REAVELEY
ENGINEERS + ASSOCIATES
Consulting Structural Engineers

2017.080
Summit Powder Mountain

SPM - PARCEL 4 BLDG

Footings and Foundations Permit

STRUCTURAL CALCULATIONS

Prepared For:
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06/27/2017

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CALCULATIONS INDEX

01
DESIGN CRITERIA

Elevation: 8615'
Latitude: 41.36283°N
Longitude: 111.74423°W

1. Design Criteria

1.1. Governing Building Code: 2015 International Building Code (IBC)
- A. Risk Category: II

1.2. Floor Live Loading:
- A. Exterior Plazas: .... 100 psf Live Load
- B. Event Space and Lobbies: .... 100 psf Live Load
- C. Exit Facilities & Corridors: .... 100 psf Live Load
- D. Gymnasium: .... 100 psf Live Load
- E. Mechanical Rooms: .... 125 psf Live Load or actual weights, if larger

1.3. Roof Live Loading:
- A. Roof Live Load: .... 20 psf
- B. Roof Snow Load: .... 184 psf or 202 psf + Drift per IBC
  1. Ground Snow Load, P_g: .... 263 psf
  2. Snow Exposure Factor, C_e: 1.0
  3. Importance Factor, I_S: 1.0
     - a. Thermal Factor,
     - b. Exterior Ventilated Roof: C_t: 1.1
     - c. All Other Roofs: 1.0

1.4. Earthquake:
- A. Seismic Design Category .... D
- B. Spectral Response Accelerations:
  - S_S = 0.81 g S_{DS} = 0.58 g
  - S_1 = 0.27 g S_{D1} = 0.27 g
- C. Soil Site Class: C
  - F_a = 1.08 F_v = 1.53
- D. Basic Seismic-Force-Resisting System:
  - Upper N-S: Steel Ordinary Moment Frames
    - R = 3.5 C_d = 3 Ω_0 = 3
  - Upper E-W: Steel Buckling-restrained Braced Frames
    - R = 8 C_d = 5 Ω_0 = 2.5
  - Lower: Special Reinforced Concrete Shear Walls
    - R = 5 C_d = 5 Ω_0 = 2.5
- E. Importance Factor, I_E: 1.0
- F. Redundancy Factor, ρ: 1.0
- G. Design Base Shear: ....
- H. Design Story Drift, Δ: .... 0.020*height of floor
- I. Analysis Procedure: .... Two Stage Equivalent Lateral Force (Static)

1.5. Wind:
- A. Ultimate Design Wind Speed V_{ult}: .... 115 mph
- B. Exposure: C
- C. Internal Pressure Coefficient, GC_{pi}: 0.18
- D. Topographic Factor, K_{ht}: 1.8
- E. Components and Cladding Design Pressure:

Design Wind Pressure (psf)					
Location		Tributary Area (ft^2)			
		< 10	50	100	> 500
Walls	Within 4.8 ft of building corner	86.7	73.2	67.4	53.8
	All other areas	70.3	63.5	60.6	53.8
Roof	Within 4.8 ft of building corner	152.6	129.6	119.7	119.7
	Within 4.8 ft of building edge	103.2	84	75.8	75.8
	All other areas	59.3	55.5	53.8	53.8

1.6. Foundation:
- A. Subsurface Conditions:
  Soils report and log of borings has not been obtained by the Owner for this parcel. The design is based upon the geotechnical report for Parcel 2C, just northeast of the site. The Owner has contracted with the Geotechnical Engineer, and fieldwork has been completed for the site. The designs in this set are subject to change once the site-specific recommendations are received and implemented. The report is expected to be completed and available in July 2017.
- B. Soils Report for Parcel 2C by IGES, dated January 18th, 2017.
- C. Soil Bearing Pressure
  1. Lower level, 8 feet below finished grade: .... 4600 psf
  2. Shallow Foundations near finished grade: .... 2300 psf
- D. Lateral Soil Pressure Fluid Equivalent Density.
  1. Active: .... 35 pcf (retaining walls)
  2. At Rest: .... 55 pcf (rigid foundation walls)
  3. Passive: .... 320 pcf
  4. Increase for Seismic: .... 13 pcf
- E. Coefficient of Friction: 0.45

1.7. Classification for Fire Rated Construction:
- A. For the purpose of determining fire-resistive assemblies, open-web steel framing members shall be considered unrestrained. All other steel floor and roof framing members shall be considered restrained.
- B. Diagonal members in vertical braced frames shall be considered as secondary members for fire proofing protection

2. Earthwork

2.1. Clearing: The entire building area shall be scraped to remove all topsoil, vegetation, and debris.

2.2. Proof rolling:

A. The natural undisturbed soil below slabs on grade shall be proof rolled prior to placing concrete. Remove all soft spots and replace with compacted structural fill. Areas requiring compacted structural fill shall be scarified to at least 6 inches deep, moisture conditioned as required to be at or above optimum moisture content and compacted to 90 percent of maximum laboratory density per ASTM D1557. Scarification and proof rolling is not required where bedrock is exposed.

B. The natural undisturbed soil below footings shall be properly prepared by proof rolling prior to placing concrete. Remove all soft spots.

2.3. Footings shall bear on one of the following, but not combined:

A. Properly prepared native soils as described above, with soft spots removed and footing elevations lowered to bear on native soils only.

B. A minimum of two feet of compacted structural fill over properly prepared native soils, with soft spots removed and replaced with compacted structural fill.

Footing excavations must be observed by the Geotechnical Engineer prior to placing compacted structural fill or any formwork, concrete, or reinforcing steel for footings.

2.4. Concrete slabs on grade shall be underlain by at least 4" of either:

A. A well-graded granular base course with no more than 5 percent passing a No. 200 sieve, compacted to 95 percent of the maximum laboratory density as determined by ASTM D1557.

B. A gap-graded free-draining granular material with a maximum size of not more than ¾ inch and no more than 5 percent passing a No. 200 sieve.

2.5. Compacted structural fill: All fill material shall be a well-graded granular material with a maximum size less than 4 inches, with less than 25 percent fines, and 10 to 60 percent sand. Native soils meeting this criteria may be used but must be approved by the geotechnical engineer prior to use. Compacted structural fill shall be compacted to 95 percent of the maximum laboratory density as determined by ASTM D1557 at or slightly above the optimum moisture content. All fill shall be tested (See Specifications and the Quality Assurance section of the GSN).

3. Concrete

3.1. Materials shall comply with the Standards specified in American Concrete Institute (ACI) 318-14, "Building Code Requirements for Structural Concrete."

A. Concrete mix design requirements shall be as follows:

Location	f'c at 28 days (psi)	Max W/C Ratio	Air Content (%)	Max Aggregate Size	Exposure Classes*		
					F	S	C
Footings	3000	0.50	-	1"	F0	S0	C0
Interior Slabs on Grade	3000	0.45	-	1"	F0	S0	C0
Walls and piers	4500	0.45	5	¾"	F1	S0	C1
Joist, Beams and Suspended Slabs	5000	0.45	-	¾"	F1	S0	C0
Concrete over Steel Deck	3000	0.45	-	¾"	F0	S0	C0
All other site cast concrete	4500	0.45	4.5	1"	F1	S0	C1

* Exposure Classes are per ACI 318, Section 4.2.1, where F, S and C are exposure categories for freezing and thawing, sulfate, and corrosion protection of reinforcement, respectively.

B. Cementitious Materials:

1. Portland Cement (ASTM C150):
   a. Type I or II for exposure class S0.
2. Fly Ash (ASTM C618, Class C or F): maximum fly ash content as a percentage of total weight of cementitious materials shall be 25 percent.

C. Steel Reinforcement:

1. ASTM A615 Grade 60, fy = 60,000 psi min. unless noted otherwise.
2. Reinforcement at concrete shear walls and all components of shear walls including coupling beams and wall piers shall comply with ASTM A706, Grade 60. ASTM A615 Grade 60 reinforcement shall be permitted if:
   a. The actual yield strength based on mill tests does not exceed 78,000 psi, and
   b. The ratio of actual tensile strength to the actual yield strength is not less than 1.25.
   c. Mill tests shall be submitted to the Engineer.

D. Wire Reinforcement:

1. Welded wire fabric (WWF): ASTM A1064.

E. Fiber Reinforcement:

1. Synthetic Micro-Fiber: fibrillated polypropylene micro-fibers engineered and designed for use in concrete, complying with ASTM C 1116, 1/2 to 1-1/2 inches long. Add to concrete at a dosage rate of 1.5 lb/cu yd where indicated.
2. Macrosynthetic Fibers: monofilament, non-fibrillating fibers made of a polypropylene/polyethylene blend. Macro fibers shall comply with ASTM C 1116, Type III, and meet the criteria of ASTM D 7508.
   a. Where noted in the Steel Deck Schedule, macrosythentic fibers shall be added to concrete over steel deck at a dosage rate determined by the fiber manufacturer but not less than 4 lb/cu yd.
   b. Do not burn off exposed fibers.

F. Admixtures:

1. Air-entraining admixtures, comply with ASTM C 260 (when used).
   a. Tolerance on air content as delivered shall be +/- 1.5%.
   b. When air content of a trowel finished floor slab exceeds 3%, there is an increased risk for delaminations and blistering to occur. When this situation is present, the contractor shall pay special attention to the finishing procedures to help minimize such risks. Refer to ACI 302.1R-96 "Guide for Concrete Floor and Slab Construction" for proper finishing guidelines.

2. Corrosion Inhibiting Admixture:
   a. Corrosion inhibiting additive containing a minimum of 30 percent calcium nitrite dosed at 3 gallons per cubic yard shall be added to all reinforced concrete with exposure class C2.
3. The use of super plasticizers and water reducers is allowed, but not required.
4. Calcium chloride or admixtures containing calcium chloride shall not be added to the concrete mix.

G. Chloride Ion: Maximum water soluble chloride ion concentrations in hardened concrete at age between 28 and 42 days contributed from the ingredients including water, aggregates, cementitious materials, and admixtures shall not exceed a maximum, by weight of cement, of 1.00% for concrete with exposure class C0, 0.30% for concrete with exposure class C1, 0.15% for concrete with exposure class C2, and 0.06% for all prestressed concrete.

H. Slump Limit: 4 inches, maximum for all concrete prior to the addition of plasticizers and water reducing admixtures. The concrete supplier shall indicate the final slump of each concrete mix in the submitted mix design.

I. Shrinkage Limit: Interior slabs on grade shall have a drying shrinkage limit of 0.040 percent tested in accordance with ASTM C157. Drying shrinkage test results shall be submitted with mix designs.

J. Only one grade or type of concrete shall be poured on the site at any given time.

K. Plastic coated tie wires and chairs shall be used to support reinforcing bars and tie bars in reinforced concrete structures that will be exposed to moisture.

3.2. Formwork shall comply with ACI Standards Publication 347 and the project specifications. The contractor shall be responsible for the design, detailing, care, placement and removal of the formwork and shores.

A. Pre-camber forms and screeds with a camber of 1/4" per every 10'-0" of span to compensate for dead load deflection, unless noted otherwise. Post tensioned concrete slabs and beams do not require formwork to be pre-cambered.

3.3. Concrete cover requirements for deformed bar reinforcing steel shall comply with ACI 318, "Building Code Requirements for Structural Concrete".

A. Cast-in-place Concrete: Specified Cover
1. Cast against and permanently exposed to earth: .... 3"
2. Formed concrete exposed to earth or weather:
   #6 thru #18 bars .... 2"
   #5 and smaller bars.. 1.1/2"
3. Concrete not exposed to weather or in contact with ground:
   Slabs, Walls, Joists; #11 bars and smaller .... 3/4"
   Beams, Columns: primary reinforcement, ties, stirrups, spirals .... 1.1/2”

3.4. Construction Joints and Control Joints:

A. Provide a surface intentionally roughened to ¼” amplitude in all wall footings. A continuous keyway shall not be used for concrete shear wall to footing connections, unless specifically indicated. Refer to project plans, schedules and details for the shear wall to footing connection requirements.

B. All horizontal and vertical construction joints shall have a surface intentionally roughened to ¼” amplitude. A continuous 2 X 4 keyway may be used on elements other than shear walls.

C. Provide reinforcement dowels to match the member reinforcement across the joint, unless noted otherwise. For dowels across construction joints and wall to footing connections of concrete shear walls, refer to specific project plans, schedules, and details.

D. Construction joints in suspended concrete pours shall be made at the center of spans.

E. Slabs on grade shall have construction or control joints spaced not to exceed 30 times the slab thickness in any direction.

F. Control joints shall be installed in slabs on grade so the length to width ratio of the slab is no more than 1.25:1. Control joints shall be completed within 12 hours of concrete placement. See typical details for joint configuration.

G. Control joints in visually exposed walls, unless noted otherwise: (Joints shall line up with masonry and architectural joints, see drawings.)
1. Vertical control joints at 10'-0" on center.
2. Reinforcing shall be continuous through control and construction joints, unless noted otherwise.
3. Control joints in concrete foundation walls shall line up with masonry control joints.

H. Control joints shall be installed in concrete slabs over steel deck by saw-cutting along girders and purlins at interior grid lines. See typical details for joint size and reinforcement. Reinforcement required shall be in addition to any slab reinforcement.

3.5. Detailing: All reinforcing, including welded wire fabric, shall be detailed, bolstered & supported to comply with ACI 315, "Details and Detailing of Concrete Reinforcement" and the Concrete Reinforcing Steel Institute (CRSI) recommendations. Reinforcing bars shall not be welded unless specifically shown on drawings.

A. Lap splice lengths shall be detailed to comply with the CONCRETE REINFORCING BAR DEVELOPMENT AND LAP SPLICE SCHEDULE.

B. All mechanical splices shall have the capacity to develop at least 1.25fy of the bar in tension or compression. Type 2 couplers have the capacity to develop the full tension capacity of the bar. Type 1 couplers shall not be used in shear wall jamb columns. Mechanical splices shall have a current ICC Code Evaluation Report; “Lenton” (ER-3967), “Taper-Lock” (ESR-2481) or “SAS Stressteel” (ESR-1163) tapered threaded rebar splices, "Bar-Lock" (ESR-2495) bolt coupling sleeves or approved equivalent may be used. Mechanical couplers on adjacent bars shall be staggered a minimum of 24" apart along the longitudinal axis of the reinforcing bars.

C. All embedded elements and dowels shall be securely tied to formwork or to adjacent reinforcing prior to the placement of concrete.

D. Use chairs or other support devices recommended by CRSI to support and tie reinforcement bars and welded wire fabric prior to placing concrete. Welded wire fabric shall be continuously supported at 36" o.c. maximum.

E. See typical details for reinforcing at wall intersections and ends, reinforcing around wall openings and suspended slab openings, vertical wall dowels, concrete column ties and splices in vertical column reinforcing.

F. See typical details for column cross-ties. The 90-degree hooks of two successive crossties engaging the same longitudinal bars shall be alternated end for end.
G. Contractor shall coordinate placement of all openings, curbs, dowels, sleeves, conduits, bolts, inserts and other embedded items prior to concrete placement.
H. All reinforcement shall be bent cold, and shall be bent only once at the same location. All reinforcement shall be shop bent, unless otherwise permitted by the engineer.

3.6. Minimum Reinforcing: Wall reinforcing shall be as follows, unless noted otherwise:

Wall Thickness	Horizontal Reinforcing	Vertical Reinforcing
6"	#4 @ 13" o.c.	#4 @ 18" o.c.
8"	#5 @ 15" o.c.	#4 @ 16" o.c.
10"	#5 @ 12" o.c.	#4 @ 13" o.c.
12"	#4 @ 13" o.c. Each Face	#4 @ 18" o.c. Each Face
Others	0.25% of Wall Area	0.15% of Wall Area

Spacing shall exceed neither three times the wall thickness nor 18". In addition to the above reinforcing, 2 - #5 x continuous horizontal bars shall be placed at the bottom of the wall (near the footing) and at each floor level, at the roof level and at the top of wall.

3.7. No aluminum conduit or product containing aluminum or any other material injurious to concrete shall be embedded in concrete.

3.8. Unless otherwise noted, all slabs on grade shall be 4" thick.

4. Structural Steel

4.1. Material:
A. W-Shapes: ASTM A992, (F_y = 50 ksi), except as noted otherwise
B. All Other Shapes and Plates: ASTM A36 (Fy = 36 ksi), except as noted otherwise
C. Rectangular and Square Hollow Structural Sections (HSS): ASTM A500, Grade C (Fy = 50 ksi)
D. Steel Deck:
   1. Galvanized Steel Sheet: ASTM A653 or A1063, Grade 50 with G60 galvanized coating.
   2. Ungalvanized Steel Sheet: ASTM A1008 or A1039, Grade 50
E. Deformed Bar Anchors (DBA): ASTM A496
F. Headed Stud Anchors (HSA): ASTM A108, with dimensions complying with AISC specifications
G. Anchor Rods: ASTM F1554, Grade 36, unless noted otherwise, with ASTM A563 heavy hex nuts and ASTM F436 hardened washers
H. Structural steel that is part of the seismic force resisting system shall be supplied with minimum Charpy V-Notch impact test results of 20 ft-lbs. absorbed energy at 70 degrees Fahrenheit, indicated below:
   1. Hot rolled shapes with flanges 1.1/2” thick and thicker tested in the alternate core location as described in ASTM A6 Supplementary requirement S30
   2. Plates 2" and thicker measured at any location permitted by ASTM A673
   3. Test Frequency: Each heat

4.2. Fabrication and construction shall comply with the following Codes and Standards:
A. American Institute of Steel Construction (AISC) 360-10, "Specification for Structural Steel Buildings"
B. AISC 341-10, “Seismic Provisions for Structural Steel Buildings”
C. AISC 303-10, "Code of Standard Practice for Steel Buildings and Bridges" excluding the following: Section 3.3 (last sentence of first paragraph), Section 4.4, Section 4.4.1, Section 4.4.2, Section 4.5, and Section 7.13.3
   1. The architectural drawings are the prime contract drawings. Consultants’ drawings by other disciplines are supplementary to the architectural drawings. The structural drawings shall be used in conjunction with the architectural drawings. Detailing and shop drawing production for structural elements will require information (including dimensions) contained in architectural, structural, and/or other consultants' drawings. Refer to the Special Instructions section of the general notes, below.
D. AISC/RCSC 2009, "Specification for Structural Joints Using ASTM A325 or A490 Bolts"
E. American Welding Society (AWS) D1.1:2010, “Structural Welding Code – Steel” (specific items do not apply when they conflict with the AISC requirements)
F. American Welding Society (AWS) D1.8:2009, “Structural Welding Code – Seismic Supplement” (specific items do not apply when they conflict with the AISC requirements)
G. Steel Joist Institute (SJI): K-10, “Standard Specification for Open Web Steel Joists, K-series;” LH/DLH-10, “Standard Specification for Longspan Steel Joists, LH-series and Deep Longspan Steel Joists, DLH-series;” JG-10, “Standard Specification for Joist Girders;” CJ-10, “Standard Specification for Composite Steel Joists, CJ-series”
H. American Iron and Steel Institute (AISI) 2007, "North American Specification for the Design of Cold-Formed Steel Structural Members"

4.3. Structural shapes and plates shall be fabricated from newly rolled (milled) one-piece sections without splices, unless specifically noted otherwise on the structural drawings. Connections for structural steel shall comply with the structural drawings, unless written approval is given by the structural engineer.

4.4. Welding:
A. It is recommended the steel erection contractor and steel fabricator contact the Quality Assurance Agency prior to beginning any welds. A program of joint preparation and welding procedures should be worked out between the two parties before the welding is started so that correct welds will be made from the beginning.

B. Certification of Welders: All shop and field welding shall be executed by AWS certified welders who have been specifically certified for the process of welding being performed. The welder's certification will be considered as being current unless the welder is not engaged in the process of welding being performed for a period exceeding six months or there is a specific reason to question a welder's ability as required by AWS. Certification and records must comply with AWS Standards. Certification and appropriate records must be provided to the architect prior to beginning work.

C. Electrodes: E-70 XX or as noted otherwise. E60 XX may be used for welding steel floor and roof decks.

D. Minimum Welds: All intersecting steel shapes that are not bolted shall be connected by a fillet weld all around, unless noted otherwise. Fillet weld sizes that are not shown shall be 1/16" less than the thinnest of the connected parts for thicknesses 1/4" and larger. Fillet welds on plates less than 1/4" shall be of the same size as the thinnest of the connected parts.

E. Reinforcing Bars: Do not weld rebar except as specifically detailed in the drawings. In such cases, use only AWS standards. Do not substitute reinforcing bars for deformed bar anchors (DBAs), machine bolts, or headed stud anchors (HSAs).

F. Bolts: Do not apply any welds, including "tack" welds to bolts, including anchor bolts, except as specifically detailed in the drawings.

G. Headed Stud Anchor (HSA) welding and Deformed Bar Anchor (DBA) welding shall conform to the manufacturer's specifications. Welding shall comply with AWS D1.1 Section 7.6 through 7.9 and Annex G.

H. Special Provisions for Welds in the SFRS (Seismic Force Resisting System): Welds used in members and connections of moment frames, braced frames, and collector elements shall comply with these requirements. Welding methods, procedures and quality control shall comply with AISC 341 Chapter J, AWS D1.1, AWS D1.8 and the following:

1. Demand Critical Welds: The following CJP groove welds are demand critical and shall comply with the special requirements for Demand Critical Welds.
   a. Beam flanges to columns, single plate shear connections to columns, and beam webs to columns in moment frames.
   b. Column splice welds including column bases in moment frames and braced frames.
   c. Link beams to columns in Eccentrically Braced Frames.
   d. Web plate to flange plate welds in built-up Eccentrically Braced Frame link beams.
   e. Other welds designated as demand critical in the drawings.
2. Welding shall be performed in accordance with AISC 341 Chapter J and a welding procedure specification (WPS) as required in AWS D1.1. WPS variables shall be within the parameters established by the filler metal manufacturer. WPS for demand critical welds shall also comply with AWS D1.8 Section 6.1.
3. Consumables for Welding:
   a. Welds used in members and connections of the SFRS shall be made with filler metals meeting the requirements specified in section 6.3 of AWS D1.8.
   b. Filler metal properties shall be as follows:

Property	70 ksi Classification	80 ksi Classification
Yield Strength, ksi	58 min	68 min
Tensile Strength, ksi	70 min	80 min
Elongation (%)	22 min	19 min
CVN Toughness, ft-lbf	20 min @ 0 degrees F	20 min @ 20 degrees F

   c. Filler metals in Demand Critical Welds shall receive Heat Input Testing that achieves the properties listed above with CVN toughness of 40 ft-lbf min @ 70 degrees F and shall comply with AWS D1.8 section 6.3.5 to 6.3.8.
   d. Diffusible Hydrogen: Welding electrodes and electrode-flux combinations shall meet the requirements of AWS D1.8 Table 6.3. The manufacturer's Certificate of Conformance shall be considered adequate proof of this requirement.
   e. Intermixed filler metals shall meet the requirements of AWS D1.8 section 6.3.4.
4. Backer bars shall be removed from the beam bottom flange to columns. The root of the weld shall be back gouged to sound metal to remove all slag and cracks. Weld the back gouged region and finish welding using a reinforcing fillet weld. Comply with AWS D1.8 sections 6.7 and 6.8. This requirement also applies to all non-fusible backing used at beam to column CJP welds. Comply with AWS D1.8 section 6.9.
5. Steel backer bars need not be removed from the beam top flange connections to columns or at continuity plate connections to columns provided that the backer bars are welded to the column flange with a continuous 5/16 inch fillet weld on the edge below the CJP groove weld for the entire length of the backer bar.
6. Backing at beam flange to column flange joints shall not be welded to the underside of the beam flange, nor tack welded at this location. If fillet welds or tack welds are placed between the backing and the beam flange in error, they shall be repaired per AWS D1.8 Section 6.9.3.
7. Details and treatment of weld tabs shall be per AWS D1.8 Section 6.11. Use weld tabs as specified in AWS D1.1 Section 5.31 except at the end of CJP welds adjacent to the column web/flange juncture at continuity plates. Remove weld tabs to within 1/8 inch of the base metal surface after welding. Where weld tabs are used at continuity plates, remove them to within 1/4 inch of the base metal surface after welding. Finish the edge where weld tabs are removed to a surface roughness of 500 micro-inches.
8. CJP joints in members with different thickness or widths (such as column splices) shall be transitioned in a manner that the slope in the transition does not exceed 1 in 2.1/2. The transition shall be accomplished by chamfering the thicker part, tapering the wider part, sloping the weld metal, or by a combination of these.
9. Quality requirements for weld access holes for all demand critical welds shall comply with AWS D1.8 Section 6.10. Weld access hole shape shall be per AWS D1.8 Figure 6.2.
10. Beam bottom flange welding sequence shall comply with AWS D1.8 Section 6.14.
11. Preheat, and interpass temperatures shall comply with AWS D1.1 Section 3.5 and AWS D1.8 Section 6.5.
12. Additional welding provisions applicable to demand critical welds only are as follows:
    a. Welding processes shall comply with AWS D1.8 Section 6.2.
    b. Filler metal packaging and exposure limitations shall comply with AWS D1.8 Section 6.4.

13. Tack welds shall comply with AWS D1.1 Section 5.18 and AWS D1.8 Section 6.6 and 6.16. Tack welds attaching backing bars and weld tabs at demand critical welds shall be placed where they are incorporated into a final weld.
14. Imperfections such as cracks, gouges, grooves, arch strikes and notches will not be permitted within the Protected Zone. Imperfections within the Protected Zone shall be repaired or removed in accordance with AWS D1.8 Section 6.15.4.
15. Braced Frame Welding: Lengths shown for fillet welds at brace-to-gusset, gusset-to-baseplate, and column-to-gusset connections are minimums, intended for establishing gusset plate dimensions. Weld entire contact length at these joints, typical.

4.5. Bolted Connections:

A. Provide snug tightened joints with ASTM A325N Type 1 bolts for steel to steel connections, as noted herein or as noted on the drawings. Snug tightened joints shall be used in connections for simple span framing and beam (or girder) to bearing plate connections. The snug tightened condition is usually attained by a few impacts of an impact wrench or the full effort of a man using an ordinary spud wrench. Bolts shall be tightened until all plies of the joint are in firm contact.

B. Provide slip critical joints with ASTM A325 Type 1 bolts for steel to steel connections subject to fatigue with load reversal, joints that utilize oversized holes, and joints that utilize slotted holes not loaded perpendicular to the long direction of the slot. Faying surfaces shall meet the requirements of a slip-critical Class A surface. Tighten bolts by the turn of the nut, calibrated wrench, or direct tension indicator method. Alternate fastener designs as defined by AISC shall be submitted to the engineer for review and acceptability prior to installation.

C. Provide hardened washers beneath the turned element of all bolts or nuts. Provide hardened beveled washers, to compensate for the lack of parallelism, where the outer face of the bolted parts has a slope greater than one in twenty with respect to the plane normal to the bolt axis. Hardened washers or plates installed over oversized holes or slotted holes shall be at least 5/16" thick and shall conform to ASTM F436. Plates or bars installed at slotted holes shall have a size sufficient to completely cover the slot after installation.

D. Where a steel to steel beam connection is not detailed in the drawings, provide a standard AISC framed connection with the capacity to support one half of the total uniform load capacity of the given shape for the span and for the steel specified.

E. Bolts, nuts and washers shall not be reused.

4.6. Beam Web Stiffener Plates:

A. Provide full-height web stiffener plates to each side of all beams above all bearing points. Unless noted otherwise, stiffener plates shall be the thickness indicated in the typical stiffener plate detail.

4.7. Composite Beams

A. Composite beams are indicated on the framing plans with the suffix ( ). The number inside the brackets indicates number of headed stud anchors for this beam. Beams shall have the studs spaced uniformly over the entire beam length.

B. Beams which have more than one number inside the symbol (, ,), such as (W, X, Y, Z), shall have "W" number of studs spaced evenly over the first "section of beam", "X" number of studs spaced evenly over the second "section of beam", "Y" number of studs spaced evenly over the third "section of beam", and "Z" number of studs spaced evenly over the fourth "section of beam". A "section of beam" is defined as that portion of beam located between the column and the nearest intersecting framing member or that portion of beam located between two adjacent intersecting framing members.

C. Use 3/4" diameter studs. Headed studs shall extend 1.1/2" minimum above the top of the steel deck after welding. Headed studs shall be applied through the metal deck to the top flange of the steel section or welded directly to the steel section.

D. The minimum center-to-center spacing of stud connectors shall be six (6) diameters along the longitudinal axis of the supporting composite beam and four (4) diameters transverse to the longitudinal axis of the supporting composite beam. The maximum center to center spacing shall not exceed 32".

E. C= XX" on the plans denotes precamber dimension (upward) in inches.

4.8. Open Web Steel Joists and Girders:

A. The steel joist supplier shall be responsible for the design of all open web steel joists and girders. Joists or girders with slopes greater than 1/2 inch per foot shall be designed to meet or exceed the load capacities, listed in the SJI load tables, of the joist or girder sizes indicated on the framing plan, as if the joists or girders were installed level.

B. Open web joists and girders shall be designed with deflection limits of L/240 for total load and L/360 for live load, where L is the joist span.

C. Where uplift loads due to wind are indicated, rows of bottom chord bridging shall be provided at the first bottom chord panel points per SJI Specifications. This is in addition to the bridging shown on the framing plans.

D. Provide special bearing ends to accommodate slopes from sloped joists, sloped girders or sloped bearing conditions.

E. Camber: All joists shall be cambered as specified in the SJI specifications, unless noted otherwise.

F. Field Modifications: Do not modify any joist or girder, including holes through the top and bottom chords, without the written consent and direction from the manufacturer.

G. Shop Drawings and Design Calculations: Shop drawings for all joists and girders used in the project shall be submitted for review. Prior to the fabrication of joists and girders, the open web steel joist and girder manufacturer shall prepare complete joist and girder calculations under the direct supervision of a professional engineer licensed in the State of Utah. Calculations shall be submitted for review for joists and girders designated as SPECIAL or SP and for all joists or girders with axial loads or additional concentrated loads (as noted on the drawings). Submitted calculations shall bear the seal of a professional engineer licensed in the State of Utah.

H. Stabilizer Plates: Provide steel joist and joist girder stabilizer plates as indicated. Stabilizer plates shall be 6" x 6" with a 13/16" diameter hole with 1.1/2" minimum edge distance, and shall extend 3" minimum below the bottom chord of the joist or girder. Plate thickness shall be equal to the chord gap minus 1/4", or 3/4", whichever is less.

I. Verify size, weight, location and configuration of all roof top equipment with architect and mechanical engineer. Provide steel frames for support of roof top equipment as indicated on structural details in the structural drawings. Coordinate openings with the mechanical and general contractor.
J. All concentrated loads greater than 100 pounds supported by open web steel joists and girders shall be located within 6 inches of joist or girder panel points or the joist or girder shall be reinforced with an additional web member. Refer to the "TYPICAL DETAIL AT ADDITIONAL CONCENTRATED POINT LOAD" in the structural drawings.
K. Concentrated point loads, single or multiple, totaling 100 pounds or less can be located at any point along the top or bottom chord of an open web steel joist or girder between adjacent panel points without meeting the requirements above. A limit of four concentrated 100# maximum point loads per joist or girder will be permitted on spans of 12' or greater, one concentrated 100# max. load on spans less than 12', unless specifically noted otherwise on the structural drawings.
L. Joist bridging shall never be used to support hanging loads.
M. Bracing of miscellaneous items (mechanical, electrical, plumbing, etc.) to the bottom chord of joists or girders will not be allowed in any instance. All lateral braces must connect to the top flange/top chord of the framing member above unless noted otherwise on the structural drawings.
N. A certificate of compliance must be submitted to the building official upon completion of fabrication in accordance with IBC Section 2207.5.

4.9. Cold-Formed Steel

A. Non-Load-Bearing Exterior Cold-Formed Steel Framing:

1. Where steel framing size designators are used in the drawings, they follow the convention established by the Steel Stud Manufacturers' Association (SSMA) and the North American Steel Framing Alliance (NASFA). Framing members provided shall comply with the designations according to this convention.
2. All load bearing stud (and/or) joist framing members along with all runner, bridging, and end track shall be of the designation shown on the plans. All studs with base metal thickness of 54 mil and 68 mil, and joists with base metal thickness of 54 mil, 68 mil and 97 mil, shall be formed from steel meeting the requirements of ASTM A570 except that the steel shall have a 50,000 psi yield stress. All track and end closures with base metal thickness of 54 mil and 68 mil, bridging with base metal thickness of 54 mil, and studs and track with base metal thickness of 43 mil and 33 mil, shall be formed from steel with a minimum yield of 33,000 psi. All components shall be galvanized.
3. Follow all manufacturers' recommendations for the use of these products.
4. Unless noted otherwise, all welded connections shall be done using 1/8" AWS type 6013 or 7014 rod with a welding heat of 60-110 amperes depending on the gauge of material and the fit of the parts. Wire tying of framing components shall not be permitted. Welds and damaged coatings on studs shall be repaired with zinc galvanizing repair paint.
5. Fasteners for steel stud construction shall be self-drilling and self-tapping meeting ASTM C1513. Screw-type fasteners shall penetrate the joined materials with a minimum of three threads exposed. Furnish, install, and tighten screws per the manufacturer's recommendations and per the sizes indicated in the details. The minimum screw-type fastener size shall be #10 for any connection or the manufacturers' minimum recommended size for framing clips and bridging. Wall studs and jamb studs shall receive a minimum of two #12 screws per stud to track connection, one each side
6. Bridging shall be provided at a maximum spacing of 4'-0" on center, typical at all wall studs. Cold-rolled channel or steel angle bridging shall not be used without suitable full-depth angle clips fastened to the studs and channel or angle to prevent stud roll-over.
7. Wall to roof connections shall use steel clips designed to accommodate vertical deflection of the floor or roof structure. See specific details for further information. Clips shall be welded to the structural steel.

B. Non-Load-Bearing Interior Cold-Formed Steel Framing:

1. All interior non-bearing steel stud walls that extend above the ceiling but do not attach to the floor or roof diaphragm (above) shall have diagonal braces at 45 degrees (+/-). The KL/r ratio of the brace shall not exceed 200 and shall not be spaced further apart than 10'-0". Connect diagonal braces to the top of the steel stud walls and to the underside of the top flange of the steel beams, open web joists or girders, etc. with 1/8" fillet welds all around. Diagonal angle braces may be connected to 8" x 8" x 1/4" steel plates which shall be anchored to the floor or roof decks which have concrete fill above with 2- 3/8" dia. expansion anchors for each plate. Connect angle to plate with a 1/4" fillet weld all around. Connect diagonal braces to roof decking which does not have concrete fill above per the non-bearing wall brace connection details shown on the structural drawings. Diagonal braces may be constructed from cold-formed light gauge steel studs but must conform to the kl/r ratio of less than 200 requirement. When diagonal brace lengths exceed 10'-0" (+/-), cold-formed box sections made from two 600S137-54 steel studs will likely be required.

C. Prefabricated Systems: Submit complete shop drawings and calculations of all elements for review. Submittals shall bear the stamp of a Professional Engineer registered in the State of Utah.

4.10. Metal Bar Grating:

A. Metal bar grating shall comply with the most recent requirements of the National Association of Architectural Metals Manufacturers, Metal Bar Grating Division (NAAMM MBG). Products shall conform to the latest edition of the Metal Bar Grating Manual, ANSI/NAAMM MBG 531, or the Heavy Duty Metal Bar Grating Manual, ANSI/NAAMM MBG 532, as appropriate.
B. Materials: Unless noted otherwise, metal bar grating of the following types shall conform with the following standards:
   1. Steel: ASTM A569 (allowable fiber unit stress F = 18,000 psi)
   2. Stainless Steel: ASTM A167, alloys 304/316 (allowable fiber unit stress F = 20,000 psi)
   3. Aluminum: ASTM B221, alloys 6063-T6/6061-T6 (allowable fiber unit stress F = 12,000 psi)
C. Metal bar grating shall be provided with mill finish, unless otherwise noted.
D. Unless noted otherwise, provide W-19-4 (1.1/2" x 3/16") Steel grating in locations where metal bar grating is specified.
E. All metal bar grating shall be firmly and positively anchored to supporting members. Unless noted otherwise, weld grating to supporting members with 3/16" fillet welds, 3/4" long. Locate welds at each end of bearing bars approximately 6 in from each side of grating panel. At intermediate supports in panel, locate one weld at middle of panel.

5. Miscellaneous

5.1. Post-Installed Anchors in Concrete and Masonry

A. Anchorage to hardened concrete and grout-filled masonry shall include all mechanical and adhesive anchors and epoxy doweled reinforcing bars of size, quantity, spacing, and embedment as shown on the drawings. Additional anchors shall not be used without approval from the Engineer prior to installation.

B. Special inspection is required during the installation of all post-installed anchors. Refer to applicable code evaluation reports and the Quality Assurance and Statement of Special Inspections sections of the General Structural Notes.

C. Anchorage to Concrete:

1. All post-installed anchors into hardened concrete shall be selected from the following pre-approved products, unless noted otherwise:

Steel Screw Anchor	Evaluation Report (ICC_ES)
Hilti KWIK HUS-EZ	ESR-3027
Powers Wedge-Bolt+	ESR-2526
Simpson Titen HD	ESR-2713

Steel Expansion/Wedge Anchor	Evaluation Report (ICC_ES)
Hilti KWIK Bolt TZ	ESR-1917
ITW Red Head Trubolt+	ESR-2427
Powers Power-Stud+ SD2	ESR-2502
Simpson Strong-Bolt 2	ESR-3037

Adhesive Anchor System	Evaluation Report (ICC_ES)
Hilti HIT-HY 200	ESR-3187
Hilti HIT-RE 500-SD	ESR-2322
ITW Red Head Epcon C6+	ESR-3577
ITW Red Head Epcon S7	ESR-2308
Powers AC100+ Gold	ESR-2582
Powers Pure 110+	ESR-3298
Simpson SET-XP	ESR-2508

2. Adhesive anchors shall be installed into concrete having a minimum age of 21 days. For installations sooner than 21 days, consult the adhesive manufacturer

D. Alternate anchors or adhesives are permitted with approval of the engineer. The Contractor shall submit the proposed anchor product data and code evaluation report demonstrating the anchor is equivalent or exceeds the capacity of the specified anchor.

E. Installation of adhesive anchors horizontally or upwardly inclined to support sustained tension loads shall be performed by personnel certified by an applicable certification program. Certification shall include written and performance tests in accordance with the ACI/CRSI Adhesive Anchor Installer Certification program, or equivalent. Proof of current certification shall be submitted to the engineer for approval prior to commencement of installation.

F. Anchors shall be installed according to the manufacturer's published instructions and applicable code evaluation reports including:

1. Hole diameter, depth, and cleaning procedure
2. Adhesive mixing, preparation, and placement
3. Installation torque

G. Locate all existing reinforcement and embedded items prior to drilling into concrete or masonry elements. Do not damage rebar or embeds while drilling or installing anchors.

H. Grout all defective or abandoned holes with non-shrink grout or an injectable epoxy adhesive matching the surrounding concrete compressive strength. Consult the Architect for additional requirements at architecturally exposed concrete.

I. Drilled anchors are not allowed in post-tensioned concrete without approval of the architect and engineer.

J. Carbon steel anchors are limited to use in dry, interior locations.

6. Special Instructions

6.1. The project specifications are not superseded by the General Structural Notes but are intended to be complementary to them. Consult the specifications for additional requirements in each section. Notes and specific details on the drawings shall take precedence over General Structural Notes and typical details.

6.2. The architectural drawings are the prime contract drawings. Consultant drawings by other disciplines are supplementary to the architectural drawings. All omissions or conflicts, including dimensions, between the various elements of the consultants' drawings and/or specifications shall be brought to the attention of the Architect before proceeding with any work involved. In case of conflict, follow the most stringent requirement as directed by the Architect without additional cost to the owner. Any work done by the contractor after discovery of such discrepancy shall be done at the contractor's risk.

6.3. The structural drawings shall be used in conjunction with the architectural drawings. Primary structural elements and overall structural layout are indicated within the structural plans and details. Some secondary elements, architectural layouts, alcoves, elevations, slopes, depressions, curbs, mechanical equipment and electrical equipment, are not indicated within the structural drawings. Detailing and shop drawing production for structural elements will require information (including dimensions) contained in the architectural, structural and/or other consultants' drawings.

6.4. Shoring and Bracing Requirements:

A. Floor and Roof Structures -- The General Contractor is responsible for the method and sequence of all structural erection. He shall provide temporary shoring and bracing as his method of erection requires to provide adequate vertical and lateral support. Shoring and bracing shall remain in place as the chosen method requires until all permanent members are in place and all final connections are completed, including all roof and floor attachments. The building shall not be considered stable until all connections are complete.

B. Foundation walls must be braced until the complete floor or roof systems is completed. Do not backfill until floor or roof systems are in place.

C. Walls above grade shall be braced until the structural system is complete. Walls shall not be considered to be self-supporting.

6.5. All expansion joints (E.J.) shown in the structural drawings shall be considered seismic separation joints, unless noted otherwise.

6.6. Submittals: A copy of all shop drawings that have been submitted for review must be kept at the construction site for reference. These drawings must bear the appropriate review stamps. The shop drawing review shall not relieve the contractor of the responsibility of completing the project according to the contract documents. The general contractor shall review and mark all shop drawings prior to submitting them to the Architect for his review. Shop Drawings made from reproductions of (these) contract drawings will be rejected.

6.7. Project Coordination: It shall be the responsibility of the general contractor to coordinate with all trades any and all items that are to be integrated into the structural system. Openings or penetrations through, or attachments to the structural system that are not indicated on these drawings shall be the responsibility of the general contractor and shall be coordinated with the Architect/Engineers. The order of construction is the responsibility of the general contractor. It is the contractor's obligation to provide all items necessary for his chosen procedure.

6.8. Contractor shall field verify all dimensions, and conditions. If the contract drawings do not represent actual conditions, contractor shall notify architect/engineer prior to fabrication or construction within that area.

6.9. Notice of Copyright: The structural drawings, plans, schedules, notes and details are hereby copyrighted by Reaveley Engineers and Associates, Inc., All Rights reserved. Submission or distribution of documents to meet official regulatory requirements or for similar purposes in connection with the project is not to be construed as publication in derogation of Reaveley Engineers and Associates, Inc.'s reserved rights. The documents defining the structure are instruments of service prepared by Reaveley Engineers and Associates, Inc. for one use only. Furthermore, these documents shall not be reproduced, or copied, in whole or in part by the contractor or his subcontractors for preparation of shop drawings or other submittals.

7. Quality Assurance

7.1. Quality Assurance Agency Requirements:

A. The Owner shall engage a qualified Quality Assurance Agency (QAA) to provide all special inspection and quality assurance testing for the project. The QAA shall provide all information necessary for the building official to determine that the agency meets the applicable requirements

1. The QAA shall be objective, competent and independent from the contractor responsible for the work being inspected. The agency shall also disclose possible conflicts of interest to confirm objectivity.
2. The QAA shall have adequate equipment to perform required tests.
3. The QAA shall employ experienced personnel educated in conducting, supervising and evaluating tests and/or inspections. Experience or training shall be considered relevant when the documented experience or training is related in complexity to the same type of special inspection activities for projects of similar complexity and material qualities.
4. Prior to the start of construction, the QAA shall submit to the building official, the owner architect and engineer copies of the following:
    a. Current calibration records for all equipment to be used for the work being inspected and/or tested.
    b. Current certification and training records for each individual performing the inspections and/or testing.
    c. Sample inspection and testing reports and the distribution list for the records.
    d. Proposed inspection procedures and frequency for each inspection required by the work.
    e. Proposed testing methods and frequency of testing required by the work.
5. The QAA shall send copies of all inspection and testing reports to the building official, owner, architect, engineer and contractor. Reports shall indicate that the work inspected was or was not completed in conformance to the approved construction documents. Discrepancies shall be brought to the immediate attention of the contractor for correction. If they are not corrected, the discrepancies shall be brought to the attention of the building official, architect and engineer.
6. The QAA shall submit a final report documenting required special inspections and correction of any discrepancies noted in the inspections. The final report shall be distributed to the building official, owner, architect and engineer in a timely manner prior to the completion of the project.

7.2. Contractor Responsibilities:

A. Each contractor responsible for the construction of a system or component requiring special inspections or testing shall submit a written statement of responsibility to the building official, owner, architect and engineer prior to the commencement of the work. The contractor's statement of responsibility shall contain the following:

1. Acknowledgement of awareness of the special requirements defined in the statement of special inspections.

2. Acknowledgement that control will be exercised in order to obtain conformance to the approved construction documents.
3. Contractor's internal quality control procedures, methods and measures to be used in order to obtain conformance to the approved construction documents. Include copies of quality control reports, frequency of reporting and distribution of reports.
4. Identification and qualifications of the person(s) responsible for quality control and their position(s) within the organization.

B. Notification of Engineer: The contractor shall notify the engineer twenty-four hours prior to the items listed in the Structural Observations by the Engineer of Record section.

C. Notification of QAA: The contractor shall notify the QAA in a timely manner so that inspection and testing may be performed as outlined in the statement of special inspections.

7.3. Structural Observations by the Engineer of Record.

A. The Engineer of Record will perform structural observations at critical phases of the project as listed below. Observations will be made on a periodic basis throughout the construction of the structural system. During this time frame, site visits will be made approximately every two weeks. Copies of the engineer's report will be distributed to the architect, contractor, owner, and building official.
1. Placing concrete in any footing, mat footing, deep foundation, grade beam, or pier
2. Closing any wall forms
3. Placing concrete in any column, beam or suspended slab
4. Grouting any masonry
5. Completing the structural steel framing
6. Completing the welding of major sections of steel decking
7. Completing the nailing of any plywood wall or deck

B. Observation visits to the site by the Engineer's field representatives shall not be construed as inspection or approval of construction.

8. Statement of Special Inspections

8.1. The following materials, systems and components require special inspection or testing per Chapter 17 of the International Building Code (IBC).

8.2. For items requiring continuous inspection, a special inspector must be present onsite during the performance of that task. In most cases, periodic inspections/tests shall be performed prior to commencing the task, intermittently during the task, and at the completion of the task.

Structural Steel per IBC Section 1705.2.1, 1705.11.1 & 1705.12.2

Item	Frequency	Detailed Instructions
Prior to Welding (Table N5.4-1, AISC 360-10):		
Verify welding procedures (WPS) and consumable certificates	Continuous	
Material identification	Periodic	Verify type and grade of material.
Welder identification	Periodic	A system shall be maintained by which a welder who has welded a joint or member can be identified.
Fit-up groove welds	Periodic	Verify joint preparation, dimensions, cleanliness, tacking, and backing.
Access holes	Periodic	Verify configuration and finish.
Fit-up of fillet welds	Periodic	Verify alignment, gaps at root, cleanliness of steel surfaces, and tack weld quality and location.
During Welding (Table N5.4-2, AISC 360-10):		
Use of qualified welders	Periodic	Verify that welders are appropriately qualified.
Control and handling of welding consumables	Periodic	Verify packaging and exposure control.
Cracked tack welds	Periodic	Verify that welding does not occur over cracked tack welds.
Environmental conditions	Periodic	Verify win speed is within limits as well as precipitation and temperature.
WPS followed	Periodic	Verify items such as settings on welding equipment, travel speed, welding materials, shielding gas type/flow rate, preheat applied, interpass temperature maintained, and proper position.
Welding techniques	Periodic	Verify interpass and final cleaning, each pass is within profile limitations, and quality of each pass.
After Welding (Table N5.4-3, AISC 360-10):		
Welds cleaned	Periodic	Verify that welds have been propyl cleaned.
Size, length, and location of welds	Continuous	
Welds meet visual acceptance criteria	Continuous	
Arc strikes	Continuous	
k-area	Continuous	
Backing & weld tabs removed	Continuous	
Repair activities	Continuous	
Document acceptance or rejection of welded joint/member	Continuous	
Nondestructive Testing (Section N5.5, AISC 360-10):		
CJP welds (Risk Cat. II)	Periodic	Ultrasonic testing shall be performed on 10% of CJP groove welds in butt, T- and corner joints subject to transversely applied tension loading in materials 5/16-inch thick or greater. Testing rate must be increased if > 5% of welds tested have unacceptable defects.
Access holes (flange > 2”)	Continuous	
Welded joints subject to fatigue	Continuous	
After Bolting (Table N5.6-3, AISC 360-10):		
Document acceptance or rejection of bolted connections	Continuous	
Other Steel Inspections (Section N5.7, AISC 360-10: Table J8-1, J10-1, AISC 341-10):		
Structural steel details	Periodic	All fabricated steel or steel frames shall be inspected to verify compliance with the details shown in the construction documents, such as braces, stiffeners, member locations, and proper application of joint details at each connection.

Item	Frequency	Detailed Instructions
Anchor rods and other embedments supporting structural steel	Periodic	Shall be on the premises during the placement of anchor rods and other embedments supporting structural steel for compliance with construction documents. Verify the diameter, grade, type, and length of the anchor rod or embedded item, and the extent or depth of embedment prior to placement of concrete.
Protected zones	Periodic	Verify that no holes or unapproved attachments are made within the protected zone *(see Table J8-1 of AISC 341-10)*.
Steel Elements of Composite Construction (Table N6.1, AISC 360-10; Tables J9-1 thru J9-3, AISC 341-11):		
Placement and installation of steel deck	Continuous	
Placement and installation of steel headed stud anchors	Continuous	
Document acceptance or rejection of steel elements	Continuous	
Reinforcing steel	Periodic	Verify appropriate reinforcement size, spacing, and orientation; that it has not been re-bent in field; that it is correctly tied and supported; and that required steel clearances have been provided.
Composite member size	Periodic	Verify that composite member is the required size.

Steel Construction Other Than Structural Steel per IBC Section 1705.2.2

Item	Frequency	Detailed Instructions
Steel Roof and Floor Decks (IBC Table 1705.2.2):		
Material verification of cold-formed steel deck	Periodic	Confirm that identification markings are provided to conform to ASTM standards specified on construction documents.
Floor and roof deck welds	Periodic	Visual inspection is required to confirm that weld meets acceptance criteria of AWS D1.3. Welder qualifications should also be verified.

Concrete Construction per IBC Sections 1705.3 &1705.12.1

Item	Frequency	Detailed Instructions
Reinforcing steel	Periodic	Verify prior to placing concrete that reinforcing is of specified type, grade and size; that it is free of oil, dirt and rust; that it is located and spaced properly; that hooks, bends, ties, stirrups and supplemental reinforcement are placed correctly; that lap lengths, stagger and offsets are provided; and that all mechanical connections are installed per the manufacturer's instructions and/or evaluation report.
Welding of reinforcing steel	Periodic	Verify weldability of reinforcing steel other than A706. Continuous inspection is required for welding of reinforcing steel used in intermediate or special concrete moment frames, boundary elements of special structural walls or shear reinforcement.
Cast-in bolts & embeds	Continuous	
Post-installed anchors or dowels		All post-installed anchors/dowels shall be specially inspected as required by the approved ICC-ES report.
Use of required mix design	Periodic	Verify that all mixes used comply with the approved construction documents; ACI 318: Ch. 4, 5.2-5.4; and IBC 1904.3, 1913.2, 1913.3.
Concrete sampling for strength tests, slump, air content, and temperature	Continuous	Samples for strength tests shall be taken in accordance with ASTM C172, cured per ASTM C31 and tested in accordance with ASTM C39. Acceptance criteria for strength tests shall be per ACI 318 Section 5.6.3.3. For each mix placed, samples shall be taken not less than once a day, nor less than once for each 150 yd^3 of concrete, nor less than once for each 5000 ft^2 of surface area for slabs or walls. At the time fresh concrete is is sampled to fabricate specimens for strength tests, perform slump and air content tests and determine the temperature of the concrete.
Concrete & shotcrete placement	Continuous	
Curing temperature and techniques	Periodic	Verify that the ambient temperature for concrete is kept at > 50°F for at least 7 days after placement. High-early-strength concrete shall be kept at > 50°F for at least 3 days. Accelerated curing methods may be used (see ACI 318: 5.11.3). The ambient temperature for shotcrete shall be > 40°F for the same period of time as noted for concrete. Shotcrete shall be kept continuously moist for at least 24 hours after shotcreting. All concrete materials, reinforcement, forms, fillers, and ground shall be free from frost. In hot weather conditions ensure that appropriate measures are taken to avoid plastic shrinkage cracking and that the specified water/cement ratio is not exceeded.
In-situ strength verification	Periodic	Verify that adequate strength has been achieved prior to the removal of shores and forms or the stressing of post-tensioned tendons.
Formwork	Periodic	Verify that the forms are placed plumb and conform to the shapes, lines, and dimensions of the members as required by the approved construction documents.
Reinforcement in special structural walls and all components of special structural walls including coupling beams and wall piers	Periodic	Verify that ASTM A 615 reinforcing steel used in these areas complies with ACI 318: 21.1.5.2 by means of certified mill test reports. If this reinforcing steel is to be welded chemical tests shall be performed in accordance with ACI 318: 3.5.2.

Soils per IBC Section 1705.6

Item	Frequency	Detailed Instructions
Verify subgrade is adequate to achieve design bearing capacity	Periodic	Prior to placement of concrete.
Verify excavations extend to proper depth and material	Periodic	Prior to placement of compacted fill or concrete.
Verify that subgrade has been appropriately prepared prior to placing compacted fill	Periodic	Prior to placement of compacted fill.
Perform classification and testing of compacted fill materials	Periodic	All materials shall be checked at each lift for proper classifications and gradations not less than once for each 10,000ft^2 of surface area.

Item	Frequency	Detailed Instructions
Verify proper materials, densities and lift thicknesses during placement and compaction.	Continuous	

BUILDING CODE FOR LIVE LOAD REDUCTION: IBC

Live Load Reduction Method: General

ROOF LOADS:

Consider Snow Loads, Ignore Roof Live Loads

DETERMINING NUMBER OF STORIES FOR LIVE LOAD REDUCTION:

Include Roof Levels:	No
Include Unreducible Levels:	Yes
Include Storage Levels:	Yes

SELF-WEIGHT:

Automatically calculate and include Self-Weight for Member Dead Loads:

Beams:	Yes
Columns:	Yes
Walls:	Yes
Slabs / Decks:	Yes

Automatically calculate and include Self-Weight for Story Masses:

Beams:	Yes
Columns:	Yes
Include half mass of columns above and below	
Walls:	Yes
Include half mass of walls above and below	
Slabs / Decks:	Yes

02
GRAVITY LOADS

REAVELEY
ENGINEERS + ASSOCIATES
Consulting Structural Engineers

SURFACE LOADS

2017.080 Powder Mountain Parcel 4

ROOF AND FLOOR LOADING

NUMBER	01	02	03	04	05	06	07	08	09	10	11	12	13	14
LABEL	TYP FLOOR	EVENT SPACE	EXIT AND CORRIDOR	GYMNASIUM	TERRACE	TERRACE OVER MECH	TYP ROOF	UNHEATED TERRACE	CONC ROOF	TYP FLOOR OVER MECH				
DEAD LOADS														
METAL DECK	2.7	2.7	2.7	2.7	2.7	2.7	2.9	0	2.7	2.7	0	0	0	0
CONCRETE TOPPING	56.3	56.3	56.3	56.3	56.3	56.3	0	100	56.3	56.3	0	0	0	0
PURLINS	3	3	3	3	3	3	3	3	3	3	0	0	0	0
GIRDERS	2	2	2	2	2	2	2	2	2	2	0	0	0	0
COLUMNS	4	4	4	4	4	4	4	4	4	4	0	0	0	0
MOMENT FRAMES	0	0	0	0	0	0	0	0	5	0	0	0	0	0
DEFL CONC WEIGHT	5	5	5	5	5	5	5	5	5	5	0	0	0	0
HOUSEKEEPING PADS	0	0	0	0	0	0	0	0	0	0	0	0	0	0
FIREPROOFING	0	0	0	0	0	0	0	0	0	0	0	0	0	0
ROOFING	0	0	0	0	55.3	55.3	5.25	55.3	55.3	0	0	0	0	0
FLOORING	3	3	3	3	0	0	0	0	0	3	0	0	0	0
CEILING BELOW	4	4	4	4	4	0	4	0	4	4	0	0	0	0
OWSJ	0	0	0	0	0	0	3.6	0	0	0	0	0	0	0
MECHANICAL	3	3	3	27	3	27	3	0	3	27	0	0	0	0
SPRINKLERS	2	2	2	2	2	2	2	0	2	2	0	0	0	0
LANDSCAPE	0	0	0	0	0	0	0	0	0	0	0	0	0	0
MISCELLANEOUS	3	3	3	3	3.2	3.2	2.7	3.2	3.2	4	0	0	0	0
TOTAL DL	88.0	88.0	88.0	112.0	140.5	160.5	37.4	172.5	145.5	113.0	0.0	0.0	0.0	0.0
SUPERIMPOSED (RSS)	20.0	20.0	20.0	44.0	72.5	92.5	25.5	63.5	72.5	45.0	0.0	0.0	0.0	0.0
CONST (RSS)	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	0.0	0.0	0.0	0.0

LIVE LOAD	100	100	100	100	100	100	20	100	20	20	0	0	0	0
REDUCIBLE	REDUCIBLE	UNREDUCIBLE	REDUCIBLE	UNREDUCIBLE	ROOF	ROOF	ROOF	ROOF	ROOF	REDUCIBLE	REDUCIBLE	REDUCIBLE	REDUCIBLE	REDUCIBLE
PARTITION LOADS	0	0	0	0	0	0	0	0	0	0	0	0	0	0
CONST LIVE (RSS)	20	20	20	20	20	20	20	20	20	20	0	0	0	0

SEISMIC MASS

METAL DECK	2.7	2.7	2.7	2.7	2.7	2.7	2.9	0	2.7	2.7	0	0	0	0
CONCRETE TOPPING	56.3	56.3	56.3	56.3	56.3	56.3	0	0	0	56.3	0	0	0	0
JOISTS	3	3	3	3	3	3	3	3	3	3	0	0	0	0
GIRDERS	2	2	2	2	2	2	2	2	2	2	0	0	0	0
COLUMNS	4	4	4	4	4	4	4	4	4	4	0	0	0	0
MOMENT FRAMES	0	0	0	0	0	0	0	0	5	0	0	0	0	0
DEFL CONC WEIGHT	5	5	5	5	5	5	5	5	5	5	0	0	0	0
HOUSEKEEPING PADS	0	0	0	0	0	0	0	0	0	0	0	0	0	0
FIREPROOFING	0	0	0	0	0	0	0	0	0	0	0	0	0	0
ROOFING	0	0	0	0	55.3	55.3	5.25	55.3	55.3	0	0	0	0	0
FLOORING	3	3	3	3	0	0	0	0	0	3	0	0	0	0
CEILING	4	4	4	4	4	0	4	0	4	4	0	0	0	0
SOFFIT	0	0	0	0	0	0	3.6	0	0	0	0	0	0	0
MECHANICAL	3	3	3	27	3	27	3	0	3	27	0	0	0	0
SPRINKLERS	2	2	2	2	2	2	2	0	2	2	0	0	0	0
LANDSCAPE	0	0	0	0	0	0	0	0	0	0	0	0	0	0
MISCELLANEOUS	3	3	3	3	3.2	3.2	2.7	3.2	3.2	4	0	0	0	0
LIVE	0	0	0	0.0	0	0	0	0	0	0	0	0	0	0
PARTITIONS	0	0	0	0	0	0	0	0	0	0	0	0	0	0
SNOW	0	0	0	0	53.4	53.4	53.4	64.1	53.4	0	0	0	0	0
TOTAL SEISM MASS	88	88	88	112.0	193.8	213.8	90.8	136.5	142.5	113.0	0.0	0.0	0.0	0
SEISMIC MASS (RSS)	20	20	20	44.0	125.8	145.8	78.9	127.5	125.8	45.0	0.0	0.0	0.0	0

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LINE LOAD SUMMARY

2017.080 Powder Mountain Parcel 4

CLADDING LOADS

CURRENT CLADDING SCHEME

DESIGN WEIGHTS

CLADDING	20 psf
WINDOWS	15 psf

GRAVITY

MAX OF 1 FLOOR ANY CLADDING OR 2 STORIES OF CURTAIN WALL

LATERAL

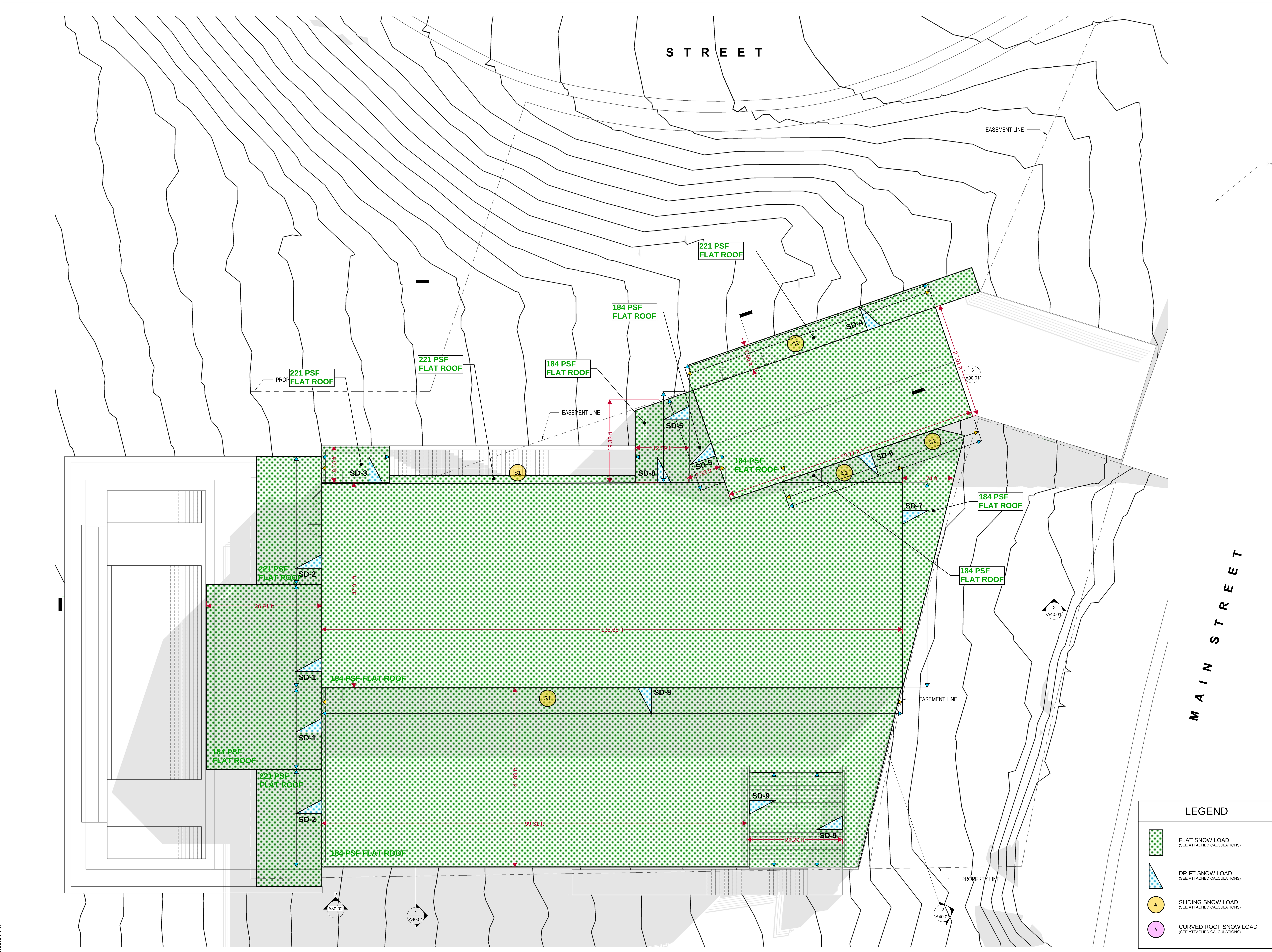
ONE FLOOR OF TRIBUTARY HEIGHT

TRIB HEIGHT	CLADDING	CURTAINWALL	BRICK	WINDOWS
	20 psf	15 psf	55 psf	15 psf
1 ft	0.020 klf	0.015 klf	0.055 klf	0.015 klf
2 ft	0.040 klf	0.030 klf	0.110 klf	0.030 klf
3 ft	0.060 klf	0.045 klf	0.165 klf	0.045 klf
4 ft	0.080 klf	0.060 klf	0.220 klf	0.060 klf
5 ft	0.100 klf	0.075 klf	0.275 klf	0.075 klf
6 ft	0.120 klf	0.090 klf	0.330 klf	0.090 klf
7 ft	0.140 klf	0.105 klf	0.385 klf	0.105 klf
8 ft	0.160 klf	0.120 klf	0.440 klf	0.120 klf
9 ft	0.180 klf	0.135 klf	0.495 klf	0.135 klf
10 ft	0.200 klf	0.150 klf	0.550 klf	0.150 klf
11 ft	0.220 klf	0.165 klf	0.605 klf	0.165 klf
12 ft	0.240 klf	0.180 klf	0.660 klf	0.180 klf
13 ft	0.260 klf	0.195 klf	0.715 klf	0.195 klf
14 ft	0.280 klf	0.210 klf	0.770 klf	0.210 klf
15 ft	0.300 klf	0.225 klf	0.825 klf	0.225 klf
16 ft	0.320 klf	0.240 klf	0.880 klf	0.240 klf
17 ft	0.340 klf	0.255 klf	0.935 klf	0.255 klf
18 ft	0.360 klf	0.270 klf	0.990 klf	0.270 klf
19 ft	0.380 klf	0.285 klf	1.045 klf	0.285 klf
20 ft	0.400 klf	0.300 klf	1.100 klf	0.300 klf
21 ft	0.420 klf	0.315 klf	1.155 klf	0.315 klf
22 ft	0.440 klf	0.330 klf	1.210 klf	0.330 klf
23 ft	0.460 klf	0.345 klf	1.265 klf	0.345 klf
24 ft	0.480 klf	0.360 klf	1.320 klf	0.360 klf
25 ft	0.500 klf	0.375 klf	1.375 klf	0.375 klf
26 ft	0.520 klf	0.390 klf	1.430 klf	0.390 klf
27 ft	0.540 klf	0.405 klf	1.485 klf	0.405 klf
28 ft	0.560 klf	0.420 klf	1.540 klf	0.420 klf
29 ft	0.580 klf	0.435 klf	1.595 klf	0.435 klf
30 ft	0.600 klf	0.450 klf	1.650 klf	0.450 klf
31 ft	0.620 klf	0.465 klf	1.705 klf	0.465 klf
32 ft	0.640 klf	0.480 klf	1.760 klf	0.480 klf

LINE LOADS

Cheek walls @ south terrace stairs

terrace_stair_cheek	1.125	klf
blw_sm_barn_west	1.5	klf



TOP OF ROOF 1/8" = 1'-0" 1

6/16/2017 2:39:56 PM

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POWDER MOUNTAIN

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SHEET TITLE

ROOF PLAN

No	Description	Date
	Update	2017/06/16

JOB NO. 236

DATE 06/16/2017

SCALE 1/8" = 1'-0"

SHEET NO.

A20.05

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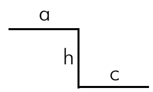
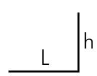
SNOW DRIFT SUMMARY

2017.080 Powder Mountain Parcel 4



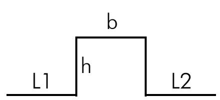
Roof Step Conditions

Mark	a (ft)	h (ft)	c (ft)	Drift (psf)	Drift (psf)	Length (ft)	Note
SD-1	136	42.3	27	409	206	27	Pool terrace drift at big barn
SD-2	136	42.3	27	445	243	27	Pool terrace drift at big barn; Unheated
SD-1	136	20.5	27	409	206	27	Pool terrace drift at upper terrace
SD-2	136	20.5	27	445	243	27	Pool terrace drift at upper terrace; Unheated
SD-3	48	25.8	9	366	302	8.6	Top of north stair; Unheated
SD-4	27	14.8	6	333	288	6	North of little barn; Unheated
SD-5	60	14.8	8	344	284	8	
SD-6	27	24.7	13	296	199	13	
SD-7	136	21.8	11.75	409	320	11.8	
SD-8	48	21.8	42	329	184	19.4	South of big barn
SD-9	99	8.0	22.3	240	184	14.9	South terrace stair



Parapet

Mark	h (ft)	L (ft)	Drift (psf)	Length (ft)	Note
SD-9	8.0	22	240	13.9	South terrace stair-alternate



Projection

Mark	L1 (ft)	h (ft)	b (ft)	L2 (ft)	Drift (psf)	Length (ft)	Note

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Calculation Sheet #________

SNOW LOADING

ASCE 7-10 Chapter 7

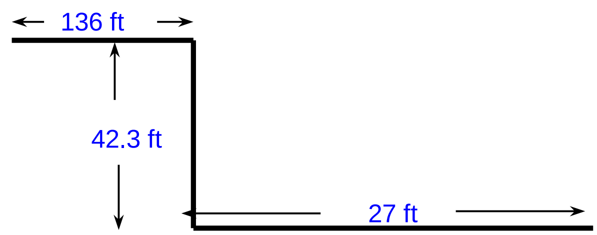
Project Name	*2017.080 Powder Mountain Parcel 4*	Engineer	*CAB*
Location	SD-1	Date	2017-06-17

General Input Information

Structure Importance Type:	*II - All Other Structures - Table 1.5-1*
Roof Exposure:	*Partially Exposed --- Table 7-2*
Terrain Category:	*Terrain Category C --- Section 26.7*
Structure Thermal Properties:	*All Other Structures --- Table 7-3*
Load Type:	*Utah Snow Load Study*

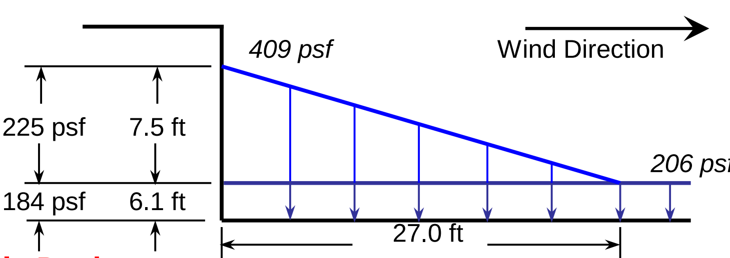
General Output Information

Building Section	14	
Ground Snow Load (p_g)	263 psf	*Snow Load Study*
Exposure Factor (C_e)	1.00	*Table 7-2*
Thermal Factor (C_t)	1.00	*Table 7-3*
Importance Factor (I_s)	1.00	*Table 1.5-2*
Flat Roof Snow Load (p_f)	184 psf	*Equation 7.3-1*
Snow Density (γ)	30.0 pcf	*Equation 7-4*



Leeward Drifting

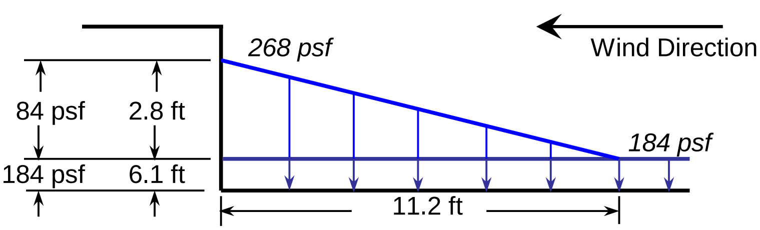
Step Height (h_c)	36.2 ft
Flat Roof Snow Depth (h_b)	6.1 ft
Roof Length (l_u)	136 ft
Drift Height (h_d)	7.5 ft
Drift Load (p_d)	225 psf
Drift Length (w)	29.9 ft



Leeward Drifting Controls Design

Windward Drifting

Step Height (hc)	36.2 ft
Flat Roof Snow Depth (hb)	6.1 ft
Roof Length (lu)	27 ft
Drift Height (hd)	2.8 ft
Drift Load (pd)	84 psf
Drift Length (w)	11.2 ft



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Calculation Sheet #________

SNOW LOADING

ASCE 7-10 Chapter 7

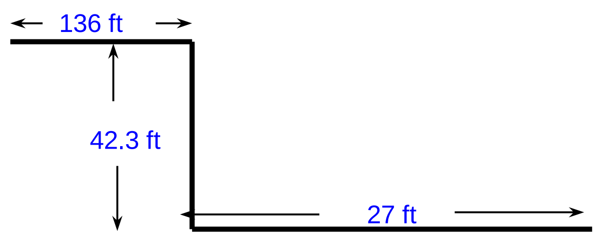
Project Name *2017.080 Powder Mountain Parcel 4* Engineer *CAB*
Location SD-2 Date 2017-06-17

General Input Information

Structure Importance Type:	*II - All Other Structures - Table 1.5-1*
Roof Exposure:	*Partially Exposed --- Table 7-2*
Terrain Category:	*Terrain Category C --- Section 26.7*
Structure Thermal Properties:	*Unheated and Open-Air Structures --- Table 7-3*
Load Type:	*Utah Snow Load Study*

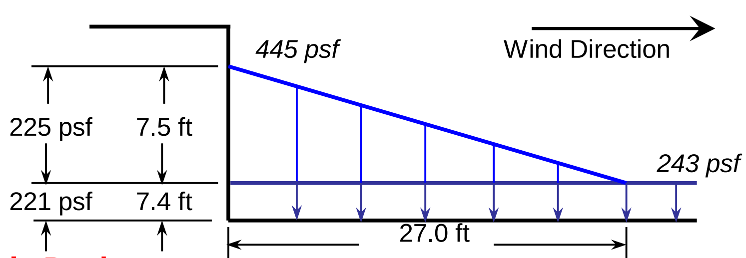
General Output Information

Building Section	14	
Ground Snow Load (p_g)	263 psf	*Snow Load Study*
Exposure Factor (C_e)	1.00	*Table 7-2*
Thermal Factor (C_t)	1.20	*Table 7-3*
Importance Factor (I_s)	1.00	*Table 1.5-2*
Flat Roof Snow Load (p_f)	221 psf	*Equation 7.3-1*
Snow Density (γ)	30.0 pcf	*Equation 7-4*



Leeward Drifting

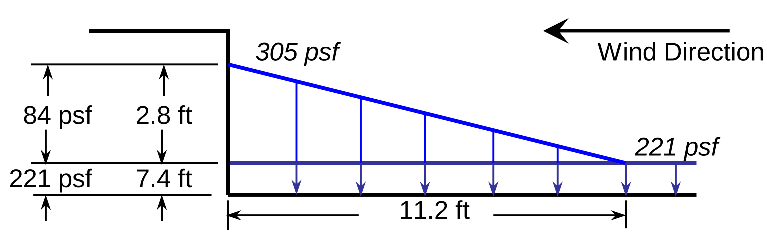
Step Height (h_c)	34.9 ft
Flat Roof Snow Depth (h_b)	7.4 ft
Roof Length (l_u)	136 ft
Drift Height (h_d)	7.5 ft
Drift Load (p_d)	225 psf
Drift Length (w)	29.9 ft



Leeward Drifting Controls Design

Windward Drifting

Step Height (hc)	34.9 ft
Flat Roof Snow Depth (hb)	7.4 ft
Roof Length (lu)	27 ft
Drift Height (hd)	2.8 ft
Drift Load (pd)	84 psf
Drift Length (w)	11.2 ft



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Calculation Sheet #________

SNOW LOADING

ASCE 7-10 Chapter 7

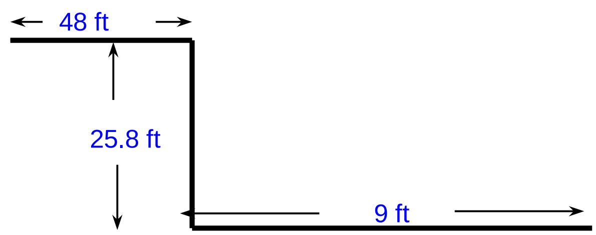
Project Name	*2017.080 Powder Mountain Parcel 4*	Engineer	*CAB*
Location	SD-3	Date	2017-06-17

General Input Information

Structure Importance Type:	*II - All Other Structures - Table 1.5-1*
Roof Exposure:	*Partially Exposed --- Table 7-2*
Terrain Category:	*Terrain Category C --- Section 26.7*
Structure Thermal Properties:	*Unheated and Open-Air Structures --- Table 7-3*
Load Type:	*Utah Snow Load Study*

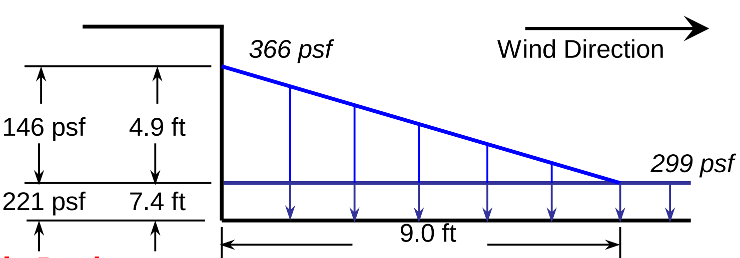
General Output Information

Building Section	14	
Ground Snow Load (p_g)	263 psf	*Snow Load Study*
Exposure Factor (C_e)	1.00	*Table 7-2*
Thermal Factor (C_t)	1.20	*Table 7-3*
Importance Factor (I_s)	1.00	*Table 1.5-2*
Flat Roof Snow Load (p_f)	221 psf	*Equation 7.3-1*
Snow Density (γ)	30.0 pcf	*Equation 7-4*



Leeward Drifting

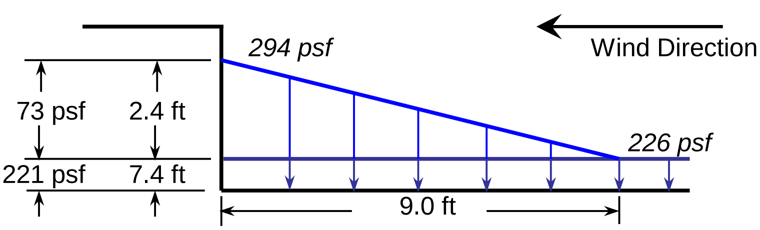
Step Height (h_c)	18.4 ft
Flat Roof Snow Depth (h_b)	7.4 ft
Roof Length (l_u)	48 ft
Drift Height (h_d)	4.9 ft
Drift Load (p_d)	146 psf
Drift Length (w)	19.4 ft



Leeward Drifting Controls Design

Windward Drifting *20 ft Length Governs lu (Figure 7-9)*

Step Height (hc)	18.4 ft
Flat Roof Snow Depth (hb)	7.4 ft
Roof Length (lu)	20 ft
Drift Height (hd)	2.4 ft
Drift Load (pd)	73 psf
Drift Length (w)	9.7 ft



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SNOW LOADING

ASCE 7-10 Chapter 7

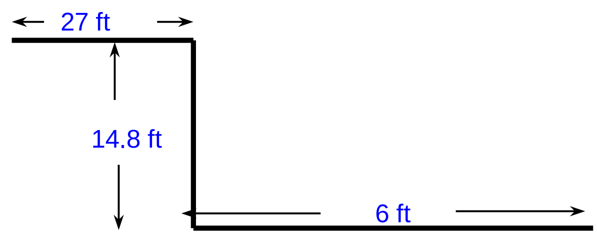
Project Name	*2017.080 Powder Mountain Parcel 4*	Engineer	*CAB*
Location	SD-4	Date	2017-06-17

General Input Information

Structure Importance Type:	*II - All Other Structures - Table 1.5-1*
Roof Exposure:	*Partially Exposed --- Table 7-2*
Terrain Category:	*Terrain Category C --- Section 26.7*
Structure Thermal Properties:	*Unheated and Open-Air Structures --- Table 7-3*
Load Type:	*Utah Snow Load Study*

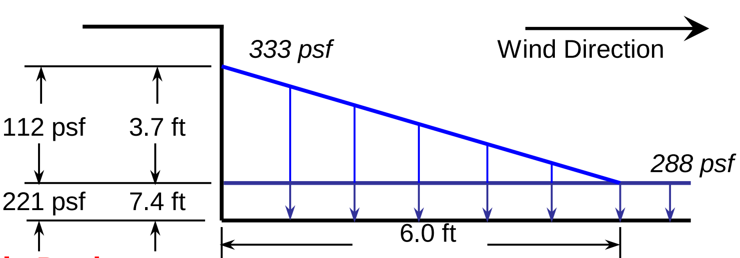
General Output Information

Building Section	14	
Ground Snow Load (p_g)	263 psf	*Snow Load Study*
Exposure Factor (C_e)	1.00	*Table 7-2*
Thermal Factor (C_t)	1.20	*Table 7-3*
Importance Factor (I_s)	1.00	*Table 1.5-2*
Flat Roof Snow Load (p_f)	221 psf	*Equation 7.3-1*
Snow Density (γ)	30.0 pcf	*Equation 7-4*



Leeward Drifting

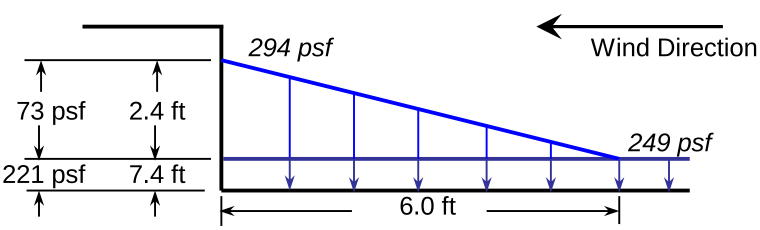
Step Height (h_c)	7.4 ft
Flat Roof Snow Depth (h_b)	7.4 ft
Roof Length (l_u)	27 ft
Drift Height (h_d)	3.7 ft
Drift Load (p_d)	112 psf
Drift Length (w)	15.0 ft



Leeward Drifting Controls Design

Windward Drifting *20 ft Length Governs lu (Figure 7-9)*

Step Height (hc)	7.4 ft
Flat Roof Snow Depth (hb)	7.4 ft
Roof Length (lu)	20 ft
Drift Height (hd)	2.4 ft
Drift Load (pd)	73 psf
Drift Length (w)	9.7 ft



Calculation Sheet #________

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SNOW LOADING

ASCE 7-10 Chapter 7

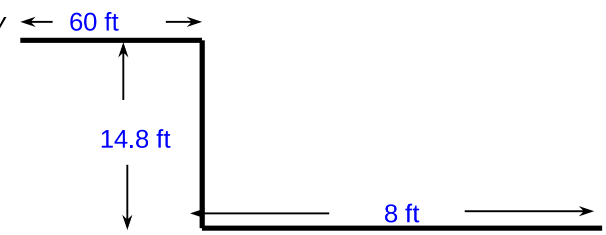
Project Name	*2017.080 Powder Mountain Parcel 4*	Engineer	*CAB*
Location	SD-5	Date	2017-06-17

General Input Information

Structure Importance Type:	*II - All Other Structures - Table 1.5-1*
Roof Exposure:	*Partially Exposed --- Table 7-2*
Terrain Category:	*Terrain Category C --- Section 26.7*
Structure Thermal Properties:	*All Other Structures --- Table 7-3*
Load Type:	*Utah Snow Load Study*

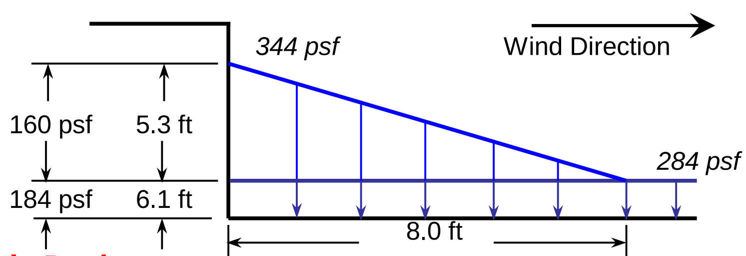
General Output Information

Building Section	14	
Ground Snow Load (p_g)	263 psf	*Snow Load Study*
Exposure Factor (C_e)	1.00	*Table 7-2*
Thermal Factor (C_t)	1.00	*Table 7-3*
Importance Factor (I_s)	1.00	*Table 1.5-2*
Flat Roof Snow Load (p_f)	184 psf	*Equation 7.3-1*
Snow Density (γ)	30.0 pcf	*Equation 7-4*



Leeward Drifting

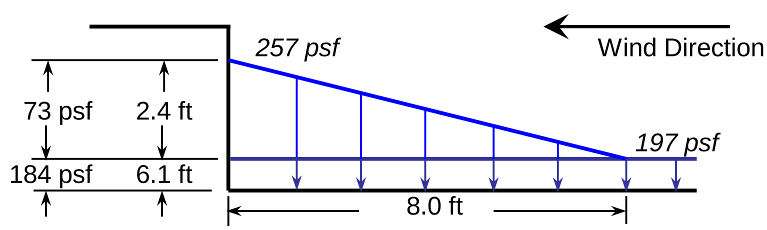
Step Height (h_c)	8.7 ft
Flat Roof Snow Depth (h_b)	6.1 ft
Roof Length (l_u)	60 ft
Drift Height (h_d)	5.3 ft
Drift Load (p_d)	160 psf
Drift Length (w)	21.4 ft



Leeward Drifting Controls Design

Windward Drifting *20 ft Length Governs lu (Figure 7-9)*

Step Height (hc)	8.7 ft
Flat Roof Snow Depth (hb)	6.1 ft
Roof Length (lu)	20 ft
Drift Height (hd)	2.4 ft
Drift Load (pd)	73 psf
Drift Length (w)	9.7 ft



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Software Version: 1.10

Calculation Sheet #________

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SNOW LOADING

ASCE 7-10 Chapter 7

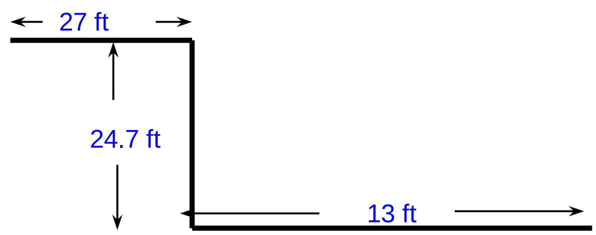
Project Name *2017.080 Powder Mountain Parcel 4*
Location SD-6
Engineer *CAB*
Date 2017-06-17

General Input Information

Structure Importance Type:	*II - All Other Structures - Table 1.5-1*
Roof Exposure:	*Partially Exposed --- Table 7-2*
Terrain Category:	*Terrain Category C --- Section 26.7*
Structure Thermal Properties:	*All Other Structures --- Table 7-3*
Load Type:	*Utah Snow Load Study*

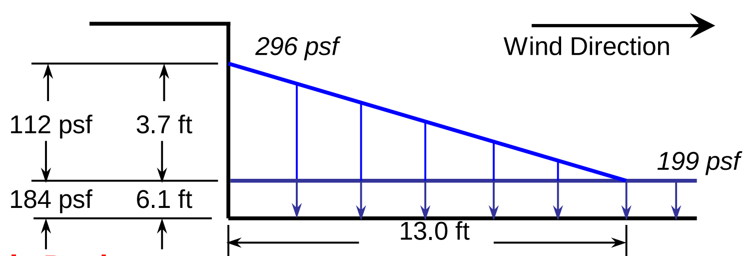
General Output Information

Building Section	14	
Ground Snow Load (p_g)	263 psf	*Snow Load Study*
Exposure Factor (C_e)	1.00	*Table 7-2*
Thermal Factor (C_t)	1.00	*Table 7-3*
Importance Factor (I_s)	1.00	*Table 1.5-2*
Flat Roof Snow Load (p_f)	184 psf	*Equation 7.3-1*
Snow Density (γ)	30.0 pcf	*Equation 7-4*



Leeward Drifting

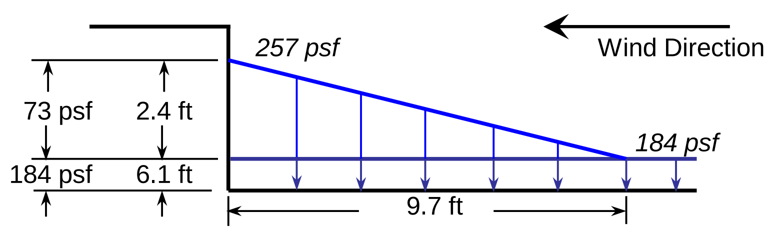
Step Height (h_c)	18.6 ft
Flat Roof Snow Depth (h_b)	6.1 ft
Roof Length (l_u)	27 ft
Drift Height (h_d)	3.7 ft
Drift Load (p_d)	112 psf
Drift Length (w)	15.0 ft



Leeward Drifting Controls Design

Windward Drifting *20 ft Length Governs lu (Figure 7-9)*

Step Height (hc)	18.6 ft
Flat Roof Snow Depth (hb)	6.1 ft
Roof Length (lu)	20 ft
Drift Height (hd)	2.4 ft
Drift Load (pd)	73 psf
Drift Length (w)	9.7 ft



Author: J. Ambrose
Software Version: 1.10

REAVELEY
ENGINEERS + ASSOCIATES
Consulting Structural Engineers

SNOW LOADING

ASCE 7-10 Chapter 7

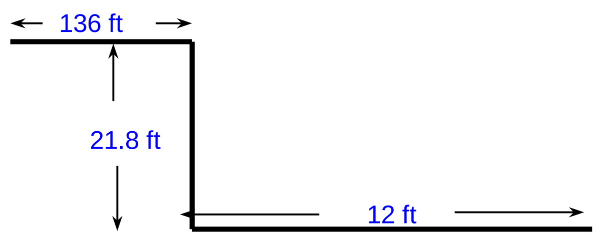
Project Name *2017.080 Powder Mountain Parcel 4* Engineer *CAB*
Location SD-7 Date 2017-06-17

General Input Information

Structure Importance Type:	*II - All Other Structures - Table 1.5-1*
Roof Exposure:	*Partially Exposed --- Table 7-2*
Terrain Category:	*Terrain Category C --- Section 26.7*
Structure Thermal Properties:	*All Other Structures --- Table 7-3*
Load Type:	*Utah Snow Load Study*

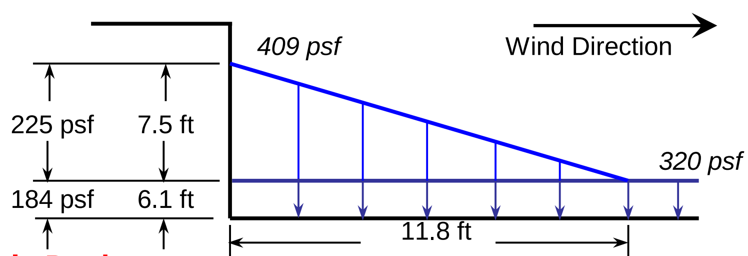
General Output Information

Building Section	14	
Ground Snow Load (p_g)	263 psf	*Snow Load Study*
Exposure Factor (C_e)	1.00	*Table 7-2*
Thermal Factor (C_t)	1.00	*Table 7-3*
Importance Factor (I_s)	1.00	*Table 1.5-2*
Flat Roof Snow Load (p_f)	184 psf	*Equation 7.3-1*
Snow Density (γ)	30.0 pcf	*Equation 7-4*



Leeward Drifting

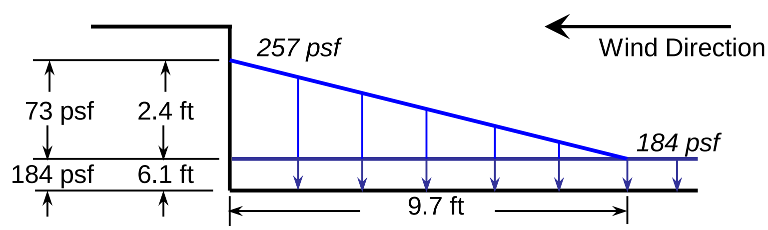
Step Height (h_c)	15.7 ft
Flat Roof Snow Depth (h_b)	6.1 ft
Roof Length (l_u)	136 ft
Drift Height (h_d)	7.5 ft
Drift Load (p_d)	225 psf
Drift Length (w)	29.9 ft



Leeward Drifting Controls Design

Windward Drifting *20 ft Length Governs lu (Figure 7-9)*

Step Height (hc)	15.7 ft
Flat Roof Snow Depth (hb)	6.1 ft
Roof Length (lu)	20 ft
Drift Height (hd)	2.4 ft
Drift Load (pd)	73 psf
Drift Length (w)	9.7 ft



Author: J. Ambrose
Software Version: 1.10

Calculation Sheet #________

REAVELEY ENGINEERS + ASSOCIATES
Consulting Structural Engineers

SNOW LOADING

ASCE 7-10 Chapter 7

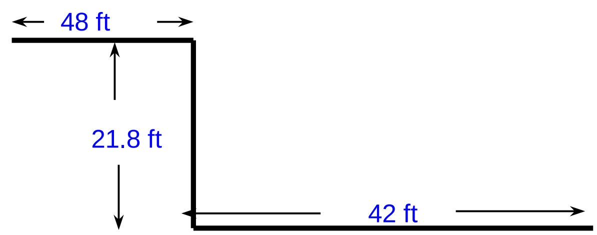
Project Name *2017.080 Powder Mountain Parcel 4*
Location SD-8
Engineer *CAB*
Date 2017-06-17

General Input Information

Structure Importance Type:	*II - All Other Structures - Table 1.5-1*
Roof Exposure:	*Partially Exposed --- Table 7-2*
Terrain Category:	*Terrain Category C --- Section 26.7*
Structure Thermal Properties:	*All Other Structures --- Table 7-3*
Load Type:	*Utah Snow Load Study*

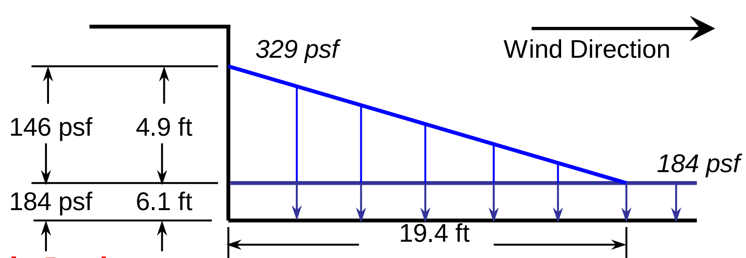
General Output Information

Building Section	14	
Ground Snow Load (p_g)	263 psf	*Snow Load Study*
Exposure Factor (C_e)	1.00	*Table 7-2*
Thermal Factor (C_t)	1.00	*Table 7-3*
Importance Factor (I_s)	1.00	*Table 1.5-2*
Flat Roof Snow Load (p_f)	184 psf	*Equation 7.3-1*
Snow Density (γ)	30.0 pcf	*Equation 7-4*



Leeward Drifting

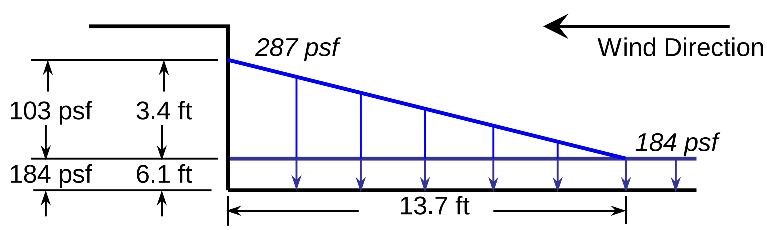
Step Height (h_c)	15.7 ft
Flat Roof Snow Depth (h_b)	6.1 ft
Roof Length (l_u)	48 ft
Drift Height (h_d)	4.9 ft
Drift Load (p_d)	146 psf
Drift Length (w)	19.4 ft



Leeward Drifting Controls Design

Windward Drifting

Step Height (hc)	15.7 ft
Flat Roof Snow Depth (hb)	6.1 ft
Roof Length (lu)	42 ft
Drift Height (hd)	3.4 ft
Drift Load (pd)	103 psf
Drift Length (w)	13.7 ft



Author: J. Ambrose
Software Version: 1.10

Calculation Sheet #________

REAVELEY ENGINEERS + ASSOCIATES
Consulting Structural Engineers

SNOW LOADING

ASCE 7-10 Chapter 7

Project Name *2017.080 Powder Mountain Parcel 4*
Location SD-9

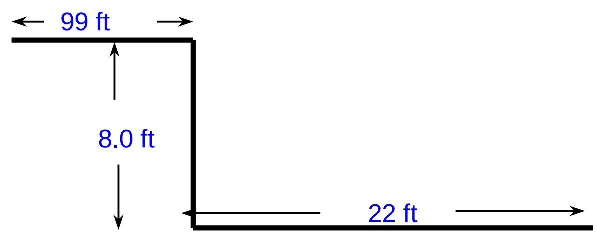
Engineer *CAB*
Date 2017-06-17

General Input Information

Structure Importance Type:	*II - All Other Structures - Table 1.5-1*
Roof Exposure:	*Partially Exposed --- Table 7-2*
Terrain Category:	*Terrain Category C --- Section 26.7*
Structure Thermal Properties:	*All Other Structures --- Table 7-3*
Load Type:	*Utah Snow Load Study*

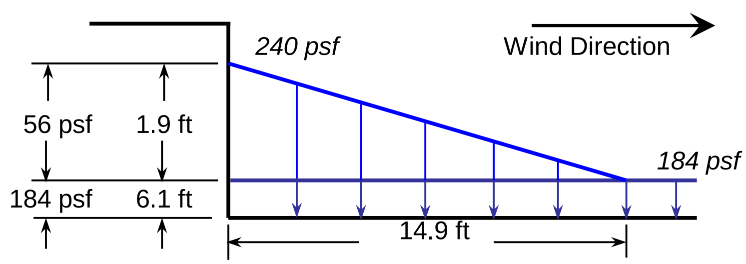
General Output Information

Building Section	14	
Ground Snow Load (p_g)	263 psf	*Snow Load Study*
Exposure Factor (C_e)	1.00	*Table 7-2*
Thermal Factor (C_t)	1.00	*Table 7-3*
Importance Factor (I_s)	1.00	*Table 1.5-2*
Flat Roof Snow Load (p_f)	184 psf	*Equation 7.3-1*
Snow Density (γ)	30.0 pcf	*Equation 7-4*



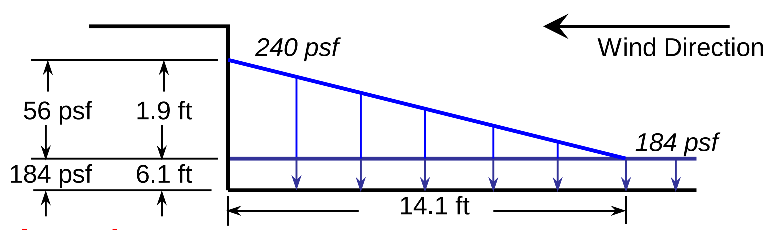
Leeward Drifting

Step Height (h_c)	1.9 ft
Flat Roof Snow Depth (h_b)	6.1 ft
Roof Length (l_u)	99 ft
Drift Height (h_d)	1.9 ft
Drift Load (p_d)	56 psf
Drift Length (w)	14.9 ft



Windward Drifting

Step Height (hc)	1.9 ft
Flat Roof Snow Depth (hb)	6.1 ft
Roof Length (lu)	22 ft
Drift Height (hd)	1.9 ft
Drift Load (pd)	56 psf
Drift Length (w)	14.1 ft



Windward Drifting Controls Design

REAVELEY
ENGINEERS + ASSOCIATES
Consulting Structural Engineers

Project: 2017.080 POWDER MOUNTAIN PARCEL 4
SNOW LOADS

Date: 2017-06-17 By: CAB

BIG BARN:
ASCE 7-10

INPUT VARIABLES:

Ground snow load: $p_g := 263\ psf$

Flat roof snow load: $p_f := 184\ psf$

Horizontal distance from eave to ridge: $W := 24\ ft$

Angle of eave from horizontal: $\theta := 26\ deg$

Snow density: $\gamma := 30.0\ pcf$

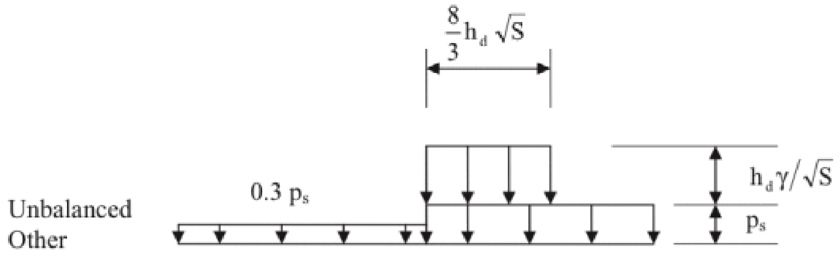
SLOPED ROOF SNOW LOAD:

$C_s := 1.0$ <-- From ASCE 7-10 Figure 7-2 p. 37

$p_s := C_s \cdot p_f = 184\ psf$ <-- ASCE 7-10 Eq. 7.4-1 p. 31

Apply the sloped roof snow load uniformly over the roof in place of the flat roof snow load.

UNBALANCED SNOW LOAD:



$l_u := W = 24\ ft$ <-- From ASCE 7-10 § 7.6.1 p. 32

$$S := \frac{1}{\tan(\theta)} = 2.05$$

$$h_d := \left(0.43 \cdot \sqrt[3]{l_u \cdot ft^{-1}} \cdot \sqrt[4]{p_g \cdot psf^{-1} + 10} - 1.5\right) \cdot ft = 3.5\ ft$$ <-- From ASCE 7-10 Figure 7-9 p. 41

$SL_{windward} := 0.3 \cdot p_s = 55.2\ psf$

$$SL_{leeward\_up} := \frac{h_d \cdot \gamma}{\sqrt{S}} = 74.2\ psf$$

$$dist_{leeward} := \frac{8 \cdot h_d \cdot \sqrt{S}}{3} = 13.5\ ft$$

$SL_{leeward\_down} := p_s = 184.0\ psf$

Apply the unbalanced snow load per the "Unbalanced Other" figure of ASCE 7-10 Figure 7-5.

SLIDING SNOW LOAD:

Length of sliding snow load perpendicular to building: $L := 15\ ft$ <-- From ASCE 7-10 § 7.9 p. 33

$$SL_{sliding} := \frac{0.4 \cdot p_f \cdot W}{L} = 118\ psf$$ <-- From ASCE 7-10 § 7.9 p. 33

Apply the uniform sliding snow load (S1) along the length of the eave at a perpendicular distance of 15' from the eave to the roof below. Superimpose the sliding snow load on the balanced snow load. Do not apply it in conjunction with drift.

NORTH EXTERIOR STAIR:

INPUT VARIABLES:

Stair height: $H := 19\ ft$

Projected stair length: $L_{proj} := 37\ ft$

Stair width: $B := 6.5\ ft$

Slab weight: $DL_{slab} := 70\ psf$

Stringer weight: $w_{DL\_stringer} := 200\ plf$

Flat snow load: $SL_{flat} := 221\ psf$

Sliding snow load: $SL_{sliding} = 118\ psf$

STRINGER REACTIONS:

$$w_{DL} := w_{DL\_stringer} + 0.5 \cdot B \cdot DL_{slab} = 0.428\ klf$$

$$w_{DL\_proj} := DL_{proj}\left(w_{DL}, L_{proj}, H\right) = 0.481\ klf$$

$$w_{SL} := 0.5 \cdot B \cdot \left(SL_{flat} + SL_{sliding}\right) = 1.101\ klf$$

$$P_{DL\_stringer} := V_{unif}\left(w_{DL\_proj}, L_{proj}\right) \cdot 120\% = 10.7\ kip$$

$$P_{SL\_stringer} := V_{unif}\left(w_{SL}, L_{proj}\right) \cdot 120\% = 24.4\ kip$$

LITTLE BARN:
ASCE 7-10

INPUT VARIABLES:

Ground snow load: $p_g := 263\ psf$

Flat roof snow load: $p_f := 184\ psf$

Horizontal distance from eave to ridge: $W := 13.5\ ft$

Angle of eave from horizontal: $\theta := 30.6\ deg$

Snow density: $\gamma := 30.0\ pcf$

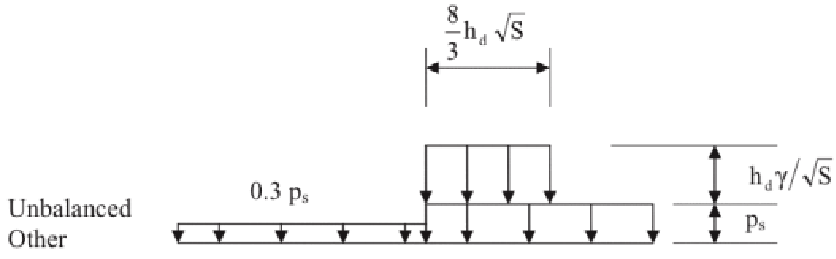
SLOPED ROOF SNOW LOAD:

$C_s := 1.0$ <-- From ASCE 7-10 Figure 7-2 p. 37

$p_s := C_s \cdot p_f = 184\ psf$ <-- ASCE 7-10 Eq. 7.4-1 p. 31

Apply the sloped roof snow load uniformly over the roof in place of the flat roof snow load.

UNBALANCED SNOW LOAD:



$l_u := W = 13.5\ ft$ <-- From ASCE 7-10 § 7.6.1 p. 32

$$S := \frac{1}{\tan(\theta)} = 1.691$$

$h_d := \left(0.43 \cdot \sqrt[3]{l_u \cdot ft^{-1}} \cdot \sqrt[4]{p_g \cdot psf^{-1} + 10} - 1.5\right) \cdot ft = 2.7\ ft$ <-- From ASCE 7-10 Figure 7-9 p. 41

$SL_{windward} := 0.3 \cdot p_s = 55.2\ psf$

$SL_{leeward\_up} := \frac{h_d \cdot \gamma}{\sqrt{S}} = 61.4\ psf$

$dist_{leeward} := \frac{8 \cdot h_d \cdot \sqrt{S}}{3} = 9.2\ ft$

$SL_{leeward\_down} := p_s = 184.0\ psf$

Apply the unbalanced snow load per the "Unbalanced Other" figure of ASCE 7-10 Figure 7-5.

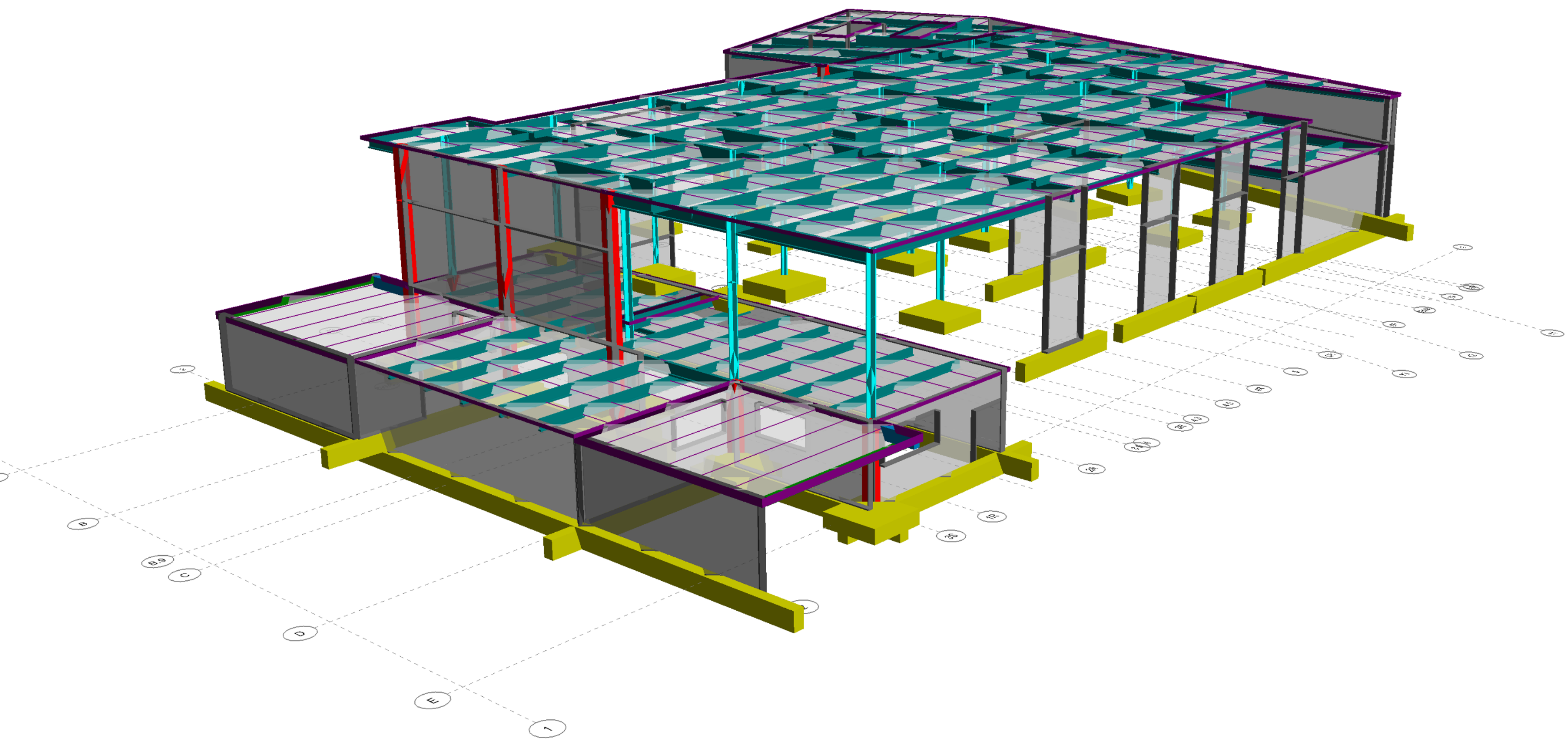
SLIDING SNOW LOAD:

Length of sliding snow load perpendicular to building: $L := 15\ ft$ <-- From ASCE 7-10 § 7.9 p. 33

$SL_{sliding} := \frac{0.4 \cdot p_f \cdot W}{L} = 66\ psf$ <-- From ASCE 7-10 § 7.9 p. 33

Apply the uniform sliding snow load (S2) along the length of the eave at a perpendicular distance of 15' from the eave to the roof below. Superimpose the sliding snow load on the balanced snow load. Do not apply it in conjunction with drift.

03
GRAVITY FRAMING

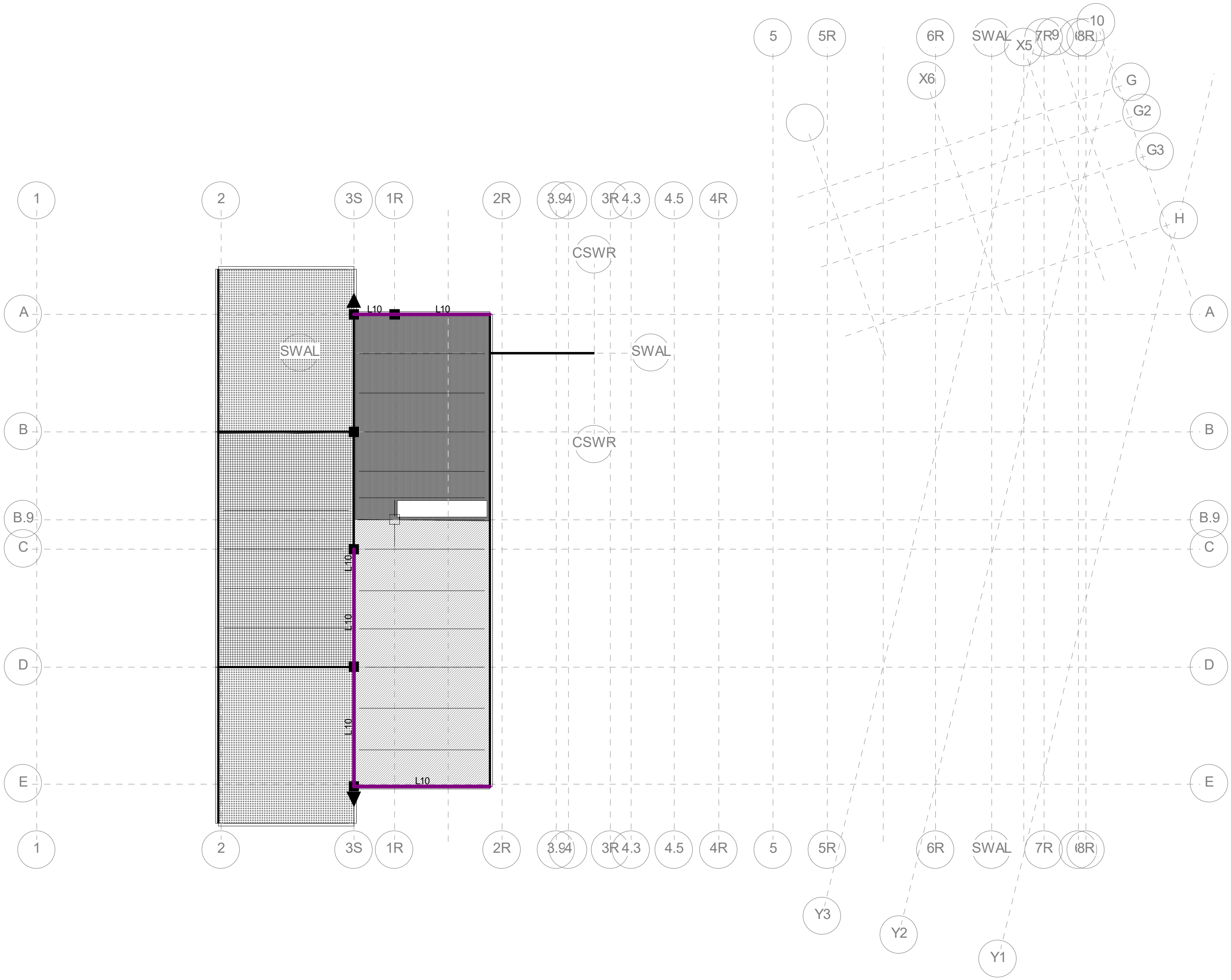

B
B.9
C
D
E
1

Floor Map

RAM Steel 15.04.00.000
Reaveley Engineers + Associates
DataBase: Summitt Powder Mtn Parcel 4 - v55
Building Code: IBC

06/26/17 22:18:16
Steel Code: AISC360-05 LRFD

Floor Type: 2-BASEMENT LEVEL



RAM Steel 15.04.00.000
Reaveley Engineers + Associates
DataBase: Summitt Powder Mtn Parcel 4 - v55
Building Code: IBC

06/26/17 22:18:16
Steel Code: AISC360-05 LRFD

Surface Loads

Label	DL psf	CDL psf	LL psf	Reduction Type	PLL psf	CLL psf	Mass DL psf
04 GYMNASIUM	44.0	5.0	100.0	Unreducible	0.0	20.0	44.0
06 TERRACE OVR MECH	92.0	5.0	100.0	Roof	0.0	20.0	146.0
08 UNHEATED TERRACE	63.0	5.0	100.0	Roof	0.0	20.0	127.5
10 TYP FLOOR OVER M	45.0	5.0	20.0	Reducible	0.0	20.0	45.0

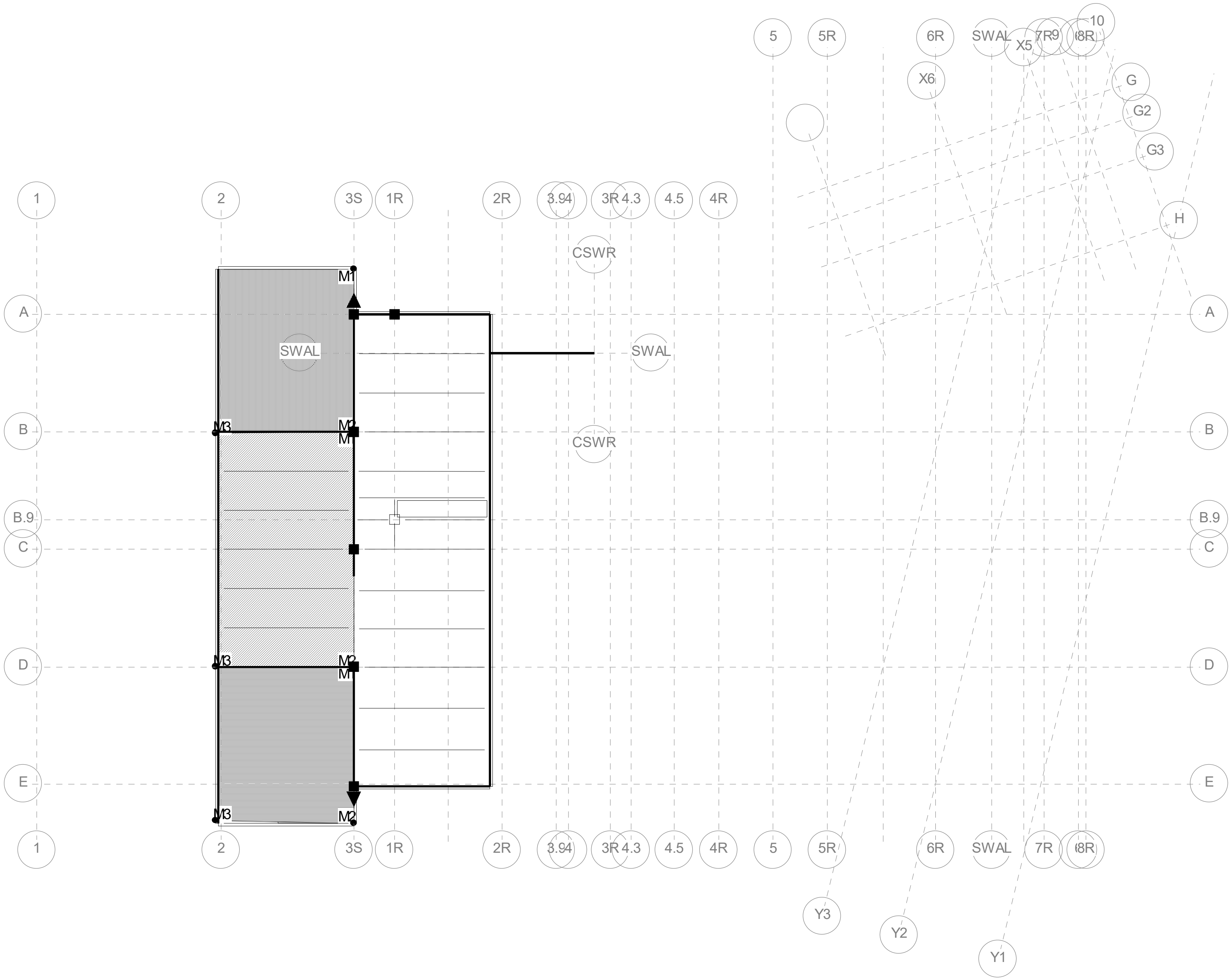
Line Loads

	Label	DL k/ft	CDL k/ft	LL k/ft	Reduction Type	PLL k/ft	CLL k/ft	Mass DL k/ft
L10	15psf15D7.5M	0.225	0.000	0.000	Unreducible	0.000	0.000	0.120

RAM Steel 15.04.00.000
Reaveley Engineers + Associates
DataBase: Summitt Powder Mtn Parcel 4 - v55
Building Code: IBC

06/26/17 22:18:16
Steel Code: AISC360-05 LRFD

Floor Type: 2-BASEMENT LEVEL



RAM Steel 15.04.00.000
Reaveley Engineers + Associates
DataBase: Summitt Powder Mtn Parcel 4 - v55
Building Code: IBC

06/26/17 22:18:16
Steel Code: AISC360-05 LRFD

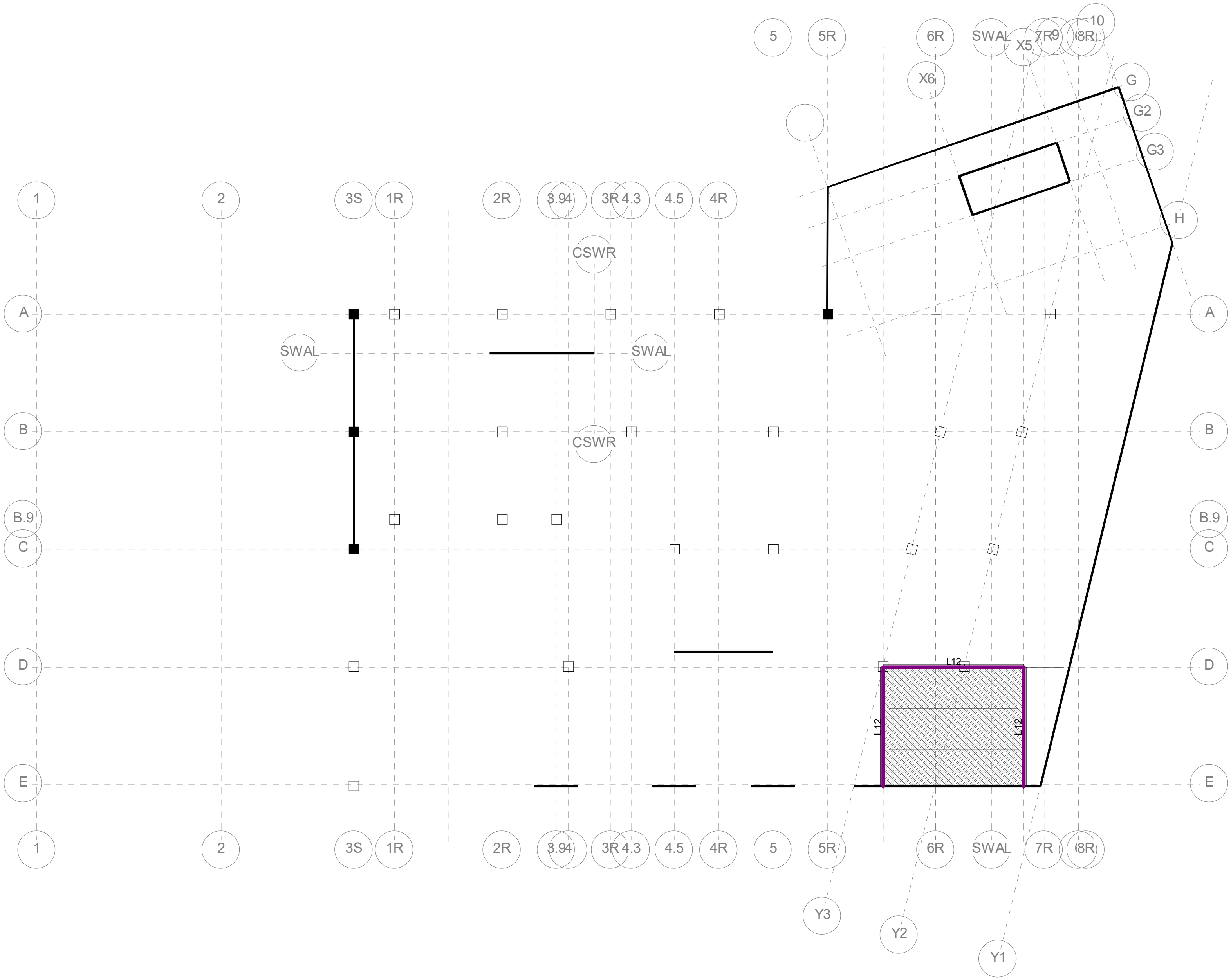
Snow Loads

Label	Type	Magnitude 1 psf	Magnitude 2 psf	Magnitude 3 psf
SL-FLAT-UNHEATED	Constant	221.000	---	---
SL-FLAT-UNHEATED	Constant	221.000	---	---
SL-FLAT	Constant	184.000	---	---
SD-2	Drift	445.000	445.000	243.000
SD-2	Drift	445.000	445.000	243.000
SD-1	Drift	409.000	409.000	206.000

RAM Steel 15.04.00.000
Reaveley Engineers + Associates
DataBase: Summitt Powder Mtn Parcel 4 - v55
Building Code: IBC

06/26/17 22:18:16
Steel Code: AISC360-05 LRFD

Floor Type: 2.5-STAIR



RAM Steel 15.04.00.000
Reaveley Engineers + Associates
DataBase: Summitt Powder Mtn Parcel 4 - v55
Building Code: IBC

06/26/17 22:18:16
Steel Code: AISC360-05 LRFD

Surface Loads

	Label	DL psf	CDL psf	LL psf	Reduction Type	PLL psf	CLL psf	Mass DL psf
	05 TERRACE	72.0	5.0	100.0	Roof	0.0	20.0	126.0

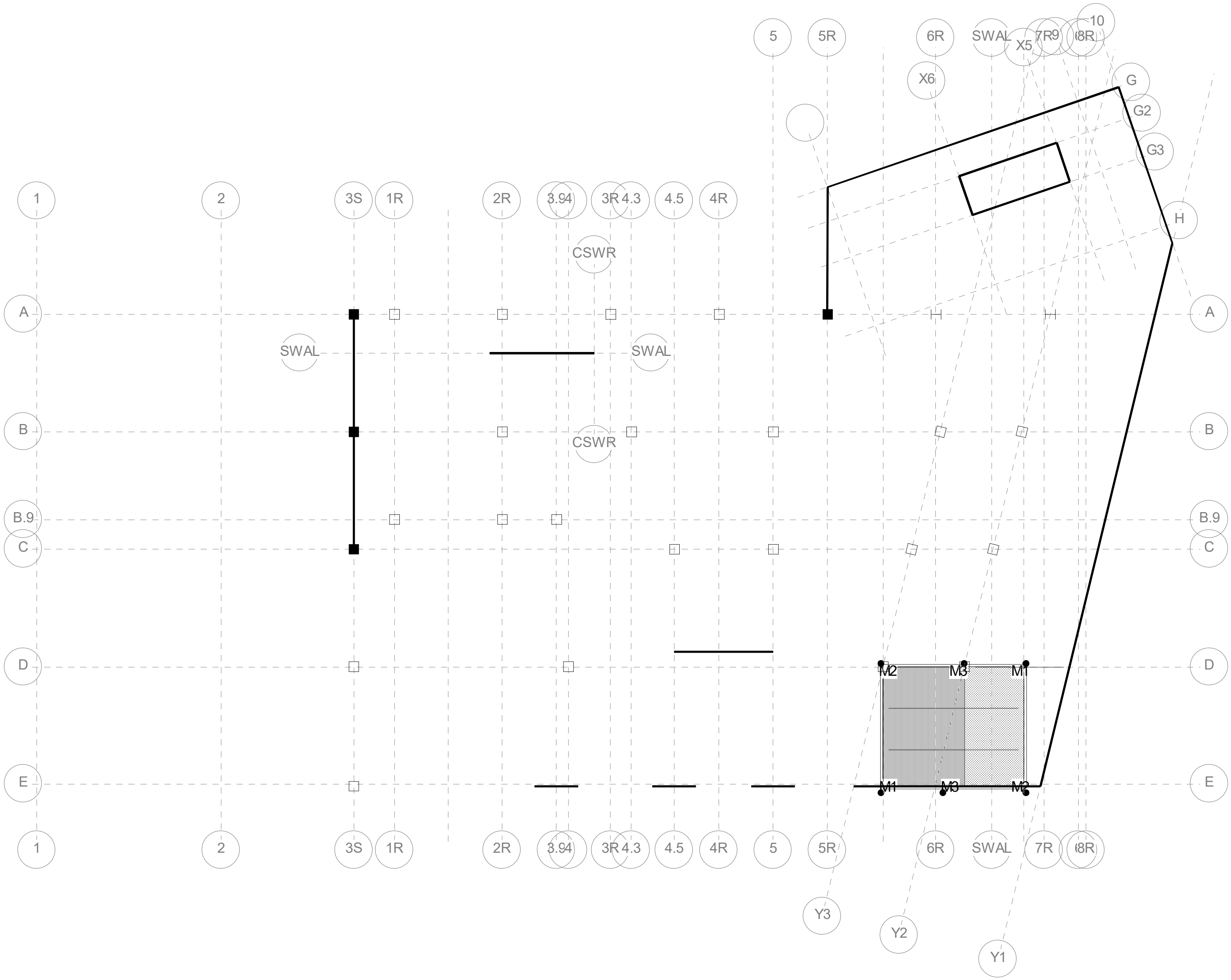
Line Loads

	Label	DL k/ft	CDL k/ft	LL k/ft	Reduction Type	PLL k/ft	CLL k/ft	Mass DL k/ft
L12	terrace_stair_cheek	1.125	0.000	0.000	Reducible	0.000	0.000	1.125

RAM Steel 15.04.00.000
Reaveley Engineers + Associates
DataBase: Summitt Powder Mtn Parcel 4 - v55
Building Code: IBC

06/26/17 22:18:16
Steel Code: AISC360-05 LRFD

Floor Type: 2.5-STAIR



Snow Loads

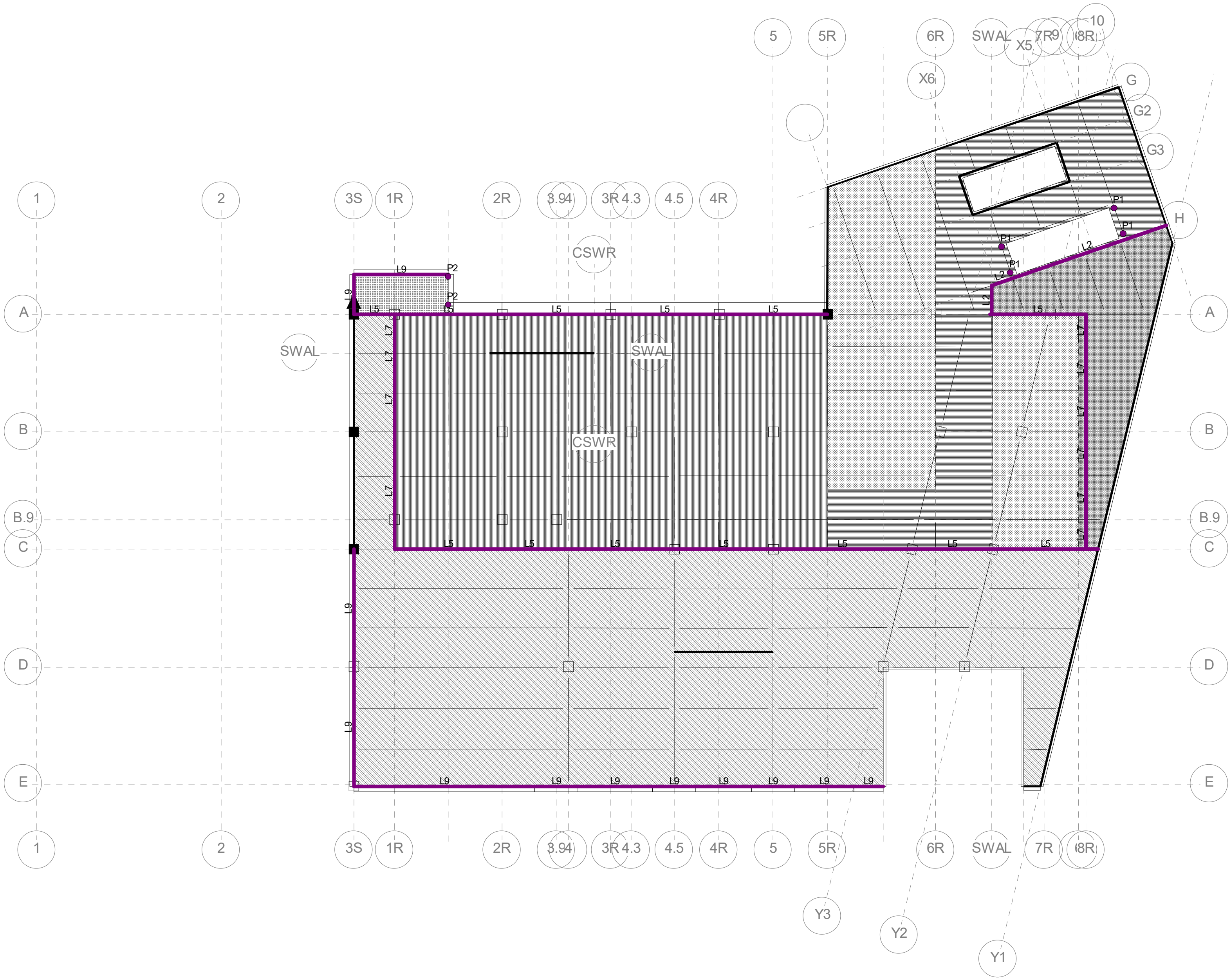
Label	Type	Magnitude 1 psf	Magnitude 2 psf	Magnitude 3 psf
SL-FLAT	Constant	184.000	---	---
SD-9	Drift	240.000	240.000	184.000
SD-9	Drift	240.000	240.000	184.000

Floor Map

RAM Steel 15.04.00.000
Reaveley Engineers + Associates
DataBase: Summitt Powder Mtn Parcel 4 - v55
Building Code: IBC

06/26/17 22:18:16
Steel Code: AISC360-05 LRFD

Floor Type: 3-GROUND LEVEL



Surface Loads

	Label	DL psf	CDL psf	LL psf	Reduction Type	PLL psf	CLL psf	Mass DL psf
	01 TYP FLOOR	20.0	5.0	100.0	Reducible	0.0	20.0	20.0
	02 EVENT SPACE	20.0	5.0	100.0	Unreducible	0.0	20.0	20.0
	03 EXIT & CORRIDOR	20.0	5.0	100.0	Reducible	0.0	20.0	20.0
	05 TERRACE	72.0	5.0	100.0	Roof	0.0	20.0	126.0
	08 UNHEATED TERRACE	63.0	5.0	100.0	Roof	0.0	20.0	127.5
	09 CONC ROOF	72.0	5.0	20.0	Roof	0.0	20.0	125.8

Line Loads

	Label	DL k/ft	CDL k/ft	LL k/ft	Reduction Type	PLL k/ft	CLL k/ft	Mass DL k/ft
L2	20psf10D5M	0.200	0.000	0.000	Unreducible	0.000	0.000	0.100
L5	20psf16D8M	0.320	0.000	0.000	Unreducible	0.000	0.000	0.160
L7	15psf28D14M	0.420	0.000	0.000	Unreducible	0.000	0.000	0.210
L9	20psf9D9M	0.180	0.000	0.000	Unreducible	0.000	0.000	0.180

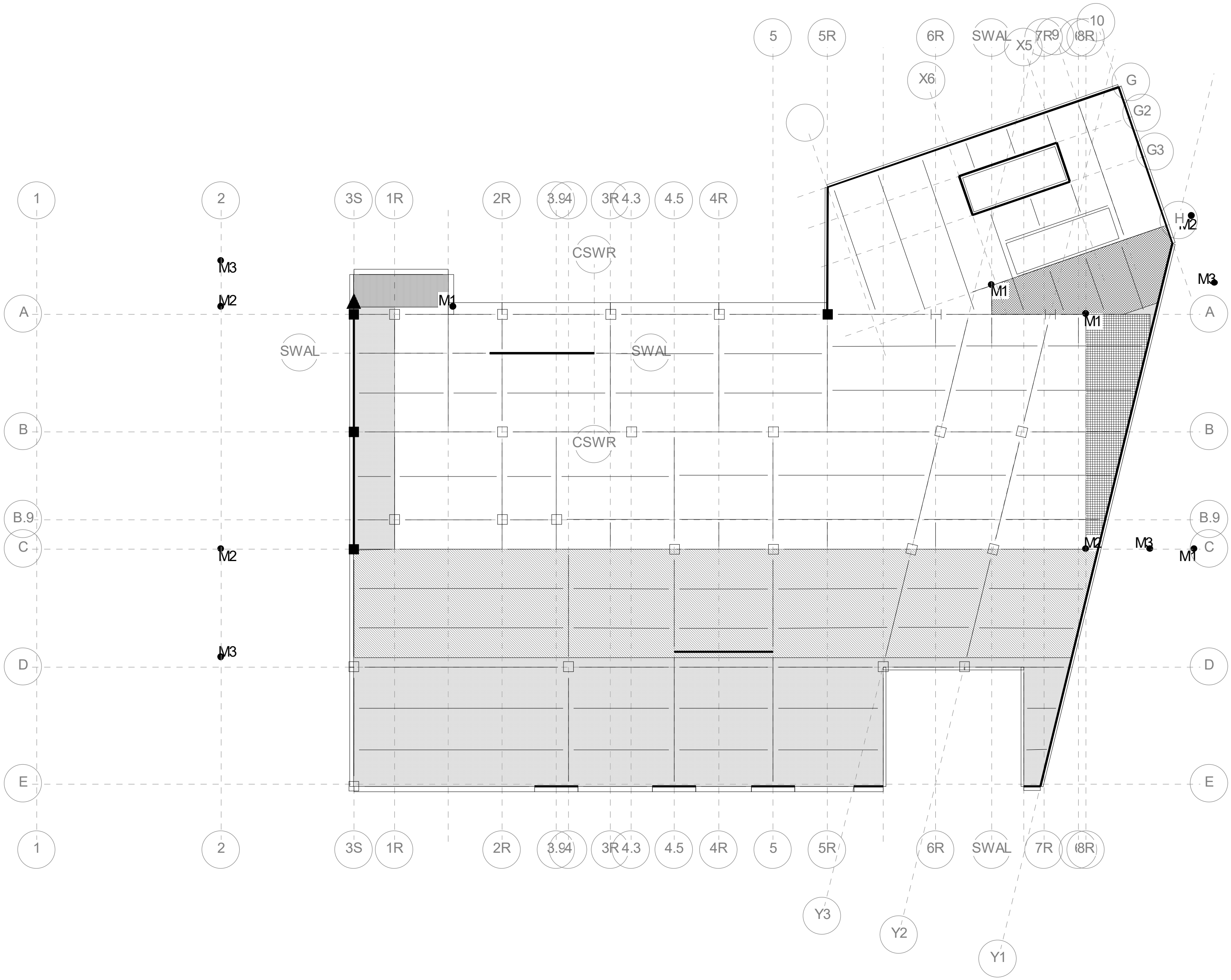
Point Loads

	Label	DL kips	CDL kips	LL kips	Reduction Type	PLL kips	CLL kips	Mass DL kips
P1	STAIR INT	1.300	1.300	2.600	Unreducible	0.000	0.520	2.600
P2	STAIR EXT	10.700	10.700	24.400	Unreducible	0.000	1.400	10.700

RAM Steel 15.04.00.000
Reaveley Engineers + Associates
DataBase: Summitt Powder Mtn Parcel 4 - v55
Building Code: IBC

06/26/17 22:18:16
Steel Code: AISC360-05 LRFD

Floor Type: 3-GROUND LEVEL



RAM Steel 15.04.00.000
Reaveley Engineers + Associates
DataBase: Summitt Powder Mtn Parcel 4 - v55
Building Code: IBC

06/26/17 22:18:16
Steel Code: AISC360-05 LRFD

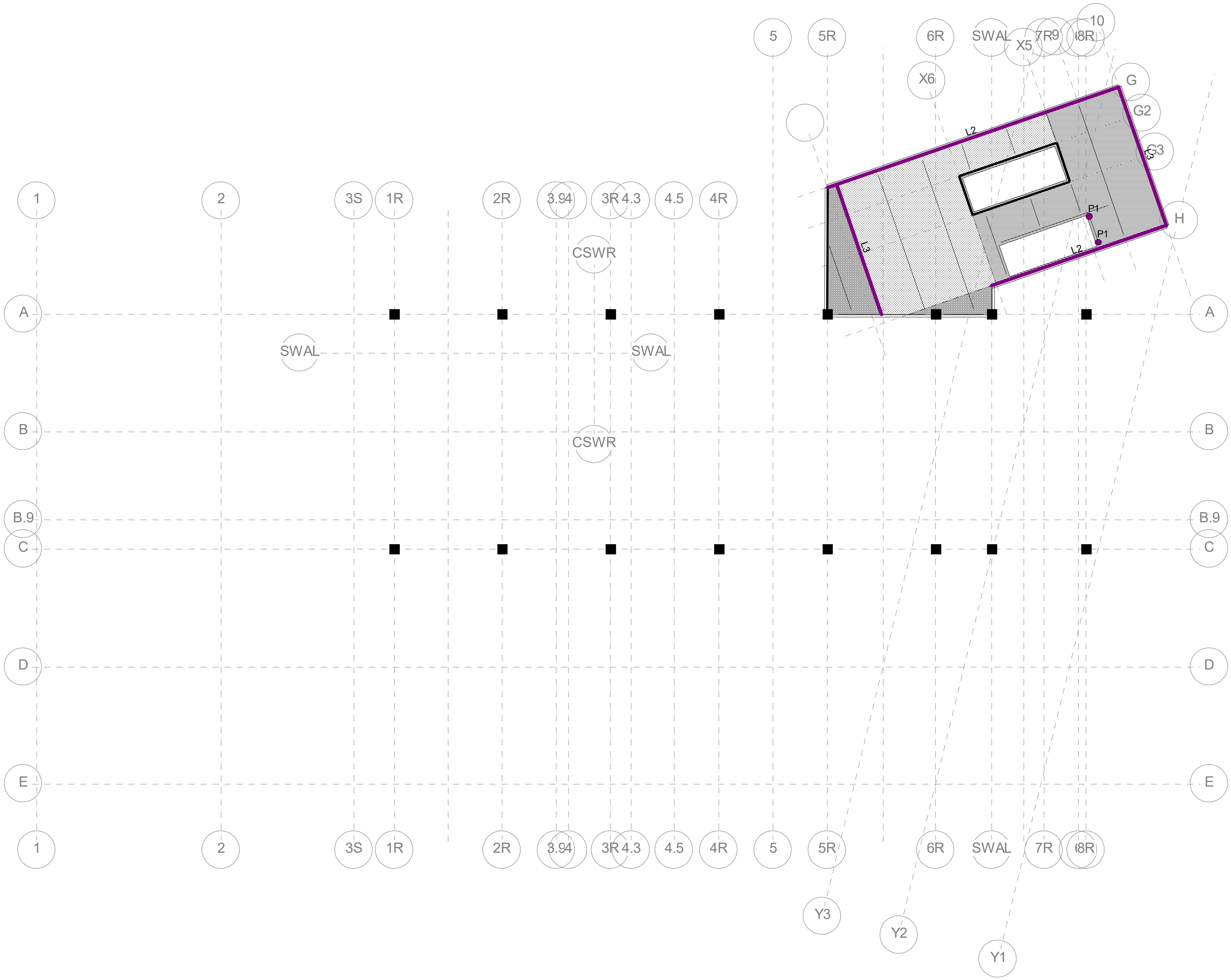
Snow Loads

Label	Type	Magnitude 1 psf	Magnitude 2 psf	Magnitude 3 psf
SL-FLAT	Constant	184.000	---	---
SL-FLAT	Constant	184.000	---	---
SD-3	Drift	366.000	366.000	302.000
SD-8	Drift	329.000	329.000	184.000
SD-7	Drift	409.000	409.000	320.000
SD-6	Drift	296.000	296.000	199.000
SL-FLAT	Constant	184.000	---	---

RAM Steel 15.04.00.000
Reaveley Engineers + Associates
DataBase: Summitt Powder Mtn Parcel 4 - v55
Building Code: IBC

06/26/17 22:18:16
Steel Code: AISC360-05 LRFD

Floor Type: 4-UPPER GROUND LVL



RAM Steel 15.04.00.000
Reaveley Engineers + Associates
DataBase: Summitt Powder Mtn Parcel 4 - v55
Building Code: IBC

06/26/17 22:18:16
Steel Code: AISC360-05 LRFD

Surface Loads

	Label	DL psf	CDL psf	LL psf	Reduction Type	PLL psf	CLL psf	Mass DL psf
	01 TYP FLOOR	20.0	5.0	100.0	Reducible	0.0	20.0	20.0
	03 EXIT & CORRIDOR	20.0	5.0	100.0	Reducible	0.0	20.0	20.0
	09 CONC ROOF	72.0	5.0	20.0	Roof	0.0	20.0	125.8

Line Loads

	Label	DL k/ft	CDL k/ft	LL k/ft	Reduction Type	PLL k/ft	CLL k/ft	Mass DL k/ft
L2	20psf10D5M	0.200	0.000	0.000	Unreducible	0.000	0.000	0.100
L3	20psf18D9M	0.360	0.000	0.000	Unreducible	0.000	0.000	0.180

Point Loads

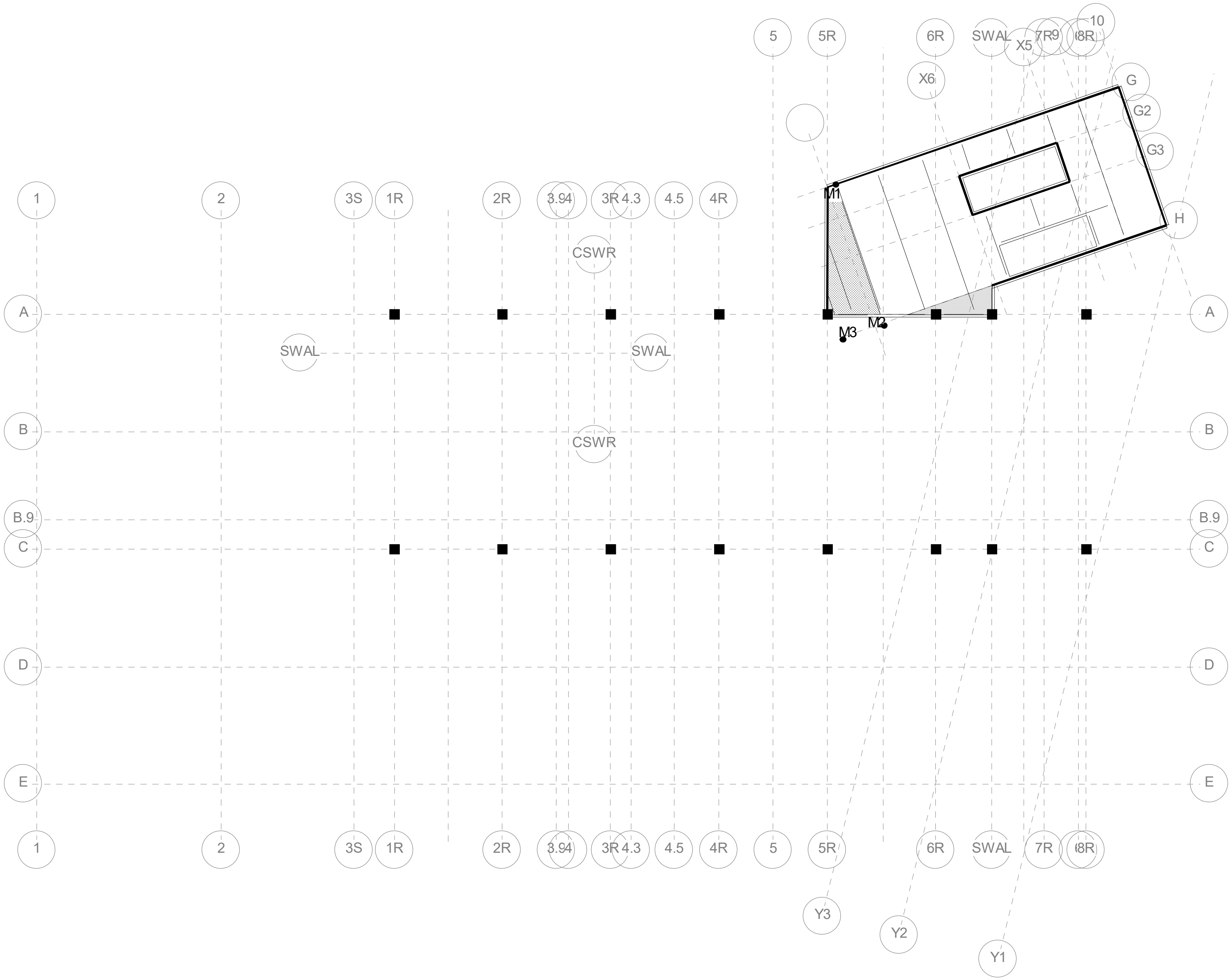
	Label	DL kips	CDL kips	LL kips	Reduction Type	PLL kips	CLL kips	Mass DL kips
P1	STAIR INT	1.300	1.300	2.600	Unreducible	0.000	0.520	2.600

Floor Map

RAM Steel 15.04.00.000
Reaveley Engineers + Associates
DataBase: Summitt Powder Mtn Parcel 4 - v55
Building Code: IBC

06/26/17 22:18:16
Steel Code: AISC360-05 LRFD

Floor Type: 4-UPPER GROUND LVL



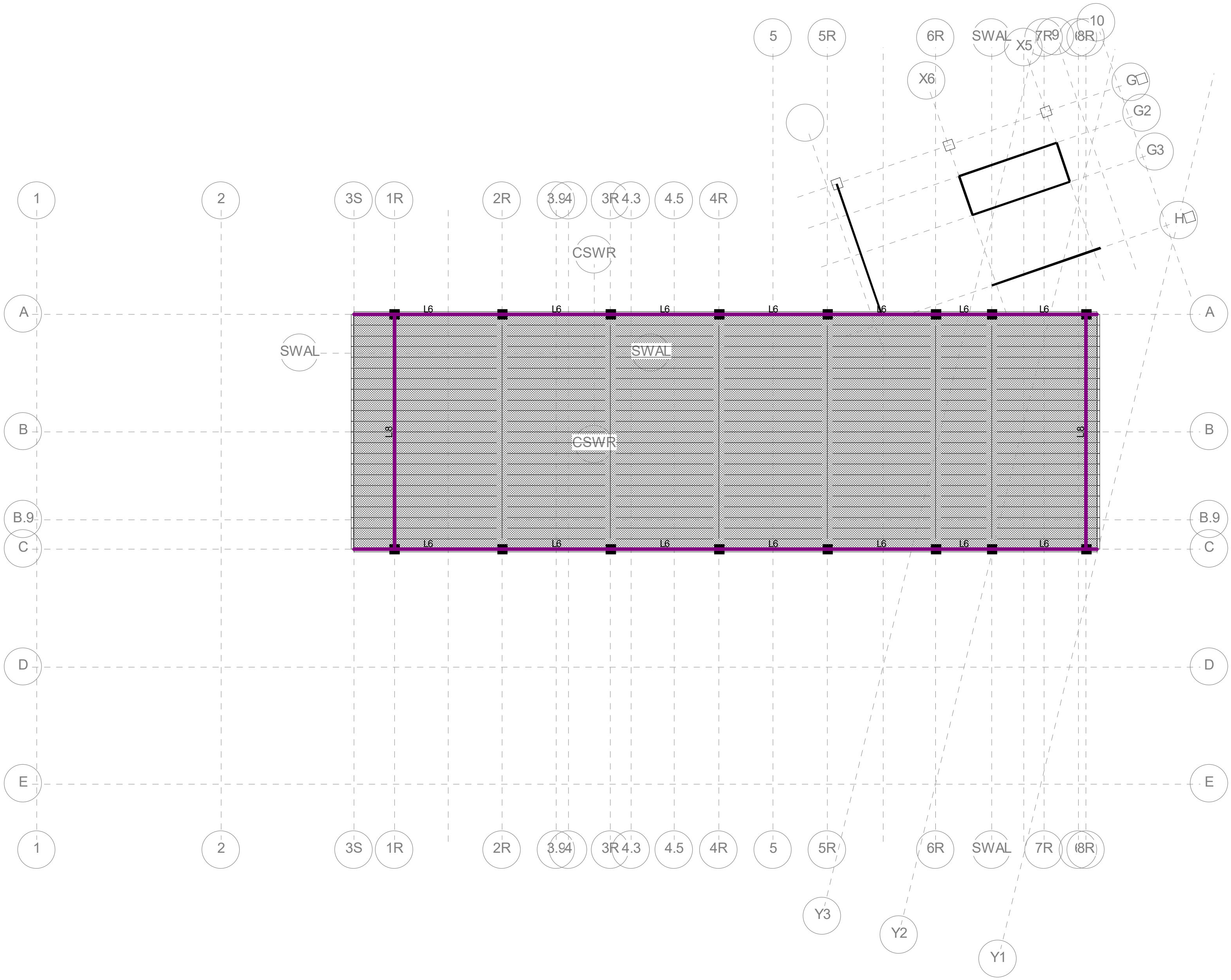
Snow Loads

Label	Type	Magnitude 1 psf	Magnitude 2 psf	Magnitude 3 psf
SL-FLAT	Constant	184.000	---	---
SL-FLAT	Constant	184.000	---	---
SD-5	Drift	344.000	344.000	284.000

RAM Steel 15.04.00.000
Reaveley Engineers + Associates
DataBase: Summitt Powder Mtn Parcel 4 - v55
Building Code: IBC

06/26/17 22:18:16
Steel Code: AISC360-05 LRFD

Floor Type: 5-B.O. ROOF



Surface Loads

	Label	DL psf	CDL psf	LL psf	Reduction Type	PLL psf	CLL psf	Mass DL psf
	07 TYP ROOF	26.0	5.0	20.0	Roof	0.0	20.0	78.9

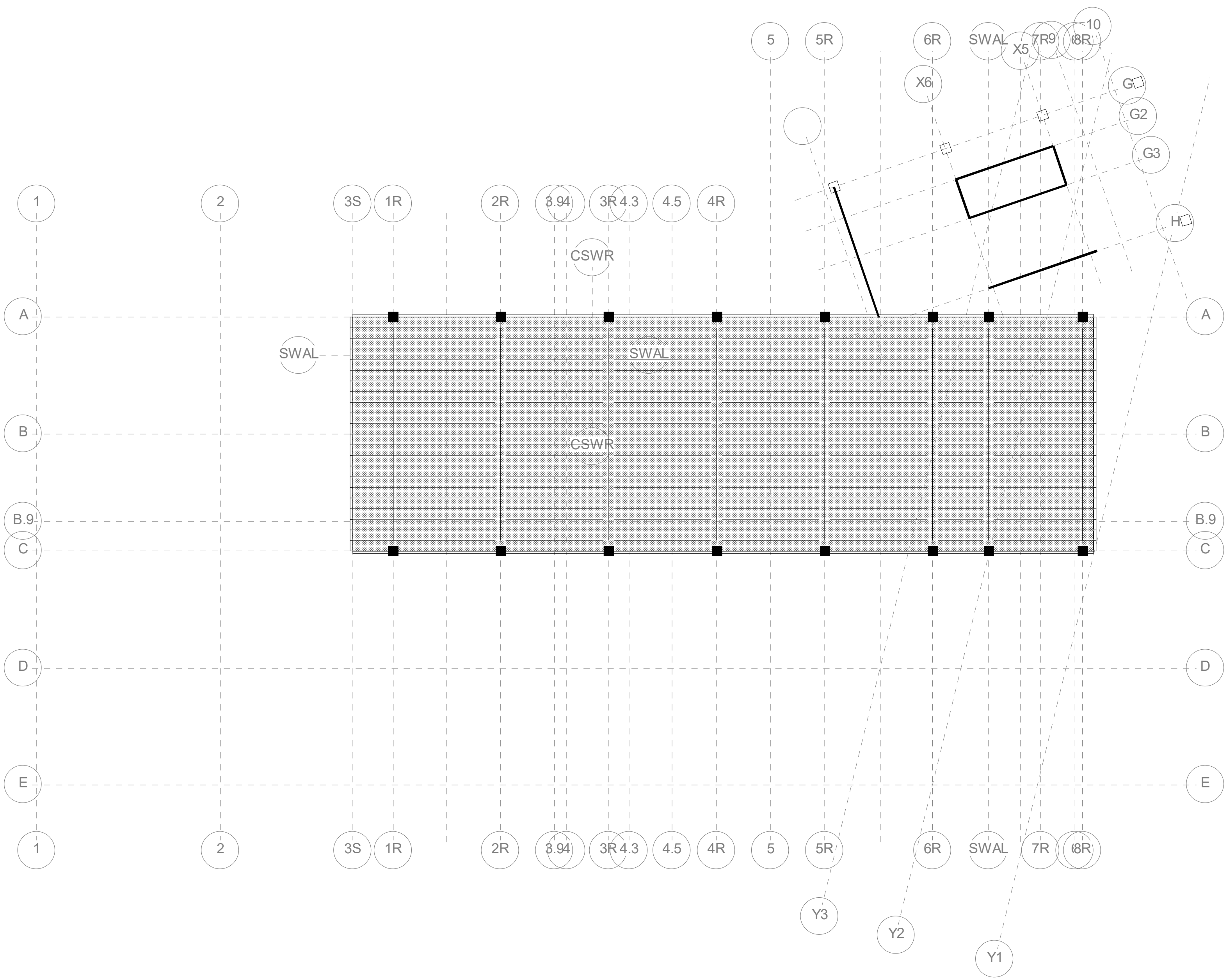
Line Loads

	Label	DL k/ft	CDL k/ft	LL k/ft	Reduction Type	PLL k/ft	CLL k/ft	Mass DL k/ft
L6	20psf0D8M	0.000	0.000	0.000	Unreducible	0.000	0.000	0.160
L8	15psf0D14M	0.000	0.000	0.000	Unreducible	0.000	0.000	0.210

RAM Steel 15.04.00.000
Reaveley Engineers + Associates
DataBase: Summitt Powder Mtn Parcel 4 - v55
Building Code: IBC

06/26/17 22:18:16
Steel Code: AISC360-05 LRFD

Floor Type: 5-B.O. ROOF



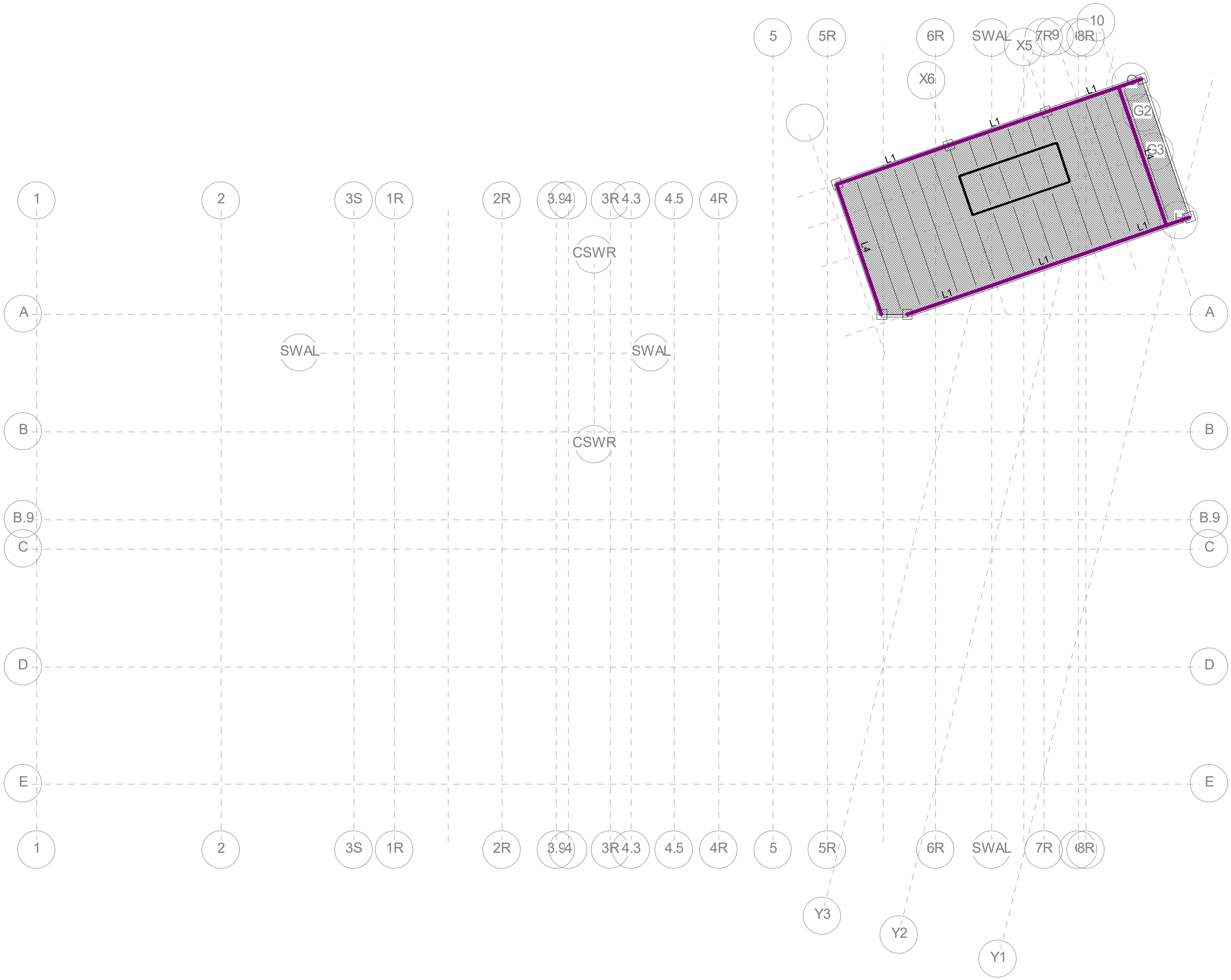
Snow Loads

	Label	Type	Magnitude 1 psf	Magnitude 2 psf	Magnitude 3 psf
	SL-FLAT	Constant	184.000	---	---

RAM Steel 15.04.00.000
Reaveley Engineers + Associates
DataBase: Summitt Powder Mtn Parcel 4 - v55
Building Code: IBC

06/26/17 22:18:16
Steel Code: AISC360-05 LRFD

Floor Type: 6-LITTLE BARN ROOF



Floor Map

Surface Loads

	Label	DL psf	CDL psf	LL psf	Reduction Type	PLL psf	CLL psf	Mass DL psf
	07 TYP ROOF	26.0	5.0	20.0	Roof	0.0	20.0	78.9

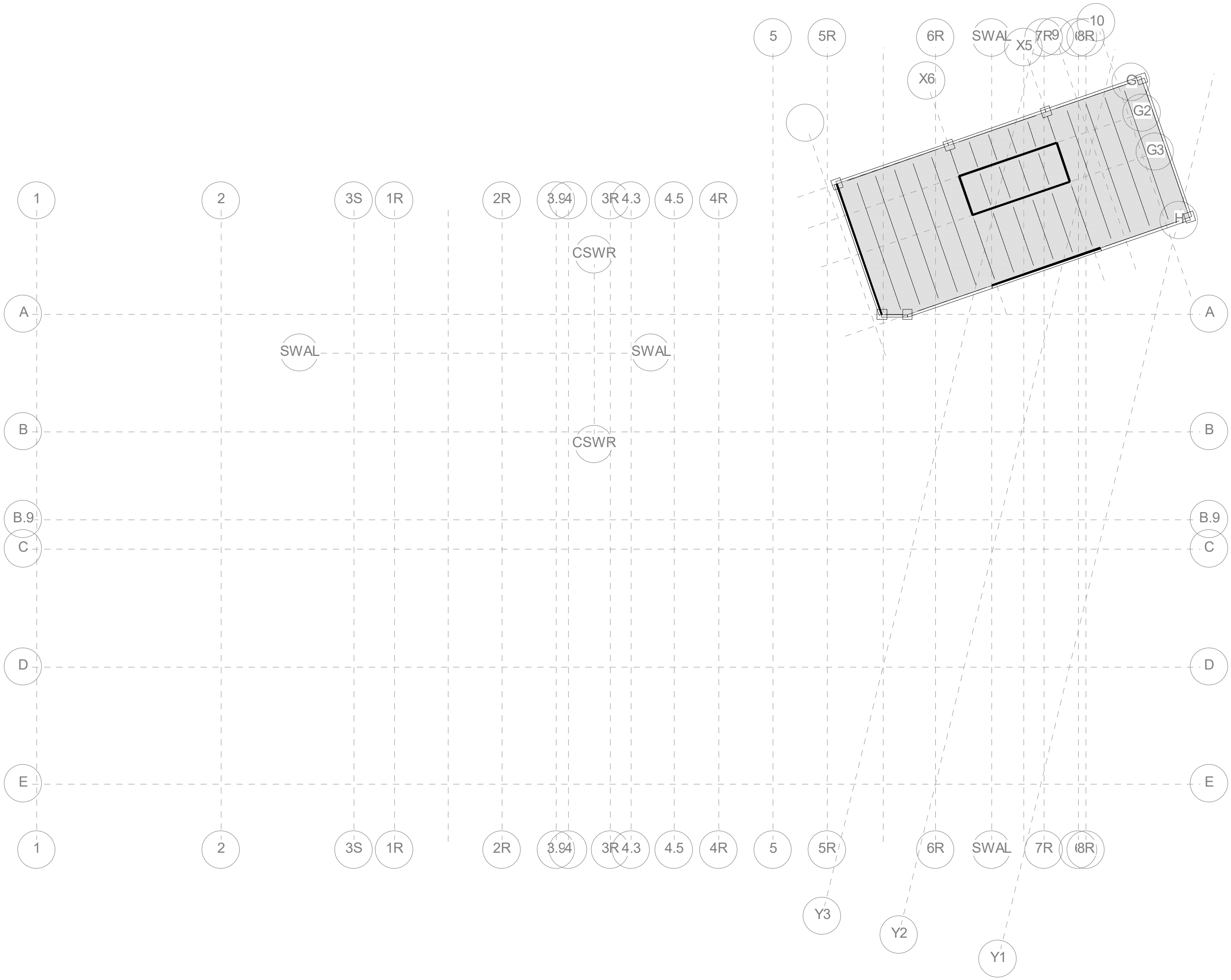
Line Loads

	Label	DL k/ft	CDL k/ft	LL k/ft	Reduction Type	PLL k/ft	CLL k/ft	Mass DL k/ft
L1	20psf0D5M	0.000	0.000	0.000	Unreducible	0.000	0.000	0.100
L4	20psf0D9M	0.000	0.000	0.000	Unreducible	0.000	0.000	0.180

RAM Steel 15.04.00.000
Reaveley Engineers + Associates
DataBase: Summitt Powder Mtn Parcel 4 - v55
Building Code: IBC

06/26/17 22:18:16
Steel Code: AISC360-05 LRFD

Floor Type: 6-LITTLE BARN ROOF



Snow Loads

	Label	Type	Magnitude 1 psf	Magnitude 2 psf	Magnitude 3 psf
	SL-FLAT	Constant	184.000	---	---

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Consulting Structural Engineers

GRAVITY BASE PLATES

2017.080 Powder Mountain Parcel 4

Base Plate Design Summary
RAM Steel
15.04.00.000
Reaveley Engineers + Associates
DataBase: Summitt Powder Mtn Parcel 4 - v55
Building Code: IBC
Steel Code: AISC 360-05 LRFD
06/26/17 21:52:59
BASE PLATES:
Design Code: AISC 360-05 LRFD

Plate Fy (ksi) .	36
Minimum Dimension From Face of Column to Edge of Plate (in) .	3
Minimum Dimension From Side of Column to Edge of Plate (in) .	0.5
Increment of Plate Dimensions (in) .	1
Increment of Plate Thickness (in) .	0.125
Minimum Footing Dimension Parallel to Web (ft) .	1.5
Minimum Footing Dimension Perpendicular to Web (ft) .	1.5
Keep Base Plate Square:.	N

Column Line	Column Size	Fy (ksi)	N (in)	B (in)	tp (in)	GL	Note	Support	BASE PLATE SCHEDULE Mark	N	B	tp	C	Type	Remarks	Pu kip	CONCRETE PIER SCHEDULE Mark	Dimensions Depth	Width	Reinf. Vertical	Ties
-17.50ft--0.58ft	HSS10X10X5/16	36	16	11	0.75	3S-E		exterior pier	SBP-6	16	13	1.25	--	T1		118.2	CP-2	24	24	8-#8	#4@8" o.c.
3S-D	HSS10X10X3/8	36	16	11	1	3S-D		exterior pier	SBP-6	16	13	1.25	--	T1		234.9	CP-2	24	24	8-#8	#4@8" o.c.
1R-B.9	HSS10X10X5/16	36	16	12	1.125	1R-B.9		int ftg	SBP-6	16	13	1.25	--	T1		309.6	N/A				
1R-A	HSS10X10X3/8	36	16	11	1	1R-A		exterior pier	SBP-6	16	13	1.25	--	T1		252.7	CP-2	24	24	8-#8	#4@8" o.c.
2R-B.9	HSS8X8X1/2	36	14	11	1.125	2R-B.9		int pier	SBP-4	14	11	1.25	--	T1		289.64	CP-1	18	18	8-#6	#4@8" o.c.
2R-B	HSS7X7X5/16	36	13	8	0.875	2R-B		int pier	SBP-2	13	8	1	--	T1		-89.53	CP-1	18	18	8-#6	#4@8" o.c.
2R-A	HSS10X10X3/8	36	16	11	1	2R-A		exterior pier	SBP-6	16	13	1.25	--	T1		241.4	CP-2	24	24	8-#8	#4@8" o.c.
3.9-B.9	HSS8X8X5/8	36	14	14	1.125	3.9-B.9		int pier	SBP-3	14	14	1.25	--	T1		361.65	CP-1	18	18	8-#6	#4@8" o.c.
4-D	HSS10X10X1/2	36	16	12	1.125	4-D		int pier	SBP-6	16	13	1.25	--	T1		329.3	CP-2	24	24	8-#8	#4@8" o.c.
3R-A	HSS10X10X5/16	36	16	11	0.875	3R-A		exterior pier	SBP-6	16	13	1.25	--	T1		191.6	CP-2	24	24	8-#8	#4@8" o.c.
4.3-B	HSS10X10X5/16	36	16	11	0.75	4.3-B		int pier	SBP-6	16	13	1.25	--	T1		121.3	CP-2	24	24	8-#8	#4@8" o.c.
4.5-C	HSS10X10X1/2	36	16	12	1.125	4.5-C		int ftg	SBP-6	16	13	1.25	--	T1		323.2	N/A				
4R-A	HSS10X10X5/16	36	16	11	0.875	4R-A		exterior pier	SBP-6	16	13	1.25	--	T1		198.1	CP-2	24	24	8-#8	#4@8" o.c.
5-C	HSS10X10X1/2	36	16	13	1.125	5-C		int ftg	SBP-6	16	13	1.25	--	T1		335.2	N/A				
5-B	HSS8X8X5/16	36	14	9	1	5-B		int ftg	SBP-5	14	9	1	--	T1		164.8	N/A				
71.71ft-110.58ft	HSS5X5X1/4	36	11	6	0.75	5R-G	little barn NW	conc wall	SBP-1	11	6	0.75	--	T1		43.2	N/A				
80.00ft-86.50ft	HSS4X4X1/4	36	10	5	0.25		little barn west post	steel beam								3.7	N/A				
80.25ft-D	HSS8X8X3/8	36	14	10	1.125	Y3-D		int ftg	SBP-4	14	11	1.25	--	T1		257.24	N/A				
85.53ft-43.17ft	HSS10X10X1/2	36	16	12	1.125	Y3-C		int ftg	SBP-6	16	13	1.25	--	T1		314.4	N/A				
6R-A	W10X100	36	18	15	1.125	6R-A	WF	int ftg	SBP-7	18	15	1.125	--	W1		397.3	N/A				
90.80ft-64.83ft	HSS8X8X1/4	36	14	9	0.75	Y3-B		int ftg	SBP-5	14	9	1	--	T1		110.6	N/A				
X6-G	HSS5X5X5/16	36	11	6	0.75	X6-G		conc wall	SBP-1	11	6	0.75	--	T1		56.4	N/A				
95.26ft-21.50ft	HSS8X8X3/8	36	14	9	1	Y2-D		int ftg	SBP-5	14	9	1	--	T1		195.8	N/A				
100.54ft-43.17ft	HSS10X10X5/8	36	16	11	1.125	Y2-C		int ftg	SBP-6	16	13	1.25	--	T1		300.4	N/A				
105.81ft-64.83ft	HSS7X7X1/4	36	13	8	0.75	Y2-B		int ftg	SBP-2	13	8	1	--	T1		90.84	N/A				
X5-G	HSS5X5X1/4	36	11	6	0.75	X5-G		conc wall	SBP-1	11	6	0.75	--	T1		48.7	N/A				
111.09ft-86.50ft	W10X100	36	18	14	1.125	Y2-A	WF	int ftg	SBP-7	18	15	1.125	--	W1		365.4	N/A				
127.96ft-129.95ft	HSS5X5X5/16	36	11	6	0.75	G-10	little barn NE	conc wall	SBP-1	11	6	0.75	--	T1		46.7	N/A				
136.76ft-104.42ft	HSS5X5X5/16	36	11	6	0.75	H-10	little barn SE	conc wall	SBP-1	11	6	0.75	--	T1		44.0	N/A				

04
LATERAL LOADS, DRIFT, AND IRREGULARITIES

ASCE 7 Windspeed | ASCE 7 Ground Snow Load | Related Resources | Sponsors | About ATC | Contact

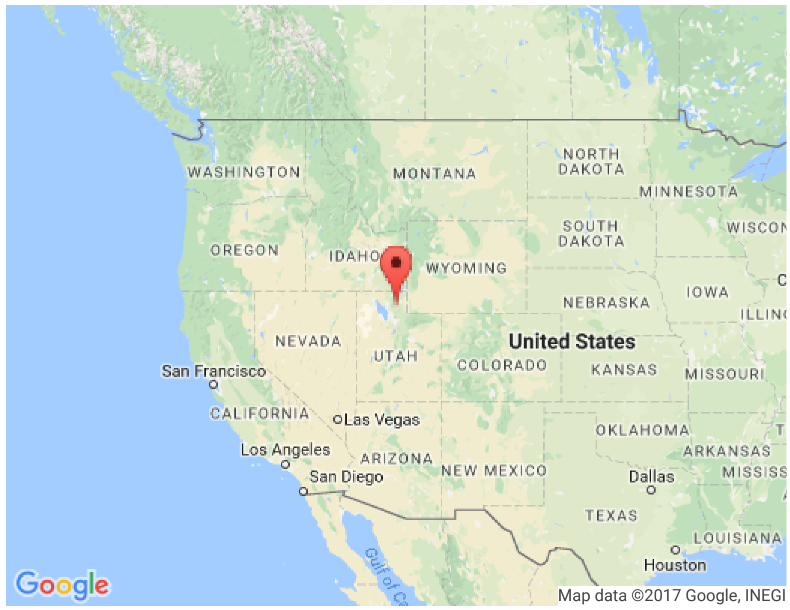
Search Results

Query Date: Thu Jun 15 2017
Latitude: 41.3628
Longitude: -111.7442

ASCE 7-10 Windspeeds
(3-sec peak gust in mph*):

Risk Category I: 105
Risk Category II: 115
Risk Category III-IV: 120
MRI 10-Year:** 76
MRI 25-Year:** 84
MRI 50-Year:** 90
MRI 100-Year:** 96

ASCE 7-05 Windspeed:
90 (3-sec peak gust in mph)
ASCE 7-93 Windspeed:
70 (fastest mile in mph)



*Miles per hour
**Mean Recurrence Interval

Users should consult with local building officials
to determine if there are community-specific wind speed
requirements that govern.

Print your results

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REAVELEY ENGINEERS + ASSOCIATES
Consulting Structural Engineers

project 2017.079 POWDER MOUNTAIN PARCEL 4

location COMPONENT AND CLADDING WIND LOADS

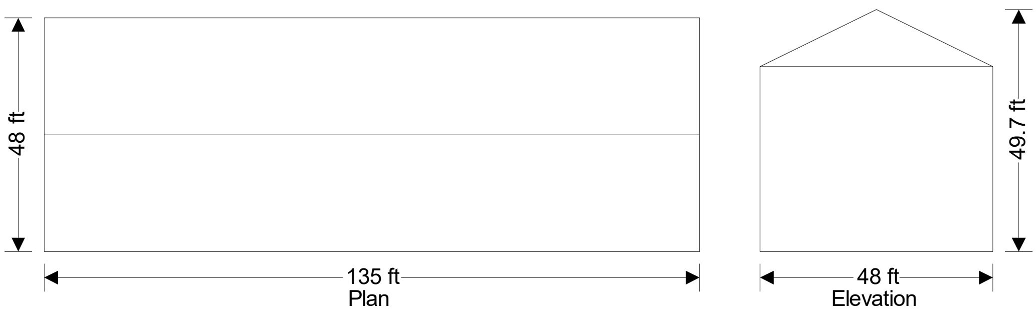
date 6/15/2017 by CAB

WIND LOADING (ASCE7-10)

In accordance with ASCE7-10 incorporating Errata No. 1 and Errata No. 2

Using the components and cladding design method

Tedds calculation version 2.0.17



Building data

Type of roof	Gable
Length of building	b = **135.00** ft
Width of building	d = **48.00** ft
Height to eaves	H = **38.00** ft
Pitch of roof	α_0 = **26.0** deg
Mean height	h = **43.85** ft

General wind load requirements

Basic wind speed	V = **115.0** mph
Risk category	II
Velocity pressure exponent coeff (Table 26.6-1)	K_d = **0.85**
Exposure category (cl.26.7.3)	C
Enclosure classification (cl.26.10)	Enclosed buildings
Internal pressure coef +ve (Table 26.11-1)	GC_{pi_p} = **0.18**
Internal pressure coef –ve (Table 26.11-1)	GC_{pi_n} = **-0.18**
Gust effect factor	G_f = **0.85**

Topography

Type of feature	Escarpment
Dist upwind of crest at half height of hill/esc.	L_h = **2000** ft
Height of topographic feature	H_{topo} = **850** ft
Distance from the crest to the building site	x_{topo} = **0** ft
Height above ground surface at building site	z_{topo} = **850** ft
Shape and max speed-up factor	$K_1 = 0.85 \times (\min(H_{topo} / L_h, 0.5))$ = **0.36**
Horizontal attenuation factor	μ = **1.50**
Height attenuation factor	γ = **2.50**
Speed-up reduction factor with distance of crest	$K_2 = \max(1 - \mathrm{abs}(x_{topo}) / (\mu \times L_h), 0)$ = **1.00**
Vertical speed up reduction factor (Fig 26.8-1)	$K_3 = \exp(-\gamma \times h / L_h)$ = **0.95**
Topographic factor	$K_{zt} = (1 + K_1 \times K_2 \times K_3)^2$ = **1.80**

REAVELEY
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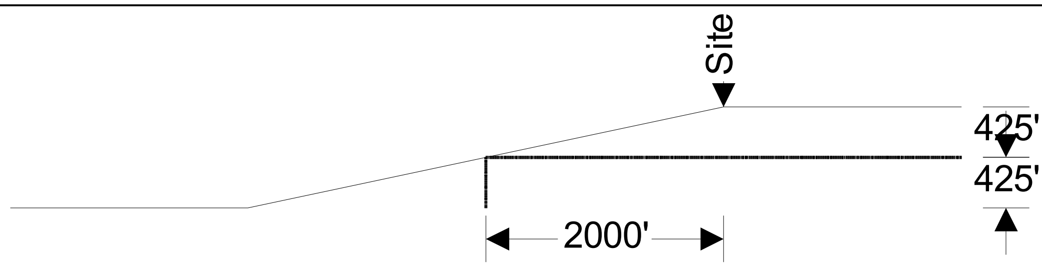
project 2017.079 POWDER MOUNTAIN PARCEL 4

location COMPONENT AND CLADDING WIND LOADS

date 6/15/2017 by CAB



Sketch showing topography

Velocity pressure

Velocity pressure coefficient (T.30.3-1) K_z = **1.06**

Velocity pressure $q_h = 0.00256 \times K_z \times K_{zt} \times K_d \times V^2 \times 1\text{psf/mph}^2$ = **54.9** psf

Peak velocity pressure for internal pressure

Peak velocity pressure – internal (as roof press.) q_i = **54.90** psf

Equations used in tables

Net pressure $p = q_h \times [GC_p - GC_{pi}]$

Components and cladding pressures - Wall (Figure 30.4-1)

Component	Zone	Length (ft)	Width (ft)	Eff. area (ft^2)	$+GC_p$	$-GC_p$	Pres (+ve) (psf)	Pres (-ve) (psf)
<10sf	4;	-;	-;	10.0;	1.00;	-1.10;	64.8;	-70.3
50sf	4;	-;	-;	50.0;	0.88;	-0.98;	58.0;	-63.5
100sf	4;	-;	-;	100.0;	0.82;	-0.92;	55.1;	-60.6
>500sf	4;	-;	-;	500.0;	0.70;	-0.80;	48.3;	-53.8
<10sf	5;	-;	-;	10.0;	1.00;	-1.40;	64.8;	-86.7
50sf	5;	-;	-;	50.0;	0.88;	-1.15;	58.0;	-73.2
100sf	5;	-;	-;	100.0;	0.82;	-1.05;	55.1;	-67.4
>500sf	5;	-;	-;	500.0;	0.70;	-0.80;	48.3;	-53.8

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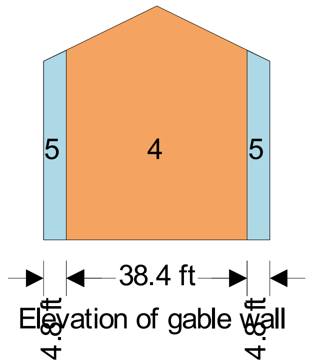
Consulting Structural Engineers

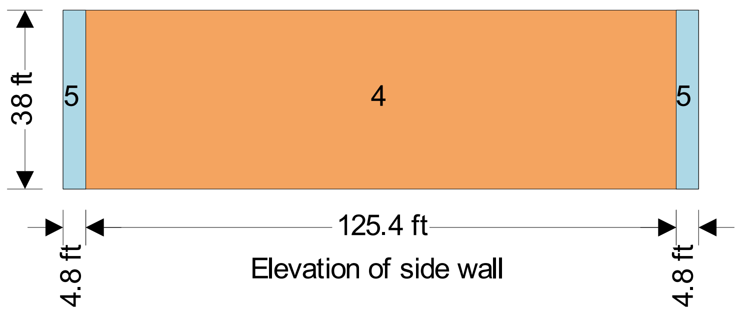
project 2017.079 POWDER MOUNTAIN PARCEL 4

location COMPONENT AND CLADDING WIND LOADS

date 6/15/2017 by CAB



Elevation of gable wall



Elevation of side wall

Components and cladding pressures - Roof (Figure 30.4-2B)

Component	Zone	Length (ft)	Width (ft)	Eff. area (ft^2)	$+GC_p$	$-GC_p$	Pres (+ve) (psf)	Pres (-ve) (psf)
<10sf	1;	-;	-;	10.0;	0.50;	-0.90;	37.3;	-59.3
50sf	1;	-;	-;	50.0;	0.36;	-0.83;	29.7;	-55.5
100sf	1;	-;	-;	100.0;	0.30;	-0.80;	26.4;	-53.8
>500sf	1;	-;	-;	500.0;	0.30;	-0.80;	26.4;	-53.8
<10sf	2;	-;	-;	10.0;	0.50;	-1.70;	37.3;	-103.2
50sf	2;	-;	-;	50.0;	0.36;	-1.35;	29.7;	-84.0
100sf	2;	-;	-;	100.0;	0.30;	-1.20;	26.4;	-75.8
>500sf	2;	-;	-;	500.0;	0.30;	-1.20;	26.4;	-75.8
<10sf	3;	-;	-;	10.0;	0.50;	-2.60;	37.3;	-152.6
50sf	3;	-;	-;	50.0;	0.36;	-2.18;	29.7;	-129.6
100sf	3;	-;	-;	100.0;	0.30;	-2.00;	26.4;	-119.7
>500sf	3;	-;	-;	500.0;	0.30;	-2.00;	26.4;	-119.7

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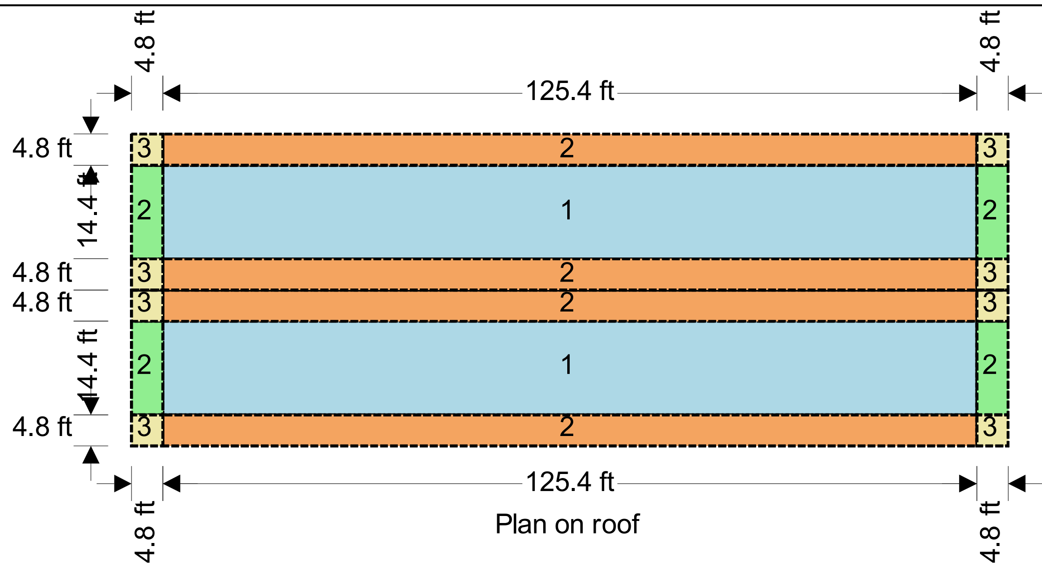
project 2017.079 POWDER MOUNTAIN PARCEL 4

location COMPONENT AND CLADDING WIND LOADS

date 6/15/2017 by CAB



Plan on roof

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project 2017.079 POWDER MOUNTAIN PARCEL 4

location DIRECTIONAL WIND LOADS

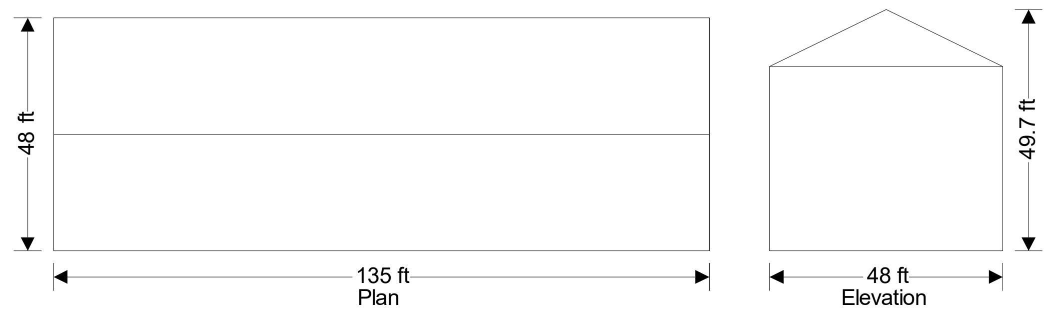
date 6/16/2017 by KN

WIND LOADING (ASCE7-10)

In accordance with ASCE7-10 incorporating Errata No. 1 and Errata No. 2

Using the directional design method

Tedds calculation version 2.0.17



Building data

Type of roof	Gable
Length of building	b = **135.00** ft
Width of building	d = **48.00** ft
Height to eaves	H = **38.00** ft
Pitch of roof	α_0 = **26.0** deg
Mean height	h = **43.85** ft

General wind load requirements

Basic wind speed	V = **115.0** mph
Risk category	II
Velocity pressure exponent coeff (Table 26.6-1)	K_d = **0.85**
Exposure category (cl.26.7.3)	C
Enclosure classification (cl.26.10)	Enclosed buildings
Internal pressure coef +ve (Table 26.11-1)	GC_{pi_p} = **0.18**
Internal pressure coef –ve (Table 26.11-1)	GC_{pi_n} = **-0.18**
Gust effect factor	G_f = **0.85**

Topography

Type of feature	Escarpment
Dist upwind of crest at half height of hill/esc.	L_h = **2000** ft
Height of topographic feature	H_{topo} = **850** ft
Distance from the crest to the building site	x_{topo} = **0** ft
Height above ground surface at building site	z_{topo} = **850** ft
Shape and max speed-up factor	$K_1 = 0.85 \times (\min(H_{topo} / L_h, 0.5))$ = **0.36**
Horizontal attenuation factor	μ = **1.50**
Height attenuation factor	γ = **2.50**
Speed-up reduction factor with distance of crest	$K_2 = \max(1 - \mathrm{abs}(x_{topo}) / (\mu \times L_h), 0)$ = **1.00**

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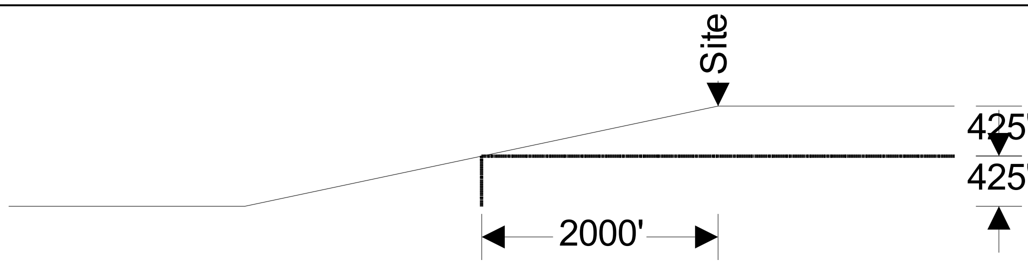
project 2017.079 POWDER MOUNTAIN PARCEL 4

location DIRECTIONAL WIND LOADS

date 6/16/2017 by KN



Sketch showing topography

Speed up reduction factor equation $K_3 = e^{-\gamma \ast z/Lh}$

Topographic factor equation $K_{zt} = (1 + K_1 \times K_2 \times K_3)^2$

Velocity pressure equation $q = 0.00256 \times K_z \times K_{zt} \times K_d \times V^2 \times 1psf/mph^2$

Velocity pressures table

z (ft)	z* (ft)	K_z (T.27.3-1)	z* / L_h	K_3	K_{zt}	q_z (psf)
15.00	7.50;	0.85;	0.00;	0.99;	1.84;	45.10;
25.00	20.00;	0.94;	0.01;	0.98;	1.83;	49.47;
35.00	30.00;	1.01;	0.02;	0.96;	1.82;	52.81;
38.00	36.50;	1.03;	0.02;	0.96;	1.81;	53.53;
43.85	40.93;	1.06;	0.02;	0.95;	1.80;	55.00;
49.71	46.78;	1.09;	0.02;	0.94;	1.80;	56.31;

Peak velocity pressure for internal pressure

Peak velocity pressure – internal (as roof press.) q_i = **55.00** psf

Pressures and forces

Net pressure $p = q \times G_f \times C_{pe} - q_i \times GC_{pi}$

Net force $F_w = p \times A_{ref}$

Roof load case 1 - Wind 0, GC_{pi} 0.18, $-c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p, (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A (-ve)	43.85	-0.43	55.00	-29.95	3604.83	-107.97
B (-ve)	43.85	-0.60	55.00	-37.95	3604.83	-136.80

Total vertical net force $F_{w,v}$ = **-220.00** kips

Total horizontal net force $F_{w,h}$ = **12.64** kips

Walls load case 1 - Wind 0, GC_{pi} 0.18, $-c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p, (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A_1	15.00	0.80	45.10	20.77	2025.00	42.06
A_2	25.00	0.80	49.47	23.74	1350.00	32.05
A_3	38.00	0.80	53.53	26.50	1755.00	46.51

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p, (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
B	43.85	-0.50	55.00	-33.27	5130.00	-170.70
C	43.85	-0.70	55.00	-42.62	2104.93	-89.72
D	43.85	-0.70	55.00	-42.62	2104.93	-89.72

Overall loading

Projected vertical plan area of wall $A_{vert\_w\_0} = b \times H =$ **5130.00** ft^2

Projected vertical area of roof $A_{vert\_r\_0} = b \times d/2 \times \tan(\alpha_0) =$ **1580.25** ft^2

Minimum overall horizontal loading $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_0} + p_{min\_r} \times A_{vert\_r\_0} =$ **94.7** kips

Leeward net force $F_l = F_{w,wB} =$ **-170.7** kips

Windward net force $F_w = F_{w,wA\_1} + F_{w,wA\_2} + F_{w,wA\_3} =$ **120.6** kips

Overall horizontal loading $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min}) =$ **304.0** kips

Roof load case 2 - Wind 0, GC_{pi} -0.18, -0c_{pe}

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p, (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A (+ve)	43.85	0.07	55.00	13.06	3604.83	47.09
B (+ve)	43.85	-0.60	55.00	-18.15	3604.83	-65.43

Total vertical net force $F_{w,v} =$ **-16.48** kips

Total horizontal net force $F_{w,h} =$ **49.33** kips

Walls load case 2 - Wind 0, GC_{pi} -0.18, -0c_{pe}

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p, (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A_1	15.00	0.80	45.10	40.57	2025.00	82.15
A_2	25.00	0.80	49.47	43.54	1350.00	58.78
A_3	38.00	0.80	53.53	46.30	1755.00	81.25
B	43.85	-0.50	55.00	-13.47	5130.00	-69.13
C	43.85	-0.70	55.00	-22.82	2104.93	-48.05
D	43.85	-0.70	55.00	-22.82	2104.93	-48.05

Overall loading

Projected vertical plan area of wall $A_{vert\_w\_0} = b \times H =$ **5130.00** ft^2

Projected vertical area of roof $A_{vert\_r\_0} = b \times d/2 \times \tan(\alpha_0) =$ **1580.25** ft^2

Minimum overall horizontal loading $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_0} + p_{min\_r} \times A_{vert\_r\_0} =$ **94.7** kips

Leeward net force $F_l = F_{w,wB} =$ **-69.1** kips

Windward net force $F_w = F_{w,wA\_1} + F_{w,wA\_2} + F_{w,wA\_3} =$ **222.2** kips

Overall horizontal loading $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min}) =$ **340.6** kips

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location DIRECTIONAL WIND LOADS

date 6/16/2017 by KN

Roof load case 3 - Wind 90, GC_{pi} 0.18, $-c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p, (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A (-ve)	43.85	-0.90	55.00	-51.97	1170.98	-60.86
B (-ve)	43.85	-0.90	55.00	-51.97	1170.98	-60.86
C (-ve)	43.85	-0.50	55.00	-33.27	2341.95	-77.93
D (-ve)	43.85	-0.30	55.00	-23.92	2525.75	-60.43

Total vertical net force $F_{w,v}$ = **-233.76** kips

Total horizontal net force $F_{w,h}$ = **0.00** kips

Walls load case 3 - Wind 90, GC_{pi} 0.18, $-c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p, (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A_1	15.00	0.80	45.10	20.77	720.00	14.95
A_2	35.00	0.80	52.81	26.01	960.00	24.97
A_3	49.71	0.80	56.31	28.39	425.04	12.07
B	43.85	-0.26	55.00	-22.03	2104.93	-46.36
C	43.85	-0.70	55.00	-42.62	5130.00	-218.67
D	43.85	-0.70	55.00	-42.62	5130.00	-218.67

Overall loading

Projected vertical plan area of wall $A_{vert\_w\_90} = d \times H + d^2 \times \tan(\alpha_0) / 4 =$ **2104.93** ft^2

Projected vertical area of roof $A_{vert\_r\_90}$ = **0.00** ft^2

Minimum overall horizontal loading $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_90} + p_{min\_r} \times A_{vert\_r\_90}$ = **33.7** kips

Leeward net force $F_l = F_{w,wB}$ = **-46.4** kips

Windward net force $F_w = F_{w,wA\_1} + F_{w,wA\_2} + F_{w,wA\_3}$ = **52.0** kips

Overall horizontal loading $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min})$ = **98.4** kips

Roof load case 4 - Wind 90, GC_{pi} -0.18, $+c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p, (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A (+ve)	43.85	-0.18	55.00	1.48	1170.98	1.74
B (+ve)	43.85	-0.18	55.00	1.48	1170.98	1.74
C (+ve)	43.85	-0.18	55.00	1.48	2341.95	3.48
D (+ve)	43.85	-0.18	55.00	1.48	2525.75	3.75

Total vertical net force $F_{w,v}$ = **9.62** kips

Total horizontal net force $F_{w,h}$ = **0.00** kips

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Walls load case 4 - Wind 90, GC_{pi} -0.18, $+c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p, (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A_1	15.00	0.80	45.10	40.57	720.00	29.21
A_2	35.00	0.80	52.81	45.81	960.00	43.98
A_3	49.71	0.80	56.31	48.19	425.04	20.48
B	43.85	-0.26	55.00	-2.23	2104.93	-4.69
C	43.85	-0.70	55.00	-22.82	5130.00	-117.09
D	43.85	-0.70	55.00	-22.82	5130.00	-117.09

Overall loading

Projected vertical plan area of wall $A_{vert\_w\_90} = d \times H + d^2 \times \tan(\alpha_0) / 4 =$ **2104.93** ft^2

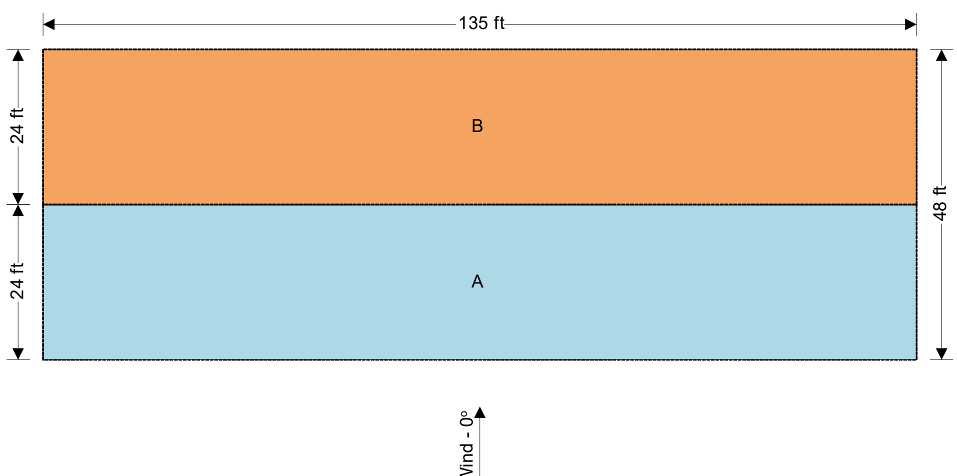
Projected vertical area of roof $A_{vert\_r\_90} =$ **0.00** ft^2

Minimum overall horizontal loading $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_90} + p_{min\_r} \times A_{vert\_r\_90} =$ **33.7** kips

Leeward net force $F_l = F_{w,wB} =$ **-4.7** kips

Windward net force $F_w = F_{w,wA\_1} + F_{w,wA\_2} + F_{w,wA\_3} =$ **93.7** kips

Overall horizontal loading $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min}) =$ **98.4** kips



Plan view - Gable roof

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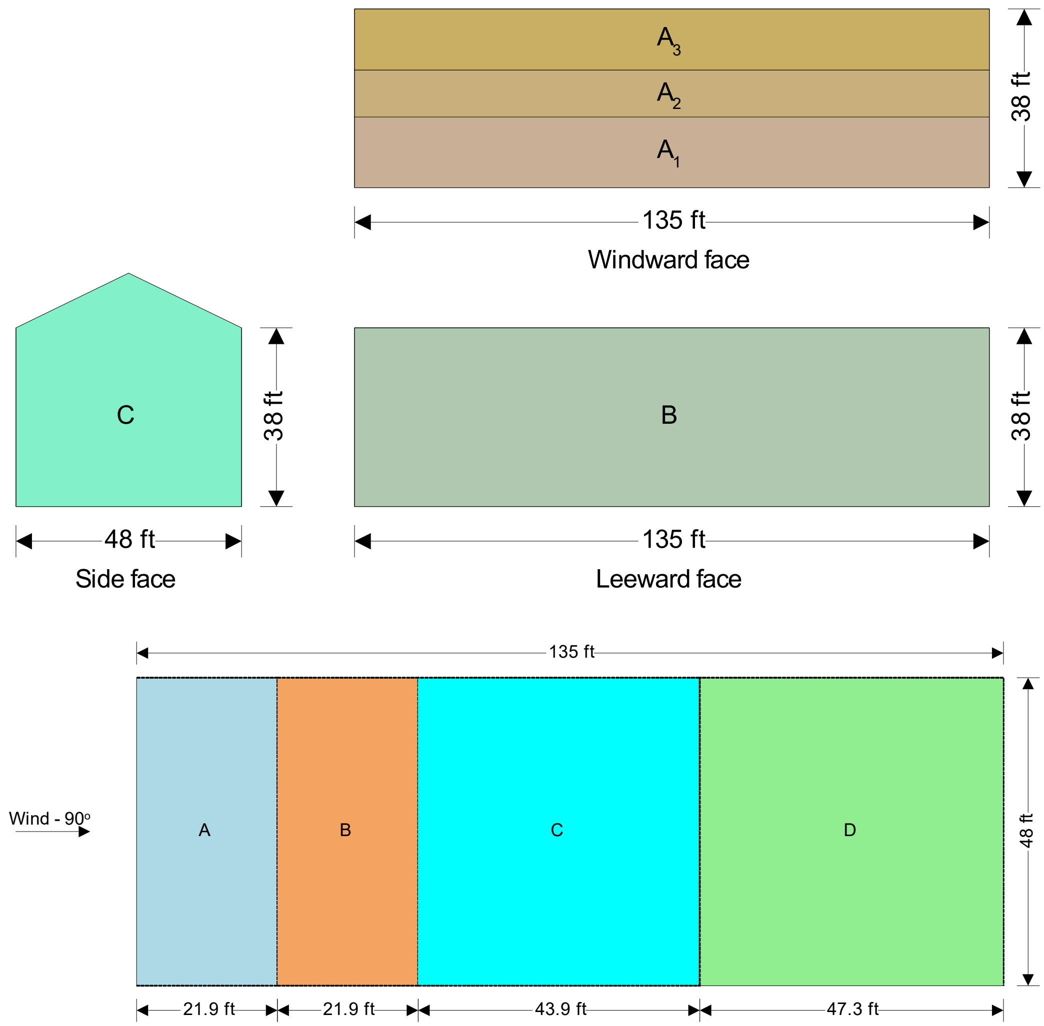

Plan view - Gable roof

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location DIRECTIONAL WIND LOADS

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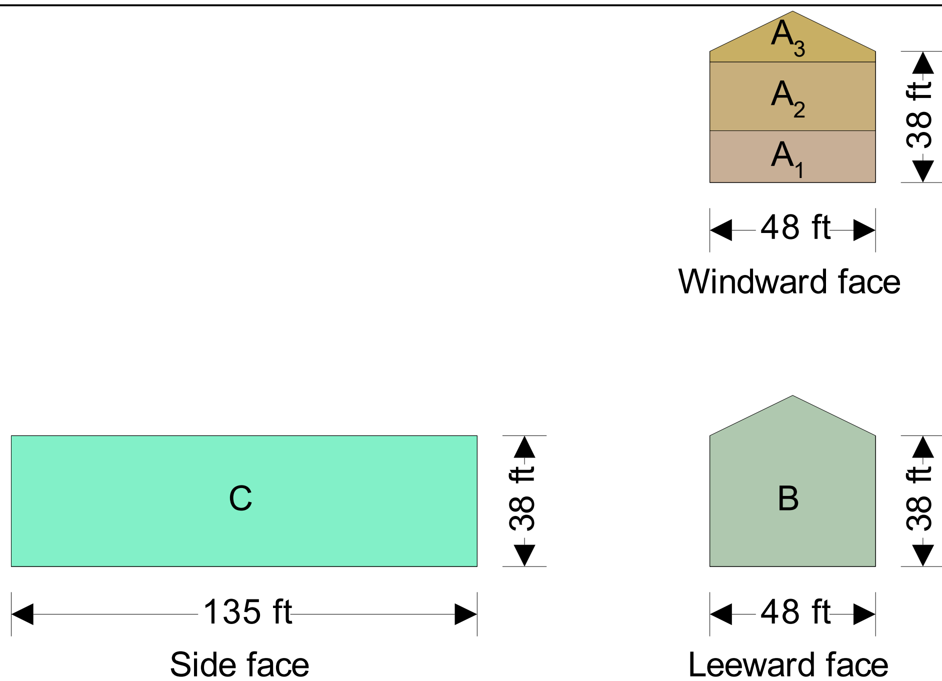


USGS Design Maps Detailed Report

2012/2015 International Building Code (41.36283°N, 111.74423°W)

Site Class D – "Stiff Soil", Risk Category I/II/III

Section 1613.3.1 — Mapped acceleration parameters

Note: Ground motion values provided below are for the direction of maximum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain S_S) and 1.3 (to obtain S_1). Maps in the 2012/2015 International Building Code are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 1613.3.3.

From Figure 1613.3.1(1) [1]	S_S = 0.810 g
From Figure 1613.3.1(2) [2]	S_1 = 0.269 g

Section 1613.3.2 — Site class definitions

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class D, based on the site soil properties in accordance with Section 1613.

2010 ASCE-7 Standard – Table 20.3-1
SITE CLASS DEFINITIONS

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard Rock	>5,000 ft/s	N/A	N/A
B. Rock	2,500 to 5,000 ft/s	N/A	N/A
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff Soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the characteristics: • Plasticity index *PI* > 20, • Moisture content *w* ≥ 40%, and • Undrained shear strength $\bar{s}_u$ < 500 psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

Section 1613.3.3 — Site coefficients and adjusted maximum considered earthquake spectral response acceleration parameters

TABLE 1613.3.3(1)
VALUES OF SITE COEFFICIENT F_a

Site Class	Mapped Spectral Response Acceleration at Short Period				
	$S_S \leq 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_S

For Site Class = D and S_S = 0.810 g, F_a = 1.176

TABLE 1613.3.3(2)
VALUES OF SITE COEFFICIENT F_v

Site Class	Mapped Spectral Response Acceleration at 1–s Period				
	$S_1 \leq 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \geq 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_1

For Site Class = D and S_1 = 0.269 g, F_v = 1.863

Equation (16-37): $S_{MS} = F_a S_S = 1.176 \times 0.810 = 0.953$ g

---

Equation (16-38): $S_{M1} = F_v S_1 = 1.863 \times 0.269 = 0.500$ g

---

Section 1613.3.4 — Design spectral response acceleration parameters

Equation (16-39): $S_{DS} = ⅔ S_{MS} = ⅔ \times 0.953 = 0.635$ g

---

Equation (16-40): $S_{D1} = ⅔ S_{M1} = ⅔ \times 0.500 = 0.333$ g

---

Section 1613.3.5 — Determination of seismic design category

TABLE 1613.3.5(1)
SEISMIC DESIGN CATEGORY BASED ON SHORT-PERIOD (0.2 second) RESPONSE ACCELERATION

VALUE OF S_{DS}	RISK CATEGORY		
	I or II	**III**	**IV**
$S_{DS} < 0.167g$	A	A	A
$0.167g \leq S_{DS} < 0.33g$	B	B	C
$0.33g \leq S_{DS} < 0.50g$	C	C	D
$0.50g \leq S_{DS}$	D	D	D

For Risk Category = I and S_{DS} = 0.635 g, Seismic Design Category = D

TABLE 1613.3.5(2)
SEISMIC DESIGN CATEGORY BASED ON 1-SECOND PERIOD RESPONSE ACCELERATION

VALUE OF S_{D1}	RISK CATEGORY		
	I or II	**III**	**IV**
$S_{D1} < 0.067g$	A	A	A
$0.067g \leq S_{D1} < 0.133g$	B	B	C
$0.133g \leq S_{D1} < 0.20g$	C	C	D
$0.20g \leq S_{D1}$	D	D	D

For Risk Category = I and S_{D1} = 0.333 g, Seismic Design Category = D

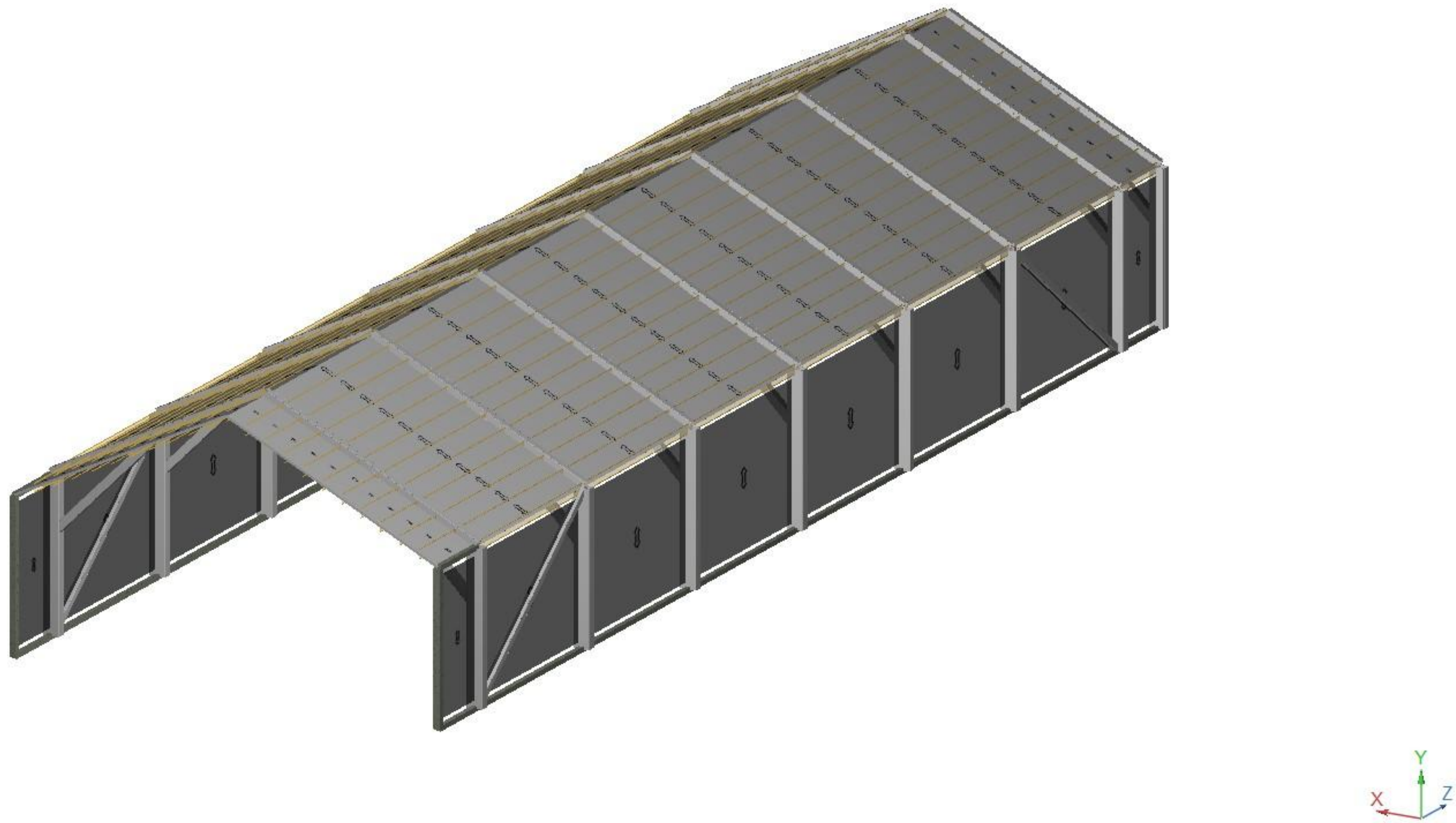
Note: When S_1 is greater than or equal to 0.75g, the Seismic Design Category is **E** for buildings in Risk Categories I, II, and III, and **F** for those in Risk Category IV, irrespective of the above.

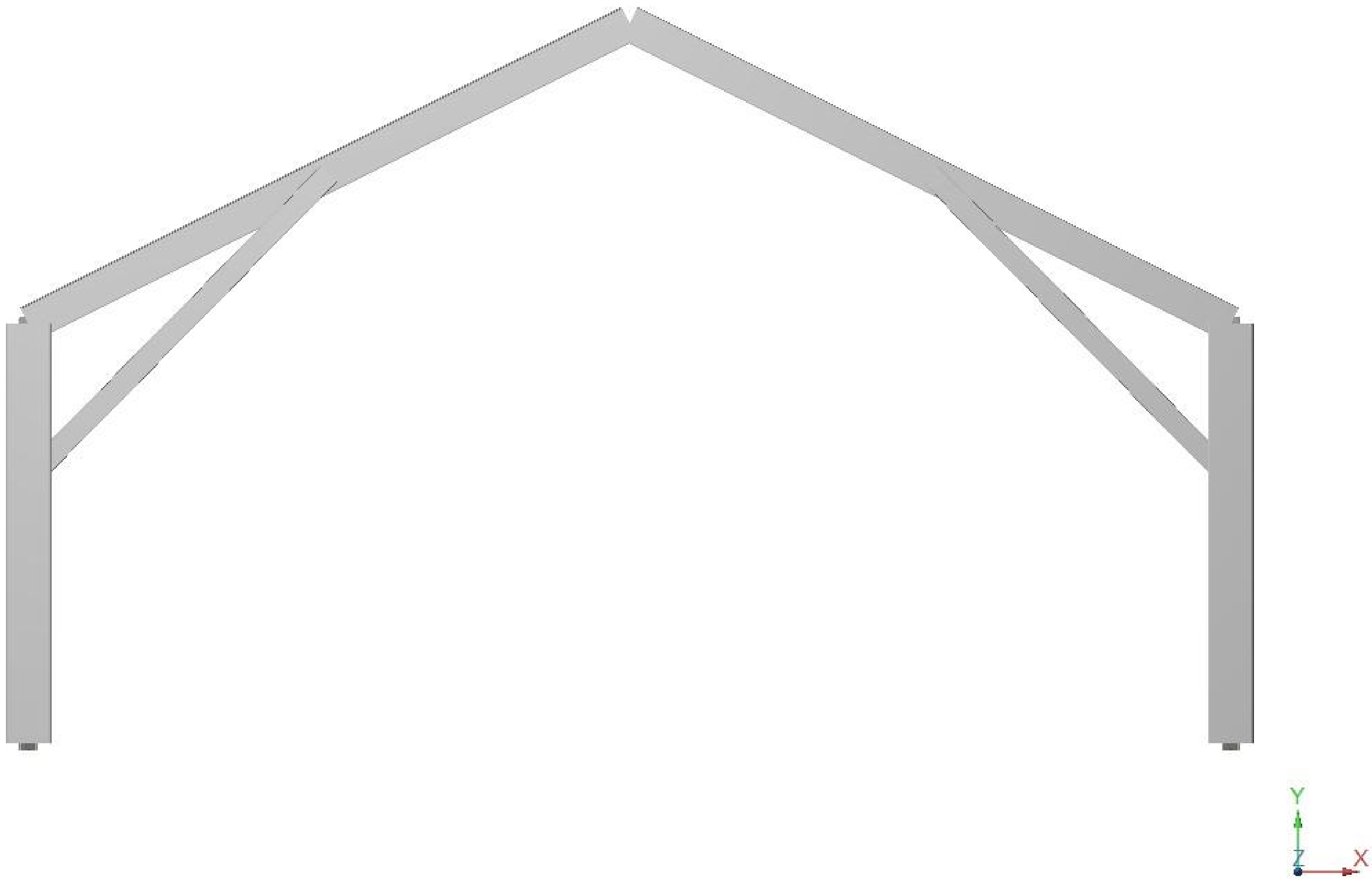
Seismic Design Category ≡ "the more severe design category in accordance with Table 1613.3.5(1) or 1613.3.5(2)" = D

Note: See Section 1613.3.5.1 for alternative approaches to calculating Seismic Design Category.

References

1. *Figure 1613.3.1(1)*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/IBC-2012-Fig1613p3p1(1).pdf
2. *Figure 1613.3.1(2)*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/IBC-2012-Fig1613p3p1(2).pdf


Y
X
Z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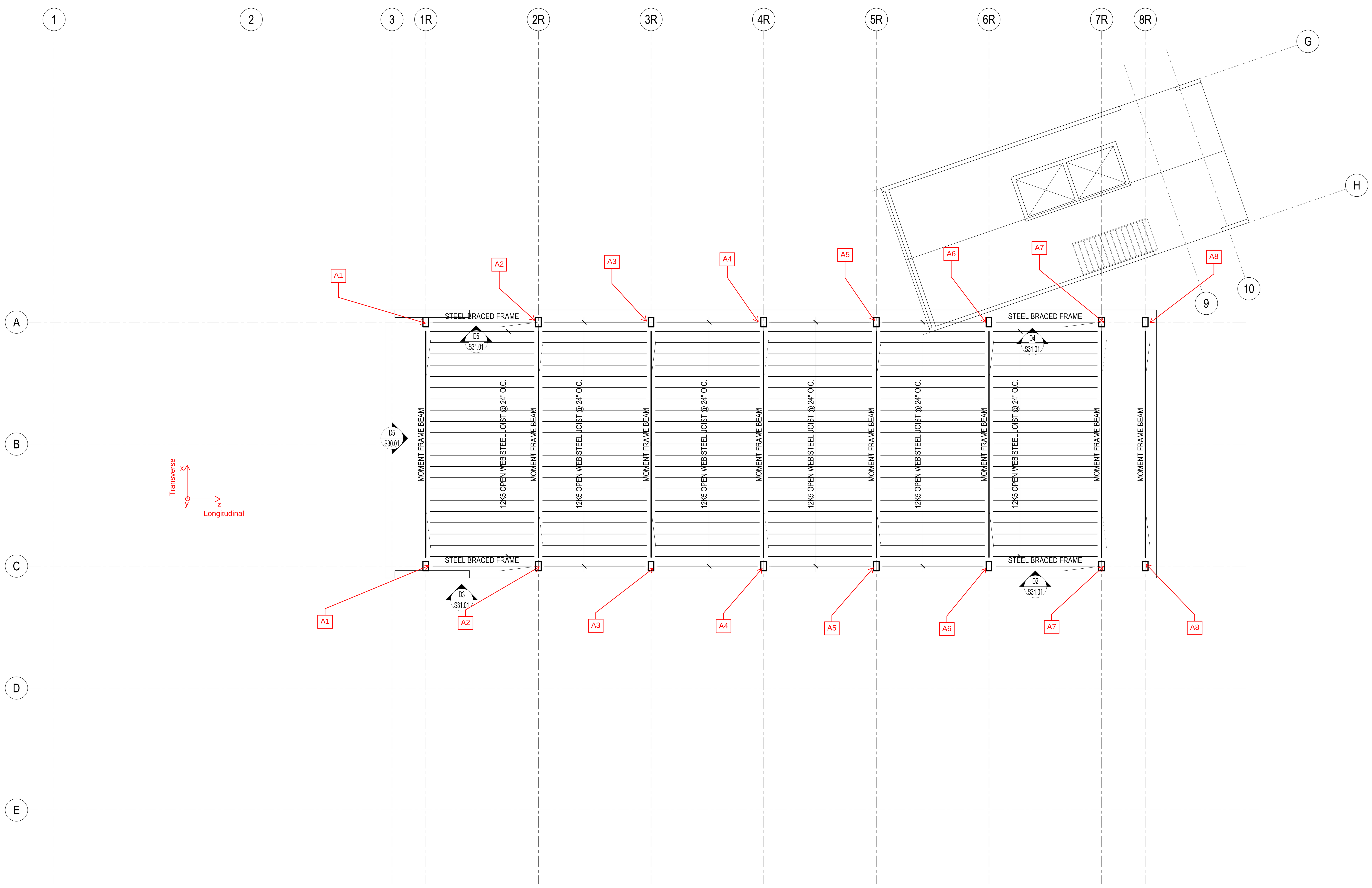
Z
X



A1 S22.04 ROOF FRAMING PLAN

SCALE: 1/8" = 1'-0"

6/22/2017 2:43:47 AM

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POWDER MOUNTAIN

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SHEET TITLE

ROOF FRAMING PLAN

No	Description	Date

JOB NO. 236

DATE 06/02/2017

SCALE 1/8" = 1'-0"

SHEET NO. S22.04

Roof Framing Column Reactions

A1		D	S	Su	Wz	WxPos	WxNeg	Ez	Ex
	Fx	6.16	31.85	26.57	-3.74	-13.88	0.424	-3.69	-25.42
	Fy	18.72	81.53	86.13	-94.28	-0.159	19.82	-62.94	-9.64
	Fz	1.38	7.47	13.56	-129.06	5.77	2.92	-77.83	10.16
A2									
	Fx	6.04	31.46	25.89	0.563	-15.64	-0.777	-0.732	-26
	Fy	16.79	70.79	65.74	100.02	-7.87	16.07	57.1	-19.62
	Fz	0	0	0	-7.39	0	0	-2.12	0
A3									
	Fx	6.34	33.22	28.6	2.83	-15.22	-0.353	0.748	-25.75
	Fy	18.16	80.7	81.97	8.55	-2.87	18.78	1.11	-11.53
	Fz	0	0	0	-7.66	0	0	2.26	0
A4									
	Fx	6.34	33.25	28.9	2.15	-14.98	-0.284	0.383	-25.29
	Fy	18.16	80.75	82.35	7.58	-2.57	18.87	0.597	-10.98
	Fz	0	0	0	-7.69	0	0	-2.27	0
A5									
	Fx	6.22	32.53	28.65	1.13	-14.61	-0.279	-0.212	-24.43
	Fy	17.97	79.63	81.83	6.07	-2.12	18.84	-0.282	-9.88
	Fz	0	0	0	-7.65	0	0	-2.25	0
A6									
	Fx	5.86	30.48	27.31	2.04	-14.21	-0.501	0.379	-22.92
	Fy	16.82	70.86	83.14	-72	6.36	20.27	-46.65	7.52
	Fz	0	0	0	-7.6	0	0	-2.19	0
A7									
	Fx	4.29	21.26	16.66	3.36	-12.59	-1.04	1.4	-20.25
	Fy	14.57	58.62	48.91	88.39	-11.38	12.37	48.99	-25.53
	Fz	-1.36	-7.35	4.6	-109.61	10.21	1.65	-63.27	20.17
A8									
	Fx	2.58	11.22	2.95	3.56	-8.46	-0.159	1.75	-17.6
	Fy	7.7	23.08	14.07	9.2	-8.22	5.52	3.17	-18.41
	Fz	0	0	0	-1.7	0	0	-0.557	0.11

RAM Frame 15.04.00.000
Reaveley Engineers + Associates
DataBase: Summitt Powder Mtn Parcel 4 - v55 CW 06/26/17 22:21:36

CRITERIA:

Rigid End Zones: Ignore Effects
Member Force Output: At Face of Joint
P-Delta: Yes Scale Factor: 1.00
Ground Level: 2-BASEMENT LEVEL
Mesh Criteria :
Max. Distance Between Nodes on Mesh Line (ft) : 4.00
Merge Node Tolerance (in) : 0.0100
Geometry Tolerance (in) : 0.0050
Walls Out-of-plane Stiffness Not Included in Analysis.
Sign considered for Dynamic Load Case Results.
Rigid Links Included at Fixed Beam-to-Wall Locations
Eigenvalue Analysis : Eigen Vectors (Subspace Iteration)

DIAPHRAGM DATA:

Story	Diaph #	Diaph Type
6-LITTLE BARN ROOF	1	Rigid
5-B.O. ROOF	1	Rigid
4-UPPER GROUND LVL	1	Rigid
3-GROUND LEVEL	1	Rigid
2.5-STAIR	1	Rigid
2-BASEMENT LEVEL	1	Rigid

Disconnect Internal Nodes of Beams: Yes
Disconnect Nodes outside Slab Boundary: Yes

STORY MASS DATA:

Includes Self Mass of:
- Beams
- Columns (Half mass of columns above and below)
- Walls (Half mass of walls above and below)
- Slabs/Deck

Calculated Values:

Story	Diaph #	Weight kips	Mass k-s2/ft	MMI ft-k-s2	Xm ft	Ym ft	EccX ft	EccY ft
6-LITTLE BARN ROOF	1	191.40	5.94	2353	103.58	107.32	3.32	2.23
5-B.O. ROOF	1	580.48	18.03	33485	51.24	64.85	6.91	2.22
	None	70.09	2.18	630	98.63	104.97	--	--
4-UPPER GROUND LVL	1	316.61	9.83	4503	99.06	105.98	3.18	2.16
3-GROUND LEVEL	1	2537.03	78.79	261544	57.73	52.85	7.62	6.54
2.5-STAIR	1	237.85	7.39	1106	93.27	10.10	1.35	1.15
	None	747.27	23.21	89146	85.95	74.61	--	--
2-BASEMENT LEVEL	1	1238.57	38.46	42710	-21.99	44.56	2.56	5.16
	None	460.60	14.30	45366	95.10	69.65	--	--

Story	Diaph #	Combine
6-LITTLE BARN ROOF	1	1-3-GROUND LEVEL
5-B.O. ROOF	1	1-3-GROUND LEVEL
	None	1-3-GROUND LEVEL
4-UPPER GROUND LVL	1	1-3-GROUND LEVEL

Story	Diaph #	Combine
3-GROUND LEVEL	1	None
2.5-STAIR	1	1-3-GROUND LEVEL
	None	1-3-GROUND LEVEL
2-BASEMENT LEVEL	1	None
	None	1-2-BASEMENT LEVEL

Combined/Merged Values:

Story	Diaph #	Weight kips	Mass k-s2/ft	MMI ft-k-s2	Xm ft	Ym ft	EccX ft	EccY ft
3-GROUND LEVEL	1	4680.7	145.36	502928	68.52	62.24	7.62	6.54
2-BASEMENT LEVEL	1	1699.2	52.77	237589	9.75	51.36	2.56	5.16

WIND EXPOSURE DATA:

Calculated Values:

Story	Diaph #	Building Extents (ft) Min X	Max X	Min Y	Max Y	Expose	Parapet ft
6-LITTLE BARN ROOF	1	71.07	137.39	86.00	130.59	Full	0.00
5-B.O. ROOF	1	-18.00	120.25	42.67	87.00	Full	0.00
4-UPPER GROUND LVL	1	69.50	133.14	86.00	129.12	Full	0.00
3-GROUND LEVEL	1	-18.25	134.18	-1.58	129.12	Full	0.00
2.5-STAIR	1	79.75	106.75	-1.08	22.00	Full	0.00
2-BASEMENT LEVEL	1	-43.00	8.25	-7.92	95.33	Full	0.00

STORY GRAVITY LOADS DATA:

Includes Weight of:

Beams
Columns
Walls
Slabs/Deck

Live Load Reduction (Calculated)

Reducible : 60.00 %
Storage : 0.00 %

Calculated Values:

Story	Diaph #	Dead kips	Xc ft	Yc ft	Live kips	Xc ft	Yc ft
6-LITTLE BARN ROOF	1	106.40	101.69	106.42	0.00	0.00	0.00
5-B.O. ROOF	1	194.42	51.20	64.90	0.00	0.00	0.00
	None	90.11	98.63	104.97	0.00	0.00	0.00
4-UPPER GROUND LVL	1	373.26	100.99	107.81	58.05	119.23	102.09
3-GROUND LEVEL	1	2424.83	60.58	56.31	560.35	28.76	69.52
2.5-STAIR	1	241.34	93.27	8.42	0.00	0.00	0.00
	None	911.22	85.95	74.61	0.00	0.00	0.00
2-BASEMENT LEVEL	1	1327.05	-20.33	43.78	135.36	-4.64	23.75
	None	30.94	17.62	79.42	0.00	0.00	0.00

Story	Diaph #	Snow kips	Xc ft	Yc ft	Combine
6-LITTLE BARN ROOF	1	310.95	104.29	107.55	1-3-GROU ND LEVEL

Story	Diaph #	Snow	Xc	Yc	Combine
5-B.O. ROOF	1	1127.75	51.13	64.83	1-3-GROU ND LEVEL
	None	0.00	0.00	0.00	1-3-GROU ND LEVEL
4-UPPER GROUND LVL	1	57.21	76.89	94.32	1-3-GROU ND LEVEL
3-GROUND LEVEL	1	1517.42	51.32	35.72	None
2.5-STAIR	1	131.26	93.34	10.46	1-3-GROU ND LEVEL
	None	0.00	0.00	0.00	1-3-GROU ND LEVEL
2-BASEMENT LEVEL	1	859.17	-30.21	43.98	None
	None	0.00	0.00	0.00	1-2-BASEM ENT LEVEL

User Specified Values:

Story	Diaph #	Dead kips	Xc ft	Yc ft	Live kips	Xc ft	Yc ft
6-LITTLE BARN ROOF	1	0.00	0.00	0.00	0.00	0.00	0.00
5-B.O. ROOF	1	0.00	0.00	0.00	0.00	0.00	0.00
4-UPPER GROUND LVL	1	0.00	0.00	0.00	0.00	0.00	0.00
3-GROUND LEVEL	1	4341.58	72.57	64.54	618.39	37.26	72.58
2.5-STAIR	1	0.00	0.00	0.00	0.00	0.00	0.00
2-BASEMENT LEVEL	1	1357.99	-19.47	44.59	135.36	-4.64	23.75

Story	Diaph #	Snow kips	Xc ft	Yc ft
6-LITTLE BARN ROOF	1	0.0	0.00	0.00
5-B.O. ROOF	1	0.0	0.00	0.00
4-UPPER GROUND LVL	1	0.0	0.00	0.00
3-GROUND LEVEL	1	3144.6	58.71	53.28
2.5-STAIR	1	0.0	0.00	0.00
2-BASEMENT LEVEL	1	859.2	-30.21	43.98

RAM Frame 15.04.00.000
Reaveley Engineers + Associates
DataBase: Summitt Powder Mtn Parcel 4 - v55 CW 06/26/17 22:21:36

LOAD CASE: EQ

Seismic ASCE 7-10 Equivalent Lateral Force

Site Class: C	Importance Factor: 1.00	Ss: 0.810 g	S1: 0.270 g	TL: 12.00 s
Fa: 1.076	Fv: 1.530	SDs: 0.581 g	SD1: 0.275 g	

Occupancy Category: II Seismic Design Category: D
Provisions for: Force
Ground Level: 2-BASEMENT LEVEL

Dir	Eccent	R	Ta Equation	Building Period-T
X	+ And -	5.0	Std,Ct=0.030,x=0.75	Calculated
Y	+ And -	5.0	Std,Ct=0.030,x=0.75	Calculated

Dir	Ta	Cu	T	T-used	Cs Eq12.8-2	Cs(max) Eq12.8-3	Cs(min) Eq12.8-5	Cs-used	k
X	0.517	1.425	0.157	0.157	0.116	0.351	0.026	0.116	1.000

Dir	Ta	Cu	T	T-used	Cs Eq12.8-2	Cs(max) Eq12.8-3	Cs(min) Eq12.8-5	Cs-used	k
Y	0.517	1.425	0.087	0.087	0.116	0.631	0.026	0.116	1.000

Total Building Weight (kips) = 4680.72

APPLIED DIAPHRAGM FORCES

Type: EQ_ASCE710_X_+E_F

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
6-LITTLE BARN ROOF	1	44.50	0.00	0.00	103.58	109.55
5-B.O. ROOF	1	39.50	0.00	0.00	51.24	67.07
4-UPPER GROUND LVL	1	30.50	0.00	0.00	99.06	108.14
3-GROUND LEVEL	1	20.50	543.94	0.00	68.52	68.78
2.5-STAIR	1	12.50	0.00	0.00	93.27	11.25

APPLIED STORY FORCES

Type: EQ_ASCE710_X_+E_F

Level	Ht ft	Fx kips	Fy kips
6-LITTLE BARN ROOF	44.50	0.00	0.00
5-B.O. ROOF	39.50	0.00	0.00
4-UPPER GROUND LVL	30.50	0.00	0.00
3-GROUND LEVEL	20.50	543.94	0.00
2.5-STAIR	12.50	0.00	0.00
		543.94	0.00

APPLIED DIAPHRAGM FORCES

Type: EQ_ASCE710_X_-E_F

Level	Diaph.#	Ht	Fx	Fy	X	Y

		ft	kips	kips	ft	ft
6-LITTLE BARN ROOF	1	44.50	0.00	0.00	103.58	105.09
5-B.O. ROOF	1	39.50	0.00	0.00	51.24	62.64
4-UPPER GROUND LVL	1	30.50	0.00	0.00	99.06	103.83
3-GROUND LEVEL	1	20.50	543.94	0.00	68.52	55.70
2.5-STAIR	1	12.50	0.00	0.00	93.27	8.95

APPLIED STORY FORCES

Type: EQ_ASCE710_X_-E_F

Level	Ht	Fx	Fy
	ft	kips	kips
6-LITTLE BARN ROOF	44.50	0.00	0.00
5-B.O. ROOF	39.50	0.00	0.00
4-UPPER GROUND LVL	30.50	0.00	0.00
3-GROUND LEVEL	20.50	543.94	0.00
2.5-STAIR	12.50	0.00	0.00
		543.94	0.00

APPLIED DIAPHRAGM FORCES

Type: EQ_ASCE710_Y_+E_F

Level	Diaph.#	Ht	Fx	Fy	X	Y
		ft	kips	kips	ft	ft
6-LITTLE BARN ROOF	1	44.50	0.00	0.00	106.89	107.32
5-B.O. ROOF	1	39.50	0.00	0.00	58.15	64.85
4-UPPER GROUND LVL	1	30.50	0.00	0.00	102.24	105.98
3-GROUND LEVEL	1	20.50	0.00	543.94	76.14	62.24
2.5-STAIR	1	12.50	0.00	0.00	94.62	10.10

APPLIED STORY FORCES

Type: EQ_ASCE710_Y_+E_F

Level	Ht	Fx	Fy
	ft	kips	kips
6-LITTLE BARN ROOF	44.50	0.00	0.00
5-B.O. ROOF	39.50	0.00	0.00
4-UPPER GROUND LVL	30.50	0.00	0.00
3-GROUND LEVEL	20.50	0.00	543.94
2.5-STAIR	12.50	0.00	0.00
		0.00	543.94

APPLIED DIAPHRAGM FORCES

Type: EQ_ASCE710_Y_-E_F

Level	Diaph.#	Ht	Fx	Fy	X	Y
		ft	kips	kips	ft	ft
6-LITTLE BARN ROOF	1	44.50	0.00	0.00	100.26	107.32
5-B.O. ROOF	1	39.50	0.00	0.00	44.32	64.85
4-UPPER GROUND LVL	1	30.50	0.00	0.00	95.88	105.98
3-GROUND LEVEL	1	20.50	0.00	543.94	60.90	62.24
2.5-STAIR	1	12.50	0.00	0.00	91.92	10.10

APPLIED STORY FORCES

Type: EQ_ASCE710_Y_-E_F

Level	Ht	Fx	Fy
	ft	kips	kips
6-LITTLE BARN ROOF	44.50	0.00	0.00
5-B.O. ROOF	39.50	0.00	0.00
4-UPPER GROUND LVL	30.50	0.00	0.00
3-GROUND LEVEL	20.50	0.00	543.94
2.5-STAIR	12.50	0.00	0.00
		0.00	543.94

RAM Frame 15.04.00.000
Reaveley Engineers + Associates
DataBase: Summitt Powder Mtn Parcel 4 - v55 CW 06/26/17 22:21:36

CRITERIA:
Rigid End Zones: Ignore Effects
Member Force Output: At Face of Joint
P-Delta: Yes Scale Factor: 1.00
Ground Level: 2-BASEMENT LEVEL
Mesh Criteria :
Max. Distance Between Nodes on Mesh Line (ft) : 4.00
Merge Node Tolerance (in) : 0.0100
Geometry Tolerance (in) : 0.0050
Walls Out-of-plane Stiffness Not Included in Analysis.
Sign considered for Dynamic Load Case Results.
Rigid Links Included at Fixed Beam-to-Wall Locations
Eigenvalue Analysis : Eigen Vectors (Subspace Iteration)

Load Case: D DeadLoad RAMUSER

Level	Diaph. #	Shear-X kips	Shear-Y kips
6-LITTLE BARN ROOF	1	0.00	0.00
4-UPPER GROUND LVL	1	-0.00	0.00
3-GROUND LEVEL	1	-0.00	-0.00
2.5-STAIR	1	0.83	-0.00
2.5-STAIR	None	-0.83	0.00
2-BASEMENT LEVEL	1	-11.30	8.64
2-BASEMENT LEVEL	None	0.40	0.00

Summary - Total Story Shears

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
6-LITTLE BARN ROOF	0.00	0.00	0.00	0.00
5-B.O. ROOF	0.00	0.00	0.00	0.00
4-UPPER GROUND LVL	-0.00	-0.00	0.00	-0.00
3-GROUND LEVEL	-0.00	-0.00	-0.00	-0.00
2.5-STAIR	-0.00	0.00	-0.00	-0.00
2-BASEMENT LEVEL	-10.90	-10.90	8.64	8.64

Load Case: Lp PosLiveLoad RAMUSER

Level	Diaph. #	Shear-X kips	Shear-Y kips
6-LITTLE BARN ROOF	1	-0.00	0.00
4-UPPER GROUND LVL	1	0.16	0.06
3-GROUND LEVEL	1	-0.04	-0.22
2.5-STAIR	1	0.20	-0.00
2.5-STAIR	None	-0.25	-0.17
2-BASEMENT LEVEL	1	-7.05	3.98
2-BASEMENT LEVEL	None	0.41	0.00

Summary - Total Story Shears

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
6-LITTLE BARN ROOF	-0.00	-0.00	0.00	0.00
5-B.O. ROOF	-0.00	-0.00	-0.00	-0.00
4-UPPER GROUND LVL	0.16	0.16	0.06	0.06

3-GROUND LEVEL	-0.04	-0.21	-0.22	-0.28
2.5-STAIR	-0.04	-0.00	-0.17	0.05
2-BASEMENT LEVEL	-6.64	-6.60	3.98	4.15

Load Case: Ln NegLiveLoad RAMUSER

Level	Diaph. #	Shear-X kips	Shear-Y kips
6-LITTLE BARN ROOF	1	0.00	-0.00
4-UPPER GROUND LVL	1	0.00	0.00
3-GROUND LEVEL	1	-0.00	0.00
2.5-STAIR	1	-0.00	-0.00
2.5-STAIR	None	0.00	0.00
2-BASEMENT LEVEL	1	0.14	0.05

Summary - Total Story Shears

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
6-LITTLE BARN ROOF	0.00	0.00	-0.00	-0.00
5-B.O. ROOF	0.00	0.00	-0.00	-0.00
4-UPPER GROUND LVL	0.00	0.00	0.00	0.00
3-GROUND LEVEL	-0.00	-0.00	0.00	0.00
2.5-STAIR	-0.00	0.00	0.00	-0.00
2-BASEMENT LEVEL	0.14	0.14	0.05	0.05

Load Case: Sp PosRoofLiveLoad RAMUSER

Level	Diaph. #	Shear-X kips	Shear-Y kips
6-LITTLE BARN ROOF	1	-0.00	0.00
4-UPPER GROUND LVL	1	-0.00	-0.00
3-GROUND LEVEL	1	0.00	-0.01
2.5-STAIR	1	0.29	-0.00
2.5-STAIR	None	-0.29	-0.00
2-BASEMENT LEVEL	1	-7.73	8.26
2-BASEMENT LEVEL	None	0.01	0.00

Summary - Total Story Shears

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
6-LITTLE BARN ROOF	-0.00	-0.00	0.00	0.00
5-B.O. ROOF	-0.00	-0.00	0.00	0.00
4-UPPER GROUND LVL	-0.00	-0.00	-0.00	-0.00
3-GROUND LEVEL	0.00	0.00	-0.01	-0.01
2.5-STAIR	0.00	0.00	-0.01	-0.00
2-BASEMENT LEVEL	-7.72	-7.73	8.26	8.26

Load Case: Sn NegRoofLiveLoad RAMUSER

Level	Diaph. #	Shear-X kips	Shear-Y kips
6-LITTLE BARN ROOF	1	0.00	-0.00
4-UPPER GROUND LVL	1	0.00	0.00
3-GROUND LEVEL	1	-0.00	0.00
2.5-STAIR	1	-0.01	-0.00

2.5-STAIR	None	0.01	0.00
2-BASEMENT LEVEL	1	0.89	0.33

Summary - Total Story Shears

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
6-LITTLE BARN ROOF	0.00	0.00	-0.00	-0.00
5-B.O. ROOF	0.00	0.00	-0.00	-0.00
4-UPPER GROUND LVL	0.00	0.00	0.00	0.00
3-GROUND LEVEL	-0.00	-0.00	0.00	0.00
2.5-STAIR	-0.00	0.00	0.00	-0.00
2-BASEMENT LEVEL	0.89	0.89	0.33	0.33

Load Case: E1 EQ EQ_ASCE710_X_+E_F

Level	Diaph. #	Shear-X kips	Shear-Y kips
6-LITTLE BARN ROOF	1	-0.00	0.00
4-UPPER GROUND LVL	1	0.00	-0.00
3-GROUND LEVEL	1	544.20	-0.08
2.5-STAIR	1	24.39	0.00
2.5-STAIR	None	519.81	-0.08
2-BASEMENT LEVEL	1	-6.31	4.76
2-BASEMENT LEVEL	None	0.84	0.00

Summary - Total Story Shears

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
6-LITTLE BARN ROOF	-0.00	-0.00	0.00	0.00
5-B.O. ROOF	-0.00	-0.00	0.00	0.00
4-UPPER GROUND LVL	0.00	0.00	-0.00	-0.00
3-GROUND LEVEL	544.20	544.20	-0.08	-0.08
2.5-STAIR	544.20	0.00	-0.08	0.00
2-BASEMENT LEVEL	-5.47	-549.67	4.76	4.85

Load Case: E2 EQ EQ_ASCE710_X_-E_F

Level	Diaph. #	Shear-X kips	Shear-Y kips
6-LITTLE BARN ROOF	1	-0.00	0.00
4-UPPER GROUND LVL	1	0.00	-0.00
3-GROUND LEVEL	1	544.21	-0.10
2.5-STAIR	1	27.49	0.00
2.5-STAIR	None	516.73	-0.10
2-BASEMENT LEVEL	1	-8.36	5.87
2-BASEMENT LEVEL	None	0.87	0.00

Summary - Total Story Shears

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
6-LITTLE BARN ROOF	-0.00	-0.00	0.00	0.00
5-B.O. ROOF	-0.00	-0.00	0.00	0.00
4-UPPER GROUND LVL	0.00	0.00	-0.00	-0.00
3-GROUND LEVEL	544.21	544.21	-0.10	-0.10
2.5-STAIR	544.21	-0.00	-0.10	-0.00

2-BASEMENT LEVEL	-7.49	-551.70	5.87	5.97

Load Case: E3 EQ EQ_ASCE710_Y_+E_F

Level	Diaph. #	Shear-X kips	Shear-Y kips
6-LITTLE BARN ROOF	1	0.00	-0.00
4-UPPER GROUND LVL	1	0.00	0.00
3-GROUND LEVEL	1	-0.08	544.07
2.5-STAIR	1	-8.55	0.00
2.5-STAIR	None	8.47	544.06
2-BASEMENT LEVEL	1	8.91	-4.99
2-BASEMENT LEVEL	None	-0.23	-0.00

Summary - Total Story Shears

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
6-LITTLE BARN ROOF	0.00	0.00	-0.00	-0.00
5-B.O. ROOF	0.00	0.00	-0.00	-0.00
4-UPPER GROUND LVL	0.00	0.00	0.00	0.00
3-GROUND LEVEL	-0.08	-0.08	544.07	544.07
2.5-STAIR	-0.08	-0.00	544.07	-0.00
2-BASEMENT LEVEL	8.68	8.76	-4.99	-549.06

Load Case: E4 EQ EQ_ASCE710_Y_-E_F

Level	Diaph. #	Shear-X kips	Shear-Y kips
6-LITTLE BARN ROOF	1	0.00	-0.00
4-UPPER GROUND LVL	1	0.00	0.00
3-GROUND LEVEL	1	-0.10	544.08
2.5-STAIR	1	-12.16	0.00
2.5-STAIR	None	12.06	544.08
2-BASEMENT LEVEL	1	11.30	-6.29
2-BASEMENT LEVEL	None	-0.26	-0.00

Summary - Total Story Shears

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
6-LITTLE BARN ROOF	0.00	0.00	-0.00	-0.00
5-B.O. ROOF	0.00	0.00	-0.00	-0.00
4-UPPER GROUND LVL	0.00	0.00	0.00	0.00
3-GROUND LEVEL	-0.10	-0.10	544.08	544.08
2.5-STAIR	-0.10	-0.00	544.08	0.00
2-BASEMENT LEVEL	11.04	11.14	-6.29	-550.37

05
LATERAL FORCE RESISTING SYSTEM

REAVELEY ENGINEERS + ASSOCIATES
Consulting Structural Engineers

Reinforced Concrete SW and Reinforced CMU SW Summary

IBC 2012 & ACI 318 & ACI 530

Project: Powder Mountain Parcel 4

Engineer: CW
Date: 6/26/2017

General Input Information

Material Properties			Seismic Data			Load Case Data	
f_y =	60	ksi	S_{DS} =	0.58	g	f_1 =	0.5
			C_d/I_E =	4.00		f_2 =	0.29

Section Cut	Length ft	Thickness in	Type	f'_c psi	Height ft	δ_{xe} in	Horizontal Reinforcement	Vertical Reinforcement	Solid Grouted?	Jamb Reinforcement Type	Length	Vertical	Bars/Cell	Ties	Flexure DCR	Shear DCR	Wall Mark	Jamb Mark
P-01	8.00	12	Concrete	4000	16.00	0.50	2 - # 5 @ 12 in. o.c.	2 - # 5 @ 12 in. o.c.	Y	2	12 in.	4 - # 5	--	# 3 @ 12 in. o.c.	**0.15**	**0.04**		
P-02	8.00	12	Concrete	4000	16.00	0.50	2 - # 5 @ 12 in. o.c.	2 - # 5 @ 12 in. o.c.	Y	2	12 in.	4 - # 5	--	# 3 @ 12 in. o.c.	**0.14**	**0.03**		
P-03	8.00	12	Concrete	4000	16.00	0.50	2 - # 5 @ 12 in. o.c.	2 - # 5 @ 12 in. o.c.	Y	2	12 in.	4 - # 5	--	# 3 @ 12 in. o.c.	**0.14**	**0.03**		
P-04	34.42	18	Concrete	4000	16.00	0.50	2 - # 5 @ 12 in. o.c.	2 - # 5 @ 12 in. o.c.	Y	1	12 in.	4 - # 5	--	# 0 @ 12 in. o.c.	**0.04**	**0.01**		
P-04-2	5.42	12	Concrete	4000	16.00	0.50	2 - # 5 @ 12 in. o.c.	2 - # 5 @ 12 in. o.c.	Y	2	12 in.	4 - # 5	--	# 3 @ 12 in. o.c.	**0.35**	**0.12**		
P-05	103.08	18	Concrete	4000	16.00	0.50	2 - # 5 @ 12 in. o.c.	2 - # 5 @ 12 in. o.c.	Y	1	12 in.	4 - # 5	--	# 0 @ 12 in. o.c.	**0.03**	**0.06**		
P-06	30.58	18	Concrete	4000	16.00	0.50	2 - # 5 @ 12 in. o.c.	2 - # 5 @ 12 in. o.c.	Y	1	12 in.	4 - # 5	--	# 0 @ 12 in. o.c.	**0.05**	**0.10**		
P-07	56.83	18	Concrete	4000	16.00	0.50	2 - # 5 @ 12 in. o.c.	2 - # 5 @ 12 in. o.c.	Y	1	12 in.	4 - # 5	--	# 0 @ 12 in. o.c.	**0.09**	**0.19**		
P-08	23.50	18	Concrete	4000	16.00	0.50	2 - # 5 @ 12 in. o.c.	2 - # 5 @ 12 in. o.c.	Y	2	12 in.	4 - # 5	--	# 3 @ 12 in. o.c.	**0.19**	**0.15**		
P-09	43.33	12	Concrete	4000	16.00	0.50	2 - # 5 @ 12 in. o.c.	2 - # 5 @ 12 in. o.c.	Y	1	12 in.	4 - # 5	--	# 0 @ 12 in. o.c.	**0.26**	**0.16**		
P-10	19.25	12	Concrete	4000	16.00	0.50	2 - # 5 @ 12 in. o.c.	2 - # 5 @ 12 in. o.c.	Y	2	12 in.	4 - # 5	--	# 3 @ 12 in. o.c.	**0.12**	**0.04**		
P-11	18.25	12	Concrete	4000	16.00	0.50	2 - # 5 @ 12 in. o.c.	2 - # 5 @ 12 in. o.c.	Y	2	12 in.	4 - # 5	--	# 3 @ 12 in. o.c.	**0.23**	**0.09**		
P-12-1	19.00	8	Concrete	4000	16.00	0.50	1 - # 5 @ 12 in. o.c.	1 - # 5 @ 12 in. o.c.	Y	1	12 in.	2 - # 5	--	# 0 @ 12 in. o.c.	**0.14**	**0.15**		
P-12-2	19.00	8	Concrete	4000	16.00	0.50	1 - # 5 @ 12 in. o.c.	1 - # 5 @ 12 in. o.c.	Y	1	12 in.	2 - # 5	--	# 0 @ 12 in. o.c.	**0.12**	**0.11**		
P-12-3	7.58	8	Concrete	4000	16.00	0.50	1 - # 5 @ 12 in. o.c.	1 - # 5 @ 12 in. o.c.	Y	1	12 in.	2 - # 5	--	# 0 @ 12 in. o.c.	**0.07**	**0.08**		
P-12-4	7.58	8	Concrete	4000	16.00	0.50	1 - # 5 @ 12 in. o.c.	1 - # 5 @ 12 in. o.c.	Y	1	12 in.	2 - # 5	--	# 0 @ 12 in. o.c.	**0.10**	**0.05**		

REAVELEY ENGINEERS + ASSOCIATES
Consulting Structural Engineers

Special Reinforced Concrete Shearwall

IBC & ACI 318

PROJECT NAME: Powder Mountain Parcel 4 **ENGINEER:**
LOCATION: P-01 **DATE:**

General Input Information

Length =	96	in.	f'_c =	4,000	psi	S_{DS} =	0.580	hw/lw =	2.00
Height =	16	ft	f_y =	60	ksi	f_1 =	0.5	β_1 =	0.85
t =	12	in.	C_d/I_E =	4.00		f_2 =	0.29		

Reinforcement

	Size	Spacing
Horizontal:	# 5	12 in. o.c.
Vertical:	# 5	12 in. o.c.
# Mats:	2	
Jamb Ties:	# 3	12 in. o.c.
Jamb Vert:	4 -	# 5

Jamb Input

Jamb Length =	12	in.
Jamb Bar Pattern:	2 Mats	
Tie Bar Cover:	1.50	in
Cover to CL Vert Bars:	2.19	in.
Jamb Bar Spacing =	7.63	in.
ρ_b =	0.0086	
Check per ACI 18.10.6.2?	N	
δ_{xe} =	0.5	in
Max # sqrt f'c =	8	8 typical (10 for a single pier)

Unfactored Loads

	D	L	S	E	Soil
P (kips)	84.51	0	67.31	0	
M (k-ft)	-16.07	0.98	-21.72	157.3	
V (kips)	2.73	-0.05	3.9	-7.67	

General Output Information

Reinforcement Check

ρ_l = 0.0043
ρ_t = 0.0043

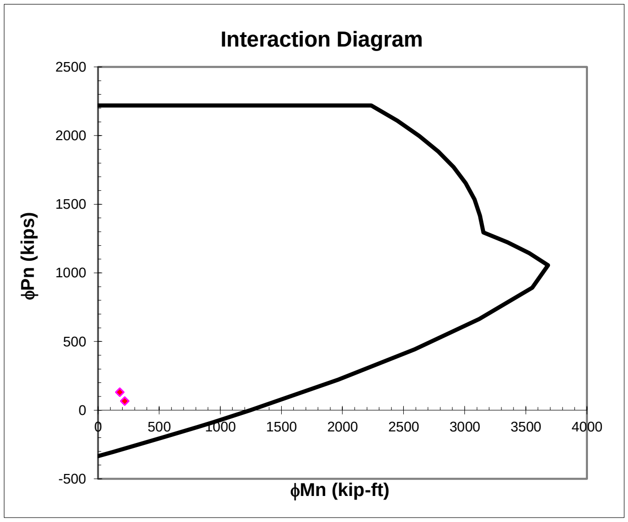
Flexural Check

Load Combination 1 - 1.32*D + 0.5*L + 0.29*S + 1.30*E + 1.00*Soil (Max Pu)

P_u =	130.7	kips	c =	11.4	in.
M_u =	178	kip-ft	a =	9.7	in.
ϕM_n =	1682	kip-ft > Mu	**DCR =**	**0.11**	

Load Combination 2 - 0.78*D + -1.30*E + 1.00*Soil (Min Pu)

P_u =	66.3	kips	c =	9.6	in.
M_u =	217	kip-ft	a =	8.2	in.
ϕM_n =	1469	kip-ft > Mu	**DCR =**	**0.15**	



Shear Check

V_u =	12.1	kips	V_{Mn} =	55.5	kips
Max V_n =	582.9	kips			
V_n =	443.3	kips			
ϕ =	0.75				
ϕV_n =	332.5	kips	**DCR =**	**0.04**	

Boundary Element

<u>ACI 318 Section 18.10.6.2</u>

δ_u =	2.00	in
δ_u/h_w =	0.010	
$l_w/(600(1.5*\delta_u/h_w))$ =	10.24	in <c
Required Length =	5.7	in
Required Height =	0	in

<u>ACI 318 Section 18.10.6.3</u>

0.2f'c =	0.800	ksi
fc =	0.215	ksi < 0.2f'c

Special Boundary Element not Required

Transverse Reinforcement

Lengthwise A_{sh} =	0.22	in^2
Req'd A_{sh} =	0.00	in^2
Crosswise A_{sh} =	0.22	in^2
Req'd A_{sh} =	0.00	in^2

Tie bar spacing exceeds maximum allowable of 8.00 in.

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Special Reinforced Concrete Shearwall

IBC & ACI 318

PROJECT NAME: Powder Mountain Parcel 4
LOCATION: P-02

ENGINEER:
DATE:

General Input Information

Length =	96	in.	f'_c =	4,000	psi	S_{DS} =	0.580		hw/lw =	2.00
Height =	16	ft	f_y =	60	ksi	f_1 =	0.5		β_1 =	0.85
t =	12	in.	C_d/I_E =	4.00		f_2 =	0.29			

Reinforcement

	Size	Spacing
Horizontal:	# 5	12 in. o.c.
Vertical:	# 5	12 in. o.c.
# Mats:	2	
Jamb Ties:	# 3	12 in. o.c.
Jamb Vert:	4 -	# 5

Jamb Input

Jamb Length =	12	in.
Jamb Bar Pattern:	2 Mats	
Tie Bar Cover:	1.50	in
Cover to CL Vert Bars:	2.19	in.
Jamb Bar Spacing =	7.63	in.
ρ_b =	0.0086	
Check per ACI 18.10.6.2?	N	
δ_{xe} =	0.5	in
Max # sqrt f'c =	8	8 typical (10 for a single pier)

Unfactored Loads

	D	L	S	E	Soil
P (kips)	66.31	0	48.23	0	
M (k-ft)	1.58	0.98	2.93	157.3	
V (kips)	-0.36	-0.05	-0.42	-7.67	

General Output Information

Reinforcement Check

ρ_l = 0.0043
ρ_t = 0.0043

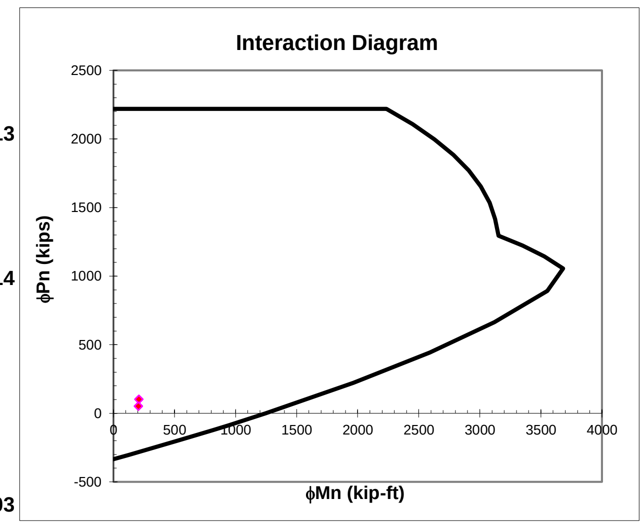
Flexural Check

Load Combination 1 - 1.32*D + 0.5*L + 0.29*S + 1.30*E + 1.00*Soil (Max Pu)

P_u =	101.3	kips		c =	10.6	in.
M_u =	208	kip-ft		a =	9.0	in.
ϕM_n =	1586	kip-ft >	Mu	**DCR =**	**0.13**	

Load Combination 2 - 0.78*D + -1.30*E + 1.00*Soil (Min Pu)

P_u =	52.0	kips		c =	9.2	in.
M_u =	203	kip-ft		a =	7.8	in.
ϕM_n =	1421	kip-ft >	Mu	**DCR =**	**0.14**	



Shear Check

V_u =	10.6	kips	V_{Mn} =	89.8	kips
Max V_n =	582.9	kips			
V_n =	443.3	kips			
ϕ =	0.75				
ϕV_n =	332.5	kips	**DCR =**	**0.03**	

Boundary Element

ACI 318 Section 18.10.6.2

δ_u =	2.00	in
δ_u/h_w =	0.010	
$l_w/(600(1.5*\delta_u/h_w))$ =	10.24	in <c
Required Length =	5.3	in
Required Height =	0	in

ACI 318 Section 18.10.6.3

0.2f'c =	0.800	ksi
fc =	0.206	ksi < 0.2f'c

Special Boundary Element not Required

Transverse Reinforcement

Lengthwise A_{sh} =	0.22	in^2
Req'd A_{sh} =	0.00	in^2
Crosswise A_{sh} =	0.22	in^2
Req'd A_{sh} =	0.00	in^2

Tie bar spacing exceeds maximum allowable of 8.00 in.

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Special Reinforced Concrete Shearwall

IBC & ACI 318

PROJECT NAME: Powder Mountain Parcel 4
LOCATION: P-03
ENGINEER:
DATE:

General Input Information

Length =	96	in.	f'_c =	4,000	psi	S_{DS} =	0.580	hw/lw =	2.00
Height =	16	ft	f_y =	60	ksi	f_1 =	0.5	β_1 =	0.85
t =	12	in.	C_d/I_E =	4.00		f_2 =	0.29		

Reinforcement

	Size	Spacing
Horizontal:	# 5	12 in. o.c.
Vertical:	# 5	12 in. o.c.
# Mats:	2	
Jamb Ties:	# 3	12 in. o.c.
Jamb Vert:	4 -	# 5

Jamb Input

Jamb Length =	12	in.
Jamb Bar Pattern:	2 Mats	
Tie Bar Cover:	1.50	in
Cover to CL Vert Bars:	2.19	in.
Jamb Bar Spacing =	7.63	in.
ρ_b =	0.0086	
Check per ACI 18.10.6.2?	N	
δ_{xe} =	0.5	in
Max # sqrt f'c =	8	8 typical (10 for a single pier)

Unfactored Loads

	D	L	S	E	Soil
P (kips)	65.62	0	47.63	0	
M (k-ft)	-1.02	0.98	0.36	157.3	
V (kips)	0.1	-0.05	0.03	-7.67	

General Output Information

Reinforcement Check

ρ_l = 0.0043
ρ_t = 0.0043

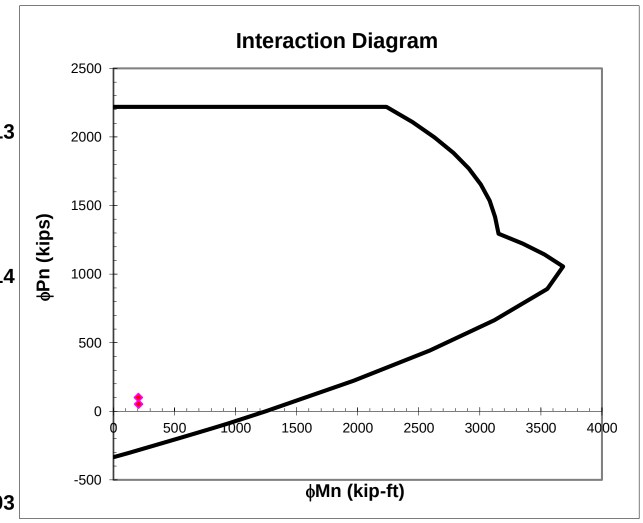
Flexural Check

Load Combination 1 - 1.32*D + 0.5*L + 0.29*S + 1.30*E + 1.00*Soil (Max Pu)

P_u =	100.2	kips		c =	10.6	in.
M_u =	204	kip-ft		a =	9.0	in.
ϕM_n =	1582	kip-ft >	Mu	**DCR =**	**0.13**	

Load Combination 2 - 0.78*D + -1.30*E + 1.00*Soil (Min Pu)

P_u =	51.4	kips		c =	9.2	in.
M_u =	205	kip-ft		a =	7.8	in.
ϕM_n =	1420	kip-ft >	Mu	**DCR =**	**0.14**	



Shear Check

V_u =	10.0	kips	V_{Mn} =	85.0	kips
Max V_n =	582.9	kips			
V_n =	443.3	kips			
ϕ =	0.75				
ϕV_n =	332.5	kips	**DCR =**	**0.03**	

Boundary Element

ACI 318 Section 18.10.6.2

δ_u =	2.00	in
δ_u/h_w =	0.010	
$l_w/(600(1.5^*\delta_u/h_w))$ =	10.24	in <c
Required Length =	5.3	in
Required Height =	0	in

ACI 318 Section 18.10.6.3

0.2f'c =	0.800	ksi
fc =	0.203	ksi < 0.2f'c

Special Boundary Element not Required

Transverse Reinforcement

Lengthwise A_{sh} =	0.22	in^2
Req'd A_{sh} =	0.00	in^2
Crosswise A_{sh} =	0.22	in^2
Req'd A_{sh} =	0.00	in^2

Tie bar spacing exceeds maximum allowable of 8.00 in.

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Special Reinforced Concrete Shearwall

IBC & ACI 318

PROJECT NAME: Powder Mountain Parcel 4
LOCATION: P-04-2

ENGINEER:
DATE:

General Input Information

Length =	65	in.	f'_c =	4,000	psi	S_{DS} =	0.580	hw/lw =	2.95
Height =	16	ft	f_y =	60	ksi	f_1 =	0.5	β_1 =	0.85
t =	12	in.	C_d/I_E =	4.00		f_2 =	0.29		

Reinforcement

	Size	Spacing
Horizontal:	# 5	12 in. o.c.
Vertical:	# 5	12 in. o.c.
# Mats:	2	
Jamb Ties:	# 3	12 in. o.c.
Jamb Vert:	4 -	# 5

Jamb Input

Jamb Length =	12	in.
Jamb Bar Pattern:	2 Mats	
Tie Bar Cover:	1.50	in
Cover to CL Vert Bars:	2.19	in.
Jamb Bar Spacing =	7.63	in.
ρ_b =	0.0086	
Check per ACI 18.10.6.2?	N	
δ_{xe} =	0.5	in
Max # sqrt f'c =	8	8 typical (10 for a single pier)

Unfactored Loads

	D	L	S	E	Soil
P (kips)	27.42	0	24.04	0	
M (k-ft)	-1.92	0.93	-0.1	150.62	
V (kips)	2.85	-0.12	3.67	-18.83	

General Output Information

Reinforcement Check

ρ_l = 0.0043
ρ_t = 0.0043

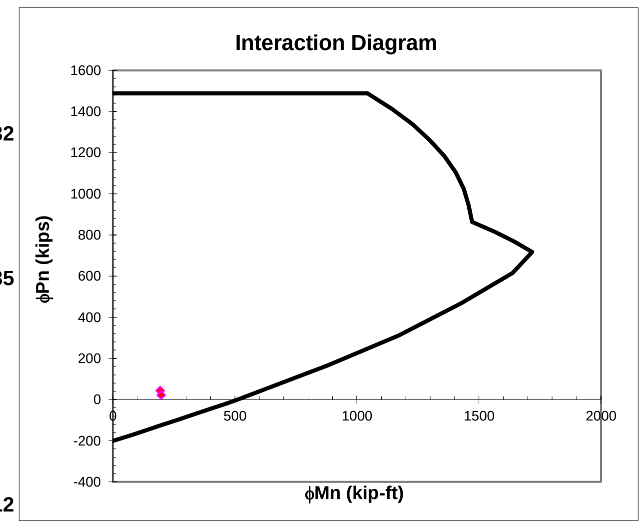
Flexural Check

Load Combination 1 - 1.32*D + 0.5*L + 0.29*S + 1.30*E + 1.00*Soil (Max Pu)

P_u =	43.1	kips		c =	5.8	in.
M_u =	194	kip-ft		a =	4.9	in.
ϕM_n =	614	kip-ft >	Mu	**DCR =**	**0.32**	

Load Combination 2 - 0.78*D + -1.30*E + 1.00*Soil (Min Pu)

P_u =	21.5	kips		c =	5.2	in.
M_u =	197	kip-ft		a =	4.4	in.
ϕM_n =	563	kip-ft >	Mu	**DCR =**	**0.35**	



Shear Check

V_u =	26.7	kips	V_{Mn} =	69.5	kips
Max V_n =	394.7	kips			
V_n =	300.2	kips			
ϕ =	0.75				
ϕV_n =	225.1	kips	**DCR =**	**0.12**	

Boundary Element

<u>ACI 318 Section 18.10.6.2</u>

δ_u =	2.00	in
δ_u/h_w =	0.010	
$l_w/(600(1.5*\delta_u/h_w))$ =	6.93	in > c
Required Length =	2.9	in
Required Height =	0	in

<u>ACI 318 Section 18.10.6.3</u>

0.2f'c =	0.800	ksi
fc =	0.280	ksi < 0.2f'c

Special Boundary Element not Required

Transverse Reinforcement

Lengthwise A_{sh} =	0.22	in^2
Req'd A_{sh} =	0.00	in^2
Crosswise A_{sh} =	0.22	in^2
Req'd A_{sh} =	0.00	in^2

Tie bar spacing exceeds maximum allowable of 8.00 in.

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Special Reinforced Concrete Shearwall

IBC & ACI 318

PROJECT NAME: Powder Mountain Parcel 4 **ENGINEER:** ____________
LOCATION: P-04 **DATE:** ____________

General Input Information

Length =	413	in.	f'_c =	4,000	psi	S_{DS} =	0.580	hw/lw =	0.46
Height =	16	ft	f_y =	60	ksi	f_1 =	0.5	β_1 =	0.85
t =	18	in.	C_d/I_E =	4.00		f_2 =	0.29		

Reinforcement

	Size	Spacing
Horizontal:	# 5	12 in. o.c.
Vertical:	# 5	12 in. o.c.
# Mats:	2	
Jamb Ties:	# 0	12 in. o.c.
Jamb Vert:	4 -	# 5

Jamb Input

Jamb Length =	12	in.
Jamb Bar Pattern:	Hooks Engaging Bars	
Tie Bar Cover:	1.50	in
Cover to CL Vert Bars:	1.81	in.
Jamb Bar Spacing =	12.00	in.
ρ_b =	0.0057	
Check per ACI 18.10.6.2?	N	
δ_{xe} =	0.5	in
Max # sqrt f'c =	8	8 typical (10 for a single pier)

Unfactored Loads

	D	L	S	E	Soil
P (kips)	189.31	-0.1	87.39	-12.46	
M (k-ft)	-186.83	2.61	-303.75	-427.21	
V (kips)	-0.57	-0.21	-0.14	10.38	

General Output Information

Reinforcement Check

ρ_l = 0.0029

ρ_t = 0.0029

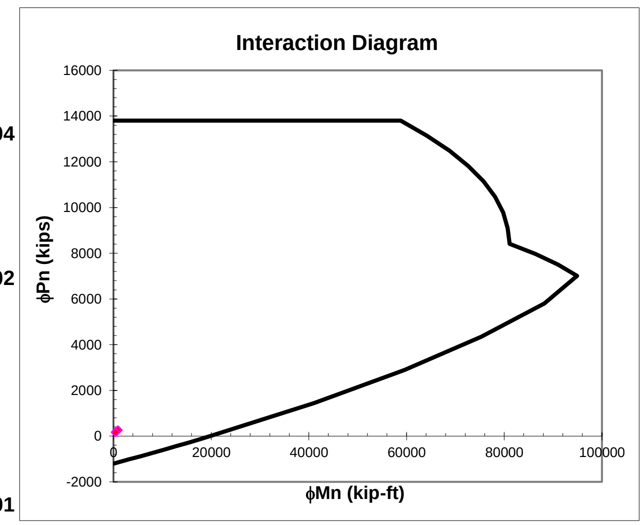
Flexural Check

Load Combination 1 - 1.32*D + 0.5*L + 0.29*S + 1.30*E + 1.00*Soil (Max Pu)

P_u =	258.2	kips	c =	27.4	in.
M_u =	888	kip-ft	a =	23.3	in.
ϕM_n =	23819	kip-ft > Mu	**DCR =**	**0.04**	

Load Combination 2 - 0.78*D + -1.30*E + 1.00*Soil (Min Pu)

P_u =	164.6	kips	c =	25.5	in.
M_u =	409	kip-ft	a =	21.7	in.
ϕM_n =	22379	kip-ft > Mu	**DCR =**	**0.02**	



Shear Check

V_u =	13.9	kips	V_{Mn} =	375.5	kips
Max V_n =	3761.3	kips			
V_n =	2690.8	kips			
ϕ =	0.75				
ϕV_n =	2018.1	kips	**DCR =**	**0.01**	

Boundary Element

ACI 318 Section 18.10.6.2

δ_u =	2.00	in
δ_u/h_w =	0.010	
$l_w/(600(1.5*\delta_u/h_w))$ =	44.05	in > c
Required Length =	0.0	in
Required Height =	0	in

ACI 318 Section 18.10.6.3

0.2f'c =	0.800	ksi
fc =	0.055	ksi < 0.2f'c

Special Boundary Element not Required

Transverse Reinforcement

Lengthwise A_{sh} =	0.00	in^2
Req'd A_{sh} =	0.00	in^2
Crosswise A_{sh} =	0.00	in^2
Req'd A_{sh} =	0.00	in^2

Tie bar spacing exceeds maximum allowable of 8.00 in.

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Special Reinforced Concrete Shearwall

IBC & ACI 318

PROJECT NAME: Powder Mountain Parcel 4 **ENGINEER:**
LOCATION: P-05 **DATE:**

General Input Information

Length =	1237	in.	f'_c =	4,000	psi	S_{DS} =	0.580		hw/lw =	0.16
Height =	16	ft	f_y =	60	ksi	f_1 =	0.5		β_1 =	0.85
t =	18	in.	C_d/I_E =	4.00		f_2 =	0.29			

Reinforcement

	Size	Spacing
Horizontal:	# 5	12 in. o.c.
Vertical:	# 5	12 in. o.c.
# Mats:	2	
Jamb Ties:	# 0	12 in. o.c.
Jamb Vert:	4 -	# 5

Jamb Input

Jamb Length =	12	in.
Jamb Bar Pattern:	Hooks Engaging Bars	
Tie Bar Cover:	1.50	in
Cover to CL Vert Bars:	1.81	in.
Jamb Bar Spacing =	12.00	in.
ρ_b =	0.0057	
Check per ACI 18.10.6.2?	N	
δ_{xe} =	0.5	in
Max # sqrt f'c =	8	8 typical (10 for a single pier)

Unfactored Loads

	D	L	S	E	Soil
P (kips)	672.85	25.59	289.41	56.59	
M (k-ft)	750.32	608.51	776.62	2882.33	
V (kips)	12.99	6.03	19.61	-206.53	

General Output Information

Reinforcement Check

ρ_l = 0.0029

ρ_t = 0.0029

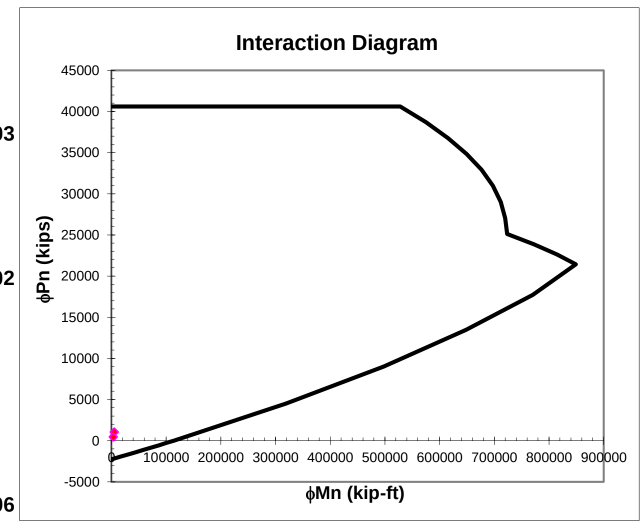
Flexural Check

Load Combination 1 - 1.32*D + 0.5*L + 0.29*S + 1.30*E + 1.00*Soil (Max Pu)

P_u =	1055.8	kips		c =	63.0	in.
M_u =	5264	kip-ft		a =	53.6	in.
ϕM_n =	164513	kip-ft >	Mu	**DCR =**	**0.03**	

Load Combination 2 - 0.78*D + -1.30*E + 1.00*Soil (Min Pu)

P_u =	453.9	kips		c =	51.5	in.
M_u =	3159	kip-ft		a =	43.8	in.
ϕM_n =	135895	kip-ft >	Mu	**DCR =**	**0.02**	



Shear Check

V_u =	278.7	kips	V_{Mn} =	8427.6	kips
Max V_n =	11265.8	kips			
V_n =	8059.4	kips			
ϕ =	0.60				
ϕV_n =	4835.6	kips	**DCR =**	**0.06**	

Boundary Element

ACI 318 Section 18.10.6.2

δ_u =	2.00	in
δ_u/h_w =	0.010	
$l_w/(600(1.5*\delta_u/h_w))$ =	131.95	in > c
Required Length =	0.0	in
Required Height =	0	in

ACI 318 Section 18.10.6.3

0.2f'c =	0.800	ksi
fc =	0.061	ksi < 0.2f'c

Special Boundary Element not Required

Transverse Reinforcement

Lengthwise A_{sh} =	0.00	in^2
Req'd A_{sh} =	0.00	in^2
Crosswise A_{sh} =	0.00	in^2
Req'd A_{sh} =	0.00	in^2

Tie bar spacing exceeds maximum allowable of 8.00 in.

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Special Reinforced Concrete Shearwall

IBC & ACI 318

PROJECT NAME: Powder Mountain Parcel 4
LOCATION: P-06
ENGINEER:
DATE:

General Input Information

Length =	367	in.	f'_c =	4,000	psi	S_{DS} =	0.580	hw/lw =	0.52
Height =	16	ft	f_y =	60	ksi	f_1 =	0.5	β_1 =	0.85
t =	18	in.	C_d/I_E =	4.00		f_2 =	0.29		

Reinforcement

	Size	Spacing
Horizontal:	# 5	12 in. o.c.
Vertical:	# 5	12 in. o.c.
# Mats:	2	
Jamb Ties:	# 0	12 in. o.c.
Jamb Vert:	4 -	# 5

Jamb Input

Jamb Length =	12	in.
Jamb Bar Pattern:	Hooks Engaging Bars	
Tie Bar Cover:	1.50	in
Cover to CL Vert Bars:	1.81	in.
Jamb Bar Spacing =	12.00	in.
ρ_b =	0.0057	
Check per ACI 18.10.6.2?	N	
δ_{xe} =	0.5	in
Max # sqrt f'c =	8	8 typical (10 for a single pier)

Unfactored Loads

	D	L	S	E	Soil
P (kips)	236.39	30.54	38.02	27.75	
M (k-ft)	117.59	18.74	187.67	-600.62	
V (kips)	6.28	0.92	4.11	105.02	

General Output Information

Reinforcement Check

ρ_l = 0.0029
ρ_t = 0.0029

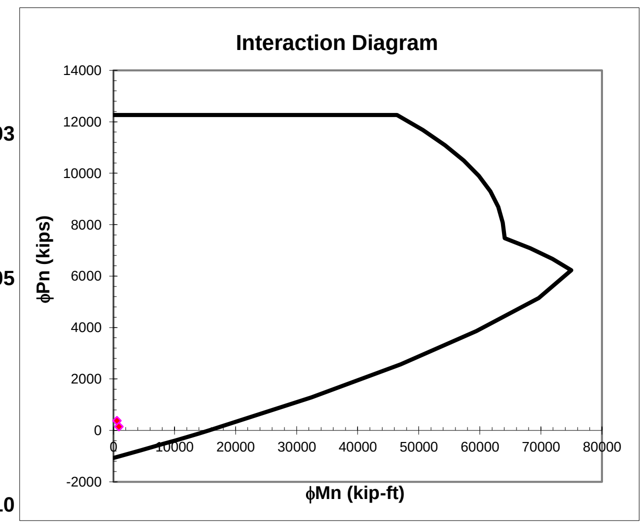
Flexural Check

Load Combination 1 - 1.32*D + 0.5*L + 0.29*S + 1.30*E + 1.00*Soil (Max Pu)

P_u =	373.5	kips		c =	27.0	in.
M_u =	562	kip-ft		a =	23.0	in.
ϕM_n =	20763	kip-ft >	Mu	**DCR =**	**0.03**	

Load Combination 2 - 0.78*D + -1.30*E + 1.00*Soil (Min Pu)

P_u =	149.3	kips		c =	22.8	in.
M_u =	873	kip-ft		a =	19.3	in.
ϕM_n =	17725	kip-ft >	Mu	**DCR =**	**0.05**	



Shear Check

V_u =	146.4	kips	V_{Mn} =	6008.5	kips
Max V_n =	3342.4	kips			
V_n =	2391.1	kips			
ϕ =	0.60				
ϕV_n =	1434.7	kips	**DCR =**	**0.10**	

Boundary Element

ACI 318 Section 18.10.6.2

δ_u =	2.00	in
δ_u/h_w =	0.010	
$l_w/(600(1.5*\delta_u/h_w))$ =	39.15	in > c
Required Length =	0.0	in
Required Height =	0	in

ACI 318 Section 18.10.6.3

0.2f'c =	0.800	ksi
fc =	0.073	ksi < 0.2f'c

Special Boundary Element not Required

Transverse Reinforcement

Lengthwise A_{sh} =	0.00	in^2
Req'd A_{sh} =	0.00	in^2
Crosswise A_{sh} =	0.00	in^2
Req'd A_{sh} =	0.00	in^2

Tie bar spacing exceeds maximum allowable of 8.00 in.

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Special Reinforced Concrete Shearwall

IBC & ACI 318

PROJECT NAME: Powder Mountain Parcel 4
LOCATION: P-07

ENGINEER:
DATE:

General Input Information

Length =	682	in.	f'_c =	4,000	psi	S_{DS} =	0.580		hw/lw =	0.28
Height =	16	ft	f_y =	60	ksi	f_1 =	0.5		β_1 =	0.85
t =	18	in.	C_d/I_E =	4.00		f_2 =	0.29			

Reinforcement

	Size	Spacing
Horizontal:	# 5	12 in. o.c.
Vertical:	# 5	12 in. o.c.
# Mats:	2	
Jamb Ties:	# 0	12 in. o.c.
Jamb Vert:	4 -	# 5

Jamb Input

Jamb Length =	12	in.
Jamb Bar Pattern:	Hooks Engaging Bars	
Tie Bar Cover:	1.50	in
Cover to CL Vert Bars:	1.81	in.
Jamb Bar Spacing =	12.00	in.
ρ_b =	0.0057	

Unfactored Loads

	D	L	S	E	Soil
P (kips)	437.6	50.32	77	-79.64	
M (k-ft)	-539.13	-130.14	-435.76	4030.8	
V (kips)	-4.33	1.51	-8.1	-388.94	

Check per ACI 18.10.6.2?	N	
δ_{xe} =	0.5	in
Max # sqrt f'c =	8	8 typical (10 for a single pier)

General Output Information

Reinforcement Check

ρ_l = 0.0029
ρ_t = 0.0029

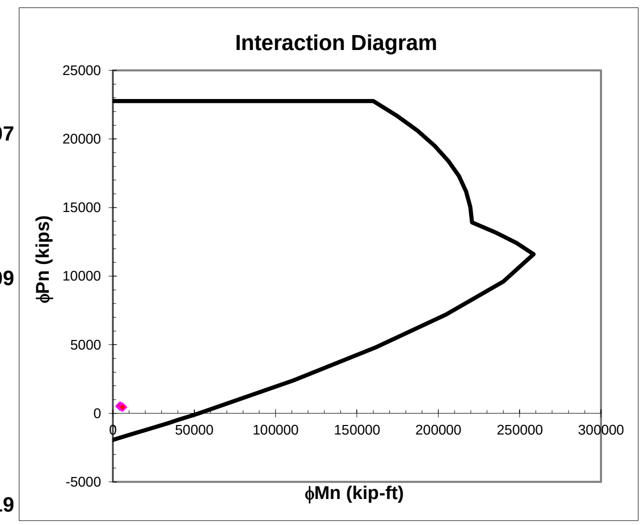
Flexural Check

Load Combination 1 - 1.32*D + 0.5*L + 0.29*S + 1.30*E + 1.00*Soil (Max Pu)

P_u =	519.8	kips		c =	46.4	in.
M_u =	4339	kip-ft		a =	39.4	in.
ϕM_n =	66022	kip-ft >	Mu	**DCR =**	**0.07**	

Load Combination 2 - 0.78*D + -1.30*E + 1.00*Soil (Min Pu)

P_u =	446.6	kips		c =	45.0	in.
M_u =	5663	kip-ft		a =	38.3	in.
ϕM_n =	64176	kip-ft >	Mu	**DCR =**	**0.09**	



Shear Check

V_u =	512.9	kips	V_{Mn} =	8671.4	kips
Max V_n =	6211.2	kips			
V_n =	4443.4	kips			
ϕ =	0.60				
ϕV_n =	2666.0	kips	**DCR =**	**0.19**	

Boundary Element

ACI 318 Section 18.10.6.2

δ_u =	2.00	in
δ_u/h_w =	0.010	
$l_w/(600(1.5*\delta_u/h_w))$ =	72.75	in > c
Required Length =	0.0	in
Required Height =	0	in

ACI 318 Section 18.10.6.3

0.2f'c =	0.800	ksi
fc =	0.084	ksi < 0.2f'c

Special Boundary Element not Required

Transverse Reinforcement

Lengthwise A_{sh} =	0.00	in^2
Req'd A_{sh} =	0.00	in^2
Crosswise A_{sh} =	0.00	in^2
Req'd A_{sh} =	0.00	in^2

Tie bar spacing exceeds maximum allowable of 8.00 in.

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Special Reinforced Concrete Shearwall

IBC & ACI 318

PROJECT NAME: Powder Mountain Parcel 4 **ENGINEER:**
LOCATION: P-08 **DATE:**

General Input Information

Length =	282	in.	f'_c =	4,000	psi	S_{DS} =	0.580	hw/lw =	0.68
Height =	16	ft	f_y =	60	ksi	f_1 =	0.5	β_1 =	0.85
t =	18	in.	C_d/I_E =	4.00		f_2 =	0.29		

Reinforcement

	Size	Spacing
Horizontal:	# 5	12 in. o.c.
Vertical:	# 5	12 in. o.c.
# Mats:	2	
Jamb Ties:	# 3	12 in. o.c.
Jamb Vert:	4 -	# 5

Jamb Input

Jamb Length =	12	in.
Jamb Bar Pattern:	2 Mats	
Tie Bar Cover:	1.50	in
Cover to CL Vert Bars:	2.19	in.
Jamb Bar Spacing =	7.63	in.
ρ_b =	0.0057	

Unfactored Loads

	D	L	S	E	Soil
P (kips)	246.32	54.41	135.12	-70.58	
M (k-ft)	-500.07	-154.78	-561.13	1435.48	
V (kips)	-6.9	-4.88	-24	-109.99	

Check per ACI 18.10.6.2?	N	
δ_{xe} =	0.5	in
Max # sqrt f'c =	8	8 typical (10 for a single pier)

General Output Information

Reinforcement Check

ρ_l = 0.0029

ρ_t = 0.0029

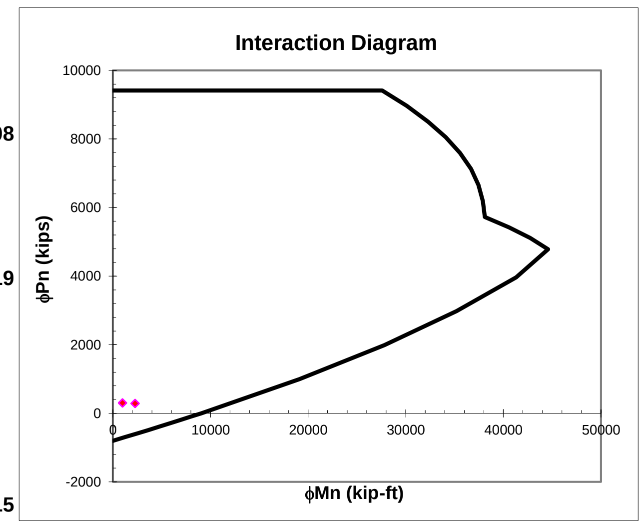
Flexural Check

Load Combination 1 - 1.32*D + 0.5*L + 0.29*S + 1.30*E + 1.00*Soil (Max Pu)

P_u =	298.8	kips	c =	20.1	in.
M_u =	968	kip-ft	a =	17.1	in.
ϕM_n =	12207	kip-ft > Mu	**DCR =**	**0.08**	

Load Combination 2 - 0.78*D + -1.30*E + 1.00*Soil (Min Pu)

P_u =	284.9	kips	c =	19.8	in.
M_u =	2258	kip-ft	a =	16.9	in.
ϕM_n =	12063	kip-ft > Mu	**DCR =**	**0.19**	



Shear Check

V_u =	161.5	kips	V_{Mn} =	2262.7	kips
Max V_n =	2568.3	kips			
V_n =	1837.3	kips			
ϕ =	0.60				
ϕV_n =	1102.4	kips	**DCR =**	**0.15**	

Boundary Element

ACI 318 Section 18.10.6.2

δ_u =	2.00	in
δ_u/h_w =	0.010	
$l_w/(600(1.5*\delta_u/h_w))$ =	30.08	in > c
Required Length =	0.0	in
Required Height =	0	in

ACI 318 Section 18.10.6.3

0.2f'c =	0.800	ksi
fc =	0.165	ksi < 0.2f'c

Special Boundary Element not Required

Transverse Reinforcement

Lengthwise A_{sh} =	0.22	in^2
Req'd A_{sh} =	0.00	in^2
Crosswise A_{sh} =	0.22	in^2
Req'd A_{sh} =	0.00	in^2

Tie bar spacing exceeds maximum allowable of 8.00 in.

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Special Reinforced Concrete Shearwall

IBC & ACI 318

PROJECT NAME: Powder Mountain Parcel 4 **ENGINEER:**

LOCATION: P-09 **DATE:**

General Input Information

Length =	520	in.	f'_c =	4,000	psi	S_{DS} =	0.580		hw/lw =	0.37
Height =	16	ft	f_y =	60	ksi	f_1 =	0.5		β_1 =	0.85
t =	12	in.	C_d/I_E =	4.00		f_2 =	0.29			

Reinforcement

	Size	Spacing
Horizontal:	# 5	12 in. o.c.
Vertical:	# 5	12 in. o.c.
# Mats:	2	
Jamb Ties:	# 0	12 in. o.c.
Jamb Vert:	4 -	# 5

Jamb Input

Jamb Length =	12	in.
Jamb Bar Pattern:	Hooks Engaging Bars	
Tie Bar Cover:	1.50	in
Cover to CL Vert Bars:	1.81	in.
Jamb Bar Spacing =	12.00	in.
ρ_b =	0.0086	
Check per ACI 18.10.6.2?	N	
δ_{xe} =	0.5	in
Max # sqrt f'c =	8	8 typical (10 for a single pier)

Unfactored Loads

	D	L	S	E	Soil
P (kips)	410.84	24.1	234.42	-4.65	
M (k-ft)	148.27	-145.4	7.61	7215.38	
V (kips)	51.78	1.71	85.49	-232.55	

General Output Information

Reinforcement Check

ρ_l = 0.0043

ρ_t = 0.0043

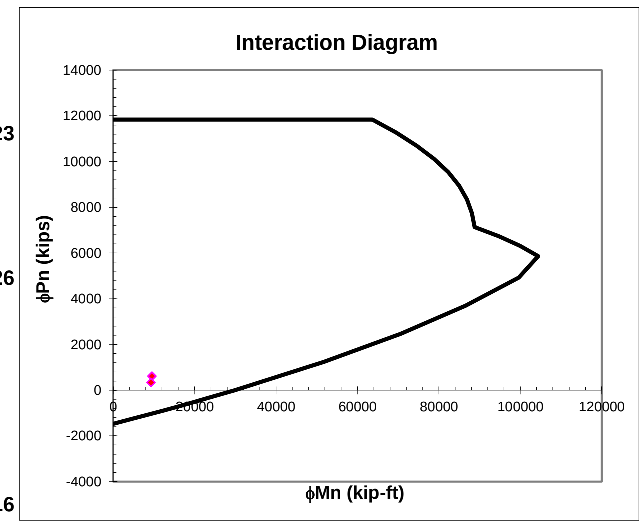
Flexural Check

Load Combination 1 - 1.32*D + 0.5*L + 0.29*S + 1.30*E + 1.00*Soil (Max Pu)

P_u =	614.7	kips		c =	55.9	in.
M_u =	9505	kip-ft		a =	47.5	in.
ϕM_n =	41156	kip-ft >	Mu	**DCR =**	**0.23**	

Load Combination 2 - 0.78*D + -1.30*E + 1.00*Soil (Min Pu)

P_u =	328.1	kips		c =	48.1	in.
M_u =	9264	kip-ft		a =	40.9	in.
ϕM_n =	36003	kip-ft >	Mu	**DCR =**	**0.26**	



Shear Check

V_u =	342.9	kips	V_{Mn} =	1003.3	kips
Max V_n =	3157.2	kips			
V_n =	2796.0	kips			
ϕ =	0.75				
ϕV_n =	2097.0	kips	**DCR =**	**0.16**	

Boundary Element

ACI 318 Section 18.10.6.2

δ_u =	2.00	in
δ_u/h_w =	0.010	
$l_w/(600(1.5*\delta_u/h_w))$ =	55.47	in <c
Required Length =	28.0	in
Required Height =	0	in

ACI 318 Section 18.10.6.3

0.2f'c =	0.800	ksi
fc =	0.305	ksi < 0.2f'c

Special Boundary Element not Required

Increase Jamb Length

Transverse Reinforcement

Lengthwise A_{sh} =	0.00	in^2
Req'd A_{sh} =	0.00	in^2
Crosswise A_{sh} =	0.00	in^2
Req'd A_{sh} =	0.00	in^2

Tie bar spacing exceeds maximum allowable of 8.00 in.

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Special Reinforced Concrete Shearwall

IBC & ACI 318

PROJECT NAME: Powder Mountain Parcel 4
LOCATION: P-10
ENGINEER:
DATE:

General Input Information

Length = 231 in. | f'_c = 4,000 psi | S_{DS} = 0.580 | hw/lw = 0.83
Height = 16 ft | f_y = 60 ksi | f_1 = 0.5 | β_1 = 0.85
t = 12 in. | C_d/I_E = 4.00 | f_2 = 0.29

Reinforcement

	Size	Spacing
Horizontal:	# 5	12 in. o.c.
Vertical:	# 5	12 in. o.c.
# Mats:	2	
Jamb Ties:	# 3	12 in. o.c.
Jamb Vert:	4 -	# 5

Jamb Input

Jamb Length = 12 in.
Jamb Bar Pattern: 2 Mats
Tie Bar Cover: 1.50 in
Cover to CL Vert Bars: 2.19 in.
Jamb Bar Spacing = 7.63 in.
ρ_b = 0.0086

Unfactored Loads

	D	L	S	E	Soil
P (kips)	80.12	23.2	0	0	
M (k-ft)	-56.09	-21.66	3.51	575.45	
V (kips)	0.17	-1.7	-0.17	-28.07	

Check per ACI 18.10.6.2? N
δ_{xe} = 0.5 in
Max # sqrt f'c = 8 — 8 typical (10 for a single pier)

General Output Information

Reinforcement Check

ρ_l = 0.0043
ρ_t = 0.0043

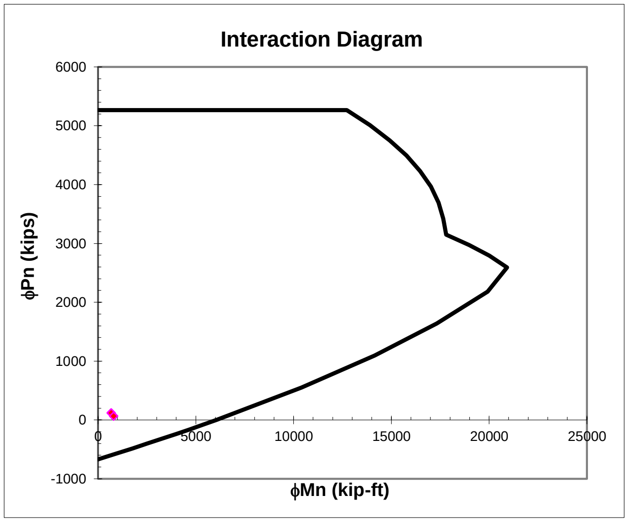
Flexural Check

Load Combination 1 - 1.32*D + 0.5*L + 0.29*S + 1.30*E + 1.00*Soil (Max Pu)

P_u = 117.0 kips | c = 20.1 in.
M_u = 664 kip-ft | a = 17.0 in.
ϕM_n = 7040 kip-ft > Mu | **DCR = 0.09**

Load Combination 2 - 0.78*D + -1.30*E + 1.00*Soil (Min Pu)

P_u = 62.8 kips | c = 18.5 in.
M_u = 792 kip-ft | a = 15.8 in.
ϕM_n = 6589 kip-ft > Mu | **DCR = 0.12**



Shear Check

V_u = 37.2 kips | V_{Mn} = 437.5 kips
Max V_n = 1402.5 kips

V_n = 1242.1 kips
ϕ = 0.75
ϕV_n = 931.5 kips | **DCR = 0.04**

Boundary Element

<u>ACI 318 Section 18.10.6.2</u>

δ_u = 2.00 in
δ_u/h_w = 0.010
$l_w/(600(1.5*\delta_u/h_w))$ = 24.64 in > c
Required Length = 10.0 in
Required Height = 0 in

<u>ACI 318 Section 18.10.6.3</u>

0.2f'c = 0.800 ksi
fc = 0.113 ksi < 0.2f'c

Special Boundary Element not Required

Transverse Reinforcement

Lengthwise A_{sh} = 0.22 in^2
Req'd A_{sh} = 0.00 in^2
Crosswise A_{sh} = 0.22 in^2
Req'd A_{sh} = 0.00 in^2

Tie bar spacing exceeds maximum allowable of 8.00 in.

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Special Reinforced Concrete Shearwall

IBC & ACI 318

PROJECT NAME: Powder Mountain Parcel 4 **ENGINEER:**
LOCATION: P-11 **DATE:**

General Input Information

Length =	219	in.	f'_c =	4,000	psi	S_{DS} =	0.580	hw/lw =	0.88
Height =	16	ft	f_y =	60	ksi	f_1 =	0.5	β_1 =	0.85
t =	12	in.	C_d/I_E =	4.00		f_2 =	0.29		

Reinforcement

	Size	Spacing
Horizontal:	# 5	12 in. o.c.
Vertical:	# 5	12 in. o.c.
# Mats:	2	
Jamb Ties:	# 3	12 in. o.c.
Jamb Vert:	4 -	# 5

Jamb Input

Jamb Length =	12	in.
Jamb Bar Pattern:	2 Mats	
Tie Bar Cover:	1.50	in
Cover to CL Vert Bars:	2.19	in.
Jamb Bar Spacing =	7.63	in.
ρ_b =	0.0086	

Unfactored Loads

	D	L	S	E	Soil
P (kips)	177.68	0	173.41	0	
M (k-ft)	-5.74	5.74	5.06	1194.12	
V (kips)	1.37	-0.28	1.49	-58.25	

Check per ACI 18.10.6.2?	N	
δ_{xe} =	0.5	in
Max # sqrt f'c =	8	8 typical (10 for a single pier)

General Output Information

Reinforcement Check

ρ_l = 0.0043
ρ_t = 0.0043

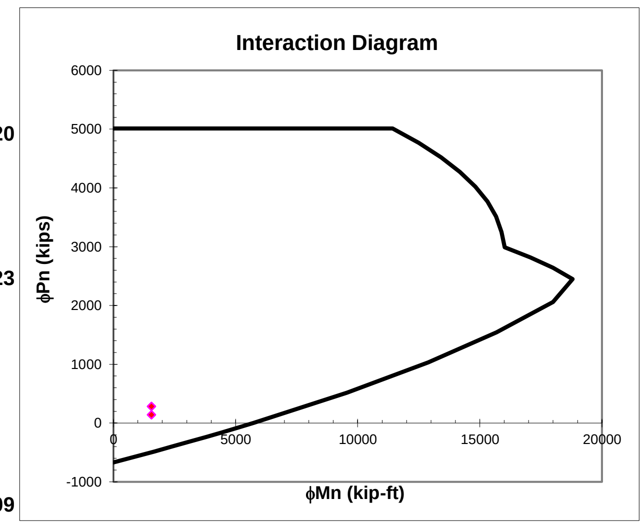
Flexural Check

Load Combination 1 - 1.32*D + 0.5*L + 0.29*S + 1.30*E + 1.00*Soil (Max Pu)

P_u =	284.1	kips		c =	24.5	in.
M_u =	1549	kip-ft		a =	20.8	in.
ϕM_n =	7905	kip-ft	> Mu	**DCR =**	**0.20**	

Load Combination 2 - 0.78*D + -1.30*E + 1.00*Soil (Min Pu)

P_u =	139.3	kips		c =	20.6	in.
M_u =	1557	kip-ft		a =	17.5	in.
ϕM_n =	6816	kip-ft	> Mu	**DCR =**	**0.23**	



Shear Check

V_u =	76.8	kips	V_{Mn} =	417.4	kips
Max V_n =	1329.7	kips			
V_n =	1177.5	kips			
ϕ =	0.75				
ϕV_n =	883.1	kips	**DCR =**	**0.09**	

Boundary Element

ACI 318 Section 18.10.6.2

δ_u =	2.00	in
δ_u/h_w =	0.010	
$l_w/(600(1.5*\delta_u/h_w))$ =	23.36	in <c
Required Length =	12.3	in
Required Height =	0	in

ACI 318 Section 18.10.6.3

0.2f'c =	0.800	ksi
fc =	0.291	ksi < 0.2f'c

Special Boundary Element not Required
Increase Jamb Length

Transverse Reinforcement

Lengthwise A_{sh} =	0.22	in^2
Req'd A_{sh} =	0.00	in^2
Crosswise A_{sh} =	0.22	in^2
Req'd A_{sh} =	0.00	in^2

Tie bar spacing exceeds maximum allowable of 8.00 in.

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Special Reinforced Concrete Shearwall

IBC & ACI 318

PROJECT NAME: Powder Mountain Parcel 4
LOCATION: P-12-1
ENGINEER:
DATE:

General Input Information

Length =	228	in.	f'_c =	4,000	psi	S_{DS} =	0.580	hw/lw =	0.84
Height =	16	ft	f_y =	60	ksi	f_1 =	0.5	β_1 =	0.85
t =	8	in.	C_d/I_E =	4.00		f_2 =	0.29		

Reinforcement

	Size	Spacing
Horizontal:	# 5	12 in. o.c.
Vertical:	# 5	12 in. o.c.
# Mats:	1	
Jamb Ties:	# 0	12 in. o.c.
Jamb Vert:	2 -	# 5

Jamb Input

Jamb Length =	12	in.
Jamb Bar Pattern:	Hooks Engaging Bars	
Tie Bar Cover:	1.50	in
Cover to CL Vert Bars:	1.81	in.
Jamb Bar Spacing =	12.00	in.
ρ_b =	0.0065	
Check per ACI 18.10.6.2?	N	
δ_{xe} =	0.5	in
Max # sqrt f'c =	8	8 typical (10 for a single pier)

Unfactored Loads

	D	L	S	E	Soil
P (kips)	119.64	21.18	38.66	24.04	
M (k-ft)	-14.14	-5.28	-6.77	371.22	
V (kips)	-3.81	-1.04	-2.64	-44.01	

General Output Information

Reinforcement Check

ρ_l = 0.0032
ρ_t = 0.0032

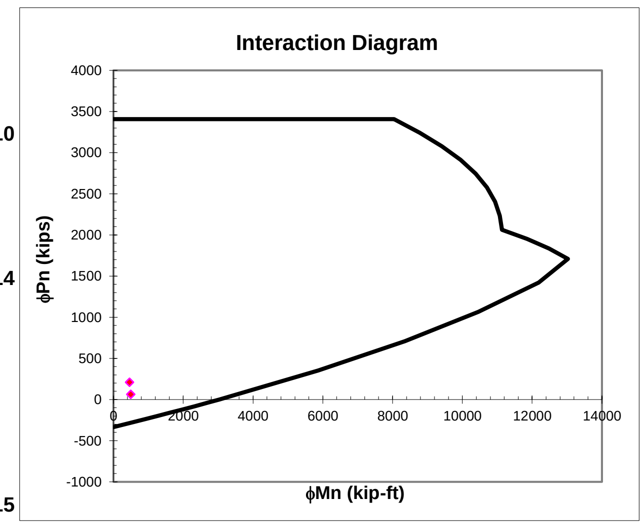
Flexural Check

Load Combination 1 - 1.32*D + 0.5*L + 0.29*S + 1.30*E + 1.00*Soil (Max Pu)

P_u =	210.5	kips	c =	22.5	in.
M_u =	459	kip-ft	a =	19.1	in.
ϕM_n =	4765	kip-ft > Mu	**DCR =**	**0.10**	

Load Combination 2 - 0.78*D + -1.30*E + 1.00*Soil (Min Pu)

P_u =	62.5	kips	c =	16.2	in.
M_u =	494	kip-ft	a =	13.8	in.
ϕM_n =	3558	kip-ft > Mu	**DCR =**	**0.14**	



Shear Check

V_u =	63.5	kips	V_{Mn} =	732.0	kips
Max V_n =	922.9	kips			
V_n =	699.5	kips			
ϕ =	0.60				
ϕV_n =	419.7	kips	**DCR =**	**0.15**	

Boundary Element

ACI 318 Section 18.10.6.2

δ_u =	2.00	in
δ_u/h_w =	0.010	
$l_w/(600(1.5*\delta_u/h_w))$ =	24.32	in > c
Required Length =	0.0	in
Required Height =	0	in

ACI 318 Section 18.10.6.3

0.2f'c =	0.800	ksi
fc =	0.191	ksi < 0.2f'c

Special Boundary Element not Required

Transverse Reinforcement

Lengthwise A_{sh} =	0.00	in^2
Req'd A_{sh} =	0.00	in^2
Crosswise A_{sh} =	0.00	in^2
Req'd A_{sh} =	0.00	in^2

Tie bar spacing exceeds maximum allowable of 8.00 in.

REAVELEY ENGINEERS + ASSOCIATES
Consulting Structural Engineers

Special Reinforced Concrete Shearwall

IBC & ACI 318

PROJECT NAME: Powder Mountain Parcel 4
LOCATION: P-12-2

ENGINEER:
DATE:

General Input Information

Length = 228 in. f'_c = 4,000 psi S_{DS} = 0.580   hw/lw = 0.84
Height = 16 ft f_y = 60 ksi f_1 = 0.5 β_1 = 0.85
t = 8 in. C_d/I_E = 4.00 f_2 = 0.29

Reinforcement

	Size	Spacing
Horizontal:	# 5	12 in. o.c.
Vertical:	# 5	12 in. o.c.
# Mats:	1	
Jamb Ties:	# 0	12 in. o.c.
Jamb Vert:	2 -	# 5

Jamb Input

Jamb Length = 12 in.
Jamb Bar Pattern: Hooks Engaging Bars
Tie Bar Cover: 1.50 in
Cover to CL Vert Bars: 1.81 in.
Jamb Bar Spacing = 12.00 in.
ρ_b = 0.0065

Unfactored Loads

	D	L	S	E	Soil
P (kips)	110.59	19.54	28.31	-24.04	
M (k-ft)	-17.11	-7.02	-8.96	373.92	
V (kips)	-2.24	-0.38	-1.6	-39.85	

Check per ACI 18.10.6.2? N
δ_{xe} = 0.5 in
Max # sqrt f'c = 8   8 typical (10 for a single pier)

General Output Information

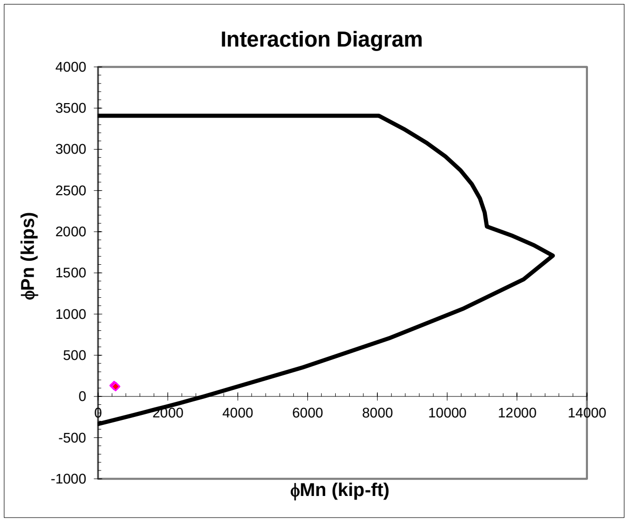
Reinforcement Check

ρ_l = 0.0032
ρ_t = 0.0032

Flexural Check

Load Combination 1 - 1.32*D + 0.5*L + 0.29*S + 1.30*E + 1.00*Soil (Max Pu)
P_u = 132.3 kips   c = 19.1 in.
M_u = 457 kip-ft   a = 16.3 in.
ϕM_n = 4133 kip-ft > Mu   **DCR = 0.11**

Load Combination 2 - 0.78*D + -1.30*E + 1.00*Soil (Min Pu)
P_u = 118.0 kips   c = 18.5 in.
M_u = 500 kip-ft   a = 15.8 in.
ϕM_n = 4016 kip-ft > Mu   **DCR = 0.12**



Shear Check

V_u = 55.4 kips V_{Mn} = 556.2 kips
Max V_n = 922.9 kips

V_n = 699.5 kips
ϕ = 0.75
ϕV_n = 524.6 kips   **DCR = 0.11**

Boundary Element

ACI 318 Section 18.10.6.2
δ_u = 2.00 in
δ_u/h_w = 0.010
$l_w/(600(1.5*\delta_u/h_w))$ = 24.32 in > c
Required Length = 0.0 in
Required Height = 0 in

ACI 318 Section 18.10.6.3
0.2f'c = 0.800 ksi
fc = 0.148 ksi < 0.2f'c

Special Boundary Element not Required

Transverse Reinforcement
Lengthwise A_{sh} = 0.00 in^2
Req'd A_{sh} = 0.00 in^2
Crosswise A_{sh} = 0.00 in^2
Req'd A_{sh} = 0.00 in^2

Tie bar spacing exceeds maximum allowable of 8.00 in.

REAVELEY ENGINEERS + ASSOCIATES
Consulting Structural Engineers

Special Reinforced Concrete Shearwall

IBC & ACI 318

PROJECT NAME: Powder Mountain Parcel 4
LOCATION: P-12-3
ENGINEER:
DATE:

General Input Information

Length =	91	in.	f'_c =	4,000	psi	S_{DS} =	0.580	hw/lw =	2.11
Height =	16	ft	f_y =	60	ksi	f_1 =	0.5	β_1 =	0.85
t =	8	in.	C_d/I_E =	4.00		f_2 =	0.29		

Reinforcement

	Size	Spacing
Horizontal:	# 5	12 in. o.c.
Vertical:	# 5	12 in. o.c.
# Mats:	1	
Jamb Ties:	# 0	12 in. o.c.
Jamb Vert:	2 -	# 5

Jamb Input

Jamb Length =	12	in.
Jamb Bar Pattern:	Hooks Engaging Bars	
Tie Bar Cover:	1.50	in
Cover to CL Vert Bars:	1.81	in.
Jamb Bar Spacing =	12.00	in.
ρ_b =	0.0065	
Check per ACI 18.10.6.2?	N	
δ_{xe} =	0.5	in
Max # sqrt f'c =	8	8 typical (10 for a single pier)

Unfactored Loads

	D	L	S	E	Soil
P (kips)	49.55	9.84	15.08	-38.96	
M (k-ft)	-4.92	-1.97	-2.88	36.87	
V (kips)	2.71	1.15	2.07	-8.58	

General Output Information

Reinforcement Check

ρ_l = 0.0032

ρ_t = 0.0032

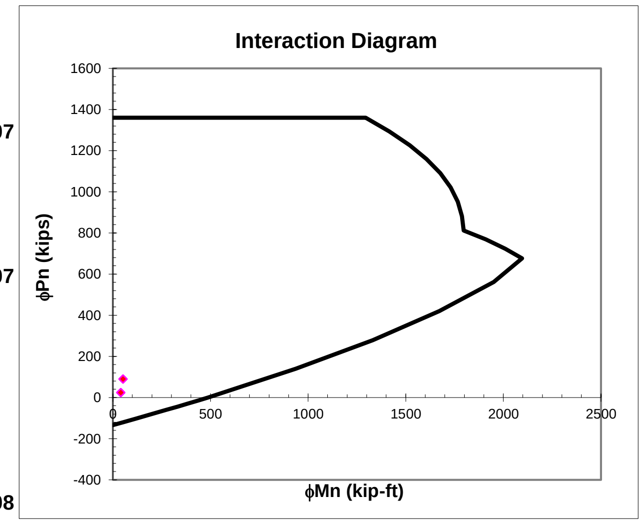
Flexural Check

Load Combination 2 - 0.78*D + -1.30*E + 1.00*Soil (Max Pu)

P_u =	89.5	kips	c =	8.9	in.
M_u =	52	kip-ft	a =	7.6	in.
ϕM_n =	783	kip-ft > Mu	**DCR =**	**0.07**	

Load Combination 1 - 1.32*D + 0.5*L + 0.29*S + 1.30*E + 1.00*Soil (Min Pu)

P_u =	23.9	kips	c =	6.0	in.
M_u =	40	kip-ft	a =	5.1	in.
ϕM_n =	568	kip-ft > Mu	**DCR =**	**0.07**	



Shear Check

V_u =	13.3	kips	V_{Mn} =	102.1	kips
Max V_n =	368.3	kips			
V_n =	233.1	kips			
ϕ =	0.75				
ϕV_n =	174.9	kips	**DCR =**	**0.08**	

Boundary Element

ACI 318 Section 18.10.6.2

δ_u =	2.00	in
δ_u/h_w =	0.010	
$l_w/(600(1.5*\delta_u/h_w))$ =	9.71	in > c
Required Length =	0.0	in
Required Height =	0	in

ACI 318 Section 18.10.6.3

0.2f'c =	0.800	ksi
fc =	0.172	ksi < 0.2f'c

Special Boundary Element not Required

Transverse Reinforcement

Lengthwise A_{sh} =	0.00	in^2
Req'd A_{sh} =	0.00	in^2
Crosswise A_{sh} =	0.00	in^2
Req'd A_{sh} =	0.00	in^2

Tie bar spacing exceeds maximum allowable of 8.00 in.

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Consulting Structural Engineers

Special Reinforced Concrete Shearwall

IBC & ACI 318

PROJECT NAME: Powder Mountain Parcel 4
LOCATION: P-12-4

ENGINEER:
DATE:

General Input Information

Length =	91	in.	f'_c =	4,000	psi	S_{DS} =	0.580	hw/lw =	2.11
Height =	16	ft	f_y =	60	ksi	f_1 =	0.5	β_1 =	0.85
t =	8	in.	C_d/I_E =	4.00		f_2 =	0.29		

Reinforcement

	Size	Spacing
Horizontal:	# 5	12 in. o.c.
Vertical:	# 5	12 in. o.c.
# Mats:	1	
Jamb Ties:	# 0	12 in. o.c.
Jamb Vert:	2 -	# 5

Jamb Input

Jamb Length =	12	in.
Jamb Bar Pattern:	Hooks Engaging Bars	
Tie Bar Cover:	1.50	in
Cover to CL Vert Bars:	1.81	in.
Jamb Bar Spacing =	12.00	in.
ρ_b =	0.0065	
Check per ACI 18.10.6.2?	N	
δ_{xe} =	0.5	in
Max # sqrt f'c =	8	8 typical (10 for a single pier)

Unfactored Loads

	D	L	S	E	Soil
P (kips)	44.57	7.97	12.37	39.58	
M (k-ft)	-0.9	-0.4	-0.19	31.49	
V (kips)	1.15	0.55	1.04	-6.42	

General Output Information

Reinforcement Check

ρ_l = 0.0032

ρ_t = 0.0032

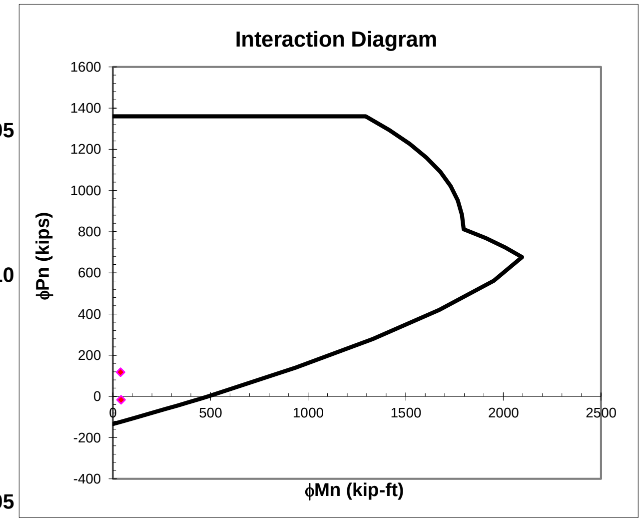
Flexural Check

Load Combination 1 - 1.32*D + 0.5*L + 0.29*S + 1.30*E + 1.00*Soil (Max Pu)

P_u =	117.7	kips	c =	10.0	in.
M_u =	39	kip-ft	a =	8.5	in.
ϕM_n =	870	kip-ft > Mu	**DCR =**	**0.05**	

Load Combination 2 - 0.78*D + -1.30*E + 1.00*Soil (Min Pu)

P_u =	-16.5	kips	c =	4.2	in.
M_u =	42	kip-ft	a =	3.6	in.
ϕM_n =	428	kip-ft > Mu	**DCR =**	**0.10**	



Shear Check

V_u = 9.2 kips V_{Mn} = 153.2 kips

Max V_n = 368.3 kips

V_n = 233.1 kips

ϕ = 0.75

ϕV_n = 174.9 kips **DCR = 0.05**

Boundary Element

<u>ACI 318 Section 18.10.6.2</u>

δ_u =	2.00	in
δ_u/h_w =	0.010	
$l_w/(600(1.5*\delta_u/h_w))$ =	9.71	in <c
Required Length =	0.0	in
Required Height =	0	in

<u>ACI 318 Section 18.10.6.3</u>

0.2f'c = 0.800 ksi

fc = 0.199 ksi < 0.2f'c

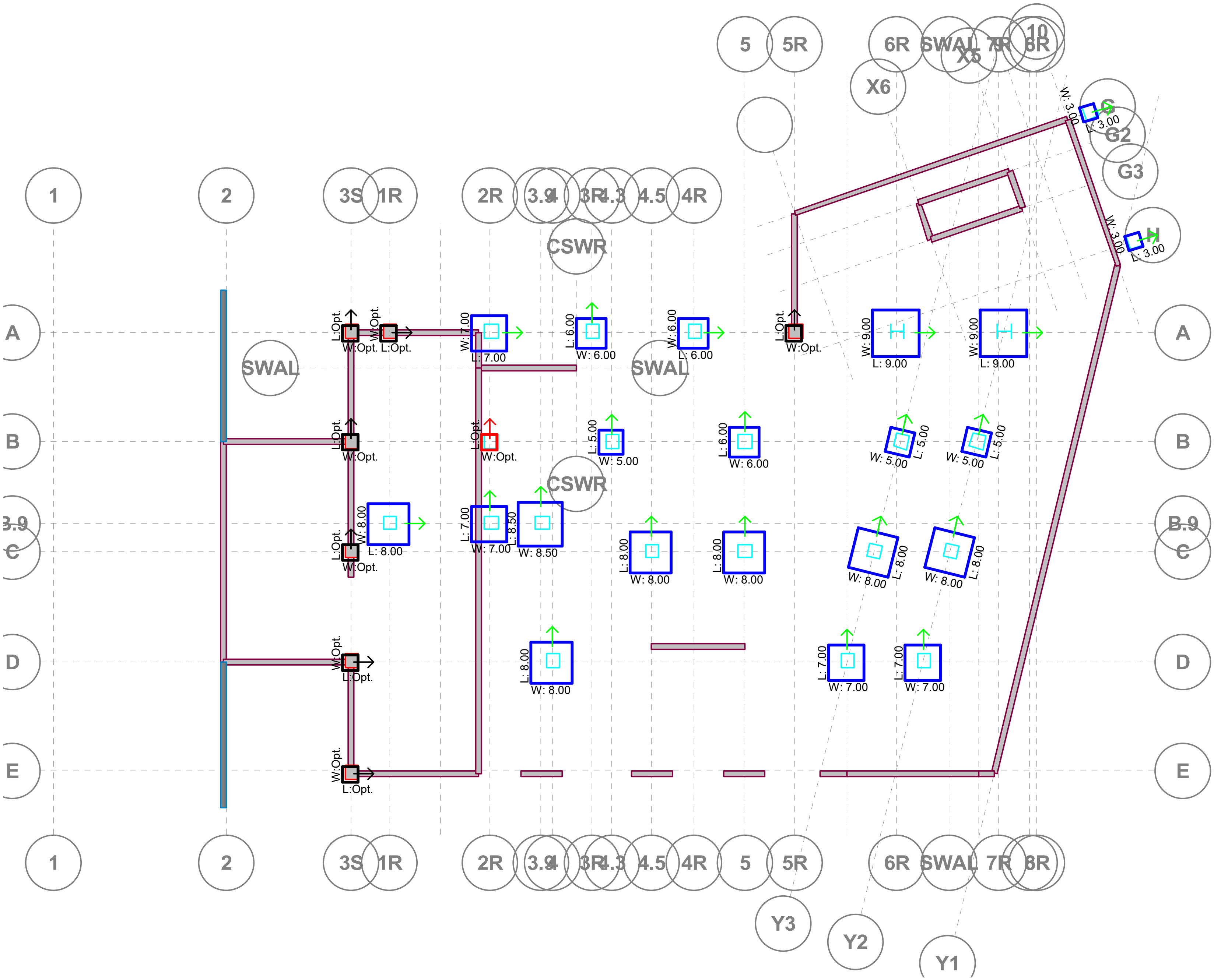
Special Boundary Element not Required

Transverse Reinforcement

Lengthwise A_{sh} =	0.00	in^2
Req'd A_{sh} =	0.00	in^2
Crosswise A_{sh} =	0.00	in^2
Req'd A_{sh} =	0.00	in^2

Tie bar spacing exceeds maximum allowable of 8.00 in.

06
FOOTINGS, PIERS, AND FOUNDATION WALLS



Calculation Sheet #________

REAVELEY ENGINEERS + ASSOCIATES
Consulting Structural Engineers

CONCRETE PIER ANALYSIS

ACI 318

Project: 2017.080 Powder Mountain Parcel 4
Designation: CP-1 SBP-2

Date: 2017-06-26
Engineer: CAB

Material Properties

f'c :	4	ksi
fy :	60	ksi
Bar#:	6	
Cover:	2.375	inches
ϕ:	0.65	

Base Plate Info

Type:	T1
B=	8
N=	13
D=	0
Anchor Ø:	0.75
Offset(y):	0

Factored Loads

Pu :	90.84	kips
Mu_x :	1	kip-ft
Mu_y :	1	kip-ft

Preliminary Calculations

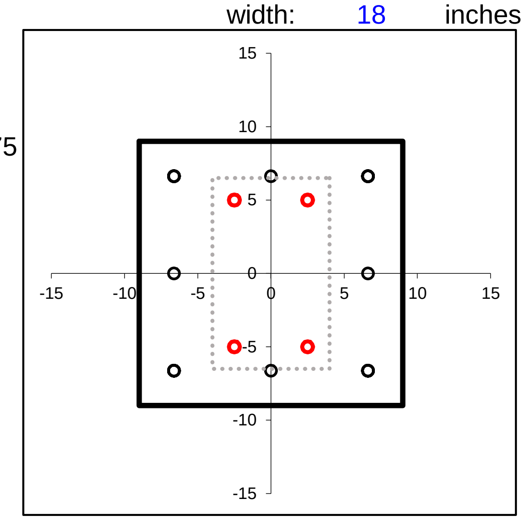
Ag =	324	in^2
Ast =	3.52	in^2
ρ =	0.010864	
β_1=	0.85	
ϕP_n =	676.4326	kips

Base Plate Size
14

Column Geometry

y
width: 18 inches

x
length: 18 inches

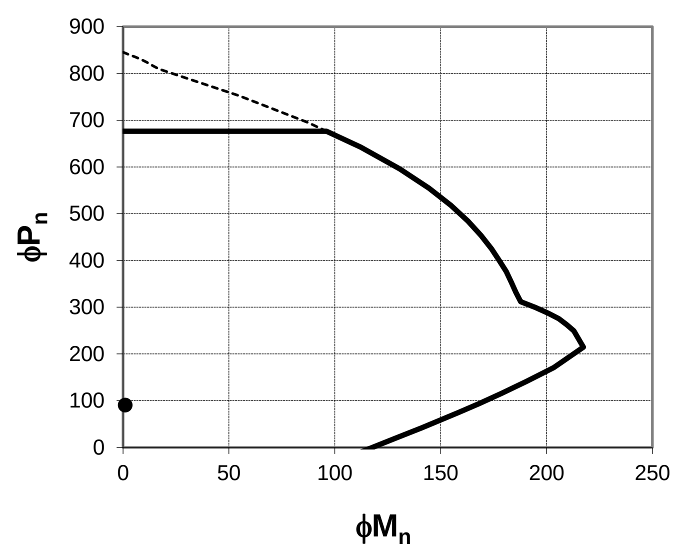


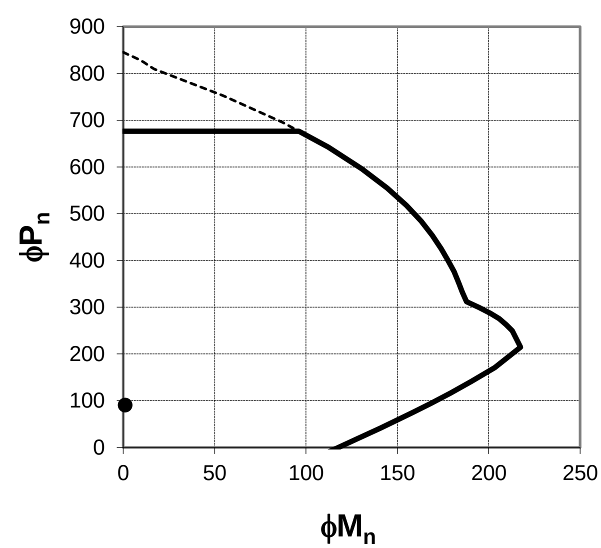
x
Bars: 3

Bars: 3
y

Minimum Tie Spacing

#3@	12	Alternate Bars along width require ties.
#4@	12	Alternate bars along length require ties.





Calculation Sheet #________

REAVELEY ENGINEERS + ASSOCIATES
Consulting Structural Engineers

CONCRETE PIER ANALYSIS

ACI 318

Project: 2017.080 Powder Mountain Parcel 4
Designation: CP-1 SBP-3

Date: 2017-06-26
Engineer: CAB

Material Properties

f'c :	4	ksi
fy :	60	ksi
Bar#:	6	
Cover:	2.375	inches
ϕ:	0.65	

Base Plate Info

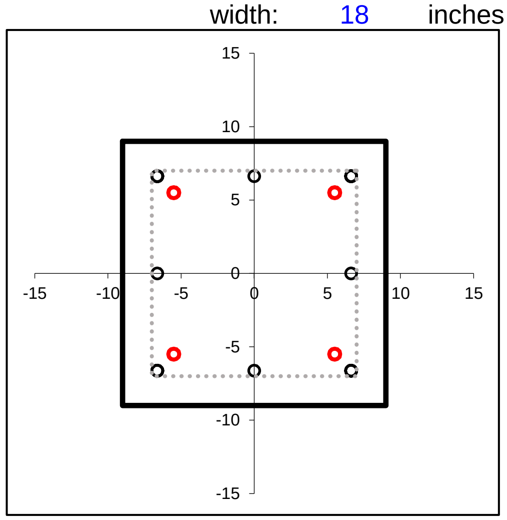
Type:	T1
B=	14
N=	14
D=	0
Anchor Ø:	0.75
Offset(y):	0

Factored Loads

Pu :	361.65	kips
Mu_x :	1	kip-ft
Mu_y :	1	kip-ft

Column Geometry

y
width: 18 inches



x
length: 18 inches

x
Bars: 3

Bars: 3
y

Preliminary Calculations

Ag =	324	in^2
Ast =	3.52	in^2
ρ =	0.010864	
β_1=	0.85	
ϕP_n =	676.4326	kips

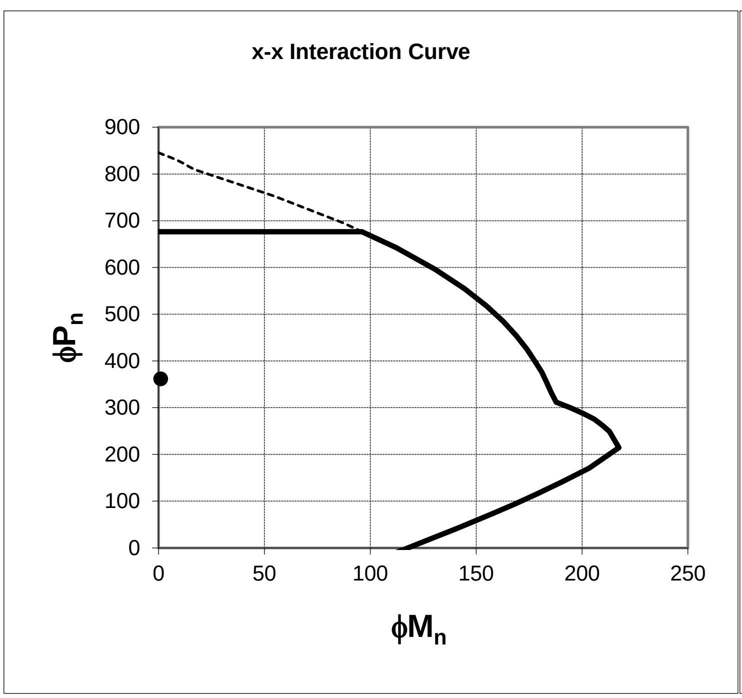
Minimum Tie Spacing

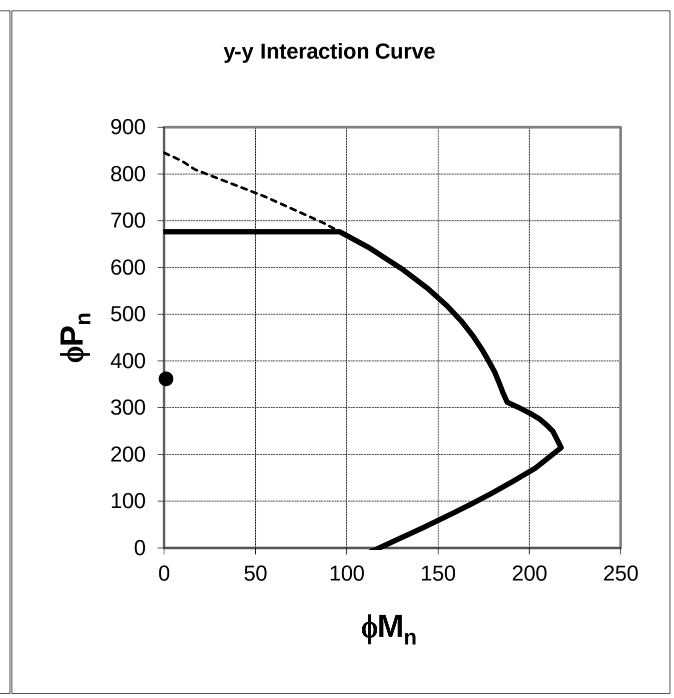
#3@	12
#4@	12

Alternate Bars along width require ties.
Alternate bars along length require ties.

Base Plate Size

14





Calculation Sheet #________

REAVELEY ENGINEERS + ASSOCIATES
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CONCRETE PIER ANALYSIS

ACI 318

Project: 2017.080 Powder Mountain Parcel 4
Designation: CP-1 SBP-4

Date: 2017-06-26
Engineer: CAB

Material Properties

f'c :	4	ksi
fy :	60	ksi
Bar#:	6	
Cover:	2.375	inches
ϕ:	0.65	

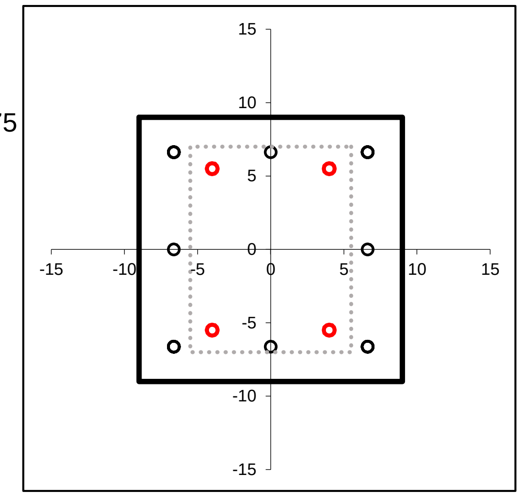
Base Plate Info

Type:	T1
B=	11
N=	14
D=	0
Anchor Ø:	0.75
Offset(y):	0

Column Geometry

y

width: 18 inches



x

Bars: 3

length: 18 inches

x

Bars: 3

y

Factored Loads

Pu :	289.64	kips
Mu_x :	1	kip-ft
Mu_y :	1	kip-ft

Preliminary Calculations

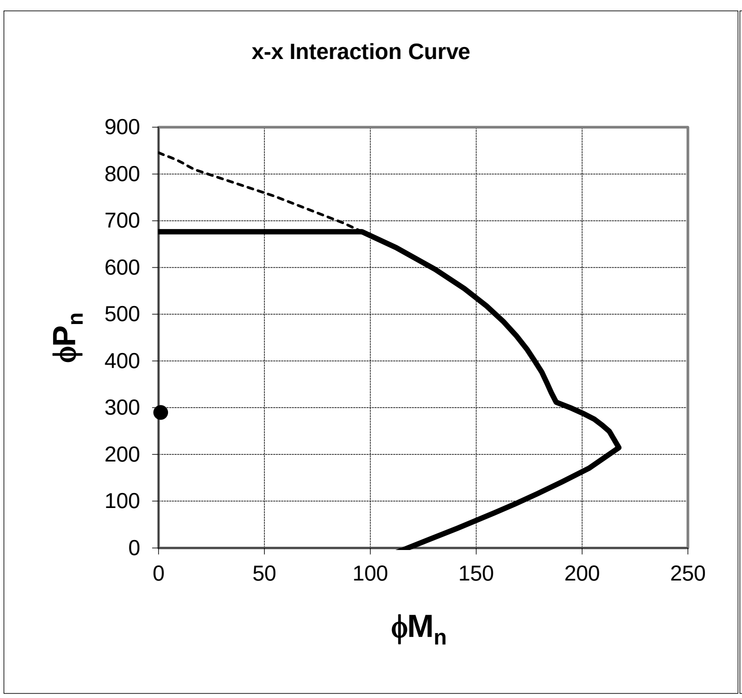
Ag =	324	in^2
Ast =	3.52	in^2
ρ =	0.010864	
β_1=	0.85	
ϕP_n =	676.4326	kips

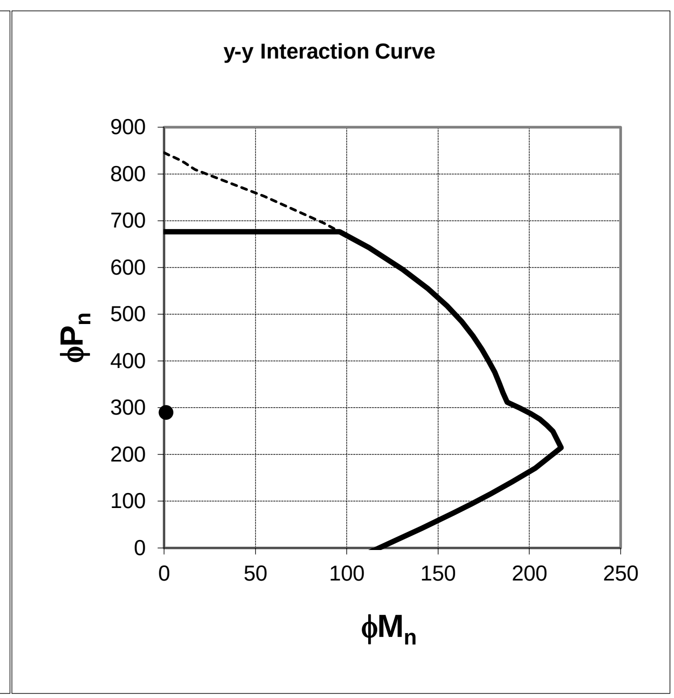
Minimum Tie Spacing

#3@ 12 Alternate Bars along width require ties.
#4@ 12 Alternate bars along length require ties.

Base Plate Size

14





Calculation Sheet #________

REAVELEY
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Consulting Structural Engineers

CONCRETE PIER ANALYSIS

ACI 318

Project: 2017.080 Powder Mountain Parcel 4
)esignation: CP-2 SBP-6

Date: 2017-06-26
Engineer: CAB

Material Properties

f'c :	4	ksi
fy :	60	ksi
Bar#:	8	
Cover:	2.5	inches
ϕ:	0.65	

Base Plate Info

Type:	T1
B=	15
N=	18
D=	0
Anchor Ø:	0.75
Offset(y):	0

Factored Loads

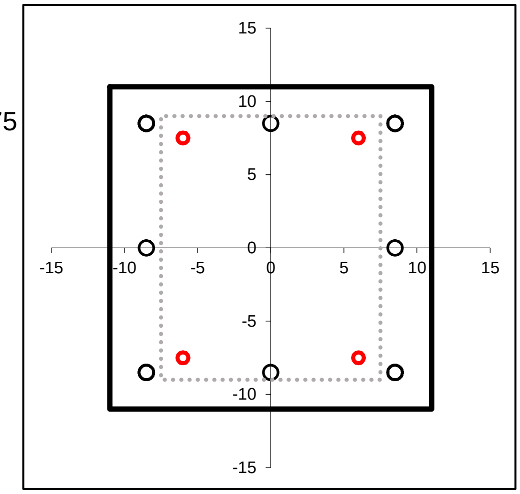
Pu :	397.3	kips
Mu_x :	1	kip-ft
Mu_y :	1	kip-ft

Column Geometry

y
width: 22 inches

x
length: 22 inches

x
Bars: 3



Bars: 3
y

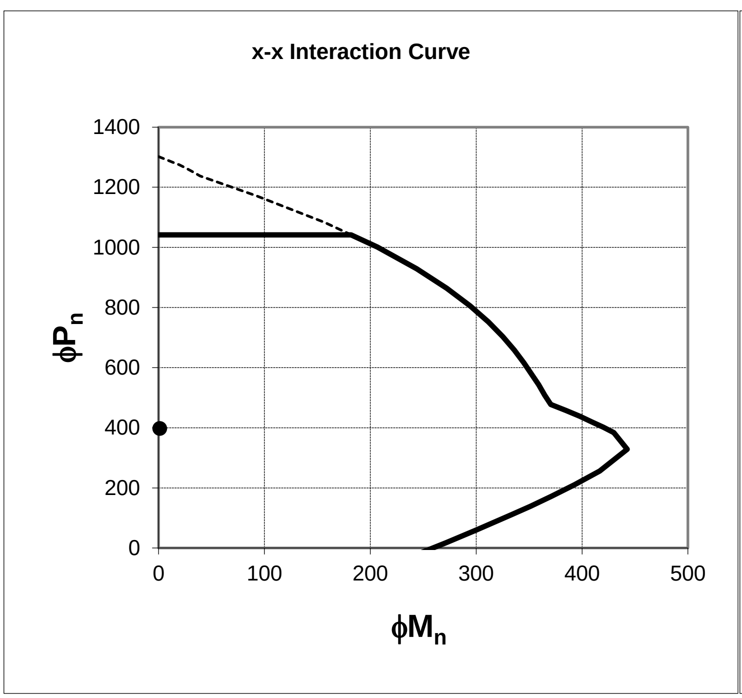
Preliminary Calculations

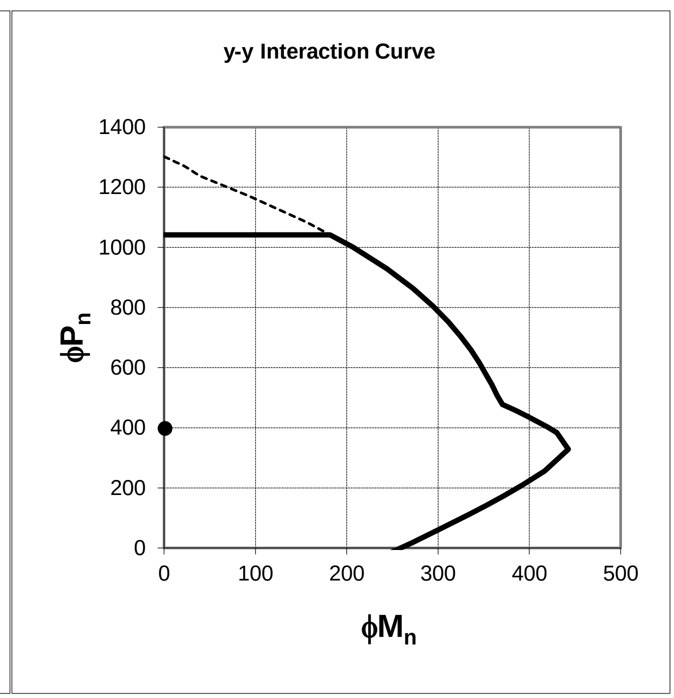
Ag =	484	in^2
Ast =	6.32	in^2
ρ =	0.013058	
β_1=	0.85	
ϕP_n =	1041.722	kips

Minimum Tie Spacing

#3@ 16 All bars along width require ties.
#4@ 16 All bars along length require ties.

Base Plate Size
14





REAVELEY ENGINEERS + ASSOCIATES
Consulting Structural Engineers

project	Powder Mountain Parcel 4
	Retaining Walls
location	Eden, Utah
date	6/26/2017
by	CW

RETAINING WALL ANALYSIS & DESIGN - POOL WALL (ACI318/MSJC)

RETAINING WALL ANALYSIS

In accordance with International Building Code 2015

Tedds calculation version 2.9.00

Retaining wall details

Stem type	Cantilever with inclined front face
Stem height	h_{stem} = **27** ft
Stem thickness	t_{stem} = **30** in
Slope length to front of stem	l_{slf} = **6** in
Angle to rear face of stem	α = **90** deg
Angle to front face of stem	α_f = **88.9** deg
Stem density	γ_{stem} = **150** pcf
Toe length	l_{toe} = **2** ft
Heel length	l_{heel} = **28** ft
Base thickness	t_{base} = **30** in
Base density	γ_{base} = **150** pcf
Height of retained soil	h_{ret} = **21.5** ft
Angle of soil surface	β = **0** deg
Depth of cover	d_{cover} = **2** ft
Depth of excavation	d_{exc} = **2** ft
Height of water	h_{water} = **12** ft
Water density	γ_w = **62** pcf

Retained soil properties

Soil type	Medium dense well graded sand
Moist density	γ_{mr} = **135** pcf
Saturated density	γ_{sr} = **145** pcf

Base soil properties

Soil type	Medium dense well graded sand
Soil density	γ_b = **115** pcf
Allowable bearing pressure	$P_{bearing}$ = **4600** psf

Loading details

Live surcharge load	$Surcharge_L$ = **350** psf

REAVELEY
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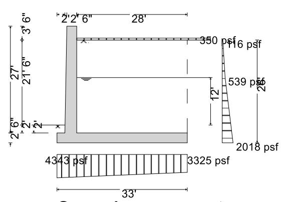
Consulting Structural Engineers

project Powder Mountain Parcel 4

Retaining Walls

location Eden, Utah

date 6/26/2017 by CW



General arrangement

Calculate retaining wall geometry

Base length	$l_{base} = l_{toe} + l_{slf} + t_{stem} + l_{heel} =$ **33** ft
Saturated soil height	$h_{sat} = h_{water} + d_{cover} =$ **14** ft
Moist soil height	$h_{moist} = h_{ret} - h_{water} =$ **9.5** ft
Length of surcharge load	$l_{sur} = l_{heel} =$ **28** ft
- Distance to vertical component	$x_{sur\_v} = l_{base} - l_{heel} / 2 =$ **19** ft
Effective height of wall	$h_{eff} = h_{base} + d_{cover} + h_{ret} =$ **26** ft
- Distance to horizontal component	$x_{sur\_h} = h_{eff} / 2 =$ **13** ft
Area of wall stem	$A_{stem} = h_{stem} \times (t_{stem} + l_{slf} / 2) =$ **74.25** ft^2
- Distance to vertical component	$x_{stem} = (h_{stem} \times t_{stem} \times (l_{toe} + l_{slf} + t_{stem} / 2) + h_{stem} \times l_{slf} / 2 \times (l_{toe} + 2 \times l_{slf} / 3)) / A_{stem} =$ **3.621** ft
Area of wall base	$A_{base} = l_{base} \times t_{base} =$ **82.5** ft^2
- Distance to vertical component	$x_{base} = l_{base} / 2 =$ **16.5** ft
Area of saturated soil	$A_{sat} = h_{sat} \times l_{heel} =$ **392** ft^2
- Distance to vertical component	$x_{sat\_v} = l_{base} - (h_{sat} \times l_{heel}^2 / 2) / A_{sat} =$ **19** ft
- Distance to horizontal component	$x_{sat\_h} = (h_{sat} + h_{base}) / 3 =$ **5.5** ft
Area of water	$A_{water} = h_{sat} \times l_{heel} =$ **392** ft^2
- Distance to vertical component	$x_{water\_v} = l_{base} - (h_{sat} \times l_{heel}^2 / 2) / A_{sat} =$ **19** ft
- Distance to horizontal component	$x_{water\_h} = (h_{sat} + h_{base}) / 3 =$ **5.5** ft
Area of moist soil	$A_{moist} = h_{moist} \times l_{heel} =$ **266** ft^2
- Distance to vertical component	$x_{moist\_v} = l_{base} - (h_{moist} \times l_{heel}^2 / 2) / A_{moist} =$ **19** ft

REAVELEY ENGINEERS + ASSOCIATES
Consulting Structural Engineers

project Powder Mountain Parcel 4
Retaining Walls
location Eden, Utah
date 6/26/2017 by CW

- Distance to horizontal component	$x_{moist\_h} = (h_{moist} \times (t_{base} + h_{sat} + h_{moist} / 3) / 2 + (h_{sat} + t_{base})^2/2) / (h_{sat} + t_{base} + h_{moist} / 2) =$ **10.802** ft
Area of base soil	$A_{pass} = d_{cover} \times (l_{toe} + l_{slf} \times d_{cover} / (2 \times h_{stem})) =$ **4.037** ft^2
- Distance to vertical component	$x_{pass\_v} = l_{base} - (d_{cover} \times l_{toe} \times (l_{base} - l_{toe} / 2) + l_{slf} \times d_{cover}^2 / (2 \times h_{stem}) \times (l_{base} - l_{toe} - l_{slf} \times d_{cover} / (3 \times h_{stem}))) / A_{pass} =$ **1.009** ft
- Distance to horizontal component	$x_{pass\_h} = (d_{cover} + h_{base}) / 3 =$ **1.5** ft
Soil coefficients	
Coefficient of friction to back of wall	$K_{fr} =$ **0.450**
Coefficient of friction to front of wall	$K_{fb} =$ **0.450**
Coefficient of friction beneath base	$K_{fbb} =$ **0.450**
Active pressure coefficient	$K_A =$ **0.330**
Passive pressure coefficient	$K_P =$ **3.000**
From IBC 2015 cl.1807.2.3 Safety factor	
Load combination 1	1.0 × Dead + 1.0 × Live + 1.0 × Lateral earth
Sliding check	
Vertical forces on wall	
Wall stem	$F_{stem} = A_{stem} \times \gamma_{stem} =$ **11138** plf
Wall base	$F_{base} = A_{base} \times \gamma_{base} =$ **12375** plf
Saturated retained soil	$F_{sat\_v} = A_{sat} \times (\gamma_{sr} - \gamma_w) =$ **32379** plf
Water	$F_{water\_v} = A_{water} \times \gamma_w =$ **24461** plf
Moist retained soil	$F_{moist\_v} = A_{moist} \times \gamma_{mr} =$ **35910** plf
Total	$F_{total\_v} = F_{stem} + F_{base} + F_{sat\_v} + F_{moist\_v} + F_{water\_v} =$ **116263** plf
Horizontal forces on wall	
Surcharge load	$F_{sur\_h} = K_A \times Surcharge_L \times h_{eff} =$ **3003** plf
Saturated retained soil	$F_{sat\_h} = K_A \times (\gamma_{sr} - \gamma_w) \times (h_{sat} + h_{base})^2 / 2 =$ **3710** plf
Water	$F_{water\_h} = \gamma_w \times (h_{water} + d_{cover} + h_{base})^2 / 2 =$ **8494** plf
Moist retained soil	$F_{moist\_h} = K_A \times \gamma_{mr} \times ((h_{eff} - h_{sat} - h_{base})^2 / 2 + (h_{eff} - h_{sat} - h_{base}) \times (h_{sat} + h_{base})) =$ **8994** plf
Total	$F_{total\_h} = F_{sat\_h} + F_{moist\_h} + F_{water\_h} + F_{sur\_h} =$ **24201** plf
Check stability against sliding	
Base soil resistance	$F_{exc\_h} = K_P \times \gamma_b \times (h_{pass} + h_{base})^2 / 2 =$ **1078** plf
Base friction	$F_{friction} = F_{total\_v} \times K_{fbb} =$ **52318** plf
Resistance to sliding	$F_{rest} = F_{exc\_h} + F_{friction} =$ **53396** plf
Factor of safety	$FoS_{sl} = F_{rest} / F_{total\_h} =$ **2.206** > 1.5
	PASS - Factor of safety against sliding is adequate
Overturning check	
Vertical forces on wall	
Wall stem	$F_{stem} = A_{stem} \times \gamma_{stem} =$ **11138** plf
Wall base	$F_{base} = A_{base} \times \gamma_{base} =$ **12375** plf
Saturated retained soil	$F_{sat\_v} = A_{sat} \times (\gamma_{sr} - \gamma_w) =$ **32379** plf
Water	$F_{water\_v} = A_{water} \times \gamma_w =$ **24461** plf
Moist retained soil	$F_{moist\_v} = A_{moist} \times \gamma_{mr} =$ **35910** plf

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Total $F_{total\_v} = F_{stem} + F_{base} + F_{sat\_v} + F_{moist\_v} + F_{water\_v} =$ **116263** plf

Horizontal forces on wall

Surcharge load $F_{sur\_h} = K_A \times Surcharge_L \times h_{eff} =$ **3003** plf

Saturated retained soil $F_{sat\_h} = K_A \times (\gamma_{sr} - \gamma_w) \times (h_{sat} + h_{base})^2 / 2 =$ **3710** plf

Water $F_{water\_h} = \gamma_w \times (h_{water} + d_{cover} + h_{base})^2 / 2 =$ **8494** plf

Moist retained soil $F_{moist\_h} = K_A \times \gamma_{mr} \times ((h_{eff} - h_{sat} - h_{base})^2 / 2 + (h_{eff} - h_{sat} - h_{base}) \times (h_{sat} + h_{base})) =$ **8994** plf

Base soil $F_{exc\_h} = -K_P \times \gamma_b \times (h_{pass} + h_{base})^2 / 2 =$ **-1078** plf

Total $F_{total\_h} = F_{sat\_h} + F_{moist\_h} + F_{exc\_h} + F_{water\_h} + F_{sur\_h} =$ **23123** plf

Overturning moments on wall

Surcharge load $M_{sur\_OT} = F_{sur\_h} \times x_{sur\_h} =$ **39039** lb_ft/ft

Saturated retained soil $M_{sat\_OT} = F_{sat\_h} \times x_{sat\_h} =$ **20408** lb_ft/ft

Water $M_{water\_OT} = F_{water\_h} \times x_{water\_h} =$ **46718** lb_ft/ft

Moist retained soil $M_{moist\_OT} = F_{moist\_h} \times x_{moist\_h} =$ **97148** lb_ft/ft

Total $M_{total\_OT} = M_{sat\_OT} + M_{moist\_OT} + M_{water\_OT} + M_{sur\_OT} =$ **203313** lb_ft/ft

Restoring moments on wall

Wall stem $M_{stem\_R} = F_{stem} \times x_{stem} =$ **40331** lb_ft/ft

Wall base $M_{base\_R} = F_{base} \times x_{base} =$ **204187** lb_ft/ft

Saturated retained soil $M_{sat\_R} = F_{sat\_v} \times x_{sat\_v} =$ **615205** lb_ft/ft

Water $M_{water\_R} = F_{water\_v} \times x_{water\_v} =$ **464755** lb_ft/ft

Moist retained soil $M_{moist\_R} = F_{moist\_v} \times x_{moist\_v} =$ **682290** lb_ft/ft

Base soil $M_{exc\_R} = -F_{exc\_h} \times x_{exc\_h} =$ **898** lb_ft/ft

Total $M_{total\_R} = M_{stem\_R} + M_{base\_R} + M_{sat\_R} + M_{moist\_R} + M_{exc\_R} + M_{water\_R} =$ **2007667** lb_ft/ft

Check stability against overturning

Factor of safety $FoS_{ot} = M_{total\_R} / M_{total\_OT} =$ **9.875** > 1.5

PASS - Factor of safety against overturning is adequate

Bearing pressure check

Vertical forces on wall

Wall stem $F_{stem} = A_{stem} \times \gamma_{stem} =$ **11138** plf

Wall base $F_{base} = A_{base} \times \gamma_{base} =$ **12375** plf

Surcharge load $F_{sur\_v} = Surcharge_L \times l_{heel} =$ **9800** plf

Saturated retained soil $F_{sat\_v} = A_{sat} \times (\gamma_{sr} - \gamma_w) =$ **32379** plf

Water $F_{water\_v} = A_{water} \times \gamma_w =$ **24461** plf

Moist retained soil $F_{moist\_v} = A_{moist} \times \gamma_{mr} =$ **35910** plf

Base soil $F_{pass\_v} = A_{pass} \times \gamma_b =$ **464** plf

Total $F_{total\_v} = F_{stem} + F_{base} + F_{sat\_v} + F_{moist\_v} + F_{pass\_v} + F_{water\_v} + F_{sur\_v} =$ **126527** plf

Horizontal forces on wall

Surcharge load $F_{sur\_h} = K_A \times Surcharge_L \times h_{eff} =$ **3003** plf

Saturated retained soil $F_{sat\_h} = K_A \times (\gamma_{sr} - \gamma_w) \times (h_{sat} + h_{base})^2 / 2 =$ **3710** plf

Water $F_{water\_h} = \gamma_w \times (h_{water} + d_{cover} + h_{base})^2 / 2 =$ **8494** plf

Moist retained soil — $F_{moist\_h} = K_A \times \gamma_{mr} \times ((h_{eff} - h_{sat} - h_{base})^2 / 2 + (h_{eff} - h_{sat} - h_{base}) \times (h_{sat} + h_{base}))$ = **8994** plf

Base soil — $F_{pass\_h} = -K_P \times \gamma_b \times (d_{cover} + h_{base})^2 / 2$ = **-3493** plf

Total — $F_{total\_h} = \max(F_{sat\_h} + F_{moist\_h} + F_{pass\_h} + F_{water\_h} + F_{sur\_h} - F_{total\_v} \times K_{fbb}, 0 \text{ plf})$ = **0** plf

Moments on wall

Wall stem — $M_{stem} = F_{stem} \times x_{stem}$ = **40331** lb_ft/ft

Wall base — $M_{base} = F_{base} \times x_{base}$ = **204187** lb_ft/ft

Surcharge load — $M_{sur} = F_{sur\_v} \times x_{sur\_v} - F_{sur\_h} \times x_{sur\_h}$ = **147161** lb_ft/ft

Saturated retained soil — $M_{sat} = F_{sat\_v} \times x_{sat\_v} - F_{sat\_h} \times x_{sat\_h}$ = **594797** lb_ft/ft

Water — $M_{water} = F_{water\_v} \times x_{water\_v} - F_{water\_h} \times x_{water\_h}$ = **418037** lb_ft/ft

Moist retained soil — $M_{moist} = F_{moist\_v} \times x_{moist\_v} - F_{moist\_h} \times x_{moist\_h}$ = **585142** lb_ft/ft

Base soil — $M_{pass} = F_{pass\_v} \times x_{pass\_v} - F_{pass\_h} \times x_{pass\_h}$ = **5708** lb_ft/ft

Total — $M_{total} = M_{stem} + M_{base} + M_{sat} + M_{moist} + M_{pass} + M_{water} + M_{sur}$ = **1995364** lb_ft/ft

Check bearing pressure

Distance to reaction — $\bar{x} = M_{total} / F_{total\_v}$ = **15.77** ft

Eccentricity of reaction — $e = \bar{x} - l_{base} / 2$ = **-0.73** ft

Loaded length of base — $l_{load} = l_{base}$ = **33** ft

Bearing pressure at toe — $q_{toe} = F_{total\_v} / l_{base} \times (1 - 6 \times e / l_{base})$ = **4343** psf

Bearing pressure at heel — $q_{heel} = F_{total\_v} / l_{base} \times (1 + 6 \times e / l_{base})$ = **3325** psf

Factor of safety — $FoS_{bp} = P_{bearing} / \max(q_{toe}, q_{heel})$ = **1.059**

PASS - Allowable bearing pressure exceeds maximum applied bearing pressure

RETAINING WALL DESIGN

In accordance with ACI 318-14

Tedds calculation version 2.9.00

Concrete details

Compressive strength of concrete — f'_c = **5000** psi

Concrete type — Normal weight

Reinforcement details

Yield strength of reinforcement — f_y = **60000** psi

Modulus of elasticity or reinforcement — E_s = **29000000** psi

Cover to reinforcement

Front face of stem — c_{sf} = **2** in

Rear face of stem — c_{sr} = **2** in

Top face of base — c_{bt} = **2** in

Bottom face of base — c_{bb} = **3** in

From IBC 2015 cl.1605.2.1 Basic load combinations

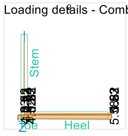
Load combination no.1 — 1.4 × Dead

Load combination no.2 — 1.2 × Dead + 1.6 × Live + 1.6 × Lateral earth

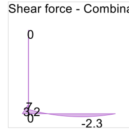
Load combination no.3 — 1.2 × Dead + 1.0 × Earthquake + 1.0 × Live + 1.6 × Lateral earth

Load combination no.4 — 0.9 × Dead + 1.0 × Earthquake + 1.6 × Lateral earth

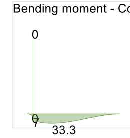
Loading details - Combination No.1 - kips/ft



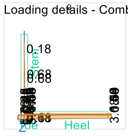
Shear force - Combination No.1 - kips/ft



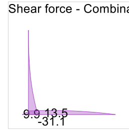
Bending moment - Combination No.1 - kips_ft/ft



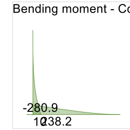
Loading details - Combination No.2 - kips/ft



Shear force - Combination No.2 - kips/ft



Bending moment - Combination No.2 - kips_ft/ft



Check stem design at base of stem

Depth of section — h = **36** in

Rectangular section in flexure - Section 22.3

Design bending moment combination 2 — M = **238212** lb_ft/ft

Depth of tension reinforcement — $d = h - c_{sr} - \phi_{sr} / 2 =$ **33.436** in

Compression reinforcement provided — No.4 bars @ 12" c/c

Area of compression reinforcement provided — $A_{sf.prov} = \pi \times \phi_{sf}^2 / (4 \times s_{sf}) =$ **0.196** in^2/ft

Tension reinforcement provided — No.9 bars @ 6" c/c

Area of tension reinforcement provided — $A_{sr.prov} = \pi \times \phi_{sr}^2 / (4 \times s_{sr}) =$ **1.999** in^2/ft

Maximum reinforcement spacing - cl.11.7.2 — $s_{max} = \min(18 \text{ in}, 3 \times h) =$ **18** in

PASS - Reinforcement is adequately spaced

Depth of compression block — $a = A_{sr.prov} \times f_y / (0.85 \times f'_c) =$ **2.351** in

Neutral axis factor - cl.22.2.2.4.3 — $\beta_1 = \min(\max(0.85 - 0.05 \times (f'_c - 4 \text{ ksi}) / 1 \text{ ksi}, 0.65), 0.85) =$ **0.8**

Depth to neutral axis — $c = a / \beta_1 =$ **2.939** in

Strain in reinforcement — $\varepsilon_t = 0.003 \times (d - c) / c =$ **0.031128**

Section is in the tension controlled zone

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Strength reduction factor ϕ_f = min(max(0.65 + (ε_t - 0.002) × (250 / 3), 0.65), 0.9) = **0.9**

Nominal flexural strength $M_n = A_{sr.prov} \times f_y \times (d - a / 2)$ = **322386** lb_ft/ft

Design flexural strength $\phi M_n = \phi_f \times M_n$ = **290148** lb_ft/ft

$M / \phi M_n$ = **0.821**

PASS - Design flexural strength exceeds factored bending moment

By iteration, reinforcement required by analysis $A_{sr.des}$ = **1.63** in^2/ft

Minimum area of reinforcement - cl.9.6.1.2 $A_{sr.min}$ = max(3 × √(f'$_c$ × 1 psi), 200 psi) × d / f_y = **1.419** in^2/ft

PASS - Area of reinforcement provided is greater than minimum area of reinforcement required

Rectangular section in shear - Section 22.5

Design shear force V = **31098** lb/ft

Concrete modification factor - cl.19.2.4 λ = **1**

Nominal concrete shear strength - eqn.22.5.5.1 $V_c = 2 \times \lambda \times \sqrt{(f'_c \times 1\ psi)} \times d$ = **56743** lb/ft

Strength reduction factor ϕ_s = **0.75**

Design concrete shear strength - cl.11.5.1.1 $\phi V_c = \phi_s \times V_c$ = **42557** lb/ft

$V / \phi V_c$ = **0.731**

PASS - No shear reinforcement is required

Check stem design at 8 ft

Depth of section h = **34.222** in

Rectangular section in flexure - Section 22.3

Design bending moment combination 2 M = **69037** lb_ft/ft

Depth of tension reinforcement $d = h - c_{sr} - \phi_{sr1} / 2$ = **31.722** in

Compression reinforcement provided No.4 bars @ 12" c/c

Area of compression reinforcement provided $A_{sf1.prov} = \pi \times \phi_{sf1}^2 / (4 \times s_{sf1})$ = **0.196** in^2/ft

Tension reinforcement provided No.8 bars @ 12" c/c

Area of tension reinforcement provided $A_{sr1.prov} = \pi \times \phi_{sr1}^2 / (4 \times s_{sr1})$ = **0.785** in^2/ft

Maximum reinforcement spacing - cl.11.7.2 s_{max} = min(18 in, 3 × h) = **18** in

PASS - Reinforcement is adequately spaced

Depth of compression block $a = A_{sr1.prov} \times f_y / (0.85 \times f'_c)$ = **0.924** in

Neutral axis factor - cl.22.2.2.4.3 β_1 = min(max(0.85 - 0.05 × (f'$_c$ - 4 ksi) / 1 ksi, 0.65), 0.85) = **0.8**

Depth to neutral axis $c = a / \beta_1$ = **1.155** in

Strain in reinforcement $\varepsilon_t = 0.003 \times (d - c) / c$ = **0.079395**

Section is in the tension controlled zone

Strength reduction factor ϕ_f = min(max(0.65 + (ε_t - 0.002) × (250 / 3), 0.65), 0.9) = **0.9**

Nominal flexural strength $M_n = A_{sr1.prov} \times f_y \times (d - a / 2)$ = **122758** lb_ft/ft

Design flexural strength $\phi M_n = \phi_f \times M_n$ = **110482** lb_ft/ft

$M / \phi M_n$ = **0.625**

PASS - Design flexural strength exceeds factored bending moment

By iteration, reinforcement required by analysis $A_{sr1.des}$ = **0.488** in^2/ft

Minimum area of reinforcement - cl.9.6.1.3 $A_{sr1.mod} = 4 \times A_{sr1.des} / 3$ = **0.651** in^2/ft

PASS - Area of reinforcement provided is greater than minimum area of reinforcement required

Rectangular section in shear - Section 22.5

Design shear force V = **12726** lb/ft

Concrete modification factor - cl.19.2.4 λ = **1**

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Nominal concrete shear strength - eqn.22.5.5.1	$V_c = 2 \times \lambda \times \sqrt{(f'_c \times 1\ psi)} \times d =$ **53834** lb/ft
Strength reduction factor	$\phi_s =$ **0.75**
Design concrete shear strength - cl.11.5.1.1	$\phi V_c = \phi_s \times V_c =$ **40376** lb/ft
	$V / \phi V_c =$ **0.315**

PASS - No shear reinforcement is required

Check stem design at 16 ft

Depth of section	h = **32.445** in

Rectangular section in flexure - Section 22.3

Design bending moment combination 2	M = **10209** lb_ft/ft
Depth of tension reinforcement	$d = h - c_{sr} - \phi_{sr2} / 2 =$ **30.07** in
Compression reinforcement provided	No.4 bars @ 12" c/c
Area of compression reinforcement provided	$A_{sf2.prov} = \pi \times \phi_{sf2}^2 / (4 \times s_{sf2}) =$ **0.196** in^2/ft
Tension reinforcement provided	No.6 bars @ 12" c/c
Area of tension reinforcement provided	$A_{sr2.prov} = \pi \times \phi_{sr2}^2 / (4 \times s_{sr2}) =$ **0.442** in^2/ft
Maximum reinforcement spacing - cl.11.7.2	$s_{max} = \min(18\ in, 3 \times h) =$ **18** in

PASS - Reinforcement is adequately spaced

Depth of compression block	$a = A_{sr2.prov} \times f_y / (0.85 \times f'_c) =$ **0.52** in
Neutral axis factor - cl.22.2.2.4.3	$\beta_1 = \min(\max(0.85 - 0.05 \times (f'_c - 4\ ksi) / 1\ ksi, 0.65), 0.85) =$ **0.8**
Depth to neutral axis	$c = a / \beta_1 =$ **0.65** in
Strain in reinforcement	$\varepsilon_t = 0.003 \times (d - c) / c =$ **0.13585**

Section is in the tension controlled zone

Strength reduction factor	$\phi_f = \min(\max(0.65 + (\varepsilon_t - 0.002) \times (250 / 3), 0.65), 0.9) =$ **0.9**
Nominal flexural strength	$M_n = A_{sr2.prov} \times f_y \times (d - a / 2) =$ **65848** lb_ft/ft
Design flexural strength	$\phi M_n = \phi_f \times M_n =$ **59263** lb_ft/ft
	$M / \phi M_n =$ **0.172**

PASS - Design flexural strength exceeds factored bending moment

By iteration, reinforcement required by analysis	$A_{sr2.des} =$ **0.076** in^2/ft
Minimum area of reinforcement - cl.9.6.1.3	$A_{sr2.mod} = 4 \times A_{sr2.des} / 3 =$ **0.101** in^2/ft

PASS - Area of reinforcement provided is greater than minimum area of reinforcement required

Rectangular section in shear - Section 22.5

Design shear force	V = **3391** lb/ft
Concrete modification factor - cl.19.2.4	$\lambda =$ **1**
Nominal concrete shear strength - eqn.22.5.5.1	$V_c = 2 \times \lambda \times \sqrt{(f'_c \times 1\ psi)} \times d =$ **51030** lb/ft
Strength reduction factor	$\phi_s =$ **0.75**
Design concrete shear strength - cl.11.5.1.1	$\phi V_c = \phi_s \times V_c =$ **38272** lb/ft
	$V / \phi V_c =$ **0.089**

PASS - No shear reinforcement is required

Horizontal reinforcement parallel to face of stem

Minimum area of reinforcement - cl.11.6.1	$A_{sx.req} = 0.0025 \times (t_{stem} + l_{slf}) =$ **1.08** in^2/ft
Transverse reinforcement provided	No.7 bars @ 13" c/c each face
Area of transverse reinforcement provided	$A_{sx.prov} = 2 \times \pi \times \phi_{sx}^2 / (4 \times s_{sx}) =$ **1.11** in^2/ft

PASS - Area of reinforcement provided is greater than area of reinforcement required

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Check base design at heel

Depth of section $h = \mathbf{30}$ in

Rectangular section in flexure - Section 22.3

Design bending moment combination 1 $M = \mathbf{33326}$ lb_ft/ft

Depth of tension reinforcement $d = h - c_{bb} - \phi_{bb} / 2 = \mathbf{26.563}$ in

Compression reinforcement provided No.10 bars @ 6" c/c

Area of compression reinforcement provided $A_{bt.prov} = \pi \times \phi_{bt}^2 / (4 \times s_{bt}) = \mathbf{2.534}$ in^2/ft

Tension reinforcement provided No.7 bars @ 10" c/c

Area of tension reinforcement provided $A_{bb.prov} = \pi \times \phi_{bb}^2 / (4 \times s_{bb}) = \mathbf{0.722}$ in^2/ft

Maximum reinforcement spacing - cl.7.7.2.3 $s_{max} = \min(18\text{ in}, 3 \times h) = \mathbf{18}$ in

PASS - Reinforcement is adequately spaced

Depth of compression block $a = A_{bb.prov} \times f_y / (0.85 \times f'_c) = \mathbf{0.849}$ in

Neutral axis factor - cl.22.2.2.4.3 $\beta_1 = \min(\max(0.85 - 0.05 \times (f'_c - 4\text{ ksi}) / 1\text{ ksi}, 0.65), 0.85) = \mathbf{0.8}$

Depth to neutral axis $c = a / \beta_1 = \mathbf{1.061}$ in

Strain in reinforcement $\varepsilon_t = 0.003 \times (d - c) / c = \mathbf{0.072095}$

Section is in the tension controlled zone

Strength reduction factor $\phi_f = \min(\max(0.65 + (\varepsilon_t - 0.002) \times (250 / 3), 0.65), 0.9) = \mathbf{0.9}$

Nominal flexural strength $M_n = A_{bb.prov} \times f_y \times (d - a / 2) = \mathbf{94304}$ lb_ft/ft

Design flexural strength $\phi M_n = \phi_f \times M_n = \mathbf{84874}$ lb_ft/ft

$M / \phi M_n = \mathbf{0.393}$

PASS - Design flexural strength exceeds factored bending moment

By iteration, reinforcement required by analysis $A_{bb.des} = \mathbf{0.281}$ in^2/ft

Minimum area of reinforcement - cl.7.6.1.1 $A_{bb.min} = 0.0018 \times h = \mathbf{0.648}$ in^2/ft

PASS - Area of reinforcement provided is greater than minimum area of reinforcement required

Rectangular section in shear - Section 22.5

Design shear force $V = \mathbf{9940}$ lb/ft

Concrete modification factor - cl.19.2.4 $\lambda = \mathbf{1}$

Nominal concrete shear strength - eqn.22.5.5.1 $V_c = 2 \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})} \times d = \mathbf{45078}$ lb/ft

Strength reduction factor $\phi_s = \mathbf{0.75}$

Design concrete shear strength - cl.7.6.3.1 $\phi V_c = \phi_s \times V_c = \mathbf{33809}$ lb/ft

$V / \phi V_c = \mathbf{0.294}$

PASS - No shear reinforcement is required

Check base design at heel

Depth of section $h = \mathbf{30}$ in

Rectangular section in flexure - Section 22.3

Design bending moment combination 2 $M = \mathbf{280902}$ lb_ft/ft

Depth of tension reinforcement $d = h - c_{bt} - \phi_{bt} / 2 = \mathbf{27.365}$ in

Compression reinforcement provided No.7 bars @ 10" c/c

Area of compression reinforcement provided $A_{bb.prov} = \pi \times \phi_{bb}^2 / (4 \times s_{bb}) = \mathbf{0.722}$ in^2/ft

Tension reinforcement provided No.10 bars @ 6" c/c

Area of tension reinforcement provided $A_{bt.prov} = \pi \times \phi_{bt}^2 / (4 \times s_{bt}) = \mathbf{2.534}$ in^2/ft

Maximum reinforcement spacing - cl.7.7.2.3 $s_{max} = \min(18\text{ in}, 3 \times h) = \mathbf{18}$ in

PASS - Reinforcement is adequately spaced

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Depth of compression block	$a = A_{bt.prov} \times f_y / (0.85 \times f'_c) =$ **2.981** in
Neutral axis factor - cl.22.2.2.4.3	$\beta_1 = \min(\max(0.85 - 0.05 \times (f'_c - 4\ ksi) / 1\ ksi, 0.65), 0.85) =$ **0.8**
Depth to neutral axis	$c = a / \beta_1 =$ **3.726** in
Strain in reinforcement	$\varepsilon_t = 0.003 \times (d - c) / c =$ **0.019034**

Section is in the tension controlled zone

Strength reduction factor	$\phi_f = \min(\max(0.65 + (\varepsilon_t - 0.002) \times (250 / 3), 0.65), 0.9) =$ **0.9**
Nominal flexural strength	$M_n = A_{bt.prov} \times f_y \times (d - a / 2) =$ **327772** lb_ft/ft
Design flexural strength	$\phi M_n = \phi_f \times M_n =$ **294995** lb_ft/ft
	$M / \phi M_n =$ **0.952**

PASS - Design flexural strength exceeds factored bending moment

By iteration, reinforcement required by analysis	$A_{bt.des} =$ **2.405** in^2/ft
Minimum area of reinforcement - cl.7.6.1.1	$A_{bt.min} = 0.0018 \times h =$ **0.648** in^2/ft

PASS - Area of reinforcement provided is greater than minimum area of reinforcement required

Rectangular section in shear - Section 22.5

Design shear force	$V =$ **13508** lb/ft
Concrete modification factor - cl.19.2.4	$\lambda =$ **1**
Nominal concrete shear strength - eqn.22.5.5.1	$V_c = 2 \times \lambda \times \sqrt{(f'_c \times 1\ psi)} \times d =$ **46440** lb/ft
Strength reduction factor	$\phi_s =$ **0.75**
Design concrete shear strength - cl.7.6.3.1	$\phi V_c = \phi_s \times V_c =$ **34830** lb/ft
	$V / \phi V_c =$ **0.388**

PASS - No shear reinforcement is required

Transverse reinforcement parallel to base

Minimum area of reinforcement - cl.76.1.1	$A_{bx.req} = 0.0018 \times t_{base} =$ **0.648** in^2/ft
Transverse reinforcement provided	No.6 bars @ 12" c/c each face
Area of transverse reinforcement provided	$A_{bx.prov} = 2 \times \pi \times \phi_{bx}^2 / (4 \times s_{bx}) =$ **0.884** in^2/ft

PASS - Area of reinforcement provided is greater than area of reinforcement required

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Retaining Walls

location Eden, Utah

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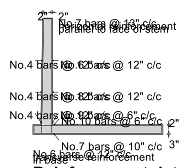

Reinforcement details

RETAINING WALL ANALYSIS & DESIGN - EAST BASEMENT WALL (ACI318/MSJC)

RETAINING WALL ANALYSIS

In accordance with International Building Code 2015

Tedds calculation version 2.9.00

Retaining wall details

Stem type	Propped cantilever pinned at the base
Stem height	h_{stem} = **21** ft
Prop height	h_{prop} = **21** ft
Stem thickness	t_{stem} = **18** in
Angle to rear face of stem	α = **90** deg
Stem density	γ_{stem} = **150** pcf
Toe length	l_{toe} = **1.75** ft
Heel length	l_{heel} = **1.75** ft
Base thickness	t_{base} = **16** in
Base density	γ_{base} = **150** pcf
Height of retained soil	h_{ret} = **20** ft
Angle of soil surface	β = **0** deg
Depth of cover	d_{cover} = **1** ft
Depth of excavation	d_{exc} = **1** ft
Height of water	h_{water} = **10** ft
Water density	γ_w = **62** pcf

Retained soil properties

Soil type	Medium dense well graded sand
Moist density	γ_{mr} = **135** pcf
Saturated density	γ_{sr} = **145** pcf

Base soil properties

Soil type	Medium dense well graded sand
Soil density	γ_b = **115** pcf
Allowable bearing pressure	$P_{bearing}$ = **4600** psf

Loading details

Live surcharge load	$Surcharge_L$ = **100** psf
Vertical line load at 0.75 ft	P_{D1} = **1000** plf
	P_{L1} = **1000** plf

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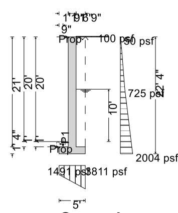
location Eden, Utah
date 6/26/2017 by CW



General arrangement

Calculate retaining wall geometry

Base length	$l_{base} = l_{toe} + t_{stem} + l_{heel} = \mathbf{5}$ ft
Saturated soil height	$h_{sat} = h_{water} + d_{cover} = \mathbf{11}$ ft
Moist soil height	$h_{moist} = h_{ret} - h_{water} = \mathbf{10}$ ft
Length of surcharge load	$l_{sur} = l_{heel} = \mathbf{1.75}$ ft
- Distance to vertical component	$x_{sur\_v} = l_{base} - l_{heel} / 2 = \mathbf{4.125}$ ft
Effective height of wall	$h_{eff} = h_{base} + d_{cover} + h_{ret} = \mathbf{22.333}$ ft
- Distance to horizontal component	$x_{sur\_h} = h_{eff} / 2 = \mathbf{11.167}$ ft
Area of wall stem	$A_{stem} = h_{stem} \times t_{stem} = \mathbf{31.5}$ ft^2
- Distance to vertical component	$x_{stem} = l_{toe} + t_{stem} / 2 = \mathbf{2.5}$ ft
Area of wall base	$A_{base} = l_{base} \times t_{base} = \mathbf{6.667}$ ft^2
- Distance to vertical component	$x_{base} = l_{base} / 2 = \mathbf{2.5}$ ft
Area of saturated soil	$A_{sat} = h_{sat} \times l_{heel} = \mathbf{19.25}$ ft^2
- Distance to vertical component	$x_{sat\_v} = l_{base} - (h_{sat} \times l_{heel}^2 / 2) / A_{sat} = \mathbf{4.125}$ ft
- Distance to horizontal component	$x_{sat\_h} = (h_{sat} + h_{base}) / 3 = \mathbf{4.111}$ ft
Area of water	$A_{water} = h_{sat} \times l_{heel} = \mathbf{19.25}$ ft^2
- Distance to vertical component	$x_{water\_v} = l_{base} - (h_{sat} \times l_{heel}^2 / 2) / A_{sat} = \mathbf{4.125}$ ft
- Distance to horizontal component	$x_{water\_h} = (h_{sat} + h_{base}) / 3 = \mathbf{4.111}$ ft
Area of moist soil	$A_{moist} = h_{moist} \times l_{heel} = \mathbf{17.5}$ ft^2
- Distance to vertical component	$x_{moist\_v} = l_{base} - (h_{moist} \times l_{heel}^2 / 2) / A_{moist} = \mathbf{4.125}$ ft

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- Distance to horizontal component	$x_{moist\_h} = (h_{moist} \times (t_{base} + h_{sat} + h_{moist} / 3) / 2 + (h_{sat} + t_{base})^2/2) / (h_{sat} + t_{base} + h_{moist} / 2)$ = **8.907** ft
Area of base soil	$A_{pass} = d_{cover} \times l_{toe}$ = **1.75** ft^2
- Distance to vertical component	$x_{pass\_v} = l_{base} - (d_{cover} \times l_{toe} \times (l_{base} - l_{toe} / 2)) / A_{pass}$ = **0.875** ft
- Distance to horizontal component	$x_{pass\_h} = (d_{cover} + h_{base}) / 3$ = **0.778** ft

Soil coefficients

Coefficient of friction to back of wall	K_{fr} = **0.450**
Coefficient of friction to front of wall	K_{fb} = **0.450**
Coefficient of friction beneath base	K_{fbb} = **0.450**
At rest pressure coefficient	K_0 = **0.500**
Passive pressure coefficient	K_P = **3.000**

From IBC 2015 cl.1807.2.3 Safety factor

Load combination 1 — 1.0 × Dead + 1.0 × Live + 1.0 × Lateral earth

Bearing pressure check

Vertical forces on wall

Wall stem	$F_{stem} = A_{stem} \times \gamma_{stem}$ = **4725** plf
Wall base	$F_{base} = A_{base} \times \gamma_{base}$ = **1000** plf
Surcharge load	$F_{sur\_v} = Surcharge_L \times l_{heel}$ = **175** plf
Line loads	$F_{P\_v} = P_{D1} + P_{L1}$ = **2000** plf
Saturated retained soil	$F_{sat\_v} = A_{sat} \times (\gamma_{sr} - \gamma_w)$ = **1590** plf
Water	$F_{water\_v} = A_{water} \times \gamma_w$ = **1201** plf
Moist retained soil	$F_{moist\_v} = A_{moist} \times \gamma_{mr}$ = **2362** plf
Base soil	$F_{pass\_v} = A_{pass} \times \gamma_b$ = **201** plf
Total	$F_{total\_v} = F_{stem} + F_{base} + F_{sat\_v} + F_{moist\_v} + F_{pass\_v} + F_{water\_v} + F_{sur\_v} + F_{P\_v}$ = **13255** plf

Horizontal forces on wall

Surcharge load	$F_{sur\_h} = K_0 \times Surcharge_L \times h_{eff}$ = **1117** plf
Saturated retained soil	$F_{sat\_h} = K_0 \times (\gamma_{sr} - \gamma_w) \times (h_{sat} + h_{base})^2 / 2$ = **3141** plf
Water	$F_{water\_h} = \gamma_w \times (h_{water} + d_{cover} + h_{base})^2 / 2$ = **4746** plf
Moist retained soil	$F_{moist\_h} = K_0 \times \gamma_{mr} \times ((h_{eff} - h_{sat} - h_{base})^2 / 2 + (h_{eff} - h_{sat} - h_{base}) \times (h_{sat} + h_{base}))$ = **11700** plf
Base soil	$F_{pass\_h} = -K_P \times \gamma_b \times (d_{cover} + h_{base})^2 / 2$ = **-939** plf
Total	$F_{total\_h} = F_{sat\_h} + F_{moist\_h} + F_{pass\_h} + F_{water\_h} + F_{sur\_h}$ = **19764** plf

Moments on wall

Wall stem	$M_{stem} = F_{stem} \times x_{stem}$ = **11812** lb_ft/ft
Wall base	$M_{base} = F_{base} \times x_{base}$ = **2500** lb_ft/ft
Surcharge load	$M_{sur} = F_{sur\_v} \times x_{sur\_v}$ = **722** lb_ft/ft
Line loads	$M_P = ((P_{D1} + P_{L1})) \times p_1$ = **1500** lb_ft/ft
Saturated retained soil	$M_{sat} = F_{sat\_v} \times x_{sat\_v}$ = **6559** lb_ft/ft
Water	$M_{water} = F_{water\_v} \times x_{water\_v}$ = **4955** lb_ft/ft
Moist retained soil	$M_{moist} = F_{moist\_v} \times x_{moist\_v}$ = **9745** lb_ft/ft
Base soil	$M_{pass} = F_{pass\_v} \times x_{pass\_v}$ = **176** lb_ft/ft

Total	$M_{total} = M_{stem} + M_{base} + M_{sat} + M_{moist} + M_{pass} + M_{water} + M_{sur} + M_P$ = **37970** lb_ft/ft

Check bearing pressure

Distance to reaction	$\bar{x} = M_{total} / F_{total\_v}$ = **2.865** ft
Eccentricity of reaction	$e = \bar{x} - l_{base} / 2$ = **0.365** ft
Loaded length of base	$l_{load} = l_{base}$ = **5** ft
Bearing pressure at toe	$q_{toe} = F_{total\_v} / l_{base} \times (1 - 6 \times e / l_{base})$ = **1491** psf
Bearing pressure at heel	$q_{heel} = F_{total\_v} / l_{base} \times (1 + 6 \times e / l_{base})$ = **3811** psf
Factor of safety	$FoS_{bp} = P_{bearing} / \max(q_{toe}, q_{heel})$ = **1.207**

PASS - Allowable bearing pressure exceeds maximum applied bearing pressure

RETAINING WALL DESIGN

In accordance with ACI 318-14

Tedds calculation version 2.9.00

Concrete details

Compressive strength of concrete	f'_c = **4500** psi
Concrete type	Normal weight

Reinforcement details

Yield strength of reinforcement	f_y = **60000** psi
Modulus of elasticity or reinforcement	E_s = **29000000** psi

Cover to reinforcement

Front face of stem	c_{sf} = **1.5** in
Rear face of stem	c_{sr} = **1.5** in
Top face of base	c_{bt} = **2** in
Bottom face of base	c_{bb} = **3** in

From IBC 2015 cl.1605.2.1 Basic load combinations

Load combination no.1	1.4 × Dead
Load combination no.2	1.2 × Dead + 1.6 × Live + 1.6 × Lateral earth
Load combination no.3	1.2 × Dead + 1.0 × Earthquake + 1.0 × Live + 1.6 × Lateral earth
Load combination no.4	0.9 × Dead + 1.0 × Earthquake + 1.6 × Lateral earth

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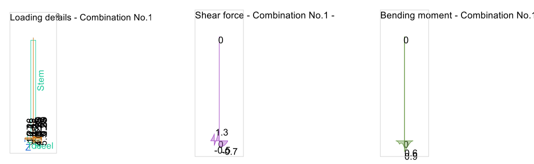

Check stem design at 8.646 ft

Depth of section	h = **18** in

Rectangular section in flexure - Section 22.3

Design bending moment combination 2	M = **75833** lb_ft/ft
Depth of tension reinforcement	d = h - c_{sf} - ϕ_{sx} - ϕ_{sfM} / 2 = **15.375** in
Compression reinforcement provided	No.5 bars @ 12" c/c
Area of compression reinforcement provided	$A_{srM.prov}$ = π × ϕ_{srM}^2 / (4 × s_{srM}) = **0.307** in^2/ft
Tension reinforcement provided	No.8 bars @ 8" c/c
Area of tension reinforcement provided	$A_{sfM.prov}$ = π × ϕ_{sfM}^2 / (4 × s_{sfM}) = **1.178** in^2/ft
Maximum reinforcement spacing - cl.11.7.2	s_{max} = min(18 in, 3 × h) = **18** in

PASS - Reinforcement is adequately spaced

Depth of compression block	a = $A_{sfM.prov}$ × f_y / (0.85 × f'$_c$) = **1.54** in
Neutral axis factor - cl.22.2.2.4.3	β_1 = min(max(0.85 - 0.05 × (f'$_c$ - 4 ksi) / 1 ksi, 0.65), 0.85) = **0.825**
Depth to neutral axis	c = a / β_1 = **1.867** in
Strain in reinforcement	ε_t = 0.003 × (d - c) / c = **0.02171**

Section is in the tension controlled zone

Strength reduction factor	ϕ_f = min(max(0.65 + (ε_t - 0.002) × (250 / 3), 0.65), 0.9) = **0.9**
Nominal flexural strength	M_n = $A_{sfM.prov}$ × f_y × (d - a / 2) = **86031** lb_ft/ft
Design flexural strength	ϕM_n = ϕ_f × M_n = **77428** lb_ft/ft
	M / ϕM_n = **0.979**

PASS - Design flexural strength exceeds factored bending moment

By iteration, reinforcement required by analysis	$A_{sfM.des}$ = **1.153** in^2/ft
Minimum area of reinforcement - cl.9.6.1.2	$A_{sfM.min}$ = max(3 × √(f'$_c$ × 1 psi), 200 psi) × d / f_y = **0.619** in^2/ft

PASS - Area of reinforcement provided is greater than minimum area of reinforcement required

Check stem design at base of stem

Depth of section	h = **18** in

Rectangular section in shear - Section 22.5

Design shear force	V = **19608** lb/ft

Concrete modification factor - cl.19.2.4	λ = **1**
Nominal concrete shear strength - eqn.22.5.5.1	$V_c = 2 \times \lambda \times \sqrt{(f'_c \times 1\ psi)} \times d$ = **24753** lb/ft
Strength reduction factor	ϕ_s = **0.75**
Design concrete shear strength - cl.11.5.1.1	$\phi V_c = \phi_s \times V_c$ = **18565** lb/ft
	$V / \phi V_c$ = **1.056**

FAIL - Shear reinforcement is required, cross-section should be revised

Check stem design at prop

Depth of section	h = **18** in

Rectangular section in shear - Section 22.5

Design shear force	V = **9390** lb/ft
Concrete modification factor - cl.19.2.4	λ = **1**
Nominal concrete shear strength - eqn.22.5.5.1	$V_c = 2 \times \lambda \times \sqrt{(f'_c \times 1\ psi)} \times d$ = **24753** lb/ft
Strength reduction factor	ϕ_s = **0.75**
Design concrete shear strength - cl.11.5.1.1	$\phi V_c = \phi_s \times V_c$ = **18565** lb/ft
	$V / \phi V_c$ = **0.506**

PASS - No shear reinforcement is required

Horizontal reinforcement parallel to face of stem

Minimum area of reinforcement - cl.11.6.1	$A_{sx.req} = 0.002 \times t_{stem}$ = **0.432** in^2/ft
Transverse reinforcement provided	No.5 bars @ 12" c/c each face
Area of transverse reinforcement provided	$A_{sx.prov} = 2 \times \pi \times \phi_{sx}^2 / (4 \times s_{sx})$ = **0.614** in^2/ft

PASS - Area of reinforcement provided is greater than area of reinforcement required

Check base design at heel

Depth of section	h = **16** in

Rectangular section in flexure - Section 22.3

Design bending moment combination 1	M = **921** lb_ft/ft
Depth of tension reinforcement	$d = h - c_{bb} - \phi_{bb} / 2$ = **12.688** in
Compression reinforcement provided	No.4 bars @ 12" c/c
Area of compression reinforcement provided	$A_{bt.prov} = \pi \times \phi_{bt}^2 / (4 \times s_{bt})$ = **0.196** in^2/ft
Tension reinforcement provided	No.5 bars @ 10" c/c
Area of tension reinforcement provided	$A_{bb.prov} = \pi \times \phi_{bb}^2 / (4 \times s_{bb})$ = **0.368** in^2/ft
Maximum reinforcement spacing - cl.7.7.2.3	s_{max} = min(18 in, 3 × h) = **18** in

PASS - Reinforcement is adequately spaced

Depth of compression block	$a = A_{bb.prov} \times f_y / (0.85 \times f'_c)$ = **0.481** in
Neutral axis factor - cl.22.2.2.4.3	β_1 = min(max(0.85 - 0.05 × (f'_c - 4 ksi) / 1 ksi, 0.65), 0.85) = **0.825**
Depth to neutral axis	$c = a / \beta_1$ = **0.583** in
Strain in reinforcement	$\varepsilon_t = 0.003 \times (d - c) / c$ = **0.06225**

Section is in the tension controlled zone

Strength reduction factor	ϕ_f = min(max(0.65 + (ε_t - 0.002) × (250 / 3), 0.65), 0.9) = **0.9**
Nominal flexural strength	$M_n = A_{bb.prov} \times f_y \times (d - a / 2)$ = **22912** lb_ft/ft
Design flexural strength	$\phi M_n = \phi_f \times M_n$ = **20621** lb_ft/ft
	$M / \phi M_n$ = **0.045**

PASS - Design flexural strength exceeds factored bending moment

By iteration, reinforcement required by analysis	$A_{bb.des}$ = **0.016** in^2/ft

Minimum area of reinforcement - cl.7.6.1.1	$A_{bb.min} = 0.0018 \times h = \mathbf{0.346}$ in^2/ft

PASS - Area of reinforcement provided is greater than minimum area of reinforcement required

Rectangular section in shear - Section 22.5

Design shear force	$V = \mathbf{1425}$ lb/ft
Concrete modification factor - cl.19.2.4	$\lambda = \mathbf{1}$
Nominal concrete shear strength - eqn.22.5.5.1	$V_c = 2 \times \lambda \times \sqrt{}(f'_c \times 1 \text{ psi}) \times d = \mathbf{20426}$ lb/ft
Strength reduction factor	$\phi_s = \mathbf{0.75}$
Design concrete shear strength - cl.7.6.3.1	$\phi V_c = \phi_s \times V_c = \mathbf{15320}$ lb/ft
	$V / \phi V_c = \mathbf{0.093}$

PASS - No shear reinforcement is required

Check base design at heel

Depth of section	$h = \mathbf{16}$ in

Rectangular section in shear - Section 22.5

Design shear force	$V = \mathbf{709}$ lb/ft
Concrete modification factor - cl.19.2.4	$\lambda = \mathbf{1}$
Nominal concrete shear strength - eqn.22.5.5.1	$V_c = 2 \times \lambda \times \sqrt{}(f'_c \times 1 \text{ psi}) \times d = \mathbf{20426}$ lb/ft
Strength reduction factor	$\phi_s = \mathbf{0.75}$
Design concrete shear strength - cl.7.6.3.1	$\phi V_c = \phi_s \times V_c = \mathbf{15320}$ lb/ft
	$V / \phi V_c = \mathbf{0.046}$

PASS - No shear reinforcement is required

Transverse reinforcement parallel to base

Minimum area of reinforcement - cl.76.1.1	$A_{bx.req} = 0.0018 \times t_{base} = \mathbf{0.346}$ in^2/ft
Transverse reinforcement provided	No.5 bars @ 12" c/c each face
Area of transverse reinforcement provided	$A_{bx.prov} = 2 \times \pi \times \phi_{bx}^2 / (4 \times s_{bx}) = \mathbf{0.614}$ in^2/ft

PASS - Area of reinforcement provided is greater than area of reinforcement required

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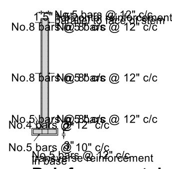
Retaining Walls

location Eden, Utah

date 6/26/2017 by CW



Reinforcement details

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Retaining Walls
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RETAINING WALL ANALYSIS & DESIGN - GRID E AT STAIR (ACI318/MSJC)

RETAINING WALL ANALYSIS

In accordance with International Building Code 2015

Tedds calculation version 2.9.00

Retaining wall details

Stem type	Propped cantilever pinned at the base
Stem height	h_{stem} = **14** ft
Prop height	h_{prop} = **14** ft
Stem thickness	t_{stem} = **12** in
Angle to rear face of stem	α = **90** deg
Stem density	γ_{stem} = **150** pcf
Toe length	l_{toe} = **1.5** ft
Heel length	l_{heel} = **1.5** ft
Base thickness	t_{base} = **16** in
Base density	γ_{base} = **150** pcf
Height of retained soil	h_{ret} = **13** ft
Angle of soil surface	β = **0** deg
Depth of cover	d_{cover} = **1** ft
Depth of excavation	d_{exc} = **1** ft
Height of water	h_{water} = **10** ft
Water density	γ_w = **62** pcf

Retained soil properties

Soil type	Medium dense well graded sand
Moist density	γ_{mr} = **135** pcf
Saturated density	γ_{sr} = **145** pcf

Base soil properties

Soil type	Medium dense well graded sand
Soil density	γ_b = **115** pcf
Allowable bearing pressure	$P_{bearing}$ = **4600** psf

Loading details

Live surcharge load	$Surcharge_L$ = **100** psf
Vertical line load at 0.75 ft	P_{D1} = **1000** plf
	P_{L1} = **1000** plf

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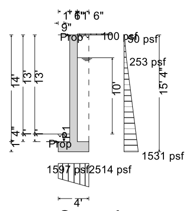

General arrangement

Calculate retaining wall geometry

Base length	$l_{base} = l_{toe} + t_{stem} + l_{heel} =$ **4** ft
Saturated soil height	$h_{sat} = h_{water} + d_{cover} =$ **11** ft
Moist soil height	$h_{moist} = h_{ret} - h_{water} =$ **3** ft
Length of surcharge load	$l_{sur} = l_{heel} =$ **1.5** ft
- Distance to vertical component	$x_{sur\_v} = l_{base} - l_{heel} / 2 =$ **3.25** ft
Effective height of wall	$h_{eff} = h_{base} + d_{cover} + h_{ret} =$ **15.333** ft
- Distance to horizontal component	$x_{sur\_h} = h_{eff} / 2 =$ **7.667** ft
Area of wall stem	$A_{stem} = h_{stem} \times t_{stem} =$ **14** ft^2
- Distance to vertical component	$x_{stem} = l_{toe} + t_{stem} / 2 =$ **2** ft
Area of wall base	$A_{base} = l_{base} \times t_{base} =$ **5.333** ft^2
- Distance to vertical component	$x_{base} = l_{base} / 2 =$ **2** ft
Area of saturated soil	$A_{sat} = h_{sat} \times l_{heel} =$ **16.5** ft^2
- Distance to vertical component	$x_{sat\_v} = l_{base} - (h_{sat} \times l_{heel}^2 / 2) / A_{sat} =$ **3.25** ft
- Distance to horizontal component	$x_{sat\_h} = (h_{sat} + h_{base}) / 3 =$ **4.111** ft
Area of water	$A_{water} = h_{sat} \times l_{heel} =$ **16.5** ft^2
- Distance to vertical component	$x_{water\_v} = l_{base} - (h_{sat} \times l_{heel}^2 / 2) / A_{sat} =$ **3.25** ft
- Distance to horizontal component	$x_{water\_h} = (h_{sat} + h_{base}) / 3 =$ **4.111** ft
Area of moist soil	$A_{moist} = h_{moist} \times l_{heel} =$ **4.5** ft^2
- Distance to vertical component	$x_{moist\_v} = l_{base} - (h_{moist} \times l_{heel}^2 / 2) / A_{moist} =$ **3.25** ft

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- Distance to horizontal component	$x_{moist\_h} = (h_{moist} \times (t_{base} + h_{sat} + h_{moist} / 3) / 2 + (h_{sat} + t_{base})^2/2) / (h_{sat} + t_{base} + h_{moist} / 2) =$ **6.944** ft
Area of base soil	$A_{pass} = d_{cover} \times l_{toe} =$ **1.5** ft^2
- Distance to vertical component	$x_{pass\_v} = l_{base} - (d_{cover} \times l_{toe} \times (l_{base} - l_{toe} / 2)) / A_{pass} =$ **0.75** ft
- Distance to horizontal component	$x_{pass\_h} = (d_{cover} + h_{base}) / 3 =$ **0.778** ft

Soil coefficients

Coefficient of friction to back of wall	$K_{fr} =$ **0.450**
Coefficient of friction to front of wall	$K_{fb} =$ **0.450**
Coefficient of friction beneath base	$K_{fbb} =$ **0.450**
At rest pressure coefficient	$K_0 =$ **0.500**
Passive pressure coefficient	$K_P =$ **3.000**

From IBC 2015 cl.1807.2.3 Safety factor

Load combination 1 — $1.0 \times$ Dead $+ 1.0 \times$ Live $+ 1.0 \times$ Lateral earth

Bearing pressure check

Vertical forces on wall

Wall stem	$F_{stem} = A_{stem} \times \gamma_{stem} =$ **2100** plf
Wall base	$F_{base} = A_{base} \times \gamma_{base} =$ **800** plf
Surcharge load	$F_{sur\_v} = Surcharge_L \times l_{heel} =$ **150** plf
Line loads	$F_{P\_v} = P_{D1} + P_{L1} =$ **2000** plf
Saturated retained soil	$F_{sat\_v} = A_{sat} \times (\gamma_{sr} - \gamma_w) =$ **1363** plf
Water	$F_{water\_v} = A_{water} \times \gamma_w =$ **1030** plf
Moist retained soil	$F_{moist\_v} = A_{moist} \times \gamma_{mr} =$ **608** plf
Base soil	$F_{pass\_v} = A_{pass} \times \gamma_b =$ **173** plf
Total	$F_{total\_v} = F_{stem} + F_{base} + F_{sat\_v} + F_{moist\_v} + F_{pass\_v} + F_{water\_v} + F_{sur\_v} + F_{P\_v} =$ **8222** plf

Horizontal forces on wall

Surcharge load	$F_{sur\_h} = K_0 \times Surcharge_L \times h_{eff} =$ **767** plf
Saturated retained soil	$F_{sat\_h} = K_0 \times (\gamma_{sr} - \gamma_w) \times (h_{sat} + h_{base})^2 / 2 =$ **3141** plf
Water	$F_{water\_h} = \gamma_w \times (h_{water} + d_{cover} + h_{base})^2 / 2 =$ **4746** plf
Moist retained soil	$F_{moist\_h} = K_0 \times \gamma_{mr} \times ((h_{eff} - h_{sat} - h_{base})^2 / 2 + (h_{eff} - h_{sat} - h_{base}) \times (h_{sat} + h_{base})) =$ **2801** plf
Base soil	$F_{pass\_h} = -K_P \times \gamma_b \times (d_{cover} + h_{base})^2 / 2 =$ **-939** plf
Total	$F_{total\_h} = F_{sat\_h} + F_{moist\_h} + F_{pass\_h} + F_{water\_h} + F_{sur\_h} =$ **10516** plf

Moments on wall

Wall stem	$M_{stem} = F_{stem} \times x_{stem} =$ **4200** lb_ft/ft
Wall base	$M_{base} = F_{base} \times x_{base} =$ **1600** lb_ft/ft
Surcharge load	$M_{sur} = F_{sur\_v} \times x_{sur\_v} =$ **487** lb_ft/ft
Line loads	$M_P = ((P_{D1} + P_{L1})) \times p_1 =$ **1500** lb_ft/ft
Saturated retained soil	$M_{sat} = F_{sat\_v} \times x_{sat\_v} =$ **4429** lb_ft/ft
Water	$M_{water} = F_{water\_v} \times x_{water\_v} =$ **3346** lb_ft/ft
Moist retained soil	$M_{moist} = F_{moist\_v} \times x_{moist\_v} =$ **1974** lb_ft/ft
Base soil	$M_{pass} = F_{pass\_v} \times x_{pass\_v} =$ **129** lb_ft/ft

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Total	$M_{total} = M_{stem} + M_{base} + M_{sat} + M_{moist} + M_{pass} + M_{water} + M_{sur} + M_P =$ **17667** lb_ft/ft

Check bearing pressure

Distance to reaction	$\bar{x} = M_{total} / F_{total\_v} =$ **2.149** ft
Eccentricity of reaction	$e = \bar{x} - l_{base} / 2 =$ **0.149** ft
Loaded length of base	$l_{load} = l_{base} =$ **4** ft
Bearing pressure at toe	$q_{toe} = F_{total\_v} / l_{base} \times (1 - 6 \times e / l_{base}) =$ **1597** psf
Bearing pressure at heel	$q_{heel} = F_{total\_v} / l_{base} \times (1 + 6 \times e / l_{base}) =$ **2514** psf
Factor of safety	$FoS_{bp} = P_{bearing} / \max(q_{toe}, q_{heel}) =$ **1.83**

PASS - Allowable bearing pressure exceeds maximum applied bearing pressure

RETAINING WALL DESIGN

In accordance with ACI 318-11

Tedds calculation version 2.9.00

Concrete details

Compressive strength of concrete	$f'_c =$ **4500** psi
Concrete type	Normal weight

Reinforcement details

Yield strength of reinforcement	$f_y =$ **60000** psi
Modulus of elasticity or reinforcement	$E_s =$ **29000000** psi

Cover to reinforcement

Front face of stem	$c_{sf} =$ **1.5** in
Rear face of stem	$c_{sr} =$ **1.5** in
Top face of base	$c_{bt} =$ **2** in
Bottom face of base	$c_{bb} =$ **3** in

From IBC 2015 cl.1605.2.1 Basic load combinations

Load combination no.1	1.4 × Dead
Load combination no.2	1.2 × Dead + 1.6 × Live + 1.6 × Lateral earth
Load combination no.3	1.2 × Dead + 1.0 × Earthquake + 1.0 × Live + 1.6 × Lateral earth
Load combination no.4	0.9 × Dead + 1.0 × Earthquake + 1.6 × Lateral earth

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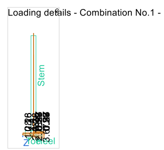

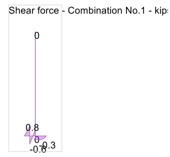

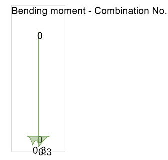

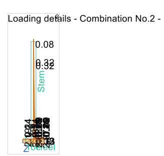

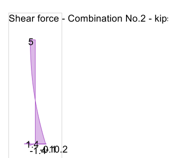

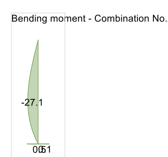

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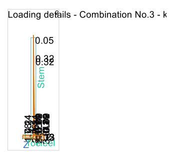
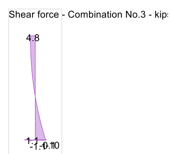
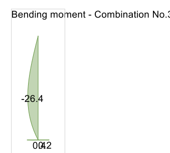

Check stem design at 5.851 ft

Depth of section $h = \mathbf{12}$ in

Rectangular section in flexure - Chapter 10

Design bending moment combination 2 $M = \mathbf{27077}$ lb_ft/ft

Depth of tension reinforcement $d = h - c_{sf} - \phi_{sx} - \phi_{sfM} / 2 = \mathbf{9.438}$ in

Compression reinforcement provided No.5 bars @ 12" c/c

Area of compression reinforcement provided $A_{srM.prov} = \pi \times \phi_{srM}^2 / (4 \times s_{srM}) = \mathbf{0.307}$ in^2/ft

Tension reinforcement provided No.7 bars @ 10" c/c

Area of tension reinforcement provided $A_{sfM.prov} = \pi \times \phi_{sfM}^2 / (4 \times s_{sfM}) = \mathbf{0.722}$ in^2/ft

Maximum reinforcement spacing - cl.14.3.5 $s_{max} = \min(18\text{ in}, 3 \times h) = \mathbf{18}$ in

PASS - Reinforcement is adequately spaced

Depth of compression block $a = A_{sfM.prov} \times f_y / (0.85 \times f'_c) = \mathbf{0.943}$ in

Neutral axis factor - cl.10.2.7.3 $\beta_1 = \min(\max(0.85 - 0.05 \times (f'_c - 4\text{ ksi}) / 1\text{ ksi}, 0.65), 0.85) = \mathbf{0.825}$

Depth to neutral axis $c = a / \beta_1 = \mathbf{1.143}$ in

Strain in reinforcement $\varepsilon_t = 0.003 \times (d - c) / c = \mathbf{0.021763}$

Section is in the tension controlled zone

Strength reduction factor $\phi_f = \min(\max(0.65 + (\varepsilon_t - 0.002) \times (250 / 3), 0.65), 0.9) = \mathbf{0.9}$

Nominal flexural strength $M_n = A_{sfM.prov} \times f_y \times (d - a / 2) = \mathbf{32348}$ lb_ft/ft

Design flexural strength $\phi M_n = \phi_f \times M_n = \mathbf{29113}$ lb_ft/ft

$M / \phi M_n = \mathbf{0.930}$

PASS - Design flexural strength exceeds factored bending moment

By iteration, reinforcement required by analysis $A_{sfM.des} = \mathbf{0.669}$ in^2/ft

Minimum area of reinforcement - eqn.10-3 $A_{sfM.min} = \max(3 \times \sqrt{(f'_c \times 1\text{ psi})}, 200\text{ psi}) \times d / f_y = \mathbf{0.38}$ in^2/ft

PASS - Area of reinforcement provided is greater than minimum area of reinforcement required

Check stem design at base of stem

Depth of section $h = \mathbf{12}$ in

Rectangular section in shear - Chapter 11

Design shear force $V = \mathbf{10202}$ lb/ft

Concrete modification factor - cl.8.6.1	λ = **1**
Nominal concrete shear strength - eqn.11-3	$V_c = 2 \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} \times d$ = **15194** lb/ft
Strength reduction factor	ϕ_s = **0.75**
Design concrete shear strength - cl.11.4.6.1	$\phi V_c = \phi_s \times V_c$ = **11396** lb/ft
	$V / \phi V_c$ = **0.895**

PASS - No shear reinforcement is required

Check stem design at prop

Depth of section	h = **12** in

Rectangular section in shear - Chapter 11

Design shear force	V = **5006** lb/ft
Concrete modification factor - cl.8.6.1	λ = **1**
Nominal concrete shear strength - eqn.11-3	$V_c = 2 \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} \times d$ = **15194** lb/ft
Strength reduction factor	ϕ_s = **0.75**
Design concrete shear strength - cl.11.4.6.1	$\phi V_c = \phi_s \times V_c$ = **11396** lb/ft
	$V / \phi V_c$ = **0.439**

PASS - No shear reinforcement is required

Horizontal reinforcement parallel to face of stem

Minimum area of reinforcement - cl.14.3.3	$A_{sx.req} = 0.002 \times t_{stem}$ = **0.288** in^2/ft
Transverse reinforcement provided	No.5 bars @ 12" c/c each face
Area of transverse reinforcement provided	$A_{sx.prov} = 2 \times \pi \times \phi_{sx}^2 / (4 \times s_{sx})$ = **0.614** in^2/ft

PASS - Area of reinforcement provided is greater than area of reinforcement required

Check base design at toe

Depth of section	h = **16** in

Rectangular section in flexure - Chapter 10

Design bending moment combination 2	M = **524** lb_ft/ft
Depth of tension reinforcement	$d = h - c_{bb} - \phi_{bb} / 2$ = **12.688** in
Compression reinforcement provided	No.4 bars @ 12" c/c
Area of compression reinforcement provided	$A_{bt.prov} = \pi \times \phi_{bt}^2 / (4 \times s_{bt})$ = **0.196** in^2/ft
Tension reinforcement provided	No.5 bars @ 10" c/c
Area of tension reinforcement provided	$A_{bb.prov} = \pi \times \phi_{bb}^2 / (4 \times s_{bb})$ = **0.368** in^2/ft
Maximum reinforcement spacing - cl.10.5.4	$s_{max} = \min(18 \text{ in}, 3 \times h)$ = **18** in

PASS - Reinforcement is adequately spaced

Depth of compression block	$a = A_{bb.prov} \times f_y / (0.85 \times f'_c)$ = **0.481** in
Neutral axis factor - cl.10.2.7.3	$\beta_1 = \min(\max(0.85 - 0.05 \times (f'_c - 4 \text{ ksi}) / 1 \text{ ksi}, 0.65), 0.85)$ = **0.825**
Depth to neutral axis	$c = a / \beta_1$ = **0.583** in
Strain in reinforcement	$\varepsilon_t = 0.003 \times (d - c) / c$ = **0.06225**

Section is in the tension controlled zone

Strength reduction factor	$\phi_f = \min(\max(0.65 + (\varepsilon_t - 0.002) \times (250 / 3), 0.65), 0.9)$ = **0.9**
Nominal flexural strength	$M_n = A_{bb.prov} \times f_y \times (d - a / 2)$ = **22912** lb_ft/ft
Design flexural strength	$\phi M_n = \phi_f \times M_n$ = **20621** lb_ft/ft
	$M / \phi M_n$ = **0.025**

PASS - Design flexural strength exceeds factored bending moment

By iteration, reinforcement required by analysis	$A_{bb.des}$ = **0.009** in^2/ft

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Minimum area of reinforcement - cl.7.12.2.1 $A_{bb.min} = 0.0018 \times h =$ **0.346** in^2/ft

PASS - Area of reinforcement provided is greater than minimum area of reinforcement required

Rectangular section in shear - Chapter 11

Design shear force $V =$ **1415** lb/ft

Concrete modification factor - cl.8.6.1 $\lambda =$ **1**

Nominal concrete shear strength - eqn.11-3 $V_c = 2 \times \lambda \times \sqrt{(f'_c \times 1\ psi)} \times d =$ **20426** lb/ft

Strength reduction factor $\phi_s =$ **0.75**

Design concrete shear strength - cl.11.4.6.1 $\phi V_c = \phi_s \times V_c =$ **15320** lb/ft

$V / \phi V_c =$ **0.092**

PASS - No shear reinforcement is required

Check base design at heel

Depth of section $h =$ **16** in

Rectangular section in shear - Chapter 11

Design shear force $V =$ **295** lb/ft

Concrete modification factor - cl.8.6.1 $\lambda =$ **1**

Nominal concrete shear strength - eqn.11-3 $V_c = 2 \times \lambda \times \sqrt{(f'_c \times 1\ psi)} \times d =$ **20426** lb/ft

Strength reduction factor $\phi_s =$ **0.75**

Design concrete shear strength - cl.11.4.6.1 $\phi V_c = \phi_s \times V_c =$ **15320** lb/ft

$V / \phi V_c =$ **0.019**

PASS - No shear reinforcement is required

Transverse reinforcement parallel to base

Minimum area of reinforcement - cl.7.12.2.1 $A_{bx.req} = 0.0018 \times t_{base} =$ **0.346** in^2/ft

Transverse reinforcement provided No.5 bars @ 12" c/c each face

Area of transverse reinforcement provided $A_{bx.prov} = 2 \times \pi \times \phi_{bx}^2 / (4 \times s_{bx}) =$ **0.614** in^2/ft

PASS - Area of reinforcement provided is greater than area of reinforcement required

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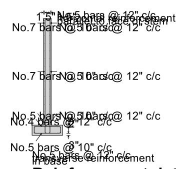

Reinforcement details

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CONCRETE RETAINING WALL SCHEDULE (CRW-1)

RETAINING WALL ANALYSIS

In accordance with International Building Code 2015

Tedds calculation version 2.9.00

Retaining wall details

Stem type	Cantilever
Stem height	h_{stem} = **6.5** ft
Stem thickness	t_{stem} = **8** in
Angle to rear face of stem	α = **90** deg
Stem density	γ_{stem} = **150** pcf
Toe length	l_{toe} = **0.833** ft
Heel length	l_{heel} = **2.5** ft
Base thickness	t_{base} = **12** in
Base density	γ_{base} = **150** pcf
Height of retained soil	h_{ret} = **4** ft
Angle of soil surface	β = **0** deg
Depth of cover	d_{cover} = **2.5** ft

Retained soil properties

Soil type	Medium dense well graded sand
Moist density	γ_{mr} = **135** pcf
Saturated density	γ_{sr} = **145** pcf

Base soil properties

Soil type	Medium dense well graded sand
Soil density	γ_{b} = **115** pcf
Allowable bearing pressure	$P_{bearing}$ = **4600** psf

Loading details

Live surcharge load	$Surcharge_{L}$ = **250** psf

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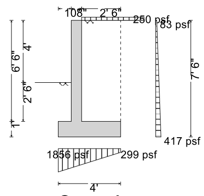

General arrangement

Calculate retaining wall geometry

Base length	$l_{base} = l_{toe} + t_{stem} + l_{heel} = \mathbf{4}$ ft
Moist soil height	$h_{moist} = h_{soil} = \mathbf{6.5}$ ft
Length of surcharge load	$l_{sur} = l_{heel} = \mathbf{2.5}$ ft
- Distance to vertical component	$x_{sur\_v} = l_{base} - l_{heel} / 2 = \mathbf{2.75}$ ft
Effective height of wall	$h_{eff} = h_{base} + d_{cover} + h_{ret} = \mathbf{7.5}$ ft
- Distance to horizontal component	$x_{sur\_h} = h_{eff} / 2 = \mathbf{3.75}$ ft
Area of wall stem	$A_{stem} = h_{stem} \times t_{stem} = \mathbf{4.333}$ ft^2
- Distance to vertical component	$x_{stem} = l_{toe} + t_{stem} / 2 = \mathbf{1.167}$ ft
Area of wall base	$A_{base} = l_{base} \times t_{base} = \mathbf{4}$ ft^2
- Distance to vertical component	$x_{base} = l_{base} / 2 = \mathbf{2}$ ft
Area of moist soil	$A_{moist} = h_{moist} \times l_{heel} = \mathbf{16.25}$ ft^2
- Distance to vertical component	$x_{moist\_v} = l_{base} - (h_{moist} \times l_{heel}^2 / 2) / A_{moist} = \mathbf{2.75}$ ft
- Distance to horizontal component	$x_{moist\_h} = h_{eff} / 3 = \mathbf{2.5}$ ft
Area of base soil	$A_{pass} = d_{cover} \times l_{toe} = \mathbf{2.083}$ ft^2
- Distance to vertical component	$x_{pass\_v} = l_{base} - (d_{cover} \times l_{toe} \times (l_{base} - l_{toe} / 2)) / A_{pass} = \mathbf{0.417}$ ft
- Distance to horizontal component	$x_{pass\_h} = (d_{cover} + h_{base}) / 3 = \mathbf{1.167}$ ft
Area of excavated base soil	$A_{exc} = h_{pass} \times l_{toe} = \mathbf{2.083}$ ft^2
- Distance to vertical component	$x_{exc\_v} = l_{base} - (h_{pass} \times l_{toe} \times (l_{base} - l_{toe} / 2)) / A_{exc} = \mathbf{0.417}$ ft
- Distance to horizontal component	$x_{exc\_h} = (h_{pass} + h_{base}) / 3 = \mathbf{1.167}$ ft

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Soil coefficients

Coefficient of friction to back of wall K_{fr} = **0.450**
Coefficient of friction to front of wall K_{fb} = **0.450**
Coefficient of friction beneath base K_{fbb} = **0.450**
Active pressure coefficient K_A = **0.330**
Passive pressure coefficient K_P = **3.000**

From IBC 2015 cl.1807.2.3 Safety factor

Load combination 1 1.0 × Dead + 1.0 × Live + 1.0 × Lateral earth

Sliding check

Vertical forces on wall

Wall stem $F_{stem} = A_{stem} \times \gamma_{stem}$ = **650** plf
Wall base $F_{base} = A_{base} \times \gamma_{base}$ = **600** plf
Moist retained soil $F_{moist\_v} = A_{moist} \times \gamma_{mr}$ = **2194** plf
Base soil $F_{exc\_v} = A_{exc} \times \gamma_b$ = **240** plf
Total $F_{total\_v} = F_{stem} + F_{base} + F_{moist\_v} + F_{exc\_v}$ = **3683** plf

Horizontal forces on wall

Surcharge load $F_{sur\_h} = K_A \times Surcharge_L \times h_{eff}$ = **619** plf
Moist retained soil $F_{moist\_h} = K_A \times \gamma_{mr} \times h_{eff}^2 / 2$ = **1253** plf
Total $F_{total\_h} = F_{moist\_h} + F_{sur\_h}$ = **1872** plf

Check stability against sliding

Base soil resistance $F_{exc\_h} = K_P \times \gamma_b \times (h_{pass} + h_{base})^2 / 2$ = **2113** plf
Base friction $F_{friction} = F_{total\_v} \times K_{fbb}$ = **1658** plf
Resistance to sliding $F_{rest} = F_{exc\_h} + F_{friction}$ = **3771** plf
Factor of safety $FoS_{sl} = F_{rest} / F_{total\_h}$ = **2.015** > 1.5

PASS - Factor of safety against sliding is adequate

Overturning check

Vertical forces on wall

Wall stem $F_{stem} = A_{stem} \times \gamma_{stem}$ = **650** plf
Wall base $F_{base} = A_{base} \times \gamma_{base}$ = **600** plf
Moist retained soil $F_{moist\_v} = A_{moist} \times \gamma_{mr}$ = **2194** plf
Base soil $F_{exc\_v} = A_{exc} \times \gamma_b$ = **240** plf
Total $F_{total\_v} = F_{stem} + F_{base} + F_{moist\_v} + F_{exc\_v}$ = **3683** plf

Horizontal forces on wall

Surcharge load $F_{sur\_h} = K_A \times Surcharge_L \times h_{eff}$ = **619** plf
Moist retained soil $F_{moist\_h} = K_A \times \gamma_{mr} \times h_{eff}^2 / 2$ = **1253** plf
Base soil $F_{exc\_h} = \max(-K_P \times \gamma_b \times (h_{pass} + h_{base})^2 / 2, -(F_{moist\_h} + F_{sur\_h}))$ = **-1872** plf
Total $F_{total\_h} = F_{moist\_h} + F_{exc\_h} + F_{sur\_h}$ = **0** plf

Overturning moments on wall

Surcharge load $M_{sur\_OT} = F_{sur\_h} \times x_{sur\_h}$ = **2320** lb_ft/ft
Moist retained soil $M_{moist\_OT} = F_{moist\_h} \times x_{moist\_h}$ = **3132** lb_ft/ft
Total $M_{total\_OT} = M_{moist\_OT} + M_{sur\_OT}$ = **5453** lb_ft/ft

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Restoring moments on wall

Wall stem $M_{stem\_R} = F_{stem} \times x_{stem} = \mathbf{758}$ lb_ft/ft

Wall base $M_{base\_R} = F_{base} \times x_{base} = \mathbf{1200}$ lb_ft/ft

Moist retained soil $M_{moist\_R} = F_{moist\_v} \times x_{moist\_v} = \mathbf{6033}$ lb_ft/ft

Base soil $M_{exc\_R} = F_{exc\_v} \times x_{exc\_v} - F_{exc\_h} \times x_{exc\_h} = \mathbf{2283}$ lb_ft/ft

Total $M_{total\_R} = M_{stem\_R} + M_{base\_R} + M_{moist\_R} + M_{exc\_R} = \mathbf{10275}$ lb_ft/ft

Check stability against overturning

Factor of safety $FoS_{ot} = M_{total\_R} / M_{total\_OT} = \mathbf{1.884} > 1.5$

PASS - Factor of safety against overturning is adequate

Bearing pressure check

Vertical forces on wall

Wall stem $F_{stem} = A_{stem} \times \gamma_{stem} = \mathbf{650}$ plf

Wall base $F_{base} = A_{base} \times \gamma_{base} = \mathbf{600}$ plf

Surcharge load $F_{sur\_v} = Surcharge_L \times l_{heel} = \mathbf{625}$ plf

Moist retained soil $F_{moist\_v} = A_{moist} \times \gamma_{mr} = \mathbf{2194}$ plf

Base soil $F_{pass\_v} = A_{pass} \times \gamma_b = \mathbf{240}$ plf

Total $F_{total\_v} = F_{stem} + F_{base} + F_{moist\_v} + F_{pass\_v} + F_{sur\_v} = \mathbf{4308}$ plf

Horizontal forces on wall

Surcharge load $F_{sur\_h} = K_A \times Surcharge_L \times h_{eff} = \mathbf{619}$ plf

Moist retained soil $F_{moist\_h} = K_A \times \gamma_{mr} \times h_{eff}^2 / 2 = \mathbf{1253}$ plf

Base soil $F_{pass\_h} = \max(-K_P \times \gamma_b \times (d_{cover} + h_{base})^2 / 2, -(F_{moist\_h} + F_{sur\_h})) = \mathbf{-1872}$ plf

Total $F_{total\_h} = \max(F_{moist\_h} + F_{pass\_h} + F_{sur\_h} - F_{total\_v} \times K_{fbb}, 0 \text{ plf}) = \mathbf{0}$ plf

Moments on wall

Wall stem $M_{stem} = F_{stem} \times x_{stem} = \mathbf{758}$ lb_ft/ft

Wall base $M_{base} = F_{base} \times x_{base} = \mathbf{1200}$ lb_ft/ft

Surcharge load $M_{sur} = F_{sur\_v} \times x_{sur\_v} - F_{sur\_h} \times x_{sur\_h} = \mathbf{-602}$ lb_ft/ft

Moist retained soil $M_{moist} = F_{moist\_v} \times x_{moist\_v} - F_{moist\_h} \times x_{moist\_h} = \mathbf{2900}$ lb_ft/ft

Base soil $M_{pass} = F_{pass\_v} \times x_{pass\_v} - F_{pass\_h} \times x_{pass\_h} = \mathbf{2283}$ lb_ft/ft

Total $M_{total} = M_{stem} + M_{base} + M_{moist} + M_{pass} + M_{sur} = \mathbf{6541}$ lb_ft/ft

Check bearing pressure

Distance to reaction $\bar{x} = M_{total} / F_{total\_v} = \mathbf{1.518}$ ft

Eccentricity of reaction $e = \bar{x} - l_{base} / 2 = \mathbf{-0.482}$ ft

Loaded length of base $l_{load} = l_{base} = \mathbf{4}$ ft

Bearing pressure at toe $q_{toe} = F_{total\_v} / l_{base} \times (1 - 6 \times e / l_{base}) = \mathbf{1856}$ psf

Bearing pressure at heel $q_{heel} = F_{total\_v} / l_{base} \times (1 + 6 \times e / l_{base}) = \mathbf{299}$ psf

Factor of safety $FoS_{bp} = P_{bearing} / \max(q_{toe}, q_{heel}) = \mathbf{2.479}$

PASS - Allowable bearing pressure exceeds maximum applied bearing pressure

<u>RETAINING WALL DESIGN</u>

In accordance with ACI 318-11

Tedds calculation version 2.9.00

Concrete details

Compressive strength of concrete $f'_c = \mathbf{4000}$ psi

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Concrete type	Normal weight

Reinforcement details

Yield strength of reinforcement	f_y = **60000** psi
Modulus of elasticity or reinforcement	E_s = **29000000** psi

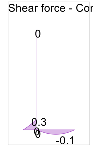
Cover to reinforcement

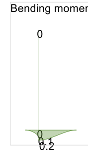
Front face of stem	c_{sf} = **1.5** in
Rear face of stem	c_{sr} = **2** in
Top face of base	c_{bt} = **2** in
Bottom face of base	c_{bb} = **3** in

From IBC 2015 cl.1605.2.1 Basic load combinations

Load combination no.1	1.4 × Dead
Load combination no.2	1.2 × Dead + 1.6 × Live + 1.6 × Lateral earth
Load combination no.3	1.2 × Dead + 1.0 × Earthquake + 1.0 × Live + 1.6 × Lateral earth
Load combination no.4	0.9 × Dead + 1.0 × Earthquake + 1.6 × Lateral earth

Loading details - Combination No.1 - kips/ft


Shear force - Combination No.1 - kips/ft


Bending moment - Combination No.1 - kips_ft/ft


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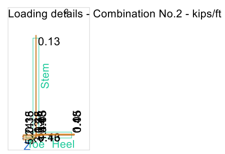

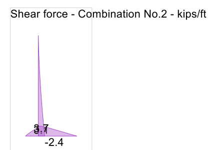

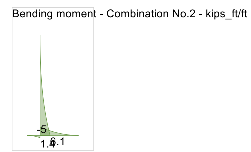

Check stem design at base of stem

Depth of section	$h = \mathbf{8}$ in

Rectangular section in flexure - Chapter 10

Design bending moment combination 2	$M = \mathbf{6051}$ lb_ft/ft
Depth of tension reinforcement	$d = h - c_{sr} - \phi_{sr} / 2 = \mathbf{5.688}$ in
Compression reinforcement provided	None
Area of compression reinforcement provided	$A_{sf.prov} = \mathbf{0}$ in^2/ft
Tension reinforcement provided	No.5 bars @ 12" c/c
Area of tension reinforcement provided	$A_{sr.prov} = \pi \times \phi_{sr}^2 / (4 \times s_{sr}) = \mathbf{0.307}$ in^2/ft
Maximum reinforcement spacing - cl.14.3.5	$s_{max} = \min(18\text{ in}, 3 \times h) = \mathbf{18}$ in

PASS - Reinforcement is adequately spaced

Depth of compression block	$a = A_{sr.prov} \times f_y / (0.85 \times f'_c) = \mathbf{0.451}$ in
Neutral axis factor - cl.10.2.7.3	$\beta_1 = \min(\max(0.85 - 0.05 \times (f'_c - 4\text{ ksi}) / 1\text{ ksi}, 0.65), 0.85) = \mathbf{0.85}$
Depth to neutral axis	$c = a / \beta_1 = \mathbf{0.531}$ in
Strain in reinforcement	$\varepsilon_t = 0.003 \times (d - c) / c = \mathbf{0.029146}$

Section is in the tension controlled zone

Strength reduction factor	$\phi_f = \min(\max(0.65 + (\varepsilon_t - 0.002) \times (250 / 3), 0.65), 0.9) = \mathbf{0.9}$
Nominal flexural strength	$M_n = A_{sr.prov} \times f_y \times (d - a / 2) = \mathbf{8378}$ lb_ft/ft
Design flexural strength	$\phi M_n = \phi_f \times M_n = \mathbf{7541}$ lb_ft/ft
	$M / \phi M_n = \mathbf{0.802}$

PASS - Design flexural strength exceeds factored bending moment

By iteration, reinforcement required by analysis	$A_{sr.des} = \mathbf{0.244}$ in^2/ft
Minimum area of reinforcement - eqn.10-3	$A_{sr.min} = \max(3 \times \sqrt{(f'_c \times 1\text{ psi})}, 200\text{ psi}) \times d / f_y = \mathbf{0.228}$ in^2/ft

PASS - Area of reinforcement provided is greater than minimum area of reinforcement required

Rectangular section in shear - Chapter 11

Design shear force	$V = \mathbf{2364}$ lb/ft
Concrete modification factor - cl.8.6.1	$\lambda = \mathbf{1}$
Nominal concrete shear strength - eqn.11-3	$V_c = 2 \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})} \times d = \mathbf{8633}$ lb/ft

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Strength reduction factor	ϕ_s = **0.75**
Design concrete shear strength - cl.11.4.6.1	$\phi V_c = \phi_s \times V_c$ = **6475** lb/ft
	$V / \phi V_c$ = **0.365**

PASS - No shear reinforcement is required

Horizontal reinforcement parallel to face of stem

Minimum area of reinforcement - cl.14.3.3	$A_{sx.req} = 0.002 \times t_{stem}$ = **0.192** in^2/ft
Transverse reinforcement provided	No.4 bars @ 12" c/c
Area of transverse reinforcement provided	$A_{sx.prov} = \pi \times \phi_{sx}^2 / (4 \times s_{sx})$ = **0.196** in^2/ft

PASS - Area of reinforcement provided is greater than area of reinforcement required

Check base design at toe

Depth of section	h = **12** in

Rectangular section in flexure - Chapter 10

Design bending moment combination 2	M = **1394** lb_ft/ft
Depth of tension reinforcement	$d = h - c_{bb} - \phi_{bb} / 2$ = **8.688** in
Compression reinforcement provided	No.5 bars @ 12" c/c
Area of compression reinforcement provided	$A_{bt.prov} = \pi \times \phi_{bt}^2 / (4 \times s_{bt})$ = **0.307** in^2/ft
Tension reinforcement provided	No.5 bars @ 12" c/c
Area of tension reinforcement provided	$A_{bb.prov} = \pi \times \phi_{bb}^2 / (4 \times s_{bb})$ = **0.307** in^2/ft
Maximum reinforcement spacing - cl.10.5.4	s_{max} = min(18 in, 3 × h) = **18** in

PASS - Reinforcement is adequately spaced

Depth of compression block	$a = A_{bb.prov} \times f_y / (0.85 \times f'_c)$ = **0.451** in
Neutral axis factor - cl.10.2.7.3	β_1 = min(max(0.85 - 0.05 × (f'$_c$ - 4 ksi) / 1 ksi, 0.65), 0.85) = **0.85**
Depth to neutral axis	$c = a / \beta_1$ = **0.531** in
Strain in reinforcement	$\varepsilon_t = 0.003 \times (d - c) / c$ = **0.046101**

Section is in the tension controlled zone

Strength reduction factor	ϕ_f = min(max(0.65 + (ε_t - 0.002) × (250 / 3), 0.65), 0.9) = **0.9**
Nominal flexural strength	$M_n = A_{bb.prov} \times f_y \times (d - a / 2)$ = **12980** lb_ft/ft
Design flexural strength	$\phi M_n = \phi_f \times M_n$ = **11682** lb_ft/ft
	$M / \phi M_n$ = **0.119**

PASS - Design flexural strength exceeds factored bending moment

By iteration, reinforcement required by analysis	$A_{bb.des}$ = **0.036** in^2/ft
Minimum area of reinforcement - cl.7.12.2.1	$A_{bb.min} = 0.0018 \times h$ = **0.259** in^2/ft

PASS - Area of reinforcement provided is greater than minimum area of reinforcement required

Rectangular section in shear - Chapter 11

Design shear force	V = **3052** lb/ft
Concrete modification factor - cl.8.6.1	λ = **1**
Nominal concrete shear strength - eqn.11-3	$V_c = 2 \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} \times d$ = **13187** lb/ft
Strength reduction factor	ϕ_s = **0.75**
Design concrete shear strength - cl.11.4.6.1	$\phi V_c = \phi_s \times V_c$ = **9890** lb/ft
	$V / \phi V_c$ = **0.309**

PASS - No shear reinforcement is required

Check base design at heel

Depth of section	h = **12** in

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Rectangular section in flexure - Chapter 10

Design bending moment combination 2	M = **5026** lb_ft/ft
Depth of tension reinforcement	$d = h - c_{bt} - \phi_{bt} / 2$ = **9.687** in
Compression reinforcement provided	No.5 bars @ 12" c/c
Area of compression reinforcement provided	$A_{bb.prov} = \pi \times \phi_{bb}^2 / (4 \times s_{bb})$ = **0.307** in^2/ft
Tension reinforcement provided	No.5 bars @ 12" c/c
Area of tension reinforcement provided	$A_{bt.prov} = \pi \times \phi_{bt}^2 / (4 \times s_{bt})$ = **0.307** in^2/ft
Maximum reinforcement spacing - cl.10.5.4	s_{max} = min(18 in, 3 × h) = **18** in

PASS - Reinforcement is adequately spaced

Depth of compression block	$a = A_{bt.prov} \times f_y / (0.85 \times f'_c)$ = **0.451** in
Neutral axis factor - cl.10.2.7.3	β_1 = min(max(0.85 - 0.05 × (f'$_c$ - 4 ksi) / 1 ksi, 0.65), 0.85) = **0.85**
Depth to neutral axis	$c = a / \beta_1$ = **0.531** in
Strain in reinforcement	$\varepsilon_t = 0.003 \times (d - c) / c$ = **0.051753**

Section is in the tension controlled zone

Strength reduction factor	ϕ_f = min(max(0.65 + (ε_t - 0.002) × (250 / 3), 0.65), 0.9) = **0.9**
Nominal flexural strength	$M_n = A_{bt.prov} \times f_y \times (d - a / 2)$ = **14514** lb_ft/ft
Design flexural strength	$\phi M_n = \phi_f \times M_n$ = **13063** lb_ft/ft
	$M / \phi M_n$ = **0.385**

PASS - Design flexural strength exceeds factored bending moment

By iteration, reinforcement required by analysis	$A_{bt.des}$ = **0.116** in^2/ft
Minimum area of reinforcement - cl.7.12.2.1	$A_{bt.min} = 0.0018 \times h$ = **0.259** in^2/ft

PASS - Area of reinforcement provided is greater than minimum area of reinforcement required

Rectangular section in shear - Chapter 11

Design shear force	V = **3674** lb/ft
Concrete modification factor - cl.8.6.1	λ = **1**
Nominal concrete shear strength - eqn.11-3	$V_c = 2 \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} \times d$ = **14705** lb/ft
Strength reduction factor	ϕ_s = **0.75**
Design concrete shear strength - cl.11.4.6.1	$\phi V_c = \phi_s \times V_c$ = **11028** lb/ft
	$V / \phi V_c$ = **0.333**

PASS - No shear reinforcement is required

Transverse reinforcement parallel to base

Minimum area of reinforcement - cl.7.12.2.1	$A_{bx.req} = 0.0018 \times t_{base}$ = **0.259** in^2/ft
Transverse reinforcement provided	No.5 bars @ 12" c/c each face
Area of transverse reinforcement provided	$A_{bx.prov} = 2 \times \pi \times \phi_{bx}^2 / (4 \times s_{bx})$ = **0.614** in^2/ft

PASS - Area of reinforcement provided is greater than area of reinforcement required

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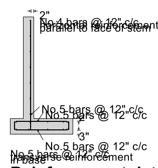

Reinforcement details

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CONCRETE RETAINING WALL SCHEDULE (CRW-2)

RETAINING WALL ANALYSIS

In accordance with International Building Code 2015

Tedds calculation version 2.9.00

Retaining wall details

Stem type	Cantilever
Stem height	h_{stem} = **9.5** ft
Stem thickness	t_{stem} = **12** in
Angle to rear face of stem	α = **90** deg
Stem density	γ_{stem} = **150** pcf
Toe length	l_{toe} = **1** ft
Heel length	l_{heel} = **4** ft
Base thickness	t_{base} = **12** in
Base density	γ_{base} = **150** pcf
Height of retained soil	h_{ret} = **7** ft
Angle of soil surface	β = **0** deg
Depth of cover	d_{cover} = **2.5** ft

Retained soil properties

Soil type	Medium dense well graded sand
Moist density	γ_{mr} = **135** pcf
Saturated density	γ_{sr} = **145** pcf

Base soil properties

Soil type	Medium dense well graded sand
Soil density	γ_b = **115** pcf
Allowable bearing pressure	$P_{bearing}$ = **4600** psf

Loading details

Live surcharge load	$Surcharge_L$ = **250** psf

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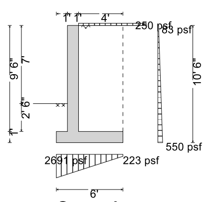

General arrangement

Calculate retaining wall geometry

Base length	$l_{base} = l_{toe} + t_{stem} + l_{heel} = \mathbf{6}$ ft
Moist soil height	$h_{moist} = h_{soil} = \mathbf{9.5}$ ft
Length of surcharge load	$l_{sur} = l_{heel} = \mathbf{4}$ ft
- Distance to vertical component	$x_{sur\_v} = l_{base} - l_{heel} / 2 = \mathbf{4}$ ft
Effective height of wall	$h_{eff} = h_{base} + d_{cover} + h_{ret} = \mathbf{10.5}$ ft
- Distance to horizontal component	$x_{sur\_h} = h_{eff} / 2 = \mathbf{5.25}$ ft
Area of wall stem	$A_{stem} = h_{stem} \times t_{stem} = \mathbf{9.5}$ ft^2
- Distance to vertical component	$x_{stem} = l_{toe} + t_{stem} / 2 = \mathbf{1.5}$ ft
Area of wall base	$A_{base} = l_{base} \times t_{base} = \mathbf{6}$ ft^2
- Distance to vertical component	$x_{base} = l_{base} / 2 = \mathbf{3}$ ft
Area of moist soil	$A_{moist} = h_{moist} \times l_{heel} = \mathbf{38}$ ft^2
- Distance to vertical component	$x_{moist\_v} = l_{base} - (h_{moist} \times l_{heel}^2 / 2) / A_{moist} = \mathbf{4}$ ft
- Distance to horizontal component	$x_{moist\_h} = h_{eff} / 3 = \mathbf{3.5}$ ft
Area of base soil	$A_{pass} = d_{cover} \times l_{toe} = \mathbf{2.5}$ ft^2
- Distance to vertical component	$x_{pass\_v} = l_{base} - (d_{cover} \times l_{toe} \times (l_{base} - l_{toe} / 2)) / A_{pass} = \mathbf{0.5}$ ft
- Distance to horizontal component	$x_{pass\_h} = (d_{cover} + h_{base}) / 3 = \mathbf{1.167}$ ft
Area of excavated base soil	$A_{exc} = h_{pass} \times l_{toe} = \mathbf{2.5}$ ft^2
- Distance to vertical component	$x_{exc\_v} = l_{base} - (h_{pass} \times l_{toe} \times (l_{base} - l_{toe} / 2)) / A_{exc} = \mathbf{0.5}$ ft
- Distance to horizontal component	$x_{exc\_h} = (h_{pass} + h_{base}) / 3 = \mathbf{1.167}$ ft

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Soil coefficients

Coefficient of friction to back of wall	K_{fr} = **0.450**
Coefficient of friction to front of wall	K_{fb} = **0.450**
Coefficient of friction beneath base	K_{fbb} = **0.450**
Active pressure coefficient	K_A = **0.330**
Passive pressure coefficient	K_P = **3.000**

From IBC 2015 cl.1807.2.3 Safety factor

Load combination 1 — 1.0 × Dead + 1.0 × Live + 1.0 × Lateral earth

Sliding check

Vertical forces on wall

Wall stem	$F_{stem} = A_{stem} \times \gamma_{stem}$ = **1425** plf
Wall base	$F_{base} = A_{base} \times \gamma_{base}$ = **900** plf
Moist retained soil	$F_{moist\_v} = A_{moist} \times \gamma_{mr}$ = **5130** plf
Base soil	$F_{exc\_v} = A_{exc} \times \gamma_b$ = **288** plf
Total	$F_{total\_v} = F_{stem} + F_{base} + F_{moist\_v} + F_{exc\_v}$ = **7743** plf

Horizontal forces on wall

Surcharge load	$F_{sur\_h} = K_A \times Surcharge_L \times h_{eff}$ = **866** plf
Moist retained soil	$F_{moist\_h} = K_A \times \gamma_{mr} \times h_{eff}^2 / 2$ = **2456** plf
Total	$F_{total\_h} = F_{moist\_h} + F_{sur\_h}$ = **3322** plf

Check stability against sliding

Base soil resistance	$F_{exc\_h} = K_P \times \gamma_b \times (h_{pass} + h_{base})^2 / 2$ = **2113** plf
Base friction	$F_{friction} = F_{total\_v} \times K_{fbb}$ = **3484** plf
Resistance to sliding	$F_{rest} = F_{exc\_h} + F_{friction}$ = **5597** plf
Factor of safety	$FoS_{sl} = F_{rest} / F_{total\_h}$ = **1.685** > 1.5

PASS - Factor of safety against sliding is adequate

Overturning check

Vertical forces on wall

Wall stem	$F_{stem} = A_{stem} \times \gamma_{stem}$ = **1425** plf
Wall base	$F_{base} = A_{base} \times \gamma_{base}$ = **900** plf
Moist retained soil	$F_{moist\_v} = A_{moist} \times \gamma_{mr}$ = **5130** plf
Base soil	$F_{exc\_v} = A_{exc} \times \gamma_b$ = **288** plf
Total	$F_{total\_v} = F_{stem} + F_{base} + F_{moist\_v} + F_{exc\_v}$ = **7743** plf

Horizontal forces on wall

Surcharge load	$F_{sur\_h} = K_A \times Surcharge_L \times h_{eff}$ = **866** plf
Moist retained soil	$F_{moist\_h} = K_A \times \gamma_{mr} \times h_{eff}^2 / 2$ = **2456** plf
Base soil	$F_{exc\_h} = -K_P \times \gamma_b \times (h_{pass} + h_{base})^2 / 2$ = **-2113** plf
Total	$F_{total\_h} = F_{moist\_h} + F_{exc\_h} + F_{sur\_h}$ = **1209** plf

Overturning moments on wall

Surcharge load	$M_{sur\_OT} = F_{sur\_h} \times x_{sur\_h}$ = **4548** lb_ft/ft
Moist retained soil	$M_{moist\_OT} = F_{moist\_h} \times x_{moist\_h}$ = **8595** lb_ft/ft
Total	$M_{total\_OT} = M_{moist\_OT} + M_{sur\_OT}$ = **13143** lb_ft/ft

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Retaining Walls
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Restoring moments on wall

Wall stem	$M_{stem\_R} = F_{stem} \times x_{stem} = \mathbf{2137}$ lb_ft/ft
Wall base	$M_{base\_R} = F_{base} \times x_{base} = \mathbf{2700}$ lb_ft/ft
Moist retained soil	$M_{moist\_R} = F_{moist\_v} \times x_{moist\_v} = \mathbf{20520}$ lb_ft/ft
Base soil	$M_{exc\_R} = F_{exc\_v} \times x_{exc\_v} - F_{exc\_h} \times x_{exc\_h} = \mathbf{2609}$ lb_ft/ft
Total	$M_{total\_R} = M_{stem\_R} + M_{base\_R} + M_{moist\_R} + M_{exc\_R} = \mathbf{27967}$ lb_ft/ft

Check stability against overturning

Factor of safety	$FoS_{ot} = M_{total\_R} / M_{total\_OT} = \mathbf{2.128} > 1.5$

PASS - Factor of safety against overturning is adequate

Bearing pressure check

Vertical forces on wall

Wall stem	$F_{stem} = A_{stem} \times \gamma_{stem} = \mathbf{1425}$ plf
Wall base	$F_{base} = A_{base} \times \gamma_{base} = \mathbf{900}$ plf
Surcharge load	$F_{sur\_v} = Surcharge_L \times l_{heel} = \mathbf{1000}$ plf
Moist retained soil	$F_{moist\_v} = A_{moist} \times \gamma_{mr} = \mathbf{5130}$ plf
Base soil	$F_{pass\_v} = A_{pass} \times \gamma_b = \mathbf{288}$ plf
Total	$F_{total\_v} = F_{stem} + F_{base} + F_{moist\_v} + F_{pass\_v} + F_{sur\_v} = \mathbf{8743}$ plf

Horizontal forces on wall

Surcharge load	$F_{sur\_h} = K_A \times Surcharge_L \times h_{eff} = \mathbf{866}$ plf
Moist retained soil	$F_{moist\_h} = K_A \times \gamma_{mr} \times h_{eff}^2 / 2 = \mathbf{2456}$ plf
Base soil	$F_{pass\_h} = -K_P \times \gamma_b \times (d_{cover} + h_{base})^2 / 2 = \mathbf{-2113}$ plf
Total	$F_{total\_h} = \max(F_{moist\_h} + F_{pass\_h} + F_{sur\_h} - F_{total\_v} \times K_{fbb}, 0 \text{ plf}) = \mathbf{0}$ plf

Moments on wall

Wall stem	$M_{stem} = F_{stem} \times x_{stem} = \mathbf{2137}$ lb_ft/ft
Wall base	$M_{base} = F_{base} \times x_{base} = \mathbf{2700}$ lb_ft/ft
Surcharge load	$M_{sur} = F_{sur\_v} \times x_{sur\_v} - F_{sur\_h} \times x_{sur\_h} = \mathbf{-548}$ lb_ft/ft
Moist retained soil	$M_{moist} = F_{moist\_v} \times x_{moist\_v} - F_{moist\_h} \times x_{moist\_h} = \mathbf{11925}$ lb_ft/ft
Base soil	$M_{pass} = F_{pass\_v} \times x_{pass\_v} - F_{pass\_h} \times x_{pass\_h} = \mathbf{2609}$ lb_ft/ft
Total	$M_{total} = M_{stem} + M_{base} + M_{moist} + M_{pass} + M_{sur} = \mathbf{18823}$ lb_ft/ft

Check bearing pressure

Distance to reaction	$\bar{x} = M_{total} / F_{total\_v} = \mathbf{2.153}$ ft
Eccentricity of reaction	$e = \bar{x} - l_{base} / 2 = \mathbf{-0.847}$ ft
Loaded length of base	$l_{load} = l_{base} = \mathbf{6}$ ft
Bearing pressure at toe	$q_{toe} = F_{total\_v} / l_{base} \times (1 - 6 \times e / l_{base}) = \mathbf{2691}$ psf
Bearing pressure at heel	$q_{heel} = F_{total\_v} / l_{base} \times (1 + 6 \times e / l_{base}) = \mathbf{223}$ psf
Factor of safety	$FoS_{bp} = P_{bearing} / \max(q_{toe}, q_{heel}) = \mathbf{1.709}$

PASS - Allowable bearing pressure exceeds maximum applied bearing pressure

RETAINING WALL DESIGN

In accordance with ACI 318-11

Tedds calculation version 2.9.00

Concrete details

Compressive strength of concrete	$f'_c = \mathbf{4500}$ psi

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Concrete type Normal weight

Reinforcement details

Yield strength of reinforcement f_y = **60000** psi

Modulus of elasticity or reinforcement E_s = **29000000** psi

Cover to reinforcement

Front face of stem c_{sf} = **1.5** in

Rear face of stem c_{sr} = **2** in

Top face of base c_{bt} = **2** in

Bottom face of base c_{bb} = **3** in

From IBC 2015 cl.1605.2.1 Basic load combinations

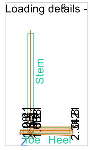
Load combination no.1 1.4 × Dead

Load combination no.2 1.2 × Dead + 1.6 × Live + 1.6 × Lateral earth

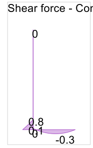
Load combination no.3 1.2 × Dead + 1.0 × Earthquake + 1.0 × Live + 1.6 × Lateral earth

Load combination no.4 0.9 × Dead + 1.0 × Earthquake + 1.6 × Lateral earth

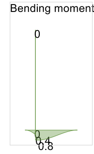
Loading details - Combination No.1 - kips/ft



Shear force - Combination No.1 - kips/ft



Bending moment - Combination No.1 - kips_ft/ft



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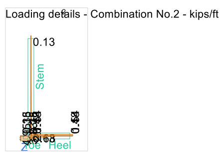

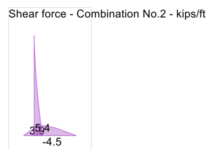

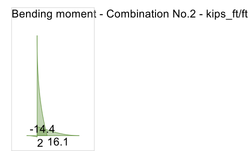

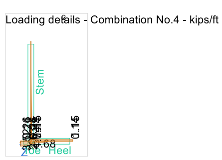

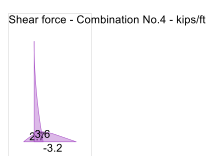

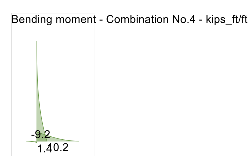

Check stem design at base of stem

Depth of section $h = 12$ in

Rectangular section in flexure - Chapter 10

Design bending moment combination 2 $M = 16142$ lb_ft/ft

Depth of tension reinforcement $d = h - c_{sr} - \phi_{sr} / 2 = 9.625$ in

Compression reinforcement provided No.4 bars @ 12" c/c

Area of compression reinforcement provided $A_{sf.prov} = \pi \times \phi_{sf}^2 / (4 \times s_{sf}) = 0.196$ in^2/ft

Tension reinforcement provided No.6 bars @ 12" c/c

Area of tension reinforcement provided $A_{sr.prov} = \pi \times \phi_{sr}^2 / (4 \times s_{sr}) = 0.442$ in^2/ft

Maximum reinforcement spacing - cl.14.3.5 $s_{max} = \min(18\text{ in}, 3 \times h) = 18$ in

PASS - Reinforcement is adequately spaced

Depth of compression block $a = A_{sr.prov} \times f_y / (0.85 \times f'_c) = 0.577$ in

Neutral axis factor - cl.10.2.7.3 β_1 = min(max(0.85 - 0.05 × (f'$_c$ - 4 ksi) / 1 ksi, 0.65), 0.85) = **0.825**

Depth to neutral axis c = a / β_1 = **0.7** in

Strain in reinforcement ε_t = 0.003 × (d - c) / c = **0.03825**

Section is in the tension controlled zone

Strength reduction factor ϕ_f = min(max(0.65 + (ε_t - 0.002) × (250 / 3), 0.65), 0.9) = **0.9**

Nominal flexural strength M_n = $A_{sr.prov}$ × f_y × (d - a / 2) = **20623** lb_ft/ft

Design flexural strength ϕM_n = ϕ_f × M_n = **18561** lb_ft/ft

M / ϕM_n = **0.870**

PASS - Design flexural strength exceeds factored bending moment

By iteration, reinforcement required by analysis $A_{sr.des}$ = **0.383** in^2/ft

Minimum area of reinforcement - eqn.10-3 $A_{sr.min}$ = max(3 × √(f'$_c$ × 1 psi), 200 psi) × d / f_y = **0.387** in^2/ft

PASS - Area of reinforcement provided is greater than minimum area of reinforcement required

Rectangular section in shear - Chapter 11

Design shear force V = **4471** lb/ft

Concrete modification factor - cl.8.6.1 λ = **1**

Nominal concrete shear strength - eqn.11-3 V_c = 2 × λ × √(f'$_c$ × 1 psi) × d = **15496** lb/ft

Strength reduction factor ϕ_s = **0.75**

Design concrete shear strength - cl.11.4.6.1 ϕV_c = ϕ_s × V_c = **11622** lb/ft

V / ϕV_c = **0.385**

PASS - No shear reinforcement is required

Horizontal reinforcement parallel to face of stem

Minimum area of reinforcement - cl.14.3.3 $A_{sx.req}$ = 0.002 × t_{stem} = **0.288** in^2/ft

Transverse reinforcement provided No.4 bars @ 12" c/c each face

Area of transverse reinforcement provided $A_{sx.prov}$ = 2 × π × ϕ_{sx}^2 / (4 × s_{sx}) = **0.393** in^2/ft

PASS - Area of reinforcement provided is greater than area of reinforcement required

Check base design at toe

Depth of section h = **12** in

Rectangular section in flexure - Chapter 10

Design bending moment combination 2 M = **2025** lb_ft/ft

Depth of tension reinforcement d = h - c_{bb} - ϕ_{bb} / 2 = **8.688** in

Compression reinforcement provided No.5 bars @ 10" c/c

Area of compression reinforcement provided $A_{bt.prov}$ = π × ϕ_{bt}^2 / (4 × s_{bt}) = **0.368** in^2/ft

Tension reinforcement provided No.5 bars @ 12" c/c

Area of tension reinforcement provided $A_{bb.prov}$ = π × ϕ_{bb}^2 / (4 × s_{bb}) = **0.307** in^2/ft

Maximum reinforcement spacing - cl.10.5.4 s_{max} = min(18 in, 3 × h) = **18** in

PASS - Reinforcement is adequately spaced

Depth of compression block a = $A_{bb.prov}$ × f_y / (0.85 × f'$_c$) = **0.401** in

Neutral axis factor - cl.10.2.7.3 β_1 = min(max(0.85 - 0.05 × (f'$_c$ - 4 ksi) / 1 ksi, 0.65), 0.85) = **0.825**

Depth to neutral axis c = a / β_1 = **0.486** in

Strain in reinforcement ε_t = 0.003 × (d - c) / c = **0.050614**

Section is in the tension controlled zone

Strength reduction factor ϕ_f = min(max(0.65 + (ε_t - 0.002) × (250 / 3), 0.65), 0.9) = **0.9**

Nominal flexural strength M_n = $A_{bb.prov}$ × f_y × (d - a / 2) = **13019** lb_ft/ft

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Design flexural strength	$\phi M_n = \phi_f \times M_n =$ **11717** lb_ft/ft
	$M / \phi M_n =$ **0.173**
	PASS - Design flexural strength exceeds factored bending moment
By iteration, reinforcement required by analysis	$A_{bb.des} =$ **0.052** in^2/ft
Minimum area of reinforcement - cl.7.12.2.1	$A_{bb.min} = 0.0018 \times h =$ **0.259** in^2/ft

PASS - Area of reinforcement provided is greater than minimum area of reinforcement required

Rectangular section in shear - Chapter 11

Design shear force	$V =$ **3863** lb/ft
Concrete modification factor - cl.8.6.1	$\lambda =$ **1**
Nominal concrete shear strength - eqn.11-3	$V_c = 2 \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} \times d =$ **13987** lb/ft
Strength reduction factor	$\phi_s =$ **0.75**
Design concrete shear strength - cl.11.4.6.1	$\phi V_c = \phi_s \times V_c =$ **10490** lb/ft
	$V / \phi V_c =$ **0.368**

PASS - No shear reinforcement is required

Check base design at heel

Depth of section	$h =$ **12** in

Rectangular section in flexure - Chapter 10

Design bending moment combination 2	$M =$ **14360** lb_ft/ft
Depth of tension reinforcement	$d = h - c_{bt} - \phi_{bt} / 2 =$ **9.687** in
Compression reinforcement provided	No.5 bars @ 12" c/c
Area of compression reinforcement provided	$A_{bb.prov} = \pi \times \phi_{bb}^2 / (4 \times s_{bb}) =$ **0.307** in^2/ft
Tension reinforcement provided	No.5 bars @ 10" c/c
Area of tension reinforcement provided	$A_{bt.prov} = \pi \times \phi_{bt}^2 / (4 \times s_{bt}) =$ **0.368** in^2/ft
Maximum reinforcement spacing - cl.10.5.4	$s_{max} = \min(18 \text{ in}, 3 \times h) =$ **18** in
	PASS - Reinforcement is adequately spaced
Depth of compression block	$a = A_{bt.prov} \times f_y / (0.85 \times f'_c) =$ **0.481** in
Neutral axis factor - cl.10.2.7.3	$\beta_1 = \min(\max(0.85 - 0.05 \times (f'_c - 4 \text{ ksi}) / 1 \text{ ksi}, 0.65), 0.85) =$ **0.825**
Depth to neutral axis	$c = a / \beta_1 =$ **0.583** in
Strain in reinforcement	$\varepsilon_t = 0.003 \times (d - c) / c =$ **0.046822**
	Section is in the tension controlled zone
Strength reduction factor	$\phi_f = \min(\max(0.65 + (\varepsilon_t - 0.002) \times (250 / 3), 0.65), 0.9) =$ **0.9**
Nominal flexural strength	$M_n = A_{bt.prov} \times f_y \times (d - a / 2) =$ **17390** lb_ft/ft
Design flexural strength	$\phi M_n = \phi_f \times M_n =$ **15651** lb_ft/ft
	$M / \phi M_n =$ **0.918**
	PASS - Design flexural strength exceeds factored bending moment
By iteration, reinforcement required by analysis	$A_{bt.des} =$ **0.337** in^2/ft
Minimum area of reinforcement - cl.7.12.2.1	$A_{bt.min} = 0.0018 \times h =$ **0.259** in^2/ft

PASS - Area of reinforcement provided is greater than minimum area of reinforcement required

Rectangular section in shear - Chapter 11

Design shear force	$V =$ **5386** lb/ft
Concrete modification factor - cl.8.6.1	$\lambda =$ **1**
Nominal concrete shear strength - eqn.11-3	$V_c = 2 \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} \times d =$ **15597** lb/ft
Strength reduction factor	$\phi_s =$ **0.75**

Design concrete shear strength - cl.11.4.6.1 $\phi V_c = \phi_s \times V_c =$ **11697** lb/ft

$V / \phi V_c =$ **0.460**

PASS - No shear reinforcement is required

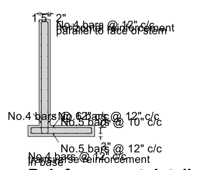
Transverse reinforcement parallel to base

Minimum area of reinforcement - cl.7.12.2.1 $A_{bx.req} = 0.0018 \times t_{base} =$ **0.259** in^2/ft

Transverse reinforcement provided No.4 bars @ 12" c/c each face

Area of transverse reinforcement provided $A_{bx.prov} = 2 \times \pi \times \phi_{bx}^2 / (4 \times s_{bx}) =$ **0.393** in^2/ft

PASS - Area of reinforcement provided is greater than area of reinforcement required



Reinforcement details

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CONCRETE RETAINING WALL SCHEDULE (CRW-3)

RETAINING WALL ANALYSIS

In accordance with International Building Code 2015

Tedds calculation version 2.9.00

Retaining wall details

Stem type	Cantilever
Stem height	h_{stem} = **12.5** ft
Stem thickness	t_{stem} = **12** in
Angle to rear face of stem	α = **90** deg
Stem density	γ_{stem} = **150** pcf
Toe length	l_{toe} = **1** ft
Heel length	l_{heel} = **6** ft
Base thickness	t_{base} = **16** in
Base density	γ_{base} = **150** pcf
Height of retained soil	h_{ret} = **10** ft
Angle of soil surface	β = **0** deg
Depth of cover	d_{cover} = **2.5** ft

Retained soil properties

Soil type	Medium dense well graded sand
Moist density	γ_{mr} = **135** pcf
Saturated density	γ_{sr} = **145** pcf

Base soil properties

Soil type	Medium dense well graded sand
Soil density	γ_b = **115** pcf
Allowable bearing pressure	$P_{bearing}$ = **4600** psf

Loading details

Live surcharge load	$Surcharge_L$ = **250** psf

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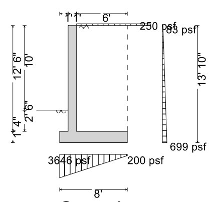

General arrangement

Calculate retaining wall geometry

Base length $l_{base} = l_{toe} + t_{stem} + l_{heel} = \mathbf{8}$ ft

Moist soil height $h_{moist} = h_{soil} = \mathbf{12.5}$ ft

Length of surcharge load $l_{sur} = l_{heel} = \mathbf{6}$ ft

- Distance to vertical component $x_{sur\_v} = l_{base} - l_{heel} / 2 = \mathbf{5}$ ft

Effective height of wall $h_{eff} = h_{base} + d_{cover} + h_{ret} = \mathbf{13.833}$ ft

- Distance to horizontal component $x_{sur\_h} = h_{eff} / 2 = \mathbf{6.917}$ ft

Area of wall stem $A_{stem} = h_{stem} \times t_{stem} = \mathbf{12.5}$ ft^2

- Distance to vertical component $x_{stem} = l_{toe} + t_{stem} / 2 = \mathbf{1.5}$ ft

Area of wall base $A_{base} = l_{base} \times t_{base} = \mathbf{10.667}$ ft^2

- Distance to vertical component $x_{base} = l_{base} / 2 = \mathbf{4}$ ft

Area of moist soil $A_{moist} = h_{moist} \times l_{heel} = \mathbf{75}$ ft^2

- Distance to vertical component $x_{moist\_v} = l_{base} - (h_{moist} \times l_{heel}^2 / 2) / A_{moist} = \mathbf{5}$ ft
- Distance to horizontal component $x_{moist\_h} = h_{eff} / 3 = \mathbf{4.611}$ ft

Area of base soil $A_{pass} = d_{cover} \times l_{toe} = \mathbf{2.5}$ ft^2

- Distance to vertical component $x_{pass\_v} = l_{base} - (d_{cover} \times l_{toe} \times (l_{base} - l_{toe} / 2)) / A_{pass} = \mathbf{0.5}$ ft
- Distance to horizontal component $x_{pass\_h} = (d_{cover} + h_{base}) / 3 = \mathbf{1.278}$ ft

Area of excavated base soil $A_{exc} = h_{pass} \times l_{toe} = \mathbf{2.5}$ ft^2

- Distance to vertical component $x_{exc\_v} = l_{base} - (h_{pass} \times l_{toe} \times (l_{base} - l_{toe} / 2)) / A_{exc} = \mathbf{0.5}$ ft
- Distance to horizontal component $x_{exc\_h} = (h_{pass} + h_{base}) / 3 = \mathbf{1.278}$ ft

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Soil coefficients

Coefficient of friction to back of wall K_{fr} = **0.450**
Coefficient of friction to front of wall K_{fb} = **0.450**
Coefficient of friction beneath base K_{fbb} = **0.450**
Active pressure coefficient K_A = **0.330**
Passive pressure coefficient K_P = **3.000**

From IBC 2015 cl.1807.2.3 Safety factor

Load combination 1 1.0 × Dead + 1.0 × Live + 1.0 × Lateral earth

Sliding check

Vertical forces on wall

Wall stem $F_{stem} = A_{stem} \times \gamma_{stem}$ = **1875** plf
Wall base $F_{base} = A_{base} \times \gamma_{base}$ = **1600** plf
Moist retained soil $F_{moist\_v} = A_{moist} \times \gamma_{mr}$ = **10125** plf
Base soil $F_{exc\_v} = A_{exc} \times \gamma_b$ = **288** plf
Total $F_{total\_v} = F_{stem} + F_{base} + F_{moist\_v} + F_{exc\_v}$ = **13888** plf

Horizontal forces on wall

Surcharge load $F_{sur\_h} = K_A \times Surcharge_L \times h_{eff}$ = **1141** plf
Moist retained soil $F_{moist\_h} = K_A \times \gamma_{mr} \times h_{eff}^2 / 2$ = **4263** plf
Total $F_{total\_h} = F_{moist\_h} + F_{sur\_h}$ = **5404** plf

Check stability against sliding

Base soil resistance $F_{exc\_h} = K_P \times \gamma_b \times (h_{pass} + h_{base})^2 / 2$ = **2535** plf
Base friction $F_{friction} = F_{total\_v} \times K_{fbb}$ = **6249** plf
Resistance to sliding $F_{rest} = F_{exc\_h} + F_{friction}$ = **8784** plf
Factor of safety $FoS_{sl} = F_{rest} / F_{total\_h}$ = **1.626** > 1.5

PASS - Factor of safety against sliding is adequate

Overturning check

Vertical forces on wall

Wall stem $F_{stem} = A_{stem} \times \gamma_{stem}$ = **1875** plf
Wall base $F_{base} = A_{base} \times \gamma_{base}$ = **1600** plf
Moist retained soil $F_{moist\_v} = A_{moist} \times \gamma_{mr}$ = **10125** plf
Base soil $F_{exc\_v} = A_{exc} \times \gamma_b$ = **288** plf
Total $F_{total\_v} = F_{stem} + F_{base} + F_{moist\_v} + F_{exc\_v}$ = **13888** plf

Horizontal forces on wall

Surcharge load $F_{sur\_h} = K_A \times Surcharge_L \times h_{eff}$ = **1141** plf
Moist retained soil $F_{moist\_h} = K_A \times \gamma_{mr} \times h_{eff}^2 / 2$ = **4263** plf
Base soil $F_{exc\_h} = -K_P \times \gamma_b \times (h_{pass} + h_{base})^2 / 2$ = **-2535** plf
Total $F_{total\_h} = F_{moist\_h} + F_{exc\_h} + F_{sur\_h}$ = **2869** plf

Overturning moments on wall

Surcharge load $M_{sur\_OT} = F_{sur\_h} \times x_{sur\_h}$ = **7894** lb_ft/ft
Moist retained soil $M_{moist\_OT} = F_{moist\_h} \times x_{moist\_h}$ = **19655** lb_ft/ft
Total $M_{total\_OT} = M_{moist\_OT} + M_{sur\_OT}$ = **27549** lb_ft/ft

Restoring moments on wall

Wall stem	$M_{stem\_R} = F_{stem} \times x_{stem} =$ **2812** lb_ft/ft
Wall base	$M_{base\_R} = F_{base} \times x_{base} =$ **6400** lb_ft/ft
Moist retained soil	$M_{moist\_R} = F_{moist\_v} \times x_{moist\_v} =$ **50625** lb_ft/ft
Base soil	$M_{exc\_R} = F_{exc\_v} \times x_{exc\_v} - F_{exc\_h} \times x_{exc\_h} =$ **3383** lb_ft/ft
Total	$M_{total\_R} = M_{stem\_R} + M_{base\_R} + M_{moist\_R} + M_{exc\_R} =$ **63220** lb_ft/ft

Check stability against overturning

Factor of safety	$FoS_{ot} = M_{total\_R} / M_{total\_OT} =$ **2.295** > 1.5

PASS - Factor of safety against overturning is adequate

Bearing pressure check

Vertical forces on wall

Wall stem	$F_{stem} = A_{stem} \times \gamma_{stem} =$ **1875** plf
Wall base	$F_{base} = A_{base} \times \gamma_{base} =$ **1600** plf
Surcharge load	$F_{sur\_v} = Surcharge_L \times l_{heel} =$ **1500** plf
Moist retained soil	$F_{moist\_v} = A_{moist} \times \gamma_{mr} =$ **10125** plf
Base soil	$F_{pass\_v} = A_{pass} \times \gamma_b =$ **288** plf
Total	$F_{total\_v} = F_{stem} + F_{base} + F_{moist\_v} + F_{pass\_v} + F_{sur\_v} =$ **15388** plf

Horizontal forces on wall

Surcharge load	$F_{sur\_h} = K_A \times Surcharge_L \times h_{eff} =$ **1141** plf
Moist retained soil	$F_{moist\_h} = K_A \times \gamma_{mr} \times h_{eff}^2 / 2 =$ **4263** plf
Base soil	$F_{pass\_h} = -K_P \times \gamma_b \times (d_{cover} + h_{base})^2 / 2 =$ **-2535** plf
Total	$F_{total\_h} = \max(F_{moist\_h} + F_{pass\_h} + F_{sur\_h} - F_{total\_v} \times K_{fbb}, 0\text{ plf}) =$ **0** plf

Moments on wall

Wall stem	$M_{stem} = F_{stem} \times x_{stem} =$ **2812** lb_ft/ft
Wall base	$M_{base} = F_{base} \times x_{base} =$ **6400** lb_ft/ft
Surcharge load	$M_{sur} = F_{sur\_v} \times x_{sur\_v} - F_{sur\_h} \times x_{sur\_h} =$ **-394** lb_ft/ft
Moist retained soil	$M_{moist} = F_{moist\_v} \times x_{moist\_v} - F_{moist\_h} \times x_{moist\_h} =$ **30970** lb_ft/ft
Base soil	$M_{pass} = F_{pass\_v} \times x_{pass\_v} - F_{pass\_h} \times x_{pass\_h} =$ **3383** lb_ft/ft
Total	$M_{total} = M_{stem} + M_{base} + M_{moist} + M_{pass} + M_{sur} =$ **43171** lb_ft/ft

Check bearing pressure

Distance to reaction	$\bar{x} = M_{total} / F_{total\_v} =$ **2.806** ft
Eccentricity of reaction	$e = \bar{x} - l_{base} / 2 =$ **-1.194** ft
Loaded length of base	$l_{load} = l_{base} =$ **8** ft
Bearing pressure at toe	$q_{toe} = F_{total\_v} / l_{base} \times (1 - 6 \times e / l_{base}) =$ **3646** psf
Bearing pressure at heel	$q_{heel} = F_{total\_v} / l_{base} \times (1 + 6 \times e / l_{base}) =$ **200** psf
Factor of safety	$FoS_{bp} = P_{bearing} / \max(q_{toe}, q_{heel}) =$ **1.262**

PASS - Allowable bearing pressure exceeds maximum applied bearing pressure

<u>RETAINING WALL DESIGN</u>

In accordance with ACI 318-14

Tedds calculation version 2.9.00

Concrete details

Compressive strength of concrete	$f'_c =$ **4500** psi

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date	6/26/2017 by CW

Concrete type Normal weight

Reinforcement details

Yield strength of reinforcement	f_y = **60000** psi
Modulus of elasticity or reinforcement	E_s = **29000000** psi

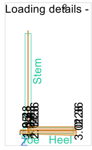
Cover to reinforcement

Front face of stem	c_{sf} = **1.5** in
Rear face of stem	c_{sr} = **2** in
Top face of base	c_{bt} = **2** in
Bottom face of base	c_{bb} = **3** in

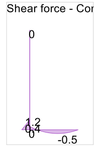
From IBC 2015 cl.1605.2.1 Basic load combinations

Load combination no.1	1.4 × Dead
Load combination no.2	1.2 × Dead + 1.6 × Live + 1.6 × Lateral earth
Load combination no.3	1.2 × Dead + 1.0 × Earthquake + 1.0 × Live + 1.6 × Lateral earth
Load combination no.4	0.9 × Dead + 1.0 × Earthquake + 1.6 × Lateral earth

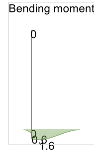
Loading details - Combination No.1 - kips/ft



Shear force - Combination No.1 - kips/ft



Bending moment - Combination No.1 - kips_ft/ft



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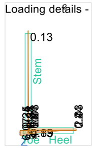
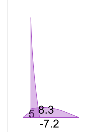
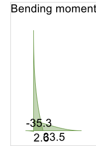
Loading details - Combination No.2 - kips/ft



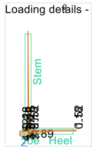
Shear force - Combination No.2 - kips/ft



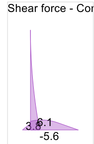
Bending moment - Combination No.2 - kips_ft/ft



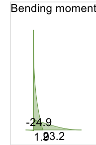
Loading details - Combination No.4 - kips/ft



Shear force - Combination No.4 - kips/ft



Bending moment - Combination No.4 - kips_ft/ft



Check stem design at base of stem

Depth of section $h = 12$ in

Rectangular section in flexure - Section 22.3

Design bending moment combination 2 $M = 33516$ lb_ft/ft

Depth of tension reinforcement $d = h - c_{sr} - \phi_{sr} / 2 = 9.562$ in

Compression reinforcement provided No.4 bars @ 12" c/c

Area of compression reinforcement provided $A_{sf.prov} = \pi \times \phi_{sf}^2 / (4 \times s_{sf}) = 0.196$ in^2/ft

Tension reinforcement provided No.7 bars @ 8" c/c

Area of tension reinforcement provided $A_{sr.prov} = \pi \times \phi_{sr}^2 / (4 \times s_{sr}) = 0.902$ in^2/ft

Maximum reinforcement spacing - cl.11.7.2 $s_{max} = \min(18 \text{ in}, 3 \times h) = 18$ in

PASS - Reinforcement is adequately spaced

Depth of compression block $a = A_{sr.prov} \times f_y / (0.85 \times f'_c) = 1.179$ in

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Neutral axis factor - cl.22.2.2.4.3	β_1 = min(max(0.85 - 0.05 × (f'$_c$ - 4 ksi) / 1 ksi, 0.65), 0.85) = **0.825**
Depth to neutral axis	c = a / β_1 = **1.429** in
Strain in reinforcement	ε_t = 0.003 × (d - c) / c = **0.017073**
	Section is in the tension controlled zone
Strength reduction factor	ϕ_f = min(max(0.65 + (ε_t - 0.002) × (250 / 3), 0.65), 0.9) = **0.9**
Nominal flexural strength	M_n = $A_{sr.prov}$ × f_y × (d - a / 2) = **40467** lb_ft/ft
Design flexural strength	ϕM_n = ϕ_f × M_n = **36421** lb_ft/ft
	M / ϕM_n = **0.920**
	PASS - Design flexural strength exceeds factored bending moment
By iteration, reinforcement required by analysis	$A_{sr.des}$ = **0.825** in^2/ft
Minimum area of reinforcement - cl.9.6.1.2	$A_{sr.min}$ = max(3 × √(f'$_c$ × 1 psi), 200 psi) × d / f_y = **0.385** in^2/ft
	PASS - Area of reinforcement provided is greater than minimum area of reinforcement required

Rectangular section in shear - Section 22.5

Design shear force	V = **7219** lb/ft
Concrete modification factor - cl.19.2.4	λ = **1**
Nominal concrete shear strength - eqn.22.5.5.1	V_c = 2 × λ × √(f'$_c$ × 1 psi) × d = **15395** lb/ft
Strength reduction factor	ϕ_s = **0.75**
Design concrete shear strength - cl.11.5.1.1	ϕV_c = ϕ_s × V_c = **11546** lb/ft
	V / ϕV_c = **0.625**
	PASS - No shear reinforcement is required

Horizontal reinforcement parallel to face of stem

Minimum area of reinforcement - cl.11.6.1	$A_{sx.req}$ = 0.002 × t_{stem} = **0.288** in^2/ft
Transverse reinforcement provided	No.4 bars @ 12" c/c each face
Area of transverse reinforcement provided	$A_{sx.prov}$ = 2 × π × ϕ_{sx}^2 / (4 × s_{sx}) = **0.393** in^2/ft
	PASS - Area of reinforcement provided is greater than area of reinforcement required

Check base design at toe

Depth of section	h = **16** in

Rectangular section in flexure - Section 22.3

Design bending moment combination 2	M = **2583** lb_ft/ft
Depth of tension reinforcement	d = h - c_{bb} - ϕ_{bb} / 2 = **12.688** in
Compression reinforcement provided	No.6 bars @ 8" c/c
Area of compression reinforcement provided	$A_{bt.prov}$ = π × ϕ_{bt}^2 / (4 × s_{bt}) = **0.663** in^2/ft
Tension reinforcement provided	No.5 bars @ 10" c/c
Area of tension reinforcement provided	$A_{bb.prov}$ = π × ϕ_{bb}^2 / (4 × s_{bb}) = **0.368** in^2/ft
Maximum reinforcement spacing - cl.7.7.2.3	s_{max} = min(18 in, 3 × h) = **18** in
	PASS - Reinforcement is adequately spaced
Depth of compression block	a = $A_{bb.prov}$ × f_y / (0.85 × f'$_c$) = **0.481** in
Neutral axis factor - cl.22.2.2.4.3	β_1 = min(max(0.85 - 0.05 × (f'$_c$ - 4 ksi) / 1 ksi, 0.65), 0.85) = **0.825**
Depth to neutral axis	c = a / β_1 = **0.583** in
Strain in reinforcement	ε_t = 0.003 × (d - c) / c = **0.06225**
	Section is in the tension controlled zone
Strength reduction factor	ϕ_f = min(max(0.65 + (ε_t - 0.002) × (250 / 3), 0.65), 0.9) = **0.9**
Nominal flexural strength	M_n = $A_{bb.prov}$ × f_y × (d - a / 2) = **22912** lb_ft/ft

Design flexural strength	$\phi M_n = \phi_f \times M_n =$ **20621** lb_ft/ft
	$M / \phi M_n =$ **0.125**
	PASS - Design flexural strength exceeds factored bending moment
By iteration, reinforcement required by analysis	$A_{bb.des} =$ **0.045** in^2/ft
Minimum area of reinforcement - cl.7.6.1.1	$A_{bb.min} = 0.0018 \times h =$ **0.346** in^2/ft

PASS - Area of reinforcement provided is greater than minimum area of reinforcement required

Rectangular section in shear - Section 22.5

Design shear force	$V =$ **5005** lb/ft
Concrete modification factor - cl.19.2.4	$\lambda =$ **1**
Nominal concrete shear strength - eqn.22.5.5.1	$V_c = 2 \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} \times d =$ **20426** lb/ft
Strength reduction factor	$\phi_s =$ **0.75**
Design concrete shear strength - cl.7.6.3.1	$\phi V_c = \phi_s \times V_c =$ **15320** lb/ft
	$V / \phi V_c =$ **0.327**

PASS - No shear reinforcement is required

Check base design at heel

Depth of section	$h =$ **16** in

Rectangular section in flexure - Section 22.3

Design bending moment combination 2	$M =$ **35344** lb_ft/ft
Depth of tension reinforcement	$d = h - c_{bt} - \phi_{bt} / 2 =$ **13.625** in
Compression reinforcement provided	No.5 bars @ 10" c/c
Area of compression reinforcement provided	$A_{bb.prov} = \pi \times \phi_{bb}^2 / (4 \times s_{bb}) =$ **0.368** in^2/ft
Tension reinforcement provided	No.6 bars @ 8" c/c
Area of tension reinforcement provided	$A_{bt.prov} = \pi \times \phi_{bt}^2 / (4 \times s_{bt}) =$ **0.663** in^2/ft
Maximum reinforcement spacing - cl.7.7.2.3	$s_{max} = \min(18 \text{ in}, 3 \times h) =$ **18** in
	PASS - Reinforcement is adequately spaced
Depth of compression block	$a = A_{bt.prov} \times f_y / (0.85 \times f'_c) =$ **0.866** in
Neutral axis factor - cl.22.2.2.4.3	$\beta_1 = \min(\max(0.85 - 0.05 \times (f'_c - 4 \text{ ksi}) / 1 \text{ ksi}, 0.65), 0.85) =$ **0.825**
Depth to neutral axis	$c = a / \beta_1 =$ **1.05** in
Strain in reinforcement	$\varepsilon_t = 0.003 \times (d - c) / c =$ **0.035929**
	Section is in the tension controlled zone
Strength reduction factor	$\phi_f = \min(\max(0.65 + (\varepsilon_t - 0.002) \times (250 / 3), 0.65), 0.9) =$ **0.9**
Nominal flexural strength	$M_n = A_{bt.prov} \times f_y \times (d - a / 2) =$ **43710** lb_ft/ft
Design flexural strength	$\phi M_n = \phi_f \times M_n =$ **39339** lb_ft/ft
	$M / \phi M_n =$ **0.898**
	PASS - Design flexural strength exceeds factored bending moment
By iteration, reinforcement required by analysis	$A_{bt.des} =$ **0.593** in^2/ft
Minimum area of reinforcement - cl.7.6.1.1	$A_{bt.min} = 0.0018 \times h =$ **0.346** in^2/ft

PASS - Area of reinforcement provided is greater than minimum area of reinforcement required

Rectangular section in shear - Section 22.5

Design shear force	$V =$ **8259** lb/ft
Concrete modification factor - cl.19.2.4	$\lambda =$ **1**
Nominal concrete shear strength - eqn.22.5.5.1	$V_c = 2 \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} \times d =$ **21936** lb/ft
Strength reduction factor	$\phi_s =$ **0.75**

Design concrete shear strength - cl.7.6.3.1 $\phi V_c = \phi_s \times V_c =$ **16452** lb/ft

$V / \phi V_c =$ **0.502**

PASS - No shear reinforcement is required

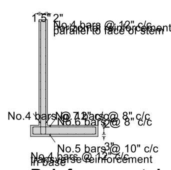
Transverse reinforcement parallel to base

Minimum area of reinforcement - cl.76.1.1 $A_{bx.req} = 0.0018 \times t_{base} =$ **0.346** in^2/ft

Transverse reinforcement provided No.4 bars @ 12" c/c each face

Area of transverse reinforcement provided $A_{bx.prov} = 2 \times \pi \times \phi_{bx}^2 / (4 \times s_{bx}) =$ **0.393** in^2/ft

PASS - Area of reinforcement provided is greater than area of reinforcement required



Reinforcement details

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Retaining Walls
location Eden, Utah
date 6/26/2017 by CW

CONCRETE RETAINING WALL SCHEDULE (CRW-4)

RETAINING WALL ANALYSIS

In accordance with International Building Code 2015

Tedds calculation version 2.9.00

Retaining wall details

Stem type	Cantilever
Stem height	h_{stem} = **16.5** ft
Stem thickness	t_{stem} = **16** in
Angle to rear face of stem	α = **90** deg
Stem density	γ_{stem} = **150** pcf
Toe length	l_{toe} = **1.333** ft
Heel length	l_{heel} = **9.333** ft
Base thickness	t_{base} = **18** in
Base density	γ_{base} = **150** pcf
Height of retained soil	h_{ret} = **14** ft
Angle of soil surface	β = **0** deg
Depth of cover	d_{cover} = **2** ft
Height of water	h_{water} = **6** ft
Water density	γ_w = **62** pcf

Retained soil properties

Soil type	Medium dense well graded sand
Moist density	γ_{mr} = **135** pcf
Saturated density	γ_{sr} = **145** pcf

Base soil properties

Soil type	Medium dense well graded sand
Soil density	γ_b = **115** pcf
Allowable bearing pressure	$P_{bearing}$ = **4600** psf

Loading details

Live surcharge load	$Surcharge_L$ = **100** psf

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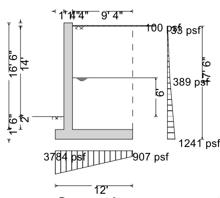

General arrangement

Calculate retaining wall geometry

Base length	$l_{base} = l_{toe} + t_{stem} + l_{heel} = \mathbf{12}$ ft
Saturated soil height	$h_{sat} = h_{water} + d_{cover} = \mathbf{8}$ ft
Moist soil height	$h_{moist} = h_{ret} - h_{water} = \mathbf{8}$ ft
Length of surcharge load	$l_{sur} = l_{heel} = \mathbf{9.333}$ ft
- Distance to vertical component	$x_{sur\_v} = l_{base} - l_{heel} / 2 = \mathbf{7.333}$ ft
Effective height of wall	$h_{eff} = h_{base} + d_{cover} + h_{ret} = \mathbf{17.5}$ ft
- Distance to horizontal component	$x_{sur\_h} = h_{eff} / 2 = \mathbf{8.75}$ ft
Area of wall stem	$A_{stem} = h_{stem} \times t_{stem} = \mathbf{22}$ ft^2
- Distance to vertical component	$x_{stem} = l_{toe} + t_{stem} / 2 = \mathbf{2}$ ft
Area of wall base	$A_{base} = l_{base} \times t_{base} = \mathbf{18}$ ft^2
- Distance to vertical component	$x_{base} = l_{base} / 2 = \mathbf{6}$ ft
Area of saturated soil	$A_{sat} = h_{sat} \times l_{heel} = \mathbf{74.667}$ ft^2
- Distance to vertical component	$x_{sat\_v} = l_{base} - (h_{sat} \times l_{heel}^2 / 2) / A_{sat} = \mathbf{7.333}$ ft
- Distance to horizontal component	$x_{sat\_h} = (h_{sat} + h_{base}) / 3 = \mathbf{3.167}$ ft
Area of water	$A_{water} = h_{sat} \times l_{heel} = \mathbf{74.667}$ ft^2
- Distance to vertical component	$x_{water\_v} = l_{base} - (h_{sat} \times l_{heel}^2 / 2) / A_{sat} = \mathbf{7.333}$ ft
- Distance to horizontal component	$x_{water\_h} = (h_{sat} + h_{base}) / 3 = \mathbf{3.167}$ ft
Area of moist soil	$A_{moist} = h_{moist} \times l_{heel} = \mathbf{74.667}$ ft^2
- Distance to vertical component	$x_{moist\_v} = l_{base} - (h_{moist} \times l_{heel}^2 / 2) / A_{moist} = \mathbf{7.333}$ ft

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- Distance to horizontal component	$x_{moist\_h} = (h_{moist} \times (t_{base} + h_{sat} + h_{moist} / 3) / 2 + (h_{sat} + t_{base})^2/2) / (h_{sat} + t_{base} + h_{moist} / 2) =$ **6.948** ft
Area of base soil	$A_{pass} = d_{cover} \times l_{toe} =$ **2.667** ft^2
- Distance to vertical component	$x_{pass\_v} = l_{base} - (d_{cover} \times l_{toe} \times (l_{base} - l_{toe} / 2)) / A_{pass} =$ **0.667** ft
- Distance to horizontal component	$x_{pass\_h} = (d_{cover} + h_{base}) / 3 =$ **1.167** ft
Area of excavated base soil	$A_{exc} = h_{pass} \times l_{toe} =$ **2.667** ft^2
- Distance to vertical component	$x_{exc\_v} = l_{base} - (h_{pass} \times l_{toe} \times (l_{base} - l_{toe} / 2)) / A_{exc} =$ **0.667** ft
- Distance to horizontal component	$x_{exc\_h} = (h_{pass} + h_{base}) / 3 =$ **1.167** ft

Soil coefficients

Coefficient of friction to back of wall	$K_{fr} =$ **0.450**
Coefficient of friction to front of wall	$K_{fb} =$ **0.450**
Coefficient of friction beneath base	$K_{fbb} =$ **0.450**
Active pressure coefficient	$K_A =$ **0.330**
Passive pressure coefficient	$K_P =$ **3.000**

From IBC 2015 cl.1807.2.3 Safety factor

Load combination 1 — 1.0 × Dead + 1.0 × Live + 1.0 × Lateral earth

Sliding check

Vertical forces on wall

Wall stem	$F_{stem} = A_{stem} \times \gamma_{stem} =$ **3300** plf
Wall base	$F_{base} = A_{base} \times \gamma_{base} =$ **2700** plf
Saturated retained soil	$F_{sat\_v} = A_{sat} \times (\gamma_{sr} - \gamma_w) =$ **6175** plf
Water	$F_{water\_v} = A_{water} \times \gamma_w =$ **4652** plf
Moist retained soil	$F_{moist\_v} = A_{moist} \times \gamma_{mr} =$ **10080** plf
Base soil	$F_{exc\_v} = A_{exc} \times \gamma_b =$ **307** plf
Total	$F_{total\_v} = F_{stem} + F_{base} + F_{sat\_v} + F_{moist\_v} + F_{exc\_v} + F_{water\_v} =$ **27213** plf

Horizontal forces on wall

Surcharge load	$F_{sur\_h} = K_A \times Surcharge_L \times h_{eff} =$ **578** plf
Saturated retained soil	$F_{sat\_h} = K_A \times (\gamma_{sr} - \gamma_w) \times (h_{sat} + h_{base})^2 / 2 =$ **1232** plf
Water	$F_{water\_h} = \gamma_w \times (h_{water} + d_{cover} + h_{base})^2 / 2 =$ **2811** plf
Moist retained soil	$F_{moist\_h} = K_A \times \gamma_{mr} \times ((h_{eff} - h_{sat} - h_{base})^2 / 2 + (h_{eff} - h_{sat} - h_{base}) \times (h_{sat} + h_{base})) =$ **4811** plf
Total	$F_{total\_h} = F_{sat\_h} + F_{moist\_h} + F_{water\_h} + F_{sur\_h} =$ **9432** plf

Check stability against sliding

Base soil resistance	$F_{exc\_h} = K_P \times \gamma_b \times (h_{pass} + h_{base})^2 / 2 =$ **2113** plf
Base friction	$F_{friction} = F_{total\_v} \times K_{fbb} =$ **12246** plf
Resistance to sliding	$F_{rest} = F_{exc\_h} + F_{friction} =$ **14359** plf
Factor of safety	$FoS_{sl} = F_{rest} / F_{total\_h} =$ **1.522** > 1.5

PASS - Factor of safety against sliding is adequate

Overturning check

Vertical forces on wall

Wall stem	$F_{stem} = A_{stem} \times \gamma_{stem} =$ **3300** plf
Wall base	$F_{base} = A_{base} \times \gamma_{base} =$ **2700** plf

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Saturated retained soil	$F_{sat\_v} = A_{sat} \times (\gamma_{sr} - \gamma_w) =$ **6175** plf
Water	$F_{water\_v} = A_{water} \times \gamma_w =$ **4652** plf
Moist retained soil	$F_{moist\_v} = A_{moist} \times \gamma_{mr} =$ **10080** plf
Base soil	$F_{exc\_v} = A_{exc} \times \gamma_b =$ **307** plf
Total	$F_{total\_v} = F_{stem} + F_{base} + F_{sat\_v} + F_{moist\_v} + F_{exc\_v} + F_{water\_v} =$ **27213** plf

Horizontal forces on wall

Surcharge load	$F_{sur\_h} = K_A \times Surcharge_L \times h_{eff} =$ **578** plf
Saturated retained soil	$F_{sat\_h} = K_A \times (\gamma_{sr} - \gamma_w) \times (h_{sat} + h_{base})^2 / 2 =$ **1232** plf
Water	$F_{water\_h} = \gamma_w \times (h_{water} + d_{cover} + h_{base})^2 / 2 =$ **2811** plf
Moist retained soil	$F_{moist\_h} = K_A \times \gamma_{mr} \times ((h_{eff} - h_{sat} - h_{base})^2 / 2 + (h_{eff} - h_{sat} - h_{base}) \times (h_{sat} + h_{base})) =$ **4811** plf
Base soil	$F_{exc\_h} = -K_P \times \gamma_b \times (h_{pass} + h_{base})^2 / 2 =$ **-2113** plf
Total	$F_{total\_h} = F_{sat\_h} + F_{moist\_h} + F_{exc\_h} + F_{water\_h} + F_{sur\_h} =$ **7319** plf

Overturning moments on wall

Surcharge load	$M_{sur\_OT} = F_{sur\_h} \times x_{sur\_h} =$ **5053** lb_ft/ft
Saturated retained soil	$M_{sat\_OT} = F_{sat\_h} \times x_{sat\_h} =$ **3900** lb_ft/ft
Water	$M_{water\_OT} = F_{water\_h} \times x_{water\_h} =$ **8902** lb_ft/ft
Moist retained soil	$M_{moist\_OT} = F_{moist\_h} \times x_{moist\_h} =$ **33427** lb_ft/ft
Total	$M_{total\_OT} = M_{sat\_OT} + M_{moist\_OT} + M_{water\_OT} + M_{sur\_OT} =$ **51283** lb_ft/ft

Restoring moments on wall

Wall stem	$M_{stem\_R} = F_{stem} \times x_{stem} =$ **6600** lb_ft/ft
Wall base	$M_{base\_R} = F_{base} \times x_{base} =$ **16200** lb_ft/ft
Saturated retained soil	$M_{sat\_R} = F_{sat\_v} \times x_{sat\_v} =$ **45283** lb_ft/ft
Water	$M_{water\_R} = F_{water\_v} \times x_{water\_v} =$ **34113** lb_ft/ft
Moist retained soil	$M_{moist\_R} = F_{moist\_v} \times x_{moist\_v} =$ **73920** lb_ft/ft
Base soil	$M_{exc\_R} = F_{exc\_v} \times x_{exc\_v} - F_{exc\_h} \times x_{exc\_h} =$ **2670** lb_ft/ft
Total	$M_{total\_R} = M_{stem\_R} + M_{base\_R} + M_{sat\_R} + M_{moist\_R} + M_{exc\_R} + M_{water\_R} =$ **178786** lb_ft/ft

Check stability against overturning

Factor of safety	$FoS_{ot} = M_{total\_R} / M_{total\_OT} =$ **3.486** > 1.5

PASS - Factor of safety against overturning is adequate

Bearing pressure check

Vertical forces on wall

Wall stem	$F_{stem} = A_{stem} \times \gamma_{stem} =$ **3300** plf
Wall base	$F_{base} = A_{base} \times \gamma_{base} =$ **2700** plf
Surcharge load	$F_{sur\_v} = Surcharge_L \times l_{heel} =$ **933** plf
Saturated retained soil	$F_{sat\_v} = A_{sat} \times (\gamma_{sr} - \gamma_w) =$ **6175** plf
Water	$F_{water\_v} = A_{water} \times \gamma_w =$ **4652** plf
Moist retained soil	$F_{moist\_v} = A_{moist} \times \gamma_{mr} =$ **10080** plf
Base soil	$F_{pass\_v} = A_{pass} \times \gamma_b =$ **307** plf
Total	$F_{total\_v} = F_{stem} + F_{base} + F_{sat\_v} + F_{moist\_v} + F_{pass\_v} + F_{water\_v} + F_{sur\_v} =$ **28147** plf

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Horizontal forces on wall

Surcharge load	$F_{sur\_h} = K_A \times Surcharge_L \times h_{eff}$ = **578** plf
Saturated retained soil	$F_{sat\_h} = K_A \times (\gamma_{sr} - \gamma_w) \times (h_{sat} + h_{base})^2 / 2$ = **1232** plf
Water	$F_{water\_h} = \gamma_w \times (h_{water} + d_{cover} + h_{base})^2 / 2$ = **2811** plf
Moist retained soil	$F_{moist\_h} = K_A \times \gamma_{mr} \times ((h_{eff} - h_{sat} - h_{base})^2 / 2 + (h_{eff} - h_{sat} - h_{base}) \times (h_{sat} + h_{base}))$ = **4811** plf
Base soil	$F_{pass\_h} = -K_P \times \gamma_b \times (d_{cover} + h_{base})^2 / 2$ = **-2113** plf
Total	$F_{total\_h} = \max(F_{sat\_h} + F_{moist\_h} + F_{pass\_h} + F_{water\_h} + F_{sur\_h} - F_{total\_v} \times K_{fbb}, 0\ plf)$ = **0** plf

Moments on wall

Wall stem	$M_{stem} = F_{stem} \times x_{stem}$ = **6600** lb_ft/ft
Wall base	$M_{base} = F_{base} \times x_{base}$ = **16200** lb_ft/ft
Surcharge load	$M_{sur} = F_{sur\_v} \times x_{sur\_v} - F_{sur\_h} \times x_{sur\_h}$ = **1791** lb_ft/ft
Saturated retained soil	$M_{sat} = F_{sat\_v} \times x_{sat\_v} - F_{sat\_h} \times x_{sat\_h}$ = **41383** lb_ft/ft
Water	$M_{water} = F_{water\_v} \times x_{water\_v} - F_{water\_h} \times x_{water\_h}$ = **25210** lb_ft/ft
Moist retained soil	$M_{moist} = F_{moist\_v} \times x_{moist\_v} - F_{moist\_h} \times x_{moist\_h}$ = **40493** lb_ft/ft
Base soil	$M_{pass} = F_{pass\_v} \times x_{pass\_v} - F_{pass\_h} \times x_{pass\_h}$ = **2670** lb_ft/ft
Total	$M_{total} = M_{stem} + M_{base} + M_{sat} + M_{moist} + M_{pass} + M_{water} + M_{sur}$ = **134347** lb_ft/ft

Check bearing pressure

Distance to reaction	$\bar{x} = M_{total} / F_{total\_v}$ = **4.773** ft
Eccentricity of reaction	$e = \bar{x} - l_{base} / 2$ = **-1.227** ft
Loaded length of base	$l_{load} = l_{base}$ = **12** ft
Bearing pressure at toe	$q_{toe} = F_{total\_v} / l_{base} \times (1 - 6 \times e / l_{base})$ = **3784** psf
Bearing pressure at heel	$q_{heel} = F_{total\_v} / l_{base} \times (1 + 6 \times e / l_{base})$ = **907** psf
Factor of safety	$FoS_{bp} = P_{bearing} / \max(q_{toe}, q_{heel})$ = **1.216**

PASS - Allowable bearing pressure exceeds maximum applied bearing pressure

<u>RETAINING WALL DESIGN</u>

In accordance with ACI 318-14

Tedds calculation version 2.9.00

Concrete details

Compressive strength of concrete	f'_c = **4500** psi
Concrete type	Normal weight

Reinforcement details

Yield strength of reinforcement	f_y = **60000** psi
Modulus of elasticity or reinforcement	E_s = **29000000** psi

Cover to reinforcement

Front face of stem	c_{sf} = **1.5** in
Rear face of stem	c_{sr} = **2** in
Top face of base	c_{bt} = **2** in
Bottom face of base	c_{bb} = **3** in

From IBC 2015 cl.1605.2.1 Basic load combinations

Load combination no.1	$1.4 \times$ Dead

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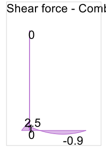
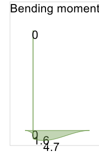
Load combination no.2	1.2 × Dead + 1.6 × Live + 1.6 × Lateral earth
Load combination no.3	1.2 × Dead + 1.0 × Earthquake + 1.0 × Live + 1.6 × Lateral earth
Load combination no.4	0.9 × Dead + 1.0 × Earthquake + 1.6 × Lateral earth

Loading details - Combination No.1 - kips/ft

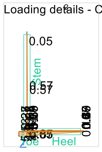
Shear force - Combination No.1 - kips/ft



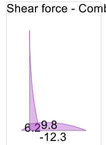
Bending moment - Combination No.1 - kips_ft/ft



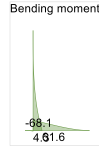
Loading details - Combination No.2 - kips/ft



Shear force - Combination No.2 - kips/ft



Bending moment - Combination No.2 - kips_ft/ft



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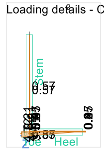
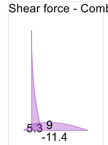
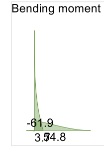
Loading details - Combination No.4 - kips/ft



Shear force - Combination No.4 - kips/ft



Bending moment - Combination No.4 - kips_ft/ft



Check stem design at base of stem

Depth of section $h = 16$ in

Rectangular section in flexure - Section 22.3

Design bending moment combination 2 $M = 61568$ lb_ft/ft

Depth of tension reinforcement $d = h - c_{sr} - \phi_{sr} / 2 = 13.5$ in

Compression reinforcement provided No.4 bars @ 12" c/c

Area of compression reinforcement provided $A_{sf.prov} = \pi \times \phi_{sf}^2 / (4 \times s_{sf}) = 0.196$ in^2/ft

Tension reinforcement provided No.8 bars @ 8" c/c

Area of tension reinforcement provided $A_{sr.prov} = \pi \times \phi_{sr}^2 / (4 \times s_{sr}) = 1.178$ in^2/ft

Maximum reinforcement spacing - cl.11.7.2 $s_{max} = \min(18 \text{ in}, 3 \times h) = 18$ in

PASS - Reinforcement is adequately spaced

Depth of compression block $a = A_{sr.prov} \times f_y / (0.85 \times f'_c) = 1.54$ in

Neutral axis factor - cl.22.2.2.4.3 $\beta_1 = \min(\max(0.85 - 0.05 \times (f'_c - 4 \text{ ksi}) / 1 \text{ ksi}, 0.65), 0.85) = 0.825$

Depth to neutral axis $c = a / \beta_1 = 1.867$ in

Strain in reinforcement $\varepsilon_t = 0.003 \times (d - c) / c = 0.018696$

Section is in the tension controlled zone

Strength reduction factor $\phi_f = \min(\max(0.65 + (\varepsilon_t - 0.002) \times (250 / 3), 0.65), 0.9) = 0.9$

Nominal flexural strength $M_n = A_{sr.prov} \times f_y \times (d - a / 2) = 74986$ lb_ft/ft

Design flexural strength $\phi M_n = \phi_f \times M_n = 67487$ lb_ft/ft

$M / \phi M_n = 0.912$

PASS - Design flexural strength exceeds factored bending moment

By iteration, reinforcement required by analysis $A_{sr.des} = 1.069$ in^2/ft

Minimum area of reinforcement - cl.9.6.1.2 $A_{sr.min} = \max(3 \times \sqrt{(f'_c \times 1 \text{ psi})}, 200 \text{ psi}) \times d / f_y = 0.543$ in^2/ft

PASS - Area of reinforcement provided is greater than minimum area of reinforcement required

Rectangular section in shear - Section 22.5

Design shear force $V = 12275$ lb/ft

Concrete modification factor - cl.19.2.4 $\lambda = 1$

Nominal concrete shear strength - eqn.22.5.5.1 $V_c = 2 \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} \times d = 21735$ lb/ft

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Strength reduction factor ϕ_s = **0.75**

Design concrete shear strength - cl.11.5.1.1 $\phi V_c = \phi_s \times V_c$ = **16301** lb/ft

$V / \phi V_c$ = **0.753**

PASS - No shear reinforcement is required

Check stem design at 8 ft

Depth of section h = **16** in

Rectangular section in flexure - Section 22.3

Design bending moment combination 2 M = **7772** lb_ft/ft

Depth of tension reinforcement $d = h - c_{sr} - \phi_{sr1} / 2$ = **13.5** in

Compression reinforcement provided No.4 bars @ 12" c/c

Area of compression reinforcement provided $A_{sf1.prov} = \pi \times \phi_{sf1}^2 / (4 \times s_{sf1})$ = **0.196** in^2/ft

Tension reinforcement provided No.8 bars @ 16" c/c

Area of tension reinforcement provided $A_{sr1.prov} = \pi \times \phi_{sr1}^2 / (4 \times s_{sr1})$ = **0.589** in^2/ft

Maximum reinforcement spacing - cl.11.7.2 s_{max} = min(18 in, 3 × h) = **18** in

PASS - Reinforcement is adequately spaced

Depth of compression block $a = A_{sr1.prov} \times f_y / (0.85 \times f'_c)$ = **0.77** in

Neutral axis factor - cl.22.2.2.4.3 β_1 = min(max(0.85 - 0.05 × (f'c - 4 ksi) / 1 ksi, 0.65), 0.85) = **0.825**

Depth to neutral axis $c = a / \beta_1$ = **0.933** in

Strain in reinforcement $\varepsilon_t = 0.003 \times (d - c) / c$ = **0.040393**

Section is in the tension controlled zone

Strength reduction factor ϕ_f = min(max(0.65 + (ε_t - 0.002) × (250 / 3), 0.65), 0.9) = **0.9**

Nominal flexural strength $M_n = A_{sr1.prov} \times f_y \times (d - a / 2)$ = **38627** lb_ft/ft

Design flexural strength $\phi M_n = \phi_f \times M_n$ = **34764** lb_ft/ft

$M / \phi M_n$ = **0.224**

PASS - Design flexural strength exceeds factored bending moment

By iteration, reinforcement required by analysis $A_{sr1.des}$ = **0.129** in^2/ft

Minimum area of reinforcement - cl.9.6.1.3 $A_{sr1.mod} = 4 \times A_{sr1.des} / 3$ = **0.172** in^2/ft

PASS - Area of reinforcement provided is greater than minimum area of reinforcement required

Rectangular section in shear - Section 22.5

Design shear force V = **2703** lb/ft

Concrete modification factor - cl.19.2.4 λ = **1**

Nominal concrete shear strength - eqn.22.5.5.1 $V_c = 2 \times \lambda \times \sqrt{(f'_c \times 1\ psi)} \times d$ = **21735** lb/ft

Strength reduction factor ϕ_s = **0.75**

Design concrete shear strength - cl.11.5.1.1 $\phi V_c = \phi_s \times V_c$ = **16301** lb/ft

$V / \phi V_c$ = **0.166**

PASS - No shear reinforcement is required

Horizontal reinforcement parallel to face of stem

Minimum area of reinforcement - cl.11.6.1 $A_{sx.req} = 0.002 \times t_{stem}$ = **0.384** in^2/ft

Transverse reinforcement provided No.4 bars @ 12" c/c each face

Area of transverse reinforcement provided $A_{sx.prov} = 2 \times \pi \times \phi_{sx}^2 / (4 \times s_{sx})$ = **0.393** in^2/ft

PASS - Area of reinforcement provided is greater than area of reinforcement required

Check base design at heel

Depth of section h = **18** in

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Rectangular section in flexure - Section 22.3

Design bending moment combination 1	M = **4719** lb_ft/ft
Depth of tension reinforcement	$d = h - c_{bb} - \phi_{bb} / 2$ = **14.625** in
Compression reinforcement provided	No.8 bars @ 9" c/c
Area of compression reinforcement provided	$A_{bt.prov} = \pi \times \phi_{bt}^2 / (4 \times s_{bt})$ = **1.047** in²/ft
Tension reinforcement provided	No.6 bars @ 12" c/c
Area of tension reinforcement provided	$A_{bb.prov} = \pi \times \phi_{bb}^2 / (4 \times s_{bb})$ = **0.442** in²/ft
Maximum reinforcement spacing - cl.7.7.2.3	s_{max} = min(18 in, 3 × h) = **18** in

PASS - Reinforcement is adequately spaced

Depth of compression block	$a = A_{bb.prov} \times f_y / (0.85 \times f'_c)$ = **0.577** in
Neutral axis factor - cl.22.2.2.4.3	β_1 = min(max(0.85 - 0.05 × (f'c - 4 ksi) / 1 ksi, 0.65), 0.85) = **0.825**
Depth to neutral axis	$c = a / \beta_1$ = **0.7** in
Strain in reinforcement	$\varepsilon_t = 0.003 \times (d - c) / c$ = **0.059679**

Section is in the tension controlled zone

Strength reduction factor	ϕ_f = min(max(0.65 + (ε_t - 0.002) × (250 / 3), 0.65), 0.9) = **0.9**
Nominal flexural strength	$M_n = A_{bb.prov} \times f_y \times (d - a / 2)$ = **31668** lb_ft/ft
Design flexural strength	$\phi M_n = \phi_f \times M_n$ = **28501** lb_ft/ft
	$M / \phi M_n$ = **0.166**

PASS - Design flexural strength exceeds factored bending moment

By iteration, reinforcement required by analysis	$A_{bb.des}$ = **0.072** in²/ft
Minimum area of reinforcement - cl.7.6.1.1	$A_{bb.min} = 0.0018 \times h$ = **0.389** in²/ft

PASS - Area of reinforcement provided is greater than minimum area of reinforcement required

Rectangular section in shear - Section 22.5

Design shear force	V = **6247** lb/ft
Concrete modification factor - cl.19.2.4	λ = **1**
Nominal concrete shear strength - eqn.22.5.5.1	$V_c = 2 \times \lambda \times \sqrt{f'_c \times 1 \text{ psi}} \times d$ = **23546** lb/ft
Strength reduction factor	ϕ_s = **0.75**
Design concrete shear strength - cl.7.6.3.1	$\phi V_c = \phi_s \times V_c$ = **17659** lb/ft
	$V / \phi V_c$ = **0.354**

PASS - No shear reinforcement is required

Check base design at heel

Depth of section	h = **18** in

Rectangular section in flexure - Section 22.3

Design bending moment combination 2	M = **68141** lb_ft/ft
Depth of tension reinforcement	$d = h - c_{bt} - \phi_{bt} / 2$ = **15.5** in
Compression reinforcement provided	No.6 bars @ 12" c/c
Area of compression reinforcement provided	$A_{bb.prov} = \pi \times \phi_{bb}^2 / (4 \times s_{bb})$ = **0.442** in²/ft
Tension reinforcement provided	No.8 bars @ 9" c/c
Area of tension reinforcement provided	$A_{bt.prov} = \pi \times \phi_{bt}^2 / (4 \times s_{bt})$ = **1.047** in²/ft
Maximum reinforcement spacing - cl.7.7.2.3	s_{max} = min(18 in, 3 × h) = **18** in

PASS - Reinforcement is adequately spaced

Depth of compression block	$a = A_{bt.prov} \times f_y / (0.85 \times f'_c)$ = **1.369** in
Neutral axis factor - cl.22.2.2.4.3	β_1 = min(max(0.85 - 0.05 × (f'c - 4 ksi) / 1 ksi, 0.65), 0.85) = **0.825**

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Depth to neutral axis	$c = a / \beta_1 =$ **1.659** in
Strain in reinforcement	$\varepsilon_t = 0.003 \times (d - c) / c =$ **0.025025**

Section is in the tension controlled zone

Strength reduction factor	$\phi_f = \min(\max(0.65 + (\varepsilon_t - 0.002) \times (250 / 3), 0.65), 0.9) =$ **0.9**
Nominal flexural strength	$M_n = A_{bt.prov} \times f_y \times (d - a / 2) =$ **77574** lb_ft/ft
Design flexural strength	$\phi M_n = \phi_f \times M_n =$ **69817** lb_ft/ft
	$M / \phi M_n =$ **0.976**

PASS - Design flexural strength exceeds factored bending moment

By iteration, reinforcement required by analysis	$A_{bt.des} =$ **1.021** in^2/ft
Minimum area of reinforcement - cl.7.6.1.1	$A_{bt.min} = 0.0018 \times h =$ **0.389** in^2/ft

PASS - Area of reinforcement provided is greater than minimum area of reinforcement required

Rectangular section in shear - Section 22.5

Design shear force	$V =$ **9768** lb/ft
Concrete modification factor - cl.19.2.4	$\lambda =$ **1**
Nominal concrete shear strength - eqn.22.5.5.1	$V_c = 2 \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} \times d =$ **24955** lb/ft
Strength reduction factor	$\phi_s =$ **0.75**
Design concrete shear strength - cl.7.6.3.1	$\phi V_c = \phi_s \times V_c =$ **18716** lb/ft
	$V / \phi V_c =$ **0.522**

PASS - No shear reinforcement is required

Transverse reinforcement parallel to base

Minimum area of reinforcement - cl.76.1.1	$A_{bx.req} = 0.0018 \times t_{base} =$ **0.389** in^2/ft
Transverse reinforcement provided	No.4 bars @ 12" c/c each face
Area of transverse reinforcement provided	$A_{bx.prov} = 2 \times \pi \times \phi_{bx}^2 / (4 \times s_{bx}) =$ **0.393** in^2/ft

PASS - Area of reinforcement provided is greater than area of reinforcement required

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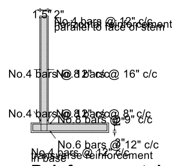
project Powder Mountain Parcel 4

Retaining Walls

location Eden, Utah

date 6/26/2017 by CW



Reinforcement details

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RETAINING WALL ANALYSIS & DESIGN - EAST MECH ROOM WALL (ACI318/MSJC)

RETAINING WALL ANALYSIS

In accordance with International Building Code 2015

Tedds calculation version 2.9.00

Retaining wall details

Stem type	Propped cantilever pinned at the base		
Stem height	h_{stem} = **12** ft		
Prop height	h_{prop} = **12** ft		
Stem thickness	t_{stem} = **12** in		
Angle to rear face of stem	α = **90** deg		
Stem density	γ_{stem} = **150** pcf		
Toe length	l_{toe} = **1** ft		
Heel length	l_{heel} = **1** ft		
Base thickness	t_{base} = **12** in		
Base density	γ_{base} = **150** pcf		
Height of retained soil	h_{ret} = **11** ft	Angle of soil surface	β = **0** deg
Depth of cover	d_{cover} = **1** ft		
Depth of excavation	d_{exc} = **1** ft		
Height of water	h_{water} = **5** ft		
Water density	γ_w = **62** pcf		

Retained soil properties

Soil type	Medium dense well graded sand
Moist density	γ_{mr} = **135** pcf
Saturated density	γ_{sr} = **145** pcf

Base soil properties

Soil type	Medium dense well graded sand
Soil density	γ_b = **115** pcf
Allowable bearing pressure	$P_{bearing}$ = **4600** psf

Loading details

Live surcharge load	$Surcharge_L$ = **100** psf
Vertical line load at 0.75 ft	P_{D1} = **1000** plf
	P_{L1} = **1000** plf

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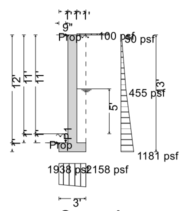

General arrangement

Calculate retaining wall geometry

Base length l_{base} = **3** ft

Saturated soil height h_{sat} = **6** ft

Moist soil height h_{moist} = **6** ft

Length of surcharge load l_{sur} = **1** ft

Vertical distance x_{sur_v} = **2.5** ft

Effective height of wall h_{eff} = **13** ft

Horizontal distance x_{sur_h} = **6.5** ft

Area of wall stem A_{stem} = **12** ft^2 — Vertical distance x_{stem} = **1.5** ft

Area of wall base A_{base} = **3** ft^2 — Vertical distance x_{base} = **1.5** ft

Area of saturated soil A_{sat} = **6** ft^2 — Vertical distance x_{sat_v} = **2.5** ft

Horizontal distance x_{sat_h} = **2.333** ft

Area of water A_{water} = **6** ft^2 — Vertical distance x_{water_v} = **2.5** ft

Horizontal distance x_{water_h} = **2.333** ft

Area of moist soil A_{moist} = **6** ft^2 — Vertical distance x_{moist_v} = **2.5** ft

Horizontal distance x_{moist_h} = **5.15** ft

Area of base soil A_{pass} = **1** ft^2 — Vertical distance x_{pass_v} = **0.5** ft

Horizontal distance x_{pass_h} = **0.667** ft

Soil coefficients

Coeff.friction to back of wall K_{fr} = **0.450**

Coeff.friction to front of wall K_{fb} = **0.450** — Coeff.friction beneath base K_{fbb} = **0.450**

At rest pressure coefficient K_0 = **0.500** — Passive pressure coefficient K_P = **3.000**

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	Retaining Walls
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From IBC 2015 cl.1807.2.3 Safety factor

Load combination 1 — 1.0 × Dead + 1.0 × Live + 1.0 × Lateral earth

Bearing pressure check

Vertical forces on wall

Total — $F_{total\_v} = F_{stem} + F_{base} + F_{sat\_v} + F_{moist\_v} + F_{pass\_v} + F_{water\_v} + F_{sur\_v} + F_{P\_v} = \mathbf{6145}$ plf

Horizontal forces on wall

Total — $F_{total\_h} = F_{sat\_h} + F_{moist\_h} + F_{pass\_h} + F_{water\_h} + F_{sur\_h} = \mathbf{6551}$ plf

Moments on wall

Total — $M_{total} = M_{stem} + M_{base} + M_{sat} + M_{moist} + M_{pass} + M_{water} + M_{sur} + M_P = \mathbf{9382}$ lb_ft/ft

Check bearing pressure

Bearing pressure at toe — $q_{toe} = \mathbf{1938}$ psf — Bearing pressure at heel — $q_{heel} = \mathbf{2158}$ psf

Factor of safety — $FoS_{bp} = \mathbf{2.131}$

PASS - Allowable bearing pressure exceeds maximum applied bearing pressure

RETAINING WALL DESIGN

In accordance with ACI 318-14

Tedds calculation version 2.9.00

Concrete details

Compressive strength — $f'_c = \mathbf{4500}$ psi — Concrete type — Normal weight

Reinforcement details

Yield strength — $f_y = \mathbf{60000}$ psi — Modulus of elasticity — $E_s = \mathbf{29000000}$ psi

Cover to reinforcement

Front face of stem — $c_{sf} = \mathbf{1.5}$ in — Rear face of stem — $c_{sr} = \mathbf{1.5}$ in

Top face of base — $c_{bt} = \mathbf{2}$ in — Bottom face of base — $c_{bb} = \mathbf{3}$ in

From IBC 2015 cl.1605.2.1 Basic load combinations

Load combination no.1 — 1.4 × Dead

Load combination no.2 — 1.2 × Dead + 1.6 × Live + 1.6 × Lateral earth

Load combination no.3 — 1.2 × Dead + 1.0 × Earthquake + 1.0 × Live + 1.6 × Lateral earth

Load combination no.4 — 0.9 × Dead + 1.0 × Earthquake + 1.6 × Lateral earth

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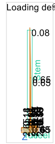
Loading details - Combination No.1

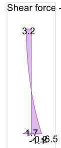
Shear force - Combination No.1 -

Bending moment - Combination No.1

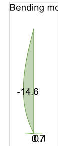
Loading details - Combination No.2

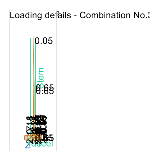


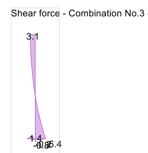
Shear force - Combination No.2 -

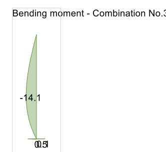


Bending moment - Combination No.2









Check stem design at 4.988 ft

Depth of section h = **12** in

Rectangular section in flexure - Section 22.3

Factored bending moment M = **14577** lb_ft/ft

Compression reinforcement No.5 bars @ 12" c/c Area provided $A_{srM.prov}$ = **0.307** in^2/ft

Tension reinforcement No.6 bars @ 12" c/c Area provided $A_{sfM.prov}$ = **0.442** in^2/ft

Max.reinforcement spacing s_{max} = **18** in

PASS - Reinforcement is adequately spaced

Nominal flexural strength M_n = **20347** lb_ft/ft Strength reduction factor ϕ_f = **0.9**

Design flexural strength ϕM_n = **18312** lb_ft/ft $M / \phi M_n$ = **0.796**

PASS - Design flexural strength exceeds factored bending moment

Reinforcement by analysis $A_{sfM.des}$ = **0.349** in^2/ft Minimum reinforcement $A_{sfM.min}$ = **0.382** in^2/ft

PASS - Area of reinforcement provided is greater than minimum area of reinforcement required

Check stem design at base of stem

Depth of section h = **12** in

Rectangular section in shear - Section 22.5

Design shear force V = **6533** lb/ft

Nominal conc.shear strength V_c = **15295** lb/ft Strength reduction factor ϕ_s = **0.75**

Design conc.shear strength ϕV_c = **11471** lb/ft $V / \phi V_c$ = **0.570**

PASS - No shear reinforcement is required

Check stem design at prop

Depth of section h = **12** in

Rectangular section in shear - Section 22.5

Design shear force V = **3246** lb/ft

Nominal conc.shear strength V_c = **15295** lb/ft Strength reduction factor ϕ_s = **0.75**

Design conc.shear strength ϕV_c = **11471** lb/ft $V / \phi V_c$ = **0.283**

PASS - No shear reinforcement is required

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Horizontal reinforcement parallel to face of stem

Min.area of reinforcement $A_{sx.req}$ = **0.288** in^2/ft

Trans.reinforcement provided No.5 bars @ 12" c/c each face Trans.reinforcement provided $A_{sx.prov}$ = **0.614** in^2/ft

PASS - Area of reinforcement provided is greater than area of reinforcement required

Check base design at toe

Depth of section h = **12** in

Rectangular section in flexure - Section 22.3

Factored bending moment M = **653** lb_ft/ft

Compression reinforcement No.4 bars @ 12" c/c Area provided $A_{bt.prov}$ = **0.196** in^2/ft

Tension reinforcement No.5 bars @ 12" c/c Area provided $A_{bb.prov}$ = **0.307** in^2/ft

Max.reinforcement spacing s_{max} = **18** in

PASS - Reinforcement is adequately spaced

Nominal flexural strength M_n = **13019** lb_ft/ft Strength reduction factor ϕ_f = **0.9**

Design flexural strength ϕM_n = **11717** lb_ft/ft $M / \phi M_n$ = **0.056**

PASS - Design flexural strength exceeds factored bending moment

Reinforcement by analysis $A_{bb.des}$ = **0.017** in^2/ft Minimum reinforcement $A_{bb.min}$ = **0.259** in^2/ft

PASS - Area of reinforcement provided is greater than minimum area of reinforcement required

Rectangular section in shear - Section 22.5

Design shear force V = **1738** lb/ft

Nominal conc.shear strength V_c = **13987** lb/ft Strength reduction factor ϕ_s = **0.75**

Design conc.shear strength ϕV_c = **10490** lb/ft $V / \phi V_c$ = **0.166**

PASS - No shear reinforcement is required

Check base design at heel

Depth of section h = **12** in

Rectangular section in shear - Section 22.5

Design shear force V = **299** lb/ft

Nominal conc.shear strength V_c = **13987** lb/ft Strength reduction factor ϕ_s = **0.75**

Design conc.shear strength ϕV_c = **10490** lb/ft $V / \phi V_c$ = **0.029**

PASS - No shear reinforcement is required

Transverse reinforcement parallel to base

Min.area of reinforcement $A_{bx.req}$ = **0.259** in^2/ft

Trans.reinforcement provided No.5 bars @ 12" c/c each face Trans.reinforcement provided $A_{bx.prov}$ = **0.614** in^2/ft

PASS - Area of reinforcement provided is greater than area of reinforcement required

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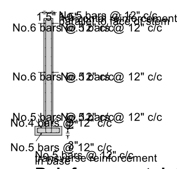

Reinforcement details

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Shear Wall Foundations

location Eden, Utah

date 6/27/2017 by CW

SHEAR WALL MAT - P-11

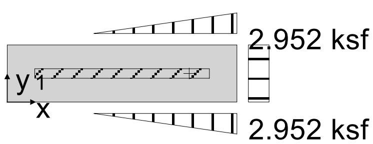
FOUNDATION ANALYSIS & DESIGN (ACI318)

In accordance with ACI318-14

Tedds calculation version 3.0.05

FOOTING ANALYSIS

Length of foundation	L_x = **24** ft	Width of foundation	L_y = **6** ft
Foundation area	A = **144** ft^2	Depth of foundation	h = **18** in
Depth of soil over foundation	h_{soil} = **12** in	Density of concrete	γ_{conc} = **150.0** lb/ft^3



Column no.1 details

Length of column	l_{x1} = **219.00** in	Width of column	l_{y1} = **12.00** in
position in x-axis	x_1 = **144.00** in	position in y-axis	y_1 = **36.00** in

Soil properties

Gross allow. bearing press.	q_{allow_Gross} = **4.6** ksf	Density of soil	γ_{soil} = **120.0** lb/ft^3
Angle of internal friction	ϕ_b = **30.0** deg	Design base friction angle	δ_{bb} = **22.0** deg
Coefficient of base friction	tan(δ_{bb}) = **0.404**		
Dead surcharge load	F_{Dsur} = **50** psf	Live surcharge load	F_{Lsur} = **100** psf
Self weight	F_{swt} = **225** psf	Soil weight	F_{soil} = **120** psf

Column no.1 loads

Dead load in z	F_{Dz1} = **177.7** kips	Snow load in z	F_{Sz1} = **175.0** kips
Seismic load in x	F_{Ex1} = **60.0** kips		
Seismic load moment in x	M_{Ex1} = **1200.0** kip_ft		

Footing analysis for soil and stability

Load combinations per ASCE 7-10

1.0D (0.354)

1.0D + 1.0L (0.376)

1.0D + 1.0Lr (0.354)

1.0D + 1.0S (0.618)

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1.0D + 1.0R (0.354)
1.0D + 0.75L + 0.75Lr (0.370)
1.0D + 0.75L + 0.75S (0.569)
1.0D + 0.75L + 0.75R (0.370)
1.0D + 0.6W (0.354)
(1.0 + 0.14 × S_{DS})D + 0.7E (0.724)
1.0D + 0.75L + 0.75Lr + 0.45W (0.370)
1.0D + 0.75L + 0.75S + 0.45W (0.569)
1.0D + 0.75L + 0.75R + 0.45W (0.370)
(1.0 + 0.10 × S_{DS})D + 0.75L + 0.75S + 0.525E (0.845)
0.6D + 0.6W (0.212)
(0.6 - 0.14 × S_{DS})D + 0.7E (0.854)

Combination 16 results: (0.6 - 0.14 × S_{DS})D + 0.7E

Forces on foundation
Force in x-axis F_{dx} = **42.0** kips Force in z-axis F_{dz} = **121.7** kips

Moments on foundation
Moment in x-axis, about x is 0 M_{dx} = **2363.4** kip_ft Moment in y-axis, about y is 0 M_{dy} = **365.1** kip_ft

Uplift verification
Vertical force F_{dz} = **121.7** kips

PASS - Foundation is not subject to uplift

Stability against overturning in x direction, moment about x is L_x
Overturning moment M_{OTxL} = **903** kip_ft Resisting moment M_{RxL} = **-1460.4** kip_ft
Factor of safety abs(M_{RxL} / M_{OTxL}) = **1.617**

PASS - Overturning moment safety factor exceeds the minimum of 1.00

Stability against sliding
Resistance due to base friction $F_{RFriction}$ = **49.17** kips

Stability against sliding in x direction
Total sliding resistance F_{Rx} = **49.17** kips Factor of safety abs(F_{Rx} / F_{dx}) = **1.17**

PASS - Sliding factor of safety exceeds the minimum of 1.00

Bearing resistance

Eccentricity of base reaction
Eccentricity in x-axis e_{dx} = **89.039** in Eccentricity in y-axis e_{dy} = **0** in
Length of bearing, x-axis L'_{xd} = **164.884** in

Pad base pressures
Min. base press. q_{min} = **0** ksf Max. base press. q_{max} = **2.952** ksf

Allowable bearing capacity
Allowable bearing capacity q_{allow} = **4.6** ksf q_{max} / q_{allow} = **0.642**

PASS - Allowable bearing capacity exceeds design base pressure

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FOOTING DESIGN (ACI318)

In accordance with ACI318-14

Material details

Compr. strength of concrete f'_c = **4000** psi
Yield strength of reinforcement f_y = **60000** psi
Cover to reinforcement c_{nom} = **3** in
Concrete type Normal weight
Concrete modification factor λ = **1.00**
Column type Concrete

Analysis and design of concrete footing

Load combinations per ASCE 7-10

1.4D (0.219)
1.2D + 1.6L + 0.5Lr (0.188)
1.2D + 1.6L + 0.5S (0.265)
1.2D + 1.6L + 0.5R (0.188)
1.2D + 1.0L + 1.6Lr (0.188)
1.2D + 1.0L + 1.6S (0.434)
1.2D + 1.0L + 1.6R (0.188)
1.2D + 1.6Lr + 0.5W (0.188)
1.2D + 1.6S + 0.5W (0.434)
1.2D + 1.6R + 0.5W (0.188)
1.2D + 1.0L + 0.5Lr + 1.0W (0.188)
1.2D + 1.0L + 0.5S + 1.0W (0.265)
1.2D + 1.0L + 0.5R + 1.0W (0.188)
(1.2 + 0.2 × S_{DS})D + 1.0L + 0.2S + 1.0E (0.237)
0.9D + 1.0W (0.141)
(0.9 - 0.2 × S_{DS})D + 1.0E (0.123)

Combination 14 results: (1.2 + 0.2 × S_{DS})D + 1.0L + 0.2S + 1.0E

Forces on foundation

Ultimate force in x-axis F_{ux} = **60.0** kips
Ultimate force in z-axis F_{uz} = **358.1** kips

Moments on foundation

Ultimate moment in x-axis, about x is 0 M_{ux} = **5587.3** kip_ft
Ultimate moment in y-axis, about y is 0 M_{uy} = **1074.3** kip_ft

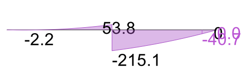
Eccentricity of base reaction

Eccentricity in x-axis e_{ux} = **43.227** in
Eccentricity in y-axis e_{uy} = **0** in

Pad base pressures

Min. ultimate base press. q_{umin} = **0.247** ksf
Max. ultimate base press. q_{umax} = **4.726** ksf

Shear diagram, x axis (kips)



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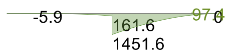
project	Powder Mountain Parcel 4
	Shear Wall Foundations
location	Eden, Utah
date	6/27/2017
by	CW

Moment diagram, x axis (kip_ft)



Moment design, x direction, positive moment

Ultimate bending moment $M_{u.x.max}$ = **97.396** kip_ft
Tension reinf. provided 7 No.6 bot. bars (10.8 in c/c)
Area of tension reinf. provided $A_{sx.bot.prov}$ = **3.08** in^2
Min. area of reinforcement $A_{s.min}$ = **2.333** in^2

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinf. s_{max} = **18** in

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinf. d = **14.625** in
Depth of compression block a = **0.755** in
Neutral axis factor β_1 = **0.85**
Depth to neutral axis c = **0.888** in
Strain in tensile reinf. ε_t = **0.04640**

PASS - Tensile strain exceeds minimum required, 0.004

Nominal moment capacity M_n = **219.412** kip_ft
Flexural strength red. factor ϕ_f = **0.900**
Design moment capacity ϕM_n = **197.471** kip_ft
$M_{u.x.max} / \phi M_n$ = **0.493**

PASS - Design moment capacity exceeds ultimate moment load

Moment design, x direction, negative moment

Ultimate bending moment $M_{u.x.min}$ = **-4.802** kip_ft
Tension reinf. provided 7 No.6 top bars (10.8 in c/c)
Area of tension reinf. provided $A_{sx.top.prov}$ = **3.08** in^2
Min. area of reinforcement $A_{s.min}$ = **2.333** in^2

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinf. s_{max} = **18** in

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinf. d = **14.625** in
Depth of compression block a = **0.755** in
Neutral axis factor β_1 = **0.85**
Depth to neutral axis c = **0.888** in
Strain in tensile reinf. ε_t = **0.04640**

PASS - Tensile strain exceeds minimum required, 0.004

Nominal moment capacity M_n = **219.412** kip_ft
Flexural strength red. factor ϕ_f = **0.900**
Design moment capacity ϕM_n = **197.471** kip_ft
abs($M_{u.x.min}$) / ϕM_n = **0.024**

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, x direction

Ultimate shear force $V_{u.x}$ = **40.696** kips
Depth to reinforcement d_v = **14.625** in
Shear strength red. factor ϕ_v = **0.75**
Nominal shear capacity V_n = **133.195** kips
Design shear capacity ϕV_n = **99.896** kips
$V_{u.x} / \phi V_n$ = **0.407**

PASS - Design shear capacity exceeds ultimate shear load

Two-way shear design at column 1

Two-way shear design does not apply. Shear perimeter falls outside extents of foundation.

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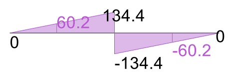
project Powder Mountain Parcel 4

Shear Wall Foundations

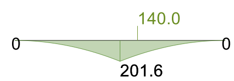
location Eden, Utah

date 6/27/2017 by CW

Shear diagram, y axis (kips)



Moment diagram, y axis (kip_ft)



Moment design, y direction, positive moment

Ultimate bending moment $M_{u.y.max}$ = **140.028** kip_ft Tension reinf. provided 22 No.6 bot. bars (13.3 in c/c)

Area of tension reinf. provided $A_{sy.bot.prov}$ = **9.68** in^2 Min. area of reinforcement $A_{s.min}$ = **9.331** in^2

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinf. s_{max} = **18** in

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinf. d = **13.875** in Depth of compression block a = **0.593** in

Neutral axis factor β_1 = **0.85** Depth to neutral axis c = **0.698** in

Strain in tensile reinf. ε_t = **0.05665**

PASS - Tensile strain exceeds minimum required, 0.004

Nominal moment capacity M_n = **657.196** kip_ft Flexural strength red. factor ϕ_f = **0.900**

Design moment capacity ϕM_n = **591.476** kip_ft $M_{u.y.max}$ / ϕM_n = **0.237**

PASS - Design moment capacity exceeds ultimate moment load

Footing geometry factor β_f = **4.000** Area of reinf. req. for uniform distr. A_{sreq} = **3.667** in^2

40% of the reinforcement shall be distributed over a 6.00 ft width band centered under the column, the remainder distributed among the remainder of the footing

One-way shear design, y direction

Ultimate shear force $V_{u.y}$ = **60.212** kips Depth to reinforcement d_v = **13.875** in

Shear strength red. factor ϕ_v = **0.75** Nominal shear capacity V_n = **505.458** kips

Design shear capacity ϕV_n = **379.094** kips $V_{u.y}$ / ϕV_n = **0.159**

PASS - Design shear capacity exceeds ultimate shear load

Two-way shear design at column 1

Two-way shear design does not apply. Shear perimeter falls outside extents of foundation.

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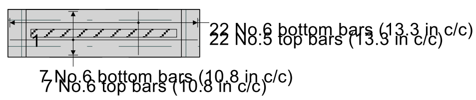

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SHEAR WALL MAT - P-10

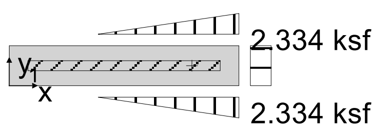
FOUNDATION ANALYSIS & DESIGN (ACI318)

In accordance with ACI318-14

Tedds calculation version 3.0.05

FOOTING ANALYSIS

Length of foundation	L_x = **23** ft	Width of foundation	L_y = **4** ft
Foundation area	A = **92** ft^2	Depth of foundation	h = **18** in
Depth of soil over foundation	h_{soil} = **12** in	Density of concrete	γ_{conc} = **150.0** lb/ft^3



Column no.1 details

Length of column	l_{x1} = **231.00** in	Width of column	l_{y1} = **12.00** in
position in x-axis	x_1 = **138.00** in	position in y-axis	y_1 = **24.00** in

Soil properties

Gross allow. bearing press.	q_{allow_Gross} = **4.6** ksf	Density of soil	γ_{soil} = **120.0** lb/ft^3
Angle of internal friction	ϕ_b = **30.0** deg	Design base friction angle	δ_{bb} = **22.0** deg
Coefficient of base friction	$\tan(\delta_{bb})$ = **0.404**		
Dead surcharge load	F_{Dsur} = **50** psf	Live surcharge load	F_{Lsur} = **100** psf
Self weight	F_{swt} = **225** psf	Soil weight	F_{soil} = **120** psf

Column no.1 loads

Dead load in z	F_{Dz1} = **80.0** kips	Live load in z	F_{Lz1} = **24.0** kips
Seismic load in x	F_{Ex1} = **30.0** kips		
Seismic load moment in x	M_{Ex1} = **575.0** kip_ft		

Footing analysis for soil and stability

Load combinations per ASCE 7-10

1.0D (0.275)

1.0D + 1.0L (0.353)

1.0D + 1.0Lr (0.275)

1.0D + 1.0S (0.275)

1.0D + 1.0R (0.275)

1.0D + 0.75L + 0.75Lr (0.334)

1.0D + 0.75L + 0.75S (0.334)
1.0D + 0.75L + 0.75R (0.334)
1.0D + 0.6W (0.275)
(1.0 + 0.14 × S_{DS})D + 0.7E (0.565)
1.0D + 0.75L + 0.75Lr + 0.45W (0.334)
1.0D + 0.75L + 0.75S + 0.45W (0.334)
1.0D + 0.75L + 0.75R + 0.45W (0.334)
(1.0 + 0.10 × S_{DS})D + 0.75L + 0.75S + 0.525E (0.550)
0.6D + 0.6W (0.165)
(0.6 - 0.14 × S_{DS})D + 0.7E (0.861)

Combination 16 results: (0.6 - 0.14 × S_{DS})D + 0.7E

Forces on foundation

Force in x-axis F_{dx} = **21.0** kips
Force in z-axis F_{dz} = **60.4** kips

Moments on foundation

Moment in x-axis, about x is 0 M_{dx} = **1128.1** kip_ft
Moment in y-axis, about y is 0 M_{dy} = **120.7** kip_ft

Uplift verification

Vertical force F_{dz} = **60.357** kips

PASS - Foundation is not subject to uplift

Stability against overturning in x direction, moment about x is L_x

Overturning moment M_{OTxL} = **434** kip_ft
Resisting moment M_{RxL} = **-694.11** kip_ft
Factor of safety abs(M_{RxL} / M_{OTxL}) = **1.599**

PASS - Overturning moment safety factor exceeds the minimum of 1.00

Stability against sliding

Resistance due to base friction $F_{RFriction}$ = **24.386** kips

Stability against sliding in x direction

Total sliding resistance F_{Rx} = **24.386** kips
Factor of safety abs(F_{Rx} / F_{dx}) = **1.16**

PASS - Sliding factor of safety exceeds the minimum of 1.00

Bearing resistance

Eccentricity of base reaction

Eccentricity in x-axis e_{dx} = **86.286** in
Eccentricity in y-axis e_{dy} = **0** in
Length of bearing, x-axis L'_{xd} = **155.141** in

Pad base pressures

Min. base press. q_{min} = **0** ksf
Max. base press. q_{max} = **2.334** ksf

Allowable bearing capacity

Allowable bearing capacity q_{allow} = **4.6** ksf
q_{max} / q_{allow} = **0.507**

PASS - Allowable bearing capacity exceeds design base pressure

<u>FOOTING DESIGN (ACI318)</u>

In accordance with ACI318-14

Material details

Compr. strength of concrete f'_c = **4000** psi
Yield strength of reinforcement f_y = **60000** psi
Cover to reinforcement c_{nom} = **3** in
Concrete type Normal weight

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Concrete modification factor λ = **1.00** Column type Concrete

Analysis and design of concrete footing

Load combinations per ASCE 7-10

1.4D (0.055)
1.2D + 1.6L + 0.5Lr (0.066)
1.2D + 1.6L + 0.5S (0.066)
1.2D + 1.6L + 0.5R (0.066)
1.2D + 1.0L + 1.6Lr (0.059)
1.2D + 1.0L + 1.6S (0.059)
1.2D + 1.0L + 1.6R (0.059)
1.2D + 1.6Lr + 0.5W (0.047)
1.2D + 1.6S + 0.5W (0.047)
1.2D + 1.6R + 0.5W (0.047)
1.2D + 1.0L + 0.5Lr + 1.0W (0.059)
1.2D + 1.0L + 0.5S + 1.0W (0.059)
1.2D + 1.0L + 0.5R + 1.0W (0.059)
(1.2 + 0.2 × S_{DS})D + 1.0L + 0.2S + 1.0E (0.064)
0.9D + 1.0W (0.036)
(0.9 - 0.2 × S_{DS})D + 1.0E (0.031)

Combination 14 results: (1.2 + 0.2 × S_{DS})D + 1.0L + 0.2S + 1.0E

Forces on foundation

Ultimate force in x-axis F_{ux} = **30.0** kips Ultimate force in z-axis F_{uz} = **186.3** kips

Moments on foundation

Ultimate moment in x-axis, about x is 0 M_{ux} = **2762.5** kip_ft Ultimate moment in y-axis, about y is 0 M_{uy} = **372.6** kip_ft

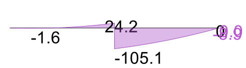
Eccentricity of base reaction

Eccentricity in x-axis e_{ux} = **39.935** in Eccentricity in y-axis e_{uy} = **0** in

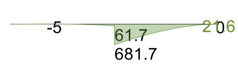
Pad base pressures

Min. ultimate base press. q_{umin} = **0.267** ksf Max. ultimate base press. q_{umax} = **3.783** ksf

Shear diagram, x axis (kips)



Moment diagram, x axis (kip_ft)



Moment design, x direction, positive moment

Ultimate bending moment $M_{u.x.max}$ = **21.57** kip_ft Tension reinf. provided 6 No.5 bot. bars (8.2 in c/c)
Area of tension reinf. provided $A_{sx.bot.prov}$ = **1.86** in^2 Min. area of reinforcement $A_{s.min}$ = **1.555** in^2

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinf. s_{max} = **18** in

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinf. d = **14.688** in Depth of compression block a = **0.684** in

Neutral axis factor β_1 = **0.85** Depth to neutral axis c = **0.804** in

Strain in tensile reinf. ε_t = **0.05177**

PASS - Tensile strain exceeds minimum required, 0.004

Nominal moment capacity M_n = **133.414** kip_ft Flexural strength red. factor ϕ_f = **0.900**

Design moment capacity ϕM_n = **120.073** kip_ft $M_{u.x.max}$ / ϕM_n = **0.180**

PASS - Design moment capacity exceeds ultimate moment load

Moment design, x direction, negative moment

Ultimate bending moment $M_{u.x.min}$ = **-1.81** kip_ft Tension reinf. provided 6 No.5 top bars (8.2 in c/c)

Area of tension reinf. provided $A_{sx.top.prov}$ = **1.86** in^2 Min. area of reinforcement $A_{s.min}$ = **1.555** in^2

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinf. s_{max} = **18** in

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinf. d = **14.688** in Depth of compression block a = **0.684** in

Neutral axis factor β_1 = **0.85** Depth to neutral axis c = **0.804** in

Strain in tensile reinf. ε_t = **0.05177**

PASS - Tensile strain exceeds minimum required, 0.004

Nominal moment capacity M_n = **133.414** kip_ft Flexural strength red. factor ϕ_f = **0.900**

Design moment capacity ϕM_n = **120.073** kip_ft abs($M_{u.x.min}$) / ϕM_n = **0.015**

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, x direction

Ultimate shear force $V_{u.x}$ = **8.873** kips Depth to reinforcement d_v = **14.688** in

Shear strength red. factor ϕ_v = **0.75** Nominal shear capacity V_n = **89.176** kips

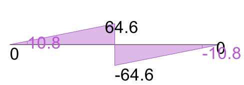
Design shear capacity ϕV_n = **66.882** kips $V_{u.x}$ / ϕV_n = **0.133**

PASS - Design shear capacity exceeds ultimate shear load

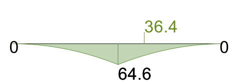
Two-way shear design at column 1

Two-way shear design does not apply. Shear perimeter falls outside extents of foundation.

Shear diagram, y axis (kips)



Moment diagram, y axis (kip_ft)



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Moment design, y direction, positive moment

Ultimate bending moment $M_{u.y.max}$ = **36.36** kip_ft
Tension reinf. provided 21 No.6 bot. bars (13.4 in c/c)
Area of tension reinf. provided $A_{sy.bot.prov}$ = **9.24** in^2
Min. area of reinforcement $A_{s.min}$ = **8.942** in^2

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinf. s_{max} = **18** in

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinf. d = **14** in
Depth of compression block a = **0.591** in
Neutral axis factor β_1 = **0.85**
Depth to neutral axis c = **0.695** in
Strain in tensile reinf. ε_t = **0.05743**

PASS - Tensile strain exceeds minimum required, 0.004

Nominal moment capacity M_n = **633.153** kip_ft
Flexural strength red. factor ϕ_f = **0.900**
Design moment capacity ϕM_n = **569.837** kip_ft
$M_{u.y.max} / \phi M_n$ = **0.064**

PASS - Design moment capacity exceeds ultimate moment load

Footing geometry factor β_f = **5.750**
Area of reinf. req. for uniform distr. A_{sreq} = **1.004** in^2

30% of the reinforcement shall be distributed over a 4.00 ft width band centered under the column, the remainder distributed among the remainder of the footing

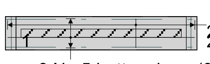
One-way shear design, y direction

Ultimate shear force $V_{u.y}$ = **10.773** kips
Depth to reinforcement d_v = **14.000** in
Shear strength red. factor ϕ_v = **0.75**
Nominal shear capacity V_n = **488.762** kips
Design shear capacity ϕV_n = **366.571** kips
$V_{u.y} / \phi V_n$ = **0.029**

PASS - Design shear capacity exceeds ultimate shear load

Two-way shear design at column 1

Two-way shear design does not apply. Shear perimeter falls outside extents of foundation.



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Shear Wall Foundations

location Eden, Utah

date 6/27/2017 by CW

SHEAR WALL MAT - P-1, P-2, P-3

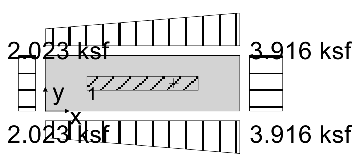
FOUNDATION ANALYSIS & DESIGN (ACI318)

In accordance with ACI318-14

Tedds calculation version 3.0.05

FOOTING ANALYSIS

Length of foundation	L_x = **14** ft	Width of foundation	L_y = **4** ft
Foundation area	A = **56** ft^2	Depth of foundation	h = **16** in
Depth of soil over foundation	h_{soil} = **12** in	Density of concrete	γ_{conc} = **150.0** lb/ft^3



Column no.1 details

Length of column	l_{x1} = **96.00** in	Width of column	l_{y1} = **12.00** in
position in x-axis	x_1 = **84.00** in	position in y-axis	y_1 = **24.00** in

Soil properties

Gross allow. bearing press.	q_{allow_Gross} = **4.6** ksf	Density of soil	γ_{soil} = **120.0** lb/ft^3
Angle of internal friction	ϕ_b = **30.0** deg	Design base friction angle	δ_{bb} = **22.0** deg
Coefficient of base friction	$\tan(\delta_{bb})$ = **0.404**		

Foundation loads

Dead surcharge load	F_{Dsur} = **50** psf	Live surcharge load	F_{Lsur} = **100** psf
Self weight	F_{swt} = **200** psf	Soil weight	F_{soil} = **120** psf

Column no.1 loads

Dead load in z	F_{Dz1} = **85.0** kips	Live load in z	F_{Lz1} = **67.0** kips
Seismic load in x	F_{Ex1} = **10.0** kips		
Dead load moment in x	M_{Dx1} = **16.0** kip_ft	Live load moment in x	M_{Lx1} = **21.0** kip_ft
Seismic load moment in x	M_{Ex1} = **160.0** kip_ft		

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Shear Wall Foundations
location Eden, Utah
date 6/27/2017 by CW

Footing analysis for soil and stability

Load combinations per ASCE 7-10

1.0D (0.437)
1.0D + 1.0L (0.754)
1.0D + 1.0Lr (0.437)
1.0D + 1.0S (0.437)
1.0D + 1.0R (0.437)
1.0D + 0.75L + 0.75Lr (0.675)
1.0D + 0.75L + 0.75S (0.675)
1.0D + 0.75L + 0.75R (0.675)
1.0D + 0.6W (0.437)
(1.0 + 0.14 × S_{DS})D + 0.7E (0.674)
1.0D + 0.75L + 0.75Lr + 0.45W (0.675)
1.0D + 0.75L + 0.75S + 0.45W (0.675)
1.0D + 0.75L + 0.75R + 0.45W (0.675)
(1.0 + 0.10 × S_{DS})D + 0.75L + 0.75S + 0.525E (0.851)
0.6D + 0.6W (0.262)
(0.6 - 0.14 × S_{DS})D + 0.7E (0.429)

Combination 14 results: (1.0 + 0.10 × S_{DS})D + 0.75L + 0.75S + 0.525E

Forces on foundation

Force in x-axis F_{dx} = **5.3** kips
Force in z-axis F_{dz} = **166.3** kips

Moments on foundation

Moment in x-axis, about x is 0 M_{dx} = **1287.8** kip_ft
Moment in y-axis, about y is 0 M_{dy} = **332.6** kip_ft

Uplift verification

Vertical force F_{dz} = **166.302** kips

PASS - Foundation is not subject to uplift

Stability against overturning in x direction, moment about x is L_x

Overturning moment M_{OTxL} = **123.68** kip_ft
Resisting moment M_{RxL} = **-1164.11** kip_ft
Factor of safety abs(M_{RxL} / M_{OTxL}) = **9.412**

PASS - Overturning moment safety factor exceeds the minimum of 1.00

Stability against sliding

Resistance due to base friction $F_{RFriction}$ = **67.19** kips

Stability against sliding in x direction

Total sliding resistance F_{Rx} = **67.19** kips
Factor of safety abs(F_{Rx} / F_{dx}) = **12.8**

PASS - Sliding factor of safety exceeds the minimum of 1.00

Bearing resistance

Eccentricity of base reaction

Eccentricity in x-axis e_{dx} = **8.924** in
Eccentricity in y-axis e_{dy} = **0** in

Pad base pressures

Min. base press. q_{min} = **2.023** ksf
Max. base press. q_{max} = **3.916** ksf

Allowable bearing capacity

Allowable bearing capacity q_{allow} = **4.6** ksf
q_{max} / q_{allow} = **0.851**

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location Eden, Utah

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PASS - Allowable bearing capacity exceeds design base pressure

FOOTING DESIGN (ACI318)

In accordance with ACI318-14

Material details

Compr. strength of concrete f'_c = **4000** psi

Yield strength of reinforcement f_y = **60000** psi

Cover to reinforcement c_{nom} = **3** in

Concrete type Normal weight

Concrete modification factor λ = **1.00**

Column type Concrete

Analysis and design of concrete footing

Load combinations per ASCE 7-10

1.4D (0.103)
1.2D + 1.6L + 0.5Lr (0.182)
1.2D + 1.6L + 0.5S (0.182)
1.2D + 1.6L + 0.5R (0.182)
1.2D + 1.0L + 1.6Lr (0.147)
1.2D + 1.0L + 1.6S (0.147)
1.2D + 1.0L + 1.6R (0.147)
1.2D + 1.6Lr + 0.5W (0.089)
1.2D + 1.6S + 0.5W (0.089)
1.2D + 1.6R + 0.5W (0.089)
1.2D + 1.0L + 0.5Lr + 1.0W (0.147)
1.2D + 1.0L + 0.5S + 1.0W (0.147)
1.2D + 1.0L + 0.5R + 1.0W (0.147)
(1.2 + 0.2 × S_{DS})D + 1.0L + 0.2S + 1.0E (0.155)
0.9D + 1.0W (0.066)
(0.9 - 0.2 × S_{DS})D + 1.0E (0.058)

Combination 14 results: (1.2 + 0.2 × S_{DS})D + 1.0L + 0.2S + 1.0E

Forces on foundation

Ultimate force in x-axis F_{ux} = **10.0** kips

Ultimate force in z-axis F_{uz} = **211.7** kips

Moments on foundation

Ultimate moment in x-axis, about x is 0 M_{ux} = **1697.5** kip_ft

Ultimate moment in y-axis, about y is 0 M_{uy} = **423.5** kip_ft

Eccentricity of base reaction

Eccentricity in x-axis e_{ux} = **12.208** in

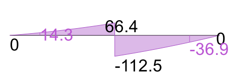
Eccentricity in y-axis e_{uy} = **0** in

Pad base pressures

Min. ultimate base press. q_{umin} = **2.132** ksf

Max. ultimate base press. q_{umax} = **5.429** ksf

Shear diagram, x axis (kips)



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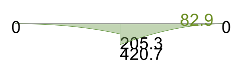
	Shear Wall Foundations
location	Eden, Utah
date	6/27/2017
by	CW

Moment diagram, x axis (kip_ft)



Moment design, x direction, positive moment

Ultimate bending moment $M_{u.x.max}$ = **82.923** kip_ft
Tension reinf. provided 5 No.5 bot. bars (10.3 in c/c)
Area of tension reinf. provided $A_{sx.bot.prov}$ = **1.55** in^2
Min. area of reinforcement $A_{s.min}$ = **1.382** in^2

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinf. s_{max} = **18** in

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinf. d = **12.688** in
Depth of compression block a = **0.57** in
Neutral axis factor β_1 = **0.85**
Depth to neutral axis c = **0.67** in
Strain in tensile reinf. ε_t = **0.05377**

PASS - Tensile strain exceeds minimum required, 0.004

Nominal moment capacity M_n = **96.12** kip_ft
Flexural strength red. factor ϕ_f = **0.900**
Design moment capacity ϕM_n = **86.508** kip_ft
$M_{u.x.max}$ / ϕM_n = **0.959**

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, x direction

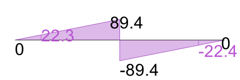
Ultimate shear force $V_{u.x}$ = **36.946** kips
Depth to reinforcement d_v = **12.688** in
Shear strength red. factor ϕ_v = **0.75**
Nominal shear capacity V_n = **77.033** kips
Design shear capacity ϕV_n = **57.775** kips
$V_{u.x}$ / ϕV_n = **0.639**

PASS - Design shear capacity exceeds ultimate shear load

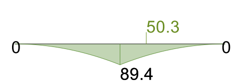
Two-way shear design at column 1

Two-way shear design does not apply. Shear perimeter falls outside extents of foundation.

Shear diagram, y axis (kips)



Moment diagram, y axis (kip_ft)



Moment design, y direction, positive moment

Ultimate bending moment $M_{u.y.max}$ = **50.304** kip_ft
Tension reinf. provided 14 No.6 bot. bars (12.4 in c/c)
Area of tension reinf. provided $A_{sy.bot.prov}$ = **6.16** in^2
Min. area of reinforcement $A_{s.min}$ = **4.838** in^2

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinf.	s_{max} = **18** in		

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinf.	d = **12** in	Depth of compression block	a = **0.647** in
Neutral axis factor	β_1 = **0.85**	Depth to neutral axis	c = **0.761** in
Strain in tensile reinf.	ε_t = **0.04429**		

PASS - Tensile strain exceeds minimum required, 0.004

Nominal moment capacity	M_n = **359.635** kip_ft	Flexural strength red. factor	ϕ_f = **0.900**
Design moment capacity	ϕM_n = **323.672** kip_ft		$M_{u.y.max}$ / ϕM_n = **0.155**

PASS - Design moment capacity exceeds ultimate moment load

Footing geometry factor	β_f = **3.500**	Area of reinf. req. for uniform distr.	A_{sreq} = **1.489** in^2

44% of the reinforcement shall be distributed over a 4.00 ft width band centered under the column, the remainder distributed among the remainder of the footing

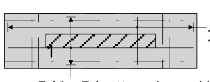
One-way shear design, y direction

Ultimate shear force	$V_{u.y}$ = **22.358** kips	Depth to reinforcement	d_v = **12.000** in
Shear strength red. factor	ϕ_v = **0.75**	Nominal shear capacity	V_n = **255.006** kips
Design shear capacity	ϕV_n = **191.255** kips	$V_{u.y}$ / ϕV_n = **0.117**	

PASS - Design shear capacity exceeds ultimate shear load

Two-way shear design at column 1

Two-way shear design does not apply. Shear perimeter falls outside extents of foundation.



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project Powder Mountain Parcel 4

Shear Wall Foundations

location Eden, Utah

date 6/27/2017 by CW

SHEAR WALL MAT - P-5

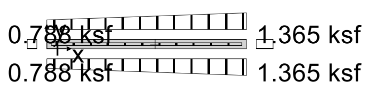
FOUNDATION ANALYSIS & DESIGN (ACI318)

In accordance with ACI318-14

Tedds calculation version 3.0.05

FOOTING ANALYSIS

Length of foundation	L_x = **108** ft	Width of foundation	L_y = **5** ft
Foundation area	A = **540** ft^2	Depth of foundation	h = **24** in
Depth of soil over foundation	h_{soil} = **48** in	Density of concrete	γ_{conc} = **150.0** lb/ft^3



Column no.1 details

Length of column	l_{x1} = **1236.00** in	Width of column	l_{y1} = **18.00** in
position in x-axis	x_1 = **648.00** in	position in y-axis	y_1 = **30.00** in

Soil properties

Gross allow. bearing press.	q_{allow_Gross} = **4.6** ksf	Density of soil	γ_{soil} = **120.0** lb/ft^3
Angle of internal friction	ϕ_b = **30.0** deg	Design base friction angle	δ_{bb} = **22.0** deg
Coefficient of base friction	$\tan(\delta_{bb})$ = **0.404**		

Foundation loads

Dead surcharge load	F_{Dsur} = **50** psf	Live surcharge load	F_{Lsur} = **100** psf
Self weight	F_{swt} = **300** psf	Soil weight	F_{soil} = **480** psf

Column no.1 loads

Dead load in z	F_{Dz1} = **672.0** kips	Live load in z	F_{Lz1} = **26.0** kips
Snow load in z	F_{Sz1} = **290.0** kips	Seismic load in x	F_{Ex1} = **285.0** kips
Dead load moment in x	M_{Dx1} = **750.0** kip_ft	Live load moment in x	M_{Lx1} = **776.0** kip_ft
Seismic load moment in x	M_{Ex1} = **2882.0** kip_ft		

Footing analysis for soil and stability

Load combinations per ASCE 7-10

1.0D (0.468)
1.0D + 1.0L (0.517)
1.0D + 1.0Lr (0.468)
1.0D + 1.0S (0.584)
1.0D + 1.0R (0.468)
1.0D + 0.75L + 0.75Lr (0.505)
1.0D + 0.75L + 0.75S (0.592)
1.0D + 0.75L + 0.75R (0.505)
1.0D + 0.6W (0.468)

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$(1.0 + 0.14 \times S_{DS})D + 0.7E$ (0.560)
1.0D + 0.75L + 0.75Lr + 0.45W (0.505)
1.0D + 0.75L + 0.75S + 0.45W (0.592)
1.0D + 0.75L + 0.75R + 0.45W (0.505)
$(1.0 + 0.10 \times S_{DS})D + 0.75L + 0.75S + 0.525E$ (0.660)
0.6D + 0.6W (0.281)
$(0.6 - 0.14 \times S_{DS})D + 0.7E$ (0.850)

Combination 16 results: $(0.6 - 0.14 \times S_{DS})D + 0.7E$

Forces on foundation

Force in x-axis F_{dx} = **199.5** kips | Force in z-axis F_{dz} = **581.2** kips

Moments on foundation

Moment in x-axis, about x is 0 M_{dx} = **34188.1** kip_ft | Moment in y-axis, about y is 0 M_{dy} = **1452.9** kip_ft

Uplift verification

Vertical force F_{dz} = **581.16** kips

PASS - Foundation is not subject to uplift

Stability against overturning in x direction, moment about x is L_x

Overturning moment M_{OTxL} = **2805.5** kip_ft | Resisting moment M_{RxL} = **-31382.63** kip_ft

Factor of safety abs(M_{RxL} / M_{OTxL}) = **11.186**

PASS - Overturning moment safety factor exceeds the minimum of 1.00

Stability against sliding

Resistance due to base friction $F_{RFriction}$ = **234.804** kips

Stability against sliding in x direction

Total sliding resistance F_{Rx} = **234.804** kips | Factor of safety abs(F_{Rx} / F_{dx}) = **1.18**

PASS - Sliding factor of safety exceeds the minimum of 1.00

Bearing resistance

Eccentricity of base reaction

Eccentricity in x-axis e_{dx} = **57.929** in | Eccentricity in y-axis e_{dy} = **0** in

Pad base pressures

Min. base press. q_{min} = **0.788** ksf | Max. base press. q_{max} = **1.365** ksf

Allowable bearing capacity

Allowable bearing capacity q_{allow} = **4.6** ksf | q_{max} / q_{allow} = **0.297**

PASS - Allowable bearing capacity exceeds design base pressure

FOOTING DESIGN (ACI318)

In accordance with ACI318-14

Material details

Compr. strength of concrete	f'_c = **4000** psi	Yield strength of reinforcement	f_y = **60000** psi
Cover to reinforcement	c_{nom} = **3** in	Concrete type	Normal weight
Concrete modification factor	λ = **1.00**	Column type	Concrete

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Shear Wall Foundations
location Eden, Utah
date 6/27/2017 by CW

Analysis and design of concrete footing

Load combinations per ASCE 7-10

1.4D (0.058)
1.2D + 1.6L + 0.5Lr (0.053)
1.2D + 1.6L + 0.5S (0.062)
1.2D + 1.6L + 0.5R (0.053)
1.2D + 1.0L + 1.6Lr (0.052)
1.2D + 1.0L + 1.6S (0.080)
1.2D + 1.0L + 1.6R (0.052)
1.2D + 1.6Lr + 0.5W (0.050)
1.2D + 1.6S + 0.5W (0.079)
1.2D + 1.6R + 0.5W (0.050)
1.2D + 1.0L + 0.5Lr + 1.0W (0.052)
1.2D + 1.0L + 0.5S + 1.0W (0.061)
1.2D + 1.0L + 0.5R + 1.0W (0.052)
(1.2 + 0.2 × S_{DS})D + 1.0L + 0.2S + 1.0E (0.060)
0.9D + 1.0W (0.038)
(0.9 - 0.2 × S_{DS})D + 1.0E (0.033)

Combination 16 results: (0.9 - 0.2 × S_{DS})D + 1.0E

Forces on foundation

Ultimate force in x-axis F_{ux} = **285.0** kips
Ultimate force in z-axis F_{uz} = **878.2** kips

Moments on foundation

Ultimate moment in x-axis, about x is 0 M_{ux} = **51464.8** kip_ft
Ultimate moment in y-axis, about y is 0 M_{uy} = **2195.6** kip_ft

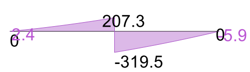
Eccentricity of base reaction

Eccentricity in x-axis e_{ux} = **55.202** in
Eccentricity in y-axis e_{uy} = **0** in

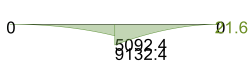
Pad base pressures

Min. ultimate base press. q_{umin} = **1.211** ksf
Max. ultimate base press. q_{umax} = **2.042** ksf

Shear diagram, x axis (kips)



Moment diagram, x axis (kip_ft)



Moment design, x direction, positive moment

Ultimate bending moment $M_{u.x.max}$ = **21.639** kip_ft
Tension reinf. provided 6 No.6 bot. bars (10.6 in c/c)
Area of tension reinf. provided $A_{sx.bot.prov}$ = **2.64** in^2
Min. area of reinforcement $A_{s.min}$ = **2.592** in^2

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinf. s_{max} = **18** in

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinf. d = **20.625** in | Depth of compression block a = **0.776** in

Neutral axis factor β_1 = **0.85** | Depth to neutral axis c = **0.913** in

Strain in tensile reinf. ε_t = **0.06473**

PASS - Tensile strain exceeds minimum required, 0.004

Nominal moment capacity M_n = **267.125** kip_ft | Flexural strength red. factor ϕ_f = **0.900**

Design moment capacity ϕM_n = **240.413** kip_ft | $M_{u.x.max}$ / ϕM_n = **0.090**

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, x direction

Ultimate shear force $V_{u.x}$ = **5.856** kips | Depth to reinforcement d_v = **20.625** in

Shear strength red. factor ϕ_v = **0.75** | Nominal shear capacity V_n = **156.533** kips

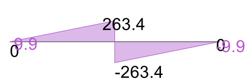
Design shear capacity ϕV_n = **117.4** kips | $V_{u.x}$ / ϕV_n = **0.050**

PASS - Design shear capacity exceeds ultimate shear load

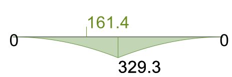
Two-way shear design at column 1

Two-way shear design does not apply. Shear perimeter falls outside extents of foundation.

Shear diagram, y axis (kips)



Moment diagram, y axis (kip_ft)



Moment design, y direction, positive moment

Ultimate bending moment $M_{u.y.max}$ = **161.408** kip_ft | Tension reinf. provided 128 No.6 bot. bars (10.1 in c/c)

Area of tension reinf. provided $A_{sy.bot.prov}$ = **56.32** in^2 | Min. area of reinforcement $A_{s.min}$ = **55.987** in^2

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinf. s_{max} = **18** in

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinf. d = **19.875** in | Depth of compression block a = **0.767** in

Neutral axis factor β_1 = **0.85** | Depth to neutral axis c = **0.902** in

Strain in tensile reinf. ε_t = **0.06309**

PASS - Tensile strain exceeds minimum required, 0.004

Nominal moment capacity M_n = **5488.823** kip_ft | Flexural strength red. factor ϕ_f = **0.900**

Design moment capacity ϕM_n = **4939.94** kip_ft | $M_{u.y.max}$ / ϕM_n = **0.033**

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location Eden, Utah
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PASS - Design moment capacity exceeds ultimate moment load

Footing geometry factor β_f = **21.600**
Area of reinf. req. for uniform distr. A_{sreq} = **3.518** in^2

9% of the reinforcement shall be distributed over a 5.00 ft width band centered under the column, the remainder distributed among the remainder of the footing

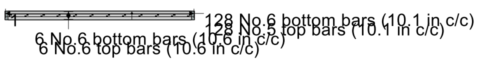
One-way shear design, y direction

Ultimate shear force $V_{u.y}$ = **9.913** kips
Depth to reinforcement d_v = **19.875** in
Shear strength red. factor ϕ_v = **0.75**
Nominal shear capacity V_n = **3258.158** kips
Design shear capacity ϕV_n = **2443.618** kips
$V_{u.y} / \phi V_n$ = **0.004**

PASS - Design shear capacity exceeds ultimate shear load

Two-way shear design at column 1

Two-way shear design does not apply. Shear perimeter falls outside extents of foundation.



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Shear Wall Foundations
location Eden, Utah
date 6/27/2017 by CW

SHEAR WALL MAT - P-7

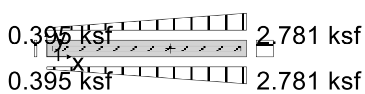
FOUNDATION ANALYSIS & DESIGN (ACI318)

In accordance with ACI318-14

Tedds calculation version 3.0.05

FOOTING ANALYSIS

Length of foundation	L_x = **60** ft	Width of foundation	L_y = **5** ft
Foundation area	A = **300** ft^2	Depth of foundation	h = **24** in
Depth of soil over foundation	h_{soil} = **90** in	Density of concrete	γ_{conc} = **150.0** lb/ft^3



Column no.1 details

Length of column	l_{x1} = **680.00** in	Width of column	l_{y1} = **18.00** in
position in x-axis	x_1 = **360.00** in	position in y-axis	y_1 = **30.00** in

Soil properties

Gross allow. bearing press.	q_{allow_Gross} = **4.6** ksf	Density of soil	γ_{soil} = **120.0** lb/ft^3
Angle of internal friction	ϕ_b = **30.0** deg	Design base friction angle	δ_{bb} = **31.0** deg
Coefficient of base friction	tan(δ_{bb}) = **0.600**		
Dead surcharge load	F_{Dsur} = **50** psf	Live surcharge load	F_{Lsur} = **100** psf
Self weight	F_{swt} = **300** psf	Soil weight	F_{soil} = **900** psf

Column no.1 loads

Dead load in z	F_{Dz1} = **438.0** kips	Live load in z	F_{Lz1} = **50.0** kips
Snow load in z	F_{Sz1} = **77.0** kips	Seismic load in z	F_{Ez1} = **78.0** kips
Seismic load in x	F_{Ex1} = **408.0** kips		
Dead load moment in x	M_{Dx1} = **540.0** kip_ft	Live load moment in x	M_{Lx1} = **566.0** kip_ft
Seismic load moment in x	M_{Ex1} = **3895.0** kip_ft		

Footing analysis for soil and stability

Load combinations per ASCE 7-10

1.0D (0.628)
1.0D + 1.0L (0.727)
1.0D + 1.0Lr (0.628)
1.0D + 1.0S (0.684)
1.0D + 1.0R (0.628)
1.0D + 0.75L + 0.75Lr (0.703)
1.0D + 0.75L + 0.75S (0.744)
1.0D + 0.75L + 0.75R (0.703)

1.0D + 0.6W (0.628)

(1.0 + 0.14 × S_{DS})D + 0.7E (0.958)

1.0D + 0.75L + 0.75Lr + 0.45W (0.703)

1.0D + 0.75L + 0.75S + 0.45W (0.744)

1.0D + 0.75L + 0.75R + 0.45W (0.703)

(1.0 + 0.10 × S_{DS})D + 0.75L + 0.75S + 0.525E (0.990)

0.6D + 0.6W (0.377)

(0.6 - 0.14 × S_{DS})D + 0.7E (0.999)

Combination 16 results: (0.6 - 0.14 × S_{DS})D + 0.7E

Forces on foundation

Force in x-axis F_{dx} = **285.6** kips Force in z-axis F_{dz} = **476.4** kips

Moments on foundation

Moment in x-axis, about x is 0 M_{dx} = **17869.4** kip_ft Moment in y-axis, about y is 0 M_{dy} = **1191.0** kip_ft

Uplift verification

Vertical force F_{dz} = **476.384** kips

PASS - Foundation is not subject to uplift

Stability against overturning in x direction, moment about x is L_x

Overturning moment M_{OTxL} = **3577.85** kip_ft Resisting moment M_{RxL} = **-14291.53** kip_ft

Factor of safety abs(M_{RxL} / M_{OTxL}) = **3.994**

PASS - Overturning moment safety factor exceeds the minimum of 1.00

Stability against sliding

Resistance due to base friction $F_{RFriction}$ = **285.831** kips

Stability against sliding in x direction

Total sliding resistance F_{Rx} = **285.831** kips Factor of safety abs(F_{Rx} / F_{dx}) = **1**

PASS - Sliding factor of safety exceeds the minimum of 1.00

Bearing resistance

Eccentricity of base reaction

Eccentricity in x-axis e_{dx} = **90.125** in Eccentricity in y-axis e_{dy} = **0** in

Pad base pressures

Min. base press. q_{min} = **0.395** ksf Max. base press. q_{max} = **2.781** ksf

Allowable bearing capacity

Allowable bearing capacity q_{allow} = **4.6** ksf q_{max} / q_{allow} = **0.604**

PASS - Allowable bearing capacity exceeds design base pressure

FOOTING DESIGN (ACI318)

In accordance with ACI318-14

Material details

Compr. strength of concrete f'_c = **4000** psi Yield strength of reinforcement f_y = **60000** psi

Cover to reinforcement c_{nom} = **3** in Concrete type Normal weight

Concrete modification factor λ = **1.00** Column type Concrete

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project Powder Mountain Parcel 4
Shear Wall Foundations
location Eden, Utah
date 6/27/2017 by CW

Analysis and design of concrete footing

Load combinations per ASCE 7-10

1.4D (0.069)
1.2D + 1.6L + 0.5Lr (0.068)
1.2D + 1.6L + 0.5S (0.072)
1.2D + 1.6L + 0.5R (0.068)
1.2D + 1.0L + 1.6Lr (0.064)
1.2D + 1.0L + 1.6S (0.078)
1.2D + 1.0L + 1.6R (0.064)
1.2D + 1.6Lr + 0.5W (0.059)
1.2D + 1.6S + 0.5W (0.073)
1.2D + 1.6R + 0.5W (0.059)
1.2D + 1.0L + 0.5Lr + 1.0W (0.064)
1.2D + 1.0L + 0.5S + 1.0W (0.069)
1.2D + 1.0L + 0.5R + 1.0W (0.064)
(1.2 + 0.2 × S_{DS})D + 1.0L + 0.2S + 1.0E (0.080)
0.9D + 1.0W (0.044)
(0.9 - 0.2 × S_{DS})D + 1.0E (0.047)

Combination 14 results: (1.2 + 0.2 × S_{DS})D + 1.0L + 0.2S + 1.0E

Forces on foundation

Ultimate force in x-axis F_{ux} = **408.0** kips Ultimate force in z-axis F_{uz} = **1243.3** kips

Moments on foundation

Ultimate moment in x-axis, about x is 0 M_{ux} = **43286.9** kip_ft Ultimate moment in y-axis, about y is 0 M_{uy} = **3108.3** kip_ft

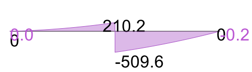
Eccentricity of base reaction

Eccentricity in x-axis e_{ux} = **57.791** in Eccentricity in y-axis e_{uy} = **0** in

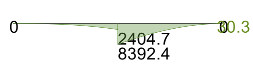
Pad base pressures

Min. ultimate base press. q_{umin} = **2.148** ksf Max. ultimate base press. q_{umax} = **6.14** ksf

Shear diagram, x axis (kips)



Moment diagram, x axis (kip_ft)



Moment design, x direction, positive moment

Ultimate bending moment $M_{u.x.max}$ = **30.266** kip_ft Tension reinf. provided 6 No.6 bot. bars (10.6 in c/c)
Area of tension reinf. provided $A_{sx.bot.prov}$ = **2.64** in^2 Min. area of reinforcement $A_{s.min}$ = **2.592** in^2

PASS - Area of reinforcement provided exceeds minimum

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Shear Wall Foundations
location Eden, Utah
date 6/27/2017 by CW

Maximum spacing of reinf.	s_{max} = **18** in		

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinf.	d = **20.625** in	Depth of compression block	a = **0.776** in
Neutral axis factor	β_1 = **0.85**	Depth to neutral axis	c = **0.913** in
Strain in tensile reinf.	ε_t = **0.06473**		

PASS - Tensile strain exceeds minimum required, 0.004

Nominal moment capacity	M_n = **267.125** kip_ft	Flexural strength red. factor	ϕ_f = **0.900**
Design moment capacity	ϕM_n = **240.413** kip_ft		$M_{u.x.max}$ / ϕM_n = **0.126**

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, x direction

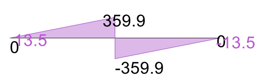
Ultimate shear force	$V_{u.x}$ = **0.229** kips	Depth to reinforcement	d_v = **20.625** in
Shear strength red. factor	ϕ_v = **0.75**	Nominal shear capacity	V_n = **156.533** kips
Design shear capacity	ϕV_n = **117.4** kips	$V_{u.x}$ / ϕV_n = **0.002**	

PASS - Design shear capacity exceeds ultimate shear load

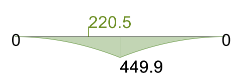
Two-way shear design at column 1

Two-way shear design does not apply. Shear perimeter falls outside extents of foundation.

Shear diagram, y axis (kips)



Moment diagram, y axis (kip_ft)



Moment design, y direction, positive moment

Ultimate bending moment	$M_{u.y.max}$ = **220.524** kip_ft	Tension reinf. provided	71 No.6 bot. bars (10.1 in c/c)
Area of tension reinf. provided	$A_{sy.bot.prov}$ = **31.24** in^2	Min. area of reinforcement	$A_{s.min}$ = **31.104** in^2

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinf.	s_{max} = **18** in		

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinf.	d = **19.875** in	Depth of compression block	a = **0.766** in
Neutral axis factor	β_1 = **0.85**	Depth to neutral axis	c = **0.901** in
Strain in tensile reinf.	ε_t = **0.06319**		

PASS - Tensile strain exceeds minimum required, 0.004

Nominal moment capacity	M_n = **3044.675** kip_ft	Flexural strength red. factor	ϕ_f = **0.900**
Design moment capacity	ϕM_n = **2740.207** kip_ft		$M_{u.y.max}$ / ϕM_n = **0.080**

PASS - Design moment capacity exceeds ultimate moment load

Footing geometry factor β_f = **12.000** Area of reinf. req. for uniform distr. A_{sreq} = **4.641** in^2

15% of the reinforcement shall be distributed over a 5.00 ft width band centered under the column, the remainder distributed among the remainder of the footing

One-way shear design, y direction

Ultimate shear force $V_{u.y}$ = **13.544** kips Depth to reinforcement d_v = **19.875** in

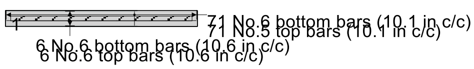
Shear strength red. factor ϕ_v = **0.75** Nominal shear capacity V_n = **1810.088** kips

Design shear capacity ϕV_n = **1357.566** kips $V_{u.y} / \phi V_n$ = **0.010**

PASS - Design shear capacity exceeds ultimate shear load

Two-way shear design at column 1

Two-way shear design does not apply. Shear perimeter falls outside extents of foundation.



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Shear Wall Foundations
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SHEAR WALL MAT - P-8

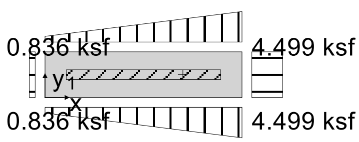
FOUNDATION ANALYSIS & DESIGN (ACI318)

In accordance with ACI318-14

Tedds calculation version 3.0.05

FOOTING ANALYSIS

Length of foundation L_x = **30** ft
Width of foundation L_y = **7** ft
Foundation area A = **210** ft^2
Depth of foundation h = **24** in
Depth of soil over foundation h_{soil} = **12** in
Density of concrete γ_{conc} = **150.0** lb/ft^3



Column no.1 details

Length of column l_{x1} = **282.00** in
Width of column l_{y1} = **18.00** in
position in x-axis x_1 = **180.00** in
position in y-axis y_1 = **42.00** in

Soil properties

Gross allow. bearing press. q_{allow_Gross} = **4.6** ksf
Density of soil γ_{soil} = **120.0** lb/ft^3
Angle of internal friction ϕ_b = **30.0** deg
Design base friction angle δ_{bb} = **31.0** deg
Coefficient of base friction $\tan(\delta_{bb})$ = **0.600**

Foundation loads

Dead surcharge load F_{Dsur} = **50** psf
Live surcharge load F_{Lsur} = **100** psf
Self weight F_{swt} = **300** psf
Soil weight F_{soil} = **120** psf

Column no.1 loads

Dead load in z F_{Dz1} = **246.0** kips
Live load in z F_{Lz1} = **55.0** kips
Snow load in z F_{Sz1} = **135.0** kips
Seismic load in z F_{Ez1} = **71.0** kips
Seismic load in x F_{Ex1} = **110.0** kips
Dead load moment in x M_{Dx1} = **500.0** kip_ft
Live load moment in x M_{Lx1} = **700.0** kip_ft
Seismic load moment in x M_{Ex1} = **1435.0** kip_ft

Footing analysis for soil and stability

Load combinations per ASCE 7-10

1.0D (0.460)

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1.0D + 1.0L (0.684)
1.0D + 1.0Lr (0.460)
1.0D + 1.0S (0.600)
1.0D + 1.0R (0.460)
1.0D + 0.75L + 0.75Lr (0.628)
1.0D + 0.75L + 0.75S (0.733)
1.0D + 0.75L + 0.75R (0.628)
1.0D + 0.6W (0.460)
(1.0 + 0.14 × S_{DS})D + 0.7E (0.789)
1.0D + 0.75L + 0.75Lr + 0.45W (0.628)
1.0D + 0.75L + 0.75S + 0.45W (0.733)
1.0D + 0.75L + 0.75R + 0.45W (0.628)
(1.0 + 0.10 × S_{DS})D + 0.75L + 0.75S + 0.525E (0.978)
0.6D + 0.6W (0.276)
(0.6 - 0.14 × S_{DS})D + 0.7E (0.562)

Combination 14 results: (1.0 + 0.10 × S_{DS})D + 0.75L + 0.75S + 0.525E

Forces on foundation

Force in x-axis F_{dx} = **57.8** kips
Force in z-axis F_{dz} = **560.2** kips

Moments on foundation

Moment in x-axis, about x is 0 M_{dx} = **10326.1** kip_ft
Moment in y-axis, about y is 0 M_{dy} = **1960.8** kip_ft

Uplift verification

Vertical force F_{dz} = **560.218** kips

PASS - Foundation is not subject to uplift

Stability against overturning in x direction, moment about x is L_x

Overturning moment M_{OTxL} = **1922.87** kip_ft
Resisting moment M_{RxL} = **-8403.26** kip_ft
Factor of safety abs(M_{RxL} / M_{OTxL}) = **4.370**

PASS - Overturning moment safety factor exceeds the minimum of 1.00

Stability against sliding

Resistance due to base friction $F_{RFriction}$ = **336.131** kips

Stability against sliding in x direction

Total sliding resistance F_{Rx} = **336.131** kips
Factor of safety abs(F_{Rx} / F_{dx}) = **5.82**

PASS - Sliding factor of safety exceeds the minimum of 1.00

Bearing resistance

Eccentricity of base reaction

Eccentricity in x-axis e_{dx} = **41.188** in
Eccentricity in y-axis e_{dy} = **0** in

Pad base pressures

Min. base press. q_{min} = **0.836** ksf
Max. base press. q_{max} = **4.499** ksf

Allowable bearing capacity

Allowable bearing capacity q_{allow} = **4.6** ksf
q_{max} / q_{allow} = **0.978**

PASS - Allowable bearing capacity exceeds design base pressure

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FOOTING DESIGN (ACI318)

In accordance with ACI318-14

Material details

Compr. strength of concrete f'_c = **4000** psi
Yield strength of reinforcement f_y = **60000** psi
Cover to reinforcement c_{nom} = **3** in
Concrete type Normal weight
Concrete modification factor λ = **1.00**
Column type Concrete

Analysis and design of concrete footing

Load combinations per ASCE 7-10

1.4D (0.134)
1.2D + 1.6L + 0.5Lr (0.149)
1.2D + 1.6L + 0.5S (0.175)
1.2D + 1.6L + 0.5R (0.149)
1.2D + 1.0L + 1.6Lr (0.136)
1.2D + 1.0L + 1.6S (0.220)
1.2D + 1.0L + 1.6R (0.136)
1.2D + 1.6Lr + 0.5W (0.115)
1.2D + 1.6S + 0.5W (0.199)
1.2D + 1.6R + 0.5W (0.115)
1.2D + 1.0L + 0.5Lr + 1.0W (0.136)
1.2D + 1.0L + 0.5S + 1.0W (0.162)
1.2D + 1.0L + 0.5R + 1.0W (0.136)
(1.2 + 0.2 × S_{DS})D + 1.0L + 0.2S + 1.0E (0.185)
0.9D + 1.0W (0.086)
(0.9 - 0.2 × S_{DS})D + 1.0E (0.103)

Combination 14 results: (1.2 + 0.2 × S_{DS})D + 1.0L + 0.2S + 1.0E

Forces on foundation

Ultimate force in x-axis F_{ux} = **110.0** kips
Ultimate force in z-axis F_{uz} = **627.6** kips

Moments on foundation

Ultimate moment in x-axis, about x is 0 M_{ux} = **12427.4** kip_ft
Ultimate moment in y-axis, about y is 0 M_{uy} = **2196.7** kip_ft

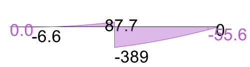
Eccentricity of base reaction

Eccentricity in x-axis e_{ux} = **57.608** in
Eccentricity in y-axis e_{uy} = **0** in

Pad base pressures

Min. ultimate base press. q_{umin} = **0.119** ksf
Max. ultimate base press. q_{umax} = **5.858** ksf

Shear diagram, x axis (kips)



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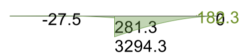
Moment diagram, x axis (kip_ft)



Moment design, x direction, positive moment

Ultimate bending moment	$M_{u.x.max}$ = **182.347** kip_ft	Tension reinf. provided	9 No.6 bot. bars (9.6 in c/c)
Area of tension reinf. provided	$A_{sx.bot.prov}$ = **3.96** in^2	Min. area of reinforcement	$A_{s.min}$ = **3.629** in^2

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinf. s_{max} = **18** in

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinf.	d = **20.625** in	Depth of compression block	a = **0.832** in
Neutral axis factor	β_1 = **0.85**	Depth to neutral axis	c = **0.979** in
Strain in tensile reinf.	ε_t = **0.06022**		

PASS - Tensile strain exceeds minimum required, 0.004

Nominal moment capacity	M_n = **400.139** kip_ft	Flexural strength red. factor	ϕ_f = **0.900**
Design moment capacity	ϕM_n = **360.125** kip_ft		$M_{u.x.max} / \phi M_n$ = **0.506**

PASS - Design moment capacity exceeds ultimate moment load

Moment design, x direction, negative moment

Ultimate bending moment	$M_{u.x.min}$ = **-14.494** kip_ft	Tension reinf. provided	9 No.6 top bars (9.6 in c/c)
Area of tension reinf. provided	$A_{sx.top.prov}$ = **3.96** in^2	Min. area of reinforcement	$A_{s.min}$ = **3.629** in^2

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinf. s_{max} = **18** in

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinf.	d = **20.625** in	Depth of compression block	a = **0.832** in
Neutral axis factor	β_1 = **0.85**	Depth to neutral axis	c = **0.979** in
Strain in tensile reinf.	ε_t = **0.06022**		

PASS - Tensile strain exceeds minimum required, 0.004

Nominal moment capacity	M_n = **400.139** kip_ft	Flexural strength red. factor	ϕ_f = **0.900**
Design moment capacity	ϕM_n = **360.125** kip_ft		abs($M_{u.x.min}$) / ϕM_n = **0.040**

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, x direction

Ultimate shear force	$V_{u.x}$ = **55.639** kips	Depth to reinforcement	d_v = **20.625** in
Shear strength red. factor	ϕ_v = **0.75**	Nominal shear capacity	V_n = **219.146** kips
Design shear capacity	ϕV_n = **164.359** kips	$V_{u.x} / \phi V_n$ = **0.339**	

PASS - Design shear capacity exceeds ultimate shear load

Two-way shear design at column 1

Two-way shear design does not apply. Shear perimeter falls outside extents of foundation.

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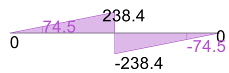
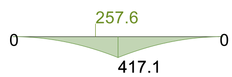
Shear diagram, y axis (kips)



Moment diagram, y axis (kip_ft)



Moment design, y direction, positive moment

Ultimate bending moment $M_{u.y.max}$ = **257.584** kip_ft
Tension reinf. provided 36 No.6 bot. bars (10.0 in c/c)
Area of tension reinf. provided $A_{sy.bot.prov}$ = **15.84** in^2
Min. area of reinforcement $A_{s.min}$ = **15.552** in^2

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinf. s_{max} = **18** in

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinf. d = **19.875** in
Depth of compression block a = **0.776** in
Neutral axis factor β_1 = **0.85**
Depth to neutral axis c = **0.913** in
Strain in tensile reinf. ε_t = **0.06227**

PASS - Tensile strain exceeds minimum required, 0.004

Nominal moment capacity M_n = **1543.352** kip_ft
Flexural strength red. factor ϕ_f = **0.900**
Design moment capacity ϕM_n = **1389.017** kip_ft
$M_{u.y.max} / \phi M_n$ = **0.185**

PASS - Design moment capacity exceeds ultimate moment load

Footing geometry factor β_f = **4.286**
Area of reinf. req. for uniform distr. A_{sreq} = **4.763** in^2

38% of the reinforcement shall be distributed over a 7.00 ft width band centered under the column, the remainder distributed among the remainder of the footing

One-way shear design, y direction

Ultimate shear force $V_{u.y}$ = **74.512** kips
Depth to reinforcement d_v = **19.875** in
Shear strength red. factor ϕ_v = **0.75**
Nominal shear capacity V_n = **905.044** kips
Design shear capacity ϕV_n = **678.783** kips
$V_{u.y} / \phi V_n$ = **0.110**

PASS - Design shear capacity exceeds ultimate shear load

Two-way shear design at column 1

Two-way shear design does not apply. Shear perimeter falls outside extents of foundation.

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