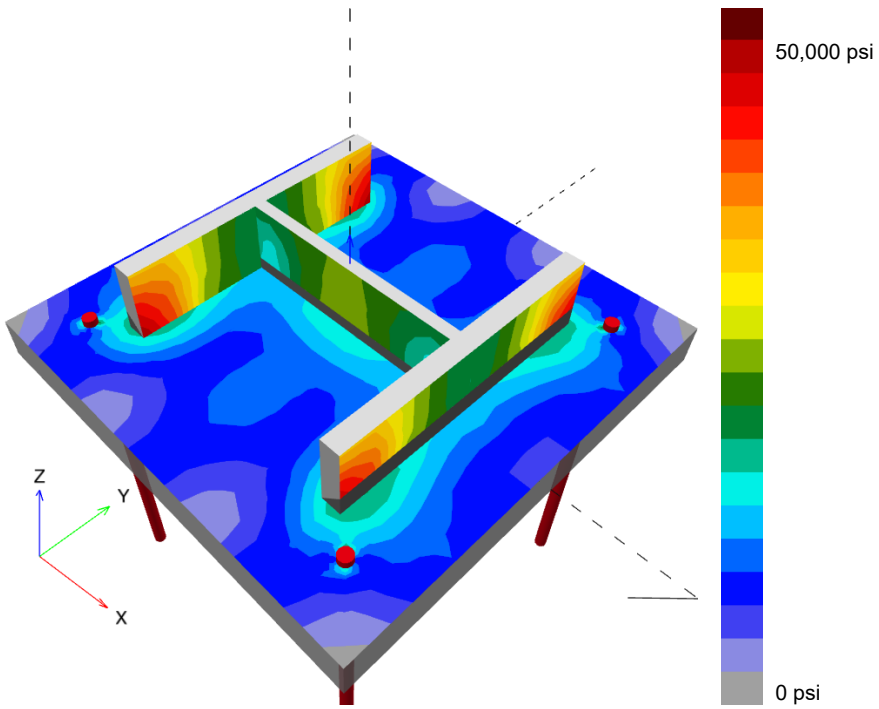
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Company:		Page:	10
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP (~400 kips)	Date:	4/24/2025
Fastening point:			

Part	Load combination	Material	f_y [psi]	ε_{lim} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Status
		Gr.50					
Profile	Combination 1	ASTM A992	50,000	5.00	48,478	0.05	OK
Profile	Combination 1	ASTM A992	50,000	5.00	48,315	0.05	OK
Profile	Combination 1	ASTM A992	50,000	5.00	27,679	0.00	OK

2.4.1.1 Equivalent stress

Results below are displayed for the decisive load combination: 1 - Combination 1

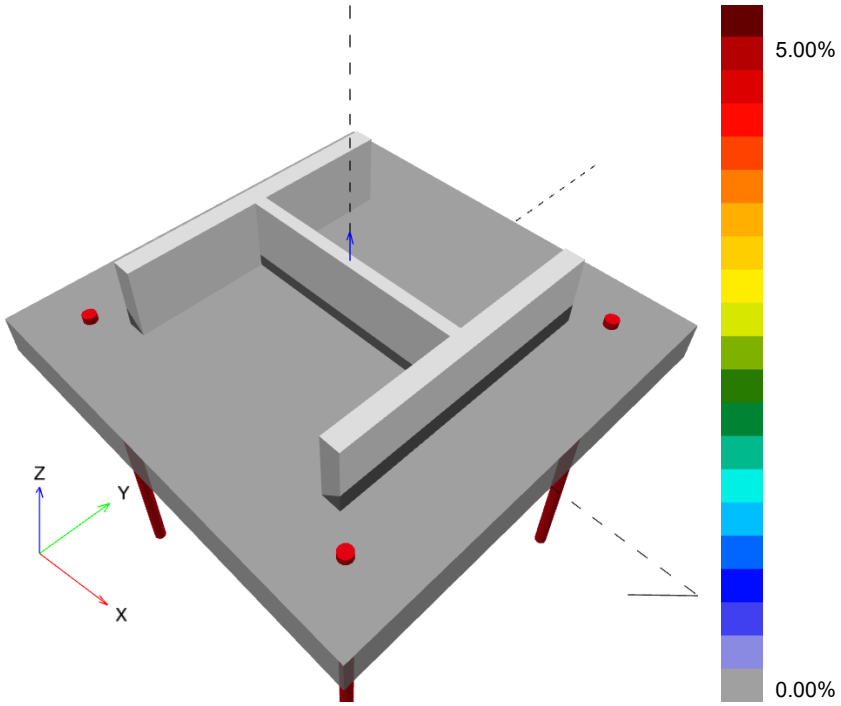


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Address:		Specifier:	
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Design:	Yellow BP (~400 kips)	Date:	4/24/2025
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2.4.1.2 Plastic strain

Results below are displayed for the decisive load combination: 1 - Combination 1



2.4.2 Plate hole bearing resistance, AISC 360-16 Section J3

Decisive load combination: 1 - Combination 1

Equations

$R_n = \min(1.2\, l_c\, t\, F_u\, ,\ 2.4\, d\, t\, F_u)$ (AISC 360-16 J3-6a, c)

$\Phi R_n = 0.75\, R_n$

$V \leq \Phi R_n$

Variables

	l_c [in]	t [in]	F_u [psi]	d [in]	R_n [kip]
Anchor 1	2.990	2.250	65,000	0.750	263.250
Anchor 2	6.139	2.250	65,000	0.750	263.250
Anchor 3	3.451	2.250	65,000	0.750	263.250
Anchor 4	26.646	2.250	65,000	0.750	263.250

Results

	V [kip]	ΦR_n [kip]	Utilization [%]	Status
Anchor 1	1.697	197.437	1	OK
Anchor 2	1.690	197.437	1	OK
Anchor 3	1.917	197.437	1	OK

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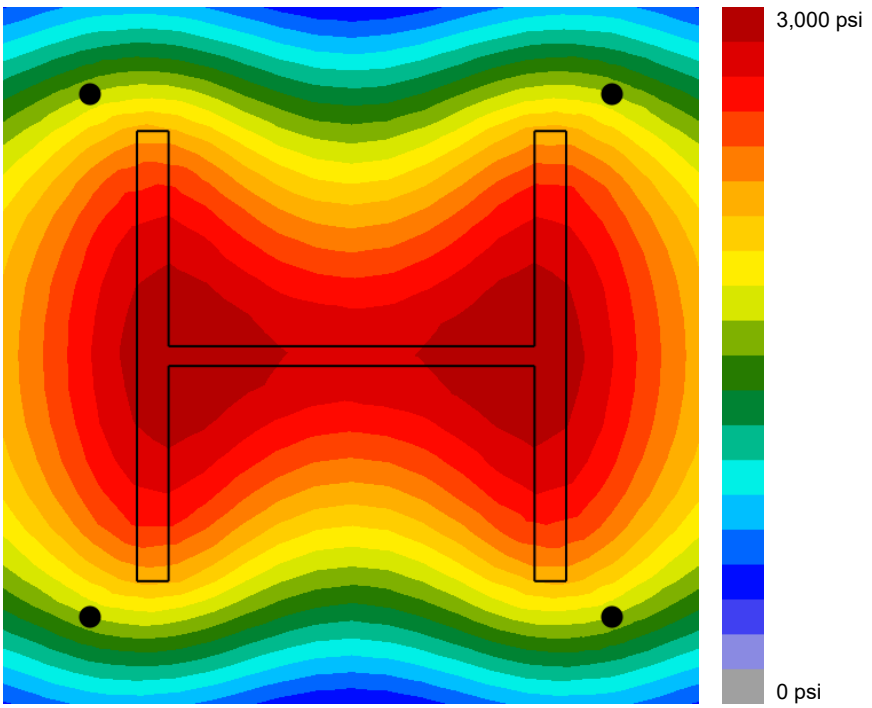
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Design:	Yellow BP (~400 kips)	Date:	4/24/2025
Fastening point:			

	V [kip]	ΦR_n [kip]	Utilization [%]	Status
Anchor 4	1.944	197.437	1	OK

2.5 Concrete

Decisive load combination: 1 - Combination 1

2.5.1 Compression in concrete under the anchor plate



2.5.2 Concrete block compressive strength resistance check, AISC 360-16 Section J8

Equations

$$F_p = \Phi f_{p,max}$$

$$f_{p,max} = 0.85 f_c' \sqrt{\left(\frac{A_2}{A_1}\right)} \le 1.7 f_c; \quad \sqrt{\left(\frac{A_2}{A_1}\right)} \le 2$$

$$\sigma = \frac{N}{A_1}$$

$$\text{Utilization} = \frac{\sigma}{F_p}$$

Variables

N [kip]	f_c' [psi]	Φ	A_1 [in²]	A_2 [in²]
1,166.185	4,500	0.65	574.13	836.71

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Company:		Page:	13
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP (~400 kips)	Date:	4/24/2025
Fastening point:			

Results

Load combination	F_p [psi]	σ [psi]	Utilization [%]	Status
Combination 1	3,001	2,031	68	OK

2.6 Symbol explanation

A_1	Loaded area of concrete
A_2	Supporting area
d	Nominal diameter of the bolt
ε_{lim}	Limit plastic strain
ε_{Pl}	Plastic strain from CBFEM results
f_c	Concrete compressive strength
f_c'	Concrete compressive strength
F_u	Specified minimum tensile strength of the connected material
F_p	Concrete block design bearing strength
$f_{p,max}$	Concrete block design bearing strength maximum
f_y	Yield strength
l_c	Clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material
N	Resulting compression force
σ	Average stress in concrete
σ_{Ed}	Equivalent stress
Φ	Resistance factor
ΦR_n	Factored resistance
t	Thickness of the anchor plate
V	Resultant of shear forces Vy, Vz in bolt.

2.7 Warnings

- By using the CBFEM calculation functionality of PROFIS Engineering you may act outside the applicable design codes and your specified anchor plate may not behave rigid. Please, validate the results with a professional designer and/or structural engineer to ensure suitability and adequacy for your specific jurisdiction and project requirements.
- The anchor is modeled considering stiffness values determined from load displacement curves tested in an independent laboratory. Please note that no simple replacement of the anchor is possible as the anchor stiffness has a major impact on the load distribution results.

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Company:		Page:	14
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP (~400 kips)	Date:	4/24/2025
Fastening point:			

3 Summary of results

Design of the anchor plate, anchors, welds and other elements are based on CBFEM (component based finite element method) and AISC.

	Load combination	Max. utilization	Status
Anchors	Combination 1	89%	OK
Anchor plate	Combination 1	44%	OK
Concrete	Combination 1	68%	OK
Profile	Combination 1	97%	OK

Fastening meets the design criteria!

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Company:		Page:	15
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP (~400 kips)	Date:	4/24/2025
Fastening point:			

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Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Purple BP (~1600 kips)	Date:	1/28/2025
Fastening point:			

Specifier's comments:

1 Anchor Design

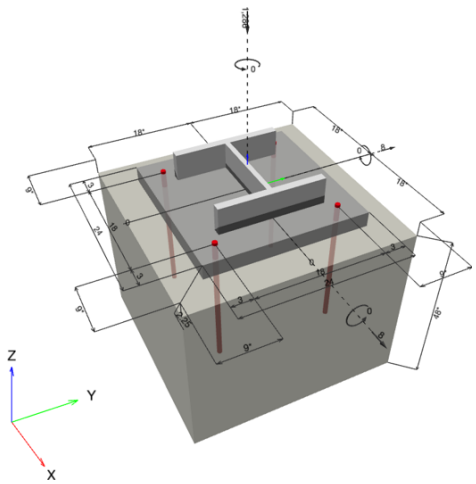
1.1 Input data



Anchor type and diameter:	**Heavy Hex Head ASTM F 1554 GR. 36 3/4**
Item number:	not available
Specification text:	∅ 3/4 in Heavy Hex Head ASTM F 1554 GR. 36 with 18 in nominal embedment depth per Technical data , cast in place installation per MPII
Effective embedment depth:	h_{ef} = 18.000 in.
Material:	ASTM F 1554
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-19 / CIP
Shear edge breakout verification:	Row closest to edge (Case 3 only from ACI 318-19 Fig. R.17.7.2.1b)
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 2.250 in.
Anchor plate[CBFEM] :	l_x x l_y x t = 24.000 in. x 24.000 in. x 2.250 in.;
Profile:	W shape (AISC), W14X159; (L x W x T x FT) = 15.000 in. x 15.600 in. x 0.745 in. x 1.190 in.
Base material:	cracked concrete, 4000, f_c' = 4,000 psi; h = 48.000 in.
Reinforcement:	tension: not present, shear: not present; edge reinforcement: none or < No. 4 bar

[CBFEM] - The anchor calculation is based on a component-based Finite Element Method (CBFEM)

Geometry [in.] & Loading [kip, ft.kip]



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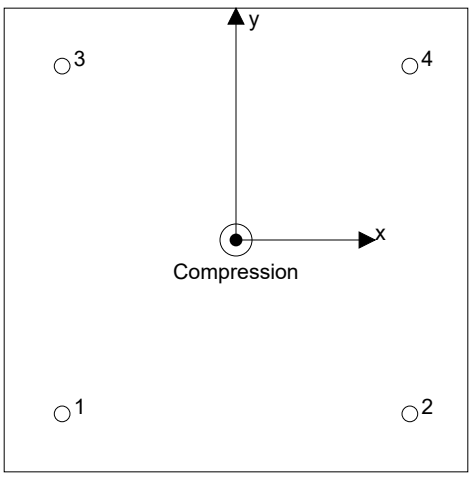
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Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Purple BP (~1600 kips)	Date:	1/28/2025
Fastening point:			

1.1.1 Design results

Case	Description	Forces [kip] / Moments [ft.kip]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = -1,286.000; V_x = 8.000; V_y = 8.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	88

1.2 Load case/Resulting anchor forces



Anchor reactions [kip]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	-0.011	2.665	2.007	1.754
2	-0.011	2.640	1.994	1.730
3	-0.011	3.003	1.992	2.247
4	-0.011	3.030	2.007	2.270

Resulting tension force in (x/y)=(-/-): 0.000 [kip]
Resulting compression force in (x/y)=(0.000/0.000): 1,285.981 [kip]

Anchor forces are calculated based on a component-based Finite Element Method (CBFEM)

1.3 Tension load

	Load N_{ua} [kip]	Capacity ϕ N_n [kip]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	-0.011	14.529	1	OK
Pullout Strength*	-0.011	20.406	-0	not recommended
Concrete Breakout Failure**	N/A	N/A	N/A	N/A
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (anchors in tension)

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Company:		Page:	3
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Purple BP (~1600 kips)	Date:	1/28/2025
Fastening point:			

1.3.1 Steel Strength

$N_{sa} = A_{se,N} f_{uta}$ ACI 318-19 Eq. (17.6.1.2)
$\phi N_{sa} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.33	58,000

Calculations

N_{sa} [kip]
19.372

Results

N_{sa} [kip]	ϕ_{steel}	ϕN_{sa} [kip]	N_{ua} [kip]
19.372	0.750	14.529	-0.011

The steel proof was done for the highest absolute force per anchor - in this case compression loading. Please be aware that buckling should be verified separately

1.3.2 Pullout Strength

$N_{pN} = \psi_{c,p} N_p$ ACI 318-19 Eq. (17.6.3.1)
$N_p = 8 A_{brg} f'_c$ ACI 318-19 Eq. (17.6.3.2.2a)
$\phi N_{pN} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$\psi_{c,p}$	A_{brg} [in.^{2}]	λ_a	f'_c [psi]
1.000	0.91	1.000	4,000

Calculations

N_p [kip]
29.152

Results

N_{pn} [kip]	$\phi_{concrete}$	ϕN_{pn} [kip]	N_{ua} [kip]
29.152	0.700	20.406	-0.011

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Company:		Page:	4
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Purple BP (~1600 kips)	Date:	1/28/2025
Fastening point:			

1.4 Shear load

	Load V_{ua} [kip]	Capacity ϕ V_n [kip]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	3.030	7.555	41	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	11.314	124.927	10	OK
Concrete edge failure in direction y+**	11.314	12.909	88	OK

* highest loaded anchor **anchor group (relevant anchors)

1.4.1 Steel Strength

$V_{sa} = 0.6\ A_{se,V}\ f_{uta}$ ACI 318-19 Eq. (17.7.1.2b)
$\phi\ V_{steel} \geq V_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.33	58,000

Calculations

V_{sa} [kip]
11.623

Results

V_{sa} [kip]	ϕ_{steel}	$\phi\ V_{sa}$ [kip]	V_{ua} [kip]
11.623	0.650	7.555	3.030

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	5
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Purple BP (~1600 kips)	Date:	1/28/2025
Fastening point:			

1.4.2 Pryout Strength

$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \right]$ ACI 318-19 Eq. (17.7.3.1b)

$\phi V_{cpg} \geq V_{ua}$ ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9 h_{ef}^2$ ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e'_N}{3 h_{ef}}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ ACI 318-19 Eq. (17.6.2.2.1)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	6.000	0.000	0.000	9.000

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.000	-	24	1.000	4,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [kip]
1,296.00	324.00	1.000	1.000	1.000	1.000	22.308

Results

V_{cpg} [kip]	$\phi_{concrete}$	ϕV_{cpg} [kip]	V_{ua} [kip]
178.467	0.700	124.927	11.314

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	6
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Purple BP (~1600 kips)	Date:	1/28/2025
Fastening point:			

1.4.3 Concrete edge failure in direction y+

$V_{cbg} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \Psi_{ec,V} \Psi_{ed,V} \Psi_{c,V} \Psi_{h,V} \Psi_{parallel,V} V_b$ — ACI 318-19 Eq. (17.7.2.1b)

$\phi V_{cbg} \geq V_{ua}$ — ACI 318-19 Table 17.5.2

A_{Vc} see ACI 318-19, Section 17.7.2.1, Fig. R 17.7.2.1(b)*

$A_{Vc0} = 4.5 c_{a1}^2$ — ACI 318-19 Eq. (17.7.2.1.3)

$\Psi_{ec,V} = \left(\frac{1}{1 + \frac{e'_v}{1.5c_{a1}}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.3.1)

$\Psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.4.1b)

$\Psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ — ACI 318-19 Eq. (17.7.2.6.1)

$V_b = 9 \lambda_a \sqrt{f'_c} c_{a1}^{1.5}$ — ACI 318-19 Eq. (17.7.2.2.1b)

Variables

c_{a1} [in.]	c_{a2} [in.]	e_{cV} [in.]	$\Psi_{c,V}$	h_a [in.]
9.000	9.000	0.001	1.000	48.000

l_e [in.]	λ_a	d_a [in.]	f'_c [psi]	$\Psi_{parallel,V}$
6.000	1.000	0.750	4,000	1.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{h,V}$	V_b [kip]
486.00	364.50	1.000	0.900	1.000	15.369

Results

V_{cbg} [kip]	$\phi_{concrete}$	ϕV_{cbg} [kip]	V_{ua} [kip]
18.442	0.700	12.909	11.314

*Anchor row defined by: Anchor 3, 4; Case 3 controls

1.5 Combined tension and shear loads, per ACI 318-19 section 17.8

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.001	0.876	1.000	74	OK

$\beta_{NV} = (\beta_N + \beta_V) / 1.2 <= 1$

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Company:		Page:	7
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Purple BP (~1600 kips)	Date:	1/28/2025
Fastening point:			

1.6 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates as per current regulations (ETAG 001/Annex C, EOTA TR029, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid base plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- The equations presented in this report are based on imperial units. When inputs are displayed in metric units, the user should be aware that the equations remain in their imperial format.
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- For additional information about ACI 318 strength design provisions, please go to https://submittals.us.hilti.com/PROFISAnchorDesignGuide/
- Attention! In case of compressive anchor forces a buckling check as well as the proof of the local load transfer into and within the base material (incl. punching) has to be done separately.
- The anchor design methods in PROFIS Engineering require rigid anchor plates, as per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means that the anchor plate should be sufficiently rigid to prevent load re-distribution to the anchors due to elastic/plastic displacements. The user accepts that the anchor plate is considered close to rigid by engineering judgment."

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Company:		Page:	8
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Purple BP (~1600 kips)	Date:	1/28/2025
Fastening point:			

1.7 Installation data

Profile: W shape (AISC), W14X159; (L x W x T x FT) = 15.000 in. x 15.600 in. x 0.745 in. x 1.190 in.

Hole diameter in the fixture: d_f = 0.812 in.

Plate thickness (input): 2.250 in.

Anchor type and diameter: Heavy Hex Head ASTM F 1554 GR. 36 3/4

Item number: not available

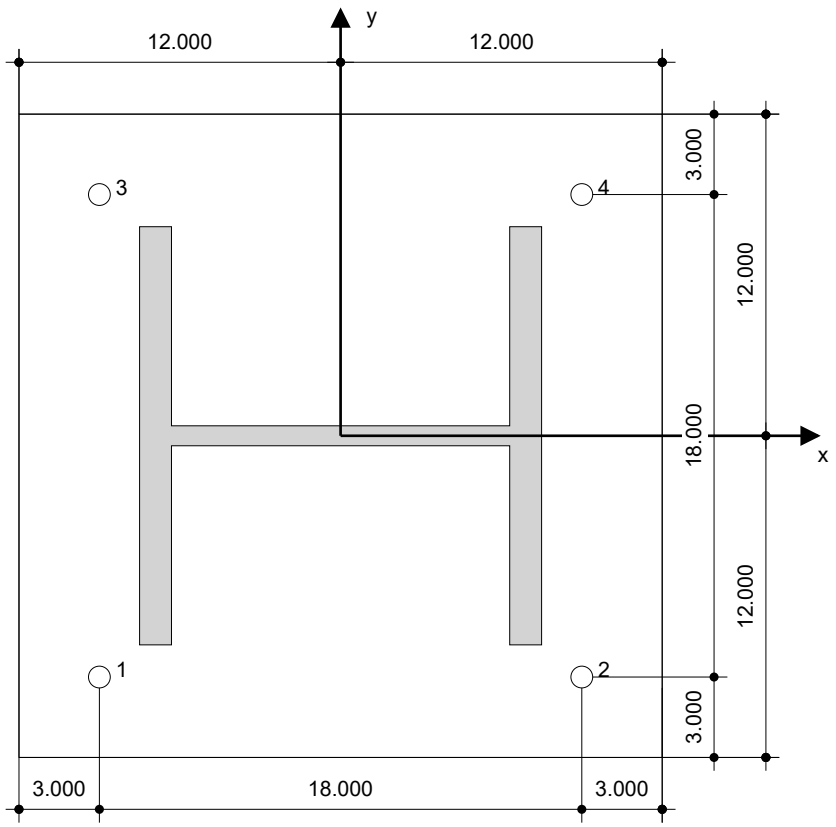
Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 18.000 in.

Minimum thickness of the base material: 19.000 in.

⌀ 3/4 in Heavy Hex Head ASTM F 1554 GR. 36 with 18 in nominal embedment depth per Technical data , cast in place installation per MPII



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-9.000	-9.000	9.000	27.000	9.000	27.000
2	9.000	-9.000	27.000	9.000	9.000	27.000
3	-9.000	9.000	9.000	27.000	27.000	9.000
4	9.000	9.000	27.000	9.000	27.000	9.000

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	9
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Purple BP (~1600 kips)	Date:	1/28/2025
Fastening point:			

2 Anchor plate design

2.1 Input data

Anchor plate:	Shape: Rectangular l_x x l_y x t = 24.000 in x 24.000 in x 2.250 in Calculation: CBFEM Material: ASTM A572 Gr.50; F_y = 50,000 psi; ε_{lim} = 5.00%
Anchor type and size:	Heavy Hex Head ASTM F 1554 GR. 36 3/4, h_{ef} = 18.000 in
Anchor stiffness:	The anchor is modeled considering stiffness values determined from load displacement curves tested in an independent laboratory. Please note that no simple replacement of the anchor is possible as the anchor stiffness has a major impact on the load distribution results.
Design method:	AISC and LRFD-based design using component-based FEM
Stand-off installation:	e_b = 0.000 in (No stand-off); t = 2.250 in
Profile:	W14X159; (L x W x T x FT) = 15.000 in x 15.600 in x 0.745 in x 1.190 in Material: ASTM A992; F_y = 50,000 psi; ε_{lim} = 5.00% Eccentricity x: 0.000 in Eccentricity y: 0.000 in
Base material:	Cracked concrete; 4000; $f_{c,cyl}$ = 4,000 psi; h = 48.000 in
Welds (profile to anchor plate):	Type of redistribution: Plastic Material: E70xx
Mesh size:	Number of elements on edge: 8 Min. size of element: 0.394 in Max. size of element: 1.969 in

2.2 Summary

	Description	Profile		Anchor plate			Concrete [%]
		σ_{Ed} [psi]	ε_{Pl} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Hole bearing [%]	
1	Combination 1	49,578	0.09	24,465	0.00	2	69

2.3 Anchor plate classification

Results below are displayed for the decisive load combinations: Combination 1

Anchor tension forces	Equivalent rigid anchor plate (CBFEM)	Component-based Finite Element Method (CBFEM) anchor plate design
Anchor 1	-0.013 kip	-0.011 kip
Anchor 2	-0.013 kip	-0.011 kip
Anchor 3	-0.013 kip	-0.011 kip
Anchor 4	-0.013 kip	-0.011 kip

User accepted to consider the selected anchor plate as rigid by his/her engineering judgement. This means the anchor design guidelines can be applied.

2.4 Profile/Stiffeners/Plate

Profile and stiffeners are verified at the level of the steel to concrete connection. The connection design does not replace the steel design for critical cross sections, which should be performed outside of PROFIS Engineering.

2.4.1 Equivalent stress and plastic strain

Part	Load combination	Material	f_y [psi]	ε_{lim} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Status
Plate	Combination 1	ASTM A572 Gr.50	50,000	5.00	24,465	0.00	OK

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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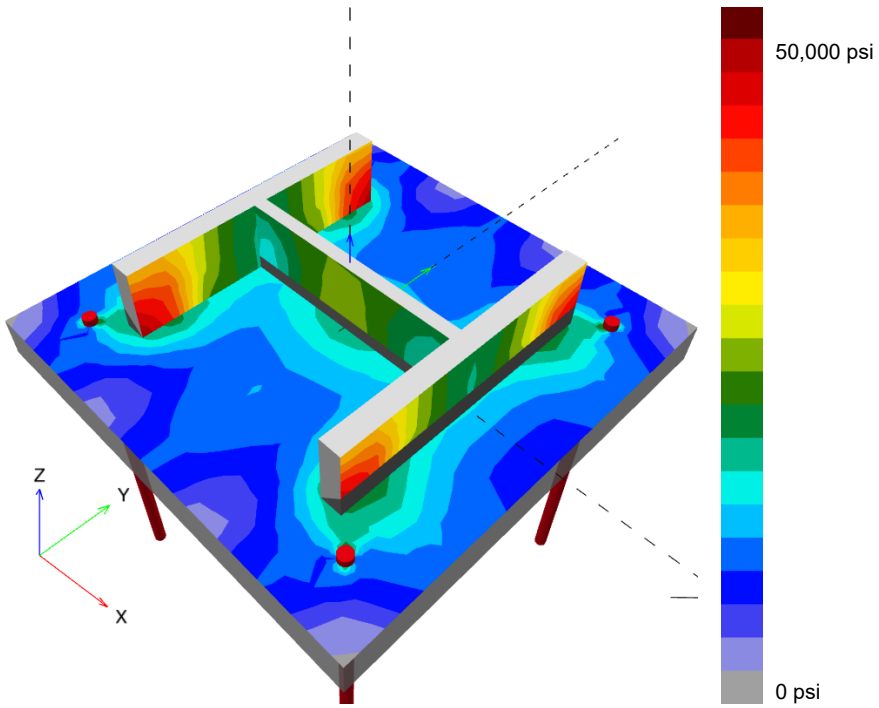
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Company:		Page:	10
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Purple BP (~1600 kips)	Date:	1/28/2025
Fastening point:			

Part	Load combination	Material	f_y [psi]	ε_{lim} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Status
Profile	Combination 1	ASTM A992	50,000	5.00	49,624	0.08	OK
Profile	Combination 1	ASTM A992	50,000	5.00	49,578	0.09	OK
Profile	Combination 1	ASTM A992	50,000	5.00	27,904	0.00	OK

2.4.1.1 Equivalent stress

Results below are displayed for the decisive load combination: 1 - Combination 1

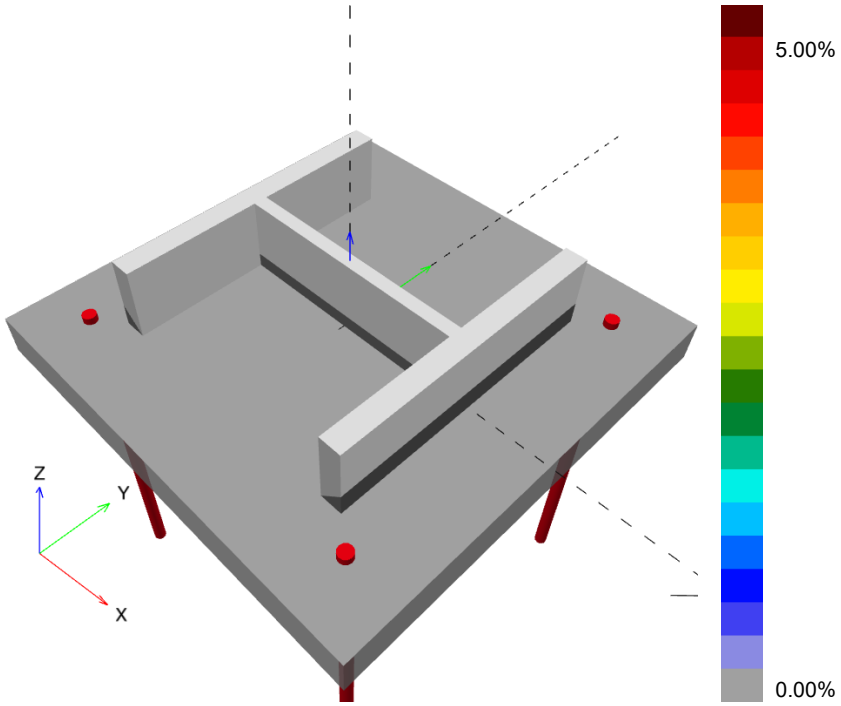


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Company:		Page:	11
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Purple BP (~1600 kips)	Date:	1/28/2025
Fastening point:			

2.4.1.2 Plastic strain

Results below are displayed for the decisive load combination: 1 - Combination 1



2.4.2 Plate hole bearing resistance, AISC 360-16 Section J3

Decisive load combination: 1 - Combination 1

Equations

$R_n = \min(1.2\, l_c\, t\, F_u\, ,\ 2.4\, d\, t\, F_u)$ (AISC 360-16 J3-6a, c)

$\Phi R_n = 0.75\, R_n$

$V \leq \Phi R_n$

Variables

	l_c [in]	t [in]	F_u [psi]	d [in]	R_n [kip]
Anchor 1	3.578	2.250	65,000	0.750	263.250
Anchor 2	4.172	2.250	65,000	0.750	263.250
Anchor 3	4.115	2.250	65,000	0.750	263.250
Anchor 4	27.626	2.250	65,000	0.750	263.250

Results

	V [kip]	ΦR_n [kip]	Utilization [%]	Status
Anchor 1	2.665	197.437	2	OK
Anchor 2	2.640	197.437	2	OK
Anchor 3	3.003	197.437	2	OK

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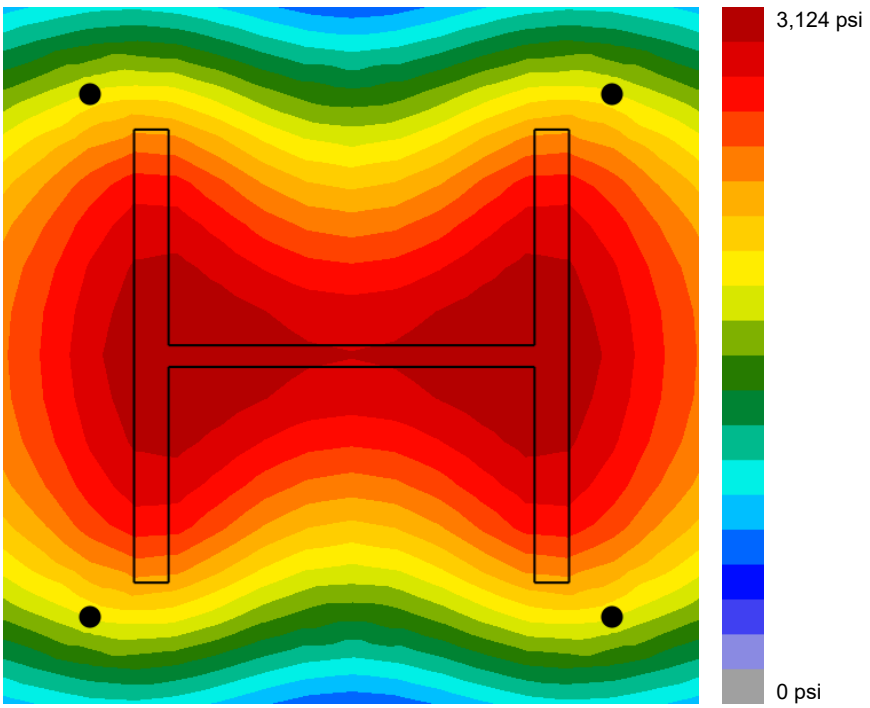
Company:		Page:	12
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Purple BP (~1600 kips)	Date:	1/28/2025
Fastening point:			

	V [kip]	ΦR_n [kip]	Utilization [%]	Status
Anchor 4	3.030	197.437	2	OK

2.5 Concrete

Decisive load combination: 1 - Combination 1

2.5.1 Compression in concrete under the anchor plate



2.5.2 Concrete block compressive strength resistance check, AISC 360-16 Section J8

Equations

$$F_p = \Phi f_{p,max}$$

$$f_{p,max} = 0.85 f_c' \sqrt{\left(\frac{A_2}{A_1}\right)} \le 1.7 f_c; \quad \sqrt{\left(\frac{A_2}{A_1}\right)} \le 2$$

$$\sigma = \frac{N}{A_1}$$

$$\text{Utilization} = \frac{\sigma}{F_p}$$

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Company:		Page:	13
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Purple BP (~1600 kips)	Date:	1/28/2025
Fastening point:			

Variables

N [kip]	f_c' [psi]	Φ	A_1 [in²]	A_2 [in²]
1,285.981	4,000	0.65	574.13	1,271.29

Results

Load combination	F_p [psi]	σ [psi]	Utilization [%]	Status
Combination 1	3,289	2,240	69	OK

2.6 Symbol explanation

A_1	Loaded area of concrete
A_2	Supporting area
d	Nominal diameter of the bolt
ε_{lim}	Limit plastic strain
ε_{Pl}	Plastic strain from CBFEM results
f_c	Concrete compressive strength
f_c'	Concrete compressive strength
F_u	Specified minimum tensile strength of the connected material
F_p	Concrete block design bearing strength
$f_{p,max}$	Concrete block design bearing strength maximum
f_y	Yield strength
l_c	Clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material
N	Resulting compression force
σ	Average stress in concrete
σ_{Ed}	Equivalent stress
Φ	Resistance factor
ΦR_n	Factored resistance
t	Thickness of the anchor plate
V	Resultant of shear forces Vy, Vz in bolt.

2.7 Warnings

- By using the CBFEM calculation functionality of PROFIS Engineering you may act outside the applicable design codes and your specified anchor plate may not behave rigid. Please, validate the results with a professional designer and/or structural engineer to ensure suitability and adequacy for your specific jurisdiction and project requirements.
- The anchor is modeled considering stiffness values determined from load displacement curves tested in an independent laboratory. Please note that no simple replacement of the anchor is possible as the anchor stiffness has a major impact on the load distribution results.

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Company:		Page:	14
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Purple BP (~1600 kips)	Date:	1/28/2025
Fastening point:			

3 Summary of results

Design of the anchor plate, anchors, welds and other elements are based on CBFEM (component based finite element method) and AISC.

	Load combination	Max. utilization	Status
Anchors	Combination 1	88%	OK
Anchor plate	Combination 1	49%	OK
Concrete	Combination 1	69%	OK
Profile	Combination 1	100%	OK

Fastening meets the design criteria!

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	15
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Purple BP (~1600 kips)	Date:	1/28/2025
Fastening point:			

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Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Purple BP (~1600 kips)	Date:	1/28/2025
Fastening point:			

Specifier's comments:

1 Anchor Design

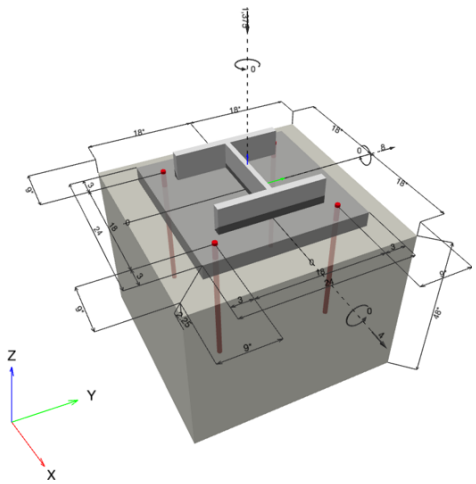
1.1 Input data



Anchor type and diameter:	**Heavy Hex Head ASTM F 1554 GR. 36 3/4**
Item number:	not available
Specification text:	∅ 3/4 in Heavy Hex Head ASTM F 1554 GR. 36 with 18 in nominal embedment depth per Technical data , cast in place installation per MPII
Effective embedment depth:	h_{ef} = 18.000 in.
Material:	ASTM F 1554
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-19 / CIP
Shear edge breakout verification:	Row closest to edge (Case 3 only from ACI 318-19 Fig. R.17.7.2.1b)
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 2.250 in.
Anchor plate[CBFEM] :	l_x x l_y x t = 24.000 in. x 24.000 in. x 2.250 in.;
Profile:	W shape (AISC), W14X159; (L x W x T x FT) = 15.000 in. x 15.600 in. x 0.745 in. x 1.190 in.
Base material:	cracked concrete, 4000, f_c' = 4,000 psi; h = 48.000 in.
Reinforcement:	tension: not present, shear: not present; edge reinforcement: none or < No. 4 bar

[CBFEM] - The anchor calculation is based on a component-based Finite Element Method (CBFEM)

Geometry [in.] & Loading [kip, ft.kip]



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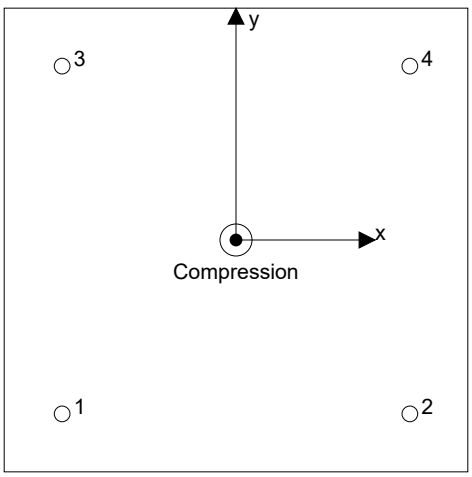
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Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Purple BP (~1600 kips)	Date:	1/28/2025
Fastening point:			

1.1.1 Design results

Case	Description	Forces [kip] / Moments [ft.kip]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = -1,375.000; V_x = 4.000; V_y = 8.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	70

1.2 Load case/Resulting anchor forces



Anchor reactions [kip]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	-0.011	2.006	1.009	1.734
2	-0.011	1.988	0.991	1.723
3	-0.011	2.473	0.990	2.267
4	-0.011	2.491	1.010	2.277

Resulting tension force in (x/y)=(-/-): 0.000 [kip]
Resulting compression force in (x/y)=(0.000/0.000): 1,375.007 [kip]

Anchor forces are calculated based on a component-based Finite Element Method (CBFEM)

1.3 Tension load

	Load N_{ua} [kip]	Capacity ϕ N_n [kip]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	-0.011	14.529	1	OK
Pullout Strength*	-0.011	20.406	-0	not recommended
Concrete Breakout Failure**	N/A	N/A	N/A	N/A
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (anchors in tension)

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	3
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Purple BP (~1600 kips)	Date:	1/28/2025
Fastening point:			

1.3.1 Steel Strength

$N_{sa} = A_{se,N} f_{uta}$ ACI 318-19 Eq. (17.6.1.2)
$\phi N_{sa} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.33	58,000

Calculations

N_{sa} [kip]
19.372

Results

N_{sa} [kip]	ϕ_{steel}	ϕN_{sa} [kip]	N_{ua} [kip]
19.372	0.750	14.529	-0.011

The steel proof was done for the highest absolute force per anchor - in this case compression loading. Please be aware that buckling should be verified separately

1.3.2 Pullout Strength

$N_{pN} = \psi_{c,p} N_p$ ACI 318-19 Eq. (17.6.3.1)
$N_p = 8 A_{brg} f'_c$ ACI 318-19 Eq. (17.6.3.2.2a)
$\phi N_{pN} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$\psi_{c,p}$	A_{brg} [in.^{2}]	λ_a	f'_c [psi]
1.000	0.91	1.000	4,000

Calculations

N_p [kip]
29.152

Results

N_{pn} [kip]	$\phi_{concrete}$	ϕN_{pn} [kip]	N_{ua} [kip]
29.152	0.700	20.406	-0.011

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Company:		Page:	4
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Purple BP (~1600 kips)	Date:	1/28/2025
Fastening point:			

1.4 Shear load

	Load V_{ua} [kip]	Capacity ϕ V_n [kip]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	2.491	7.555	33	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	8.944	124.927	8	OK
Concrete edge failure in direction x+**	8.944	12.909	70	OK

* highest loaded anchor **anchor group (relevant anchors)

1.4.1 Steel Strength

$V_{sa} = 0.6\ A_{se,V}\ f_{uta}$ ACI 318-19 Eq. (17.7.1.2b)
$\phi\ V_{steel} \geq V_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.33	58,000

Calculations

V_{sa} [kip]
11.623

Results

V_{sa} [kip]	ϕ_{steel}	$\phi\ V_{sa}$ [kip]	V_{ua} [kip]
11.623	0.650	7.555	2.491

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Company:		Page:	5
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Purple BP (~1600 kips)	Date:	1/28/2025
Fastening point:			

1.4.2 Pryout Strength

$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \right]$ ACI 318-19 Eq. (17.7.3.1b)

$\phi V_{cpg} \geq V_{ua}$ ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9 h_{ef}^2$ ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e'_N}{3 h_{ef}}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ ACI 318-19 Eq. (17.6.2.2.1)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	6.000	0.000	0.000	9.000

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.000	-	24	1.000	4,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [kip]
1,296.00	324.00	1.000	1.000	1.000	1.000	22.308

Results

V_{cpg} [kip]	$\phi_{concrete}$	ϕV_{cpg} [kip]	V_{ua} [kip]
178.467	0.700	124.927	8.944

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Company:		Page:	6
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Purple BP (~1600 kips)	Date:	1/28/2025
Fastening point:			

1.4.3 Concrete edge failure in direction x+

$V_{cbg} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \psi_{ec,V} \psi_{ed,V} \psi_{c,V} \psi_{h,V} \psi_{parallel,V} V_b$ ACI 318-19 Eq. (17.7.2.1b)

$\phi V_{cbg} \geq V_{ua}$ ACI 318-19 Table 17.5.2

A_{Vc} see ACI 318-19, Section 17.7.2.1, Fig. R 17.7.2.1(b)*

$A_{Vc0} = 4.5 c_{a1}^2$ ACI 318-19 Eq. (17.7.2.1.3)

$\psi_{ec,V} = \left(\frac{1}{1 + \frac{e'_v}{1.5c_{a1}}}\right) \leq 1.0$ ACI 318-19 Eq. (17.7.2.3.1)

$\psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ ACI 318-19 Eq. (17.7.2.4.1b)

$\psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ ACI 318-19 Eq. (17.7.2.6.1)

$V_b = 9 \lambda_a \sqrt{f'_c} c_{a1}^{1.5}$ ACI 318-19 Eq. (17.7.2.2.1b)

Variables

c_{a1} [in.]	c_{a2} [in.]	e_{cV} [in.]	$\psi_{c,V}$	h_a [in.]
9.000	9.000	0.000	1.000	48.000

l_e [in.]	λ_a	d_a [in.]	f'_c [psi]	$\psi_{parallel,V}$
6.000	1.000	0.750	4,000	1.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\psi_{ec,V}$	$\psi_{ed,V}$	$\psi_{h,V}$	V_b [kip]
486.00	364.50	1.000	0.900	1.000	15.369

Results

V_{cbg} [kip]	$\phi_{concrete}$	ϕV_{cbg} [kip]	V_{ua} [kip]
18.442	0.700	12.909	8.944

*Anchor row defined by: Anchor 2, 4; Case 3 controls

1.5 Combined tension and shear loads, per ACI 318-19 section 17.8

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.001	0.693	5/3	55	OK

$\beta_{NV} = \beta_N^{\zeta} + \beta_V^{\zeta} <= 1$

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Company:		Page:	7
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Purple BP (~1600 kips)	Date:	1/28/2025
Fastening point:			

1.6 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates as per current regulations (ETAG 001/Annex C, EOTA TR029, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid base plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- The equations presented in this report are based on imperial units. When inputs are displayed in metric units, the user should be aware that the equations remain in their imperial format.
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- For additional information about ACI 318 strength design provisions, please go to https://submittals.us.hilti.com/PROFISAnchorDesignGuide/
- Attention! In case of compressive anchor forces a buckling check as well as the proof of the local load transfer into and within the base material (incl. punching) has to be done separately.
- The anchor design methods in PROFIS Engineering require rigid anchor plates, as per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means that the anchor plate should be sufficiently rigid to prevent load re-distribution to the anchors due to elastic/plastic displacements. The user accepts that the anchor plate is considered close to rigid by engineering judgment."

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Company:		Page:	8
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Purple BP (~1600 kips)	Date:	1/28/2025
Fastening point:			

1.7 Installation data

Profile: W shape (AISC), W14X159; (L x W x T x FT) = 15.000 in. x 15.600 in. x 0.745 in. x 1.190 in.

Hole diameter in the fixture: d_f = 0.812 in.

Plate thickness (input): 2.250 in.

Anchor type and diameter: Heavy Hex Head ASTM F 1554 GR. 36 3/4

Item number: not available

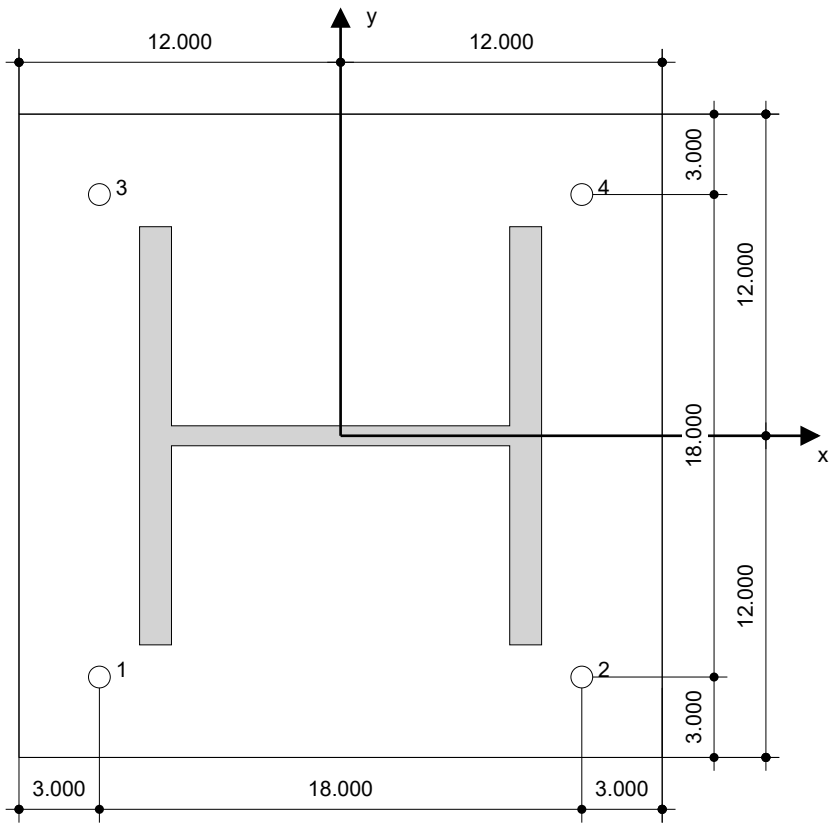
Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 18.000 in.

Minimum thickness of the base material: 19.000 in.

⌀ 3/4 in Heavy Hex Head ASTM F 1554 GR. 36 with 18 in nominal embedment depth per Technical data , cast in place installation per MPII



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-9.000	-9.000	9.000	27.000	9.000	27.000
2	9.000	-9.000	27.000	9.000	9.000	27.000
3	-9.000	9.000	9.000	27.000	27.000	9.000
4	9.000	9.000	27.000	9.000	27.000	9.000

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	9
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Purple BP (~1600 kips)	Date:	1/28/2025
Fastening point:			

2 Anchor plate design

2.1 Input data

Anchor plate:	Shape: Rectangular l_x x l_y x t = 24.000 in x 24.000 in x 2.250 in Calculation: CBFEM Material: ASTM A572 Gr.50; F_y = 50,000 psi; ε_{lim} = 5.00%
Anchor type and size:	Heavy Hex Head ASTM F 1554 GR. 36 3/4, h_{ef} = 18.000 in
Anchor stiffness:	The anchor is modeled considering stiffness values determined from load displacement curves tested in an independent laboratory. Please note that no simple replacement of the anchor is possible as the anchor stiffness has a major impact on the load distribution results.
Design method:	AISC and LRFD-based design using component-based FEM
Stand-off installation:	e_b = 0.000 in (No stand-off); t = 2.250 in
Profile:	W14X159; (L x W x T x FT) = 15.000 in x 15.600 in x 0.745 in x 1.190 in Material: ASTM A992; F_y = 50,000 psi; ε_{lim} = 5.00% Eccentricity x: 0.000 in Eccentricity y: 0.000 in
Base material:	Cracked concrete; 4000; $f_{c,cyl}$ = 4,000 psi; h = 48.000 in
Welds (profile to anchor plate):	Type of redistribution: Plastic Material: E70xx
Mesh size:	Number of elements on edge: 8 Min. size of element: 0.394 in Max. size of element: 1.969 in

2.2 Summary

	Description	Profile		Anchor plate			Concrete [%]
		σ_{Ed} [psi]	ε_{Pl} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Hole bearing [%]	
1	Combination 1	50,038	0.13	26,240	0.00	2	73

2.3 Anchor plate classification

Results below are displayed for the decisive load combinations: Combination 1

Anchor tension forces	Equivalent rigid anchor plate (CBFEM)	Component-based Finite Element Method (CBFEM) anchor plate design
Anchor 1	-0.014 kip	-0.011 kip
Anchor 2	-0.014 kip	-0.011 kip
Anchor 3	-0.014 kip	-0.011 kip
Anchor 4	-0.014 kip	-0.011 kip

User accepted to consider the selected anchor plate as rigid by his/her engineering judgement. This means the anchor design guidelines can be applied.

2.4 Profile/Stiffeners/Plate

Profile and stiffeners are verified at the level of the steel to concrete connection. The connection design does not replace the steel design for critical cross sections, which should be performed outside of PROFIS Engineering.

2.4.1 Equivalent stress and plastic strain

Part	Load combination	Material	f_y [psi]	ε_{lim} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Status
Plate	Combination 1	ASTM A572 Gr.50	50,000	5.00	26,240	0.00	OK

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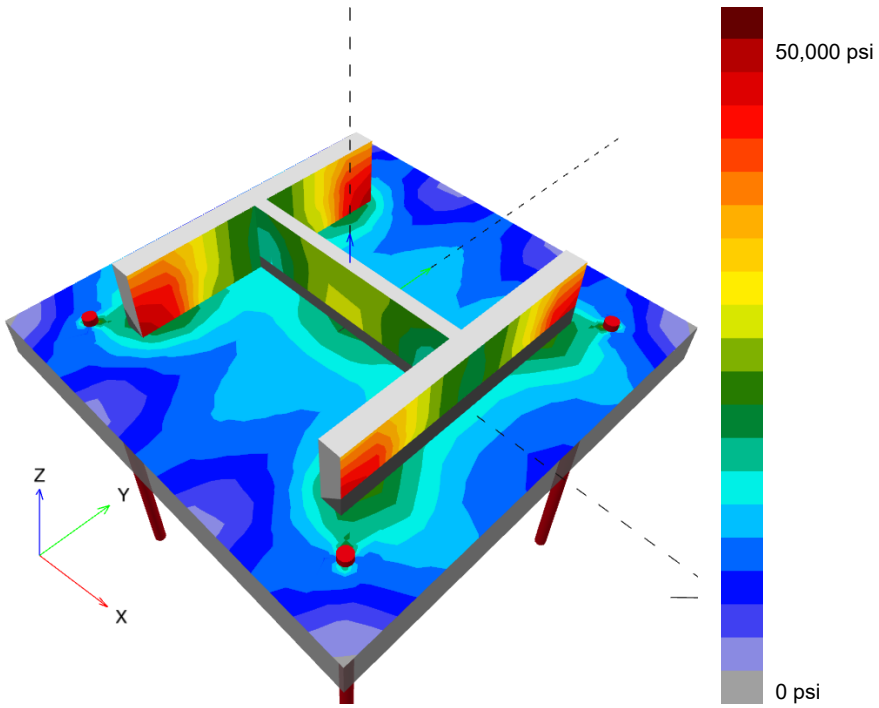
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Company:		Page:	10
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Purple BP (~1600 kips)	Date:	1/28/2025
Fastening point:			

Part	Load combination	Material	f_y [psi]	ε_{lim} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Status
Profile	Combination 1	ASTM A992	50,000	5.00	50,037	0.13	OK
Profile	Combination 1	ASTM A992	50,000	5.00	50,038	0.13	OK
Profile	Combination 1	ASTM A992	50,000	5.00	29,467	0.00	OK

2.4.1.1 Equivalent stress

Results below are displayed for the decisive load combination: 1 - Combination 1

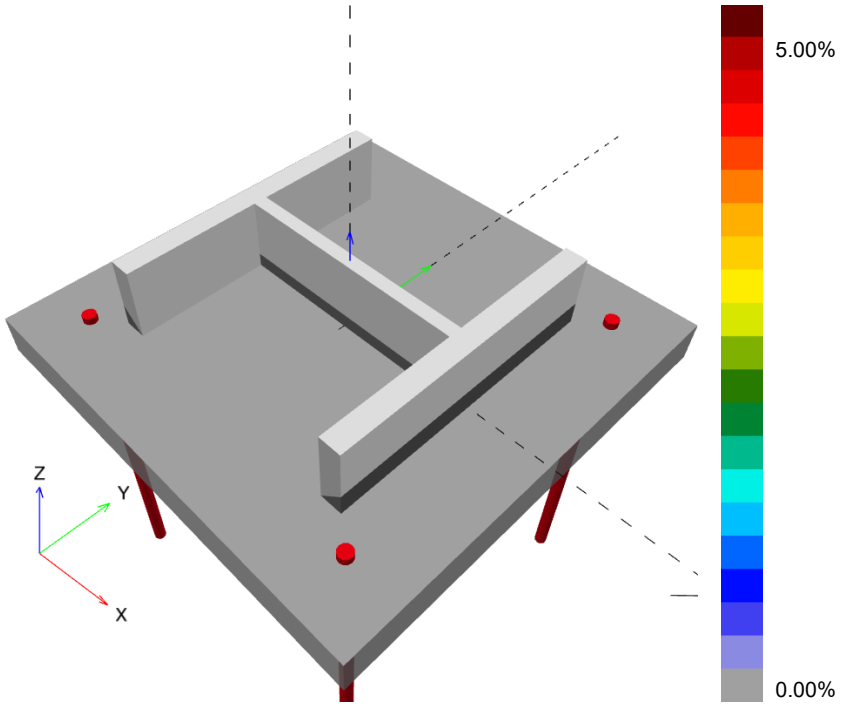


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Company:		Page:	11
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Purple BP (~1600 kips)	Date:	1/28/2025
Fastening point:			

2.4.1.2 Plastic strain

Results below are displayed for the decisive load combination: 1 - Combination 1



2.4.2 Plate hole bearing resistance, AISC 360-16 Section J3

Decisive load combination: 1 - Combination 1

Equations

$R_n = \min(1.2\, l_c\, t\, F_u\, ,\ 2.4\, d\, t\, F_u)$ (AISC 360-16 J3-6a, c)

$\Phi R_n = 0.75\, R_n$

$V \leq \Phi R_n$

Variables

	l_c [in]	t [in]	F_u [psi]	d [in]	R_n [kip]
Anchor 1	3.065	2.250	65,000	0.750	263.250
Anchor 2	3.055	2.250	65,000	0.750	263.250
Anchor 3	7.089	2.250	65,000	0.750	263.250
Anchor 4	22.566	2.250	65,000	0.750	263.250

Results

	V [kip]	ΦR_n [kip]	Utilization [%]	Status
Anchor 1	2.006	197.437	2	OK
Anchor 2	1.988	197.437	2	OK
Anchor 3	2.473	197.437	2	OK

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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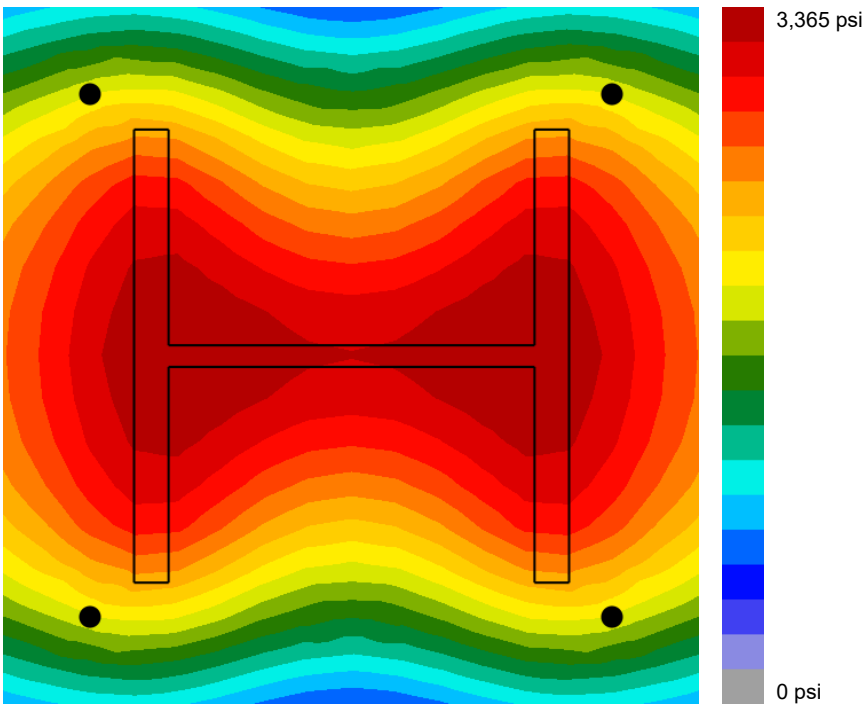
Company:		Page:	12
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Purple BP (~1600 kips)	Date:	1/28/2025
Fastening point:			

	V [kip]	ΦR_n [kip]	Utilization [%]	Status
Anchor 4	2.491	197.437	2	OK

2.5 Concrete

Decisive load combination: 1 - Combination 1

2.5.1 Compression in concrete under the anchor plate



2.5.2 Concrete block compressive strength resistance check, AISC 360-16 Section J8

Equations

$$F_p = \Phi f_{p,max}$$

$$f_{p,max} = 0.85 f_c' \sqrt{\left(\frac{A_2}{A_1}\right)} \le 1.7 f_c; \quad \sqrt{\left(\frac{A_2}{A_1}\right)} \le 2$$

$$\sigma = \frac{N}{A_1}$$

$$\text{Utilization} = \frac{\sigma}{F_p}$$

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Company:		Page:	13
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Purple BP (~1600 kips)	Date:	1/28/2025
Fastening point:			

Variables

N [kip]	f_c' [psi]	Φ	A_1 [in²]	A_2 [in²]
1,375.007	4,000	0.65	574.13	1,271.29

Results

Load combination	F_p [psi]	σ [psi]	Utilization [%]	Status
Combination 1	3,289	2,395	73	OK

2.6 Symbol explanation

A_1	Loaded area of concrete
A_2	Supporting area
d	Nominal diameter of the bolt
ε_{lim}	Limit plastic strain
ε_{Pl}	Plastic strain from CBFEM results
f_c	Concrete compressive strength
f_c'	Concrete compressive strength
F_u	Specified minimum tensile strength of the connected material
F_p	Concrete block design bearing strength
$f_{p,max}$	Concrete block design bearing strength maximum
f_y	Yield strength
l_c	Clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material
N	Resulting compression force
σ	Average stress in concrete
σ_{Ed}	Equivalent stress
Φ	Resistance factor
ΦR_n	Factored resistance
t	Thickness of the anchor plate
V	Resultant of shear forces Vy, Vz in bolt.

2.7 Warnings

- By using the CBFEM calculation functionality of PROFIS Engineering you may act outside the applicable design codes and your specified anchor plate may not behave rigid. Please, validate the results with a professional designer and/or structural engineer to ensure suitability and adequacy for your specific jurisdiction and project requirements.
- The anchor is modeled considering stiffness values determined from load displacement curves tested in an independent laboratory. Please note that no simple replacement of the anchor is possible as the anchor stiffness has a major impact on the load distribution results.

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Company:		Page:	14
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Purple BP (~1600 kips)	Date:	1/28/2025
Fastening point:			

3 Summary of results

Design of the anchor plate, anchors, welds and other elements are based on CBFEM (component based finite element method) and AISC.

	Load combination	Max. utilization	Status
Anchors	Combination 1	70%	OK
Anchor plate	Combination 1	53%	OK
Concrete	Combination 1	73%	OK
Profile	Combination 1	100%	OK

Fastening meets the design criteria!

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Company:		Page:	15
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Purple BP (~1600 kips)	Date:	1/28/2025
Fastening point:			

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Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Purple BP (~1600 kips)	Date:	1/28/2025
Fastening point:			

Specifier's comments:

1 Anchor Design

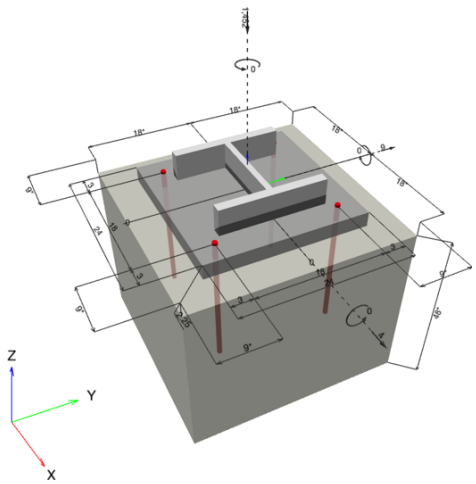
1.1 Input data



Anchor type and diameter:	**Heavy Hex Head ASTM F 1554 GR. 36 3/4**
Item number:	not available
Specification text:	∅ 3/4 in Heavy Hex Head ASTM F 1554 GR. 36 with 18 in nominal embedment depth per Technical data , cast in place installation per MPII
Effective embedment depth:	h_{ef} = 18.000 in.
Material:	ASTM F 1554
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-19 / CIP
Shear edge breakout verification:	Row closest to edge (Case 3 only from ACI 318-19 Fig. R.17.7.2.1b)
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 2.250 in.
Anchor plate[CBFEM] :	l_x x l_y x t = 24.000 in. x 24.000 in. x 2.250 in.;
Profile:	W shape (AISC), W14X193; (L x W x T x FT) = 15.500 in. x 15.700 in. x 0.890 in. x 1.440 in.
Base material:	cracked concrete, 4000, f_c' = 4,000 psi; h = 48.000 in.
Reinforcement:	tension: not present, shear: not present; edge reinforcement: none or < No. 4 bar

[CBFEM] - The anchor calculation is based on a component-based Finite Element Method (CBFEM)

Geometry [in.] & Loading [kip, ft.kip]



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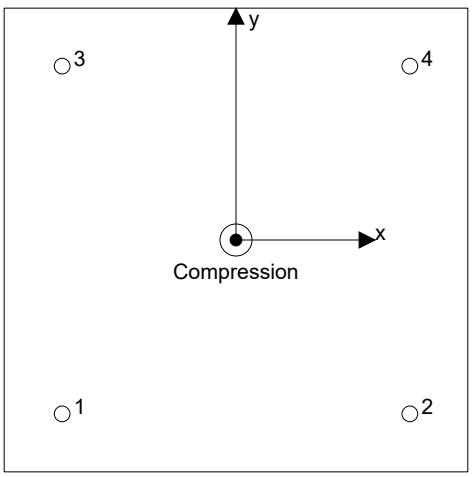
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Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Purple BP (~1600 kips)	Date:	1/28/2025
Fastening point:			

1.1.1 Design results

Case	Description	Forces [kip] / Moments [ft.kip]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = -1,452.000; V_x = 4.000; V_y = 9.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	77

1.2 Load case/Resulting anchor forces



Anchor reactions [kip]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	-0.013	2.191	1.018	1.941
2	-0.013	2.166	0.982	1.930
3	-0.013	2.747	0.998	2.559
4	-0.013	2.758	1.002	2.570

Resulting tension force in (x/y)=(-/-): 0.000 [kip]
Resulting compression force in (x/y)=(0.000/0.000): 1,452.137 [kip]

Anchor forces are calculated based on a component-based Finite Element Method (CBFEM)

1.3 Tension load

	Load N_{ua} [kip]	Capacity ϕ N_n [kip]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	-0.013	14.529	1	OK
Pullout Strength*	-0.013	20.406	-0	not recommended
Concrete Breakout Failure**	N/A	N/A	N/A	N/A
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (anchors in tension)

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Company:		Page:	3
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Purple BP (~1600 kips)	Date:	1/28/2025
Fastening point:			

1.3.1 Steel Strength

$N_{sa} = A_{se,N} f_{uta}$ ACI 318-19 Eq. (17.6.1.2)
$\phi N_{sa} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.33	58,000

Calculations

N_{sa} [kip]
19.372

Results

N_{sa} [kip]	ϕ_{steel}	ϕN_{sa} [kip]	N_{ua} [kip]
19.372	0.750	14.529	-0.013

The steel proof was done for the highest absolute force per anchor - in this case compression loading. Please be aware that buckling should be verified separately

1.3.2 Pullout Strength

$N_{pN} = \psi_{c,p} N_p$ ACI 318-19 Eq. (17.6.3.1)
$N_p = 8 A_{brg} f_c'$ ACI 318-19 Eq. (17.6.3.2.2a)
$\phi N_{pN} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$\psi_{c,p}$	A_{brg} [in.^{2}]	λ_a	f_c' [psi]
1.000	0.91	1.000	4,000

Calculations

N_p [kip]
29.152

Results

N_{pn} [kip]	$\phi_{concrete}$	ϕN_{pn} [kip]	N_{ua} [kip]
29.152	0.700	20.406	-0.013

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Company:		Page:	4
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Purple BP (~1600 kips)	Date:	1/28/2025
Fastening point:			

1.4 Shear load

	Load V_{ua} [kip]	Capacity ϕ V_n [kip]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	2.758	7.555	37	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	9.849	124.927	8	OK
Concrete edge failure in direction y+**	9.849	12.910	77	OK

* highest loaded anchor **anchor group (relevant anchors)

1.4.1 Steel Strength

$V_{sa} = 0.6\ A_{se,V}\ f_{uta}$ ACI 318-19 Eq. (17.7.1.2b)
$\phi\ V_{steel} \geq V_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.33	58,000

Calculations

V_{sa} [kip]
11.623

Results

V_{sa} [kip]	ϕ_{steel}	$\phi\ V_{sa}$ [kip]	V_{ua} [kip]
11.623	0.650	7.555	2.758

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Company:		Page:	5
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Purple BP (~1600 kips)	Date:	1/28/2025
Fastening point:			

1.4.2 Pryout Strength

$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \right]$ ACI 318-19 Eq. (17.7.3.1b)

$\phi V_{cpg} \geq V_{ua}$ ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9 h_{ef}^2$ ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e'_N}{3 h_{ef}}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ ACI 318-19 Eq. (17.6.2.2.1)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	6.000	0.000	0.000	9.000

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.000	-	24	1.000	4,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [kip]
1,296.00	324.00	1.000	1.000	1.000	1.000	22.308

Results

V_{cpg} [kip]	$\phi_{concrete}$	ϕV_{cpg} [kip]	V_{ua} [kip]
178.467	0.700	124.927	9.849

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Company:		Page:	6
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Purple BP (~1600 kips)	Date:	1/28/2025
Fastening point:			

1.4.3 Concrete edge failure in direction y+

$V_{cbg} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \Psi_{ec,V} \Psi_{ed,V} \Psi_{c,V} \Psi_{h,V} \Psi_{parallel,V} V_b$ — ACI 318-19 Eq. (17.7.2.1b)

$\phi V_{cbg} \geq V_{ua}$ — ACI 318-19 Table 17.5.2

A_{Vc} see ACI 318-19, Section 17.7.2.1, Fig. R 17.7.2.1(b)*

$A_{Vc0} = 4.5 c_{a1}^2$ — ACI 318-19 Eq. (17.7.2.1.3)

$\Psi_{ec,V} = \left(\frac{1}{1 + \frac{e'_v}{1.5c_{a1}}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.3.1)

$\Psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.4.1b)

$\Psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ — ACI 318-19 Eq. (17.7.2.6.1)

$V_b = 9 \lambda_a \sqrt{f'_c} c_{a1}^{1.5}$ — ACI 318-19 Eq. (17.7.2.2.1b)

Variables

c_{a1} [in.]	c_{a2} [in.]	e_{cV} [in.]	$\Psi_{c,V}$	h_a [in.]
9.000	9.000	0.000	1.000	48.000

l_e [in.]	λ_a	d_a [in.]	f'_c [psi]	$\Psi_{parallel,V}$
6.000	1.000	0.750	4,000	1.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{h,V}$	V_b [kip]
486.00	364.50	1.000	0.900	1.000	15.369

Results

V_{cbg} [kip]	$\phi_{concrete}$	ϕV_{cbg} [kip]	V_{ua} [kip]
18.442	0.700	12.910	9.849

*Anchor row defined by: Anchor 3, 4; Case 3 controls

1.5 Combined tension and shear loads, per ACI 318-19 section 17.8

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.001	0.763	1.000	64	OK

$\beta_{NV} = (\beta_N + \beta_V) / 1.2 <= 1$

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Company:		Page:	7
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Purple BP (~1600 kips)	Date:	1/28/2025
Fastening point:			

1.6 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates as per current regulations (ETAG 001/Annex C, EOTA TR029, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid base plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- The equations presented in this report are based on imperial units. When inputs are displayed in metric units, the user should be aware that the equations remain in their imperial format.
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- For additional information about ACI 318 strength design provisions, please go to https://submittals.us.hilti.com/PROFISAnchorDesignGuide/
- Attention! In case of compressive anchor forces a buckling check as well as the proof of the local load transfer into and within the base material (incl. punching) has to be done separately.
- The anchor design methods in PROFIS Engineering require rigid anchor plates, as per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means that the anchor plate should be sufficiently rigid to prevent load re-distribution to the anchors due to elastic/plastic displacements. The user accepts that the anchor plate is considered close to rigid by engineering judgment."

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Company:		Page:	8
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Purple BP (~1600 kips)	Date:	1/28/2025
Fastening point:			

1.7 Installation data

Profile: W shape (AISC), W14X193; (L x W x T x FT) = 15.500 in. x 15.700 in. x 0.890 in. x 1.440 in.

Hole diameter in the fixture: d_f = 0.812 in.

Plate thickness (input): 2.250 in.

Anchor type and diameter: Heavy Hex Head ASTM F 1554 GR. 36 3/4

Item number: not available

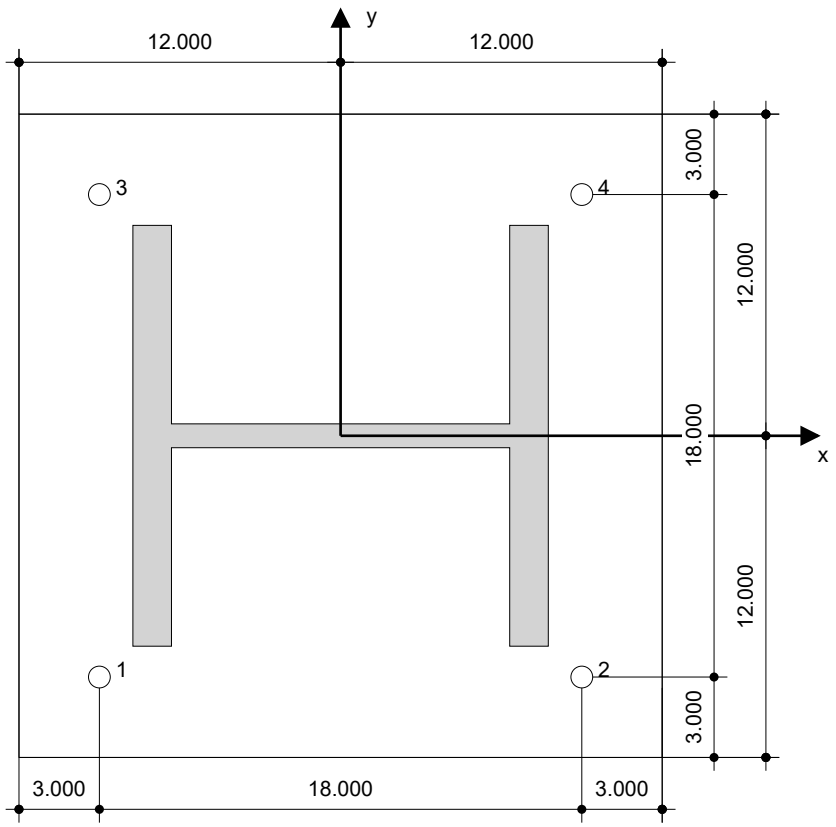
Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 18.000 in.

Minimum thickness of the base material: 19.000 in.

⌀ 3/4 in Heavy Hex Head ASTM F 1554 GR. 36 with 18 in nominal embedment depth per Technical data , cast in place installation per MPII



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-9.000	-9.000	9.000	27.000	9.000	27.000
2	9.000	-9.000	27.000	9.000	9.000	27.000
3	-9.000	9.000	9.000	27.000	27.000	9.000
4	9.000	9.000	27.000	9.000	27.000	9.000

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Company:		Page:	9
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Purple BP (~1600 kips)	Date:	1/28/2025
Fastening point:			

2 Anchor plate design

2.1 Input data

Anchor plate:	Shape: Rectangular l_x x l_y x t = 24.000 in x 24.000 in x 2.250 in Calculation: CBFEM Material: ASTM A572 Gr.50; F_y = 50,000 psi; ε_{lim} = 5.00%
Anchor type and size:	Heavy Hex Head ASTM F 1554 GR. 36 3/4, h_{ef} = 18.000 in
Anchor stiffness:	The anchor is modeled considering stiffness values determined from load displacement curves tested in an independent laboratory. Please note that no simple replacement of the anchor is possible as the anchor stiffness has a major impact on the load distribution results.
Design method:	AISC and LRFD-based design using component-based FEM
Stand-off installation:	e_b = 0.000 in (No stand-off); t = 2.250 in
Profile:	W14X193; (L x W x T x FT) = 15.500 in x 15.700 in x 0.890 in x 1.440 in Material: ASTM A992; F_y = 50,000 psi; ε_{lim} = 5.00% Eccentricity x: 0.000 in Eccentricity y: 0.000 in
Base material:	Cracked concrete; 4000; $f_{c,cyl}$ = 4,000 psi; h = 48.000 in
Welds (profile to anchor plate):	Type of redistribution: Plastic Material: E70xx
Mesh size:	Number of elements on edge: 8 Min. size of element: 0.394 in Max. size of element: 1.969 in

2.2 Summary

	Description	Profile		Anchor plate			Concrete [%]
		σ_{Ed} [psi]	ε_{Pl} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Hole bearing [%]	
1	Combination 1	48,976	0.08	25,914	0.00	2	77

2.3 Anchor plate classification

Results below are displayed for the decisive load combinations: Combination 1

Anchor tension forces	Equivalent rigid anchor plate (CBFEM)	Component-based Finite Element Method (CBFEM) anchor plate design
Anchor 1	-0.015 kip	-0.013 kip
Anchor 2	-0.015 kip	-0.013 kip
Anchor 3	-0.015 kip	-0.013 kip
Anchor 4	-0.015 kip	-0.013 kip

User accepted to consider the selected anchor plate as rigid by his/her engineering judgement. This means the anchor design guidelines can be applied.

2.4 Profile/Stiffeners/Plate

Profile and stiffeners are verified at the level of the steel to concrete connection. The connection design does not replace the steel design for critical cross sections, which should be performed outside of PROFIS Engineering.

2.4.1 Equivalent stress and plastic strain

Part	Load combination	Material	f_y [psi]	ε_{lim} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Status
Plate	Combination 1	ASTM A572 Gr.50	50,000	5.00	25,914	0.00	OK

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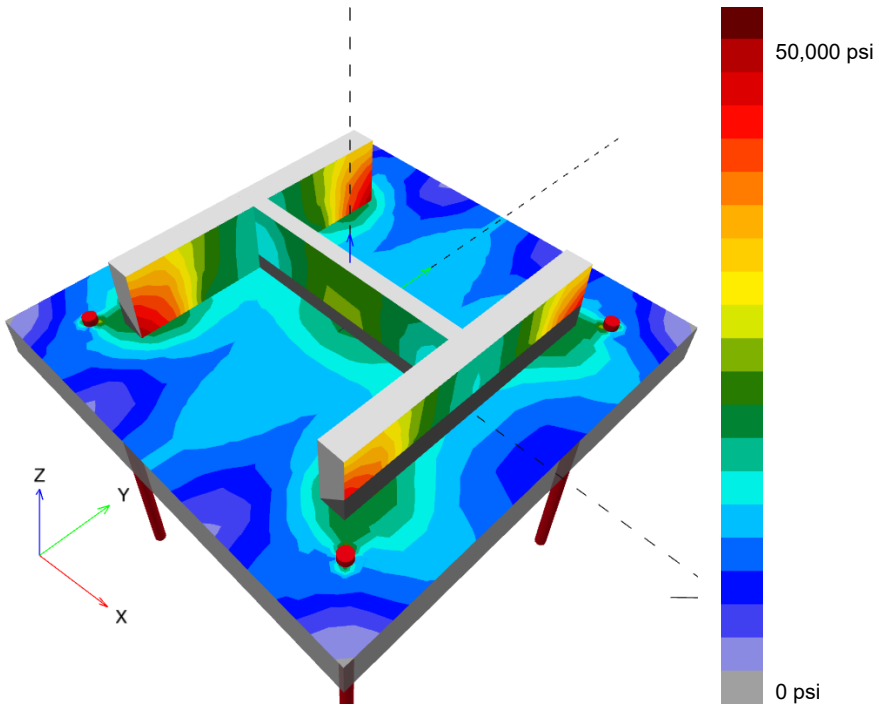
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Company:		Page:	10
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Purple BP (~1600 kips)	Date:	1/28/2025
Fastening point:			

Part	Load combination	Material	f_y [psi]	ε_{lim} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Status
Profile	Combination 1	ASTM A992	50,000	5.00	48,840	0.08	OK
Profile	Combination 1	ASTM A992	50,000	5.00	48,976	0.08	OK
Profile	Combination 1	ASTM A992	50,000	5.00	26,553	0.00	OK

2.4.1.1 Equivalent stress

Results below are displayed for the decisive load combination: 1 - Combination 1

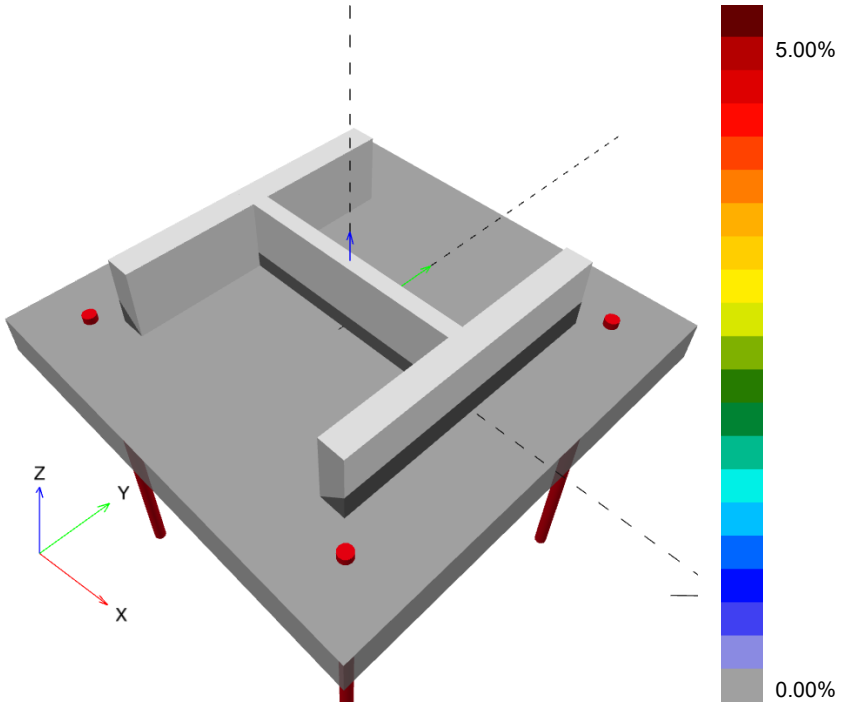


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Company:		Page:	11
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Purple BP (~1600 kips)	Date:	1/28/2025
Fastening point:			

2.4.1.2 Plastic strain

Results below are displayed for the decisive load combination: 1 - Combination 1



2.4.2 Plate hole bearing resistance, AISC 360-16 Section J3

Decisive load combination: 1 - Combination 1

Equations

$R_n = \min(1.2\, l_c\, t\, F_u\, ,\ 2.4\, d\, t\, F_u)$ (AISC 360-16 J3-6a, c)

$\Phi R_n = 0.75\, R_n$

$V \leq \Phi R_n$

Variables

	l_c [in]	t [in]	F_u [psi]	d [in]	R_n [kip]
Anchor 1	2.981	2.250	65,000	0.750	263.250
Anchor 2	2.960	2.250	65,000	0.750	263.250
Anchor 3	7.849	2.250	65,000	0.750	263.250
Anchor 4	22.132	2.250	65,000	0.750	263.250

Results

	V [kip]	ΦR_n [kip]	Utilization [%]	Status
Anchor 1	2.192	197.437	2	OK
Anchor 2	2.166	197.437	2	OK
Anchor 3	2.747	197.437	2	OK

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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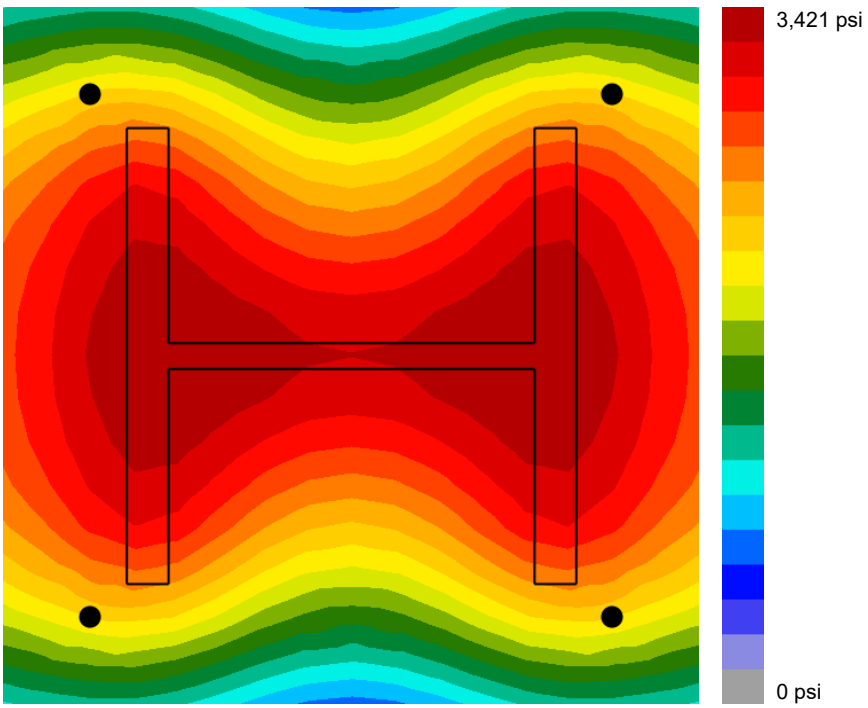
Company:		Page:	12
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Purple BP (~1600 kips)	Date:	1/28/2025
Fastening point:			

	V [kip]	ΦR_n [kip]	Utilization [%]	Status
Anchor 4	2.758	197.437	2	OK

2.5 Concrete

Decisive load combination: 1 - Combination 1

2.5.1 Compression in concrete under the anchor plate



2.5.2 Concrete block compressive strength resistance check, AISC 360-16 Section J8

Equations

$$F_p = \Phi f_{p,max}$$

$$f_{p,max} = 0.85 f_c' \sqrt{\left(\frac{A_2}{A_1}\right)} \le 1.7 f_c;\ \sqrt{\left(\frac{A_2}{A_1}\right)} \le 2$$

$$\sigma = \frac{N}{A_1}$$

$$\text{Utilization} = \frac{\sigma}{F_p}$$

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	13
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Purple BP (~1600 kips)	Date:	1/28/2025
Fastening point:			

Variables

N [kip]	f_c' [psi]	Φ	A_1 [in²]	A_2 [in²]
1,452.137	4,000	0.65	574.13	1,271.29

Results

Load combination	F_p [psi]	σ [psi]	Utilization [%]	Status
Combination 1	3,289	2,529	77	OK

2.6 Symbol explanation

A_1	Loaded area of concrete
A_2	Supporting area
d	Nominal diameter of the bolt
ε_{lim}	Limit plastic strain
ε_{Pl}	Plastic strain from CBFEM results
f_c	Concrete compressive strength
f_c'	Concrete compressive strength
F_u	Specified minimum tensile strength of the connected material
F_p	Concrete block design bearing strength
$f_{p,max}$	Concrete block design bearing strength maximum
f_y	Yield strength
l_c	Clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material
N	Resulting compression force
σ	Average stress in concrete
σ_{Ed}	Equivalent stress
Φ	Resistance factor
ΦR_n	Factored resistance
t	Thickness of the anchor plate
V	Resultant of shear forces Vy, Vz in bolt.

2.7 Warnings

- By using the CBFEM calculation functionality of PROFIS Engineering you may act outside the applicable design codes and your specified anchor plate may not behave rigid. Please, validate the results with a professional designer and/or structural engineer to ensure suitability and adequacy for your specific jurisdiction and project requirements.
- The anchor is modeled considering stiffness values determined from load displacement curves tested in an independent laboratory. Please note that no simple replacement of the anchor is possible as the anchor stiffness has a major impact on the load distribution results.

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	14
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Purple BP (~1600 kips)	Date:	1/28/2025
Fastening point:			

3 Summary of results

Design of the anchor plate, anchors, welds and other elements are based on CBFEM (component based finite element method) and AISC.

	Load combination	Max. utilization	Status
Anchors	Combination 1	77%	OK
Anchor plate	Combination 1	52%	OK
Concrete	Combination 1	77%	OK
Profile	Combination 1	98%	OK

Fastening meets the design criteria!

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	15
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Purple BP (~1600 kips)	Date:	1/28/2025
Fastening point:			

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Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Green BP (~2000 kips)	Date:	1/28/2025
Fastening point:			

Specifier's comments:

1 Anchor Design

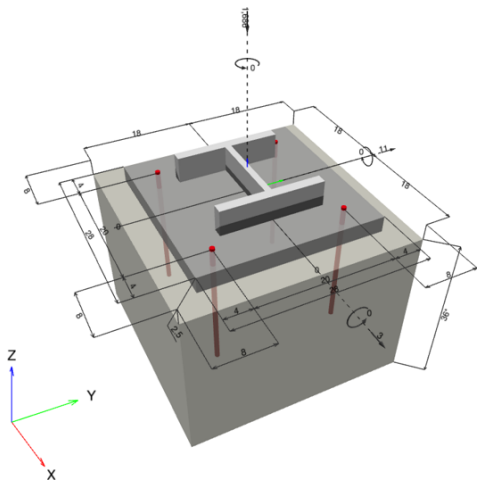
1.1 Input data



Anchor type and diameter:	**Heavy Hex Head ASTM F 1554 GR. 36 3/4**
Item number:	not available
Specification text:	∅ 3/4 in Heavy Hex Head ASTM F 1554 GR. 36 with 18 in nominal embedment depth per Technical data , cast in place installation per MPII
Effective embedment depth:	h_{ef} = 18.000 in.
Material:	ASTM F 1554
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-19 / CIP
Shear edge breakout verification:	Row closest to edge (Case 3 only from ACI 318-19 Fig. R.17.7.2.1b)
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 2.500 in.
Anchor plate[CBFEM] :	l_x x l_y x t = 28.000 in. x 28.000 in. x 2.500 in.;
Profile:	W shape (AISC), W14X211; (L x W x T x FT) = 15.700 in. x 15.800 in. x 0.980 in. x 1.560 in.
Base material:	cracked concrete, 4000, f_c' = 4,000 psi; h = 36.000 in.
Reinforcement:	tension: not present, shear: not present; edge reinforcement: none or < No. 4 bar

[CBFEM] - The anchor calculation is based on a component-based Finite Element Method (CBFEM)

Geometry [in.] & Loading [kip, ft.kip]



Input data and results must be checked for conformity with the existing conditions and for plausibility!
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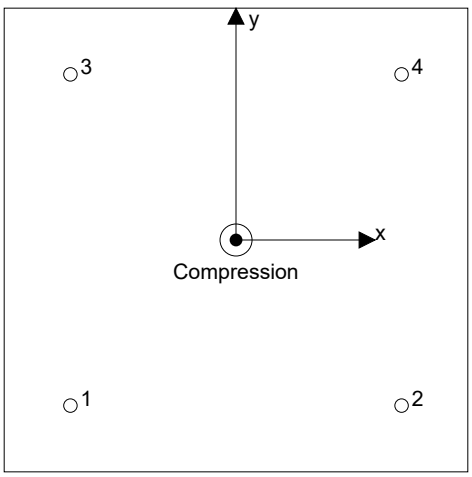
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Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Green BP (~2000 kips)	Date:	1/28/2025
Fastening point:			

1.1.1 Design results

Case	Description	Forces [kip] / Moments [ft.kip]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = -1,686.000; V_x = 3.000; V_y = 11.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	94

1.2 Load case/Resulting anchor forces



Anchor reactions [kip]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	-0.009	2.591	0.732	2.486
2	-0.009	2.596	0.768	2.480
3	-0.009	3.097	0.710	3.015
4	-0.009	3.121	0.790	3.020

Resulting tension force in (x/y)=(-/-): 0.000 [kip]
Resulting compression force in (x/y)=(0.002/-0.000): 1,686.504 [kip]

Anchor forces are calculated based on a component-based Finite Element Method (CBFEM)

1.3 Tension load

	Load N_{ua} [kip]	Capacity ϕ N_n [kip]	Utilization $\beta_N = N_{ua}/\phi N_n$	Status
Steel Strength*	-0.009	14.529	1	OK
Pullout Strength*	-0.009	20.406	-0	not recommended
Concrete Breakout Failure**	N/A	N/A	N/A	N/A
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (anchors in tension)

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Company:		Page:	3
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Green BP (~2000 kips)	Date:	1/28/2025
Fastening point:			

1.3.1 Steel Strength

$N_{sa} = A_{se,N} f_{uta}$ ACI 318-19 Eq. (17.6.1.2)
$\phi N_{sa} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.33	58,000

Calculations

N_{sa} [kip]
19.372

Results

N_{sa} [kip]	ϕ_{steel}	ϕN_{sa} [kip]	N_{ua} [kip]
19.372	0.750	14.529	-0.009

The steel proof was done for the highest absolute force per anchor - in this case compression loading. Please be aware that buckling should be verified separately

1.3.2 Pullout Strength

$N_{pN} = \psi_{c,p} N_p$ ACI 318-19 Eq. (17.6.3.1)
$N_p = 8 A_{brg} f_c'$ ACI 318-19 Eq. (17.6.3.2.2a)
$\phi N_{pN} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$\psi_{c,p}$	A_{brg} [in.^{2}]	λ_a	f_c' [psi]
1.000	0.91	1.000	4,000

Calculations

N_p [kip]
29.152

Results

N_{pn} [kip]	$\phi_{concrete}$	ϕN_{pn} [kip]	N_{ua} [kip]
29.152	0.700	20.406	-0.009

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Company:		Page:	4
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Green BP (~2000 kips)	Date:	1/28/2025
Fastening point:			

1.4 Shear load

	Load V_{ua} [kip]	Capacity ϕ V_n [kip]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	3.121	7.555	42	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	11.402	111.405	11	OK
Concrete edge failure in direction x+**	11.402	12.171	94	OK

* highest loaded anchor **anchor group (relevant anchors)

1.4.1 Steel Strength

$V_{sa} = 0.6\ A_{se,V}\ f_{uta}$ ACI 318-19 Eq. (17.7.1.2b)
$\phi\ V_{steel} \geq V_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.33	58,000

Calculations

V_{sa} [kip]
11.623

Results

V_{sa} [kip]	ϕ_{steel}	$\phi\ V_{sa}$ [kip]	V_{ua} [kip]
11.623	0.650	7.555	3.121

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Company:		Page:	5
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Green BP (~2000 kips)	Date:	1/28/2025
Fastening point:			

1.4.2 Pryout Strength

$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \right]$ ACI 318-19 Eq. (17.7.3.1b)

$\phi V_{cpg} \geq V_{ua}$ ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9 h_{ef}^2$ ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e'_N}{3 h_{ef}}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ ACI 318-19 Eq. (17.6.2.2.1)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	6.667	0.000	0.000	8.000

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.000	-	24	1.000	4,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [kip]
1,296.00	400.00	1.000	1.000	0.940	1.000	26.128

Results

V_{cpg} [kip]	$\phi_{concrete}$	ϕV_{cpg} [kip]	V_{ua} [kip]
159.150	0.700	111.405	11.402

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Company:		Page:	6
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Green BP (~2000 kips)	Date:	1/28/2025
Fastening point:			

1.4.3 Concrete edge failure in direction x+

$V_{cbg} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \Psi_{ec,V} \Psi_{ed,V} \Psi_{c,V} \Psi_{h,V} \Psi_{parallel,V} V_b$ — ACI 318-19 Eq. (17.7.2.1b)

$\phi V_{cbg} \geq V_{ua}$ — ACI 318-19 Table 17.5.2

A_{Vc} see ACI 318-19, Section 17.7.2.1, Fig. R 17.7.2.1(b)*

$A_{Vc0} = 4.5 c_{a1}^2$ — ACI 318-19 Eq. (17.7.2.1.3)

$\Psi_{ec,V} = \left(\frac{1}{1 + \frac{e'_v}{1.5c_{a1}}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.3.1)

$\Psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.4.1b)

$\Psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ — ACI 318-19 Eq. (17.7.2.6.1)

$V_b = 9 \lambda_a \sqrt{f'_c} c_{a1}^{1.5}$ — ACI 318-19 Eq. (17.7.2.2.1b)

Variables

c_{a1} [in.]	c_{a2} [in.]	e_{cV} [in.]	$\Psi_{c,V}$	h_a [in.]
8.000	8.000	0.000	1.000	36.000

l_e [in.]	λ_a	d_a [in.]	f'_c [psi]	$\Psi_{parallel,V}$
6.000	1.000	0.750	4,000	1.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{h,V}$	V_b [kip]
432.00	288.00	1.000	0.900	1.000	12.880

Results

V_{cbg} [kip]	$\phi_{concrete}$	ϕV_{cbg} [kip]	V_{ua} [kip]
17.387	0.700	12.171	11.402

*Anchor row defined by: Anchor 2, 4; Case 3 controls

1.5 Combined tension and shear loads, per ACI 318-19 section 17.8

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.001	0.937	1.000	79	OK

$\beta_{NV} = (\beta_N + \beta_V) / 1.2 <= 1$

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Company:		Page:	7
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Green BP (~2000 kips)	Date:	1/28/2025
Fastening point:			

1.6 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates as per current regulations (ETAG 001/Annex C, EOTA TR029, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid base plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- The equations presented in this report are based on imperial units. When inputs are displayed in metric units, the user should be aware that the equations remain in their imperial format.
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- For additional information about ACI 318 strength design provisions, please go to https://submittals.us.hilti.com/PROFISAnchorDesignGuide/
- Attention! In case of compressive anchor forces a buckling check as well as the proof of the local load transfer into and within the base material (incl. punching) has to be done separately.
- The anchor design methods in PROFIS Engineering require rigid anchor plates, as per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means that the anchor plate should be sufficiently rigid to prevent load re-distribution to the anchors due to elastic/plastic displacements. The user accepts that the anchor plate is considered close to rigid by engineering judgment."

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Company:		Page:	8
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Green BP (~2000 kips)	Date:	1/28/2025
Fastening point:			

1.7 Installation data

Profile: W shape (AISC), W14X211; (L x W x T x FT) = 15.700 in. x 15.800 in. x 0.980 in. x 1.560 in.

Hole diameter in the fixture: d_f = 0.812 in.

Plate thickness (input): 2.500 in.

Anchor type and diameter: Heavy Hex Head ASTM F 1554 GR. 36 3/4

Item number: not available

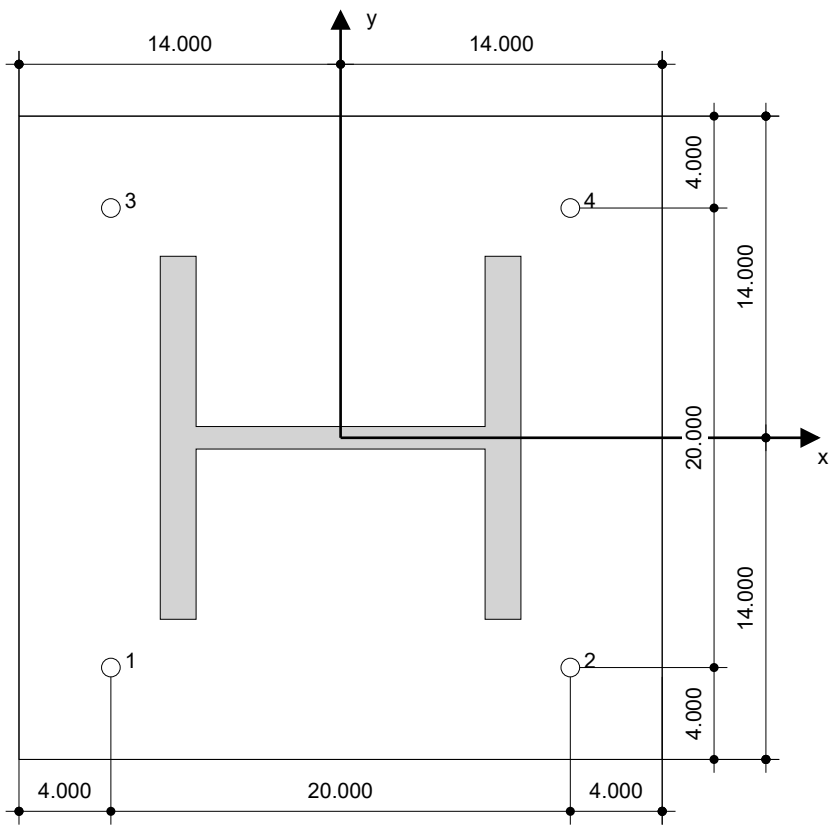
Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 18.000 in.

Minimum thickness of the base material: 19.000 in.

⌀ 3/4 in Heavy Hex Head ASTM F 1554 GR. 36 with 18 in nominal embedment depth per Technical data , cast in place installation per MPII



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-10.000	-10.000	8.000	28.000	8.000	28.000
2	10.000	-10.000	28.000	8.000	8.000	28.000
3	-10.000	10.000	8.000	28.000	28.000	8.000
4	10.000	10.000	28.000	8.000	28.000	8.000

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Company:		Page:	9
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Green BP (~2000 kips)	Date:	1/28/2025
Fastening point:			

2 Anchor plate design

2.1 Input data

Anchor plate:	Shape: Rectangular l_x x l_y x t = 28.000 in x 28.000 in x 2.500 in Calculation: CBFEM Material: ASTM A572 Gr.50; F_y = 50,000 psi; ε_{lim} = 5.00%
Anchor type and size:	Heavy Hex Head ASTM F 1554 GR. 36 3/4, h_{ef} = 18.000 in
Anchor stiffness:	The anchor is modeled considering stiffness values determined from load displacement curves tested in an independent laboratory. Please note that no simple replacement of the anchor is possible as the anchor stiffness has a major impact on the load distribution results.
Design method:	AISC and LRFD-based design using component-based FEM
Stand-off installation:	e_b = 0.000 in (No stand-off); t = 2.500 in
Profile:	W14X211; (L x W x T x FT) = 15.700 in x 15.800 in x 0.980 in x 1.560 in Material: ASTM A992; F_y = 50,000 psi; ε_{lim} = 5.00% Eccentricity x: 0.000 in Eccentricity y: 0.000 in
Base material:	Cracked concrete; 4000; $f_{c,cyl}$ = 4,000 psi; h = 36.000 in
Welds (profile to anchor plate):	Type of redistribution: Plastic Material: E70xx
Mesh size:	Number of elements on edge: 8 Min. size of element: 0.394 in Max. size of element: 1.969 in

2.2 Summary

	Description	Profile		Anchor plate			Concrete [%]
		σ_{Ed} [psi]	ε_{Pl} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Hole bearing [%]	
1	Combination 1	50,039	0.13	27,299	0.00	2	78

2.3 Anchor plate classification

Results below are displayed for the decisive load combinations: Combination 1

Anchor tension forces	Equivalent rigid anchor plate (CBFEM)	Component-based Finite Element Method (CBFEM) anchor plate design
Anchor 1	-0.013 kip	-0.009 kip
Anchor 2	-0.013 kip	-0.009 kip
Anchor 3	-0.013 kip	-0.009 kip
Anchor 4	-0.013 kip	-0.009 kip

User accepted to consider the selected anchor plate as rigid by his/her engineering judgement. This means the anchor design guidelines can be applied.

2.4 Profile/Stiffeners/Plate

Profile and stiffeners are verified at the level of the steel to concrete connection. The connection design does not replace the steel design for critical cross sections, which should be performed outside of PROFIS Engineering.

2.4.1 Equivalent stress and plastic strain

Part	Load combination	Material	f_y [psi]	ε_{lim} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Status
Plate	Combination 1	ASTM A572 Gr.50	50,000	5.00	27,299	0.00	OK

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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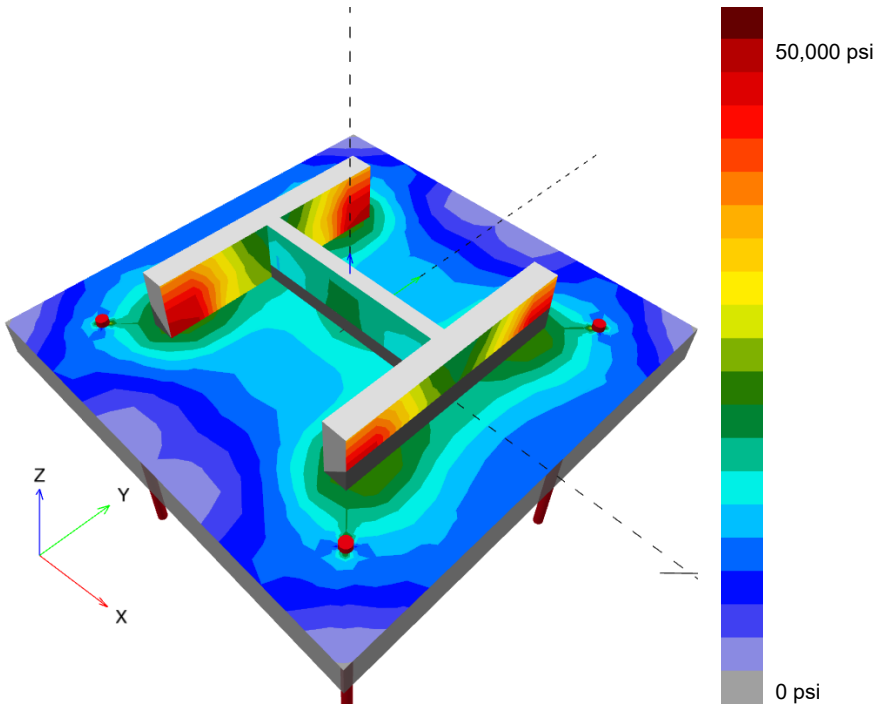
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Company:		Page:	10
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Green BP (~2000 kips)	Date:	1/28/2025
Fastening point:			

Part	Load combination	Material	f_y [psi]	ε_{lim} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Status
Profile	Combination 1	ASTM A992	50,000	5.00	50,039	0.13	OK
Profile	Combination 1	ASTM A992	50,000	5.00	50,039	0.13	OK
Profile	Combination 1	ASTM A992	50,000	5.00	26,989	0.00	OK

2.4.1.1 Equivalent stress

Results below are displayed for the decisive load combination: 1 - Combination 1

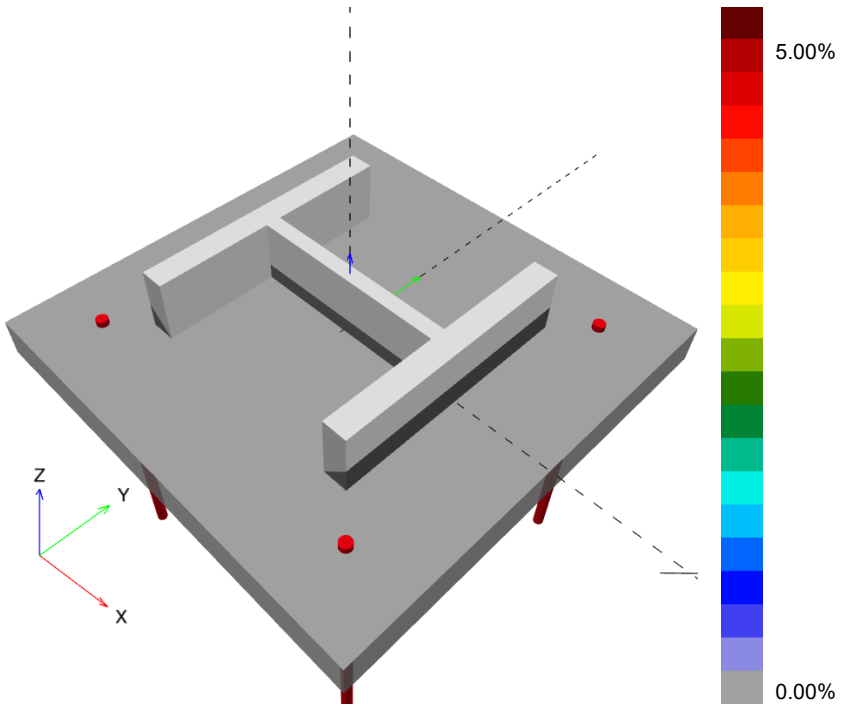


Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	11
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Green BP (~2000 kips)	Date:	1/28/2025
Fastening point:			

2.4.1.2 Plastic strain

Results below are displayed for the decisive load combination: 1 - Combination 1



2.4.2 Plate hole bearing resistance, AISC 360-16 Section J3

Decisive load combination: 1 - Combination 1

Equations

$R_n = \min(1.2\, l_c\, t\, F_u\, ,\ 2.4\, d\, t\, F_u)$ (AISC 360-16 J3-6a, c)

$\Phi R_n = 0.75\, R_n$

$V \leq \Phi R_n$

Variables

	l_c [in]	t [in]	F_u [psi]	d [in]	R_n [kip]
Anchor 1	3.764	2.500	65,000	0.750	292.500
Anchor 2	3.781	2.500	65,000	0.750	292.500
Anchor 3	17.045	2.500	65,000	0.750	292.500
Anchor 4	24.401	2.500	65,000	0.750	292.500

Results

	V [kip]	ΦR_n [kip]	Utilization [%]	Status
Anchor 1	2.591	219.375	2	OK
Anchor 2	2.596	219.375	2	OK
Anchor 3	3.097	219.375	2	OK

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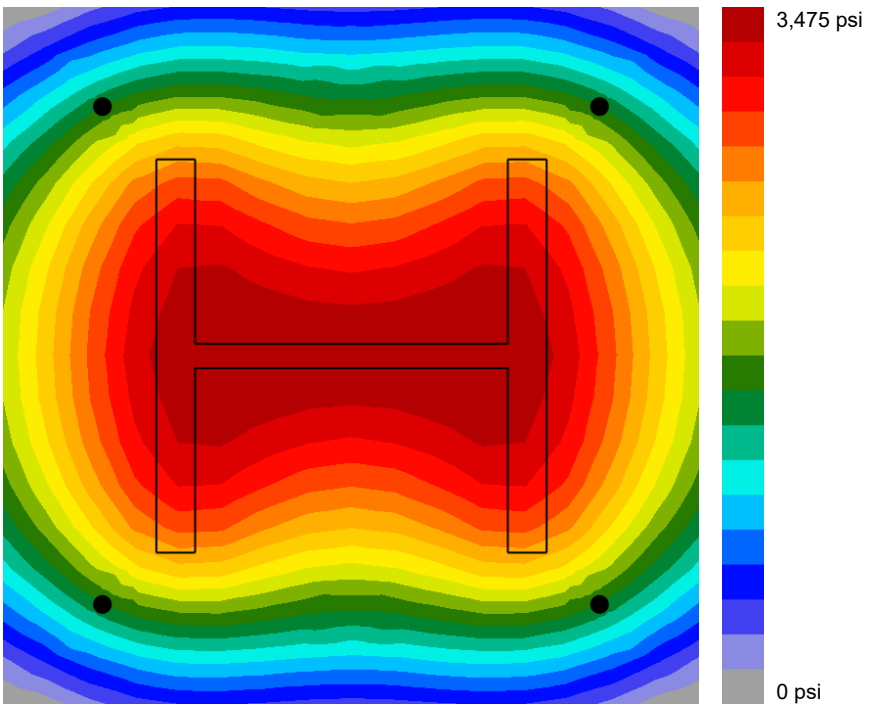
Company:		Page:	12
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Green BP (~2000 kips)	Date:	1/28/2025
Fastening point:			

	V [kip]	ΦR_n [kip]	Utilization [%]	Status
Anchor 4	3.121	219.375	2	OK

2.5 Concrete

Decisive load combination: 1 - Combination 1

2.5.1 Compression in concrete under the anchor plate



2.5.2 Concrete block compressive strength resistance check, AISC 360-16 Section J8

Equations

$$F_p = \Phi f_{p,max}$$

$$f_{p,max} = 0.85 f_c' \sqrt{\left(\frac{A_2}{A_1}\right)} \le 1.7 f_c; \quad \sqrt{\left(\frac{A_2}{A_1}\right)} \le 2$$

$$\sigma = \frac{N}{A_1}$$

$$\text{Utilization} = \frac{\sigma}{F_p}$$

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	13
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Green BP (~2000 kips)	Date:	1/28/2025
Fastening point:			

Variables

N [kip]	f_c' [psi]	Φ	A_1 [in²]	A_2 [in²]
1,686.504	4,000	0.65	768.73	1,246.58

Results

Load combination	F_p [psi]	σ [psi]	Utilization [%]	Status
Combination 1	2,814	2,194	78	OK

2.6 Symbol explanation

A_1	Loaded area of concrete
A_2	Supporting area
d	Nominal diameter of the bolt
ε_{lim}	Limit plastic strain
ε_{Pl}	Plastic strain from CBFEM results
f_c	Concrete compressive strength
f_c'	Concrete compressive strength
F_u	Specified minimum tensile strength of the connected material
F_p	Concrete block design bearing strength
$f_{p,max}$	Concrete block design bearing strength maximum
f_y	Yield strength
l_c	Clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material
N	Resulting compression force
σ	Average stress in concrete
σ_{Ed}	Equivalent stress
Φ	Resistance factor
ΦR_n	Factored resistance
t	Thickness of the anchor plate
V	Resultant of shear forces Vy, Vz in bolt.

2.7 Warnings

- By using the CBFEM calculation functionality of PROFIS Engineering you may act outside the applicable design codes and your specified anchor plate may not behave rigid. Please, validate the results with a professional designer and/or structural engineer to ensure suitability and adequacy for your specific jurisdiction and project requirements.
- The anchor is modeled considering stiffness values determined from load displacement curves tested in an independent laboratory. Please note that no simple replacement of the anchor is possible as the anchor stiffness has a major impact on the load distribution results.

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	14
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Green BP (~2000 kips)	Date:	1/28/2025
Fastening point:			

3 Summary of results

Design of the anchor plate, anchors, welds and other elements are based on CBFEM (component based finite element method) and AISC.

	Load combination	Max. utilization	Status
Anchors	Combination 1	94%	OK
Anchor plate	Combination 1	55%	OK
Concrete	Combination 1	78%	OK
Profile	Combination 1	100%	OK

Fastening meets the design criteria!

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	15
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Green BP (~2000 kips)	Date:	1/28/2025
Fastening point:			

4 Remarks; Your Cooperation Duties

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Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Green BP (~2000 kips)	Date:	1/28/2025
Fastening point:			

Specifier's comments:

1 Anchor Design

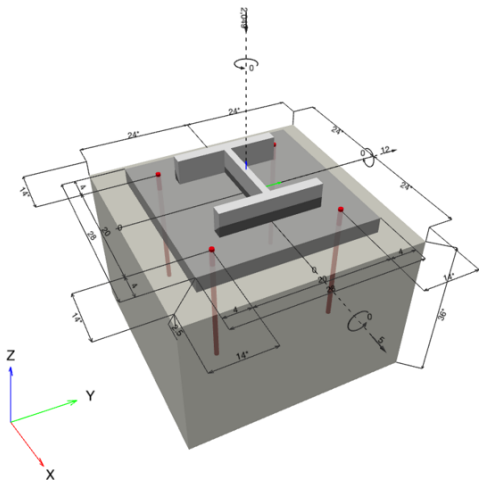
1.1 Input data



Anchor type and diameter:	**Heavy Hex Head ASTM F 1554 GR. 36 3/4**
Item number:	not available
Specification text:	∅ 3/4 in Heavy Hex Head ASTM F 1554 GR. 36 with 18 in nominal embedment depth per Technical data , cast in place installation per MPII
Effective embedment depth:	h_{ef} = 18.000 in.
Material:	ASTM F 1554
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-19 / CIP
Shear edge breakout verification:	Row closest to edge (Case 3 only from ACI 318-19 Fig. R.17.7.2.1b)
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 2.500 in.
Anchor plate[CBFEM] :	l_x x l_y x t = 28.000 in. x 28.000 in. x 2.500 in.;
Profile:	W shape (AISC), W14X233; (L x W x T x FT) = 16.000 in. x 15.900 in. x 1.070 in. x 1.720 in.
Base material:	cracked concrete, Custom, f_c' = 4,500 psi; h = 36.000 in.
Reinforcement:	tension: not present, shear: not present; edge reinforcement: none or < No. 4 bar

[CBFEM] - The anchor calculation is based on a component-based Finite Element Method (CBFEM)

Geometry [in.] & Loading [kip, ft.kip]



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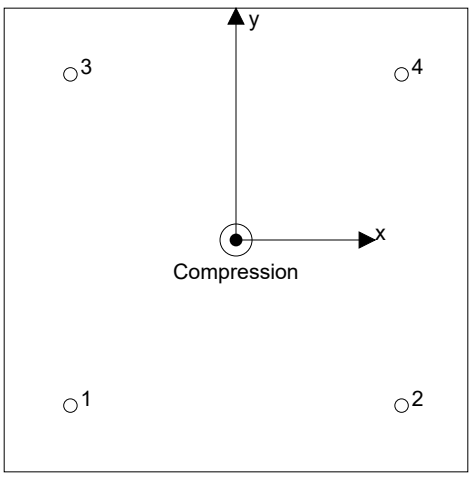
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Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Green BP (~2000 kips)	Date:	1/28/2025
Fastening point:			

1.1.1 Design results

Case	Description	Forces [kip] / Moments [ft.kip]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = -2,049.000; V_x = 5.000; V_y = 12.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	58

1.2 Load case/Resulting anchor forces



Anchor reactions [kip]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	-0.010	2.957	1.230	2.690
2	-0.010	2.967	1.271	2.681
3	-0.010	3.523	1.206	3.310
4	-0.010	3.562	1.294	3.319

Resulting tension force in (x/y)=(-/-): 0.000 [kip]
Resulting compression force in (x/y)=(0.002/-0.000): 2,049.503 [kip]

Anchor forces are calculated based on a component-based Finite Element Method (CBFEM)

1.3 Tension load

	Load N_{ua} [kip]	Capacity ϕ N_n [kip]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	-0.010	14.529	1	OK
Pullout Strength*	-0.010	22.957	-0	not recommended
Concrete Breakout Failure**	N/A	N/A	N/A	N/A
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (anchors in tension)

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	3
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Green BP (~2000 kips)	Date:	1/28/2025
Fastening point:			

1.3.1 Steel Strength

$N_{sa} = A_{se,N} f_{uta}$ ACI 318-19 Eq. (17.6.1.2)
$\phi N_{sa} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.33	58,000

Calculations

N_{sa} [kip]
19.372

Results

N_{sa} [kip]	ϕ_{steel}	ϕN_{sa} [kip]	N_{ua} [kip]
19.372	0.750	14.529	-0.010

The steel proof was done for the highest absolute force per anchor - in this case compression loading. Please be aware that buckling should be verified separately

1.3.2 Pullout Strength

$N_{pN} = \psi_{c,p} N_p$ ACI 318-19 Eq. (17.6.3.1)
$N_p = 8 A_{brg} f'_c$ ACI 318-19 Eq. (17.6.3.2.2a)
$\phi N_{pN} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$\psi_{c,p}$	A_{brg} [in.^{2}]	λ_a	f'_c [psi]
1.000	0.91	1.000	4,500

Calculations

N_p [kip]
32.796

Results

N_{pn} [kip]	$\phi_{concrete}$	ϕN_{pn} [kip]	N_{ua} [kip]
32.796	0.700	22.957	-0.010

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Company:		Page:	4
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Green BP (~2000 kips)	Date:	1/28/2025
Fastening point:			

1.4 Shear load

	Load V_{ua} [kip]	Capacity ϕ V_n [kip]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	3.562	7.555	48	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	13.000	188.872	7	OK
Concrete edge failure in direction y+**	13.000	22.770	58	OK

* highest loaded anchor **anchor group (relevant anchors)

1.4.1 Steel Strength

$V_{sa} = 0.6\ A_{se,V}\ f_{uta}$ ACI 318-19 Eq. (17.7.1.2b)
$\phi\ V_{steel} \geq V_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.33	58,000

Calculations

V_{sa} [kip]
11.623

Results

V_{sa} [kip]	ϕ_{steel}	$\phi\ V_{sa}$ [kip]	V_{ua} [kip]
11.623	0.650	7.555	3.562

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Company:		Page:	5
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Green BP (~2000 kips)	Date:	1/28/2025
Fastening point:			

1.4.2 Pryout Strength

$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \right]$ ACI 318-19 Eq. (17.7.3.1b)

$\phi V_{cpg} \geq V_{ua}$ ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9 h_{ef}^2$ ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e'_N}{3 h_{ef}}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ ACI 318-19 Eq. (17.6.2.2.1)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	9.333	0.000	0.000	14.000

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.000	-	24	1.000	4,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [kip]
2,304.00	784.00	1.000	1.000	1.000	1.000	45.906

Results

V_{cpg} [kip]	$\phi_{concrete}$	ϕV_{cpg} [kip]	V_{ua} [kip]
269.817	0.700	188.872	13.000

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Company:		Page:	6
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Green BP (~2000 kips)	Date:	1/28/2025
Fastening point:			

1.4.3 Concrete edge failure in direction y+

$V_{cbg} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \Psi_{ec,V} \Psi_{ed,V} \Psi_{c,V} \Psi_{h,V} \Psi_{parallel,V} V_b$ — ACI 318-19 Eq. (17.7.2.1b)

$\phi V_{cbg} \geq V_{ua}$ — ACI 318-19 Table 17.5.2

A_{Vc} see ACI 318-19, Section 17.7.2.1, Fig. R 17.7.2.1(b)*

$A_{Vc0} = 4.5 c_{a1}^2$ — ACI 318-19 Eq. (17.7.2.1.3)

$\Psi_{ec,V} = \left(\frac{1}{1 + \frac{e'_v}{1.5c_{a1}}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.3.1)

$\Psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.4.1b)

$\Psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ — ACI 318-19 Eq. (17.7.2.6.1)

$V_b = 9 \lambda_a \sqrt{f'_c} c_{a1}^{1.5}$ — ACI 318-19 Eq. (17.7.2.2.1b)

Variables

c_{a1} [in.]	c_{a2} [in.]	e_{cV} [in.]	$\Psi_{c,V}$	h_a [in.]
14.000	14.000	0.000	1.000	36.000

l_e [in.]	λ_a	d_a [in.]	f'_c [psi]	$\Psi_{parallel,V}$
6.000	1.000	0.750	4,500	1.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{h,V}$	V_b [kip]
1,008.00	882.00	1.000	0.900	1.000	31.626

Results

V_{cbg} [kip]	$\phi_{concrete}$	ϕV_{cbg} [kip]	V_{ua} [kip]
32.529	0.700	22.770	13.000

*Anchor row defined by: Anchor 3, 4; Case 3 controls

1.5 Combined tension and shear loads, per ACI 318-19 section 17.8

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.001	0.571	5/3	40	OK

$\beta_{NV} = \beta_N^{\zeta} + \beta_V^{\zeta} <= 1$

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Company:		Page:	7
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Green BP (~2000 kips)	Date:	1/28/2025
Fastening point:			

1.6 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates as per current regulations (ETAG 001/Annex C, EOTA TR029, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid base plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- The equations presented in this report are based on imperial units. When inputs are displayed in metric units, the user should be aware that the equations remain in their imperial format.
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- For additional information about ACI 318 strength design provisions, please go to https://submittals.us.hilti.com/PROFISAnchorDesignGuide/
- Attention! In case of compressive anchor forces a buckling check as well as the proof of the local load transfer into and within the base material (incl. punching) has to be done separately.
- The anchor design methods in PROFIS Engineering require rigid anchor plates, as per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means that the anchor plate should be sufficiently rigid to prevent load re-distribution to the anchors due to elastic/plastic displacements. The user accepts that the anchor plate is considered close to rigid by engineering judgment."

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Company:		Page:	8
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Green BP (~2000 kips)	Date:	1/28/2025
Fastening point:			

1.7 Installation data

Anchor type and diameter: Heavy Hex Head ASTM F 1554 GR. 36 3/4

Item number: not available

Profile: W shape (AISC), W14X233; (L x W x T x FT) = 16.000 in. x 15.900 in. x 1.070 in. x 1.720 in.

Hole diameter in the fixture: d_f = 0.812 in.

Plate thickness (input): 2.500 in.

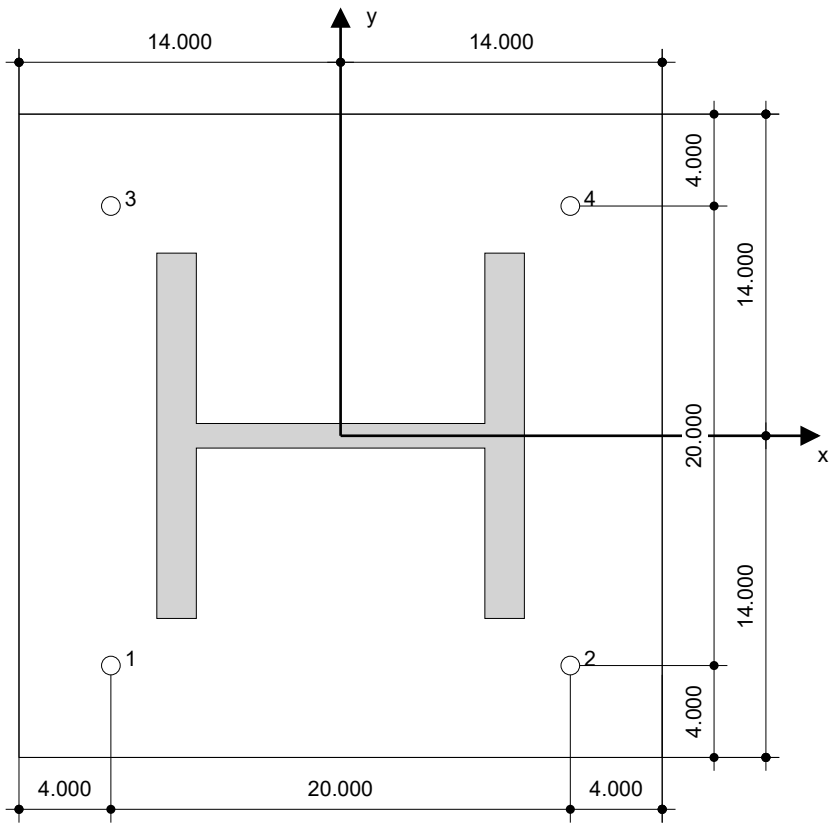
Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 18.000 in.

Minimum thickness of the base material: 19.000 in.

⌀ 3/4 in Heavy Hex Head ASTM F 1554 GR. 36 with 18 in nominal embedment depth per Technical data , cast in place installation per MPII



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-10.000	-10.000	14.000	34.000	14.000	34.000
2	10.000	-10.000	34.000	14.000	14.000	34.000
3	-10.000	10.000	14.000	34.000	34.000	14.000
4	10.000	10.000	34.000	14.000	34.000	14.000

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Company:		Page:	9
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Green BP (~2000 kips)	Date:	1/28/2025
Fastening point:			

2 Anchor plate design

2.1 Input data

Anchor plate:	Shape: Rectangular l_x x l_y x t = 28.000 in x 28.000 in x 2.500 in Calculation: CBFEM Material: ASTM A572 Gr.50; F_y = 50,000 psi; ε_{lim} = 5.00%
Anchor type and size:	Heavy Hex Head ASTM F 1554 GR. 36 3/4, h_{ef} = 18.000 in
Anchor stiffness:	The anchor is modeled considering stiffness values determined from load displacement curves tested in an independent laboratory. Please note that no simple replacement of the anchor is possible as the anchor stiffness has a major impact on the load distribution results.
Design method:	AISC and LRFD-based design using component-based FEM
Stand-off installation:	e_b = 0.000 in (No stand-off); t = 2.500 in
Profile:	W14X233; (L x W x T x FT) = 16.000 in x 15.900 in x 1.070 in x 1.720 in Material: ASTM A992; F_y = 50,000 psi; ε_{lim} = 5.00% Eccentricity x: 0.000 in Eccentricity y: 0.000 in
Base material:	Cracked concrete; Custom; $f_{c,cyl}$ = 4,500 psi; h = 36.000 in; E = 4,351,132 psi; G = 1,740,453 psi; v = 0.20; D = 156.07 lb/ft³
Welds (profile to anchor plate):	Type of redistribution: Plastic Material: E70xx
Mesh size:	Number of elements on edge: 8 Min. size of element: 0.394 in Max. size of element: 1.969 in

2.2 Summary

	Description	Profile		Anchor plate			Concrete [%]
		σ_{Ed} [psi]	ε_{Pl} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Hole bearing [%]	
1	Combination 1	50,058	0.20	31,473	0.00	2	65

2.3 Anchor plate classification

Results below are displayed for the decisive load combinations: Combination 1

Anchor tension forces	Equivalent rigid anchor plate (CBFEM)	Component-based Finite Element Method (CBFEM) anchor plate design
Anchor 1	-0.014 kip	-0.010 kip
Anchor 2	-0.014 kip	-0.010 kip
Anchor 3	-0.014 kip	-0.010 kip
Anchor 4	-0.014 kip	-0.010 kip

User accepted to consider the selected anchor plate as rigid by his/her engineering judgement. This means the anchor design guidelines can be applied.

2.4 Profile/Stiffeners/Plate

Profile and stiffeners are verified at the level of the steel to concrete connection. The connection design does not replace the steel design for critical cross sections, which should be performed outside of PROFIS Engineering.

2.4.1 Equivalent stress and plastic strain

Part	Load combination	Material	f_y [psi]	ε_{lim} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Status
Plate	Combination 1	ASTM A572	50,000	5.00	31,473	0.00	OK

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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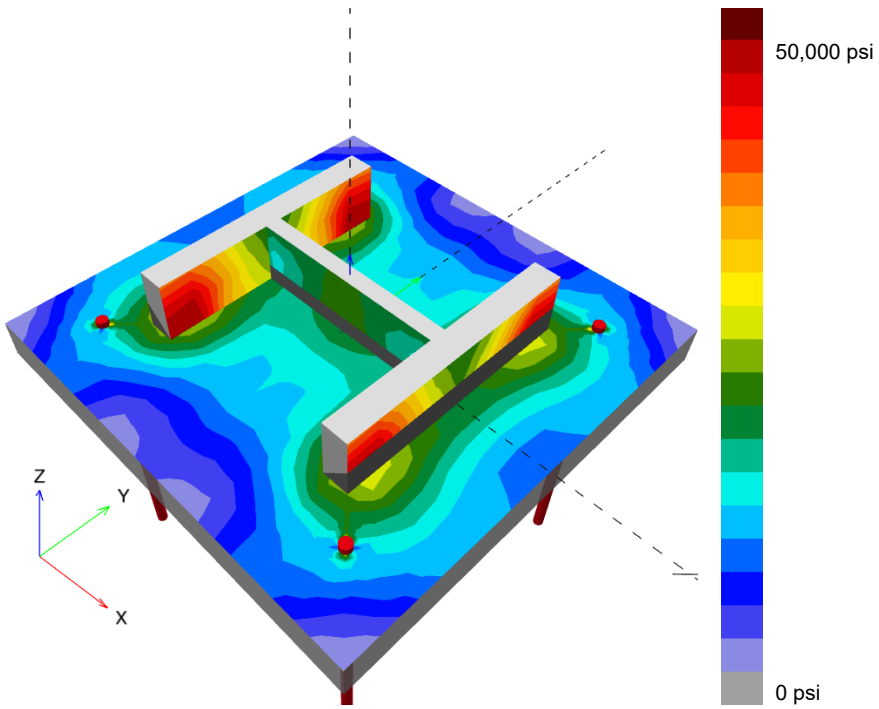
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Company:		Page:	10
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Green BP (~2000 kips)	Date:	1/28/2025
Fastening point:			

Part	Load combination	Material	f_y [psi]	ε_{lim} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Status
		Gr.50					
Profile	Combination 1	ASTM A992	50,000	5.00	50,058	0.20	OK
Profile	Combination 1	ASTM A992	50,000	5.00	50,058	0.20	OK
Profile	Combination 1	ASTM A992	50,000	5.00	29,691	0.00	OK

2.4.1.1 Equivalent stress

Results below are displayed for the decisive load combination: 1 - Combination 1

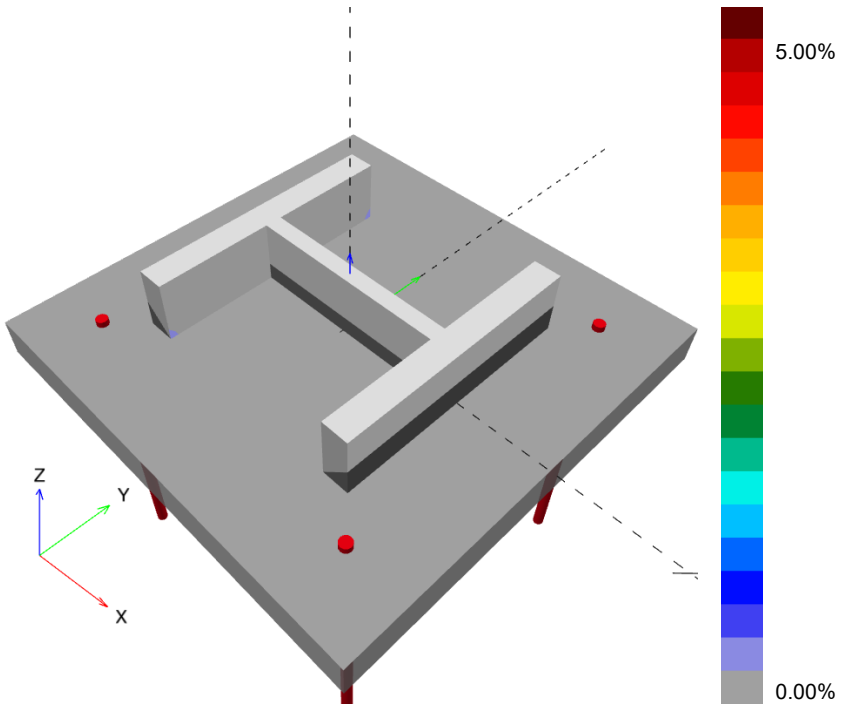


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Company:		Page:	11
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Green BP (~2000 kips)	Date:	1/28/2025
Fastening point:			

2.4.1.2 Plastic strain

Results below are displayed for the decisive load combination: 1 - Combination 1



2.4.2 Plate hole bearing resistance, AISC 360-16 Section J3

Decisive load combination: 1 - Combination 1

Equations

$R_n = \min(1.2\, l_c\, t\, F_u\, ,\ 2.4\, d\, t\, F_u)$ (AISC 360-16 J3-6a, c)

$\Phi R_n = 0.75\, R_n$

$V \leq \Phi R_n$

Variables

	l_c [in]	t [in]	F_u [psi]	d [in]	R_n [kip]
Anchor 1	3.992	2.500	65,000	0.750	292.500
Anchor 2	4.020	2.500	65,000	0.750	292.500
Anchor 3	11.276	2.500	65,000	0.750	292.500
Anchor 4	25.352	2.500	65,000	0.750	292.500

Results

	V [kip]	ΦR_n [kip]	Utilization [%]	Status
Anchor 1	2.958	219.375	2	OK
Anchor 2	2.967	219.375	2	OK
Anchor 3	3.523	219.375	2	OK

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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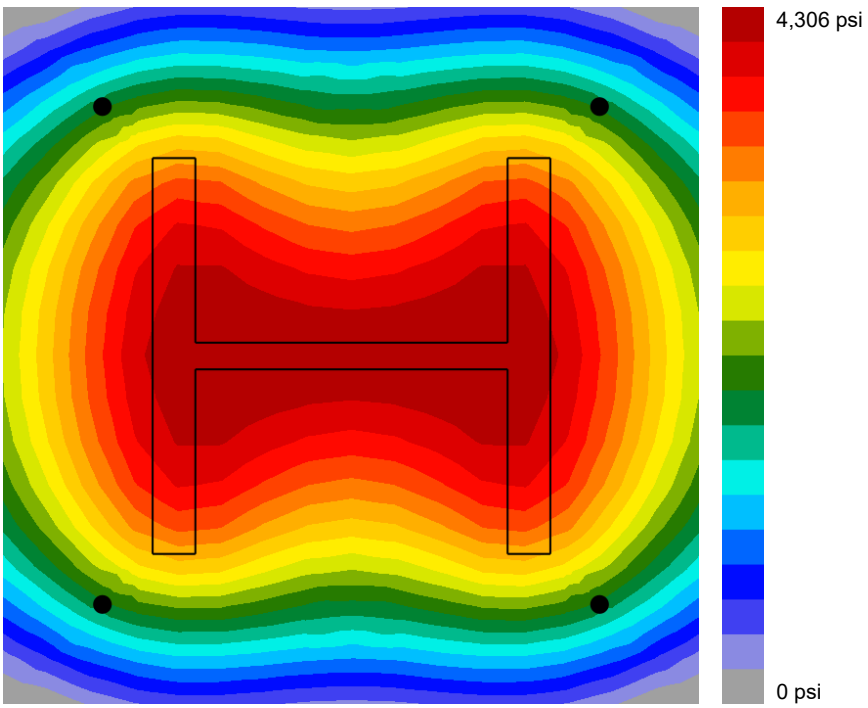
Company:		Page:	12
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Green BP (~2000 kips)	Date:	1/28/2025
Fastening point:			

	V [kip]	ΦR_n [kip]	Utilization [%]	Status
Anchor 4	3.562	219.375	2	OK

2.5 Concrete

Decisive load combination: 1 - Combination 1

2.5.1 Compression in concrete under the anchor plate



2.5.2 Concrete block compressive strength resistance check, AISC 360-16 Section J8

Equations

$$F_p = \Phi f_{p,max}$$

$$f_{p,max} = 0.85 f_c' \sqrt{\left(\frac{A_2}{A_1}\right)} \le 1.7 f_c; \quad \sqrt{\left(\frac{A_2}{A_1}\right)} \le 2$$

$$\sigma = \frac{N}{A_1}$$

$$\text{Utilization} = \frac{\sigma}{F_p}$$

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Company:		Page:	13
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Green BP (~2000 kips)	Date:	1/28/2025
Fastening point:			

Variables

N [kip]	f_c' [psi]	Φ	A_1 [in²]	A_2 [in²]
2,049.503	4,500	0.65	762.31	2,136.03

Results

Load combination	F_p [psi]	σ [psi]	Utilization [%]	Status
Combination 1	4,162	2,689	65	OK

2.6 Symbol explanation

A_1	Loaded area of concrete
A_2	Supporting area
d	Nominal diameter of the bolt
ε_{lim}	Limit plastic strain
ε_{Pl}	Plastic strain from CBFEM results
f_c	Concrete compressive strength
f_c'	Concrete compressive strength
F_u	Specified minimum tensile strength of the connected material
F_p	Concrete block design bearing strength
$f_{p,max}$	Concrete block design bearing strength maximum
f_y	Yield strength
l_c	Clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material
N	Resulting compression force
σ	Average stress in concrete
σ_{Ed}	Equivalent stress
Φ	Resistance factor
ΦR_n	Factored resistance
t	Thickness of the anchor plate
V	Resultant of shear forces Vy, Vz in bolt.

2.7 Warnings

- By using the CBFEM calculation functionality of PROFIS Engineering you may act outside the applicable design codes and your specified anchor plate may not behave rigid. Please, validate the results with a professional designer and/or structural engineer to ensure suitability and adequacy for your specific jurisdiction and project requirements.
- The anchor is modeled considering stiffness values determined from load displacement curves tested in an independent laboratory. Please note that no simple replacement of the anchor is possible as the anchor stiffness has a major impact on the load distribution results.

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	14
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Green BP (~2000 kips)	Date:	1/28/2025
Fastening point:			

3 Summary of results

Design of the anchor plate, anchors, welds and other elements are based on CBFEM (component based finite element method) and AISC.

	Load combination	Max. utilization	Status
Anchors	Combination 1	58%	OK
Anchor plate	Combination 1	63%	OK
Concrete	Combination 1	65%	OK
Profile	Combination 1	100%	OK

Fastening meets the design criteria!

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	15
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Green BP (~2000 kips)	Date:	1/28/2025
Fastening point:			

4 Remarks; Your Cooperation Duties

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Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Red BP (~2631 kips)	Date:	1/28/2025
Fastening point:			

Specifier's comments:

1 Anchor Design

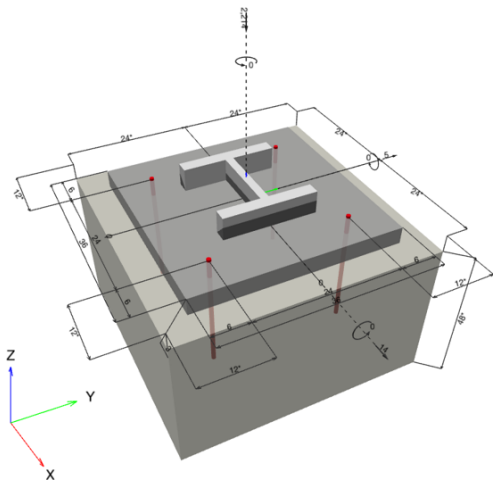
1.1 Input data



Anchor type and diameter:	**Heavy Hex Head ASTM F 1554 GR. 36 3/4**
Item number:	not available
Specification text:	∅ 3/4 in Heavy Hex Head ASTM F 1554 GR. 36 with 18 in nominal embedment depth per Technical data , cast in place installation per MPII
Effective embedment depth:	h_{ef} = 18.000 in.
Material:	ASTM F 1554
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-19 / CIP
Shear edge breakout verification:	Row closest to edge (Case 3 only from ACI 318-19 Fig. R.17.7.2.1b)
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 3.000 in.
Anchor plate[CBFEM] :	l_x x l_y x t = 36.000 in. x 36.000 in. x 3.000 in.;
Profile:	W shape (AISC), W14X311; (L x W x T x FT) = 17.100 in. x 16.200 in. x 1.410 in. x 2.260 in.
Base material:	cracked concrete, 4000, f_c' = 4,000 psi; h = 48.000 in.
Reinforcement:	tension: not present, shear: not present; edge reinforcement: none or < No. 4 bar

[CBFEM] - The anchor calculation is based on a component-based Finite Element Method (CBFEM)

Geometry [in.] & Loading [kip, ft.kip]



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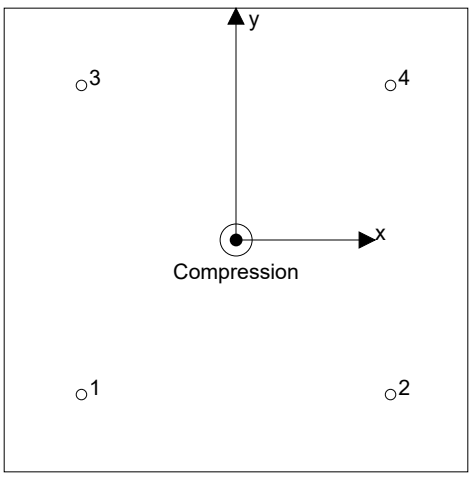
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Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Red BP (~2631 kips)	Date:	1/28/2025
Fastening point:			

1.1.1 Design results

Case	Description	Forces [kip] / Moments [ft.kip]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = -2,214.000; V_x = 14.000; V_y = 5.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	75

1.2 Load case/Resulting anchor forces



Anchor reactions [kip]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	-0.008	3.586	3.442	1.006
2	-0.008	3.691	3.558	0.981
3	-0.008	3.748	3.438	1.494
4	-0.008	3.872	3.562	1.519

Resulting tension force in (x/y)=(-/-): 0.000 [kip]
Resulting compression force in (x/y)=(0.010/0.001): 2,217.429 [kip]

Anchor forces are calculated based on a component-based Finite Element Method (CBFEM)

1.3 Tension load

	Load N_{ua} [kip]	Capacity ϕ N_n [kip]	Utilization $\beta_N = N_{ua}/\phi N_n$	Status
Steel Strength*	-0.008	14.529	1	OK
Pullout Strength*	-0.008	20.406	-0	not recommended
Concrete Breakout Failure**	N/A	N/A	N/A	N/A
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (anchors in tension)

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Company:		Page:	3
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Red BP (~2631 kips)	Date:	1/28/2025
Fastening point:			

1.3.1 Steel Strength

$N_{sa} = A_{se,N} f_{uta}$ ACI 318-19 Eq. (17.6.1.2)
$\phi N_{sa} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.33	58,000

Calculations

N_{sa} [kip]
19.372

Results

N_{sa} [kip]	ϕ_{steel}	ϕN_{sa} [kip]	N_{ua} [kip]
19.372	0.750	14.529	-0.008

The steel proof was done for the highest absolute force per anchor - in this case compression loading. Please be aware that buckling should be verified separately

1.3.2 Pullout Strength

$N_{pN} = \psi_{c,p} N_p$ ACI 318-19 Eq. (17.6.3.1)
$N_p = 8 A_{brg} f'_c$ ACI 318-19 Eq. (17.6.3.2.2a)
$\phi N_{pN} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$\psi_{c,p}$	A_{brg} [in.^{2}]	λ_a	f'_c [psi]
1.000	0.91	1.000	4,000

Calculations

N_p [kip]
29.152

Results

N_{pn} [kip]	$\phi_{concrete}$	ϕN_{pn} [kip]	N_{ua} [kip]
29.152	0.700	20.406	-0.008

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Company:		Page:	4
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Red BP (~2631 kips)	Date:	1/28/2025
Fastening point:			

1.4 Shear load

	Load V_{ua} [kip]	Capacity ϕ V_n [kip]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	3.872	7.555	52	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	14.866	192.338	8	OK
Concrete edge failure in direction x+**	14.866	19.876	75	OK

* highest loaded anchor **anchor group (relevant anchors)

1.4.1 Steel Strength

$V_{sa} = 0.6\ A_{se,V}\ f_{uta}$ ACI 318-19 Eq. (17.7.1.2b)
$\phi\ V_{steel} \geq V_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.33	58,000

Calculations

V_{sa} [kip]
11.623

Results

V_{sa} [kip]	ϕ_{steel}	$\phi\ V_{sa}$ [kip]	V_{ua} [kip]
11.623	0.650	7.555	3.872

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	5
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Red BP (~2631 kips)	Date:	1/28/2025
Fastening point:			

1.4.2 Pryout Strength

$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \right]$ ACI 318-19 Eq. (17.7.3.1b)

$\phi V_{cpg} \geq V_{ua}$ ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9 h_{ef}^2$ ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e'_N}{3 h_{ef}}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ ACI 318-19 Eq. (17.6.2.2.1)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	8.000	0.000	0.000	12.000

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.000	-	24	1.000	4,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [kip]
2,304.00	576.00	1.000	1.000	1.000	1.000	34.346

Results

V_{cpg} [kip]	$\phi_{concrete}$	ϕV_{cpg} [kip]	V_{ua} [kip]
274.768	0.700	192.338	14.866

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Company:		Page:	6
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Red BP (~2631 kips)	Date:	1/28/2025
Fastening point:			

1.4.3 Concrete edge failure in direction x+

$V_{cbg} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \psi_{ec,V}\ \psi_{ed,V}\ \psi_{c,V}\ \psi_{h,V}\ \psi_{parallel,V}\ V_b$ — ACI 318-19 Eq. (17.7.2.1b)

$\phi\ V_{cbg} \geq V_{ua}$ — ACI 318-19 Table 17.5.2

A_{Vc} see ACI 318-19, Section 17.7.2.1, Fig. R 17.7.2.1(b)*

$A_{Vc0} = 4.5\ c_{a1}^2$ — ACI 318-19 Eq. (17.7.2.1.3)

$\psi_{ec,V} = \left(\frac{1}{1 + \frac{e'_v}{1.5c_{a1}}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.3.1)

$\psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.4.1b)

$\psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ — ACI 318-19 Eq. (17.7.2.6.1)

$V_b = 9\ \lambda_a\ \sqrt{f'_c}\ c_{a1}^{1.5}$ — ACI 318-19 Eq. (17.7.2.2.1b)

Variables

c_{a1} [in.]	c_{a2} [in.]	e_{cV} [in.]	$\psi_{c,V}$	h_a [in.]
12.000	12.000	0.000	1.000	48.000

l_e [in.]	λ_a	d_a [in.]	f'_c [psi]	$\psi_{parallel,V}$
6.000	1.000	0.750	4,000	1.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\psi_{ec,V}$	$\psi_{ed,V}$	$\psi_{h,V}$	V_b [kip]
864.00	648.00	1.000	0.900	1.000	23.662

Results

V_{cbg} [kip]	$\phi_{concrete}$	$\phi\ V_{cbg}$ [kip]	V_{ua} [kip]
28.394	0.700	19.876	14.866

*Anchor row defined by: Anchor 2, 4; Case 3 controls

1.5 Combined tension and shear loads, per ACI 318-19 section 17.8

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.001	0.748	5/3	62	OK

$\beta_{NV} = \beta_N^{\zeta} + \beta_V^{\zeta} <= 1$

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	7
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Red BP (~2631 kips)	Date:	1/28/2025
Fastening point:			

1.6 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates as per current regulations (ETAG 001/Annex C, EOTA TR029, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid base plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- The equations presented in this report are based on imperial units. When inputs are displayed in metric units, the user should be aware that the equations remain in their imperial format.
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- For additional information about ACI 318 strength design provisions, please go to https://submittals.us.hilti.com/PROFISAnchorDesignGuide/
- Attention! In case of compressive anchor forces a buckling check as well as the proof of the local load transfer into and within the base material (incl. punching) has to be done separately.
- The anchor design methods in PROFIS Engineering require rigid anchor plates, as per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means that the anchor plate should be sufficiently rigid to prevent load re-distribution to the anchors due to elastic/plastic displacements. The user accepts that the anchor plate is considered close to rigid by engineering judgment."

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	8
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Red BP (~2631 kips)	Date:	1/28/2025
Fastening point:			

1.7 Installation data

Profile: W shape (AISC), W14X311; (L x W x T x FT) = 17.100 in. x 16.200 in. x 1.410 in. x 2.260 in.

Hole diameter in the fixture: d_f = 0.812 in.

Plate thickness (input): 3.000 in.

Anchor type and diameter: Heavy Hex Head ASTM F 1554 GR. 36 3/4

Item number: not available

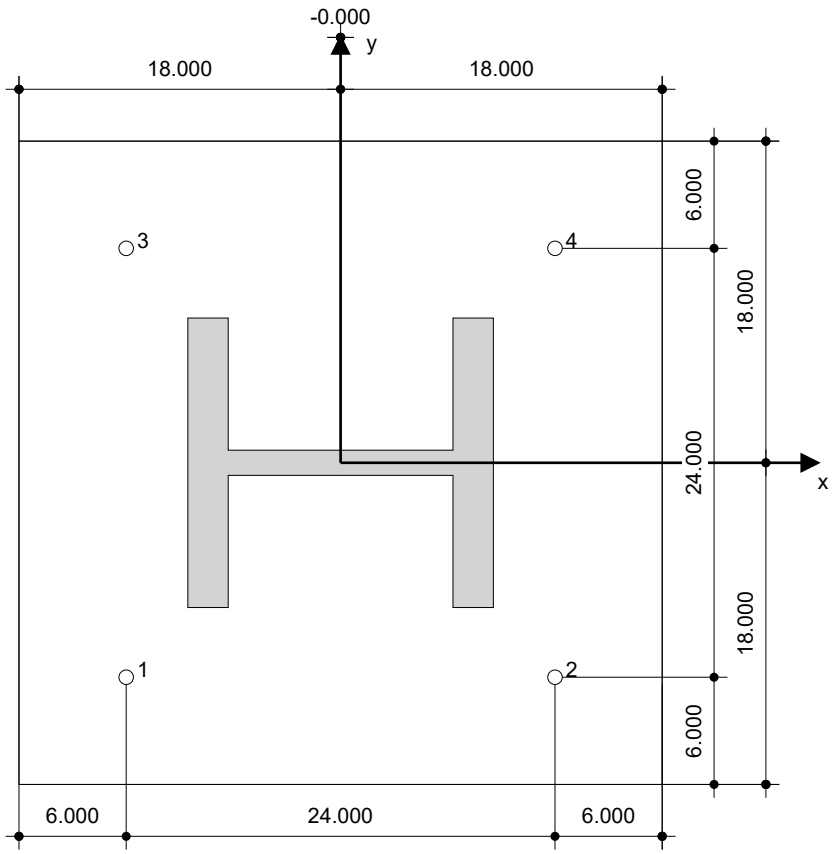
Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 18.000 in.

Minimum thickness of the base material: 19.000 in.

⌀ 3/4 in Heavy Hex Head ASTM F 1554 GR. 36 with 18 in nominal embedment depth per Technical data , cast in place installation per MPII



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-12.000	-12.000	12.000	36.000	12.000	36.000
2	12.000	-12.000	36.000	12.000	12.000	36.000
3	-12.000	12.000	12.000	36.000	36.000	12.000
4	12.000	12.000	36.000	12.000	36.000	12.000

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Company:		Page:	9
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Red BP (~2631 kips)	Date:	1/28/2025
Fastening point:			

2 Anchor plate design

2.1 Input data

Anchor plate:	Shape: Rectangular l_x x l_y x t = 36.000 in x 36.000 in x 3.000 in Calculation: CBFEM Material: ASTM A572 Gr.50; F_y = 50,000 psi; ε_{lim} = 5.00%
Anchor type and size:	Heavy Hex Head ASTM F 1554 GR. 36 3/4, h_{ef} = 18.000 in
Anchor stiffness:	The anchor is modeled considering stiffness values determined from load displacement curves tested in an independent laboratory. Please note that no simple replacement of the anchor is possible as the anchor stiffness has a major impact on the load distribution results.
Design method:	AISC and LRFD-based design using component-based FEM
Stand-off installation:	e_b = 0.000 in (No stand-off); t = 3.000 in
Profile:	W14X311; (L x W x T x FT) = 17.100 in x 16.200 in x 1.410 in x 2.260 in Material: ASTM A992; F_y = 50,000 psi; ε_{lim} = 5.00% Eccentricity x: -0.000 in Eccentricity y: 0.000 in
Base material:	Cracked concrete; 4000; $f_{c,cyl}$ = 4,000 psi; h = 48.000 in
Welds (profile to anchor plate):	Type of redistribution: Plastic Material: E70xx
Mesh size:	Number of elements on edge: 8 Min. size of element: 0.394 in Max. size of element: 1.969 in

2.2 Summary

	Description	Profile		Anchor plate			Concrete [%]
		σ_{Ed} [psi]	ε_{Pl} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Hole bearing [%]	
1	Combination 1	50,057	0.20	31,330	0.00	2	69

2.3 Anchor plate classification

Results below are displayed for the decisive load combinations: Combination 1

Anchor tension forces	Equivalent rigid anchor plate (CBFEM)	Component-based Finite Element Method (CBFEM) anchor plate design
Anchor 1	-0.013 kip	-0.008 kip
Anchor 2	-0.013 kip	-0.008 kip
Anchor 3	-0.013 kip	-0.008 kip
Anchor 4	-0.013 kip	-0.008 kip

User accepted to consider the selected anchor plate as rigid by his/her engineering judgement. This means the anchor design guidelines can be applied.

2.4 Profile/Stiffeners/Plate

Profile and stiffeners are verified at the level of the steel to concrete connection. The connection design does not replace the steel design for critical cross sections, which should be performed outside of PROFIS Engineering.

2.4.1 Equivalent stress and plastic strain

Part	Load combination	Material	f_y [psi]	ε_{lim} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Status
Plate	Combination 1	ASTM A572 Gr.50	50,000	5.00	31,330	0.00	OK

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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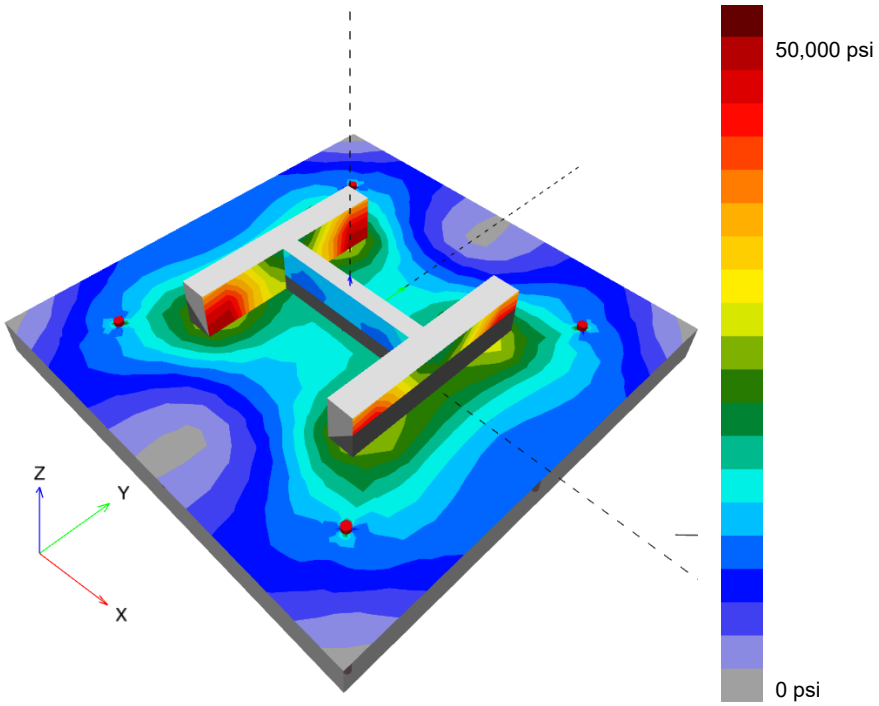
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Company:		Page:	10
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Red BP (~2631 kips)	Date:	1/28/2025
Fastening point:			

Part	Load combination	Material	f_y [psi]	ε_{lim} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Status
Profile	Combination 1	ASTM A992	50,000	5.00	50,057	0.20	OK
Profile	Combination 1	ASTM A992	50,000	5.00	50,057	0.19	OK
Profile	Combination 1	ASTM A992	50,000	5.00	24,114	0.00	OK

2.4.1.1 Equivalent stress

Results below are displayed for the decisive load combination: 1 - Combination 1

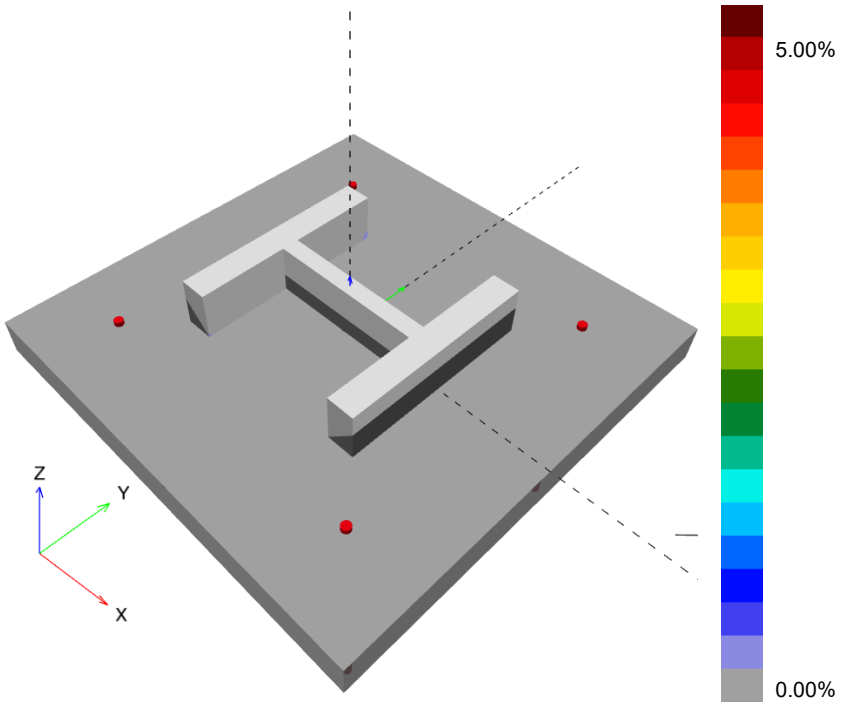


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Company:		Page:	11
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Red BP (~2631 kips)	Date:	1/28/2025
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2.4.1.2 Plastic strain

Results below are displayed for the decisive load combination: 1 - Combination 1



2.4.2 Plate hole bearing resistance, AISC 360-16 Section J3

Decisive load combination: 1 - Combination 1

Equations

$R_n = \min(1.2\, l_c\, t\, F_u\, ,\ 2.4\, d\, t\, F_u)$ (AISC 360-16 J3-6a, c)

$\Phi R_n = 0.75\, R_n$

$V \leq \Phi R_n$

Variables

	l_c [in]	t [in]	F_u [psi]	d [in]	R_n [kip]
Anchor 1	5.845	3.000	65,000	0.750	351.000
Anchor 2	22.167	3.000	65,000	0.750	351.000
Anchor 3	6.136	3.000	65,000	0.750	351.000
Anchor 4	32.208	3.000	65,000	0.750	351.000

Results

	V [kip]	ΦR_n [kip]	Utilization [%]	Status
Anchor 1	3.586	263.250	2	OK
Anchor 2	3.691	263.250	2	OK
Anchor 3	3.748	263.250	2	OK

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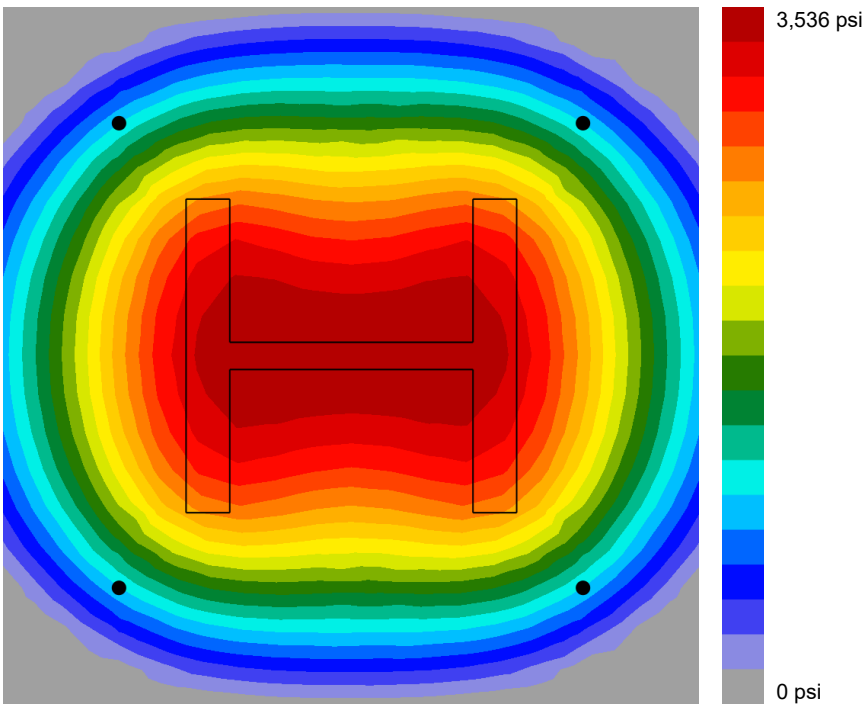
Company:		Page:	12
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Red BP (~2631 kips)	Date:	1/28/2025
Fastening point:			

	V [kip]	ΦR_n [kip]	Utilization [%]	Status
Anchor 4	3.872	263.250	2	OK

2.5 Concrete

Decisive load combination: 1 - Combination 1

2.5.1 Compression in concrete under the anchor plate



2.5.2 Concrete block compressive strength resistance check, AISC 360-16 Section J8

Equations

$$F_p = \Phi f_{p,max}$$

$$f_{p,max} = 0.85 f_c' \sqrt{\left(\frac{A_2}{A_1}\right)} \le 1.7 f_c; \quad \sqrt{\left(\frac{A_2}{A_1}\right)} \le 2$$

$$\sigma = \frac{N}{A_1}$$

$$\text{Utilization} = \frac{\sigma}{F_p}$$

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Company:		Page:	13
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Red BP (~2631 kips)	Date:	1/28/2025
Fastening point:			

Variables

N [kip]	f_c' [psi]	Φ	A_1 [in²]	A_2 [in²]
2,217.428	4,000	0.65	1,096.64	1,933.83

Results

Load combination	F_p [psi]	σ [psi]	Utilization [%]	Status
Combination 1	2,935	2,022	69	OK

2.6 Symbol explanation

A_1	Loaded area of concrete
A_2	Supporting area
d	Nominal diameter of the bolt
ε_{lim}	Limit plastic strain
ε_{Pl}	Plastic strain from CBFEM results
f_c	Concrete compressive strength
f_c'	Concrete compressive strength
F_u	Specified minimum tensile strength of the connected material
F_p	Concrete block design bearing strength
$f_{p,max}$	Concrete block design bearing strength maximum
f_y	Yield strength
l_c	Clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material
N	Resulting compression force
σ	Average stress in concrete
σ_{Ed}	Equivalent stress
Φ	Resistance factor
ΦR_n	Factored resistance
t	Thickness of the anchor plate
V	Resultant of shear forces Vy, Vz in bolt.

2.7 Warnings

- By using the CBFEM calculation functionality of PROFIS Engineering you may act outside the applicable design codes and your specified anchor plate may not behave rigid. Please, validate the results with a professional designer and/or structural engineer to ensure suitability and adequacy for your specific jurisdiction and project requirements.
- The anchor is modeled considering stiffness values determined from load displacement curves tested in an independent laboratory. Please note that no simple replacement of the anchor is possible as the anchor stiffness has a major impact on the load distribution results.

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	14
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Red BP (~2631 kips)	Date:	1/28/2025
Fastening point:			

3 Summary of results

Design of the anchor plate, anchors, welds and other elements are based on CBFEM (component based finite element method) and AISC.

	Load combination	Max. utilization	Status
Anchors	Combination 1	75%	OK
Anchor plate	Combination 1	63%	OK
Concrete	Combination 1	69%	OK
Profile	Combination 1	100%	OK

Fastening meets the design criteria!

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Company:		Page:	15
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Red BP (~2631 kips)	Date:	1/28/2025
Fastening point:			

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Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Red BP (~2631 kips)	Date:	1/28/2025
Fastening point:			

Specifier's comments:

1 Anchor Design

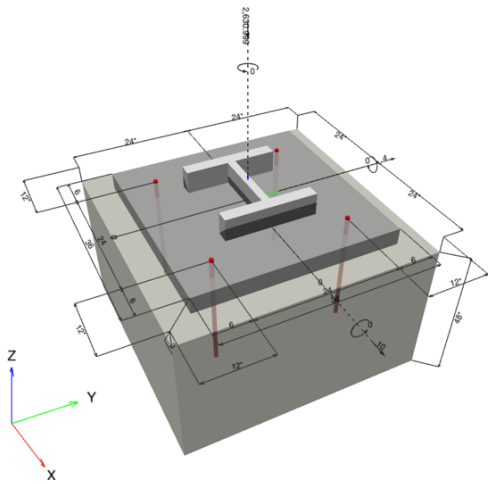
1.1 Input data



Anchor type and diameter:	**Heavy Hex Head ASTM F 1554 GR. 36 3/4**
Item number:	not available
Specification text:	∅ 3/4 in Heavy Hex Head ASTM F 1554 GR. 36 with 18 in nominal embedment depth per Technical data , cast in place installation per MPII
Effective embedment depth:	h_{ef} = 18.000 in.
Material:	ASTM F 1554
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-19 / CIP
Shear edge breakout verification:	Row closest to edge (Case 3 only from ACI 318-19 Fig. R.17.7.2.1b)
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 3.000 in.
Anchor plate[CBFEM] :	l_x x l_y x t = 36.000 in. x 36.000 in. x 3.000 in.;
Profile:	W shape (AISC), W14X311; (L x W x T x FT) = 17.100 in. x 16.200 in. x 1.410 in. x 2.260 in.
Base material:	cracked concrete, 4000, f_c' = 4,000 psi; h = 48.000 in.
Reinforcement:	tension: not present, shear: not present; edge reinforcement: none or < No. 4 bar

[CBFEM] - The anchor calculation is based on a component-based Finite Element Method (CBFEM)

Geometry [in.] & Loading [kip, ft.kip]



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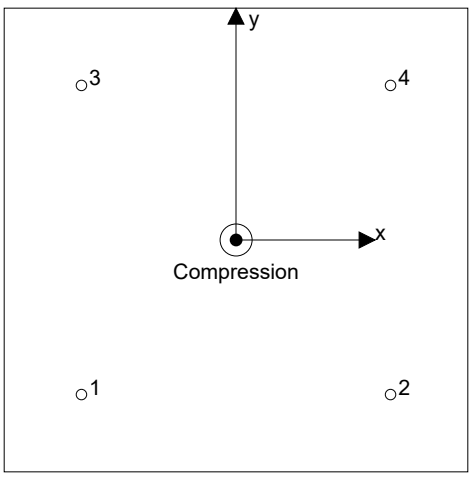
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Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Red BP (~2631 kips)	Date:	1/28/2025
Fastening point:			

1.1.1 Design results

Case	Description	Forces [kip] / Moments [ft.kip]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = -2,631.000; V_x = 10.000; V_y = 4.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	55

1.2 Load case/Resulting anchor forces



Anchor reactions [kip]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	-0.009	2.539	2.433	0.727
2	-0.009	2.663	2.567	0.705
3	-0.009	2.742	2.428	1.273
4	-0.009	2.879	2.572	1.295

Resulting tension force in (x/y)=(-/-): 0.000 [kip]
Resulting compression force in (x/y)=(0.008/0.001): 2,635.522 [kip]

Anchor forces are calculated based on a component-based Finite Element Method (CBFEM)

1.3 Tension load

	Load N_{ua} [kip]	Capacity ϕ N_n [kip]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	-0.009	14.529	1	OK
Pullout Strength*	-0.009	20.406	-0	not recommended
Concrete Breakout Failure**	N/A	N/A	N/A	N/A
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (anchors in tension)

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Company:		Page:	3
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Red BP (~2631 kips)	Date:	1/28/2025
Fastening point:			

1.3.1 Steel Strength

$N_{sa} = A_{se,N} f_{uta}$ ACI 318-19 Eq. (17.6.1.2)
$\phi N_{sa} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.33	58,000

Calculations

N_{sa} [kip]
19.372

Results

N_{sa} [kip]	ϕ_{steel}	ϕN_{sa} [kip]	N_{ua} [kip]
19.372	0.750	14.529	-0.009

The steel proof was done for the highest absolute force per anchor - in this case compression loading. Please be aware that buckling should be verified separately

1.3.2 Pullout Strength

$N_{pN} = \psi_{c,p} N_p$ ACI 318-19 Eq. (17.6.3.1)
$N_p = 8 A_{brg} f'_c$ ACI 318-19 Eq. (17.6.3.2.2a)
$\phi N_{pN} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$\psi_{c,p}$	A_{brg} [in.^{2}]	λ_a	f'_c [psi]
1.000	0.91	1.000	4,000

Calculations

N_p [kip]
29.152

Results

N_{pn} [kip]	$\phi_{concrete}$	ϕN_{pn} [kip]	N_{ua} [kip]
29.152	0.700	20.406	-0.009

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Company:		Page:	4
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Red BP (~2631 kips)	Date:	1/28/2025
Fastening point:			

1.4 Shear load

	Load V_{ua} [kip]	Capacity ϕ V_n [kip]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	2.879	7.555	39	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	10.770	192.338	6	OK
Concrete edge failure in direction y+**	10.770	19.875	55	OK

* highest loaded anchor **anchor group (relevant anchors)

1.4.1 Steel Strength

$V_{sa} = 0.6\ A_{se,V}\ f_{uta}$ ACI 318-19 Eq. (17.7.1.2b)
$\phi\ V_{steel} \geq V_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.33	58,000

Calculations

V_{sa} [kip]
11.623

Results

V_{sa} [kip]	ϕ_{steel}	$\phi\ V_{sa}$ [kip]	V_{ua} [kip]
11.623	0.650	7.555	2.879

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	5
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Red BP (~2631 kips)	Date:	1/28/2025
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1.4.2 Pryout Strength

$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \right]$ ACI 318-19 Eq. (17.7.3.1b)

$\phi V_{cpg} \geq V_{ua}$ ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9 h_{ef}^2$ ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e'_N}{3 h_{ef}}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ ACI 318-19 Eq. (17.6.2.2.1)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	8.000	0.000	0.000	12.000

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.000	-	24	1.000	4,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [kip]
2,304.00	576.00	1.000	1.000	1.000	1.000	34.346

Results

V_{cpg} [kip]	$\phi_{concrete}$	ϕV_{cpg} [kip]	V_{ua} [kip]
274.768	0.700	192.338	10.770

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	6
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Red BP (~2631 kips)	Date:	1/28/2025
Fastening point:			

1.4.3 Concrete edge failure in direction y+

$V_{cbg} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \psi_{ec,V}\ \psi_{ed,V}\ \psi_{c,V}\ \psi_{h,V}\ \psi_{parallel,V}\ V_b$ — ACI 318-19 Eq. (17.7.2.1b)

$\phi\ V_{cbg} \geq V_{ua}$ — ACI 318-19 Table 17.5.2

A_{Vc} see ACI 318-19, Section 17.7.2.1, Fig. R 17.7.2.1(b)*

$A_{Vc0} = 4.5\ c_{a1}^2$ — ACI 318-19 Eq. (17.7.2.1.3)

$\psi_{ec,V} = \left(\frac{1}{1 + \frac{e'_v}{1.5c_{a1}}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.3.1)

$\psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.4.1b)

$\psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ — ACI 318-19 Eq. (17.7.2.6.1)

$V_b = 9\ \lambda_a\ \sqrt{f'_c}\ c_{a1}^{1.5}$ — ACI 318-19 Eq. (17.7.2.2.1b)

Variables

c_{a1} [in.]	c_{a2} [in.]	e_{cV} [in.]	$\psi_{c,V}$	h_a [in.]
12.000	12.000	0.000	1.000	48.000

l_e [in.]	λ_a	d_a [in.]	f'_c [psi]	$\psi_{parallel,V}$
6.000	1.000	0.750	4,000	1.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\psi_{ec,V}$	$\psi_{ed,V}$	$\psi_{h,V}$	V_b [kip]
864.00	648.00	1.000	0.900	1.000	23.662

Results

V_{cbg} [kip]	$\phi_{concrete}$	$\phi\ V_{cbg}$ [kip]	V_{ua} [kip]
28.393	0.700	19.875	10.770

*Anchor row defined by: Anchor 3, 4; Case 3 controls

1.5 Combined tension and shear loads, per ACI 318-19 section 17.8

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.001	0.542	5/3	37	OK

$\beta_{NV} = \beta_N^{\zeta} + \beta_V^{\zeta} <= 1$

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Company:		Page:	7
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Red BP (~2631 kips)	Date:	1/28/2025
Fastening point:			

1.6 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates as per current regulations (ETAG 001/Annex C, EOTA TR029, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid base plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- The equations presented in this report are based on imperial units. When inputs are displayed in metric units, the user should be aware that the equations remain in their imperial format.
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- For additional information about ACI 318 strength design provisions, please go to https://submittals.us.hilti.com/PROFISAnchorDesignGuide/
- Attention! In case of compressive anchor forces a buckling check as well as the proof of the local load transfer into and within the base material (incl. punching) has to be done separately.
- The anchor design methods in PROFIS Engineering require rigid anchor plates, as per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means that the anchor plate should be sufficiently rigid to prevent load re-distribution to the anchors due to elastic/plastic displacements. The user accepts that the anchor plate is considered close to rigid by engineering judgment."

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Company:		Page:	8
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Red BP (~2631 kips)	Date:	1/28/2025
Fastening point:			

1.7 Installation data

Profile: W shape (AISC), W14X311; (L x W x T x FT) = 17.100 in. x 16.200 in. x 1.410 in. x 2.260 in.

Hole diameter in the fixture: d_f = 0.812 in.

Plate thickness (input): 3.000 in.

Anchor type and diameter: Heavy Hex Head ASTM F 1554 GR. 36 3/4

Item number: not available

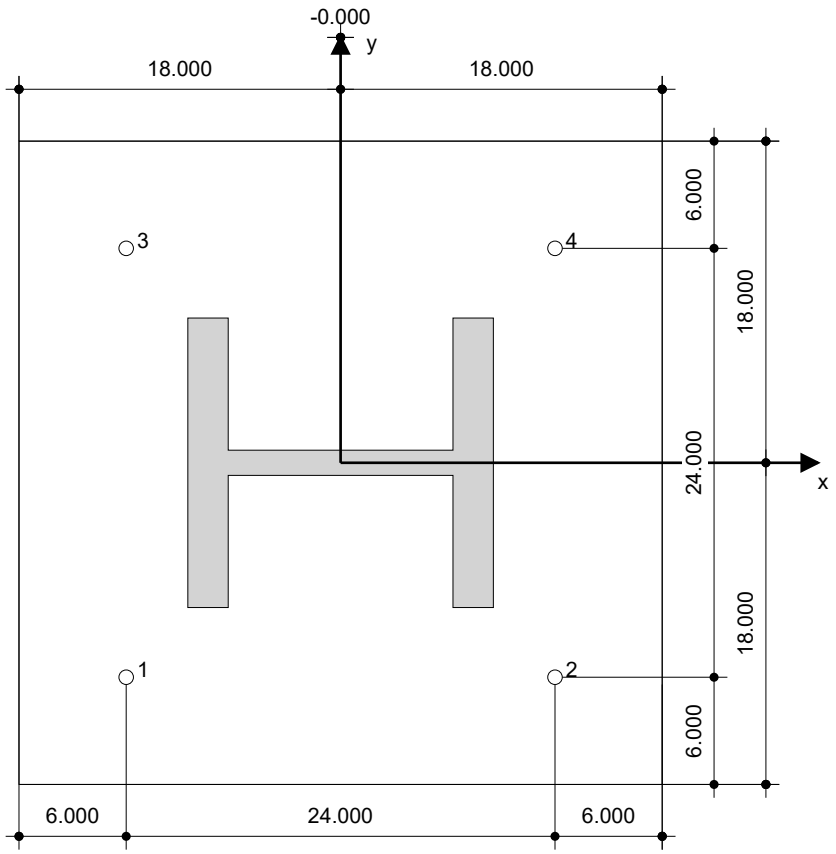
Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 18.000 in.

Minimum thickness of the base material: 19.000 in.

⌀ 3/4 in Heavy Hex Head ASTM F 1554 GR. 36 with 18 in nominal embedment depth per Technical data , cast in place installation per MPII



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-12.000	-12.000	12.000	36.000	12.000	36.000
2	12.000	-12.000	36.000	12.000	12.000	36.000
3	-12.000	12.000	12.000	36.000	36.000	12.000
4	12.000	12.000	36.000	12.000	36.000	12.000

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	9
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Red BP (~2631 kips)	Date:	1/28/2025
Fastening point:			

2 Anchor plate design

2.1 Input data

Anchor plate:	Shape: Rectangular l_x x l_y x t = 36.000 in x 36.000 in x 3.000 in Calculation: CBFEM Material: ASTM A572 Gr.50; F_y = 50,000 psi; ε_{lim} = 5.00%
Anchor type and size:	Heavy Hex Head ASTM F 1554 GR. 36 3/4, h_{ef} = 18.000 in
Anchor stiffness:	The anchor is modeled considering stiffness values determined from load displacement curves tested in an independent laboratory. Please note that no simple replacement of the anchor is possible as the anchor stiffness has a major impact on the load distribution results.
Design method:	AISC and LRFD-based design using component-based FEM
Stand-off installation:	e_b = 0.000 in (No stand-off); t = 3.000 in
Profile:	W14X311; (L x W x T x FT) = 17.100 in x 16.200 in x 1.410 in x 2.260 in Material: ASTM A992; F_y = 50,000 psi; ε_{lim} = 5.00% Eccentricity x: -0.000 in Eccentricity y: 0.000 in
Base material:	Cracked concrete; 4000; $f_{c,cyl}$ = 4,000 psi; h = 48.000 in
Welds (profile to anchor plate):	Type of redistribution: Plastic Material: E70xx
Mesh size:	Number of elements on edge: 8 Min. size of element: 0.394 in Max. size of element: 1.969 in

2.2 Summary

	Description	Profile		Anchor plate			Concrete [%]
		σ_{Ed} [psi]	ε_{Pl} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Hole bearing [%]	
1	Combination 1	50,108	0.37	36,801	0.00	2	84

2.3 Anchor plate classification

Results below are displayed for the decisive load combinations: Combination 1

Anchor tension forces	Equivalent rigid anchor plate (CBFEM)	Component-based Finite Element Method (CBFEM) anchor plate design
Anchor 1	-0.015 kip	-0.009 kip
Anchor 2	-0.015 kip	-0.009 kip
Anchor 3	-0.015 kip	-0.009 kip
Anchor 4	-0.015 kip	-0.009 kip

User accepted to consider the selected anchor plate as rigid by his/her engineering judgement. This means the anchor design guidelines can be applied.

2.4 Profile/Stiffeners/Plate

Profile and stiffeners are verified at the level of the steel to concrete connection. The connection design does not replace the steel design for critical cross sections, which should be performed outside of PROFIS Engineering.

2.4.1 Equivalent stress and plastic strain

Part	Load combination	Material	f_y [psi]	ε_{lim} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Status
Plate	Combination 1	ASTM A572 Gr.50	50,000	5.00	36,801	0.00	OK

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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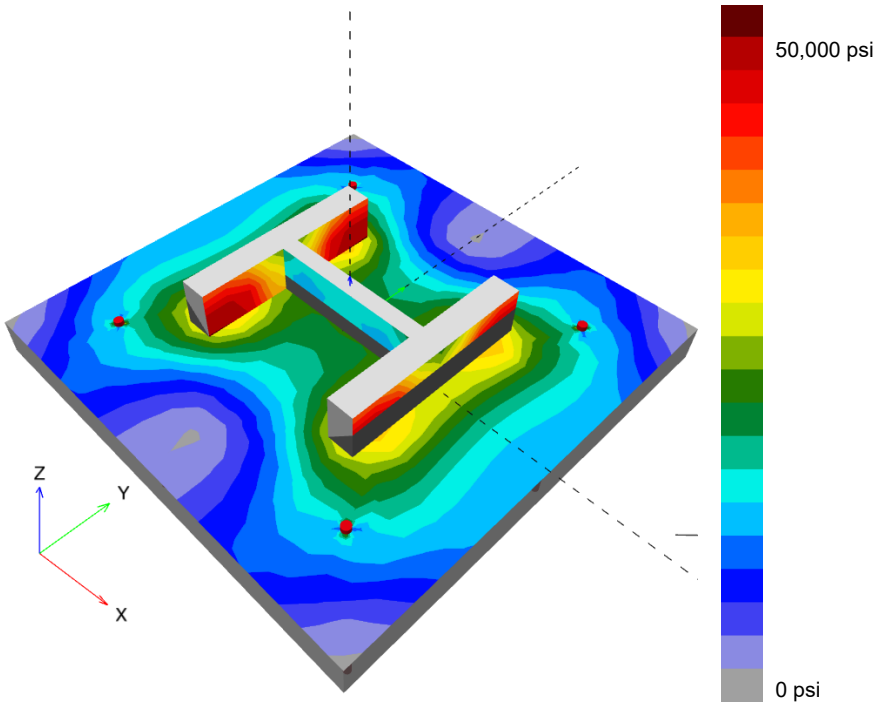
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Company:		Page:	10
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Red BP (~2631 kips)	Date:	1/28/2025
Fastening point:			

Part	Load combination	Material	f_y [psi]	ε_{lim} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Status
Profile	Combination 1	ASTM A992	50,000	5.00	50,108	0.37	OK
Profile	Combination 1	ASTM A992	50,000	5.00	50,107	0.37	OK
Profile	Combination 1	ASTM A992	50,000	5.00	28,363	0.00	OK

2.4.1.1 Equivalent stress

Results below are displayed for the decisive load combination: 1 - Combination 1



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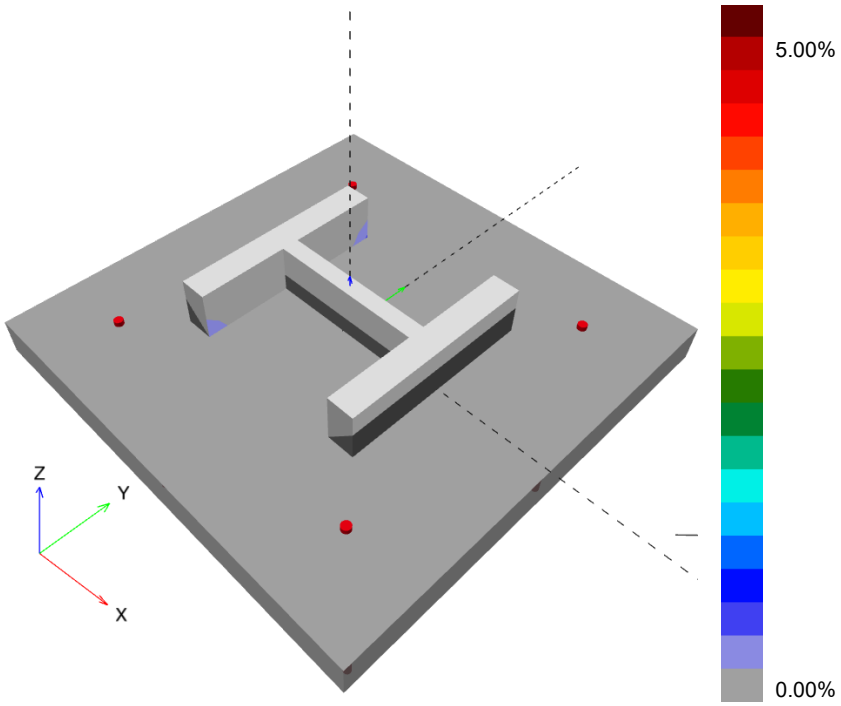
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Company:		Page:	11
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Red BP (~2631 kips)	Date:	1/28/2025
Fastening point:			

2.4.1.2 Plastic strain

Results below are displayed for the decisive load combination: 1 - Combination 1



2.4.2 Plate hole bearing resistance, AISC 360-16 Section J3

Decisive load combination: 1 - Combination 1

Equations

$R_n = \min(1.2\, l_c\, t\, F_u\, ,\ 2.4\, d\, t\, F_u)$ (AISC 360-16 J3-6a, c)

$\Phi R_n = 0.75\, R_n$

$V \leq \Phi R_n$

Variables

	l_c [in]	t [in]	F_u [psi]	d [in]	R_n [kip]
Anchor 1	5.856	3.000	65,000	0.750	351.000
Anchor 2	22.248	3.000	65,000	0.750	351.000
Anchor 3	6.369	3.000	65,000	0.750	351.000
Anchor 4	33.181	3.000	65,000	0.750	351.000

Results

	V [kip]	ΦR_n [kip]	Utilization [%]	Status
Anchor 1	2.539	263.250	1	OK
Anchor 2	2.663	263.250	2	OK
Anchor 3	2.742	263.250	2	OK

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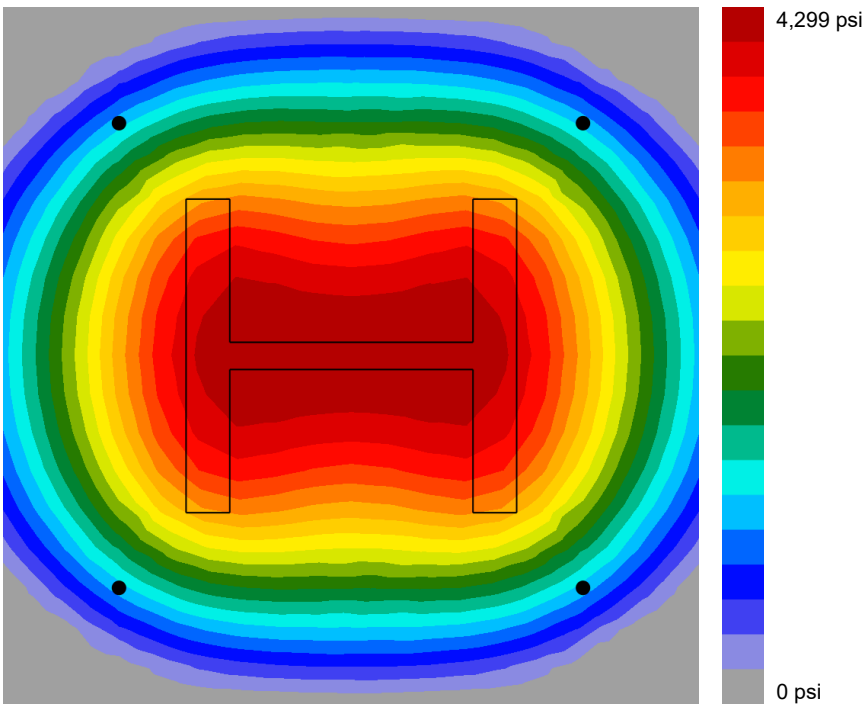
Company:		Page:	12
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Red BP (~2631 kips)	Date:	1/28/2025
Fastening point:			

	V [kip]	ΦR_n [kip]	Utilization [%]	Status
Anchor 4	2.879	263.250	2	OK

2.5 Concrete

Decisive load combination: 1 - Combination 1

2.5.1 Compression in concrete under the anchor plate



2.5.2 Concrete block compressive strength resistance check, AISC 360-16 Section J8

Equations

$$F_p = \Phi f_{p,max}$$

$$f_{p,max} = 0.85 f_c' \sqrt{\left(\frac{A_2}{A_1}\right)} \le 1.7 f_c;\ \sqrt{\left(\frac{A_2}{A_1}\right)} \le 2$$

$$\sigma = \frac{N}{A_1}$$

$$\text{Utilization} = \frac{\sigma}{F_p}$$

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	13
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Red BP (~2631 kips)	Date:	1/28/2025
Fastening point:			

Variables

N [kip]	f_c' [psi]	Φ	A_1 [in²]	A_2 [in²]
2,635.522	4,000	0.65	1,076.64	1,907.88

Results

Load combination	F_p [psi]	σ [psi]	Utilization [%]	Status
Combination 1	2,942	2,448	84	OK

2.6 Symbol explanation

A_1	Loaded area of concrete
A_2	Supporting area
d	Nominal diameter of the bolt
ε_{lim}	Limit plastic strain
ε_{Pl}	Plastic strain from CBFEM results
f_c	Concrete compressive strength
f_c'	Concrete compressive strength
F_u	Specified minimum tensile strength of the connected material
F_p	Concrete block design bearing strength
$f_{p,max}$	Concrete block design bearing strength maximum
f_y	Yield strength
l_c	Clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material
N	Resulting compression force
σ	Average stress in concrete
σ_{Ed}	Equivalent stress
Φ	Resistance factor
ΦR_n	Factored resistance
t	Thickness of the anchor plate
V	Resultant of shear forces Vy, Vz in bolt.

2.7 Warnings

- By using the CBFEM calculation functionality of PROFIS Engineering you may act outside the applicable design codes and your specified anchor plate may not behave rigid. Please, validate the results with a professional designer and/or structural engineer to ensure suitability and adequacy for your specific jurisdiction and project requirements.
- The anchor is modeled considering stiffness values determined from load displacement curves tested in an independent laboratory. Please note that no simple replacement of the anchor is possible as the anchor stiffness has a major impact on the load distribution results.

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	14
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Red BP (~2631 kips)	Date:	1/28/2025
Fastening point:			

3 Summary of results

Design of the anchor plate, anchors, welds and other elements are based on CBFEM (component based finite element method) and AISC.

	Load combination	Max. utilization	Status
Anchors	Combination 1	55%	OK
Anchor plate	Combination 1	74%	OK
Concrete	Combination 1	84%	OK
Profile	Combination 1	100%	OK

Fastening meets the design criteria!

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	15
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Red BP (~2631 kips)	Date:	1/28/2025
Fastening point:			

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Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP (W12x79 Elevator Pit)	Date:	1/28/2025
Fastening point:			

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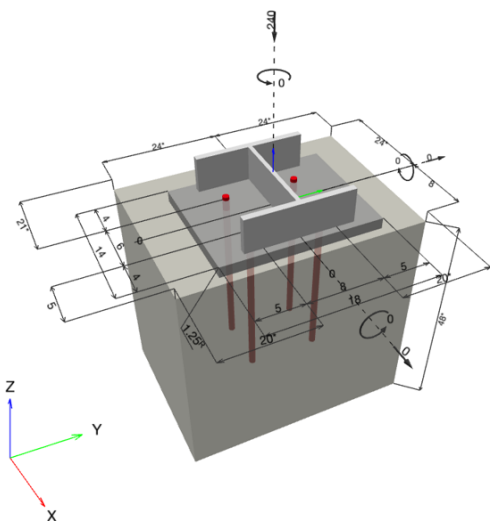
1 Input data



Anchor type and diameter:	**Heavy Hex Head ASTM F 1554 GR. 36 3/4**
Item number:	not available
Specification text:	∅ 3/4 in Heavy Hex Head ASTM F 1554 GR. 36 with 18 in nominal embedment depth per Technical data , cast in place installation per MPII
Effective embedment depth:	h_{ef} = 18.000 in.
Material:	ASTM F 1554
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-19 / CIP
Shear edge breakout verification:	Row closest to edge (Case 3 only from ACI 318-19 Fig. R.17.7.2.1b)
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 1.250 in.
Anchor plate[R] :	l_x x l_y x t = 14.000 in. x 18.000 in. x 1.250 in.;
Profile:	W shape (AISC), W12X79; (L x W x T x FT) = 12.400 in. x 12.100 in. x 0.470 in. x 0.735 in.
Base material:	cracked concrete, 4000, f_c' = 4,000 psi; h = 48.000 in.
Reinforcement:	tension: not present, shear: not present; edge reinforcement: none or < No. 4 bar

[R] - The anchor calculation is based on a rigid anchor plate assumption.

Geometry [in.] & Loading [kip, ft.kip]



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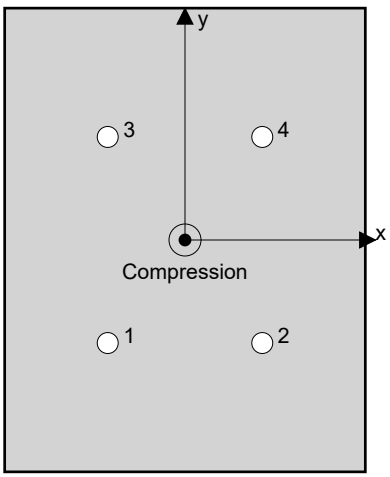
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Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP (W12x79 Elevator Pit)	Date:	1/28/2025
Fastening point:			

1.1 Design results

Case	Description	Forces [kip] / Moments [ft.kip]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = -240.000; V_x = 0.000; V_y = 0.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	-

2 Load case/Resulting anchor forces



Anchor reactions [kip]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	0.000	0.000	0.000	0.000
2	0.000	0.000	0.000	0.000
3	0.000	0.000	0.000	0.000
4	0.000	0.000	0.000	0.000

Max. concrete compressive strain: 0.00 [‰]
Max. concrete compressive stress: 952 [psi]
Resulting tension force in (x/y)=(-/-): 0.000 [kip]
Resulting compression force in (x/y)=(0.000/0.000): 240.000 [kip]

Anchor forces are calculated based on the assumption of a rigid anchor plate.

3 Tension load

	Load N_{ua} [kip]	Capacity ϕ N_n [kip]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	N/A	N/A	N/A	N/A
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	N/A	N/A	N/A	N/A
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (anchors in tension)

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	3
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP (W12x79 Elevator Pit)	Date:	1/28/2025
Fastening point:			

4 Shear load

	Load V_{ua} [kip]	Capacity ϕ V_n [kip]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	N/A	N/A	N/A	N/A
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength*	N/A	N/A	N/A	N/A
Concrete edge failure in direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (relevant anchors)

5 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Since no anchor rod forces exist, the anchor rod size can be determined based on the OSHA requirements, and practical considerations.
- The equations presented in this report are based on imperial units. When inputs are displayed in metric units, the user should be aware that the equations remain in their imperial format.
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- For additional information about ACI 318 strength design provisions, please go to https://submittals.us.hilti.com/PROFISAnchorDesignGuide/

Fastening meets the design criteria!

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Company:		Page:	4
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP (W12x79 Elevator Pit)	Date:	1/28/2025
Fastening point:			

6 Anchor plate and concrete bearing stress check

	Load	Capacity	Utilization [%]	Status
Concentric Compression	8,240.76 [in.lb/in.]	17,578.13 [in.lb/in.]	47	OK
Concrete bearing	952 [psi]	2,526 [psi]	38	OK
Tension Interface	N/A	N/A	N/A	N/A
Uniaxial Moment (Strong Axis)	N/A	N/A	N/A	N/A
Uniaxial Moment (Weak Axis)	N/A	N/A	N/A	N/A

6.1 Concrete bearing check (per AISC DG1, section 3.1.1)

$A_1 = B \cdot N$

$A_2 = L \cdot W$

$f_{pu(max)} = \phi \left(0.85 f'_c \sqrt{\frac{A_2}{A_1}} \right) , \sqrt{\frac{A_2}{A_1}} \leq 2$

$f_{pu} = \frac{P_u}{BN}$

$f_{pu} \leq f_{p(max)}$

Variables

B [in.]	N [in.]	L [in.]	W [in.]	f_c' [psi]	ϕ	P_u [kip]
14.000	18.000	20.571	16.000	4,000	0.650	240.000

Calculations

A_1 [in.^{2}]	A_2 [in.^{2}]
252.00	329.14

Results

f_{pu} [psi]	$f_{pu(max)}$ [psi]
952	2,526

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	5
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP (W12x79 Elevator Pit)	Date:	1/28/2025
Fastening point:			

6.2 Plate bending, compression (per AISC DG1, sections 3.1.2, 3.1.3)

$A_1 = B \cdot N$

$A_2 = L \cdot W$

$P_p = 0.85 f'_c A_1 \sqrt{\frac{A_2}{A_1}}, \sqrt{\frac{A_2}{A_1}} \leq 2$

$X = \left\{ \frac{4 d b_f}{(d + b_f)^2} \right\} \frac{P_u}{\phi_c P_p}$

$\lambda = \frac{2\sqrt{X}}{1 + \sqrt{1 - X}} \leq 1$

$m = \frac{B - 0.95d}{2}$

$n = \frac{N - 0.80 b_f}{2}$

$\lambda n' = \lambda \frac{\sqrt{d b_f}}{4}$

$l = \max(m, n, \lambda n')$

$f_{pu} = \frac{P_u}{BN}$

$M_{pl} = \frac{f_{pu} \cdot l^2}{2}$

$\phi M_n = \phi \cdot F_y \cdot \frac{t_p^2}{4}$

$M_{pl} \leq \phi M_n$

Variables

B [in.]	N [in.]	L [in.]	W [in.]	d [in.]
14.000	18.000	20.571	16.000	12.400

b_f [in.]	F_y [psi]	ϕ	t_p [in.]	P_u' [kip]
12.100	50,000	0.900	1.250	240.000

Calculations

A_1 [in.^{2}]	A_2 [in.^{2}]	P_p [kip]	X	λ
252.00	329.14	636.480	0.580	0.924

m [in.]	n [in.]	$\lambda n'$ [in.]	l [in.]	f_{pu} [psi]
1.110	4.160	2.830	4.160	952

Results

M_n [in.lb/in.]	ϕ	ϕM_n [in.lb/in.]	M_{pl} [in.lb/in.]
19,531.26	0.900	17,578.13	8,240.76

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Company:		Page:	6
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP (W12x79 Elevator Pit)	Date:	1/28/2025
Fastening point:			

7 Installation data

Profile: W shape (AISC), W12X79; (L x W x T x FT) = 12.400 in. x 12.100 in. x 0.470 in. x 0.735 in.

Hole diameter in the fixture: d_f = 0.812 in.

Plate thickness (input): 1.250 in.

Anchor type and diameter: Heavy Hex Head ASTM F 1554 GR. 36 3/4

Item number: not available

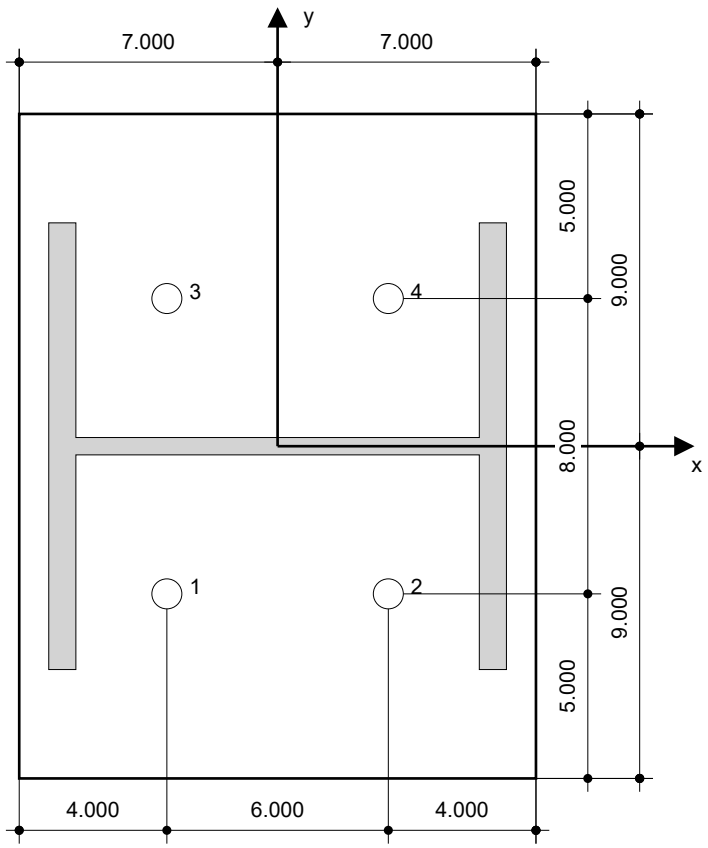
Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 18.000 in.

Minimum thickness of the base material: 19.000 in.

⌀ 3/4 in Heavy Hex Head ASTM F 1554 GR. 36 with 18 in nominal embedment depth per Technical data , cast in place installation per MPII



Coordinates Anchor in.

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-3.000	-4.000	21.000	11.000	20.000	28.000
2	3.000	-4.000	27.000	5.000	20.000	28.000
3	-3.000	4.000	21.000	11.000	28.000	20.000
4	3.000	4.000	27.000	5.000	28.000	20.000

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Company:		Page:	7
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP (W12x79 Elevator Pit)	Date:	1/28/2025
Fastening point:			

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Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP (W14x90 Elevator Pit)	Date:	1/28/2025
Fastening point:			

Specifier's comments:

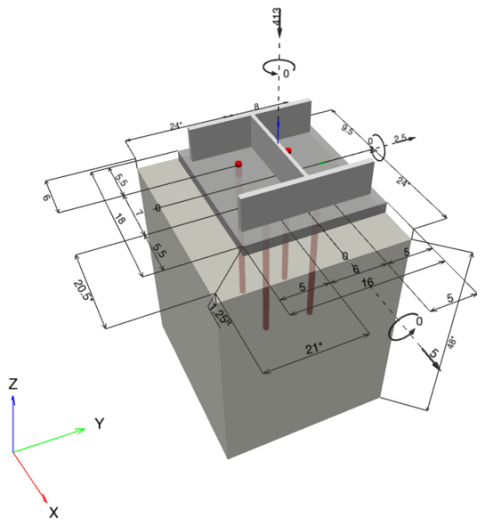
1 Input data



Anchor type and diameter:	**Heavy Hex Head ASTM F 1554 GR. 36 3/4**
Item number:	not available
Specification text:	∅ 3/4 in Heavy Hex Head ASTM F 1554 GR. 36 with 18 in nominal embedment depth per Technical data , cast in place installation per MPII
Effective embedment depth:	h_{ef} = 18.000 in.
Material:	ASTM F 1554
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-19 / CIP
Shear edge breakout verification:	Row closest to edge (Case 3 only from ACI 318-19 Fig. R.17.7.2.1b)
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 1.250 in.
Anchor plate[R] :	l_x x l_y x t = 18.000 in. x 16.000 in. x 1.250 in.;
Profile:	W shape (AISC), W14X90; (L x W x T x FT) = 14.000 in. x 14.500 in. x 0.440 in. x 0.710 in.
Base material:	cracked concrete, Custom, f_c' = 4,500 psi; h = 48.000 in.
Reinforcement:	tension: not present, shear: not present;
	edge reinforcement: none or < No. 4 bar

[R] - The anchor calculation is based on a rigid anchor plate assumption.

Geometry [in.] & Loading [kip, ft.kip]



Input data and results must be checked for conformity with the existing conditions and for plausibility!
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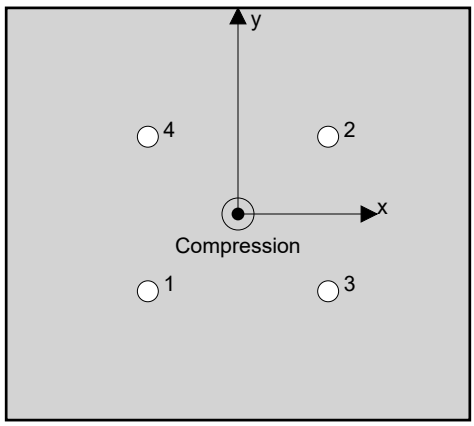
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Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP (W14x90 Elevator Pit)	Date:	1/28/2025
Fastening point:			

1.1 Design results

Case	Description	Forces [kip] / Moments [ft.kip]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = -413.000; V_x = 5.000; V_y = 2.500; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	93

2 Load case/Resulting anchor forces



Anchor reactions [kip]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	0.000	1.398	1.250	0.625
2	0.000	1.398	1.250	0.625
3	0.000	1.398	1.250	0.625
4	0.000	1.398	1.250	0.625

Max. concrete compressive strain:	0.00 [‰]
Max. concrete compressive stress:	1,434 [psi]
Resulting tension force in (x/y)=(-/-):	0.000 [kip]
Resulting compression force in (x/y)=(0.000/0.000):	413.000 [kip]

Anchor forces are calculated based on the assumption of a rigid anchor plate.

3 Tension load

	Load N_{ua} [kip]	Capacity ϕ N_n [kip]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	N/A	N/A	N/A	N/A
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	N/A	N/A	N/A	N/A
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (anchors in tension)

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Company:		Page:	3
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP (W14x90 Elevator Pit)	Date:	1/28/2025
Fastening point:			

4 Shear load

	Load V_{ua} [kip]	Capacity ϕ V_n [kip]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	1.398	7.555	19	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	5.590	57.287	10	OK
Concrete edge failure in direction y+**	5.590	6.070	93	OK

* highest loaded anchor **anchor group (relevant anchors)

4.1 Steel Strength

$V_{sa} = 0.6\ A_{se,V}\ f_{uta}$ ACI 318-19 Eq. (17.7.1.2b)
$\phi\ V_{steel} \geq V_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.33	58,000

Calculations

V_{sa} [kip]
11.623

Results

V_{sa} [kip]	ϕ_{steel}	$\phi\ V_{sa}$ [kip]	V_{ua} [kip]
11.623	0.650	7.555	1.398

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Company:		Page:	4
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP (W14x90 Elevator Pit)	Date:	1/28/2025
Fastening point:			

4.2 Pryout Strength

$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \right]$ — ACI 318-19 Eq. (17.7.3.1b)

$\phi V_{cpg} \geq V_{ua}$ — ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9 h_{ef}^2$ — ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e'_N}{3 h_{ef}}} \right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = 16 \lambda_a \sqrt{f'_c} h_{ef}^{5/3}$ — ACI 318-19 Eq. (17.6.2.2.3)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	14.000	0.000	0.000	5.000

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.000	-	16	1.000	4,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [kip]
1,072.00	1,764.00	1.000	1.000	0.771	1.000	87.285

Results

V_{cpg} [kip]	$\phi_{concrete}$	ϕV_{cpg} [kip]	V_{ua} [kip]
81.839	0.700	57.287	5.590

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Company:		Page:	5
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP (W14x90 Elevator Pit)	Date:	1/28/2025
Fastening point:			

4.3 Concrete edge failure in direction y+

$V_{cbg} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \Psi_{ec,V} \Psi_{ed,V} \Psi_{c,V} \Psi_{h,V} \Psi_{parallel,V} V_b$ — ACI 318-19 Eq. (17.7.2.1b)

$\phi V_{cbg} \geq V_{ua}$ — ACI 318-19 Table 17.5.2

A_{Vc} see ACI 318-19, Section 17.7.2.1, Fig. R 17.7.2.1(b)*

$A_{Vc0} = 4.5 c_{a1}^2$ — ACI 318-19 Eq. (17.7.2.1.3)

$\Psi_{ec,V} = \left(\frac{1}{1 + \frac{e'_v}{1.5c_{a1}}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.3.1)

$\Psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.4.1b)

$\Psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ — ACI 318-19 Eq. (17.7.2.6.1)

$V_b = 9 \lambda_a \sqrt{f'_c} c_{a1}^{1.5}$ — ACI 318-19 Eq. (17.7.2.2.1b)

Variables

c_{a1} [in.]	c_{a2} [in.]	e_{cV} [in.]	$\Psi_{c,V}$	h_a [in.]
5.000	6.000	0.000	1.000	48.000

l_e [in.]	λ_a	d_a [in.]	f'_c [psi]	$\Psi_{parallel,V}$
6.000	1.000	0.750	4,500	1.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{h,V}$	V_b [kip]
153.75	112.50	1.000	0.940	1.000	6.750

Results

V_{cbg} [kip]	$\phi_{concrete}$	ϕV_{cbg} [kip]	V_{ua} [kip]
8.671	0.700	6.070	5.590

*Anchor row defined by: Anchor 2, 4; Case 3 controls

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Company:		Page:	6
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP (W14x90 Elevator Pit)	Date:	1/28/2025
Fastening point:			

5 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- User is responsible for evaluating the hole bearing capacity in case of shear forces.
- The equations presented in this report are based on imperial units. When inputs are displayed in metric units, the user should be aware that the equations remain in their imperial format.
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- For additional information about ACI 318 strength design provisions, please go to https://submittals.us.hilti.com/PROFISAnchorDesignGuide/

Fastening meets the design criteria!

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Company:		Page:	7
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP (W14x90 Elevator Pit)	Date:	1/28/2025
Fastening point:			

6 Anchor plate and concrete bearing stress check

	Load	Capacity	Utilization [%]	Status
Concentric Compression	9,097.11 [in.lb/in.]	17,578.13 [in.lb/in.]	52	OK
Concrete bearing	1,434 [psi]	2,486 [psi]	58	OK
Tension Interface	N/A	N/A	N/A	N/A
Uniaxial Moment (Strong Axis)	N/A	N/A	N/A	N/A
Uniaxial Moment (Weak Axis)	N/A	N/A	N/A	N/A

6.1 Concrete bearing check (per AISC DG1, section 3.1.1)

$A_1 = B \cdot N$

$A_2 = L \cdot W$

$f_{pu(max)} = \phi \left(0.85 f'_c \sqrt{\frac{A_2}{A_1}} \right) , \sqrt{\frac{A_2}{A_1}} \leq 2$

$f_{pu} = \frac{P_u}{BN}$

$f_{pu} \leq f_{p(max)}$

Variables

B [in.]	N [in.]	L [in.]	W [in.]	f_c' [psi]	ϕ	P_u [kip]
16.000	18.000	18.000	16.000	4,500	0.650	413.000

Calculations

A_1 [in.^{2}]	A_2 [in.^{2}]
288.00	288.00

Results

f_{pu} [psi]	$f_{pu(max)}$ [psi]
1,434	2,486

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Company:		Page:	8
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP (W14x90 Elevator Pit)	Date:	1/28/2025
Fastening point:			

6.2 Plate bending, compression (per AISC DG1, sections 3.1.2, 3.1.3)

$A_1 = B \cdot N$

$A_2 = L \cdot W$

$P_p = 0.85 f'_c A_1 \sqrt{\frac{A_2}{A_1}}, \sqrt{\frac{A_2}{A_1}} \leq 2$

$X = \left\{ \frac{4 d b_f}{(d + b_f)^2} \right\} \frac{P_u}{\phi_c P_p}$

$\lambda = \frac{2\sqrt{X}}{1 + \sqrt{1 - X}} \leq 1$

$m = \frac{N - 0.95d}{2}$

$n = \frac{B - 0.80 b_f}{2}$

$\lambda n' = \lambda \frac{\sqrt{d b_f}}{4}$

$l = \max(m, n, \lambda n')$

$f_{pu} = \frac{P_u}{BN}$

$M_{pl} = \frac{f_{pu} \cdot l^2}{2}$

$\phi M_n = \phi \cdot F_y \cdot \frac{t_p^2}{4}$

$M_{pl} \leq \phi M_n$

Variables

B [in.]	N [in.]	L [in.]	W [in.]	d [in.]
16.000	18.000	18.000	16.000	14.000

b_f [in.]	F_y [psi]	ϕ	t_p [in.]	P_u' [kip]
14.500	50,000	0.900	1.250	413.000

Calculations

A_1 [in.^{2}]	A_2 [in.^{2}]	P_p [kip]	X	λ
288.00	288.00	716.040	0.887	1.000

m [in.]	n [in.]	$\lambda n'$ [in.]	l [in.]	f_{pu} [psi]
2.350	2.200	3.562	3.562	1,434

Results

M_n [in.lb/in.]	ϕ	ϕM_n [in.lb/in.]	M_{pl} [in.lb/in.]
19,531.26	0.900	17,578.13	9,097.11

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Company:		Page:	9
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP (W14x90 Elevator Pit)	Date:	1/28/2025
Fastening point:			

7 Installation data

Profile: W shape (AISC), W14X90; (L x W x T x FT) = 14.000 in. x 14.500 in. x 0.440 in. x 0.710 in.

Hole diameter in the fixture: d_f = 0.812 in.

Plate thickness (input): 1.250 in.

Anchor type and diameter: Heavy Hex Head ASTM F 1554 GR. 36 3/4

Item number: not available

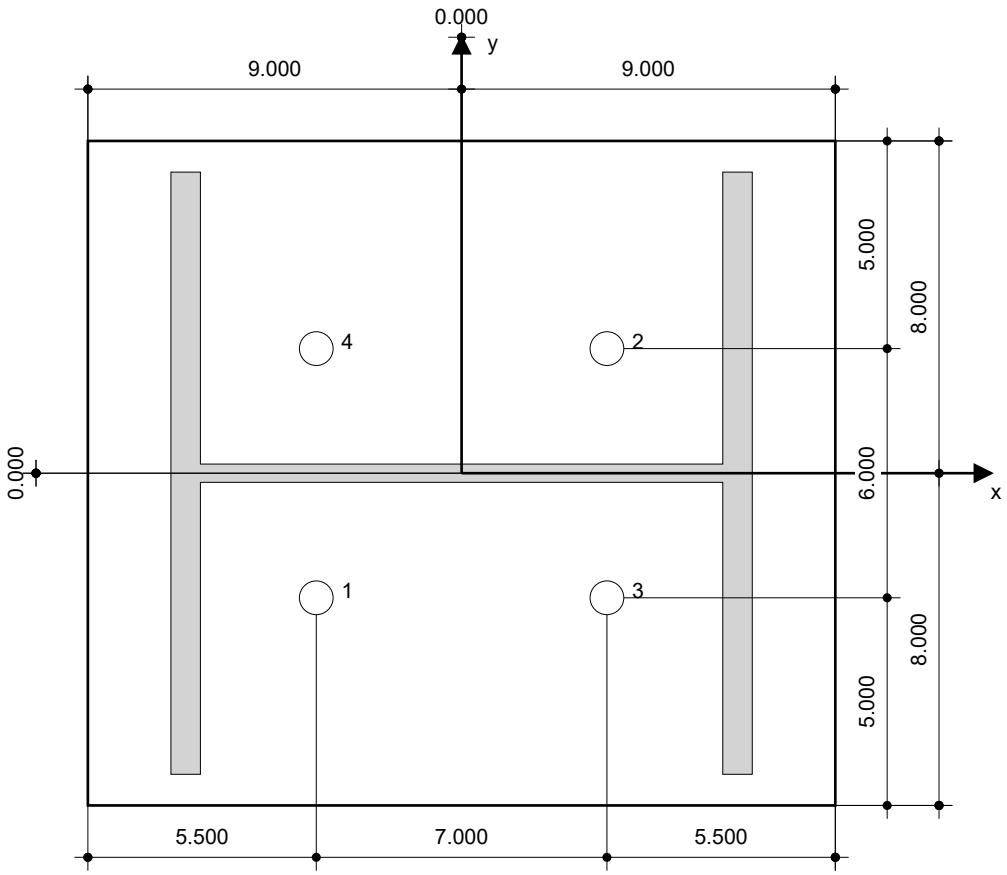
Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 18.000 in.

Minimum thickness of the base material: 19.000 in.

⌀ 3/4 in Heavy Hex Head ASTM F 1554 GR. 36 with 18 in nominal embedment depth per Technical data , cast in place installation per MPII



Coordinates Anchor in.

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-3.500	-3.000	6.000	27.500	21.000	11.000
2	3.500	3.000	13.000	20.500	27.000	5.000
3	3.500	-3.000	13.000	20.500	21.000	11.000
4	-3.500	3.000	6.000	27.500	27.000	5.000

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	10
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP (W14x90 Elevator Pit)	Date:	1/28/2025
Fastening point:			

8 Remarks; Your Cooperation Duties

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Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Green BP (Uplift Column)	Date:	4/24/2025
Fastening point:			

Specifier's comments:

1 Anchor Design

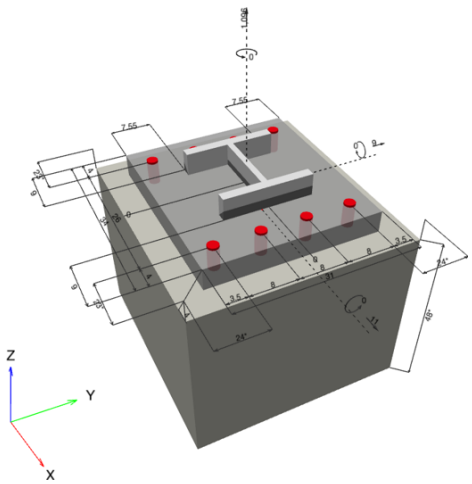
1.1 Input data



Anchor type and diameter:	**Heavy Hex Head ASTM F 1554 GR. 105 2**
Item number:	not available
Specification text:	∅ 2 in Heavy Hex Head ASTM F 1554 GR. 105 with 24 in nominal embedment depth per Technical data , cast in place installation per MPII,
Effective embedment depth:	h_{ef} = 24.000 in.
Material:	ASTM F 1554
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-19 / CIP
Shear edge breakout verification:	Row closest to edge (Case 3 only from ACI 318-19 Fig. R.17.7.2.1b)
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 4.000 in.
Anchor plate[CBFEM] :	l_x x l_y x t = 34.000 in. x 31.000 in. x 4.000 in.;
Profile:	W shape (AISC), W14X233; (L x W x T x FT) = 16.000 in. x 15.900 in. x 1.070 in. x 1.720 in.
Base material:	cracked concrete, 4000, f_c' = 4,000 psi; h = 48.000 in.
Reinforcement:	tension: not present, shear: not present; edge reinforcement: none or < No. 4 bar

[CBFEM] - The anchor calculation is based on a component-based Finite Element Method (CBFEM)

Geometry [in.] & Loading [kip, ft.kip]



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Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Green BP (Uplift Column)	Date:	4/24/2025
Fastening point:			

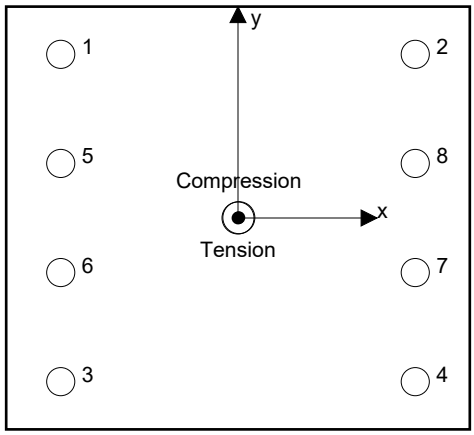
1.1.1 Design results

Case	Description	Forces [kip] / Moments [ft.kip]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 1,096.000; V_x = 11.000; V_y = 9.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	713

1.2 Load case/Resulting anchor forces

Anchor reactions [kip]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	108.148	1.958	1.906	0.450
2	108.336	0.943	0.812	0.479
3	108.195	2.643	1.953	1.781
4	108.345	1.910	0.777	1.745
5	176.742	2.753	2.649	0.746
6	176.642	3.065	2.653	1.535
7	176.498	1.533	0.148	1.526
8	176.546	0.745	0.103	0.738



Resulting tension force in (x/y)=(-0.000/-0.000): 1,139.452 [kip]
Resulting compression force in (x/y)=(0.063/0.025): 48.584 [kip]

Anchor forces are calculated based on a component-based Finite Element Method (CBFEM)

1.3 Tension load

	Load N_{ua} [kip]	Capacity ϕ N_n [kip]	Utilization $\beta_N = N_{ua}/\phi N_n$	Status
Steel Strength*	176.742	234.375	76	OK
Pullout Strength*	176.742	119.078	149 28.1	not recommended
Concrete Breakout Failure**	1,139.452	159.894	713 47.6	not recommended
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (anchors in tension)

SEE SUPPLEMENTAL CALCULATION FOR BP-11

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Company:		Page:	3
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Green BP (Uplift Column)	Date:	4/24/2025
Fastening point:			

1.3.1 Steel Strength

$N_{sa} = A_{se,N} f_{uta}$ ACI 318-19 Eq. (17.6.1.2)
$\phi N_{sa} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
2.50	125,001

Calculations

N_{sa} [kip]
312.500

Results

N_{sa} [kip]	ϕ_{steel}	ϕN_{sa} [kip]	N_{ua} [kip]
312.500	0.750	234.375	176.742

1.3.2 Pullout Strength

$N_{pN} = \psi_{c,p} N_p$ ACI 318-19 Eq. (17.6.3.1)
$N_p = 8 A_{brg} f_c'$ ACI 318-19 Eq. (17.6.3.2.2a)
$\phi N_{pN} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$\psi_{c,p}$	A_{brg} [in.^{2}]	λ_a	f_c' [psi]
1.000	5.32	1.000	4,000

Calculations

N_p [kip]
170.112

Results

N_{pn} [kip]	$\phi_{concrete}$	ϕN_{pn} [kip]	N_{ua} [kip]
170.112	0.700	119.078	176.742

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Company:		Page:	4
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Green BP (Uplift Column)	Date:	4/24/2025
Fastening point:			

1.3.3 Concrete Breakout Failure

$N_{cbg} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ec,N}\ \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b$ — ACI 318-19 Eq. (17.6.2.1b)

$\phi\ N_{cbg} \geq N_{ua}$ — ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9\ h_{ef}^2$ — ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\ e'_N}{3\ h_{ef}}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3\left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = 16\ \lambda_a\ \sqrt{f'_c}\ h_{ef}^{5/3}$ — ACI 318-19 Eq. (17.6.2.2.3)

Variables

h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
16.000	0.000	0.000	23.000	1.000

c_{ac} [in.]	k_c	λ_a	f'_c [psi]
-	16	1.000	4,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [kip]
5,184.00	2,304.00	1.000	1.000	0.988	1.000	102.806

Results

N_{cbg} [kip]	$\phi_{concrete}$	$\phi\ N_{cbg}$ [kip]	N_{ua} [kip]
228.420	0.700	159.894	1,139.452

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Company:		Page:	5
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Green BP (Uplift Column)	Date:	4/24/2025
Fastening point:			

1.4 Shear load

	Load V_{ua} [kip]	Capacity ϕ V_n [kip]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	3.065	121.875	3	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	14.213	319.789	5	OK
Concrete edge failure in direction x+**	14.213	41.646	35	OK

* highest loaded anchor **anchor group (relevant anchors)

1.4.1 Steel Strength

$V_{sa} = 0.6\ A_{se,V}\ f_{uta}$ ACI 318-19 Eq. (17.7.1.2b)
$\phi\ V_{steel} \geq V_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
2.50	125,001

Calculations

V_{sa} [kip]
187.500

Results

V_{sa} [kip]	ϕ_{steel}	$\phi\ V_{sa}$ [kip]	V_{ua} [kip]
187.500	0.650	121.875	3.065

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Company:		Page:	6
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Green BP (Uplift Column)	Date:	4/24/2025
Fastening point:			

1.4.2 Pryout Strength

$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \right]$ — ACI 318-19 Eq. (17.7.3.1b)

$\phi V_{cpg} \geq V_{ua}$ — ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9 h_{ef}^2$ — ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e'_N}{3 h_{ef}}} \right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = 16 \lambda_a \sqrt{f'_c} h_{ef}^{5/3}$ — ACI 318-19 Eq. (17.6.2.2.3)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	16.000	0.000	0.000	23.000

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.000	-	16	1.000	4,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [kip]
5,184.00	2,304.00	1.000	1.000	0.988	1.000	102.806

Results

V_{cpg} [kip]	$\phi_{concrete}$	ϕV_{cpg} [kip]	V_{ua} [kip]
456.842	0.700	319.789	14.213

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Company:		Page:	7
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Green BP (Uplift Column)	Date:	4/24/2025
Fastening point:			

1.4.3 Concrete edge failure in direction x+

$V_{cbg} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \Psi_{ec,V} \Psi_{ed,V} \Psi_{c,V} \Psi_{h,V} \Psi_{parallel,V} V_b$ — ACI 318-19 Eq. (17.7.2.1b)

$\phi V_{cbg} \geq V_{ua}$ — ACI 318-19 Table 17.5.2

A_{Vc} see ACI 318-19, Section 17.7.2.1, Fig. R 17.7.2.1(b)*

$A_{Vc0} = 4.5 c_{a1}^2$ — ACI 318-19 Eq. (17.7.2.1.3)

$\Psi_{ec,V} = \left(\frac{1}{1 + \frac{e'_v}{1.5c_{a1}}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.3.1)

$\Psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.4.1b)

$\Psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ — ACI 318-19 Eq. (17.7.2.6.1)

$V_b = 9 \lambda_a \sqrt{f'_c} c_{a1}^{1.5}$ — ACI 318-19 Eq. (17.7.2.2.1b)

Variables

c_{a1} [in.]	c_{a2} [in.]	e_{cV} [in.]	$\Psi_{c,V}$	h_a [in.]
23.000	24.000	0.023	1.000	48.000

l_e [in.]	λ_a	d_a [in.]	f'_c [psi]	$\Psi_{parallel,V}$
16.000	1.000	2.000	4,000	1.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{h,V}$	V_b [kip]
2,484.00	2,380.50	0.999	0.909	1.000	62.786

Results

V_{cbg} [kip]	$\phi_{concrete}$	ϕV_{cbg} [kip]	V_{ua} [kip]
59.494	0.700	41.646	14.213

*Anchor row defined by: Anchor 2, 4, 7, 8; Case 3 controls

1.5 Combined tension and shear loads, per ACI 318-19 section 17.8

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
~~7.126~~ 0.76	0.341	1.000	~~623~~ 91.8	not recommended

$\beta_{NV} = (\beta_N + \beta_V) / 1.2 <= 1$

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Company:		Page:	8
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Green BP (Uplift Column)	Date:	4/24/2025
Fastening point:			

1.6 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates as per current regulations (ETAG 001/Annex C, EOTA TR029, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid base plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- The equations presented in this report are based on imperial units. When inputs are displayed in metric units, the user should be aware that the equations remain in their imperial format.
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- For additional information about ACI 318 strength design provisions, please go to https://viewer.joomag.com/profis-design-guide-us-en-summer-2021/0841849001625154758?short&/
- The anchor design methods in PROFIS Engineering require rigid anchor plates, as per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means that the anchor plate should be sufficiently rigid to prevent load re-distribution to the anchors due to elastic/plastic displacements. The user accepts that the anchor plate is considered close to rigid by engineering judgment."

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Company:		Page:	9
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Green BP (Uplift Column)	Date:	4/24/2025
Fastening point:			

1.7 Installation data

Profile: W shape (AISC), W14X233; (L x W x T x FT) = 16.000 in. x 15.900 in. x 1.070 in. x 1.720 in.

Hole diameter in the fixture: d_f = 2.062 in.

Plate thickness (input): 4.000 in.

Anchor type and diameter: Heavy Hex Head ASTM F 1554 GR. 105 2

Item number: not available

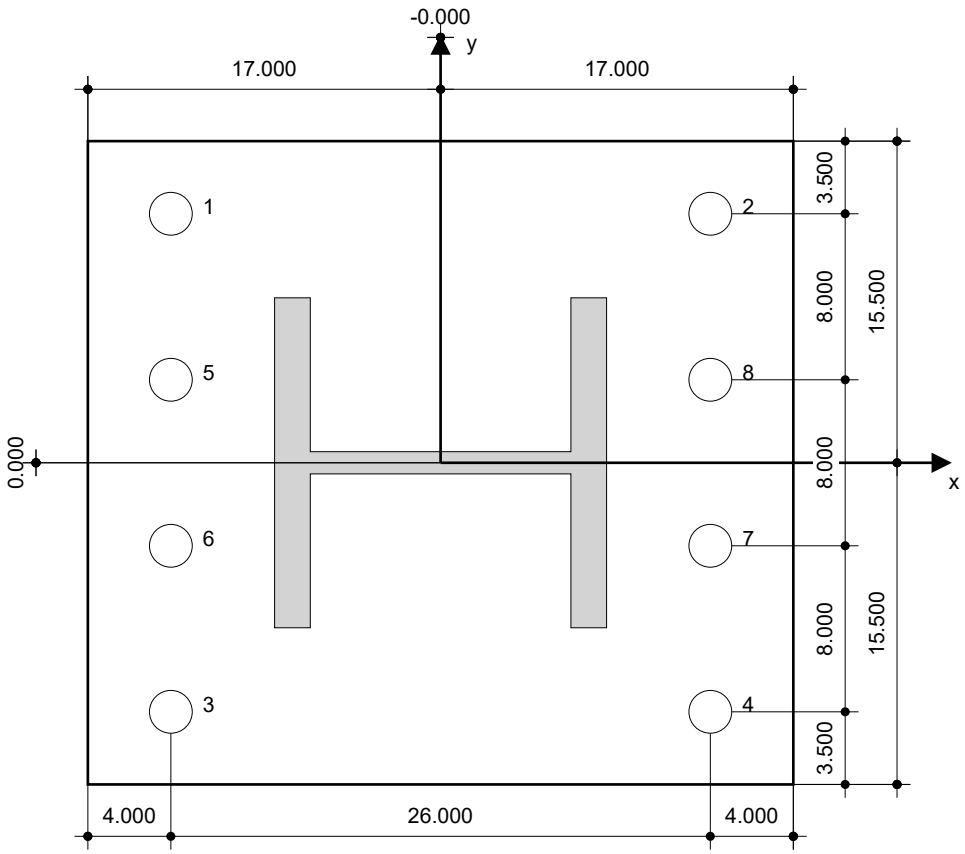
Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 24.000 in.

Minimum thickness of the base material: 25.844 in.

⌀ 2 in Heavy Hex Head ASTM F 1554 GR. 105 with 24 in nominal embedment depth per Technical data , cast in place installation per MPII



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-13.000	12.000	23.000	49.000	48.000	24.000
2	13.000	12.000	49.000	23.000	48.000	24.000
3	-13.000	-12.000	23.000	49.000	24.000	48.000
4	13.000	-12.000	49.000	23.000	24.000	48.000

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
5	-13.000	4.000	23.000	49.000	40.000	32.000
6	-13.000	-4.000	23.000	49.000	32.000	40.000
7	13.000	-4.000	49.000	23.000	32.000	40.000
8	13.000	4.000	49.000	23.000	40.000	32.000

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Company:		Page:	10
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Green BP (Uplift Column)	Date:	4/24/2025
Fastening point:			

2 Anchor plate design

2.1 Input data

Anchor plate:	Shape: Rectangular l_x x l_y x t = 34.000 in x 31.000 in x 4.000 in Calculation: CBFEM Material: ASTM A572 Gr.50; F_y = 50,000 psi; ε_{lim} = 5.00%
Anchor type and size:	Heavy Hex Head ASTM F 1554 GR. 105 2, h_{ef} = 24.000 in
Anchor stiffness:	The anchor is modeled considering stiffness values determined from load displacement curves tested in an independent laboratory. Please note that no simple replacement of the anchor is possible as the anchor stiffness has a major impact on the load distribution results.
Design method:	AISC and LRFD-based design using component-based FEM
Stand-off installation:	e_b = 0.000 in (No stand-off); t = 4.000 in
Profile:	W14X233; (L x W x T x FT) = 16.000 in x 15.900 in x 1.070 in x 1.720 in Material: ASTM A992; F_y = 50,000 psi; ε_{lim} = 5.00% Eccentricity x: -0.000 in Eccentricity y: -0.000 in
Base material:	Cracked concrete; 4000; $f_{c,cyl}$ = 4,000 psi; h = 48.000 in
Welds (profile to anchor plate):	Type of redistribution: Plastic Material: E70xx
Mesh size:	Number of elements on edge: 8 Min. size of element: 0.394 in Max. size of element: 1.969 in

2.2 Summary

	Description	Profile		Anchor plate			Concrete [%]
		σ_{Ed} [psi]	ε_{Pl} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Hole bearing [%]	
1	Combination 1	49,506	0.02	30,156	0.00	1	9

2.3 Anchor plate classification

Results below are displayed for the decisive load combinations: Combination 1

Anchor tension forces	Equivalent rigid anchor plate (CBFEM)	Component-based Finite Element Method (CBFEM) anchor plate design
Anchor 1	136.993 kip	108.148 kip
Anchor 2	136.997 kip	108.336 kip
Anchor 3	136.986 kip	108.195 kip
Anchor 4	136.991 kip	108.345 kip
Anchor 5	136.993 kip	176.742 kip
Anchor 6	136.991 kip	176.642 kip
Anchor 7	136.995 kip	176.498 kip
Anchor 8	136.997 kip	176.546 kip

User accepted to consider the selected anchor plate as rigid by his/her engineering judgement. This means the anchor design guidelines can be applied.

2.4 Profile/Stiffeners/Plate

Profile and stiffeners are verified at the level of the steel to concrete connection. The connection design does not replace the steel design for critical cross sections, which should be performed outside of PROFIS Engineering.

2.4.1 Equivalent stress and plastic strain

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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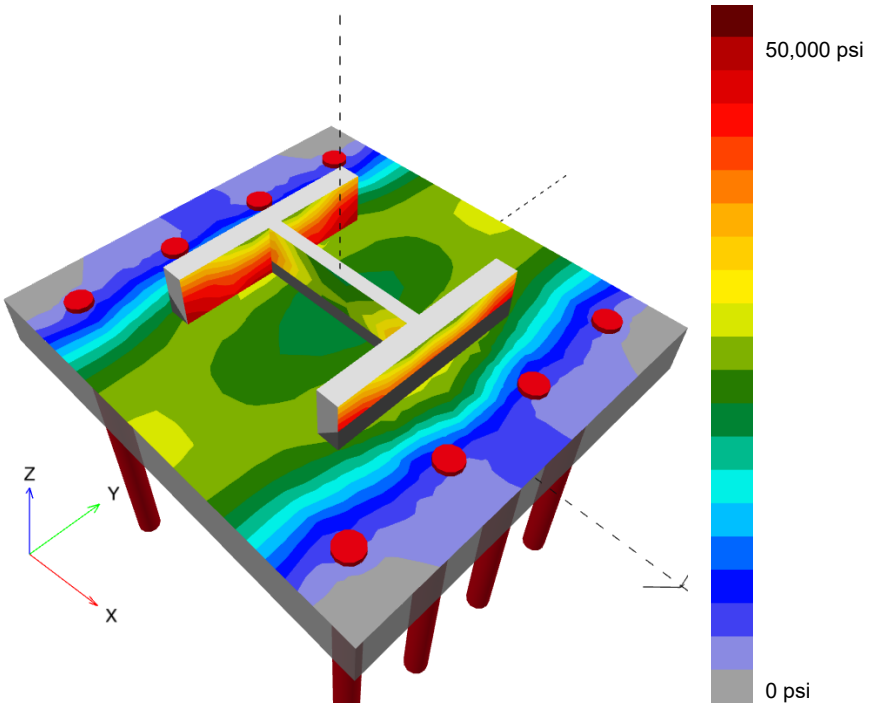
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Company:		Page:	11
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Green BP (Uplift Column)	Date:	4/24/2025
Fastening point:			

Part	Load combination	Material	f_y [psi]	ε_{lim} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Status
Plate	Combination 1	ASTM A572 Gr.50	50,000	5.00	30,156	0.00	OK
Profile	Combination 1	ASTM A992	50,000	5.00	49,212	0.02	OK
Profile	Combination 1	ASTM A992	50,000	5.00	49,506	0.02	OK
Profile	Combination 1	ASTM A992	50,000	5.00	46,284	0.00	OK

2.4.1.1 Equivalent stress

Results below are displayed for the decisive load combination: 1 - Combination 1

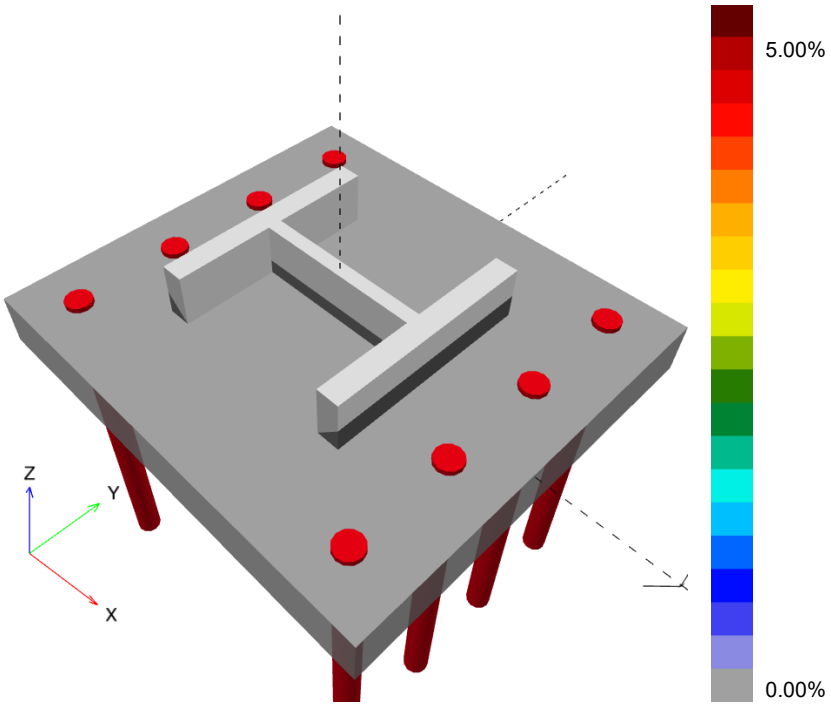


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Company:		Page:	12
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Green BP (Uplift Column)	Date:	4/24/2025
Fastening point:			

2.4.1.2 Plastic strain

Results below are displayed for the decisive load combination: 1 - Combination 1



2.4.2 Plate hole bearing resistance, AISC 360-16 Section J3

Decisive load combination: 1 - Combination 1

Equations

$R_n = \min(1.2\, l_c\, t\, F_u\, ,\ 2.4\, d\, t\, F_u)$ (AISC 360-16 J3-6a, c)

$\Phi R_n = 0.75\, R_n$

$V \leq \Phi R_n$

Variables

	l_c [in]	t [in]	F_u [psi]	d [in]	R_n [kip]
Anchor 1	3.079	4.000	65,000	2.000	960.574
Anchor 2	28.466	4.000	65,000	2.000	1,247.999
Anchor 3	4.163	4.000	65,000	2.000	1,247.999
Anchor 4	2.800	4.000	65,000	2.000	873.543
Anchor 5	3.124	4.000	65,000	2.000	974.840
Anchor 6	3.590	4.000	65,000	2.000	1,120.212
Anchor 7	5.938	4.000	65,000	2.000	1,247.999
Anchor 8	5.938	4.000	65,000	2.000	1,247.999

Results

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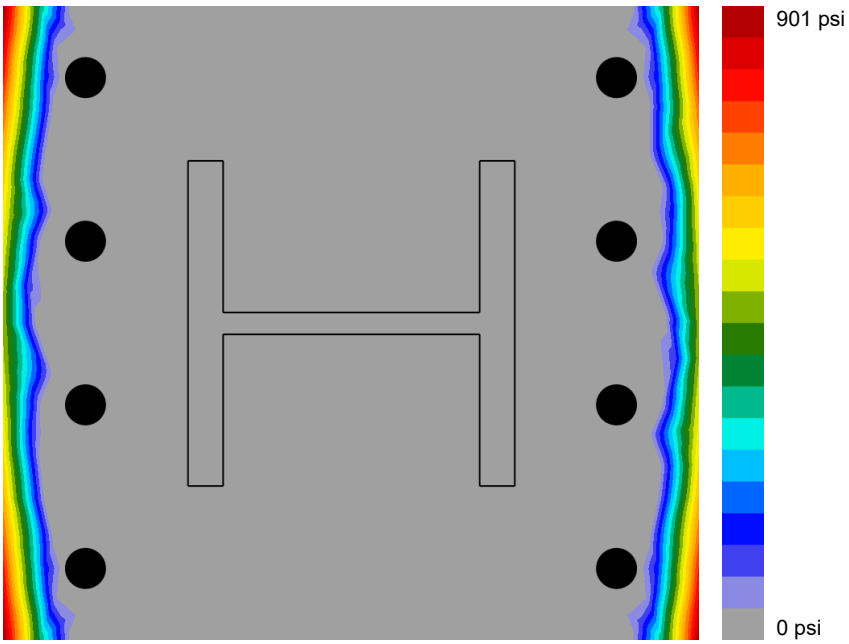
Company:		Page:	13
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Green BP (Uplift Column)	Date:	4/24/2025
Fastening point:			

	V [kip]	ΦR_n [kip]	Utilization [%]	Status
Anchor 1	1.958	720.431	1	OK
Anchor 2	0.943	935.999	1	OK
Anchor 3	2.643	935.999	1	OK
Anchor 4	1.910	655.157	1	OK
Anchor 5	2.753	731.130	1	OK
Anchor 6	3.065	840.159	1	OK
Anchor 7	1.533	935.999	1	OK
Anchor 8	0.745	935.999	1	OK

2.5 Concrete

Decisive load combination: 1 - Combination 1

2.5.1 Compression in concrete under the anchor plate



2.5.2 Concrete block compressive strength resistance check, AISC 360-16 Section J8

Equations

$F_p = \Phi f_{p,max}$

$f_{p,max} = 0.85 f_c' \sqrt{\left(\frac{A_2}{A_1}\right)} \le 1.7 f_c; \quad \sqrt{\left(\frac{A_2}{A_1}\right)} \le 2$

$\sigma = \frac{N}{A_1}$

$\text{Utilization} = \frac{\sigma}{F_p}$

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Company:		Page:	14
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Green BP (Uplift Column)	Date:	4/24/2025
Fastening point:			

Variables

N [kip]	f_c' [psi]	Φ	A_1 [in²]	A_2 [in²]
48.584	4,000	0.65	125.10	4,648.95

Results

Load combination	F_p [psi]	σ [psi]	Utilization [%]	Status
Combination 1	4,420	388	9	OK

2.6 Symbol explanation

A_1	Loaded area of concrete
A_2	Supporting area
d	Nominal diameter of the bolt
ε_{lim}	Limit plastic strain
ε_{Pl}	Plastic strain from CBFEM results
f_c	Concrete compressive strength
f_c'	Concrete compressive strength
F_u	Specified minimum tensile strength of the connected material
F_p	Concrete block design bearing strength
$f_{p,max}$	Concrete block design bearing strength maximum
f_y	Yield strength
l_c	Clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material
N	Resulting compression force
σ	Average stress in concrete
σ_{Ed}	Equivalent stress
Φ	Resistance factor
ΦR_n	Factored resistance
t	Thickness of the anchor plate
V	Resultant of shear forces Vy, Vz in bolt.

2.7 Warnings

- By using the CBFEM calculation functionality of PROFIS Engineering you may act outside the applicable design codes and your specified anchor plate may not behave rigid. Please, validate the results with a professional designer and/or structural engineer to ensure suitability and adequacy for your specific jurisdiction and project requirements.
- The anchor is modeled considering stiffness values determined from load displacement curves tested in an independent laboratory. Please note that no simple replacement of the anchor is possible as the anchor stiffness has a major impact on the load distribution results.

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Company:		Page:	15
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Green BP (Uplift Column)	Date:	4/24/2025
Fastening point:			

3 Summary of results

Design of the anchor plate, anchors, welds and other elements are based on CBFEM (component based finite element method) and AISC.

	Load combination	Max. utilization	Status
Anchors	Combination 1	713%	NOT OK
Anchor plate	Combination 1	61%	OK
Concrete	Combination 1	9%	OK
Profile	Combination 1	100%	OK

Fastening does not meet the design criteria!

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Company:		Page:	16
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Green BP (Uplift Column)	Date:	4/24/2025
Fastening point:			

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Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	HSS COLUMNS BP	Date:	1/29/2025
Fastening point:			

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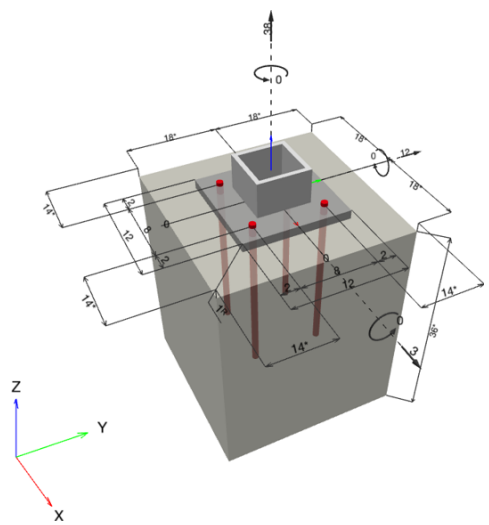
1 Input data



Anchor type and diameter:	**Heavy Hex Head ASTM F 1554 GR. 36 3/4**
Item number:	not available
Specification text:	∅ 3/4 in Heavy Hex Head ASTM F 1554 GR. 36 with 18 in nominal embedment depth per Technical data , cast in place installation per MPII
Effective embedment depth:	h_{ef} = 18.000 in.
Material:	ASTM F 1554
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-19 / CIP
Shear edge breakout verification:	Row closest to edge (Case 3 only from ACI 318-19 Fig. R.17.7.2.1b)
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 1.000 in.
Anchor plate[R] :	l_x x l_y x t = 12.000 in. x 12.000 in. x 1.000 in.;
Profile:	Square HSS (AISC), HSS6X6X.500; (L x W x T) = 6.000 in. x 6.000 in. x 0.500 in.
Base material:	cracked concrete, Custom, f_c' = 4,500 psi; h = 36.000 in.
Reinforcement:	tension: not present, shear: not present;
	edge reinforcement: none or < No. 4 bar

[R] - The anchor calculation is based on a rigid anchor plate assumption.

Geometry [in.] & Loading [kip, ft.kip]



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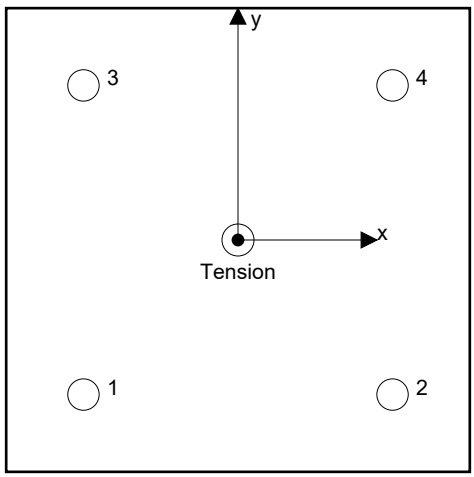
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Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	HSS COLUMNS BP	Date:	1/29/2025
Fastening point:			

1.1 Design results

Case	Description	Forces [kip] / Moments [ft.kip]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 38.000; V_x = 3.000; V_y = 12.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	116

2 Load case/Resulting anchor forces



Anchor reactions [kip]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	9.500	3.092	0.750	3.000
2	9.500	3.092	0.750	3.000
3	9.500	3.092	0.750	3.000
4	9.500	3.092	0.750	3.000

Max. concrete compressive strain: - [‰]
Max. concrete compressive stress: - [psi]
Resulting tension force in (x/y)=(0.000/-0.000): 38.000 [kip]
Resulting compression force in (x/y)=(-/-): 0.000 [kip]

Anchor forces are calculated based on the assumption of a rigid anchor plate.

3 Tension load

	Load N_{ua} [kip]	Capacity ϕ N_n [kip]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	9.500	14.529	66	OK
Pullout Strength*	9.500	22.957	42	OK
Concrete Breakout Failure**	38.000	53.120	72	OK
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (anchors in tension)

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Company:		Page:	3
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	HSS COLUMNS BP	Date:	1/29/2025
Fastening point:			

3.1 Steel Strength

$N_{sa} = A_{se,N} f_{uta}$ ACI 318-19 Eq. (17.6.1.2)
$\phi N_{sa} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.33	58,000

Calculations

N_{sa} [kip]
19.372

Results

N_{sa} [kip]	ϕ_{steel}	ϕN_{sa} [kip]	N_{ua} [kip]
19.372	0.750	14.529	9.500

3.2 Pullout Strength

$N_{pN} = \psi_{c,p} N_p$ ACI 318-19 Eq. (17.6.3.1)
$N_p = 8 A_{brg} f'_c$ ACI 318-19 Eq. (17.6.3.2.2a)
$\phi N_{pN} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$\psi_{c,p}$	A_{brg} [in.^{2}]	λ_a	f'_c [psi]
1.000	0.91	1.000	4,500

Calculations

N_p [kip]
32.796

Results

N_{pn} [kip]	$\phi_{concrete}$	ϕN_{pn} [kip]	N_{ua} [kip]
32.796	0.700	22.957	9.500

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Company:		Page:	4
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	HSS COLUMNS BP	Date:	1/29/2025
Fastening point:			

3.3 Concrete Breakout Failure

$N_{cbg} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ec,N}\ \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b$ — ACI 318-19 Eq. (17.6.2.1b)

$\phi\ N_{cbg} \geq N_{ua}$ — ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9\ h_{ef}^2$ — ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\ e'_N}{3\ h_{ef}}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3\left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c\ \lambda_a\ \sqrt{f'_c}\ h_{ef}^{1.5}$ — ACI 318-19 Eq. (17.6.2.2.1)

Variables

h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
9.333	0.000	0.000	14.000	1.000

c_{ac} [in.]	k_c	λ_a	f'_c [psi]
-	24	1.000	4,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [kip]
1,296.00	784.00	1.000	1.000	1.000	1.000	45.906

Results

N_{cbg} [kip]	$\phi_{concrete}$	$\phi\ N_{cbg}$ [kip]	N_{ua} [kip]
75.886	0.700	53.120	38.000

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Company:		Page:	5
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	HSS COLUMNS BP	Date:	1/29/2025
Fastening point:			

4 Shear load

	Load V_{ua} [kip]	Capacity ϕ V_n [kip]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	3.092	7.555	41	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	12.369	106.240	12	OK
Concrete edge failure in direction y+**	12.369	17.078	73	OK

* highest loaded anchor **anchor group (relevant anchors)

4.1 Steel Strength

$V_{sa} = 0.6\ A_{se,V}\ f_{uta}$ ACI 318-19 Eq. (17.7.1.2b)
$\phi\ V_{steel} \geq V_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.33	58,000

Calculations

V_{sa} [kip]
11.623

Results

V_{sa} [kip]	ϕ_{steel}	ϕ V_{sa} [kip]	V_{ua} [kip]
11.623	0.650	7.555	3.092

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Company:		Page:	6
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	HSS COLUMNS BP	Date:	1/29/2025
Fastening point:			

4.2 Pryout Strength

$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \right]$ ACI 318-19 Eq. (17.7.3.1b)

$\phi V_{cpg} \geq V_{ua}$ ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9 h_{ef}^2$ ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e'_N}{3 h_{ef}}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ ACI 318-19 Eq. (17.6.2.2.1)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	9.333	0.000	0.000	14.000

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.000	-	24	1.000	4,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [kip]
1,296.00	784.00	1.000	1.000	1.000	1.000	45.906

Results

V_{cpg} [kip]	$\phi_{concrete}$	ϕV_{cpg} [kip]	V_{ua} [kip]
151.772	0.700	106.240	12.369

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Company:		Page:	7
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	HSS COLUMNS BP	Date:	1/29/2025
Fastening point:			

4.3 Concrete edge failure in direction y+

$V_{cbg} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \Psi_{ec,V} \Psi_{ed,V} \Psi_{c,V} \Psi_{h,V} \Psi_{parallel,V} V_b$ — ACI 318-19 Eq. (17.7.2.1b)

$\phi V_{cbg} \geq V_{ua}$ — ACI 318-19 Table 17.5.2

A_{Vc} see ACI 318-19, Section 17.7.2.1, Fig. R 17.7.2.1(b)*

$A_{Vc0} = 4.5 c_{a1}^2$ — ACI 318-19 Eq. (17.7.2.1.3)

$\Psi_{ec,V} = \left(\frac{1}{1 + \frac{e'_v}{1.5c_{a1}}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.3.1)

$\Psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.4.1b)

$\Psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ — ACI 318-19 Eq. (17.7.2.6.1)

$V_b = 9 \lambda_a \sqrt{f'_c} c_{a1}^{1.5}$ — ACI 318-19 Eq. (17.7.2.2.1b)

Variables

c_{a1} [in.]	c_{a2} [in.]	e_{cV} [in.]	$\Psi_{c,V}$	h_a [in.]
14.000	14.000	0.000	1.000	36.000

l_e [in.]	λ_a	d_a [in.]	f'_c [psi]	$\Psi_{parallel,V}$
6.000	1.000	0.750	4,500	1.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{h,V}$	V_b [kip]
756.00	882.00	1.000	0.900	1.000	31.626

Results

V_{cbg} [kip]	$\phi_{concrete}$	ϕV_{cbg} [kip]	V_{ua} [kip]
24.397	0.700	17.078	12.369

*Anchor row defined by: Anchor 3, 4; Case 3 controls

5 Combined tension and shear loads, per ACI 318-19 section 17.8

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.715	0.724	5/3	116	not recommended

$\beta_{NV} = \beta_N^{\zeta} + \beta_V^{\zeta} <= 1$

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	8
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	HSS COLUMNS BP	Date:	1/29/2025
Fastening point:			

6 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- User is responsible for evaluating the hole bearing capacity in case of shear forces.
- The equations presented in this report are based on imperial units. When inputs are displayed in metric units, the user should be aware that the equations remain in their imperial format.
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- For additional information about ACI 318 strength design provisions, please go to https://submittals.us.hilti.com/PROFISAnchorDesignGuide/

Fastening does not meet the design criteria!

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Company:		Page:	9
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	HSS COLUMNS BP	Date:	1/29/2025
Fastening point:			

7 Anchor plate and concrete bearing stress check

	Load	Capacity	Utilization [%]	Status
Concentric Compression	N/A	N/A	N/A	N/A
Concrete bearing	N/A	N/A	N/A	N/A
Tension Interface	4,750.00 [in.lb/in.]	11,250.00 [in.lb/in.]	43	OK
Uniaxial Moment (Strong Axis)	N/A	N/A	N/A	N/A
Uniaxial Moment (Weak Axis)	N/A	N/A	N/A	N/A

7.1 Plate bending, tension (per AISC DG1, sections 3.2, 3.3)

$$m = \frac{N-0.95d}{2}$$

$$n = \frac{B-0.95b_f}{2}$$

$$M_{pl} = \frac{T_u \cdot x}{b_{eff}}$$

$$\phi M_n = \phi \cdot F_y \cdot \frac{t_p^2}{4}$$

$$M_{pl} \leq \phi M_n$$

Variables

B [in.]	N [in.]	d [in.]	b_f [in.]	F_y [psi]
12.000	12.000	6.000	6.000	50,000

ϕ	t_p [in.]	P_u [kip]	M_u [ft.kip]
0.900	1.000	38.000	0.00000

Calculations

m [in.]	n [in.]
3.150	3.150

T_u [kip]	x [in.]	b_{eff} [in.]
9.500	1.150	2.300

Results

M_n [in.lb/in.]	ϕ	ϕM_n [in.lb/in.]	M_{pl} [in.lb/in.]
12,500.00	0.900	11,250.00	4,750.00

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Company:		Page:	10
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	HSS COLUMNS BP	Date:	1/29/2025
Fastening point:			

8 Installation data

Profile: Square HSS (AISC), HSS6X6X.500; (L x W x T) = 6.000 in. x 6.000 in. x 0.500 in.

Hole diameter in the fixture: d_f = 0.812 in.

Plate thickness (input): 1.000 in.

Anchor type and diameter: Heavy Hex Head ASTM F 1554 GR. 36 3/4

Item number: not available

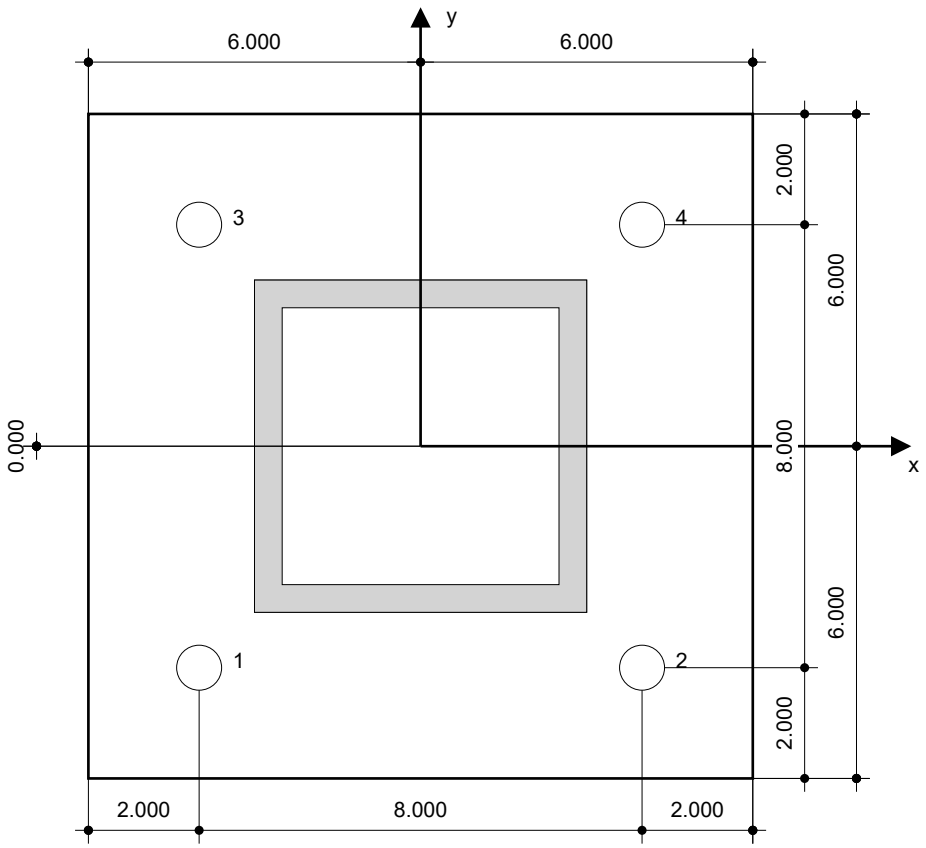
Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 18.000 in.

Minimum thickness of the base material: 19.000 in.

⌀ 3/4 in Heavy Hex Head ASTM F 1554 GR. 36 with 18 in nominal embedment depth per Technical data , cast in place installation per MPII



Coordinates Anchor in.

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-4.000	-4.000	14.000	22.000	14.000	22.000
2	4.000	-4.000	22.000	14.000	14.000	22.000
3	-4.000	4.000	14.000	22.000	22.000	14.000
4	4.000	4.000	22.000	14.000	22.000	14.000

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Company:		Page:	11
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	HSS COLUMNS BP	Date:	1/29/2025
Fastening point:			

9 Remarks; Your Cooperation Duties

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Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	HSS COLUMNS BP	Date:	1/29/2025
Fastening point:			

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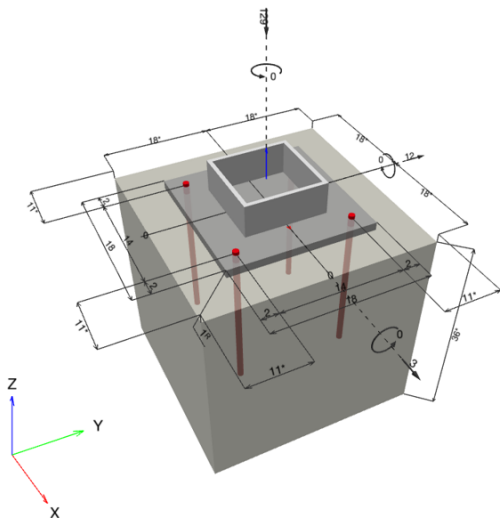
1 Input data



Anchor type and diameter:	**Heavy Hex Head ASTM F 1554 GR. 36 3/4**
Item number:	not available
Specification text:	∅ 3/4 in Heavy Hex Head ASTM F 1554 GR. 36 with 18 in nominal embedment depth per Technical data , cast in place installation per MPII
Effective embedment depth:	h_{ef} = 18.000 in.
Material:	ASTM F 1554
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-19 / CIP
Shear edge breakout verification:	Row closest to edge (Case 3 only from ACI 318-19 Fig. R.17.7.2.1b)
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 1.000 in.
Anchor plate[R] :	l_x x l_y x t = 18.000 in. x 18.000 in. x 1.000 in.;
Profile:	Square HSS (AISC), HSS10X10X.500; (L x W x T) = 10.000 in. x 10.000 in. x 0.500 in.
Base material:	cracked concrete, Custom, f_c' = 4,500 psi; h = 36.000 in.
Reinforcement:	tension: not present, shear: not present;
	edge reinforcement: none or < No. 4 bar

[R] - The anchor calculation is based on a rigid anchor plate assumption.

Geometry [in.] & Loading [kip, ft.kip]



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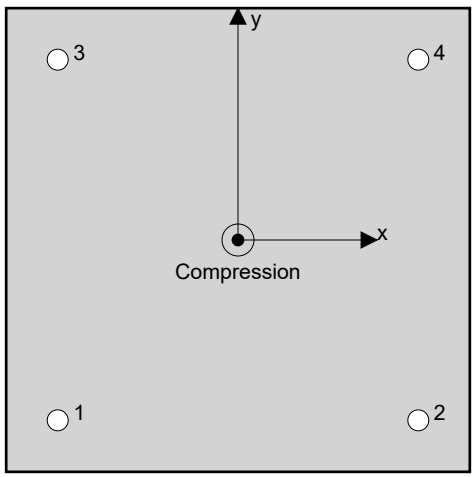
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Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	HSS COLUMNS BP	Date:	1/29/2025
Fastening point:			

1.1 Design results

Case	Description	Forces [kip] / Moments [ft.kip]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = -129.000; V_x = 3.000; V_y = 12.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	82

2 Load case/Resulting anchor forces



Anchor reactions [kip]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	0.000	3.092	0.750	3.000
2	0.000	3.092	0.750	3.000
3	0.000	3.092	0.750	3.000
4	0.000	3.092	0.750	3.000

Max. concrete compressive strain:	0.00 [‰]
Max. concrete compressive stress:	398 [psi]
Resulting tension force in (x/y)=(-/-):	0.000 [kip]
Resulting compression force in (x/y)=(0.000/-0.000):	129.000 [kip]

Anchor forces are calculated based on the assumption of a rigid anchor plate.

3 Tension load

	Load N_{ua} [kip]	Capacity ϕ N_n [kip]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	N/A	N/A	N/A	N/A
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	N/A	N/A	N/A	N/A
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (anchors in tension)

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	3
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	HSS COLUMNS BP	Date:	1/29/2025
Fastening point:			

4 Shear load

	Load V_{ua} [kip]	Capacity ϕ V_n [kip]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	3.092	7.555	41	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	12.369	119.855	11	OK
Concrete edge failure in direction y+**	12.369	15.138	82	OK

* highest loaded anchor **anchor group (relevant anchors)

4.1 Steel Strength

$V_{sa} = 0.6\ A_{se,V}\ f_{uta}$ ACI 318-19 Eq. (17.7.1.2b)
$\phi\ V_{steel} \geq V_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.33	58,000

Calculations

V_{sa} [kip]
11.623

Results

V_{sa} [kip]	ϕ_{steel}	ϕ V_{sa} [kip]	V_{ua} [kip]
11.623	0.650	7.555	3.092

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Company:		Page:	4
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	HSS COLUMNS BP	Date:	1/29/2025
Fastening point:			

4.2 Pryout Strength

$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \right]$ — ACI 318-19 Eq. (17.7.3.1b)

$\phi V_{cpg} \geq V_{ua}$ — ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9 h_{ef}^2$ — ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e'_N}{3 h_{ef}}} \right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ — ACI 318-19 Eq. (17.6.2.2.1)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	7.333	0.000	0.000	11.000

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.000	-	24	1.000	4,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [kip]
1,296.00	484.00	1.000	1.000	1.000	1.000	31.972

Results

V_{cpg} [kip]	$\phi_{concrete}$	ϕV_{cpg} [kip]	V_{ua} [kip]
171.222	0.700	119.855	12.369

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Company:		Page:	5
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	HSS COLUMNS BP	Date:	1/29/2025
Fastening point:			

4.3 Concrete edge failure in direction y+

$V_{cbg} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \psi_{ec,V}\ \psi_{ed,V}\ \psi_{c,V}\ \psi_{h,V}\ \psi_{parallel,V}\ V_b$ — ACI 318-19 Eq. (17.7.2.1b)

$\phi\ V_{cbg} \geq V_{ua}$ — ACI 318-19 Table 17.5.2

A_{Vc} see ACI 318-19, Section 17.7.2.1, Fig. R 17.7.2.1(b)*

$A_{Vc0} = 4.5\ c_{a1}^2$ — ACI 318-19 Eq. (17.7.2.1.3)

$\psi_{ec,V} = \left(\frac{1}{1 + \frac{e'_v}{1.5c_{a1}}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.3.1)

$\psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.4.1b)

$\psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ — ACI 318-19 Eq. (17.7.2.6.1)

$V_b = 9\ \lambda_a\ \sqrt{f'_c}\ c_{a1}^{1.5}$ — ACI 318-19 Eq. (17.7.2.2.1b)

Variables

c_{a1} [in.]	c_{a2} [in.]	e_{cV} [in.]	$\psi_{c,V}$	h_a [in.]
11.000	11.000	0.000	1.000	36.000

l_e [in.]	λ_a	d_a [in.]	f'_c [psi]	$\psi_{parallel,V}$
6.000	1.000	0.750	4,500	1.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\psi_{ec,V}$	$\psi_{ed,V}$	$\psi_{h,V}$	V_b [kip]
594.00	544.50	1.000	0.900	1.000	22.026

Results

V_{cbg} [kip]	$\phi_{concrete}$	$\phi\ V_{cbg}$ [kip]	V_{ua} [kip]
21.626	0.700	15.138	12.369

*Anchor row defined by: Anchor 3, 4; Case 3 controls

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Company:		Page:	6
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	HSS COLUMNS BP	Date:	1/29/2025
Fastening point:			

5 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- User is responsible for evaluating the hole bearing capacity in case of shear forces.
- The equations presented in this report are based on imperial units. When inputs are displayed in metric units, the user should be aware that the equations remain in their imperial format.
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- For additional information about ACI 318 strength design provisions, please go to https://submittals.us.hilti.com/PROFISAnchorDesignGuide/

Fastening meets the design criteria!

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Company:		Page:	7
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	HSS COLUMNS BP	Date:	1/29/2025
Fastening point:			

6 Anchor plate and concrete bearing stress check

	Load	Capacity	Utilization [%]	Status
Concentric Compression	3,595.78 [in.lb/in.]	11,250.00 [in.lb/in.]	32	OK
Concrete bearing	398 [psi]	4,972 [psi]	9	OK
Tension Interface	N/A	N/A	N/A	N/A
Uniaxial Moment (Strong Axis)	N/A	N/A	N/A	N/A
Uniaxial Moment (Weak Axis)	N/A	N/A	N/A	N/A

6.1 Concrete bearing check (per AISC DG1, section 3.1.1)

$A_1 = B \cdot N$

$A_2 = L \cdot W$

$f_{pu(max)} = \phi \left(0.85 f'_c \sqrt{\frac{A_2}{A_1}} \right), \sqrt{\frac{A_2}{A_1}} \leq 2$

$f_{pu} = \frac{P_u}{BN}$

$f_{pu} \leq f_{p(max)}$

Variables

B [in.]	N [in.]	L [in.]	W [in.]	f_c' [psi]	ϕ	P_u [kip]
18.000	18.000	36.000	36.000	4,500	0.650	129.000

Calculations

A_1 [in.^{2}]	A_2 [in.^{2}]
324.00	1,296.00

Results

f_{pu} [psi]	$f_{pu(max)}$ [psi]
398	4,972

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	8
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	HSS COLUMNS BP	Date:	1/29/2025
Fastening point:			

6.2 Plate bending, compression (per AISC DG1, sections 3.1.2, 3.1.3)

$$m = \frac{N-0.95d}{2}$$

$$n = \frac{B-0.95b_f}{2}$$

$$l = \max(m,n)$$

$$f_{pu} = \frac{P_u}{BN}$$

$$M_{pl} = \frac{f_{pu} \cdot l^2}{2}$$

$$\phi M_n = \phi \cdot F_y \cdot \frac{t_p^2}{4}$$

$$M_{pl} \leq \phi M_n$$

Variables

B [in.]	N [in.]	L [in.]	W [in.]	d [in.]
18.000	18.000	36.000	36.000	10.000

b_f [in.]	F_y [psi]	ϕ	t_p [in.]	P_u' [kip]
10.000	50,000	0.900	1.000	129.000

Calculations

A_1 [in.^{2}]	A_2 [in.^{2}]	P_p [kip]	X	λ
324.00	1,296.00	-	-	-

m [in.]	n [in.]	λ n' [in.]	l [in.]	f_{pu} [psi]
4.250	4.250	-	4.250	398

Results

M_n [in.lb/in.]	ϕ	ϕM_n [in.lb/in.]	M_{pl} [in.lb/in.]
12,500.00	0.900	11,250.00	3,595.78

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Company:		Page:	9
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	HSS COLUMNS BP	Date:	1/29/2025
Fastening point:			

7 Installation data

Profile: Square HSS (AISC), HSS10X10X.500; (L x W x T) = 10.000 in. x 10.000 in. x 0.500 in.

Hole diameter in the fixture: d_f = 0.812 in.

Plate thickness (input): 1.000 in.

Anchor type and diameter: Heavy Hex Head ASTM F 1554 GR. 36 3/4

Item number: not available

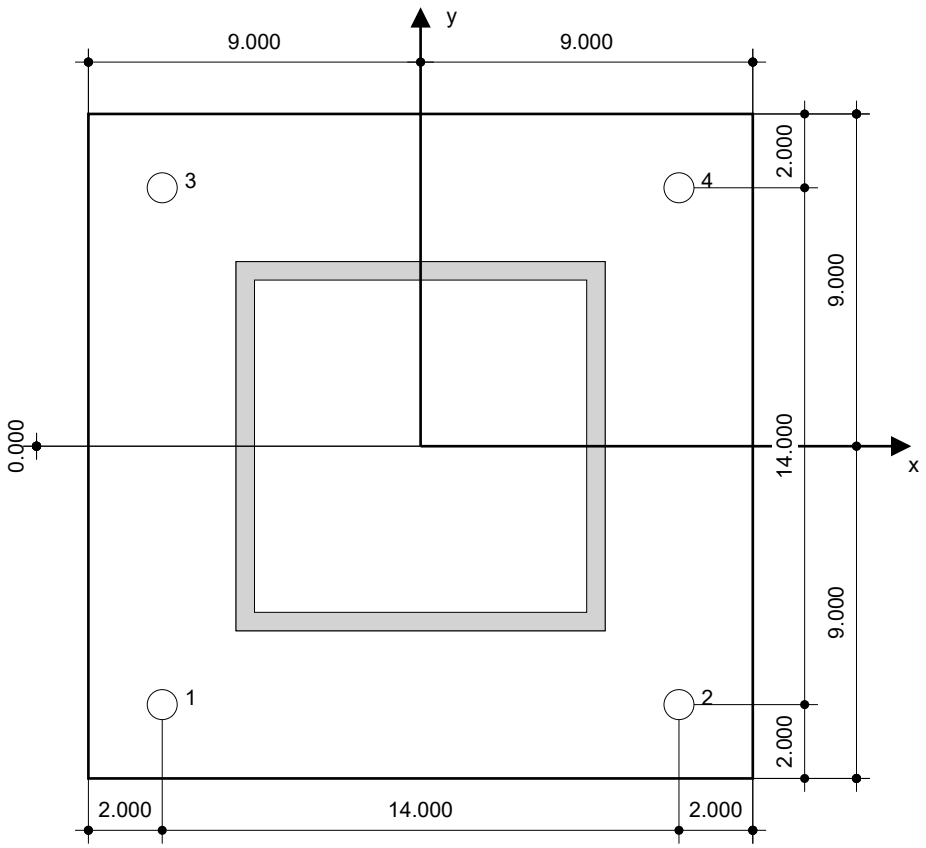
Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 18.000 in.

Minimum thickness of the base material: 19.000 in.

⌀ 3/4 in Heavy Hex Head ASTM F 1554 GR. 36 with 18 in nominal embedment depth per Technical data , cast in place installation per MPII



Coordinates Anchor in.

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-7.000	-7.000	11.000	25.000	11.000	25.000
2	7.000	-7.000	25.000	11.000	11.000	25.000
3	-7.000	7.000	11.000	25.000	25.000	11.000
4	7.000	7.000	25.000	11.000	25.000	11.000

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	10
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	HSS COLUMNS BP	Date:	1/29/2025
Fastening point:			

8 Remarks; Your Cooperation Duties

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Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Weak Axis FOH BP Brace (size)	Date:	4/24/2025
Fastening point:			

Specifier's comments:

1 Anchor Design

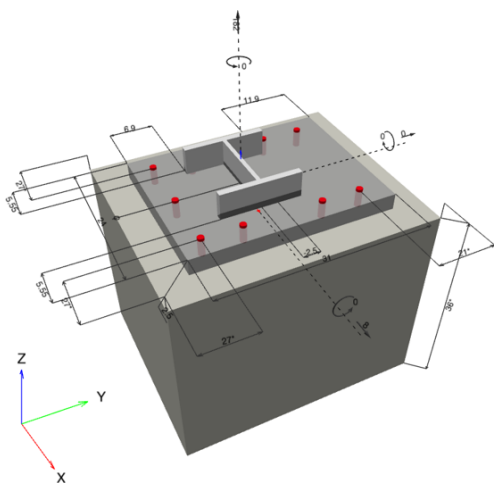
1.1 Input data



Anchor type and diameter:	**Heavy Hex Head ASTM F 1554 GR. 105 1**
Item number:	not available
Specification text:	∅ 1 in Heavy Hex Head ASTM F 1554 GR. 105 with 24 in nominal embedment depth per Technical data , cast in place installation per MPII,
Effective embedment depth:	h_{ef} = 24.000 in.
Material:	ASTM F 1554
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-19 / CIP
Shear edge breakout verification:	Row closest to edge (Case 3 only from ACI 318-19 Fig. R.17.7.2.1b)
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 2.500 in.
Anchor plate[CBFEM] :	l_x x l_y x t = 24.000 in. x 31.000 in. x 2.500 in.;
Profile:	W shape (AISC), W12X106; (L x W x T x FT) = 12.900 in. x 12.200 in. x 0.610 in. x 0.990 in.
Base material:	cracked concrete, Custom, f_c' = 4,500 psi; h = 36.000 in.
Reinforcement:	tension: present, shear: not present; edge reinforcement: none or < No. 4 bar

[CBFEM] - The anchor calculation is based on a component-based Finite Element Method (CBFEM)

Geometry [in.] & Loading [kip, ft.kip]



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Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Weak Axis FOH BP Brace (size)	Date:	4/24/2025
Fastening point:			

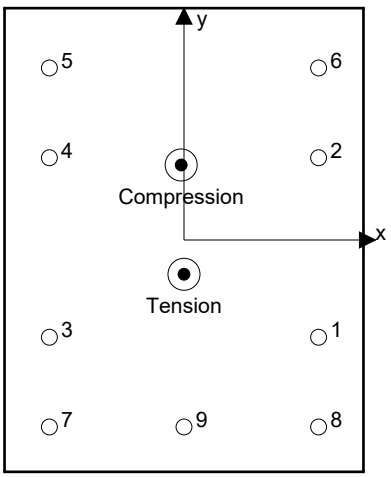
1.1.1 Design results

Case	Description	Forces [kip] / Moments [ft.kip]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 182.000; V_x = 8.000; V_y = 0.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	115

1.2 Load case/Resulting anchor forces

Anchor reactions [kip]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	33.617	0.909	0.900	0.128
2	30.825	0.846	0.838	-0.118
3	33.747	0.959	0.955	0.089
4	30.843	0.930	0.908	-0.199
5	8.055	0.869	0.846	-0.201
6	8.093	0.812	0.806	-0.103
7	9.938	0.952	0.943	0.131
8	10.036	0.901	0.887	0.157
9	21.133	0.925	0.917	0.116



Resulting tension force in (x/y)=(-0.001/-2.291): 186.287 [kip]
Resulting compression force in (x/y)=(-0.188/5.016): 4.645 [kip]

Anchor forces are calculated based on a component-based Finite Element Method (CBFEM)

1.3 Tension load

	Load N_{ua} [kip]	Capacity ϕ N_n [kip]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	33.747	56.812	60	OK
Pullout Strength*	33.747	37.825	90	OK
Concrete Breakout Failure**	186.287	162.371	115 59	not recommended
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (anchors in tension)

SEE SUPPLEMENTAL CALCULATION

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Company:		Page:	3
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Weak Axis FOH BP Brace (size)	Date:	4/24/2025
Fastening point:			

1.3.1 Steel Strength

$N_{sa} = A_{se,N} f_{uta}$ ACI 318-19 Eq. (17.6.1.2)
$\phi N_{sa} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.61	125,001

Calculations

N_{sa} [kip]
75.750

Results

N_{sa} [kip]	ϕ_{steel}	ϕN_{sa} [kip]	N_{ua} [kip]
75.750	0.750	56.812	33.747

1.3.2 Pullout Strength

$N_{pN} = \psi_{c,p} N_p$ ACI 318-19 Eq. (17.6.3.1)
$N_p = 8 A_{brg} f'_c$ ACI 318-19 Eq. (17.6.3.2.2a)
$\phi N_{pN} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$\psi_{c,p}$	A_{brg} [in.^{2}]	λ_a	f'_c [psi]
1.000	1.50	1.000	4,500

Calculations

N_p [kip]
54.036

Results

N_{pn} [kip]	$\phi_{concrete}$	ϕN_{pn} [kip]	N_{ua} [kip]
54.036	0.700	37.825	33.747

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Company:		Page:	4
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Weak Axis FOH BP Brace (size)	Date:	4/24/2025
Fastening point:			

1.3.3 Concrete Breakout Failure

$N_{cbg} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ec,N}\ \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b$ — ACI 318-19 Eq. (17.6.2.1b)

$\phi\ N_{cbg} \geq N_{ua}$ — ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9\ h_{ef}^2$ — ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\ e'_N}{3\ h_{ef}}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3\left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = 16\ \lambda_a\ \sqrt{f'_c}\ h_{ef}^{5/3}$ — ACI 318-19 Eq. (17.6.2.2.3)

Variables

h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
18.000	0.001	0.458	21.000	1.000

c_{ac} [in.]	k_c	λ_a	f'_c [psi]
-	16	1.000	4,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [kip]
5,184.00	2,916.00	1.000	0.983	0.933	1.000	132.693

Results

N_{cbg} [kip]	$\phi_{concrete}$	$\phi\ N_{cbg}$ [kip]	N_{ua} [kip]
216.495	0.750	162.371	186.287

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Company:		Page:	5
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Weak Axis FOH BP Brace (size)	Date:	4/24/2025
Fastening point:			

1.4 Shear load

	Load V_{ua} [kip]	Capacity ϕ V_n [kip]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	0.959	29.542	4	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	8.000	300.813	3	OK
Concrete edge failure in direction x+**	8.024	41.495	20	OK

* highest loaded anchor **anchor group (relevant anchors)

1.4.1 Steel Strength

$V_{sa} = 0.6\ A_{se,V}\ f_{uta}$ ACI 318-19 Eq. (17.7.1.2b)
$\phi\ V_{steel} \geq V_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.61	125,001

Calculations

V_{sa} [kip]
45.450

Results

V_{sa} [kip]	ϕ_{steel}	$\phi\ V_{sa}$ [kip]	V_{ua} [kip]
45.450	0.650	29.542	0.959

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Company:		Page:	6
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Weak Axis FOH BP Brace (size)	Date:	4/24/2025
Fastening point:			

1.4.2 Pryout Strength

$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \right]$ ACI 318-19 Eq. (17.7.3.1b)

$\phi V_{cpg} \geq V_{ua}$ ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9 h_{ef}^2$ ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e'_N}{3 h_{ef}}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = 16 \lambda_a \sqrt{f'_c} h_{ef}^{5/3}$ ACI 318-19 Eq. (17.6.2.2.3)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	18.000	0.000	0.667	21.000

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.000	-	16	1.000	4,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [kip]
5,184.00	2,916.00	1.000	0.976	0.933	1.000	132.693

Results

V_{cpg} [kip]	$\phi_{concrete}$	ϕV_{cpg} [kip]	V_{ua} [kip]
429.732	0.700	300.813	8.000

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Company:		Page:	7
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Weak Axis FOH BP Brace (size)	Date:	4/24/2025
Fastening point:			

1.4.3 Concrete edge failure in direction x+

$V_{cbg} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \Psi_{ec,V} \Psi_{ed,V} \Psi_{c,V} \Psi_{h,V} \Psi_{parallel,V} V_b$ — ACI 318-19 Eq. (17.7.2.1b)

$\phi V_{cbg} \geq V_{ua}$ — ACI 318-19 Table 17.5.2

A_{Vc} see ACI 318-19, Section 17.7.2.1, Fig. R 17.7.2.1(b)*

$A_{Vc0} = 4.5 c_{a1}^2$ — ACI 318-19 Eq. (17.7.2.1.3)

$\Psi_{ec,V} = \left(\frac{1}{1 + \frac{e'_v}{1.5c_{a1}}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.3.1)

$\Psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.4.1b)

$\Psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ — ACI 318-19 Eq. (17.7.2.6.1)

$V_b = 9 \lambda_a \sqrt{f'_c} c_{a1}^{1.5}$ — ACI 318-19 Eq. (17.7.2.2.1b)

Variables

c_{a1} [in.]	c_{a2} [in.]	e_{cV} [in.]	$\Psi_{c,V}$	h_a [in.]
24.000	21.000	1.720	1.000	36.000

l_e [in.]	λ_a	d_a [in.]	f'_c [psi]	$\Psi_{parallel,V}$
8.000	1.000	1.000	4,500	1.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{h,V}$	V_b [kip]
2,592.00	2,592.00	0.954	0.875	1.000	70.985

Results

V_{cbg} [kip]	$\phi_{concrete}$	ϕV_{cbg} [kip]	V_{ua} [kip]
59.279	0.700	41.495	8.024

*Anchor row defined by: Anchor 1, 2, 6, 8; Case 3 controls

1.5 Combined tension and shear loads, per ACI 318-19 section 17.8

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
~~1.147~~ 0.90	0.193	1.000	112 91.7	not recommended

$\beta_{NV} = (\beta_N + \beta_V) / 1.2 <= 1$

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Company:		Page:	8
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Weak Axis FOH BP Brace (size)	Date:	4/24/2025
Fastening point:			

1.6 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates as per current regulations (ETAG 001/Annex C, EOTA TR029, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid base plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- The equations presented in this report are based on imperial units. When inputs are displayed in metric units, the user should be aware that the equations remain in their imperial format.
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- For additional information about ACI 318 strength design provisions, please go to https://viewer.joomag.com/profis-design-guide-us-en-summer-2021/0841849001625154758?short&/
- The anchor design methods in PROFIS Engineering require rigid anchor plates, as per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means that the anchor plate should be sufficiently rigid to prevent load re-distribution to the anchors due to elastic/plastic displacements. The user accepts that the anchor plate is considered close to rigid by engineering judgment."

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Company:		Page:	9
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Weak Axis FOH BP Brace (size)	Date:	4/24/2025
Fastening point:			

1.7 Installation data

Profile: W shape (AISC), W12X106; (L x W x T x FT) = 12.900 in. x 12.200 in. x 0.610 in. x 0.990 in.

Hole diameter in the fixture: d_f = 1.062 in.

Plate thickness (input): 2.500 in.

Anchor type and diameter: Heavy Hex Head ASTM F 1554 GR. 105 1

Item number: not available

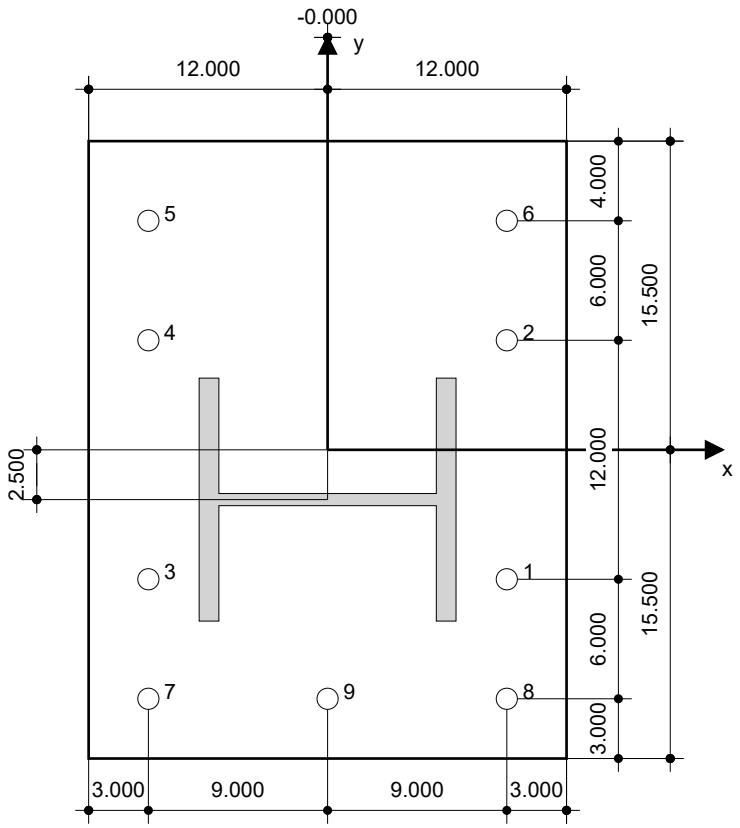
Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 24.000 in.

Minimum thickness of the base material: 25.172 in.

⌀ 1 in Heavy Hex Head ASTM F 1554 GR. 105 with 24 in nominal embedment depth per Technical data , cast in place installation per MPII



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	9.000	-6.500	45.000	27.000	33.000	39.000
2	9.000	5.500	45.000	27.000	45.000	27.000
3	-9.000	-6.500	27.000	45.000	33.000	39.000
4	-9.000	5.500	27.000	45.000	45.000	27.000
5	-9.000	11.500	27.000	45.000	51.000	21.000

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
6	9.000	11.500	45.000	27.000	51.000	21.000
7	-9.000	-12.500	27.000	45.000	27.000	45.000
8	9.000	-12.500	45.000	27.000	27.000	45.000
9	-0.000	-12.500	36.000	36.000	27.000	45.000

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Company:		Page:	10
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Weak Axis FOH BP Brace (size)	Date:	4/24/2025
Fastening point:			

2 Anchor plate design

2.1 Input data

Anchor plate:	Shape: Rectangular l_x x l_y x t = 24.000 in x 31.000 in x 2.500 in Calculation: CBFEM Material: ASTM A572 Gr.50; F_y = 50,000 psi; ε_{lim} = 5.00%
Anchor type and size:	Heavy Hex Head ASTM F 1554 GR. 105 1, h_{ef} = 24.000 in
Anchor stiffness:	The anchor is modeled considering stiffness values determined from load displacement curves tested in an independent laboratory. Please note that no simple replacement of the anchor is possible as the anchor stiffness has a major impact on the load distribution results.
Design method:	AISC and LRFD-based design using component-based FEM
Stand-off installation:	e_b = 0.000 in (No stand-off); t = 2.500 in
Profile:	W12X106; (L x W x T x FT) = 12.900 in x 12.200 in x 0.610 in x 0.990 in Material: ASTM A992; F_y = 50,000 psi; ε_{lim} = 5.00% Eccentricity x: -0.000 in Eccentricity y: -2.500 in
Base material:	Cracked concrete; Custom; $f_{c,cyl}$ = 4,500 psi; h = 36.000 in; E = 4,351,132 psi; G = 1,740,453 psi; v = 0.20; D = 156.07 lb/ft³
Welds (profile to anchor plate):	Type of redistribution: Plastic Material: E70xx
Mesh size:	Number of elements on edge: 8 Min. size of element: 0.394 in Max. size of element: 1.969 in

2.2 Summary

	Description	Profile		Anchor plate			Concrete [%]
		σ_{Ed} [psi]	ε_{Pl} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Hole bearing [%]	
1	Combination 1	25,662	0.00	9,530	0.00	1	4

2.3 Anchor plate classification

Results below are displayed for the decisive load combinations: Combination 1

Anchor tension forces	Equivalent rigid anchor plate (CBFEM)	Component-based Finite Element Method (CBFEM) anchor plate design
Anchor 1	20.893 kip	33.617 kip
Anchor 2	19.175 kip	30.825 kip
Anchor 3	20.888 kip	33.747 kip
Anchor 4	19.171 kip	30.843 kip
Anchor 5	18.312 kip	8.055 kip
Anchor 6	18.316 kip	8.093 kip
Anchor 7	21.746 kip	9.938 kip
Anchor 8	21.750 kip	10.036 kip
Anchor 9	21.748 kip	21.133 kip

User accepted to consider the selected anchor plate as rigid by his/her engineering judgement. This means the anchor design guidelines can be applied.

2.4 Profile/Stiffeners/Plate

Profile and stiffeners are verified at the level of the steel to concrete connection. The connection design does not replace the steel design for critical cross sections, which should be performed outside of PROFIS Engineering.

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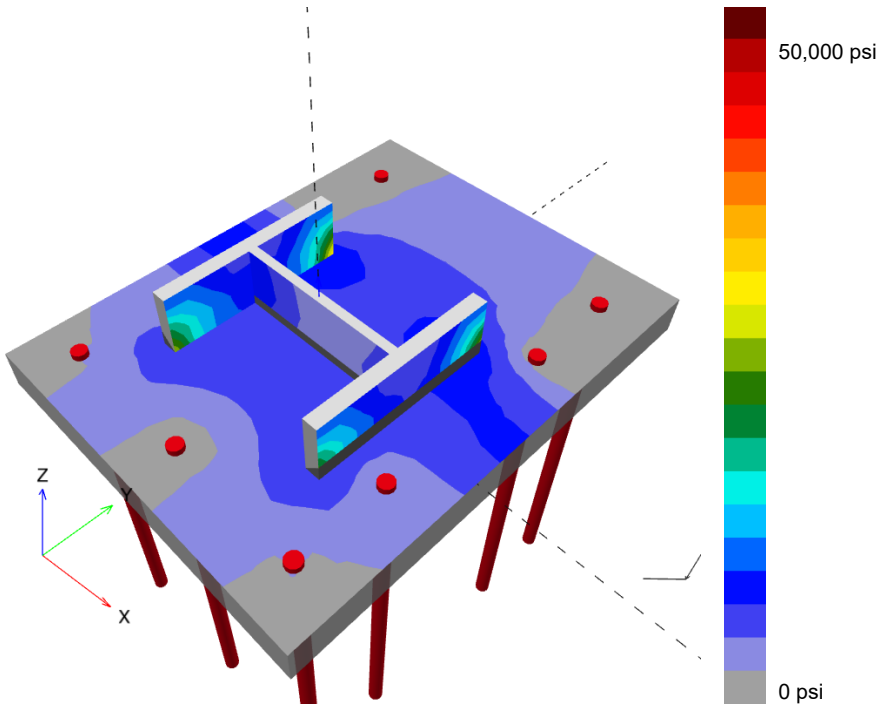
Company:		Page:	11
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Weak Axis FOH BP Brace (size)	Date:	4/24/2025
Fastening point:			

2.4.1 Equivalent stress and plastic strain

Part	Load combination	Material	f_y [psi]	ε_{lim} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Status
Plate	Combination 1	ASTM A572 Gr.50	50,000	5.00	9,530	0.00	OK
Profile	Combination 1	ASTM A992	50,000	5.00	25,116	0.00	OK
Profile	Combination 1	ASTM A992	50,000	5.00	25,662	0.00	OK
Profile	Combination 1	ASTM A992	50,000	5.00	10,632	0.00	OK

2.4.1.1 Equivalent stress

Results below are displayed for the decisive load combination: 1 - Combination 1

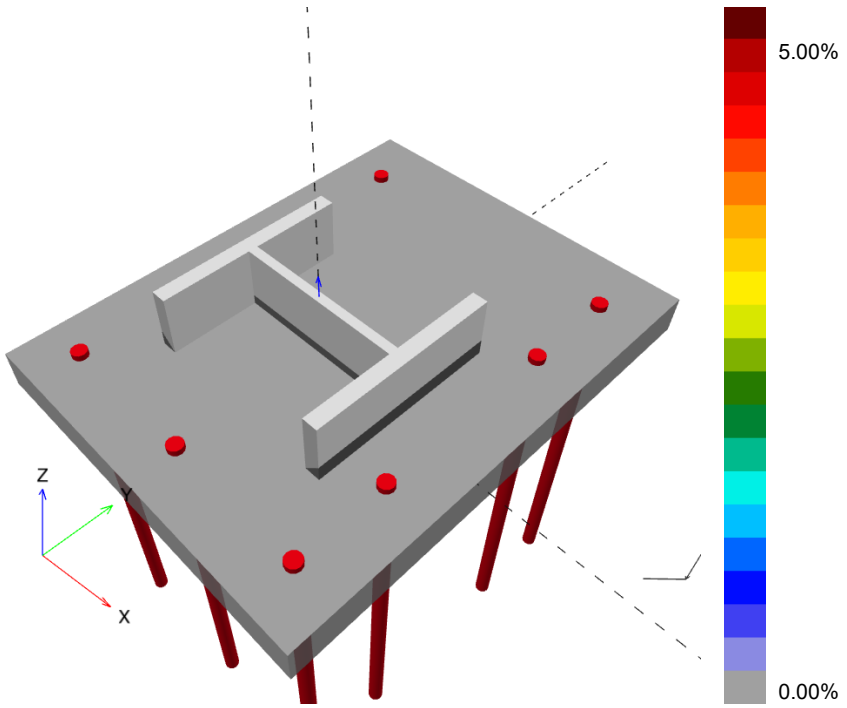


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Company:		Page:	12
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Weak Axis FOH BP Brace (size)	Date:	4/24/2025
Fastening point:			

2.4.1.2 Plastic strain

Results below are displayed for the decisive load combination: 1 - Combination 1



2.4.2 Plate hole bearing resistance, AISC 360-16 Section J3

Decisive load combination: 1 - Combination 1

Equations

$R_n = \min(1.2\, l_c\, t\, F_u\, ,\ 2.4\, d\, t\, F_u)$ (AISC 360-16 J3-6a, c)

$\Phi R_n = 0.75\, R_n$

$V \leq \Phi R_n$

Variables

	l_c [in]	t [in]	F_u [psi]	d [in]	R_n [kip]
Anchor 1	20.680	2.500	65,000	1.000	390.000
Anchor 2	20.677	2.500	65,000	1.000	390.000
Anchor 3	2.482	2.500	65,000	1.000	390.000
Anchor 4	2.540	2.500	65,000	1.000	390.000
Anchor 5	2.552	2.500	65,000	1.000	390.000
Anchor 6	20.640	2.500	65,000	1.000	390.000
Anchor 7	2.497	2.500	65,000	1.000	390.000
Anchor 8	16.633	2.500	65,000	1.000	390.000
Anchor 9	11.565	2.500	65,000	1.000	390.000

Results

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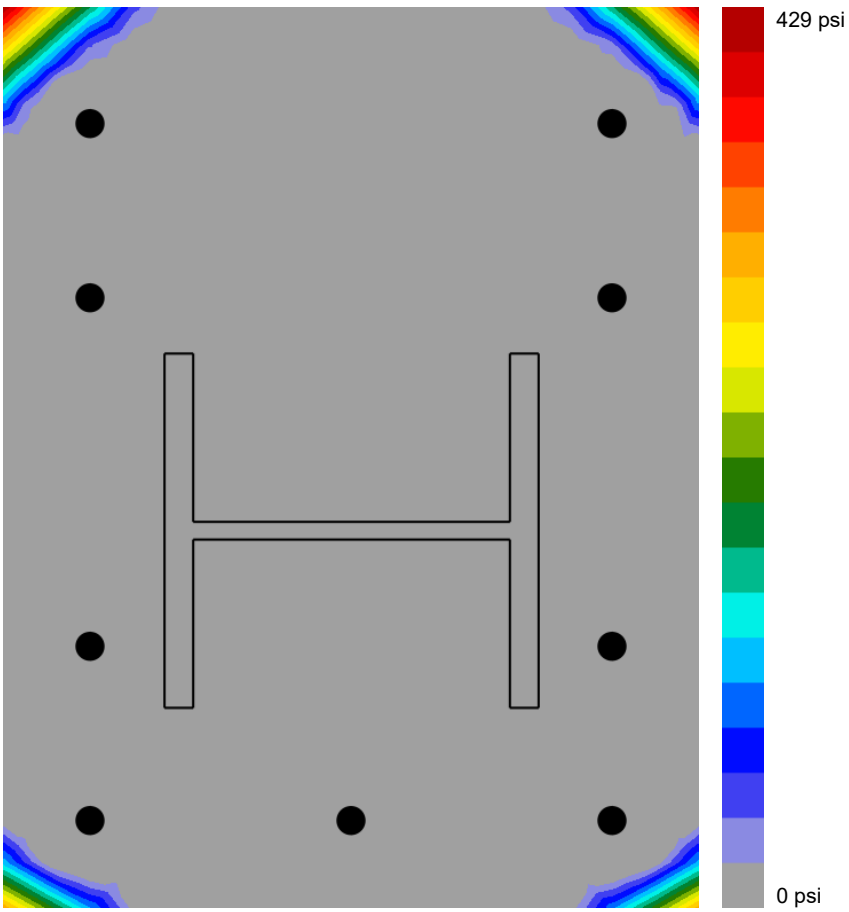
Company:		Page:	13
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Weak Axis FOH BP Brace (size)	Date:	4/24/2025
Fastening point:			

	V [kip]	ΦR_n [kip]	Utilization [%]	Status
Anchor 1	0.909	292.500	1	OK
Anchor 2	0.846	292.500	1	OK
Anchor 3	0.959	292.500	1	OK
Anchor 4	0.930	292.500	1	OK
Anchor 5	0.870	292.500	1	OK
Anchor 6	0.812	292.500	1	OK
Anchor 7	0.952	292.500	1	OK
Anchor 8	0.901	292.500	1	OK
Anchor 9	0.925	292.500	1	OK

2.5 Concrete

Decisive load combination: 1 - Combination 1

2.5.1 Compression in concrete under the anchor plate



2.5.2 Concrete block compressive strength resistance check, AISC 360-16 Section J8

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Company:		Page:	14
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Weak Axis FOH BP Brace (size)	Date:	4/24/2025
Fastening point:			

Equations

$F_p = \Phi f_{p,max}$

$f_{p,max} = 0.85 f_c' \sqrt{(\frac{A_2}{A_1})} \le 1.7 f_c; \quad \sqrt{(\frac{A_2}{A_1})} \le 2$

$\sigma = \frac{N}{A_1}$

$\text{Utilization} = \frac{\sigma}{F_p}$

Variables

N [kip]	f_c' [psi]	Φ	A_1 [in²]	A_2 [in²]
4.645	4,500	0.65	28.65	3,550.62

Results

Load combination	F_p [psi]	σ [psi]	Utilization [%]	Status
Combination 1	4,973	162	4	OK

2.6 Symbol explanation

A_1	Loaded area of concrete
A_2	Supporting area
d	Nominal diameter of the bolt
ε_{lim}	Limit plastic strain
ε_{Pl}	Plastic strain from CBFEM results
f_c	Concrete compressive strength
f_c'	Concrete compressive strength
F_u	Specified minimum tensile strength of the connected material
F_p	Concrete block design bearing strength
$f_{p,max}$	Concrete block design bearing strength maximum
f_y	Yield strength
l_c	Clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material
N	Resulting compression force
σ	Average stress in concrete
σ_{Ed}	Equivalent stress
Φ	Resistance factor
ΦR_n	Factored resistance
t	Thickness of the anchor plate
V	Resultant of shear forces Vy, Vz in bolt.

2.7 Warnings

- By using the CBFEM calculation functionality of PROFIS Engineering you may act outside the applicable design codes and your specified anchor plate may not behave rigid. Please, validate the results with a professional designer and/or structural engineer to ensure suitability and adequacy for your specific jurisdiction and project requirements.
- The anchor is modeled considering stiffness values determined from load displacement curves tested in an independent laboratory. Please note that no simple replacement of the anchor is possible as the anchor stiffness has a major impact on the load distribution results.

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Company:		Page:	15
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Weak Axis FOH BP Brace (size)	Date:	4/24/2025
Fastening point:			

3 Summary of results

Design of the anchor plate, anchors, welds and other elements are based on CBFEM (component based finite element method) and AISC.

	Load combination	Max. utilization	Status
Anchors	Combination 1	115%	NOT OK
Anchor plate	Combination 1	20%	OK
Concrete	Combination 1	4%	OK
Profile	Combination 1	52%	OK

Fastening does not meet the design criteria!

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Company:		Page:	16
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Weak Axis FOH BP Brace (size)	Date:	4/24/2025
Fastening point:			

4 Remarks; Your Cooperation Duties

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Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis BP Brace (FOH) - bp-11	Date:	4/24/2025
Fastening point:			

Specifier's comments:

1 Anchor Design

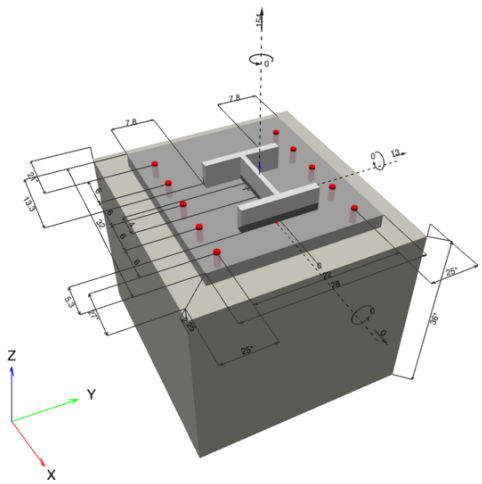
1.1 Input data



Anchor type and diameter:	**Heavy Hex Head ASTM F 1554 GR. 105 1**
Item number:	not available
Specification text:	∅ 1 in Heavy Hex Head ASTM F 1554 GR. 105 with 24 in nominal embedment depth per Technical data , cast in place installation per MPII,
Effective embedment depth:	h_{ef} = 24.000 in.
Material:	ASTM F 1554
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-19 / CIP
Shear edge breakout verification:	Row closest to edge (Case 3 only from ACI 318-19 Fig. R.17.7.2.1b)
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 2.250 in.
Anchor plate[CBFEM] :	l_x x l_y x t = 32.000 in. x 28.000 in. x 2.250 in.;
Profile:	W shape (AISC), W12X136; (L x W x T x FT) = 13.400 in. x 12.400 in. x 0.790 in. x 1.250 in.
Base material:	cracked concrete, Custom, f_c' = 4,500 psi; h = 36.000 in.
Reinforcement:	tension: present, shear: present; edge reinforcement: none or < No. 4 bar

[CBFEM] - The anchor calculation is based on a component-based Finite Element Method (CBFEM)

Geometry [in.] & Loading [kip, ft.kip]



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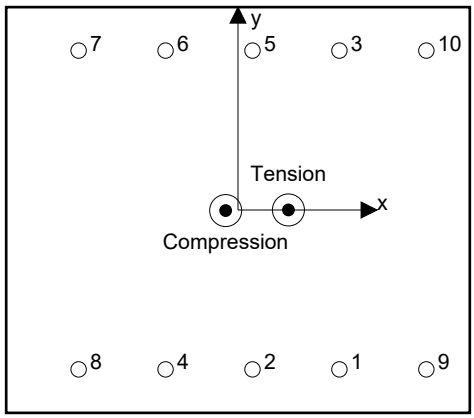
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Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis BP Brace (FOH) - bp-11	Date:	4/24/2025
Fastening point:			

1.1.1 Design results

Case	Description	Forces [kip] / Moments [ft.kip]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 154.000; V_x = 0.000; V_y = 13.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	117

1.2 Load case/Resulting anchor forces



Anchor reactions [kip]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	19.988	1.777	0.208	1.765
2	18.382	1.621	0.213	1.607
3	20.011	1.110	-0.200	1.092
4	15.117	1.434	0.242	1.413
5	18.384	1.038	-0.236	1.010
6	15.128	0.961	-0.201	0.939
7	7.724	0.994	-0.210	0.971
8	7.707	1.140	0.246	1.114
9	22.596	1.883	0.180	1.874
10	22.616	1.238	-0.241	1.214

Resulting tension force in (x/y)=(3.481/0.005): 167.654 [kip]
Resulting compression force in (x/y)=(-0.851/-0.040): 15.377 [kip]

Anchor forces are calculated based on a component-based Finite Element Method (CBFEM)

1.3 Tension load

	Load N_{ua} [kip]	Capacity ϕ N_n [kip]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	22.616	56.812	40	OK
Pullout Strength*	22.616	37.825	60	OK
Concrete Breakout Failure**	167.654	151.207	111 47.2	not recommended
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (anchors in tension)

SEE SUPPLEMENTAL CALCULATIONS

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Company:		Page:	3
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis BP Brace (FOH) - bp-11	Date:	4/24/2025
Fastening point:			

1.3.1 Steel Strength

$N_{sa} = A_{se,N} f_{uta}$ ACI 318-19 Eq. (17.6.1.2)
$\phi N_{sa} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.61	125,001

Calculations

N_{sa} [kip]
75.750

Results

N_{sa} [kip]	ϕ_{steel}	ϕN_{sa} [kip]	N_{ua} [kip]
75.750	0.750	56.812	22.616

1.3.2 Pullout Strength

$N_{pN} = \psi_{c,p} N_p$ ACI 318-19 Eq. (17.6.3.1)
$N_p = 8 A_{brg} f'_c$ ACI 318-19 Eq. (17.6.3.2.2a)
$\phi N_{pN} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$\psi_{c,p}$	A_{brg} [in.^{2}]	λ_a	f'_c [psi]
1.000	1.50	1.000	4,500

Calculations

N_p [kip]
54.036

Results

N_{pn} [kip]	$\phi_{concrete}$	ϕN_{pn} [kip]	N_{ua} [kip]
54.036	0.700	37.825	22.616

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Company:		Page:	4
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis BP Brace (FOH) - bp-11	Date:	4/24/2025
Fastening point:			

1.3.3 Concrete Breakout Failure

$N_{cbg} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ec,N}\ \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b$ — ACI 318-19 Eq. (17.6.2.1b)

$\phi\ N_{cbg} \geq N_{ua}$ — ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9\ h_{ef}^2$ — ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\ e'_N}{3\ h_{ef}}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3\left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = 16\ \lambda_a\ \sqrt{f'_c}\ h_{ef}^{5/3}$ — ACI 318-19 Eq. (17.6.2.2.3)

Variables

h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
18.000	2.481	0.005	21.000	1.000

c_{ac} [in.]	k_c	λ_a	f'_c [psi]
-	16	1.000	4,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [kip]
5,184.00	2,916.00	0.916	1.000	0.933	1.000	132.693

Results

N_{cbg} [kip]	$\phi_{concrete}$	$\phi\ N_{cbg}$ [kip]	N_{ua} [kip]
201.609	0.750	151.207	167.654

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Company:		Page:	5
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis BP Brace (FOH) - bp-11	Date:	4/24/2025
Fastening point:			

1.4 Shear load

	Load V_{ua} [kip]	Capacity ϕ V_n [kip]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	1.883	29.542	7	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	13.000	277.416	5	OK
Concrete edge failure in direction y+**	13.045	45.135	29	OK

* highest loaded anchor **anchor group (relevant anchors)

1.4.1 Steel Strength

$V_{sa} = 0.6\ A_{se,V}\ f_{uta}$ ACI 318-19 Eq. (17.7.1.2b)
$\phi\ V_{steel} \geq V_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.61	125,001

Calculations

V_{sa} [kip]
45.450

Results

V_{sa} [kip]	ϕ_{steel}	$\phi\ V_{sa}$ [kip]	V_{ua} [kip]
45.450	0.650	29.542	1.883

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Company:		Page:	6
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis BP Brace (FOH) - bp-11	Date:	4/24/2025
Fastening point:			

1.4.2 Pryout Strength

$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \right]$ ACI 318-19 Eq. (17.7.3.1b)

$\phi V_{cpg} \geq V_{ua}$ ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9 h_{ef}^2$ ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e'_N}{3 h_{ef}}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = 16 \lambda_a \sqrt{f'_c} h_{ef}^{5/3}$ ACI 318-19 Eq. (17.6.2.2.3)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	18.000	3.000	0.000	21.000

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.000	-	16	1.000	4,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [kip]
5,184.00	2,916.00	0.900	1.000	0.933	1.000	132.693

Results

V_{cpg} [kip]	$\phi_{concrete}$	ϕV_{cpg} [kip]	V_{ua} [kip]
396.309	0.700	277.416	13.000

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Company:		Page:	7
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis BP Brace (FOH) - bp-11	Date:	4/24/2025
Fastening point:			

1.4.3 Concrete edge failure in direction y+

$V_{cbg} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \Psi_{ec,V} \Psi_{ed,V} \Psi_{c,V} \Psi_{h,V} \Psi_{parallel,V} V_b$ — ACI 318-19 Eq. (17.7.2.1b)

$\phi V_{cbg} \geq V_{ua}$ — ACI 318-19 Table 17.5.2

A_{Vc} see ACI 318-19, Section 17.7.2.1, Fig. R 17.7.2.1(b)*

$A_{Vc0} = 4.5 c_{a1}^2$ — ACI 318-19 Eq. (17.7.2.1.3)

$\Psi_{ec,V} = \left(\frac{1}{1 + \frac{e'_v}{1.5c_{a1}}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.3.1)

$\Psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.4.1b)

$\Psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ — ACI 318-19 Eq. (17.7.2.6.1)

$V_b = 9 \lambda_a \sqrt{f'_c} c_{a1}^{1.5}$ — ACI 318-19 Eq. (17.7.2.2.1b)

Variables

c_{a1} [in.]	c_{a2} [in.]	e_{cV} [in.]	$\Psi_{c,V}$	h_a [in.]
24.000	21.000	1.155	1.000	36.000

l_e [in.]	λ_a	d_a [in.]	f'_c [psi]	$\Psi_{parallel,V}$
8.000	1.000	1.000	4,500	1.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{h,V}$	V_b [kip]
2,592.00	2,592.00	0.969	0.875	1.000	70.985

Results

V_{cbg} [kip]	$\phi_{concrete}$	ϕV_{cbg} [kip]	V_{ua} [kip]
60.181	0.750	45.135	13.045

*Anchor row defined by: Anchor 3, 5-7, 10; Case 3 controls

1.5 Combined tension and shear loads, per ACI 318-19 section 17.8

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
~~1.109~~ 0.60	0.289	1.000	~~117~~ 74.1	not recommended

$\beta_{NV} = (\beta_N + \beta_V) / 1.2 <= 1$

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Company:		Page:	8
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis BP Brace (FOH) - bp-11	Date:	4/24/2025
Fastening point:			

1.6 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates as per current regulations (ETAG 001/Annex C, EOTA TR029, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid base plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- The equations presented in this report are based on imperial units. When inputs are displayed in metric units, the user should be aware that the equations remain in their imperial format.
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- For additional information about ACI 318 strength design provisions, please go to https://viewer.joomag.com/profis-design-guide-us-en-summer-2021/0841849001625154758?short&/
- The anchor design methods in PROFIS Engineering require rigid anchor plates, as per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means that the anchor plate should be sufficiently rigid to prevent load re-distribution to the anchors due to elastic/plastic displacements. The user accepts that the anchor plate is considered close to rigid by engineering judgment."

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Company:		Page:	9
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis BP Brace (FOH) - bp-11	Date:	4/24/2025
Fastening point:			

1.7 Installation data

Profile: W shape (AISC), W12X136; (L x W x T x FT) = 13.400 in. x 12.400 in. x 0.790 in. x 1.250 in.

Hole diameter in the fixture: d_f = 1.062 in.

Plate thickness (input): 2.250 in.

Anchor type and diameter: Heavy Hex Head ASTM F 1554 GR. 105 1

Item number: not available

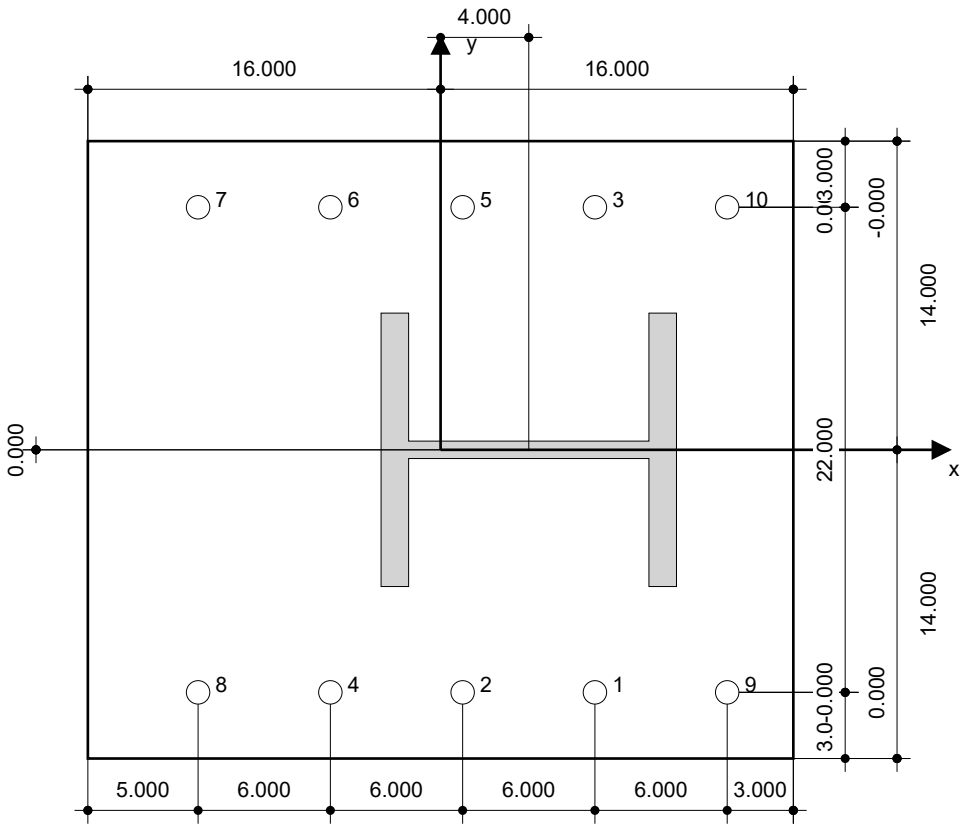
Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 24.000 in.

Minimum thickness of the base material: 25.172 in.

⌀ 1 in Heavy Hex Head ASTM F 1554 GR. 105 with 24 in nominal embedment depth per Technical data , cast in place installation per MPII



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	7.000	-11.000	39.000	33.000	25.000	47.000
2	1.000	-11.000	33.000	39.000	25.000	47.000
3	7.000	11.000	39.000	33.000	47.000	25.000
4	-5.000	-11.000	27.000	45.000	25.000	47.000
5	1.000	11.000	33.000	39.000	47.000	25.000

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
6	-5.000	11.000	27.000	45.000	47.000	25.000
7	-11.000	11.000	21.000	51.000	47.000	25.000
8	-11.000	-11.000	21.000	51.000	25.000	47.000
9	13.000	-11.000	45.000	27.000	25.000	47.000
10	13.000	11.000	45.000	27.000	47.000	25.000

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Company:		Page:	10
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis BP Brace (FOH) - bp-11	Date:	4/24/2025
Fastening point:			

2 Anchor plate design

2.1 Input data

Anchor plate:	Shape: Rectangular l_x x l_y x t = 32.000 in x 28.000 in x 2.250 in Calculation: CBFEM Material: ASTM A572 Gr.50; F_y = 50,000 psi; ε_{lim} = 5.00%
Anchor type and size:	Heavy Hex Head ASTM F 1554 GR. 105 1, h_{ef} = 24.000 in
Anchor stiffness:	The anchor is modeled considering stiffness values determined from load displacement curves tested in an independent laboratory. Please note that no simple replacement of the anchor is possible as the anchor stiffness has a major impact on the load distribution results.
Design method:	AISC and LRFD-based design using component-based FEM
Stand-off installation:	e_b = 0.000 in (No stand-off); t = 2.250 in
Profile:	W12X136; (L x W x T x FT) = 13.400 in x 12.400 in x 0.790 in x 1.250 in Material: ASTM A992; F_y = 50,000 psi; ε_{lim} = 5.00% Eccentricity x: 4.000 in Eccentricity y: 0.000 in
Base material:	Cracked concrete; Custom; $f_{c,cyl}$ = 4,500 psi; h = 36.000 in; E = 4,351,132 psi; G = 1,740,453 psi; v = 0.20; D = 156.07 lb/ft³
Welds (profile to anchor plate):	Type of redistribution: Plastic Material: E70xx
Mesh size:	Number of elements on edge: 8 Min. size of element: 0.394 in Max. size of element: 1.969 in

2.2 Summary

	Description	Profile		Anchor plate			Concrete [%]
		σ_{Ed} [psi]	ε_{Pl} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Hole bearing [%]	
1	Combination 1	29,187	0.00	17,125	0.00	1	4

2.3 Anchor plate classification

Results below are displayed for the decisive load combinations: Combination 1

Anchor tension forces	Equivalent rigid anchor plate (CBFEM)	Component-based Finite Element Method (CBFEM) anchor plate design
Anchor 1	19.248 kip	19.988 kip
Anchor 2	15.398 kip	18.382 kip
Anchor 3	19.252 kip	20.011 kip
Anchor 4	11.548 kip	15.117 kip
Anchor 5	15.402 kip	18.384 kip
Anchor 6	11.552 kip	15.128 kip
Anchor 7	7.702 kip	7.724 kip
Anchor 8	7.698 kip	7.707 kip
Anchor 9	23.098 kip	22.596 kip
Anchor 10	23.102 kip	22.616 kip

User accepted to consider the selected anchor plate as rigid by his/her engineering judgement. This means the anchor design guidelines can be applied.

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Company:		Page:	11
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis BP Brace (FOH) - bp-11	Date:	4/24/2025
Fastening point:			

2.4 Profile/Stiffeners/Plate

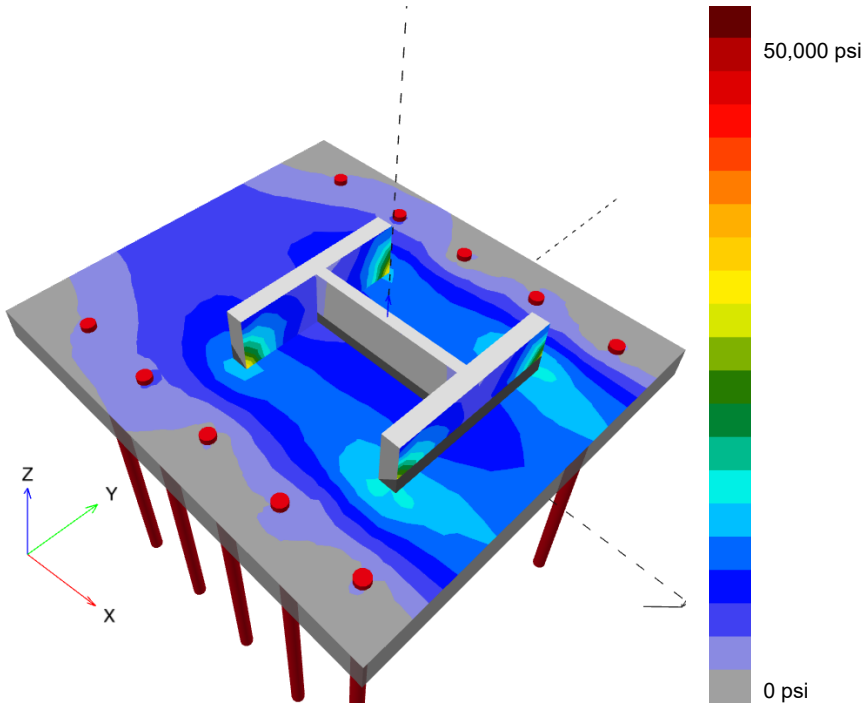
Profile and stiffeners are verified at the level of the steel to concrete connection. The connection design does not replace the steel design for critical cross sections, which should be performed outside of PROFIS Engineering.

2.4.1 Equivalent stress and plastic strain

Part	Load combination	Material	f_y [psi]	ε_{lim} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Status
Plate	Combination 1	ASTM A572 Gr.50	50,000	5.00	17,125	0.00	OK
Profile	Combination 1	ASTM A992	50,000	5.00	29,187	0.00	OK
Profile	Combination 1	ASTM A992	50,000	5.00	24,713	0.00	OK
Profile	Combination 1	ASTM A992	50,000	5.00	3,992	0.00	OK

2.4.1.1 Equivalent stress

Results below are displayed for the decisive load combination: 1 - Combination 1

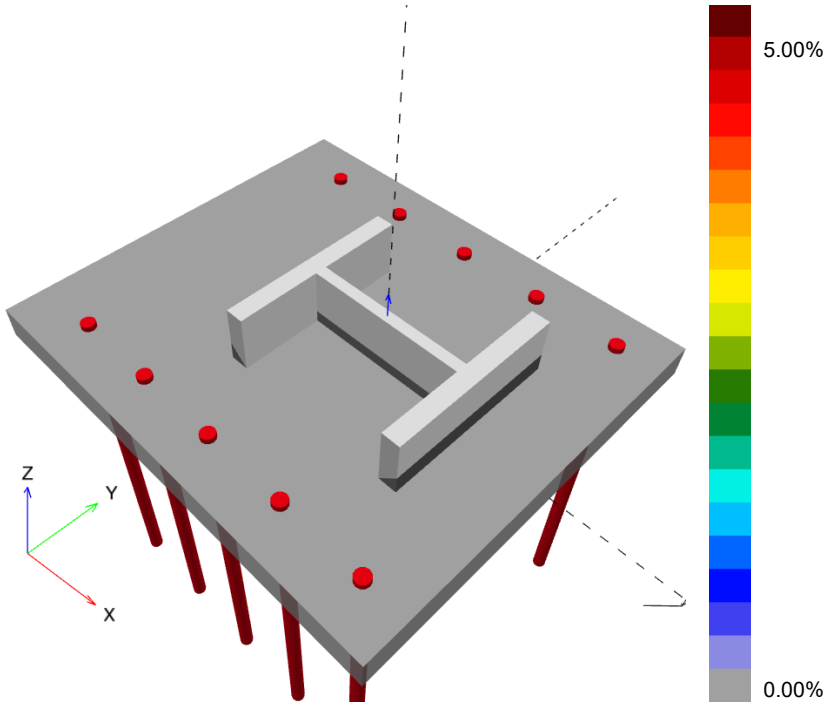


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Company:		Page:	12
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis BP Brace (FOH) - bp-11	Date:	4/24/2025
Fastening point:			

2.4.1.2 Plastic strain

Results below are displayed for the decisive load combination: 1 - Combination 1



2.4.2 Plate hole bearing resistance, AISC 360-16 Section J3

Decisive load combination: 1 - Combination 1

Equations

$R_n = \min(1.2\, l_c\, t\, F_u\, ,\ 2.4\, d\, t\, F_u)$ (AISC 360-16 J3-6a, c)

$\Phi R_n = 0.75\, R_n$

$V \leq \Phi R_n$

Variables

	l_c [in]	t [in]	F_u [psi]	d [in]	R_n [kip]
Anchor 1	2.489	2.250	65,000	1.000	351.000
Anchor 2	2.495	2.250	65,000	1.000	351.000
Anchor 3	24.884	2.250	65,000	1.000	351.000
Anchor 4	2.512	2.250	65,000	1.000	351.000
Anchor 5	21.741	2.250	65,000	1.000	351.000
Anchor 6	25.034	2.250	65,000	1.000	351.000
Anchor 7	25.047	2.250	65,000	1.000	351.000
Anchor 8	2.541	2.250	65,000	1.000	351.000
Anchor 9	2.483	2.250	65,000	1.000	351.000
Anchor 10	14.895	2.250	65,000	1.000	351.000

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Company:		Page:	13
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis BP Brace (FOH) - bp-11	Date:	4/24/2025
Fastening point:			

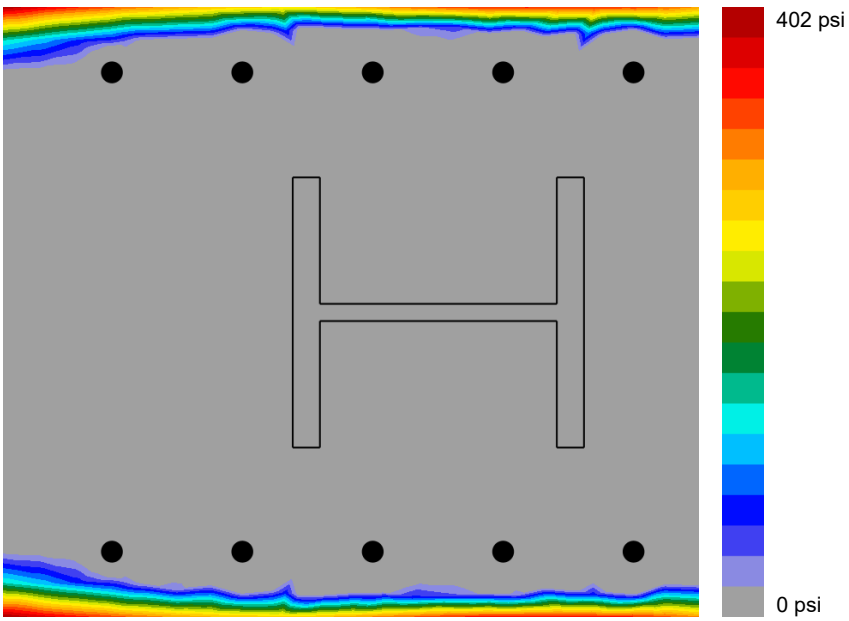
Results

	V [kip]	ΦR_n [kip]	Utilization [%]	Status
Anchor 1	1.777	263.250	1	OK
Anchor 2	1.621	263.250	1	OK
Anchor 3	1.110	263.250	1	OK
Anchor 4	1.434	263.250	1	OK
Anchor 5	1.038	263.250	1	OK
Anchor 6	0.961	263.250	1	OK
Anchor 7	0.994	263.250	1	OK
Anchor 8	1.140	263.250	1	OK
Anchor 9	1.883	263.250	1	OK
Anchor 10	1.238	263.250	1	OK

2.5 Concrete

Decisive load combination: 1 - Combination 1

2.5.1 Compression in concrete under the anchor plate



2.5.2 Concrete block compressive strength resistance check, AISC 360-16 Section J8

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Company:		Page:	14
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis BP Brace (FOH) - bp-11	Date:	4/24/2025
Fastening point:			

Equations

$F_p = \Phi f_{p,max}$

$f_{p,max} = 0.85 f_c' \sqrt{(\frac{A_2}{A_1})} \le 1.7 f_c; \quad \sqrt{(\frac{A_2}{A_1})} \le 2$

$\sigma = \frac{N}{A_1}$

$\text{Utilization} = \frac{\sigma}{F_p}$

Variables

N [kip]	f_c' [psi]	Φ	A_1 [in²]	A_2 [in²]
15.377	4,500	0.65	85.58	3,607.98

Results

Load combination	F_p [psi]	σ [psi]	Utilization [%]	Status
Combination 1	4,973	180	4	OK

2.6 Symbol explanation

A_1	Loaded area of concrete
A_2	Supporting area
d	Nominal diameter of the bolt
ε_{lim}	Limit plastic strain
ε_{Pl}	Plastic strain from CBFEM results
f_c	Concrete compressive strength
f_c'	Concrete compressive strength
F_u	Specified minimum tensile strength of the connected material
F_p	Concrete block design bearing strength
$f_{p,max}$	Concrete block design bearing strength maximum
f_y	Yield strength
l_c	Clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material
N	Resulting compression force
σ	Average stress in concrete
σ_{Ed}	Equivalent stress
Φ	Resistance factor
ΦR_n	Factored resistance
t	Thickness of the anchor plate
V	Resultant of shear forces Vy, Vz in bolt.

2.7 Warnings

- By using the CBFEM calculation functionality of PROFIS Engineering you may act outside the applicable design codes and your specified anchor plate may not behave rigid. Please, validate the results with a professional designer and/or structural engineer to ensure suitability and adequacy for your specific jurisdiction and project requirements.
- The anchor is modeled considering stiffness values determined from load displacement curves tested in an independent laboratory. Please note that no simple replacement of the anchor is possible as the anchor stiffness has a major impact on the load distribution results.

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Company:		Page:	15
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis BP Brace (FOH) - bp-11	Date:	4/24/2025
Fastening point:			

3 Summary of results

Design of the anchor plate, anchors, welds and other elements are based on CBFEM (component based finite element method) and AISC.

	Load combination	Max. utilization	Status
Anchors	Combination 1	117%	NOT OK
Anchor plate	Combination 1	35%	OK
Concrete	Combination 1	4%	OK
Profile	Combination 1	59%	OK

Fastening does not meet the design criteria!

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Company:		Page:	16
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis BP Brace (FOH) - bp-11	Date:	4/24/2025
Fastening point:			

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Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis BP Brace (FOH) - Grid F	Date:	1/31/2025
Fastening point:			

Specifier's comments:

1 Anchor Design

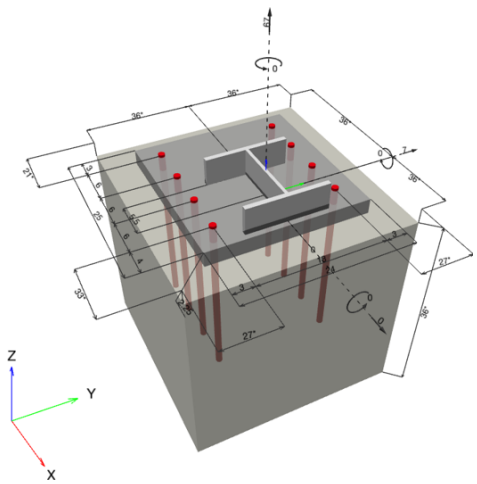
1.1 Input data



Anchor type and diameter:	**Heavy Hex Head ASTM F 1554 GR. 105 1**
Item number:	not available
Specification text:	∅ 1 in Heavy Hex Head ASTM F 1554 GR. 105 with 24 in nominal embedment depth per Technical data , cast in place installation per MPII
Effective embedment depth:	h_{ef} = 24.000 in.
Material:	ASTM F 1554
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-19 / CIP
Shear edge breakout verification:	Row closest to edge (Case 3 only from ACI 318-19 Fig. R.17.7.2.1b)
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 2.250 in.
Anchor plate[CBFEM] :	l_x x l_y x t = 25.000 in. x 24.000 in. x 2.250 in.;
Profile:	W shape (AISC), W12X79; (L x W x T x FT) = 12.400 in. x 12.100 in. x 0.470 in. x 0.735 in.
Base material:	cracked concrete, Custom, f_c' = 4,500 psi; h = 36.000 in.
Reinforcement:	tension: present, shear: present; edge reinforcement: none or < No. 4 bar

[CBFEM] - The anchor calculation is based on a component-based Finite Element Method (CBFEM)

Geometry [in.] & Loading [kip, ft.kip]



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Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis BP Brace (FOH) - Grid F	Date:	1/31/2025
Fastening point:			

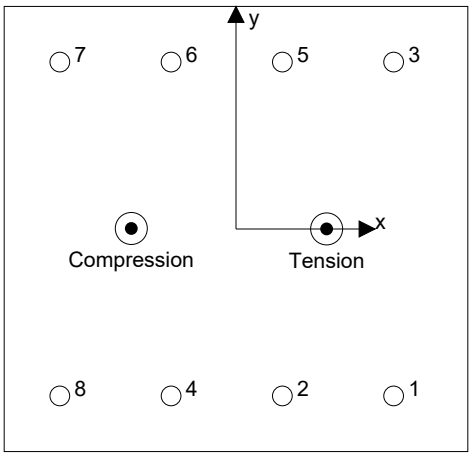
1.1.1 Design results

Case	Description	Forces [kip] / Moments [ft.kip]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 67.000; V_x = 0.000; V_y = 7.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	56

1.2 Load case/Resulting anchor forces

Anchor reactions [kip]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	20.729	1.487	0.407	1.430
2	8.787	1.165	0.369	1.105
3	20.743	1.139	-0.320	1.093
4	4.784	0.888	0.354	0.814
5	8.796	0.977	-0.383	0.898
6	4.792	0.778	-0.382	0.678
7	0.959	0.628	-0.393	0.490
8	0.954	0.602	0.347	0.492



Resulting tension force in (x/y)=(4.888/0.005): 70.544 [kip]
Resulting compression force in (x/y)=(-5.645/0.028): 4.028 [kip]

Anchor forces are calculated based on a component-based Finite Element Method (CBFEM)

1.3 Tension load

	Load N_{ua} [kip]	Capacity ϕ N_n [kip]	Utilization $\beta_N = N_{ua}/\phi N_n$	Status
Steel Strength*	20.743	56.812	37	OK
Pullout Strength*	20.743	37.825	55	OK
Concrete Breakout Failure**	70.544	126.716	56	OK
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (anchors in tension)

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Company:		Page:	3
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis BP Brace (FOH) - Grid F	Date:	1/31/2025
Fastening point:			

1.3.1 Steel Strength

$N_{sa} = A_{se,N} f_{uta}$ ACI 318-19 Eq. (17.6.1.2)
$\phi N_{sa} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.61	125,001

Calculations

N_{sa} [kip]
75.750

Results

N_{sa} [kip]	ϕ_{steel}	ϕN_{sa} [kip]	N_{ua} [kip]
75.750	0.750	56.812	20.743

1.3.2 Pullout Strength

$N_{pN} = \psi_{c,p} N_p$ ACI 318-19 Eq. (17.6.3.1)
$N_p = 8 A_{brg} f'_c$ ACI 318-19 Eq. (17.6.3.2.2a)
$\phi N_{pN} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$\psi_{c,p}$	A_{brg} [in.^{2}]	λ_a	f'_c [psi]
1.000	1.50	1.000	4,500

Calculations

N_p [kip]
54.036

Results

N_{pn} [kip]	$\phi_{concrete}$	ϕN_{pn} [kip]	N_{ua} [kip]
54.036	0.700	37.825	20.743

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Company:		Page:	4
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis BP Brace (FOH) - Grid F	Date:	1/31/2025
Fastening point:			

1.3.3 Concrete Breakout Failure

$N_{cbg} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ec,N}\ \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b$ — ACI 318-19 Eq. (17.6.2.1b)

$\phi\ N_{cbg} \geq N_{ua}$ — ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9\ h_{ef}^2$ — ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\ e'_N}{3\ h_{ef}}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3\left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = 16\ \lambda_a\ \sqrt{f'_c}\ h_{ef}^{5/3}$ — ACI 318-19 Eq. (17.6.2.2.3)

Variables

h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
22.000	5.388	0.005	21.000	1.000

c_{ac} [in.]	k_c	λ_a	f'_c [psi]
-	16	1.000	4,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [kip]
5,184.00	4,356.00	0.860	1.000	0.891	1.000	185.395

Results

N_{cbg} [kip]	$\phi_{concrete}$	$\phi\ N_{cbg}$ [kip]	N_{ua} [kip]
168.955	0.750	126.716	70.544

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	5
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis BP Brace (FOH) - Grid F	Date:	1/31/2025
Fastening point:			

1.4 Shear load

	Load V_{ua} [kip]	Capacity ϕ V_n [kip]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	1.487	29.542	6	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	7.000	232.855	4	OK
Concrete edge failure in direction y+**	7.154	43.955	17	OK

* highest loaded anchor **anchor group (relevant anchors)

1.4.1 Steel Strength

$V_{sa} = 0.6\ A_{se,V}\ f_{uta}$ ACI 318-19 Eq. (17.7.1.2b)
$\phi\ V_{steel} \geq V_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.61	125,001

Calculations

V_{sa} [kip]
45.450

Results

V_{sa} [kip]	ϕ_{steel}	$\phi\ V_{sa}$ [kip]	V_{ua} [kip]
45.450	0.650	29.542	1.487

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	6
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis BP Brace (FOH) - Grid F	Date:	1/31/2025
Fastening point:			

1.4.2 Pryout Strength

$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \right]$ ACI 318-19 Eq. (17.7.3.1b)

$\phi V_{cpg} \geq V_{ua}$ ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9 h_{ef}^2$ ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e'_N}{3 h_{ef}}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = 16 \lambda_a \sqrt{f'_c} h_{ef}^{5/3}$ ACI 318-19 Eq. (17.6.2.2.3)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	22.000	6.000	0.000	21.000

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.000	-	16	1.000	4,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [kip]
5,184.00	4,356.00	0.846	1.000	0.891	1.000	185.395

Results

V_{cpg} [kip]	$\phi_{concrete}$	ϕV_{cpg} [kip]	V_{ua} [kip]
332.650	0.700	232.855	7.000

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Company:		Page:	7
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis BP Brace (FOH) - Grid F	Date:	1/31/2025
Fastening point:			

1.4.3 Concrete edge failure in direction y+

$V_{cbg} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \Psi_{ec,V} \Psi_{ed,V} \Psi_{c,V} \Psi_{h,V} \Psi_{parallel,V} V_b$ — ACI 318-19 Eq. (17.7.2.1b)

$\phi V_{cbg} \geq V_{ua}$ — ACI 318-19 Table 17.5.2

A_{Vc} see ACI 318-19, Section 17.7.2.1, Fig. R 17.7.2.1(b)*

$A_{Vc0} = 4.5 c_{a1}^2$ — ACI 318-19 Eq. (17.7.2.1.3)

$\Psi_{ec,V} = \left(\frac{1}{1 + \frac{e'_v}{1.5c_{a1}}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.3.1)

$\Psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.4.1b)

$\Psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ — ACI 318-19 Eq. (17.7.2.6.1)

$V_b = 9 \lambda_a \sqrt{f'_c} c_{a1}^{1.5}$ — ACI 318-19 Eq. (17.7.2.2.1b)

Variables

c_{a1} [in.]	c_{a2} [in.]	e_{cV} [in.]	$\Psi_{c,V}$	h_a [in.]
24.000	21.000	2.153	1.000	36.000

l_e [in.]	λ_a	d_a [in.]	f'_c [psi]	$\Psi_{parallel,V}$
8.000	1.000	1.000	4,500	1.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{h,V}$	V_b [kip]
2,592.00	2,592.00	0.944	0.875	1.000	70.985

Results

V_{cbg} [kip]	$\phi_{concrete}$	ϕV_{cbg} [kip]	V_{ua} [kip]
58.607	0.750	43.955	7.154

*Anchor row defined by: Anchor 3, 5-7; Case 3 controls

1.5 Combined tension and shear loads, per ACI 318-19 section 17.8

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.557	0.163	5/3	43	OK

$\beta_{NV} = \beta_N^{\zeta} + \beta_V^{\zeta} <= 1$

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Company:		Page:	8
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis BP Brace (FOH) - Grid F	Date:	1/31/2025
Fastening point:			

1.6 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates as per current regulations (ETAG 001/Annex C, EOTA TR029, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid base plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- The equations presented in this report are based on imperial units. When inputs are displayed in metric units, the user should be aware that the equations remain in their imperial format.
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- For additional information about ACI 318 strength design provisions, please go to https://submittals.us.hilti.com/PROFISAnchorDesignGuide/
- The anchor design methods in PROFIS Engineering require rigid anchor plates, as per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means that the anchor plate should be sufficiently rigid to prevent load re-distribution to the anchors due to elastic/plastic displacements. The user accepts that the anchor plate is considered close to rigid by engineering judgment."

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Company:		Page:	9
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis BP Brace (FOH) - Grid F	Date:	1/31/2025
Fastening point:			

1.7 Installation data

Profile: W shape (AISC), W12X79; (L x W x T x FT) = 12.400 in. x 12.100 in. x 0.470 in. x 0.735 in.

Hole diameter in the fixture: d_f = 1.062 in.

Plate thickness (input): 2.250 in.

Anchor type and diameter: Heavy Hex Head ASTM F 1554 GR. 105 1

Item number: not available

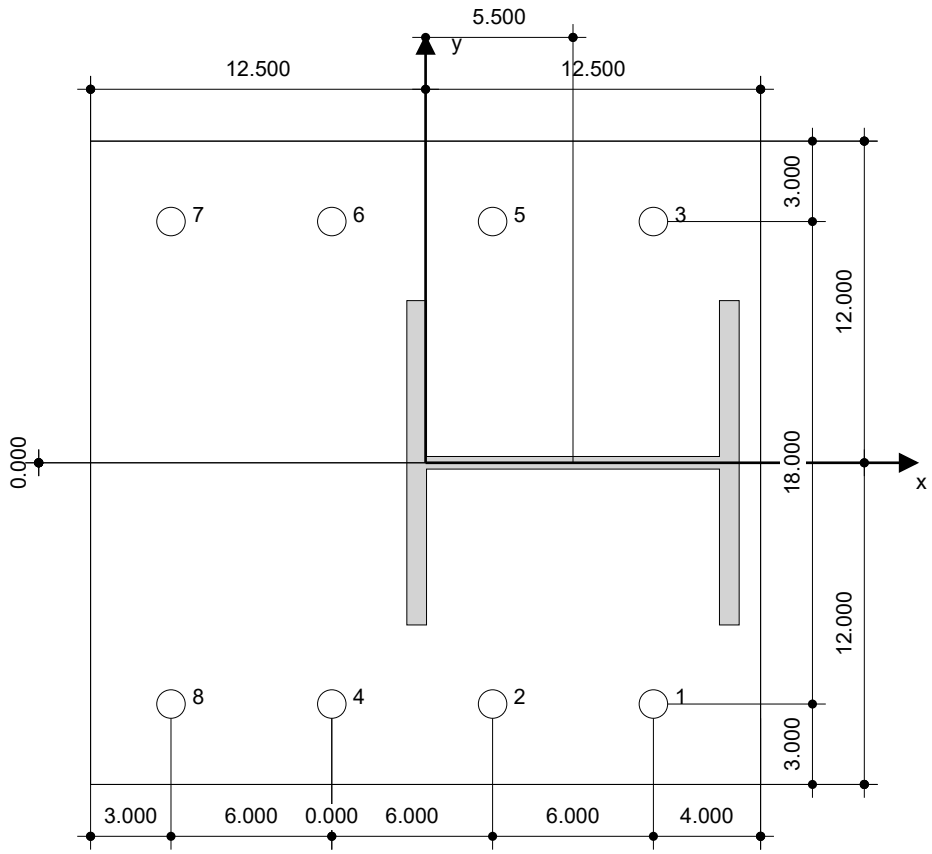
Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 24.000 in.

Minimum thickness of the base material: 25.172 in.

⌀ 1 in Heavy Hex Head ASTM F 1554 GR. 105 with 24 in nominal embedment depth per Technical data , cast in place installation per MPII



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	8.500	-9.000	39.000	33.000	27.000	45.000
2	2.500	-9.000	33.000	39.000	27.000	45.000
3	8.500	9.000	39.000	33.000	45.000	27.000
4	-3.500	-9.000	27.000	45.000	27.000	45.000

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
5	2.500	9.000	33.000	39.000	45.000	27.000
6	-3.500	9.000	27.000	45.000	45.000	27.000
7	-9.500	9.000	21.000	51.000	45.000	27.000
8	-9.500	-9.000	21.000	51.000	27.000	45.000

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	10
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis BP Brace (FOH) - Grid F	Date:	1/31/2025
Fastening point:			

2 Anchor plate design

2.1 Input data

Anchor plate:	Shape: Rectangular l_x x l_y x t = 25.000 in x 24.000 in x 2.250 in Calculation: CBFEM Material: ASTM A572 Gr.50; F_y = 50,000 psi; ε_{lim} = 5.00%
Anchor type and size:	Heavy Hex Head ASTM F 1554 GR. 105 1, h_{ef} = 24.000 in
Anchor stiffness:	The anchor is modeled considering stiffness values determined from load displacement curves tested in an independent laboratory. Please note that no simple replacement of the anchor is possible as the anchor stiffness has a major impact on the load distribution results.
Design method:	AISC and LRFD-based design using component-based FEM
Stand-off installation:	e_b = 0.000 in (No stand-off); t = 2.250 in
Profile:	W12X79; (L x W x T x FT) = 12.400 in x 12.100 in x 0.470 in x 0.735 in Material: ASTM A992; F_y = 50,000 psi; ε_{lim} = 5.00% Eccentricity x: 5.500 in Eccentricity y: -0.000 in
Base material:	Cracked concrete; Custom; $f_{c,cyl}$ = 4,500 psi; h = 36.000 in; E = 4,351,132 psi; G = 1,740,453 psi; v = 0.20; D = 156.07 lb/ft³
Welds (profile to anchor plate):	Type of redistribution: Plastic Material: E70xx
Mesh size:	Number of elements on edge: 8 Min. size of element: 0.394 in Max. size of element: 1.969 in

2.2 Summary

	Description	Profile		Anchor plate			Concrete [%]
		σ_{Ed} [psi]	ε_{Pl} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Hole bearing [%]	
1	Combination 1	18,959	0.00	8,543	0.00	1	2

2.3 Anchor plate classification

Results below are displayed for the decisive load combinations: Combination 1

Anchor tension forces	Equivalent rigid anchor plate (CBFEM)	Component-based Finite Element Method (CBFEM) anchor plate design
Anchor 1	17.879 kip	20.729 kip
Anchor 2	11.834 kip	8.787 kip
Anchor 3	17.883 kip	20.743 kip
Anchor 4	5.789 kip	4.784 kip
Anchor 5	11.838 kip	8.796 kip
Anchor 6	5.793 kip	4.792 kip
Anchor 7	0.000 kip	0.959 kip
Anchor 8	0.000 kip	0.954 kip

User accepted to consider the selected anchor plate as rigid by his/her engineering judgement. This means the anchor design guidelines can be applied.

2.4 Profile/Stiffeners/Plate

Profile and stiffeners are verified at the level of the steel to concrete connection. The connection design does not replace the steel design for critical cross sections, which should be performed outside of PROFIS Engineering.

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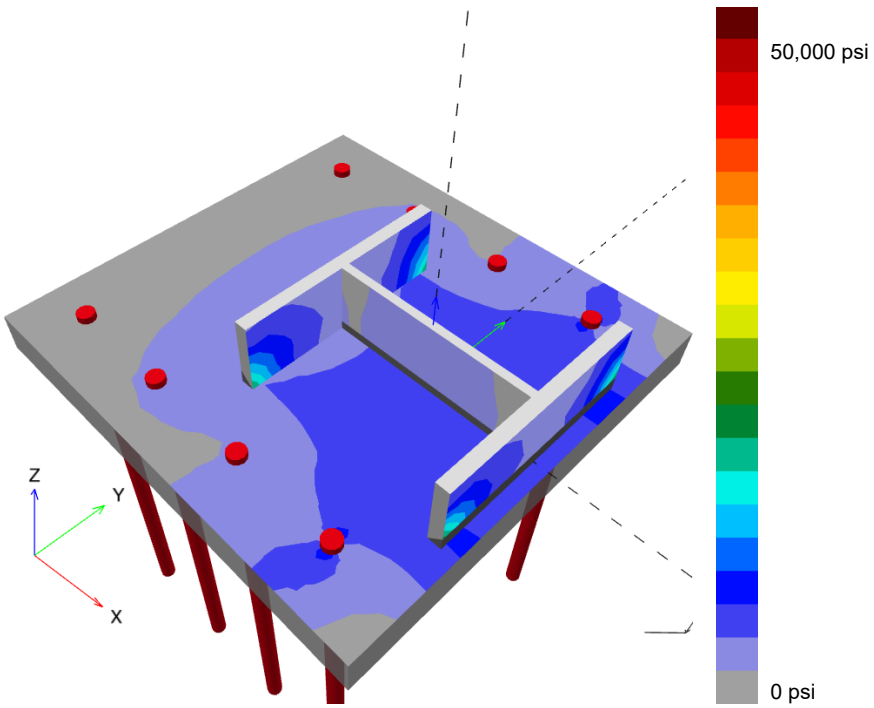
Company:		Page:	11
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis BP Brace (FOH) - Grid F	Date:	1/31/2025
Fastening point:			

2.4.1 Equivalent stress and plastic strain

Part	Load combination	Material	f_y [psi]	ε_{lim} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Status
Plate	Combination 1	ASTM A572 Gr.50	50,000	5.00	8,543	0.00	OK
Profile	Combination 1	ASTM A992	50,000	5.00	18,959	0.00	OK
Profile	Combination 1	ASTM A992	50,000	5.00	15,307	0.00	OK
Profile	Combination 1	ASTM A992	50,000	5.00	3,898	0.00	OK

2.4.1.1 Equivalent stress

Results below are displayed for the decisive load combination: 1 - Combination 1

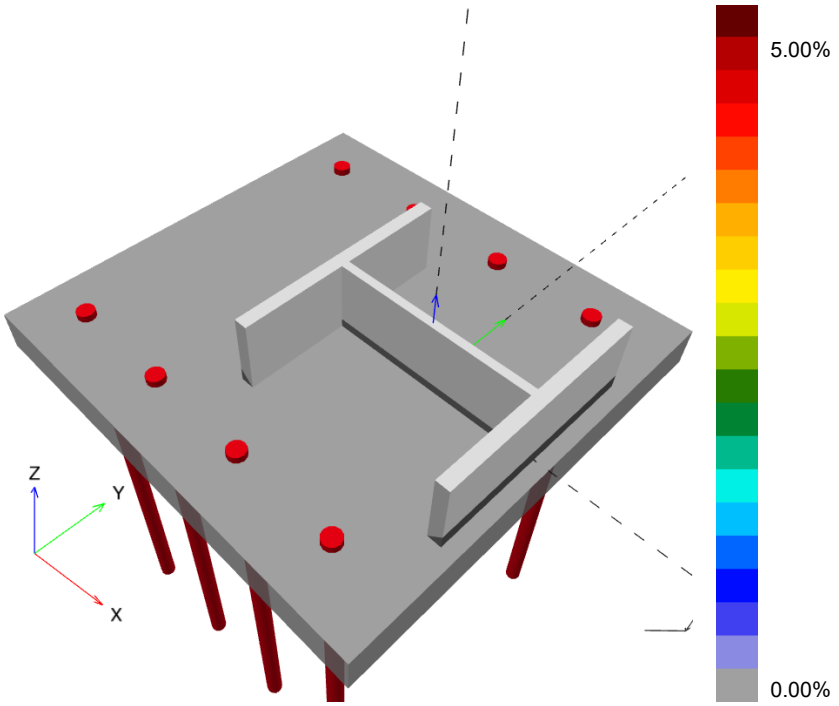


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Company:		Page:	12
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis BP Brace (FOH) - Grid F	Date:	1/31/2025
Fastening point:			

2.4.1.2 Plastic strain

Results below are displayed for the decisive load combination: 1 - Combination 1



2.4.2 Plate hole bearing resistance, AISC 360-16 Section J3

Decisive load combination: 1 - Combination 1

Equations

$R_n = \min(1.2\, l_c\, t\, F_u\, ,\ 2.4\, d\, t\, F_u)$ (AISC 360-16 J3-6a, c)

$\Phi R_n = 0.75\, R_n$

$V \leq \Phi R_n$

Variables

	l_c [in]	t [in]	F_u [psi]	d [in]	R_n [kip]
Anchor 1	2.588	2.250	65,000	1.000	351.000
Anchor 2	2.632	2.250	65,000	1.000	351.000
Anchor 3	13.700	2.250	65,000	1.000	351.000
Anchor 4	2.740	2.250	65,000	1.000	351.000
Anchor 5	22.297	2.250	65,000	1.000	351.000
Anchor 6	23.572	2.250	65,000	1.000	351.000
Anchor 7	26.388	2.250	65,000	1.000	351.000
Anchor 8	3.141	2.250	65,000	1.000	351.000

Results

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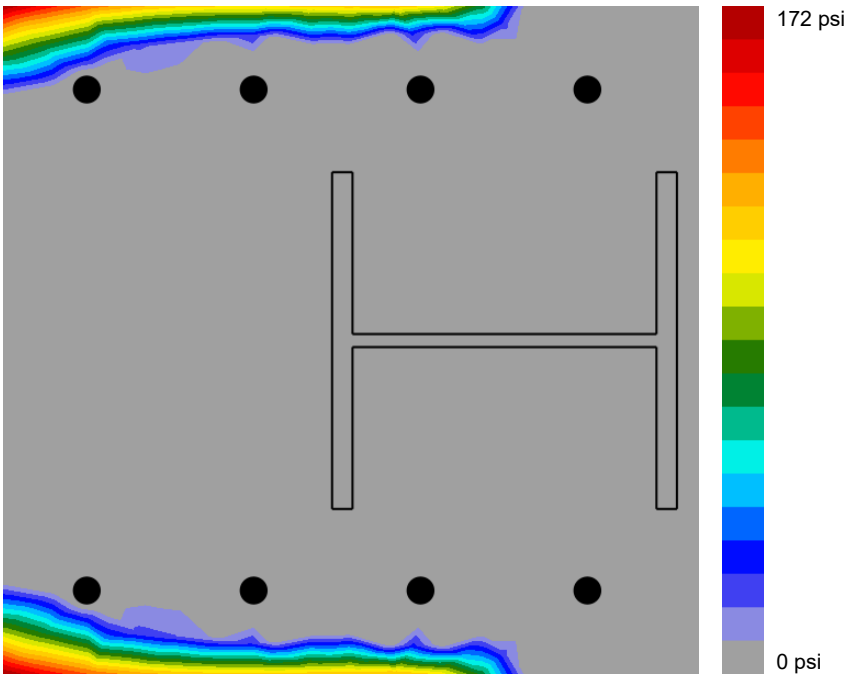
Company:		Page:	13
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis BP Brace (FOH) - Grid F	Date:	1/31/2025
Fastening point:			

	V [kip]	ΦR_n [kip]	Utilization [%]	Status
Anchor 1	1.487	263.250	1	OK
Anchor 2	1.165	263.250	1	OK
Anchor 3	1.139	263.250	1	OK
Anchor 4	0.888	263.250	1	OK
Anchor 5	0.977	263.250	1	OK
Anchor 6	0.778	263.250	1	OK
Anchor 7	0.628	263.250	1	OK
Anchor 8	0.602	263.250	1	OK

2.5 Concrete

Decisive load combination: 1 - Combination 1

2.5.1 Compression in concrete under the anchor plate



2.5.2 Concrete block compressive strength resistance check, AISC 360-16 Section J8

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	14
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis BP Brace (FOH) - Grid F	Date:	1/31/2025
Fastening point:			

Equations

$$F_p = \Phi f_{p,max}$$

$$f_{p,max} = 0.85 f_c' \sqrt{\left(\frac{A_2}{A_1}\right)} \le 1.7 f_c; \quad \sqrt{\left(\frac{A_2}{A_1}\right)} \le 2$$

$$\sigma = \frac{N}{A_1}$$

$$\text{Utilization} = \frac{\sigma}{F_p}$$

Variables

N [kip]	f_c' [psi]	Φ	A_1 [in²]	A_2 [in²]
4.028	4,500	0.65	54.62	3,061.73

Results

Load combination	F_p [psi]	σ [psi]	Utilization [%]	Status
Combination 1	4,973	74	2	OK

2.6 Symbol explanation

A_1	Loaded area of concrete
A_2	Supporting area
d	Nominal diameter of the bolt
ε_{lim}	Limit plastic strain
ε_{Pl}	Plastic strain from CBFEM results
f_c	Concrete compressive strength
f_c'	Concrete compressive strength
F_u	Specified minimum tensile strength of the connected material
F_p	Concrete block design bearing strength
$f_{p,max}$	Concrete block design bearing strength maximum
f_y	Yield strength
l_c	Clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material
N	Resulting compression force
σ	Average stress in concrete
σ_{Ed}	Equivalent stress
Φ	Resistance factor
ΦR_n	Factored resistance
t	Thickness of the anchor plate
V	Resultant of shear forces Vy, Vz in bolt.

2.7 Warnings

- By using the CBFEM calculation functionality of PROFIS Engineering you may act outside the applicable design codes and your specified anchor plate may not behave rigid. Please, validate the results with a professional designer and/or structural engineer to ensure suitability and adequacy for your specific jurisdiction and project requirements.
- The anchor is modeled considering stiffness values determined from load displacement curves tested in an independent laboratory. Please note that no simple replacement of the anchor is possible as the anchor stiffness has a major impact on the load distribution results.

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Company:		Page:	15
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis BP Brace (FOH) - Grid F	Date:	1/31/2025
Fastening point:			

3 Summary of results

Design of the anchor plate, anchors, welds and other elements are based on CBFEM (component based finite element method) and AISC.

	Load combination	Max. utilization	Status
Anchors	Combination 1	56%	OK
Anchor plate	Combination 1	18%	OK
Concrete	Combination 1	2%	OK
Profile	Combination 1	38%	OK

Fastening meets the design criteria!

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Company:		Page:	16
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis BP Brace (FOH) - Grid F	Date:	1/31/2025
Fastening point:			

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Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP Brace (size)	Date:	2/2/2025
Fastening point:			

Specifier's comments:

1 Anchor Design

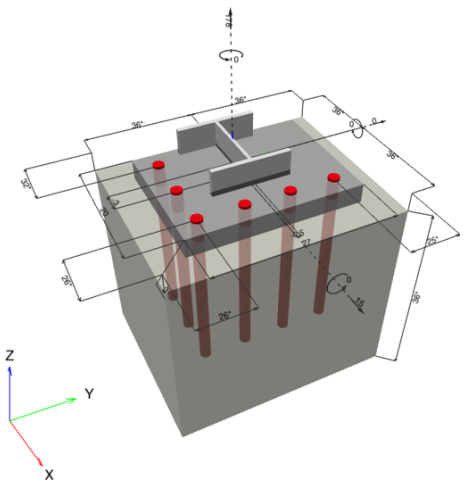
1.1 Input data



Anchor type and diameter:	**Heavy Hex Head ASTM F 1554 GR. 105 1 3/4**
Item number:	not available
Specification text:	∅ 1 3/4 in Heavy Hex Head ASTM F 1554 GR. 105 with 24 in nominal embedment depth per Technical data , cast in place installation per MPII
Effective embedment depth:	h_{ef} = 24.000 in.
Material:	ASTM F 1554
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-19 / CIP
Shear edge breakout verification:	Row closest to edge (Case 3 only from ACI 318-19 Fig. R.17.7.2.1b)
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 3.000 in.
Anchor plate[CBFEM] :	l_x x l_y x t = 20.000 in. x 27.000 in. x 3.000 in.;
Profile:	W shape (AISC), W12X87; (L x W x T x FT) = 12.500 in. x 12.100 in. x 0.515 in. x 0.810 in.
Base material:	cracked concrete, Custom, f_c' = 4,500 psi; h = 36.000 in.
Reinforcement:	tension: present, shear: present; edge reinforcement: none or < No. 4 bar

[CBFEM] - The anchor calculation is based on a component-based Finite Element Method (CBFEM)

Geometry [in.] & Loading [kip, ft.kip]



Input data and results must be checked for conformity with the existing conditions and for plausibility!
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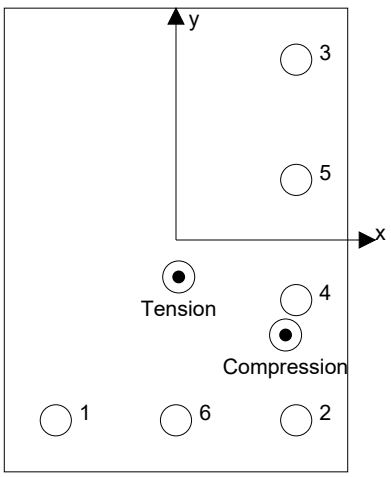
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Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP Brace (size)	Date:	2/2/2025
Fastening point:			

1.1.1 Design results

Case	Description	Forces [kip] / Moments [ft.kip]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 178.000; V_x = 15.000; V_y = 0.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	195

1.2 Load case/Resulting anchor forces



Anchor reactions [kip]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	127.229	2.389	1.863	1.496
2	-0.005	2.135	2.089	-0.439
3	66.281	3.046	3.011	-0.458
4	11.452	2.893	2.859	-0.438
5	55.572	3.152	3.116	-0.471
6	8.964	2.084	2.061	0.311

Resulting tension force in (x/y)=(0.158/-2.151): 269.493 [kip]
Resulting compression force in (x/y)=(6.382/-5.531): 94.806 [kip]

Anchor forces are calculated based on a component-based Finite Element Method (CBFEM)

1.3 Tension load

	Load N_{ua} [kip]	Capacity ϕ N_n [kip]	Utilization $\beta_N = N_{ua}/\phi N_n$		Status
Steel Strength*	127.229	178.125	72		OK
Pullout Strength*	127.229	104.429	122	31.7	not recommended
Concrete Breakout Failure**	269.499	138.593	195	27.8	not recommended
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A		N/A

* highest loaded anchor **anchor group (anchors in tension)

SEE SUPPLEMENTAL CALCULATIONS

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Company:		Page:	3
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP Brace (size)	Date:	2/2/2025
Fastening point:			

1.3.1 Steel Strength

$N_{sa} = A_{se,N} f_{uta}$ ACI 318-19 Eq. (17.6.1.2)
$\phi N_{sa} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
1.90	125,001

Calculations

N_{sa} [kip]
237.500

Results

N_{sa} [kip]	ϕ_{steel}	ϕN_{sa} [kip]	N_{ua} [kip]
237.500	0.750	178.125	127.229

1.3.2 Pullout Strength

$N_{pN} = \psi_{c,p} N_p$ ACI 318-19 Eq. (17.6.3.1)
$N_p = 8 A_{brg} f_c'$ ACI 318-19 Eq. (17.6.3.2.2a)
$\phi N_{pN} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$\psi_{c,p}$	A_{brg} [in.^{2}]	λ_a	f_c' [psi]
1.000	4.14	1.000	4,500

Calculations

N_p [kip]
149.184

Results

N_{pn} [kip]	$\phi_{concrete}$	ϕN_{pn} [kip]	N_{ua} [kip]
149.184	0.700	104.429	127.229

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Company:		Page:	4
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP Brace (size)	Date:	2/2/2025
Fastening point:			

1.3.3 Concrete Breakout Failure

$N_{cbg} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ec,N}\ \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b$ — ACI 318-19 Eq. (17.6.2.1b)

$\phi\ N_{cbg} \geq N_{ua}$ — ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9\ h_{ef}^2$ — ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\ e'_N}{3\ h_{ef}}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = 16\ \lambda_a\ \sqrt{f'_c}\ h_{ef}^{5/3}$ — ACI 318-19 Eq. (17.6.2.2.3)

Variables

h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
21.333	2.642	0.051	25.000	1.000

c_{ac} [in.]	k_c	λ_a	f'_c [psi]
-	16	1.000	4,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [kip]
4,987.00	4,096.00	0.924	0.998	0.934	1.000	176.126

Results

N_{cbg} [kip]	$\phi_{concrete}$	$\phi\ N_{cbg}$ [kip]	N_{ua} [kip]
184.790	0.750	138.593	269.499

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Company:		Page:	5
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP Brace (size)	Date:	2/2/2025
Fastening point:			

1.4 Shear load

	Load V_{ua} [kip]	Capacity ϕ V_n [kip]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	3.152	92.625	4	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	15.000	256.520	6	OK
Concrete edge failure in direction y+**	15.108	42.634	36	OK

* highest loaded anchor **anchor group (relevant anchors)

1.4.1 Steel Strength

$V_{sa} = 0.6\ A_{se,V}\ f_{uta}$ ACI 318-19 Eq. (17.7.1.2b)
$\phi\ V_{steel} \geq V_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
1.90	125,001

Calculations

V_{sa} [kip]
142.500

Results

V_{sa} [kip]	ϕ_{steel}	$\phi\ V_{sa}$ [kip]	V_{ua} [kip]
142.500	0.650	92.625	3.152

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Company:		Page:	6
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP Brace (size)	Date:	2/2/2025
Fastening point:			

1.4.2 Pryout Strength

$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \right]$ ACI 318-19 Eq. (17.7.3.1b)

$\phi V_{cpg} \geq V_{ua}$ ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9 h_{ef}^2$ ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e'_N}{3 h_{ef}}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = 16 \lambda_a \sqrt{f'_c} h_{ef}^{5/3}$ ACI 318-19 Eq. (17.6.2.2.3)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	21.333	0.000	3.000	25.000

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.000	-	16	1.000	4,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [kip]
4,988.00	4,096.00	1.000	0.914	0.934	1.000	176.126

Results

V_{cpg} [kip]	$\phi_{concrete}$	ϕV_{cpg} [kip]	V_{ua} [kip]
366.457	0.700	256.520	15.000

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Company:		Page:	7
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP Brace (size)	Date:	2/2/2025
Fastening point:			

1.4.3 Concrete edge failure in direction y+

$V_{cb} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \psi_{ed,V} \psi_{c,V} \psi_{h,V} \psi_{parallel,V} V_b$ — ACI 318-19 Eq. (17.7.2.1a)

$\phi V_{cb} \geq V_{ua}$ — ACI 318-19 Table 17.5.2

A_{Vc} see ACI 318-19, Section 17.7.2.1, Fig. R 17.7.2.1(b)*

$A_{Vc0} = 4.5 c_{a1}^2$ — ACI 318-19 Eq. (17.7.2.1.3)

$\psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.4.1b)

$\psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ — ACI 318-19 Eq. (17.7.2.6.1)

$V_b = 9 \lambda_a \sqrt{f'_c} c_{a1}^{1.5}$ — ACI 318-19 Eq. (17.7.2.2.1b)

Variables

c_{a1} [in.]	c_{a2} [in.]	$\psi_{c,V}$	h_a [in.]	l_e [in.]
25.000	26.000	1.000	36.000	14.000

λ_a	d_a [in.]	f'_c [psi]	$\psi_{parallel,V}$
1.000	1.750	4,500	1.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\psi_{ed,V}$	$\psi_{h,V}$	V_b [kip]
2,286.00	2,812.50	0.908	1.021	75.467

Results

V_{cb} [kip]	$\phi_{concrete}$	ϕV_{cb} [kip]	V_{ua} [kip]
56.845	0.750	42.634	15.108

*Anchor row defined by: Anchor 3; Case 3 controls

1.5 Combined tension and shear loads, per ACI 318-19 section 17.8

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
~~1.945~~ 0.72	0.354	1.000	~~192~~ 89.5	not recommended

$\beta_{NV} = (\beta_N + \beta_V) / 1.2 <= 1$

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Company:		Page:	8
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP Brace (size)	Date:	2/2/2025
Fastening point:			

1.6 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates as per current regulations (ETAG 001/Annex C, EOTA TR029, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid base plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- The equations presented in this report are based on imperial units. When inputs are displayed in metric units, the user should be aware that the equations remain in their imperial format.
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- For additional information about ACI 318 strength design provisions, please go to https://submittals.us.hilti.com/PROFISAnchorDesignGuide/
- Attention! In case of compressive anchor forces a buckling check as well as the proof of the local load transfer into and within the base material (incl. punching) has to be done separately.
- The anchor design methods in PROFIS Engineering require rigid anchor plates, as per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means that the anchor plate should be sufficiently rigid to prevent load re-distribution to the anchors due to elastic/plastic displacements. The user accepts that the anchor plate is considered close to rigid by engineering judgment."

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Company:		Page:	9
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP Brace (size)	Date:	2/2/2025
Fastening point:			

1.7 Installation data

Profile: W shape (AISC), W12X87; (L x W x T x FT) = 12.500 in. x 12.100 in. x 0.515 in. x 0.810 in.

Hole diameter in the fixture: d_f = 1.812 in.

Plate thickness (input): 3.000 in.

Anchor type and diameter: Heavy Hex Head ASTM F 1554 GR. 105 1 3/4

Item number: not available

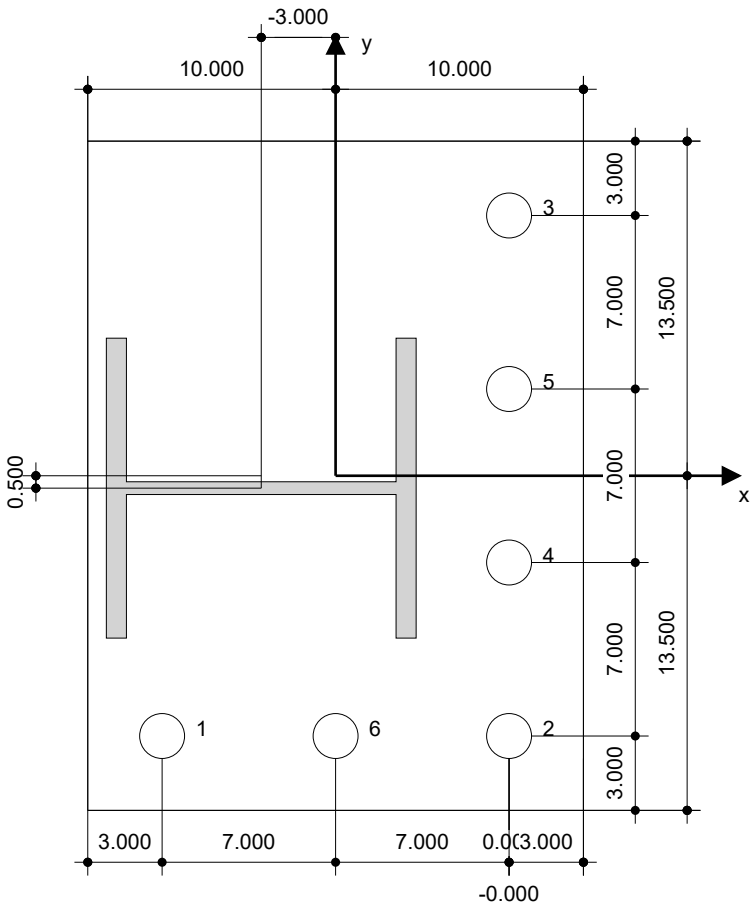
Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 24.000 in.

Minimum thickness of the base material: 25.656 in.

⌀ 1 3/4 in Heavy Hex Head ASTM F 1554 GR. 105 with 24 in nominal embedment depth per Technical data , cast in place installation per MPII



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-7.000	-10.500	32.000	40.000	26.000	46.000
2	7.000	-10.500	46.000	26.000	26.000	46.000
3	7.000	10.500	46.000	26.000	47.000	25.000
4	7.000	-3.500	46.000	26.000	33.000	39.000
5	7.000	3.500	46.000	26.000	40.000	32.000
6	0.000	-10.500	39.000	33.000	26.000	46.000

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Company:		Page:	10
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP Brace (size)	Date:	2/2/2025
Fastening point:			

2 Anchor plate design

2.1 Input data

Anchor plate:	Shape: Rectangular l_x x l_y x t = 20.000 in x 27.000 in x 3.000 in Calculation: CBFEM Material: ASTM A572 Gr.50; F_y = 50,000 psi; ε_{lim} = 5.00%
Anchor type and size:	Heavy Hex Head ASTM F 1554 GR. 105 1 3/4, h_{ef} = 24.000 in
Anchor stiffness:	The anchor is modeled considering stiffness values determined from load displacement curves tested in an independent laboratory. Please note that no simple replacement of the anchor is possible as the anchor stiffness has a major impact on the load distribution results.
Design method:	AISC and LRFD-based design using component-based FEM
Stand-off installation:	e_b = 0.000 in (No stand-off); t = 3.000 in
Profile:	W12X87; (L x W x T x FT) = 12.500 in x 12.100 in x 0.515 in x 0.810 in Material: ASTM A992; F_y = 50,000 psi; ε_{lim} = 5.00% Eccentricity x: -3.000 in Eccentricity y: -0.500 in
Base material:	Cracked concrete; Custom; $f_{c,cyl}$ = 4,500 psi; h = 36.000 in; E = 4,351,132 psi; G = 1,740,453 psi; v = 0.20; D = 156.07 lb/ft³
Welds (profile to anchor plate):	Type of redistribution: Plastic Material: E70xx
Mesh size:	Number of elements on edge: 8 Min. size of element: 0.394 in Max. size of element: 1.969 in

2.2 Summary

	Description	Profile		Anchor plate			Concrete [%]
		σ_{Ed} [psi]	ε_{Pl} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Hole bearing [%]	
1	Combination 1	44,794	0.04	32,613	0.00	1	18

2.3 Anchor plate classification

Results below are displayed for the decisive load combinations: Combination 1

Anchor tension forces	Equivalent rigid anchor plate (CBFEM)	Component-based Finite Element Method (CBFEM) anchor plate design
Anchor 1	119.533 kip	127.229 kip
Anchor 2	-0.005 kip	-0.005 kip
Anchor 3	76.560 kip	66.281 kip
Anchor 4	-0.002 kip	11.452 kip
Anchor 5	19.743 kip	55.571 kip
Anchor 6	12.818 kip	8.964 kip

User accepted to consider the selected anchor plate as rigid by his/her engineering judgement. This means the anchor design guidelines can be applied.

2.4 Profile/Stiffeners/Plate

Profile and stiffeners are verified at the level of the steel to concrete connection. The connection design does not replace the steel design for critical cross sections, which should be performed outside of PROFIS Engineering.

2.4.1 Equivalent stress and plastic strain

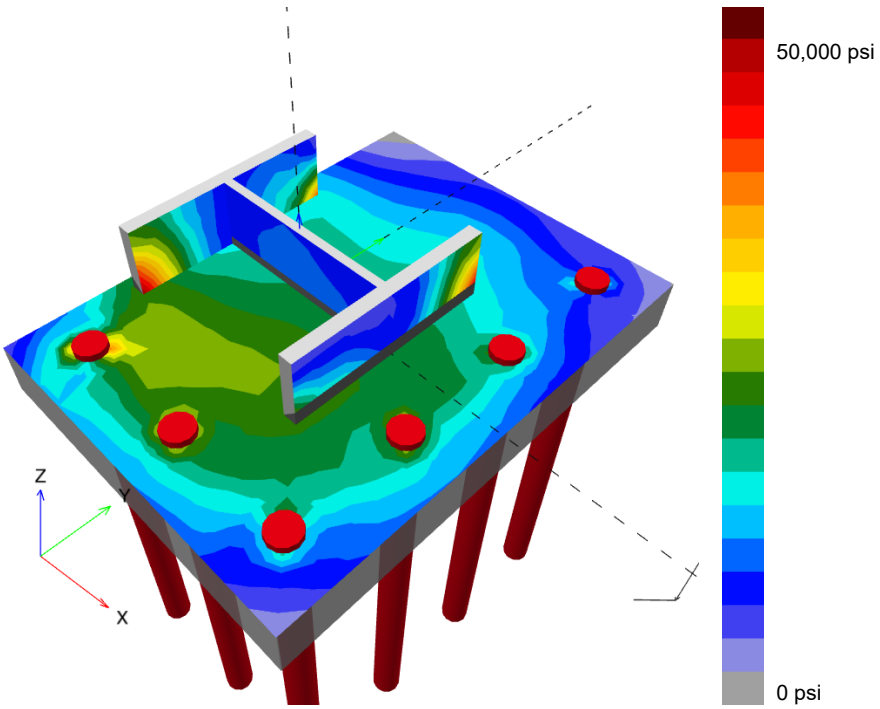
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Company:		Page:	11
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP Brace (size)	Date:	2/2/2025
Fastening point:			

Part	Load combination	Material	f_y [psi]	ε_{lim} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Status
Plate	Combination 1	ASTM A572 Gr.50	50,000	5.00	32,613	0.00	OK
Profile	Combination 1	ASTM A992	50,000	5.00	42,311	0.00	OK
Profile	Combination 1	ASTM A992	50,000	5.00	44,794	0.04	OK
Profile	Combination 1	ASTM A992	50,000	5.00	12,487	0.00	OK

2.4.1.1 Equivalent stress

Results below are displayed for the decisive load combination: 1 - Combination 1

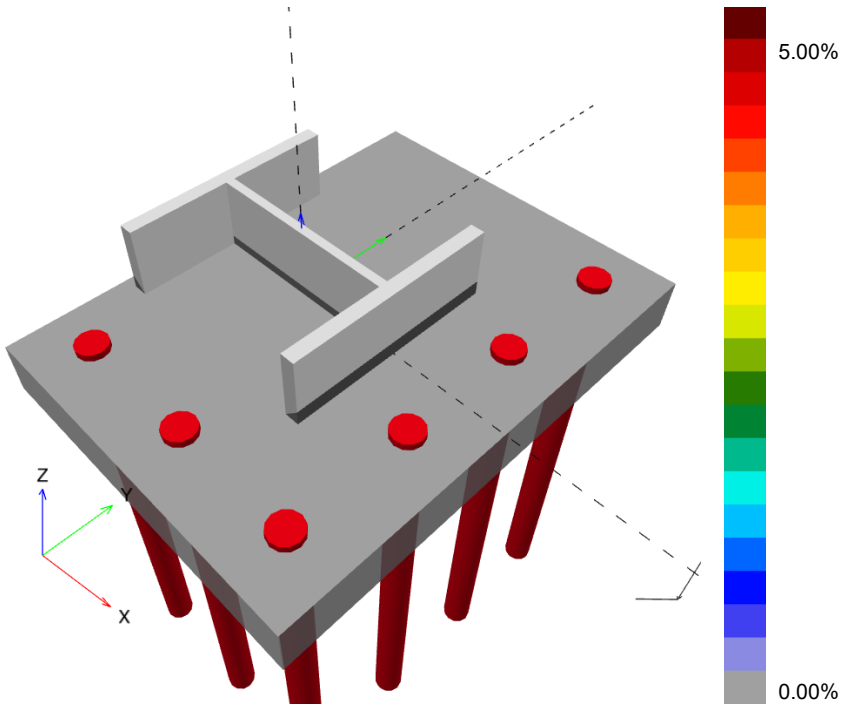


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Company:		Page:	12
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP Brace (size)	Date:	2/2/2025
Fastening point:			

2.4.1.2 Plastic strain

Results below are displayed for the decisive load combination: 1 - Combination 1



2.4.2 Plate hole bearing resistance, AISC 360-16 Section J3

Decisive load combination: 1 - Combination 1

Equations

$R_n = \min(1.2\, l_c\, t\, F_u\, ,\ 2.4\, d\, t\, F_u)$ (AISC 360-16 J3-6a, c)

$\Phi R_n = 0.75\, R_n$

$V \leq \Phi R_n$

Variables

	l_c [in]	t [in]	F_u [psi]	d [in]	R_n [kip]
Anchor 1	2.941	3.000	65,000	1.750	688.291
Anchor 2	5.188	3.000	65,000	1.750	818.999
Anchor 3	16.289	3.000	65,000	1.750	818.999
Anchor 4	16.292	3.000	65,000	1.750	818.999
Anchor 5	16.287	3.000	65,000	1.750	818.999
Anchor 6	5.188	3.000	65,000	1.750	818.999

Results

	V [kip]	ΦR_n [kip]	Utilization [%]	Status
Anchor 1	2.389	516.218	1	OK

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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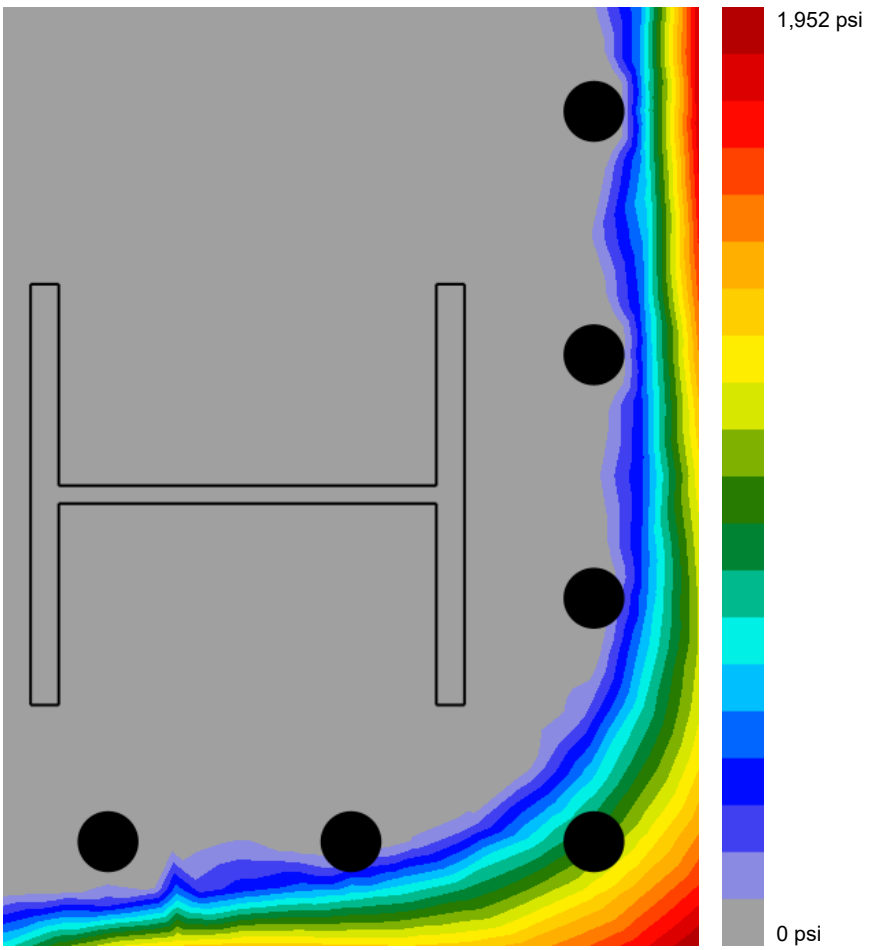
Company:		Page:	13
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP Brace (size)	Date:	2/2/2025
Fastening point:			

	V [kip]	ΦR_n [kip]	Utilization [%]	Status
Anchor 2	2.135	614.250	1	OK
Anchor 3	3.046	614.250	1	OK
Anchor 4	2.893	614.250	1	OK
Anchor 5	3.152	614.250	1	OK
Anchor 6	2.084	614.250	1	OK

2.5 Concrete

Decisive load combination: 1 - Combination 1

2.5.1 Compression in concrete under the anchor plate



2.5.2 Concrete block compressive strength resistance check, AISC 360-16 Section J8

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	14
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP Brace (size)	Date:	2/2/2025
Fastening point:			

Equations

$$F_p = \Phi f_{p,max}$$

$$f_{p,max} = 0.85 f_c' \sqrt{\left(\frac{A_2}{A_1}\right)} \le 1.7 f_c; \quad \sqrt{\left(\frac{A_2}{A_1}\right)} \le 2$$

$$\sigma = \frac{N}{A_1}$$

$$\text{Utilization} = \frac{\sigma}{F_p}$$

Variables

N [kip]	f_c' [psi]	Φ	A_1 [in²]	A_2 [in²]
94.806	4,500	0.65	111.71	3,680.74

Results

Load combination	F_p [psi]	σ [psi]	Utilization [%]	Status
Combination 1	4,973	849	18	OK

2.6 Symbol explanation

A_1	Loaded area of concrete
A_2	Supporting area
d	Nominal diameter of the bolt
ε_{lim}	Limit plastic strain
ε_{Pl}	Plastic strain from CBFEM results
f_c	Concrete compressive strength
f_c'	Concrete compressive strength
F_u	Specified minimum tensile strength of the connected material
F_p	Concrete block design bearing strength
$f_{p,max}$	Concrete block design bearing strength maximum
f_y	Yield strength
l_c	Clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material
N	Resulting compression force
σ	Average stress in concrete
σ_{Ed}	Equivalent stress
Φ	Resistance factor
ΦR_n	Factored resistance
t	Thickness of the anchor plate
V	Resultant of shear forces Vy, Vz in bolt.

2.7 Warnings

- By using the CBFEM calculation functionality of PROFIS Engineering you may act outside the applicable design codes and your specified anchor plate may not behave rigid. Please, validate the results with a professional designer and/or structural engineer to ensure suitability and adequacy for your specific jurisdiction and project requirements.
- The anchor is modeled considering stiffness values determined from load displacement curves tested in an independent laboratory. Please note that no simple replacement of the anchor is possible as the anchor stiffness has a major impact on the load distribution results.

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	15
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP Brace (size)	Date:	2/2/2025
Fastening point:			

3 Summary of results

Design of the anchor plate, anchors, welds and other elements are based on CBFEM (component based finite element method) and AISC.

	Load combination	Max. utilization	Status
Anchors	Combination 1	195%	NOT OK
Anchor plate	Combination 1	66%	OK
Concrete	Combination 1	18%	OK
Profile	Combination 1	90%	OK

Fastening does not meet the design criteria!

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Company:		Page:	16
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Yellow BP Brace (size)	Date:	2/2/2025
Fastening point:			

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Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Weak Axis DH BP Brace (size)	Date:	4/24/2025
Fastening point:			

Specifier's comments:

1 Anchor Design

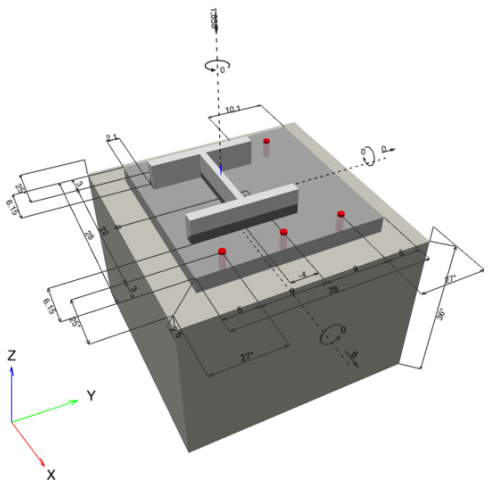
1.1 Input data



Anchor type and diameter:	**Heavy Hex Head ASTM F 1554 GR. 105 1**
Item number:	not available
Specification text:	∅ 1 in Heavy Hex Head ASTM F 1554 GR. 105 with 18 in nominal embedment depth per Technical data , cast in place installation per MPII,
Effective embedment depth:	h_{ef} = 18.000 in.
Material:	ASTM F 1554
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-19 / CIP
Shear edge breakout verification:	Row closest to edge (Case 3 only from ACI 318-19 Fig. R.17.7.2.1b)
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 2.500 in.
Anchor plate[CBFEM] :	l_x x l_y x t = 28.000 in. x 28.000 in. x 2.500 in.;
Profile:	W shape (AISC), W14X211; (L x W x T x FT) = 15.700 in. x 15.800 in. x 0.980 in. x 1.560 in.
Base material:	cracked concrete, Custom, f_c' = 4,500 psi; h = 36.000 in.
Reinforcement:	tension: present, shear: not present; edge reinforcement: none or < No. 4 bar

[CBFEM] - The anchor calculation is based on a component-based Finite Element Method (CBFEM)

Geometry [in.] & Loading [kip, ft.kip]



Input data and results must be checked for conformity with the existing conditions and for plausibility!
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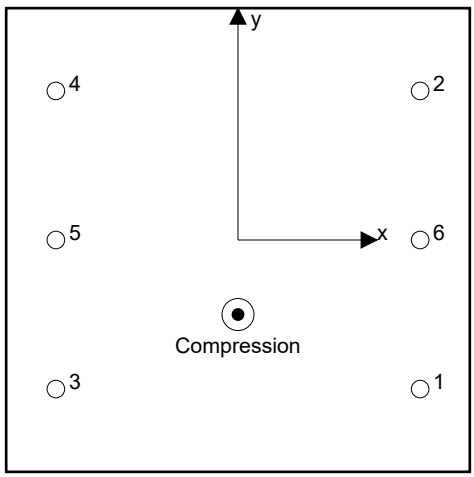
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Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Weak Axis DH BP Brace (size)	Date:	4/24/2025
Fastening point:			

1.1.1 Design results

Case	Description	Forces [kip] / Moments [ft.kip]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = -1,858.000; V_x = 8.000; V_y = 0.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	19

1.2 Load case/Resulting anchor forces



Anchor reactions [kip]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	-0.016	1.731	1.705	-0.295
2	-0.002	1.357	1.105	0.787
3	-0.016	1.746	1.514	-0.868
4	-0.002	0.976	0.973	0.078
5	-0.014	1.332	1.317	-0.197
6	-0.014	1.472	1.386	0.496

Resulting tension force in (x/y)=(-/-): 0.000 [kip]
Resulting compression force in (x/y)=(0.000/-4.500): 1,859.615 [kip]

Anchor forces are calculated based on a component-based Finite Element Method (CBFEM)

1.3 Tension load

	Load N_{ua} [kip]	Capacity ϕ N_n [kip]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	-0.016	56.812	1	OK
Pullout Strength*	-0.002	37.825	-0	not recommended
Concrete Breakout Failure**	N/A	N/A	N/A	N/A
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (anchors in tension)

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	3
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Weak Axis DH BP Brace (size)	Date:	4/24/2025
Fastening point:			

1.3.1 Steel Strength

$N_{sa} = A_{se,N} f_{uta}$ ACI 318-19 Eq. (17.6.1.2)
$\phi N_{sa} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.61	125,001

Calculations

N_{sa} [kip]
75.750

Results

N_{sa} [kip]	ϕ_{steel}	ϕN_{sa} [kip]	N_{ua} [kip]
75.750	0.750	56.812	-0.016

The steel proof was done for the highest absolute force per anchor - in this case compression loading. Please be aware that buckling should be verified separately

1.3.2 Pullout Strength

$N_{pN} = \psi_{c,p} N_p$ ACI 318-19 Eq. (17.6.3.1)
$N_p = 8 A_{brg} f'_c$ ACI 318-19 Eq. (17.6.3.2.2a)
$\phi N_{pN} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$\psi_{c,p}$	A_{brg} [in.^{2}]	λ_a	f'_c [psi]
1.000	1.50	1.000	4,500

Calculations

N_p [kip]
54.036

Results

N_{pn} [kip]	$\phi_{concrete}$	ϕN_{pn} [kip]	N_{ua} [kip]
54.036	0.700	37.825	-0.002

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Company:		Page:	4
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Weak Axis DH BP Brace (size)	Date:	4/24/2025
Fastening point:			

1.4 Shear load

	Load V_{ua} [kip]	Capacity ϕ V_n [kip]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	1.746	29.542	6	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	8.000	281.251	3	OK
Concrete edge failure in direction y+**	8.115	43.176	19	OK

* highest loaded anchor **anchor group (relevant anchors)

1.4.1 Steel Strength

$V_{sa} = 0.6\ A_{se,V}\ f_{uta}$ ACI 318-19 Eq. (17.7.1.2b)
$\phi\ V_{steel} \geq V_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.61	125,001

Calculations

V_{sa} [kip]
45.450

Results

V_{sa} [kip]	ϕ_{steel}	$\phi\ V_{sa}$ [kip]	V_{ua} [kip]
45.450	0.650	29.542	1.746

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	5
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Weak Axis DH BP Brace (size)	Date:	4/24/2025
Fastening point:			

1.4.2 Pryout Strength

$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \right]$ — ACI 318-19 Eq. (17.7.3.1b)

$\phi V_{cpg} \geq V_{ua}$ — ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9 h_{ef}^2$ — ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e'_N}{3 h_{ef}}} \right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = 16 \lambda_a \sqrt{f'_c} h_{ef}^{5/3}$ — ACI 318-19 Eq. (17.6.2.2.3)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	18.000	0.000	4.000	25.000

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.000	-	16	1.000	4,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [kip]
5,184.00	2,916.00	1.000	0.871	0.978	1.000	132.693

Results

V_{cpg} [kip]	$\phi_{concrete}$	ϕV_{cpg} [kip]	V_{ua} [kip]
401.788	0.700	281.251	8.000

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Company:		Page:	6
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Weak Axis DH BP Brace (size)	Date:	4/24/2025
Fastening point:			

1.4.3 Concrete edge failure in direction y+

$V_{cbg} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \Psi_{ec,V} \Psi_{ed,V} \Psi_{c,V} \Psi_{h,V} \Psi_{parallel,V} V_b$ — ACI 318-19 Eq. (17.7.2.1b)

$\phi V_{cbg} \geq V_{ua}$ — ACI 318-19 Table 17.5.2

A_{Vc} see ACI 318-19, Section 17.7.2.1, Fig. R 17.7.2.1(b)*

$A_{Vc0} = 4.5 c_{a1}^2$ — ACI 318-19 Eq. (17.7.2.1.3)

$\Psi_{ec,V} = \left(\frac{1}{1 + \frac{e'_v}{1.5c_{a1}}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.3.1)

$\Psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.4.1b)

$\Psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ — ACI 318-19 Eq. (17.7.2.6.1)

$V_b = 9 \lambda_a \sqrt{f'_c} c_{a1}^{1.5}$ — ACI 318-19 Eq. (17.7.2.2.1b)

Variables

c_{a1} [in.]	c_{a2} [in.]	e_{cV} [in.]	$\Psi_{c,V}$	h_a [in.]
24.000	25.000	1.633	1.000	36.000

l_e [in.]	λ_a	d_a [in.]	f'_c [psi]	$\Psi_{parallel,V}$
8.000	1.000	1.000	4,500	1.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{h,V}$	V_b [kip]
2,592.00	2,592.00	0.957	0.908	1.000	70.985

Results

V_{cbg} [kip]	$\phi_{concrete}$	ϕV_{cbg} [kip]	V_{ua} [kip]
61.680	0.700	43.176	8.115

*Anchor row defined by: Anchor 2, 4; Case 3 controls

1.5 Combined tension and shear loads, per ACI 318-19 section 17.8

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.000	0.188	5/3	7	OK

$\beta_{NV} = \beta_N^{\zeta} + \beta_V^{\zeta} <= 1$

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Company:		Page:	7
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Weak Axis DH BP Brace (size)	Date:	4/24/2025
Fastening point:			

1.6 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates as per current regulations (ETAG 001/Annex C, EOTA TR029, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid base plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- The equations presented in this report are based on imperial units. When inputs are displayed in metric units, the user should be aware that the equations remain in their imperial format.
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- For additional information about ACI 318 strength design provisions, please go to https://viewer.joomag.com/profis-design-guide-us-en-summer-2021/0841849001625154758?short&/
- Attention! In case of compressive anchor forces a buckling check as well as the proof of the local load transfer into and within the base material (incl. punching) has to be done separately.
- The anchor design methods in PROFIS Engineering require rigid anchor plates, as per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means that the anchor plate should be sufficiently rigid to prevent load re-distribution to the anchors due to elastic/plastic displacements. The user accepts that the anchor plate is considered close to rigid by engineering judgment."

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Company:		Page:	8
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Weak Axis DH BP Brace (size)	Date:	4/24/2025
Fastening point:			

1.7 Installation data

Profile: W shape (AISC), W14X211; (L x W x T x FT) = 15.700 in. x 15.800 in. x 0.980 in. x 1.560 in.

Hole diameter in the fixture: d_f = 1.062 in.

Plate thickness (input): 2.500 in.

Anchor type and diameter: Heavy Hex Head ASTM F 1554 GR. 105 1

Item number: not available

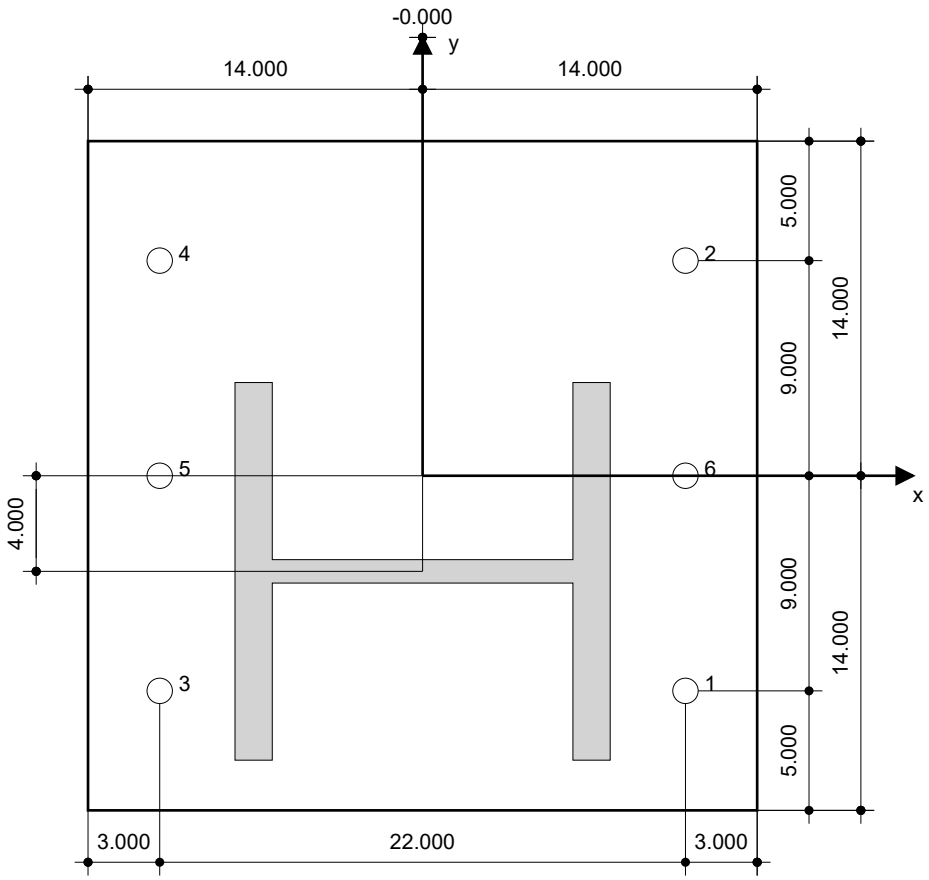
Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 18.000 in.

Minimum thickness of the base material: 19.172 in.

⌀ 1 in Heavy Hex Head ASTM F 1554 GR. 105 with 18 in nominal embedment depth per Technical data , cast in place installation per MPII



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	11.000	-9.000	47.000	25.000	27.000	45.000
2	11.000	9.000	47.000	25.000	45.000	27.000
3	-11.000	-9.000	25.000	47.000	27.000	45.000
4	-11.000	9.000	25.000	47.000	45.000	27.000
5	-11.000	-0.000	25.000	47.000	36.000	36.000
6	11.000	-0.000	47.000	25.000	36.000	36.000

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Company:		Page:	9
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Weak Axis DH BP Brace (size)	Date:	4/24/2025
Fastening point:			

2 Anchor plate design

2.1 Input data

Anchor plate:	Shape: Rectangular l_x x l_y x t = 28.000 in x 28.000 in x 2.500 in Calculation: CBFEM Material: ASTM A572 Gr.50; F_y = 50,000 psi; ε_{lim} = 5.00%
Anchor type and size:	Heavy Hex Head ASTM F 1554 GR. 105 1, h_{ef} = 18.000 in
Anchor stiffness:	The anchor is modeled considering stiffness values determined from load displacement curves tested in an independent laboratory. Please note that no simple replacement of the anchor is possible as the anchor stiffness has a major impact on the load distribution results.
Design method:	AISC and LRFD-based design using component-based FEM
Stand-off installation:	e_b = 0.000 in (No stand-off); t = 2.500 in
Profile:	W14X211; (L x W x T x FT) = 15.700 in x 15.800 in x 0.980 in x 1.560 in Material: ASTM A992; F_y = 50,000 psi; ε_{lim} = 5.00% Eccentricity x: -0.000 in Eccentricity y: -4.000 in
Base material:	Cracked concrete; Custom; $f_{c,cyl}$ = 4,500 psi; h = 36.000 in; E = 4,351,132 psi; G = 1,740,453 psi; v = 0.20; D = 156.07 lb/ft³
Welds (profile to anchor plate):	Type of redistribution: Plastic Material: E70xx
Mesh size:	Number of elements on edge: 8 Min. size of element: 0.394 in Max. size of element: 1.969 in

2.2 Summary

	Description	Profile		Anchor plate			Concrete [%]
		σ_{Ed} [psi]	ε_{Pl} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Hole bearing [%]	
1	Combination 1	50,039	0.13	30,222	0.00	1	58

2.3 Anchor plate classification

Results below are displayed for the decisive load combinations: Combination 1

Anchor tension forces	Equivalent rigid anchor plate (CBFEM)	Component-based Finite Element Method (CBFEM) anchor plate design
Anchor 1	-0.020 kip	-0.016 kip
Anchor 2	-0.006 kip	-0.002 kip
Anchor 3	-0.020 kip	-0.016 kip
Anchor 4	-0.006 kip	-0.002 kip
Anchor 5	-0.013 kip	-0.014 kip
Anchor 6	-0.013 kip	-0.014 kip

User accepted to consider the selected anchor plate as rigid by his/her engineering judgement. This means the anchor design guidelines can be applied.

2.4 Profile/Stiffeners/Plate

Profile and stiffeners are verified at the level of the steel to concrete connection. The connection design does not replace the steel design for critical cross sections, which should be performed outside of PROFIS Engineering.

2.4.1 Equivalent stress and plastic strain

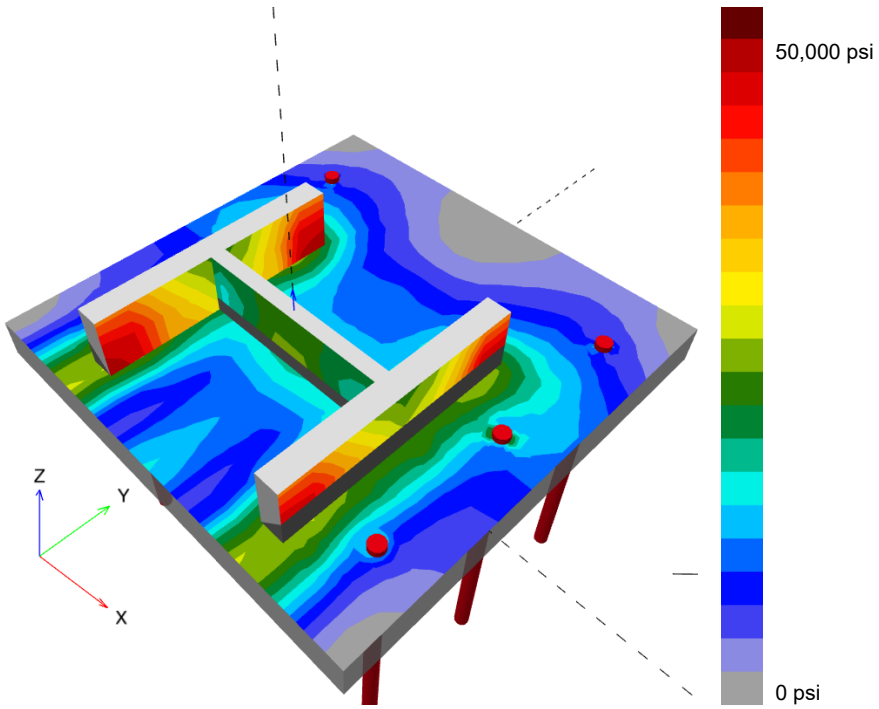
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Company:		Page:	10
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Weak Axis DH BP Brace (size)	Date:	4/24/2025
Fastening point:			

Part	Load combination	Material	f_y [psi]	ε_{lim} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Status
Plate	Combination 1	ASTM A572 Gr.50	50,000	5.00	30,222	0.00	OK
Profile	Combination 1	ASTM A992	50,000	5.00	50,039	0.13	OK
Profile	Combination 1	ASTM A992	50,000	5.00	50,038	0.13	OK
Profile	Combination 1	ASTM A992	50,000	5.00	29,937	0.00	OK

2.4.1.1 Equivalent stress

Results below are displayed for the decisive load combination: 1 - Combination 1

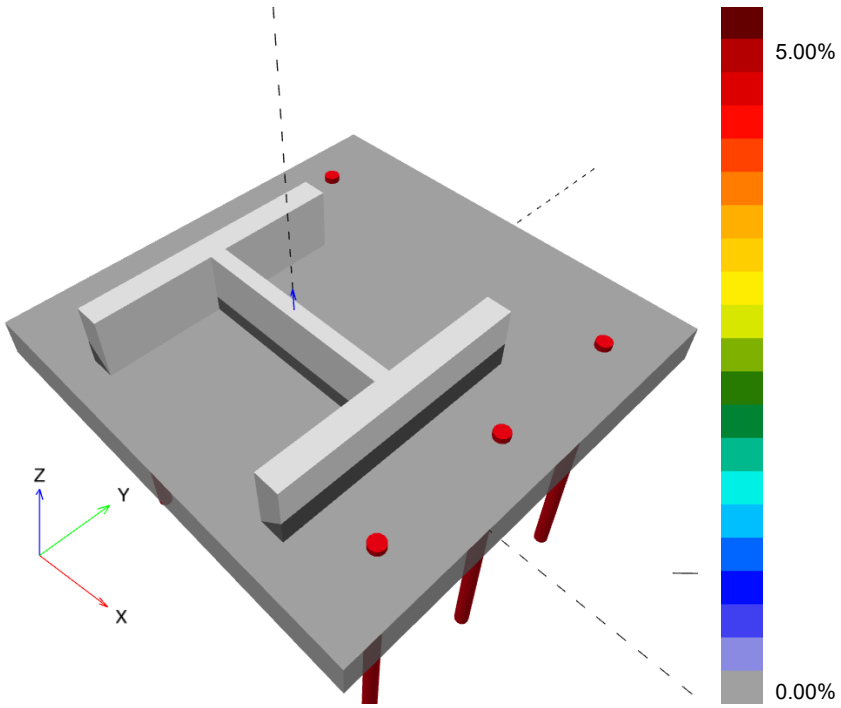


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Company:		Page:	11
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Weak Axis DH BP Brace (size)	Date:	4/24/2025
Fastening point:			

2.4.1.2 Plastic strain

Results below are displayed for the decisive load combination: 1 - Combination 1



2.4.2 Plate hole bearing resistance, AISC 360-16 Section J3

Decisive load combination: 1 - Combination 1

Equations

$R_n = \min(1.2\, l_c\, t\, F_u\, ,\ 2.4\, d\, t\, F_u)$ (AISC 360-16 J3-6a, c)

$\Phi R_n = 0.75\, R_n$

$V \leq \Phi R_n$

Variables

	l_c [in]	t [in]	F_u [psi]	d [in]	R_n [kip]
Anchor 1	24.841	2.500	65,000	1.000	390.000
Anchor 2	30.161	2.500	65,000	1.000	390.000
Anchor 3	2.927	2.500	65,000	1.000	390.000
Anchor 4	2.478	2.500	65,000	1.000	390.000
Anchor 5	2.502	2.500	65,000	1.000	390.000
Anchor 6	22.707	2.500	65,000	1.000	390.000

Results

	V [kip]	ΦR_n [kip]	Utilization [%]	Status
Anchor 1	1.731	292.500	1	OK

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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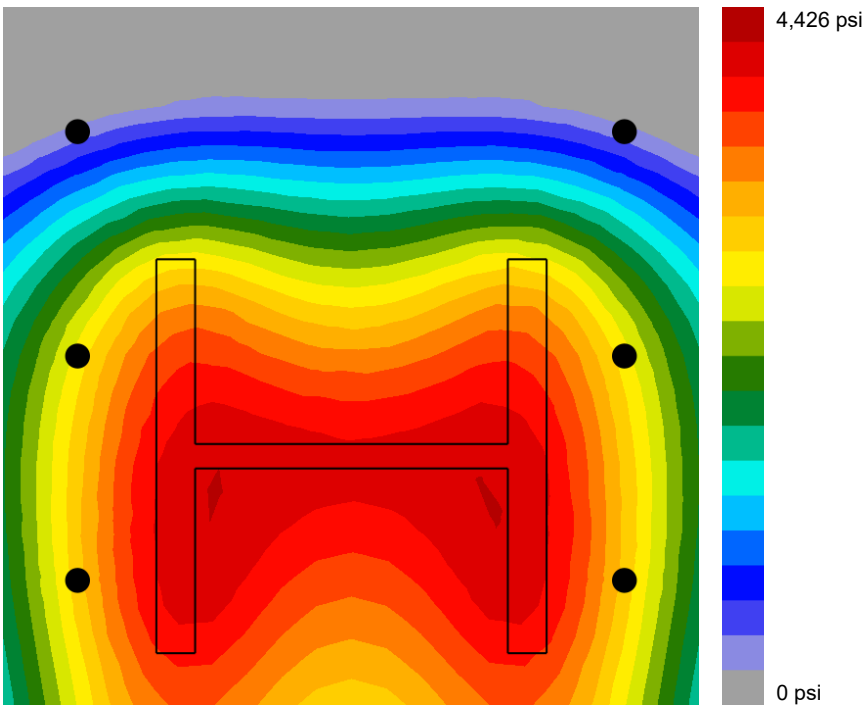
Company:		Page:	12
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Weak Axis DH BP Brace (size)	Date:	4/24/2025
Fastening point:			

	V [kip]	ΦR_n [kip]	Utilization [%]	Status
Anchor 2	1.356	292.500	1	OK
Anchor 3	1.746	292.500	1	OK
Anchor 4	0.976	292.500	1	OK
Anchor 5	1.332	292.500	1	OK
Anchor 6	1.472	292.500	1	OK

2.5 Concrete

Decisive load combination: 1 - Combination 1

2.5.1 Compression in concrete under the anchor plate



2.5.2 Concrete block compressive strength resistance check, AISC 360-16 Section J8

Equations

$F_p = \Phi f_{p,max}$

$f_{p,max} = 0.85 f_c' \sqrt{\left(\frac{A_2}{A_1}\right)} \le 1.7 f_c; \quad \sqrt{\left(\frac{A_2}{A_1}\right)} \le 2$

$\sigma = \frac{N}{A_1}$

$\text{Utilization} = \frac{\sigma}{F_p}$

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Company:		Page:	13
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Weak Axis DH BP Brace (size)	Date:	4/24/2025
Fastening point:			

Variables

N [kip]	f_c' [psi]	Φ	A_1 [in²]	A_2 [in²]
1,859.615	4,500	0.65	644.85	4,416.90

Results

Load combination	F_p [psi]	σ [psi]	Utilization [%]	Status
Combination 1	4,973	2,884	58	OK

2.6 Symbol explanation

A_1	Loaded area of concrete
A_2	Supporting area
d	Nominal diameter of the bolt
ε_{lim}	Limit plastic strain
ε_{Pl}	Plastic strain from CBFEM results
f_c	Concrete compressive strength
f_c'	Concrete compressive strength
F_u	Specified minimum tensile strength of the connected material
F_p	Concrete block design bearing strength
$f_{p,max}$	Concrete block design bearing strength maximum
f_y	Yield strength
l_c	Clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material
N	Resulting compression force
σ	Average stress in concrete
σ_{Ed}	Equivalent stress
Φ	Resistance factor
ΦR_n	Factored resistance
t	Thickness of the anchor plate
V	Resultant of shear forces Vy, Vz in bolt.

2.7 Warnings

- By using the CBFEM calculation functionality of PROFIS Engineering you may act outside the applicable design codes and your specified anchor plate may not behave rigid. Please, validate the results with a professional designer and/or structural engineer to ensure suitability and adequacy for your specific jurisdiction and project requirements.
- The anchor is modeled considering stiffness values determined from load displacement curves tested in an independent laboratory. Please note that no simple replacement of the anchor is possible as the anchor stiffness has a major impact on the load distribution results.

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	14
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Weak Axis DH BP Brace (size)	Date:	4/24/2025
Fastening point:			

3 Summary of results

Design of the anchor plate, anchors, welds and other elements are based on CBFEM (component based finite element method) and AISC.

	Load combination	Max. utilization	Status
Anchors	Combination 1	19%	OK
Anchor plate	Combination 1	61%	OK
Concrete	Combination 1	58%	OK
Profile	Combination 1	100%	OK

Fastening meets the design criteria!

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Company:		Page:	15
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Weak Axis DH BP Brace (size)	Date:	4/24/2025
Fastening point:			

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Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Weak Axis DH BP Brace (GRID F)	Date:	4/24/2025
Fastening point:			

Specifier's comments:

1 Anchor Design

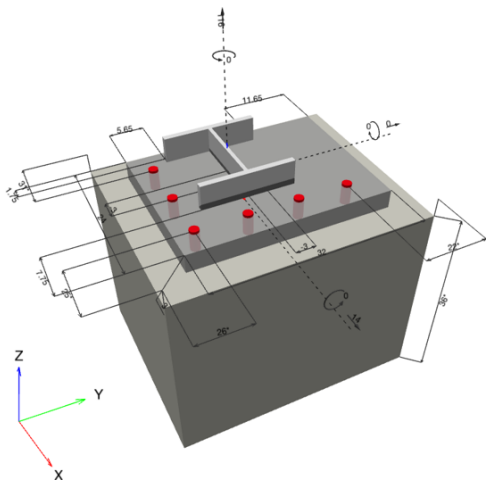
1.1 Input data



Anchor type and diameter:	**Heavy Hex Head ASTM F 1554 GR. 105 1 1/2**
Item number:	not available
Specification text:	∅ 1 1/2 in Heavy Hex Head ASTM F 1554 GR. 105 with 24 in nominal embedment depth per Technical data , cast in place installation per MPII,
Effective embedment depth:	h_{ef} = 24.000 in.
Material:	ASTM F 1554
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-19 / CIP
Shear edge breakout verification:	Row closest to edge (Case 3 only from ACI 318-19 Fig. R.17.7.2.1b)
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 3.000 in.
Anchor plate[CBFEM] :	l_x x l_y x t = 24.000 in. x 32.000 in. x 3.000 in.;
Profile:	W shape (AISC), W14X120; (L x W x T x FT) = 14.500 in. x 14.700 in. x 0.590 in. x 0.940 in.
Base material:	cracked concrete, Custom, f_c' = 4,500 psi; h = 36.000 in.
Reinforcement:	tension: present, shear: not present; edge reinforcement: none or < No. 4 bar

[CBFEM] - The anchor calculation is based on a component-based Finite Element Method (CBFEM)

Geometry [in.] & Loading [kip, ft.kip]



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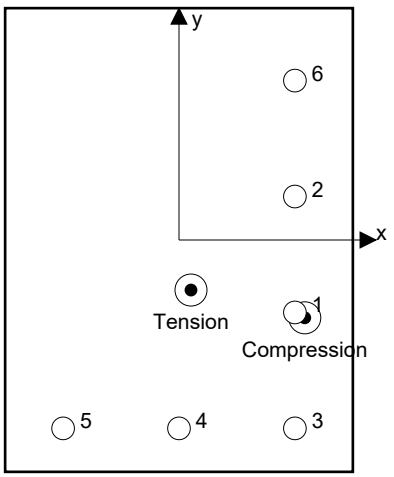
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Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Weak Axis DH BP Brace (GRID F)	Date:	4/24/2025
Fastening point:			

1.1.1 Design results

Case	Description	Forces [kip] / Moments [ft.kip]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 116.000; V_x = 14.000; V_y = 0.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	138

1.2 Load case/Resulting anchor forces



Anchor reactions [kip]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	13.871	2.561	2.550	-0.229
2	35.954	2.664	2.650	-0.269
3	-0.002	2.119	2.109	-0.213
4	10.237	2.100	2.094	0.155
5	71.595	2.131	1.971	0.809
6	39.477	2.637	2.625	-0.253

Resulting tension force in (x/y)=(0.828/-3.454): 171.132 [kip]
Resulting compression force in (x/y)=(8.681/-5.370): 57.265 [kip]

Anchor forces are calculated based on a component-based Finite Element Method (CBFEM)

1.3 Tension load

	Load N_{ua} [kip]	Capacity ϕ N_n [kip]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	71.595	132.187	55	OK
Pullout Strength*	71.595	78.574	92	OK
Concrete Breakout Failure**	171.134	136.245	126	not recommended
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

23.3
17.7

* highest loaded anchor **anchor group (anchors in tension)

SEE SUPPLEMENTAL CALCULATIONS
NOTE: PULLOUT IS NOT FAILING BUT COMBINED UNITY WITH SHEAR WILL BE OVER SO WASHERS ARE PROVIDED

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Company:		Page:	3
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Weak Axis DH BP Brace (GRID F)	Date:	4/24/2025
Fastening point:			

1.3.1 Steel Strength

$N_{sa} = A_{se,N} f_{uta}$ ACI 318-19 Eq. (17.6.1.2)
$\phi N_{sa} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
1.41	125,001

Calculations

N_{sa} [kip]
176.250

Results

N_{sa} [kip]	ϕ_{steel}	ϕN_{sa} [kip]	N_{ua} [kip]
176.250	0.750	132.187	71.595

1.3.2 Pullout Strength

$N_{pN} = \psi_{c,p} N_p$ ACI 318-19 Eq. (17.6.3.1)
$N_p = 8 A_{brg} f'_c$ ACI 318-19 Eq. (17.6.3.2.2a)
$\phi N_{pN} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$\psi_{c,p}$	A_{brg} [in.^{2}]	λ_a	f'_c [psi]
1.000	3.12	1.000	4,500

Calculations

N_p [kip]
112.248

Results

N_{pn} [kip]	$\phi_{concrete}$	ϕN_{pn} [kip]	N_{ua} [kip]
112.248	0.700	78.574	71.595

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Company:		Page:	4
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Weak Axis DH BP Brace (GRID F)	Date:	4/24/2025
Fastening point:			

1.3.3 Concrete Breakout Failure

$N_{cbg} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ec,N}\ \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b$ — ACI 318-19 Eq. (17.6.2.1b)

$\phi\ N_{cbg} \geq N_{ua}$ — ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9\ h_{ef}^2$ — ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\ e'_N}{3\ h_{ef}}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3\left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = 16\ \lambda_a\ \sqrt{f'_c}\ h_{ef}^{5/3}$ — ACI 318-19 Eq. (17.6.2.2.3)

Variables

h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
20.667	2.372	0.054	22.000	1.000

c_{ac} [in.]	k_c	λ_a	f'_c [psi]
-	16	1.000	4,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [kip]
4,938.00	3,844.00	0.929	0.998	0.913	1.000	167.049

Results

N_{cbg} [kip]	$\phi_{concrete}$	$\phi\ N_{cbg}$ [kip]	N_{ua} [kip]
181.660	0.750	136.245	171.134

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Company:		Page:	5
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Weak Axis DH BP Brace (GRID F)	Date:	4/24/2025
Fastening point:			

1.4 Shear load

	Load V_{ua} [kip]	Capacity ϕ V_n [kip]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	2.664	68.737	4	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	14.000	257.952	6	OK
Concrete edge failure in direction y+**	14.033	35.536	40	OK

* highest loaded anchor **anchor group (relevant anchors)

1.4.1 Steel Strength

$V_{sa} = 0.6\ A_{se,V}\ f_{uta}$ ACI 318-19 Eq. (17.7.1.2b)
$\phi\ V_{steel} \geq V_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
1.41	125,001

Calculations

V_{sa} [kip]
105.750

Results

V_{sa} [kip]	ϕ_{steel}	$\phi\ V_{sa}$ [kip]	V_{ua} [kip]
105.750	0.650	68.737	2.664

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Company:		Page:	6
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Weak Axis DH BP Brace (GRID F)	Date:	4/24/2025
Fastening point:			

1.4.2 Pryout Strength

$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \right]$ ACI 318-19 Eq. (17.7.3.1b)

$\phi V_{cpg} \geq V_{ua}$ ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9 h_{ef}^2$ ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e'_N}{3 h_{ef}}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = 16 \lambda_a \sqrt{f'_c} h_{ef}^{5/3}$ ACI 318-19 Eq. (17.6.2.2.3)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	20.667	0.000	2.000	22.000

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.000	-	16	1.000	4,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [kip]
4,944.00	3,844.00	1.000	0.939	0.913	1.000	167.049

Results

V_{cpg} [kip]	$\phi_{concrete}$	ϕV_{cpg} [kip]	V_{ua} [kip]
368.503	0.700	257.952	14.000

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Company:		Page:	7
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Weak Axis DH BP Brace (GRID F)	Date:	4/24/2025
Fastening point:			

1.4.3 Concrete edge failure in direction y+

$V_{cb} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \psi_{ed,V} \psi_{c,V} \psi_{h,V} \psi_{parallel,V} V_b$ — ACI 318-19 Eq. (17.7.2.1a)

$\phi V_{cb} \geq V_{ua}$ — ACI 318-19 Table 17.5.2

A_{Vc} see ACI 318-19, Section 17.7.2.1, Fig. R 17.7.2.1(b)*

$A_{Vc0} = 4.5 c_{a1}^2$ — ACI 318-19 Eq. (17.7.2.1.3)

$\psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.4.1b)

$\psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ — ACI 318-19 Eq. (17.7.2.6.1)

$V_b = 9 \lambda_a \sqrt{f'_c} c_{a1}^{1.5}$ — ACI 318-19 Eq. (17.7.2.2.1b)

Variables

c_{a1} [in.]	c_{a2} [in.]	$\psi_{c,V}$	h_a [in.]	l_e [in.]
22.000	25.000	1.000	36.000	12.000

λ_a	d_a [in.]	f'_c [psi]	$\psi_{parallel,V}$
1.000	1.500	4,500	1.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\psi_{ed,V}$	$\psi_{h,V}$	V_b [kip]
1,914.00	2,178.00	0.927	1.000	62.299

Results

V_{cb} [kip]	$\phi_{concrete}$	ϕV_{cb} [kip]	V_{ua} [kip]
50.766	0.700	35.536	14.033

*Anchor row defined by: Anchor 6; Case 3 controls

1.5 Combined tension and shear loads, per ACI 318-19 section 17.8

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
~~1.256~~ 0.55	0.395	1.000	~~138~~ 78.8	not recommended

$\beta_{NV} = (\beta_N + \beta_V) / 1.2 <= 1$

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Company:		Page:	8
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Weak Axis DH BP Brace (GRID F)	Date:	4/24/2025
Fastening point:			

1.6 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates as per current regulations (ETAG 001/Annex C, EOTA TR029, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid base plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- The equations presented in this report are based on imperial units. When inputs are displayed in metric units, the user should be aware that the equations remain in their imperial format.
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- For additional information about ACI 318 strength design provisions, please go to https://viewer.joomag.com/profis-design-guide-us-en-summer-2021/0841849001625154758?short&/
- Attention! In case of compressive anchor forces a buckling check as well as the proof of the local load transfer into and within the base material (incl. punching) has to be done separately.
- The anchor design methods in PROFIS Engineering require rigid anchor plates, as per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means that the anchor plate should be sufficiently rigid to prevent load re-distribution to the anchors due to elastic/plastic displacements. The user accepts that the anchor plate is considered close to rigid by engineering judgment."

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Company:		Page:	9
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Weak Axis DH BP Brace (GRID F)	Date:	4/24/2025
Fastening point:			

1.7 Installation data

Profile: W shape (AISC), W14X120; (L x W x T x FT) = 14.500 in. x 14.700 in. x 0.590 in. x 0.940 in.

Hole diameter in the fixture: d_f = 1.562 in.

Plate thickness (input): 3.000 in.

Anchor type and diameter: Heavy Hex Head ASTM F 1554 GR. 105 1 1/2

Item number: not available

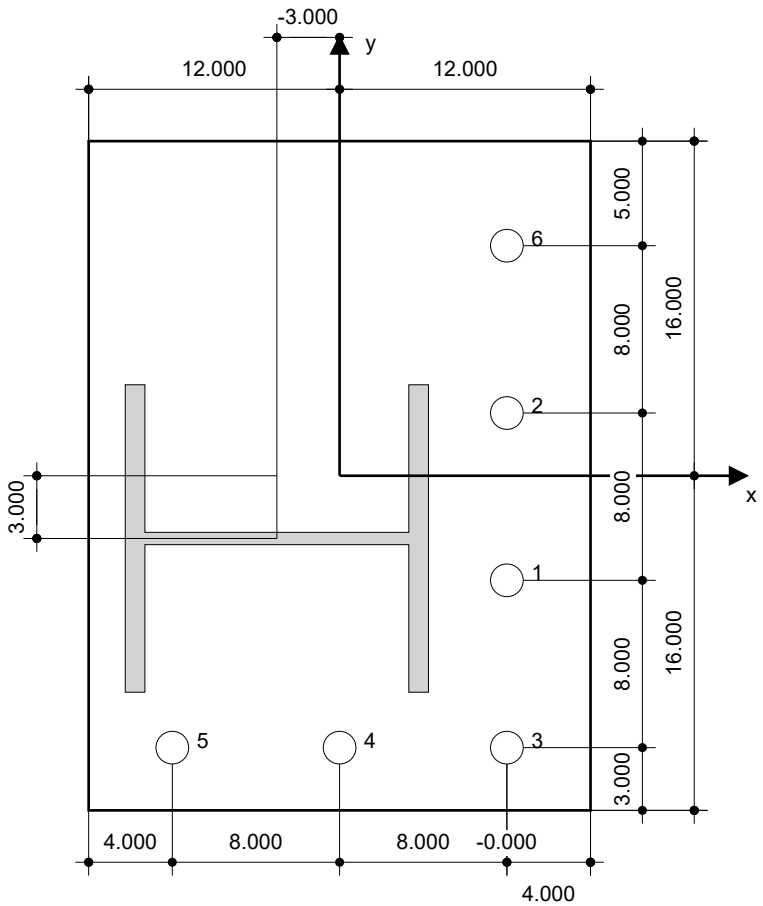
Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 24.000 in.

Minimum thickness of the base material: 25.500 in.

⌀ 1 1/2 in Heavy Hex Head ASTM F 1554 GR. 105 with 24 in nominal embedment depth per Technical data , cast in place installation per MPII



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	8.000	-5.000	47.000	25.000	34.000	38.000
2	8.000	3.000	47.000	25.000	42.000	30.000
3	8.000	-13.000	47.000	25.000	26.000	46.000
4	0.000	-13.000	39.000	33.000	26.000	46.000
5	-8.000	-13.000	31.000	41.000	26.000	46.000
6	8.000	11.000	47.000	25.000	50.000	22.000

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Company:		Page:	10
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Weak Axis DH BP Brace (GRID F)	Date:	4/24/2025
Fastening point:			

2 Anchor plate design

2.1 Input data

Anchor plate:	Shape: Rectangular l_x x l_y x t = 24.000 in x 32.000 in x 3.000 in Calculation: CBFEM Material: ASTM A572 Gr.50; F_y = 50,000 psi; ε_{lim} = 5.00%
Anchor type and size:	Heavy Hex Head ASTM F 1554 GR. 105 1 1/2, h_{ef} = 24.000 in
Anchor stiffness:	The anchor is modeled considering stiffness values determined from load displacement curves tested in an independent laboratory. Please note that no simple replacement of the anchor is possible as the anchor stiffness has a major impact on the load distribution results.
Design method:	AISC and LRFD-based design using component-based FEM
Stand-off installation:	e_b = 0.000 in (No stand-off); t = 3.000 in
Profile:	W14X120; (L x W x T x FT) = 14.500 in x 14.700 in x 0.590 in x 0.940 in Material: ASTM A992; F_y = 50,000 psi; ε_{lim} = 5.00% Eccentricity x: -3.000 in Eccentricity y: -3.000 in
Base material:	Cracked concrete; Custom; $f_{c,cyl}$ = 4,500 psi; h = 36.000 in; E = 4,351,132 psi; G = 1,740,453 psi; v = 0.20; D = 156.07 lb/ft³
Welds (profile to anchor plate):	Type of redistribution: Plastic Material: E70xx
Mesh size:	Number of elements on edge: 8 Min. size of element: 0.394 in Max. size of element: 1.969 in

2.2 Summary

	Description	Profile		Anchor plate			Concrete [%]
		σ_{Ed} [psi]	ε_{Pl} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Hole bearing [%]	
1	Combination 1	25,705	0.00	15,637	0.00	1	9

2.3 Anchor plate classification

Results below are displayed for the decisive load combinations: Combination 1

Anchor tension forces	Equivalent rigid anchor plate (CBFEM)	Component-based Finite Element Method (CBFEM) anchor plate design
Anchor 1	-0.000 kip	13.871 kip
Anchor 2	16.452 kip	35.954 kip
Anchor 3	-0.002 kip	-0.002 kip
Anchor 4	18.529 kip	10.236 kip
Anchor 5	68.042 kip	71.595 kip
Anchor 6	40.168 kip	39.477 kip

User accepted to consider the selected anchor plate as rigid by his/her engineering judgement. This means the anchor design guidelines can be applied.

2.4 Profile/Stiffeners/Plate

Profile and stiffeners are verified at the level of the steel to concrete connection. The connection design does not replace the steel design for critical cross sections, which should be performed outside of PROFIS Engineering.

2.4.1 Equivalent stress and plastic strain

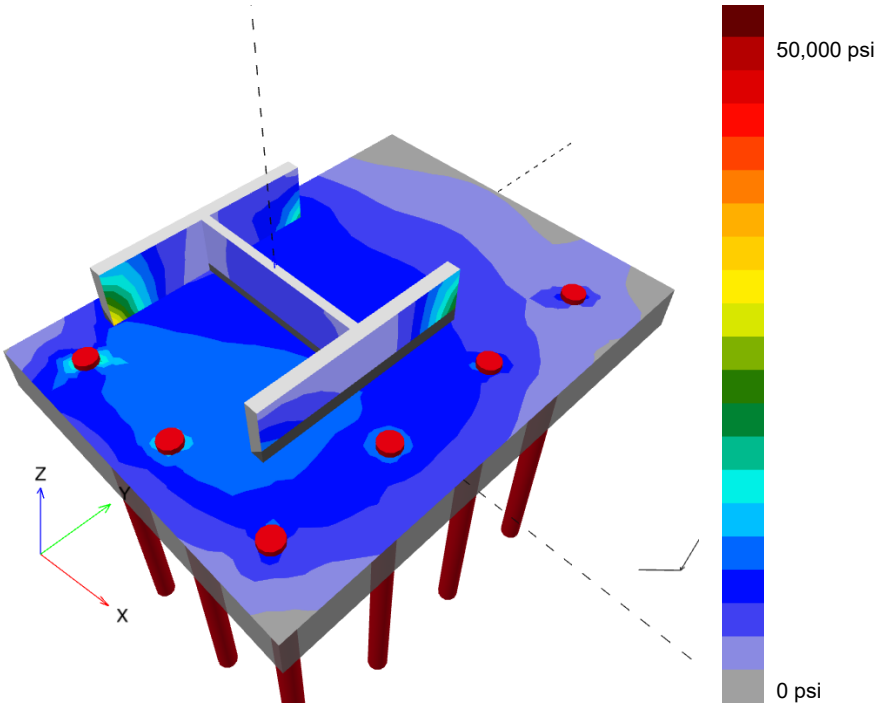
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Company:		Page:	11
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Weak Axis DH BP Brace (GRID F)	Date:	4/24/2025
Fastening point:			

Part	Load combination	Material	f_y [psi]	ε_{lim} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Status
Plate	Combination 1	ASTM A572 Gr.50	50,000	5.00	15,637	0.00	OK
Profile	Combination 1	ASTM A992	50,000	5.00	21,697	0.00	OK
Profile	Combination 1	ASTM A992	50,000	5.00	25,705	0.00	OK
Profile	Combination 1	ASTM A992	50,000	5.00	7,536	0.00	OK

2.4.1.1 Equivalent stress

Results below are displayed for the decisive load combination: 1 - Combination 1



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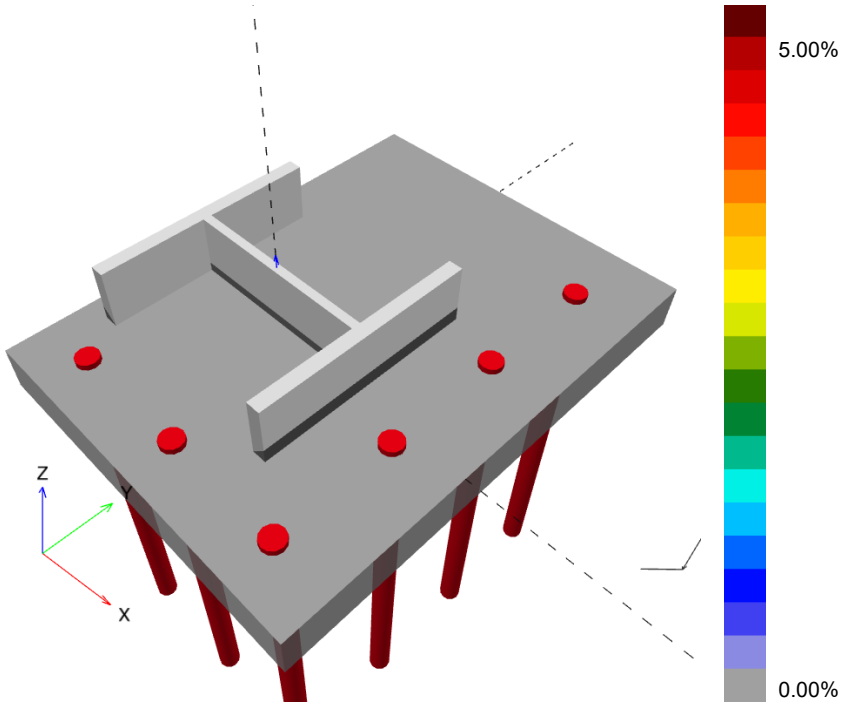
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Company:		Page:	12
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Weak Axis DH BP Brace (GRID F)	Date:	4/24/2025
Fastening point:			

2.4.1.2 Plastic strain

Results below are displayed for the decisive load combination: 1 - Combination 1



2.4.2 Plate hole bearing resistance, AISC 360-16 Section J3

Decisive load combination: 1 - Combination 1

Equations

$R_n = \min(1.2\, l_c\, t\, F_u\, ,\ 2.4\, d\, t\, F_u)$ (AISC 360-16 J3-6a, c)

$\Phi R_n = 0.75\, R_n$

$V \le \Phi R_n$

Variables

	l_c [in]	t [in]	F_u [psi]	d [in]	R_n [kip]
Anchor 1	19.299	3.000	65,000	1.500	702.000
Anchor 2	19.322	3.000	65,000	1.500	702.000
Anchor 3	6.438	3.000	65,000	1.500	702.000
Anchor 4	6.438	3.000	65,000	1.500	702.000
Anchor 5	3.543	3.000	65,000	1.500	702.000
Anchor 6	19.312	3.000	65,000	1.500	702.000

Results

	V [kip]	ΦR_n [kip]	Utilization [%]	Status
Anchor 1	2.561	526.500	1	OK

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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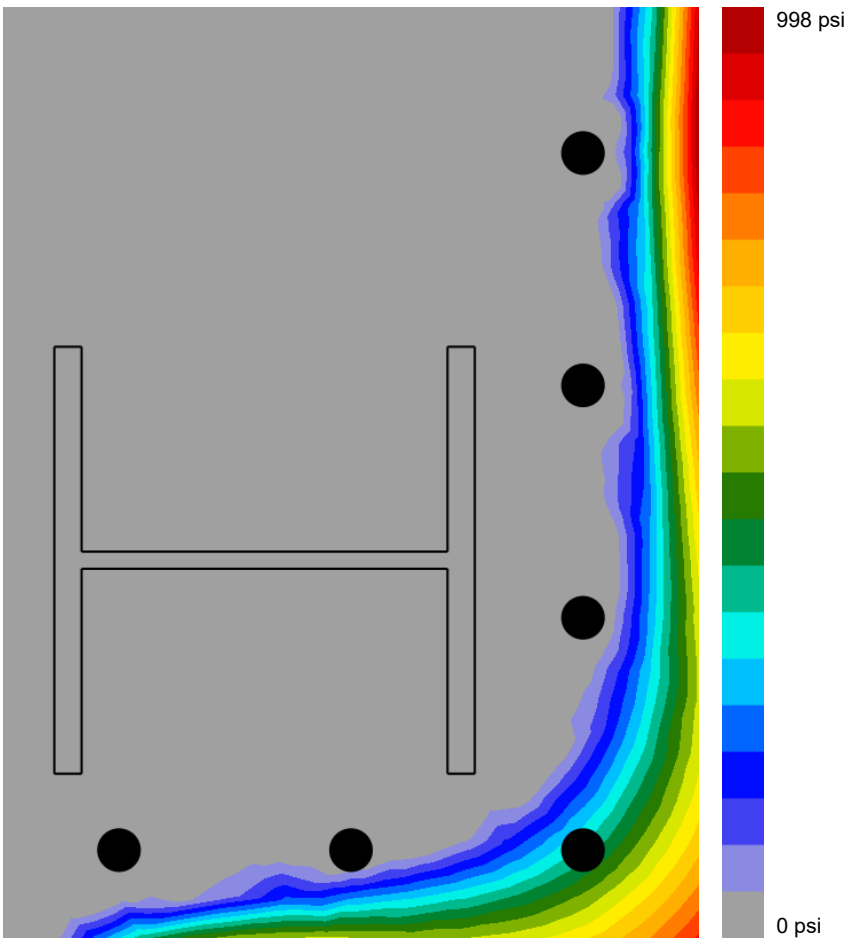
Company:		Page:	13
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Weak Axis DH BP Brace (GRID F)	Date:	4/24/2025
Fastening point:			

	V [kip]	ΦR_n [kip]	Utilization [%]	Status
Anchor 2	2.664	526.500	1	OK
Anchor 3	2.119	526.500	1	OK
Anchor 4	2.100	526.500	1	OK
Anchor 5	2.131	526.500	1	OK
Anchor 6	2.637	526.500	1	OK

2.5 Concrete

Decisive load combination: 1 - Combination 1

2.5.1 Compression in concrete under the anchor plate



2.5.2 Concrete block compressive strength resistance check, AISC 360-16 Section J8

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	14
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Weak Axis DH BP Brace (GRID F)	Date:	4/24/2025
Fastening point:			

Equations

$F_p = \Phi f_{p,max}$

$f_{p,max} = 0.85 f_c' \sqrt{(\frac{A_2}{A_1})} \le 1.7 f_c; \quad \sqrt{(\frac{A_2}{A_1})} \le 2$

$\sigma = \frac{N}{A_1}$

$\text{Utilization} = \frac{\sigma}{F_p}$

Variables

N [kip]	f_c' [psi]	Φ	A_1 [in²]	A_2 [in²]
57.265	4,500	0.65	136.33	2,839.95

Results

Load combination	F_p [psi]	σ [psi]	Utilization [%]	Status
Combination 1	4,973	420	9	OK

2.6 Symbol explanation

A_1	Loaded area of concrete
A_2	Supporting area
d	Nominal diameter of the bolt
ε_{lim}	Limit plastic strain
ε_{Pl}	Plastic strain from CBFEM results
f_c	Concrete compressive strength
f_c'	Concrete compressive strength
F_u	Specified minimum tensile strength of the connected material
F_p	Concrete block design bearing strength
$f_{p,max}$	Concrete block design bearing strength maximum
f_y	Yield strength
l_c	Clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material
N	Resulting compression force
σ	Average stress in concrete
σ_{Ed}	Equivalent stress
Φ	Resistance factor
ΦR_n	Factored resistance
t	Thickness of the anchor plate
V	Resultant of shear forces Vy, Vz in bolt.

2.7 Warnings

- By using the CBFEM calculation functionality of PROFIS Engineering you may act outside the applicable design codes and your specified anchor plate may not behave rigid. Please, validate the results with a professional designer and/or structural engineer to ensure suitability and adequacy for your specific jurisdiction and project requirements.
- The anchor is modeled considering stiffness values determined from load displacement curves tested in an independent laboratory. Please note that no simple replacement of the anchor is possible as the anchor stiffness has a major impact on the load distribution results.

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	15
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Weak Axis DH BP Brace (GRID F)	Date:	4/24/2025
Fastening point:			

3 Summary of results

Design of the anchor plate, anchors, welds and other elements are based on CBFEM (component based finite element method) and AISC.

	Load combination	Max. utilization	Status
Anchors	Combination 1	138%	NOT OK
Anchor plate	Combination 1	32%	OK
Concrete	Combination 1	9%	OK
Profile	Combination 1	52%	OK

Fastening does not meet the design criteria!

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Company:		Page:	16
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Weak Axis DH BP Brace (GRID F)	Date:	4/24/2025
Fastening point:			

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Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis DH BP Brace (Double Brace)	Date:	4/24/2025
Fastening point:			

Specifier's comments:

1 Anchor Design

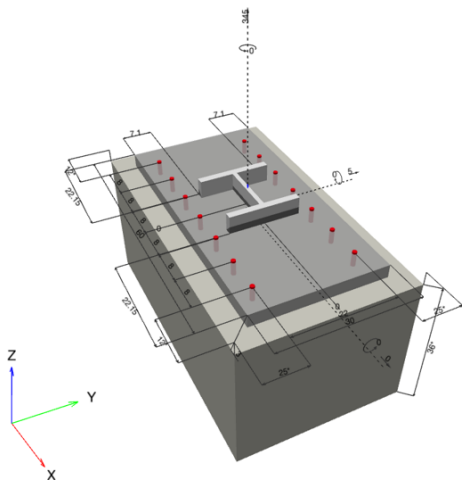
1.1 Input data



Anchor type and diameter:	**Heavy Hex Head ASTM F 1554 GR. 105 1**
Item number:	not available
Specification text:	∅ 1 in Heavy Hex Head ASTM F 1554 GR. 105 with 24 in nominal embedment depth per Technical data , cast in place installation per MPII,
Effective embedment depth:	h_{ef} = 24.000 in.
Material:	ASTM F 1554
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-19 / CIP
Shear edge breakout verification:	Row closest to edge (Case 3 only from ACI 318-19 Fig. R.17.7.2.1b)
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 3.000 in.
Anchor plate[CBFEM] :	l_x x l_y x t = 60.000 in. x 30.000 in. x 3.000 in.;
Profile:	W shape (AISC), W14X211; (L x W x T x FT) = 15.700 in. x 15.800 in. x 0.980 in. x 1.560 in.
Base material:	cracked concrete, Custom, f_c' = 4,500 psi; h = 36.000 in.
Reinforcement:	tension: not present, shear: not present; edge reinforcement: none or < No. 4 bar

[CBFEM] - The anchor calculation is based on a component-based Finite Element Method (CBFEM)

Geometry [in.] & Loading [kip, ft.kip]



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Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis DH BP Brace (Double Brace)	Date:	4/24/2025
Fastening point:			

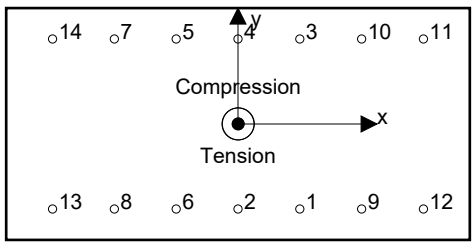
1.1.1 Design results

Case	Description	Forces [kip] / Moments [ft.kip]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 345.000; V_x = 0.000; V_y = 5.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	252

1.2 Load case/Resulting anchor forces

Anchor reactions [kip]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	42.194	0.701	-0.033	0.700
2	45.655	0.637	0.000	0.637
3	42.184	0.038	-0.021	0.032
4	45.623	0.120	-0.000	0.120
5	42.191	0.038	0.021	0.032
6	42.171	0.700	0.033	0.699
7	21.515	0.248	0.028	0.247
8	21.499	0.467	0.048	0.465
9	21.501	0.467	-0.048	0.465
10	21.513	0.248	-0.028	0.247
11	3.536	0.331	-0.004	0.331
12	3.539	0.349	-0.037	0.347
13	3.538	0.349	0.037	0.347
14	3.541	0.331	0.005	0.331



Resulting tension force in (x/y)=(0.000/0.000): 360.198 [kip]
Resulting compression force in (x/y)=(0.044/0.024): 15.792 [kip]

Anchor forces are calculated based on a component-based Finite Element Method (CBFEM)

1.3 Tension load

	Load N_{ua} [kip]	Capacity ϕ N_n [kip]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	45.655	56.812	81	OK
Pullout Strength*	45.655	37.825	121 20.3	not recommended
Concrete Breakout Failure**	360.198	142.988	252 65.1	not recommended
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (anchors in tension)

SEE SUPPLEMENTAL CALCULATIONS

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Company:		Page:	3
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis DH BP Brace (Double Brace)	Date:	4/24/2025
Fastening point:			

1.3.1 Steel Strength

$N_{sa} = A_{se,N} f_{uta}$ ACI 318-19 Eq. (17.6.1.2)
$\phi N_{sa} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.61	125,001

Calculations

N_{sa} [kip]
75.750

Results

N_{sa} [kip]	ϕ_{steel}	ϕN_{sa} [kip]	N_{ua} [kip]
75.750	0.750	56.812	45.655

1.3.2 Pullout Strength

$N_{pN} = \psi_{c,p} N_p$ ACI 318-19 Eq. (17.6.3.1)
$N_p = 8 A_{brg} f'_c$ ACI 318-19 Eq. (17.6.3.2.2a)
$\phi N_{pN} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$\psi_{c,p}$	A_{brg} [in.^{2}]	λ_a	f'_c [psi]
1.000	1.50	1.000	4,500

Calculations

N_p [kip]
54.036

Results

N_{pn} [kip]	$\phi_{concrete}$	ϕN_{pn} [kip]	N_{ua} [kip]
54.036	0.700	37.825	45.655

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Company:		Page:	4
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis DH BP Brace (Double Brace)	Date:	4/24/2025
Fastening point:			

1.3.3 Concrete Breakout Failure

$N_{cbg} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ec,N}\ \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b$ — ACI 318-19 Eq. (17.6.2.1b)

$\phi\ N_{cbg} \geq N_{ua}$ — ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9\ h_{ef}^2$ — ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\ e'_N}{3\ h_{ef}}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3\left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = 16\ \lambda_a\ \sqrt{f'_c}\ h_{ef}^{5/3}$ — ACI 318-19 Eq. (17.6.2.2.3)

Variables

h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
16.667	0.000	0.000	12.000	1.000

c_{ac} [in.]	k_c	λ_a	f'_c [psi]
-	16	1.000	4,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [kip]
5,184.00	2,500.00	1.000	1.000	0.844	1.000	116.719

Results

N_{cbg} [kip]	$\phi_{concrete}$	$\phi\ N_{cbg}$ [kip]	N_{ua} [kip]
204.269	0.700	142.988	360.198

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	5
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis DH BP Brace (Double Brace)	Date:	4/24/2025
Fastening point:			

1.4 Shear load

	Load V_{ua} [kip]	Capacity ϕ V_n [kip]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	0.701	29.542	3	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	5.000	285.980	2	OK
Concrete edge failure in direction x-**	5.003	28.080	18	OK

* highest loaded anchor **anchor group (relevant anchors)

1.4.1 Steel Strength

$V_{sa} = 0.6\ A_{se,V}\ f_{uta}$ ACI 318-19 Eq. (17.7.1.2b)
$\phi\ V_{steel} \geq V_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.61	125,001

Calculations

V_{sa} [kip]
45.450

Results

V_{sa} [kip]	ϕ_{steel}	$\phi\ V_{sa}$ [kip]	V_{ua} [kip]
45.450	0.650	29.542	0.701

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	6
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis DH BP Brace (Double Brace)	Date:	4/24/2025
Fastening point:			

1.4.2 Pryout Strength

$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \right]$ ACI 318-19 Eq. (17.7.3.1b)

$\phi V_{cpg} \geq V_{ua}$ ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9 h_{ef}^2$ ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e'_N}{3 h_{ef}}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = 16 \lambda_a \sqrt{f'_c} h_{ef}^{5/3}$ ACI 318-19 Eq. (17.6.2.2.3)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	16.667	0.000	0.000	12.000

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.000	-	16	1.000	4,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [kip]
5,184.00	2,500.00	1.000	1.000	0.844	1.000	116.719

Results

V_{cpg} [kip]	$\phi_{concrete}$	ϕV_{cpg} [kip]	V_{ua} [kip]
408.543	0.700	285.980	5.000

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Company:		Page:	7
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis DH BP Brace (Double Brace)	Date:	4/24/2025
Fastening point:			

1.4.3 Concrete edge failure in direction x-

$V_{cbg} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \Psi_{ec,V} \Psi_{ed,V} \Psi_{c,V} \Psi_{h,V} \Psi_{parallel,V} V_b$ — ACI 318-19 Eq. (17.7.2.1b)

$\phi V_{cbg} \geq V_{ua}$ — ACI 318-19 Table 17.5.2

A_{Vc} see ACI 318-19, Section 17.7.2.1, Fig. R 17.7.2.1(b)*

$A_{Vc0} = 4.5 c_{a1}^2$ — ACI 318-19 Eq. (17.7.2.1.3)

$\Psi_{ec,V} = \left(\frac{1}{1 + \frac{e'_v}{1.5c_{a1}}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.3.1)

$\Psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.4.1b)

$\Psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ — ACI 318-19 Eq. (17.7.2.6.1)

$V_b = 9 \lambda_a \sqrt{f'_c} c_{a1}^{1.5}$ — ACI 318-19 Eq. (17.7.2.2.1b)

Variables

c_{a1} [in.]	c_{a2} [in.]	e_{cV} [in.]	$\Psi_{c,V}$	h_a [in.]
12.000	25.000	0.143	1.000	36.000

l_e [in.]	λ_a	d_a [in.]	f'_c [psi]	$\Psi_{parallel,V}$
8.000	1.000	1.000	4,500	1.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{h,V}$	V_b [kip]
1,044.00	648.00	0.992	1.000	1.000	25.097

Results

V_{cbg} [kip]	$\phi_{concrete}$	ϕV_{cbg} [kip]	V_{ua} [kip]
40.115	0.700	28.080	5.003

*Anchor row defined by: Anchor 13, 14; Case 3 controls

1.5 Combined tension and shear loads, per ACI 318-19 section 17.8

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
~~2.519~~ 0.81	0.178	1.000	~~225~~ 82.3	not recommended

$\beta_{NV} = (\beta_N + \beta_V) / 1.2 <= 1$

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Company:		Page:	8
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis DH BP Brace (Double Brace)	Date:	4/24/2025
Fastening point:			

1.6 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates as per current regulations (ETAG 001/Annex C, EOTA TR029, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid base plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- The equations presented in this report are based on imperial units. When inputs are displayed in metric units, the user should be aware that the equations remain in their imperial format.
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- For additional information about ACI 318 strength design provisions, please go to https://viewer.joomag.com/profis-design-guide-us-en-summer-2021/0841849001625154758?short&/
- The anchor design methods in PROFIS Engineering require rigid anchor plates, as per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means that the anchor plate should be sufficiently rigid to prevent load re-distribution to the anchors due to elastic/plastic displacements. The user accepts that the anchor plate is considered close to rigid by engineering judgment."

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Company:		Page:	9
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis DH BP Brace (Double Brace)	Date:	4/24/2025
Fastening point:			

1.7 Installation data

Profile: W shape (AISC), W14X211; (L x W x T x FT) = 15.700 in. x 15.800 in. x 0.980 in. x 1.560 in.

Hole diameter in the fixture: d_f = 1.062 in.

Plate thickness (input): 3.000 in.

Anchor type and diameter: Heavy Hex Head ASTM F 1554 GR. 105 1

Item number: not available

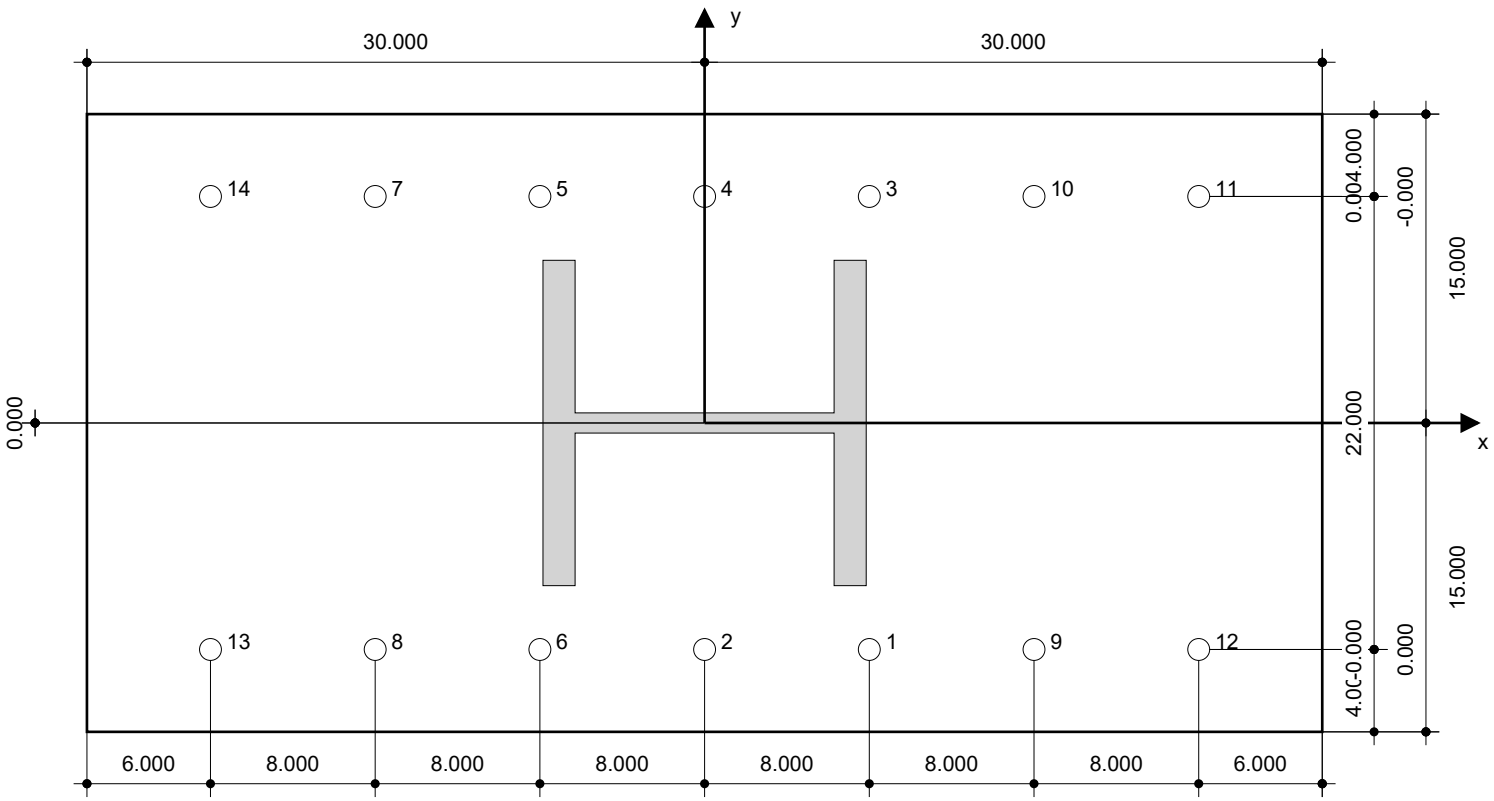
Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 24.000 in.

Minimum thickness of the base material: 25.172 in.

⌀ 1 in Heavy Hex Head ASTM F 1554 GR. 105 with 24 in nominal embedment depth per Technical data , cast in place installation per MPII



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Company:		Page:	10
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis DH BP Brace (Double Brace)	Date:	4/24/2025
Fastening point:			

Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	8.000	-11.000	44.000	28.000	25.000	47.000
2	0.000	-11.000	36.000	36.000	25.000	47.000
3	8.000	11.000	44.000	28.000	47.000	25.000
4	0.000	11.000	36.000	36.000	47.000	25.000
5	-8.000	11.000	28.000	44.000	47.000	25.000
6	-8.000	-11.000	28.000	44.000	25.000	47.000
7	-16.000	11.000	20.000	52.000	47.000	25.000
8	-16.000	-11.000	20.000	52.000	25.000	47.000
9	16.000	-11.000	52.000	20.000	25.000	47.000
10	16.000	11.000	52.000	20.000	47.000	25.000
11	24.000	11.000	60.000	12.000	47.000	25.000
12	24.000	-11.000	60.000	12.000	25.000	47.000
13	-24.000	-11.000	12.000	60.000	25.000	47.000
14	-24.000	11.000	12.000	60.000	47.000	25.000

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Company:		Page:	11
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis DH BP Brace (Double Brace)	Date:	4/24/2025
Fastening point:			

2 Anchor plate design

2.1 Input data

Anchor plate:	Shape: Rectangular l_x x l_y x t = 60.000 in x 30.000 in x 3.000 in Calculation: CBFEM Material: ASTM A572 Gr.50; F_y = 50,000 psi; ε_{lim} = 5.00%
Anchor type and size:	Heavy Hex Head ASTM F 1554 GR. 105 1, h_{ef} = 24.000 in
Anchor stiffness:	The anchor is modeled considering stiffness values determined from load displacement curves tested in an independent laboratory. Please note that no simple replacement of the anchor is possible as the anchor stiffness has a major impact on the load distribution results.
Design method:	AISC and LRFD-based design using component-based FEM
Stand-off installation:	e_b = 0.000 in (No stand-off); t = 3.000 in
Profile:	W14X211; (L x W x T x FT) = 15.700 in x 15.800 in x 0.980 in x 1.560 in Material: ASTM A992; F_y = 50,000 psi; ε_{lim} = 5.00% Eccentricity x: 0.000 in Eccentricity y: -0.000 in
Base material:	Cracked concrete; Custom; $f_{c,cyl}$ = 4,500 psi; h = 36.000 in; E = 4,351,132 psi; G = 1,740,453 psi; v = 0.20; D = 156.07 lb/ft³
Welds (profile to anchor plate):	Type of redistribution: Plastic Material: E70xx
Mesh size:	Number of elements on edge: 8 Min. size of element: 0.394 in Max. size of element: 1.969 in

2.2 Summary

	Description	Profile		Anchor plate			Concrete [%]
		σ_{Ed} [psi]	ε_{Pl} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Hole bearing [%]	
1	Combination 1	26,248	0.00	11,142	0.00	1	3

2.3 Anchor plate classification

Results below are displayed for the decisive load combinations: Combination 1

Anchor tension forces	Equivalent rigid anchor plate (CBFEM)	Component-based Finite Element Method (CBFEM) anchor plate design
Anchor 1	24.650 kip	42.194 kip
Anchor 2	24.650 kip	45.655 kip
Anchor 3	24.651 kip	42.184 kip
Anchor 4	24.652 kip	45.623 kip
Anchor 5	24.651 kip	42.191 kip
Anchor 6	24.650 kip	42.171 kip
Anchor 7	24.649 kip	21.515 kip
Anchor 8	24.648 kip	21.499 kip
Anchor 9	24.648 kip	21.501 kip
Anchor 10	24.649 kip	21.513 kip
Anchor 11	24.647 kip	3.536 kip
Anchor 12	24.645 kip	3.539 kip
Anchor 13	24.645 kip	3.538 kip
Anchor 14	24.647 kip	3.541 kip

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Company:		Page:	12
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis DH BP Brace (Double Brace)	Date:	4/24/2025
Fastening point:			

User accepted to consider the selected anchor plate as rigid by his/her engineering judgement. This means the anchor design guidelines can be applied.

2.4 Profile/Stiffeners/Plate

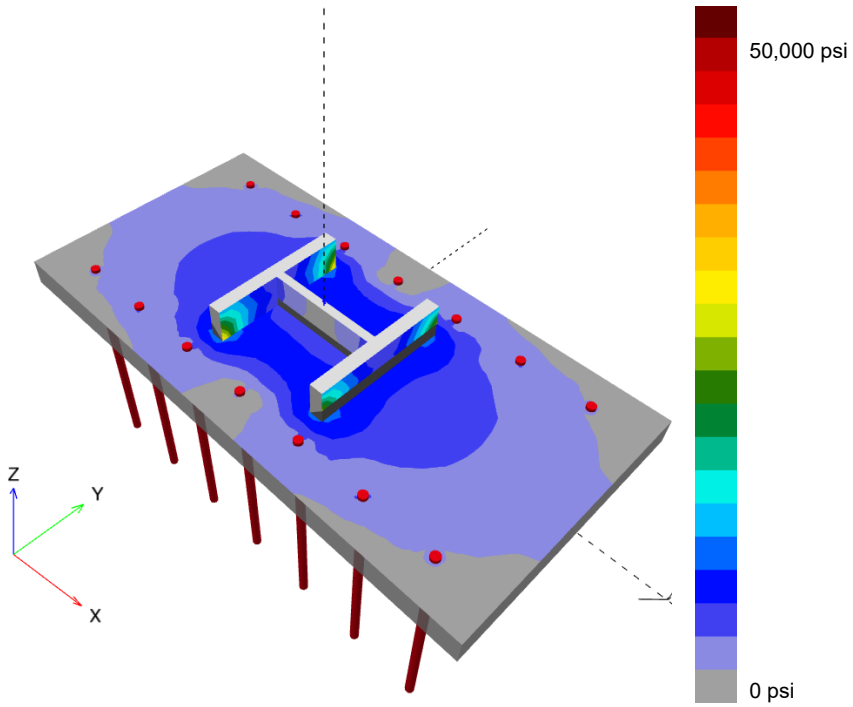
Profile and stiffeners are verified at the level of the steel to concrete connection. The connection design does not replace the steel design for critical cross sections, which should be performed outside of PROFIS Engineering.

2.4.1 Equivalent stress and plastic strain

Part	Load combination	Material	f_y [psi]	ε_{lim} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Status
Plate	Combination 1	ASTM A572 Gr.50	50,000	5.00	11,142	0.00	OK
Profile	Combination 1	ASTM A992	50,000	5.00	26,248	0.00	OK
Profile	Combination 1	ASTM A992	50,000	5.00	26,195	0.00	OK
Profile	Combination 1	ASTM A992	50,000	5.00	6,845	0.00	OK

2.4.1.1 Equivalent stress

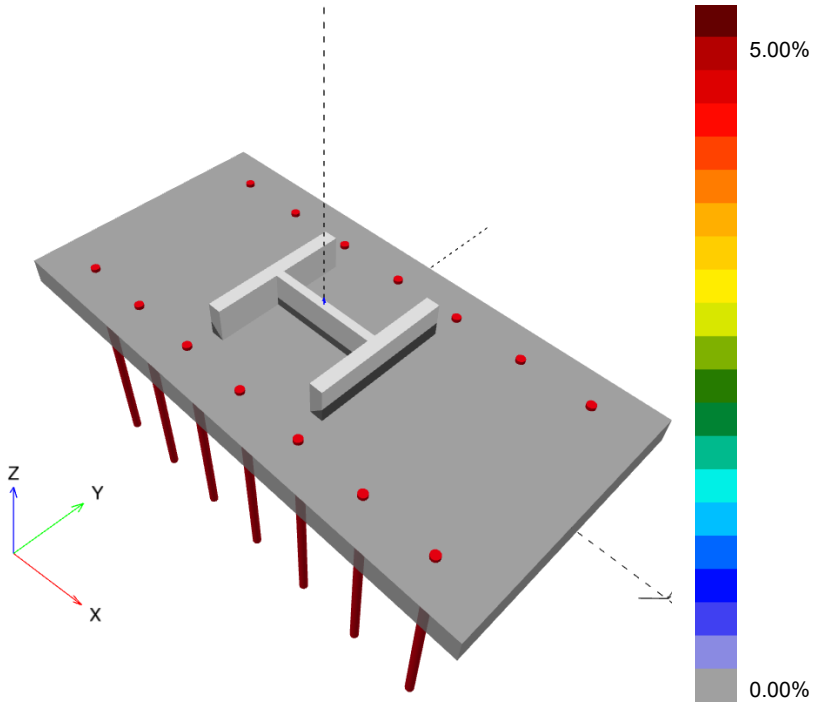
Results below are displayed for the decisive load combination: 1 - Combination 1



Company:		Page:	13
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis DH BP Brace (Double Brace)	Date:	4/24/2025
Fastening point:			

2.4.1.2 Plastic strain

Results below are displayed for the decisive load combination: 1 - Combination 1



2.4.2 Plate hole bearing resistance, AISC 360-16 Section J3

Decisive load combination: 1 - Combination 1

Equations

$R_n = \min(1.2\, l_c\, t\, F_u\, ,\ 2.4\, d\, t\, F_u)$ (AISC 360-16 J3-6a, c)

$\Phi R_n = 0.75\, R_n$

$V \le \Phi R_n$

Variables

	l_c [in]	t [in]	F_u [psi]	d [in]	R_n [kip]
Anchor 1	3.473	3.000	65,000	1.000	468.000
Anchor 2	3.469	3.000	65,000	1.000	468.000
Anchor 3	30.666	3.000	65,000	1.000	468.000
Anchor 4	20.938	3.000	65,000	1.000	468.000
Anchor 5	30.448	3.000	65,000	1.000	468.000
Anchor 6	3.473	3.000	65,000	1.000	468.000
Anchor 7	25.638	3.000	65,000	1.000	468.000
Anchor 8	3.490	3.000	65,000	1.000	468.000
Anchor 9	3.490	3.000	65,000	1.000	468.000
Anchor 10	25.633	3.000	65,000	1.000	468.000

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Company:		Page:	14
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis DH BP Brace (Double Brace)	Date:	4/24/2025
Fastening point:			

	l_c [in]	t [in]	F_u [psi]	d [in]	R_n [kip]
Anchor 11	20.938	3.000	65,000	1.000	468.000
Anchor 12	3.492	3.000	65,000	1.000	468.000
Anchor 13	3.492	3.000	65,000	1.000	468.000
Anchor 14	20.938	3.000	65,000	1.000	468.000

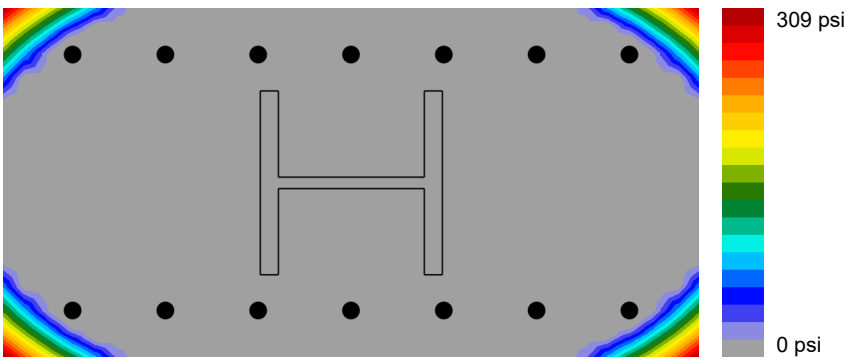
Results

	V [kip]	ΦR_n [kip]	Utilization [%]	Status
Anchor 1	0.701	351.000	1	OK
Anchor 2	0.637	351.000	1	OK
Anchor 3	0.038	351.000	1	OK
Anchor 4	0.120	351.000	1	OK
Anchor 5	0.038	351.000	1	OK
Anchor 6	0.700	351.000	1	OK
Anchor 7	0.248	351.000	1	OK
Anchor 8	0.467	351.000	1	OK
Anchor 9	0.467	351.000	1	OK
Anchor 10	0.248	351.000	1	OK
Anchor 11	0.330	351.000	1	OK
Anchor 12	0.349	351.000	1	OK
Anchor 13	0.349	351.000	1	OK
Anchor 14	0.331	351.000	1	OK

2.5 Concrete

Decisive load combination: 1 - Combination 1

2.5.1 Compression in concrete under the anchor plate



2.5.2 Concrete block compressive strength resistance check, AISC 360-16 Section J8

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Company:		Page:	15
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis DH BP Brace (Double Brace)	Date:	4/24/2025
Fastening point:			

Equations

$F_p = \Phi f_{p,max}$

$f_{p,max} = 0.85 f_c' \sqrt{(\frac{A_2}{A_1})} \le 1.7 f_c; \quad \sqrt{(\frac{A_2}{A_1})} \le 2$

$\sigma = \frac{N}{A_1}$

$\text{Utilization} = \frac{\sigma}{F_p}$

Variables

N [kip]	f_c' [psi]	Φ	A_1 [in²]	A_2 [in²]
15.792	4,500	0.65	124.51	1,334.62

Results

Load combination	F_p [psi]	σ [psi]	Utilization [%]	Status
Combination 1	4,973	127	3	OK

2.6 Symbol explanation

A_1	Loaded area of concrete
A_2	Supporting area
d	Nominal diameter of the bolt
ε_{lim}	Limit plastic strain
ε_{Pl}	Plastic strain from CBFEM results
f_c	Concrete compressive strength
f_c'	Concrete compressive strength
F_u	Specified minimum tensile strength of the connected material
F_p	Concrete block design bearing strength
$f_{p,max}$	Concrete block design bearing strength maximum
f_y	Yield strength
l_c	Clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material
N	Resulting compression force
σ	Average stress in concrete
σ_{Ed}	Equivalent stress
Φ	Resistance factor
ΦR_n	Factored resistance
t	Thickness of the anchor plate
V	Resultant of shear forces Vy, Vz in bolt.

2.7 Warnings

- By using the CBFEM calculation functionality of PROFIS Engineering you may act outside the applicable design codes and your specified anchor plate may not behave rigid. Please, validate the results with a professional designer and/or structural engineer to ensure suitability and adequacy for your specific jurisdiction and project requirements.
- The anchor is modeled considering stiffness values determined from load displacement curves tested in an independent laboratory. Please note that no simple replacement of the anchor is possible as the anchor stiffness has a major impact on the load distribution results.

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	16
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis DH BP Brace (Double Brace)	Date:	4/24/2025
Fastening point:			

3 Summary of results

Design of the anchor plate, anchors, welds and other elements are based on CBFEM (component based finite element method) and AISC.

	Load combination	Max. utilization	Status
Anchors	Combination 1	252%	NOT OK
Anchor plate	Combination 1	23%	OK
Concrete	Combination 1	3%	OK
Profile	Combination 1	53%	OK

Fastening does not meet the design criteria!

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	17
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis DH BP Brace (Double Brace)	Date:	4/24/2025
Fastening point:			

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Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis DH BP Brace (Single Brace)	Date:	4/24/2025
Fastening point:			

Specifier's comments:

1 Anchor Design

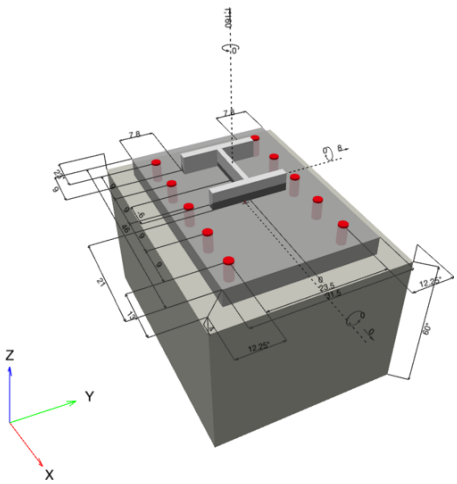
1.1 Input data



Anchor type and diameter:	**Heavy Hex Head ASTM F 1554 GR. 105 2**
Item number:	not available
Specification text:	∅ 2 in Heavy Hex Head ASTM F 1554 GR. 105 with 24 in nominal embedment depth per Technical data , cast in place installation per MPII,
Effective embedment depth:	h_{ef} = 24.000 in.
Material:	ASTM F 1554
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-19 / CIP
Shear edge breakout verification:	Row closest to edge (Case 3 only from ACI 318-19 Fig. R.17.7.2.1b)
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 4.000 in.
Anchor plate[CBFEM] :	l_x x l_y x t = 46.000 in. x 31.500 in. x 4.000 in.;
Profile:	W shape (AISC), W14X233; (L x W x T x FT) = 16.000 in. x 15.900 in. x 1.070 in. x 1.720 in.
Base material:	cracked concrete, Custom, f_c' = 4,500 psi; h = 60.000 in.
Reinforcement:	tension: present, shear: present; edge reinforcement: none or < No. 4 bar

[CBFEM] - The anchor calculation is based on a component-based Finite Element Method (CBFEM)

Geometry [in.] & Loading [kip, ft.kip]



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Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis DH BP Brace (Single Brace)	Date:	4/24/2025
Fastening point:			

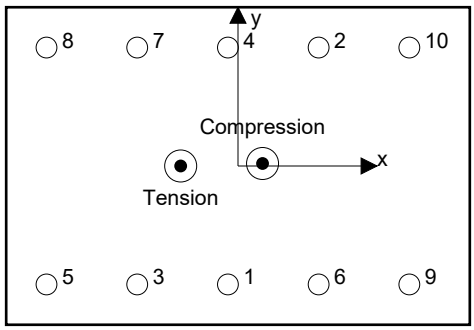
1.1.1 Design results

Case	Description	Forces [kip] / Moments [ft.kip]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 1,160.000; V_x = 0.000; V_y = 8.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	1,342

1.2 Load case/Resulting anchor forces

Anchor reactions [kip]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	174.553	2.646	-0.198	2.639
2	101.472	0.271	-0.006	-0.271
3	179.597	2.831	-0.102	2.829
4	174.698	0.990	0.119	-0.983
5	128.440	2.298	0.117	2.295
6	101.416	1.664	-0.312	1.634
7	179.731	0.985	0.147	-0.974
8	128.090	0.453	0.387	-0.236
9	12.073	0.675	-0.259	0.624
10	12.226	0.455	0.105	0.442



Resulting tension force in (x/y)=(-5.687/0.001): 1,192.297 [kip]
Resulting compression force in (x/y)=(2.436/0.248): 36.376 [kip]

Anchor forces are calculated based on a component-based Finite Element Method (CBFEM)

1.3 Tension load

	Load N_{ua} [kip]	Capacity ϕ N_n [kip]	Utilization $\beta_N = N_{ua}/\phi$ N_n		Status
Steel Strength*	179.731	234.375	77	52.2	OK
Pullout Strength*	179.731	133.963	135	21.6	not recommended
Concrete Breakout Failure**	1,192.297	88.864	1,342	49.8	not recommended
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A		N/A

* highest loaded anchor **anchor group (anchors in tension)

SEE SUPPLEMENTAL CALCULATIONS

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Company:		Page:	3
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis DH BP Brace (Single Brace)	Date:	4/24/2025
Fastening point:			

1.3.1 Steel Strength

$N_{sa} = A_{se,N} f_{uta}$ ACI 318-19 Eq. (17.6.1.2)
$\phi N_{sa} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
2.50	125,001

Calculations

N_{sa} [kip]
312.500

Results

N_{sa} [kip]	ϕ_{steel}	ϕN_{sa} [kip]	N_{ua} [kip]
312.500	0.750	234.375	179.731

1.3.2 Pullout Strength

$N_{pN} = \psi_{c,p} N_p$ ACI 318-19 Eq. (17.6.3.1)
$N_p = 8 A_{brg} f_c'$ ACI 318-19 Eq. (17.6.3.2.2a)
$\phi N_{pN} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$\psi_{c,p}$	A_{brg} [in.^{2}]	λ_a	f_c' [psi]
1.000	5.32	1.000	4,500

Calculations

N_p [kip]
191.376

Results

N_{pn} [kip]	$\phi_{concrete}$	ϕN_{pn} [kip]	N_{ua} [kip]
191.376	0.700	133.963	179.731

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Company:		Page:	4
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis DH BP Brace (Single Brace)	Date:	4/24/2025
Fastening point:			

1.3.3 Concrete Breakout Failure

$N_{cbg} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ec,N}\ \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b$ — ACI 318-19 Eq. (17.6.2.1b)

$\phi\ N_{cbg} \geq N_{ua}$ — ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9\ h_{ef}^2$ — ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\ e'_N}{3\ h_{ef}}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3\left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = 16\ \lambda_a\ \sqrt{f'_c}\ h_{ef}^{5/3}$ — ACI 318-19 Eq. (17.6.2.2.3)

Variables

h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
15.333	4.687	0.001	12.250	1.000

c_{ac} [in.]	k_c	λ_a	f'_c [psi]
-	16	1.000	4,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [kip]
3,456.00	2,116.00	0.831	1.000	0.860	1.000	101.575

Results

N_{cbg} [kip]	$\phi_{concrete}$	$\phi\ N_{cbg}$ [kip]	N_{ua} [kip]
118.485	0.750	88.864	1,192.297

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Company:		Page:	5
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis DH BP Brace (Single Brace)	Date:	4/24/2025
Fastening point:			

1.4 Shear load

	Load V_{ua} [kip]	Capacity ϕ V_n [kip]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	2.831	121.875	3	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	8.000	164.033	5	OK
Concrete edge failure in direction x+**	10.500	22.385	47	OK

* highest loaded anchor **anchor group (relevant anchors)

1.4.1 Steel Strength

$V_{sa} = 0.6\ A_{se,V}\ f_{uta}$ ACI 318-19 Eq. (17.7.1.2b)
$\phi\ V_{steel} \geq V_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
2.50	125,001

Calculations

V_{sa} [kip]
187.500

Results

V_{sa} [kip]	ϕ_{steel}	$\phi\ V_{sa}$ [kip]	V_{ua} [kip]
187.500	0.650	121.875	2.831

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www.hilti.com

Company:		Page:	6
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis DH BP Brace (Single Brace)	Date:	4/24/2025
Fastening point:			

1.4.2 Pryout Strength

$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \right]$ ACI 318-19 Eq. (17.7.3.1b)

$\phi V_{cpg} \geq V_{ua}$ ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9 h_{ef}^2$ ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e'_N}{3 h_{ef}}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = 16 \lambda_a \sqrt{f'_c} h_{ef}^{5/3}$ ACI 318-19 Eq. (17.6.2.2.3)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	15.333	5.000	0.000	12.250

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.000	-	16	1.000	4,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [kip]
3,456.00	2,116.00	0.821	1.000	0.860	1.000	101.575

Results

V_{cpg} [kip]	$\phi_{concrete}$	ϕV_{cpg} [kip]	V_{ua} [kip]
234.333	0.700	164.033	8.000

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Company:		Page:	7
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis DH BP Brace (Single Brace)	Date:	4/24/2025
Fastening point:			

1.4.3 Concrete edge failure in direction x+

$V_{cbg} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \Psi_{ec,V} \Psi_{ed,V} \Psi_{c,V} \Psi_{h,V} \Psi_{parallel,V} V_b$ — ACI 318-19 Eq. (17.7.2.1b)

$\phi V_{cbg} \geq V_{ua}$ — ACI 318-19 Table 17.5.2

A_{Vc} see ACI 318-19, Section 17.7.2.1, Fig. R 17.7.2.1(b)*

$A_{Vc0} = 4.5 c_{a1}^2$ — ACI 318-19 Eq. (17.7.2.1.3)

$\Psi_{ec,V} = \left(\frac{1}{1 + \frac{e'_v}{1.5c_{a1}}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.3.1)

$\Psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.4.1b)

$\Psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ — ACI 318-19 Eq. (17.7.2.6.1)

$V_b = 9 \lambda_a \sqrt{f'_c} c_{a1}^{1.5}$ — ACI 318-19 Eq. (17.7.2.2.1b)

Variables

c_{a1} [in.]	c_{a2} [in.]	e_{cV} [in.]	$\Psi_{c,V}$	h_a [in.]
13.000	12.250	0.717	1.000	60.000

l_e [in.]	λ_a	d_a [in.]	f'_c [psi]	$\Psi_{parallel,V}$
16.000	1.000	2.000	4,500	1.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{h,V}$	V_b [kip]
936.00	760.50	0.965	0.888	1.000	28.299

Results

V_{cbg} [kip]	$\phi_{concrete}$	ϕV_{cbg} [kip]	V_{ua} [kip]
29.847	0.750	22.385	10.500

*Anchor row defined by: Anchor 9, 10; Case 3 controls

1.5 Combined tension and shear loads, per ACI 318-19 section 17.8

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
~~13.417~~ 0.498	0.469	1.000	~~1,158~~ 80.6	not recommended

$\beta_{NV} = (\beta_N + \beta_V) / 1.2 <= 1$

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Company:		Page:	8
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis DH BP Brace (Single Brace)	Date:	4/24/2025
Fastening point:			

1.6 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates as per current regulations (ETAG 001/Annex C, EOTA TR029, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid base plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- The equations presented in this report are based on imperial units. When inputs are displayed in metric units, the user should be aware that the equations remain in their imperial format.
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- For additional information about ACI 318 strength design provisions, please go to https://viewer.joomag.com/profis-design-guide-us-en-summer-2021/0841849001625154758?short&/
- The anchor design methods in PROFIS Engineering require rigid anchor plates, as per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means that the anchor plate should be sufficiently rigid to prevent load re-distribution to the anchors due to elastic/plastic displacements. The user accepts that the anchor plate is considered close to rigid by engineering judgment."

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Company:		Page:	9
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis DH BP Brace (Single Brace)	Date:	4/24/2025
Fastening point:			

1.7 Installation data

Profile: W shape (AISC), W14X233; (L x W x T x FT) = 16.000 in. x 15.900 in. x 1.070 in. x 1.720 in.

Hole diameter in the fixture: d_f = 2.062 in.

Plate thickness (input): 4.000 in.

Anchor type and diameter: Heavy Hex Head ASTM F 1554 GR. 105 2

Item number: not available

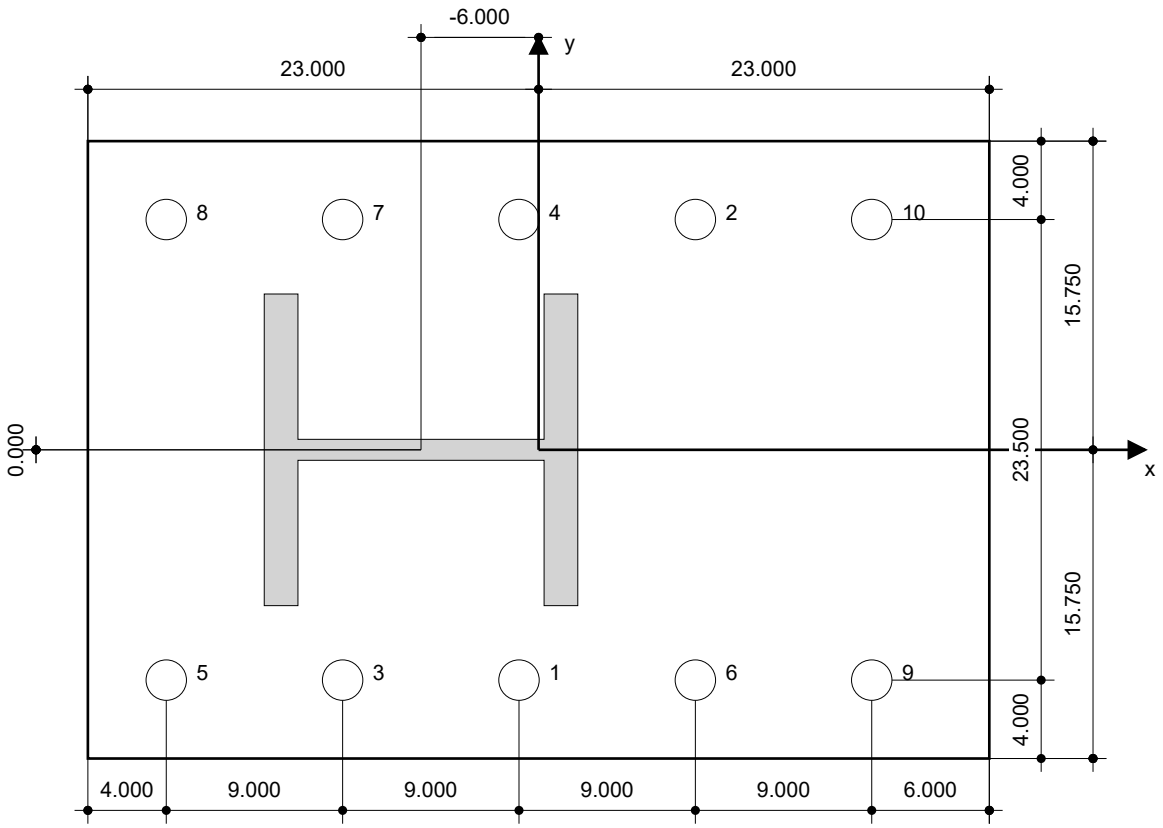
Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 24.000 in.

Minimum thickness of the base material: 25.844 in.

⌀ 2 in Heavy Hex Head ASTM F 1554 GR. 105 with 24 in nominal embedment depth per Technical data , cast in place installation per MPII



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-1.000	-11.750	41.000	31.000	12.250	35.750
2	8.000	11.750	50.000	22.000	35.750	12.250
3	-10.000	-11.750	32.000	40.000	12.250	35.750
4	-1.000	11.750	41.000	31.000	35.750	12.250
5	-19.000	-11.750	23.000	49.000	12.250	35.750

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
6	8.000	-11.750	50.000	22.000	12.250	35.750
7	-10.000	11.750	32.000	40.000	35.750	12.250
8	-19.000	11.750	23.000	49.000	35.750	12.250
9	17.000	-11.750	59.000	13.000	12.250	35.750
10	17.000	11.750	59.000	13.000	35.750	12.250

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Company:		Page:	10
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis DH BP Brace (Single Brace)	Date:	4/24/2025
Fastening point:			

2 Anchor plate design

2.1 Input data

Anchor plate:	Shape: Rectangular l_x x l_y x t = 46.000 in x 31.500 in x 4.000 in Calculation: CBFEM Material: ASTM A572 Gr.50; F_y = 50,000 psi; ε_{lim} = 5.00%
Anchor type and size:	Heavy Hex Head ASTM F 1554 GR. 105 2, h_{ef} = 24.000 in
Anchor stiffness:	The anchor is modeled considering stiffness values determined from load displacement curves tested in an independent laboratory. Please note that no simple replacement of the anchor is possible as the anchor stiffness has a major impact on the load distribution results.
Design method:	AISC and LRFD-based design using component-based FEM
Stand-off installation:	e_b = 0.000 in (No stand-off); t = 4.000 in
Profile:	W14X233; (L x W x T x FT) = 16.000 in x 15.900 in x 1.070 in x 1.720 in Material: ASTM A992; F_y = 50,000 psi; ε_{lim} = 5.00% Eccentricity x: -6.000 in Eccentricity y: -0.000 in
Base material:	Cracked concrete; Custom; $f_{c,cyl}$ = 4,500 psi; h = 60.000 in; E = 4,351,132 psi; G = 1,740,453 psi; v = 0.20; D = 156.07 lb/ft³
Welds (profile to anchor plate):	Type of redistribution: Plastic Material: E70xx
Mesh size:	Number of elements on edge: 8 Min. size of element: 0.394 in Max. size of element: 1.969 in

2.2 Summary

	Description	Profile		Anchor plate			Concrete [%]
		σ_{Ed} [psi]	ε_{Pl} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Hole bearing [%]	
1	Combination 1	50,057	0.19	26,854	0.00	1	5

2.3 Anchor plate classification

Results below are displayed for the decisive load combinations: Combination 1

Anchor tension forces	Equivalent rigid anchor plate (CBFEM)	Component-based Finite Element Method (CBFEM) anchor plate design
Anchor 1	116.004 kip	174.553 kip
Anchor 2	83.781 kip	101.472 kip
Anchor 3	148.224 kip	179.597 kip
Anchor 4	116.006 kip	174.698 kip
Anchor 5	180.442 kip	128.440 kip
Anchor 6	83.779 kip	101.416 kip
Anchor 7	148.227 kip	179.731 kip
Anchor 8	180.444 kip	128.090 kip
Anchor 9	51.551 kip	12.073 kip
Anchor 10	51.554 kip	12.226 kip

User accepted to consider the selected anchor plate as rigid by his/her engineering judgement. This means the anchor design guidelines can be applied.

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	11
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis DH BP Brace (Single Brace)	Date:	4/24/2025
Fastening point:			

2.4 Profile/Stiffeners/Plate

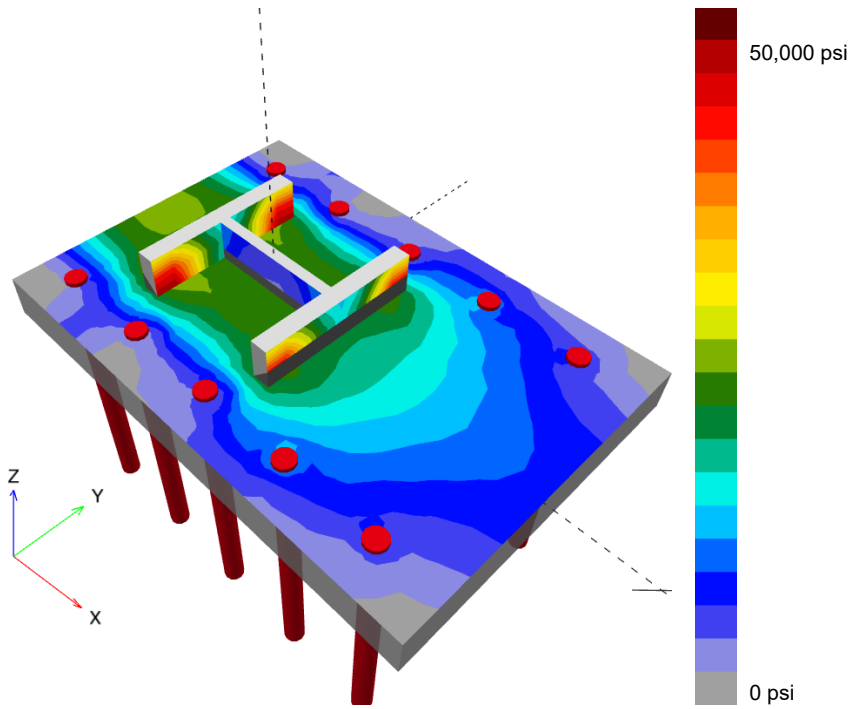
Profile and stiffeners are verified at the level of the steel to concrete connection. The connection design does not replace the steel design for critical cross sections, which should be performed outside of PROFIS Engineering.

2.4.1 Equivalent stress and plastic strain

Part	Load combination	Material	f_y [psi]	ε_{lim} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Status
Plate	Combination 1	ASTM A572 Gr.50	50,000	5.00	26,854	0.00	OK
Profile	Combination 1	ASTM A992	50,000	5.00	50,037	0.13	OK
Profile	Combination 1	ASTM A992	50,000	5.00	50,057	0.19	OK
Profile	Combination 1	ASTM A992	50,000	5.00	16,553	0.00	OK

2.4.1.1 Equivalent stress

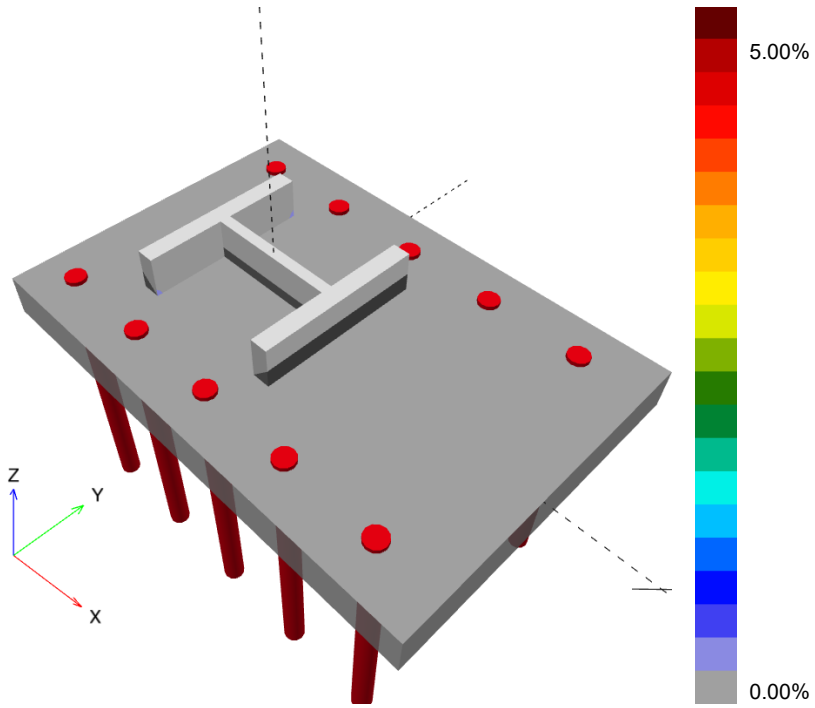
Results below are displayed for the decisive load combination: 1 - Combination 1



Company:		Page:	12
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis DH BP Brace (Single Brace)	Date:	4/24/2025
Fastening point:			

2.4.1.2 Plastic strain

Results below are displayed for the decisive load combination: 1 - Combination 1



2.4.2 Plate hole bearing resistance, AISC 360-16 Section J3

Decisive load combination: 1 - Combination 1

Equations

$R_n = \min(1.2\, l_c\, t\, F_u\, ,\ 2.4\, d\, t\, F_u)$ (AISC 360-16 J3-6a, c)

$\Phi R_n = 0.75\, R_n$

$V \leq \Phi R_n$

Variables

	l_c [in]	t [in]	F_u [psi]	d [in]	R_n [kip]
Anchor 1	2.980	4.000	65,000	2.000	929.741
Anchor 2	2.970	4.000	65,000	2.000	926.514
Anchor 3	2.971	4.000	65,000	2.000	927.053
Anchor 4	2.998	4.000	65,000	2.000	935.381
Anchor 5	2.974	4.000	65,000	2.000	927.879
Anchor 6	3.041	4.000	65,000	2.000	948.791
Anchor 7	3.014	4.000	65,000	2.000	940.376
Anchor 8	3.655	4.000	65,000	2.000	1,140.228
Anchor 9	3.300	4.000	65,000	2.000	1,029.483
Anchor 10	27.239	4.000	65,000	2.000	1,247.999

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	13
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis DH BP Brace (Single Brace)	Date:	4/24/2025
Fastening point:			

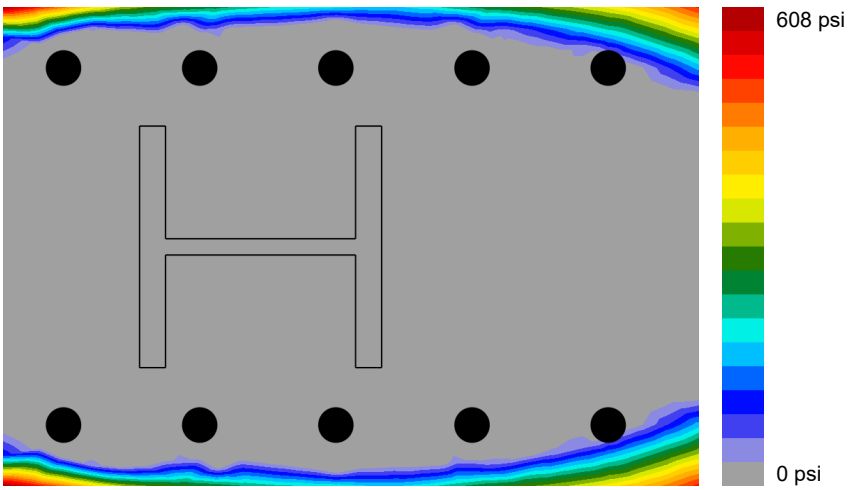
Results

	V [kip]	ΦR_n [kip]	Utilization [%]	Status
Anchor 1	2.646	697.306	1	OK
Anchor 2	0.271	694.885	1	OK
Anchor 3	2.831	695.289	1	OK
Anchor 4	0.990	701.536	1	OK
Anchor 5	2.298	695.909	1	OK
Anchor 6	1.664	711.593	1	OK
Anchor 7	0.985	705.282	1	OK
Anchor 8	0.453	855.171	1	OK
Anchor 9	0.675	772.112	1	OK
Anchor 10	0.455	935.999	1	OK

2.5 Concrete

Decisive load combination: 1 - Combination 1

2.5.1 Compression in concrete under the anchor plate



2.5.2 Concrete block compressive strength resistance check, AISC 360-16 Section J8

Equations

$$F_p = \Phi f_{p,max}$$

$$f_{p,max} = 0.85 f_c' \sqrt{\left(\frac{A_2}{A_1}\right)} \le 1.7 f_c; \quad \sqrt{\left(\frac{A_2}{A_1}\right)} \le 2$$

$$\sigma = \frac{N}{A_1}$$

$$\text{Utilization} = \frac{\sigma}{F_p}$$

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Company:		Page:	14
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis DH BP Brace (Single Brace)	Date:	4/24/2025
Fastening point:			

Variables

N [kip]	f_c' [psi]	Φ	A_1 [in²]	A_2 [in²]
36.376	4,500	0.65	166.51	1,918.17

Results

Load combination	F_p [psi]	σ [psi]	Utilization [%]	Status
Combination 1	4,973	218	5	OK

2.6 Symbol explanation

A_1	Loaded area of concrete
A_2	Supporting area
d	Nominal diameter of the bolt
ε_{lim}	Limit plastic strain
ε_{Pl}	Plastic strain from CBFEM results
f_c	Concrete compressive strength
f_c'	Concrete compressive strength
F_u	Specified minimum tensile strength of the connected material
F_p	Concrete block design bearing strength
$f_{p,max}$	Concrete block design bearing strength maximum
f_y	Yield strength
l_c	Clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material
N	Resulting compression force
σ	Average stress in concrete
σ_{Ed}	Equivalent stress
Φ	Resistance factor
ΦR_n	Factored resistance
t	Thickness of the anchor plate
V	Resultant of shear forces Vy, Vz in bolt.

2.7 Warnings

- By using the CBFEM calculation functionality of PROFIS Engineering you may act outside the applicable design codes and your specified anchor plate may not behave rigid. Please, validate the results with a professional designer and/or structural engineer to ensure suitability and adequacy for your specific jurisdiction and project requirements.
- The anchor is modeled considering stiffness values determined from load displacement curves tested in an independent laboratory. Please note that no simple replacement of the anchor is possible as the anchor stiffness has a major impact on the load distribution results.

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	15
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis DH BP Brace (Single Brace)	Date:	4/24/2025
Fastening point:			

3 Summary of results

Design of the anchor plate, anchors, welds and other elements are based on CBFEM (component based finite element method) and AISC.

	Load combination	Max. utilization	Status
Anchors	Combination 1	1,342%	NOT OK
Anchor plate	Combination 1	54%	OK
Concrete	Combination 1	5%	OK
Profile	Combination 1	100%	OK

Fastening does not meet the design criteria!

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	16
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis DH BP Brace (Single Brace)	Date:	4/24/2025
Fastening point:			

4 Remarks; Your Cooperation Duties

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Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Weak Axis FOH BP Brace (size) - no HSS	Date:	4/24/2025
Fastening point:			

Specifier's comments:

1 Anchor Design

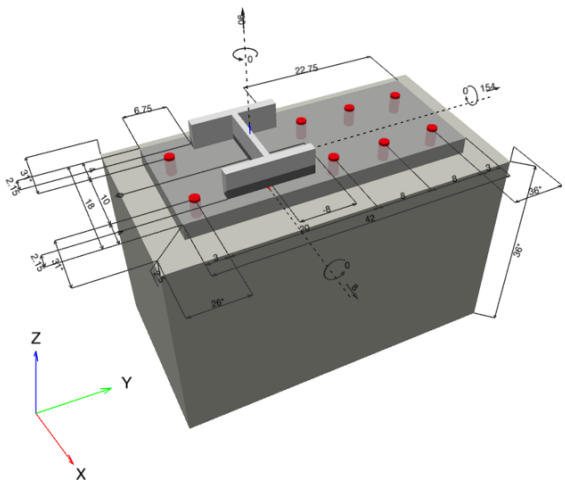
1.1 Input data



Anchor type and diameter:	**Heavy Hex Head ASTM F 1554 GR. 105 1 1/2**
Item number:	not available
Specification text:	∅ 1 1/2 in Heavy Hex Head ASTM F 1554 GR. 105 with 24 in nominal embedment depth per Technical data , cast in place installation per MPII,
Effective embedment depth:	h_{ef} = 24.000 in.
Material:	ASTM F 1554
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-19 / CIP
Shear edge breakout verification:	Row closest to edge (Case 3 only from ACI 318-19 Fig. R.17.7.2.1b)
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 2.500 in.
Anchor plate[CBFEM] :	l_x x l_y x t = 18.000 in. x 42.000 in. x 2.500 in.;
Profile:	W shape (AISC), W12X152; (L x W x T x FT) = 13.700 in. x 12.500 in. x 0.870 in. x 1.400 in.
Base material:	cracked concrete, Custom, f_c' = 4,500 psi; h = 36.000 in.
Reinforcement:	tension: present, shear: not present; edge reinforcement: none or < No. 4 bar

[CBFEM] - The anchor calculation is based on a component-based Finite Element Method (CBFEM)

Geometry [in.] & Loading [kip, ft.kip]



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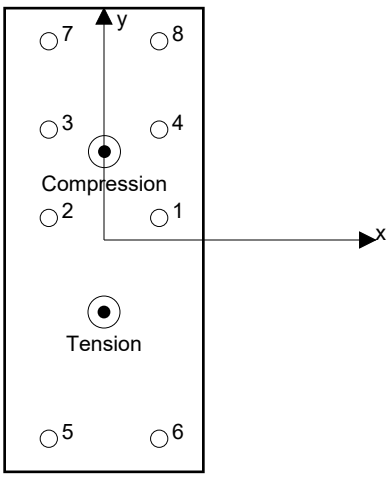
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Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Weak Axis FOH BP Brace (size) - no HSS	Date:	4/24/2025
Fastening point:			

1.1.1 Design results

Case	Description	Forces [kip] / Moments [ft.kip]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 90.000; V_x = 8.000; V_y = 154.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	325

1.2 Load case/Resulting anchor forces



Anchor reactions [kip]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	27.986	19.776	1.298	19.733
2	28.028	19.192	0.980	19.167
3	0.000	18.342	0.506	18.335
4	0.000	18.944	0.734	18.930
5	20.794	20.615	2.030	20.515
6	20.822	20.996	2.165	20.884
7	0.000	17.920	0.096	17.920
8	0.000	18.517	0.192	18.516

Resulting tension force in (x/y)=(-0.001/-6.525): 97.630 [kip]
Resulting compression force in (x/y)=(0.043/8.005): 8.310 [kip]

Anchor forces are calculated based on a component-based Finite Element Method (CBFEM)

1.3 Tension load

	Load N_{ua} [kip]	Capacity ϕ N_n [kip]	Utilization $\beta_N = N_{ua}/\phi N_n$	Status
Steel Strength*	28.028	132.187	22	OK
Pullout Strength*	28.028	78.574	36	OK
Concrete Breakout Failure**	97.630	164.138	60	OK
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (anchors in tension)

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Company:		Page:	3
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Weak Axis FOH BP Brace (size) - no HSS	Date:	4/24/2025
Fastening point:			

1.3.1 Steel Strength

$N_{sa} = A_{se,N} f_{uta}$ ACI 318-19 Eq. (17.6.1.2)
$\phi N_{sa} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
1.41	125,001

Calculations

N_{sa} [kip]
176.250

Results

N_{sa} [kip]	ϕ_{steel}	ϕN_{sa} [kip]	N_{ua} [kip]
176.250	0.750	132.187	28.028

1.3.2 Pullout Strength

$N_{pN} = \psi_{c,p} N_p$ ACI 318-19 Eq. (17.6.3.1)
$N_p = 8 A_{brg} f'_c$ ACI 318-19 Eq. (17.6.3.2.2a)
$\phi N_{pN} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$\psi_{c,p}$	A_{brg} [in.^{2}]	λ_a	f'_c [psi]
1.000	3.12	1.000	4,500

Calculations

N_p [kip]
112.248

Results

N_{pn} [kip]	$\phi_{concrete}$	ϕN_{pn} [kip]	N_{ua} [kip]
112.248	0.700	78.574	28.028

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Company:		Page:	4
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Weak Axis FOH BP Brace (size) - no HSS	Date:	4/24/2025
Fastening point:			

1.3.3 Concrete Breakout Failure

$N_{cbg} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ec,N}\ \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b$ — ACI 318-19 Eq. (17.6.2.1b)

$\phi\ N_{cbg} \geq N_{ua}$ — ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9\ h_{ef}^2$ — ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\ e'_N}{3\ h_{ef}}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3\left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = 16\ \lambda_a\ \sqrt{f'_c}\ h_{ef}^{5/3}$ — ACI 318-19 Eq. (17.6.2.2.3)

Variables

h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
20.667	0.001	1.475	26.000	1.000

c_{ac} [in.]	k_c	λ_a	f'_c [psi]
-	16	1.000	4,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [kip]
5,544.00	3,844.00	1.000	0.955	0.952	1.000	167.049

Results

N_{cbg} [kip]	$\phi_{concrete}$	$\phi\ N_{cbg}$ [kip]	N_{ua} [kip]
218.851	0.750	164.138	97.630

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Company:		Page:	5
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Weak Axis FOH BP Brace (size) - no HSS	Date:	4/24/2025
Fastening point:			

1.4 Shear load

	Load V_{ua} [kip]	Capacity ϕ V_n [kip]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	20.996	68.737	31	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	154.208	380.310	41	OK
Concrete edge failure in direction y+**	154.208	47.528	325 48.9	not recommended

* highest loaded anchor **anchor group (relevant anchors)

SEE SUPPLEMENTAL CALCULATIONS

1.4.1 Steel Strength

$V_{sa} = 0.6\ A_{se,V}\ f_{uta}$ ACI 318-19 Eq. (17.7.1.2b)
$\phi\ V_{steel} \geq V_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
1.41	125,001

Calculations

V_{sa} [kip]
105.750

Results

V_{sa} [kip]	ϕ_{steel}	$\phi\ V_{sa}$ [kip]	V_{ua} [kip]
105.750	0.650	68.737	20.996

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Company:		Page:	6
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Weak Axis FOH BP Brace (size) - no HSS	Date:	4/24/2025
Fastening point:			

1.4.2 Pryout Strength

$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \right]$ ACI 318-19 Eq. (17.7.3.1b)

$\phi V_{cpg} \geq V_{ua}$ ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9 h_{ef}^2$ ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e'_N}{3 h_{ef}}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = 16 \lambda_a \sqrt{f'_c} h_{ef}^{5/3}$ ACI 318-19 Eq. (17.6.2.2.3)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	20.667	0.570	0.030	26.000

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.000	-	16	1.000	4,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [kip]
6,696.00	3,844.00	0.982	0.999	0.952	1.000	167.049

Results

V_{cpg} [kip]	$\phi_{concrete}$	ϕV_{cpg} [kip]	V_{ua} [kip]
543.300	0.700	380.310	154.208

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	7
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Weak Axis FOH BP Brace (size) - no HSS	Date:	4/24/2025
Fastening point:			

1.4.3 Concrete edge failure in direction y+

$V_{cbg} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \Psi_{ec,V} \Psi_{ed,V} \Psi_{c,V} \Psi_{h,V} \Psi_{parallel,V} V_b$ — ACI 318-19 Eq. (17.7.2.1b)

$\phi V_{cbg} \geq V_{ua}$ — ACI 318-19 Table 17.5.2

A_{Vc} see ACI 318-19, Section 17.7.2.1, Fig. R 17.7.2.1(b)*

$A_{Vc0} = 4.5 c_{a1}^2$ — ACI 318-19 Eq. (17.7.2.1.3)

$\Psi_{ec,V} = \left(\frac{1}{1 + \frac{e'_v}{1.5c_{a1}}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.3.1)

$\Psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.4.1b)

$\Psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ — ACI 318-19 Eq. (17.7.2.6.1)

$V_b = 9 \lambda_a \sqrt{f'_c} c_{a1}^{1.5}$ — ACI 318-19 Eq. (17.7.2.2.1b)

Variables

c_{a1} [in.]	c_{a2} [in.]	e_{cV} [in.]	$\Psi_{c,V}$	h_a [in.]
24.000	31.000	0.069	1.000	36.000

l_e [in.]	λ_a	d_a [in.]	f'_c [psi]	$\Psi_{parallel,V}$
12.000	1.000	1.500	4,500	1.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{h,V}$	V_b [kip]
2,592.00	2,592.00	0.998	0.958	1.000	70.985

Results

V_{cbg} [kip]	$\phi_{concrete}$	ϕV_{cbg} [kip]	V_{ua} [kip]
67.897	0.700	47.528	154.208

*Anchor row defined by: Anchor 7, 8; Case 3 controls

1.5 Combined tension and shear loads, per ACI 318-19 section 17.8

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.595	~~3.245~~ 0.489	1.000	~~320~~ 90.3	not recommended

$\beta_{NV} = (\beta_N + \beta_V) / 1.2 <= 1$

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	8
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Weak Axis FOH BP Brace (size) - no HSS	Date:	4/24/2025
Fastening point:			

1.6 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates as per current regulations (ETAG 001/Annex C, EOTA TR029, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid base plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- The equations presented in this report are based on imperial units. When inputs are displayed in metric units, the user should be aware that the equations remain in their imperial format.
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- For additional information about ACI 318 strength design provisions, please go to https://viewer.joomag.com/profis-design-guide-us-en-summer-2021/0841849001625154758?short&/
- The anchor design methods in PROFIS Engineering require rigid anchor plates, as per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means that the anchor plate should be sufficiently rigid to prevent load re-distribution to the anchors due to elastic/plastic displacements. The user accepts that the anchor plate is considered close to rigid by engineering judgment."

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	9
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Weak Axis FOH BP Brace (size) - no HSS	Date:	4/24/2025
Fastening point:			

1.7 Installation data

Profile: W shape (AISC), W12X152; (L x W x T x FT) = 13.700 in. x 12.500 in. x 0.870 in. x 1.400 in.

Hole diameter in the fixture: d_f = 1.562 in.

Plate thickness (input): 2.500 in.

Anchor type and diameter: Heavy Hex Head ASTM F 1554 GR. 105 1 1/2

Item number: not available

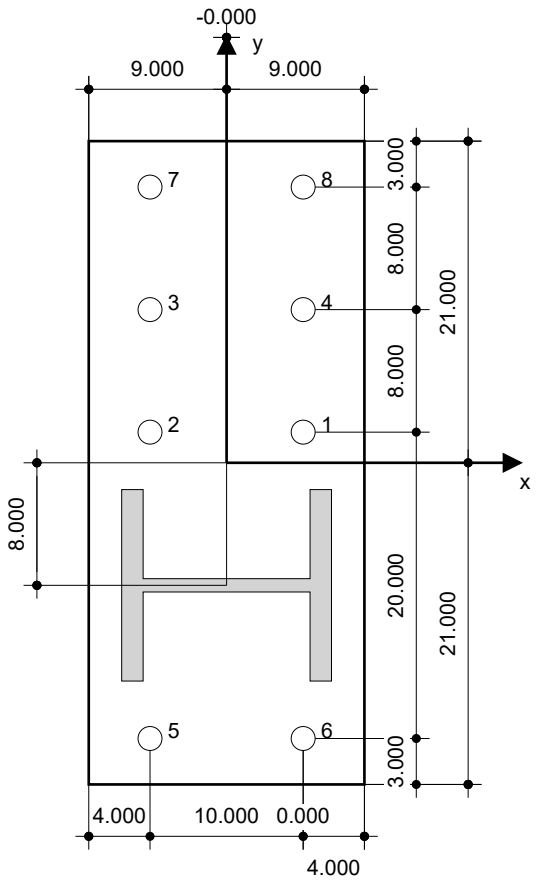
Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 24.000 in.

Minimum thickness of the base material: 25.500 in.

⌀ 1 1/2 in Heavy Hex Head ASTM F 1554 GR. 105 with 24 in nominal embedment depth per Technical data , cast in place installation per MPII



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	5.000	2.000	41.000	31.000	46.000	52.000
2	-5.000	2.000	31.000	41.000	46.000	52.000
3	-5.000	10.000	31.000	41.000	54.000	44.000
4	5.000	10.000	41.000	31.000	54.000	44.000

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
5	-5.000	-18.000	31.000	41.000	26.000	72.000
6	5.000	-18.000	41.000	31.000	26.000	72.000
7	-5.000	18.000	31.000	41.000	62.000	36.000
8	5.000	18.000	41.000	31.000	62.000	36.000

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Company:		Page:	10
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Weak Axis FOH BP Brace (size) - no HSS	Date:	4/24/2025
Fastening point:			

2 Anchor plate design

2.1 Input data

Anchor plate:	Shape: Rectangular l_x x l_y x t = 18.000 in x 42.000 in x 2.500 in Calculation: CBFEM Material: ASTM A572 Gr.50; F_y = 50,000 psi; ε_{lim} = 5.00%
Anchor type and size:	Heavy Hex Head ASTM F 1554 GR. 105 1 1/2, h_{ef} = 24.000 in
Anchor stiffness:	The anchor is modeled considering stiffness values determined from load displacement curves tested in an independent laboratory. Please note that no simple replacement of the anchor is possible as the anchor stiffness has a major impact on the load distribution results.
Design method:	AISC and LRFD-based design using component-based FEM
Stand-off installation:	e_b = 0.000 in (No stand-off); t = 2.500 in
Profile:	W12X152; (L x W x T x FT) = 13.700 in x 12.500 in x 0.870 in x 1.400 in Material: ASTM A992; F_y = 50,000 psi; ε_{lim} = 5.00% Eccentricity x: -0.000 in Eccentricity y: -8.000 in
Base material:	Cracked concrete; Custom; $f_{c,cyl}$ = 4,500 psi; h = 36.000 in; E = 4,351,132 psi; G = 1,740,453 psi; v = 0.20; D = 156.07 lb/ft³
Welds (profile to anchor plate):	Type of redistribution: Plastic Material: E70xx
Mesh size:	Number of elements on edge: 8 Min. size of element: 0.394 in Max. size of element: 1.969 in

2.2 Summary

	Description	Profile		Anchor plate			Concrete [%]
		σ_{Ed} [psi]	ε_{Pl} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Hole bearing [%]	
1	Combination 1	41,407	0.00	8,707	0.00	7	1

2.3 Anchor plate classification

Results below are displayed for the decisive load combinations: Combination 1

Anchor tension forces	Equivalent rigid anchor plate (CBFEM)	Component-based Finite Element Method (CBFEM) anchor plate design
Anchor 1	11.966 kip	27.986 kip
Anchor 2	11.959 kip	28.028 kip
Anchor 3	6.461 kip	0.000 kip
Anchor 4	6.468 kip	0.000 kip
Anchor 5	25.694 kip	20.794 kip
Anchor 6	25.702 kip	20.822 kip
Anchor 7	0.963 kip	0.000 kip
Anchor 8	0.970 kip	0.000 kip

User accepted to consider the selected anchor plate as rigid by his/her engineering judgement. This means the anchor design guidelines can be applied.

2.4 Profile/Stiffeners/Plate

Profile and stiffeners are verified at the level of the steel to concrete connection. The connection design does not replace the steel design for critical cross sections, which should be performed outside of PROFIS Engineering.

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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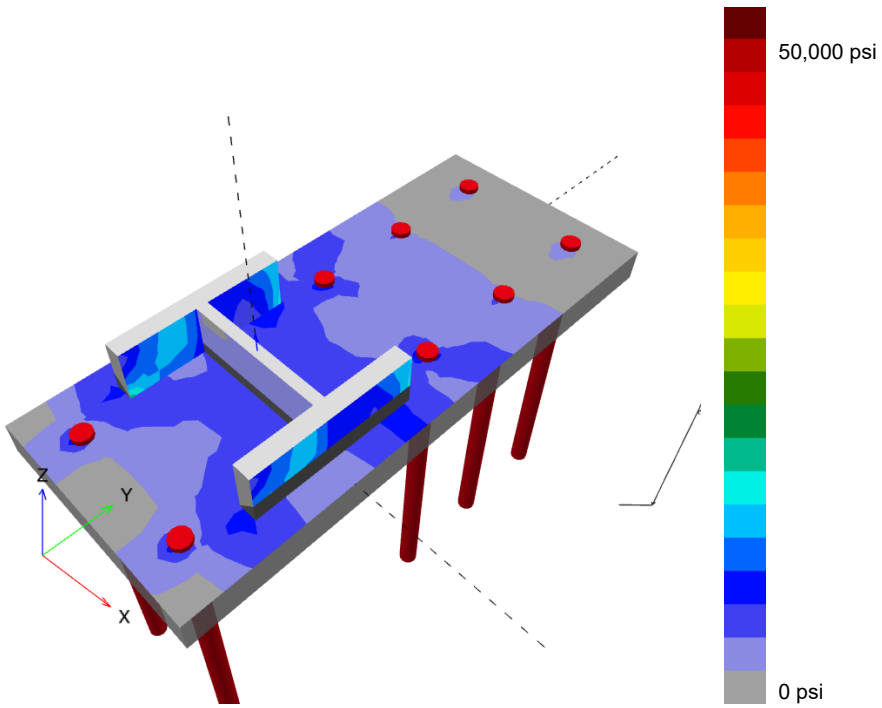
Company:		Page:	11
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Weak Axis FOH BP Brace (size) - no HSS	Date:	4/24/2025
Fastening point:			

2.4.1 Equivalent stress and plastic strain

Part	Load combination	Material	f_y [psi]	ε_{lim} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Status
Plate	Combination 1	ASTM A572 Gr.50	50,000	5.00	8,707	0.00	OK
Profile	Combination 1	ASTM A992	50,000	5.00	41,407	0.00	OK
Profile	Combination 1	ASTM A992	50,000	5.00	40,179	0.00	OK
Profile	Combination 1	ASTM A992	50,000	5.00	5,725	0.00	OK

2.4.1.1 Equivalent stress

Results below are displayed for the decisive load combination: 1 - Combination 1

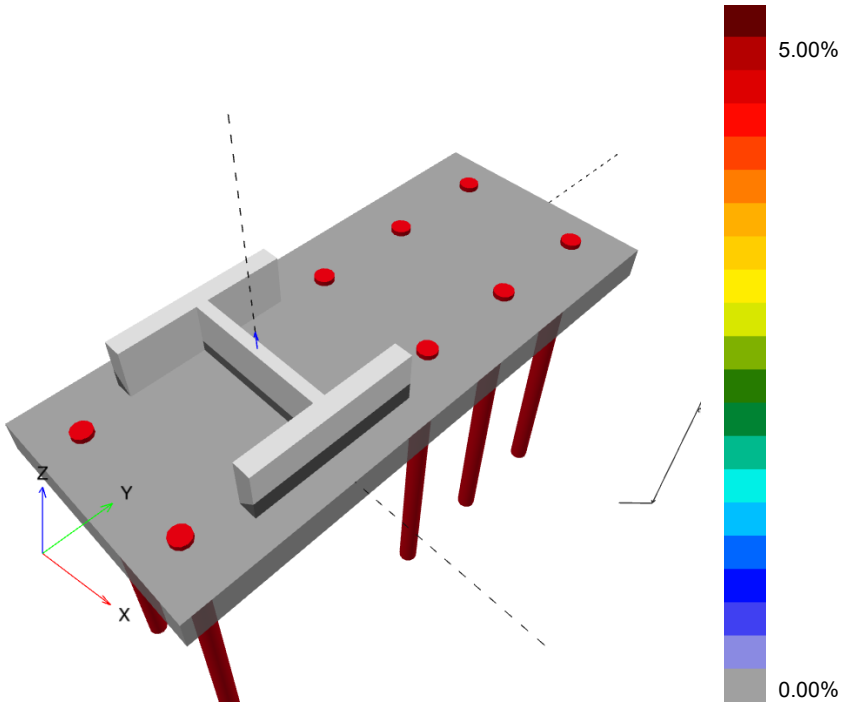


Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	12
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Weak Axis FOH BP Brace (size) - no HSS	Date:	4/24/2025
Fastening point:			

2.4.1.2 Plastic strain

Results below are displayed for the decisive load combination: 1 - Combination 1



2.4.2 Plate hole bearing resistance, AISC 360-16 Section J3

Decisive load combination: 1 - Combination 1

Equations

$R_n = \min(1.2\, l_c\, t\, F_u\, ,\ 2.4\, d\, t\, F_u)$ (AISC 360-16 J3-6a, c)

$\Phi R_n = 0.75\, R_n$

$V \leq \Phi R_n$

Variables

	l_c [in]	t [in]	F_u [psi]	d [in]	R_n [kip]
Anchor 1	18.438	2.500	65,000	1.500	585.000
Anchor 2	18.438	2.500	65,000	1.500	585.000
Anchor 3	6.438	2.500	65,000	1.500	585.000
Anchor 4	6.438	2.500	65,000	1.500	585.000
Anchor 5	2.233	2.500	65,000	1.500	435.513
Anchor 6	2.235	2.500	65,000	1.500	435.790
Anchor 7	6.438	2.500	65,000	1.500	585.000
Anchor 8	6.438	2.500	65,000	1.500	585.000

Results

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	13
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Weak Axis FOH BP Brace (size) - no HSS	Date:	4/24/2025
Fastening point:			

	V [kip]	ΦR_n [kip]	Utilization [%]	Status
Anchor 1	19.776	438.750	5	OK
Anchor 2	19.192	438.750	5	OK
Anchor 3	18.342	438.750	5	OK
Anchor 4	18.944	438.750	5	OK
Anchor 5	20.615	326.635	7	OK
Anchor 6	20.996	326.843	7	OK
Anchor 7	17.920	438.750	5	OK
Anchor 8	18.517	438.750	5	OK

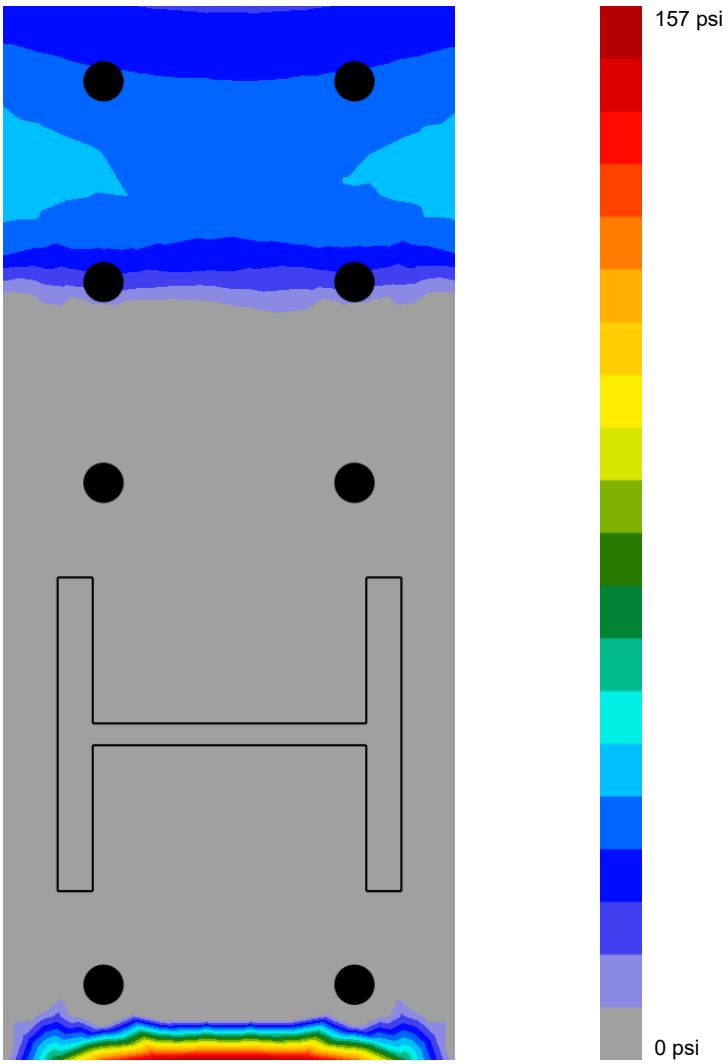
2.5 Concrete

Decisive load combination: 1 - Combination 1

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
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Fastening point:			

2.5.1 Compression in concrete under the anchor plate



2.5.2 Concrete block compressive strength resistance check, AISC 360-16 Section J8

Equations

$F_p = \Phi f_{p,max}$

$f_{p,max} = 0.85 f_c' \sqrt{\left(\frac{A_2}{A_1}\right)} \le 1.7 f_c; \quad \sqrt{\left(\frac{A_2}{A_1}\right)} \le 2$

$\sigma = \frac{N}{A_1}$

$\text{Utilization} = \frac{\sigma}{F_p}$

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	15
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Weak Axis FOH BP Brace (size) - no HSS	Date:	4/24/2025
Fastening point:			

Variables

N [kip]	f_c' [psi]	Φ	A_1 [in²]	A_2 [in²]
8.310	4,500	0.65	219.84	5,197.42

Results

Load combination	F_p [psi]	σ [psi]	Utilization [%]	Status
Combination 1	4,973	38	1	OK

2.6 Symbol explanation

A_1	Loaded area of concrete
A_2	Supporting area
d	Nominal diameter of the bolt
ε_{lim}	Limit plastic strain
ε_{Pl}	Plastic strain from CBFEM results
f_c	Concrete compressive strength
f_c'	Concrete compressive strength
F_u	Specified minimum tensile strength of the connected material
F_p	Concrete block design bearing strength
$f_{p,max}$	Concrete block design bearing strength maximum
f_y	Yield strength
l_c	Clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material
N	Resulting compression force
σ	Average stress in concrete
σ_{Ed}	Equivalent stress
Φ	Resistance factor
ΦR_n	Factored resistance
t	Thickness of the anchor plate
V	Resultant of shear forces Vy, Vz in bolt.

2.7 Warnings

- By using the CBFEM calculation functionality of PROFIS Engineering you may act outside the applicable design codes and your specified anchor plate may not behave rigid. Please, validate the results with a professional designer and/or structural engineer to ensure suitability and adequacy for your specific jurisdiction and project requirements.
- The anchor is modeled considering stiffness values determined from load displacement curves tested in an independent laboratory. Please note that no simple replacement of the anchor is possible as the anchor stiffness has a major impact on the load distribution results.

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	16
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Weak Axis FOH BP Brace (size) - no HSS	Date:	4/24/2025
Fastening point:			

3 Summary of results

Design of the anchor plate, anchors, welds and other elements are based on CBFEM (component based finite element method) and AISC.

	Load combination	Max. utilization	Status
Anchors	Combination 1	325%	NOT OK
Anchor plate	Combination 1	18%	OK
Concrete	Combination 1	1%	OK
Profile	Combination 1	83%	OK

Fastening does not meet the design criteria!

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	17
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Weak Axis FOH BP Brace (size) - no HSS	Date:	4/24/2025
Fastening point:			

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Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis DH BP Brace (Single Brace)	Date:	4/24/2025
Fastening point:			

Specifier's comments:

1 Anchor Design

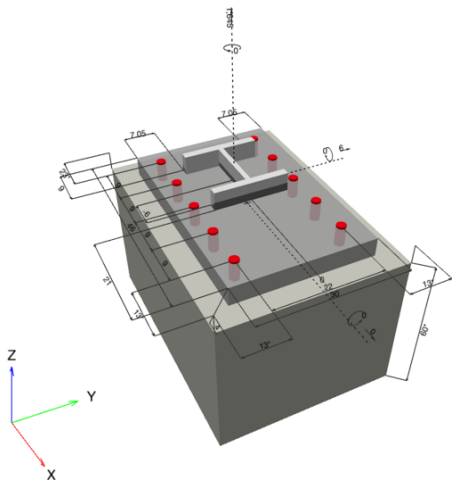
1.1 Input data



Anchor type and diameter:	**Heavy Hex Head ASTM F 1554 GR. 105 2**
Item number:	not available
Specification text:	∅ 2 in Heavy Hex Head ASTM F 1554 GR. 105 with 24 in nominal embedment depth per Technical data , cast in place installation per MPII,
Effective embedment depth:	h_{ef} = 24.000 in.
Material:	ASTM F 1554
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-19 / CIP
Shear edge breakout verification:	Row closest to edge (Case 3 only from ACI 318-19 Fig. R.17.7.2.1b)
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 4.000 in.
Anchor plate[CBFEM] :	l_x x l_y x t = 46.000 in. x 30.000 in. x 4.000 in.;
Profile:	W shape (AISC), W14X233; (L x W x T x FT) = 16.000 in. x 15.900 in. x 1.070 in. x 1.720 in.
Base material:	cracked concrete, Custom, f_c' = 4,500 psi; h = 60.000 in.
Reinforcement:	tension: present, shear: present; edge reinforcement: none or < No. 4 bar

[CBFEM] - The anchor calculation is based on a component-based Finite Element Method (CBFEM)

Geometry [in.] & Loading [kip, ft.kip]



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Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis DH BP Brace (Single Brace)	Date:	4/24/2025
Fastening point:			

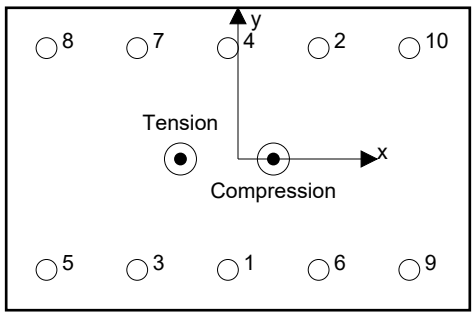
1.1.1 Design results

Case	Description	Forces [kip] / Moments [ft.kip]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 1,645.000; V_x = 0.000; V_y = 6.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	1,874

1.2 Load case/Resulting anchor forces

Anchor reactions [kip]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	250.401	2.707	-0.164	2.702
2	143.269	0.590	-0.083	-0.584
3	256.249	2.844	-0.060	2.844
4	250.261	1.464	0.083	-1.462
5	177.384	2.247	0.216	2.237
6	143.255	1.625	-0.314	1.594
7	255.785	1.436	0.126	-1.430
8	177.837	0.796	0.412	-0.681
9	13.944	0.555	-0.242	0.500
10	14.084	0.281	0.026	0.280



Resulting tension force in (x/y)=(-5.707/0.000): 1,682.468 [kip]
Resulting compression force in (x/y)=(3.513/-0.004): 43.864 [kip]

Anchor forces are calculated based on a component-based Finite Element Method (CBFEM)

1.3 Tension load

	Load N_{ua} [kip]	Capacity ϕ N_n [kip]	Utilization $\beta_N = N_{ua}/\phi$ N_n		Status
Steel Strength*	256.249	234.375	110	74.5	not recommended
Pullout Strength*	256.249	133.963	192	30.9	not recommended
Concrete Breakout Failure**	1,682.468	89.815	1,874	70.2	not recommended
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A		N/A

* highest loaded anchor **anchor group (anchors in tension)

SEE SUPPLEMENTAL CALCULATIONS

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Company:		Page:	3
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis DH BP Brace (Single Brace)	Date:	4/24/2025
Fastening point:			

1.3.1 Steel Strength

$N_{sa} = A_{se,N} f_{uta}$ ACI 318-19 Eq. (17.6.1.2)
$\phi N_{sa} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
2.50	125,001

Calculations

N_{sa} [kip]
312.500

Results

N_{sa} [kip]	ϕ_{steel}	ϕN_{sa} [kip]	N_{ua} [kip]
312.500	0.750	234.375	256.249

1.3.2 Pullout Strength

$N_{pN} = \psi_{c,p} N_p$ ACI 318-19 Eq. (17.6.3.1)
$N_p = 8 A_{brg} f_c'$ ACI 318-19 Eq. (17.6.3.2.2a)
$\phi N_{pN} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$\psi_{c,p}$	A_{brg} [in.^{2}]	λ_a	f_c' [psi]
1.000	5.32	1.000	4,500

Calculations

N_p [kip]
191.376

Results

N_{pn} [kip]	$\phi_{concrete}$	ϕN_{pn} [kip]	N_{ua} [kip]
191.376	0.700	133.963	256.249

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	4
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis DH BP Brace (Single Brace)	Date:	4/24/2025
Fastening point:			

1.3.3 Concrete Breakout Failure

$N_{cbg} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ec,N}\ \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b$ — ACI 318-19 Eq. (17.6.2.1b)

$\phi\ N_{cbg} \geq N_{ua}$ — ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9\ h_{ef}^2$ — ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\ e'_N}{3\ h_{ef}}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3\left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = 16\ \lambda_a\ \sqrt{f'_c}\ h_{ef}^{5/3}$ — ACI 318-19 Eq. (17.6.2.2.3)

Variables

h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
15.333	4.707	0.000	13.000	1.000

c_{ac} [in.]	k_c	λ_a	f'_c [psi]
-	16	1.000	4,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [kip]
3,456.00	2,116.00	0.830	1.000	0.870	1.000	101.575

Results

N_{cbg} [kip]	$\phi_{concrete}$	$\phi\ N_{cbg}$ [kip]	N_{ua} [kip]
119.754	0.750	89.815	1,682.468

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	5
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis DH BP Brace (Single Brace)	Date:	4/24/2025
Fastening point:			

1.4 Shear load

	Load V_{ua} [kip]	Capacity ϕ V_n [kip]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	2.844	121.875	3	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	6.000	165.900	4	OK
Concrete edge failure in direction x+**	10.193	22.961	45	OK

* highest loaded anchor **anchor group (relevant anchors)

1.4.1 Steel Strength

$V_{sa} = 0.6\ A_{se,V}\ f_{uta}$ ACI 318-19 Eq. (17.7.1.2b)
$\phi\ V_{steel} \geq V_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
2.50	125,001

Calculations

V_{sa} [kip]
187.500

Results

V_{sa} [kip]	ϕ_{steel}	$\phi\ V_{sa}$ [kip]	V_{ua} [kip]
187.500	0.650	121.875	2.844

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Company:		Page:	6
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis DH BP Brace (Single Brace)	Date:	4/24/2025
Fastening point:			

1.4.2 Pryout Strength

$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \right]$ ACI 318-19 Eq. (17.7.3.1b)

$\phi V_{cpg} \geq V_{ua}$ ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9 h_{ef}^2$ ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e'_N}{3 h_{ef}}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = 16 \lambda_a \sqrt{f'_c} h_{ef}^{5/3}$ ACI 318-19 Eq. (17.6.2.2.3)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	15.333	5.000	0.000	13.000

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.000	-	16	1.000	4,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [kip]
3,456.00	2,116.00	0.821	1.000	0.870	1.000	101.575

Results

V_{cpg} [kip]	$\phi_{concrete}$	ϕV_{cpg} [kip]	V_{ua} [kip]
236.999	0.700	165.900	6.000

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Hilti PROFIS Engineering 3.1.14

www.hilti.com

Company:		Page:	7
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis DH BP Brace (Single Brace)	Date:	4/24/2025
Fastening point:			

1.4.3 Concrete edge failure in direction x+

$V_{cbg} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \Psi_{ec,V} \Psi_{ed,V} \Psi_{c,V} \Psi_{h,V} \Psi_{parallel,V} V_b$ — ACI 318-19 Eq. (17.7.2.1b)

$\phi V_{cbg} \geq V_{ua}$ — ACI 318-19 Table 17.5.2

A_{Vc} see ACI 318-19, Section 17.7.2.1, Fig. R 17.7.2.1(b)*

$A_{Vc0} = 4.5 c_{a1}^2$ — ACI 318-19 Eq. (17.7.2.1.3)

$\Psi_{ec,V} = \left(\frac{1}{1 + \frac{e'_v}{1.5c_{a1}}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.3.1)

$\Psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.4.1b)

$\Psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ — ACI 318-19 Eq. (17.7.2.6.1)

$V_b = 9 \lambda_a \sqrt{f'_c} c_{a1}^{1.5}$ — ACI 318-19 Eq. (17.7.2.2.1b)

Variables

c_{a1} [in.]	c_{a2} [in.]	e_{cV} [in.]	$\Psi_{c,V}$	h_a [in.]
13.000	13.000	0.465	1.000	60.000

l_e [in.]	λ_a	d_a [in.]	f'_c [psi]	$\Psi_{parallel,V}$
16.000	1.000	2.000	4,500	1.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{h,V}$	V_b [kip]
936.00	760.50	0.977	0.900	1.000	28.299

Results

V_{cbg} [kip]	$\phi_{concrete}$	ϕV_{cbg} [kip]	V_{ua} [kip]
30.615	0.750	22.961	10.193

*Anchor row defined by: Anchor 9, 10; Case 3 controls

1.5 Combined tension and shear loads, per ACI 318-19 section 17.8

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
~~18.733~~ 0.745	0.444	1.000	~~1,599~~ 99.1	not recommended

$\beta_{NV} = (\beta_N + \beta_V) / 1.2 <= 1$

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Company:		Page:	8
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis DH BP Brace (Single Brace)	Date:	4/24/2025
Fastening point:			

1.6 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates as per current regulations (ETAG 001/Annex C, EOTA TR029, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid base plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- The equations presented in this report are based on imperial units. When inputs are displayed in metric units, the user should be aware that the equations remain in their imperial format.
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- For additional information about ACI 318 strength design provisions, please go to https://viewer.joomag.com/profis-design-guide-us-en-summer-2021/0841849001625154758?short&/
- The anchor design methods in PROFIS Engineering require rigid anchor plates, as per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means that the anchor plate should be sufficiently rigid to prevent load re-distribution to the anchors due to elastic/plastic displacements. The user accepts that the anchor plate is considered close to rigid by engineering judgment."

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	9
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis DH BP Brace (Single Brace)	Date:	4/24/2025
Fastening point:			

1.7 Installation data

Profile: W shape (AISC), W14X233; (L x W x T x FT) = 16.000 in. x 15.900 in. x 1.070 in. x 1.720 in.

Hole diameter in the fixture: d_f = 2.062 in.

Plate thickness (input): 4.000 in.

Anchor type and diameter: Heavy Hex Head ASTM F 1554 GR. 105 2

Item number: not available

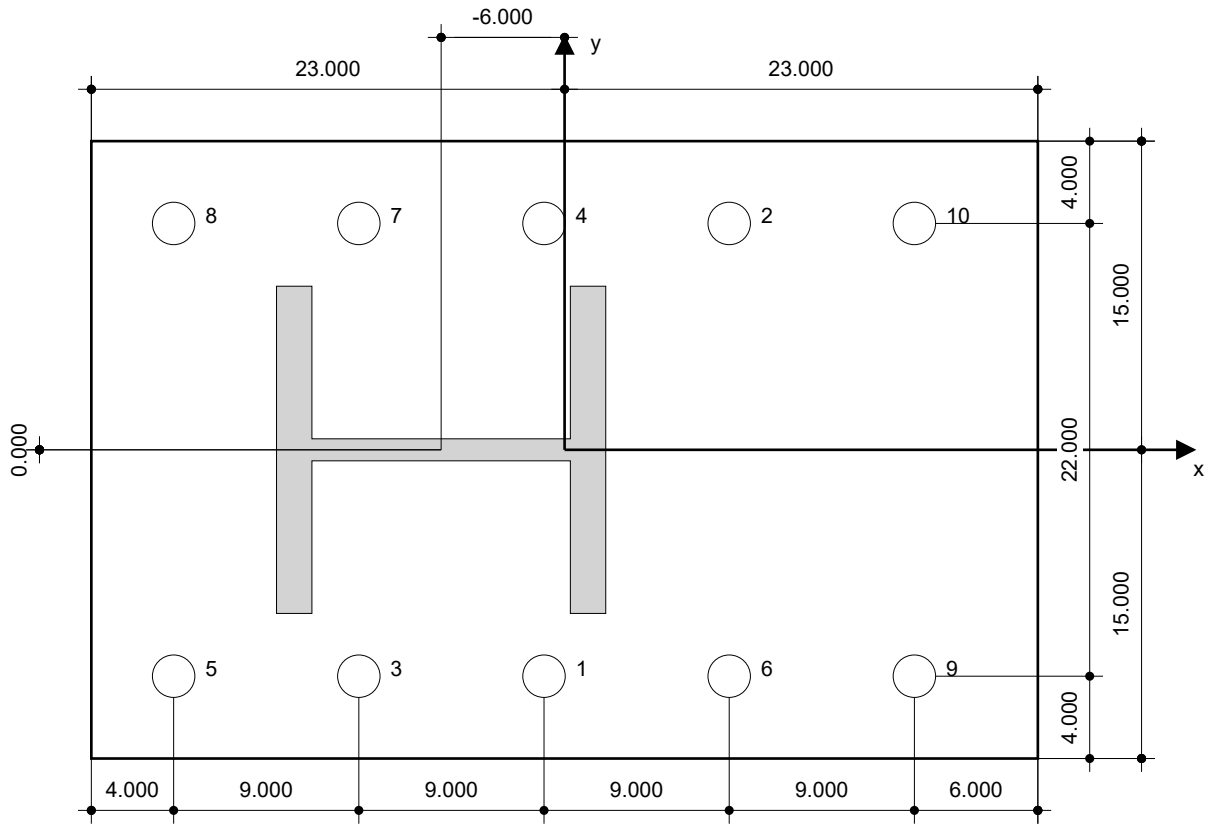
Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 24.000 in.

Minimum thickness of the base material: 25.844 in.

⌀ 2 in Heavy Hex Head ASTM F 1554 GR. 105 with 24 in nominal embedment depth per Technical data , cast in place installation per MPII



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-1.000	-11.000	41.000	31.000	13.000	35.000
2	8.000	11.000	50.000	22.000	35.000	13.000
3	-10.000	-11.000	32.000	40.000	13.000	35.000
4	-1.000	11.000	41.000	31.000	35.000	13.000
5	-19.000	-11.000	23.000	49.000	13.000	35.000

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
6	8.000	-11.000	50.000	22.000	13.000	35.000
7	-10.000	11.000	32.000	40.000	35.000	13.000
8	-19.000	11.000	23.000	49.000	35.000	13.000
9	17.000	-11.000	59.000	13.000	13.000	35.000
10	17.000	11.000	59.000	13.000	35.000	13.000

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Company:		Page:	10
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis DH BP Brace (Single Brace)	Date:	4/24/2025
Fastening point:			

2 Anchor plate design

2.1 Input data

Anchor plate:	Shape: Rectangular l_x x l_y x t = 46.000 in x 30.000 in x 4.000 in Calculation: CBFEM Material: ASTM A572 Gr.50; F_y = 50,000 psi; ε_{lim} = 5.00%
Anchor type and size:	Heavy Hex Head ASTM F 1554 GR. 105 2, h_{ef} = 24.000 in
Anchor stiffness:	The anchor is modeled considering stiffness values determined from load displacement curves tested in an independent laboratory. Please note that no simple replacement of the anchor is possible as the anchor stiffness has a major impact on the load distribution results.
Design method:	AISC and LRFD-based design using component-based FEM
Stand-off installation:	e_b = 0.000 in (No stand-off); t = 4.000 in
Profile:	W14X233; (L x W x T x FT) = 16.000 in x 15.900 in x 1.070 in x 1.720 in Material: ASTM A992; F_y = 50,000 psi; ε_{lim} = 5.00% Eccentricity x: -6.000 in Eccentricity y: -0.000 in
Base material:	Cracked concrete; Custom; $f_{c,cyl}$ = 4,500 psi; h = 60.000 in; E = 4,351,132 psi; G = 1,740,453 psi; v = 0.20; D = 156.07 lb/ft³
Welds (profile to anchor plate):	Type of redistribution: Plastic Material: E70xx
Mesh size:	Number of elements on edge: 8 Min. size of element: 0.394 in Max. size of element: 1.969 in

2.2 Summary

	Description	Profile		Anchor plate			Concrete [%]
		σ_{Ed} [psi]	ε_{Pl} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Hole bearing [%]	
1	Combination 1	50,149	0.51	37,031	0.00	1	6

2.3 Anchor plate classification

Results below are displayed for the decisive load combinations: Combination 1

Anchor tension forces	Equivalent rigid anchor plate (CBFEM)	Component-based Finite Element Method (CBFEM) anchor plate design
Anchor 1	164.497 kip	250.401 kip
Anchor 2	118.803 kip	143.269 kip
Anchor 3	210.187 kip	256.249 kip
Anchor 4	164.500 kip	250.261 kip
Anchor 5	255.871 kip	177.384 kip
Anchor 6	118.801 kip	143.255 kip
Anchor 7	210.189 kip	255.785 kip
Anchor 8	255.873 kip	177.837 kip
Anchor 9	73.101 kip	13.944 kip
Anchor 10	73.104 kip	14.084 kip

User accepted to consider the selected anchor plate as rigid by his/her engineering judgement. This means the anchor design guidelines can be applied.

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Company:		Page:	11
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis DH BP Brace (Single Brace)	Date:	4/24/2025
Fastening point:			

2.4 Profile/Stiffeners/Plate

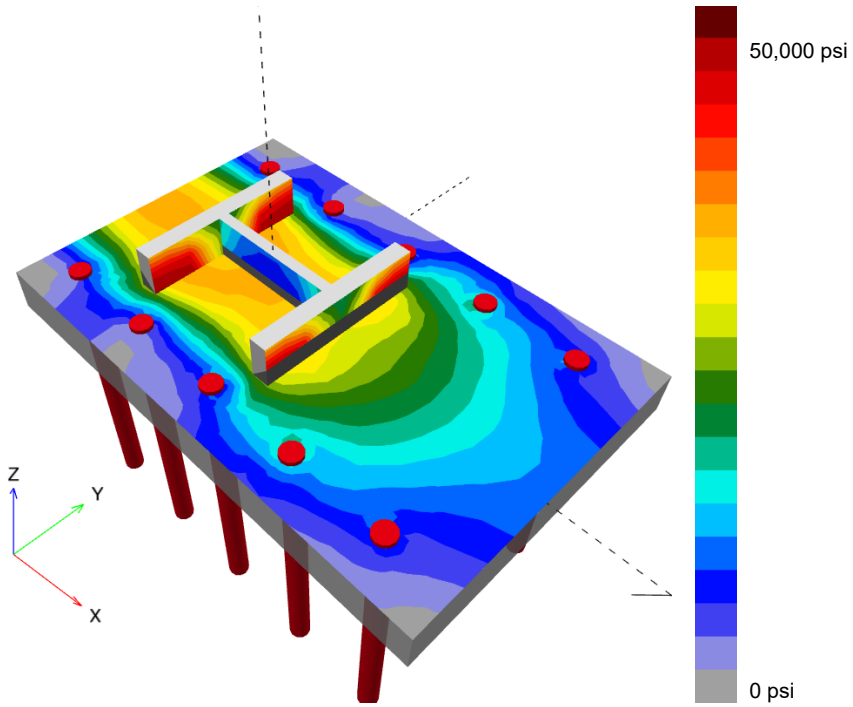
Profile and stiffeners are verified at the level of the steel to concrete connection. The connection design does not replace the steel design for critical cross sections, which should be performed outside of PROFIS Engineering.

2.4.1 Equivalent stress and plastic strain

Part	Load combination	Material	f_y [psi]	ε_{lim} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Status
Plate	Combination 1	ASTM A572 Gr.50	50,000	5.00	37,031	0.00	OK
Profile	Combination 1	ASTM A992	50,000	5.00	50,117	0.40	OK
Profile	Combination 1	ASTM A992	50,000	5.00	50,149	0.51	OK
Profile	Combination 1	ASTM A992	50,000	5.00	23,470	0.00	OK

2.4.1.1 Equivalent stress

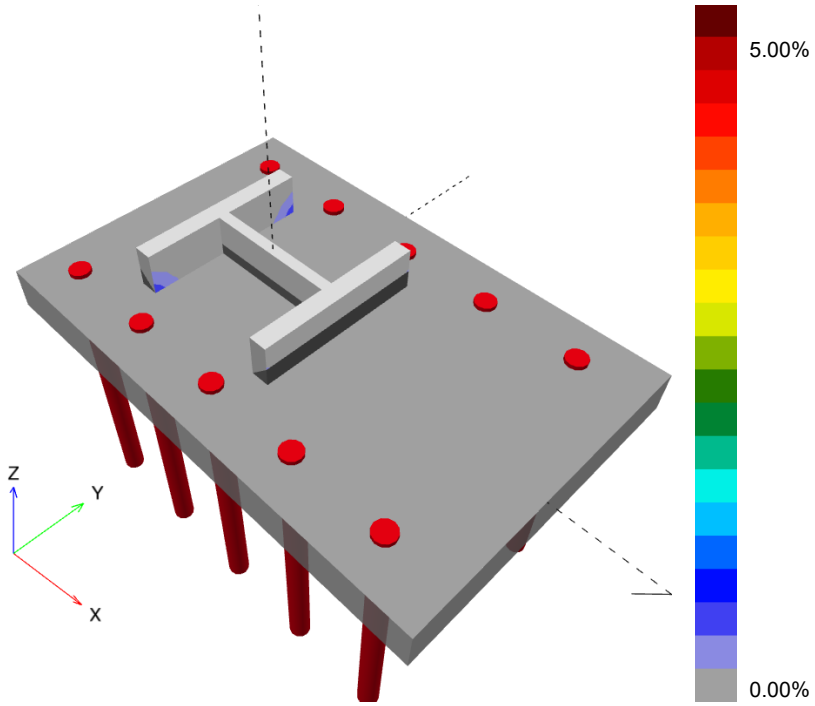
Results below are displayed for the decisive load combination: 1 - Combination 1



Company:		Page:	12
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis DH BP Brace (Single Brace)	Date:	4/24/2025
Fastening point:			

2.4.1.2 Plastic strain

Results below are displayed for the decisive load combination: 1 - Combination 1



2.4.2 Plate hole bearing resistance, AISC 360-16 Section J3

Decisive load combination: 1 - Combination 1

Equations

$R_n = \min(1.2\, l_c\, t\, F_u\, ,\ 2.4\, d\, t\, F_u)$ (AISC 360-16 J3-6a, c)

$\Phi R_n = 0.75\, R_n$

$V \leq \Phi R_n$

Variables

	l_c [in]	t [in]	F_u [psi]	d [in]	R_n [kip]
Anchor 1	2.976	4.000	65,000	2.000	928.540
Anchor 2	3.009	4.000	65,000	2.000	938.897
Anchor 3	2.970	4.000	65,000	2.000	926.526
Anchor 4	2.975	4.000	65,000	2.000	928.261
Anchor 5	2.987	4.000	65,000	2.000	932.044
Anchor 6	3.046	4.000	65,000	2.000	950.277
Anchor 7	2.984	4.000	65,000	2.000	931.087
Anchor 8	3.645	4.000	65,000	2.000	1,137.318
Anchor 9	3.412	4.000	65,000	2.000	1,064.525
Anchor 10	19.938	4.000	65,000	2.000	1,247.999

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Company:		Page:	13
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis DH BP Brace (Single Brace)	Date:	4/24/2025
Fastening point:			

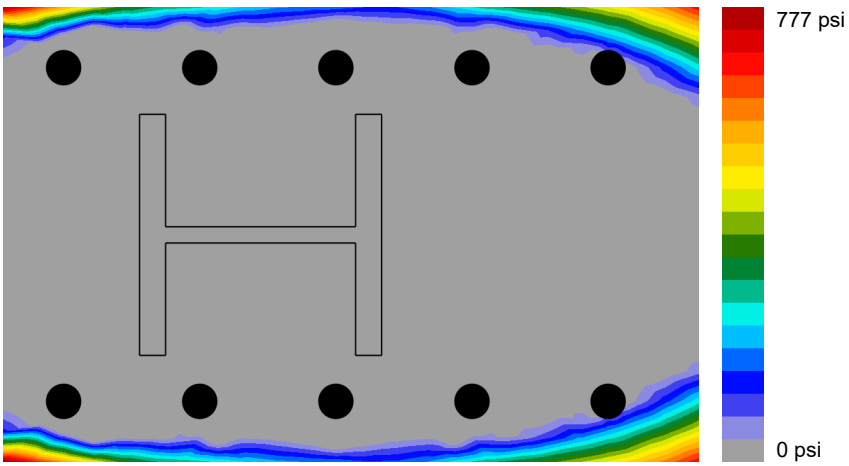
Results

	V [kip]	ΦR_n [kip]	Utilization [%]	Status
Anchor 1	2.707	696.405	1	OK
Anchor 2	0.590	704.173	1	OK
Anchor 3	2.844	694.894	1	OK
Anchor 4	1.465	696.196	1	OK
Anchor 5	2.247	699.033	1	OK
Anchor 6	1.625	712.707	1	OK
Anchor 7	1.436	698.316	1	OK
Anchor 8	0.796	852.988	1	OK
Anchor 9	0.555	798.394	1	OK
Anchor 10	0.281	935.999	1	OK

2.5 Concrete

Decisive load combination: 1 - Combination 1

2.5.1 Compression in concrete under the anchor plate



2.5.2 Concrete block compressive strength resistance check, AISC 360-16 Section J8

Equations

$$F_p = \Phi f_{p,max}$$

$$f_{p,max} = 0.85 f_c' \sqrt{\left(\frac{A_2}{A_1}\right)} \le 1.7 f_c; \quad \sqrt{\left(\frac{A_2}{A_1}\right)} \le 2$$

$$\sigma = \frac{N}{A_1}$$

$$\text{Utilization} = \frac{\sigma}{F_p}$$

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Company:		Page:	14
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis DH BP Brace (Single Brace)	Date:	4/24/2025
Fastening point:			

Variables

N [kip]	f_c' [psi]	Φ	A_1 [in²]	A_2 [in²]
43.864	4,500	0.65	161.80	1,924.21

Results

Load combination	F_p [psi]	σ [psi]	Utilization [%]	Status
Combination 1	4,973	271	6	OK

2.6 Symbol explanation

A_1	Loaded area of concrete
A_2	Supporting area
d	Nominal diameter of the bolt
ε_{lim}	Limit plastic strain
ε_{Pl}	Plastic strain from CBFEM results
f_c	Concrete compressive strength
f_c'	Concrete compressive strength
F_u	Specified minimum tensile strength of the connected material
F_p	Concrete block design bearing strength
$f_{p,max}$	Concrete block design bearing strength maximum
f_y	Yield strength
l_c	Clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material
N	Resulting compression force
σ	Average stress in concrete
σ_{Ed}	Equivalent stress
Φ	Resistance factor
ΦR_n	Factored resistance
t	Thickness of the anchor plate
V	Resultant of shear forces Vy, Vz in bolt.

2.7 Warnings

- By using the CBFEM calculation functionality of PROFIS Engineering you may act outside the applicable design codes and your specified anchor plate may not behave rigid. Please, validate the results with a professional designer and/or structural engineer to ensure suitability and adequacy for your specific jurisdiction and project requirements.
- The anchor is modeled considering stiffness values determined from load displacement curves tested in an independent laboratory. Please note that no simple replacement of the anchor is possible as the anchor stiffness has a major impact on the load distribution results.

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	15
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis DH BP Brace (Single Brace)	Date:	4/24/2025
Fastening point:			

3 Summary of results

Design of the anchor plate, anchors, welds and other elements are based on CBFEM (component based finite element method) and AISC.

	Load combination	Max. utilization	Status
Anchors	Combination 1	1,874%	NOT OK
Anchor plate	Combination 1	75%	OK
Concrete	Combination 1	6%	OK
Profile	Combination 1	100%	OK

Fastening does not meet the design criteria!

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	16
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Strong Axis DH BP Brace (Single Brace)	Date:	4/24/2025
Fastening point:			

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SUPPLEMENTAL CALCULATIONS FOR BASE PLATES

BP-11

TENSION PULLOUT (ACI 318-19, SECTION 17.6.3.2.2

Nu = 177 KIPS

	Load N_{ua} [kip]
Pullout Strength*	176.742

17.6.3.2.2 For single anchors, it shall be permitted to evaluate the pullout strength in tension, N_p, for use in Eq. (17.6.3.1) in accordance with (a) or (b). Alternatively, it shall be permitted to use values of N_p based on the 5 percent fractile of tests performed and evaluated in the same manner as the ACI 355.2 procedures but without the benefit of friction.

(a) For cast-in headed studs and headed bolts, N_p shall be calculated by:

$$N_p = 8A_{brg}f_c' \qquad (17.6.3.2.2a)$$

Phi*Np = 0.7*8*Abrg*f'c
- f'c = 4500 psi
- Abrg = 5"x5" washer = 25 in^2

Phi*Np = 0.7*8*25 in^2* 4500 psi
= 630 kips > 177 kips (U = 0.281)

Tension Reinforcement

Nu = 1140 KIPS

	Load N_{ua} [kip]
Concrete Breakout Failure**	1,139.452

Grid of 11x11 #6 single leg stirrups provided per detail 4/S3-21
As for #6 bar = 0.44 in^2

Phi*Np = N*0.75*As*fy
- N = 121 bars
- fy = 60 ksi

Phi*Np = 121*0.75*0.44 in^2*60ksi
= 2395.8 kips > 1140 kips (U = 0.476)

BF-1

Tension Reinforcement

Nu = 187 KIPS

	Load N_{ua} [kip]
Concrete Breakout Failure**	186.287

Tension reinforcement per detail 5/S5-00 (2 #6 bars at each anchor)
As for #6 bar = 0.44 in^2
of anchors = 8 anchors (only counting anchors on outer lines)
of # 6 bars = 8*2 = 16 bars

Phi*Np = N*0.75*As*fy
- N = 16 bars
- fy = 60 ksi

Phi*Np = 16*0.75*0.44 in^2*60ksi
= 316.8 kips > 187 kips (U = 0.59)

SUPPLEMENTAL CALCULATIONS FOR BASE PLATES

BF-2

Tension Reinforcement

Nu = 168 KIPS

	Load N_{ua} [kip]
Concrete Breakout Failure**	167.654

Tension reinforcement per detail 5/S5-00 (2 #6 bars at each anchor)
As for #6 bar = 0.44 in^2
of anchors = 10 anchors
of # 6 bars = 10*2 = 20 bars

Phi*Np = N*0.75*As*fy
- N = 20 bars
- fy = 60 ksi

Phi*Np = 20*0.75*0.44 in^2*60ksi
= 396 kips > 168 kips (U = 0.424)

BF-4

TENSION PULLOUT (ACI 318-19, SECTION 17.6.3.2.2

Nu = 128 KIPS

	Load N_{ua} [kip]
Pullout Strength*	127.229

17.6.3.2.2 For single anchors, it shall be permitted to evaluate the pullout strength in tension, N_p, for use in Eq. (17.6.3.1) in accordance with (a) or (b). Alternatively, it shall be permitted to use values of N_p based on the 5 percent fractile of tests performed and evaluated in the same manner as the ACI 355.2 procedures but without the benefit of friction.

(a) For cast-in headed studs and headed bolts, N_p shall be calculated by:

$$N_p = 8A_{brg}f_c' \qquad (17.6.3.2.2a)$$

Phi*Np = 0.7*8*Abrg*f'c
- f'c = 4500 psi
- Abrg = 4"x4" washer = 16 in^2

Phi*Np = 0.7*8*16 in^2* 4500 psi
= 403.2 kips > 128 kips (U = 0.317)

Tension Reinforcement

Nu = 270 KIPS

	Load N_{ua} [kip]
Concrete Breakout Failure**	269.499

Grid of 7x7 #6 single leg stirrups provided per detail 5/S3-21
As for #6 bar = 0.44 in^2

Phi*Np = N*0.75*As*fy
- N = 49 bars
- fy = 60 ksi

Phi*Np = 49*0.75*0.44 in^2*60ksi
= 970.2 kips > 270 kips (U = 0.278)

SUPPLEMENTAL CALCULATIONS FOR BASE PLATES

BF-6

TENSION PULLOUT (ACI 318-19, SECTION 17.6.3.2.2

Nu = 72 KIPS

	Load N_{ua} [kip]

17.6.3.2.2 For single anchors, it shall be permitted to evaluate the pullout strength in tension, N_p, for use in Eq. (17.6.3.1) in accordance with (a) or (b). Alternatively, it shall be permitted to use values of N_p based on the 5 percent fractile of tests performed and evaluated in the same manner as the ACI 355.2 procedures but without the benefit of friction.

(a) For cast-in headed studs and headed bolts, N_p shall be calculated by:

$$N_p = 8A_{brg}f_c' \qquad (17.6.3.2.2a)$$

Phi*Np = 0.7*8*Abrg*f'c
- f'c = 4500 psi
- Abrg = 3 1/2"x3 1/2" washer = 12.25 in^2

Phi*Np = 0.7*8*12.25 in^2* 4500 psi
= 308.7 kips > 72 kips (U = 0.233)

Tension Reinforcement

Nu = 172 KIPS

	Load N_{ua} [kip]
Concrete Breakout Failure**	171.134

Grid of 7x7 #6 single leg stirrups provided per detail 5/S3-21
As for #6 bar = 0.44 in^2

Phi*Np = N*0.75*As*fy
- N = 49 bars
- fy = 60 ksi

Phi*Np = 49*0.75*0.44 in^2*60ksi
= 970.2 kips > 172 kips (U = 0.177)

BF-7

TENSION PULLOUT (ACI 318-19, SECTION 17.6.3.2.2

Nu = 46 KIPS

	Load N_{ua} [kip]
Pullout Strength*	45.655

17.6.3.2.2 For single anchors, it shall be permitted to evaluate the pullout strength in tension, N_p, for use in Eq. (17.6.3.1) in accordance with (a) or (b). Alternatively, it shall be permitted to use values of N_p based on the 5 percent fractile of tests performed and evaluated in the same manner as the ACI 355.2 procedures but without the benefit of friction.

(a) For cast-in headed studs and headed bolts, N_p shall be calculated by:

$$N_p = 8A_{brg}f_c' \qquad (17.6.3.2.2a)$$

Phi*Np = 0.7*8*Abrg*f'c
- f'c = 4500 psi
- Abrg = 3"x3" washer = 9 in^2

Phi*Np = 0.7*8*9 in^2* 4500 psi
= 226.8 kips > 46 kips (U = 0.203)

Tension Reinforcement

Nu = 361 KIPS

	Load N_{ua} [kip]
Concrete Breakout Failure**	360.198

Tension reinforcement per detail 5/S5-00 (2 #6 bars at each anchor)
As for #6 bar = 0.44 in^2
of anchors = 14 anchors
of # 6 bars = 8*2 = 28 bars

Phi*Np = N*0.75*As*fy
- N = 28 bars
- fy = 60 ksi

Phi*Np = 28*0.75*0.44 in^2*60ksi
= 554.4 kips > 361 kips (U = 0.651)

SUPPLEMENTAL CALCULATIONS FOR BASE PLATES

BF-8

ANCHOR ROD FAILURE

Max anchor rod size in Hilti is 2" dia. but 2 1/2" dia. used for BF-8

	Load N_{ua} [kip]
Steel Strength*	179.731

Table 3.1. ASTM F1554 Anchor Rod (rod only) Available Tensile Strength, kips

Rod Diameter, in.	Rod Area, A_b, in.²	LRFD ϕR_n ϕ = 0.75			ASD R_n/Ω Ω = 2.00		
		Grade 36 kips	Grade 55 kips	Grade 105 kips	Grade 36 kips	Grade 55 kips	Grade 105 kips
⅝	0.307	10.0	12.9	21.6	6.68	8.63	14.4
¾	0.442	14.4	18.6	31.1	9.60	12.4	20.7
⅞	0.601	19.6	25.4	42.3	13.1	16.9	28.2
1	0.785	25.6	33.1	55.2	17.1	22.1	36.8
1⅛	0.994	32.4	41.9	69.9	21.6	28.0	46.6
1¼	1.23	40.0	51.8	86.3	26.7	34.5	57.5
1½	1.77	57.7	74.6	124	38.4	49.7	82.8
1¾	2.41	78.5	102	169	52.3	67.6	113
2	3.14	103	133	221	68.3	88.4	147
2¼	3.98	130	168	280	86.5	112	186
2½	4.91	160	207	345	107	138	230

PhiRn = 345 kips for 2 1/2" dia GR 105 AR per AISC DG-1, Table 3.1
= 345 kips > 180 kips (U = 0.522)

TENSION PULLOUT (ACI 318-19, SECTION 17.6.3.2.2

Nu = 180 KIPS

	Load N_{ua} [kip]
Pullout Strength*	179.731

17.6.3.2.2 For single anchors, it shall be permitted to evaluate the pullout strength in tension, N_p, for use in Eq. (17.6.3.1) in accordance with (a) or (b). Alternatively, it shall be permitted to use values of N_p based on the 5 percent fractile of tests performed and evaluated in the same manner as the ACI 355.2 procedures but without the benefit of friction.

(a) For cast-in headed studs and headed bolts, N_p shall be calculated by:

$$N_p = 8A_{brg}f_c' \qquad (17.6.3.2.2a)$$

Phi*Np = 0.7*8*Abrg*f'c
- f'c = 4500 psi
- Abrg = 5 3/4"x5 3/4" washer = 33 in^2

Phi*Np = 0.7*8*33 in^2* 4500 psi
= 833 kips > 180 kips (U = 0.216

Tension Reinforcement

Nu = 1193 KIPS

	Load N_{ua} [kip]
Concrete Breakout Failure**	1,192.297

Grid of 11x11 #6 single leg stirrups provided per detail 4/S3-21
As for #6 bar = 0.44 in^2

Phi*Np = N*0.75*As*fy
- N = 121 bars
- fy = 60 ksi

Phi*Np = 121*0.75*0.44 in^2*60ksi
= 2395.8 kips > 1193 kips (U = 0.603)

BF-9

Shear Reinforcement

Vu = 155 KIPS

	Load V_{ua} [kip]
Concrete edge failure in direction y+**	154.208

Shear reinforcement per detail 5/S5-00 (4 #6 U bar at each anchor group accounted for, 2 legs each)
Av for #6 bar = 0.44 in^2
of anchors groups = 2 anchors groups
of # 6 bars = 4*4 = 16 bars

Phi*Vn = N*0.75*Av*fy
- N = 16 bars
- fy = 60 ksi

Phi*Np = 16*0.75*0.44 in^2*60ksi
= 316.8 kips > 155 kips (U = 0.489)

SUPPLEMENTAL CALCULATIONS FOR BASE PLATES

BF-10

Table 3.1. ASTM F1554 Anchor Rod (rod only) Available Tensile Strength, kips

Rod Diameter, in.	Rod Area, A_b, in.^{2}	LRFD ϕR_n ϕ = 0.75			ASD R_n/Ω Ω = 2.00		
		Grade 36 kips	Grade 55 kips	Grade 105 kips	Grade 36 kips	Grade 55 kips	Grade 105 kips
⅝	0.307	10.0	12.9	21.6	6.68	8.63	14.4
¾	0.442	14.4	18.6	31.1	9.60	12.4	20.7
⅞	0.601	19.6	25.4	42.3	13.1	16.9	28.2
1	0.785	25.6	33.1	55.2	17.1	22.1	36.8
1⅛	0.994	32.4	41.9	69.9	21.6	28.0	46.6
1¼	1.23	40.0	51.8	86.3	26.7	34.5	57.5
1½	1.77	57.7	74.6	124	38.4	49.7	82.8
1¾	2.41	78.5	102	169	52.3	67.6	113
2	3.14	103	133	221	68.3	88.4	147
2¼	3.98	130	168	280	86.5	112	186
2½	4.91	160	207	345	107	138	230

ANCHOR ROD FAILURE

Max anchor rod size in Hilti is 2" dia. but 2 1/2" dia. used for BF-8

	Load N_{ua} [kip]
Steel Strength*	256.249

PhiRn = 345 kips for 2 1/2" dia GR 105 AR per AISC DG-1, Table 3.1
= 345 kips > 257 kips (U = 0.745)

TENSION PULLOUT (ACI 318-19, SECTION 17.6.3.2.2

Nu = 257 KIPS

	Load N_{ua} [kip]
Pullout Strength*	256.249

17.6.3.2.2 For single anchors, it shall be permitted to evaluate the pullout strength in tension, N_p, for use in Eq. (17.6.3.1) in accordance with (a) or (b). Alternatively, it shall be permitted to use values of N_p based on the 5 percent fractile of tests performed and evaluated in the same manner as the ACI 355.2 procedures but without the benefit of friction.

(a) For cast-in headed studs and headed bolts, N_p shall be calculated by:

$$N_p = 8A_{brg}f_c' \qquad (17.6.3.2a)$$

Phi*Np = 0.7*8*Abrg*f'c
- f'c = 4500 psi
- Abrg = 5 3/4"x5 3/4" washer = 33 in^2

Phi*Np = 0.7*8*33 in^2* 4500 psi
= 833 kips > 257 kips (U = 0.309)

Tension Reinforcement

Nu = 1193 KIPS

	Load N_{ua} [kip]
Concrete Breakout Failure**	1,682.468

Grid of 11x11 #6 single leg stirrups provided per detail 4/S3-21
As for #6 bar = 0.44 in^2

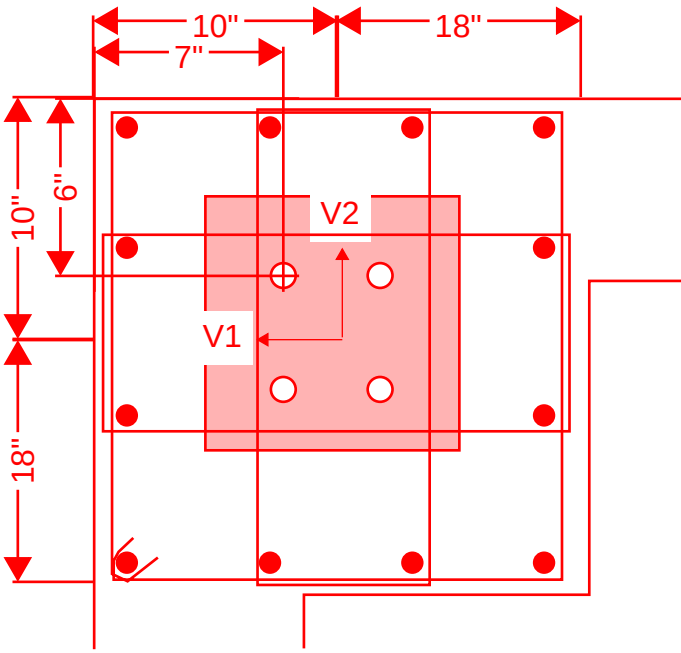
Phi*Np = N*0.75*As*fy
- N = 121 bars
- fy = 60 ksi

Phi*Np = 121*0.75*0.44 in^2*60ksi
= 2395.8 kips > 1683 kips (U = 0.702)

SUPPLEMENTAL CALCULATIONS FOR BASE PLATES

CONCRETE PEDESTAL SHEAR REINFORCEMENT CALCULATION

CALCULATION PER REINFORCED CONCRETE DESIGN HANDBOOK CHAPTER 15 EXAMPLE 17b



LOADS
V1 = 23 KIPS
V2 = 12 KIPS

Will look at outer stirrup for resistance at worse case

CALCULATION

Vu1 = 23 Kips

Strut angle = 40.6 degrees

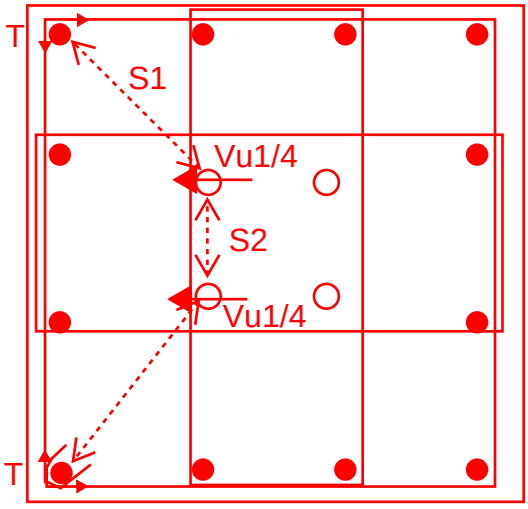
Vu1/4 = 5.75 kips

Vu1/4 = S1*cos(40.6)

T = S1*sin(40.6)

T= Vu1/4*sin(40.6)/cos(40.6) = Vu1/4*tan(40.6) = 4.93 kips

As,req1 = 4.93 kips/ (0.75*60 ksi) = 0.11 in^2



Vu2 = 12 Kips

Strut angle = 49.4 degrees

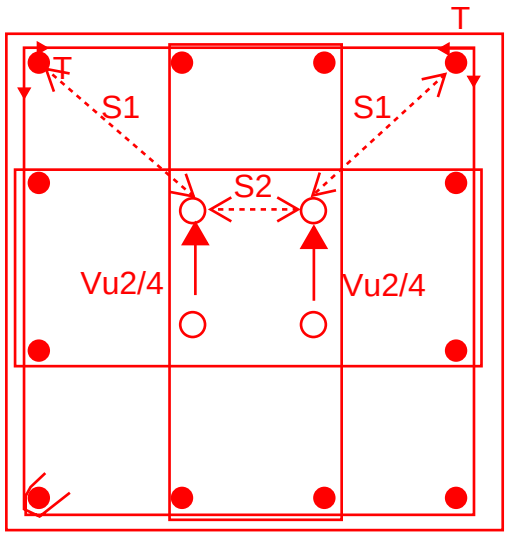
Vu2/4 = 3.0 kips

Vu2/4 = S1*cos(49.4)

T = S1*sin(49.4)

T= Vu2/4*sin(49.4)/cos(49.4) = Vu2/4*tan(49.4) = 3.5 kips

As,req2 = 3.5 kips/ (0.75*60 ksi) = 0.078 in^2



As,req = 0.11 in^2 + 0.078 in^2 = 0.188 in^2

of #4 ties required = 0.188 in^2 / 0.2 in^2 = 1 ties
2 #4 ties provided in to of pedestal

CONCLUSION

(2) #4 bars are required in top 5" of pedestal