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NPS GATE Floyd Bennett Field Hangars 3 and 4 – Rehabilitation

PMIS / PKG No. GATE 308504

Structural Design Calculation Package
Final Submission

MFSE Project No. 23-086.1
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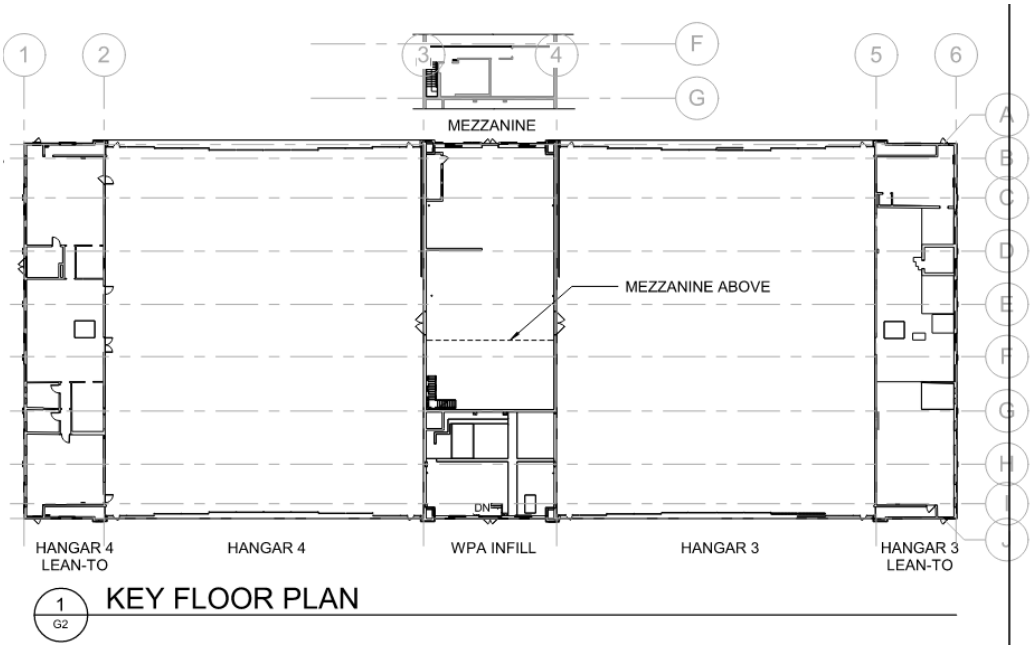
I. Description of Project

This project concerns the rehabilitation of Hangars 3 & 4 at Floyd Bennett Field in the Gateway National Recreation Area, a unit of the National Park Service. The last two of eight total hangars to be rehabilitated at the site, the structures were originally constructed between 1929 and 1938 on land created by infilling the channels between Barren Island and several smaller islands and the rest of Brooklyn with sand pumped from the bottom of Jamaica Bay. While the hangars do not sit within a flood zone, they are located directly adjacent to flood zones.

The hangars are planned to be restored to their original appearance and condition. The National Register Nomination for the Floyd Bennett Field Historic District identifies Hangars 3 and 4 as a contributing resource, and as such, the rehabilitation will follow the Secretary of the Interior Standards for the Treatment of Historic Properties. Interventions will include repair of the masonry building envelope, fenestration and interior finishes; replacement of the roof; replacement of MEP systems; and installation of new accessible restrooms. The hangar complex includes (2) 16,600 SF hangars, (2) 4100 SF lean-tos, and a 6,600 SF infill building between the two hangars.



a. Hangar Construction and Conditions

The hangar roof framing consists of steel channel purlins spanning in the north-south direction and supporting 2x tongue and groove wood decking. The purlins bear on steel trusses spanning 120 feet in the east-west direction to 14” wide flange columns. Vertical x-bracing braces the truss top and bottom chords at the centerline and two additional lines. Horizontal x-bracing exists at the truss top chord level within the northernmost, southernmost and center bays to reinforce the roof diaphragm. At the truss bottom chord level, diagonal horizontal braces laterally brace the top of the brick masonry walls and the hangar doors. The wood decking, wood nailers and wood plates at the tops of the masonry walls are deteriorated and it is assumed that they require significant replacement. The roof diaphragm will be reinforced with an additional layer of plywood.

The steel columns are embedded in the east and west brick masonry walls and are supported on reinforced concrete pile caps which are, in turn, supported on 14” square reinforced concrete piles. The brick walls are supported on reinforced concrete grade beams supported on lines of the same piles. The embedded steel columns are experiencing varying levels of corrosion and will be cleaned and coated and, where required, structurally reinforced.

The upper portion of the east and west hangar walls contains clerestory windows framed out with steel beams spanning in between the steel columns. The steel beams also support bands of brick veneer. Significant corrosion of the steel beams and hung lintels and related movement and deterioration of the brick were observed. Brick masonry is to be rebuilt in these areas and steel is to be replaced or cleaned and coated and locally reinforced where required.

The hangar doors are constructed with aluminum wide flange and channel vertical members with horizontal aluminum channels and tees spanning in between them. The lower portion has solid aluminum panels whereas the upper portion is glazed. Each door panel has two wheels which bear on and are guided by steel rails embedded in a grade beam. Steel angle guides also exist at the top of the doors. The aluminum elements of the hangar doors and their connections exhibit varying degrees of corrosion and require localized replacement. The embedded rails at the base are generally corroded, with associated spalling of the surrounding concrete. Rails are to be replaced in kind and concrete repaired.

The hangar floors consist of concrete slabs-on-grade. There has been differential settlement of the slabs-on-grade relative to the pile-supported elements, in some areas up to 6-1/2 inches. The proposed rehabilitation includes subgrade improvement per recommendations in the geotechnical report and a replacement of the slabs with new reinforced concrete slabs on grade, thickened at the edges and doweled into the surrounding grade beams.

b. Lean-to Construction and Conditions

The Lean-to roof framing consists of 10”-deep and 8”-deep steel channel purlins spanning in the north-south direction and supported on 21”-deep and 16”-deep steel girders spanning in the east-west direction. The girders connect to the steel columns embedded in the masonry wall shared with the hangars and pocket into piers in the exterior masonry walls. The pocketed beam and purlin ends are typically severely corroded and require cleaning, reinforcement, and coating. Bearing plates typically require replacement. The roofing system is similar to that of the hangars, with 2x wood decking connected to wood nailers bolted to the steel purlins. The wood decking, wood nailers and wood plates at the tops of the masonry walls are deteriorated and it is assumed that they require full replacement. The roof diaphragm will be reinforced with an additional layer of plywood.

Lintels typically consist of double steel back-to-back angles. They have been replaced in Lean-to 3 with new galvanized steel lintels. Hangar 4 still has the original lintels, which are severely corroded and are displacing the brick masonry above. In these areas, the lintels are to be replaced with new galvanized steel lintels and the brick masonry rebuilt.

At the north and south walls of the lean-tos there is a large pocket to contain the hangar doors when they are opened. The pocket is built of 8” brick masonry which have a long unbraced length both vertically and horizontally compared to their thickness. These brick walls are exhibiting significant distress in the form of cracking and separation at the corners. The structural repairs include masonry repairs and reinforcement of the masonry pocket, designed for out-of-plane wind loading.

Existing floors in the lean-tos are a mix of concrete slabs and wood floors, all of which are significantly deteriorated. The proposed rehabilitation includes subgrade improvement per recommendations in the geotechnical report and installation of new reinforced concrete slabs on grade, thickened at the edges and doweled into the surrounding grade beams.

c. WPA Infill Building / Boiler Room Construction and Conditions

The roof framing over the northern portion of the WPA Infill Building is constructed similarly to the hangars. Steel channel purlins span in the north-south direction between steel trusses, but here they are flush framed with the truss top chord. The steel trusses span 50 feet in the east-west direction between the steel columns embedded in the east and west brick masonry walls. The northernmost truss has two lines of out-of plane bracing spanning to the north wall. The northernmost and southernmost purlins pocket into the north exterior brick wall and the brick wall separating the main space from the boiler room. The roofing system is similar to that of the hangars, with 2x wood decking connected to wood nailers bolted to the steel purlins. The wood decking and wood nailers are deteriorated in areas and it is assumed that they require significant replacement. The roof diaphragm will be reinforced with an additional layer of plywood. The existing skylights will be restored as part of this rehabilitation.

Directly to the north of the Boiler Room is a one-bay mezzanine. The floor is a one-way concrete slab supported on concrete-encased steel beams. The beams bear on the brick wall to the south and a deep steel truss to the north. The bottom chord of the truss delineates the edge of the mezzanine.

The Boiler Room is at the south end of the WPA Infill Building. The floor slab on grade is at a lower elevation than the hanger floor, likely to accommodate the tall mechanical equipment. There are localized areas of concrete damage that will require repair. Existing trenches are to be covered with new diamond plate or steel plank.

The roof framing over the Boiler Room consists of a one-way reinforced concrete slab spanning in the east-west direction between concrete-encased steel beams spanning in the north-south direction. The steel beams bear on brick masonry walls at each end. One large skylight exists over the center of the space with four small skylights adjacent to it. There is significant deterioration of the concrete slab and concrete encasement of the steel beams, likely due to water infiltration through deteriorated roofing, and concrete repairs will be required.

Lintels over the door and window openings are assumed to be steel angles based on a destructive probe performed at one lintel bearing location. Lintels are typically corroded at the bearing ends and, in places, along the lintel span. It is assumed that most lintels will require replacement with new galvanized steel lintels and surrounding brick will need to be rebuilt.

II. Geotechnical Investigation Summary

A geotechnical investigation was conducted by GHD, with three (3) boreholes advanced directly outside the hangars at the northeast, southeast and southwest corners and one (1) borehole advanced within Hangar 4. The field work was carried out from April 15 to 18, 2025. The following is a brief summary of the findings and recommendations. The full report is included in Appendix B.

- Soil Conditions:
  - Sand fill was encountered under the concrete slab and asphalt pavement and extended to approximately 13.5 to 18.5 feet below the ground surface (bgs) at all borehole locations. SPT 'N' values obtained within the sandy silt and sandy clay fill was 2 blows per foot (bpf), indicating a very soft consistency.
  - Sand (USCS classification of SP, SP-SC, SC, SM) with variable amounts of silt, clay, and gravel was encountered below the fill at all borehole locations and extended to the borehole termination depth of 50 feet bgs. The SPT N values ranged from 0 to 5 bpf to depths ranging from 23.5 to 28.5 feet bgs indicating a very loose to loose relative density. Below these depths and extending to the borehole termination depths, the SPT N values increased to values ranging from 11 to 40 bpf, indicating a medium dense to dense relative density.
- Groundwater:
  - The soils encountered in the investigation were generally moist to wet. Water was observed at two borehole locations during drilling at a depth of 10 feet bgs.
- Corrosion Potential:
  - Soil testing results indicated low corrosion potential based on evaluation using the American Water Works Association (AWWA) C-105 Standard, Caltrans Corrosion Guidelines, and National Association of Corrosion Engineers Corrosion Basics.
  - The sulfate content for the samples tested indicated a "negligible" risk of sulfate exposure potential for concrete contacting soil per ACI 318; therefore, normal Portland cement is acceptable for use in below-grade concrete structures.
- Recommendations for Site Preparation and Foundation Design:
  - Due to the history of slab settlement on the site and the soil conditions observed / tested on site by GHD, the report provided recommendations for soil improvement by overexcavating the soil, proofrolling, installing geogrid, and backfilling with compacted, crushed aggregate. The specific recommendations are included in the geotechnical report in Appendix B and represented in the structural drawings.
    - Soil improvement per the recommendations is noted to be suitable for support of slabs-on-grade with loads of 150 psf or less, to be designed using a modulus of subgrade reaction (k) value of 115 pounds per cubic inch (pci).
    - Foundation pads with subgrades prepared as described in the geotechnical report should be designed for a net allowable bearing capacity of 1,200 pounds per square foot (psf) and a coefficient of sliding friction of 0.35 between the concrete and foundation subgrade material. The pad foundations should be structurally separate (isolated) from the adjacent slabs-on-grade and existing foundations.
- Seismic Considerations:
  - The Seismic Site Class, for calculations of earthquake design forces based on a two percent probability of exceedance in 50 years in accordance with ASCE 7−22, is a function of soil profile and is based on the average properties of the subsurface strata to a depth of 100 feet below the ground surface. The average SPT value for the top 100 feet at the Site is estimated to be less than 15 bpf. and therefore Site Class E is recommended for seismic design of the Site.

III. Structural Design Criteria

a. Applicable Codes and Standards

The following code is currently required by the National Park Service, the Authority Having Jurisdiction:

1. 2024 International Existing Building Code (IEBC).

The following structural design standards relevant to this project are referenced by the currently adopted codes or required by the National Park Service:

2. ASCE 7-22, Minimum Design Loads for Buildings and other Structures
3. ACI 318-19, Building Code Requirements for Structural Concrete
4. AISC 360-22, Specification for Structural Steel Buildings
5. TMS 402/602-16, Building Code Requirements and Specification for Masonry Structures
6. NDS-24, National Design Specification for Wood Construction with 2018 NDS Supplement

b. Structural Loading

1. Risk Category

Risk Category, which is based on a building's primary occupancy and its occupant load, affects the magnitude of structural design loading per IBC and ASCE 7 for the effects of wind, earthquakes, flooding, snow, and ice. The previous use as an airplane hangar would be considered Risk Category II. For purposes of this project, the building use has been determined to be S-2, low-hazard storage. The design of repairs and reinforcements for this rehabilitation is based on Risk Category II.

The future use of Hangars 3 and 4 is currently unknown and they may remain as a Risk Category II structure. However, the following occupancies would change the structure to be Risk Category III per the IBC:

- Buildings and other structures whose primary occupancy is public assembly with an occupant load greater than 300.
- Buildings and other structures containing quantities of toxic or explosive materials that exceed maximum allowable quantities per control area as given in Table 307.1(1) or 307.1(2).

2. Additions or Alterations to Existing Buildings

The proposed work for this building is considered a Level 2 Alteration, since the work area is less than 50 percent of the building area. As such, per the IEBC, any structural element for which an addition or alteration to the existing building causes an increase in the demand due to gravity loads on the element or decrease in its gravity load-resisting capacity exceeding 5% shall be strengthened or replaced as needed to meet the provisions of the currently adopted Building Code for new buildings (IBC). Also per the IEBC, any structural element for which an addition or alteration to the existing building causes an increase in the demand due to lateral (wind or seismic) loads on

the element or decrease in its lateral load-resisting capacity exceeding 10% (or where an alteration results in a structural irregularity as defined by ASCE 7) shall be strengthened or replaced as needed to meet the provisions of the currently adopted Building Code for new buildings (IBC). Calculations of any increases in the demand-to-capacity ratio for structural elements shall account for the cumulative effect of all alterations and additions since the original construction, per the IEBC.

3. Live Loading

The following minimum design live load values are specified by ASCE 7. Since the potential future use(s) of the space are currently unknown, a limited list of uses and corresponding live load requirements is provided below:

Occupancy or Use	Live Loadings	
	Uniform	Concentrated
Assembly areas	100 psf	-
Light Storage	125 psf	-
Offices	50 psf	-
Roof	20 psf	300 lb

Interior partition walls shall be designed to resist a minimum 5 psf out-of-plane live load per ASCE 7.

4. Snow Loading

Flat roof snow loads and snow drift shall be calculated in accordance with ASCE 7 based on the following parameters:

Ground Snow Load (p_g)	45 psf
Exposure Category	D
Exposure Factor (C_e)	0.8 (Fully Exposed)
Thermal Factor (C_t)	1.2 (Structure heated to 40 – 50 degrees F)

5. Wind Loading

Wind loads shall be calculated in accordance with ASCE 7 based on the following parameters. The site is in a hurricane-prone region as defined in ASCE 7-22 Section 26.2. However, glazed openings need not be protected against wind-borne debris.

Ultimate Design Wind Speed	118 mph
Exposure Category	D

6. Seismic Loading

Seismic loads shall be calculated in accordance with ASCE 7 based on the following parameters (for a Risk Category II building):

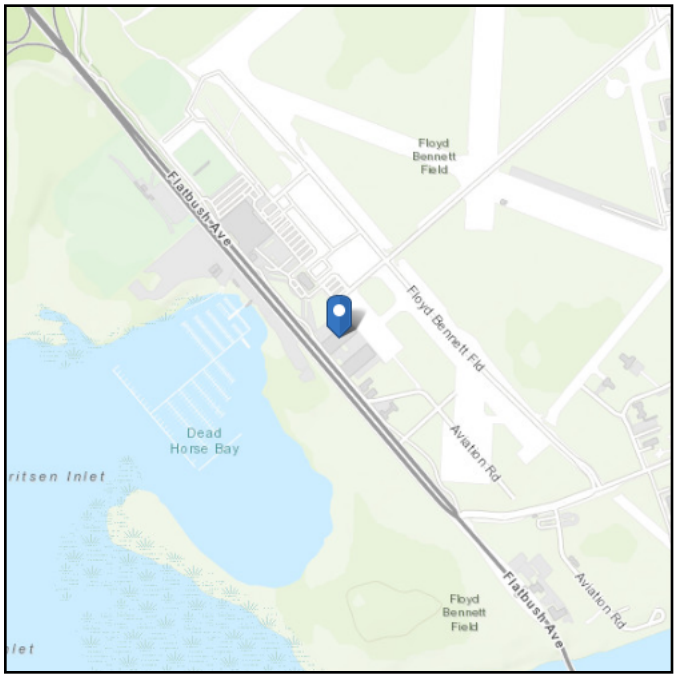
Site Class	D
Mapped short-period spectral acceleration (S_s)	0.26 g
Mapped spectral acceleration for 1 s period (S_1)	0.049 g
Design spectral response acceleration (S_{DS})	0.17 g
Design spectral acceleration for 1 s period (S_{D1})	0.064 g
Seismic Design Category	B
Seismic Importance Factor	1.00
Response Modification Coefficient (R):	
Ordinary Plain Masonry Shear Walls	1.5
Ordinary Steel Moment Frames	3.5

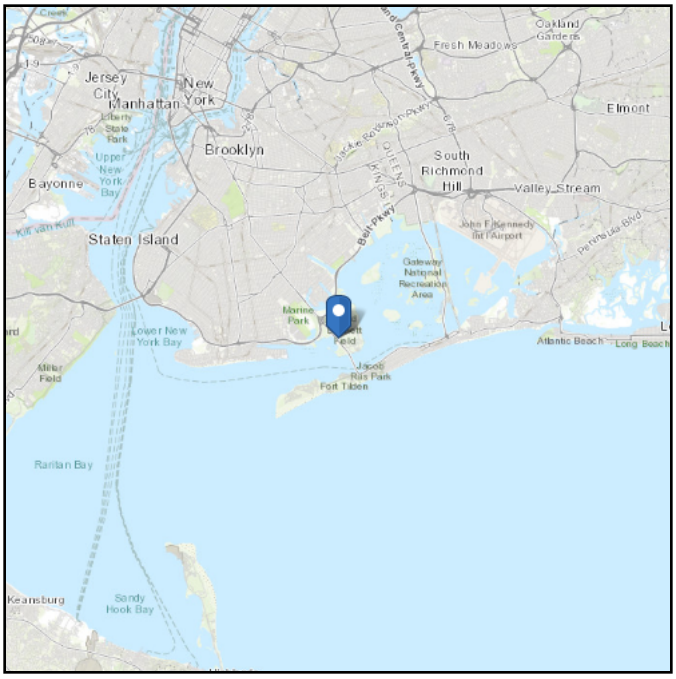
ASCE Hazards Report

Address:
No Address at This Location

Standard: ASCE/SEI 7-22
Risk Category: II
Soil Class: E - Soft Clay Soil

Latitude: 40.586618
Longitude: -73.89575
Elevation: 12.89370924133884 ft (NAVD 88)





Wind

Results:

Wind Speed	118 Vmph
10-year MRI	75 Vmph
25-year MRI	85 Vmph
50-year MRI	91 Vmph
100-year MRI	98 Vmph
300-year MRI	109 Vmph
700-year MRI	118 Vmph
1,700-year MRI	127 Vmph
3,000-year MRI	132 Vmph
10,000-year MRI	142 Vmph
100,000-year MRI	162 Vmph
1,000,000-year MRI	187 Vmph

Data Source: ASCE/SEI 7-22, Fig. 26.5-1B and Figs. CC.2-1–CC.2-4, and Section 26.5.2
Date Accessed: Tue May 20 2025

Value provided is 3-second gust wind speeds at 33 ft above ground for Exposure C Category, based on linear interpolation between contours. Wind speeds are interpolated in accordance with the 7-22 Standard. Wind speeds correspond to approximately a 7% probability of exceedance in 50 years (annual exceedance probability = 0.00143, MRI = 700 years). Values for 10-year MRI, 25-year MRI, 50-year MRI and 100-year MRI are Service Level wind speeds, all other wind speeds are Ultimate wind speeds.

Site is in a hurricane-prone region as defined in ASCE/SEI 7-22 Section 26.2. Glazed openings need not be protected against wind-borne debris.

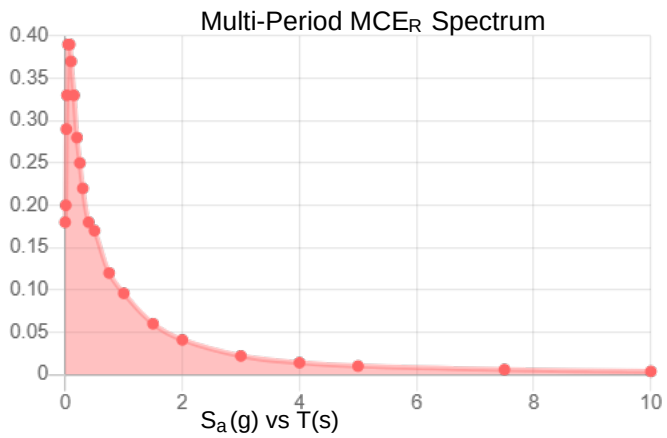
Seismic

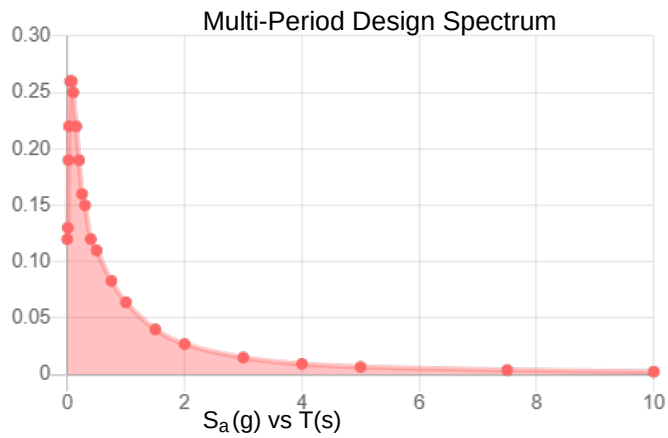
Site Soil Class: E - Soft Clay Soil

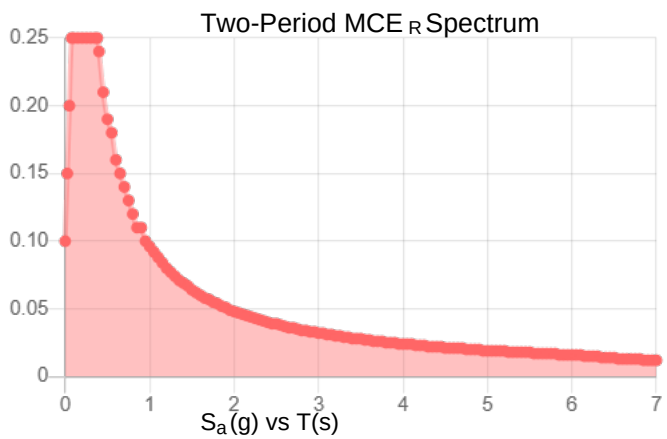
Results:

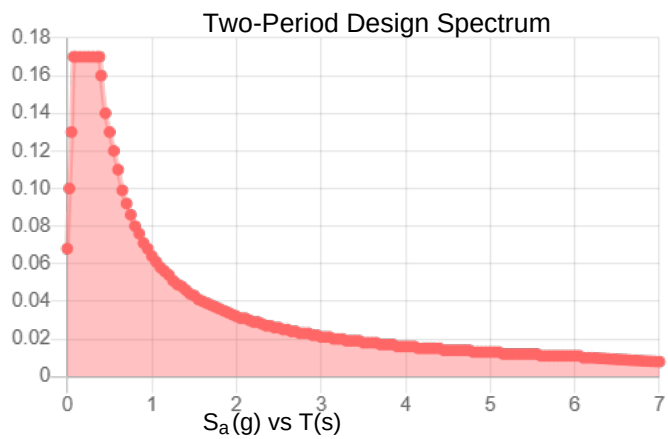
PGA_M :	0.16	T_L :	6
S_{MS} :	0.25	S_S :	0.26
S_{M1} :	0.096	S_1 :	0.049
S_{DS} :	0.17	V_{S30} :	150
S_{D1} :	0.064		

Seismic Design Category: B









MCE_R Vertical Response Spectrum

Vertical ground motion data has not yet been made available by USGS.

Design Vertical Response Spectrum

Vertical ground motion data has not yet been made available by USGS.

Data Accessed: **Tue May 20 2025**

Date Source:
USGS Seismic Design Maps based on ASCE/SEI 7-22 and ASCE/SEI 7-22 Table 1.5-2. Additional data for site-specific ground motion procedures in accordance with ASCE/SEI 7-22 Ch. 21 are available from USGS.

Snow

Results:

Ground Snow Load, p_g :	45 lb/ft^2
20-year MRI Value:	16.26 lb/ft^2
Winter Wind Parameter:	0.55
Mapped Elevation:	4.7 ft
Data Source:	ASCE/SEI 7-22, Figures 7.6-1 and 7.6-2 A-D
Date Accessed:	Tue May 20 2025
	Values provided are ground snow loads. In areas designated "case study required," extreme local variations in ground snow loads preclude mapping at this scale. Site-specific case studies are required to establish ground snow loads at elevations not covered.
	Snow load values are mapped to a 0.5 mile resolution. This resolution can create a mismatch between the mapped elevation and the site-specific elevation in topographically complex areas. Engineers should consult the local authority having jurisdiction in locations where the reported ‘elevation’ and ‘mapped elevation’ differ significantly from each other.
Ground Snow Loads for IRC only, $p_{g(asd)}$:	31.5 lb/ft^2

The ASCE Hazard Tool is provided for your convenience, for informational purposes only, and is provided “as is” and without warranties of any kind. The location data included herein has been obtained from information developed, produced, and maintained by third party providers; or has been extrapolated from maps incorporated in the ASCE standard. While ASCE has made every effort to use data obtained from reliable sources or methodologies, ASCE does not make any representations or warranties as to the accuracy, completeness, reliability, currency, or quality of any data provided herein. Any third-party links provided by this Tool should not be construed as an endorsement, affiliation, relationship, or sponsorship of such third-party content by or from ASCE.

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Main Wind Force Resisting System Wind Pressures - IBC 2024 / ASCE 7-22

Project:	NPS GATE Hangars 3&4	Notes:	Hangars
Project #:	23-086.1		

Enclosure Classification	Enclosed	(t 26.12)	**For Roof Slopes LESS THAN 10 degrees**					
Risk Category	II	(IBC 1604.5)						
Exposure	D	(t 26.7)	GC_{pi} =	+/- 0.18	(Table 26.13-1)			
Mean Roof Height, h =	37	ft	V (mph) =	118	(t 26.5.1)			
B1, Length =	140	ft	K_{zt} =	1.00	(t 26.8.2)	z =	37	
B2, Width =	120	ft	K_d =	0.85	(t 26.6)	z_g =	1935	(Table 26.11-1)
Roof Slope Angle, θ =	2	degrees	G =	0.85	(t 26.11)	α =	11.5	(Table 26.11-1)
Parapet Height =	0	ft	K_e =	1.00	(t 26.9)			

WALL PRESSURES

$C_{p,WW}$ =	0.80	(Fig. 27.3-1)	$q_z = 0.00256\ K_z K_{zt} K_e V^2$		(lb/ft^2)	(Eq. 26.10-1)
$C_{p1,LW \,||\, B2}$ =	-0.50	(Fig. 27.3-1)	$p = q\ K_d\ GC_p - q_i\ K_d\ (GC_{pi})$		(lb/ft^2)	(Eq. 27.3-1)
$C_{p2,LW \,||\, B1}$ =	-0.47	(Fig. 27.3-1)	$qK_dGC_{p\ leeward\ ||\ B2}$	-15.59		
$C_{p,SW}$ =	-0.70	(Fig. 27.3-1)	$qK_dGC_{p\ leeward\ ||\ B1}$	-14.55		

z (ft)	K_z	q (psf)	WW	internal	Pos. Int. Pressure			Neg. Int. Pressure			Total	
			qK_dGC_p	$qK_d(GC_{pi})$	WW	LW||B1	LW||B2	WW	LW||B1	LW||B2	Wind || to B1	Wind || to B2
37	1.211	43.2	25.0	+/- 6.6	18.35	-21.16	-22.20	31.6	-7.95	-8.99	**39.5** psf	**40.5** psf
30	1.168	41.6	24.1	+/- 6.6	17.45	-21.16	-22.20	30.7	-7.95	-8.99	**38.6** psf	**39.7** psf
22	1.106	39.4	22.8	+/- 6.6	16.19	-21.16	-22.20	29.4	-7.95	-8.99	**37.3** psf	**38.4** psf
15	1.035	36.9	21.3	+/- 6.6	14.72	-21.16	-22.20	27.9	-7.95	-8.99	**35.9** psf	**36.9** psf

PARAPET PRESSURES

K_z =	1.211		Windward Parapet =	55.0	psf
q_p =	43.2		Leeward Parapet =	-36.7	psf
GC_{pn} =	1.5	windward	Total Pressure =	**91.7**	**psf**
GC_{pn} =	-1.0	leeward	Total Pressure assumes a parapet of equal height on windward and leeward side of building		

ROOF PRESSURES

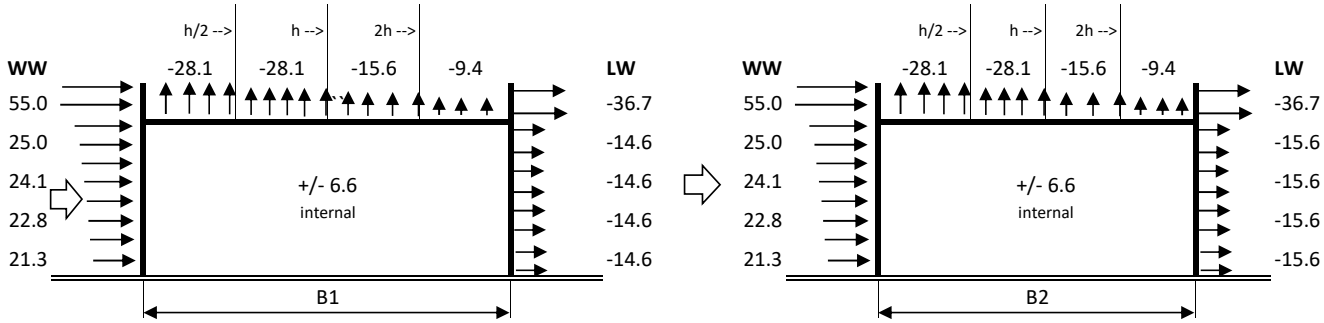
Wind Parallel to B1 h/B1 = 0.26

Dist from WW edge	C_p	p	
0 to h/2	-0.90	-28.1	psf
h/2 to h	-0.90	-28.1	psf
h to 2h	-0.50	-15.6	psf
> 2h	-0.30	-9.4	psf

Wind Parallel to B2 h/B2 = 0.31

Dist from WW edge	C_p	p	
0 to h/2	-0.90	-28.1	psf
h/2 to h	-0.90	-28.1	psf
h to 2h	-0.50	-15.6	psf
> 2h	-0.30	-9.4	psf

SUMMARY OF PRESSURES - pressures in psf acting normal to the surface



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Main Wind Force Resisting System Wind Pressures - IBC 2024 / ASCE 7-22

Project:	NPS GATE Hangars 3&4	Notes:	Lean-tos
Project #:	23-086.1		

Enclosure Classification	Enclosed	(t 26.12)	**For Roof Slopes LESS THAN 10 degrees**					
Risk Category	II	(IBC 1604.5)						
Exposure	D	(t 26.7)	GC_{pi} =	+/- 0.18	(Table 26.13-1)			
Mean Roof Height, h =	15	ft	V (mph) =	118	(t 26.5.1)			
B1, Length =	140	ft	K_{zt} =	1.00	(t 26.8.2)	z =	15	
B2, Width =	30	ft	K_d =	0.85	(t 26.6)	z_g =	1935	(Table 26.11-1)
Roof Slope Angle, θ =	2	degrees	G =	0.85	(t 26.11)	α =	11.5	(Table 26.11-1)
Parapet Height =	13	ft	K_e =	1.00	(t 26.9)			

WALL PRESSURES

$C_{p,WW}$ =	0.80	(Fig. 27.3-1)	$q_z = 0.00256\ K_z K_{zt} K_e V^2$		(lb/ft^2)	(Eq. 26.10-1)
$C_{p1,LW \,||\, B2}$ =	-0.50	(Fig. 27.3-1)	$p = q\ K_d\ GC_p - q_i\ K_d\ (GC_{pi})$		(lb/ft^2)	(Eq. 27.3-1)
$C_{p2,LW \,||\, B1}$ =	-0.20	(Fig. 27.3-1)	$qK_dGC_{p\ leeward\ ||\ B2}$	-13.33		
$C_{p,SW}$ =	-0.70	(Fig. 27.3-1)	$qK_dGC_{p\ leeward\ ||\ B1}$	-5.33		

z (ft)	K_z	q (psf)	WW	internal	Pos. Int. Pressure			Neg. Int. Pressure			Total	
			qK_dGC_p	$qK_d(GC_{pi})$	WW	LW||B1	LW||B2	WW	LW||B1	LW||B2	Wind || to B1	Wind || to B2
15	1.035	36.9	21.3	+/- 5.64	15.68	-10.98	-18.97	27.0	0.31	-7.68	**26.7** psf	**34.7** psf
15	1.035	36.9	21.3	+/- 5.64	15.68	-10.98	-18.97	27.0	0.31	-7.68	**26.7** psf	**34.7** psf
15	1.035	36.9	21.3	+/- 5.64	15.68	-10.98	-18.97	27.0	0.31	-7.68	**26.7** psf	**34.7** psf
15	1.035	36.9	21.3	+/- 5.64	15.68	-10.98	-18.97	27.0	0.31	-7.68	**26.7** psf	**34.7** psf

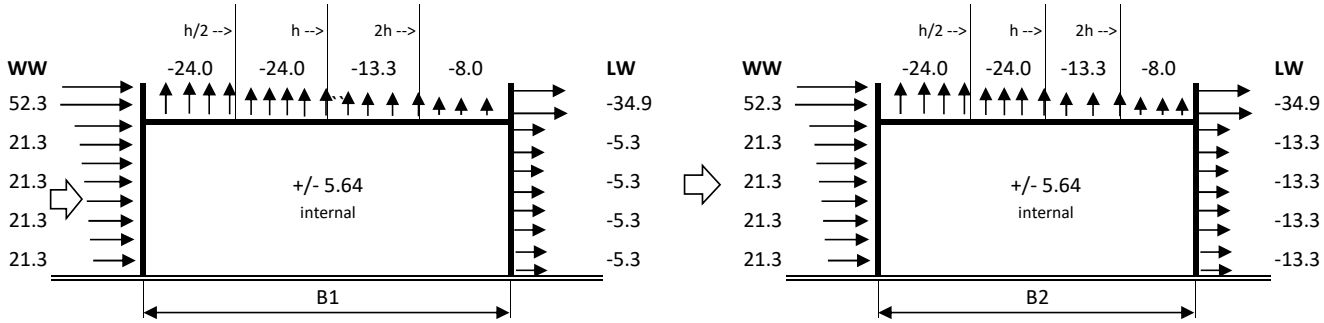
PARAPET PRESSURES

K_z =	1.152		Windward Parapet =	52.3	psf
q_p =	41.0		Leeward Parapet =	-34.9	psf
GC_{pn} =	1.5	windward	Total Pressure =	**87.2**	**psf**
GC_{pn} =	-1.0	leeward	Total Pressure assumes a parapet of equal height on windward and leeward side of building		

ROOF PRESSURES

Wind Parallel to B1	h/B1 =	0.11		**Wind Parallel to B2**	h/B2 =	0.50	
Dist from WW edge	C_p	p		Dist from WW edge	C_p	p	
0 to h/2	-0.90	-24.0	psf	0 to h/2	-0.90	-24.0	psf
h/2 to h	-0.90	-24.0	psf	h/2 to h	-0.90	-24.0	psf
h to 2h	-0.50	-13.3	psf	h to 2h	-0.50	-13.3	psf
> 2h	-0.30	-8.0	psf	> 2h	-0.30	-8.0	psf

SUMMARY OF PRESSURES - pressures in psf acting normal to the surface



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Main Wind Force Resisting System Wind Pressures - IBC 2024 / ASCE 7-22

Project:	NPS GATE Hangars 3&4	Notes:	Infill
Project #:	23-086.1		

Enclosure Classification	Enclosed	(t 26.12)	**For Roof Slopes LESS THAN 10 degrees**					
Risk Category	II	(IBC 1604.5)						
Exposure	D	(t 26.7)	GC_{pi} =	+/- 0.18	(Table 26.13-1)			
Mean Roof Height, h =	34	ft	V (mph) =	118	(t 26.5.1)			
B1, Length =	140	ft	K_{zt} =	1.00	(t 26.8.2)	z =	34	
B2, Width =	50	ft	K_d =	0.85	(t 26.6)	z_g =	1935	(Table 26.11-1)
Roof Slope Angle, θ =	2	degrees	G =	0.85	(t 26.11)	α =	11.5	(Table 26.11-1)
Parapet Height =	13	ft	K_e =	1.00	(t 26.9)			

WALL PRESSURES

$C_{p,WW}$ =	0.80	(Fig. 27.3-1)	$q_z = 0.00256\ K_z K_{zt} K_e V^2$		(lb/ft^2)	(Eq. 26.10-1)
$C_{p1,LW \,||\, B2}$ =	-0.50	(Fig. 27.3-1)	$p = q\ K_d\ GC_p - q_i\ K_d\ (GC_{pi})$		(lb/ft^2)	(Eq. 27.3-1)
$C_{p2,LW \,||\, B1}$ =	-0.26	(Fig. 27.3-1)	$qK_dGC_{p\ leeward\ ||\ B2}$	-15.37		
$C_{p,SW}$ =	-0.70	(Fig. 27.3-1)	$qK_dGC_{p\ leeward\ ||\ B1}$	-7.99		

z (ft)	K_z	q (psf)	WW	internal	Pos. Int. Pressure			Neg. Int. Pressure			Total	
			qK_dGC_p	$qK_d(GC_{pi})$	WW	LW||B1	LW||B2	WW	LW||B1	LW||B2	Wind || to B1	Wind || to B2
34	1.193	42.5	24.6	+/- 6.51	18.08	-14.50	-21.87	31.1	-1.48	-8.86	**32.6** psf	**40.0** psf
28	1.154	41.1	23.8	+/- 6.51	17.26	-14.50	-21.87	30.3	-1.48	-8.86	**31.8** psf	**39.1** psf
21	1.097	39.1	22.6	+/- 6.51	16.10	-14.50	-21.87	29.1	-1.48	-8.86	**30.6** psf	**38.0** psf
15	1.035	36.9	21.3	+/- 6.51	14.82	-14.50	-21.87	27.8	-1.48	-8.86	**29.3** psf	**36.7** psf

PARAPET PRESSURES

K_z =	1.262		Windward Parapet =	57.4	psf
q_p =	45.0		Leeward Parapet =	-38.3	psf
GC_{pn} =	1.5	windward	Total Pressure =	**95.6**	**psf**
GC_{pn} =	-1.0	leeward	Total Pressure assumes a parapet of equal height on windward and leeward side of building		

ROOF PRESSURES

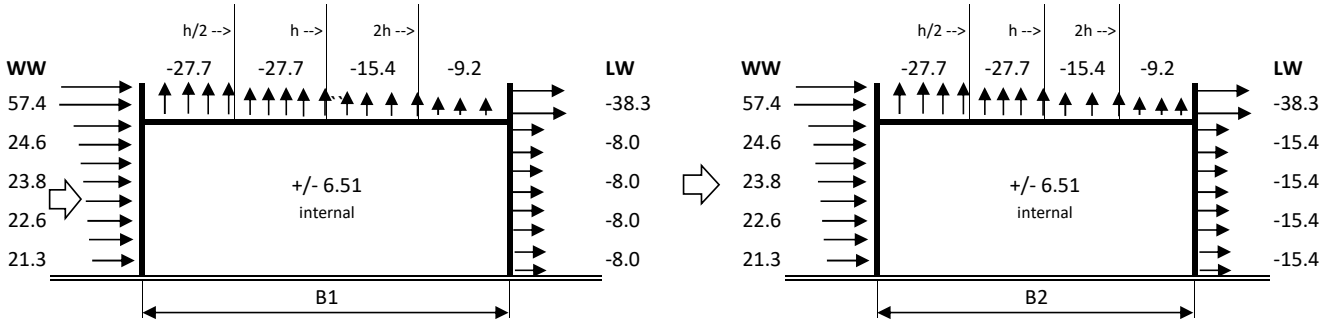
Wind Parallel to B1 h/B1 = 0.24

Dist from WW edge	C_p	p	
0 to h/2	-0.90	-27.7	psf
h/2 to h	-0.90	-27.7	psf
h to 2h	-0.50	-15.4	psf
> 2h	-0.30	-9.2	psf

Wind Parallel to B2 h/B2 = 0.68

Dist from WW edge	C_p	p	
0 to h/2	-0.90	-27.7	psf
h/2 to h	-0.90	-27.7	psf
h to 2h	-0.50	-15.4	psf
> 2h	-0.30	-9.2	psf

SUMMARY OF PRESSURES - pressures in psf acting normal to the surface



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Components & Cladding Wind Pressures - IBC 2024 / ASCE 7-22 (h <=60 ft)

Project:	GATE Hangars	Notes:	Hangars
Project #:	23-086.1		

Pressures shown on this sheet are conservative. Reduced values can be determined per roof pitch, etc. See appliable code sections

Risk Category	II	(IBC 1604.5)							
Exposure	D	(t 26.7)							
Enclosure Classification	Enclosed		GC_{pi} =	+/-	0.18	(t 26.13.1)			
h, Mean roof height =	37	ft	V (mph) =		118	(t 26.5.1)			
L, Long Dimension of Building =	140	ft	K_{zt} =		1.00	(t 26.8.2)	z =	37	ft
B, Short Dimension of Building =	120	ft	K_d =		0.85	(t 26.6)	zg =	1935	ft
h_p, Parapet height =	2	ft	G =		0.85	(t 26.11)	α =	11.5	
			K_e =		1.00	(t 26.9)			

Wall Pressures (ASCE 7-22, Part 1 - Section 30.3)

q_h =	43.17	(t 26.10)	$q_h = 0.00256\ K_z K_{zt} K_e V^2$	psf	(Eq. 26.10-1)
K_z =	1.211	(Table 26.10-1)	$p = q_h K_d [(GC_p) - (GC_{pi})]$	psf	(Eq. 30.3-1)
a (end zone 5 dim) =	12. ft	(Fig. 30.3-1)			

	-GCp		Wall Pressures (psf)	
EWA (ft^2)	Zone 4	Zone 5	Zone 4	Zone 5
10	-1.10	-1.40	-47.0	-58.0
20	-1.05	-1.30	-45.1	-54.3
50	-0.98	-1.15	-42.6	-48.8
100	-0.92	-1.05	-40.4	-45.1
200	-0.88	-0.95	-38.9	-41.5
500	-0.80	-0.80	-36.0	-36.0

1. (EWA Denotes Effective Wind Area) ; 2. Only Negative GCp is shown, Positive GCp produces lower pressures

Parapet Pressures (ASCE 7-22, Part 1 - Section 30.6)

q_p =	(at 39') =	43.56	(Eq. 26.10-1)	$q_p = 0.00256\ K_z K_{zt} K_e V^2$	psf	(Eq. 26.10-1)
K_z =	(at 39') =	1.222	(Table 26.10-1)	$p = q_p K_d (GC_p - GC_{pi})$	psf	(Eq. 30.6-1)
a (end zone 5 dim) =		12. ft	(Fig. 30.3-1)	See Figure 30.6-1 for applicable pressures		

	+GCp (Wall)		-GCp (Wall)		-GCp (Roof)		Parapet Pressures (psf)	
EWA (ft^2)	Zone 4	Zone 5	Zone 4	Zone 5	Zone 2	Zone 3	Zone 4	Zone 5
10	1.00	1.00	-1.10	-1.40	-2.30	-3.20	122.2	155.5
20	0.95	0.95	-1.05	-1.30	-2.15	-2.90	114.8	142.6
50	0.90	0.90	-0.98	-1.15	-1.93	-2.50	104.8	125.9
100	0.82	0.82	-0.92	-1.05	-1.78	-2.15	96.3	110.0
200	0.78	0.78	-0.88	-0.95	-1.60	-1.80	88.1	95.5
500	0.70	0.70	-0.80	-0.80	-1.40	-1.40	77.8	77.8

1. EWA Denotes Effective Wind Area ; 2. Two conditions should be checked: Windward Parapet = -GCp (Roof) - +GCp (Wall) and Leeward Parapet = -GCp(Wall) - +GCp (Wall). Windward always controls and is the final pressure shown above.

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Components & Cladding Wind Pressures - IBC 2024 / ASCE 7-22 (h <=60 ft)

Project: GATE Hangars
Project #: 23-086.1
Notes: Hangars

Roof Screen and RTU Pressures (ASCE 7-22, Sections 29.3.1 and 29.4.1)

h', Screen Height =		13.50	ft	$q_h = 0.00256\ K_z K_{zt} K_e V^2$		psf	(Eq. 26.10-1)
L', Screen Length || to L =		157.5	ft	RTU :	$p = q_h K_d (GC_r)$	psf	(Eq. 29.4-2)
B', Screen length || to B =		33	ft	Screen :	$p = q_z G C_f$		
A', Side area of RTU =		50	ft^2	GC_r (RTU) =	1.90		(t 29.4.1)
q_h	(at 37.') =	43.17	(Eq. 26.10-1)	C_f (Screen) =	1.55		(t 29.4.1)
K_h	(at 37.') =	1.211	(Table 26.10-1)	B/s =	3.1	Minimum	
q_z	(at 50.5') =	45.57	(Eq. 26.10-1)	s/h =	0.80	Assumed	
K_z	(at 50.5') =	1.278	(Table 26.10-1)	If bottom of screen is elevated more than 20% of the screen height, hand calc.			

NOT USED

Pressures :		
Screen =	71.4	psf
RTU =	69.7	psf

Screen pressure based on Case B which is 40% greater than increase in pressure from Case A due to offset of the force

Roof Uplift Pressures (ASCE 7-22, Part 1 - Section 30.3)

q_h =	30.60	(Eq. 26.10-1)	$q_h = 0.00256\ K_z K_{zt} K_d K_e V^2$	psf	(Eq. 26.10-1)
K_z =	1.010	(Table 26.10-1)	$p = q_h [(GC_p) - (GC_{pi})]$	psf	(Eq. 30.3-1)
0.6h =	22.2	ft (Fig. 30.3-2A)	0.2h =	7 ft	(Fig. 30.3-2A)

	-GCp			Roof Pressures (psf)		
EWA (ft^2)	Zone 1	Zone 2	Zone 3	Zone 1	Zone 2	Zone 3
10	-1.90	-2.30	-3.20	-63.7	-75.89	-103.44
20	-1.60	-2.15	-2.90	-54.5	-71.30	-94.25
50	-1.40	-1.93	-2.50	-48.4	-64.57	-82.01
100	-1.30	-1.78	-2.15	-45.3	-59.98	-71.30
200	-1.20	-1.60	-1.80	-42.2	-54.47	-60.59
500	-1.00	-1.40	-1.40	-36.1	-48.35	-48.35

1. (EWA Denotes Effective Wind Area) ; 2. This is raw uplift, DL may be used to offset uplift

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Components & Cladding Wind Pressures - IBC 2024 / ASCE 7-22 (h <=60 ft)

Project: GATE Hangars
Project #: 23-086.1

Notes: Infill

Risk Category	II	(IBC 1604.5)							
Exposure	D	(t 26.7)							
Enclosure Classification	Enclosed		GC_{pi} =	+/-	0.18	(t 26.13.1)			
h, Mean roof height =	34	ft	V (mph) =		118	(t 26.5.1)			
L, Long Dimension of Building =	140	ft	K_{zt} =		1.00	(t 26.8.2)	z =	34	ft
B, Short Dimension of Building =	50	ft	K_d =		0.85	(t 26.6)	zg =	1935	ft
h_p, Parapet height =	2	ft	G =		0.85	(t 26.11)	α =	11.5	
			K_e =		1.00	(t 26.9)			

Pressures shown on this sheet are conservative. Reduced values can be determined per roof pitch, etc. See appliable code sections

Wall Pressures (ASCE 7-22, Part 1 - Section 30.3)

q_h = 42.54 (t 26.10) $q_h = 0.00256\ K_z K_{zt} K_e V^2$ psf (Eq. 26.10-1)

K_z = 1.193 (Table 26.10-1) $p = q_h K_d [(GC_p) - (GC_{pi})]$ psf (Eq. 30.3-1)

a (end zone 5 dim) = 5. ft (Fig. 30.3-1)

	-GCp		Wall Pressures (psf)	
EWA (ft^2)	Zone 4	Zone 5	Zone 4	Zone 5
10	-1.10	-1.40	-46.3	-57.1
20	-1.05	-1.30	-44.5	-53.5
50	-0.98	-1.15	-41.9	-48.1
100	-0.92	-1.05	-39.8	-44.5
200	-0.88	-0.95	-38.3	-40.9
500	-0.80	-0.80	-35.4	-35.4

1. (EWA Denotes Effective Wind Area) ; 2. Only Negative GCp is shown, Positive GCp produces lower pressures

Parapet Pressures (ASCE 7-22, Part 1 - Section 30.6)

q_p = (at 36') = 42.96 (Eq. 26.10-1) $q_p = 0.00256\ K_z K_{zt} K_e V^2$ psf (Eq. 26.10-1)

K_z = (at 36') = 1.205 (Table 26.10-1) $p = q_p K_d (GC_p - GC_{pi})$ psf (Eq. 30.6-1)

a (end zone 5 dim) = 5. ft (Fig. 30.3-1) See Figure 30.6-1 for applicable pressures

	+GCp (Wall)		-GCp (Wall)		-GCp (Roof)		Parapet Pressures (psf)	
EWA (ft^2)	Zone 4	Zone 5	Zone 4	Zone 5	Zone 2	Zone 3	Zone 4	Zone 5
10	1.00	1.00	-1.10	-1.40	-2.30	-3.20	120.5	153.4
20	0.95	0.95	-1.05	-1.30	-2.15	-2.90	113.2	140.6
50	0.90	0.90	-0.98	-1.15	-1.93	-2.50	103.3	124.2
100	0.82	0.82	-0.92	-1.05	-1.78	-2.15	94.9	108.5
200	0.78	0.78	-0.88	-0.95	-1.60	-1.80	86.9	94.2
500	0.70	0.70	-0.80	-0.80	-1.40	-1.40	76.7	76.7

1. EWA Denotes Effective Wind Area ; 2. Two conditions should be checked: Windward Parapet = -GCp (Roof) - +GCp (Wall) and Leeward Parapet = -GCp(Wall) - +GCp (Wall). Windward always controls and is the final pressure shown above.

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Components & Cladding Wind Pressures - IBC 2024 / ASCE 7-22 (h <=60 ft)

Project: GATE Hangars
Project #: 23-086.1
Notes: Infill

Roof Screen and RTU Pressures (ASCE 7-22, Sections 29.3.1 and 29.4.1)

h', Screen Height =		13.50	ft	$q_h = 0.00256\ K_z K_{zt} K_e V^2$		psf	(Eq. 26.10-1)
L', Screen Length || to L =		157.5	ft	RTU :	$p = q_h K_d (GC_r)$	psf	(Eq. 29.4-2)
B', Screen length || to B =		33	ft	Screen :	$p = q_z G C_f$		
A', Side area of RTU =		50	ft^2	GC_r (RTU) =	1.90		(t 29.4.1)
q_h	(at 34.') =	42.54	(Eq. 26.10-1)	C_f (Screen) =	1.55		(t 29.4.1)
K_h	(at 34.') =	1.193	(Table 26.10-1)	B/s = NOT USED	3.1	Minimum	
q_z	(at 47.5') =	45.08	(Eq. 26.10-1)	s/h =	0.80	Assumed	
K_z	(at 47.5') =	1.265	(Table 26.10-1)	If bottom of screen is elevated more than 20% of the screen height, hand calc.			

Pressures :		
Screen =	70.7	psf
RTU =	68.7	psf

Screen pressure based on Case B which is 40% greater than increase in pressure from Case A due to offset of the force

Roof Uplift Pressures (ASCE 7-22, Part 1 - Section 30.3)

q_h =	30.16	(Eq. 26.10-1)	$q_h = 0.00256\ K_z K_{zt} K_d K_e V^2$	psf	(Eq. 26.10-1)
K_z =	0.995	(Table 26.10-1)	$p = q_h [(GC_p) - (GC_{pi})]$	psf	(Eq. 30.3-1)
0.6h =	20.4	ft (Fig. 30.3-2A)	0.2h =	7 ft (Fig. 30.3-2A)	

	-GCp			Roof Pressures (psf)		
EWA (ft^2)	Zone 1	Zone 2	Zone 3	Zone 1	Zone 2	Zone 3
10	-1.90	-2.30	-3.20	-62.7	-74.79	-101.93
20	-1.60	-2.15	-2.90	-53.7	-70.26	-92.88
50	-1.40	-1.93	-2.50	-47.6	-63.63	-80.82
100	-1.30	-1.78	-2.15	-44.6	-59.10	-70.26
200	-1.20	-1.60	-1.80	-41.6	-53.68	-59.71
500	-1.00	-1.40	-1.40	-35.6	-47.65	-47.65

1. (EWA Denotes Effective Wind Area) ; 2. This is raw uplift, DL may be used to offset uplift

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Components & Cladding Wind Pressures - IBC 2024 / ASCE 7-22 (h <=60 ft)

Project: GATE Hangars
Project #: 23-086.1

Notes: Lean-tos

Risk Category	II	(IBC 1604.5)							
Exposure	D	(t 26.7)							
Enclosure Classification	Enclosed		GC_{pi} =	+/-	0.18	(t 26.13.1)			
h, Mean roof height =	15	ft	V (mph) =		118	(t 26.5.1)			
L, Long Dimension of Building =	140	ft	K_{zt} =		1.00	(t 26.8.2)	z =	15	ft
B, Short Dimension of Building =	30	ft	K_d =		0.85	(t 26.6)	zg =	1935	ft
h_p, Parapet height =	2	ft	G =		0.85	(t 26.11)	α =	11.5	
			K_e =		1.00	(t 26.9)			

Pressures shown on this sheet are conservative. Reduced values can be determined per roof pitch, etc. See appliable code sections

Wall Pressures (ASCE 7-22, Part 1 - Section 30.3)

q_h = 36.89 (t 26.10) $q_h = 0.00256\ K_z K_{zt} K_e V^2$ psf (Eq. 26.10-1)

K_z = 1.035 (Table 26.10-1) $p = q_h K_d [(GC_p) - (GC_{pi})]$ psf (Eq. 30.3-1)

a (end zone 5 dim) = 3. ft (Fig. 30.3-1)

	-GCp		Wall Pressures (psf)	
EWA (ft^2)	Zone 4	Zone 5	Zone 4	Zone 5
10	-1.10	-1.40	-40.1	-49.5
20	-1.05	-1.30	-38.6	-46.4
50	-0.98	-1.15	-36.4	-41.7
100	-0.92	-1.05	-34.5	-38.6
200	-0.88	-0.95	-33.2	-35.4
500	-0.80	-0.80	-30.7	-30.7

1. (EWA Denotes Effective Wind Area) ; 2. Only Negative GCp is shown, Positive GCp produces lower pressures

Parapet Pressures (ASCE 7-22, Part 1 - Section 30.6)

q_p = (at 17') = 37.71 (Eq. 26.10-1) $q_p = 0.00256\ K_z K_{zt} K_e V^2$ psf (Eq. 26.10-1)

K_z = (at 17') = 1.058 (Table 26.10-1) $p = q_p K_d (GC_p - GC_{pi})$ psf (Eq. 30.6-1)

a (end zone 5 dim) = 3. ft (Fig. 30.3-1) See Figure 30.6-1 for applicable pressures

	+GCp (Wall)		-GCp (Wall)		-GCp (Roof)		Parapet Pressures (psf)	
EWA (ft^2)	Zone 4	Zone 5	Zone 4	Zone 5	Zone 2	Zone 3	Zone 4	Zone 5
10	1.00	1.00	-1.10	-1.40	-2.30	-3.20	105.8	134.6
20	0.95	0.95	-1.05	-1.30	-2.15	-2.90	99.4	123.4
50	0.90	0.90	-0.98	-1.15	-1.93	-2.50	90.7	109.0
100	0.82	0.82	-0.92	-1.05	-1.78	-2.15	83.3	95.2
200	0.78	0.78	-0.88	-0.95	-1.60	-1.80	76.3	82.7
500	0.70	0.70	-0.80	-0.80	-1.40	-1.40	67.3	67.3

1. EWA Denotes Effective Wind Area ; 2. Two conditions should be checked: Windward Parapet = -GCp (Roof) - +GCp (Wall) and Leeward Parapet = -GCp(Wall) - +GCp (Wall). Windward always controls and is the final pressure shown above.

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Components & Cladding Wind Pressures - IBC 2024 / ASCE 7-22 (h <=60 ft)

Project: GATE Hangars
Project #: 23-086.1

Notes: Lean-tos

Roof Screen and RTU Pressures (ASCE 7-22, Sections 29.3.1 and 29.4.1)

h', Screen Height =		13.50	ft	$q_h = 0.00256\ K_z K_{zt} K_e V^2$		psf	(Eq. 26.10-1)
L', Screen Length || to L =		157.5	ft	RTU :	$p = q_h K_d (GC_r)$	psf	(Eq. 29.4-2)
B', Screen length || to B =		33	ft	Screen :	$p = q_z G C_f$		
A', Side area of RTU =		50	ft^2	GC_r (RTU) =	1.90		(t 29.4.1)
q_h	(at 15.') =	36.89	(Eq. 26.10-1)	C_f (Screen) =	1.55		(t 29.4.1)
K_h	(at 15.') =	1.035	(Table 26.10-1)	B/s =	3.1	Minimum	
q_z	(at 28.5') =	41.25	(Eq. 26.10-1)	s/h =	0.80	Assumed	
K_z	(at 28.5') =	1.157	(Table 26.10-1)	If bottom of screen is elevated more than 20% of the screen height, hand calc.			

NOT USED

Pressures :		
Screen =	64.7	psf
RTU =	59.6	psf

Screen pressure based on Case B which is 40% greater than increase in pressure from Case A due to offset of the force

Roof Uplift Pressures (ASCE 7-22, Part 1 - Section 30.3)

q_h =	26.16	(Eq. 26.10-1)	$q_h = 0.00256\ K_z K_{zt} K_d K_e V^2$	psf	(Eq. 26.10-1)
K_z =	0.863	(Table 26.10-1)	$p = q_h [(GC_p) - (GC_{pi})]$	psf	(Eq. 30.3-1)
0.6h =	9.0	ft (Fig. 30.3-2A)	0.2h =	3 ft	(Fig. 30.3-2A)

	-GCp			Roof Pressures (psf)		
EWA (ft^2)	Zone 1	Zone 2	Zone 3	Zone 1	Zone 2	Zone 3
10	-1.90	-2.30	-3.20	-54.4	-64.87	-88.40
20	-1.60	-2.15	-2.90	-46.6	-60.94	-80.56
50	-1.40	-1.93	-2.50	-41.3	-55.19	-70.10
100	-1.30	-1.78	-2.15	-38.7	-51.26	-60.94
200	-1.20	-1.60	-1.80	-36.1	-46.56	-51.79
500	-1.00	-1.40	-1.40	-30.9	-41.33	-41.33

1. (EWA Denotes Effective Wind Area) ; 2. This is raw uplift, DL may be used to offset uplift

SNOW LOADS: DRIFTING SNOW

Hangar Roof:

- Rise is 2' over 60', so 0.4V:12H
- There are no parapets on the hangar roof. The snow can slide off of the hangar roof onto the Lean-to or WPA Infill Building Roof below (Leeward drift).

Lean-to Roof:

- Rise is 9'"over 30', so 0.3V:12H
- Windward drifts at parapets and wall projections must be considered.

WPA Infill Building Roof:

- Rise is 1' over 24', so 0.5V:12H
- Windward drifts at parapets and wall projections must be considered.

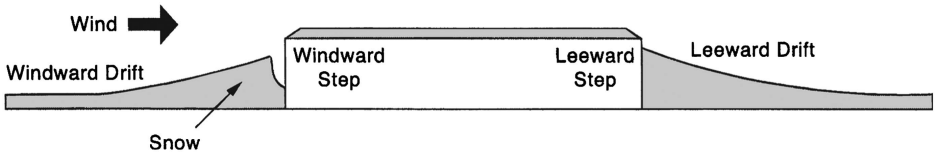
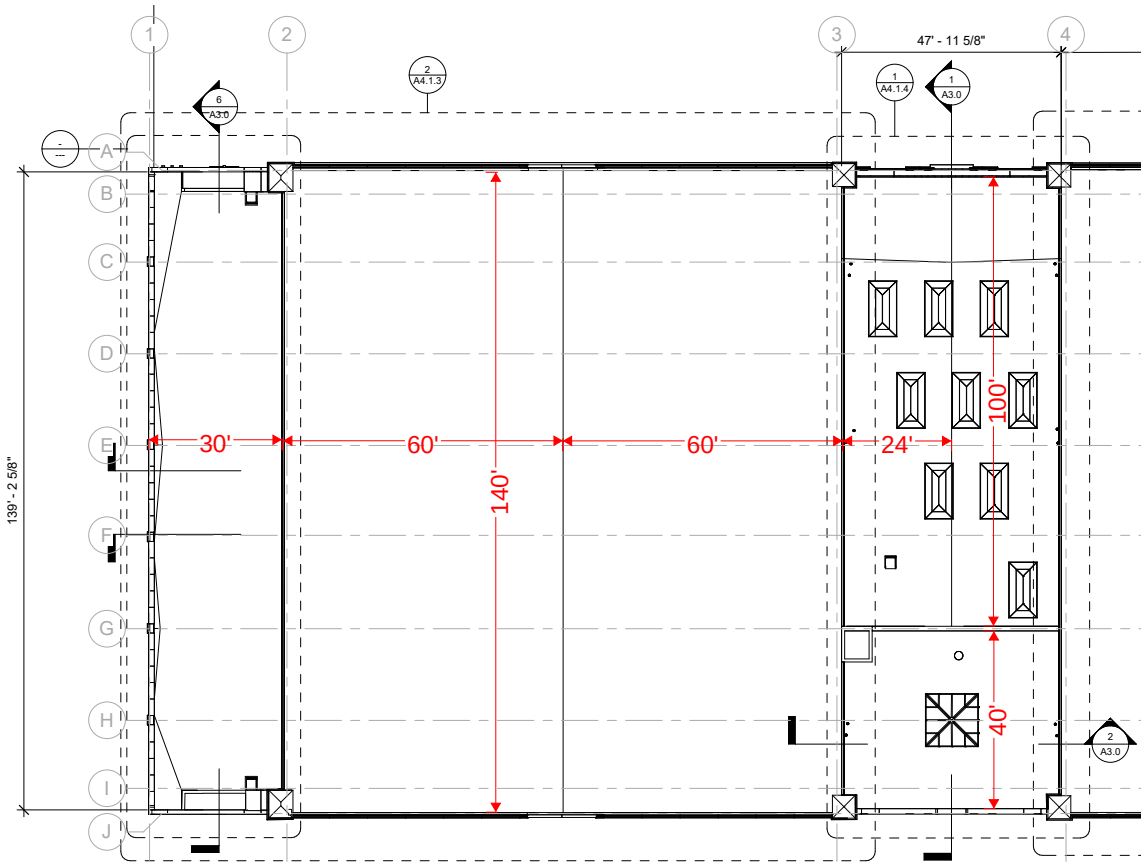


Figure 7.7-1. Drifts formed at windward and leeward steps.



1 A1.1 OVERALL ROOF PLAN
SCALE A

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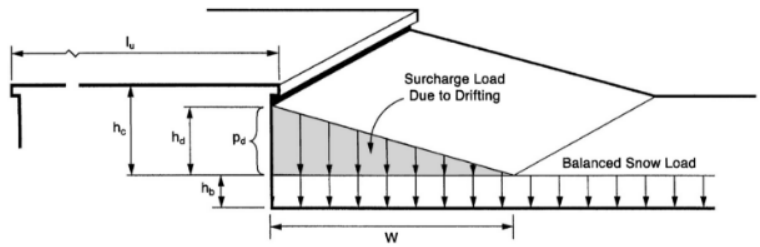
SNOW LOADS
ASCE 7-22

Project: **NPS GATE Hangars**
Project No.: 23-086.1
Description:

Basic Info:		
p_g =	45	psf
C_e =	0.8	
C_t =	1.2	
RC =	II	

Sloped and Sliding:			
Slope:	.25H	:12V =	1.2 degrees
C_s =	1.00		
a conservative value of Cs is calculated here, but can be overridden			
W =	60.0	ft - horizontal distance from eave to ridge of upper sloped roof for Sliding	

Drifting:		
$L_{u,lw}$ =	60	ft - Roof fetch for leeward drift
$L_{u,ww}$ =	48	ft - Roof fetch for windward drift
Step, h =	4.0	ft - Total step height for drift calculation
W_2 =	0.55	Winter Wind Parameter, Fig. 7.6-1



7.3 Flat Roof Snow Loads, p_f

$p_f = 0.7\ C_e\ C_t\ p_g$ = 30.2 psf
$p_m = p_g$ not to exceed $p_{m,max}$ 30.0 psf
p_m is minimum for roofs with slope less than 15 degrees

7.4 Sloped Roof Snow Loads, p_s

$p_s = C_s\ p_f$ = 30.24 psf

7.7 Drifts on Lower Roofs

$\gamma = 0.13\ pg + 14 <= 30$	19.85	pcf
$h_b = p_s/\gamma$ =	1.52	ft
$h_c = h - h_b$ =	2.48	ft
$h_{d,lw} = (1.5)\ (P_g^{0.74}\ L_{u,lw}^{0.7}\ W_2^{1.7}/\ \gamma)^{1/2}$ =	3.47	
$h_{d,ww} = (3/4)\ (1.5)\ (P_g^{0.74}\ L_{u,ww}^{0.7}\ W_2^{1.7}/\ \gamma)^{1/2}$ =	2.41	
$h_{d,max}$ =	3.47	ft
Drift Load, p_d =	68.9	psf
$h_d <= h_c$?	NO	
Drift width, w_{lw} =	19.5	ft
for $h_d<=h_c$ $4h_d$ =	13.9	
for $h_d>h_c$ $4h_d^2/h_c$ =	19.5	
max, w_{lw} = $8h_c$ =	19.8	
Drift width, w_{ww} = $8h_{d,ww}$	19.3	ft

Drift Summary

Drift Load, p_d =	68.9	psf
Total Load, $p_b + p_d$ =	99.2	psf
Drift Length, w_{lw} =	19.5	ft
w_{ww} =	19.3	ft
Drift Length to p_g =	15.3	ft

Drift length to pg is the distance to the edge of drift which is truncated at pg instead of ps

7.8 Roof Projections and Parapets

Lean-to Roof, Drift against E/W Walls

$L_{u,ww}$ =	30	ft - Roof fetch for windward drift
$h_{d,ww} = (3/4)\ (1.5)\ (P_g^{0.74}\ L_{u,ww}^{0.7}\ W_2^{1.7}/\ \gamma)^{1/2}$ =	2.04	ft
Drift Load, p_d =	40.6	psf
Drift width, w_{ww} = $8h_{d,ww}$	16.3	ft

Lean-to Roof, Drift against N/S Pocket Walls

$L_{u,ww}$ =	130	ft - Roof fetch for windward drift
$h_{d,ww} = (3/4)\ (1.5)\ (P_g^{0.74}\ L_{u,ww}^{0.7}\ W_2^{1.7}/\ \gamma)^{1/2}$ =	3.41	ft
Drift Load, p_d =	67.8	psf
Drift width, w_{ww} = $8h_{d,ww}$	27.3	ft

Roof Projection and Parapet Summary

Total Load, $p_b + p_d$ = 70.8 psf
Equivalent height = 3.57 ft
This height can develop at clerestory side.
Drift height will be limited to parapet height at opposite end.
= 2'-6" with p = 49.6 psf

Total Load, $p_b + p_d$ = 98.0 psf
Equivalent height = 4.94 ft
This height can develop at pocket wall side.
Drift height will be limited to parapet height adjacent to pocket.
= 2'-6" with p = 49.6 psf

7.9 Sliding Snow (from Hangar roof to WPA Infill or Lean-to lower roofs)

Total sliding snow = $0.4\ p_f\ W$ =	0	plf
Sliding Snow distributed over width of…	15	ft
Sliding Snow Load =	0.0	psf

IV. Foundation Design

Design of Slab for Estimated Scissor Lift Loading

Exterior Slabs - Scissor Lift

From materials, site, and designer:

Concrete compressive strength: f'c = 4000 psi
Modulus of rupture: MOR= 569 psi
Subgrade modulus: k = 115 pci
Safety factor: SF = 2.0

From Scissor lift specifications:
Lift truck capacity = 0.5 kips
Total Weight = 3.354 kips
Total Axle Load (Assuming 60% to one axle) = 2.01 kips

Wheel spacing = 27.7 inches
Tire pressure= 165 psi
Wheel contact area = 20.32 square inches (one wheel)
Effective diameter of contact area= 5.1 inches

Allowable stress = 569/2 = 284 psi
Stress per 1000 lbs = 284/2.01 = 140 psi per 1000 lbs

--->Required thickness, tr = 4.5 in.
---> USE t = 5 inches

EXAMPLE CALC FROM PCA

3.2 — Using PCA charts to design for axle loading: AUTHORS' CHOICE

The PCA design charts appear in the appendix along with larger copies of all other design graphs. Reduced size charts are used in these examples for illustrative purposes. These PCA charts were originally published in *References 5* and *12*.

For the first problem using PCA charts, the following values will be used:

From materials, site, and designer:

Concrete compressive strength: $f_c' = 4500$ psi
Modulus of rupture (using $9 \times \sqrt{f_c'}$): $MOR = 604$ psi
Subgrade modulus: $k = 200$ pci
Safety factor: SF = 2.0

From lift truck specifications:

CONTACT AREA
= 0.01 P
1/2.5 SHAPE FACTOR

Lift truck capacity = 20 kips
Vehicle weight = 22 kips
Total axle load = 42 kips *(Assume 100% on front loaded axle)*
Axle: Single wheels at each end of axle
Wheel spacing = 42 inches
Tire pressure = 250 psi *(Rational value of 250 is used to represent a solid tire, to obtain contact area.)*
Wheel contact area = 84 square inches (one wheel) *(Calculated here from a single wheel load divided by tire pressure [21,000/250 = 84 square inches]. However, the authors recommend that you use the manufacturer's specification sheet if it is available for the vehicle.)*
Effective diameter of contact area = 10.34 inches *(Needed for use of WRI charts)*

The slab thickness is then selected using *Figure 14* in the following manner:

- Calculate the stress per 1000 pounds of axle load. First find the allowable stress, which is the modulus of rupture divided by the factor of safety = 604/2 = 302 psi. This is the acceptable flexural tensile stress for the concrete.
- Divide this by the axle load. This gives:

  302/42 = 7.19 psi per 1000 pounds of axle load

  which is to be located on the left hand vertical axis of *Figure 14*.

A.3 — Portland Cement Association, Wire Reinforcement Institute, and Corps of Engineers Charts

SCISSOR LIFT

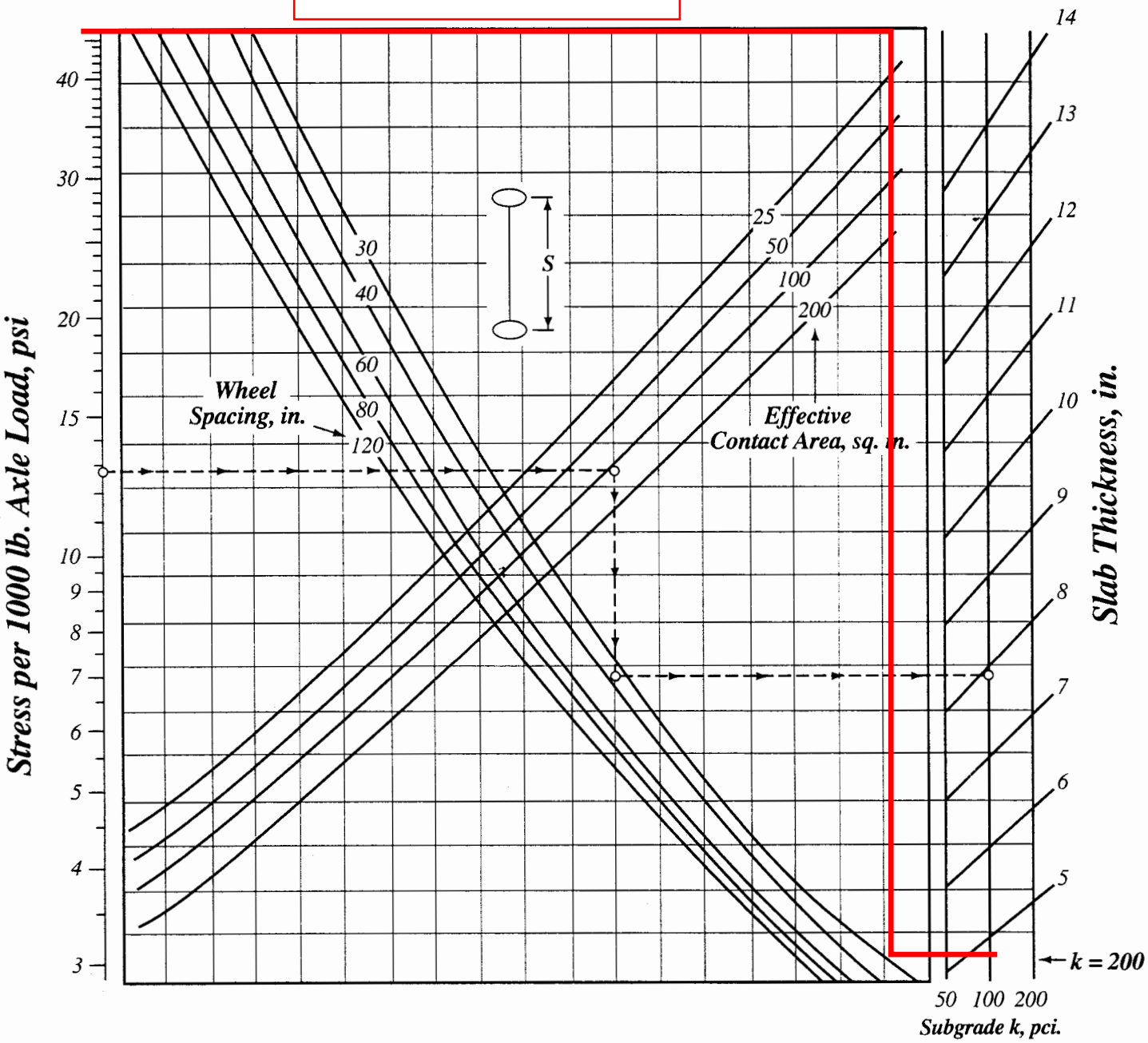


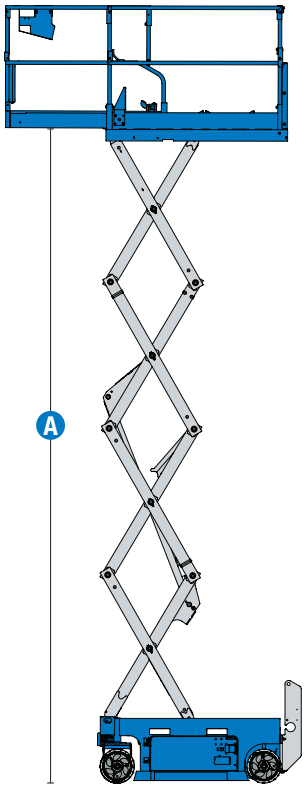
Figure A.7 *Use of a PCA design chart to select slab thickness for single axle loading.*

Genie® GS™-1930 & GS-1932

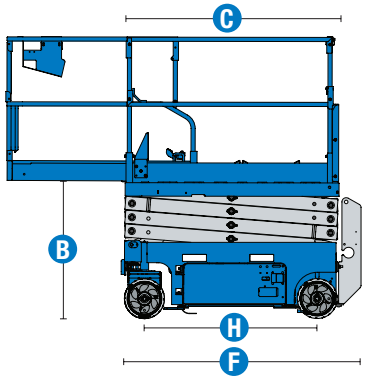
Specifications

EXAMPLE CUTSHEET FOR ESTIMATING LIFT LOADS.

Models	GS-1930		GS-1932	
Measurements	**US**	**Metric**	**US**	**Metric**
Working height max. - indoor[1]	25 ft 3 in	7.85 m	25 ft 3 in	7.85 m
Working height max. - outdoor[1]	20 ft 8 in	6.47 m	20 ft 8 in	6.47 m
A Platform height max. - indoor	19 ft 3 in	5.85 m	19 ft 3 in	5.85 m
A Platform height max. - outdoor	14 ft 8 in	4.47 m	14 ft 8 in	4.47 m
B Platform height - stowed	3 ft 5 in	1.04 m	3 ft 5 in	1.04 m
C Platform length - outside	5 ft 5 in	1.64 m	5 ft 5 in	1.66 m
C Platform length - outside, extended	8 ft 5 in	2.55 m	8 ft 5 in	2.55 m
Slide-out platform extension deck - length	2 ft 11.8 in	0.91 m	3 ft	0.91 m
D Platform width - outside	2 ft 5.9 in	0.76 m	29.9 in	0.76 m
Guardrail height	3 ft 7 in	1.10 m	3 ft 7 in	1.10 m
Toeboard height	6 in	15 cm	6 in	15 cm
E Machine Height - stowed (folding rails)	7 ft 1 in	2.16 m	7 ft 1 in	2.16 m
Machine Height - stowed (rails lowered)	5 ft 11 in	1.80 m	5 ft 11 in	1.80 m
F Length machine - stowed	6 ft	1.83 m	6 ft	1.83 m
F Length machine - stowed, extended	8 ft 11 in	2.73 m	8 ft 11 in	2.72 m
G Width	2 ft 6.2 in	0.77 m	32.2 in	0.82 m
H Wheelbase	4 ft 4 in	1.32 m	4 ft 4 in	1.32 m
I Ground clearance (center) - pothole guards retracted	3.2 in	8 cm	3.2 in	8 cm
I Ground clearance - pothole guards deployed	0.9 in	2.30 cm	0.9 in	2.30 cm



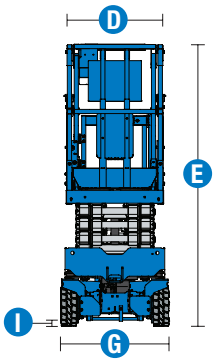
Productivity				
Max. platform occupancy (in/out)	2/1		2/1	
Lift capacity	500 lb	227 kg	500 lb	227 kg
Lift capacity - extension deck extended	250 lb	113 kg	250 lb	113 kg
Max. platform drive height	19 ft 3 in	5.85 m	19 ft 3 in	5.85 m
Drive speed - stowed	2.5 mph	4.0 km/h	2.5 mph	4.0 km/h
Drive speed - raised	0.5 mph	0.8 km/h	0.5 mph	0.8 km/h
Gradeability - stowed[2]	25 %		25 %	
Turning radius - inside/outside	0/5 ft 1 in	0/1.55 m	0/5 ft 1 in	0/1.55 m
Lifting speed - raise/lower	26/22 sec		26/22 sec	
Tilt sensor activation (front to back/side to side)	3° / 1.5°		3° / 1.5°	
Tires - solid non-marking	12×4.5 in	31×11 cm	12×4.5 in	30.5×11.5 cm



Power				
Power - FLA	24V DC (4×6V 210 Ah)		24V DC (4×6V 210 Ah)	
Power - Lithium-Ion	24V DC (1×24V 90 Ah)		24V DC (1×24V 90 Ah)	
Hydraulic tank capacity (E-Drive)	1 gal	3.8 L	1 gal	3.8 L

Weight(3) and Floor Loading[4]				
Weight	3,209 lb	1,456 kg	2,929 lb	1,329 kg
Weight max. (adding all possible options)	3,354 lb	1,521 kg	3,205 lb	1,454 kg
Tire load max.	1,483 lb	672 kg	1,473 lb	668 kg
Tire contact pressure	165 psi	1,136 kPa	164 psi	1,128 kPa
Occupied floor pressure	287 psf	13.73 kPa	258 psf	12.38 kPa

Sound and Vibration Levels				
Sound Pressure level (ground workstation)	< 70 dBA		< 70 dBA	
Sound Pressure level (platform workstation)	< 70 dBA		< 70 dBA	
Vibrations	< 8 ft 2 in/s²	< 2.5 m/s²	< 8 ft 2 in/s²	< 2.5 m/s²



(1) The metric equivalent of working height adds 2 m to platform height. US adds 6 ft to platform height.
(2) Gradeability applies to driving on slopes and may vary depending on options and machine configurations. See operator's manual for details regarding slope ratings.
(3) Weight refers to standard configuration and may vary depending on options and/or country standards.
(4) Floor loading information is approximate and may vary depending on options and machine configurations. It should be used only with adequate safety factors.

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Project:	GATE Hangars	Job No.:	23-086.1	Page:	1
Subject:	Slab on Grade Design	By:	JCT	Date:	2-Jun-25

Design of new slab on grade reinforcing to span 4 ft void.

Geometry

Span	l	4	ft	
Minimum slab depth	h_{min}	2.4	in	*(Span/20)*
Total slab depth	h	6	in	
Effective depth	d	3	in	
Unit width	b	12	in	

Material Strengths and Design Parameters

Compressive Strength	f'_c	4	ksi			
Yield Strength	F_y	60	ksi (rebar)	or	65	ksi (WWF)
Strength Reduction Factors	ϕ_M	0.90				
	ϕ_V	0.75				

Design Forces

Dead Load (6" SOG)	w_{DL}	75	psf			
Live Load (Restaurant / Retail)	w_{LL}	150	psf	or	P_{LL} 2000	lb concentrated
Factored Total Load	w_u	330	psf		P_u 3200	lb concentrated
Maximum Negative Bending Moment	M_{u-}	1.60	k-ft / ft			
Maximum Positive Bending Moment	M_{u+}	1.60	k-ft / ft			
Maximum Shear	V_u	3.2	k / ft			

Flexural Design

Design of Positive Bending Reinforcement

Coefficient of resistance	k_{req}	0.1975	Solve for ρ (rebar):		Solve for ρ (WWF):		
	ρ_{req}	0.0034	ω	0.050902	ω	0.049468	
	ρ min1	0.0033	k	0.197493	ρ_{req}	0.0030	
	ρ min2	0.0045		*(Use data seek function to set k equal to k_{req} by changing ρ_{req})*	k	0.197516	*(Use data seek function to set k equal to k_{req} by changing ρ_{req})*
	ρ	**0.0034**					
Area of Steel used	As	0.12	in^2 / ft		As	0.11	in^2 / ft
		#4 @ 12, As = 0.20 in^2				4x4-W4.0-W4.0, As = 0.12 in^2	

Design of Negative Bending Reinforcement

Coefficient of resistance	k_{req}	0.1975	Solve for ρ:		
	ρ_{req}	0.0034	ω	0.050896	
	ρ min1	0.0033	k	0.19747	*(Use data seek function to set k equal to calculated k by changing ρ_{req})*
	ρ min2	0.0045			
	ρ	**0.0034**			
Area of Steel used	As	0.12	in^2 / ft		
		#4 @ 12, As = 0.20 in^2			

One-Way Shear Check

Concrete Shear Capacity	ϕV_c	**3.42 k / ft**	**> Vu, OK**

Table I—Common stock styles of welded wire fabric

STYLE DESIGNATION |		Steel Area, square inches per foota		Approximate Weight, pounds per 100 square feet6
New Designation (By W-number)	Old Designation (By Steel wire Gage)	Longitudinal	Transverse	
ROLLS				
6x6—W1.4xW1.4	6x6—10x10	.028	.028	21
6x6—W2.0xW2.0	6x6—8x8 c	.040	.040	29
6x6—W2.9xW2.9	6x6—6x6	.058	.058	42
6x6—W4.0xW4.0	6x6—4x4	.080	.080	58
4x4—W1.4xW1.4	4x4—10x10	.042	.042	31
4x4—W2.0xW2.0	4x4—8x8c	.060	.060	43
4x4—W2.9xW2.9	4x4 6x6	.087	.087	62
4x4—W4.0xW4.0	4x4—4x4	.120	.120	85
SHEETS				
6x6—W2.9xW2.9	6x6—6x6	.058	.058	42
6x6—W4.0xW4.0	6x6—4x4	.080	.080	58
6x6—W5.5xW5.5	6x6—2x2d	.110	.110	80
4x4—W4.0xW4.0	4x4—4x4	.120	.120	85

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Design of Typical Shear Dowel at New Hangar Slab Edge

Project:	GATE Hangars	Job No.:	23-086.1	Page:	1
Subject:	Shear Dowel	By:	JCT	Date:	27-Jun-25

Geometry

Slab Depth at Joint	**t**	12	in
Unit width	**b**	12	in
Angle of Reinforcement	**α**	90	

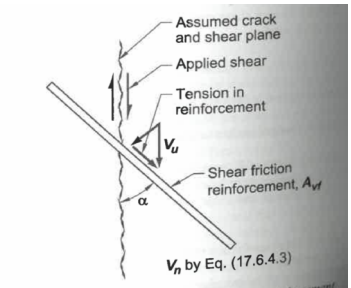


Fig. R22.9.4.3a—Tension in shear friction reinforcement.

Material Strengths and Design Parameters

Compressive Strength	**f'c**	4	ksi	
Yield Strength	**Fy**	60	ksi	
Lamda	**λ**	1.0		
Friction factor		0.6		See Table 22.9.4.2
Coefficient of Friction	**μ**	0.6		

Design Forces

Dead Load (6" Slab)	w_{DL}	75	psf				
Live Load (Storage)	w_{LL}	150	psf	or	P_{LL}	1000	lb concentrated
Factored Total Load	w_u	330	psf		P_u	1600	lb concentrated
Trib Width (2ft + downturn)		3	ft				
Maximum Shear	V_u	1.6	k				

Joint Shear Check

		Bar No. (#)	Spacing (in.)	
		4	24	
Shear Reinforcement	A_{vf}	0.098	in2	
Shear Plane treatment		Roughened		See Table 22.9.4.4
Max Shear	$V_{n,\,max}$	115.2	k	
Concrete Shear Capacity	ϕV_c	**2.65**	**k**	**> Vu, OK with #4 @ 24" OC**

Table 22.9.4.2—Coefficients of friction

Contact surface condition	Coefficient of friction μ[1]	
Concrete placed monolithically	1.4λ	(a)
Concrete placed against hardened concrete that is clean, free of laitance, and intentionally roughened to a full amplitude of approximately 1/4 in.	1.0λ	(b)
Concrete placed against hardened concrete that is clean, free of laitance, and not intentionally roughened	0.6λ	(c)
Concrete placed against as-rolled structural steel that is clean, free of paint, and with shear transferred across the contact surface by headed studs or by welded deformed bars or wires.	0.7λ	(d)

[1] λ = 1.0 for normalweight concrete; λ = 0.75 for all lightweight concrete. Otherwise, λ is calculated based on volumetric proportions of lightweight and normalweight aggregate as given in 19.2.4, but shall not exceed 0.85.

Table 22.9.4.4—Maximum V_n across the assumed shear plane

Condition	Maximum V_n		
Normalweight concrete placed monolithically or placed against hardened concrete intentionally roughened to a full amplitude of approximately 1/4 in.	Least of (a), (b), and (c)	$0.2f_c'A_c$	(a)
		$(480 + 0.08f_c')A_c$	(b)
		$1600A_c$	(c)
Other cases	Lesser of (d) and (e)	$0.2f_c'A_c$	(d)
		$800A_c$	(e)

SUMMARY OF INTERIOR EQUIPMENT ON PADS:

1) SWITCHBOARD

UNIT FOOTPRINT = 48" x 216"
WEIGHT = 7,043 LB
EQUIPMENT PAD FOOTPRINT = 4'-8" x 23'-0" → A = 107.4 sf

ASSUME 12"-THICK CONCRETE PAD (150 psf)

→ W_{TOTAL} = 7,043# + (107.4 sf)(150 psf) = 23,144#

q = 23,144# / 107.4 sf = 216 psf

2) TRANSFORMER

UNIT FOOTPRINT = 24.88" x 21.13"
WEIGHT = 362#
EQUIPMENT PAD FOOTPRINT = 2'-6" x 2'-8" → A = 6.67 sf

q = 150 psf + 362# / 6.67 sf = 205 psf

3) INVERTER (2 BATTERY CABINETS)

WEIGHT = 4,063# TOTAL FOR BOTH CABINETS
UNIT FOOTPRINT = 30" x 47" EACH
EQUIPMENT PAD FOOTPRINT = 3'-7" x 9'-4" → A = 33.44 sf

q = 150 psf + 4,063#/33.44 sf = 272 psf

DESIGN PAD REINFORCEMENT

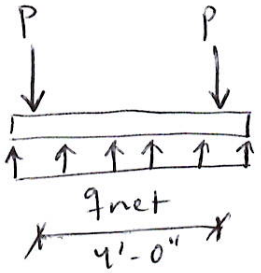
MIN REINF. FOR 12" THICKNESS:

$0.0018(12")(12") = 0.2592 \text{ in}^2/\text{ft}$ T&S

→ NEED #5 @ 14" OR #4 @ 9 E.W.

SWITCHBOARD:

HAS (3) RAILS ALONG LENGTH, 4' APART & ALONG ℄. DISTRIBUTION OF WT. IS UNKNOWN. CONSERVATIVELY APPLY JUST @ OUTER RAILS:



$P = 7043\#/2/18' = 196 \text{ plf}$

$q = 196 \text{ plf} \times 2/4.67' = 84 \text{ psf NET}$

$M_u = 1.4(84 \text{ psf})(4')^2/8 = 236 \text{ \#-ft/ft}$

$d = 12" - 3" - 5/8" - 5/16" = 8.06"$

$f'_c = 4000 \text{ psi}$, $f_y = 60 \text{ ksi}$

$$\rho_{req} = \frac{M_u}{\phi b d^2} = \frac{(0.236)(12) \text{ k-in}}{0.9(12")(8.06")^2} = 0.004$$

→ $\rho_{req} < 0.001$ SO T&S STEEL OK.

V. Analysis and Design of Roof Elements

MATTEO | FERRAN
STRUCTURAL ENGINEERS

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Alexandria, VA 22314
703.350.4151

Project: NPS GATE Hangar 3 & 4
Project No.: 23-086.1

By: JCT / NF

Existing vs. New Load Comparison

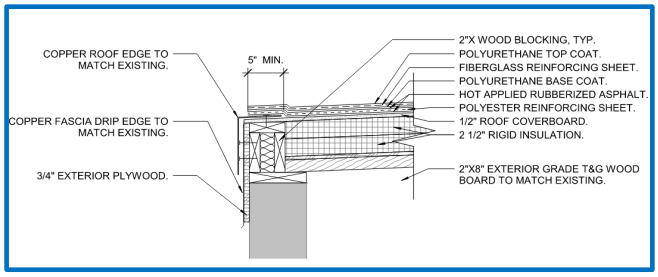
Note: Original roofing was not indicated on original drawings. Existing loads below reflect the information in the 1981 HSR.

Roof system weights (Dead Loads)

	EXISTING
Bituminous, gravel-covered roofing	5.50 psf
2x Existing Decking	5.00 psf
Purlin 10"Channel x 15.3# @ 6'-0" O.C.	2.60 psf
CMEP Allowance	4.00 psf
	17.10 psf

	NEW
Polyurethane Roofing	5.00 psf
1/2" Roof coverboard	0.50 psf
5" Rigid Insulation	1.25 psf
1/2" New Plywood	1.50 psf
2x New Decking	5.00 psf
Purlin 10"Channel x 15.3# @ 6'-0" O.C.	2.60 psf
CMEP Allowance	4.00 psf
	19.85 psf

Each hangar was basically the same. The floor space measured 120 by 140 feet. The hangars were built on 125 thirty-five-foot concrete piles. The floors were steel reinforced concrete. The doors had an overhead clearance of twenty-two feet and were made of an aluminum alloy and glass that could be moved by hand. They consisted of twelve partitions that could be moved completely out of the way. The structure had a tar and gravel roof. In addition, each hangar had a 30-by 140-foot lean-to attached to either the east or west side for use as office space. The lean-tos were constructed of the same brick and artificial stone used in the other buildings. Their floorplans varied.[16]



Increase in weight with new system (neglecting self weight of purlins): 19.0%
--> This is greater than the 5% weight increase allowed by the IEBC so existing framing needs to be checked for load increase.

Live Loads

Current code specifies 20 psf minimum live load for roofs. The 1929 New York City Building Code specified 40 psf live load for roofs with a slope less than 20 degrees and 30 psf live load for roofs with steeper slopes.

Snow Loads

The 1929 New York City Building Code did not specify snow loads. It is assumed that those were included in the live load design value.

Per current code, balanced roof snow load is 30.3 psf (see detailed calculation in this package.
Roof slope is less than 2.38 deg, therefore unbalanced snow loading is not required.

Note: Unbalanced loads need not be considered for $\theta > 30.2°$ (7 on 12) or for $\theta \leq 2.38°$ (1/2 on 12).

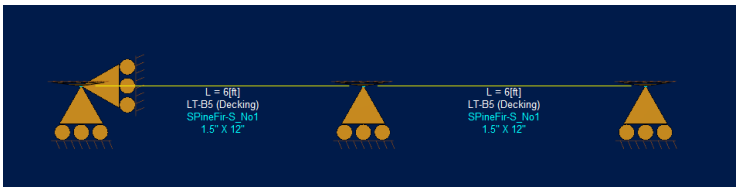
Drift from higher roofs and against parapets and wall projections must be considered for Lean-to roofs and WPA Infill Building roof per current code. See detailed calculations.

NAME: LT-B5
DESCRIPTION: New sheathing spanning between purlins

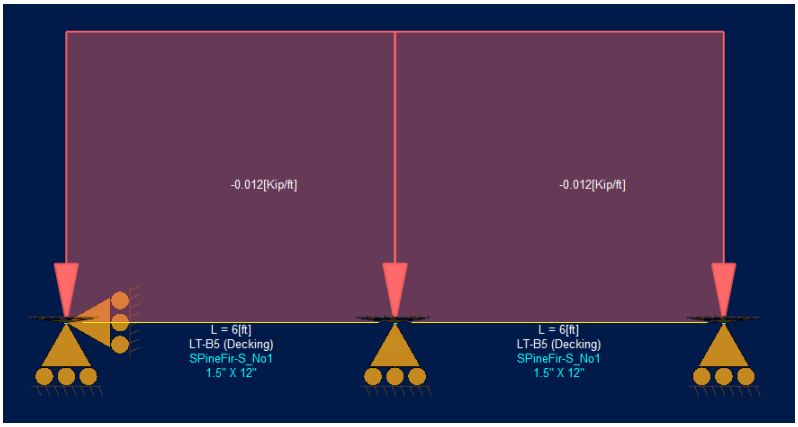
Typical New Decking Design Check

SPAN: 6 ft Assume 2-span minimum Purlins spaced @ 6'-0" typ.

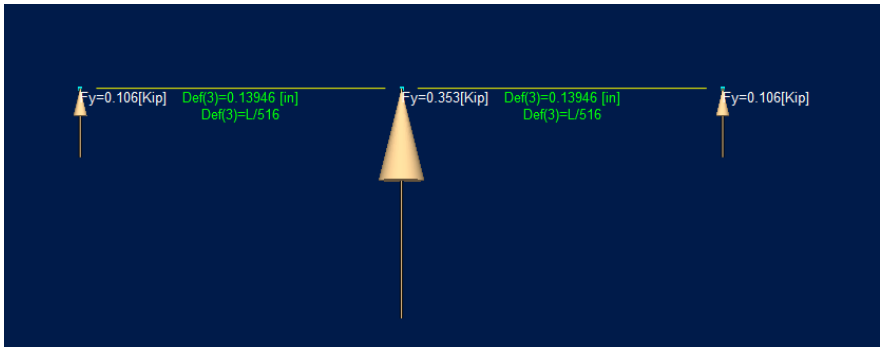
LOAD NO.	LEVEL	DESCRIPTION	TYPE	LOAD CONDITION (psf)						TRIB	
				DL	LL	RLL	SL	WL	EQ		
1		Ex Roof Load	UNIFORM	12		20	32	7.9		1	ft.
			TOTAL	12.0	0.0	20.0	32.0	7.9	0.0	PLF	



DECKING PROPERTIES



EXAMPLE LOAD CASE - DEAD LOAD



EXAMPLE REACTIONS & DEFLECTIONS - DEAD + SNOW

RAM Elements **Matteo Ferran**

Current Date: 6/25/2025 3:45 PM
Units system: English
File name: P:\23-086.1 NPS GATE Rehabilitate Hangars 3 & 4 at Floyd Bennett Field PD-CD\6_Calcs\RAM files\250625 Lean-To Framing_GATE Hangars.retx

Wood Design
Concise Report

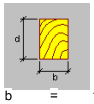
ANSI/AWC NDS-2018 ASD

Member : 6 (LT-B5 (Decking)) - OK

Section information

Section name: 1.5" X 12" **(US)**

Dimensions



b	=	1.500	[in]	Width
d	=	12.000	[in]	Height

Properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	18.000	
Moment of Inertia (principal axes) (I')	[in4]	216.000	3.375
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	36.000	4.500

Material : SPineFir-S_No1

Properties	Value
Type:	Lumber
Species:	Spruce-Pine-Fir (South)
Grade:	No.1
Coefficient of variation:	0.25

Design Criteria

Description	Unit	Value
Repetitive member:	--	No
End notches at top:	--	Top

Description	Unit	Major axis	Minor axis
Physical length	[ft]	6.00	

Design Checks

Bending capacity about y-y axis ✓

Ratio	:	0.45		
Capacity	:	0.94 [Kip/in2]	Reference	: Sec. 3.3
Demand	:	0.42 [Kip/in2]	Ctrl Eq.	: D15 at 6.00%

Intermediate results	Unit	Value	Reference
Bending capacity about y-y axis:	[Kip/in2]	0.85	Sec. 3.3

Intermediate results	Unit	Value	Reference
Bending capacity about y-y axis:	[Kip/in2]	0.94	Sec. 3.3

Intermediate results	Unit	Value	Reference
Bending capacity about y-y axis:	[Kip/in2]	1.09	Sec. 3.3

Intermediate results	Unit	Value	Reference
Bending capacity about y-y axis:	[Kip/in2]	0.94	Sec. 3.3

Intermediate results	Unit	Value	Reference
Bending capacity about y-y axis:	[Kip/in2]	1.09	Sec. 3.3

Intermediate results	Unit	Value	Reference
Bending capacity about y-y axis:	[Kip/in2]	1.51	Sec. 3.3

Intermediate results	Unit	Value	Reference
Bending capacity about y-y axis:	[Kip/in2]	1.51	Sec. 3.3

Intermediate results	Unit	Value	Reference
Bending capacity about y-y axis:	[Kip/in2]	1.51	Sec. 3.3

Intermediate results	Unit	Value	Reference
Bending capacity about y-y axis:	[Kip/in2]	1.09	Sec. 3.3

Typical New Decking
Wood Design Output

ANCHORAGE OF ROOF PECKING
FOR NET UPLIFT DUE TO WIND

MAX. ZONE 2 ASCE 7-22 LRFD WIND LOAD = 75 psf

15 psf ± DEAD LOAD

NET ASD UPLIFT = (0.6)(75) − (0.6)(15) = 36 psf

6' ± PURLIN SPACING ⇒ (6)(36) = 216 lb/ft ASD NET UPLIFT

16d PLAIN SHANK NAIL IN ASSUMED HEM-FIR (G=0.43) FRAMING:
W = 27 lb/in

p = 3.5 − 1.5 = 2"

W' = (1.6)(2)(27) = 86 lb/NAIL (FACE NAIL)
W' = (0.83)(86) = 71 lb/NAIL (TOE-NAIL)

(2) FACE NAILS + (1) TOE-NAIL
⇒ TOTAL W' = (2)(86) + 71 = 243 lb/ BOARD

8" WIDTH ⇒ (12/8)(243) = 365 lb/ft OK

HANGAR ROOF DIAPHRAGM

$R = 1.5$ URM SHEAR WALLS
$S_{ds} = 0.170$ (ASCE 7 HAZARD TOOL)
$I = 1.0$ (CURRENT CONDITION)
$I = 1.25$ (LIKELY FUTURE CONDITION – RISK CATEGORY III)

$C_s = \frac{(0.170)(1.0)}{1.5} = 0.113$ EXISTING

$C_s = \frac{(0.170)(1.25)}{1.5} = 0.142$ FUTURE

$0.2 S_{ds} I w_p$ MIN. DIAPHRAGM FORCE DOESN'T CONTROL BY INSPECTION

ASSUME 20 psf D ROOF × 120'
ASSUME 40 psf D EXT. WALL (2) SIDES × 33'-4" ± TALL.
ASSUME SIMPLE SPAN OF WALL, NEGLECTING ADD'L TRIB HT DUE TO PARAPET. 40 psf USED TO ACCOUNT FOR OPNGS (50% SOLID 8" BRICK WALL @ 80 psf, FOR EXAMPLE)

$\Rightarrow w_p = (20)(120) + (2)(40)(33.3/2) = 3730 \text{ lb/ft}$

$F_p = \begin{cases} (0.113)(3.73) = 0.421 \text{ k/ft} & \text{EXISTING} \\ (0.142)(3.73) = 0.530 \text{ k/ft} & \text{FUTURE} \end{cases}$

$V = \begin{cases} (0.421)(140)/2 = 29.5 \text{ k} & \text{EXISTING} \quad (1.0E) \text{ LRFD} \\ (0.530)(140)/2 = 37.1 \text{ k} & \text{FUTURE} \end{cases}$

$V = \begin{cases} (0.7)(29.5) = 20.7 \text{ k} & \text{EXISTING} \quad (0.7E) \text{ ASD} \\ (0.7)(37.1) = 26.0 \text{ k} & \text{FUTURE} \end{cases}$

ASSUME SIMPLE SPAN OF DIAPHRAGM

$v = \begin{cases} (20.7)(1000)/120 = \underline{173\ ^{lb}/_{ft}} & \text{EXISTING (0.7E) ASD} \\ (26.0)(1000)/120 = \underline{217\ ^{lb}/_{ft}} & \text{FUTURE} \end{cases}$

STRAIGHT-SHEATHED 2X
LUMBER DIAPHRAGM
$v_{ALL} = 140/2.8 = \underline{50\ ^{lb}/_{ft}}$ (SDPWS TABLE 4.2D) SEISMIC

⇒ SEISMIC DEMAND ≈ 4x CAPACITY

⇒ INSTALL PLYWOOD SHEATHING OVERLAY

$\frac{15}{32}$" STRUCT I PLYWOOD SHEATHING w/ 10d NAILS
@ 4" o.c. @ PANEL EDGES & DIAPHRAGM BOUNDARIES
(BLOCKED w/ EFFECTIVE FRAMING WIDTH ≥ 3x)

$v_{ALL} = (1345/2.8)[1-(0.5-0.43)] = (480)(0.93) = \underline{446\ ^{lb}/_{ft}}$ (INCL. REDUCTION FOR POSSIBLE HEM-FIR SPECIES DECKING)

CONSIDER 20 psf ASD WIND (0.6W)
IN PERPENDICULAR DIRECTION:

$w = (33.3/2)(20) = 333\ ^{lb}/_{ft}$
$V = (0.333)(120)/2 = 20.0\ k$
$v = (20.0)(1000)/140 = \underline{143\ ^{lb}/_{ft}}$

$v_{ALL} = (1345/2.0)(0.93) = (673)(0.93) = \underline{626\ ^{lb}/_{ft}}$ OK

PER ATTACHED ICC REPORT DIAPHRAGM
ALLOWABLE SHEAR w/ 1 $\frac{3}{4}$" LONG SIMPSON
WSV (#9) SCREWS EQUIVALENT TO 10d NAILS

LEAN-TO ROOF DIAPHRAGM

TRIB = 15/2 = 7.5 ft 12" BRICK WALL @ 60 psf
(ASSUME 50% SOLID)
TRIB = 2.5 ft 8" BRICK PARAPET @ 80 psf

ASSUME 20 psf D ROOF × 30'

WORST CASE, ASSUME SHARED WALL
W/ HANGAR BRACED BY LEAN-TO
ROOF DIAPHRAGM:

ASSUME 50% SOLID 8" BRICK @ 40 pcf
TRIB = 15/2 + (33.3 − 15)/2 = 16.7 ft

$w_p = (7.5)(60) + (2.5)(80) + (30)(20) + (16.7)(40) = 1920 \frac{lb}{ft}$

$F_p = (0.142)(1.92) = 0.273 \frac{k}{ft}$ (FUTURE)

$V = (0.273)(140)/2 = 19.1\ k$
$v = (19.1)(1000)/30 = 637 \frac{lb}{ft}$ } LRFD (1.0E)

$v = (0.7)(637) = \underline{\underline{446 \frac{lb}{ft}}}$ ASD (0.7E) (FUTURE) = v_{ALL} $\underline{\underline{OK}}$

CONSIDER 20 psf ASD WIND (0.6W):

$w = [7.5 + 2.5 + (33.3 - 15)/2](20) = \underline{\underline{383 \frac{lb}{ft}}}$ DOESN'T CONTROL

⇒ SAME DIAPHRAGM OVERLAY OK

OUT-OF-PLANE BRACING OF
REBUILT LEAN-TO EXT. WALLS

DIAPHRAGM SPAN > 100 ft ⇒ $k_a = 2.0$

$F_p = 0.4 S_{DS} k_a I W_p \geq 0.2 k_a I W_p$
$= (0.2)(2.0)(1.0) W_p = 0.40 W_p$
$W_p = (7.5)(60) + (2.5)(80) = 650 \frac{lb}{ft}$ (SEE P. 3)

ANCHORS @ 4'-0" o.c. (MAX. PER ASCE 7 UNLESS WALL DESIGNED TO SPAN BETWEEN ANCHORS)
⇒ $W_p = (4)(650) = 2600$ lb
$F_p = \begin{cases} (0.40)(2600) = 1040 \text{ lb/ANCHOR LRFD } (1.0E) \\ (0.7)(1040) = \underline{728 \text{ lb/ANCHOR}} \text{ ASD } (0.7E) \end{cases}$

PER CALCS ON P. 4A + 4B: 1240 lb/ANCHOR ASD WIND (0.6W)
CONTROLS

PER ATTACHED ICC REPORT
3/4" ϕ THREADED ROD @ 22.5° w/
HILTI-HY 270 ADHESIVE
T_{ALL} = 1200 lb OK (SAY 3% OVERLOAD FOR WIND OK)

SIMPSON LTTP2 w/ 3/4" ϕ ANCHOR ROD
ϕ (12) #9 x 1 1/2" SIMPSON SD SCREWS
T_{ALL} = 2045 lb FOR SPF/HF OK

CONNXN OF BLKG TO 2x HF DECKING:
1240/6 = 207 lb/ft
16d NAIL Z' = (1.6)(122) = 195 lb
(195/207)(12) = 11.3" MAX. SPACING ⇒ 10" SPACING OK

LEAN-TO PARAPET WIND LOAD

EXPOSURE D
MEAN ROOF HEIGHT $h = 15' \pm$
PARAPET HEIGHT $= 2'-6'' \pm$
$\Rightarrow K_h = (1.03 + 1.08)/2 = 1.06$
$q_p = (0.00256)(1.06)(1.0)(1.0)(118)^2 = 37.8$ psf
$K_d = 0.85$
$GC_{pi} = \phi$

SAY EFFECTIVE WIND AREA $= \phi$ (CONSERVATIVE) (ANCHORAGE TRIB AREA $= (4)(2.5) = 10$ ft^2)

$$GC_p = \begin{cases} +1.0 & \text{ZONES 4 \& 5 WALLS} \\ -1.2 & \text{ZONE 4} \\ -1.4 & \text{ZONE 5} \end{cases}$$

$$GC_p = \begin{cases} -2.3 & \text{ZONE 2} \\ -3.2 & \text{ZONE 3} \end{cases}$$

LOAD CASE A (WINDWARD PARAPET):

$p = (37.8)(0.85)(1.0) = 32.1$ psf WINDWARD SURFACE
$p = (37.8)(0.85)(-3.2) = -103$ psf LEEWARD SURFACE
NET TOTAL $= 32 + 103 = 135$ psf

LOAD CASE B (LEEWARD PARAPET):

$p = (37.8)(0.85)(1.0) = 32.1$ psf
$p = (37.8)(0.85)(-1.4) = -45.0$ psf
NET TOTAL $= 32 + 45 = 77$ psf

LEAN-TO WALL WIND LOAD

$h = 15' \pm \Rightarrow K_h = 1.03$

$q_p = (0.00256)(1.03)(1.0)(1.0)(118)^2 = 36.7$ psf

$GC_{pi} = \pm 0.18$

ANCHORAGE TRIB AREA $= (4)(\frac{15}{2}) = 30$ ft^2

$$GC_p = \begin{cases} +0.9 & \text{ZONES 4 \& 5} \\ -1.0 & \text{ZONE 4} \\ -1.2 & \text{ZONE 5} \end{cases}$$

$p = (36.7)(0.85)(0.9 + 0.18) = 33.7$ psf MAX. PRESSURE

$p = (36.7)(0.85)(-1.2 - 0.18) = -43.0$ psf MAX. SUCTION

ANCHORAGE

ANCHORS @ 4'-0" o.c.

$\Rightarrow T_u = (4)(2.5)(77) + (4)(15/2)(43) = 2060$ lb (1.0W)

$T = (0.6)(2060) = \underline{1240 \text{ lb ASD}}$ (0.6W)

CONNXN OF $\frac{15}{32}$" PLYWOOD OVERLAY TO 2x HF DECKING:

#9 SCREW

$Z' = (1.6)(70) = 112$ lb PER NDS FOR $\frac{1}{2}$" THK HF SIDE MEMBER FOR 10D PENETRATION

$p = 1.75 - \frac{15}{32} = 1.28$"

$1.28/0.177 = 7.23$

$\Rightarrow Z' = (7.23/10)(112) = \underline{81.0 \text{ lb}}$

$(81/207)(12) = 4.69$" ~~MAX.~~ SPACING

BUT ALLOWABLE DIAPHRAGM SHEAR FOR 6" SPACING OF SIMPSON WSV SCREWS = 320 lb/ft MIN. FOR 2x FRAMING (x 0.93 FOR HF = 298 lb/ft) FOR $\frac{15}{32}$" STRUCT I SHEATHING $\Rightarrow$ 6" SPACING OK

IN-PLANE SHEAR TRANSFER FROM DIAPHRAGM TO URM WALL

$\frac{3}{4}$"ϕ BOLT IN 2x HF LEDGER $Z' = (1.6)(1200) = 1920$ lb

ADHESIVE ANCHOR $V_{all} = 1000$ lb PER ICC REPORT CONTROLS

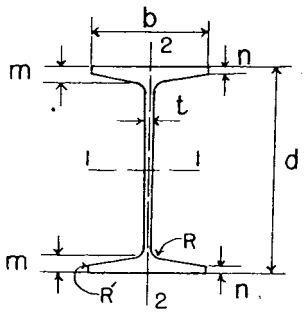
4' SPACING $\Rightarrow V_{all} = 1000/4 = \underline{250 \text{ lb/ft}}$ ASSUME OK FOR SHEAR IN LONG DIRECTION OF DIAPHRAGM

16d NAIL FROM LEDGER TO WOOD FILLER/PLATE ON TOP OF WALL:

$Z' = 195$ lb

6" SPACING $\Rightarrow V_{all} = 195/0.5 = 390 \text{ lb/ft} > 250 \text{ lb/ft}$

SECTION PROPERTIES FOR EXISTING STEEL FRAMING MEMBERS



I-shaped Members:

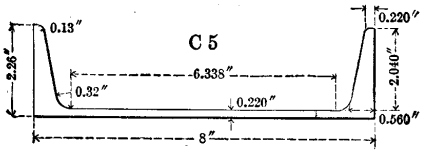
- Based on AISC's "Iron and Steel Beams 1873 to 1952."
- Generally, sections identified in original 1929 drawings appeared to match observed members where accessible or where able to be scaled visually. The exception was the Lean-to roofs, which had 21" and 16" deep girders instead of the 18" and 12" deep girders indicated on the original framing plan.

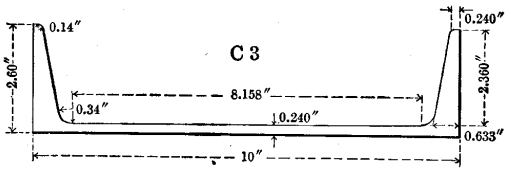
ID ON DWGS	SECT. NO. OR NOM. SIZE	COL. (1)	WEIGHT PER FOOT	AREA	DEPTH d	FLANGE WIDTH b	WEB THICK t	DIMENSIONS m	n	R	R'	SLOPE INSIDE FLANGE	AXIS 1—1 I	S	r	AXIS 2—2 I	S	r
			Lb.	Sq.In.	In.	In.	In.	In.	In.	In.	In.	%	In.⁴	In.³	In.	In.⁴	In.³	In.
14" CB 86#	CB146 14X15	8	86.0	25.28	13.714	15.008	.414	.662	.662	.65	0	0	923.0	134.6	6.04	373.1	49.7	3.84
14" CB 68#	CB144 14X10	7	68.0	19.99	14.238	10.043	.425	.714	.714	.55	0	0	738.8	103.8	6.08	120.6	24.0	2.46
12" CB 36#	CB122 12X6½	17	36.0	10.59	12.236	6.568	.308	.538	.538	.35	0	0	280.1	45.8	5.14	25.4	7.7	1.55
10" CB 36#	CB102	15	36.0	10.58	10.000	8.147	.467	.381	.381	.30	0	0	175.6	35.1	4.07	34.4	8.5	1.80
10" CB 31#	CB102	15	31.0	9.11	10.000	8.000	.320	.381	.381	.30	0	0	163.4	32.7	4.23	32.5	8.1	1.89
10" CB 21#	CB101	15	21.0	6.17	9.902	6.000	.230	.332	.332	.30	0	0	107.6	21.7	4.18	12.0	4.0	1.39
10" I 27.9#	B66,	7	27.9	8.15	12.0	6.000	.284	.540	.280	.26	–	9.1 *	199.4	33.2	4.95	12.6	4.2	1.24
4" I 7.7#	5,6,8,10,11,13, 14,17,26,28, 29,30		7.7	2.21	4.0	2.660	.190	.396	.19	.29	.11	16⅔	6.0	3.0	1.64	.77	.58	.59
21" CB 55#	CB211 21X8	9	55.0	16.17	20.890	8.000	.360	.553	.553	.55	0	0	1166.7	111.7	8.49	47.3	11.8	1.71
16" B 45#	B16 16X7¼	3	45.0	13.26	16.120	7.285	.330	.708	.418	.40	0	8⅓ *	594.5	73.8	6.69	31.9	8.75	1.55

Channels:

- Based on "Pocket Companion for Engineers, Architects and Builders" by Carnegie Steel Company, 1923 Edition.

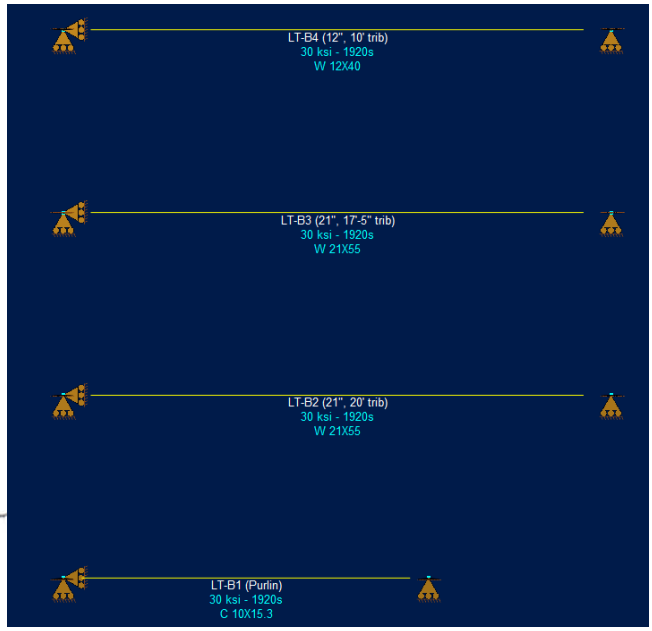
Section Index	Depth of Channel, Inches	Weight per Foot, Pounds	Flange Width, Inches Decimal	Flange Width, Inches Fractional	Web Thickness, Inches Decimal	Web Thickness, Inches Fractional
C 5	8	21.25	2.619	2⅝	0.579	37/64
		18.75	2.527	2 17/32	0.487	31/64
		16.25	2.435	2 7/16	0.395	25/64
		13.75	2.343	2 11/32	0.303	19/64
		11.5	2.260	2 17/64	0.220	7/32
C 3	10	35.0	3.180	3 3/16	0.820	13/16
		30.0	3.033	3 1/32	0.673	43/64
		25.0	2.886	2 57/64	0.526	17/32
		20.0	2.739	2 47/64	0.379	⅜
		15.3	2.600	2 19/32	0.240	15/64

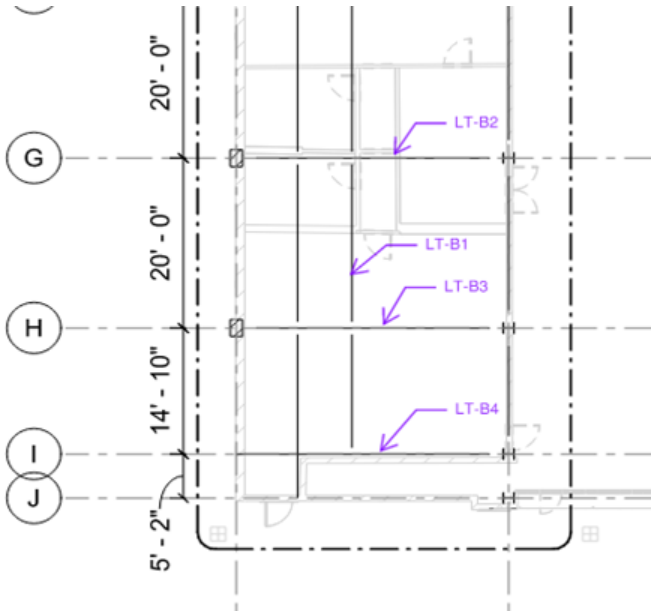


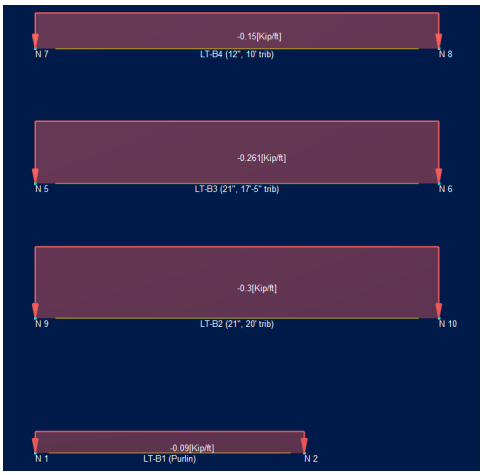


LEAN-TO FRAMING DESIGN CHECKS

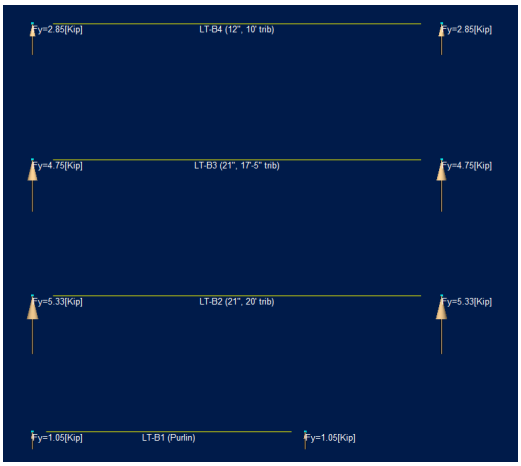
- Beams were checked for 15 psf DL, 20 psf RLL, 32 psf SL and -24 psf WL using RAM Elements software.







EXAMPLE LOAD CASE - DEAD LOAD



EXAMPLE REACTIONS - DEAD LOAD

RAM Elements **Matteo Ferran**

Current Date: 6/25/2025 3:25 PM
Units system: English
File name: P:\23-086.1 NPS GATE Rehabilitate Hangars 3 & 4 at Floyd Bennett Field PD-CD\6_Calcs\RAM files\250625 Lean-To Framing_GATE Hangars.retx

Steel Code Check
Concise Report

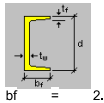
AISC 360-2016 LRFD (Hot-rolled)

Member : 1 (LT-B1 (Purlin)) - OK

Section information

Section name: C 10X15.3 **(US)**

Dimensions



bf	=	2.600	[in]	Width
d	=	10.000	[in]	Depth
k	=	1.000	[in]	Distance k
tf	=	0.436	[in]	Flange thickness
tw	=	0.240	[in]	Web thickness

Properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	4.480	
Moment of Inertia (local axes) (I)	[in4]	67.300	2.270
Moment of Inertia (principal axes) (I')	[in4]	67.300	2.270
Bending constant for moments (principal axis) (J')	[in]	0.000	5.620
Radius of gyration (local axes) (r)	[in]	3.876	0.712
Radius of gyration (principal axes) (r')	[in]	3.876	0.712
Saint-Venant torsion constant. (J)	[in4]	0.209	
Section warping constant. (Cw)	[in6]	45.500	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	-1.517	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	13.500	1.150
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	13.500	3.924
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	13.500	1.150
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	13.500	3.924
Plastic section modulus (local axis) (Z)	[in3]	15.900	2.340
Plastic section modulus (principal axis) (Z')	[in3]	15.900	2.340
Polar radius of gyration. (ro)	[in]	4.190	
Area for shear (Aw)	[in2]	2.270	2.400
Torsional constant. (C)	[in3]	0.415	

Material : 30 ksi - 1920s

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	30.00
Tensile strength (Fu):	[Kip/in2]	55.00
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11153.85

Design Criteria

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	20.00

Distance between member lateral bracing points

Length (Lb) [ft] Top	Bottom
2.00	20.00

Laterally unbraced length

Length [ft] Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Effective length factor Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
20.00	20.00	20.00	1.0	1.0	1.0

Additional assumptions

Continuous lateral torsional restraint	No
Tension field action	No
Continuous flexural torsional restraint	No
Effective length factor value type	None
Major axis frame type	Sway
Minor axis frame type	Sway

Design Checks

Axial Tension Design ✓

Axial tension

Ratio	:	0.00		
Capacity	:	120.96 [Kip]	Reference	: Cl.D2
Demand	:	0.00 [Kip]	Ctrl Eq.	: D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(ϕP_n):	[Kip]	120.96	Cl.D2

Axial Compression Design ✓

Compression in the major axis 33

Ratio	:	0.00		
Capacity	:	102.23 [Kip]	Reference	: Cl.E3
Demand	:	0.00 [Kip]	Ctrl Eq.	: D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored flexural buckling strength(ϕP_{n33}):	[Kip]	102.23	Cl.E3

Compression in the minor axis 22

Ratio	:	0.00		
Capacity	:	8.90 [Kip]	Reference	: Cl.E3
Demand	:	0.00 [Kip]	Ctrl Eq.	: D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored flexural buckling strength(ϕP_{n22}):	[Kip]	8.90	Cl.E3
Factored torsional or flexural-torsional buckling strength(ϕP_{n11}):	[Kip]	79.06	Cl.E4

Lean-To Framing RAM Elements
Design Output

Flexural Design ✔

Bending about major axis, M33

Ratio	:	0.61			
Capacity	:	35.78 [Kip*ft]	Reference	:	Cl.F2.1
Demand	:	21.68 [Kip*ft]	Ctrl Eq.	:	D5 at 50.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored yielding strength(ϕM_n):	[Kip*ft]	35.78	Cl.F2.1
Factored lateral-torsional buckling strength(ϕM_n):	[Kip*ft]	12.74	Cl.F2.2

Bending about minor axis, M22

Ratio	:	0.00			
Capacity	:	4.14 [Kip*ft]	Reference	:	Cl.F6.1
Demand	:	0.00 [Kip*ft]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored yielding strength about a geometric axis(ϕM_n):	[Kip*ft]	4.14	Cl.F6.1

Shear Design ✔

Shear in major axis 33

Ratio	:	0.00			
Capacity	:	36.73 [Kip]	Reference	:	Cl.G1
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	36.73	Cl.G1

Shear in minor axis 22

Ratio	:	0.11			
Capacity	:	38.88 [Kip]	Reference	:	Cl.G1
Demand	:	4.34 [Kip]	Ctrl Eq.	:	D5 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	38.88	Cl.G1

Combined Actions Design ✔

Combined flexure and axial

Ratio	:	0.61			
Ctrl Eq.	:	D5 at 50.00%	Reference	:	Eq.H1-1b

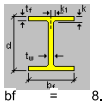
Intermediate results	Unit	Value	Reference
Interaction of flexure and axial force:	--	0.61	Eq.H1-1b

Member : 3 (LT-B3 (21", 17'-5" trib)) - OK

Section information

Section name: W 21X55 **(US)**

Dimensions



bf	=	8.220	[in]	Width
d	=	20.800	[in]	Depth
k	=	1.020	[in]	Distance k
k1	=	0.813	[in]	Distance k1
tf	=	0.522	[in]	Flange thickness
tw	=	0.375	[in]	Web thickness

Properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	16.200	
Moment of Inertia (local axes) (I)	[in4]	1140.000	48.400
Moment of Inertia (principal axes) (I')	[in4]	1140.000	48.400
Bending constant for moments (principal axis) (J')	[in]	0.000	0.000
Radius of gyration (local axes) (r)	[in]	8.389	1.728
Radius of gyration (principal axes) (r')	[in]	8.389	1.728
Saint-Venant torsion constant. (J)	[in4]	1.240	
Section warping constant. (Cw)	[in6]	4980.000	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	0.000	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	110.000	11.800
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	110.000	11.800
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	110.000	11.800
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	110.000	11.800
Plastic section modulus (local axis) (Z)	[in3]	126.000	18.400
Plastic section modulus (principal axis) (Z')	[in3]	126.000	18.400
Polar radius of gyration. (ro)	[in]	8.565	
Area for shear (Aw)	[in2]	8.580	7.800
Torsional constant. (C)	[in3]	2.176	

Material : 30 ksi - 1920s

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	30.00
Tensile strength (Fu):	[Kip/in2]	55.00
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11153.85

Design Criteria

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	30.00

Distance between member lateral bracing points

Length (Lb) [ft] Top	Bottom
6.00	30.00

Laterally unbraced length

Length [ft]			Effective length factor		
Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
30.00	30.00	30.00	1.0	1.0	1.0

Additional assumptions

Continuous lateral torsional restraint	No
Tension field action	No
Continuous flexural torsional restraint	No
Effective length factor value type	None
Major axis frame type	Sway
Minor axis frame type	Sway

Design Checks

Axial Tension Design ✔

Axial tension

Ratio	: 0.00			
Capacity	: 437.40 [Kip]	Reference	:	Cl.D2
Demand	: 0.00 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(ϕP_n):	[Kip]	437.40	Cl.D2

Axial Compression Design ✔

Compression in the major axis 33

Ratio	: 0.00			
Capacity	: 399.04 [Kip]	Reference	:	Cl.E3
Demand	: 0.00 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored flexural buckling strength(ϕP_{n33}):	[Kip]	399.04	Cl.E3

Compression in the minor axis 22

Ratio	: 0.00			
Capacity	: 84.37 [Kip]	Reference	:	Cl.E3
Demand	: 0.00 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored flexural buckling strength(ϕP_{n22}):	[Kip]	84.37	Cl.E3
Factored torsional or flexural-torsional buckling strength(ϕP_{n11}):	[Kip]	239.81	Cl.E4

Flexural Design ✔

Bending about major axis, M33

Ratio	: 0.50			
Capacity	: 283.50 [Kip*ft]	Reference	:	Cl.F2.1
Demand	: 143.04 [Kip*ft]	Ctrl Eq.	:	D5 at 50.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored yielding strength(ϕM_n):	[Kip*ft]	283.50	Cl.F2.1
Factored lateral-torsional buckling strength(ϕM_n):	[Kip*ft]	182.25	Cl.F2.2

Bending about minor axis, M22

Ratio	: 0.00			
Capacity	: 41.40 [Kip*ft]	Reference	:	Cl.F6.1
Demand	: 0.00 [Kip*ft]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored yielding strength about a geometric axis(ϕM_n):	[Kip*ft]	41.40	Cl.F6.1

Shear Design ✔

Shear in major axis 33

Ratio	: 0.00			
Capacity	: 139.02 [Kip]	Reference	:	Cl.G1
Demand	: 0.00 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	139.02	Cl.G1

Shear in minor axis 22

Ratio	: 0.14			
Capacity	: 140.40 [Kip]	Reference	:	Cl.G1
Demand	: 19.07 [Kip]	Ctrl Eq.	:	D5 at 100.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	140.40	Cl.G1

Combined Actions Design ✔

Combined flexure and axial

Ratio	: 0.50			
Ctrl Eq.	: D5 at 50.00%	Reference	:	Eq.H1-1b

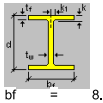
Intermediate results	Unit	Value	Reference
Interaction of flexure and axial force:	–	0.50	Eq.H1-1b

Member : 4 (LT-B4 (12", 10' trib)) - OK

Section information

Section name: W 12X40 **(US)**

Dimensions



bf	=	8.010	[in]	Width
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k	=	1.020	[in]	Distance k
k1	=	0.875	[in]	Distance k1
tf	=	0.515	[in]	Flange thickness
tw	=	0.295	[in]	Web thickness

Properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	11.700	
Moment of Inertia (local axes) (I)	[in4]	307.000	44.100
Moment of Inertia (principal axes) (I')	[in4]	307.000	44.100
Bending constant for moments (principal axis) (J')	[in]	0.000	0.000
Radius of gyration (local axes) (r)	[in]	5.122	1.941
Radius of gyration (principal axes) (r')	[in]	5.122	1.941
Saint-Venant torsion constant. (J)	[in4]	0.906	
Section warping constant. (Cw)	[in6]	1440.000	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	0.000	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	51.500	11.000
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	51.500	11.000
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	51.500	11.000
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	51.500	11.000
Plastic section modulus (local axis) (Z)	[in3]	57.000	16.800
Plastic section modulus (principal axis) (Z')	[in3]	57.000	16.800
Polar radius of gyration. (ro)	[in]	5.478	
Area for shear (Aw)	[in2]	8.250	3.510
Torsional constant. (C)	[in3]	1.605	

Material : 30 ksi - 1920s

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	30.00
Tensile strength (Fu):	[Kip/in2]	55.00
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11153.85

Design Criteria

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	30.00

Distance between member lateral bracing points

Length (Lb) [ft] Top	Bottom
6.00	30.00

Laterally unbraced length

Length [ft] Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Effective length factor Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
30.00	30.00	30.00	1.0	1.0	1.0

Additional assumptions

Continuous lateral torsional restraint	No
Tension field action	No
Continuous flexural torsional restraint	No
Effective length factor value type	None
Major axis frame type	Sway
Minor axis frame type	Sway

Design Checks

Axial Tension Design ✔

Axial tension

Ratio	:	0.00			
Capacity	:	315.90 [Kip]	Reference	:	Cl.D2
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(ϕP_n):	[Kip]	315.90	Cl.D2

Axial Compression Design ✔

Compression in the major axis 33

Ratio	:	0.00			
Capacity	:	254.36 [Kip]	Reference	:	Cl.E3
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored flexural buckling strength(ϕP_{n33}):	[Kip]	254.36	Cl.E3

Compression in the minor axis 22

Ratio	:	0.00			
Capacity	:	76.87 [Kip]	Reference	:	Cl.E3
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored flexural buckling strength(ϕP_{n22}):	[Kip]	76.87	Cl.E3
Factored torsional or flexural-torsional buckling strength(ϕP_{n11}):	[Kip]	226.69	Cl.E4

Flexural Design ✔

Bending about major axis, M33

Ratio	:	0.65			
Capacity	:	128.25 [Kip*ft]	Reference	:	Cl.F2.1
Demand	:	83.23 [Kip*ft]	Ctrl Eq.	:	D5 at 50.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored yielding strength(ϕM_n):	[Kip*ft]	128.25	Cl.F2.1
Factored lateral-torsional buckling strength(ϕM_n):	[Kip*ft]	120.00	Cl.F2.2

Bending about minor axis, M22

Ratio	:	0.00			
Capacity	:	37.80 [Kip*ft]	Reference	:	Cl.F6.1
Demand	:	0.00 [Kip*ft]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored yielding strength about a geometric axis(ϕM_n):	[Kip*ft]	37.80	Cl.F6.1

Shear Design ✔

Shear in major axis 33

Ratio	:	0.00		
Capacity	:	133.65 [Kip]	Reference	: Cl.G1
Demand	:	0.00 [Kip]	Ctrl Eq.	: D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	133.65	Cl.G1

Shear in minor axis 22

Ratio	:	0.18		
Capacity	:	63.18 [Kip]	Reference	: Cl.G1
Demand	:	11.10 [Kip]	Ctrl Eq.	: D5 at 100.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	63.18	Cl.G1

Combined Actions Design ✔

Combined flexure and axial

Ratio	:	0.65		
Ctrl Eq.	:	D5 at 50.00%	Reference	: Eq.H1-1b

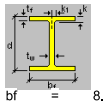
Intermediate results	Unit	Value	Reference
Interaction of flexure and axial force:	--	0.65	Eq.H1-1b

Member : 5 (LT-B2 (21", 20' trib)) - OK

Section information

Section name: W 21X55 **(US)**

Dimensions



bf	=	8.220	[in]	Width
d	=	20.800	[in]	Depth
k	=	1.020	[in]	Distance k
k1	=	0.813	[in]	Distance k1
tf	=	0.522	[in]	Flange thickness
tw	=	0.375	[in]	Web thickness

Properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	16.200	
Moment of Inertia (local axes) (I)	[in4]	1140.000	48.400
Moment of Inertia (principal axes) (I')	[in4]	1140.000	48.400
Bending constant for moments (principal axis) (J')	[in]	0.000	0.000
Radius of gyration (local axes) (r)	[in]	8.389	1.728
Radius of gyration (principal axes) (r')	[in]	8.389	1.728
Saint-Venant torsion constant. (J)	[in4]	1.240	
Section warping constant. (Cw)	[in6]	4980.000	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	0.000	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	110.000	11.800
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	110.000	11.800
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	110.000	11.800
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	110.000	11.800
Plastic section modulus (local axis) (Z)	[in3]	126.000	18.400
Plastic section modulus (principal axis) (Z')	[in3]	126.000	18.400
Polar radius of gyration. (ro)	[in]	8.565	
Area for shear (Aw)	[in2]	8.580	7.800
Torsional constant. (C)	[in3]	2.176	

Material : 30 ksi - 1920s

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	30.00
Tensile strength (Fu):	[Kip/in2]	55.00
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11153.85

Design Criteria

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	30.00

Distance between member lateral bracing points

Length (Lb) [ft] Top	Bottom
6.00	30.00

Laterally unbraced length

Length [ft] Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Effective length factor Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
30.00	30.00	30.00	1.0	1.0	1.0

Additional assumptions

Continuous lateral torsional restraint	No
Tension field action	No
Continuous flexural torsional restraint	No
Effective length factor value type	None
Major axis frame type	Sway
Minor axis frame type	Sway

Design Checks

Axial Tension Design ✔

Axial tension

Ratio	:	0.00		
Capacity	:	437.40 [Kip]	Reference	: Cl.D2
Demand	:	0.00 [Kip]	Ctrl Eq.	: D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(ϕP_n):	[Kip]	437.40	Cl.D2

Axial Compression Design ✔

Compression in the major axis 33

Ratio	:	0.00		
Capacity	:	399.04 [Kip]	Reference	: Cl.E3
Demand	:	0.00 [Kip]	Ctrl Eq.	: D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored flexural buckling strength(ϕP_{n33}):	[Kip]	399.04	Cl.E3

Compression in the minor axis 22

Ratio	:	0.00		
Capacity	:	84.37 [Kip]	Reference	: Cl.E3
Demand	:	0.00 [Kip]	Ctrl Eq.	: D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored flexural buckling strength(ϕP_{n22}):	[Kip]	84.37	Cl.E3
Factored torsional or flexural-torsional buckling strength(ϕP_{n11}):	[Kip]	239.81	Cl.E4

Flexural Design ✔

Bending about major axis, M33

Ratio	:	0.58		
Capacity	:	283.50 [Kip*ft]	Reference	: Cl.F2.1
Demand	:	163.15 [Kip*ft]	Ctrl Eq.	: D5 at 50.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored yielding strength(ϕM_n):	[Kip*ft]	283.50	Cl.F2.1
Factored lateral-torsional buckling strength(ϕM_n):	[Kip*ft]	185.02	Cl.F2.2

Bending about minor axis, M22

Ratio	:	0.00		
Capacity	:	41.40 [Kip*ft]	Reference	: Cl.F6.1
Demand	:	0.00 [Kip*ft]	Ctrl Eq.	: D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored yielding strength about a geometric axis(ϕM_n):	[Kip*ft]	41.40	Cl.F6.1

Shear Design ✔

Shear in major axis 33

Ratio	:	0.00		
Capacity	:	139.02 [Kip]	Reference	: Cl.G1
Demand	:	0.00 [Kip]	Ctrl Eq.	: D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	139.02	Cl.G1

Shear in minor axis 22

Ratio	:	0.15		
Capacity	:	140.40 [Kip]	Reference	: Cl.G1
Demand	:	21.75 [Kip]	Ctrl Eq.	: D5 at 100.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	140.40	Cl.G1

Combined Actions Design ✔

Combined flexure and axial

Ratio	:	0.58		
Ctrl Eq.	:	D5 at 50.00%	Reference	: Eq.H1-1b

Intermediate results	Unit	Value	Reference
Interaction of flexure and axial force:	–	0.58	Eq.H1-1b

RAM Elements **Matteo Ferran**

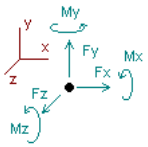
Lean-To Framing RAM Elements
Node Reactions

Current Date: 6/25/2025 3:23 PM
Units system: English
File name: P:\23-086.1 NPS GATE Rehabilitate Hangars 3 & 4 at Floyd Bennett Field PD-CD\6_Calcs\RAM files\250625 Lean-To Framing_GATE Hangars.retx

Analysis result

Nodes

Reactions



Direction of positive forces and moments

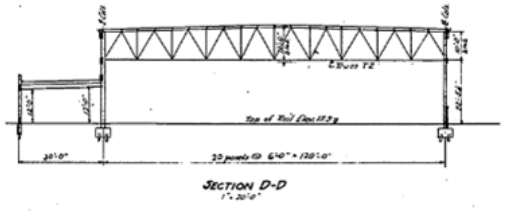
Node	Forces [Kip] FX	FY	FZ	Moments [Kip*ft] MX	MY	MZ
Condition **DL=Dead Load**						
1	0.00000	1.05268	0.00000	0.00000	0.00000	0.00000
2	0.00000	1.05268	0.00000	0.00000	0.00000	0.00000
5	0.00000	4.74764	0.00000	0.00000	0.00000	0.00000
6	0.00000	4.74764	0.00000	0.00000	0.00000	0.00000
7	0.00000	2.84810	0.00000	0.00000	0.00000	0.00000
8	0.00000	2.84810	0.00000	0.00000	0.00000	0.00000
9	0.00000	5.32814	0.00000	0.00000	0.00000	0.00000
10	0.00000	5.32814	0.00000	0.00000	0.00000	0.00000
SUM	0.00000	27.95314	0.00000	0.00000	0.00000	0.00000
Condition **LLR=Roof Live Load**						
1	0.00000	1.20000	0.00000	0.00000	0.00000	0.00000
2	0.00000	1.20000	0.00000	0.00000	0.00000	0.00000
5	0.00000	5.22450	0.00000	0.00000	0.00000	0.00000
6	0.00000	5.22450	0.00000	0.00000	0.00000	0.00000
7	0.00000	3.00000	0.00000	0.00000	0.00000	0.00000
8	0.00000	3.00000	0.00000	0.00000	0.00000	0.00000
9	0.00000	6.00000	0.00000	0.00000	0.00000	0.00000
10	0.00000	6.00000	0.00000	0.00000	0.00000	0.00000
SUM	0.00000	30.84900	0.00000	0.00000	0.00000	0.00000
Condition **SL=Snow Load**						
1	0.00000	1.92000	0.00000	0.00000	0.00000	0.00000
2	0.00000	1.92000	0.00000	0.00000	0.00000	0.00000
5	0.00000	8.35950	0.00000	0.00000	0.00000	0.00000
6	0.00000	8.35950	0.00000	0.00000	0.00000	0.00000
7	0.00000	4.80000	0.00000	0.00000	0.00000	0.00000
8	0.00000	4.80000	0.00000	0.00000	0.00000	0.00000
9	0.00000	9.60000	0.00000	0.00000	0.00000	0.00000
10	0.00000	9.60000	0.00000	0.00000	0.00000	0.00000
SUM	0.00000	49.35900	0.00000	0.00000	0.00000	0.00000
Condition **WL=Wind Load**						
1	0.00000	-0.48000	0.00000	0.00000	0.00000	0.00000
2	0.00000	-0.48000	0.00000	0.00000	0.00000	0.00000
5	0.00000	-5.48531	0.00000	0.00000	0.00000	0.00000
6	0.00000	-3.56719	0.00000	0.00000	0.00000	0.00000
7	0.00000	-3.14906	0.00000	0.00000	0.00000	0.00000
8	0.00000	-2.04844	0.00000	0.00000	0.00000	0.00000
9	0.00000	-6.29812	0.00000	0.00000	0.00000	0.00000
10	0.00000	-4.09688	0.00000	0.00000	0.00000	0.00000
SUM	0.00000	-25.60500	0.00000	0.00000	0.00000	0.00000

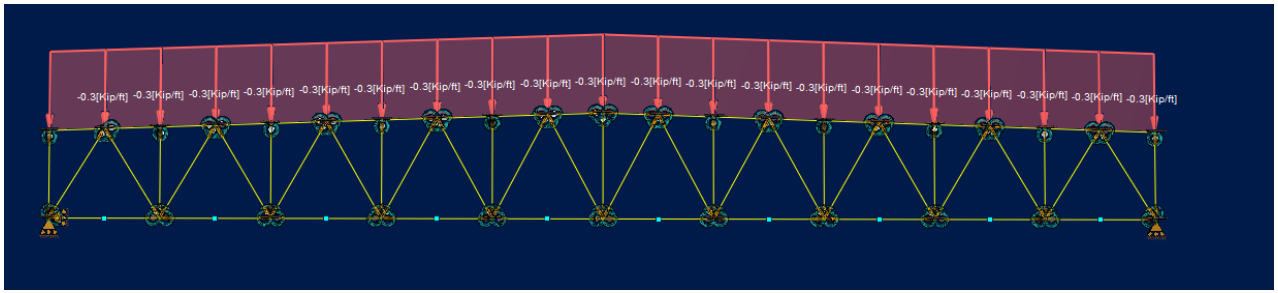
NAME: Truss T2
DESCRIPTION: Typical Truss at Hangar - Type 2

Truss T2 at Hangars Design Check

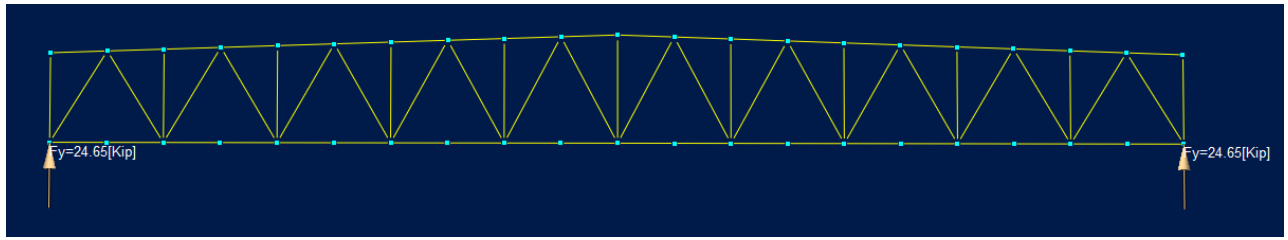
SPAN: 120 ft

LOAD NO.	LEVEL	DESCRIPTION	TYPE	LOAD CONDITION (psf)						TRIB	EXTENTS (ft)		NOTES
				DL	LL	RLL	SL	WL	EQ		x	y	
1		Ex Roof Load	UNIFORM	15		20	32	-28.1		20 ft.	FULL LENGTH		Wind is a stepped load (see model)
			TOTAL	300.0	0.0	400.0	640.0	-562.0	0.0	PLF	0	0	

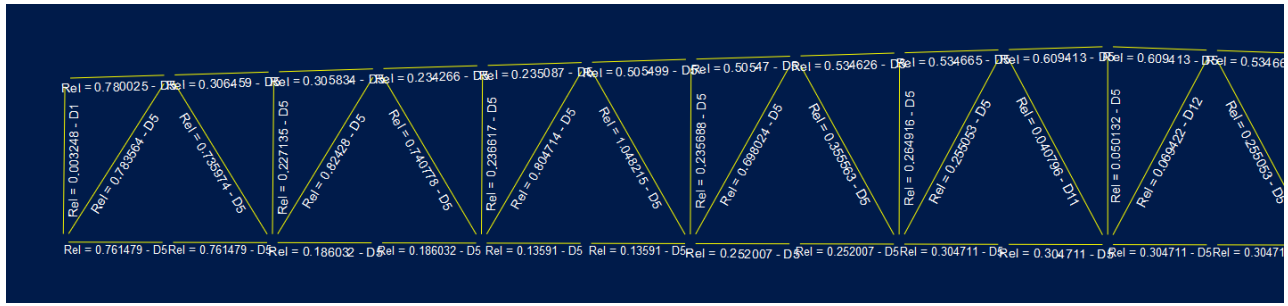




EXAMPLE LOAD CASE - DEAD LOAD



EXAMPLE REACTIONS - DEAD LOAD



DESIGN UNITY RATIO AT GOVERNING LOAD COMBINATION

RAM Elements **Matteo Ferran**

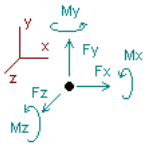
Truss T2 Node Reactions

Current Date: 6/25/2025 2:20 PM
Units system: English
File name: P:\23-086.1 NPS GATE Rehabilitate Hangars 3 & 4 at Floyd Bennett Field PD-CD\6_Calcs\RAM files\250604 Roof Truss T2_GATE Hangars.retx

Analysis result

Nodes

Reactions



Direction of positive forces and moments

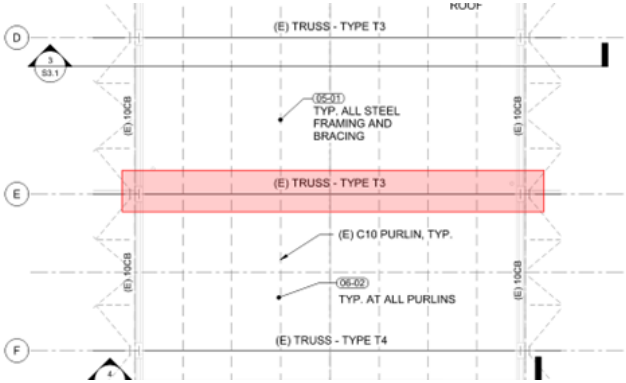
	Forces [Kip]			Moments [Kip*ft]		
Node	**FX**	**FY**	**FZ**	**MX**	**MY**	**MZ**
Condition **DL=Dead Load**						
1	0.00000	24.64895	0.00000	0.00000	0.00000	0.00000
2	0.00000	24.64895	0.00000	0.00000	0.00000	0.00000
SUM	0.00000	49.29790	0.00000	0.00000	0.00000	0.00000
Condition **LLR=Roof Live Load**						
1	0.00000	24.01333	0.00000	0.00000	0.00000	0.00000
2	0.00000	24.01333	0.00000	0.00000	0.00000	0.00000
SUM	0.00000	48.02666	0.00000	0.00000	0.00000	0.00000
Condition **SL=Snow Load**						
1	0.00000	38.42133	0.00000	0.00000	0.00000	0.00000
2	0.00000	38.42133	0.00000	0.00000	0.00000	0.00000
SUM	0.00000	76.84265	0.00000	0.00000	0.00000	0.00000
Condition **WL=Wind Load**						
1	0.00000	-29.52139	0.00000	0.00000	0.00000	0.00000
2	0.00000	-19.22567	0.00000	0.00000	0.00000	0.00000
SUM	0.00000	-48.74706	0.00000	0.00000	0.00000	0.00000

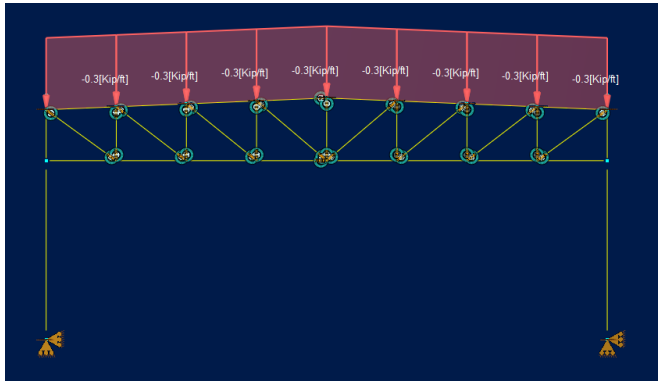
NAME: Truss T3
DESCRIPTION: Typical Truss at WPA Infill - Type 3

Truss T3 at Infill Design Check

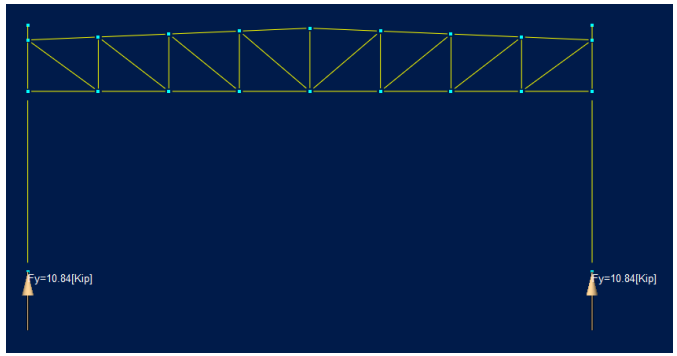
SPAN: 50 ft

LOAD NO.	LEVEL	DESCRIPTION	TYPE	LOAD CONDITION (psf)						TRIB		EXTENTS (ft)		NOTES
				DL	LL	RLL	SL	WL	EQ			x	y	
1		Ex Roof Load	UNIFORM	15		20	32	-27.7		20	ft.	FULL LENGTH		Wind is a stepped load (see model)
			TOTAL	300.0	0.0	400.0	640.0	-554.0	0.0	PLF		0	0	





EXAMPLE LOAD CASE - DEAD LOAD



EXAMPLE REACTIONS - DEAD LOAD

RAM Elements **Matteo Ferran**

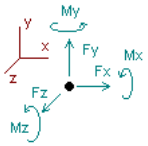
Truss T3 Node Reactions

Current Date: 6/25/2025 2:26 PM
Units system: English
File name: P:\23-086.1 NPS GATE Rehabilitate Hangars 3 & 4 at Floyd Bennett Field PD-CD\6_Calcs\RAM files\240829 Roof Truss T3_GATE Hangars.retx

Analysis result

Nodes

Reactions



Direction of positive forces and moments

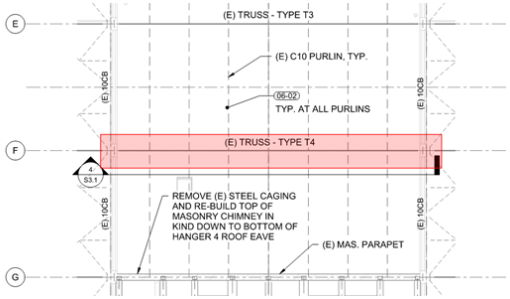
	Forces [Kip]			Moments [Kip*ft]		
Node	**FX**	**FY**	**FZ**	**MX**	**MY**	**MZ**
Condition **DL=Dead Load**						
1	0.91515	10.84284	0.00000	0.00000	0.00000	0.00000
2	-0.91515	10.84284	0.00000	0.00000	0.00000	0.00000
SUM	0.00000	21.68567	0.00000	0.00000	0.00000	0.00000
Condition **LLR=Roof Live Load**						
1	1.01450	10.00873	0.00000	0.00000	0.00000	0.00000
2	-1.01450	10.00873	0.00000	0.00000	0.00000	0.00000
SUM	0.00000	20.01747	0.00000	0.00000	0.00000	0.00000
Condition **SL=Snow Load**						
1	1.62319	16.01397	0.00000	0.00000	0.00000	0.00000
2	-1.62319	16.01397	0.00000	0.00000	0.00000	0.00000
SUM	0.00000	32.02794	0.00000	0.00000	0.00000	0.00000
Condition **WL=Wind Load**						
1	-1.04157	-12.12934	0.00000	0.00000	0.00000	0.00000
2	1.04157	-7.88812	0.00000	0.00000	0.00000	0.00000
SUM	0.00000	-20.01746	0.00000	0.00000	0.00000	0.00000

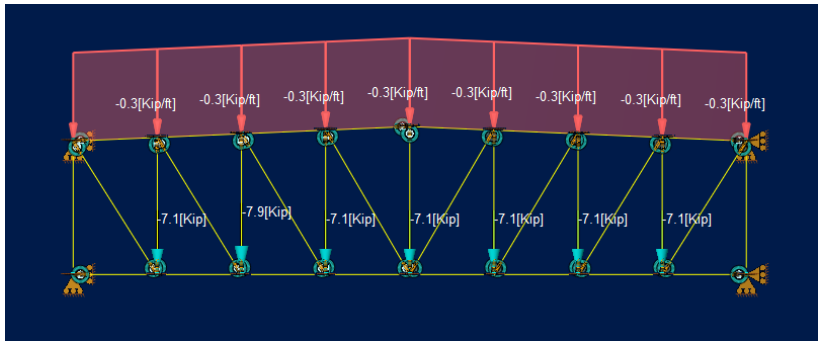
NAME: Truss T4
DESCRIPTION: Truss at WPA Infill Mezzanine - Type 4

Truss T4 at Infill Design Check

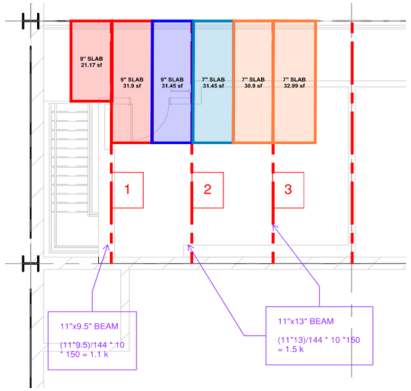
SPAN: 50 ft

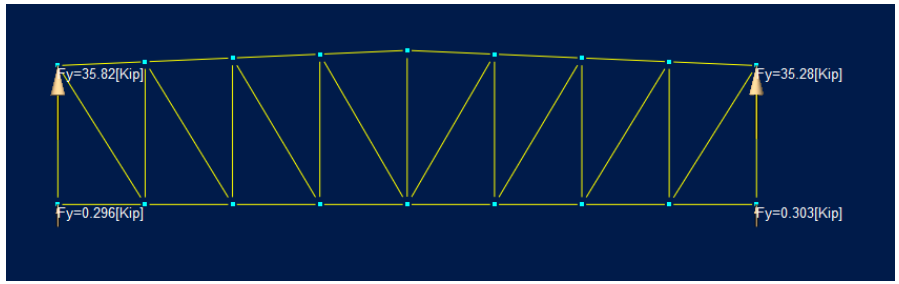
LOAD NO.	LEVEL	DESCRIPTION	TYPE	LOAD CONDITION (psf)						TRIB		EXTENTS (ft)		NOTES
				DL	LL	RLL	SL	WL	EQ			x	y	
1		Ex Roof Load	UNIFORM	15		20	32	-27.7		20	ft.	FULL LENGTH		Wind is a stepped load (see model)
			TOTAL	**300.0**	**0.0**	**400.0**	**640.0**	**-554.0**	**0.0**	**PLF**		0	0	
2		Load from concrete	POINT	113	65					53	sq.ft.	FULL LENGTH		Line Type 1 (9" slab only)
		floor at bottom	**TOTAL**	**6.0**	**3.4**	**0.0**	**0.0**	**0.0**	**0.0**	**KIPS**		0	0	
3		Load from concrete	POINT	1100						1	sq.ft.	FULL LENGTH		Line 1 beam
		encased beam	**TOTAL**	**1.1**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**KIPS**		0	0	
4		Load from concrete	POINT	100	65					64	sq.ft.	FULL LENGTH		Line Type 2 (avg. of 7" and 9")
		floor at bottom	**TOTAL**	**6.4**	**4.2**	**0.0**	**0.0**	**0.0**	**0.0**	**KIPS**		0	0	
5		Load from concrete	POINT	1500						1	sq.ft.	FULL LENGTH		Line 2 beam
		encased beam	**TOTAL**	**1.5**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**KIPS**		0	0	
6		Load from concrete	POINT	88	65					64	sq.ft.	FULL LENGTH		Line Type 3 (7" slab only)
		floor at bottom	**TOTAL**	**5.6**	**4.2**	**0.0**	**0.0**	**0.0**	**0.0**	**KIPS**		0	0	
7		Load from concrete	POINT	1500						1	sq.ft.	FULL LENGTH		Line 3 beam
		encased beam	**TOTAL**	**1.5**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**KIPS**		0	0	





EXAMPLE LOAD CASE - DEAD LOAD





EXAMPLE REACTIONS - DEAD LOAD

RAM Elements **Matteo Ferran**

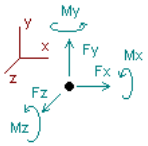
Truss T4 Node Reactions

Current Date: 6/25/2025 2:32 PM
Units system: English
File name: P:\23-086.1 NPS GATE Rehabilitate Hangars 3 & 4 at Floyd Bennett Field PD-CD\6_Calcs\RAM files\240904 Roof Truss T4_GATE Hangars.retx

Analysis result

Nodes

Reactions



Direction of positive forces and moments

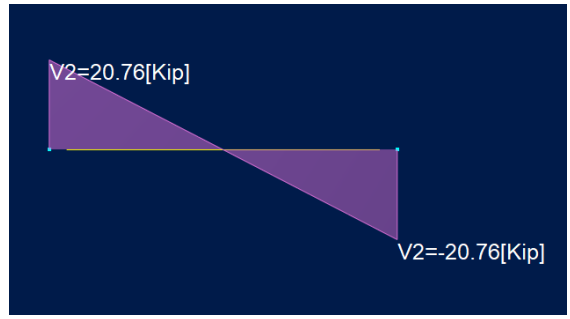
Node	Forces [Kip] FX	FY	FZ	Moments [Kip*ft] MX	MY	MZ
Condition **DL=Dead Load**						
28	-31.30159	35.81988	0.00000	0.00000	0.00000	0.00000
29	30.96719	35.28054	0.00000	0.00000	0.00000	0.00000
31	25.82428	0.29647	0.00000	0.00000	0.00000	0.00000
32	-25.48988	0.30317	0.00000	0.00000	0.00000	0.00000
SUM	0.00000	71.70006	0.00000	0.00000	0.00000	0.00000
Condition **LLR=Roof Live Load**						
28	-7.85024	10.06152	0.00000	0.00000	0.00000	0.00000
29	7.85024	10.06152	0.00000	0.00000	0.00000	0.00000
31	6.72520	-0.05213	0.00000	0.00000	0.00000	0.00000
32	-6.72520	-0.05213	0.00000	0.00000	0.00000	0.00000
SUM	0.00000	20.01877	0.00000	0.00000	0.00000	0.00000
Condition **SL=Snow Load**						
28	-12.56038	16.09843	0.00000	0.00000	0.00000	0.00000
29	12.56038	16.09843	0.00000	0.00000	0.00000	0.00000
31	10.76032	-0.08341	0.00000	0.00000	0.00000	0.00000
32	-10.76032	-0.08341	0.00000	0.00000	0.00000	0.00000
SUM	0.00000	32.03004	0.00000	0.00000	0.00000	0.00000
Condition **WL=Wind Load**						
28	8.62539	-12.41552	0.00000	0.00000	0.00000	0.00000
29	-7.48576	-7.71204	0.00000	0.00000	0.00000	0.00000
31	-7.47486	0.05937	0.00000	0.00000	0.00000	0.00000
32	6.33523	0.04941	0.00000	0.00000	0.00000	0.00000
SUM	0.00000	-20.01877	0.00000	0.00000	0.00000	0.00000
Condition **LL=Live Load**						
28	-13.25586	14.01387	0.00000	0.00000	0.00000	0.00000
29	13.56517	14.73178	0.00000	0.00000	0.00000	0.00000
31	10.78042	-0.07522	0.00000	0.00000	0.00000	0.00000
32	-11.08973	-0.07043	0.00000	0.00000	0.00000	0.00000
SUM	0.00000	28.60000	0.00000	0.00000	0.00000	0.00000

VI. Design of Reinforcement of Corroded Steel Framing Elements

WEB SHEAR DESIGN - LT-B2

Member Size: 21" CB 55#

V = 20.8 K (LRFD, FROM RAM MODEL, LC D5)



SHEAR STRENGTH

Vn = 0.6FyAwCv

Vn = 0.6*30*(0.36*20.89)*1.0 = 135.4 KIPS

h = 19.78 in.

h/tw = 19.78 / 0.36 = 54.96 < 69.64

kv = 5 for h/tw < 260, tw > 0.08

tw,min > V / (0.6*Fy*D*Cv) = 20.8 / (0.6*30*20.89*1.0)
= 0.0553

1.10 (kv*E / Fy)^0.5 = 76.5 for tw = 0.26

1.37 (kv*E / Fy)^0.5 = 95.2 for tw = 0.21, Cv = 0.81
- Vn = 0.6*30*.21*20.89*0.81 = 63.96 KIPS

for tw = 0.1875, Cv = 0.66
- Vn = 0.6*30*.1875*20.89*0.66 = 46.2 KIPS OK

for tw = 0.125, Cv = 0.29
- Vn = 0.6*30*.125*20.89*0.29 = 13.63 KIPS N.G.

G2. MEMBERS WITH UNSTIFFENED OR STIFFENED WEBS

1. Shear Strength

This section applies to webs of singly or doubly symmetric members and channels subject to shear in the plane of the web.

The *nominal shear strength*, V_n, of unstiffened or stiffened webs, according to the *limit states* of *shear yielding* and *shear buckling*, is

$$V_n = 0.6F_yA_wC_v \quad \text{(G2-1)}$$

(a) For webs of rolled I-shaped members with $h/t_w \le 2.24\sqrt{E/F_y}$:

$$\phi_v = 1.00 \text{ (LRFD)} \qquad \Omega_v = 1.50 \text{ (ASD)}$$

and

$$C_v = 1.0 \quad \text{(G2-2)}$$

(b) For webs of all other doubly symmetric shapes and singly symmetric shapes and channels, except round *HSS*, the web shear coefficient, C_v, is determined as follows:

(i) For $h/t_w \le 1.10\sqrt{k_vE/F_y}$

$$C_v = 1.0 \quad \text{(G2-3)}$$

(ii) For $1.10\sqrt{k_vE/F_y} < h/t_w \le 1.37\sqrt{k_vE/F_y}$

$$C_v = \frac{1.10\sqrt{k_vE/F_y}}{h/t_w} \quad \text{(G2-4)}$$

(iii) For $h/t_w > 1.37\sqrt{k_vE/F_y}$

$$C_v = \frac{1.51Ek_v}{(h/t_w)^2F_y} \quad \text{(G2-5)}$$

The web plate shear buckling co-efficient, k_v, is determined as follows:

- For webs without transverse stiffeners and with $h/t_w < 260$:

$$k_v = 5$$

except for the stem of tee shapes where $k_v = 1.2$.

- For webs with transverse stiffeners:

$$k_v = 5 + \frac{5}{\left(\frac{a}{h}\right)^2} \quad \text{(G2-6)}$$

$$= 5 \text{ when } a/h > 3.0 \text{ or } a/h > \left[\frac{260}{(h/t_w)}\right]^2$$

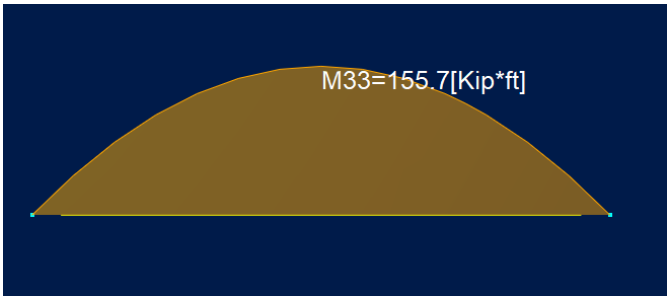
where

a= clear distance between transverse stiffeners, in.

FLANGE FLEXURAL DESIGN - LT-B2

M = 40.48 K-FT (LRFD, from RAM MODEL, LC D5)
--> at 1'-6" from bearing end

= 485.76 K-IN



FLEXURAL STRENGTH

--> Assume limit state of Compression Flange Local Buckling

Mn = [Mp - (Mp - 0.7FySx) * (Y - Ypf / Yrf - Ypf)]

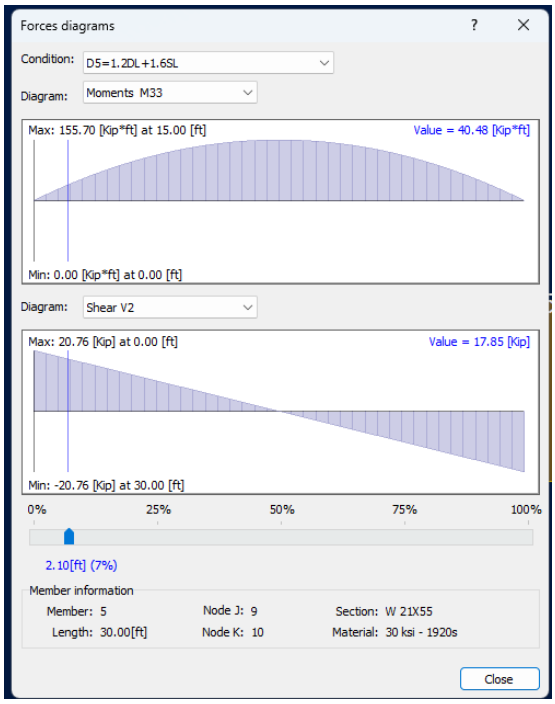
Mp = FyZx = Fy*(bf*tf*D)

Sx = B / 6 * (H^2 - h^3/H)

Y = bf / 2*tf
Ypf = 0.38*(E/Fy)^0.5 = 11.8
Yrf = 1.0*(E/Fy)^0.5 = 31.1

ORIGINAL tf = 0.553
Mp = 3756 K-IN

--> See calculations for minimum flange thickness in RAM spreadsheet for LT-B2 beam.



NAME: LT-B1
DESCRIPTION: Purlin at Lean-to

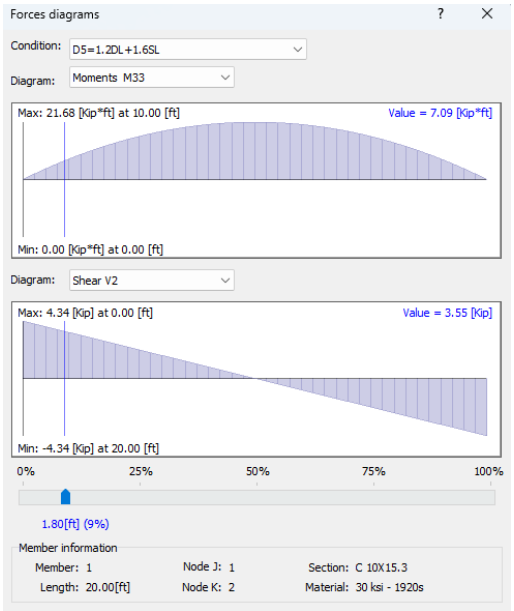
SPAN: 20 ft

SHEAR CAPACITY CALC

	C10x15.3	
Fy =	30	ksi
E =	29000	ksi
bf =	2.6	in
D =	10	in
tw =	0.065	in
tf =	0.436	in
h =	9.1	in
kv =	5	

Minimum web thickness to resist required shear

1.10*(kv*E / Fy)^0.5 =	76.47	
1.37*(kv*E / Fy)^0.5 =	95.25	
h/tw =	140.34	
Cv =	0.37	
Vn =	4.3 kips	G2-1
phi*Vn =	4.3 kips	
Vu =	4.34 kips	Demand
D/C =	1.00	



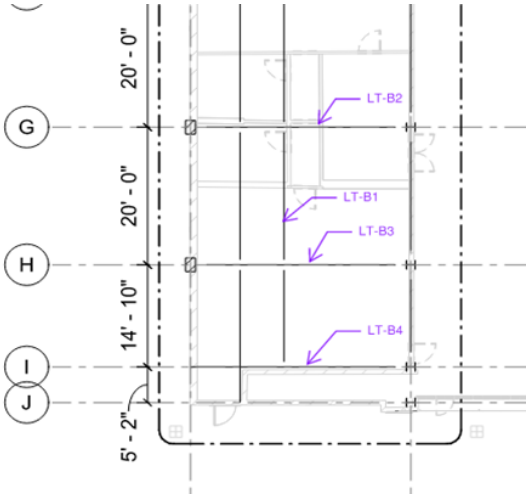
FLEXURAL CAPACITY CALC

Fy =	30	ksi
bf =	2.6	in
D =	9.65	in
tw =	0.24	in
tf =	0.083	in
Mp =	223.7	k-in
Sx =	5.6	in^3
Zx =	7.5	in^3
kc =	0.64	

Minimum flange thickness to resist required flexure

2.05 Sx without web

	Y =	31.32	For Channel
	Ypf =	11.81	
	Yrf =	31.09	
	Y' =	1.01	
Non-Compact	Mn =	115.9 k-in	(F3-1)
	phi*Mn =	104.3 k-in	
Slender	Mn =	94.5 k-in	(F3-2)
	phi*Mn =	85.0 k-in	
	Mu =	85.08 k-in	Demand
	govern, phi*Mn =	85.04	
	D/C =	1.00	



NAME: LT-B2
DESCRIPTION: Beam at lean-to

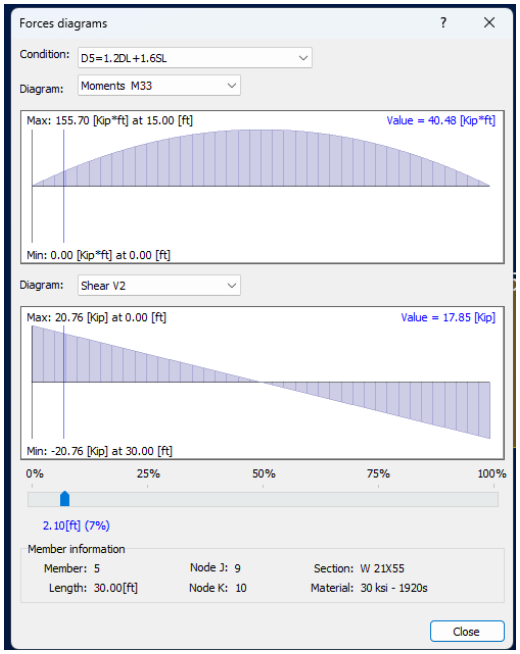
SPAN: 30 ft

SHEAR CAPACITY CALC

	CB21x55	
Fy =	30	ksi
E =	29000	ksi
bf =	8	in
D =	20.89	in
tw =	0.144	in
tf =	0.553	in
h =	19.8	in
kv =	5	

Minimum web thickness to resist required shear

1.10*(kv*E / Fy)^0.5 =	76.47	
1.37*(kv*E / Fy)^0.5 =	95.25	
h/tw =	137.69	
Cv =	0.38	
Vn =	20.8 kips	G2-1
phi*Vn =	20.8 kips	
Vu =	20.8 kips	Demand
D/C =	1.00	

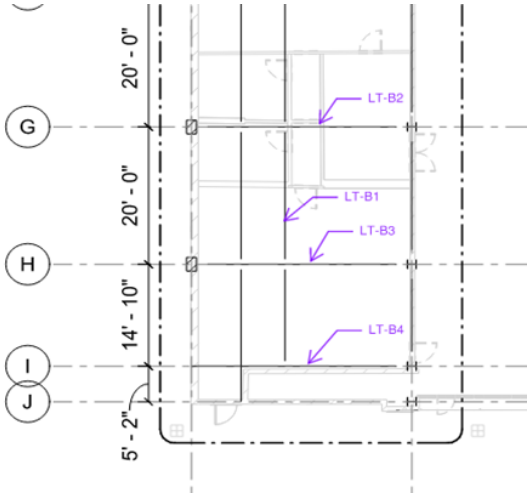


FLEXURAL CAPACITY CALC

Fy =	30	ksi	
bf =	8	in	
D =	20.5	in	
tw =	0.36	in	
tf =	0.119	in	
Mp =	1686.1	k-in	
Sx =	43.5	in^3	19.30 Sx without web
Zx =	56.2	in^3	
kc =	0.53		
Y =	33.52		For I-shaped member
Ypf =	11.81		
Yrf =	31.09		
Y' =	1.13		

Minimum flange thickness to resist required flexure

Non-Compact	Mn =	817.0 k-in	(F3-1)
	phi*Mn =	735.3 k-in	
Slender	Mn =	539.8 k-in	(F3-2)
	phi*Mn =	485.8 k-in	
	Mu =	485.76 k-in	Demand
	govern, phi*Mn =	485.81	
	D/C =	1.00	



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Bearing Plate on Brick Masonry

Overal Status: PASS

Code: AISC 360-16 (ASD) & ACI 530-13

Plate Thickness= **1/2"**

Project:	NPS GATE Floyd Bennett Field Hangars 3&4
Project No.:	23-086.1
Description:	Lean-to Girder 21 CB 55#

			k =	0.930	in
Reaction, R_a =	14.1	kips (ASD)	d =	20.800	in
f'_m =	0.6	ksi	t_w =	0.375	in
Beam Size =	W21X55		t_f =	0.522	in
Bearing Plate Size:			$f_{y,\ beam}$ =	32	ksi
N (length) =	12	in	$f_{y,\ plate}$ =	36	ksi
B (width) =	12	in	E =	29	x 10^3 ksi
Brick Pier Size:					
L (length) =	24	in	A_{wall} =	384.00	in^2
t (thickness) =	16	in			

Check Web Local Yielding (Ω = 1.50) *[AISC - J10.2]*

$R_n = (2.5\ k + N)\ F_{yw}\ t_w$

R_n = 171.9 kips

PASS R_n / Ω = 114.6 kips

Check Web Crippling (Ω = 2.00) *[AISC - J10.3]*

For $N/d \leq 0.2$

$$R_n = 0.40 t_w^2 \left[1 + 3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{EF_{yw}t_f}{t_w}} \qquad \text{(J10-5a)}$$

For $N/d > 0.2$

$$R_n = 0.40 t_w^2 \left[1 + \left(\frac{4N}{d} - 0.2\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{EF_{yw}t_f}{t_w}} \qquad \text{(J10-5b)}$$

N / d = 0.58 ==> Equation J10-5b is applicable

R_n = 146.0 kips

PASS R_n / Ω = 73.0 kips

Check Bearing on Masonry *[ACI 530 - 8.1.5; 4.3.4]*

$R_n = P_p = 0.33\ f'_m\ A_1$ A_1 = N x B = 144.0 in^2

PASS R_n = 28.5 kips

Design Plate Thickness (Ω = 1.67) *[AISC - F11]*

w_p = Ra / (B x N) =	0.098	ksi	(uniform pressure under plate)
n = B/2 - k =	5.07	in	(bearing plate cantilever dimension)
$M_a = n^2 / 2 * w_p$ =	1.26	k-in / in	
$t = (6.68\ M_a / F_y)^{1/2}$ =	0.483	in	(solved $\Omega\ M_a = F_y Z$ for 't' with Ω = 1.67)
Plate Thickness =	1/2"	in	

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Bearing Plate on Brick Masonry

Overal Status: **PASS**

Code: AISC 360-16 (ASD) & ACI 530-13

Plate Thickness= **1/4"**

Project:	**NPS GATE Floyd Bennett Field Hangars 3&4**
Project No.:	**23-086.1**
Description:	**WPA Infill Building - 10"C Purlin**

			k =	1.000	in
Reaction, R_a =	**3.5**	kips (ASD)	d =	10.000	in
f'_m =	**0.6**	ksi	t_w =	0.240	in
Beam Size =	C10x15.3		t_f =	0.436	in
Bearing Plate Size:			$f_{y,\ beam}$ =	32	ksi
N (length) =	**6**	in	$f_{y,\ plate}$ =	36	ksi
B (width) =	**8**	in	E =	29	x 10^3 ksi
Brick Wall:					
t (thickness) =	**12**	in			

Check Web Local Yielding (Ω = 1.50) *[AISC - J10.2]*

$R_n = (2.5\ k + N)\ F_{yw}\ t_w$

R_n = 65.3 kips

PASS R_n / Ω = 43.5 kips

Check Web Crippling (Ω = 2.00) *[AISC - J10.3]*

For $N/d \leq 0.2$

$$R_n = 0.40 t_w^2 \left[1 + 3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{EF_{yw}t_f}{t_w}} \qquad \text{(J10-5a)}$$

For $N/d > 0.2$

$$R_n = 0.40 t_w^2 \left[1 + \left(\frac{4N}{d} - 0.2\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{EF_{yw}t_f}{t_w}} \qquad \text{(J10-5b)}$$

N / d = 0.60 ==> Equation J10-5b is applicable

R_n = 56.8 kips

PASS R_n / Ω = 28.4 kips

Check Bearing on Masonry *[ACI 530 - 8.1.5; 4.3.4]*

$R_n = P_p = 0.33\ f'_m\ A_1$ A_1 = N x B = 48.0 in^2

PASS R_n = 9.5 kips

Design Plate Thickness (Ω = 1.67) *[AISC - F11]*

w_p = Ra / (B x N) =	0.073	ksi	(uniform pressure under plate)
n = B/2 - k =	3	in	(bearing plate cantilever dimension)
$M_a = n^2 / 2 * w_p$ =	0.33	k-in / in	
$t = (6.68\ M_a / F_y)^{1/2}$ =	0.247	in	(solved $\Omega\ M_a = F_y Z$ for 't' with Ω = 1.67)
Plate Thickness =	1/4"	in	

Loads to Line 2 Frame for Analysis of Corroded Column Bases

Column 2/C at Lean-To	20' Trib				
	DL	LL	RLL	SL	WL
R1 - Lean-To Girder	5.33		6	9.6	-6.3
R2 - Hangar Truss	24.7		24	38.4	-29.5
Wall Load	17.12				
Overturning Column 2/C					5.3
TOTAL	47.15	0	30	48	-30.5
1.4D	66.0				
1.2D+1.6L+0.5Lr	71.6				
1.2D+1.6L+0.3SL	71.0				
1.2D+1.6Lr+L	104.6				
1.2D+SL+L	104.6				
1.2D+1.6Lr+0.5W	89.3				
1.2D+SL+0.5W	89.3				
1.2D+WL+L+0.5Lr	40.5				
1.2D+WL+L+0.3SL	40.5				
0.9D+WL	11.9	MAX =	**104.6**		

HANGAR N-S WALL (LEAN-TO)

Description	Quantity	Unit
2 WYTHE (SELF-SUPPORTED)	140.00	sf
2 WYTHE (STORY 1)	80.75	sf
2 WYTHE (STORY 2)	171.11	sf
2 WYTHE (STORY 3)	129.27	sf
GLAZING (STORY 2)	124.29	sf
GLAZING (STORY 3)	70.33	sf

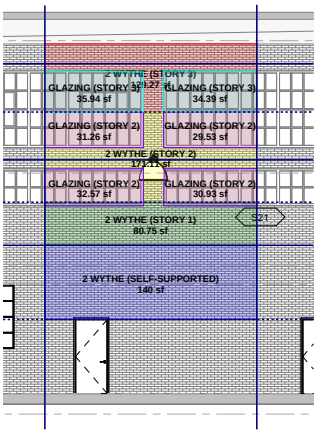
HANGAR N-S WALL (LEAN-TO)

STORY 3:
2 WYTHE:
130 SF - 70 SF (GLAZING) = 60 SF
60 SF * 82 PSF = 4920 lb
GLAZING:
70 SF * 10 PSF = 700 lb

STORY 2:
2 WYTHE:
170 SF - 125 SF = 45 SF
45 SF * 82 PSF = 3690 lb
GLAZING:
125 SF * 10 PSF = 1250 lb

STORY 1:
2 WYTHE:
80 SF * 82 PSF = 6560 lb

SELF-SUPPORTED AREA DOES NOT ADD AXIAL LOAD TO COLUMN, SO NEGLECT.



Column 2/A at Lean-To	10' Trib				
	DL	LL	RLL	SL	WL
R1 - Lean-To Girder					
R2 - Hangar Truss	12.35	0	12	19.2	-14.75
Wall Load	8.56				
Overturning Column 2/C					5.3
TOTAL	20.91	0	12	19.2	-9.45
1.4D	29.3				
1.2D+1.6L+0.5Lr	31.1				
1.2D+1.6L+0.3SL	30.9				
1.2D+1.6Lr+L	44.3				
1.2D+SL+L	44.3				
1.2D+1.6Lr+0.5W	39.6				
1.2D+SL+0.5W	39.6				
1.2D+WL+L+0.5Lr	21.4				
1.2D+WL+L+0.3SL	21.4				
0.9D+WL	9.4	MAX =	**44.3**		

HANGAR N-S WALL (INFILL)

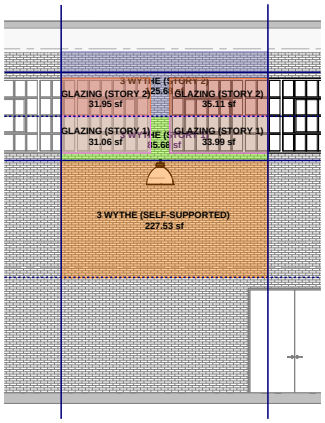
Description	Quantity	Unit
3 WYTHE (SELF-SUPPORTED)	227.53	sf
3 WYTHE (STORY 1)	85.68	sf
3 WYTHE (STORY 2)	125.68	sf
GLAZING (STORY 1)	65.05	sf
GLAZING (STORY 2)	67.07	sf

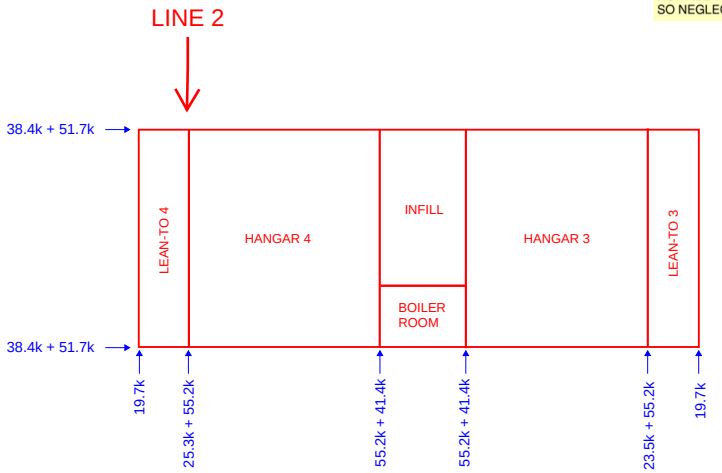
HANGAR N-S WALL (INFILL)

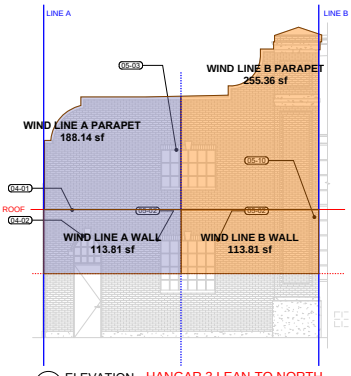
STORY 2:
3 WYTHE:
125 SF - 65 SF = 60 SF
60 SF * 132 PSF = 7920 lb
GLAZING:
65 SF * 10 PSF = 650 lb

STORY 1:
3 WYTHE:
85 SF - 65 SF = 20 SF
20 SF * 132 PSF = 2640 lb
GLAZING:
65 SF * 10 PSF = 650 lb

SELF-SUPPORTED AREA DOES NOT ADD AXIAL LOAD TO COLUMN, SO NEGLECT.







2 ELEVATION - HANGAR 3 LEAN-TO NORTH
3/16" = 1'-0"

LINE B:

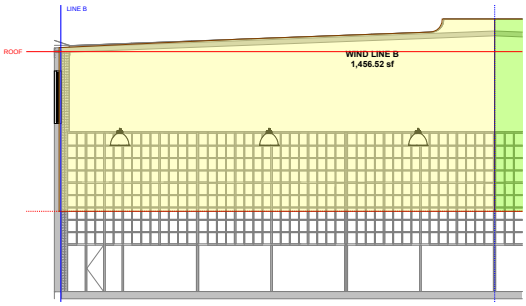
WALL FORCE = 26.7 PSF * 115 SF = 3.07k
PARAPET FORCE = 87.2 PSF * 255 SF = 22.2k
TOTAL = 25.3k

Wind Loads to Frame at Line 2

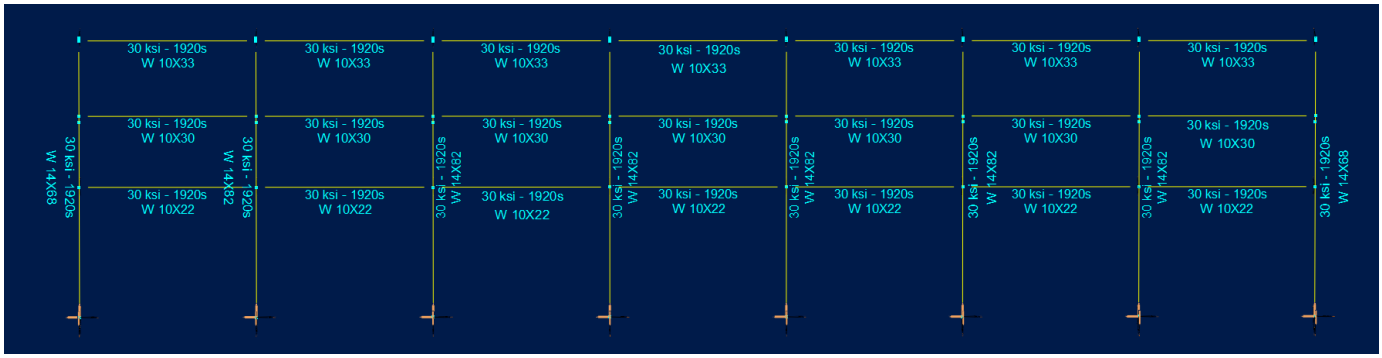
WIND PRESSURE PARALLEL TO LONG WALL = AVG(35.9 PSF, 37.3 PSF, 38.6 PSF, 39.5 PSF) = 37.8 PSF

LINE B:

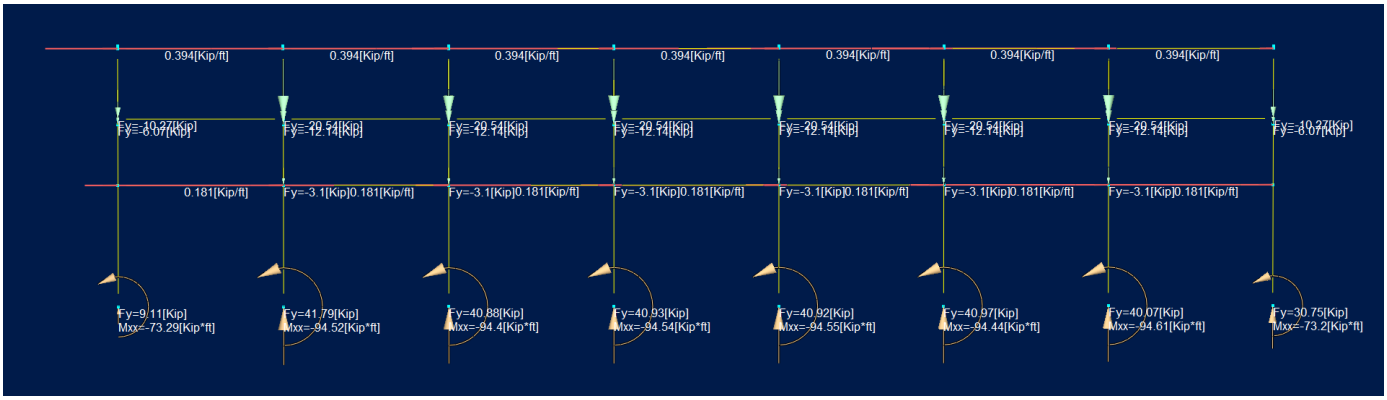
TOTAL WALL FORCE = 37.8 PSF * 1460 SF = 55.2k



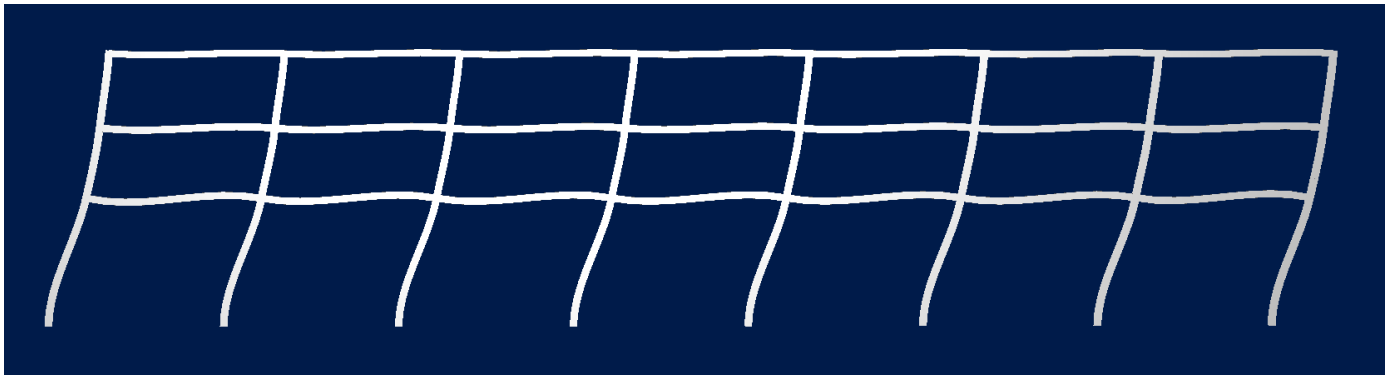
1 ELEVATION - HANGAR 4 SOUTH
SCALE



FRAME PROPERTIES - SECTION, MATERIAL, BOUNDARY CONDITIONS, AND HINGING (NONE)

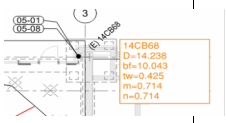


LOAD CASE 15 - LOADING & REACTIONS
1.2DL + WL + 0.5LLr



LOAD CASE 15 - DEFLECTED SHAPE

DETERMINATION OF MINIMUM WEB & FLANGE THICKNESSES AT HANGAR END COLUMNS - LINE 2



Original Axial Capacity

Shape=	14CB68		
Fy =	30	ksi	
E =	29000	ksi	
Kx=	1		
Ky=	1		
L=	22		
Ag=	19.79	in2	
D=	14.238	in	
tw=	0.425	in	
bf=	10.043	in	
tf=	0.714	in	
h=	12.81	in	
Ix=	730.8	in4	
Iy=	120.6	in4	
rx=	6.08	in	
ry=	2.47	in	
KxL/rx=	43.4		
KxL/ry=	106.9		
h/tw=	30.14		
Fe=	25.0	ksi	
Fy/Fe=	1.20	<= 2.25	**Use EQ. E3-2**
Fcr=	18.2	ksi	E3-2
Fcr=	22.0	ksi	E3-3
Governing Fcr=	18.2		
Pn=	359.5	kips	E3-1
ϕPn=	**323.5**	**kips**	

Section Slenderness for Compression

b/t=	7.03	Flanges	
limiting b/t=	17.41		Non-slender Flange Tabel B4.1a
h/tw=	30.14	Web	
limiting h/tw=	46.33		Non-slender Web Tabel B4.1a

Non-slender Section

Qs=	1		E7-4,5,6
bef=	-51.01671		E7-17
bew=	12.81		E7-17
h=	12.81		
Aef=	14.34	in2	
Aew=	5.44	in2	
Ae=	19.79		
Qa=	1.00		E7-16
Q=	1.000		
QFy/Fe=	1.20	<= 2.25	**Use EQ. E7-2**
Fcr=	18.2	ksi	E7-2
Fcr=	22.0	ksi	E7-3
Governing Fcr=	18.2	ksi	
Pn=	359.5	kips	E7-1
ϕPn=	**323.5**	**kips**	

Minimal Web Thickness

Shape=	14CB86			
Fy =	30	ksi		
E =	29000	ksi		
Kx=	1			
Ky=	1			
L=	22			
Ag=	15.94	in2		
D=	14.238	in		
tw=	0.125	in		0.300 in
bf=	10.043	in		
tf=	0.714	in		
h=	12.81	in		
Ix=	678.3	in4		
Iy=	120.5	in4		
rx=	6.52	in		
ry=	2.75	in		
KxL/rx=	40.5			
KxL/ry=	96.0			
h/tw=	102.48			
Fe=	31.1	ksi		
Fy/Fe=	0.97	<= 2.25	**Use EQ. E3-2**	
Fcr=	20.0	ksi	E3-2	
Fcr=	27.2	ksi	E3-3	
Governing Fcr=	20.0			
Pn=	319.2	kips	E3-1	
ϕPn=	**287.3**	**kips**		

MINIMUM WEB THICKNESS

Section Slenderness for Compression

b/t=	7.03	Flanges	
limiting b/t=	17.41		Non-slender Flange Tabel B4.1a
h/tw=	102.48	Web	
limiting h/tw=	46.33		Slender Web Tabel B4.1a

Slender Section

Qs=	1		E7-4,5,6
bef=	-43.8201581		E7-17
bew=	7.98067253		E7-17
h=	12.81		
Aef=	14.34	in2	
Aew=	1.00	in2	
Ae=	15.34		
Qa=	0.96		E7-16
Q=	0.962		
QFy/Fe=	0.93	<= 2.25	**Use EQ. E7-2**
Fcr=	19.6	ksi	E7-2
Fcr=	27.2	ksi	E7-3
Governing Fcr=	19.6	ksi	
Pn=	311.9	kips	E7-1
ϕPn=	**280.7**	**kips**	

Minimal Flange Thickness

Shape=	14CB86			
Fy =	30	ksi		
E =	29000	ksi		
Kx=	1			
Ky=	1			
L=	22			
Ag=	18.46	in2		
D=	14.238	in		
tw=	0.425	in		
bf=	10.043	in		
tf=	0.645	in		0.069 in
h=	12.94761	in		
Ix=	675.9	in4		
Iy=	109.0	in4		
rx=	6.05	in		
ry=	2.43	in		
KxL/rx=	43.6			
KxL/ry=	108.6			
h/tw=	30.46			
Fe=	24.2	ksi		
Fy/Fe=	1.24	<= 2.25	**Use EQ. E3-2**	
Fcr=	17.9	ksi	E3-2	
Fcr=	21.3	ksi	E3-3	
Governing Fcr=	17.9			
Pn=	330.0	kips	E3-1	
ϕPn=	**297.0**	**kips**		

MINIMUM FLANGE THICKNESS

Section Slenderness for Compression

b/t=	7.78	Flanges	
limiting b/t=	17.41		Non-slender Flange Tabel B4.1a
h/tw=	30.46	Web	
limiting h/tw=	46.33		Non-slender Web Tabel B4.1a

Non-slender Section

Qs=	1		E7-4,5,6
bef=	-37.90388		E7-17
bew=	12.94761		E7-17
h=	12.94761		
Aef=	12.96	in2	
Aew=	5.50	in2	
Ae=	18.46		
Qa=	1.00		E7-16
Q=	1.000		
QFy/Fe=	1.24	<= 2.25	**Use EQ. E7-2**
Fcr=	17.9	ksi	E7-2
Fcr=	21.3	ksi	E7-3
Governing Fcr=	17.9	ksi	
Pn=	330.0	kips	E7-1
ϕPn=	**297.0**	**kips**	

Minimal Web & Flange Thicknesses Jointly

Shape=	14CB86			
Fy =	30	ksi		
E =	29000	ksi		
Kx=	1			
Ky=	1			
L=	22			
Ag=	14.88	in2		
D=	14.238	in		
tw=	0.125	in		0.300 in
bf=	10.043	in		
tf=	0.661	in		0.053 in
h=	12.9168389	in		
Ix=	634.4	in4		
Iy=	111.5	in4		
rx=	6.53	in		
ry=	2.74	in		
KxL/rx=	40.4			
KxL/ry=	96.4			
h/tw=	103.33			
Fe=	30.8	ksi		
Fy/Fe=	0.97	<= 2.25	**Use EQ. E3-2**	
Fcr=	19.9	ksi	E3-2	
Fcr=	27.0	ksi	E3-3	
Governing Fcr=	19.9			
Pn=	296.9	kips	E3-1	
ϕPn=	**267.2**	**kips**		

MINIMUM COMBINED WEB & FLANGE THICKNESSES

Section Slenderness for Compression

b/t=	7.60	Flanges	
limiting b/t=	17.41		Non-slender Flange Tabel B4.1a
h/tw=	103.33	Web	
limiting h/tw=	46.33		Slender Web Tabel B4.1a

Slender Section

Qs=	1		E7-4,5,6
bef=	-34.1093141		E7-17
bew=	8.00271426		E7-17
h=	12.9168389		
Aef=	13.27	in2	
Aew=	1.00	in2	
Ae=	14.27		
Qa=	0.96		E7-16
Q=	0.959		
QFy/Fe=	0.93	<= 2.25	**Use EQ. E7-2**
Fcr=	19.5	ksi	E7-2
Fcr=	27.0	ksi	E7-3
Governing Fcr=	19.5	ksi	
Pn=	289.5	kips	E7-1
ϕPn=	**260.5**	**kips**	

Original Bending Capacity

Fy =	30	ksi	
Zy =	36.6	in3	
Sy =	24.0	in3	
Mp =	1097.6	k-in	F6-1
lambda =	7.03		
Lam,pf =	11.81		Compact Flange Table B4.1a
kc =	0.73		
FL =	21.0	ksi	
Lam,rf =	30.1		Non-Compact Flange Table B4.1a
Mn =	1252.4	k-in	F6-2
Fcr=	404.6		F6-4
Mn =	9718.0	k-in	F6-3
Mn, gov =	1097.6	k-in	
ϕMn,y=	**987.8**	k-in	
Pr =	30.75	kips	
Mr,y =	878.4	k-in	LC D15
Pr / Pc =	0.10		
Mr / Mc =	0.89		
Interaction =	**0.94**		

FORCES PER RAM ELEMENTS MODEL AT LINE 2

Minimal Web Thickness

Fy =	30	ksi	
Zy =	36.1	in3	
Sy =	24.0	in3	
Mp =	1081.7	k-in	F6-1
lambda =	7.03		
Lam,pf =	11.81		Compact Flange Table B4.1a
kc =	0.40		
FL =	21.0	ksi	
Lam,rf =	22.2		Non-Compact Flange Table B4.1a
Mn =	1347.9	k-in	F6-2
Fcr=	404.6		F6-4
Mn =	9711.5	k-in	F6-3
Mn, gov =	1081.7	k-in	Governing
ϕMn,y=	973.6	k-in	
Pr =	30.75	kips	
Mr,y =	878.4	k-in	LC D15
Pr / Pc =	0.11		
Mr / Mc =	0.90		
Interaction =	**0.96**		

2%

Minimal Flange Thickness

Fy =	30	ksi	
Zy =	33.1	in3	
Sy =	21.7	in3	
Mp =	993.7	k-in	F6-1
lambda =	7.78		
Lam,pf =	11.81		Compact Flange Table B4.1a
kc =	0.72		
FL =	21.0	ksi	
Lam,rf =	30.1		Non-Compact Flange Table B4.1a
Mn =	1112.6	k-in	F6-2
Fcr=	330.3		F6-4
Mn =	7171.1	k-in	F6-3
Mn, gov =	993.7	k-in	Governing
ϕMn,y=	894.3	k-in	
Pr =	30.75	kips	
Mr,y =	878.4	k-in	LC D15
Pr / Pc =	0.10		
Mr / Mc =	0.98		
Interaction =	**1.03**		

10%

Minimal Web & Flange Thicknesses Jointly

Fy =	30	ksi	
Zy =	33.4	in3	
Sy =	22.2	in3	
Mp =	1000.9	k-in	F6-1
lambda =	7.60		
Lam,pf =	11.81		Compact Flange Table B4.1a
kc =	0.39		
FL =	21.0	ksi	
Lam,rf =	22.1		Non-Compact Flange Table B4.1a
Mn =	1218.9	k-in	F6-2
Fcr=	346.3		F6-4
Mn =	7690.8	k-in	F6-3
Mn, gov =	1000.9	k-in	Governing
ϕMn,y=	900.8	k-in	
Pr =	30.75	kips	
Mr,y =	878.4	k-in	LC D15
Pr / Pc =	0.12		
Mr / Mc =	0.98		
Interaction =	**1.03**		

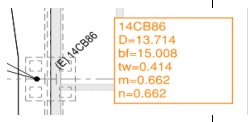
10%

INTERACTION EQUATION FOR COMBINED AXIAL AND MOMENT FORCES FOR ORIGINAL MEMBER WITHOUT SECTION LOSS.

2% INCREASE IN MEMBER STRESSES WHEN COLUMN WEB REDUCED TO MINIMUM TOLERABLE THICKNESS.

10% INCREASE IS MAX ALLOWABLE INCREASE IN STRESSES FOR AN EXISTING LATERAL LOAD-CARRYING MEMBER PER IBC.

DETERMINATION OF MINIMUM WEB & FLANGE THICKNESSES AT HANGAR INTERIOR COLUMNS - LINE 2



Original Axial Capacity

Shape=	14CB86		
Fy =	30	ksi	
E =	29000	ksi	
Kx=	1		
Ky=	1		
L=	22		
Ag=	25.00	in2	
D=	13.714	in	
tw=	0.414	in	
bf=	15.008	in	
tf=	0.662	in	
h=	12.39	in	
Ix=	912.6	in4	
Iy=	373.0	in4	
rx=	6.04	in	
ry=	3.86	in	
KxL/rx=	43.7		
KxL/ry=	68.3		
h/tw=	29.93		
Fe=	61.3	ksi	
Fy/Fe=	0.49	<= 2.25	**Use EQ. E3-2**
Fcr=	24.4	ksi	E3-2
Fcr=	53.7	ksi	E3-3
Governing Fcr=	24.4		
Pn=	611.0	kips	E3-1
ϕPn=	**549.9**	**kips**	

Section Slenderness for Compression

b/t=	11.34	Flanges	
limiting b/t=	17.41		Non-slender Flange Tabel B4.1a
h/tw=	29.93	Web	
limiting h/tw=	46.33		Non-slender Web Tabel B4.1a

Non-slender Section

Qs=	1		E7-4,5,6
bef=	-1.452857		E7-17
bew=	12.39		E7-17
h=	12.39		
Aef=	19.87	in2	
Aew=	5.13	in2	
Ae=	25.00		
Qa=	1.00		E7-16
Q=	1.000		
QFy/Fe=	0.49	<= 2.25	**Use EQ. E7-2**
Fcr=	24.4	ksi	E7-2
Fcr=	53.7	ksi	E7-3
Governing Fcr=	24.4	ksi	
Pn=	611.0	kips	E7-1
ϕPn=	**549.9**	**kips**	

Minimal Web Thickness

Shape=	14CB86			
Fy =	30	ksi		
E =	29000	ksi		
Kx=	1			
Ky=	1			
L=	22			
Ag=	21.42	in2		
D=	13.714	in		
tw=	0.125	in		0.289 in
bf=	15.008	in		
tf=	0.662	in		
h=	12.39	in		
Ix=	866.8	in4		
Iy=	373.0	in4		
rx=	6.36	in		
ry=	4.17	in		
KxL/rx=	41.5			
KxL/ry=	63.3			
h/tw=	99.12			
Fe=	71.5	ksi		
Fy/Fe=	0.42	<= 2.25	**Use EQ. E3-2**	
Fcr=	25.2	ksi	E3-2	
Fcr=	62.7	ksi	E3-3	
Governing Fcr=	25.2			
Pn=	539.1	kips	E3-1	
ϕPn=	**485.2**	**kips**		

MINIMUM WEB THICKNESS

Section Slenderness for Compression

b/t=	11.34	Flanges	
limiting b/t=	17.41		Non-slender Flange Tabel B4.1a
h/tw=	99.12	Web	
limiting h/tw=	46.33		Slender Web Tabel B4.1a

Slender Section

Qs=	1		E7-4,5,6
bef=	-0.7831		E7-17
bew=	7.198093		E7-17
h=	12.39		
Aef=	19.87	in2	
Aew=	0.90	in2	
Ae=	20.77		
Qa=	0.97		E7-16
Q=	0.970		
QFy/Fe=	0.41	<= 2.25	**Use EQ. E7-2**
Fcr=	24.5	ksi	E7-2
Fcr=	62.7	ksi	E7-3
Governing Fcr=	24.5	ksi	
Pn=	525.6	kips	E7-1
ϕPn=	**473.0**	**kips**	

Minimal Flange Thickness

Shape=	14CB86			
Fy =	30	ksi		
E =	29000	ksi		
Kx=	1			
Ky=	1			
L=	22			
Ag=	23.47	in2		
D=	13.714	in		
tw=	0.414	in		
bf=	15.008	in		
tf=	0.610	in		0.052 in
h=	12.49468	in		
Ix=	853.5	in4		
Iy=	343.6	in4		
rx=	6.03	in		
ry=	3.83	in		
KxL/rx=	43.8			
KxL/ry=	69.0			
h/tw=	30.18			
Fe=	60.1	ksi		
Fy/Fe=	0.50	<= 2.25	**Use EQ. E3-2**	
Fcr=	24.3	ksi	E3-2	
Fcr=	52.7	ksi	E3-3	
Governing Fcr=	24.3			
Pn=	571.4	kips	E3-1	
ϕPn=	**514.3**	**kips**		

MINIMUM FLANGE THICKNESS

Section Slenderness for Compression

b/t=	12.31	Flanges	
limiting b/t=	17.41		Non-slender Flange Tabel B4.1a
h/tw=	30.18	Web	
limiting h/tw=	46.33		Non-slender Web Tabel B4.1a

Non-slender Section

Qs=	1		E7-4,5,6
bef=	1.882856		E7-17
bew=	12.49468		E7-17
h=	12.49468		
Aef=	18.30	in2	
Aew=	5.17	in2	
Ae=	23.47		
Qa=	1.00		E7-16
Q=	1.000		
QFy/Fe=	0.50	<= 2.25	**Use EQ. E7-2**
Fcr=	24.3	ksi	E7-2
Fcr=	52.7	ksi	E7-3
Governing Fcr=	24.3	ksi	
Pn=	571.4	kips	E7-1
ϕPn=	**514.3**	**kips**	

Minimal Web & Flange Thicknesses Jointly

Shape=	14CB86			
Fy =	30	ksi		
E =	29000	ksi		
Kx=	1			
Ky=	1			
L=	22			
Ag=	20.17	in2		
D=	13.714	in		
tw=	0.125	in		0.289 in
bf=	15.008	in		
tf=	0.620	in		0.042 in
h=	12.4739145	in		
Ix=	818.5	in4		
Iy=	349.3	in4		
rx=	6.37	in		
ry=	4.16	in		
KxL/rx=	41.4			
KxL/ry=	63.4			
h/tw=	99.79			
Fe=	71.1	ksi		
Fy/Fe=	0.42	<= 2.25	**Use EQ. E3-2**	
Fcr=	25.1	ksi	E3-2	
Fcr=	62.4	ksi	E3-3	
Governing Fcr=	25.1			
Pn=	507.2	kips	E3-1	
ϕPn=	**456.5**	**kips**		

MINIMUM COMBINED WEB & FLANGE THICKNESSES

Section Slenderness for Compression

b/t=	12.10	Flanges	
limiting b/t=	17.41		Non-slender Flange Tabel B4.1a
h/tw=	99.79	Web	
limiting h/tw=	46.33		Slender Web Tabel B4.1a

Slender Section

Qs=	1		E7-4,5,6
bef=	1.85675342		E7-17
bew=	7.20745155		E7-17
h=	12.4739145		
Aef=	18.61	in2	
Aew=	0.90	in2	
Ae=	19.51		
Qa=	0.97		E7-16
Q=	0.967		
QFy/Fe=	0.41	<= 2.25	**Use EQ. E7-2**
Fcr=	24.5	ksi	E7-2
Fcr=	62.4	ksi	E7-3
Governing Fcr=	24.5	ksi	
Pn=	493.5	kips	E7-1
ϕPn=	**444.1**	**kips**	

Original Bending Capacity

Fy =	30	ksi	
Zy =	75.1	in3	
Sy =	49.7	in3	
Mp =	2252.6	k-in	F6-1
lambda =	11.34		
Lam,pf =	11.81		Compact Flange Table B4.1a
kc =	0.73		
FL =	21.0	ksi	
Lam,rf =	30.2		Non-Compact Flange Table B4.1a
Mn =	2284.1	k-in	F6-2
Fcr=	155.7		F6-4
Mn =	7741.9	k-in	F6-3
Mn, gov =	2252.6	k-in	
ϕMn,y=	**2027.3**	k-in	
Pr =	40.07	kips	
Mr,y =	1135.32	k-in	LC D15
Pr / Pc =	0.07		
Mr / Mc =	0.56		
Interaction =	**0.596**		

FORCES PER RAM ELEMENTS MODEL AT LINE 2

INTERACTION EQUATION FOR COMBINED AXIAL AND MOMENT FORCES FOR ORIGINAL MEMBER WITHOUT SECTION LOSS.

Minimal Web Thickness

Fy =	30	ksi	
Zy =	74.6	in3	
Sy =	49.7	in3	
Mp =	2238.1	k-in	F6-1
lambda =	11.34		
Lam,pf =	11.81		Compact Flange Table B4.1a
kc =	0.40		
FL =	21.0	ksi	
Lam,rf =	22.4		Non-Compact Flange Table B4.1a
Mn =	2292.3	k-in	F6-2
Fcr=	155.7		F6-4
Mn =	7740.4	k-in	F6-3
Mn, gov =	2238.1	k-in	Governing
ϕMn,y=	2014.3	k-in	
Pr =	40.07	kips	
Mr,y =	1135.32	k-in	LC D15
Pr / Pc =	0.08		
Mr / Mc =	0.56		
Interaction =	**0.61**		

2%

2% INCREASE IN MEMBER STRESSES WHEN COLUMN WEB REDUCED TO MINIMUM TOLERABLE THICKNESS.

Minimal Flange Thickness

Fy =	30	ksi	
Zy =	69.2	in3	
Sy =	45.8	in3	
Mp =	2075.9	k-in	F6-1
lambda =	12.31		
Lam,pf =	11.81		Compact Flange Table B4.1a
kc =	0.73		
FL =	21.0	ksi	
Lam,rf =	30.1		Non-Compact Flange Table B4.1a
Mn =	2045.8	k-in	F6-2
Fcr=	132.1		F6-4
Mn =	6047.0	k-in	F6-3
Mn, gov =	2045.8	k-in	Governing
ϕMn,y=	1841.2	k-in	
Pr =	40.07	kips	
Mr,y =	1135.32	k-in	LC D15
Pr / Pc =	0.08		
Mr / Mc =	0.62		
Interaction =	**0.66**		

10%

Minimal Web & Flange Thicknesses Jointly

Fy =	30	ksi	
Zy =	69.9	in3	
Sy =	46.6	in3	
Mp =	2096.3	k-in	F6-1
lambda =	12.10		
Lam,pf =	11.81		Compact Flange Table B4.1a
kc =	0.40		
FL =	21.0	ksi	
Lam,rf =	22.3		Non-Compact Flange Table B4.1a
Mn =	2065.8	k-in	F6-2
Fcr=	136.6		F6-4
Mn =	6359.9	k-in	F6-3
Mn, gov =	2065.8	k-in	Governing
ϕMn,y=	1859.2	k-in	
Pr =	40.07	kips	
Mr,y =	1135.32	k-in	LC D15
Pr / Pc =	0.09		
Mr / Mc =	0.61		
Interaction =	**0.66**		

10%

10% INCREASE IS MAX ALLOWABLE INCREASE IN STRESSES FOR AN EXISTING LATERAL LOAD-CARRYING MEMBER PER IBC.

VII. Reinforcement / Repair Of Hangar Door Pocket Walls

C&C WIND LOAD
PER ASCE 7-22

CONSIDER ENCLOSED BLDG w/ $h = 30'\pm$

$V = 118$ mph (RISK CATEGORY II) (ASCE 7 HAZARD TOOL)
EXPOSURE D (ASCE 7 26.7.4.4)
$K_d = 0.85$
$K_h = 1.16$
$GC_{pi} = \pm 0.18$
$q_h = (0.00256)(1.16)(1.0)(1.0)(118)^2 = 41.3$ psf

$GC_p = \begin{cases} +1.0/-1.1 & \text{ZONE 4} \\ +1.0/-1.4 & \text{ZONE 5} \end{cases}$ EFFECTIVE WIND AREA $\leq 10\ ft^2$

$GC_p = \begin{cases} +0.8/-0.9 & \text{ZONE 4} \\ +0.8/-1.05 & \text{ZONE 5} \end{cases}$ EFFECTIVE WIND AREA $= 100\ ft^2$

$GC_p = +0.7/-0.8$ ZONE 4 & 5 EFFECTIVE WIND AREA $\geq 500\ ft^2$

FOR WALL HORZ. SPAN OF $20'\pm$
EFFECTIVE WIND AREA $\geq 20^2/3 = 133\ ft^2$
$\Rightarrow$ SAY $100\ ft^2$

$\Rightarrow$ SAY $GC_p = +0.8/-1.0$

$p = (41.3)(0.85)\overbrace{(-1.0 - 0.18)}^{\text{MAX.}} = -41.4$ psf LRFD (1.0W)

$p = (0.6)(-41.4) = -24.8$ psf ASD (0.6W)

NOTE: IF $h = 15'$, $K_h = 1.03 \Rightarrow p = \left(\frac{1.03}{1.16}\right)(24.8) = 22.0$ psf ASD

OUT-OF-PLANE SEISMIC
LOAD PER ASCE 7-22

$F_p = 0.4 S_{DS} I W$
$= (0.4)(0.17)(1.0) W$
$= 0.068 W$
$\Rightarrow$ MIN. $F_p = 0.10 W$ CONTROLS

ANCHORAGE: $F_p = 0.4 S_{DS} k_a I W_p$

$k_a \leq 2.0 \Rightarrow F_p \leq (2)(0.068) W_p = 0.136 W_p$

MIN. $F_p = 0.2 k_a I W_p = (0.2)(2)(1.0) W_p = 0.4 W_p$
CONTROLS FOR $k_a = 2.0$ MAX.

$\Rightarrow$ OUT-OF-PLANE SEISMIC LOAD ON WALL
FOR 8" UFM (80 psf) = (0.10)(80) = 8 psf LRFD (1.0E) DOESN'T CONTROL OVER WIND

EFFECTIVE LOAD FOR ANCHORAGE
= (0.4)(80) = 32 psf MAX. LRFD (1.0E)

(0.7)(32) = 22.4 psf MAX. ASD (0.7E)
≈ 20 psf ASD WIND LOAD
$\Rightarrow$ OK TO DESIGN ANCHORAGE FOR
20 psf ASD WIND LOAD IF DESIGN
IS ≥ 10% CONSERVATIVE

SPAN EXTERIOR WALL HORIZONTALLY
FROM RETURN WALL TO NEW STRONGBACK

$l = 18'-9" \pm$

SPANDREL BETWEEN WINDOW OPNGS
$TRIB = (5.5 + 6.25/2 + 4.75/2)(18.75) = (11)(18.75)$
$= 206\ ft^2 > 18.75^2/3 = 117\ ft^2$

$\Rightarrow GC_p \approx -0.95$ MAX.
$p = (41.3)(0.85)(-0.95 - 0.18) = -39.7$ psf LRFD (1.0W)
$p = (0.6)(-39.7) = -23.8$ psf ASD (0.6W)

$M_u = [(11)(40)](18.75)^2/8 = 19300$ ft-lb $= 19.3$ ft-k

MAX. WIRE SIZE $= 3/8"/2 = 3/16" = 0.1875"$ FOR $3/8"$ MORTAR JOINTS
$\Rightarrow$ D2 DEFORMED WIRE (0.160" ϕ)
$A_s = 0.020\ in^2$
SAY $d = 6.5"$ (NOTE: SLOT IS $1\frac{1}{2}"$ DEEP $\Rightarrow$ CONSERVATIVE)

REQ'D $A_s \approx \dfrac{(19.3)(12)}{(0.9)(75)(0.9)(6.5)} = 0.587\ in^2$ ($f_y = 75$ ksi PER ASTM A1022 FOR DEFORMED SS WIRE)

$0.587/0.020 = 29.4 \Rightarrow (30)$ D2 WIRES

ASSUME ORIGINAL DESIGN WOULD BE FOR
20 psf ASD WIND LOAD
$\Rightarrow$ REQ'D $A_s \approx (20/24)(0.587) = 0.489\ in^2$

$0.489/0.020 = 24.5 \approx$ # OF MORTAR JOINTS IN SPANDREL
$\Rightarrow$ REINFORCE EVERY JOINT

~21 JOINTS $\Rightarrow A_s = (21)(0.020) = 0.420\ in^2$ (NOTE: # OF JOINTS ACCOUNTS FOR HEADER COURSE/SILL @ WINDOWS BLOCKING SOME JOINTS)

$$\rho = \frac{0.420}{(5.5)(12)(6.5)} = 0.000979$$

ASSUME $f'_m = 750$ psi

$$\Rightarrow \rho_{MAX} = (0.8)(0.8)\left(\frac{750}{75000}\right)\left[\frac{0.0035}{(0.0035+0.005)}\right] = 0.00264$$

FOR TENSION-CONTROLLED ($\phi = 0.9$)

$\underline{\rho < \rho_{MAX} \Rightarrow \phi = 0.9}$

$$a = \frac{(0.420)(75)}{(0.80)(0.750)(5.5)(12)} = 0.795''$$

$$d - \frac{a}{2} = 6.5 - \frac{0.795}{2} = 6.10'' = 0.938d$$

$$\phi M_n = (0.9)(0.420)(75)(6.10)/12 = \underline{14.4\ ft\text{-}k}$$

FOR ASSUMED ORIGINAL ASD DESIGN WIND LOAD OF 20 psf (33 psf LRFD)

$$M_u = (33/40)(19.3) = \underline{15.9\ ft\text{-}k}$$

SAY 10% OVERLOAD <u>OK</u>

NOTE: NEGLECTING WINDOW OPENINGS, FOR A SOLID URM WALL SPANNING 18'-9" HORIZONTALLY WITH 20 psf ASD WIND LOAD

$M = (20)(18.75)^2/8 = 879\ ft\text{-}lb/ft$

$S_g = (12)(8)^2/6 = 128\ in^3/ft$

$f_t = (879)(12)/128 = 82.4$ psi $\approx F_t = 80$ psi FOR TYPE N PORTLAND CEMENT/LIME MORTAR, PARALLEL TO BED JOINTS, SOLID UNITS IN RUNNING BOND

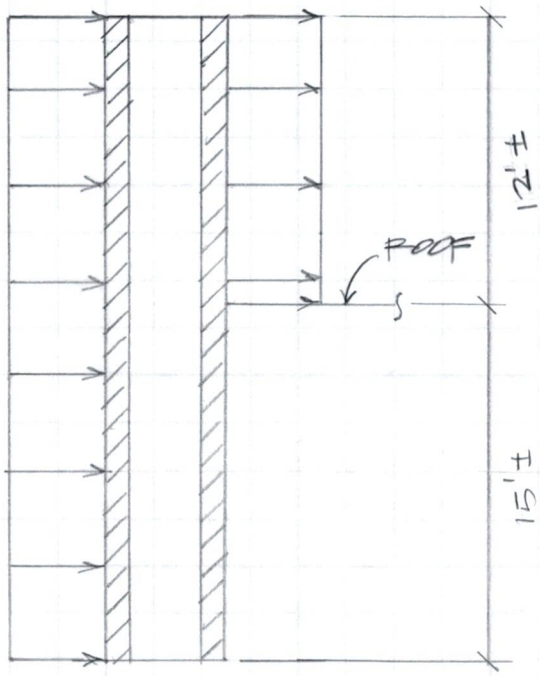
$f_r = 200$ psi

$M_{cr} = (5.5)(128)(0.2)/12 = 11.7$ ft-k

$1.3M_{cr} = (1.3)(11.7) = \underline{\underline{15.2 \text{ ft-k}}}$

$M_n = 14.4/0.9 = \underline{\underline{16.0 \text{ ft-k}}} > 1.3M_{cr}$ $\underline{\underline{OK}}$

PARAPET WIND LOAD



$h = 27' \pm$ TO TOP OF PARAPET/DOOR POCKET

EXPOSURE D

$\Rightarrow K_h = 1.14 \pm$

$q_p = (0.00256)(1.14)(1.0)(1.0)(118)^2 = 40.6$ psf

$K_d = 0.85$

$GC_{pi} = \pm 0.18$

$(12)(12)/3 = 48\ ft^2 \Rightarrow$ SAY EFFECTIVE WIND AREA $= 50\ ft^2$

$$\Rightarrow GC_p \approx \begin{cases} +0.9 & \text{ZONE 4 \& 5} \\ -1.0 & \text{ZONE 4} \\ -1.2 & \text{ZONE 5} \end{cases} \quad \text{WALLS (FIG. 30.3-1)}$$

$GC_p \approx -1.9$ ZONE 2 ROOF (FIG. 30.3-2A)

LOAD CASE A (WINDWARD PARAPET):

WINDWARD WALL SURFACE $p = \begin{cases} (40.6)(0.85)(0.9+0.18) = 37.3 \text{ psf} \\ (40.6)(0.85)(0.9-0.18) = 24.8 \text{ psf} \end{cases}$

LEEWARD WALL SURFACE $p = \begin{cases} (40.6)(0.85)(-1.9+0.18) = -59.4 \text{ psf} \\ (40.6)(0.85)(-1.9-0.18) = -71.8 \text{ psf} \end{cases}$

LOAD CASE B (LEEWARD PARAPET):

WINDWARD WALL SURFACE $p = \begin{cases} 37.3 \text{ psf} \\ 24.8 \text{ psf} \end{cases}$

LEEWARD WALL SURFACE $p = \begin{cases} (40.6)(0.85)(-1.2+0.18) = -35.2 \text{ psf} \\ (40.6)(0.85)(-1.2-0.18) = -47.6 \text{ psf} \end{cases}$

WORST CASE (ON ONE SURFACE)
71.8 psf LRFD (1.0W)
(0.6)(71.8) = 43.1 psf ASD (0.6W)

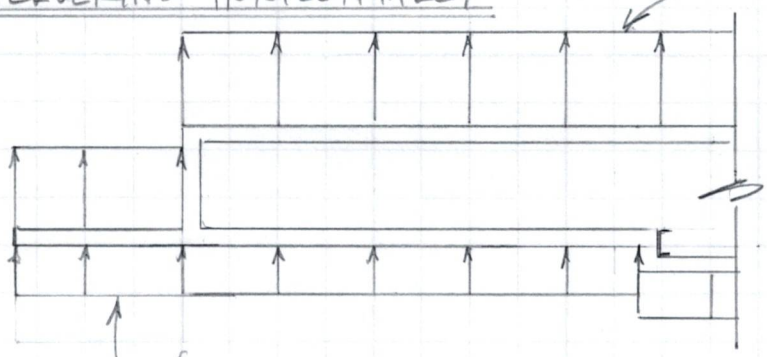
$M = (43.1)(12)^2/2 = 3100 \frac{\text{ft-lb}}{\text{ft}}$
$S_g = (12)(8)^2/6 = 128 \frac{\text{in}^3}{\text{ft}}$
$f_t = (3100)(12)/128 = 291$ psi NG

⇒ SPAN PARAPET TO NEW DIAPHRAGM @ LEVEL OF EXISTING DOOR TRACKS

CONSIDER WALL SPANNING VERTICALLY B/W ROOF & NEW DIAPHRAGM @ DOOR TRACK LEVEL:
$l = 8' \pm$
$M = (43.1)(8)^2/8 = 345 \frac{\text{ft-lb}}{\text{ft}}$
(CONSERVATIVE)
⇒ $f_t = 32.3$ psi OK

⇒ PARAPET HT = 4' ±
$M = (43.1)(4)^2/2 = 345 \frac{\text{ft-lb}}{\text{ft}}$
$f_t = (345)(12)/128 = 32.3$ psi < $F_t = 40$ psi FOR TYPE N PORTLAND CEMENT/LIME MORTAR, NORMAL TO BED JOINTS, SOLID UNITS
OK

CONSIDER FRONT PARAPET "WING WALL"
CANTILEVERING HORIZONTALLY



CASE 1: 59.4 psf
CASE 2: 71.8 psf

CASE 3: -37.3 psf
CASE 4: -24.8 psf

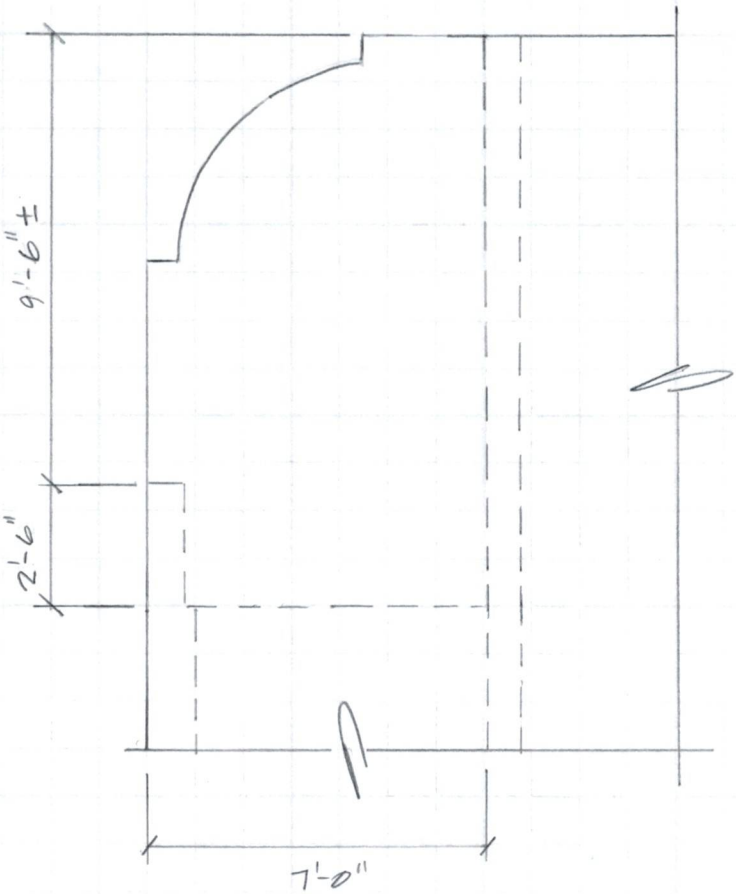
CASE 1: 37.3 psf
CASE 2: 24.8 psf

CASE 3: -35.2 psf
CASE 4: -47.6 psf

(NOTE: + & - IS BASED ON DIRECTION OF LOAD SHOWN IN FIGURE, NOT PRESSURE/SUCTION)

CASE 1 & 2: WORST CASE FOR EXTENT OF NEGATIVE MOMENT IN BACKSPAN IS CASE 2 (TENSION ON OUTSIDE FACE)

CASE 3 & 4: WORST CASE FOR EXTENT OF NEGATIVE MOMENT IN BACKSPAN IS CASE 3 (TENSION ON INSIDE FACE)



MAX. PARAPET WALL LOAD = { 96.7 psf LRFD (1.0 W)
(TOTAL PRESSURE + SUCTION) { (0.6)(96.7) = 58.0 psf ASD (0.6 W)

CANTILEVER WALL HORIZONTALLY

$\Rightarrow M = (58.0)(7)^2/2 = 1420$ ft-lb/ft

$f_t = (1420)(12)/128 = \underline{133 \text{ psi}}$ NO AS URM

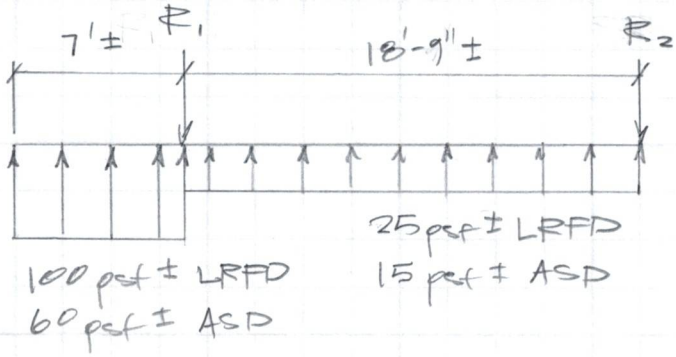
$\Rightarrow$ ADD JOINT REINFORCEMENT

$M_u = (96.7)(7)^2/2 = 2370$ ft-lb/ft

SAY $d = 6.5''$

REQ'D $A_s \approx \dfrac{(2.37)(12)}{(0.9)(75)(0.9)(6.5)} = 0.0720$ in²/ft

$0.0720/0.020 = 3.6 \Rightarrow$ (4) D2 WIRES PER FT
(WIRE IN EVERY JOINT)



STRONGBACK @ FRONT WALL

SAY 20 psf ASD WIND (0.6W)
USE 70% OF ASD WIND FOR SERVICEABILITY (0.42W)
(REDUCE FROM 700-YR MRE TO 100-YR MRE)

$(0.7)(20) = 14$ psf (0.42W)
TRIB $= 18.75/2 + 4/2 = 11.375'$
$w = (11.375)(14) = 159$ lb/ft

VERT. SPAN TO NEW DIAPHRAGM $= 24' \pm$

$\Rightarrow$ REQ'D $I \geq \dfrac{(600)(5)(159)(24)^3(144)}{(384)(29\times10^6)} = 85.3 \text{ in}^4$ FOR $\Delta \leq L/600$

TRY MC12x31 $I = 202 \text{ in}^4$

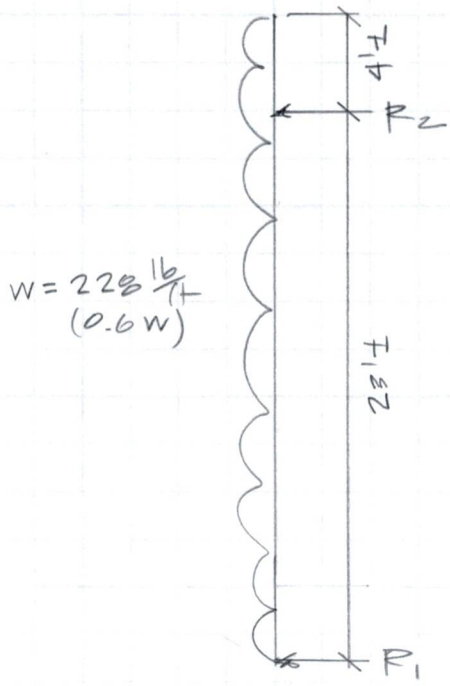
$\Delta = \dfrac{(5)(159)(24)^4(1728)}{(384)(29\times10^6)(202)} = \underline{0.203''} < L/600 = 0.480'' \approx 3/8''$ OK

20 psf ASD WIND $\Rightarrow$ 33 psf LRFD
$w_u = (11.375)(33) = 375$ lb/ft
$M_u = (0.375)(24)^2/8 = \underline{27.0 \text{ ft-k}}$

MC12x31 $\phi_b M_n (L_b = 24') = \underline{52 \text{ ft-k}} \pm (C_b = 1.0)$ OK

$w = (11.375)(20) = 228$ lb/ft (0.6W)
$R = (228)(24)/2 = \cancel{2740}$ lb

CONSIDER WALL ABV NEW DIAPHRAGM @
TRACK LEVEL ALSO SPANNING HORIZONTALLY
TO STRONGBACK



$R_2 \approx (228)(23/2 + 4)$
$= \underline{\underline{3530 \text{ lb}}}$ (0.6w)

CONNXN OF WALL TO STRONGBACK

GROUTED ANCHOR SHEAR CAPACITY = 3380 lb ASD (SEE P. 16)

ANCHORS @ 4' MAX. O.C. $\Rightarrow (228)(4) = \underline{912 \text{ lb}}$ (0.6w) OK

CONNXN OF STRONGBACK TO DIAPHRAGM

C6x10.5 STRUT $r_y = 0.529''$, $A = 3.07 \text{ in}^2$
SAY $K = 1.0$ & $L = 3.67$ ft
$KL/r_y = (1.0)(3.67)(12)/0.529 = 83$
$\Rightarrow \phi_c F_{cr} = 22.5$ ksi FOR $F_y = 36$ ksi
$\phi_c P_n = (22.5)(3.07) = \underline{69.1 \text{ k}} \gg P_u = 3.53/0.6 = 5.88$ k OK

$\frac{3}{4}$" ϕ BOLTS FROM C6 WEB ($t_w = \frac{5}{16}$")
TO (2) 2x (3" THK) HEM-FIR NAILER:

$Z'_{\parallel} \approx 1780$ lb (SEE P. 16)
$3530/1780 = 1.98 \Rightarrow$ (2) BOLTS REQ'D
(4) BOLTS PROVIDED OK

10d NAILS IN $\frac{15}{32}$" PLYWOOD SHEATHING
TO HEM-FIR NAILER:

$Z' = C_D C_{di} Z = (1.6)(1.1)(73) = 128 \frac{lb}{NAIL}$ (FOR G=0.42 MIN. PANELS)
$3530/128 = 27.6 \Rightarrow$ (28) NAILS REQ'D

(2) ROWS @ $2\frac{1}{2}$" o.c. $\Rightarrow (2)(3.67)\frac{(12)}{2.5} =$ (35) NAILS PROVIDED OK

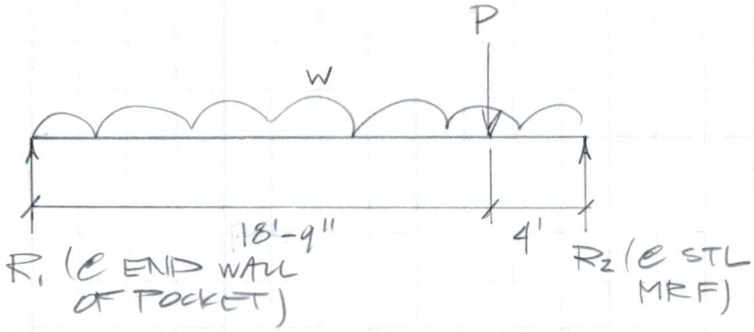
SPIKING OF DBL 2x HEM-FIR NAILER:

(2) ROWS 10d NAILS @ $2\frac{1}{2}$" o.c. OK BY INSPECTION

DIAPHRAGM @ TRACK LEVEL

CONSIDER (3) CASES:

1) 20 psf ASD WIND LOAD ON FRONT WALL ONLY
2) 20 psf ASD WIND LOAD ON REAR WALL ONLY
3) 10 psf ASD WIND LOAD ON EACH WALL



CASE 1: P = 3530 lb (0.6w) FROM STRONGBACK

w = 0

MAX. V = (3530)(18.75)/22.75 = 2910 lb = R_2

R_1 = 3530 − 2910 = 620 lb

M = (3530)(18.75)(4)/22.75 = 11600 ft-lb

DIAPHRAGM DEPTH = 5' − (2)(8") = 3'-8"

v = 2910/3.67 = 793 lb/ft

15/32" STRUCT I PLYWOOD SHEATHING ON 2x HEM-FIR FRAMING w/ 10d NAILS @ 2½" o.c.

v_{all} = (1790/2)[1 − (0.5 − 0.43)] = 832 lb/ft OK

CHORD FORCE T = 11600/3.67 = 3160 lb

$f_t = \frac{3160}{(1.5)(6)}$ = 351 psi ⇒ 1½" × 6" HEM-FIR CHORD OK

(F_t' = (1.6)(525) = 840 psi)

CASE 2: $P = \phi$

REAR WALL TFIB $= 8/2 + 4 = 8$ ft

$w = (8)(20) = 160$ lb/ft

$V = (160)(22.75)/2 = \underline{1820}$ lb $(= R_1 = R_2)$

$M = (160)(22.75)^2/8 = \underline{10400}$ ft-lb

DOESN'T CONTROL

CASE 3: $P = 3530/2 = 1770$ lb

$w = 160/2 = 80$ lb/ft

MAX. $V = (2910 + 1820)/2 = \underline{2370}$ lb

$M < (11600 + 10400)/2 = \underline{11000}$ ft-lb

DOESN'T CONTROL

CHECK DEFLECTION

CONSIDER CASE 2 (UNIFORM LOAD ONLY):

$v = 1820/3.67 = 496$ lb/ft

$G_a = (1.2)(15) = 18$ k/in FOR 4- OR 5-PLY PANELS

$E = 1.3 \times 10^6$ psi HEM-FIR #2

$A = (1.5)(6) = 9$ in^2

$$\delta = \frac{(5)(496)(22.75)^3}{(8)(1.3\times10^6)(9)(3.67)} + \frac{(0.25)(496)(22.75)}{(1000)(18)}$$

$= 0.0850 + 0.157 = \underline{0.242''} = L/1130$

IF NO CHORD SPLICES OR IF SPLICE SLIP IS NEGLIGIBLE

COULD ALSO CONSIDER 70% OF WIND LOAD (100-YR MRE) FOR SERVICEABILITY CONDITION

$\Rightarrow$ SAY <u>OK</u> BY INSPECTION FOR ALL CASES & CHORD SLIP

TRANSFER SHEAR TO STL MRF
& URM END WALL OF POCKET

R @ STL MRF = 2910 lb MAX. (CASE 1)
R @ URM WALL = 1820 lb MAX. (CASE 2)
} (0.6W)

@ STL MRF: 3/4" ϕ BOLT IN 3" THK HEM-FIR NAILER TO 3"± THK HEM-FIR FILLER IN STL BEAM:

$Z'_{\parallel} > (1.6)(870) = 1390$ lb (FOR 2½" THK MAIN MEMBER w/ 1½" THK SIDE MEMBER)

3/4" ϕ BOLT IN 3"± THK HEM-FIR FILLER TO STL BEAM WEB:

$Z'_{H} \approx (1.6)(1110) = 1780$ lb (FOR 2½" THK WOOD MAIN MEMBER w/ ¼" THK STL SIDE PL)

$\frac{2910}{1390} = 2.09 \Rightarrow$ (3) BOLTS REQ'D

(4) BOLTS PROVIDED OK

E.D. = 3" = 4D OK

@ URM WALL: 3/4" ϕ THREADED ROD IN 2½" ϕ GROUTED HOLE X 6" DEEP

ASSUME $f'_m = 750$ psi ⇒ ALLOWABLE BRG STRESS = 750/3 = 250 psi

(2.5)(6)(250) = 3750 lb/ANCHOR CAPACITY

(3) ANCHORS PROVIDED OK (FOR 1820 lb)

@ URM WALL: 3/4" ϕ BOLT IN 2x HEM-FIR LEDGER
(CONT'D)

$Z'_{\parallel} \approx (1.6)(1200) = \underline{1920 \text{ lb}}$ CONTROLS

(3) ANCHORS PROVIDED OK (FOR 1820 lb)

USE PYTHON MT SCREW ANCHOR w/ 6½" EMBED.
T_{ALL} = 1135 lb ⇒ 32" o.c.
$\left(T = \left(\frac{32}{12}\right)(8)(43.1) = 919 \text{ lb}\right)$

WALL ANCHORAGE

NEW POCKET DIAPHRAGM: WALL TRIB = 8'
ANCHORS @ ~~4'~~ o.c.
⇒ (8)(4)(43.1) = ~~1380 lb~~ ASD (0.6W)
CONTROLS

(E) LEAN-TO ROOF DIAPHRAGM: WALL TRIB = $\frac{8}{2} + \frac{15}{2}$ = 11.5'
ANCHORS @ 4' o.c.
⇒ (11.5)(4)(22.4) = 1030 lb ASD (0.7E)
CONTROLS

SIMPSON LTTP2 w/ ¾" ϕ ANCHOR ROD &
(12) #9 x 1½" SIMPSON SD SCREWS

T_{ALL} = 2045 lb FOR SPF/HF OK

4"x4" BRG PL ON MASONRY:

SAY f'm = 750 psi ⇒ ALLOWABLE BRG STRESS = $\frac{750}{3}$ = 250 psi

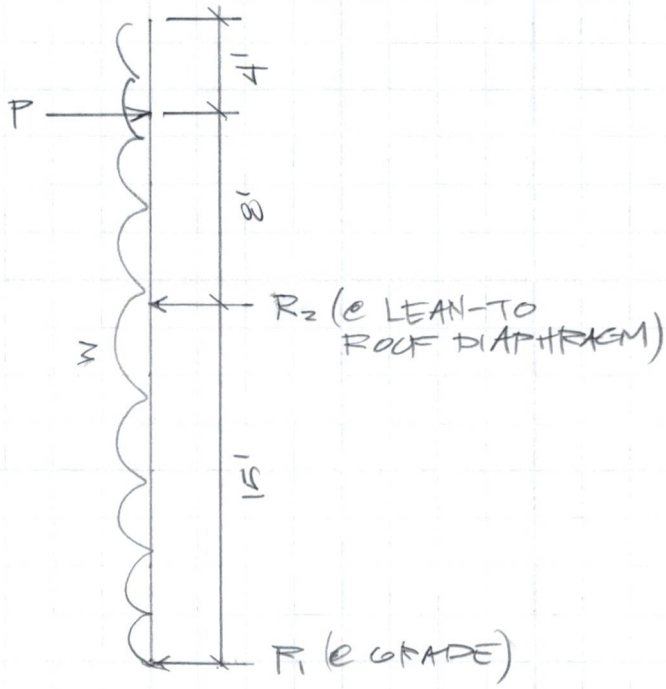
NET BRG A = $(4)^2 - \pi(2.5)^2/4$ = 11.1 in² FOR ANCHOR ROD IN 2¼" ϕ GROUTED HOLE

(11.1)(250) = 2780 lb OK

CONSERVATIVELY CHECK BRG PL SPANNING ONE-WAY:

$M_u \lesssim (1.5)\left(\frac{2.78}{2}\right)\left(\frac{4}{2}\right) = 4.17$ in-k
$\phi_b M_n = (0.9)\left[\frac{(4)(0.375)^2}{4}\right](36) = 4.56$ in-k OK TO DEVELOP FULL BRG CAPACITY ABOVE

CHECK END WALL OF POCKET
AS A URM SHEAR WALL



CASE 1: $P = 620$ lb (SEE P. 14)

WALL TRIB $= \frac{18.75}{2} + \frac{7}{2} = 12.875'$

$w = (12.875)(20) = 258$ lb/ft

$V = 620 + (258)(12) = \underline{3720 \text{ lb}}$

$f_v = \frac{(3)(3720)}{(2)(8)(5)(12)} = \underline{11.6 \text{ psi}}$ OK

$M = (620)(8) + (258)(12)^2/2 = \underline{23500 \text{ ft-lb}}$

$f_t = \frac{(6)(23500)(12)}{(8)[(5)(12)]^2} = \underline{58.8 \text{ psi}}$ (NEGLECTING RETURNS IN WALL EA. END ⇒ CONSERVATIVE)

80 psf SELF-WT $\Rightarrow f_c = \frac{(12)(80)}{(8)(12)} = 10$ psi

NET FLEXURAL TENSILE STRESS = 58.8 − 10 = 48.8 psi
> F_t = 40 psi (SEE P. 7) NG

⇒ CONSIDER RETURN = WALL THICKNESS (8")
EA. END FOR FLANGED "C" SECTION

$I = (8)(60)^3/12 + (2)\left[(8)(8)\left(\frac{60}{2} - \frac{8}{2}\right)^2 + (8)(8)^3/12\right]$
$= 231000 \text{ in}^4$
$S = 231000/(60/2) = 7700 \text{ in}^3$

$f_t = \frac{(23500)(12)}{7700} = 36.6$ psi OK

CASE 2: P = 1820 lb (SEE P. 15)
REAR WALL TRIB = 7/2 = 3.5 ft
w = (3.5)(20) = 70 lb/ft
V = 1820 + (70)(12) = 2660 lb
M = (1820)(8) + (70)(12)²/2 = 19600 ft-lb
} DOESN'T CONTROL

CASE 3: P = 620/2 + 1820/2 = 1220 lb
w = 258/2 + 70/2 = 164 lb/ft
V = 1220 + (164)(12) = 3190 lb
M = (1220)(8) + (164)(12)²/2 = 21600 ft-lb
} DOESN'T CONTROL

SHEAR TRANSFER TO LEAN-TO ROOF DIAPHRAGM

$R_2 \approx 3720 + (258)(15)/2 = 5660$ lb (0-6 W)

GROUTED 3/4" ϕ ANCHOR IN 2x HF LEDGER CAPACITY = 1920 lb ASD (SEE P. 16A)

5660/1920 = 2.95 ⇒ (3) ANCHORS REQ'D
(3) ANCHORS PROVIDED OK

VIII. Miscellaneous Design Items

DESIGN 1-WAY TRENCH COVERS FOR BOILER RM.

USE TABLES 3-18a & 3-18b IN AISC MANUAL.

LOADING: DL : PLATE SELF-WT.
LL : 100 psf

MAX. SPAN = 4'-0" CLR. FOR WIDE TRENCH.
SMALLER TRENCHES HAVE 1'-6" MAX. CLEAR SPAN.

LIMIT DEFLECTIONS TO L/180 TL, L/240 LL
(VALUES IN TABLE 3-18a ARE BASED ON L/100 DEFL.)

FOR 1'-6" SPAN: $w_{Am} = 499$ psf, OK

3/16" PL → $w_s = 8.70$ psf + $w_u = 100$ psf = 108.7 psf TL

$\Delta = L/100 = 18''/100 = 0.18''$ FOR $w_{TL} = 302$ psf

$\Delta_{LL} = \frac{100}{302}(0.18'') = 0.06'' = L/302 < L/240$ OK

TO LIMIT PIECES TO LIFTABLE WEIGHTS, TRY 75# PER PIECE:

$(8.70 \text{ psf})(1.5')L \leq 75$ psf
$L \leq 5.75'$

IF 6' EVEN, $w_{MAX} = 78.3$# OK

⇒ USE 3/16" PL, 1'-6" WIDE MAX, 6' LONG MAX.

FOR 4'-0" SPAN:

1/2" PL → $w_s = 21.5$ psf + $w_u = 100$ psf = 121.5 psf TL

$\Delta = L/100 = 48''/100 = 0.48''$ FOR $w_{TL} = 302$ psf

4'-0" SPAN, CONT'D:

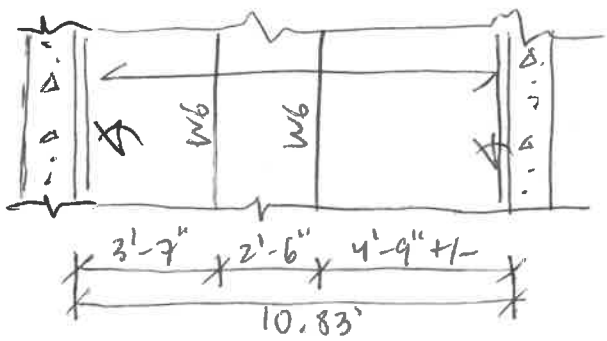
BY COMPARISON, 1/2" PL OK FOR DEFLECTION.
$w_{AM} = 499$ psf ✓ OK

LIMIT WEIGHT TO ~ 75 POUNDS

$(21.5 \text{ psf})(4') L \leq 75\#$
$L \leq 0.87'$ NOT PRACTICAL

→ PROVIDE OPTION FOR USING STEEL PLANK

DESIGN REPLACEMENT BMS & STEEL PLANK TO COVER CES PIT IN SE CORNER



PLAN VIEW

ASSUME (2) NEW W6 BMS WILL REPLACE EXISTING "BRIDGE" BEAMS.

LONGEST PLANK SPAN = 4'-9" = 57"

McNICHOLS TRACTION TREAD PLANK, 1½" THK.:
13 GA., 60" SPAN PER LOAD TABLE HAS 104 psf CAPACITY W/ 0.15" DEFLECTION. ✓ OK
w_s = 4.93 psf
(11 GA. HAS w_s = 6.90 psf)

BEAM SPAN ≅ 8'-3"
→ SAY 9'-0"

CONSERVATIVELY USE ½ TOTAL WIDTH FOR TRIB. WIDTH = 10.83'/2
= 5.42'

w_{DL} = 6.90 psf , w_u = 100 psf

M_{MAX} = (107 psf)(5.42')(9')²/8 = 5.87 K-FT

FOR F_y = 50 ksi , l_u = 9'-0" , W8x10 HAS $M_{AU} \cong$ 12 K-FT
W8x10 HAS S_x = 7.81 in³ , S_y = 1.06 in³
W6x16 HAS S_x = 10.2 in³ , S_y = 2.20 in³ SO OK BY COMPARISON

$$\Delta_u = \frac{(5)(100\,psf)(5.42')(9')^4(12^3)/1000}{384\,(29{,}000\,ksi)(32.1\,in^4)} = 0.086" = L/1256$$

OK

→ REDUCE TO W6x12 SINCE LOADS CONSERV. & OVERSIZED.

$$\Delta_u = \frac{32.1}{22.1}(0.086") = 0.125" = L/864$$

OK

PRIM MATL	GAUGE/ THICK	DEPTH (mm)	LBS./LF (kg/m)	LOAD/ DEFL	CLEAR SPAN																
					24"	30"	36"	42"	48"	54"	60"	66"	72"	78"	84"	90"	96"	108"	120"	132"	144"
ALUMINUM	.1250"	1-1/2" (38.1)	2.37 (3.52)	U	238	195	141	101	74	58	46	42	37	27	20	--	--	--	--	--	--
				D	0.02	0.04	0.06	0.08	0.10	0.13	0.15	0.20	0.25	0.25	0.25	--	--	--	--	--	--
		2" (50.8)	2.52 (3.75)	U	487	399	288	207	152	119	93	85	75	55	41	31	24	--	--	--	--
				D	0.02	0.04	0.06	0.08	0.10	0.13	0.15	0.20	0.25	0.25	0.25	0.25	0.25	--	--	--	--
CARBON & GALVANIZED STEEL	13	1-1/2" (38.1)	4.93 (7.33)	U	544	445	322	232	170	133	104	95	84	61	45	--	--	--	--	--	--
				D	0.02	0.04	0.06	0.08	0.10	0.13	0.15	0.20	0.25	0.25	0.25	--	--	--	--	--	--
		2" (50.8)	5.24 (7.80)	U	1101	902	652	469	344	268	211	192	170	123	92	70	54	--	--	--	--
				D	0.02	0.04	0.06	0.08	0.10	0.13	0.15	0.20	0.25	0.25	0.25	0.25	0.25	--	--	--	--
		3" (76.2)	5.57 (8.29)	U	2986	2446	1769	1273	933	728	573	522	461	335	249	189	146	91	60	41	29
				D	0.02	0.04	0.06	0.08	0.10	0.13	0.15	0.20	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25
	11	1-1/2" (38.1)	6.90 (10.3)	U	682	559	404	291	213	166	131	119	105	76	57	--	--	--	--	--	--
				D	0.02	0.04	0.06	0.08	0.10	0.13	0.15	0.20	0.25	0.25	0.25	--	--	--	--	--	--
		2" (50.8)	7.30 (10.9)	U	1395	1143	827	595	436	340	268	244	215	156	116	88	68	--	--	--	--
				D	0.02	0.04	0.06	0.08	0.10	0.13	0.15	0.20	0.25	0.25	0.25	0.25	0.25	--	--	--	--
		3" (76.2)	8.23 (12.3)	U	3831	3138	2270	1634	1197	934	736	670	591	429	319	242	187	117	77	52	37
				D	0.02	0.04	0.06	0.08	0.10	0.13	0.15	0.20	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25

U - Uniform Load - Lbs./Sf. **D** - Deflection in Inches | Span and loading values to the left of the bolded black line produce a deflection of 1/4" or less under a uniform load of 100 Lbs./Sf., allowing for safe pedestrian comfort. Span and loading values to the right of the bolded black line are applicable to other types of loads at the discretion of a licensed engineer. Technical information provided is theoretical and for evaluation by technically skilled persons, with any use thereof to be at their independent discretion and risk. **McNICHOLS** shall have no responsibility or liability for results obtained or damages resulting from improper evaluation or use of Plank Grating.

ADA compliant surface when installed using multiple pieces to width regardless of the direction of travel.

MATTEO | FERRAN
STRUCTURAL ENGINEERS

210 N Lee Street, Ste 210
Alexandria, VA 22314
703.350.4151

Bearing Plate on Concrete

Overal Status: **PASS**

Code: AISC 360-16 (ASD)

Plate Thickness= **1/4"**

Project:	**NPS GATE Floyd Bennett Field Hangars 3&4**
Project No.:	**23-086.1**
Description:	**Boiler Room Beams at Pit Infill**

			k =	0.530	in
Reaction, R_a =	**2.61**	kips (ASD)	d =	6.030	in
f'_c =	**3**	ksi (Assumed)	t_w =	0.230	in
Beam Size =	W6X12		t_f =	0.280	in
Bearing Plate Size:			$f_{y,\ beam}$ =	50	ksi
N (length) =	**4**	in	$f_{y,\ plate}$ =	36	ksi
B (width) =	**6**	in	E =	29	x 10^3 ksi

Check Web Local Yielding (Ω = 1.50) *[AISC - J10.2]*

$R_n = (2.5\ k + N)\ F_{yw}\ t_w$

R_n = 61.2 kips

PASS R_n / Ω = 40.8 kips

Check Web Crippling (Ω = 2.00) *[AISC - J10.3]*

For $N/d \leq 0.2$

$$R_n = 0.40 t_w^2 \left[1 + 3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{EF_{yw}t_f}{t_w}} \qquad \text{(J10-5a)}$$

For $N/d > 0.2$

$$R_n = 0.40 t_w^2 \left[1 + \left(\frac{4N}{d} - 0.2\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{EF_{yw}t_f}{t_w}} \qquad \text{(J10-5b)}$$

N / d = 0.66 ==> Equation J10-5b is applicable

R_n = 79.5 kips

PASS R_n / Ω = 39.7 kips

Check Bearing on Concrete (Ω = 2.31) *[AISC - J8]*

$R_n = P_p = 0.85\ f'_c\ A_1$ A1 = N x B = 24.0 in^2

R_n = 61.2 kips

PASS R_n / Ω = 26.5 kips

Design Plate Thickness (Ω = 1.67) *[AISC - F11]*

w_p = Ra / (B x N) = 0.109 ksi (uniform pressure under plate)

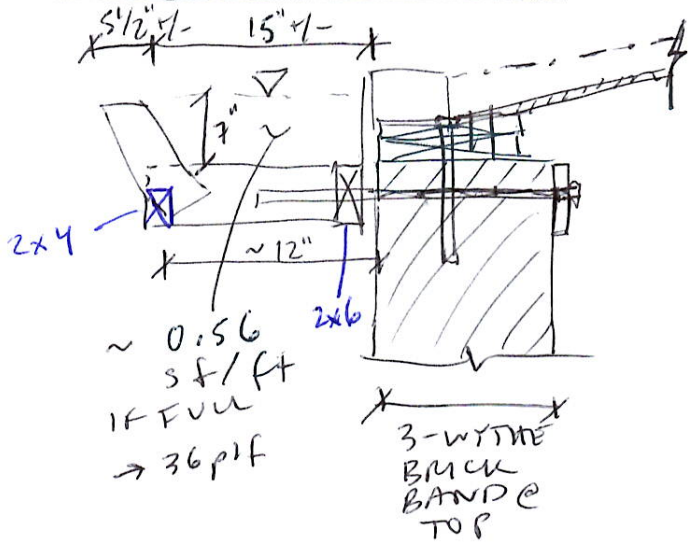
n = B/2 - k = 2.47 in (bearing plate cantilever dimension)

$M_a = n^2 / 2 * w_p$ = 0.33 k-in / in

$t = (6.68\ M_a / F_y)^{1/2}$ = 0.248 in (solved $\Omega\ M_a = F_y Z$ for 't' with Ω = 1.67)

Plate Thickness = 1/4" in

GUTTER ATTACHMENT



ASSUME MISC. FRAMING & GUTTER MAT'L ≈

2x FRAMING → 3 psf
2 LAYERS 3/4" PLY → 5 psf
COPPER LINER → 2 psf
COPPER GUTTER → 2 psf
12 psf

→ SAY 15 psf TO ALLOW FOR MISC. HARDWARE, ETC.

$(15\ psf)(20.5''/12) = 25.6\ plf$

TOTAL LOAD:

$$w_{TOT} = 36 + 26\ plf = 62\ plf$$

$$M_{MAX} = (36\ plf)\left(\frac{(10'' + 13'')/2}{2}\right) + (26\ plf)(20.5''/2)$$

$$= (36\ plf)\left(\frac{5.75''}{12}\right) + (26\ plf)\left(\frac{10.25''}{12}\right)$$

$$= (17.25 + 22.21)\ \#\text{-FT/FT}$$

$$= 39.5\ \#\text{-FT/FT}$$

IF GUTTER STRUCTURE IS LOWERED AND SIMPSON LTTP2 IS INSTALLED @ TOP FACE OF 2x6 OUTRIGGER & ANCHORED THROUGH BRICK:

$T_{ALL} = 1{,}695\#$ w/ (15) 0.148 x 2½ FASTENERS INSTALLED IN NARROW FACE

MOMENT ARM = 5½", SO $M_{ALL} = (1{,}695\#)(5\tfrac{1}{2}'') = 776\#\text{-FT}$

MAX. SPACING FOR TENSION TIE: 776#-FT / 39.5#-FT/FT = 19.6'

IF TENSION STRAP IS FASTENED TO SIDE OF JOIST, MOMENT ARM ≈ 3½". TRY DTT1Z TIE (ONLY 7" LONG)

$M_{AH} = (840\#)(3\tfrac{1}{2}") = 2{,}940\ \#\text{-IN} = 245\ \#\text{-FT}$

$S_{MAX} = 245\#\text{-FT} / 39.5\#\text{-FT/FT} = 6.2'$

INSTALL CONTINUOUS MEMBER @ OUTER EDGE TO SPAN BTW. OUTRIGGERS:

USE 2x4 & BIRDSMOUTH DIAG. EDGE MEMBER OVER IT.

ASSUME HEM-FIR #2: $F_b = 850$ psi, $F_v = 150$ psi, $E = 1.3 \times 10^6$ psi, S.G. = 0.43

$A = 5.25\ in^2$
$S = 3.06\ in^3$
$I = 5.36\ in^4$

½ OF 62 plf LD GOES TO LEDGER, OTHER ½ TO 2x4:
$w = 31$ plf

$$\frac{M}{S} = \frac{(31\ plf)\,L^2\,(12''/')}{(8)(3.06\ in^3)} \le 850\ psi$$

$$L \le 7.48'$$

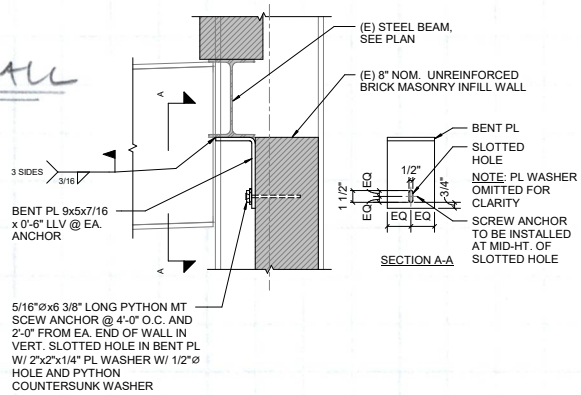
IF JOISTS SPACED @ 2'-0" O.C., EVERY 3 JOISTS (6'-0")
IF JOISTS SPACED @ 16" O.C., @ 6'-8", EVERY 5 JOISTS

CHECK IF 2x6 CAN SUPPORT END RXN @ 2" NOTCHED END:

$$V_r' = \frac{2}{3} F_v' b d_n \left[\frac{d_n}{d}\right]^2 = \frac{2}{3}(150\ psi)(1.5")(2")\left(\frac{2"}{5.5"}\right)^2$$
$$= 39.7\ \#$$

$V = (2')(62\ plf)/2 = 62\# > V_r'$ ⇒ NEED TO ADD SIMPSON ANGLE OR TOENAILS

OUT-OF-PLANE BRACING OF URM INFILL WALL



MIN. $F_p = 0.2 k_a W_p$ $k_a = 2.0$

$F_p = (0.2)(2.0) W_p = 0.4 W_p$

$W_p = 80$ psf 8" BRICK MASONRY

ANCHORAGE SPACING = 4' MAX. PER ASCE 7 12.11.2.1
WALL HT = 13'-9" ±

$\Rightarrow F_p = \begin{cases} (0.4)(4)(13.75/2)(80) = 880 \text{ lb} & \text{LRFD } (1.0E) \\ (0.7)(880) = 616 \text{ lb} & \text{ASD } (0.7E) \end{cases}$

~~4" x 4" BRG AREA~~ $\Rightarrow$ ~~$\frac{616}{(4)(4)}$ = 38.5 psi BRG ON URM OK~~

BENDING OF 7/16" THK BENT PL x 6" WIDE:

$M_u = (0.880)(\cancel{4}\ 7.5) = \cancel{3.52}\ 6.60$ k-in

$\phi M_n = (0.9)[(6)(\cancel{0.375}\ 0.4375)^2/4](36) = \cancel{6.83}\ 9.30$ k-in OK

$\Delta = \dfrac{(0.616)(\cancel{4}\ 7.5)^3}{(3)(29 \times 10^3)[(6)(\cancel{0.375}\ 0.4375)^3/12]} = \cancel{0.0172"}\ 0.0713"$ OK

PYTHON MT SCREW ANCHOR w/ 6 1/8" MIN.
EDGE DISTANCE:
6 3/8" LENGTH − 1/2" BENT PL − 1/4" PL WASHER = 5.625" > 4" OK
ALLOWABLE TENSION ≥ (0.6)(1060) = 636 lb > 616 lb OK
(FOR 4.5" EMBEDMENT w/ REDUCTION FACTOR
PER ESR-5183 FOR < 16" EDGE DISTANCE)

APPENDIX A: ICC-ES Evaluation Reports

- ESR-1472 for Simpson Strong-Tie WSV Screws
- ESR-4144 for Hilti HIT-HY 270 Adhesive Anchor System in Unreinforced Masonry
- ESR=5183 for Python C and Python MT URM Screw Anchors for Use with Brick and Ungrouted Concrete Block (CMU)

ICC-ES Evaluation Report

ESR-1472

Reissued September 2024

Revised May 2025

Subject to renewal September 2025

This report also contains:

- City of LA Supplement

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DIVISION: 06 00 00 - WOOD, PLASTICS AND COMPOSITES Section: 06 05 23—Wood, Plastic, and Composite Fastenings Section: 06 05 23.20—Screws	REPORT HOLDER: SIMPSON STRONG-TIE COMPANY INC.	EVALUATION SUBJECT: SIMPSON STRONG-TIE® WSV SCREWS	

1.0 EVALUATION SCOPE

Compliance with the following codes:

- 2024, 2021, 2018, 2015, and 2012 *International Building Code®* (IBC)
- 2024, 2021, 2018, 2015, and 2012 *International Residential Code®* (IRC)

Property evaluated:

- Structural

2.0 USES

The Simpson Strong-Tie® WSV screws are used in engineered horizontal wood structural panel (WSP) diaphragms; in engineered diagonally sheathed lumber diaphragms; for prescriptive attachment of WSP to wood framing; and for prescriptive connections described in this report. WSV screws are also used in wood-to-wood connections that are designed in accordance with the IBC. The WSV screws may be used under the IRC when an engineered design is submitted in accordance with IRC Section R301.1.3.

3.0 DESCRIPTION

3.1 Notation and Symbols:

D	=	Outside thread diameter
D_H	=	Diameter of fastener head or integral washer
D_r	=	Minor thread (root) diameter
D_s	=	Unthreaded shank diameter
$F_{yb,spec}$	=	Minimum specified bending yield strength, determined in accordance with ASTM F1575 using D_r.
L	=	Fastener length measured from top of head to tip
$L_{emb,w}$	=	Minimum required embedded thread length in holding member, including tip, applicable to tabulated withdrawal design values

L_m	=	Dowel bearing length in the main member
L_{thread}	=	Length of thread including tip
L_{tip}	=	Length of tip
N_a	=	Allowable tension strength of the fastener for use in ASD
N_u	=	Design tension strength of the fastener, for use in LRFD
SG_{eq}	=	Equivalent specific gravity for structural composite lumber, as reported by ICC-ES
SG_{NDS}	=	Assigned specific gravity (See Section 3.3.)
$t_{s,w}$	=	Thickness of wood side member
V_a	=	Allowable shear strength of the fastener for use in ASD
V_u	=	Design shear strength of the fastener, for use in LRFD
W	=	Reference unit withdrawal design value for fasteners installed perpendicular to face of the wood
W_H	=	Reference head pull-through design value
Z	=	Reference lateral design value

3.2 WSV Screws:

The screws are proprietary No. 9 screws. See Table 1A for dimensions and other descriptions of the WSV screws. The screws have a countersunk flat head with four equally spaced nibs and a T25 drive recess and are partially threaded with a hi-low thread pattern of 5.5 and 11.5 threads per inch. The screws have either a yellow zinc coating or a proprietary coating, as shown in Table 1A. See Figure 1 for a depiction of a typical WSV screw.

3.3 Wood Members:

For engineered wood-to-wood connections using WSV screws designed in accordance with Section 4.1.3, wood main members may be either sawn lumber or engineered wood (e.g. LVL, PSL, LSL). Wood side members may be either solid-sawn lumber, wood structural panel (OSB or plywood) or engineered wood. Engineered wood must have a minimum E value of 1.3E for lateral loading and 1.55E for withdrawal loading. The engineered wood must be addressed in an ICC-ES evaluation report and must have $SG_{eq} \geq 0.50$.

For purposes of connection design, sawn lumber, glulam and CLT members must have SG_{NDS} as indicated in the tables in this report and the moisture content must be less than or equal to 19 percent at the time of screw installation and while in service. SG_{NDS} for sawn lumber is the assigned specific gravity for the applicable grade mark, which must be determined in accordance with Table 12.3.3A of the ANSI/AWC National Design Specification for Wood Construction® (NDS) or the latest NDS Supplement (Table 11.3.3A of the 2012 NDS for the 2012 IBC). SG_{NDS} for glulam members is the Specific Gravity for Fastener Design addressed in Tables 5A through 5D of the NDS Supplement. When designing connections with screws installed into CLT panels, all of the laminations must have a minimum SG_{NDS} as indicated in the tables in this report.

Wood structural panels must comply with DOC PS1 or PS2 and the grade shown in this report. For purposes of lateral connection design, wood structural panels have an assigned specific gravity of 0.42 or 0.50 in accordance with Table 12.3.3B of the NDS (Table 11.3.3B of the 2012 NDS for the 2012 IBC).

For engineered wood, the moisture content at the time of screw installation and in service must be in accordance with the applicable ICC-ES evaluation report on the engineered wood product.

For wood-to-wood connections, the tabulated side member thickness is an absolute value (not a minimum or maximum value). The thickness of the wood main member must be equal to or greater than the screw length less the thickness of the side member.

Where the WSV screws are used as substitutes for fasteners prescribed in the code, the wood members must comply with the code. Wood members used in engineered diaphragms must comply with the 2021 AWC Special Design Provisions for Wind and Seismic (SDPWS) for the 2024 and 2021 IBC (SDPWS-15 for the 2018 and 2015 IBC, SDPWS-08 for the 2012 IBC).

4.0 DESIGN AND INSTALLATION

4.1 Design:

4.1.1 Diaphragm Design - General: Framing members and wood structural panel sheathing must be selected based on the spacing of the framing and the anticipated loads in accordance with the applicable code. Diaphragms described in this report have been evaluated for use in all Seismic Design Categories.

4.1.1.1 Engineered Horizontal WSP Diaphragms: The WSV screws may be used to attach WSP sheathing to sawn lumber framing to create floor diaphragms as shown in Table 2. WSV screws may also be used to attach WSP sheathing to sawn lumber framing to create high load diaphragms as shown in Table 3. Refer to Table 2 and 3 for the required screw penetration. Allowable shear values for horizontal diaphragms consisting of WSP attached to Douglas fir–larch or Southern pine lumber framing are shown in Table 2 and 3 based on the values for 8d and 10d common nails in Tables 4.2A, 4.2B and 4.2C of SDPWS. Allowable shear values for other wood species must be determined in accordance with the footnotes to each table. The allowable shear capacities shown in Table 2 and are permitted to be increased by 40 percent for wind design. Diaphragm deflections must be calculated in accordance with SDPWS-21 Section 4.2.3 (SDPWS-15 and SDPWS-08 Section 4.2.2), using the applicable G_a value given in Tables 4.2A, 4.2B and 4.2C of SDPWS.

4.1.1.2 Engineered Diaphragms Diagonally Sheathed with One Layer of Lumber: Minimum 2-inch-long WSV screws may be substituted for the 8d common nails prescribed in SDPWS Table 4.2D. Diaphragm deflections must be calculated in accordance with SDPWS-21 Section 4.3.2 (SDPWS-15 Section 4.2.2), using the applicable G_a value given in SDPWS Table 4.2D.

4.1.2 Prescriptive Fastening:

4.1.2.1 Prescriptive Attachment of Sheathing: Minimum 2-inch-long WSV screws may be used as substitutes for the 8d and 10d common nails prescribed in Items 30 through 32 and 35 to 37 of 2024 and 2021 IBC Table 2304.10.2 (Items 30 through 32 and 35 through 37 of 2018 IBC Table 2304.10.1, Items 31 through 33 and 36 through 38 of 2015 IBC Table 2304.10.1; similar in earlier code editions) and in Items 31 through 33 and 38 through 40 of 2024 and 2021 IRC Table R602.3(1) [Items 30 through 32 and 37 through 39 of 2018 and 2015 IRC Table R602.3(1), similar in earlier code editions] for the attachment of subflooring to framing; provided the penetration into the framing members is a minimum of $1^3/_8$ inches (35 mm).

4.1.2.2 Prescriptive Framing Connections: Prescriptive fastening requirements for select framing connections, which are alternatives to what is shown in 2024 and 2021 IBC Table 2304.10.2 (2018 and 2015 IBC Table 2304.10.1, similar for IRC and earlier code editions), are described in Table 4.

4.1.3 Wood-to-wood Connection Design:

4.1.3.1 General: The design values in this report are intended to aid the designer in meeting the requirements of IBC Section 1604.2. For connections not completely described in this report, determination of the suitability of the WSV screws for the specific application is the responsibility of the designer and is outside the scope of this report. The designer is responsible for determining the available strengths for the connection, considering all applicable limit states, and for considering serviceability issues.

4.1.3.2 Screw Strength: Allowable screw shear and tension strengths (ASD), design screw shear and tension strengths (LRFD) and specified bending yield strength for the WSV screws are shown in Table 1B.

4.1.3.3 Reference Withdrawal and Pull-through Design Values: Reference withdrawal (W) design values for screws installed perpendicular to the face of the wood member and reference head pull-through values (W_H) are shown in Table 6.

4.1.3.4 Lateral Connections in Accordance with the NDS: Select reference lateral design values based on calculation in accordance with the NDS are shown in Table 5. For other connection configurations, the reference lateral design strength for connections made with WSV screws may be determined in accordance with the NDS and the following:

1. $F_{yb,spec}$ from Table 1B must be used for design.
2. D_r must be used where 'D' is referenced in Tables 12.3.1A, 12.3.1B and 12.3.3 of the NDS (Tables 11.3.1B, 11.3.3 and 11.3.1 A of the 2012 NDS for the 2012 IBC).
3. Wood must have SG_{NDS} of 0.55 or less, in accordance with Table 12.3.3A of the NDS (Table 11.3.3A of the 2012 NDS for the 2012 IBC).
4. The side member thickness must be a minimum of $^{19}/_{32}$ inch (15.1 mm).
5. The penetration into the main member must be a minimum of 1 inch (25.4 mm).
6. The dowel bearing length must be determined in accordance with Section 12.3.5.3 of the NDS, using L_{tip} shown in Table 1A.
7. Spacing, edge and end distance must be in accordance with Table 7, and as needed to prevent splitting of the wood.

4.1.3.5 Adjustments to Reference Design Values: Reference design values in the report must be adjusted in accordance with the NDS provisions for dowel-type fasteners to determine the allowable strengths for use with ASD and the design strengths for use with LRFD.

4.1.3.6 Connections with Multiple Screws: See Sections 11.1.2, 11.2.2 and 12.6 of the NDS (Sections 10.1.2, 10.2.2 and 11.6 of the NDS for the 2012 IBC) regarding multiple fastener connections and consideration of local stresses in the wood members.

4.1.3.7 Combined Loading: Where the WSV screws are subjected to combined lateral and withdrawal loads, connections shall be designed in accordance with Section 12.4.1 of the NDS (Section 11.4.1 of the NDS for the 2012 IBC), as applicable.

4.2 Installation:

The screws must be installed in accordance with Section 12.1.5 of the NDS (Section 11.1.5 of the NDS for the 2012 IBC), with the exception that no predrilling is required. Predrilled holes up to $0.70D_r$ may be used to reduce the likelihood of splitting.

The WSV screws used in diaphragms must be installed with minimum panel edge and end distances of $^3/_8$ inch (9.5 mm). The use of adhesive between the connected members is not required.

The WSV screws used in wood-to-wood connections must be installed perpendicular to the face of the wood side member. The top of the screw head must be flush with the surface of the wood side member. Screws must not be overdriven. Minimum connection geometries must comply with Table 7, or be as needed to prevent splitting of the wood. The screws must be installed by turning with a power driver, not by driving with a hammer.

4.3 Special Inspection:

When special inspections are required by 2024 and 2021 IBC Section 1705.12 (2018 and 2015 IBC Section 1705.11, 2012 IBC Section 1705.10), periodic special inspection of the installation of WSV screws in diaphragms is required when the screw spacing is 4 inches (102 mm) or less, in accordance with 2024 and 2021 IBC Section 1705.12.1 (2018 and 2015 IBC Section 1705.11.1, 2012 IBC Section 1705.10.2).

When special inspections are required by 2024 and 2021 IBC Section 1705.13 (2018 and 2015 IBC Section 1705.12, 2012 IBC Section 1705.11), periodic special inspection of installation of the WSV screws in diaphragms installed in Seismic Design Category C, D, E or F is required when the screw spacing is 4 inches (102 mm) or less, in accordance with 2024 and 2021 IBC Section 1705.13.2 (2018 and 2015 IBC Section 1705.12.2, 2012 IBC Section 1705.11.2).

Special inspection of the construction of high-load diaphragms is required in accordance with IBC Section 1705.5.1.

5.0 CONDITIONS OF USE

The Simpson Strong-Tie WSV wood screws described in this report comply with, or are suitable alternatives to what is specified in, those codes listed in Section 1.0 of this report, subject to the following conditions:

5.1 Screws must be installed in accordance with Simpson Strong-Tie's installation instructions and this report. In the case of conflict between this report and the installation instructions, this report governs.

5.2 Shear loads applied to horizontal wood structural panel diaphragms must be less than or equal to the allowable loads shown in Table 2 and 3, as applicable.

5.3 Shear loads applied to diagonally sheathed lumber diaphragms must be less than or equal to the applicable allowable loads indicated in SDPWS.

5.4 Use of the screws is limited to dry-service conditions where the moisture content of the wood members complies with Section 3.3.

5.5 The screws are manufactured under a quality control program with inspections by ICC-ES.

6.0 EVIDENCE SUBMITTED

6.1 Data in accordance with the ICC-ES Acceptance Criteria for Dowel-type Threaded Fasteners Used in Wood (AC233), dated June 2023 (editorially revised June 2024).

6.2 Data in accordance with the ICC-ES Acceptance Criteria for Wood-frame Horizontal Diaphragms, Vertical Shear Walls and Braced Walls with Alternative Fasteners (AC120), dated February 2017 (editorially revised May 2024).

7.0 IDENTIFICATION

7.1 The ICC-ES mark of conformity, electronic labeling, or the evaluation report number (ICC-ES ESR-1472) along with the name, registered trademark, or registered logo of the report holder must be included in the product label.

7.2 In addition, packages of screws bear a label with the product designation and size, and each screw is identified on the head by "≠" (the "no equal" sign) and a letter designating the screw length.

7.3 The report holder's contact information is the following:

SIMPSON STRONG-TIE COMPANY INC.
5956 WEST LAS POSITAS BOULEVARD
PLEASANTON, CALIFORNIA 94588
(800) 999-5099
www.strongtie.com

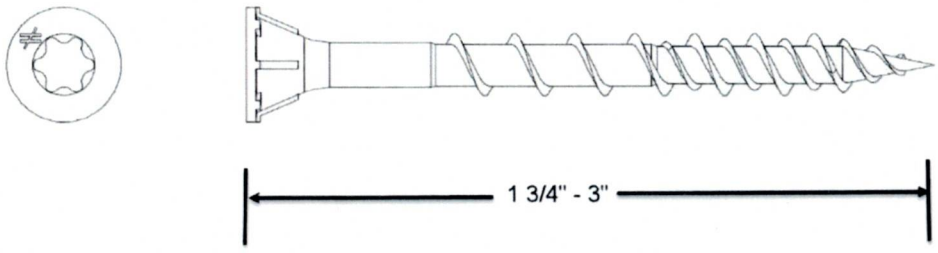

FIGURE 1—WSV SCREW

TABLE 1A—WSV SCREW SPECIFICATIONS

FASTENER DESIGNATION	COATING	SCREW DIMENSIONS (inches)								
		L	L_{thread}[1]	Head Mark	D_r	D_s	Outside Thread Diameters		D_H	L_{tip}
							High Thread, D	Low Thread		
WSV134	Zinc	1.75	1.20	K	0.121 / 0.109[(2)]	0.132	0.182	0.151	0.333	0.268
WSV200	Zinc	2.00	1.45	M						
WSVF200	Proprietary									
WSV212	Zinc	2.50	1.95	Q						
WSVF212	Proprietary									
WSV300	Zinc	3.00	2.17	U						
WSVF300	Proprietary									

For **SI:** 1 inch = 25.4 mm.

[1] Length of thread includes tip.

[2] This screw has a stepped shank design, as shown in Figure 1. For lateral connection design in accordance with Section 4.1.3.4, use a value of 0.113 inch for 'D' described in Section 4.1.3.4 Item 2.

TABLE 1B—WSV SCREW STRENGTHS

FASTENER DESIGNATION	$F_{yb,spec}$ (psi)[1]	ALLOWABLE SCREW STRENGTH (ASD)		DESIGN SCREW STRENGTH (LRFD)	
		N_a (lbf)	V_a (lbf)	N_u (lbf)	V_u (lbf)
WSV134	200,000	710	460	1,065	690
WSV200					
WSVF200					
WSV212					
WSVF212					
WSV300					
WSVF300					

For **SI:** 1 psi = 6.89 kPa, 1 lbf = 4.45 N.

[1]Bending yield strength determined in accordance with ASTM F1575 using the minor thread (root) diameter, D_r.

TABLE 2—ALLOWABLE SHEAR (POUNDS PER FOOT) FOR WOOD STRUCTURAL PANEL DIAPHRAGMS UTILIZING WSV SCREWS WITH FRAMING OF DOUGLAS FIR–LARCH OR SOUTHERN PINE FOR WIND OR SEISMIC LOADING[1,2,3,4,5]

PANEL GRADE	MINIMUM WSV SCREW PENETRATION IN FRAMING (inches)	MINIMUM NOMINAL PANEL THICKNESS (inches)	MINIMUM NOMINAL WIDTH OF FRAMING MEMBERS AT ADJOINING PANEL EDGES AND BOUNDARIES (inches)	BLOCKED DIAPHRAGMS				UNBLOCKED DIAPHRAGMS	
				Screw spacing (inches) at diaphragm boundaries (all cases), at continuous panel edges parallel to load (Cases 3 and 4), and at all edges (Cases 5 and 6)				Screws spaced 6 inches maximum, at support edges	
				6	4	$2^1/_2$	2	Case 1 (no unblocked edges or continuous joints parallel to load)	All other configurations (Cases 2, 3, 4, 5, and 6)
				Screw spacing (inches) at other panel edges					
				6	6	4	3		
Structural I	$1^1/_4$	$^3/_8$	2	270	360	530	600	240	180
			3	300	400	600	675	265	200
		$^{15}/_{32}$	2	320	425	640	730	285	215
			3	360	480	720	820	320	240
Sheathing and single floor		$^3/_8$	2	240	320	480	545	215	160
			3	270	360	540	610	240	180
		$^7/_{16}$	2	255	340	505	575	230	170
			3	285	380	570	645	255	190
		$^{15}/_{32}$	2	290	385	575	655	255	190
			3	325	430	650	735	290	215
		$^{19}/_{32}$	2	320	425	640	730	285	215
			3	360	480	720	820	320	240

For **SI:** 1 inch = 25.4 mm, 1 pound per foot = 14.6 N/m.

[1]For framing of other species, the allowable diaphragm shear capacity is found by: (1) Determining the specific gravity for the applicable species of lumber in the NDS. (2) Finding the allowable diaphragm shear value from the table above and multiplying this value by the Specific Gravity Adjustment Factor = [1-(0.5-SG)], where SG = Specific Gravity of the framing lumber. This adjustment factor must not be greater than 1.0.

[2]For shear loads of normal or permanent load duration as defined by the NDS, the values in the table above must be multiplied by 0.63 or 0.56, respectively.

[3]Shear loads can be increased 40% for wind loading.

[4]Diaphragm construction must be in accordance with Sections 4.2.7 and 4.2.8 of SDPWS-21 (Sections 4.2.6 and 4.2.7 of SDPWS-15 for the 2018 and 2015 IBC and of SDPWS-08 for the 2012 IBC), as applicable.

[5]See Figure 2 for Case diagrams.

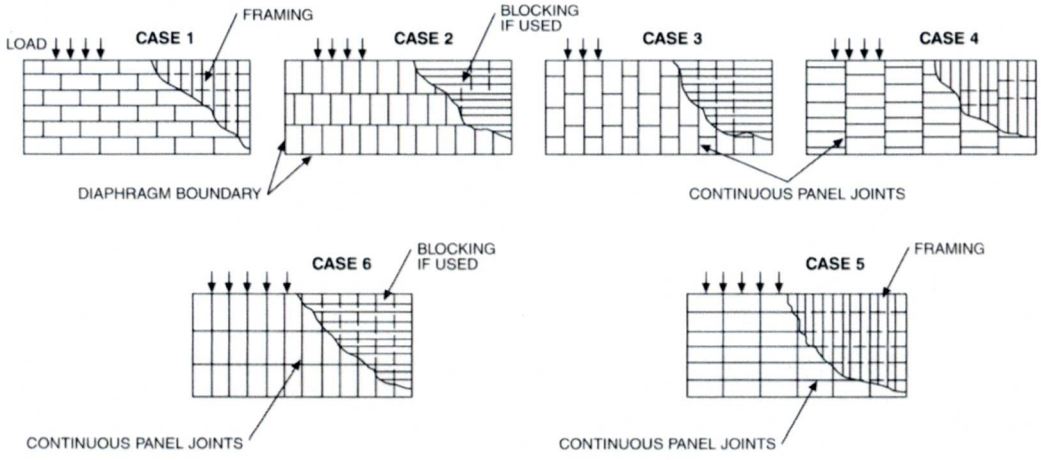

FIGURE 2—DIAPHRAGM CASES

TABLE 3—ALLOWABLE SHEAR (POUNDS PER FOOT) FOR WOOD STRUCTURAL PANEL BLOCKED DIAPHRAGMS UTILIZING MULTIPLE ROWS OF WSV SCREWS (HIGH LOAD DIAPHRAGMS) WITH FRAMING OF DOUGLAS FIR–LARCH OR SOUTHERN PINE FOR WIND OR SEISMIC LOADING[1,2,3,4]

PANEL GRADE	MINIMUM WSV SCREW PENETRATION IN FRAMING (inches)	MINIMUM NOMINAL PANEL THICKNESS (inches)	MINIMUM NOMINAL WIDTH OF FRAMING MEMBERS AT ADJOINING PANEL EDGES AND BOUNDARIES (inches)	LINES OF SCREWS	BLOCKED DIAPHRAGMS			
					Cases 1 and 2[5,6]			
					Screw Spacing Per Line at Boundaries (inches)			
					4		$2^1/_2$	
					Screw Spacing Per Line at Other Panel Edges (inches)			
					6	4	4	3
Structural I	$1^1/_4$	$^{15}/_{32}$	3	2	605	815	875	1,150
			4	2	700	915	1,005	1,290
			4	3	875	1,220	1,285	1,395
		$^{19}/_{32}$	3	2	670	880	965	1,255
			4	2	780	990	1,110	1,440
			4	3	965	1,320	1,405	1,790
		$^{23}/_{32}$	3	2	730	955	1,050	1,365
			4	2	855	1,070	1,210	1,565
			4	3	1,050	1,430	1,525	1,800
Sheathing and single floor	$1^1/_4$	$^{15}/_{32}$	3	2	525	725	765	1,010
			4	2	605	815	875	1,105
			4	3	765	1,085	1,130	1,195
		$^{19}/_{32}$	3	2	650	860	935	1,225
			4	2	755	965	1,080	1,370
			4	3	935	1,290	1,365	1,485
		$^{23}/_{32}$	3	2	710	935	1,020	1,335
			4	2	825	1,050	1,175	1,445
			4	3	1,020	1,400	1,480	1,565

For **SI:** 1 inch = 25.4 mm, 1 pound per foot = 14.6 N/m.

[1]For framing of other species, the allowable diaphragm shear capacity is found by: (1) Determining the specific gravity for applicable species of lumber in the NDS. (2) Finding the allowable diaphragm shear value from the table above multiplying this value by the Specific Gravity Adjustment Factor = [1-(0.5-SG)], where SG = Specific Gravity of the framing lumber. This adjustment factor must not be greater than 1.0.

[2]For shear loads of normal or permanent load duration as defined by the NDS, the values in the table above must be multiplied by 0.63 or 0.56, respectively.

[3]Shear loads can be increased 40% for wind loading.

[4]Diaphragm construction must be in accordance with Sections 4.2.7 and 4.2.8 of SDPWS-21 (Sections 4.2.6 and 4.2.7 of SDPWS-15 for the 2018 and 2015 IBC and of SDPWS-08 for the 2012 IBC), as applicable.

[5]This table gives shear values for Cases 1 and 2. The values shown are applicable to Cases 3 and 4, provided screws at all continuous panel edges are spaced in accordance with the boundary screw spacing; and are applicable to Cases 5 and 6, provided screws at all panel edges are spaced in accordance with the boundary screw spacing.

[6]See Figure 2 for Case diagrams.

TABLE 4—IBC EQUIVALENT PRESCRIPTIVE FASTENING SCHEDULE FOR WSV SCREWS

CONNECTION	FASTENING[1]	LOCATION
2024 and 2021 IBC Table 2304.10.2 and 2018 IBC Table 2304.10.1		
18. 1" brace to each stud and plate	2 screws	face
20. 1" x 8" and wider sheathing to each bearing	3 screws	face
23. 1" x 6" subfloor or less to each joist	2 screws	face
2015 IBC Table 2304.10.1		
19. 1" brace to each stud and plate	2 screws	face
21. 1" x 8" and wider sheathing to each bearing	3 screws	face
24. 1" x 6" subfloor or less to each joist	2 screws	face
2012 IBC Table 2304.9.1		
3. 1" X 6" subfloor or less to each joist	2 screws	face
4. Wider than 1" x 6" subfloor to each joist	3 screws	face
20. 1" diagonal brace to each stud and plate	2 screws	face
21. 1" x 8" sheathing to each bearing	3 screws	face
22. Wider than 1" x 8" sheathing to each bearing	3 screws	face

For **SI:** 1 inch = 25.4 mm.

[1]WSV screws must be a minimum of 2 inches long.

TABLE 5—REFERENCE LATERAL DESIGN VALUES (Z) FOR SINGLE SHEAR WOOD-TO-WOOD CONNECTIONS WITH WSV SCREWS (lbf)[1,2]

SCREW	$t_{s,w}$ (inches)	Z (lbf)	
		$0.42 \le SG_{NDS} < 0.50$	$0.50 \le SG_{NDS}/SG_{eq}$
WSV212, WSVF212	$1^1/_2$	72	92
WSV300, WSVF300		87	102

For **SI:** 1 inch = 25.4 mm, 1 lbf = 4.45 N.

[1]Tabulated values are based on attachment of the side member to a minimum 1½-inch-thick main member of the same species.
[2]Tabulated lateral design values (Z) must be multiplied by all applicable adjustment factors, including the load duration factor, C_D, from the NDS as referenced in the IBC or IRC.

TABLE 6—REFERENCE PULL THROUGH AND WITHDRAWAL DESIGN VALUES FOR WSV SCREWS INSTALLED IN THE SIDE GRAIN OF WOOD MEMBERS[1,2,3]

SCREW	L (inch)	$L_{emb,w}$ (inch)	REFERENCE PULL-THROUGH DESIGN VALUES, W_H (lbf)						REFERENCE WITHDRAWAL DESIGN VALUE, W			
			Nominal OSB/Plywood Rated Sheathing Panel Thickness (inch)				Solid Sawn Wood Min. Thickness (inch)					
							$0.42 \le SG_{NDS} < 0.50$	$0.50 \le SG_{NDS}/SG_{eq}$	$0.42 \le SG_{NDS} < 0.50$		$0.50 \le SG_{NDS}/SG_{eq}$	
			$^7/_{16}$	$^{15}/_{32}$	$^{19}/_{32}$	$^{23}/_{32}$	$1^1/_2$	$1^1/_2$	W (lbf/inch)	W_{max} (lbf)	W (lbf/inch)	W_{max} (lbf)
WSV134	1.75	1.20	66	66	96	109	141	195	98	117	123	147
WSV200	2.00	1.45							99	144	128	185
WSV212	2.50	1.95							117	233	128	256
WSV300	3.00	2.17							121	266	141	311

For **SI:** 1 inch = 25.4 mm, 1 lbf = 4.45 N.

[1]Use the lower of the pull-through or withdrawal values to determine axial design value.
[2]Reference withdrawal values, *W*, are in pounds per inch of the thread penetration in to the main member. W_{max} is the maximum reference withdrawal value.
[3]Tabulated design values must be multiplied by all applicable adjustment factors, including the load duration factor, C_D, from the NDS as referenced in the IBC or IRC.

TABLE 7—WSV CONNECTION GEOMETRY[1,2,3]

CONDITION	DIRECTION OF LOAD TO GRAIN	ID	MINIMUM DISTANCE OR SPACING (in.)	
			$SG_{NDS} < 0.50$	$SG_{NDS} \geq 0.50$
Edge Distance	Perpendicular	1	1 1/2	1 1/2
	Parallel	1	1	1 1/2
End Distance	Perpendicular	2	2	2 3/4
	Parallel	2	2 3/4	3
Spacing Between Fasteners in a Row	Perpendicular	3	1	1 1/2
	Parallel	4	2 3/4	2 3/4
Spacing Between Rows of Fasteners	Perpendicular	5	2	2
	Parallel	6	1	1 1/2
Spacing Between Staggered Rows	Perpendicular or Parallel	7	1/2	3/4

For **SI:** 1 inch = 25.4 mm.

[1]Edge distances, end distances and spacing of the screws must be sufficient to prevent splitting of the wood, or as required by this table, whichever is the more restrictive. See Figure 3 for a depiction of the requirements.

[2]Values for spacing between staggered rows apply where screws in adjacent rows are offset by 1/2 inch for $SG_{NDS} < 0.50$ or 3/4 inch when $SG_{NDS} \geq 0.50$.

[3]For screws which are axially loaded, edge distance, measured in the direction perpendicular to grain, must be a minimum of 3/4 inch; end distance, measured in the direction of grain, must be a minimum of 2 inches; the minimum perpendicular to grain spacing between screws must be 3/4 inch, and the minimum parallel to grain spacing between screws must be 1 1/2 inches.

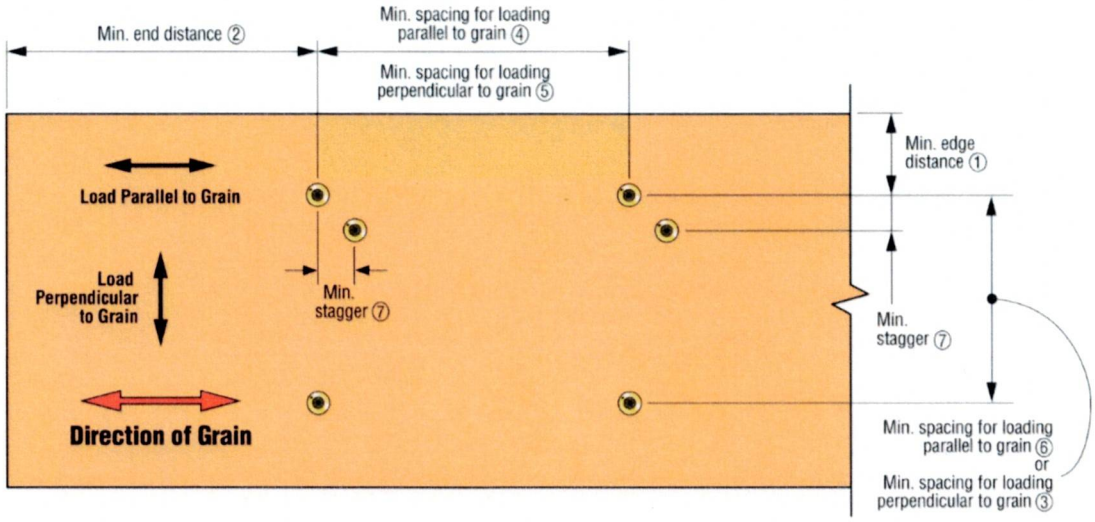

FIGURE 3—WSV CONNECTION GEOMETRY
Circled numbers refer to information in Table 7

ICC EVALUATION SERVICE®

ICC-ES Evaluation Report

ESR-4144

Reissued August 2024

This report also contains:

- LABC Supplement

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DIVISION: 04 00 00—MASONRY Section: 04 05 19.16—Masonry Anchors	REPORT HOLDER: HILTI, INC.	EVALUATION SUBJECT: HILTI HIT-HY 270 ADHESIVE ANCHOR SYSTEM IN UNREINFORCED MASONRY	

1.0 EVALUATION SCOPE

Compliance with the following codes:

- 2021, 2018, and 2015 *International Building Code®* (IBC)
- 2021, 2018, and 2015 *International Residential Code®* (IRC)
- 2021, 2018, and 2015 *International Existing Building Code®* (IEBC)

For evaluation for compliance with codes adopted by the *Los Angeles Department of Building and Safety* (LADBS), see ESR-4144 LABC, LARC and LAEBC supplement.

Property evaluated:

- Structural

2.0 USES

The Hilti HIT-HY 270 Adhesive Anchor System is used to anchor threaded steel rods or deformed steel reinforcement bars in unreinforced brick masonry. Anchors installed in unreinforced masonry with the HIT-HY 270 adhesive system are designed to resist short-term loads imposed by earthquake or wind, as noted in Section 4.0 of this report. The anchor system is an alternative to anchors described in Section 8.1.3 of TMS 402 (2016 or 2013 editions) as referenced in Section 2107 of the 2021, 2018, and 2015 IBC, respectively, as applicable. The anchors are an alternative to bolts described in Section A107.4 and Section A113.1 of the IEBC. The anchor system may also be used where an engineered design is submitted in accordance with Section R301.1.3 of the IRC.

3.0 DESCRIPTION

3.1 General:

The Hilti HIT-HY 270 Adhesive Anchor System consists of steel threaded rods or reinforcing bars, plastic mesh screen tube(s), the adhesive, and installation equipment as described in this report.

3.2 Materials:

3.2.1 Hilti HIT-HY 270 Adhesive: The Hilti HIT-HY 270 adhesive is an injectable hybrid adhesive mortar consisting of urethane methacrylate resin, hardener, cement and water. The resin and cement are separated from the hardener and water by means of a dual-cylinder foil pack attached to a manifold. An injection nozzle with an internal mixing element is attached to the manifold, and the adhesive components are dispensed through the injection nozzle to ensure proper mixing of the separate adhesive components. The injection nozzle may be replaced to permit multiple uses of the foil packs. Available foil pack sizes include total mixed volumes of 11.1 ounces (330 mL) and 16.9 ounces (500 mL).

3.2.2 The adhesive expiration date is printed on the manifold of each foil pack (month/year). The shelf life, as indicated by the expiration date, is for an unopened foil pack stored in a cool, dry, dark environment at temperatures between 41°F and 77°F (5°C and 25°C). Gel and curing times for the Hilti HIT-HY 270 adhesive and the respective masonry temperature during installation and cure are shown in Table 1.

3.2.3 Threaded Steel Rods and Reinforcing Bars: The threaded steel rods are $^3/_4$ inch (19.1 mm) in diameter, and must comply with the minimum mechanical properties of ASTM A307. Alternatively, the $^3/_4$-inch threaded rods may be used in a prebent $22^1/_2$-degree configuration. Threaded rods are supplied with nuts conforming to ASTM A563, Grade A, hex style, and with washers conforming to ANSI B 18.22.1, type A, Plain. The steel reinforcing bars are No. 4, No. 5, or No. 6 deformed bars complying with ASTM A615, A706, A767, or A996, Grade 60.

3.2.4 Plastic Mesh Screen Tubes: The mesh screen tubes are plastic with a black driving collar at the open end. They are available in diameters and lengths as described in Section 4.1. Different lengths can be achieved by assembling multiple screen tubes together, as depicted in Figure 3.

3.2.5 Unreinforced Masonry:

The existing unreinforced masonry walls must have a minimum nominal thickness of 13 inches (330 mm) [three wythes of brick]. The average in-place mortar shear strength of the building's unreinforced masonry determined in accordance with Section A106.2.3 (2021 and 2018 IEBC) or A106.3.3 (2015 IEBC), as applicable, must be no less than 50 psi (345 kPa) net.

4.0 DESIGN AND INSTALLATION

4.1 General:

Two types of anchor assemblies are available: Configuration A (shear anchor or rebar dowel) and Configuration B ($22^1/_2$-degree combination anchor).

4.1.1 Configuration A, Threaded Rods or Steel Reinforcing Bars in Shear (Shear Anchor or Rebar Dowel): Configuration A is the anchor assembly resisting shear loads only. Configuration A consists of a $^3/_4$-inch-diameter (19.1 mm), ASTM A307 threaded rod or a No. 4, No. 5 or No. 6 reinforcing bar and a 1-inch-diameter-by-8-inch-long (26 mm by 200 mm) plastic-mesh screen tube (HIT-SC 26x200). Figure 1 shows details of an installed shear-resisting assembly.

4.1.2 Configuration B, Bent Threaded Rods in Shear and Tension ($22^1/_2$-degree Combination Anchor): Configuration B is the anchor assembly resisting a combination of tension and shear loads. Configuration B consists of a $^3/_4$-inch-diameter (19.1 mm), ASTM A307 threaded rod prebent at an angle of $22^1/_2$ degrees and a 1-inch-diameter-by-13-inch-long (26 mm by 325 mm) plastic-mesh screen tube obtained by combining 1-inch-diameter-by-8-inch-long (26 mm by 200 mm) and 1-inch-diameter-by-5-inch-long (26 mm by 125 mm) plastic-mesh screen tubes (HIT-SC 26x200 and HIT-SC 26x125). The anchor must be embedded a minimum of 13 inches (330 mm) at a downward angle of $22^1/_2$ degrees to the horizontal. Figure 2 shows details of an installed shear- and tension-resisting assembly.

4.2 Design:

The Hilti HIT-HY 270 adhesive anchors are intended to resist only short-term loads imposed by wind or earthquake. The anchors must be approved by a registered design professional and installed under special inspection in accordance with Section 4.5 of this report. The edge distance and vertical and horizontal spacing for the two types of anchor assemblies described in Section 4.1 must comply with Table 2.

Conditions of acceptance for threaded rods and reinforcing bars in unreinforced brick masonry are as follows:

4.2.1 Configuration A, Threaded Rods or Steel Reinforcing Bars in Shear (Shear Anchor or Rebar Dowel):

a. Installation of assemblies using threaded rods or reinforcing bars intended to resist shear loads only must comply with Sections 4.1 and 4.3, and Figure 3 of this report.

b. The allowable shear load for the $^3/_4$-inch-diameter (19.1 mm) threaded rod is 1,000 pounds (4450 N), as shown in Table 3. The allowable shear loads for the No. 4, No. 5, and No. 6 reinforcing bars are 500, 750, and 1,000 pounds (2225, 3338, and 4450 N), respectively, as shown in Table 3. No adjustment for wind or earthquake loading is permitted.

c. The allowable shear loads noted above are applicable only to anchors installed in walls where in-place shear tests indicate a minimum mortar strength of 50 psi (345 kPa) net in accordance with Section A106.2.3 (2021 and 2018 IEBC) or A106.3.3 (2015 IEBC), as applicable.

4.2.2 Configuration B, Bent Threaded Rods in Shear and Tension ($22^1/_2$-degree Combination Anchor):

a. Installation of assemblies using prebent threaded rods intended to resist a combination of tension and shear loads must comply with Sections 4.1 and 4.3, and Figure 3 of this report.

b. The maximum allowable tension load for the $^3/_4$-inch-diameter (19.1 mm) prebent threaded rod (Configuration B) is 1,200 pounds (5340 N), as shown in Table 3. No adjustment for wind or earthquake loading is permitted.

c. The maximum allowable shear load for the $^3/_4$-inch-diameter (19.1 mm) prebent threaded rod (Configuration B) is 1,000 pounds (4,450 N), as shown in Table 3. No adjustment for wind or earthquake loading is permitted.

d. The maximum allowable load for the bent rod anchors subjected to combined tension and shear loads is determined by the following equation:

$$\left(\frac{P_s}{P_t}\right) + \left(\frac{V_s}{V_t}\right) \leq 1.0$$

where:

P_s = Applied tension load

P_t = Allowable tension load

V_s = Applied shear load

V_t = Allowable shear load

e. The allowable tension and shear values as determined above are applicable only to anchors installed in walls where in-place shear tests indicate minimum mortar strength of 50 psi (345 kPa) net in accordance with Section A106.2.3 (2021 and 2018 IEBC) or A106.3.3 (2015 IEBC), as applicable

4.3 Installation:

4.3.1 General: 1-inch-diameter (25.4 mm) holes must be drilled using standard carbide-tipped masonry drill bits complying with ANSI Specification B212.15-1994, or diamond core drill bits, as detailed in Figure 3 of this report. Holes must be drilled using a rotary drill or a rotary hammer drill set on "rotation only", or core drill, as applicable. Impact tools are not permitted. Assembly installation must be in accordance with Section 4.3.2 or 4.3.3, and Figure 3 of this report. The adhesive must be allowed to cure for the full curing time listed in Table 1 before anchors are loaded. Cure time refers to that period of cure after which hardware may be placed and nuts tightened. Design loads may not be applied until the full curing time has transpired. Installation must not occur when masonry temperatures are below 41 °F (5 °C). The Hilti HIT HY 270 Adhesive Anchor System is intended to resist only short-term loads imposed by wind or earthquake. The anchors must be approved by the registered professional and installed under special inspection in accordance with Section 4.5 of this report. The edge distance, and vertical and horizontal spacing for all anchor configurations described in Sections 4.3.2 and 4.3.3 of this report, must comply with Table 2.

4.3.2 Configuration A: Holes for threaded rods or reinforcing bars intended to resist shear only must be drilled perpendicular to the wall to a minimum embedment depth of 8 inches (203 mm). The holes must be cleaned in accordance with Figure 3 of this report to remove debris. The Hilti HIT-HY 270 adhesive must be injected into the supplied plastic mesh screen tube that is then inserted into the predrilled hole. The threaded rod or reinforcing bar must be inserted and pushed into the screen tube in a rotating manner to force the adhesive into the hole. Figure 1 illustrates an anchor installed in this configuration. Figure 3 illustrates the Manufacturer's Published Installation Instructions (MPII).

4.3.3 Configuration B: Holes for installation of assemblies using prebent threaded rods intended to resist shear and tension must be drilled at a $22^1/_2$-degree angle to within 1 inch (25.4 mm) of the opposing surface. The holes must be cleaned in accordance with Figure 3 of this report to remove debris. The Hilti HIT-HY 270 adhesive must be injected into the supplied plastic mesh screen tube that is then inserted into the predrilled hole. The prebent threaded rod must be inserted and pushed into the screen tube in a rotating manner to force the adhesive into the hole. Figure 2 illustrates an anchor installed in this configuration. Figure 3 illustrates the MPII.

4.4 Field Tests:

a. Tests for in-place mortar shear strength of the building must be done in accordance with Section A106.2.3 (2021 and 2018 IEBC) or A106.3.3 (2015 IEBC), as applicable. In-place mortar shear strength testing must indicate a minimum mortar strength of 50 psi (345 kPa).

b. Anchors resisting tension forces or a combination of tension and shear forces must be tested in accordance with Section A107.5 of the IEBC. The test report must include:

1. Test location(s)
2. Brick/mortar condition
3. Bolt movement/elongation
4. Embedment depth and masonry wall thickness
5. Applied load, loading procedure, load increments, and rate of loading.

4.5 Special Inspection:

4.5.1 IBC and IRC: Continuous special inspection must be performed in accordance with Sections 1704 and 1705 of the IBC.

4.5.2 IEBC: Periodic inspection, direct-tension tests, and calibrated torque wrench tests must be performed in accordance with Section A107.5 of the IEBC. In lieu of testing and periodic inspection, the IEBC permits continuous special inspection during installation of bolts resisting shear forces only.

5.0 CONDITIONS OF USE:

The Hilti HIT-HY 270 Adhesive Anchoring System for Unreinforced Masonry described in this report complies with, or is a suitable alternative to what is specified in, those codes listed in Section 1.0 of this report, subject to the following conditions:

5.1 Use and installation must be as set forth in this evaluation report and the Manufacturer's Published Installation Instructions (MPII) illustrated in Figure 3 of this report. In case of conflict, this report governs.

5.2 Calculations and details must be submitted to the code official for approval.

5.3 Special inspection must be in accordance with Section 4.5 of this evaluation report.

5.4 Use of the anchor system must be approved by the registered design professional.

5.5 The existing mortar shall have a minimum in-place shear strength of 50 psi (345 kPa) in accordance with Section 3.3 of this report prior to installation of the adhesive anchors.

5.6 Anchors must be limited to resisting transient wind or seismic loads only.

5.7 Anchors are installed in holes predrilled with a carbide-tipped masonry drill bit complying with ANSI B212.15-1994, or diamond core drill bits, in accordance with Figure 3 of this report. Impact tools must not be used for drilling holes or for tightening steel anchors or nuts.

5.8 The adhesive is not used after the expiration date stamped on the cartridge.

5.9 The Hilti HIT-HY 270 adhesive is manufactured by Hilti GmbH in Kaufering, Germany, under a quality control program with inspections by ICC-ES.

5.10 The Hilti HIT-SC screens are manufactured by Herbert Kaut GmbH & Co. KG in Sigmaringen, Germany, under a quality-control program with inspections by ICC-ES.

6.0 EVIDENCE SUBMITTED

Data in accordance with the ICC-ES Acceptance Criteria for Adhesive Anchors in Unreinforced Masonry Elements (AC60), dated December 2009 (editorially revised March 2021); and quality-control documentation.

7.0 IDENTIFICATION

7.1 The ICC-ES mark of conformity, electronic labeling, or the evaluation report number (ICC-ES ESR-4144) along with the name, registered trademark, or registered logo of the report holder must be included in the product label.

7.2 In addition, Hilti HIT-HY 270 adhesive cartridges are identified by a label displaying the product name, lot number, expiration date, a description of the product, and installation instructions. The Hilti HIT-SC screens are identified by a label displaying the product name, lot number, a description of the product, and installation instructions. Threaded rods, nuts, washers, and deformed reinforcing bars are standard elements and must conform to applicable national or international specifications.

7.3 The report holder's contact information is the following:

HILTI, INC.
7250 DALLAS PARKWAY, SUITE 1000
PLANO, TEXAS 75024
(800) 879-8000
www.us.hilti.com
HiltiTechEng@us.hilti.com

TABLE 1—HILTI'S GEL AND CURING TIMES FOR HILTI HIT-HY 270 ADHESIVE

MINIMUM BASE MATERIAL TEMPERATURE[1]		GEL TIME[2]	CURING TIME[3]
°F	°C		
41	5	10 minutes	4 hours
42-50	6-10	7 minutes	2.5 hours
51-68	11-20	4 minutes	1.5 hours
69-86	21-30	2 minutes	30 minutes
87-104	31-40	1 minute	20 minutes

For **SI:** °C = $^5/_9$ (°F – 32).

[1]The temperatures listed above refer to the base-material temperature, not ambient air temperature.
[2]The anchors may be adjusted during the gel time following installation.
[3]Anchors must not be disturbed between gel time and curing time. The anchors may be loaded after the full curing time has elapsed.

TABLE 2—SPACING AND EDGE DISTANCE REQUIREMENTS FOR HILTI HIT-HY 270 ADHESIVE INSTALLED IN UNREINFORCED BRICK MASONRY

ANCHOR ASSEMBLY	MINIMUM VERTICAL SPACING (inches)	MINIMUM HORIZONTAL SPACING (inches)	MINIMUM EDGE DISTANCE (inches)
Configuration A, Threaded Rods or Reinforcing Bars in Shear (see Figure 1)	16	16	16
Configuration B, Bent Threaded Rods in Shear and Tension (22$^1/_2$° Combination Anchor) (see Figure 2)	16	16	16

For **SI:** 1 inch = 25.4 mm.

TABLE 3—ALLOWABLE TENSION AND SHEAR LOADS FOR THREADED RODS AND REINFORCING BARS FOR HILTI HIT-HY 270 ADHESIVE INSTALLED IN UNREINFORCED BRICK MASONRY[1,2]

CONFIGURATION A – SHEAR ANCHOR OR REBAR DOWEL (FIGURE 1)				
Threaded Rod Dia. (inch) or Rebar Size	**Minimum Embedment (inches)**	**Minimum Wall Thickness (inches)**	**Allowable Tension Load (pounds)**	**Allowable Shear Load (pounds)**
$^3/_4$	8	13	-	1,000
No. 4	8	13	-	500
No. 5	8	13	-	750
No. 6	8	13	-	1,000
CONFIGURATION B – 22$^1/_2$° COMBINATION ANCHOR (FIGURE 2)				
Threaded Rod Dia. (inch)	**Minimum Embedment**	**Minimum Wall Thickness (inches)**	**Allowable Tension Load[3] (pounds)**	**Allowable Shear Load (pounds)**
$^3/_4$	Within 1 inch of opposite wall surface	13	1,200	1,000

For **SI:** 1 inch = 25.4 mm, 1 lbf = 4.45 N, 1 foot-pound = 1.356 N-m, 1 psi = 6.89 Pa.

[1]Allowable load values are applicable only to anchors where in-place shear tests indicate minimum mortar strength of 50 psi, net.
[2]No increase for short-term loading is permitted, such as loading induced by wind or earthquake.
[3]Anchors must be tested in accordance with Section 4.4 for use with the IEBC.

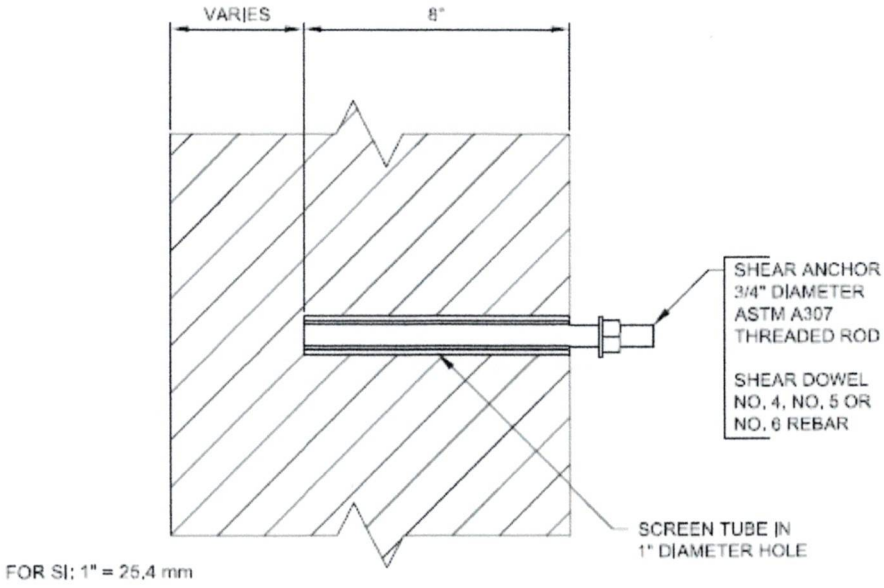

FIGURE 1—HILTI HIT-HY 270 ADHESIVE SHEAR ANCHOR OR DOWEL (CONFIGURATON A)

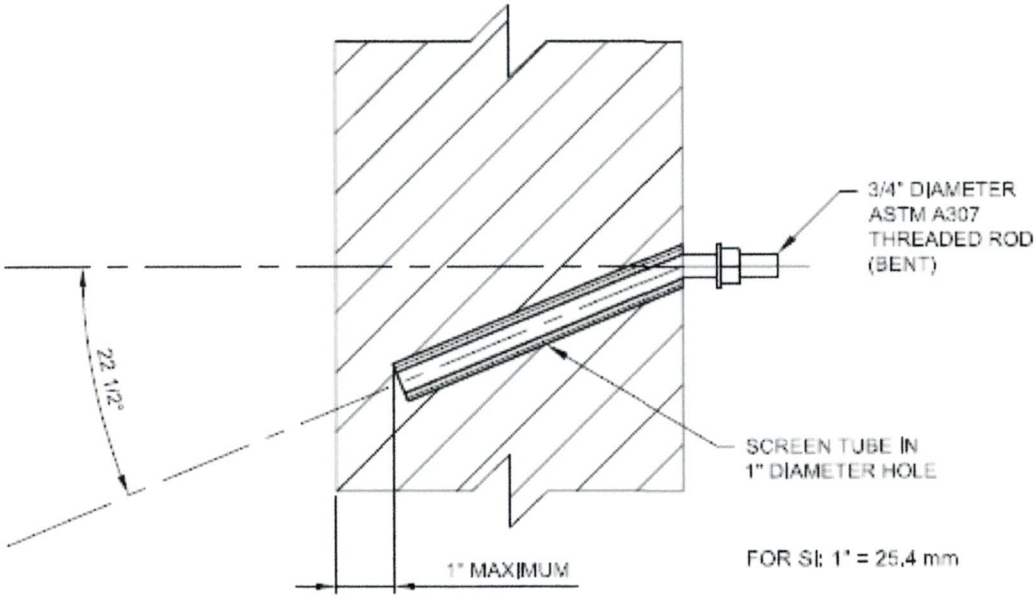

FIGURE 2—HILTI HIT-HY 270 ADHESIVE 22$^1/_2$° COMBINATION ANCHOR (CONFIGURATION B)

ICC-ES Evaluation Report

ESR-5183

Reissued December 2024

Subject to renewal December 2025

This report also contains:

-City of LA Supplement

-CA Supplement

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DIVISION: 04 00 00 - MASONRY Section: 04 05 19.16 – Masonry Anchors	REPORT HOLDER: TERREMOTO LTD (dba PYTHON FASTENERS)	EVALUATION SUBJECT: PYTHON C AND PYTHON MT URM SCREW ANCHORS FOR USE WITH BRICK AND UNGROUTED CONCRETE BLOCK (CMU)	

1.0 EVALUATION SCOPE

Compliance with the following codes:

- 2024, 2021 and 2018 *International Existing Building Code®* (IEBC)

Property evaluated:

- Structural

2.0 USES

The Python C and Python MT URM screw anchors for use with brick and ungrouted concrete block (CMU) are used as post installed connections for URM wall anchorages and URM element strengthening applications, to resist short-term tension and shear loads imposed by seismic or wind. The URM screw anchors are also used as connections for URM wall veneer strengthening and restraining applications.

The URM screw anchors are alternatives to "New wall anchors" described in Section A107.4 of Appendix A of the IEBC.

3.0 DESCRIPTION

3.1 Python C URM screw anchor:

The Python C URM screw anchor is a nominal ¼ - inch (6 mm) shank diameter, heat treated, carbon steel anchor, with a spiral threaded pitch and an unthreaded mid-section, available in a standard length of 9.45-inches (240 mm). The anchor has an electro-galvanized zinc-nickel coating. The Python C URM screw anchor is illustrated in Figures 1 and 2.

3.2 Python MT URM screw anchor:

The Python MT URM screw anchor is a nominal $^{5}/_{16}$ - inch (8 mm) shank diameter, heat treated, carbon steel anchor, with a spiral threaded pitch throughout the anchor's body, available in three lengths: 6.3-inches, 9.1-inches, and 13.4-inches (160 mm, 230 mm, and 340 mm). The anchor has an electro-galvanized zinc-nickel coating. The Python MT URM screw anchor is illustrated in Figures 1 and 2.

3.3 Unreinforced Masonry:

The existing unreinforced masonry walls with clay bricks may be single, double, or triple wythes as defined in the IEBC. Unreinforced masonry walls with ungrouted concrete blocks must be as defined in the IEBC. The existing unreinforced masonry wall average in-place mortar shear strength must be no less than 30 psi (207 kPa) net.

4.0 DESIGN AND INSTALLATION

4.1 Design: Anchors described in this report provide tension and shear loads for designs based on allowable stress design (working stress design) as indicated in Tables 1, 2 and 3. The anchors are intended to resist only short-term loads imposed by wind or earthquake.

Allowable shear and tension loads may be combined in accordance with the following equation:

$[P_s/P_t] + [V_s/V_t] \leq 1.0$

where:

P_s = Applied service tension load.

P_t = Allowable tension load.

V_s = Applied service shear load.

V_t = Allowable shear load.

4.2 Installation: URM screw anchors must be installed in accordance with this report and the manufacturer's published installation instructions (MPII) as represented in Figures 3 through 7. The drill bit size, hole diameter, embedment depth, spacing, edge distance and base material must comply with the requirements of this report. For impact drivers used to tighten URM screw anchors, the maximum torque capacity of the impact driver must be 1550 in-lbf. The anchors must be installed under special inspection in accordance with Section 4.4 of this report.

4.3 Field Tests:

a. Tests for in-place mortar shear strength of the building must be conducted in accordance with 2024, 2021 or 2018 Section A106.2.3 of the IEBC. In-place mortar shear strengths must indicate a minimum mortar strength of 30 psi (275 kPa).

b. Anchors resisting tension forces, or a combination of tension and shear forces must be tested in accordance with Section A107.4 of the IEBC. The test report must include:

1. Test location(s)
2. Brick/mortar/concrete block condition
3. URM screw anchor movement/elongation
4. Embedment depth and masonry wall thickness
5. Applied load, loading procedure, load increments, and rate of loading.

4.4 Special Inspection: Periodic inspection and field direct-tension tests must be conducted in accordance with Section A107.4 and A107.5.1 of the IEBC, respectively, with modification to Section A107.5.1 of the IEBC as follow:

The test apparatus must be supported by the URM element. URM screw anchors must be subject to a direct tension load of not less than 2.5 times the allowable design load for 5 minutes without noticeable deformation or damage to the URM screw anchors. The field direct-tension test requirements and methods must be specified by the registered design professional and submitted to the code official for approval.

5.0 CONDITIONS OF USE:

The Python C and Python MT URM screw anchors for use with brick and ungrouted concrete block (CMU) described in this report comply with, or are a suitable alternative to what is specified in, those codes listed in Section 1.0 of this report, subject to the following conditions:

5.1 Use and installation must be as set forth in this evaluation report and the manufacturer's printed installation instructions (MPII) illustrated in Figures 3 to 7 of this report. In case of conflict, this report governs.

5.2 Calculations and details, justifying the capacity of the URM elements to resist the demand loads from the URM screw anchors must be submitted to the code official for approval. The calculations and details must include a complete load path as part of the component design. The calculations and details must be prepared by a registered design professional where required by the statutes of the jurisdiction in which the project is to be constructed.

5.3 URM screw anchors must be designed by the registered design professional as single screw. Unless reduction factors for reduced edge distances are applied as indicated in Tables 1, 2 or 3, the URM screw anchors resisting shear only must have with a minimum spacing and edge distance of 12-inches (30.48 cm) and the URM screw anchors resisting tension, or combined tension and shear must have a minimum spacing and edge distance of 16-inches (40.64 cm).

5.4 The in-place mortar shear strength of the building must be at least equal to the value published in the evaluation report under Section 3.3 and also comply with requirements in IEBC Section A106.2.3.6 (2024 or 2021 IEBC), or Section A106.2.3.5 (2018 IEBC).

5.5 The URM screw anchors are limited to dry, interior locations.

5.6 The URM screw anchors are used to resist short-term loading due to wind or seismic forces under the IEBC.

5.7 URM screw anchors, URM element and URM wall veneer configurations included in this evaluation report must meet or exceed the member parameters described in accordance with Section 3.0 of this report.

5.8 URM screw anchors must be installed under special inspection in accordance with Section 4.4 of this evaluation report. The URM screw anchors may be backed out of one full turn maximum and retightened for the direct-tension test described in Section 4.4 of this evaluation report.

5.9 The URM screw anchor allowable loads must be limited to the allowable values for existing materials in accordance with Table A108.1(2) of the IEBC (refer to Table A108.1(2) footnotes 'e' and 'f' of the IEBC to determine allowable values).

5.10 URM screw anchors must not be installed in URM elements constructed using stack bond pattern, and its evaluation is outside the scope of this criteria.

5.11 URM screw anchors are installed in holes predrilled with a carbide-tipped masonry drill bit complying with ANSI B212.15. Holes must be drilled using a rotary drill, or a rotary hammer drill set on "rotation only" (non-impact). Hammer action tools must not be used for drilling holes or for tightening URM screw anchors.

5.12 The Python C and Python MT URM screw anchors for use with brick and ungrouted concrete block (CMU) are manufactured under a quality control program with inspections by ICC-ES.

6.0 EVIDENCE SUBMITTED

6.1 Data in accordance with the ICC-ES Acceptance Criteria for 5/16-in (8mm) diameter and smaller unreinforced masonry (URM) (AC545), dated February 2023 and editorially revised November 2023.

6.2 Quality control documentation.

7.0 IDENTIFICATION

7.1 The ICC-ES mark of conformity, electronic labeling, or the evaluation report number (ICC-ES ESR-5183) along with the name, registered trademark, or registered logo of the report holder must be included in the product label.

7.2 In addition, the Python C and Python MT URM screw anchors for use with brick and ungrouted concrete block (CMU) are identified with the URM screw anchor size and length on the packaging labels.

7.3 The report holder's contact information is the following:

TERREMOTO LTD (dba PYTHON FASTENERS)
100 N. HOWARD ST. Ste R
SPOKANE, WA 99201
(206) 841-8745
https://www.pythonfasteners.com
info@pythonfasteners.com

TABLE 1 – ALLOWABLE TENSION AND SHEAR OF PYTHON C IN UNREINFORCED BRICK MASONRY WALL APPLICATIONS[2,3]

Location	Embedment[1], in (mm)	Mortar Shear Strength = 30 psi (0.2 MPa)		Mortar Shear Strength ≥ 48 psi (0.3 MPa)	
		Tension, lbf (kN)	Shear, lbf (kN)	Tension, lbf (kN)	Shear, lbf (kN)
Brick Face	2 (50)	145[4] (0.6)	275[4] (1.2)	235[4] (1.0)	435[4] (1.9)
	2.75 (70)	300 (1.3)	-	480 (2.1)	-
Mortar Joint	2 (50)	80[4] (0.4)	195[4] (0.9)	125[4] (0.6)	315[4] (1.4)
	2.75 (70)	125 (0.6)	-	200 (0.9)	-

For **SI:** 1 inch = 25.4 mm, 1 lbf = 4.45 N, 1 psi = 6.89 Pa.

1. Embedment of anchor into substrate.
2. Interpolation between mortar shear strengths is allowed.
3. Values are based on a Factor of Safety of 5.
4. For edge distances less than 12-in (305 mm) for shear loading and 16-in (406 mm) for tension or combined tension and shear loading, the following reduction factors must be applied where the edge distance is no less than 6 1/8-inches (155mm):
   a. Allowable shear loads 0.8
   b. Allowable tension (or combined shear and tension) loads 1.0

TABLE 2 – ALLOWABLE TENSION AND SHEAR OF PYTHON MT IN UNREINFORCED BRICK MASONRY WALL APPLICATIONS[2,3]

Location	Embedment[1], in (mm)	Mortar Shear Strength = 30 psi (0.2 MPa)		Mortar Shear Strength ≥ 48 psi (0.3 MPa)	
		Tension, lbf (kN)	Shear, lbf (kN)	Tension, lbf (kN)	Shear, lbf (kN)
Brick Face	2 (50)	235[4] (1.0)	510[4] (2.3)	375[4] (1.7)	815[4] (3.6)
	2.75 (70)	300 (1.3)	-	480 (2.1)	-
	4.5 (115)	660 (3.0)	590 (2.6)	1060 (4.7)	940 (4.2)
	6.5 (165)	705 (3.1)	-	1135 (5.0)	-
	8 (200)	875[4] (3.9)	855[4] (3.8)	1400[4] (6.2)	1370[4] (6.1)
	10 (250)	1295 (5.8)	-	2075 (9.2)	-
Mortar Joint	2 (50)	95[4] (0.4)	255[4] (1.1)	150[4] (0.7)	410[4] (1.8)
	4.5 (115)	410 (1.8)	450 (2.0)	650 (2.9)	720 (3.2)
	8 (200)	590[4] (2.6)	710[4] (3.2)	940[4] (4.2)	1140[4] (5.1)
	10 (250)	1035 (4.6)	-	1665 (7.4)	-

For **SI:** 1 inch = 25.4 mm, 1 lbf = 4.45 N, 1 psi = 6.89 Pa.

1. Embedment of anchor into substrate.
2. Interpolation between mortar shear strengths is allowed.
3. Values are based on a Factor of Safety of 5.
4. For edge distances less than 12-in (305 mm) for shear loading and 16-in (406 mm) for tension or combined tension and shear loading, the following reduction factors must be applied where the edge distance is no less than 6 1/8-inches (155mm):
   a. Allowable shear loads: 1.0
   b. Allowable tension (or combined shear and tension) loads 0.6

TABLE 3 – ALLOWABLE TENSION AND SHEAR OF PYTHON MT IN UNGROUTED CONCRETE MASONRY BLOCK WALL APPLICATIONS[2,3,5]

Location	Embedment[1], in (mm)	Mortar Shear Strength = 30 psi (0.2 MPa)		Mortar Shear Strength ≥ 48 psi (0.3 MPa)	
		Tension, lbf (kN)	Shear, lbf (kN)	Tension, lbf (kN)	Shear, lbf (kN)
Face Shell	1.25 (32)	150 (0.7)	200 (0.9)	240 (1.1)	315 (1.4)
	1.25 (32) + 3/8 (10)[4]	190 (0.9)	255 (1.1)	315 (1.4)	410 (1.8)

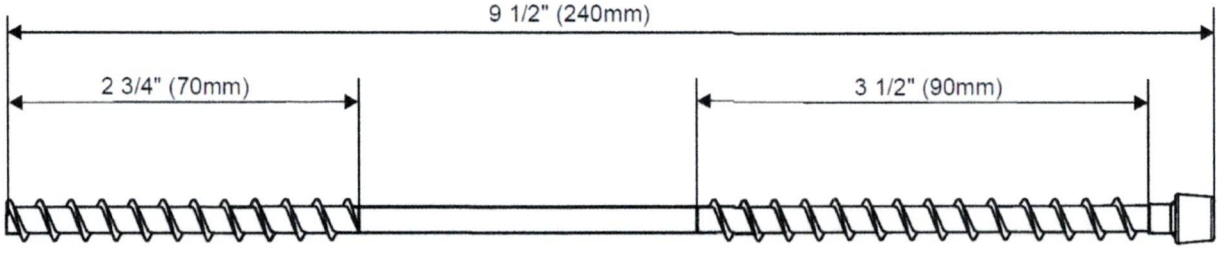
For **SI:** 1 inch = 25.4 mm, 1 lbf = 4.45 N, 1 psi = 6.89 Pa.

1. Embedment of anchor into substrate.
2. Interpolation between mortar shear strengths is allowed.
3. Values are based on ungrouted and unreinforced concrete masonry walls and a Factor of Safety of 5.
4. Anchor is installed though front face shell with a minimum embedment of 3/8 in in the rear face shell.
5. For edge distances less than 12-in (305 mm) for shear loading and 16-in (406 mm) for tension or combined tension and shear loading, the following reduction factors must be applied where the edge distance is no less than 6 1/8-inches (155mm):
   c. Allowable shear loads where embedment is less than or equal to 1.25 in (32mm): 1.0
   d. Allowable shear loads where embedment is greater than 1.25 in (32mm): 0.95
   e. Allowable tension (or combined shear and tension) loads where embedment is less than or equal to 1.25 in (32mm): 1.0
   f. Allowable tension (or combined shear and tension) loads where embedment is greater than 1.25 in (32mm): 0.8

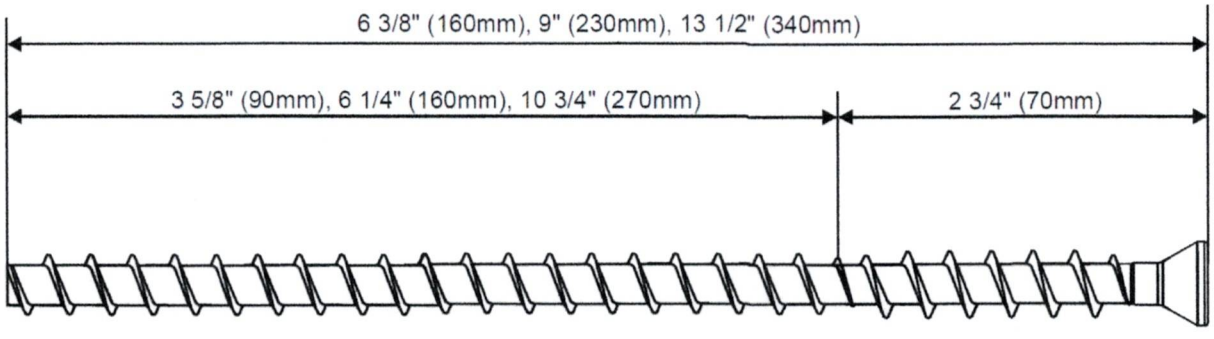
PYTHON MT ANCHORS

PYTHON C ANCHOR

FIGURE 1—PYTHON URM SCREW ANCHORS



PYTHON C ANCHOR

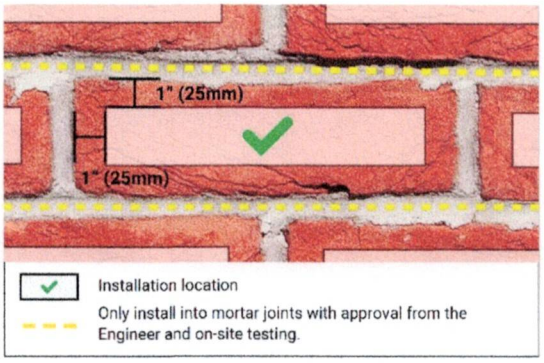


PYTHON MT ANCHOR

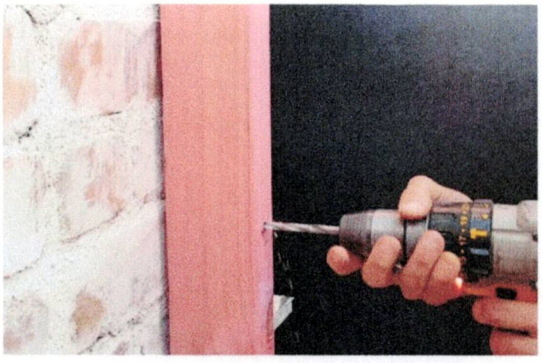
FIGURE 2—PYTHON URM SCREW ANCHOR DIMENSIONS

PYTHON MT: TIMBER TO MASONRY

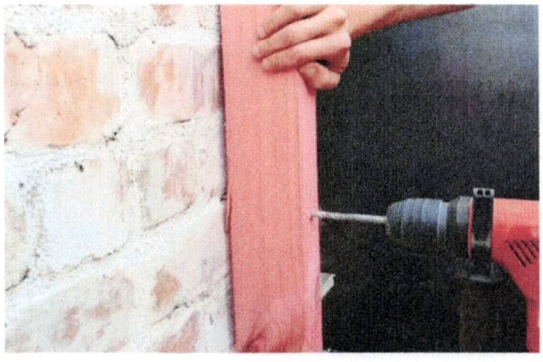
PYTHON® MASONRY FASTENERS



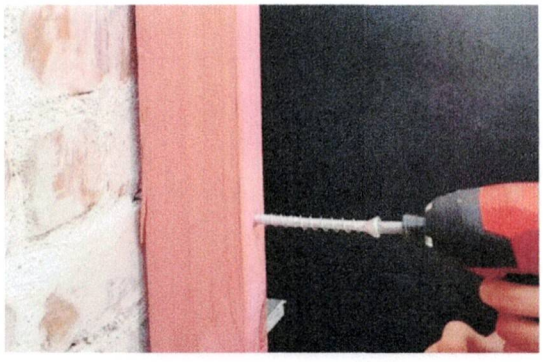
01 Mark the location for screw installation on the timber. Ensure the location is at least 1" (25mm) from the brick edge.



02 Drill a 5/16" (8mm) diameter hole through the timber with a timber drill bit.



03 Drill an 8mm diameter hole 1/2" (13mm) deeper than the intended screw embedment depth with a PYTHON masonry drill bit and rotary only drill.



04 Install the PYTHON MT using a TORX T50 bit and impact driver.

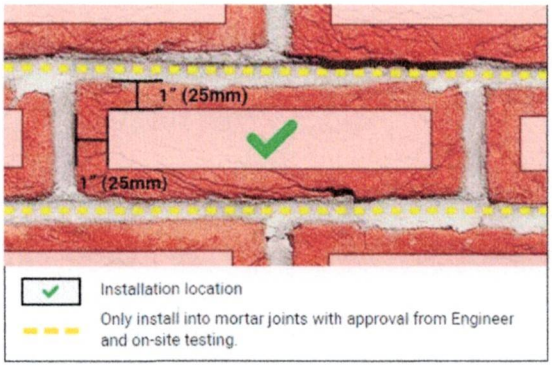
05 Countersink the screw head for a smooth finish.

WARNING: Do not over-torque or countersink the head past flush.

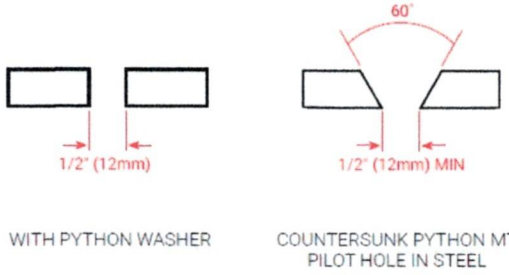
FIGURE 4 – PYTHON MT MANUFACTURER'S PUBLISHED INSTALLATION INSTRUCTIONS FOR TIMBER TO URM (MPII)

PYTHON MT: STEEL TO MASONRY

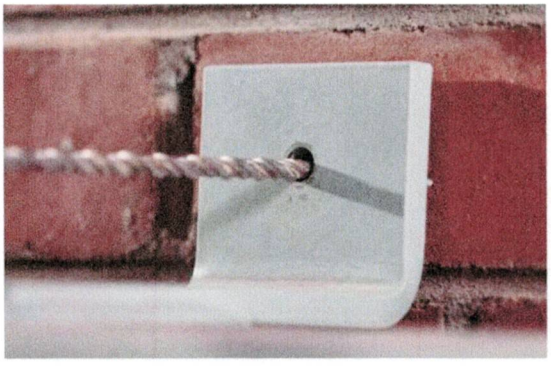
PYTHON® MASONRY FASTENERS



01 Mark the installation location on the brick. Ensure the location is at least 1" (25mm) from the brick edge.

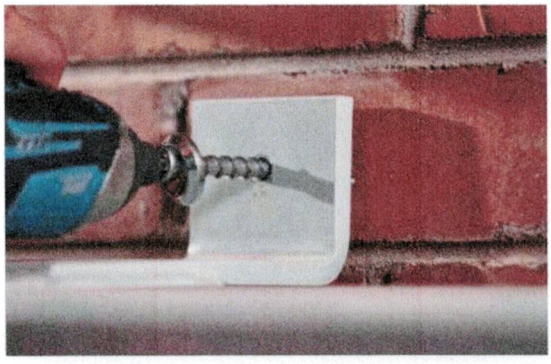


02 Pre-fabricate 1/2" (12mm) diameter holes for use with PYTHON Washers OR cone holes with a 60° angle for countersunk option.

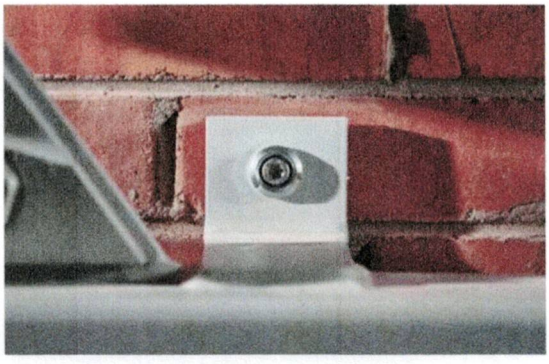


03 Drill an 8mm diameter hole 1/2" (13mm) deeper than the intended screw embedment depth with a PYTHON masonry drill bit and rotary only drill.

04 Thread the PYTHON MT through the center of the PYTHON Washer (if applicable).



05 Install the PYTHON MT using a TORX T50 bit and impact driver.



06 PYTHON MT installed with washer.

WARNING: Do not over-torque.

FIGURE 5 – PYTHON MT MANUFACTURER'S PUBLISHED INSTALLATION INSTRUCTIONS FOR STEEL TO URM (MPII)

APPENDIX B: Geotechnical Investigation Report

Geotechnical Investigation Report

NPS Floyd Bennett Field Hangars 3 & 4 Rehabilitation

Quinn Evans Architects, Inc.

June 25, 2025

The Power of Commitment

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The Power of Commitment

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Appendix A	Borehole Logs
Appendix B	Geotechnical Laboratory Test Results
Appendix C	Corrosion Potential Laboratory Test Results

1. Introduction

GHD Services Inc. (GHD) was retained by Quinn Evans Architects, Inc. (QE) to conduct a geotechnical evaluation for the rehabilitation of Hangars 3 & 4 at Floyd Bennett Field in the Gateway National Recreation Area located at 3159 Flatbush Avenue in Jamaica Bay Unit, Brooklyn, New York (Site). A Site Location Map showing the project Site is included as Figure 1.

Per the Hangar Foundations Drawings dated November 15, 1929, the existing masonry walls of the hangars, lean-tos and the hangar door tracks bear on grade beams and pile caps supported by 14-inch square reinforced concrete piles driven to a depth of about 35 feet. The existing hangar floors are comprised of a 6-inch concrete slab-on-grade. Differential settlement of up to 6.5 inches of the slab-on-grade relative to the pile-supported elements has been documented. Existing masonry non-load bearing partitions in the lean-tos have also experienced some differential settlement. GHD understands that no signs of differential settlement between the pile-supported elements have been observed above-grade.

GHD was provided with the General Requirements for Subsurface Exploration and Analysis (GRSEA) document prepared by Matteo Ferran Structural Engineers (MFSE) and dated September 6, 2024. Per review of the GRSEA as well as subsequent e-mail correspondence and a project meeting with QE and MFSE on May 19, 2025, GHD understands the hangars will be restored to their original appearance and condition and the proposed rehabilitation will include the following:

- Existing exterior masonry walls will remain in place founded on the existing pile-supported foundation system.
- Removal and replacement of the existing slab-on-grade and interior walls and constructing a new slab-on-grade including thickening the slab edges and doweling the slab into the existing grade beams.
- Installation of mechanical and electrical equipment in the areas of the existing lean-tos.

The future use of the hangars is not presently known. The planned rehabilitation is to include an anticipated maximum floor loads of 150 pounds per square foot (psf) in slab-on-grade areas and less than 300 psf loading from the mechanical equipment pads. Future development of the interior space of the hangars could include various heavier structures and enclosures.

This report contains a description and findings of GHD's geotechnical evaluation, as well as professional opinions and geotechnical recommendations to assist others in the design and construction of the new slab-on-grade, pavements, mechanical and electrical foundations, as well as providing options for deep foundation support of heavier structures.

2. Field and Laboratory Programs

2.1 Field Program

The field work for this geotechnical investigation was carried out from April 15 to 18, 2025. ADT-Cascade completed the drilling, in-situ testing and sampling operations. A GHD geo professional provided oversight of the drilling operations and logged the boreholes. Four (4) geotechnical boreholes (B-1 to B-4) were advanced at the Site. The borehole locations are shown on the provided Borehole Locations Map, included as Figure 2. The borehole locations were estimated from measurements from existing Site features and ground surface elevations were estimated from topographic data prepared by VHB dated May 30, 2024.

At each borehole location, a Geoprobe 7822 DT track-mounted drill rig was used to advance the boreholes to a depth of 50 feet below ground surface (bgs).

Hand augering was performed to a depth of 5 feet bgs for verification of clearance of obstructions. Drilling then commenced using 4.25-inch I.D. hollow-stem augers to a depth of 10 feet bgs followed by wash rotary using a

3-7/8-inch tri-cone roller-bit with bentonite and water as drilling mud to advance the borehole to 50 feet bgs. The Standard Penetration Test (SPT) method per ASTM D1586 was performed continuously to a depth of 10 feet and at a maximum of 5-foot intervals thereafter. The results of the penetration tests from the split spoon samples are reported as N-values on the borehole logs at their corresponding depths. The split spoon sampler was driven 18 inches into the ground using a 140-pound hammer falling 30 inches. The sum of the number of blows required to advance the split spoon the second and third 6-inch intervals (last 12 inches) is termed the N-value.

Groundwater level observations were made prior to the introduction of drilling mud at 10 feet bgs. Upon completion of drilling, the boreholes were backfilled with soil cuttings and the surface patched to match the surrounding area.

Geotechnical soil samples obtained from the boreholes were field classified upon retrieval for type, texture, color, and moisture condition. The detailed results of the boreholes are recorded on the accompanying borehole logs in Appendix A.

2.2 Laboratory Program

The split spoon samples were sealed in clean, airtight, glass jars and transported to GHD's geotechnical laboratory in Farmington Hills, Michigan for further examination and testing. Grab samples from auger cuttings were obtained from the upper 10 feet of boreholes B-1 and B-4, which were transported to Geotechnics in East Pittsburg, Pennsylvania for testing for corrosion potential.

Geotechnical samples received in the GHD laboratory were visually examined by a geotechnical professional and classified based on type, texture, plasticity, color, and consistency in general accordance with the Unified Soil Classification System (ASTM D2487). Moisture content determinations were completed on all recovered split-spoon samples. Grain size analyses (ASTM D422) and Atterberg limit tests (liquid and plastic limits, ASTM D4318) were completed on select samples. Results of the moisture content and liquid and plastic limit tests are included on the individual borehole log at their respective depths.

Samples received by Geotechnics were tested for parameters to characterize the corrosion potential in accordance with the soil-test evaluation in the American Water Works Association (AWWA) C-105 Standard. Testing conducted by Geotechnics included pH (AASHTO T289-91), minimum resistivity (AASHTO T288-12), sulfate content (ASTM T290–95), and chloride content (ASTM T291-94). Geotechnics shipped portions of each sample to Eurofins' laboratory in Pittsburgh, PA for oxidation reduction (redox) potential (SM 2580B) and sulfide content (SW846 EPA 9034) testing. The results of the tests were used to designate a total aggressivity score and soil aggressivity classification for each sample. Geotechnics provided a Laboratory Test Results Transmittal dated May 12, 2025, which included a Eurofins Analytical Report also dated May 12, 2025.

The detailed results of the boreholes are recorded on the accompanying borehole logs in Appendix A. In addition, GHD's laboratory test results are provided in Appendix B, and the laboratory test results from Geotechnics and Eurofins are included in Appendix C.

3. Regional Geology

Jamaica Bay is located along the southern shore of Long Island, New York, and has a geological profile shaped by both natural processes and human activities. The area is primarily covered by Late Wisconsin an glacial outwash, consisting of unconsolidated sediments like sand and gravel, which are obscured by modern shore and urban deposits. The region features diverse depositional environments, including salt marshes, beaches, dunes, and urban fill. Beneath these sedimentary layers lies the southeastward dipping crystalline basement complex of the New York City series, with bedrock buried under significant thicknesses of sediment in Brooklyn and Queens. Geologic mapping indicates bedrock is located between Elevation -700 and -800 feet mean sea level (msl).

Human activities have significantly altered the geological processes in Jamaica Bay, impacting both the surface and subsurface geology. The GRSEA indicates the structures in the project vicinity were originally built between 1929 and 1938 on land created by infilling the channels located between Barren Island, several smaller islands and portions of Brooklyn with sand pumped from the bottom sediments of Jamaica Bay. The GRSEA also indicates Hangars 3 & 4 are not located within a flood zone but are located immediately adjacent to mapped flood zones.

4. Site and Subsurface Conditions

Details of the subsurface conditions encountered at the Site are summarized below, and are also presented on the accompanying borehole logs in Appendix A.

It should be noted that the conditions indicated on the borehole logs are for specific locations only and can vary beyond the borehole locations. The soil boundaries indicated on the borehole logs are inferred from non-continuous sampling and observations during drilling and excavation. These boundaries are intended to reflect approximate transition zones and should not be interpreted as exact planes of geological change.

In addition, the descriptions provided on the borehole logs are inferred from a variety of factors, including visual observations of the soil samples retrieved, laboratory testing, and measurements prior to and after drilling.

4.1 Soil Conditions

4.1.1 Concrete Slab and Asphalt Pavement

At borehole location B-1 performed inside the existing hangars, approximately 6 inches of concrete slab was encountered at the ground surface. At borehole locations B-2 through B-4 performed just outside the hangars, approximately 3 to 6 inches of asphalt pavement was encountered at the ground surface.

4.1.2 Fill

Sand fill was encountered under the concrete slab and asphalt pavement and extended to approximately 13.5 to 18.5 feet bgs at all borehole locations. The sand fill consisted of variable amounts of silt and gravel. The fill was generally light brown to brown and gray. SPT ‘N’ values obtained within the sand fill ranged from 0 (weight of hammer) to 12 blows per foot (bpf), indicating a very loose to medium dense relative density. The sand was generally moist to wet with moisture content by weight ranging from 3% to 36%.

Sandy silt and sandy clay fill with variable amounts of organics, wood chips and roots were encountered under the sand fill layer at B-1 and B-3 and extended to depths ranging from 19.0 to 19.5 feet bgs. The sandy silt encountered at borehole B-3 had a high liquid limit value of 53 and a low plasticity index of eight, which is considered an elastic silt. The sandy silt and sandy clay fill were generally dark gray and gray. SPT ‘N’ values obtained within the sandy silt and sandy clay fill was 2 bpf, indicating a very soft consistency. The sandy silt and sandy clay fill were generally moist to wet with moisture content by weight ranging from 42% to 50%.

4.1.3 Sand (SP, SP-SC, SC, SM)

Sand (USCS classification of SP, SP-SC, SC, SM) with variable amounts of silt, clay, and gravel was encountered below the fill at all borehole locations and extended to the borehole termination depth of 50 feet bgs. Generally, the sand was gray and dark gray. Trace amounts of shells were encountered within the clayey sand at B-4 at a depth of 18.5 to 23.5 feet bgs. The SPT N values in the sand ranged from 0 (weight of hammer) to 5 bpf to depths ranging from 23.5 to 28.5 feet bgs indicating a very loose to loose relative density. Below these depths and extending to the borehole termination depths, the SPT N values increased to values ranging from 11 to 40 bpf, indicating a medium

dense to dense relative density. The sand was moist to wet with moisture content by weight measured between 15% and 44%.

4.2 Groundwater Conditions

The soils encountered in the investigation were generally moist to wet. Water was observed at the locations of boreholes B-2 through B-4 during drilling at a depth of 10 feet bgs. Water was not observed at the borehole location B-1 prior to introducing drilling mud (water and bentonite) for wash rotary drilling operations. Groundwater observations were not able to be made at the completion of drilling due to the drilling mud placement in the boreholes.

Groundwater observations during drilling are often not reliable indicators of the static water table due to the drilling methods employed that may seal off the natural paths for groundwater flow.

It should be noted that groundwater levels are transient and tend to fluctuate with the seasons and periods of precipitation and temperature. Typically, monitoring wells or piezometers are used to measure groundwater levels in soils over a long period of time. No monitoring wells were installed in the soil boreholes that pertain to this report.

4.3 Laboratory Results

Moisture content determinations were completed on all recovered samples. Grain size analyses and Atterberg limit tests were carried out on representative split-spoon samples. The test results are summarized below, and the detailed geotechnical laboratory test results are provided in Appendix B.

Table 1 Summary of Soil Laboratory Test Results

Borehole No.	Sample No.	Sample Depth (ft bgs)	% Passing No. 200 Sieve	W_n (%)	Atterberg Limits (%)			Soil Description
					LL	PL	PI	
B-1	SS-4	8.5	1	13	-	-	-	(SP) SAND
B-1	SS-7	23.5	1	44	-	-	-	(SP) SAND
B-2	SS-4	8.5	1	22	-	-	-	(SP) SAND
B-2	SS-6	18.5	9	24	NP	NP	NP	(SP-SC) SAND with clay/silt
B-2	SS-10	38.5	2	15	-	-	-	(SP) SAND
B-3	SS-5	13.5	53	50	53	45	8	(MH) Sandy SILT with clay
B-3	SS-8	28.5	1	27	-	-	-	(SP) SAND
B-4	SS-3	7.0	1	8	-	-	-	(SP) SAND
B-4	SS-6	18.5	48	32	35	21	14	(SC) Clayey SAND with silt
B-4	SS-9	33.5	1	26	-	-	-	(SP) SAND

Notes:
W_n – Natural Water Content; LL – Liquid Limit; PL – Plastic Limit; PI – Plasticity Index
Soil classification based on Unified Soil Classification System (ASTM D2487)

Table 2 Summary Results of Grain Size Analysis

Borehole No.	Sample No.	Sample Depth (feet bgs)	Soil Description	Percent Gravel	Percent Sand	Percent Fines
B-1	SS-4	8.5	(SP) SAND	4	95	1
B-1	SS-7	23.5	(SP) SAND	0	99	1
B-2	SS-4	8.5	(SP) SAND	0	99	1
B-2	SS-6	18.5	(SP-SC) SAND with clay/silt	0	91	9
B-2	SS-10	38.5	(SP) SAND	0	98	2
B-3	SS-5	13.5	(MH) Sandy SILT with clay	0	47	53
B-3	SS-8	28.5	(SP) SAND	0	99	1
B-4	SS-3	7.0	(SP) SAND	0	99	1
B-4	SS-6	18.5	(SC) Clayey SAND with silt	0	52	48
B-4	SS-9	33.5	(SP) SAND	0	99	1
Fines – passing the #200 sieve						

4.3.1 Testing for Corrosion Potential

Representative soil samples recovered from the boreholes were tested for parameters to characterize the corrosion potential in accordance with the soil-test evaluation in the American Water Works Association (AWWA) C-105 Standard. Geotechnics conducted pH, minimum resistivity, sulfate content and chloride content testing and Eurofins conducted oxidation reduction (redox) and sulfide content testing. The results of the tests were used to designate a total aggressivity score and soil aggressivity classification for each sample. The aggressivity scores are summarized below in Table 3. The detailed soil corrosion test results are provided in Appendix C.

Table 3 Summary of Test Results for Corrosion Potential

Borehole	Sample No.	Sample Depth (ft bgs)	pH	Minimum Resistivity (ohm-cm)	Sulfate Content (ppm)	Chloride Content (ppm)	Redox Potential (mV)	Sulfide Content (mg/kg)	Aggressivity Score
B-1	GS-1*	0 – 10	8.05	6,200	68.3	7.38	200	ND	1
B-4	GS-1*	0 – 10	7.67	32,000	42.6	2.78	180	ND	1
*Grab sample (GS-1) taken from auger cuttings from the upper 10 feet									

The AWWA assigns points based on the results of pH, resistivity, sulfides, redox potential, and the moisture condition of the soil. A soil that has a total point score of 10 or more may be potentially corrosive to ductile iron pipe. Based on the results obtained for the samples submitted, less than 10 points (aggressivity score, see Table 3) are assigned to each sample. Thus, the soils have a low corrosion potential.

Per Caltrans Corrosion Guidelines Version 3.2 dated May 2021, for structural elements, the Department considers a Site to be corrosive if one or more of the following conditions exist for the representative soil samples taken at the Site:

- Chloride concentration is 500 ppm or greater.
- Sulfate concentration is 1,500 ppm or greater.
- The pH is 5.5 or less.

Test results indicated chloride contents range from about 2.78 to 7.38 ppm, pH values range from 7.67 to 8.05 and sulfates contents range from about 42.6 to 68.3 ppm. Thus, the tested values fall below the range to be considered corrosive per Caltrans.

Minimum resistivity ranged from 6,200 to 32,000 ohm-cm indicative of mildly to negligible corrosive conditions per NACE (National Association of Corrosion Engineers) Corrosion Basics.

Based on laboratory test results, the subsurface conditions have a low potential for corrosion for buried ductile iron or steel elements.

According to ACI 318, soils containing less than 0.10% sulfates by weight are a “negligible” risk of sulfate exposure potential for concrete contacting soil. The sulfate content for the samples tested indicates less than 0.10% by weight. Therefore, normal Portland cement can be used for below grade concrete structures.

5. Discussion and Recommendations

5.1 General

The purpose of this investigation was to obtain geotechnical information regarding the subsurface and groundwater conditions and to provide geotechnical recommendations for the design and construction of the new slab-on-grade, pavements, mechanical and electrical foundations, as well as providing options for deep foundation support of heavier structures and enclosures.

Based upon the review of the GRSEA, subsequent e-mail correspondence and project meeting with QE and MFSE on May 19, 2025, GHD understands the hangars will be restored to their original appearance and condition. The existing exterior masonry walls will remain in place founded on the existing pile-supported foundation system including pile caps and grade beams. The existing slab-on-grade and interior walls will be removed and replaced. The new slab-on-grade is planned to include thickened slab edges that will be doweled into the existing grade beams. The anticipated maximum slab-on-grade load is 150 psf. New mechanical and electrical equipment are planned to be installed on 12-inch-thick concrete pads with the following proposed dimensions and maximum loads:

– Switchboard: 7,043 lb equipment on 108 square feet (sf) pad (4’-8” x 23’-0”), bearing pressure of 216 psf
– Transformer: 362 lb equipment on 6.7 sf pad (2’-6” x 2’-8”), bearing pressure of 205 psf
– Inverter: 4,063 lb equipment on 33.5 sf pad (3’-7” x 9’-4”), bearing pressure of 272 psf

The future use of the hangars is not presently known. Future development of the interior space of the hangars could include various heavier structures and enclosures.

Based on the above information summary and data obtained from the boreholes, and assuming it to be representative of the subsurface conditions, the following comments and recommendations are offered in relation to the proposed design and construction of the various Site improvements.

5.2 Earthwork

5.2.1 Site Preparation and Grading

GHD recommends that all existing asphalt pavement, concrete slab, surficial debris, surficial fill, existing structure foundations and other deleterious materials be completely removed from areas where the new pavements, slabs-on-grade and various structures will be constructed.

In planned pavement areas, the exposed subgrade should be compacted under the observation of qualified geotechnical personnel using a smooth vibratory roller if granular soils are present and a sheep’s foot roller if cohesive soils are present. The compacted subgrade should then be proof rolled to detect unstable areas under the observations of qualified geotechnical personnel. Proof rolling shall be performed by making a minimum of two complete passes in two perpendicular directions with a fully-loaded 20-ton minimum tandem-axle dump truck, or other approved rubber-tired vehicle. Areas showing excessive deflection or rutting (generally defined as greater than

½-inch) and as identified by the geotechnical personnel should be reworked to an appropriate moisture condition and re-compacted to a stable subgrade, or removed and replaced with engineered fill if judged to be unsuitable soil. If proof rolling is not feasible due to confined conditions, then visual inspection of the subgrade shall be performed by the geotechnical personnel to identify potentially unsuitable/unstable areas that require remediation. If necessary, use of geosynthetics (i.e., geogrid and/or geo-fabric) can reduce the thickness of the undercuts of unstable/unsuitable soils.

In slab-on-grade areas, the exposed subgrade should be undercut to a depth of 18 inches below the proposed base of slab. Then, InterAx NX750 geogrid (or engineer approved equivalent) should be placed in accordance with the manufacturer specifications including the required minimum overlaps. Backfill comprised of Type 4 crushed aggregate per Table 733-04A of the NYSDOT Standard Specifications should be placed per Section 5.2.2 over the geogrid to the planned top of subgrade. A final proof roll should be performed as described above prior to installation of the slab.

Refer to Sections 5.4 and 5.5 for subgrade preparation in foundation areas.

5.2.2 Backfilling

Engineered fill should consist of non-frozen, inorganic natural soils and/or granular fill free of deleterious materials or debris. The majority of the on-Site soils may be reused as engineered fill but may require moisture conditioning prior to reuse. Any silt, clay and/or soils that contain organics or deleterious materials are not suitable for reuse. Backfill should be as uniform as practical and mixing soil types should be avoided. An approved testing laboratory should be engaged to verify the proposed backfill material. Fine-grained backfill soils should be analyzed for plasticity to verify low to moderate plasticity and should have a liquid limit of less than 40 and a plasticity index of less than 20. The fill should be placed in maximum loose lifts of 10 inches (8 inches in foundation areas) and compacted by a heavy sheep's foot or smooth drum roller to a minimum of 95% of the maximum dry density as estimated by Modified Proctor (ASTM D1557). Engineered fill placement and compaction should be under the full-time observation of the on-Site Geotechnical Engineer or his/her designated representative (i.e., qualified geotechnical technician). In-situ density tests should be performed frequently to verify the specified degree of compaction has been met.

5.2.3 Utility Abandonment

For existing utilities encountered on-Site, if abandoned in place, it is recommended that utility pipes be bulk-headed and filled with flowable grout with a minimum compressive strength of 150 psi to avoid potential collapse in the future. Should the utility lines be removed from the Site, the resultant excavations should be properly backfilled and compacted as indicated above.

5.2.4 Excavation and Dewatering

All excavations should be carried out in accordance with Occupational Safety and Health Administration Regulations (Standards CFR 29). Temporary slopes in trenches greater than 4 feet in depth should be properly sloped or supported by means of a trench box or shield or other form of shoring. Excavations greater than 20 feet are required by OSHA to be designed by a professional engineer with bracing, shoring, or other appropriate method. A "competent person", as defined by OSHA, should continually evaluate soil conditions during trench excavation and decide on the appropriate trench protection on an ongoing and daily basis during excavation activities. OSHA regulations classify soils according to the following three categories:

Type A is meant for:

Cohesive soils with an unconfined compressive strength of 1.5 tsf or greater. However, no soil is Type A if:

- The soil is fissured.
- The soil is subject to vibration from heavy traffic, pile driving or similar effects.
- The soil has been previously disturbed.

– The soil is part of a sloped, layered system where the layers dip into the excavation on a slope of four horizontal to one vertical (4H:1V) or greater.
– The material is subject to other factors that would require it to be classified as a less stable material.

Type B is meant for:

– Cohesive soil with an unconfined compressive strength greater than 0.5 tsf but less than 1.5 tsf.
– Granular cohesionless soils including angular gravel (similar to crushed rock), silt, silt loam, sandy loam, and, in some cases, silty clay loam and sandy clay loam.
– Previously disturbed soils except those which would otherwise be classified as Type C soil.
– Soil that meets the unconfined compressive strength or cementation requirements for Type A but is fissured or subject to vibration.
– Dry rock that is not stable.
– Material that is part of a sloped, layered system where the layers dip into the excavation on a slope less steep than four horizontal to one vertical (4H:1V), but only if the material would otherwise be classified as Type B.

Type C is meant for:

– Cohesive soils with an unconfined compressive strength of 0.5 tsf or less.
– Granular soils including gravel, sand, and loamy sand.
– Submerged soil or soil from which water is freely seeping.
– Submerged rock that is unstable.
– Material in a sloped, layered system where the layers dip into the excavation or a slope of four horizontal to one vertical (4H:1V) or steeper.

Using the above OSHA guidelines, the soils encountered at the borehole locations are anticipated to be classified as Type ‘C’. The excavation contractor’s ‘responsible person’ as defined by OSHA is responsible for determining the OSHA soil classifications.

Where workers must enter a trench or excavation carried deeper than 4 feet, it must be suitably sloped and/or shored in accordance with the regulation requirements. The regulation stipulates the following maximum slopes for the excavation:

Table 4 Maximum Allowable Excavation Slopes

Soil or Rock Type	Maximum Allowable Slopes (H:V) for Excavations Less than 20 feet deep
Stable Rock	Vertical (90°)
Type A	0.75:1 (53°)
Type B	1:1 (45°)
Type C	1.5:1 (34°)

OSHA allows a short-term maximum slope of 0.5H:1V (63º) in an excavation in Type A soil that is 12 feet or less in depth. If there is an excavation that is comprised of more than one type of soil and the lower (worse) soil classification is lower in the excavation than the higher (better) soil classification, then the whole excavation must be considered with the lower (worse) soil classification. For example, if a Type C soil underlies a Type B soil, then the whole excavation should be considered as Type C soil. OSHA provides procedures for classifying soils in the field which should be conducted by the "competent person" as defined by OSHA.

Temporary construction loads (including, but not limited to, equipment and soil stockpiles) should be placed no closer than 5 feet from the slope crest and considered in the final excavation plan. If the excavation cannot be constructed with open stepped cuts due to the proximity to existing machinery, equipment and/or building foundations, then

temporary earth retention support will be required. For stepped slopes, the height of each step layer should be less than 3 feet. The existing foundations should be shored or underpinned. Care should be taken during construction to avoid damage or undermining of the drilled shafts during construction activities.

There are areas where temporary control of groundwater or seepage will be necessary, where high or perched groundwater levels are encountered. Groundwater could be expected to be encountered in excavations because of the presence of water bearing granular layers at the Site. Free water was observed in the boreholes as shallow as 10 feet bgs. If groundwater is encountered in shallow excavations (above 10 feet bgs), the flow and volume could be controllable during construction with a system of sumps and pumps. In excavations that extend into granular layers below the water table, significant groundwater inflow could be anticipated which would require well points.

5.2.5 Grading and Drainage

Grading should be designed so that water is not allowed to pond in the areas of the proposed Site upgrades. Grades should slope away from the planned structures with a minimum grade of 2%. Utility or equipment trenches that will run beneath proposed structures should be sealed to prevent water intrusion into the trenches that could collect beneath the structures and adversely impact the foundational support.

5.3 Slabs-on-Grade

Section 5.2.1 provides recommendations for preparation of the slab-on-grade subgrade with geogrid reinforcement. A geogrid reinforced subgrade is only suitable for support of slabs-on-grade with loads of 150 psf or less. Subgrade soils prepared as discussed in Section 5.2.1 are recommended to be designed using a modulus of subgrade reaction (k) value of 115 pounds per cubic inch (pci).

As discussed in Section 5.1, the new slab-on-grade is planned to include thickened slab edges that will be doweled into the existing grade beams. Typically, it is recommended to isolate all foundations from slabs-on-grade to allow for differential movement between the slabs and the foundations to take place. If the slabs will be structurally connected to the existing grade beams, the grade beams should be assessed to verify they are able to accommodate the additional loading from the slab connections. Additionally, as differential movement would be anticipated between the slabs-on-grade and the existing foundation system, the slab-on-grade design and structural connections should account for this to avoid formation of cracks in the slab.

To minimize the potential damage due to concrete shrinkage, control joints and expansion joints should be provided at a proper timing and at suitable intervals, per ACI guidelines.

5.4 Pad Foundations

This section provides recommendations for foundation pads (mats) that may potentially support the mechanical and electrical equipment and other lightly loaded at-grade structures inside the hangar. The various planned equipment pad sizes and loadings are summarized in Section 5.1. It is anticipated the top of the pads will project above the adjacent slabs-on-grade with the base of the pads projecting slightly below the bearing elevation of the slabs-on-grade.

The pad foundation subgrades should be prepared as discussed in Section 5.2.1. In non-heated areas, it is recommended the pad bear directly on capillary resistant granular engineered fill that extends to a minimum depth of 4 feet below final Site grades to protect against frost heave. The base of the pad foundations should bear no less than 8 inches below final Site grades. Foundation subgrades are anticipated to consist of very loose to medium dense sand fill that is considered unsuitable for support of the proposed pad foundations. Therefore, as an alternate to deep undercuts and replacement with engineered fill, GHD recommends a geogrid reinforced pad be constructed as follows.

– Step 1 – Over excavate to a minimum depth of 18 inches below the bottom of the foundation. As discussed above, this over excavation should extend even further to a minimum depth of 4 feet in non-heated areas with

placement and compaction of capillary resistant granular engineered fill up to a depth of 18 inches below the bottom of the foundation. The over excavation should extend a minimum of 3 feet beyond the edges of the proposed foundation in all directions.

- Step 2 – Place a layer of InterAx NX750 geogrid at a depth of 18 inches below the bottom of the foundation.
- Step 3 – Place and compact 12 inches of Type 4 crushed aggregate per Table 733-04A of the NYSDOT Standard Specifications.
- Step 4 – Place another layer of InterAx NX750 geogrid.
- Step 5 – Place and compact 6 inches of Type 4 crushed aggregate per Table 733-04A.

Note that the base of the pads are anticipated to project slightly below the adjacent slabs-on-grade (around 2 inches). In this case, the layer of geogrid that will be installed 18 inches below the base of the slabs-on-grade can transition to a depth of 18 inches below the base of the pads within the 3-foot zone of geogrid extension beyond the edge of the pads to allow for one continuous layer of geogrid.

If a single roll of geogrid is not sufficiently wide enough to complete coverage with a single pass, overlapping seams of geogrid will be required. The geogrid overlaps need to be a minimum of 3 feet.

Prior to placement of the geogrid and aggregate, the exposed subgrade of the over excavation should be visually inspected by the on-Site geotechnical engineer or their designated representative (i.e., qualified geotechnical technician) to verify the condition and composition of the subgrade. Should the subgrade conditions encountered vary from those found in the boreholes, GHD requests to be notified immediately to permit a reassessment of the recommendations in this section.

Foundation pads with subgrades prepared as described in this section should be designed for a net allowable bearing capacity of 1,200 pounds per square foot (psf) and a coefficient of sliding friction of 0.35 between the concrete and foundation subgrade material. The net allowable bearing capacity is the pressure that can be transmitted to the bearing strata in excess of the final minimum overburden pressure near the footing. The pad foundation constructed as recommended herein is anticipated to have maximum total and differential settlement of 1 inch and 0.5 inch, respectively, provided the pad foundation has a maximum width of 6 feet and maximum length of 25 feet. If pad foundations larger than this are desired or pad loads will exceed 1,200 psf, then alternate foundation support options should be utilized. Refer to Section 5.5 for discussion of other potential support options.

The pad foundations should be structurally separate (isolated) from the adjacent slabs-on-grade and existing foundations.

Foundations should be constructed as soon as possible after foundation excavation activities. If foundation excavations will be left open for an extended period of time, it is recommended the foundation subgrades be protected from weather and construction activities by placement of a thin mat of lean concrete. Water should not be allowed to pond in foundation excavations and foundation concrete shall not be placed on frozen or flooded subgrades.

5.5 Alternate Subgrade Support Options

The following sections provide recommendations for alternate support options for various structures if the following loading and configurations apply:

- Slabs-on-grade with floor loads more than 150 psf
- Foundations pads with bearing pressures more than 1,200 psf and/or a width of greater than 6 feet and/or a length greater than 25 feet

5.5.1 Micropiles

5.5.1.1 Allowable Axial Capacity

The following recommendations are for general planning and cost estimating purposes. The micropiles should be designed by an experienced micropile design-build contractor.

Micropiles utilized to support foundations and/or slabs-on-grade should have a minimum length of 45 feet with minimum diameter of 6 inches and full-length pile reinforcement. It is also recommended to have a minimum of 20 feet of permanent steel casing measured from the top of the pile and a minimum 25 feet grouted length. The grouted length of the micropiles will support the new axial loads via frictional resistance between the soil and grout interface. The micropiles are recommended to bear a minimum of 15 feet into the medium dense to dense sand layer. The depth of medium dense to dense sand encountered in the boreholes was variable, between about 23 to 28 feet bgs. Based upon the above recommended micropile configuration, an allowable axial capacity of 40 kips per micropile can be achieved. Total settlement of less than 0.5 inches is anticipated not including the elastic compression of the pile. Micropiles should be spaced no closer than 3 pile diameters apart (minimum of 18 inches). Recommended soil parameters to use for design of micropiles based upon axial loading are provided in Table 5.

Portland cement grout with a minimum 28-day compressive strength of 5,000 psi is recommended to be used for the grouting of the micropiles.

5.5.1.2 Allowable Lateral Capacity

If lateral loads on the micropiles are anticipated, the micropiles should be designed to accommodate the lateral loads or a portion of the micropiles should be battered. If a lateral load analysis is required, GHD recommends conducting an LPILE analysis. Recommended soil parameters to use in an LPILE analysis are provided in Table 5.

5.5.1.3 Micropile Loading Testing Recommendations

Typically, verification load testing is completed on a minimum of one pre-production pile (e.g., test pile) to verify adequacy of the installation method and design assumptions. The test pile will be used under high loads as a “sacrificial” pile to test the ultimate capacity and should not be used as part of the final structure. It is recommended to complete one verification load test consisting of a static load test conducted in accordance with ASTM D1143 with compressive loads up to 2.5 times the design strength of the pile. It should be verified that acceptance criteria have been met for the verification loading per ASTM D1143 prior to construction of production piles.

Proof load-testing of production piles should also occur after the grouted piles reach at least 50% of the design strength to verify the piles can carry the design loads without excess movement. The proof tests should be completed with loads that will not cause failure or damage to the micropile. It is recommended that proof testing be completed on a minimum of 3 piles or 5 percent of the production piles, whichever is more, with compressive loads up to 160 percent of the design load. The results of the proof testing should be evaluated to verify acceptance criteria per FHWA NHI-05-39 have been met. The micropile design may need to be modified if the testing fails to meet the acceptance criteria.

Loading schedules for the verification and proof tests should follow Deep Foundation Institute codes (2004).

5.5.2 Helical Piers

Helical piers are a proprietary system and should be designed and installed by an experienced design-build contractor. The design should address axial and lateral loads and uplift forces. Installation depths are dependent on installation torque and may vary from location to location due to Site subsurface variability. It is anticipated the helical piers will need to penetrate a minimum of 10 feet into the medium dense to dense sand. Recommended soil parameters to develop the design of helical piers are provided in Table 5.

All helical piers should be torqued to the required capacity, which may occur as soon as the helix (helices) penetrate a competent soil layer. The helical piers should be installed in the presence of the on-Site Geotechnical Engineer to confirm that the appropriate torque has been achieved. Helical pier torque tests can be performed at the Site using a test pier that matches the anticipated production piers. This can aid in determining the average pier depth throughout the Site. Torque readings should be recorded at 1-foot increments until the required design torque value is achieved.

As groundwater will be in contact with the piers, it is recommended the piers be galvanized in accordance with either ASTM A164 or ASTM B633. If refusal conditions or issues arise at premature depths during advancement of the pier, the installation should be halted, and the galvanized coating inspected for damage. If there is no damage observed, the pier can be installed in an offset location.

Helical pier foundation systems, if designed using industry standard methods and installed to the required torque value, typically do not settle beyond the limits required by structural engineers.

5.5.3 Soil Remediation

Shallow foundations and slabs-on-grade can be supported on remediated soils using aggregate or grout columns to improve the soil bearing capacity and minimize settlements. Such remediation options include vibro stone columns (VSCs), rammed aggregate piers (RAPs) or controlled modulus columns (CMCs). This option could eliminate the need for geogrid reinforced subgrades and/or deep foundations support and can provide flexibility for future retrofits to the hangars.

During the installation of VSCs and RAPs, the aggregate and surrounding soil is compacted through vibration techniques and down pressure, or a dynamic ramming process for RAPs. This increases the lateral stress in the adjacent soils and stiffens the composite aggregate/soil mass. VSCs typically consist of 22 to 24-inch diameter holes created by inserting a vibratory probe into the soil and laterally displacing it and minimizing the production of spoils. Once the hole is open, open-graded aggregate is then placed in lifts down the hole and compacted with vibration and down pressure which produces dense stone columns. RAPs vary from VSCs in that the holes are drilled, not displaced, which produces spoils. The holes are backfilled with open-graded aggregate in lifts that are compacted by applying a high frequency impact ramming energy. A third option for soil improvement is the installation of CMCs which can be installed either by soil displacement or soil drilling methods similar to VSCs and RAPs except that grout or concrete is placed by low pressure injection through the hollow core of the drilling tool. This generates rigid inclusions in the soil that provide a stiffened composite soil mass.

Prior to installation of the VSCs, RAPs or CMCs, the Site should be brought to grade and a 24-inch aggregate pad (often referred to as the load transfer pad or LTP) should be installed in the building areas. This will provide a stable working platform for installation of the elements. The aggregate pad should consist of open-graded aggregate, placed in 8-to-12-inch lifts compacted to a minimum of 95% of the Modified Proctor value. VSCs, RAPs and CMCs can improve the allowable soil bearing capacity to 3,000 to 8,000 psf depending on the size, spacing and depths of the individual foundation elements. Based on soil conditions encountered in the boreholes, it is anticipated these elements would be installed to depths ranging from 30 to 40 feet bgs. Once installed, including the aggregate pad, shallow foundations and slabs-on-grade can be constructed to be supported on the stiffened composite soil matrix.

The vibrations generated by installation of these type of subgrade remediation elements are generally much less than vibrations generated by driving piles into the ground. Also, these soil improvement methods are typically more cost effective than traditional deep foundations such as driven piles, drilled shafts and auger cast-in-place piles. The design of VSCs, RAPs and CMCs are typically part of the foundation contractor’s design-build scope of work. The contractor should be responsible for providing detailed design calculations and plans sealed by a professional engineer licensed in the State of New York. Column/pier diameter, installation depth and spacing, aggregate or concrete/grout requirements, and settlement and bearing capacity evaluations should be included in the design. The contractor should be provided with this geotechnical report. The contractor may request that additional Site investigations and laboratory testing be performed to confirm their design. A qualified geotechnical consultant should be contracted to oversee the load tests and construction of the reinforcement elements. It is recommended that Menard and/or Geopier Foundation Company, Inc. be contacted to further explore the feasibility of this type of soil remediation method.

5.6 Geogrid Removal for Future Development

The future use of the rehabilitated hangars is not presently known. It may be possible that retrofits for heavier structures will be implemented that could require penetrations through and/or removal of geogrid reinforced subgrades. If removal of geogrid is required, it is recommended the removals include clean cuts of the geogrid with appropriate tooling and avoid rotary action that can result in torquing or pulling on the geogrid such as could occur with augering, helical pier advancement or other similar drilling advancement methodologies. Care should be taken if excavator buckets are used to remove the geogrid reinforced subgrades to avoid tugging or pulling on the geogrid.

5.7 Soil Parameters Summary

The recommended soil parameters for use in the design of the Site rehabilitation are as follows:

Table 5 Soil Parameters and Lateral Earth Pressure Coefficients

Soil Layer / LPILE p-y Curve Type	Depth (feet bgs)	Total Unit Weight (lb/ft^3)	Effective Unit Weight (lb/ft^3)	φ' – Drained Angle of Internal Friction (degrees)[1]	Undrained Shear Strength (psf)[1]	Strain at 50% Max. Stress (e_{50})[2]	Lateral Subgrade Modulus, k (pci)[2]	Earth Pressure Coefficients		
								K_a	K_o	K_p
Fill Sand / Sand (Reese)	0 – 15	115	53	29	--	N/A	25	0.35	0.52	2.88
Fill Very Soft Clay and Silt / Soft Clay (Matlock)	15 – 20	120	58	26	400	0.02	N/A	0.39	0.56	2.56
Native Very Loose to Loose Sand / Sand (Reese)	20 – 25	115	53	28	--	N/A	20	0.36	0.53	2.77
Native Medium Dense to Very Dense Sand / Sand (Reese)	25 – 50	120	58	36	--	N/A	110	0.26	0.41	3.85

Notes:

1. Drained angle of internal friction shall be used for drained analyses using a drained cohesion (c') of 0 psf. The undrained shear strength shall be used for undrained analyses using an angle of internal friction (φ) of 0 degrees.
2. Parameters specific to p-y curve models in LPILE

The calculation of the earth pressure coefficients is based on Rankine theory, which provides a conservative estimate as no friction between the soil and the structure is accounted for in the estimate. The earth pressure coefficients provided above are only applicable for flat ground surfaces beyond the structure and must be increased for sloping ground surfaces.

The pressure coefficients referenced within the above table are a function of the friction angle of the adjacent soil, and both the degree and direction of movement of the structure subjected to unbalanced lateral earth pressures. For structures that are restrained at the top, the at-rest earth pressure coefficient will apply. For structures that allow for 1 to 10% of exposed shoring height movement into the soil, the full passive earth pressure coefficient will apply.

5.8 Seismic Considerations

The International Building Code (IBC) requires the assignment of a Seismic Site Class, in accordance with ASCE 7–22, for calculations of earthquake design forces and the structural design based on a two percent probability of

exceedance in 50 years. The Seismic Site Class is a function of soil profile and is based on the average properties of the subsurface strata to a depth of 100 feet below the ground surface. The IBC provides the following three methods to obtain the average properties for the top 100 feet of the subsoil strata:

– Average shear wave velocity.
– Average SPT values (uncorrected for overburden).
– Average undrained shear strength.

The geotechnical data comprised of SPT N value measurements that extended to a maximum depth of 50 feet below existing grade. Based on this data obtained from the boreholes and GHD's knowledge of the local geology, the average SPT value $\overline{N}$ for the top 100 feet at the Site is estimated to be less than 15 bpf. Per the IBC 2024 and ASCE 7-22, Site Class E is recommended for seismic design of the Site.

Based on the Site location, Seismic Site Class E and an assumed Risk Category II, the mapped seismic design parameters for this Site are as follows:

– PGA = 0.160 g.
– S_S = 0.260 g.
– S_1 = 0.049 g.
– S_{DS} = 0.170 g.
– S_{D1} = 0.064 g.

5.9 Pavement

The following recommendations apply to pavements with subgrades prepared as outlined in Section 5.2.1.

5.9.1 Pavement Design Recommendations

The design criteria considered in our evaluation for flexible pavement for the Site is summarized in Table 6.

Table 6 Flexible Pavement Basis of Design

Pavement Design Parameter	Assumed Parameter Value
Basis of Design (yrs)	20
20 Year Standard Duty ESAL (18 kip)	25,000
20 Year Heavy Duty ESAL (18 kip)	200,000
Initial Serviceability	4.5
Terminal Serviceability	2.25
Standard Deviation	0.45
Reliability	85%

The following bituminous concrete and aggregate base thickness for a flexible pavement may be used for the design of the exterior pavement area. A California Bearing Ratio (CBR) value of 7 was used for pavement design based on the soil conditions encountered in the boreholes.

Table 7 Pavement Structure Summary - Flexible

Pavement Layer	Light Duty Pavement Design	Heavy Duty Pavement Design
Surface Course Asphalt Concrete	1.5	1.5
Base Course Asphalt Concrete	2.5	3.5
Sub-base (Type 4 crushed aggregate, Table 733-04A)	6.0	6.0

Asphaltic concrete is not designed for standing loads and if used can lead to major rutting under wheel loads over time, leading to water ponding and associated problems. A rigid pavement structure comprised of Portland cement concrete (PCC) is recommended to limit life-cycle costs where heavy-duty vehicles will be parked for a long-term basis. A 10-inch-thick PCC slab can be placed on a granular material (minimum of 6 inches of Type 4 crushed aggregate per Table 733-04A), compacted to 95% of the Modified Proctor maximum dry density to provide proper grade and cross fall to drainage facilities. The PCC should be 4,500 psi with maximum 1.5-inch nominal size aggregate with some air-entrainment to tolerate freeze-thaw cycles. Either plain concrete with suitably spaced construction/shrinkage joints and load-transfer dowels or continuously reinforced PCC pavement can be considered. The design of PCC slabs should be performed by a structural engineer, based on the anticipated loading conditions.

It is recommended underdrains be utilized where catch basins are proposed and other low-lying areas of the pavement to limit the accumulation of water below the pavement section. At curbs, surface edge drains should be used.

5.10 Construction Monitoring and Testing

Qualified geotechnical consulting personnel should inspect and test the subgrade surfaces during construction. Specifically, they should verify that the materials and conditions exposed at the subgrade depth are consistent with the findings of this geotechnical report. Should subsoil conditions be encountered that vary considerably from those described in this report, our office should be informed immediately so that in-situ conditions can be assessed, and appropriate recommendations can be made.

The conditions will vary across the Site depending on the final design grades and therefore, the preparation of the subgrade and compaction of all fill should be monitored at the time of construction to confirm their suitability. In addition, qualified geotechnical personnel should provide materials testing services prior to and during subgrade preparation (i.e., materials suitability assessment of on-Site and imported fills, compaction testing, etc.).

All backfilling should be supervised so that proper materials are used, and that adequate compaction is achieved. Field density testing can be performed on subgrade layers to verify proper compaction and readiness for concrete placement upon completion of the backfilling process. Strict quality control guidelines should be followed during the placement of fill materials.

6. Limitations of the Report

This report has been prepared by GHD for Quinn Evans Architects (Client) and other parties explicitly identified in the report and is prohibited for use by others without GHD’s prior written consent. GHD otherwise disclaims responsibility to any person other than Client and approved parties arising in connection with this report. GHD also excludes implied warranties and conditions, to the extent legally permissible. The services undertaken by GHD in connection with preparing this report were limited to those specifically detailed in the report and are subject to the scope limitations set out in the report. No portion of this report may be used as a separate entity; it is to be read in its entirety and shall include all supporting drawings and appendices.

The opinions, conclusions and any recommendations in this report are based on GHD's present understanding of the project, conditions encountered, current Site use, ground surface elevations, and information reviewed at the date of

preparation of the report. The opinions, conclusions and any recommendations in this report are based on assumptions made by GHD described in this report. GHD disclaims liability arising from any of the assumptions being incorrect.

GHD has no responsibility or obligation to update this report to account for events or changes occurring subsequent to the date that the report was prepared. GHD should, however, be retained to review the recommendations when the drawings and specifications for the project are complete. Without this review, GHD will not be liable for any misunderstanding of the recommendations or their application and adaptation into the final design.

The services were performed in a manner consistent with that level of care and skill ordinarily exercised by members of geotechnical engineering professions currently practicing under similar conditions in the same locality. No other representations, and no warranties or representations of any kind, either expressed or implied, are made. Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties.

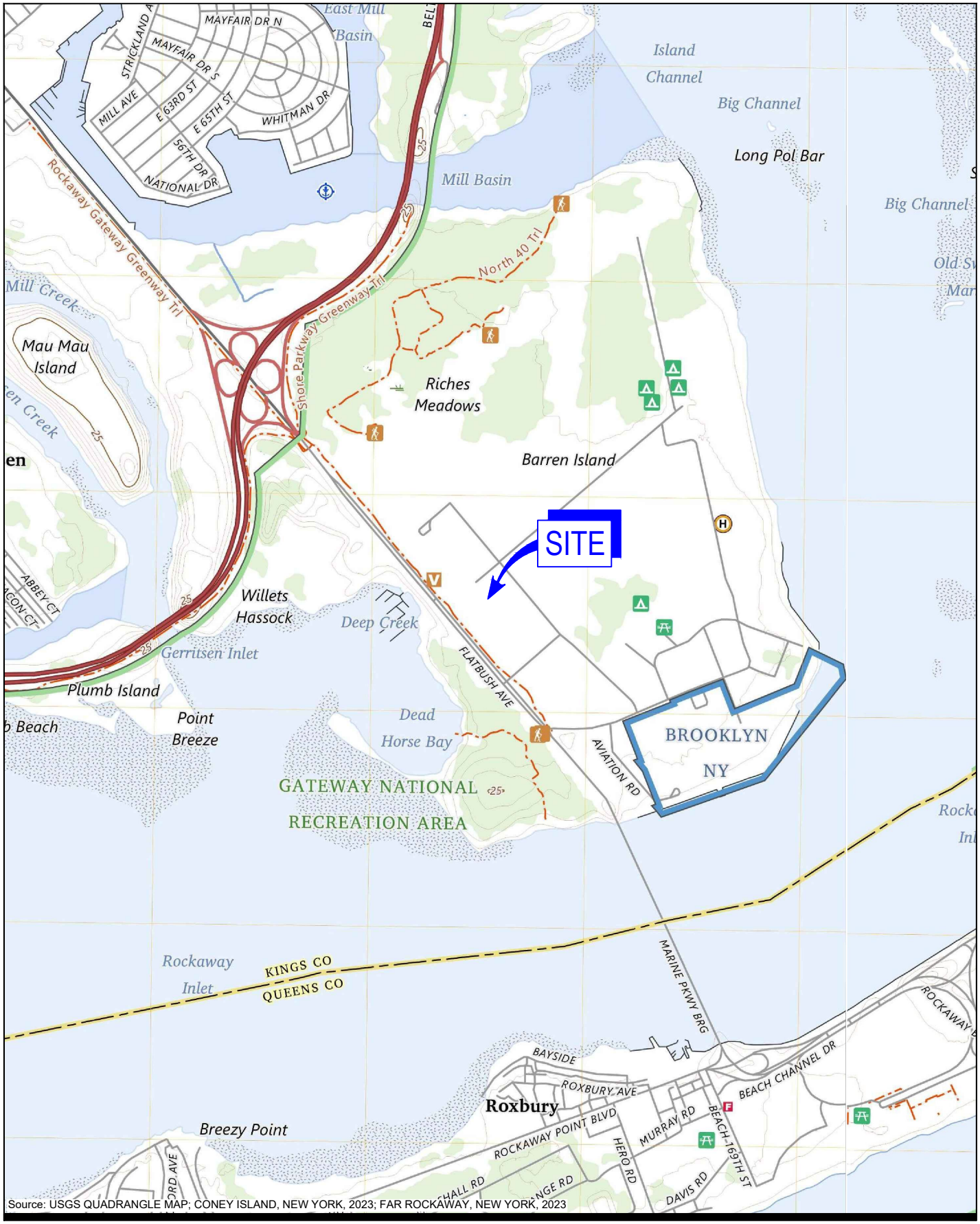
By issuing this report, GHD is the Geotechnical Engineer of Record. It is recommended that GHD be retained during construction and earthwork operations to confirm the conditions of the subsoil are actually similar to those observed during the study. The intent of this requirement is to verify that conditions encountered during construction are consistent with the findings in the report and that inherent knowledge developed as part of the study is correctly carried forward to the construction phases.

It is important to emphasize that a soil investigation is a random sampling of a Site and the comments included in this report are based on the results obtained at the test locations only (4 borehole locations). The subsurface conditions confirmed at the test locations may vary at other locations. The subsurface conditions can also be significantly modified by the construction activities on Site (excavation, dewatering and drainage, etc.). These conditions can also be modified by exposure of soils or bedrock to humidity, dry periods or frost. Soil and groundwater conditions beyond the test location may differ both horizontally and vertically from those encountered at the test location and conditions may become apparent during construction which could not be detected or anticipated at the time of GHD's investigation. Should any conditions at the Site be encountered which differ from those found at the test locations, GHD requests to be notified immediately to permit a reassessment of the recommendations in this report. If changed conditions are identified during construction, no matter how minor, the recommendations in this report shall be considered invalid until sufficient review and written assessment of said conditions by GHD is completed.

All of which is respectfully submitted,

Jeff Snyder
Senior Geotechnical Engineer

Scott Adamowski
Senior Engineer



Source: USGS QUADRANGLE MAP; CONEY ISLAND, NEW YORK, 2023; FAR ROCKAWAY, NEW YORK, 2023

0 1000 2000 ft

1" = 2000 ft

Coordinate System:
STATE PLANE
NY83-LIF

N

GHD

FLOYD BENNETT FIELD HANGARS
3&4 REHABILITATION
3159 FLATBUSH AVENUE
BROOKLYN, NEW YORK

SITE LOCATION

Project No. **12667502**
Date **April 2025**

FIGURE 1

Filename: N:\US\Detroit\Projects\563\12667502\Digital_Design\ACAD\Figures\RPT001\12667502-GHD-00-00-RPT-EN-D101_DE-001.dwg
Plot Date: 24 April 2025 11:06 AM

0 50 100 ft

1" = 100 ft

Coordinate System:
STATE PLANE
NY83-LIF

N

GHD

FLOYD BENNETT FIELD HANGARS
3&4 REHABILITATION
3159 FLATBUSH AVENUE
BROOKLYN, NEW YORK

BOREHOLE LOCATIONS

Project No. **12667502**
Date **April 2025**

FIGURE 2

Filename: N:\US\Detroit\Projects\563\12667502\Digital_Design\ACAD\Figures\RPT001\12667502-GHD-00-00-RPT-EN-D102_DE-001.dwg
Plot Date: 24 April 2025 11:07 AM

Appendices

Appendix A

Borehole Logs

USCS CLASSIFICATION OF SOILS (ASTM D2487)

Coarse grained or granular soils have more than 50% of their dry weight retained on a #200 sieve; they are described as boulders, cobbles, gravel or sand. Fine grained soils have less than 50% of their dry weight retained on a #200 sieve; they are described as clay or clayey silt if they are cohesive, and silt if they are non-cohesive. In addition to gradation, granular soils are defined on the bases of their relative in-place density and fine grained soils on the basis of their strength or consistency and their plasticity.

Major Division		Classification Criteria		Group Symbol and Name	Modified Descriptors*
Coarse Grained Soils >50% retained on No. 200 sieve	Gravel % Gravel > % Sand	< 5% Fines	Cu ≥ 4 and 1 ≤ Cc ≤ 3	**GW** - Well Graded GRAVEL	<5% fines = trace clay and/or silt <15% sand = trace sand 15% - 30% sand = with sand 30+% sand = sandy
			Cu < 4 and/or (Cc < 1 or Cc > 3)	**GP** - Poorly Graded GRAVEL	
		5% - 12% Fines	Use GW/GP gradation and GM/GC fines descriptors	**[GW or GP] - [GM or GC]** - Well or Poorly Graded GRAVEL with Silt or Clay or Silty Clay	
		>12% Fines	fines = ML or MH	**GM** - Silty GRAVEL	
			fines = CL or CH	**GC**- Clayey GRAVEL	
			fines = CL-ML	**GC-GM** - Silty, Clayey GRAVEL	
	Sand % Sand > % Gravel	< 5% Fines	Cu ≥ 6 and 1 ≤ Cc ≤ 3	**SW** - Well Graded SAND	<5% fines = trace clay and/or silt <15% gravel = trace gravel 15% - 30% gravel = with gravel >30% gravel = gravelly
			Cu < 6 and/or (Cc < 1 or Cc > 3)	**SP** - Poorly Graded SAND	
		5% - 12% Fines	Use SW/SP gradation and SM/SC fines descriptors	**[SW or SP] - [SM or SC]** - Well or Poorly Graded SAND with Silt or Clay or Silty Clay	
		>12% Fines	fines = ML or MH	**SM** - Silty SAND	
			fines = CL or CH	**SC** - Clayey SAND	
			fines = CL-ML	**SM-SC** - Silty, Clayey SAND	

NOTE: $C_u = \frac{D_{60}}{D_{10}}$; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$

Grain Size Classification

Grain Type	Grain Size
Cobbles	Greater than 3 inches
Gravel	3 in - No. 4 (7.76 mm)
Sand	No. 4 (4.76 mm) - No. 200 (0.074 mm)
Silt	No. 200 (0.074 mm) - 0.002 mm
Clay	Less than 0.002 mm

Granular Soils Relative Density

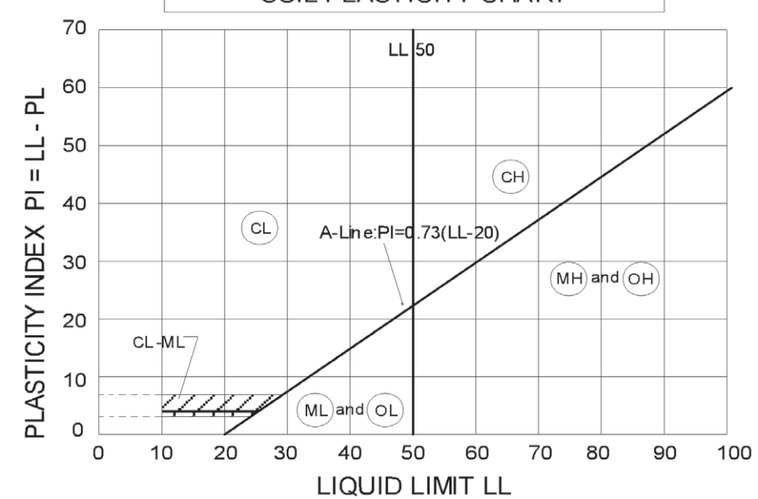
Relative Density	Blows Per Foot (N-Value)
Very Loose	<5
Loose	5 - 9
Medium Dense	10 - 29
Dense	30 - 50
Very Dense	>50

Cohesive (Clayey) Soils Consistency

Consistency	Unconfined Compressive Strength, Qu (tsf)
Very Soft	<0.25
Soft	0.25 - 0.49
Medium Stiff	0.50 - 0.99
Stiff	1.00 - 1.99
Very Stiff	2.00 - 3.99
Hard	>4.00

Major Division		Classification Criteria	Group Symbol and Name	Additional Descriptors		Modified Descriptors*
Fine Grained Soils > 50% passing No. 200 sieve	Clay Plots on or Above 'A'-Line	LL ≥ 50	**CH** - Fat CLAY	15% -29% plus No. 200: % Gravel > % Sand - with Gravel % Sand > % Gravel - with Sand	>30% plus No. 200 % Gravel > % Sand : Gravelly (with Sand if >15% sand) % Sand > % Gravel : Sandy (with Gravel if >15% gravel)	<15% sand/gravel = trace sand/gravel <15% silt/clay = trace silt/clay 15%-30% silt/clay = with silt/clay 30-50% silt/clay = silty/clayey 50%+ = and silt/clay
		LL < 50, PI > 7	**CL** - Lean CLAY			
	Silt Plots Below 'A'-Line	LL ≥ 50	**MH** - Elastic SILT			
		LL < 50, PI < 7	**ML** - SILT			
	Silty Clay Plots on or Above 'A'-Line	4 ≤ PI ≤ 7	**CL- ML** - SILTY CLAY			

SOIL PLASTICITY CHART



*Modified descriptions are not included in ASTM D2487 and provide additional information on the soil

REFERENCE No.: 12667502

ENCLOSURE No.:

BOREHOLE No.: B-1

ELEVATION: 13.7 ft

BOREHOLE REPORT

CLIENT: Quinn Evans Architects

PROJECT: NPS Hangars 3/4 Rehab Geotech

LOCATION: Brooklyn, NY

DESCRIBED BY: S. King

CHECKED BY: J. Snyder

DATE (START): April 15, 2025

DATE (FINISH): April 15, 2025

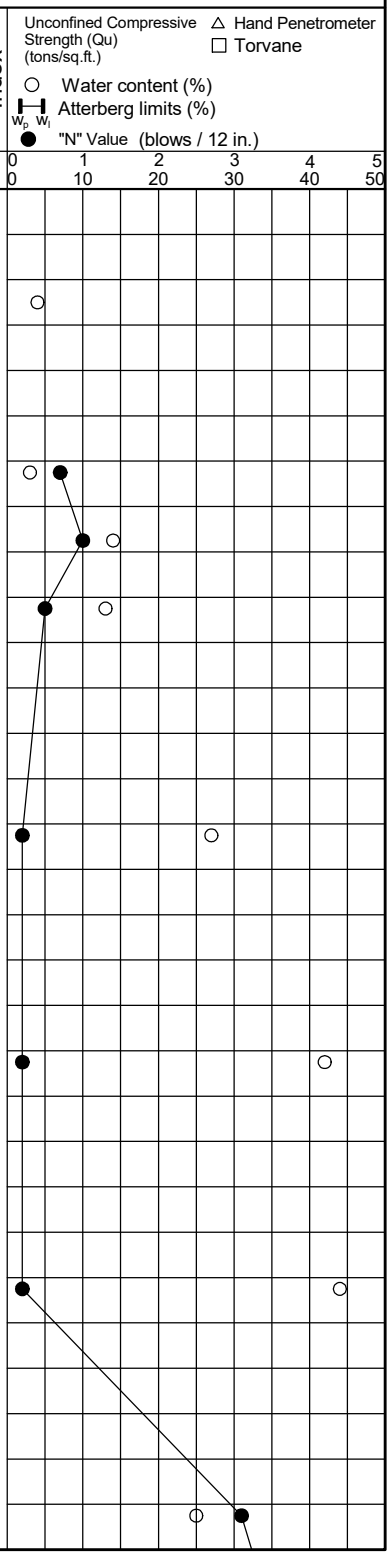
DRILLER: ADT-Cascade

DRILL RIG: Geoprobe 7822 DT

LEGEND

- SS - SPLIT SPOON
- ST - SHELBY TUBE
- RC - ROCK CORE
- WATER LEVEL
- GS - GRAB SAMPLE

Depth (Feet)	Elevation (ft)	Stratigraphy	DESCRIPTION OF SOIL AND BEDROCK	State	Type and Number	Recovery (RQD) %	Moisture Content	Blows per 6 in.	Penetraion Index N
	13.70		GROUND SURFACE						
	13.20		0.5 CONCRETE SLAB						
			(FILL) SAND, trace silt, brown and light brown, loose to medium dense, moist		GS-1		4	--	--
5					SS-2	100	3	3-3-4	7
					SS-3	100	14	4-5-5	10
	5.20		8.5 (FILL) SAND, trace silt and gravel, brown, loose, moist to wet		SS-4	58	13	2-3-2	5
10									
	0.20		13.5 (FILL) Silty SAND, trace gravel, gray, very loose, wet		SS-5	94	27	1-1-1	2
	-0.90		14.6 (FILL) Sandy CLAY, trace organics, roots, debris and wood chips, dark gray, very soft, moist						
15									
	-5.80		19.5 (SP) Sand, trace silt, gray, very loose, moist to wet		SS-6	94	42	1-1-1	2
20									
					SS-7	44	44	1-1-1	2
25									
	-14.80		28.5 (SP) Sand, trace silt, gray, medium dense to dense, moist to wet		SS-8	61	25	10-15-16	31



SOIL LOG WITH GRAPH 12667502 NPS FLOYD BENNET FIELD.GPJ CRA PLYMOUTH.GDT 5/8/25

REFERENCE No.: 12667502

ENCLOSURE No.:

BOREHOLE No.: B-1

ELEVATION: 13.7 ft

BOREHOLE REPORT

CLIENT: Quinn Evans Architects

PROJECT: NPS Hangars 3/4 Rehab Geotech

LOCATION: Brooklyn, NY

DESCRIBED BY: S. King

CHECKED BY: J. Snyder

DATE (START): April 15, 2025

DATE (FINISH): April 15, 2025

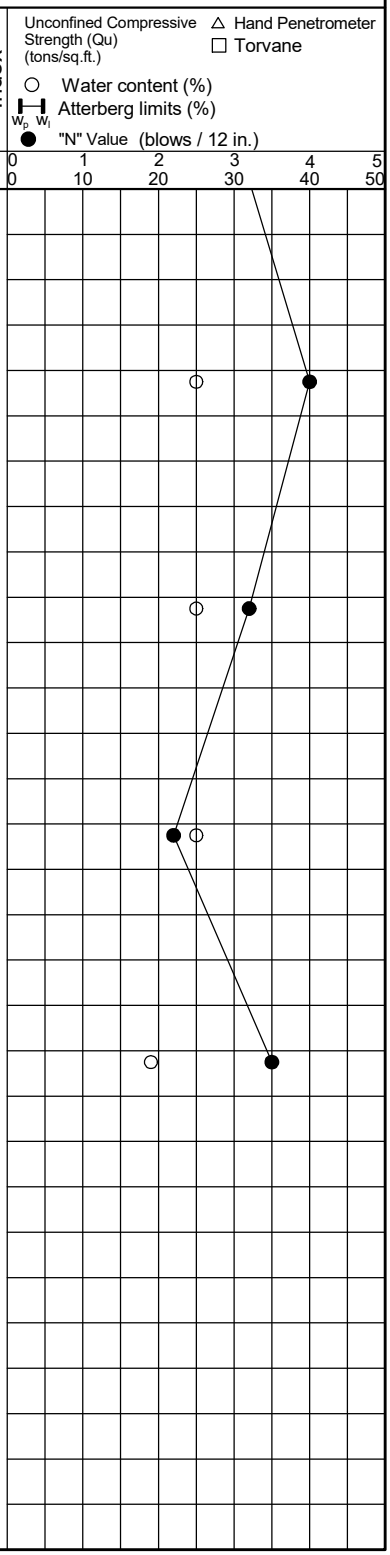
DRILLER: ADT-Cascade

DRILL RIG: Geoprobe 7822 DT

LEGEND

- SS - SPLIT SPOON
- ST - SHELBY TUBE
- RC - ROCK CORE
- WATER LEVEL
- GS - GRAB SAMPLE

Depth	Elevation (ft)	Stratigraphy	DESCRIPTION OF SOIL AND BEDROCK	State	Type and Number	Recovery (RQD)	Moisture Content	Blows per 6 in.	Penetraion Index
Feet	13.70		GROUND SURFACE			%			N
35					SS-9	55	25	14-21-19	40
40					SS-10	55	25	9-15-17	32
45					SS-11	50	25	11-11-11	22
50	-36.30		50.0 END OF BOREHOLE AT 50 FT BELOW GROUND SURFACE (BGS). HAND AUGER USED TO A DEPTH OF 5 FT BGS. SAMPLE GS-1 TAKEN FROM HAND AUGER CUTTINGS. BOREHOLE THEN ADVANCED TO 10 FT BGS USING 4.25-IN I.D. HOLLOW STEM AUGERS. THEN SWITCHED TO WASH ROTARY USING 3-7/8-IN TRI-CONE ROLLER BIT WITH BENTONITE/WATER AS DRILLING MUD TO ADVANCE BOREHOLE TO 50 FT BGS. BOREHOLE BACKFILLED WITH SOIL CUTTINGS AND SURFACE PATCHED. WL: N/A ft. WL (AB): N/A ft.		SS-12	58	19	16-18-17	35
55									



SOIL LOG WITH GRAPH 12667502 NPS FLOYD BENNET FIELD.GPJ CRA PLYMOUTH.GDT 5/8/25

REFERENCE No.: 12667502

ENCLOSURE No.:

BOREHOLE No.: B-2

ELEVATION: 13.5 ft

BOREHOLE REPORT

CLIENT: Quinn Evans Architects

PROJECT: NPS Hangars 3/4 Rehab Geotech

LOCATION: Brooklyn, NY

DESCRIBED BY: S. King

CHECKED BY: J. Snyder

DATE (START): April 16, 2025

DATE (FINISH): April 17, 2025

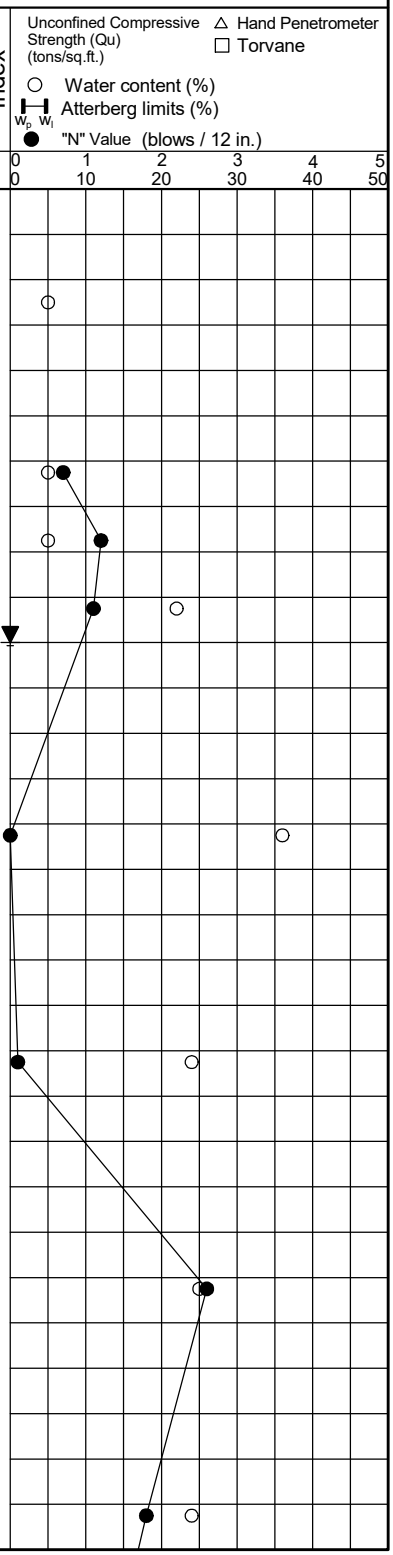
DRILLER: ADT-Cascade

DRILL RIG: Geoprobe 7822 DT

LEGEND

- SS - SPLIT SPOON
- ST - SHELBY TUBE
- RC - ROCK CORE
- WATER LEVEL
- GS - GRAB SAMPLE

Depth (Feet)	Elevation (ft)	Stratigraphy	DESCRIPTION OF SOIL AND BEDROCK	State	Type and Number	Recovery (RQD) %	Moisture Content	Blows per 6 in.	Penetraion Index N
	13.50		GROUND SURFACE						
	13.25		0.3 ASPHALT						
			(FILL) SAND, trace silt, brown and light brown, very loose to medium dense, moist						
					GS-1		5	--	--
5					SS-2	89	5	3-3-4	7
					SS-3	100	5	17-5-7	12
					SS-4	78	22	6-5-6	11
10									
					SS-5	5	36	0-0-0	0
15									
	-5.00		18.5 (SP-SC) SAND with clay and silt, gray, very loose, moist to wet		SS-6	100	24	1-0-1	1
20									
	-10.00		23.5 (SP) SAND, trace silt and gravel, gray, medium dense to dense, moist to wet		SS-7	78	25	10-14-12	26
25									
					SS-8	67	24	8-10-8	18



REFERENCE No.: 12667502

ENCLOSURE No.:

BOREHOLE No.: B-2

ELEVATION: 13.5 ft

BOREHOLE REPORT

CLIENT: Quinn Evans Architects

PROJECT: NPS Hangars 3/4 Rehab Geotech

LOCATION: Brooklyn, NY

DESCRIBED BY: S. King

CHECKED BY: J. Snyder

DATE (START): April 16, 2025

DATE (FINISH): April 17, 2025

DRILLER: ADT-Cascade

DRILL RIG: Geoprobe 7822 DT

LEGEND

- SS - SPLIT SPOON
- ST - SHELBY TUBE
- RC - ROCK CORE
- WATER LEVEL
- GS - GRAB SAMPLE

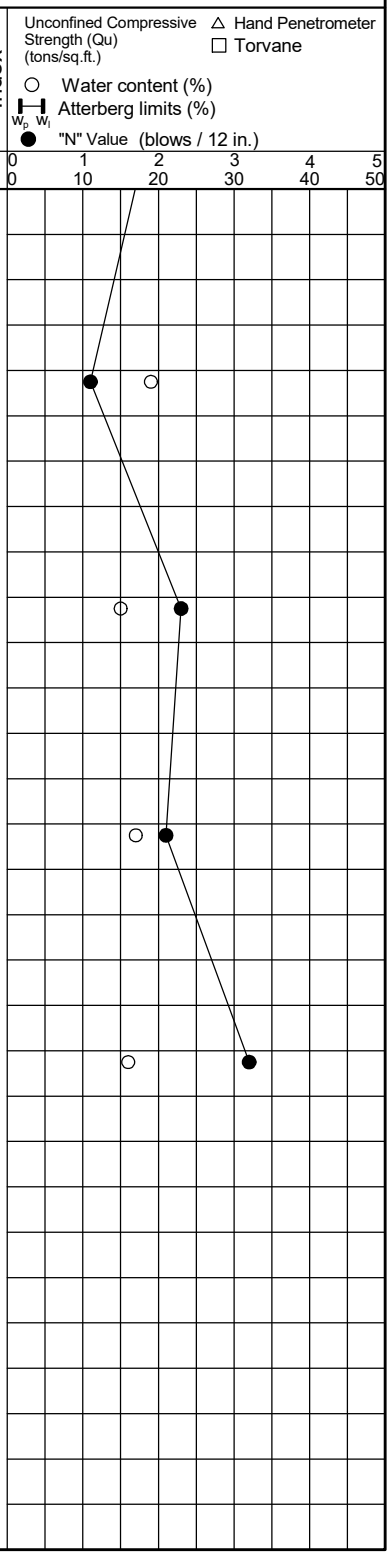
Depth	Elevation (ft)	Stratigraphy	DESCRIPTION OF SOIL AND BEDROCK	State	Type and Number	Recovery (RQD)	Moisture Content	Blows per 6 in.	Penetraion Index
Feet	13.50		GROUND SURFACE			%			N
35					SS-9	72	19	10-7-4	11
40					SS-10	78	15	10-11-12	23
45					SS-11	58	17	8-9-12	21
50	-36.50		50.0		SS-12	56	16	11-17-15	32
55									

END OF BOREHOLE AT 50 FT BELOW GROUND SURFACE (BGS). HAND AUGER USED TO A DEPTH OF 5 FT BGS. SAMPLE GS-1 TAKEN FROM HAND AUGER CUTTINGS. BOREHOLE THEN ADVANCED TO 10 FT BGS USING 4.25-IN I.D. HOLLOW STEM AUGERS. THEN SWITCHED TO WASH ROTARY USING 3-7/8-IN TRI-CONE ROLLER BIT WITH BENTONITE/WATER AS DRILLING MUD TO ADVANCE BOREHOLE TO 50 FT BGS.

BOREHOLE BACKFILLED WITH SOIL CUTTINGS AND SURFACE PATCHED.

WL: 10 ft.

WL (AB): N/A ft.



SOIL LOG WITH GRAPH 12667502 NPS FLOYD BENNET FIELD.GPJ CRA PLYMOUTH.GDT 5/8/25

REFERENCE No.: 12667502

ENCLOSURE No.:

BOREHOLE No.: B-3

ELEVATION: 13.5 ft

BOREHOLE REPORT

CLIENT: Quinn Evans Architects

PROJECT: NPS Hangars 3/4 Rehab Geotech

LOCATION: Brooklyn, NY

DESCRIBED BY: S. King

CHECKED BY: J. Snyder

DATE (START): April 18, 2025

DATE (FINISH): April 18, 2025

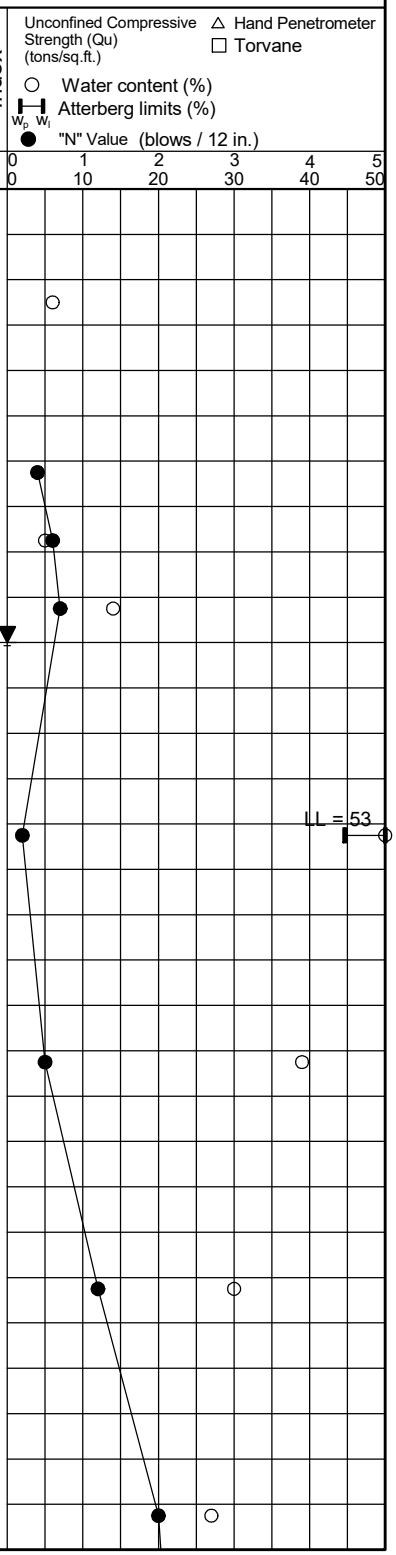
DRILLER: ADT-Cascade

DRILL RIG: Geoprobe 7822 DT

LEGEND

- SS - SPLIT SPOON
- ST - SHELBY TUBE
- RC - ROCK CORE
- WATER LEVEL
- GS - GRAB SAMPLE

Depth (Feet)	Elevation (ft)	Stratigraphy	DESCRIPTION OF SOIL AND BEDROCK	State	Type and Number	Recovery (RQD) %	Moisture Content	Blows per 6 in.	Penetraion Index N
	13.50		GROUND SURFACE						
	13.00		0.5 ASPHALT						
			(FILL) SAND, trace silt, brown and light brown, very loose to loose, moist		GS-1		6	--	--
5					SS-2	83	4	3-2-2	4
					SS-3	89	5	3-3-3	6
					SS-4	94	14	4-3-4	7
10									
	0.00		13.5 (FILL) Sandy SILT with clay, trace organics, roots and wood chips, gray, very soft, moist to wet, LL=53, PL=45, PI=8		SS-5	100	50	2-1-1	2
15									
	-5.50		19.0 (SP) SAND, trace silt, gray, loose, moist to wet		SS-6	100	39	1-1-4	5
20									
	-10.00		23.5 (SP) SAND, trace silt, gray, medium dense to dense, moist to wet		SS-7	67	30	4-5-7	12
25									
					SS-8	81	27	10-10-10	20



REFERENCE No.: 12667502

ENCLOSURE No.:

BOREHOLE No.: B-3

ELEVATION: 13.5 ft

BOREHOLE REPORT

CLIENT: Quinn Evans Architects

PROJECT: NPS Hangars 3/4 Rehab Geotech

LOCATION: Brooklyn, NY

DESCRIBED BY: S. King

CHECKED BY: J. Snyder

DATE (START): April 18, 2025

DATE (FINISH): April 18, 2025

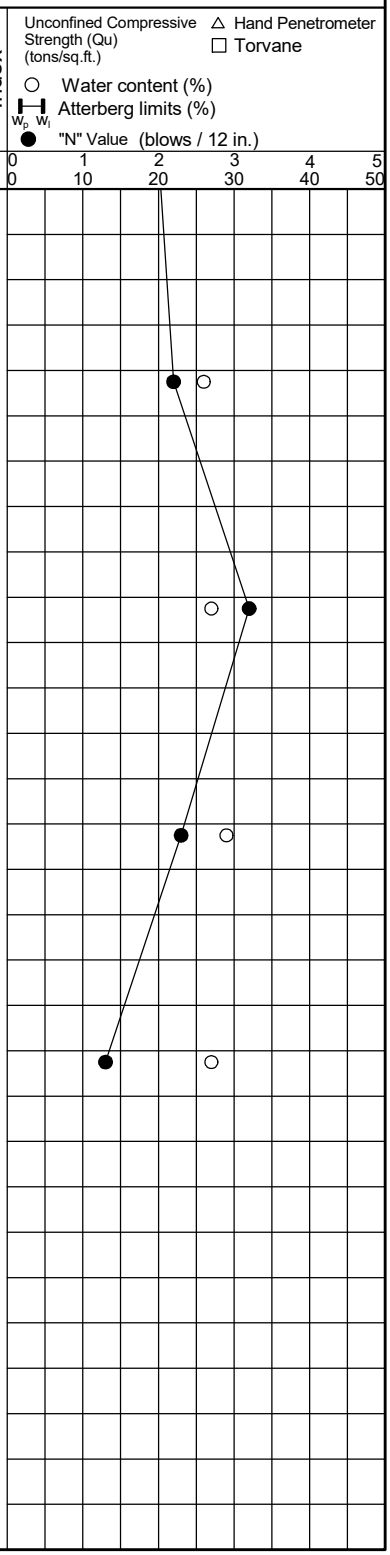
DRILLER: ADT-Cascade

DRILL RIG: Geoprobe 7822 DT

LEGEND

- SS - SPLIT SPOON
- ST - SHELBY TUBE
- RC - ROCK CORE
- WATER LEVEL
- GS - GRAB SAMPLE

Depth	Elevation (ft)	Stratigraphy	DESCRIPTION OF SOIL AND BEDROCK	State	Type and Number	Recovery (RQD)	Moisture Content	Blows per 6 in.	Penetraion Index
Feet	13.50		GROUND SURFACE			%			N
35					SS-9	69	26	7-10-12	22
40					SS-10	69	27	10-13-19	32
45					SS-11	75	29	9-12-11	23
50	-36.50		50.0		SS-12	67	27	5-8-5	13
55									



END OF BOREHOLE AT 50 FT BELOW GROUND SURFACE (BGS). HAND AUGER USED TO A DEPTH OF 5 FT BGS. SAMPLE GS-1 TAKEN FROM HAND AUGER CUTTINGS. BOREHOLE THEN ADVANCED TO 10 FT BGS USING 4.25-IN I.D. HOLLOW STEM AUGERS. THEN SWITCHED TO WASH ROTARY USING 3-7/8-IN TRI-CONE ROLLER BIT WITH BENTONITE/WATER AS DRILLING MUD TO ADVANCE BOREHOLE TO 50 FT BGS.

BOREHOLE BACKFILLED WITH SOIL CUTTINGS AND SURFACE PATCHED.

WL: 10 ft.

WL (AB): N/A ft.

REFERENCE No.: 12667502

ENCLOSURE No.:

BOREHOLE No.: B-4

ELEVATION: 13.5 ft

BOREHOLE REPORT

CLIENT: Quinn Evans Architects

PROJECT: NPS Hangars 3/4 Rehab Geotech

LOCATION: Brooklyn, NY

DESCRIBED BY: S. King

CHECKED BY: J. Snyder

DATE (START): April 17, 2025

DATE (FINISH): April 17, 2025

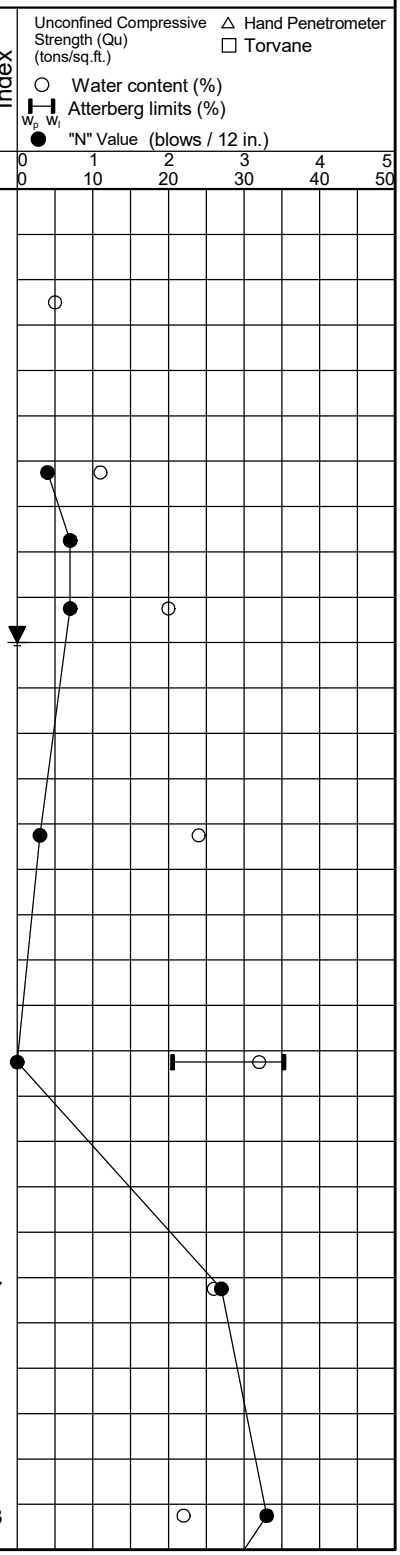
DRILLER: ADT-Cascade

DRILL RIG: Geoprobe 7822 DT

LEGEND

- SS - SPLIT SPOON
- ST - SHELBY TUBE
- RC - ROCK CORE
- WATER LEVEL
- GS - GRAB SAMPLE

Depth (Feet)	Elevation (ft)	Stratigraphy	DESCRIPTION OF SOIL AND BEDROCK	State	Type and Number	Recovery (RQD) %	Moisture Content	Blows per 6 in.	Penetraion Index N
	13.50		GROUND SURFACE						
	13.25		0.3 ASPHALT						
			(FILL) SAND, trace silt, brown and light brown, very loose to loose, moist		GS-1		5	--	--
5					SS-2	100	11	2-2-2	4
					SS-3	89	7	3-3-4	7
					SS-4	78	20	6-4-3	7
10									
	0.00		13.5 (SM) Silty SAND, trace gravel, dark gray, very loose, moist to wet		SS-5	42	24	3-2-1	3
15									
	-5.00		18.5 (SC) Clayey SAND with silt, trace shells, gray, very loose, moist to wet		SS-6	100	32	0-0-0	0
20									
	-10.00		23.5 (SP) SAND, trace silt, gray, medium dense to dense, moist to wet		SS-7	83	26	11-14-13	27
25									
					SS-8	58	22	10-17-16	33



SOIL LOG WITH GRAPH 12667502 NPS FLOYD BENNET FIELD.GPJ CRA PLYMOUTH.GDT 5/8/25

REFERENCE No.: 12667502

ENCLOSURE No.:

BOREHOLE No.: B-4

ELEVATION: 13.5 ft

BOREHOLE REPORT

CLIENT: Quinn Evans Architects

PROJECT: NPS Hangars 3/4 Rehab Geotech

LOCATION: Brooklyn, NY

DESCRIBED BY: S. King

CHECKED BY: J. Snyder

DATE (START): April 17, 2025

DATE (FINISH): April 17, 2025

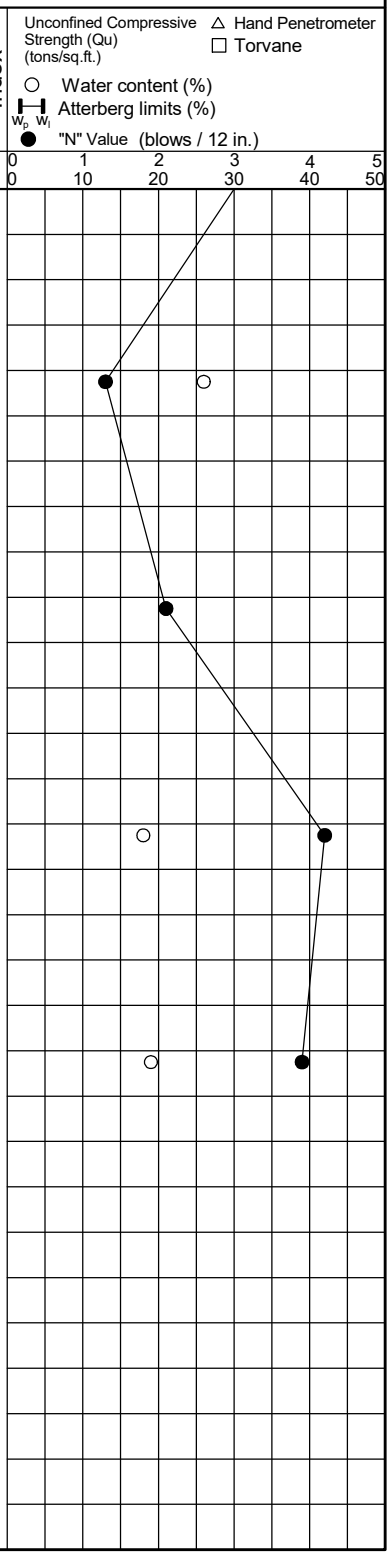
DRILLER: ADT-Cascade

DRILL RIG: Geoprobe 7822 DT

LEGEND

- SS - SPLIT SPOON
- ST - SHELBY TUBE
- RC - ROCK CORE
- WATER LEVEL
- GS - GRAB SAMPLE

Depth	Elevation (ft)	Stratigraphy	DESCRIPTION OF SOIL AND BEDROCK	State	Type and Number	Recovery (RQD)	Moisture Content	Blows per 6 in.	Penetraion Index
Feet	13.50		GROUND SURFACE			%			N
35					SS-9	58	26	7-6-7	13
40					SS-10	39	21	7-11-10	21
45					SS-11	58	18	20-21-21	42
50	-36.50		50.0 END OF BOREHOLE AT 50 FT BELOW GROUND SURFACE (BGS). HAND AUGER USED TO A DEPTH OF 5 FT BGS. SAMPLE GS-1 TAKEN FROM HAND AUGER CUTTINGS. BOREHOLE THEN ADVANCED TO 10 FT BGS USING 4.25-IN I.D. HOLLOW STEM AUGERS. THEN SWITCHED TO WASH ROTARY USING 3-7/8-IN TRI-CONE ROLLER BIT WITH BENTONITE/WATER AS DRILLING MUD TO ADVANCE BOREHOLE TO 50 FT BGS. BOREHOLE BACKFILLED WITH SOIL CUTTINGS AND SURFACE PATCHED. WL: 10 ft. WL (AB): N/A ft.		SS-12	53	19	15-16-23	39
55									



Appendix B

Geotechnical Laboratory Test Results

Borehole	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	Maximum Size (mm)	%<#200 Sieve	Class-ification	Water Content (%)	Dry Density (pcf)	Bulk Density (pcf)	Void Ratio
B-1	0							4.1			
B-1	5.5							2.9			
B-1	7							14.0			
B-1	8.5				9.375	1	SP	12.9			
B-1	13.5							26.9			
B-1	18.5							41.5			
B-1	23.5				2	1	SP	44.2			
B-1	28.5							25.1			
B-1	33.5							24.6			
B-1	38.5							24.5			
B-1	43.5							24.8			
B-1	48.5							18.9			
B-2	0							4.8			
B-2	5.5							4.5			
B-2	7							5.0			
B-2	8.5				2	1	SP	22.2			
B-2	13.5							35.8			
B-2	18.5	NP	NP	NP	2.36	9		24.4			
B-2	23.5							25.1			
B-2	28.5							24.2			
B-2	33.5							19.0			
B-2	38.5				9.375	2	SP	15.2			
B-2	43.5							16.5			
B-2	48.5							16.2			
B-3	0							5.9			
B-3	5.5							4.0			
B-3	7							4.8			
B-3	8.5							14.1			
B-3	13.5	53	45	8	4.75	53	MH	49.7			
B-3	18.5							39.3			
B-3	23.5							29.6			
B-3	28.5				2.36	1	SP	27.2			
B-3	33.5							25.9			
B-3	38.5							27.3			
B-3	43.5							28.9			
B-3	48.5							27.2			
B-4	0							4.9			
B-4	5.5							11.1			
B-4	7				4.75	1	SP	7.5			
B-4	8.5							20.1			
B-4	13.5							24.1			
B-4	18.5	35	21	14	4.75	48	SC	31.9			

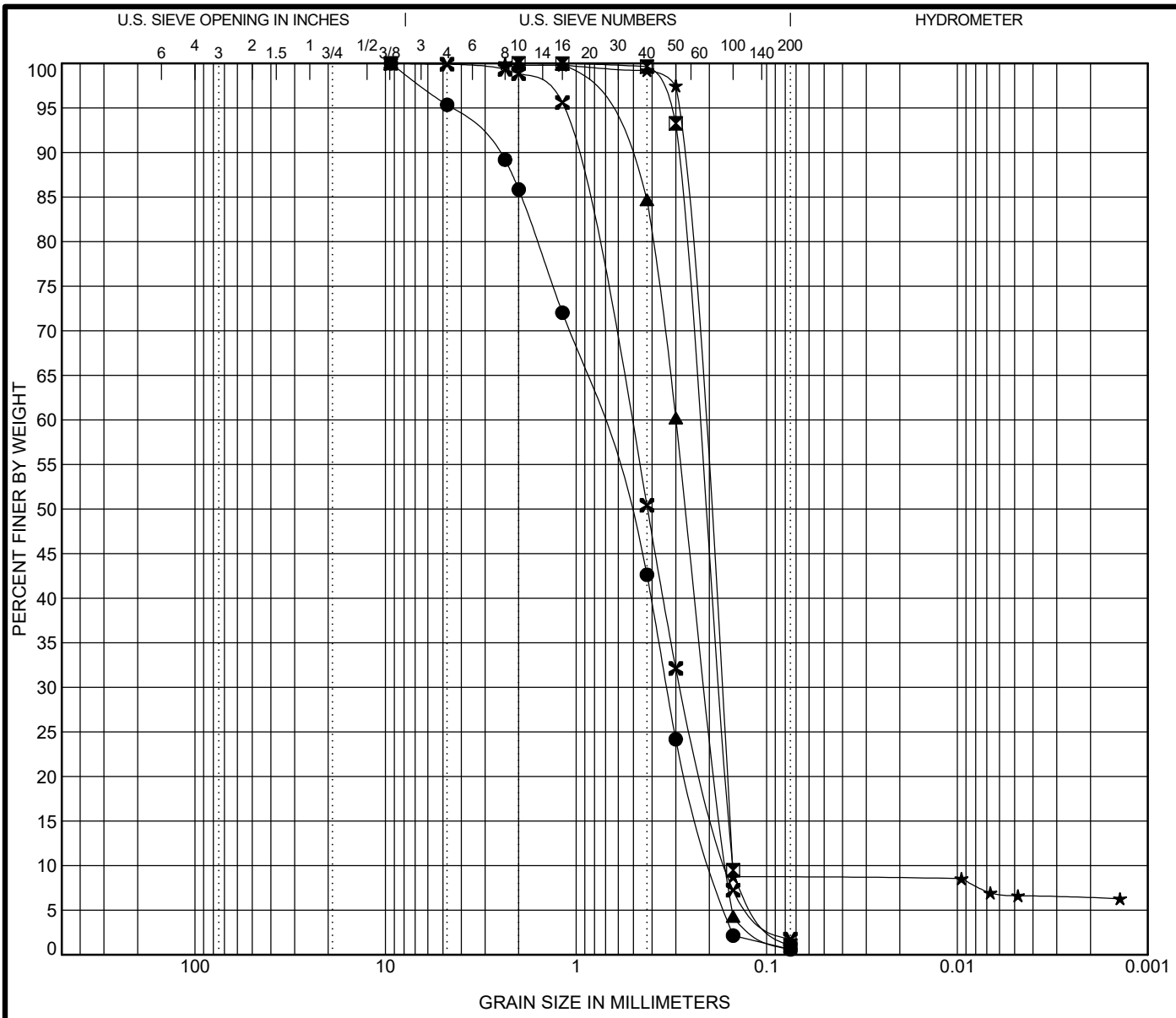
Summary of Laboratory Results

Project Name: NPS Hangars 3/4 Rehab Geotech
Project Number: 12667502
Client: Quinn Evans Architects
Location: Brooklyn, NY

Borehole	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	Maximum Size (mm)	%<#200 Sieve	Class-ification	Water Content (%)	Dry Density (pcf)	Bulk Density (pcf)	Void Ratio
B-4	23.5							25.7			
B-4	28.5							22.4			
B-4	33.5				4.75	1	SP	26.3			
B-4	38.5							21.3			
B-4	43.5							18.2			
B-4	48.5							19.3			

Summary of Laboratory Results

Project Name: NPS Hangars 3/4 Rehab Geotech
Project Number: 12667502
Client: Quinn Evans Architects
Location: Brooklyn, NY



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen Identification		Classification	LL	PL	PI	Cc	Cu
●	B-1	8.5	(SP) SAND, trace silt and gravel				0.75	4.05
⊠	B-1	23.5	(SP) SAND, trace silt				0.92	1.51
▲	B-2	8.5	(SP) SAND, trace silt				0.88	1.86
★	B-2	18.5	(SP-SC) SAND with clay and silt	NP	NP	NP	0.92	1.48
✕	B-2	38.5	(SP) SAND, trace silt and gravel				0.93	3.26

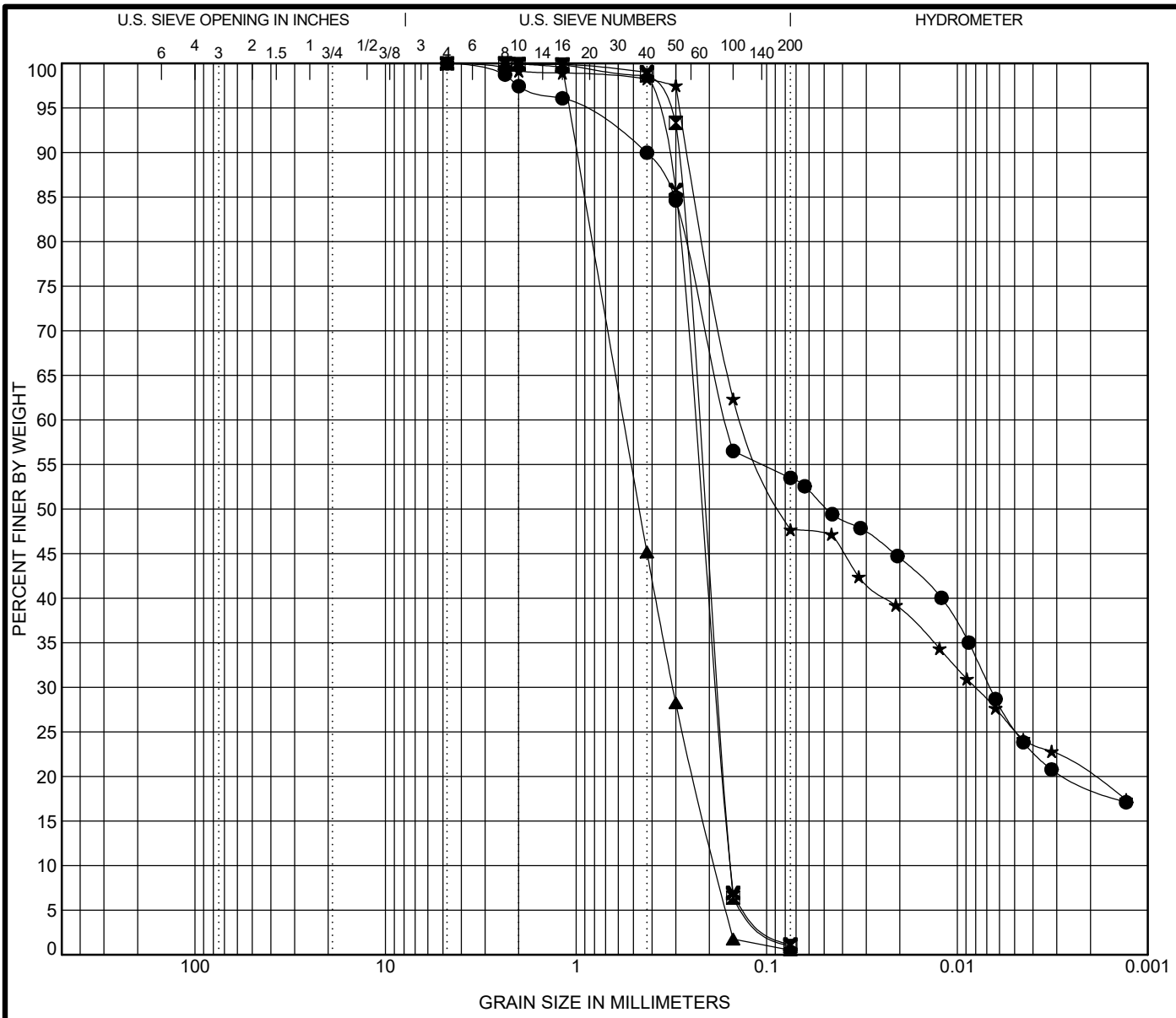
	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	B-1	8.5	9.375	0.777	0.335	0.192	4.6	94.8	0.6	
⊠	B-1	23.5	2	0.228	0.178	0.151	0.0	99.0	1.0	
▲	B-2	8.5	2	0.299	0.206	0.161	0.0	99.4	0.6	
★	B-2	18.5	2.36	0.224	0.177	0.151	0.0	91.3	2.3	6.4
✕	B-2	38.5	9.375	0.528	0.283	0.162	0.1	98.2	1.8	

GRAIN SIZE DISTRIBUTION (ASTM D422)

Project Name: NPS Hangars 3/4 Rehab Geotech
Project Number: 12667502
Client: Quinn Evans Architects
Location: Brooklyn, NY

Test by: K. Kube **Checked by:** J. Snyder **Test Date:** 5/5/2025

GRAIN SIZE 12667502 NPS FLOYD BENNET FIELD.GPJ GINT STD AGS 3_1.GDT 5/8/25



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen Identification		Classification	LL	PL	PI	Cc	Cu
●	B-3	13.5	(MH) Sandy SILT with clay	53	45	8		
⊠	B-3	28.5	(SP) SAND, trace silt				0.92	1.49
▲	B-4	7	(SP) SAND, trace silt				0.93	3.02
★	B-4	18.5	(SC) Clayey SAND with silt	35	21	14		
✕	B-4	33.5	(SP) SAND, trace silt				0.92	1.55

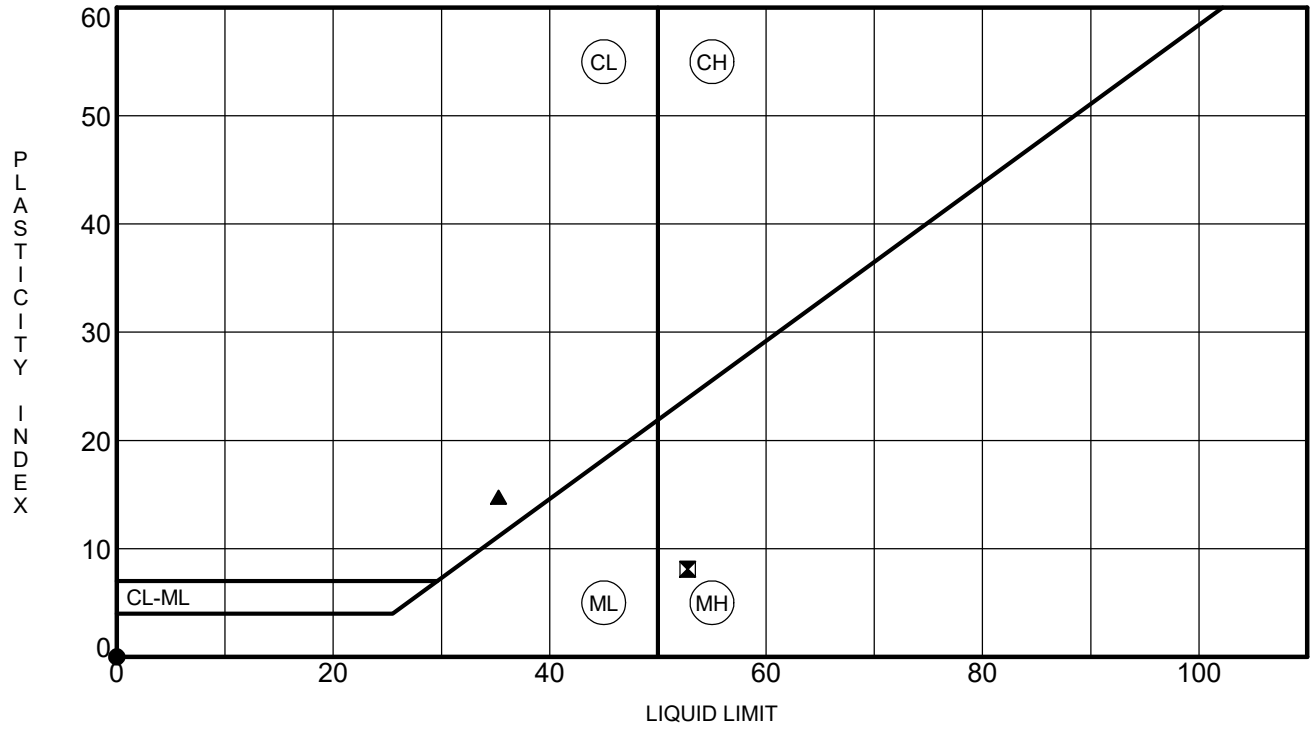
	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	B-3	13.5	4.75	0.163	0.007		0.0	46.5	34.6	18.9
⊠	B-3	28.5	2.36	0.23	0.181	0.154	0.0	99.0	1.0	
▲	B-4	7	4.75	0.561	0.311	0.186	0.0	99.5	0.5	
★	B-4	18.5	4.75	0.134	0.008		0.0	52.3	27.7	20.0
✕	B-4	33.5	4.75	0.239	0.184	0.154	0.0	98.8	1.2	

GRAIN SIZE DISTRIBUTION (ASTM D422)

Project Name: NPS Hangars 3/4 Rehab Geotech
Project Number: 12667502
Client: Quinn Evans Architects
Location: Brooklyn, NY

Test by: K. Kube **Checked by:** J. Snyder **Test Date:** 5/5/2025

GRAIN SIZE 12667502 NPS FLOYD BENNET FIELD.GPJ GINT STD AGS 3 1.GDT 5/8/25



	Specimen Identification		LL	PL	PI	Fines	Classification
●	**B-2**	**18.5**	**NP**	**NP**	**NP**	**9**	**(SP-SC) SAND with clay and silt**
⊠	**B-3**	**13.5**	**53**	**45**	**8**	**53**	**(MH) Sandy SILT with clay**
▲	**B-4**	**18.5**	**35**	**21**	**14**	**48**	**(SC) Clayey SAND with silt**

ATTERBERG LIMITS RESULTS (ASTM D4318)

Project Name: NPS Hangars 3/4 Rehab Geotech
Project Number: 12667502
Client: Quinn Evans Architects
Location: Brooklyn, NY

Test By: K. Kube **Checked By:** J. Snyder **Test Date:** 5/6/2025

Appendix C

Corrosion Potential Laboratory Test Results

May 12, 2025

Project No. 2025-252-001

Jeff Snyder
GHD Services Inc.
26850 Haggerty Rd.
Farmington Hills, MI 48170

Transmittal
Laboratory Test Results
Brooklyn, NY 1266 7502

Please find attached the laboratory test results for the above referenced project. The tests were outlined on the Project Verification Form that was transmitted to your firm prior to the testing. The testing was performed in general accordance with the methods listed on the enclosed data sheets. The test results are believed to be representative of the samples that were submitted for testing and are indicative only of the specimens that were evaluated. We have no direct knowledge of the origin of the samples and imply no position with regard to the nature of the test results, i.e. pass/fail and no claims as to the suitability of the material for its intended use.

The test data and all associated project information provided shall be held in strict confidence and disclosed to other parties only with authorization by our Client. The test data submitted herein is considered integral with this report and is not to be reproduced except in whole and only with the authorization of the Client and Geotechnics. The remaining sample materials for this project will be retained for a minimum of 90 days as directed by the Geotechnics' Quality Program.

We are pleased to provide these testing services. Should you have any questions or if we may be of further assistance, please contact our office.

Respectfully submitted,
Geotechnics, Inc.

Nathan Melaro
Director of Operations

We understand that you have a choice in your laboratory services and we thank you for choosing Geotechnics.

DCN: Data Transmittal Letter Date: 1/28/05 Rev.: 1

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CHLORIDE ION CONTENT IN SOILS

AASHTO T 291 - 94 (2018) (*Method B*)

Client:	GHD Services, Inc.	Boring No.:	B-1
Client Reference:	Brooklyn, NY 1266 7502	Depth (ft):	0.0-10.0'
Project No.:	2025-252-001	Sample No.:	GS-1
Lab ID:	2025-252-001-001	Description:	Brown

(- # 10 Sieve material)

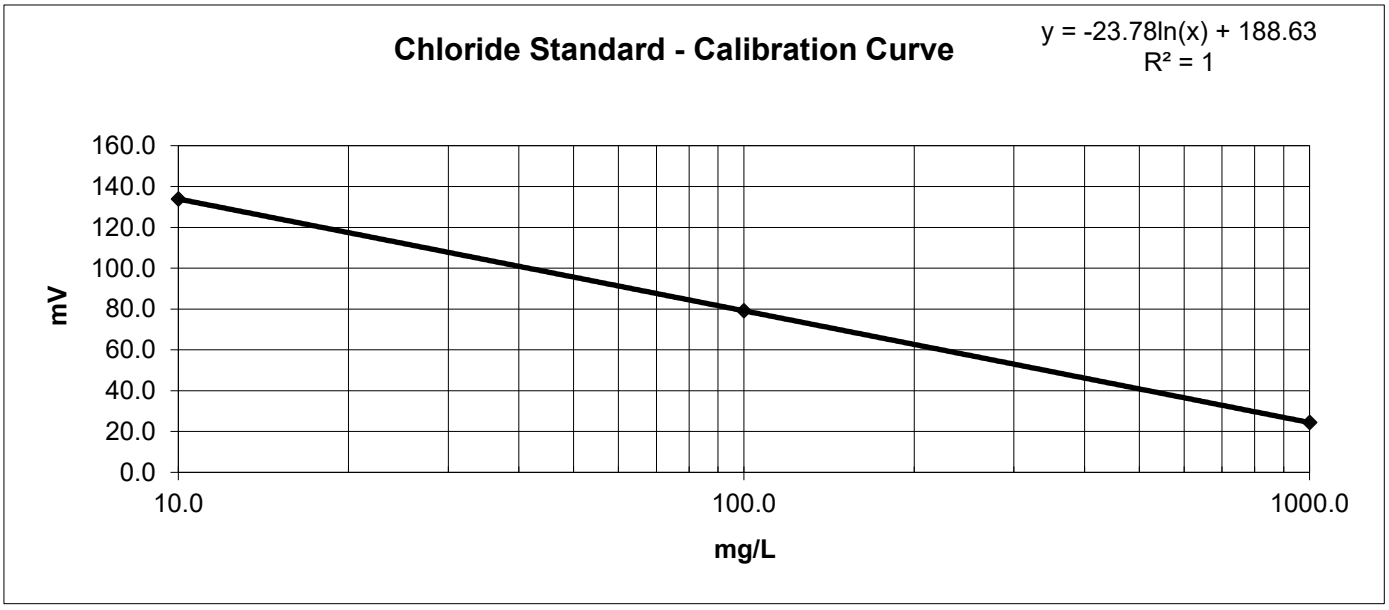
CHLORIDE STANDARD: CALIBRATION CURVE

STANDARD		MILLIVOLTS (mV)
10.0	mg/L	133.9
100.0	mg/L	79.1
1000.0	mg/L	24.4

MEASUREMENT OF CHLORIDES

		CONCENTRATION (mg/L)	CONCENTRATION (mg/kg)
Sample Weight (g):	100.0		
Water added to Sample (ml):	100.0		
Size of Sample Aliquot (ml):	25.0		
Sample Reading (mV):	141.1	7.38	7.38

Notes: 1) Samples and standards were buffered by the addition of an equal volume of the 0.2 M KNO_3 solution (1:1 volume).
2) Samples were dried for a minimum of 12 hours at 110 $^{+}/_{-}$ 5°C.



Notes:

Tested By	JAM	Date	5/8/25	Checked By	EG	Date	5/9/25

DCN: CT-S63A DATE: 6/2/14 REVISION: 1

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pH OF SOILS

AASHTO T 289-91 (2013)

Client:	GHD Services, Inc.
Client Reference:	Brooklyn, NY 1266 7502
Project No.:	2025-252-001

Lab ID:	001
Boring No.:	B-1
Depth (ft):	0.0-10.0'
Sample No.:	GS-1
Drying Tare No.:	2003
Testing Tare No.:	Z
Temperature (°C):	22.7
pH of Sample:	**8.05**

Meter Calibration (as used each day)		
Buffer pH	Meter Reading	Meter Model
4.00	4.00	ORION 720A
7.00	7.00	
10.00	10.00	

Tested By JAM Date 5/7/25 Checked By EG Date 5/8/25

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Minimum Resistivity

AASHTO T288-12

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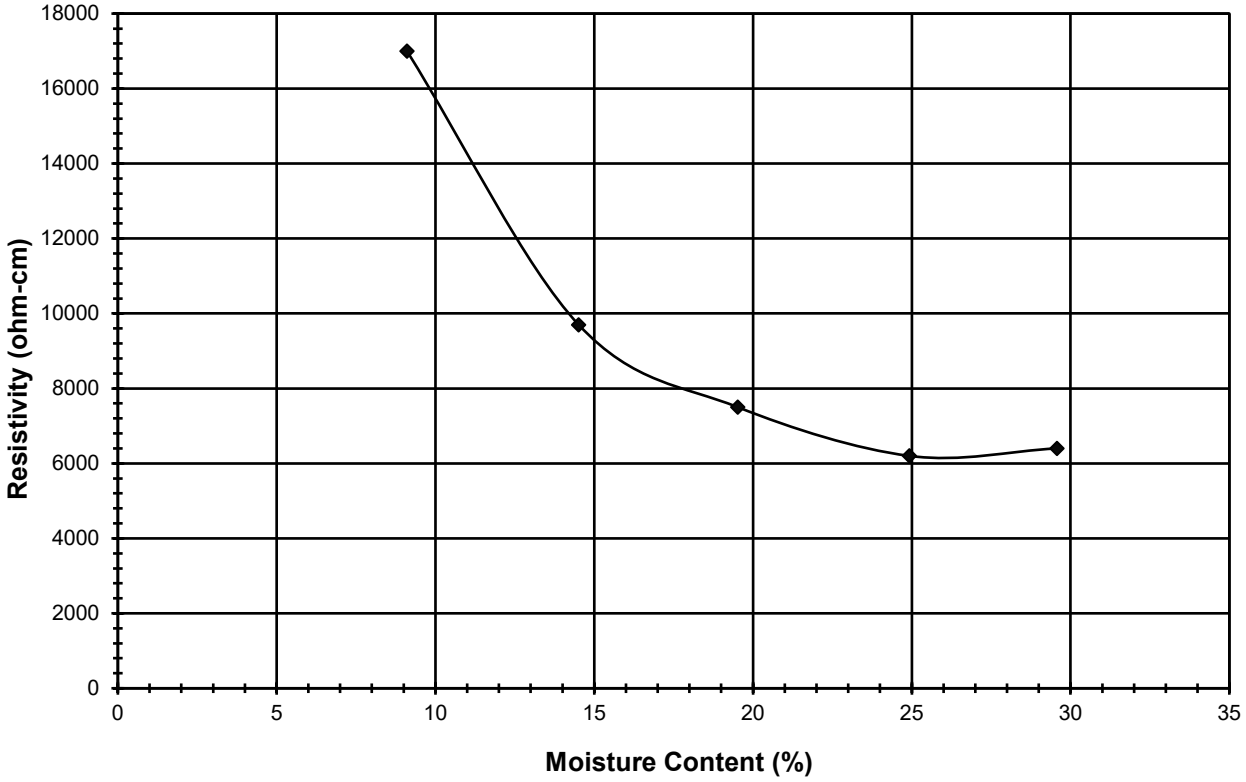
Client:	GHD Services, Inc	Boring No.:	B-1
Client Reference:	Brooklyn, NY 1266 7502	Depth (ft):	0.0-10.0'
Project No.:	2025-252-001	Sample No.:	GS-1
Lab ID:	2025-252-001-001	Visual Description:	Brown
		(- #10 Sieve material)	

Tare No.:	717	705	733	719	768
Tare & Wet Specimen (g):	53.71	47.05	46.15	43.25	45.23
Tare & Dry Specimen (g):	51.11	43.81	41.98	39.13	39.18
Tare Weight (g):	22.55	21.48	20.62	22.60	18.72
Moisture Content (%):	**9.1**	**14.5**	**19.5**	**24.9**	**29.6**
Resistance (ohm):	**17000**	**9700**	**7500**	**6200**	**6400**
Resistivity (ohm-cm):	**17000**	**9700**	**7500**	**6200**	**6400**

Note: The ratio of Miller Box area versus distance between electrodes is equal to 1.



Soil Class	*Corrosion Resistance*	*Specific Resistivity (ohm-cm)*
1	Excellent	10,000 - 6,000
2	Good	6,000 - 4,500
3	Fair	4,500 - 2,000
4	Bad	2,000 - 0

Tested By JAM *Date* 5/8/2025 *Checked By* EG *Date* 5/9/2025

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Water-Soluble Sulfate Ion Content in Soil

AASHTO T 290-95 (2020)

Client:	GHD Services, Inc.	Boring No.:	B-1
Client Reference:	Brooklyn NY 1266 7502	Depth (ft):	0.0-10.0'
Project No.:	2025-252-001	Sample No.:	GS-1
Lab ID:	2025-252-001-001	Soil Description:	Brown

Sulfate Standard - Calibration Curve Spectrophotometer Readings

Sulfate Ion Concentrations (mg/L)								
0.0	4.0	10.0	20.0	30.0	40.0	60.0	80.0	100.0
Spectrophotometer Readings (FAU)								
Underrange	Underrange	8	17	44	68	129	184	227

Measurement of Barium Chloride Turbidity

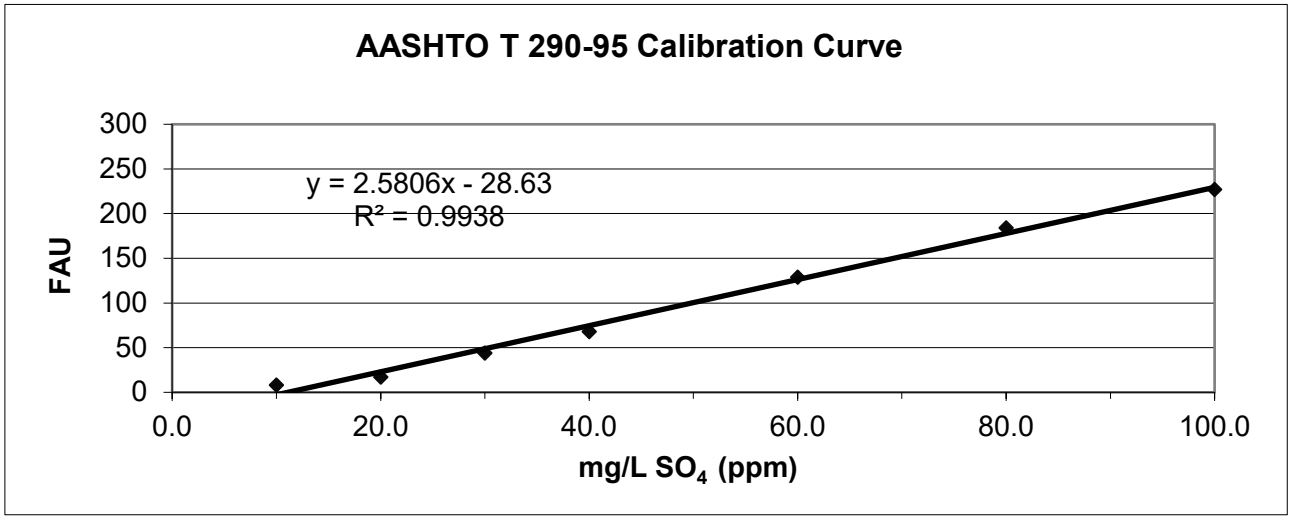
(Sample contains 5.0 mL NaCl solution and 0.3 g $BaCl_2 \cdot 2H_2O$)

Sample Weight (g):	100.0
Water added to Sample (mL):	300.0
Size of Sample Aliquot (mL):	50.0
Sample Reading (FAU):	30
Sample Diluted:	No
Sulfate Solution Added (ml):	0

Sample Moisture Content

Tare Number:	483
Weight of Tare & Wet Sample (g):	209.15
Weight of Tare & Dry Sample (g):	208.91
Weight of Tare (g):	97.30
Weight of Water (g):	0.24
Weight of Dry Sample (g):	111.61
Moisture Content (%):	0.22

Sample Sulfate Ion Concentration:	22.72	**mg/L SO_4 (ppm)**
Sample Sulfate Ion Content:	68.2	**mg/Kg SO_4 (not corrected for moisture)**
Sample Sulfate Ion Content:	68.3	**mg/Kg SO_4 (corrected for moisture)**



Tested by: JAM Date: 5/7/25 Checked by: EG Date: 5/8/2025

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CHLORIDE ION CONTENT IN SOILS

AASHTO T 291 - 94 (2018) (*Method B*)

Client:	GHD Services, Inc.	Boring No.:	B-4
Client Reference:	Brooklyn, NY 1266 7502	Depth (ft):	0.0-10.0'
Project No.:	2025-252-001	Sample No.:	GS-2
Lab ID:	2025-252-001-002	Description:	Brown

(- # 10 Sieve material)

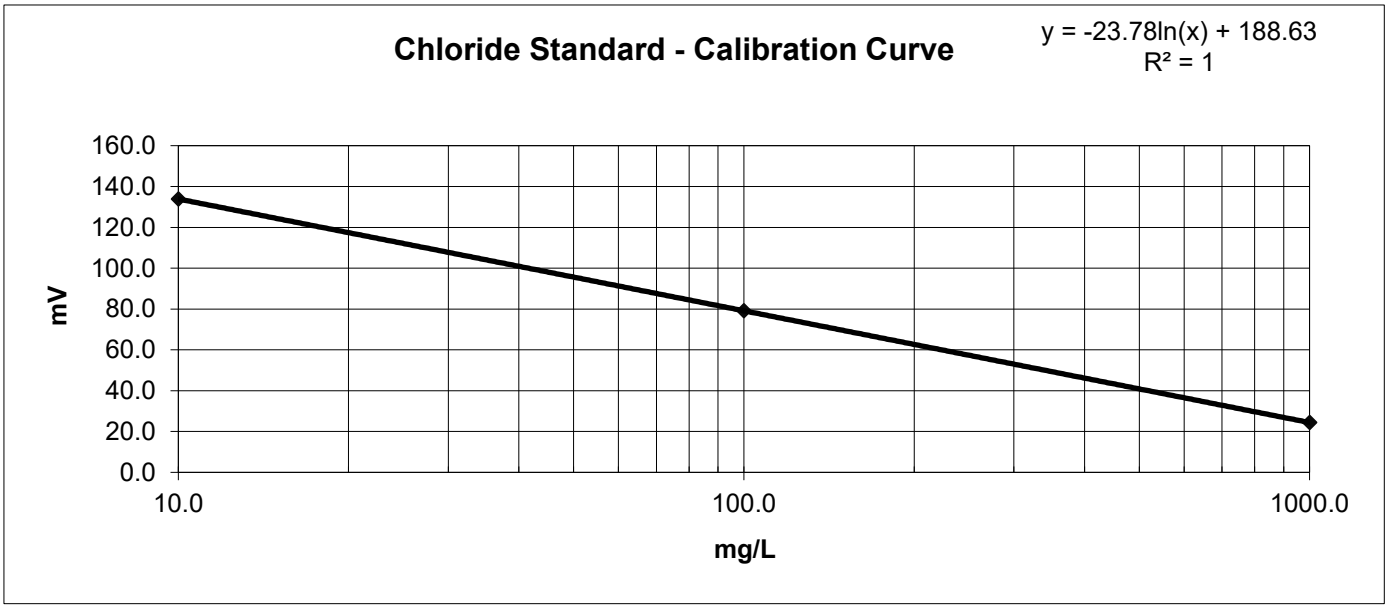
CHLORIDE STANDARD: CALIBRATION CURVE

STANDARD		MILLIVOLTS (mV)
10.0	mg/L	133.9
100.0	mg/L	79.1
1000.0	mg/L	24.4

MEASUREMENT OF CHLORIDES

		CONCENTRATION (mg/L)	CONCENTRATION (mg/kg)
Sample Weight (g):	100.0		
Water added to Sample (ml):	100.0		
Size of Sample Aliquot (ml):	25.0		
Sample Reading (mV):	164.3	2.78	2.78

Notes: 1) Samples and standards were buffered by the addition of an equal volume of the 0.2 M KNO_3 solution (1:1 volume).
2) Samples were dried for a minimum of 12 hours at 110 $^{+}/_{-}$ 5°C.



Notes:

Tested By	JAM	Date	5/8/25	Checked By	EG	Date	5/9/25

 DCN: CT-S63A DATE: 6/2/14 REVISION: 1

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pH OF SOILS

AASHTO T 289-91 (2013)

Client:	GHD Services, Inc.
Client Reference:	Brooklyn, NY 1266 7502
Project No.:	2025-252-001

Lab ID:	002
Boring No.:	B-4
Depth (ft):	0.0-10.0'
Sample No.:	GS-2
Drying Tare No.:	2004
Testing Tare No.:	Y
Temperature (°C):	22.7
pH of Sample:	**7.67**

Meter Calibration (as used each day)		
Buffer pH	Meter Reading	Meter Model
4.00	4.00	ORION 720A
7.00	7.00	
10.00	10.00	

Tested By JAM Date 5/7/25 Checked By EG Date 5/8/25

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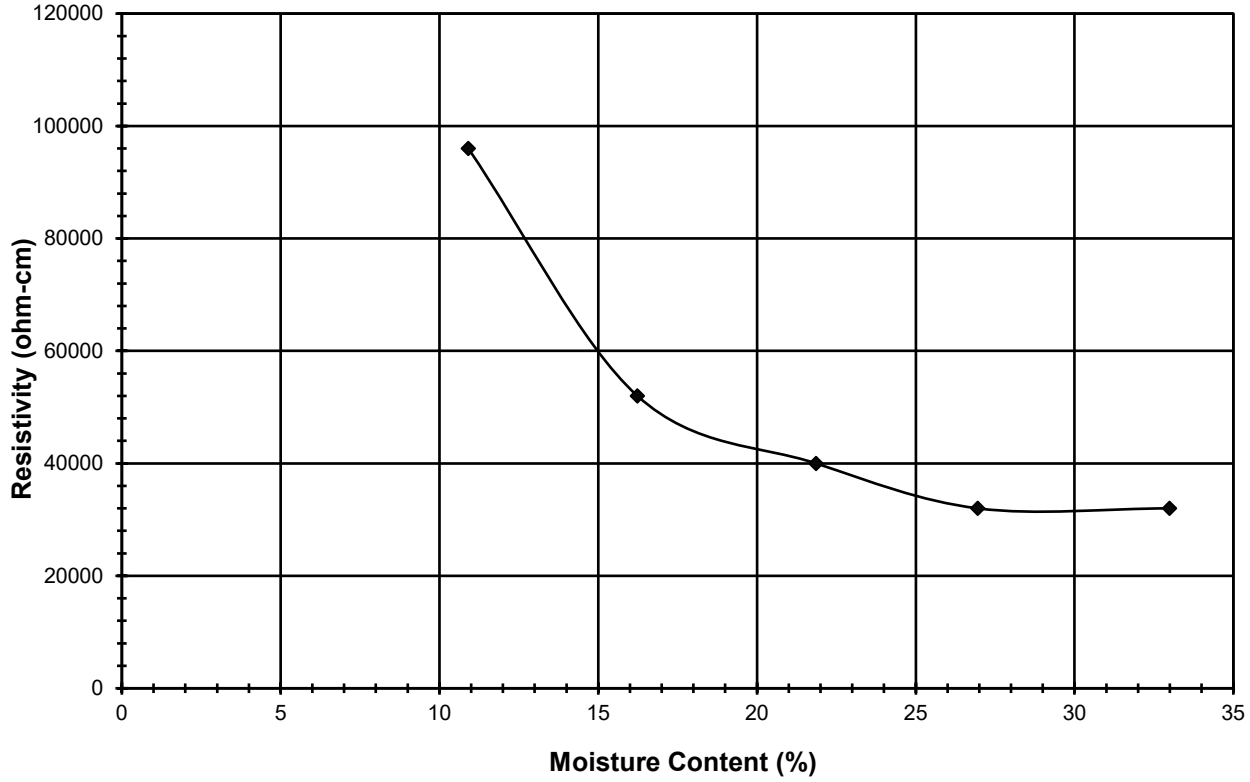
Minimum Resistivity

AASHTO T288-12

Client:	GHD Services, Inc.	Boring No.:	B-4
Client Reference:	Brooklyn, NY 1266 7522	Depth (ft):	0.0-10.0'
Project No.:	2025-252-001	Sample No.:	GS-2
Lab ID:	2025-252-001-002	Visual Description:	Brown
		(- #10 Sieve material)	

Tare No.:	329	292	168	279	290
Tare & Wet Specimen (g):	42.97	40.37	46.22	55.41	53.23
Tare & Dry Specimen (g):	40.61	37.54	41.17	48.27	44.47
Tare Weight (g):	18.98	20.11	18.07	21.78	17.92
Moisture Content (%):	**10.9**	**16.2**	**21.9**	**27.0**	**33.0**
Resistance (ohm):	**96000**	**52000**	**40000**	**32000**	**32000**
Resistivity (ohm-cm):	**96000**	**52000**	**40000**	**32000**	**32000**

Note: The ratio of Miller Box area versus distance between electrodes is equal to 1.



Soil Class	*Corrosion Resistance*	*Specific Resistivity (ohm-cm)*
1	Excellent	10,000 - 6,000
2	Good	6,000 - 4,500
3	Fair	4,500 - 2,000
4	Bad	2,000 - 0

Tested By DDA *Date* 5/7/2025 *Checked By* EG *Date* 5/9/2025

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Water-Soluble Sulfate Ion Content in Soil

AASHTO T 290-95 (2020)

Client:	GHD Services, Inc.	Boring No.:	B-4
Client Reference:	Brooklyn NY 1266 7502	Depth (ft):	0.0-10.0'
Project No.:	2025-252-001	Sample No.:	GS-2
Lab ID:	2025-252-001-002	Soil Description:	Brown

Sulfate Standard - Calibration Curve Spectrophotometer Readings

Sulfate Ion Concentrations (mg/L)								
0.0	4.0	10.0	20.0	30.0	40.0	60.0	80.0	100.0
Spectrophotometer Readings (FAU)								
Underrange	Underrange	8	17	44	68	129	184	227

Measurement of Barium Chloride Turbidity

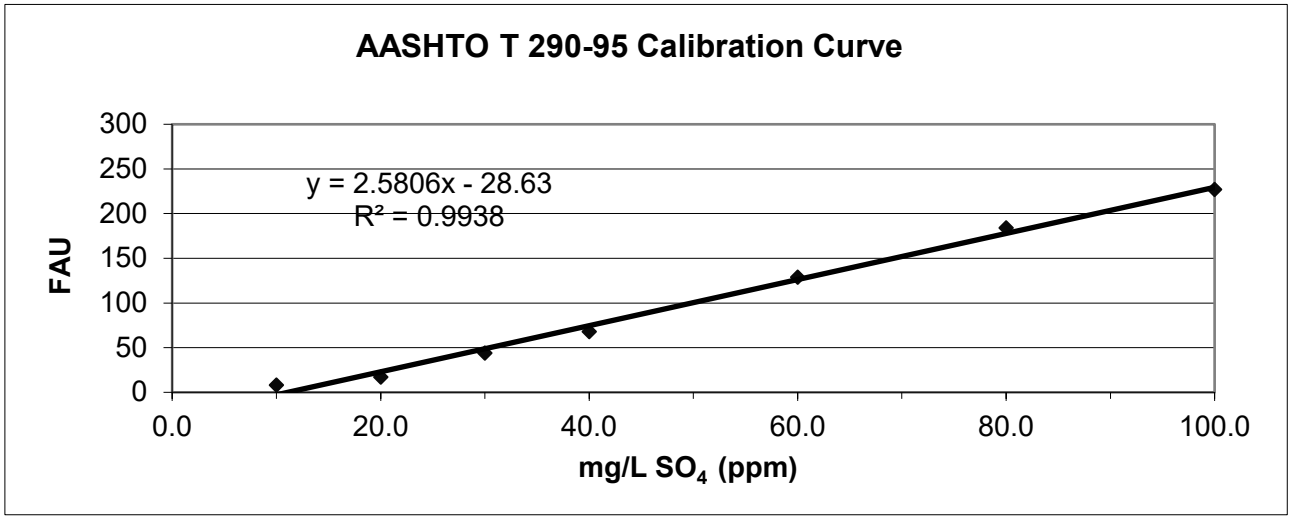
(Sample contains 5.0 mL NaCl solution and 0.3 g $BaCl_2 \cdot 2H_2O$)

Sample Weight (g):	100.0
Water added to Sample (mL):	300.0
Size of Sample Aliquot (mL):	50.0
Sample Reading (FAU):	8
Sample Diluted:	No
Sulfate Solution Added (ml):	0

Sample Moisture Content

Tare Number:	479
Weight of Tare & Wet Sample (g):	215.21
Weight of Tare & Dry Sample (g):	214.96
Weight of Tare (g):	47.73
Weight of Water (g):	0.25
Weight of Dry Sample (g):	167.23
Moisture Content (%):	0.15

Sample Sulfate Ion Concentration:	14.19	**mg/L SO_4 (ppm)**
Sample Sulfate Ion Content:	42.6	**mg/Kg SO_4 (not corrected for moisture)**
Sample Sulfate Ion Content:	42.6	**mg/Kg SO_4 (corrected for moisture)**



Tested by: JAM Date: 5/7/25 Checked by: EG Date: 5/8/2025

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ANALYTICAL REPORT

PREPARED FOR

Attn: Caleb Kyper
Geotechnics Inc.
544 Braddock Ave
East Pittsburgh, Pennsylvania 15112

Generated 5/12/2025 7:21:31 AM

JOB DESCRIPTION

Geotechnics, GHD Services, Inc.

JOB NUMBER

180-189869-1

Eurofins Pittsburgh
301 Alpha Drive
RIDC Park
Pittsburgh PA 15238

See page two for job notes and contact information.

my EOL

Eurofins Pittsburgh

Job Notes

This report may not be reproduced except in full, and with written approval from the laboratory. The results relate only to the samples tested. For questions please contact the Project Manager at the e-mail address or telephone number listed on this page.

PA Lab ID: 02-00416

The test results in this report relate only to the samples as received by the laboratory and will meet all requirements of the methodology, with any exceptions noted. This report shall not be reproduced except in full, without the express written approval of the laboratory. All questions should be directed to the Eurofins Pittsburgh Project Manager.

Authorization

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5/12/2025 7:21:31 AM

Authorized for release by
David Dunlap, Senior Project Manager
David.Dunlap@et.eurofinsus.com
(412)963-2432

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Chain of Custody 14
Receipt Checklists 15

Job Narrative
180-189869-1

Comments
The samples were received on the last day of the holding time for the sulfide analysis. At the direction of the client, the analysis was to be completed.

Receipt
The samples were received on 4/29/2025 4:00 PM. Unless otherwise noted below, the samples arrived in good condition. The temperature of the cooler at receipt was 21.9º C. The samples were not submitted on ice.

There was no initial relinquished/received time on the COC.

General Chemistry
Method 9034: The sulfide analysis was completed past the 7 day holding time.

No additional analytical or quality issues were noted, other than those described above or in the Definitions/Glossary page.

Definitions/Glossary

Client: Geotechnics Inc.
Project/Site: Geotechnics, GHD Services, Inc.

Job ID: 180-189869-1

Qualifiers

General Chemistry

Qualifier	Qualifier Description
!	Laboratory is not accredited for this parameter.
H	Sample was prepped or analyzed beyond the specified holding time. This does not meet regulatory requirements.

Glossary

Abbreviation	These commonly used abbreviations may or may not be present in this report.
☼	Listed under the "D" column to designate that the result is reported on a dry weight basis
%R	Percent Recovery
CFL	Contains Free Liquid
CFU	Colony Forming Unit
CNF	Contains No Free Liquid
DER	Duplicate Error Ratio (normalized absolute difference)
Dil Fac	Dilution Factor
DL	Detection Limit (DoD/DOE)
DL, RA, RE, IN	Indicates a Dilution, Re-analysis, Re-extraction, or additional Initial metals/anion analysis of the sample
DLC	Decision Level Concentration (Radiochemistry)
EDL	Estimated Detection Limit (Dioxin)
LOD	Limit of Detection (DoD/DOE)
LOQ	Limit of Quantitation (DoD/DOE)
MCL	EPA recommended "Maximum Contaminant Level"
MDA	Minimum Detectable Activity (Radiochemistry)
MDC	Minimum Detectable Concentration (Radiochemistry)
MDL	Method Detection Limit
ML	Minimum Level (Dioxin)
MPN	Most Probable Number
MQL	Method Quantitation Limit
NC	Not Calculated
ND	Not Detected at the reporting limit (or MDL or EDL if shown)
NEG	Negative / Absent
POS	Positive / Present
PQL	Practical Quantitation Limit
PRES	Presumptive
QC	Quality Control
RER	Relative Error Ratio (Radiochemistry)
RL	Reporting Limit or Requested Limit (Radiochemistry)
RPD	Relative Percent Difference, a measure of the relative difference between two points
TEF	Toxicity Equivalent Factor (Dioxin)
TEQ	Toxicity Equivalent Quotient (Dioxin)
TNTC	Too Numerous To Count

Accreditation/Certification Summary

Client: Geotechnics Inc.
Job ID: 180-189869-1
Project/Site: Geotechnics, GHD Services, Inc.

Laboratory: Eurofins Pittsburgh

Unless otherwise noted, all analytes for this laboratory were covered under each accreditation/certification below.

Authority	Program	Identification Number	Expiration Date
Pennsylvania	NELAP	02-00416	04-30-26

The following analytes are included in this report, but the laboratory is not certified by the governing authority. This list may include analytes for which the agency does not offer certification.

Analysis Method	Prep Method	Matrix	Analyte
SM 2580B		Solid	Oxidation Reduction Potential

Sample Summary

Client: Geotechnics Inc.
Project/Site: Geotechnics, GHD Services, Inc.

Job ID: 180-189869-1

Lab Sample ID	Client Sample ID	Matrix	Collected	Received
180-189869-1	2025-252-001-001	Solid	04/22/25 00:00	04/29/25 16:00
180-189869-2	2025-252-001-002	Solid	04/22/25 00:00	04/29/25 16:00

Method Summary

Client: Geotechnics Inc.
Project/Site: Geotechnics, GHD Services, Inc.

Job ID: 180-189869-1

Method	Method Description	Protocol	Laboratory
2540G	SM 2540G	SM22	EET PIT
EPA 9034	Sulfide, Acid soluble and Insoluble (Titrimetric)	SW846	EET PIT
SM 2580B	Reduction-Oxidation (REDOX) Potential	SM	EET PIT
9030B	Sulfide, Distillation (Acid Soluble and Insoluble)	SW846	EET PIT
DI Leach	Deionized Water Leaching Procedure	ASTM	EET PIT

Protocol References:

ASTM = ASTM International
SM = "Standard Methods For The Examination Of Water And Wastewater"
SM22 = Standard Methods For The Examination Of Water And Wastewater, 22nd Edition
SW846 = "Test Methods For Evaluating Solid Waste, Physical/Chemical Methods", Third Edition, November 1986 And Its Updates.

Laboratory References:

EET PIT = Eurofins Pittsburgh, 301 Alpha Drive, RIDC Park, Pittsburgh, PA 15238, TEL (412)963-7058

Lab Chronicle

Client: Geotechnics Inc.
Project/Site: Geotechnics, GHD Services, Inc.

Job ID: 180-189869-1

Client Sample ID: 2025-252-001-001 — Lab Sample ID: 180-189869-1

Date Collected: 04/22/25 00:00
Date Received: 04/29/25 16:00
Matrix: Solid

Prep Type	Batch Type	Batch Method	Run	Dil Factor	Initial Amount	Final Amount	Batch Number	Prepared or Analyzed	Analyst	Lab
Total/NA	Analysis	2540G		1			494992	05/02/25 10:35	MTW	EET PIT
	Instrument ID: NOEQUIP									
Soluble	Leach	DI Leach			19.90 g	20 mL	495457	05/08/25 10:56	ELS	EET PIT
Soluble	Analysis	SM 2580B		1			495451	05/08/25 14:09	ELS	EET PIT
	Instrument ID: NOEQUIP									

Client Sample ID: 2025-252-001-001 — Lab Sample ID: 180-189869-1

Date Collected: 04/22/25 00:00
Date Received: 04/29/25 16:00
Matrix: Solid
Percent Solids: 92.4

Prep Type	Batch Type	Batch Method	Run	Dil Factor	Initial Amount	Final Amount	Batch Number	Prepared or Analyzed	Analyst	Lab
Total/NA	Prep	9030B			5.02 g	50 mL	495269	05/06/25 12:57	BAB	EET PIT
Total/NA	Analysis	EPA 9034		1			495287	05/06/25 16:24	SNR	EET PIT
	Instrument ID: TITRAT1									

Client Sample ID: 2025-252-001-002 — Lab Sample ID: 180-189869-2

Date Collected: 04/22/25 00:00
Date Received: 04/29/25 16:00
Matrix: Solid

Prep Type	Batch Type	Batch Method	Run	Dil Factor	Initial Amount	Final Amount	Batch Number	Prepared or Analyzed	Analyst	Lab
Total/NA	Analysis	2540G		1			494992	05/02/25 10:35	MTW	EET PIT
	Instrument ID: NOEQUIP									
Soluble	Leach	DI Leach			19.98 g	20 mL	495457	05/08/25 10:56	ELS	EET PIT
Soluble	Analysis	SM 2580B		1			495451	05/08/25 14:26	ELS	EET PIT
	Instrument ID: NOEQUIP									

Client Sample ID: 2025-252-001-002 — Lab Sample ID: 180-189869-2

Date Collected: 04/22/25 00:00
Date Received: 04/29/25 16:00
Matrix: Solid
Percent Solids: 95.1

Prep Type	Batch Type	Batch Method	Run	Dil Factor	Initial Amount	Final Amount	Batch Number	Prepared or Analyzed	Analyst	Lab
Total/NA	Prep	9030B			4.98 g	50 mL	495269	05/06/25 12:57	BAB	EET PIT
Total/NA	Analysis	EPA 9034		1			495287	05/06/25 16:26	SNR	EET PIT
	Instrument ID: TITRAT1									

[1] This procedure uses a method stipulated length of time for the process. Both start and end times are displayed.

Laboratory References:

EET PIT = Eurofins Pittsburgh, 301 Alpha Drive, RIDC Park, Pittsburgh, PA 15238, TEL (412)963-7058

Lab Chronicle

Client: Geotechnics Inc.
Project/Site: Geotechnics, GHD Services, Inc.

Job ID: 180-189869-1

Analyst References:

Lab: EET PIT

- Batch Type: Leach
  - ELS = Edwin Shireman
- Batch Type: Prep
  - BAB = Brooke Batyi
- Batch Type: Analysis
  - ELS = Edwin Shireman
  - MTW = Michael Wesoloski
  - SNR = Sabra Richart

Client Sample Results

Client: Geotechnics Inc.
Project/Site: Geotechnics, GHD Services, Inc.

Job ID: 180-189869-1

Client Sample ID: 2025-252-001-001
Lab Sample ID: 180-189869-1
Date Collected: 04/22/25 00:00
Matrix: Solid
Date Received: 04/29/25 16:00

General Chemistry

Analyte	Result	Qualifier	RL	MDL	Unit	D	Prepared	Analyzed	Dil Fac
Percent Moisture (SM22 2540G)	7.6		0.1	0.1	%			05/02/25 10:35	1
Percent Solids (SM22 2540G)	92.4		0.1	0.1	%			05/02/25 10:35	1

General Chemistry - Soluble

Analyte	Result	Qualifier	RL	MDL	Unit	D	Prepared	Analyzed	Dil Fac
Oxidation Reduction Potential (SM 2580B)	200	!	10	10	millivolts			05/08/25 14:09	1

Client Sample ID: 2025-252-001-001
Lab Sample ID: 180-189869-1
Date Collected: 04/22/25 00:00
Matrix: Solid
Date Received: 04/29/25 16:00
Percent Solids: 92.4

General Chemistry

Analyte	Result	Qualifier	RL	MDL	Unit	D	Prepared	Analyzed	Dil Fac
Sulfide (SW846 EPA 9034)	ND	H	32	11	mg/Kg	☼	05/06/25 12:57	05/06/25 16:24	1

Client Sample ID: 2025-252-001-002
Lab Sample ID: 180-189869-2
Date Collected: 04/22/25 00:00
Matrix: Solid
Date Received: 04/29/25 16:00

General Chemistry

Analyte	Result	Qualifier	RL	MDL	Unit	D	Prepared	Analyzed	Dil Fac
Percent Moisture (SM22 2540G)	4.9		0.1	0.1	%			05/02/25 10:35	1
Percent Solids (SM22 2540G)	95.1		0.1	0.1	%			05/02/25 10:35	1

General Chemistry - Soluble

Analyte	Result	Qualifier	RL	MDL	Unit	D	Prepared	Analyzed	Dil Fac
Oxidation Reduction Potential (SM 2580B)	180	!	10	10	millivolts			05/08/25 14:26	1

Client Sample ID: 2025-252-001-002
Lab Sample ID: 180-189869-2
Date Collected: 04/22/25 00:00
Matrix: Solid
Date Received: 04/29/25 16:00
Percent Solids: 95.1

General Chemistry

Analyte	Result	Qualifier	RL	MDL	Unit	D	Prepared	Analyzed	Dil Fac
Sulfide (SW846 EPA 9034)	ND	H	32	11	mg/Kg	☼	05/06/25 12:57	05/06/25 16:26	1

QC Sample Results

Client: Geotechnics Inc.
Project/Site: Geotechnics, GHD Services, Inc.

Job ID: 180-189869-1

Method: 2540G - SM 2540G

Lab Sample ID: 180-189869-1 DU
Matrix: Solid
Analysis Batch: 494992

Client Sample ID: 2025-252-001-001
Prep Type: Total/NA

Analyte	Sample Result	Sample Qualifier		DU Result	DU Qualifier	Unit	D			RPD	RPD Limit
Percent Moisture	7.6			7.1		%				6	10
Percent Solids	92.4			92.9		%				0.5	10

Method: EPA 9034 - Sulfide, Acid soluble and Insoluble (Titrimetric)

Lab Sample ID: MB 180-495269/2-A
Matrix: Solid
Analysis Batch: 495287

Client Sample ID: Method Blank
Prep Type: Total/NA
Prep Batch: 495269

Analyte	MB Result	MB Qualifier	RL	MDL	Unit	D	Prepared	Analyzed	Dil Fac
Sulfide	ND		30	9.9	mg/Kg		05/06/25 12:57	05/06/25 16:10	1

Lab Sample ID: LCS 180-495269/1-A
Matrix: Solid
Analysis Batch: 495287

Client Sample ID: Lab Control Sample
Prep Type: Total/NA
Prep Batch: 495269

Analyte			Spike Added	LCS Result	LCS Qualifier	Unit	D	%Rec	%Rec Limits		
Sulfide			177	150		mg/Kg		85	85 - 115		

Method: SM 2580B - Reduction-Oxidation (REDOX) Potential

Lab Sample ID: LCS 180-495451/1
Matrix: Solid
Analysis Batch: 495451

Client Sample ID: Lab Control Sample
Prep Type: Total/NA

Analyte			Spike Added	LCS Result	LCS Qualifier	Unit	D	%Rec	%Rec Limits		
Oxidation Reduction Potential			475	468		millivolts		99	98 - 102		

QC Association Summary

Client: Geotechnics Inc.
Project/Site: Geotechnics, GHD Services, Inc.

Job ID: 180-189869-1

General Chemistry

Analysis Batch: 494992

Lab Sample ID	Client Sample ID	Prep Type	Matrix	Method	Prep Batch
180-189869-1	2025-252-001-001	Total/NA	Solid	2540G	
180-189869-2	2025-252-001-002	Total/NA	Solid	2540G	
180-189869-1 DU	2025-252-001-001	Total/NA	Solid	2540G	

Prep Batch: 495269

Lab Sample ID	Client Sample ID	Prep Type	Matrix	Method	Prep Batch
180-189869-1	2025-252-001-001	Total/NA	Solid	9030B	
180-189869-2	2025-252-001-002	Total/NA	Solid	9030B	
MB 180-495269/2-A	Method Blank	Total/NA	Solid	9030B	
LCS 180-495269/1-A	Lab Control Sample	Total/NA	Solid	9030B	

Analysis Batch: 495287

Lab Sample ID	Client Sample ID	Prep Type	Matrix	Method	Prep Batch
180-189869-1	2025-252-001-001	Total/NA	Solid	EPA 9034	495269
180-189869-2	2025-252-001-002	Total/NA	Solid	EPA 9034	495269
MB 180-495269/2-A	Method Blank	Total/NA	Solid	EPA 9034	495269
LCS 180-495269/1-A	Lab Control Sample	Total/NA	Solid	EPA 9034	495269

Analysis Batch: 495451

Lab Sample ID	Client Sample ID	Prep Type	Matrix	Method	Prep Batch
180-189869-1	2025-252-001-001	Soluble	Solid	SM 2580B	495457
180-189869-2	2025-252-001-002	Soluble	Solid	SM 2580B	495457
LCS 180-495451/1	Lab Control Sample	Total/NA	Solid	SM 2580B	

Leach Batch: 495457

Lab Sample ID	Client Sample ID	Prep Type	Matrix	Method	Prep Batch
180-189869-1	2025-252-001-001	Soluble	Solid	DI Leach	
180-189869-2	2025-252-001-002	Soluble	Solid	DI Leach	

GEOTECHNICS TEST REQUEST

geotechnics

Date: 04/22/25
Laboratory: Eurofins Env. Testing Northeast, LLC
Disposal: By Lab
Total Samples: 2
Sampling Date: 4/22/25 (Perform testing regardless of hold time expiration)

DATE DATA NEEDED: **Normal TAT**

Send Report To: ckyper@geotechnics.net
kcummings@geotechnics.net

Client: GHD Services, Inc.
Client Project / Project No.: Brooklyn, NY 1266 7502
Geotechnics Project No.: 2025-252-001

Send Invoice To: Geotechnics, Inc.
544 Braddock Ave.
East Pittsburgh, PA 15112
pcummings@geotechnics.net

Geotechnics Contact: Caleb Kyper

BORING	DEPTH	SAMPLE No.	Lab ID No.	CONTAINER	QUANTITY	WATER	SOIL	AGGR	TESTING / REMARKS
B-1	0.0-10.0'	GS-1	2025-252-001-001	ZIP BAG	1		X		Sulfide Testing & Redox Potential
B-4	0.0-10.0'	GS-2	2025-252-001-002	ZIP BAG	1		X		Sulfide Testing & Redox Potential
									C6 #25 NoIce
									22.3 − 0.4 = 21.9°C
									CF

180-189869 Chain of Custody

RELINQUISHED BY	FIRM	DATE	RECEIVED BY	FIRM	DATE
Caleb Kyper	Geotechnics	4/22/25	John Muron	Geotechnics	4/29/25
John Muron	Geotechnics	4/29/25 11:00	[illegible]	EPNE	4/29/25 1600

Geotechnics, Inc. 544 Braddock Ave. East Pittsburgh, PA 15112
Phone: 412-823-7600 Fax: 412-823-8999 www.geotechnics.net

Login Sample Receipt Checklist

Client: Geotechnics Inc.

Job Number: 180-189869-1

Login Number: 189869
List Number: 1
Creator: Abernathy, Eric L

List Source: Eurofins Pittsburgh

Question	Answer	Comment
Radioactivity wasn't checked or is </= background as measured by a survey meter.	N/A	
The cooler's custody seal, if present, is intact.	True	
Sample custody seals, if present, are intact.	True	
The cooler or samples do not appear to have been compromised or tampered with.	True	
Samples were received on ice.	False	
Cooler Temperature is acceptable.	False	
Cooler Temperature is recorded.	True	
COC is present.	True	
COC is filled out in ink and legible.	True	
COC is filled out with all pertinent information.	True	
Is the Field Sampler's name present on COC?	False	
There are no discrepancies between the containers received and the COC.	True	
Samples are received within Holding Time (excluding tests with immediate HTs)	False	rec. on last day of sulfide hold time
Sample containers have legible labels.	True	
Containers are not broken or leaking.	True	
Sample collection date/times are provided.	True	
Appropriate sample containers are used.	True	
Sample bottles are completely filled.	True	
Sample Preservation Verified.	N/A	
There is sufficient vol. for all requested analyses, incl. any requested MS/MSDs	True	
Containers requiring zero headspace have no headspace or bubble is <6mm (1/4").	True	
Multiphasic samples are not present.	True	
Samples do not require splitting or compositing.	True	
Residual Chlorine Checked.	N/A	

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