BSC

Design Calculation or Analysis Cover Sheet

Complete only applicable items.

1. QA: QA
2. Page 1

3. System	4. Document Identifier
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CRCF Interior Structural Steel Design

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A. CRCF Interior Structural Steel Design	3
-	-
-	-

RECORD OF REVISIONS

9. No.	10. Reason For Revision	11. Total # of Pgs.	12. Last Pg. #	13. Originator (Print/Sign/Date)	14. Checker (Print/Sign/Date)	15. EGS (Print/Sign/Date)	16. Approved/Accepted (Print/Sign/Date)
00A	Initial Draft	111	A-3	Jonathan Char	Kiritkumar Parikh	Michael Denlinger	Raj Rajagopal
00B	Extensive Revision: -Added 15 psf roof topping dead load to all roof framing members (revised beams B2, B3, B6). Pages 12, 17-30, 36-46, 64-93. -Revised roof slab thickness (+64'-0") from 1'-6" to 2'-9" at gridline 4-6 / E-G (revised girder G5) Pages 24, 25, 43, 44, 67, 92, A-2. -Revised load combination factor to 1.5 for all axial loaded members; truss chords & columns per AISC N690 Supplement No. 2 (revised members sizes in truss T1, and T2). Pages 8, 64-91, 93. -Added steel framing members around openings in floor slab (beam B11, girders G13, G14, and G15). Pages 35-36, 58-63, 67, 92, A-1. -Revised girder G11 for clearance. Pages 54, 55, 67, 92, A-1. -Removed Tornado Evaluation calculations Section 6.6, and tornado load combinations and tables. Pages 3, 4, 8, 15, 67, and Tables 1 & 6. -Re-verification required per EG-PRO-3DP-G04B-00037 Rev. 9, Section 3.6.2.	97	A-3	Jonathan Char Jon Char 8/15/07	John Rast 8/15/07	Michael Denlinger 8/15/07	Raj Rajagopal 8/16/07

DISCLAIMER

The calculations contained in this document were developed by Bechtel SAIC Company, LLC (BSC) and are intended solely for the use of BSC in its work for the Yucca Mountain Project.

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ACRONYMS AND ABBREVIATIONS

kip	1000 pound force
ksi	kips per square inch
lbs	pounds
pcf	pounds per cubic foot
psf	pounds per square foot
AISC	American Institute of Steel Construction
ANSI	American National Standards Institute
ASCE	American Society of Civil Engineers
ASD	Allowable Stress Design
ASTM	American Society for Testing and Materials
BSC	Bechtel SAIC Company
CRCF	Canister Receipt and Closure Facility
CHF	Canister Handling Facility
DBGM	design basis ground motion
D/C	Demand / Capacity Ratio
DG	Draft Guide
DIRS	Document Input Reference System
HVAC	heating, ventilation, air-conditioning
Hz	Hertz (cycles per second)
ICC	International Code Council
ITS	important to safety
NRC	Nuclear Regulatory Commission
NUREG	Nuclear Regulatory Commission technical report designation
ORD	Office of Repository Development

PDCD	Project Design Criteria Document
QMD	quality management directive
SAIC	Science Applications International Corporation
SSC's	structures, systems, and components
SASSI	System for Analysis of Soil-Structure Interaction
TAD	transportation aging and disposal
TIC	Technical Information Center
ZPA	Zero Period Acceleration

1. PURPOSE

The purpose of this calculation is to design the structural steel framing that supports the reinforced concrete floor and roof slabs of the Canister Receipt and Closure Facility (CRCF). The design of the interior steel framing system includes steel floor and roof decking, structural steel beams, girders, trusses and columns.

2. REFERENCES

2.1 PROJECT PROCEDURES/DIRECTIVES

2.1.1 BSC (Bechtel SAIC Company) 2007. EG-PRO-3DP-G04B-00037, Rev.009, ICN0. *Calculations and Analyses*. Las Vegas, Nevada: Bechtel SAIC Company. ACC:ENG.20070717.0004.

2.1.2 BSC (Bechtel SAIC Company) 2007. IT-PRO-0011 Rev.006, ICN0. *Software Management.* Las Vegas, Nevada, Bechtel SAIC Company. ACC: DOC.20070802.0001

2.1.3 ORD (Office of Repository Development) 2007. *Repository Project Management Automation Plan.* 000-PLN-MGR0-00200-000, Rev. 00E. Las Vegas, Nevada: U.S. Department of Energy, Office of Repository Development. ACC: ENG.20070326.0019

2.2 DESIGN INPUTS

2.2.1 BSC (Bechtel SAIC Company) 2006. *Project Design Criteria Document.* 000-3DR-MGR0-00100-000-006. Las Vegas, Nevada: Bechtel SAIC Company. ACC: ENG.20061201.0005.

2.2.2 BSC (Bechtel SAIC Company) 2006. *Basis of Design for the TAD Canister-Based Repository Design Concept.* 000-3DR-MGR0-00300-000-000. Las Vegas, Nevada: Bechtel SAIC Company. ACC: ENG.20061023.0002.

2.2.3 ASCE 7-98. 2000. *Minimum Design Loads for Buildings and Other Structures.* Revision of ANSI/ASCE 7-95. Reston, Virginia: American Society of Civil Engineers. TIC: 247427. [DIRS 149921]

2.2.4 ICC (International Code Council) 2000. *International Building Code 2000.* Falls Church, Virginia: International Code Council. TIC: 251054 [DIRS 159179]

2.2.5 AISC (American Institute of Steel Construction) 1997. *Manual of Steel Construction, Allowable Stress Design.* 9th Edition, 2nd Revision, 2nd Impression. Chicago, Illinois: American Institute of Steel Construction. TIC: 240772. [DIRS 107063]

2.2.6 ANSI/AISC N690-1994. 1994. *American National Standard Specification for the Design, Fabrication, and Erection of Steel Safety-Related Structures for Nuclear Facilities.* Chicago, Illinois: American Institute of Steel Construction. TIC: 252734. [DIRS 158835]

2.2.7 Not Used

2.2.8 Not Used

2.2.9 Not Used

2.2.10 BSC (Bechtel SAIC Company) 2006. *Canister Receipt and Closure Facility (CRCF) Seismic Analysis.* 060-SYC-CR00-00400-000-00A. Las Vegas, Nevada: Bechtel SAIC Company. ACC: ENG.20061220.0029.

2.2.11 Vulcraft 2001. Steel Roof and Floor Deck. Florence, South Carolina: Vulcraft. TIC: 256157. [DIRS 169951]

2.2.12 Pecknold, D.A. 1997. "Structural Analysis." Section 1 of *Structural Engineering Handbook.* 4th Edition. Gaylord, E.H., Jr.; Gaylord, C.N.; and Stallmeyer, J.E., eds. New York, New York: McGraw-Hill. TIC: 255067. [DIRS 172508]

2.2.13 BSC (Bechtel SAIC Company) 2007 *Canister Receipt and Closure Facility #1 Preliminary Layout Ground Floor* Plan. 060-P0K-CR00-10101-000-00B. Las Vegas, Nevada: Bechtel SAIC Company. ACC: ENG.20070418.0003.

2.2.14 BSC (Bechtel SAIC Company) 2007 *Canister Receipt and Closure Facility #1 Preliminary Layout Second Floor* Plan. 060-P0K-CR00-10102-000-00B. Las Vegas, Nevada: Bechtel SAIC Company. ACC: ENG.20070418.0004.

2.2.15 BSC (Bechtel SAIC Company) 2007 *Canister Receipt and Closure Facility #1 Preliminary Layout Third Floor* Plan. 060-P0K-CR00-10103-000-00B. Las Vegas, Nevada: Bechtel SAIC Company. ACC: ENG.20070418.0005.

2.2.16 ANSI/AISC 341-02. 2002. *Seismic Provisions for Structural Steel Buildings.* Chicago, Illinois: American Institute of Steel Construction. TIC: 256615. [DIRS 171789]

2.2.17 BSC (Bechtel SAIC Company) 2005. *CHF Slab Stiffness Evaluation.* 190-SYC-SY00-01600-000 REV 00A. Las Vegas, Nevada: Bechtel SAIC Company. ACC: ENG.20051019.0003.

2.2.18 BSC (Bechtel SAIC Company) 2004. *Analysis (SASSI) for Sample In-Structure Response Spectra for CHF with Two Closure Cells.* 190-SYC-SY00-01000-000-00A. Las Vegas, Nevada: Bechtel SAIC Company. ACC: ENG.20041101.0001.

2.2.19 ANSI/AISC N690-1994(R2004)s2. 2005. *Supplement No. 2 to the Specification for the Design, Fabrication, and Erection of Steel Safety-Related Structures for Nuclear Facilities.* Chicago, Illinois: American Institute of Steel Construction. TIC: 252734; 258040. [DIRS 177028]

2.3 DESIGN CONSTRAINTS

None

2.4 DESIGN OUTPUTS

Results from this calculation will be used as input for structural steel framing drawings for the Canister Receipt and Closure Facility (CRCF). Currently, document numbers have not been assigned for these drawings.

3. ASSUMPTIONS

3.1 ASSUMPTIONS REQUIRING VERIFICATION

3.1.1 Uniform design live load used is 100 pounds per square foot on floor slabs and the roof live load is 25 pounds per square foot (psf). (Ref. 2.2.3, & 2.2.4)

Rationale: A 100psf live load (Ref. 2.2.3) on floor slabs and 25psf live load (Ref. 2.2.4) on roof slabs are reasonable for buildings of this type. Elevated floor slabs are primarily mechanical rooms with HVAC fans and filters. The minimum required roof live load per code is 20psf. A roof live load of 25psf was chosen to encompass ash load, and snow loads as well.

Where used: Section 6.1.3 Design Loads

3.1.2 Forklift will be used to deliver lids into the closure maintenance rooms (between Gridline 5 to 6, from D to E, and G to H). Concentrated forklift wheel live load will be 16,000 lbs per wheel, spaced at 3’-9”.

Rationale: A forklift will be used in the closure support room with a maximum lifted capacity of 15,500 lbs. The front wheel load will be (32000lbs /2 wheels = 16000lbs) 16,000 lbs spaced at 3’-9” apart per model specifications. Loads will be considered on all participating members to design for maximum bending, shear, and deflection values. These values were taken from a representative forklift manufacturer’s data.

Where used: Section 6.1.3 Design Loads

3.1.3 Structural Steel framing dead loads are assumed to be 50 psf.

Rationale: The structural beam and girder steel represents a small fraction of the total mass of the *Canister Receipt and Closure Facility (CRCF)* structure. Actual steel weights will be used in the detailed design phase of the project.

Where used: Section 6.1.3 Design Loads

3.1.4 Heavy steel trusses are assumed to have a uniform dead load of 1000 pounds per linear foot.

Rationale: The 1000 plf represents an approximate weight of the trusses for preliminary design. Actual weight will be used in the detailed design phase.

Where used: Section 6.1.3 Design Loads

3.1.5 Miscellaneous hanging equipment (cable trays, ductwork, etc.) is 10 psf

Rationale: 10 psf for miscellaneous hanging equipment is an allotment for potential piping, electrical and HVAC components.

Where used: Section 6.1.3 Design Loads

3.1.6 Uniform equipment load is 50 psf.

Rationale: The *Canister Receipt and Closure Facility (CRCF)* elevated floors support primarily HVAC equipment. 50 psf is assumed in the preliminary design, actual equipment weights will be used as they are defined later in the detail design phase.

Where used: Section 6.1.3 Design Loads

3.1.7 Steel framing along column line 10 at elevation 32 ft. will support the floor slab at elevation 32 and the first wall placement assumed to be a 25 ft. lift.

Rationale: Steel framing at floor elevation 32' will support floor slab and wall (first concrete pour of wall) construction loads only. The first concrete placement is assumed to be a maximum 25 feet high, and subsequent placements will make up the remaining wall height. After the first lift cures, the wall itself will become a deep beam and support the remaining wall pours as well as the tributary roof load.

Where used: Section 6.4.2 Floor Girder Calculation

3.1.8 Amplified Slab Acceleration

The amplified slab acceleration to be considered for out of plane seismic loads is 2.0 times the floor acceleration obtained from the Canister Receipt and Closure Facility (CRCF) Seismic Analysis (Ref 2.2.10).

Rationale: The Tier-1 Seismic Analysis models did not include the effects of vertical floor flexibility, i.e. the floors were considered as rigid diaphragms. To obtain amplified vertical floor accelerations to be used in the design of floor slabs and supporting steel, the following process was used.

A SASSI analysis was performed on the Canister Handling Facility (CHF) (Ref. 2.2.18) which developed in-structure response spectra at hard points on the walls. Using the 7% damped vertical response spectra given in fig. F-3 (Ref. 2.2.17), a response ratio between the wall Zero Period Acceleration (ZPA) and the in-structure response was computed at various frequencies. A plot was generated of response ratio versus frequency.

A study was performed for the CHF where floor frequencies were computed for various slab geometry's (Ref. 2.2.18). Looking at the results of this study one can determine the fundamental vertical floor mode and obtain the frequency and mass participation for the various conditions studied. For an 18" floor with columns spaced at approximately 20' on centers the fundamental mode is approximately 25 Hz with a mass participation of 50%. Thus 50% of the mass is responding at this frequency and 50% of the mass responds at the ZPA. The following equation may be written:

$$\text{Response} = (0.5 * \text{mass} * \text{ZPA}) + (0.5 * \text{ratio} * \text{mass} * \text{ZPA})$$

Using the Response Ratio versus frequency plot described above the ratio for 25 Hz was found to be 2.3. The reference and method for the Response Ratio determination is provided in the last paragraph of page 11. Using this value in the response equation above results in:

$$\text{Response} = 0.5 * \text{mass} * \text{ZPA} + 0.5 * \text{mass} * 2.3 * \text{ZPA}$$

$$\text{Response} = 1.65 * \text{mass} * \text{ZPA}$$

Where ZPA is the vertical acceleration obtained from the CRCF Seismic Analysis (Ref. 2.2.10) at the floor level under consideration.

This procedure was done for various slabs and the results indicated that 2.0 * ZPA is a reasonable and conservative approximation of the vertical floor amplification for this type of structural configuration.

Where used: Section 6.1.3

3.1.9 Roofing dead load is 15 psf

Rationale: This is a reasonable assumption that allows for a lightweight roofing material to be applied over the concrete slab with an average thickness of 12 inches.

Where used: Section 6.1.3 Design Loads

3.1.10 Roof slab at gridline 5 to 6 from E to G is 2'-9" thick.

Rationale: The roof slab thickness shall be revised from 1'-6" to 2'-9" to maintain concrete diaphragm continuity between gridlines E to G at the +64'-0" roof elevation.

Where used: Section 6.3.1, and 6.4.1 Roof beam and girder design

Assumptions requiring verification will be verified in detailed design and are being tracked in Calctrac.

3.2 ASSUMPTIONS NOT REQUIRING VERIFICATION

3.2.1 Steel beams, girders and trusses are designed as Type 2 construction per section Q1.2 (Ref. 2.2.6) using simple framing. All formulas used to calculate moment, shear, and deflections are based on uniform and/or concentrated loads along the beam span, where maximum calculated values would be used for design.

Rationale– Design of beams assumed as simple span beams will produce the maximum possible moment about the strong axis and deflection values in long span beams which will control the design in a majority of steel members within this framing system. Formulas for simple span beam design are listed in the Beam Diagrams and Formulas Part 2, AISC 1997, (Ref. 2.2.5, & 2.2.6).

Where used: Assumption is used in the entire calculation

3.2.2 Composite action is not considered between the concrete slabs and the supporting structural steel beams, girders, and trusses.

Rationale- Not considering composite action between concrete and steel framing will produce the most conservative results.

Where used: Assumption is used in the entire calculation

3.2.3 Snow Load Importance Factor used is 1.2.

Rationale- Building is considered an essential facility Category IV. 1.2 is a bounding value for the importance factor Table 7-4 (Ref. 2.2.3).

Where used: Section 6.1.3.

3.2.4 Decking will provide full lateral support to top flanges of beams and girders during construction.

Rationale- Standard engineering practice.

Where used: Assumption is used in the entire calculation

4. METHODOLOGY

4.1 QUALITY ASSURANCE

This calculation was prepared in accordance with EG-PRO-3DP-G04B-00037, *Calculations and Analyses* (Ref. 2.1.1). Section 4.1.2 of the *Basis of Design for the TAD Canister–Based Repository Design Concept* (Ref. 2.2.2) classifies the CRCF structure as ITS. Therefore, the approved record version of this calculation is designated as QA: QA.

4.2 USE OF SOFTWARE

The commercial off-the-shelf software MathCAD 13 was used in this calculation. It is classified as Level 2 software usage as defined in IT-PRO-0011, *Software Management* (Ref. 2.1.2). MathCAD version 13 is listed on the Controlled Software Report (SW Tracking Number 611161-13-00) as well as the Repository Project Management Automation Plan (Ref. 2.1.3). The calculation process and equations are documented in Section 6 for checking by manual calculation.

Excel 2000 and Word 2000, which are part of the Microsoft Office 2000 Professional suite of programs, were used in this calculation. Word 2000 was used in the text preparation of this document, no calculation functions contained in Word were used in this document. Microsoft Office 2000 Professional as used in this calculation is classified as Level 2 software usage as defined in IT-PRO-0011 (Ref. 2.1.2). Microsoft Office 2000 Professional is listed on the Controlled Software Report (SW Tracking Number 610236-2000-00) as well as the *Repository Project Management Automation Plan* (Ref. 2.1.3).

The software was executed on a PC system running Microsoft Windows 2000 operating system. Results were confirmed by visual inspection and by performing hand calculations.

Computer based calculations will be used in this analysis. Hand calculation verification results may vary compared to computer calculations due to rounding in the computer software output.

4.3 DESIGN METHODOLOGY

The Canister Receipt and Closure Facility (CRCF) framing plans in Attachment A were developed from the general arrangement sketches (Ref. 2.2.13, 2.2.14, and 2.2.15). The following steps are performed to accomplish the design:

- Determine applicable loads and load combinations from the Project Design Criteria Document (Ref. 2.2.1), the Seismic Analysis and Design Approach Document (Ref. 2.2.10).

- Determine allowable spans for steel floor decking assuming unshored construction, using the section properties outlined in the Vulcraft Steel Roof and Floor Deck Catalog (Ref. 2.2.11). Note: this aspect of the design is not ITS. The steel decking is relied on only to support construction loads. The decking is not considered in service and extreme load combinations for the CRCF.

- W-shaped members tend to buckle out of the plane of bending due to bending about the strong axis. Therefore, lateral bracing is required and will be provided as follows:

    - Steel beams framing into trusses will provide lateral bracing to truss top and bottom chords. Lateral support for truss top chord is determined by the spacing of intermediate framing along the span of the truss.

    - Decking will provide full lateral support to top flanges of beams and girders during construction (Assumption 3.2.4). Concrete slabs provide lateral support to the top flanges of beams, girders, and truss top chords during service.

- Determine efficient floor and roof framing layout based on a maximum center to center beam spacing on results of the decking calculation described above. Beam spans will be limited from 20ft to approximately 30 ft. to achieve a balance between reduction of steel weight and minimizing the number of columns, girders and trusses.

- Structural steel will not be used for the 4’-0” slabs located between Gridline 4 to 9, from E to G. The use of steel framing and metal decking does not offer any advantage over shored concrete construction, as the allowable deck spans under construction loads would be minimal.

- Design structural steel beams, girders, trusses, and columns using Allowable Stress Design ASD (Ref 2.2.5) and ANSI N690 (Ref. 2.2.6). The structural steel framing system provides all vertical support for the concrete slabs for all applicable service and extreme load combinations. The use of steel framing and metal decking offers an advantage over shored concrete construction as an efficient means to support the concrete

roof and floor diaphragms during and after construction. The steel framing will be designed for construction loads, service loads, extreme abnormal loading and will remain as permanent framing in the building structural system.

- Verify that deflections are acceptable (Ref. 2.2.6)
- Demand/Capacity Ratio will be created in tabular format for every section of the calculation.
- The response of structural steel roof framing members to tornado missiles will be evaluated in a separate calculation.

5. LIST OF ATTACHMENTS

		Number of Pages
Attachment A.	Canister Receipt and Closure Facility Steel Framing Plans	3

6.0 BODY OF CALCULATION

6.1 INPUTS

6.1.1 CONSTANTS

$ksi \equiv 1000lbf \cdot in^{-2}$ $kcf \equiv 1000lbf \cdot ft^{-3}$ $kip \equiv 1000lbf$ $psf \equiv lbf \cdot ft^{-2}$ $ksf \equiv 1000lbf \cdot ft^{-2}$ $kpf \equiv 1000lbf \cdot ft^{-1}$

$k \equiv 1000lbf$ $fps \equiv ft \cdot sec^{-1}$

6.1.2 MATERIAL PROPERTIES, Ref. 2.2.1

Material properties are taken from Project Design Criteria Document 000-3DR-MGR0-00100-000-006, Section 4.2.11.6 (Ref. 2.2.1)

Structural Steel

$F_y := 50ksi$ Structural steel yield strength for ASTM A992 (Ref. 2.2.1)

$F_u := 65ksi$ Structural steel ultimate tensile strength for ASTM A992 (Ref. 2.2.1)

$F_{y36} := 36ksi$ Structural plate yield strength, ASTM A36 for plate girders & stiffeners (Ref. 2.2.1)

$F_{u36} := 58ksi$ Structural steel ultimate tensile strength for ASTM A36 (Ref. 2.2.1)

$E_s := 29000ksi$ Steel modulus of elasticity (Ref. 2.2.1)

$\gamma_s := 490pcf$ Unit weight of structural steel (Ref. 2.2.1)

Concrete

$\gamma_c := 150pcf$ Unit weight of normal concrete (Ref. 2.2.1)

6.1.3 DESIGN LOADS; Section 4.2.11.3, Ref. 2.2.1

Ash Loads (A) 4.2.11.3.4, Ref. 2.2.1

The design basis for volcanic ash is a 4" depth at a density of 63 pcf.
per PDC, sections 4.2.11.3.4, and 6.1.11 (Ref. 2.2.1)

$\gamma_a := 63pcf$ Ash density per section 6.1.11 (Ref. 2.2.1)

$d_a := 4in$ Ash depth per section 6.1.11 (Ref. 2.2.1)

$A := \gamma_a \cdot d_a$ $A = 21\,psf$ Ash load

Construction Loads (C) 4.2.11.3.16, Ref. 2.2.1

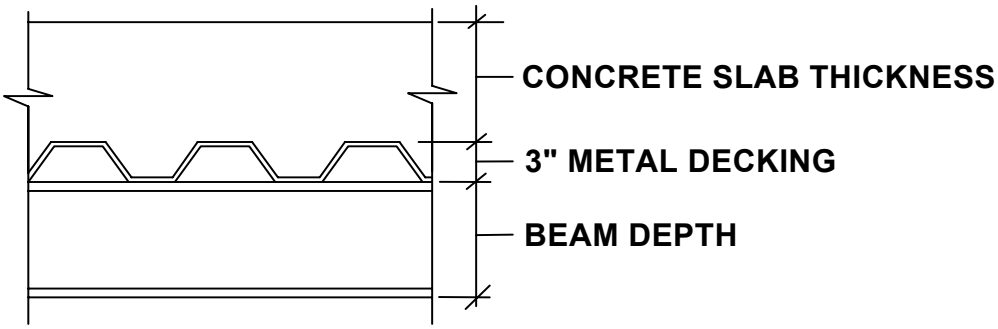
Steel deck supporting concrete shall be designed for the weight of concrete plus a 50 psf uniformly distributed load.

Structural Steel Framing supporting a steel deck shall be designed for the following load case: the wet weight of concrete plus a 50 psf uniformly distributed load. In addition a 5,000 lb concentrated load shall be placed in the middle of the span to design for maximum moment and at the end of the span to design for maximum shear.

$C := 50psf$ construction load to account for steel deck supporting wet concrete

$C_p := 5000lbf$ Construction load placed anywhere along span for max moment & shear

Dead Loads (D) 4.2.11.3.1, Ref. 2.2.1



1'-6" thick concrete slab on top of 3" metal decking (Total slab thickness t = 1'-9")

$t_{cs} := 1.5ft$ concrete slab thickness $t_{deck} := 0.25ft$ deck thickness

$D_{slab1.5} := \left(t_{cs} + \frac{t_{deck}}{2}\right) \cdot \gamma_c$ $D_{slab1.5} = 244psf$ concrete weight

2'-9" thick concrete slab on top of 3" metal decking (Total slab thickness t = 3'-0")

$t_{cs3} := 2.75ft$ concrete slab thickness $t_{deck3} := 0.25ft$ deck thickness

$D_{slab3.0} := \left(t_{cs3} + \frac{t_{deck3}}{2}\right) \cdot \gamma_c$ $D_{slab3.0} = 431psf$ concrete weight

Miscellaneous dead loads

$D_{hang} := 10psf$ Miscellaneous hanging equipment (Assumption 3.1.5)

$D_{roofing} := 15psf$ Roofing load (Assumption 3.1.9)

$D_{equip} := 50.0psf$ Equipment load, including HVAC, Piping, and Electrical, after concrete has set. See Note below. (Assumption 3.1.6)

$D_{beam} := 50psf$ Minimum considered design load (Assumption 3.1.3)

$D_{gird} := 1000plf$ Heavy steel truss (Assumption 3.1.4)

$D_{1.5rf} := D_{slab1.5} + D_{hang} + D_{roofing} + D_{equip} + D_{beam}$ $D_{1.5rf} = 369psf$ total distributed roof dead load for 1'-9" thick slab

$D_{1.5fl} := D_{slab1.5} + D_{hang} + D_{equip} + D_{beam}$ $D_{1.5fl} = 354psf$ total distributed floor dead load for 1'-9" thick slab

$D_{3.0} := D_{slab3.0} + D_{hang} + D_{roofing} + D_{equip} + D_{beam}$ $D_{3.0} = 556psf$ total distributed dead load for 3'-0" thick slab

Live Loads (L and L_r) 4.2.11.3.2

$L := 100psf$ (Assumption 3.1.1)

$L_p := 16.0kip$ Concentrated floor live load, per forklift wheel load spaced at 3'-9" o.c. located between gridlines 5 and 6, from D to E, and G to H. See beam B9 section 6.3, and girders G9 & G10 section 6.4. (Assumption 3.1.2)

$L_r := 25psf$ Roof Live Load conservatively used 25psf to envelope ash load, and snow live loads; (Assumption 3.1.1), The minimum Roof Live Load required is 20 psf per IBC2000 Section 1607.11.2 (Ref. 2.2.4, Eqn. 16-4: $L_r = 20*R_1*R_2$ (R_1 = 1.0, R_2 = 1.0) L_r = 20psf)

Snow Loads (S_n) 4.2.11.3.3

The maximum daily snowfall depth is 6 inches per PDC, section 4.2.11.3.3 (Ref. 2.2.1), converted into a snow load of 18 psf (depth x density x importance factor = load) using the maximum density (30 pcf) per ASCE 7-98 (Ref. 2.2.3)

$d_{sn} := 6in$ daily snowfall depth based on section 6.1.1 (Ref. 2.2.1)

$\gamma_{sn} := 30pcf$ maximum snow density based on equation 7-4 (max density required for design), ASCE 7-98 (Ref. 2.2.3.)

$P_g := d_{sn} \cdot \gamma_{sn}$ $P_g = 15\,psf$ Calculated ground snow load

$I := 1.2$ (Assumption 3.2.3)

$P_f := I \cdot P_g$ $P_f = 18\,psf$ Flat roof snow load per section 7.3, ASCE 7-98 (Ref. 2.2.3)

$S_n := P_f$ $S_n = 18\,psf$ Roof snow load

Seismic Loads (E) 4.2.11.3.8

The seismic analysis and design for the CRCF is considered an ITS SSC's and is assigned a design basis ground motion (DBGM) postulated category. For the purposes of this calculation, the CRCF is assigned a DBGM-2 Mean annual probability of exceedence of $5x10^{-4}$ (2000 year return period). Canister Receipt and Closure Facility (CRCF) Seismic Analysis, Table 13 (Ref. 2.2.10) lists the maximum vertical acceleration as 0.82 at level 100 ft. This value is conservative for the lower diaphragm levels, therefore is acceptable for preliminary design. Actual design accelerations for each diaphragm level shall be considered in final detailed design.

$S_{aV} := 0.82$ Maximum diaphragm vertical acceleration per Table 13 (Diaphragm Acceleration for DBGM-2, SRSS Combination) Canister Receipt and Closure Facility Seismic Analysis; Document No. 060-SYC-CR00-00400-000-00A (Ref. 2.2.10)

$Amp := 2.0$ Amplified slab acceleration factor (Assumption 3.1.8)

$E_{1.5} := (D_{1.5rf} + 0.25 \cdot L_r) \cdot S_{aV} \cdot Amp$ $E_{1.5} = 615\,psf$ Vertical earthquake load for 1'-9" thick slab at roof

$E_{1.5_floor} := (D_{1.5fl} + 0.25 \cdot L) \cdot S_{aV} \cdot Amp$ $E_{1.5_floor} = 621\,psf$ Vertical earthquake load for 1'-9" thick slab at floor

$E_{3.0} := (D_{3.0} + 0.25 \cdot L_r) \cdot S_{aV} \cdot Amp$ $E_{3.0} = 923\,psf$ Vertical earthquake load for 3'-0" thick slab at roof

Operating Pipe Reactions (R_o) 4.2.11.3.12; There are no operating pipes in this building, therefore, pipe reaction loading are not considered in this calculation.

Thermal Loads (T_o) 4.2.11.3.9; There are no thermal loads in this building, therefore thermal loads are not considered in this calculation.

6.1.4 LOAD COMBINATIONS FOR ROOF & FLOOR FRAMING DESIGN, Section 4.2.11.4.6, Ref. 2.2.1

Normal and Abnormal load combinations are derived from the following combinations to be considered for roof and floor loading per section 4.2.11.4.6 (Ref. 2.2.1) as follows:

1.	$S = D + L + (L_r \text{ or } A)$	normal load combination 1
2.	$S = D + L + S_N$	normal load combination 2
9.	$1.6S = D + L + L_r + R_o + T_o + E$	abnormal load combination 9 (1.4S for consideration of shear loads and 1.5S for consideration of axial compression loads)

The structural steel framing considered in this calculation is subjected to vertical loads only. Lateral loads are resisted by the concrete diaphragm and shear wall system. An evaluation of the load combinations contained in the Project Design Criteria Document; Revision 006 section 4.2.11.4.6 (Ref. 2.2.1) has determined the following load combinations will govern. A comprehensive evaluation of all load combinations will be made during final detailed design.

NORMAL LOAD COMBINATIONS for 1'-9" THICK SLAB 4.2.11.4.6, Ref. 2.2.1

$S1_a := D_{1.5rf} + L_r$	$S1_a = 393.8\,psf$	roof load combination
$S1_b := D_{1.5rf} + A$	$S1_b = 389.8\,psf$	roof load combination
$S2_a := D_{1.5rf} + S_n$	$S2_a = 386.8\,psf$	roof load combination
$S1_c := D_{1.5fl} + L$	$S1_c = 453.8\,psf$	floor load combination

NOTE: Forklift truck load will be considered separately in addition to uniform loading for floor load combination $S1_c$. see beam B9 section 6.3, and girders G9, and G10 section 6.4.

ABNORMAL LOAD COMBINATIONS for 1'-9" THICK SLAB 4.2.11.4.6, Ref. 2.2.1

$S9_a := \frac{D_{1.5rf} + 0.25L_r + E_{1.5}}{1.6}$	$S9_a = 618.8\,psf$	roof flexure load combination
$S9_b := \frac{D_{1.5rf} + 0.25L_r + E_{1.5}}{1.4}$	$S9_b = 707.1\,psf$	roof shear load combination
$S9_c := \frac{D_{1.5fl} + 0.25L + E_{1.5_floor}}{1.6}$	$S9_c = 624.9\,psf$	floor flexure load combination
$S9_d := \frac{D_{1.5fl} + 0.25L + E_{1.5_floor}}{1.4}$	$S9_d = 714.2\,psf$	floor shear load combination

NORMAL LOAD COMBINATIONS for 3'-0" THICK SLAB 4.2.11.4.6, Ref. 2.2.1

$S1_d := D_{3.0} + L_r$	$S1_d = 581.3\,psf$	roof load combination
$S1_e := D_{3.0} + A$	$S1_e = 577.3\,psf$	roof load combination
$S2_b := D_{3.0} + S_n$	$S2_b = 574.3\,psf$	roof load combination

ABNORMAL LOAD COMBINATIONS for 3'-0" THICK SLAB 4.2.11.4.6, Ref. 2.2.1

$S9_e := \frac{D_{3.0} + 0.25L_r + E_{3.0}}{1.6}$ $S9_e = 928.1\,psf$ roof flexure load combination

$S9_f := \frac{D_{3.0} + 0.25L_r + E_{3.0}}{1.4}$ $S9_f = 1060.7\,psf$ roof shear load combination

MAX LOADING AT 1'-9" THICK CONCRETE SLAB

$BMax_{rf} := \max(S1_a, S1_b, S2_a, S9_a)$ $BMax_{rf} = 618.8\,psf$ Maximum roof load for bending

$VMax_{rf} := \max(S1_a, S1_b, S2_a, S9_b)$ $VMax_{rf} = 707\,psf$ Maximum roof load for shear

$BMax_{fl} := \max(S1_c, S9_c)$ $BMax_{fl} = 625\,psf$ Maximum floor load for bending

$VMax_{fl} := \max(S1_c, S9_d)$ $VMax_{fl} = 714\,psf$ Maximum floor load for shear

MAX LOADING AT 3'-0" THICK CONCRETE SLAB

$BMax_{rf3} := \max(S1_d, S1_e, S2_b, S9_e)$ $BMax_{rf3} = 928\,psf$ Maximum roof load for bending

$VMax_{rf3} := \max(S1_d, S1_e, S2_b, S9_f)$ $VMax_{rf3} = 1061\,psf$ Maximum roof load for shear

CONSTRUCTION LOAD COMBINATION

$S19 := D_{1.5rf} + C - D_{equip}$ $S19 = 369\,psf$ distributed construction loading; equipment loading will not be placed until after construction, therefore the equipment loading has been removed

$S20 := D_{3.0} + C - D_{equip}$ $S20 = 556\,psf$ distributed construction loading; equipment loading will not be placed until after construction, therefore the equipment loading has been removed

6.2 DECK SPAN CALCULATION, Ref. 2.2.11

Metal Deck Maximum Span

Use Vulcraft deck type **3C16** non-composite section properties data (Ref. 2.2.11)

$F_{y_deck} := 33ksi$ Minimum yield strength specified for Vulcraft non-composite steel form deck

$F_{b_deck} := F_{y_deck} \cdot (0.60)$ $F_{b_deck} = 19.8\,ksi$ SDI specifications and commentary for non-composite steel floor deck (Ref. 2.2.11)

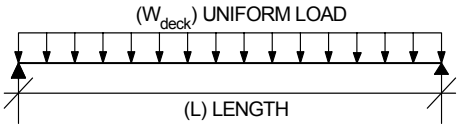
$I_{deck} := 1.58in^4$ $S_{deck} := 1.013in^3$ Section properties for positive and negative moment (Ref. 2.2.11)

Allowable maximum deck span, $L_{1.5}$, for 1'-6" thick (nominal) concrete slab

$w_{deck1.5} := (D_{slab1.5} + C) \cdot 1ft$ $w_{deck1.5} = 293.8\,plf$ per 1 foot width

$M_{deck1.5} := F_{b_deck} \cdot S_{deck}$ $M_{deck1.5} = 1.67\,k \cdot ft$ Allowable moment of metal deck

Derived from the simple bending moment formula (Beam Diagram and Formulas, AISC 1997, Ref. 2.2.5) assume a single span loaded metal deck:



$L_{1.5} := \left(\frac{8 \cdot M_{deck1.5}}{w_{deck1.5}}\right)^{.5}$ $L_{1.5} = 6.75\ ft$ (clear span) USE 6'-9" max. span from center to center of supports

$\Delta_{allow} := \frac{L_{1.5}}{240}$ $\Delta_{allow} = 0.337\,in$ maximum allowable deflection (Ref. 2.2.4) Section 1604, Table 1604.3

$\Delta_{1.5} := \frac{5 \cdot w_{deck1.5} \cdot L_{1.5}^{4}}{384 \cdot E_s \cdot I_{deck}}$ $\Delta_{1.5} = 0.299\,in$ calculated deflection based on maximum clear span

$deflection_check := if\left(\Delta_{1.5} \le \Delta_{allow}, "OK", "N.G!"\right)$ deflection_check = "OK"

Allowable maximum deck span, $L_{3.0}$, for 2'-9" thick (nominal) concrete slab

$w_{deck3.0} := \left(D_{slab3.0} + C\right) \cdot 1ft$ $w_{deck3.0} = 481.3\,plf$ per 1 foot width

$M_{deck3.0} := F_{b_deck} \cdot S_{deck}$ $M_{deck3.0} = 1.67\,k \cdot ft$ (Allowable moment of metal deck)

Derived from the simple bending moment formula from Beam Diagram and Formulas, AISC 1997 (Ref. 2.2.5) assume a single span loaded metal deck as shown above:

$L_{3.0} := \left(\frac{8 \cdot M_{deck3.0}}{w_{deck3.0}}\right)^{.5}$ $L_{3.0} = 5.27\ ft$ ft. (clear span) USE 5'-6" max. span from center to center of supports. (Clear span + min 5" flange width = 5.69') which is greater than 5'-6" O.C. Therefore 5'-6" spacing is OK.

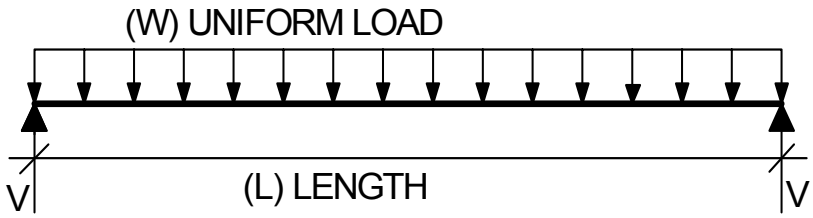
$\Delta_{allow} := \frac{L_{3.0}}{240}$ $\Delta_{allow} = 0.264\,in$ maximum allowable deflection (Ref. 2.2.4) Section 1604, Table 1604.3

$\Delta_{3.0} := \frac{5 \cdot w_{deck3.0} \cdot L_{3.0}^{4}}{384 \cdot E_s \cdot I_{deck}}$ $\Delta_{3.0} = 0.182\,in$ calculated deflection based on maximum clear span

$deflection_check := if\left(\Delta_{3.0} \le \Delta_{allow}, "OK", "N.G!"\right)$ deflection_check = "OK"

6.3 BEAM CALCULATIONS (BEAM SIZES SHOWN ON PAGE 67)

6.3.1 ROOF BEAM CALCULATION, Ref. 2.2.5, Ref. 2.2.6



Typical Beam loading diagram, Part 2, Beam Diagrams and Formulas, AISC 1997 (Ref. 2.2.5)

ROOF BEAM B1

ROOF ELEV. 64'-0" - Steel Framing Design between Gridline 2 to 5, and 9 to 12 from D to E, and G to H.
Nominal concrete slab thickness = 1'-6" (Attachment A-2)

BEAM PROPERTIES per AISC 1997 (Ref. 2.2.5)

$B1 := \text{"W16X31"}$ $M_r := 130k\cdot ft$ $d := 15.88in$ $t_w := 0.275in$ $I := 375in^4$ $t_f := 0.440in$

$L1 := 14.5ft$	$trib_{b1} := 6.5ft$	Max. span and spacing
$\omega_{b1} := BMax_{rf}\cdot trib_{b1}$	$\omega_{b1} = 4.02\,klf$	Uniform beam loading for flexure
$\omega_{b1v} := VMax_{rf}\cdot trib_{b1}$	$\omega_{b1v} = 4.6\,klf$	Uniform beam loading for shear
$\omega_{b1_const} := S19\cdot trib_{b1}$	$\omega_{b1_const} = 2.4\,klf$	Uniform beam construction loading
$M_{b1} := 0.125\omega_{b1}\cdot L1^2$	$M_{b1} = 105.7\,k\cdot ft$	beam moment
$M_{b1_const} := 0.125(\omega_{b1_const})\cdot L1^2 + 0.25C_p\cdot L1$	$M_{b1_const} = 81.1\,k\cdot ft$	beam moment w/ construction loading
$V_{b1} := \frac{\omega_{b1v}\cdot L1}{2}$	$V_{b1} = 33.3\,kip$	beam shear
$V_{b1_const} := \frac{\omega_{b1_const}\cdot L1}{2} + C_p$	$V_{b1_const} = 22.4\,kip$	beam shear w/ construction loading
$M_{b1} := \max(M_{b1}, M_{b1_const})$	$M_{b1} = 105.7\,k\cdot ft$	max beam moment
$V_{b1} := \max(V_{b1}, V_{b1_const})$	$V_{b1} = 33.3\,kip$	max beam shear

Allowable shear stress per ANSI/AISC N690-1994 section Q1.5.1.2.1 (Ref. 2.2.6)

$F_v := 0.4\cdot F_y$	$F_v = 20\,ksi$	allowable shear stress
$f_v := \frac{V_{b1}}{d\cdot t_w}$	$f_v = 7.6\,ksi$	calculated shear stress
$DC_f := \frac{M_{b1}}{M_r}$	$DC_f = 0.81$	flexure demand/capacity ratio
$DC_v := \frac{f_v}{F_v}$	$DC_v = 0.38$	shear demand/capacity ratio

$\Delta_{B1} := \frac{5\cdot(S1_a\cdot trib_{b1})\cdot L1^4}{384\cdot E_s\cdot I}$ $\Delta_{B1} = 0.234\,in$ max (D + L_r) beam deflection

$\Delta_{allow} := \frac{L1}{240}$ $\Delta_{allow} = 0.725\,in$ allowable deflection(Ref. 2.2.4) $\frac{\Delta_{B1}}{\Delta_{allow}} = 0.32$ deflection demand/capacity ratio

shear_check := if($f_v \le F_v$, "OK", "N.G!") shear_check = "OK"

moment_check := if($M_{b1} \le M_r$, "OK", "N.G!") moment_check = "OK"

deflection_check := if($\Delta_{B1} \le \Delta_{allow}$, "OK", "N.G!") deflection_check = "OK"

depth_check1 := if$\left(d \ge \frac{F_y\cdot L1}{1000\cdot ksi}, \text{"OK"}, \text{"N.G!"}\right)$ ANSI/AISC N60-1994 section Q1.13, CQ1.13 (Ref. 2.2.6) depth_check1 = "OK"

depth_check2 := if$\left(d \ge \frac{L1}{20}, \text{"OK"}, \text{"N.G!"}\right)$ ANSI/AISC N60-1994 section Q1.13, CQ1.13 (Ref. 2.2.6) depth_check2 = "OK"

ROOF BEAM B1 USE: B1 = "W16X31"

ROOF BEAM B2
ROOF ELEV. 64'-0" - Steel Framing Design between Gridline 5 and 6, from D to E, and G to H.
Nominal concrete slab thickness = 1'-6" (Attachment A-2)

BEAM PROPERTIES per AISC 1997, Ref. 2.2.5

B2 := "W24X68" $M_r := 424k\cdot ft$ $d := 23.73in$ $t_w := 0.415in$ $I := 1830in^4$ $t_f := 0.585in$

$L2 := 26ft$	$trib_{b2} := 6.0ft$	Max. span and spacing
$\omega_{b2} := BMax_{rf}\cdot trib_{b2}$	$\omega_{b2} = 3.71\,klf$	Uniform beam loading for flexure
$\omega_{b2v} := VMax_{rf}\cdot trib_{b2}$	$\omega_{b2v} = 4.2\,klf$	Uniform beam loading for shear
$\omega_{b2_const} := S19\cdot trib_{b2}$	$\omega_{b2_const} = 2.2\,klf$	Uniform beam construction loading
$M_{b2} := 0.125\omega_{b2}\cdot L2^2$	$M_{b2} = 313.7\,k\cdot ft$	beam moment
$M_{b2_const} := 0.125(\omega_{b2_const})\cdot L2^2 + 0.25C_p\cdot L2$	$M_{b2_const} = 219.5\,k\cdot ft$	beam moment w/ construction loading
$V_{b2} := \frac{\omega_{b2v}\cdot L2}{2}$	$V_{b2} = 55.2\,kip$	beam shear
$V_{b2_const} := \frac{\omega_{b2_const}\cdot L2}{2} + C_p$	$V_{b2_const} = 33.8\,kip$	beam shear w/ construction loading
$M_{b2} := \max(M_{b2}, M_{b2_const})$	$M_{b2} = 313.7\,k\cdot ft$	max beam moment
$V_{b2} := \max(V_{b2}, V_{b2_const})$	$V_{b2} = 55.2\,kip$	max beam shear

Allowable shear stress per ANSI/AISC N690-1994 section Q1.5.1.2.1 (Ref. 2.2.6)

$F_v := 0.4\cdot F_y$ $F_v = 20\,ksi$ allowable shear stress

$f_v := \frac{V_{b2}}{d\cdot t_w}$ $f_v = 5.6\,ksi$ calculated shear stress

$DC_f := \frac{M_{b2}}{M_r}$ $DC_f = 0.74$ flexure demand/capacity ratio

$DC_v := \frac{f_v}{F_v}$ $DC_v = 0.28$ shear demand/capacity ratio

$\Delta_{B2} := \frac{5 \cdot (S1_a \cdot trib_{b2}) \cdot L2^4}{384 \cdot E_s \cdot I}$ $\Delta_{B2} = 0.458\,in$ max (D + L_r) beam deflection

$\Delta_{allow} := \frac{L2}{240}$ $\Delta_{allow} = 1.3\,in$ allowable deflection (Ref.2.2.4) $\frac{\Delta_{B2}}{\Delta_{allow}} = 0.35$ deflection demand/capacity ratio

$shear_check := if(f_v \le F_v, "OK", "N.G!")$ shear_check = "OK"

$moment_check := if(M_{b2} \le M_r, "OK", "N.G!")$ moment_check = "OK"

$deflection_check := if(\Delta_{B2} \le \Delta_{allow}, "OK", "N.G!")$ deflection_check = "OK"

$depth_check1 := if\left(d \ge \frac{F_y \cdot L2}{1000 \cdot ksi}, "OK", "N.G!"\right)$ ANSI/AISC N690-1994 section Q1.13, CQ1.13 (Ref. 2.2.6) depth_check1 = "OK"

$depth_check2 := if\left(d \ge \frac{L2}{20}, "OK", "N.G!"\right)$ ANSI/AISC N690-1994 section Q1.13, CQ1.13 (Ref. 2.2.6) depth_check2 = "OK"

ROOF BEAM B2 USE: B2 = "W24X68"

ROOF BEAM B3
ROOF ELEV. 64'-0" - Steel Framing Design between Gridline 4 and 6, from E to G.
Nominal concrete slab thickness = 2'-9" (Attachment A-2) assumption 3.1.10

BEAM PROPERTIES per AISC 1997, Ref. 2.2.5

$B3 := "W24X55"$ $M_r := 314k \cdot ft$ $d := 23.57in$ $t_w := 0.395in$ $I := 1350in^4$ $t_f := 0.505in$

$L3 := 22.5ft$ $trib_{b3} := 4.5ft$ Max. span and spacing

$\omega_{b3} := BMax_{rf3} \cdot trib_{b3}$ $\omega_{b3} = 4.18\,klf$ Uniform beam loading for flexure

$\omega_{b3v} := VMax_{rf3} \cdot trib_{b3}$ $\omega_{b3v} = 4.8\,klf$ Uniform beam loading for shear

$\omega_{b3_const} := S20 \cdot trib_{b3}$ $\omega_{b3_const} = 2.5\,klf$ Uniform beam construction loading

$M_{b3} := 0.125 \omega_{b3} \cdot L3^2$ $M_{b3} = 264.3\,k \cdot ft$ beam moment

$M_{b3_const} := 0.125(\omega_{b3_const}) \cdot L3^2 + 0.25 C_p \cdot L3$ $M_{b3_const} = 186.5\,k \cdot ft$ beam moment w/ construction loading

$V_{b3} := \frac{\omega_{b3v} \cdot L3}{2}$ $V_{b3} = 53.7\,kip$ beam shear

$V_{b3_const} := \frac{\omega_{b3_const} \cdot L3}{2} + C_p$ $V_{b3_const} = 33.2\,kip$ beam shear w/ construction loading

$M_{b3} := \max(M_{b3}, M_{b3_const})$ $M_{b3} = 264.3\,k \cdot ft$ max beam moment

$V_{b3} := \max(V_{b3}, V_{b3_const})$ $V_{b3} = 53.7\,kip$ max beam shear

Allowable shear stress per ANSI/AISC N690-1994 section Q1.5.1.2.1 (Ref. 2.2.6)

$F_v := 0.4 \cdot F_y$ $F_v = 20\,ksi$ allowable shear stress

$f_v := \frac{V_{b3}}{d \cdot t_w}$ $f_v = 5.8\,ksi$ calculated shear stress

$DC_f := \frac{M_{b3}}{M_r}$ $DC_f = 0.84$ flexure demand/capacity ratio

$DC_v := \frac{f_v}{F_v}$ $DC_v = 0.29$ shear demand/capacity ratio

$\Delta_{B3} := \frac{5 \cdot (S1_d \cdot trib_{b3}) \cdot L3^4}{384 \cdot E_s \cdot I}$ $\Delta_{B3} = \blacksquare\,in$ max (D + L_r) beam deflection

$\Delta_{allow} := \frac{L3}{240}$ $\Delta_{allow} = 1.125\,in$ allowable deflection (Ref. 2.2.4) $\frac{\Delta_{B3}}{\Delta_{allow}} = 0.34$ deflection demand/capacity ratio

$shear_check := if(f_v \le F_v, "OK", "N.G!")$ shear_check = "OK"

$moment_check := if(M_{b3} \le M_r, "OK", "N.G!")$ moment_check = "OK"

$deflection_check := if(\Delta_{B3} \le \Delta_{allow}, "OK", "N.G!")$ deflection_check = "OK"

$depth_check1 := if\left(d \ge \frac{F_y \cdot L3}{1000 \cdot ksi}, "OK", "N.G!"\right)$ ANSI/AISC N690-1994 section Q1.13, CQ1.13 (Ref. 2.2.6) depth_check1 = "OK"

$depth_check2 := if\left(d \ge \frac{L3}{20}, "OK", "N.G!"\right)$ ANSI/AISC N690-1994 section Q1.13, CQ1.13 (Ref. 2.2.6) depth_check2 = "OK"

ROOF BEAM B3 USE: B3 = "W24X55"

ROOF BEAM B4
ROOF ELEV. 72'-0" - Steel Framing Design between Gridline 9 and 12, from E to G.
Nominal concrete slab thickness = 1'-6" (Attachment A-3)

BEAM PROPERTIES per AISC 1997, Ref. 2.2.5

$B4 := "W18X40"$ $M_r := 188k \cdot ft$ $d := 17.90in$ $t_w := 0.315in$ $I := 612in^4$ $t_f := 0.525in$

$L4 := 17.75ft$ $trib_{b4} := 6.5ft$ Max. span and spacing

$\omega_{b4} := BMax_{rf} \cdot trib_{b4}$ $\omega_{b4} = 4.02\,klf$ Uniform beam loading for flexure

$\omega_{b4v} := VMax_{rf} \cdot trib_{b4}$ $\omega_{b4v} = 4.6\,klf$ Uniform beam loading for shear

$\omega_{b4_const} := S19 \cdot trib_{b4}$ $\omega_{b4_const} = 2.4\,klf$ Uniform beam construction loading

$M_{b4} := 0.125 \omega_{b4} \cdot L4^2$ $M_{b4} = 158.4\,k \cdot ft$ beam moment

$M_{b4_const} := 0.125(\omega_{b4_const}) \cdot L4^2 + 0.25 C_p \cdot L4$ $M_{b4_const} = 116.6\,k \cdot ft$ beam moment w/ construction loading

$V_{b4} := \frac{\omega_{b4v} \cdot L4}{2}$ $V_{b4} = 40.8\,kip$ beam shear

$V_{b4_const} := \frac{\omega_{b4_const} \cdot L4}{2} + C_p$ $V_{b4_const} = 26.3\,kip$ beam shear w/ construction loading

$M_{b4} := \max(M_{b4}, M_{b4_const})$ $M_{b4} = 158.4\,k \cdot ft$ max beam moment

$V_{b4} := \max(V_{b4}, V_{b4_const})$ $V_{b4} = 40.8\,kip$ max beam shear

Allowable shear stress per ANSI/AISC N690-1994 section Q1.5.1.2.1 (Ref. 2.2.6)

$F_v := 0.4 \cdot F_y$ $F_v = 20\,ksi$ allowable shear stress

$f_v := \frac{V_{b4}}{d \cdot t_w}$ $f_v = 7.2\,ksi$ calculated shear stress

$DC_f := \frac{M_{b4}}{M_r}$ $DC_f = 0.84$ flexure demand/capacity ratio

$DC_v := \frac{f_v}{F_v}$ $DC_v = 0.36$ shear demand/capacity ratio

$\Delta_{B4} := \frac{5 \cdot (S1_a \cdot trib_{b4}) \cdot L4^4}{384 \cdot E_s \cdot I}$ $\Delta_{B4} = 0.322\,in$ max (D + L_r) beam deflection

$\Delta_{allow} := \frac{L4}{240}$ $\Delta_{allow} = 0.888\,in$ allowable deflection (Ref. 2.2.4) $\frac{\Delta_{B4}}{\Delta_{allow}} = 0.36$ deflection demand/capacity ratio

$shear_check := if(f_v \le F_v, "OK", "N.G!")$ shear_check = "OK"

$moment_check := if(M_{b4} \le M_r, "OK", "N.G!")$ moment_check = "OK"

$deflection_check := if(\Delta_{B4} \le \Delta_{allow}, "OK", "N.G!")$ deflection_check = "OK"

$depth_check1 := if\left(d \ge \frac{F_y \cdot L4}{1000 \cdot ksi}, "OK", "N.G!"\right)$ ANSI/AISC N690-1994 section Q1.13, CQ1.13 (Ref. 2.2.6) depth_check1 = "OK"

$depth_check2 := if\left(d \ge \frac{L4}{20}, "OK", "N.G!"\right)$ ANSI/AISC N690-1994 section Q1.13, CQ1.13 (Ref. 2.2.6) depth_check2 = "OK"

ROOF BEAM B4 USE: B4 = "W18X40"

ROOF BEAM B5
ROOF ELEV. 64'-0" - Steel Framing Design between Gridline 2 and 4, from E to G.
Nominal concrete slab thickness = 2'-9" (Attachment A-2)

BEAM PROPERTIES per AISC 1997 (Ref. 2.2.5)

$B5 := \text{"W12X30"}$ $M_r := 106k \cdot ft$ $d := 12.34in$ $t_w := 0.26in$ $I := 238in^4$ $t_f := 0.44in$

$L5 := 12ft$	$trib_{b5} := 5.3ft$	Max. span and spacing
$\omega_{b5} := BMax_{rf3} \cdot trib_{b5}$	$\omega_{b5} = 4.92\,klf$	Uniform beam loading for flexure
$\omega_{b5v} := VMax_{rf3} \cdot trib_{b5}$	$\omega_{b5v} = 5.6\,klf$	Uniform beam loading for shear
$\omega_{b5_const} := S20 \cdot trib_{b5}$	$\omega_{b5_const} = 2.9\,klf$	Uniform beam construction loading
$M_{b5} := 0.125\omega_{b5} \cdot L5^2$	$M_{b5} = 88.5\,k \cdot ft$	beam moment
$M_{b5_const} := 0.125(\omega_{b5_const}) \cdot L5^2 + 0.25C_p \cdot L5$	$M_{b5_const} = 68.1\,k \cdot ft$	beam moment w/ construction loading
$V_{b5} := \frac{\omega_{b5v} \cdot L5}{2}$	$V_{b5} = 33.7\,kip$	beam shear
$V_{b5_const} := \frac{\omega_{b5_const} \cdot L5}{2} + C_p$	$V_{b5_const} = 22.7\,kip$	beam shear w/ construction loading
$M_{b5} := \max(M_{b5}, M_{b5_const})$	$M_{b5} = 88.5\,k \cdot ft$	max beam moment
$V_{b5} := \max(V_{b5}, V_{b5_const})$	$V_{b5} = 33.7\,kip$	max beam shear

Allowable shear stress per ANSI/AISC N690-1994 section Q1.5.1.2.1 (Ref. 2.2.6)

$F_v := 0.4F_y$	$F_v = 20\,ksi$	allowable shear stress
$f_v := \frac{V_{b5}}{d \cdot t_w}$	$f_v = 10.5\,ksi$	calculated shear stress
$DC_f := \frac{M_{b5}}{M_r}$	$DC_f = 0.84$	flexure demand/capacity ratio
$DC_v := \frac{f_v}{F_v}$	$DC_v = 0.53$	shear demand/capacity ratio
$\Delta_{B5} := \frac{5 \cdot (S1_d \cdot trib_{b5}) \cdot L5^4}{384 \cdot E_s \cdot I}$	$\Delta_{B5} = 0.208\,in$	max (D + L_r) beam deflection
$\Delta_{allow} := \frac{L5}{240}$ $\Delta_{allow} = \blacksquare\,in$ allowable deflection (Ref. 2.2.4)	$\frac{\Delta_{B5}}{\Delta_{allow}} = 0.35$	deflection demand/capacity ratio

$shear_check := if(f_v \le F_v, \text{"OK"}, \text{"N.G!"})$ shear_check = "OK"

$moment_check := if(M_{b5} \le M_r, \text{"OK"}, \text{"N.G!"})$ moment_check = "OK"

$deflection_check := if(\Delta_{B5} \le \Delta_{allow}, \text{"OK"}, \text{"N.G!"})$ deflection_check = "OK"

$depth_check1 := if\left(d \geq \frac{F_y \cdot L5}{1000ksi}, "OK", "N.G!"\right)$ ANSI/AISC N690-1994 section Q1.13, CQ1.13 (Ref. 2.2.6) $depth_check1 = "OK"$

$depth_check2 := if\left(d \geq \frac{L5}{20}, "OK", "N.G!"\right)$ ANSI/AISC N690-1994 section Q1.13, CQ1.13 (Ref. 2.2.6) $depth_check2 = "OK"$

ROOF BEAM B5 USE: $B5 = "W12X30"$

ROOF BEAM B6
ROOF ELEV. 100'-0" - Steel Framing Design between Gridline 6 and 9, from D to H.
Nominal concrete slab thickness = 1'-6" (Attachment A-3)

BEAM PROPERTIES per AISC ASD 1997 (Ref. 2.2.5)

$B6 := "W18X55"$ $M_r := 270k \cdot ft$ $d := 18.11in$ $t_w := 0.390in$ $I := 890in^4$ $t_f := 0.630in$

$L6 := 21.167ft$	$trib_{b6} := 6.43ft$	Max. span and spacing
$\omega_{b6} := BMax_{rf} \cdot trib_{b6}$	$\omega_{b6} = 3.98\,klf$	Uniform beam loading for flexure
$\omega_{b6v} := VMax_{rf} \cdot trib_{b6}$	$\omega_{b6v} = 4.5\,klf$	Uniform beam loading for shear
$\omega_{b6_const} := S19 \cdot trib_{b6}$	$\omega_{b6_const} = 2.4\,klf$	Uniform beam construction loading
$M_{b6} := 0.125 \cdot \omega_{b6} \cdot L6^2$	$M_{b6} = 222.8\,k \cdot ft$	beam moment
$M_{b6_const} := 0.125\left(\omega_{b6_const}\right) \cdot L6^2 + 0.25C_p \cdot L6$	$M_{b6_const} = 159.3\,k \cdot ft$	beam moment w/ construction loading
$V_{b6} := \frac{\omega_{b6v} \cdot L6}{2}$	$V_{b6} = 48.1\,kip$	beam shear
$V_{b6_const} := \frac{\omega_{b6_const} \cdot L6}{2} + C_p$	$V_{b6_const} = 30.1\,kip$	beam shear w/ construction loading
$M_{b6} := \max\left(M_{b6}, M_{b6_const}\right)$	$M_{b6} = 222.8\,k \cdot ft$	max beam moment
$V_{b6} := \max\left(V_{b6}, V_{b6_const}\right)$	$V_{b6} = 48.1\,kip$	max beam shear

Allowable shear stress per ANSI/AISC N690-1994 section Q1.5.1.2.1 (Ref. 2.2.6)

$F_v := 0.4 \cdot F_y$	$F_v = 20\,ksi$	allowable shear stress
$f_v := \frac{V_{b6}}{d \cdot t_w}$	$f_v = 6.8\,ksi$	calculated shear stress
$DC_f := \frac{M_{b6}}{M_r}$	$DC_f = 0.83$	flexure demand/capacity ratio
$DC_v := \frac{f_v}{F_v}$	$DC_v = 0.34$	shear demand/capacity ratio
$\Delta_{B6} := \frac{5 \cdot \left(S1_a \cdot trib_{b6}\right) \cdot L6^4}{384 \cdot E_s \cdot I}$	$\Delta_{B6} = 0.443\,in$	max (D + L_r) beam deflection
$\Delta_{allow} := \frac{L6}{240}$ $\Delta_{allow} = 1.058\,in$ allowable deflection (Ref. 2.2.4)	$\frac{\Delta_{B6}}{\Delta_{allow}} = ▮$	deflection demand/capacity ratio

$shear_check := if(f_v \le F_v, "OK", "N.G!")$ shear_check = "OK"

$moment_check := if(M_{b6} \le M_r, "OK", "N.G!")$ moment_check = "OK"

$deflection_check := if(\Delta_{B6} \le \Delta_{allow}, "OK", "N.G!")$ deflection_check = "OK"

$depth_check1 := if\left(d \ge \frac{F_y \cdot L6}{1000 \cdot ksi}, "OK", "N.G!"\right)$ ANSI/AISC N690-1994 section Q1.13, CQ1.13 (Ref. 2.2.6) depth_check1 = "OK"

$depth_check2 := if\left(d \ge \frac{L6}{20}, "OK", "N.G!"\right)$ ANSI/AISC N690-1994 section Q1.13, CQ1.13 (Ref. 2.2.6) depth_check2 = "OK"

ROOF BEAM B6 USE: B6 = "W18X55"

ROOF BEAM B7

ROOF ELEV. 36'-0" - Steel Framing Design between Gridline 12 and 13, from E to G at Entrance Vestibule.

Nominal concrete slab thickness = 1'-6" (Attachment A-1). Beam B7 envelopes roof beam at Exit Vestibule between Gridline 1 and 2, from E to G. Therefore, beam will be used at Exit Vestibule as well.

BEAM PROPERTIES per AISC ASD 1997 (Ref. 2.2.5)

$B7 := "W21X44"$ $M_r := 224k \cdot ft$ $d := 20.66in$ $t_w := 0.35in$ $I := 843in^4$ $t_f := 0.45in$

$L7 := 18.80ft$	$trib_{b7} := 6.67ft$	Max. span and spacing
$\omega_{b7} := BMax_{rf} \cdot trib_{b7}$	$\omega_{b7} = 4.13\,klf$	Uniform beam loading for flexure
$\omega_{b7v} := VMax_{rf} \cdot trib_{b7}$	$\omega_{b7v} = 4.7\,klf$	Uniform beam loading for shear
$\omega_{b7_const} := S19 \cdot trib_{b7}$	$\omega_{b7_const} = 2.5\,klf$	Uniform beam construction loading
$M_{b7} := 0.125 \cdot \omega_{b7} \cdot L7^2$	$M_{b7} = 182.3\,k \cdot ft$	beam moment
$M_{b7_const} := 0.125(\omega_{b7_const}) \cdot L7^2 + 0.25C_p \cdot L6$	$M_{b7_const} = 135.1\,k \cdot ft$	beam moment w/ construction loading
$V_{b7} := \frac{\omega_{b7v} \cdot L7}{2}$	$V_{b7} = 44.3\,kip$	beam shear
$V_{b7_const} := \frac{\omega_{b7_const} \cdot L7}{2} + C_p$	$V_{b7_const} = 28.1\,kip$	beam shear w/ construction loading
$M_{b7} := \max(M_{b7}, M_{b7_const})$	$M_{b7} = 182.3\,k \cdot ft$	max beam moment
$V_{b7} := \max(V_{b7}, V_{b7_const})$	$V_{b7} = 44.3\,kip$	max beam shear

Allowable shear stress per ANSI/AISC N690-1994 section Q1.5.1.2.1 (Ref. 2.2.6)

$F_v := 0.4 \cdot F_y$	$F_v = 20\,ksi$	allowable shear stress
$f_v := \frac{V_{b7}}{d \cdot t_w}$	$f_v = 6.1\,ksi$	calculated shear stress
$DC_f := \frac{M_{b7}}{M_r}$	$DC_f = 0.81$	flexure demand/capacity ratio
$DC_v := \frac{f_v}{F_v}$	$DC_v = 0.31$	shear demand/capacity ratio

$\Delta_{B7} := \frac{5 \cdot (S1_a \cdot trib_{b7}) \cdot L7^4}{384 \cdot E_s \cdot I}$ $\Delta_{B7} = 0.302\,in$ max (D + L_r) beam deflection

$\Delta_{allow} := \frac{L7}{240}$ $\Delta_{allow} = 0.94\,in$ allowable deflection (Ref. 2.2.4) $\frac{\Delta_{B7}}{\Delta_{allow}} = 0.32$ deflection demand/capacity ratio

$shear_check := if(f_v \le F_v, "OK", "N.G!")$ shear_check = "OK"

$moment_check := if(M_{b7} \le M_r, "OK", "N.G!")$ moment_check = "OK"

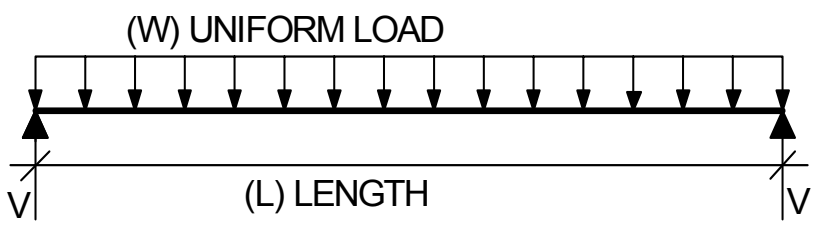
$deflection_check := if(\Delta_{B7} \le \Delta_{allow}, "OK", "N.G!")$ deflection_check = "OK"

$depth_check1 := if\left(d \ge \frac{F_y \cdot L7}{1000 \cdot ksi}, "OK", "N.G!"\right)$ ANSI/AISC N690-1994 section Q1.13, CQ1.13 (Ref. 2.2.6) depth_check1 = "OK"

$depth_check2 := if\left(d \ge \frac{L7}{20}, "OK", "N.G!"\right)$ ANSI/AISC N690-1994 section Q1.13, CQ1.13 (Ref. 2.2.6) depth_check2 = "OK"

ROOF BEAM B7 USE: B7 = "W21X44"

6.3.2 FLOOR BEAM CALCULATION, Ref. 2.2.5, Ref. 2.2.6



Typical Beam loading diagram, Part 2, Beam Diagrams and Formulas, AISC 1997 (Ref. 2.2.5)

FLOOR BEAM B8
FLOOR ELEV. 32'-0" - Steel Framing Design between Gridline 2 to 5 and 6 to 12, from D to E and G to H, and Gridline 9 to 10, from E to G. Nominal concrete slab thickness = 1'-6" (Attachment A-1)

BEAM PROPERTIES per AISC 1997 (Ref. 2.2.5)

$B8 := "W18X35"$ $M_r := 158k \cdot ft$ $d := 17.70in$ $t_w := 0.30in$ $I := 510in^4$ $t_f := 0.425in$

$L8 := 14.5ft$ $trib_{b8} := 6.5ft$ Max. span and spacing

$\omega_{b8} := BMax_{fl} \cdot trib_{b8}$ $\omega_{b8} = 4.06\,klf$ Uniform beam loading for flexure

$\omega_{b8v} := VMax_{fl} \cdot trib_{b8}$ $\omega_{b8v} = 4.6\,klf$ Uniform beam loading for shear

$\omega_{b8_const} := S19 \cdot trib_{b8}$ $\omega_{b8_const} = 2.4\,klf$ Uniform beam construction loading

$M_{b8} := 0.125 \omega_{b8} \cdot L8^2$ $M_{b8} = 106.8\,k \cdot ft$ beam moment

$M_{b8_const} := 0.125(\omega_{b8_const}) \cdot L8^2 + 0.25 C_p \cdot L8$ $M_{b8_const} = 81.1\,k \cdot ft$ beam moment w/ construction loading

$V_{b8} := \frac{\omega_{b8v} \cdot L8}{2}$ $V_{b8} = 33.7\,kip$ beam shear

$V_{b8_const} := \frac{\omega_{b8_const} \cdot L8}{2} + C_p$ $V_{b8_const} = 22.4\,kip$ beam shear w/ construction loading

$M_{b8} := \max(M_{b8}, M_{b8_const})$ $M_{b8} = 106.8\,k \cdot ft$ max beam moment

$V_{b8} := \max(V_{b8}, V_{b8_const})$ $V_{b8} = 33.7\,kip$ max beam shear

Allowable shear stress per ANSI/AISC N690-1994 section Q1.5.1.2.1 (Ref. 2.2.6)

$F_v := 0.4 \cdot F_y$ $F_v = 20\,ksi$ allowable shear stress

$f_v := \frac{V_{b8}}{d \cdot t_w}$ $f_v = 6.3\,ksi$ calculated shear stress

$DC_f := \frac{M_{b8}}{M_r}$ $DC_f = 0.68$ flexure demand/capacity ratio

$DC_v := \frac{f_v}{F_v}$ $DC_v = 0.32$ shear demand/capacity ratio

$\Delta_{B8} := \frac{5 \cdot (S1_c \cdot trib_{b8}) \cdot L8^4}{384 \cdot E_s \cdot I}$ $\Delta_{B8} = 0.198\,in$ max (D + L) beam deflection

$\Delta_{allow} := \frac{L8}{240}$ $\Delta_{allow} = 0.725\,in$ allowable deflection (Ref. 2.2.4) $\frac{\Delta_{B8}}{\Delta_{allow}} = 0.27$ deflection demand/capacity ratio

$shear_check := if(f_v \le F_v, "OK", "N.G!")$ shear_check = ▮

$moment_check := if(M_{b8} \le M_r, "OK", "N.G!")$ moment_check = "OK"

$deflection_check := if(\Delta_{B8} \le \Delta_{allow}, "OK", "N.G!")$ deflection_check = "OK"

$depth_check1 := if\left(d \ge \frac{F_y L8}{800ksi}, "OK", "N.G!"\right)$ ANSI/AISC N690-1994 section Q1.13, CQ1.13 (Ref. 2.2.6) depth_check1 = "OK"

$depth_check2 := if\left(d \ge \frac{L8}{20}, "OK", "N.G!"\right)$ ANSI/AISC N690-1994 section Q1.13, CQ1.13 (Ref. 2.2.6) depth_check2 = "OK"

FLOOR BEAM B8 USE: B8 = "W18X35"

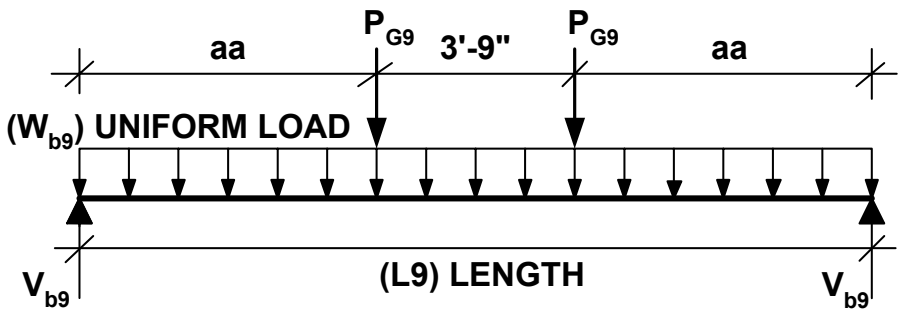
FLOOR BEAM B9 with Forklift loading
FLOOR ELEV. 32'-0" - Steel Framing Design between Gridline 5 and 6, from D to E and G to H.
Nominal concrete slab thickness = 1'-6" (Attachment A-1)

BEAM PROPERTIES per AISC 1997 (Ref. 2.2.5)

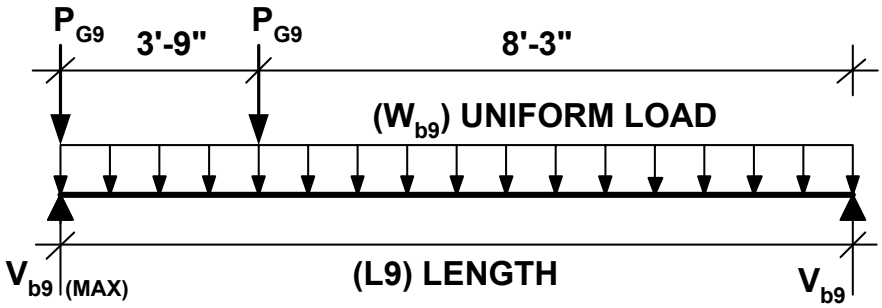
$B9 := \text{"W18X40"}$ $M_r := 188k\cdot ft$ $d := 17.90in$ $t_w := 0.315in$ $I := 612in^4$ $t_f := 0.525in$

Beam loading diagram, Part 2, Beam Diagrams and Formulas, AISC 1997 (Ref. 2.2.5)

P_{G9} reactions from forklift wheel load (Assumption 3.1.2)



MAX FLEXURE LOADING



MAX SHEAR LOADING

$L9 := 12.0ft$	$trib_{b9} := 6.5ft$	Max. span and spacing
$\omega_{b9} := BMax_{fl}\cdot trib_{b9}$	$\omega_{b9} = 4.06\,klf$	Uniform beam loading for flexure
$\omega_{b9v} := VMax_{fl}\cdot trib_{b9}$	$\omega_{b9v} = 4.6\,klf$	Uniform beam loading for shear
$\omega_{b9_const} := S19\cdot trib_{b9}$	$\omega_{b9_const} = 2.4\,klf$	Uniform beam construction loading
$P_{G9} := L_p$	$P_{G9} = 16\,kip$	Concentrated beam load from forklift wheel load spaced at 3'-9" o.c. (Assumption 3.1.2)
$aa := \frac{L9 - 3.75ft}{2}$	$aa = 4.1\ ft$	length from beam end to forklift wheel load
$M_{b9} := 0.125\omega_{b9}\cdot L9^2 + P_{G9}\cdot aa$	$M_{b9} = 139.1\,k\cdot ft$	beam moment
$M_{b9_const} := 0.125(\omega_{b9_const})\cdot L9^2 + 0.25C_p\cdot L9$	$M_{b9_const} = 58.1\,k\cdot ft$	beam moment w/ construction loading
$V_{b9} := \frac{\omega_{b9v}\cdot L9}{2} + P_{G9} + \frac{P_{G9}\cdot(L9 - 3.75ft)}{L9}$	$V_{b9} = 54.9\,kip$	beam shear

$V_{b9_const} := \frac{\omega_{b9_const} \cdot L9}{2} + C_p$ $V_{b9_const} = 19.4\,kip$ beam shear w/ construction loading

$M_{b9} := \max(M_{b9}, M_{b9_const})$ $M_{b9} = 139.1\,k \cdot ft$ max beam moment

$V_{b9} := \max(V_{b9}, V_{b9_const})$ $V_{b9} = 54.9\,kip$ max beam shear

Allowable shear stress per ANSI/AISC N690-1994 section Q1.5.1.2.1 (Ref. 2.2.6)

$F_v := 0.4F_y$ $F_v = 20\,ksi$ allowable shear stress

$f_v := \frac{V_{b9}}{d \cdot t_w}$ $f_v = 9.7\,ksi$ calculated shear stress

$DC_f := \frac{M_{b9}}{M_r}$ $DC_f = 0.74$ flexure demand/capacity ratio

$DC_v := \frac{f_v}{F_v}$ $DC_v = 0.49$ shear demand/capacity ratio

$\Delta_{B9} := \left[\frac{5 \cdot (S1_c \cdot trib_{b9}) \cdot L9^4}{384 \cdot E_s \cdot I}\right] + \left[\frac{P_{G9} \cdot aa}{24 \cdot E_s \cdot I} \cdot (3 \cdot L9^2 - 4 \cdot aa^2)\right]$ $\Delta_{B9} = 0.175\,in$ max (D + L) beam deflection, Case 9. (Ref. 2.2.5)

$\Delta_{allow} := \frac{L9}{240}$ $\Delta_{allow} = \blacksquare\,in$ allowable deflection (Ref. 2.2.4) $\frac{\Delta_{B9}}{\Delta_{allow}} = 0.29$ deflection demand/capacity ratio

$shear_check := if(f_v \le F_v, "OK", "N.G!")$ shear_check = "OK"

$moment_check := if(M_{b9} \le M_r, "OK", "N.G!")$ moment_check = "OK"

$deflection_check := if(\Delta_{B9} \le \Delta_{allow}, "OK", "N.G!")$ deflection_check = "OK"

$depth_check1 := if\left(d \ge \frac{F_y \cdot L9}{800ksi}, "OK", "N.G!"\right)$ ANSI/AISC N690-1994 section Q1.13, CQ1.13 (Ref. 2.2.6) depth_check1 = "OK"

$depth_check2 := if\left(d \ge \frac{L9}{20}, "OK", "N.G!"\right)$ ANSI/AISC N690-1994 section Q1.13, CQ1.13 (Ref. 2.2.6) depth_check2 = "OK"

FLOOR BEAM B9 USE: B9 = "W18X40"

FLOOR BEAM B10
FLOOR ELEV. 32'-0" - Steel Framing Design between Gridline 9 to 10, from E to G. Nominal concrete slab thickness = 1'-6" (Attachment A-1)

BEAM PROPERTIES per AISC 1997 (Ref. 2.2.5)

$B10 := "W10X39"$ $M_r := 116k \cdot ft$ $d := 9.92in$ $t_w := 0.315in$ $I := 209in^4$ $t_f := 0.530in$

$L10 := 13.0ft$ $trib_{b10} := 6.4ft$ Max. span and spacing

$\omega_{b10} := BMax_{fl} \cdot trib_{b10}$ $\omega_{b10} = 4\,klf$ Uniform beam loading for flexure

$\omega_{b10v} := VMax_{fl} \cdot trib_{b10}$ $\omega_{b10v} = 4.6\,klf$ Uniform beam loading for shear

$\omega_{b10_const} := S19 \cdot trib_{b10}$ $\omega_{b10_const} = 2.4\,klf$ Uniform beam construction loading

$M_{b10} := 0.125\omega_{b10} \cdot L10^2$ $M_{b10} = 84.5\,k \cdot ft$ beam moment

$M_{b10_const} := 0.125(\omega_{b10_const}) \cdot L10^2 + 0.25C_p \cdot L10$ $M_{b10_const} = 66.1\,k \cdot ft$ beam moment w/ construction loading

$V_{b10} := \frac{\omega_{b10v} \cdot L10}{2}$ $V_{b10} = 29.7\,kip$ beam shear

$V_{b10_const} := \frac{\omega_{b10_const} \cdot L10}{2} + C_p$ $V_{b10_const} = 20.3\,kip$ beam shear w/ construction loading

$M_{b10} := \max(M_{b10}, M_{b10_const})$ $M_{b10} = 84.5\,k \cdot ft$ max beam moment

$V_{b10} := \max(V_{b10}, V_{b10_const})$ $V_{b10} = 29.7\,kip$ max beam shear

Allowable shear stress per ANSI/AISC N690-1994 section Q1.5.1.2.1 (Ref. 2.2.6)

$F_v := 0.4 \cdot F_y$ $F_v = 20\,ksi$ allowable shear stress

$f_v := \frac{V_{b10}}{d \cdot t_w}$ $f_v = 9.5\,ksi$ calculated shear stress

$DC_f := \frac{M_{b10}}{M_r}$ $DC_f = 0.73$ flexure demand/capacity ratio

$DC_v := \frac{f_v}{F_v}$ $DC_v = 0.48$ shear demand/capacity ratio

$\Delta_{B10} := \frac{5 \cdot (S1_c \cdot trib_{b10}) \cdot L10^4}{384 \cdot E_s \cdot I}$ $\Delta_{B10} = 0.308\,in$ max (D + L) beam deflection

$\Delta_{allow} := \frac{L10}{240}$ $\Delta_{allow} = 0.65\,in$ allowable deflection (Ref. 2.2.4) $\frac{\Delta_{B10}}{\Delta_{allow}} = \blacksquare$ deflection demand/capacity ratio

$shear_check := if(f_v \le F_v, "OK", "N.G!")$ shear_check = "OK"

$moment_check := if(M_{b10} \le M_r, "OK", "N.G!")$ moment_check = "OK"

$deflection_check := if(\Delta_{B10} \le \Delta_{allow}, "OK", "N.G!")$ deflection_check = "OK"

$depth_check1 := if\left(d \ge \frac{F_y L10}{800ksi}, "OK", "N.G!"\right)$ ANSI/AISC N690-1994 section Q1.13, CQ1.13 (Ref. 2.2.6) depth_check1 = "OK"

$depth_check2 := if\left(d \ge \frac{L10}{20}, "OK", "N.G!"\right)$ ANSI/AISC N690-1994 section Q1.13, CQ1.13 (Ref. 2.2.6) depth_check2 = "OK"

FLOOR BEAM B10 USE: B10 = "W10X39"

FLOOR BEAM B11
FLOOR ELEV. 32'-0" - Steel Framing Design around opening between Gridline 8 to 9, from D to E and G to H.
Nominal concrete slab thickness = 1'-6" (Attachment A-1)

BEAM PROPERTIES per AISC 1997 (Ref. 2.2.5)

$B11 := \text{"W18X35"}$ $M_r := 158k\cdot ft$ $d := 17.70in$ $t_w := 0.30in$ $I := 510in^4$ $t_f := 0.425in$

$L11 := 7.67ft$	$trib_{b11} := 6.5ft$	Max. span and spacing
$\omega_{b11} := BMax_{fl}\cdot trib_{b11}$	$\omega_{b11} = 4.1\,klf$	Uniform beam loading for flexure
$\omega_{b11v} := VMax_{fl}\cdot trib_{b11}$	$\omega_{b11v} = 4.6\,klf$	Uniform beam loading for shear
$\omega_{b11_const} := S19\cdot trib_{b11}$	$\omega_{b11_const} = 2.4\,klf$	Uniform beam construction loading
$M_{b11} := 0.125\omega_{b11}\cdot L11^2$	$M_{b11} = 29.9\,k\cdot ft$	beam moment
$M_{b11_const} := 0.125(\omega_{b11_const})\cdot L11^2 + 0.25C_p\cdot L11$	$M_{b11_const} = 27.2\,k\cdot ft$	beam moment w/ construction loading
$V_{b11} := \frac{\omega_{b11}\cdot L11}{2}$	$V_{b11} = 15.6\,kip$	beam reaction for girder loading see girder G13
$V_{b11v} := \frac{\omega_{b11v}\cdot L11}{2}$	$V_{b11v} = 17.8\,kip$	beam shear
$V_{b11_const} := \frac{\omega_{b11_const}\cdot L11}{2} + C_p$	$V_{b11_const} = 14.2\,kip$	beam shear w/ construction loading
$M_{b11} := \max(M_{b11}, M_{b11_const})$	$M_{b11} = 29.9\,k\cdot ft$	max beam moment
$V_{b11} := \max(V_{b11v}, V_{b11_const})$	$V_{b11} = 17.8\,kip$	max beam shear

Allowable shear stress per ANSI/AISC N690-1994 section Q1.5.1.2.1 (Ref. 2.2.6)

$F_v := 0.4\cdot F_y$	$F_v = 20\,ksi$	allowable shear stress
$f_v := \frac{V_{b11}}{d\cdot t_w}$	$f_v = 3.4\,ksi$	calculated shear stress
$DC_f := \frac{M_{b11}}{M_r}$	$DC_f = 0.19$	flexure demand/capacity ratio
$DC_v := \frac{f_v}{F_v}$	$DC_v = 0.17$	shear demand/capacity ratio
$\Delta_{B11} := \frac{5\cdot(S1_c\cdot trib_{b11})\cdot L11^4}{384\cdot E_s\cdot I}$	$\Delta_{B11} = 0.016\,in$	max (D + L) beam deflection

$\Delta_{allow} := \frac{L11}{240}$ $\Delta_{allow} = 0.384\,in$ allowable deflection (Ref. 2.2.4) $\frac{\Delta_{B11}}{\Delta_{allow}} = 0.04$ deflection demand/capacity ratio

$shear_check := if(f_v \le F_v, "OK", "N.G!")$ shear_check = "OK"

$moment_check := if(M_{b11} \le M_r, "OK", "N.G!")$ moment_check = "OK"

$deflection_check := if(\Delta_{B11} \le \Delta_{allow}, "OK", "N.G!")$ deflection_check = "OK"

$depth_check1 := if\left(d \ge \frac{F_y L11}{800ksi}, "OK", "N.G!"\right)$ ANSI/AISC N690-1994 section Q1.13, CQ1.13 (Ref. 2.2.6) depth_check1 = "OK"

$depth_check2 := if\left(d \ge \frac{L11}{20}, "OK", "N.G!"\right)$ ANSI/AISC N690-1994 section Q1.13, CQ1.13 (Ref. 2.2.6) depth_check2 = "OK"

FLOOR BEAM B11 USE: B11 = "W18X35"

6.4 GIRDER CALCULATIONS (GIRDER SIZES SHOWN ON PAGE 67)

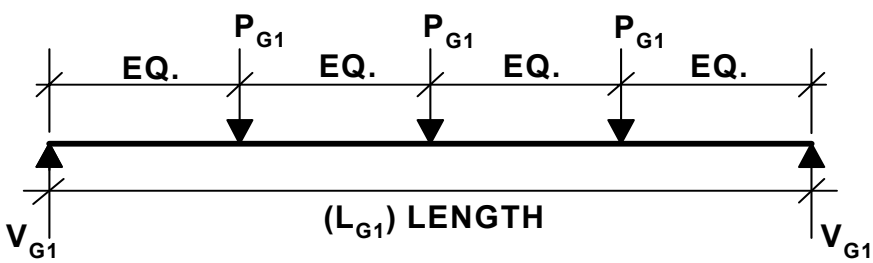
6.4.1 ROOF GIRDER CALCULATION, Ref. 2.2.5, Ref. 2.2.6

ROOF GIRDER G1

ROOF ELEV. 64'-0" - Girder Design between Gridline 2 to 5, and 9 to 12, from D to E and G to H.
Nominal concrete slab thickness = 1'-6" (Attachment A-2)

BEAM PROPERTIES per AISC 1997, Ref. 2.2.5

$G1 := "W30X116"$ $M_r := 905k \cdot ft$ $d := 30.01in$ $t_w := 0.565in$ $I := 4930in^4$ $t_f := 0.850in$



Girder loading diagram, Part 2, Beam Diagrams and Formulas, AISC 1997 (Ref. 2.2.5)

P_{G1} girder reactions from B1 (each side of girder)

$L_{G1} := 26ft$ Girder span

$P_{G1} := 2 \cdot \omega_{b1} \cdot \frac{L1}{2}$ $P_{G1} = 58.3\,kip$ Concentrated beam load for bending (See beam B1 design)

$P_{G1v} := 2\omega_{b1v} \cdot \frac{L1}{2}$ $P_{G1v} = 66.6\,kip$ Concentrated beam load for shear

$P_{G1const} := 2 \cdot \left(\omega_{b1_const} \cdot \frac{L1}{2}\right)$ $P_{G1const} = 34.8\,kip$ Concentrated beam construction load

$a_{G1} := 0.5$
$c_{G1} := 1.5$
$e_{G1} := 0.0495$

Coefficients for 3 equally spaced concentrated loads on beam, AISC (2-295) Beam Diagrams and Formulas; Table of concentrated load equivalents, Case 4 (Ref. 2.2.5.)

$M_{g1} := a_{G1} \cdot P_{G1} \cdot L_{G1}$ $M_{g1} = 758.1\,k \cdot ft$ Girder moment

$M_{g1const} := \left(a_{G1} \cdot P_{G1const} \cdot L_{G1}\right) + \frac{C_p \cdot L_{G1}}{4}$ $M_{g1const} = 484.3\,k \cdot ft$ Girder moment w/ construction loading

$V_{g1} := c_{G1} \cdot P_{G1v}$ $V_{g1} = 100\,kip$ Girder shear

$V_{g1const} := c_{G1} \cdot P_{G1const} + C_p$ $V_{g1const} = 57.1\,kip$ Girder shear w/ construction loading

$M_{G1} := \max\left(M_{g1}, M_{g1const}\right)$ $M_{G1} = 758.1\,k \cdot ft$ max girder beam moment

$V_{G1} := \max\left(V_{g1}, V_{g1const}\right)$ $V_{G1} = 100\,kip$ max girder beam shear

Allowable shear stress per ANSI/AISC N690-1994 section Q1.5.1.2.1, Ref. 2.2.6

$F_v := 0.4 \cdot F_y$ $F_v = 20\,ksi$ allowable shear stress

$f_v := \frac{V_{G1}}{d \cdot t_w}$ $f_v = 5.9\,ksi$ calculated shear stress

$DC_f := \frac{M_{G1}}{M_r}$ $DC_f = 0.84$ flexure demand/capacity ratio

$DC_v := \frac{f_v}{F_v}$ $DC_v = 0.29$ shear demand/capacity ratio

$\Delta_{G1} := \frac{e_{G1} \cdot \left(S1_a \cdot trib_{b1} \cdot L1\right) \cdot L_{G1}^3}{E_s \cdot I}$ $\Delta_{G1} = 0.39\,in$ max (D + L) girder deflection (Ref. 2.2.5)

$\Delta_{allow} := \frac{L_{G1}}{240}$ $\Delta_{allow} = 1.3\,in$ allowable girder deflection (Ref. 2.2.4) $\frac{\Delta_{G1}}{\Delta_{allow}} = 0.3$ deflection demand/capacity ratio

$shear_check := if\left(f_v \le F_v, "OK", "N.G!"\right)$ shear_check = "OK"

$moment_check := if\left(M_{G1} \le M_r, "OK", "N.G!"\right)$ moment_check = "OK"

$deflection_check := if\left(\Delta_{G1} \le \Delta_{allow}, "OK", "N.G!"\right)$ deflection_check = "OK"

$depth_check1 := if\left(d \ge \frac{F_y \cdot L_{G1}}{1000ksi}, "OK", "N.G!"\right)$ ANSI/AISC N690-1994 section Q1.13, CQ1.13 (Ref. 2.2.6) depth_check1 = "OK"

$depth_check2 := if\left(d \ge \frac{L_{G1}}{20}, "OK", "N.G!"\right)$ ANSI/AISC N690-1994 section Q1.13, CQ1.13 (Ref. 2.2.6) depth_check2 = "OK"

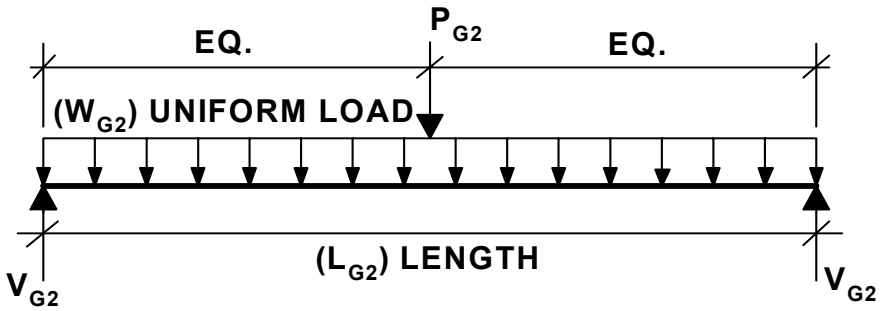
ROOF GIRDER G1 USE: G1 = "W30X116"

ROOF GIRDER G2

ROOF ELEV. 64'-0" - Girder Design between Gridline 2 to 5, from D to E and G to H.
Nominal concrete slab thickness = 1'-6" (Attachment A-2)

BEAM PROPERTIES per AISC 1997 (Ref. 2.2.5)

$G2 := \text{"W36x210"}$ $M_r := 1980k \cdot ft$ $d := 36.69in$ $t_w := 0.830in$ $I := 13200in^4$ $t_f := 1.360in$



Girder loading diagram, Part 2, Beam Diagrams and Formulas, AISC 1997 (Ref. 2.2.5)

P_{G2} reaction from girder G1 (each side of girder)

$L_{G2} := 29ft$	$trib_{G2} := 6.5ft$	Max girder span/ tributary loading
$\omega_{G2} := BMax_{rf} \cdot trib_{G2}$	$\omega_{G2} = 4.02\,klf$	Uniform girder loading for flexure
$\omega_{G2v} := VMax_{rf} \cdot trib_{G2}$	$\omega_{G2v} = 4.6\,klf$	Uniform girder loading for shear
$\omega_{G2const} := S19 \cdot trib_{G2}$	$\omega_{G2const} = 2.4\,klf$	Uniform girder construction loading
$P_{G2} := 2 \cdot c_{G1} \cdot P_{G1}$	$P_{G2} = 175\,kip$	Concentrated beam load for flexure (see girder G1 design)
$P_{G2v} := 2 \cdot c_{G1} \cdot P_{G1v}$	$P_{G2v} = 199.9\,kip$	Concentrated beam load for shear
$P_{G2const} := 2(c_{G1} \cdot P_{G1const})$	$P_{G2const} = 104.3\,kip$	Concentrated beam construction load
$a1_{G2} := 0.125$		Coefficients for uniform load on beam, AISC Beam Diagrams and Formulas; Table of concentrated load equivalents, Case oo (Ref. 2.2.5)
$c1_{G2} := 0.5$		
$e1_{G2} := 0.013$		
$a2_{G2} := 0.25$		Coefficients for single concentrated load on beam, AISC Beam Diagrams and Formulas; Table of concentrated load equivalents, Case 2 (Ref. 2.2.5)
$c2_{G2} := 0.5$		
$e2_{G2} := 0.0208$		

$M_{g2} := a1_{G2} \cdot (\omega_{G2} \cdot L_{G2}) \cdot L_{G2} + a2_{G2} \cdot P_{G2} \cdot L_{G2}$ $M_{g2} = 1691.2\,k \cdot ft$ Girder moment

$$M_{G2const} := a1_{G2} \cdot (\omega_{G2const} \cdot L_{G2}) \cdot L_{G2} + (a2_{G2} \cdot P_{G2const} \cdot L_{G2}) + \frac{C_p \cdot L_{G2}}{4}$$

$M_{G2const} = 1044.1\,k \cdot ft$ Girder moment w/ construction loading

$V_{g2} := c1_{G2} \cdot (\omega_{G2v} \cdot L_{G2}) + c2_{G2} \cdot P_{G2v}$ $V_{g2} = 166.6\,kip$ Girder shear

$V_{g2const} := c1_{G2} \cdot (\omega_{G2const} \cdot L_{G2}) + (c2_{G2} \cdot P_{G2const}) + C_p$

$V_{g2const} = 91.9\,kip$ Girder shear w/ construction loading

$M_{G2} := \max(M_{g2}, M_{G2const})$ $M_{G2} = 1691.2\,k\cdot ft$ max girder beam moment

$V_{G2} := \max(V_{g2}, V_{g2const})$ $V_{G2} = 166.6\,kip$ max girder beam shear

Allowable shear stress per ANSI/AISC N690-1994 section Q1.5.1.2.1 (Ref. 2.2.6)

$F_v = 20\,ksi$ allowable shear stress

$f_v := \frac{V_{G2}}{d\cdot t_w}$ $f_v = 5.5\,ksi$ calculated shear stress

$DC_f := \frac{M_{G2}}{M_r}$ $DC_f = 0.85$ flexure demand/capacity ratio

$DC_v := \frac{f_v}{F_v}$ $DC_v = 0.27$ shear demand/capacity ratio

$\Delta_{G2} := \left[\frac{e1_{G2}\cdot(S1_a\cdot trib_{G2}\cdot L_{G2})\cdot L_{G2}^3}{E_s\cdot I}\right] + \left[\frac{e2_{G2}\cdot(S1_a\cdot L1\cdot L_{G1})\cdot L_{G2}^3}{E_s\cdot I}\right]$ $\Delta_{G2} = 0.446\,in$ max (D + L) girder deflection

$\Delta_{allow} := \frac{L_{G2}}{240}$ $\Delta_{allow} = 1.45\,in$ allowable girder deflection (Ref. 2.2.4) $\frac{\Delta_{G2}}{\Delta_{allow}} = 0.31$ deflection demand/capacity ratio

$shear_check := if(f_v \le F_v, "OK", "N.G!")$ shear_check = "OK"

$moment_check := if(M_{G2} \le M_r, "OK", "N.G!")$ moment_check = "OK"

$deflection_check := if(\Delta_{G2} \le \Delta_{allow}, "OK", "N.G!")$ deflection_check = "OK"

$depth_check1 := if\left(d \ge \frac{F_y\cdot L_{G2}}{1000\cdot ksi}, "OK", "N.G!"\right)$ ANSI/AISC N690-1994 section Q1.13, CQ1.13 (Ref. 2.2.6) depth_check1 = "OK"

$depth_check2 := if\left(d \ge \frac{L_{G2}}{20}, "OK", "N.G!"\right)$ ANSI/AISC N690-1994 section Q1.13, CQ1.13 (Ref. 2.2.6) depth_check2 = "OK"

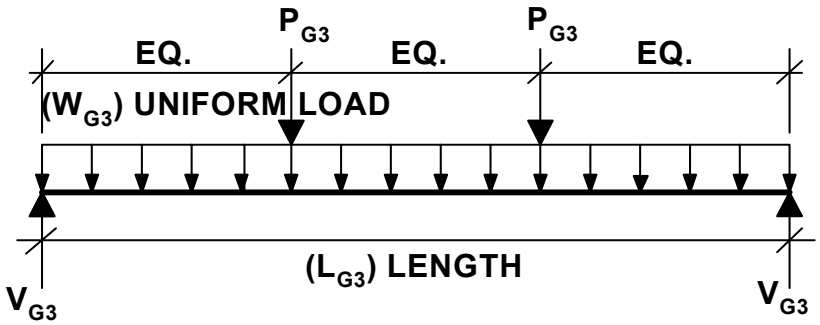
ROOF GIRDER G2 USE: G2 = "W36x210"

ROOF GIRDER G3

ROOF ELEV. 64'-0" - Girder Design between Gridline 3 to 5, and 10 to 12, from D to E and G to H.
Nominal concrete slab thickness = 1'-6" (Attachment A-2)

BEAM PROPERTIES per AISC 1997, Ref. 2.2.5

$G3 := \text{"W36x359"}$ $M_r := 3630k \cdot ft$ $d := 37.40in$ $t_w := 1.12in$ $I := 24800in^4$ $t_f := 2.01in$



Girder loading diagram, Part 2, Beam Diagrams and Formulas, AISC 1997 (Ref. 2.2.5)

P_{G3} reactions are from girder G1 (each side of girder)

$L_{G3} := 38ft$ $trib_{G3} := 6.5ft$ Max girder span/ tributary loading

$\omega_{G3} := BMax_{rf} \cdot trib_{G3}$ $\omega_{G3} = 4.02\,klf$ Uniform girder loading for flexure

$\omega_{G3v} := VMax_{rf} \cdot trib_{G3}$ $\omega_{G3v} = 4.6\,klf$ Uniform girder loading for shear

$\omega_{G3const} := S19 \cdot trib_{G3}$ $\omega_{G3const} = 2.4\,klf$ Uniform girder construction loading

$P_{G3} := 2 \cdot c_{G1} \cdot P_{G1}$ $P_{G3} = 175\,kip$ Concentrated beam load for flexure (see girder G1 design)

$P_{G3v} := 2 \cdot c_{G1} \cdot P_{G1v}$ $P_{G3v} = 199.9\,kip$ Concentrated beam load for shear

$P_{G3const} := 2\left(c_{G1} \cdot P_{G1const}\right)$ $P_{G3const} = 104.3\,kip$ Concentrated beam construction load

$a1_{G3} := 0.125$
$c1_{G3} := 0.5$
$e1_{G3} := 0.013$

Coefficients for uniform load on beam, AISC Beam Diagrams and Formulas; Table of concentrated load equivalents, Case oo (Ref. 2.2.5)

$a2_{G3} := 0.3333$
$c2_{G3} := 1.0$
$e2_{G3} := 0.0355$

Coefficients for 2-concentrated loads on beam, AISC Beam Diagrams and Formulas; Table of concentrated load equivalents, Case 3 (Ref. 2.2.5)

$M_{g3} := a1_{G3} \cdot \left(\omega_{G3} \cdot L_{G3}\right) \cdot L_{G3} + a2_{G3} \cdot P_{G3} \cdot L_{G3}$ $M_{g3} = 2941.8\,k \cdot ft$ Girder moment

$M_{G3const} := a1_{G3} \cdot \left(\omega_{G3const} \cdot L_{G3}\right) \cdot L_{G3} + \left(a2_{G3} \cdot P_{G3const} \cdot L_{G3}\right) + \dfrac{C_p \cdot L_{G3}}{4}$

$M_{G3const} = 1800.7\,k \cdot ft$ Girder moment w/ construction loading

$V_{g3} := c1_{G3} \cdot \left(\omega_{G3v} \cdot L_{G3}\right) + c2_{G3} \cdot P_{G3v}$ $V_{g3} = 287.3\,kip$ Girder shear

$V_{g3const} := c1_{G3} \cdot \left(\omega_{G3const} \cdot L_{G3}\right) + \left(c2_{G3} \cdot P_{G3const}\right) + C_p$ $V_{g3const} = 154.8\,kip$ Girder shear w/ construction loading

$M_{G3} := \max(M_{g3}, M_{G3const})$ $M_{G3} = 2941.8\,k\cdot ft$ max girder beam moment

$V_{G3} := \max(V_{g3}, V_{g3const})$ $V_{G3} = 287.3\,kip$ max girder beam shear

Allowable shear stress per ANSI/AISC N690-1994 section Q1.5.1.2.1 (Ref. 2.2.6)

$F_v := 0.4\cdot F_y$ $F_v = 20\,ksi$ allowable shear stress

$f_v := \frac{V_{G3}}{d\cdot t_w}$ $f_v = 6.9\,ksi$ calculated shear stress

$DC_f := \frac{M_{G3}}{M_r}$ $DC_f = 0.81$ flexure demand/capacity ratio

$DC_v := \frac{f_v}{F_v}$ $DC_v = 0.34$ shear demand/capacity ratio

$\Delta_{G3} := \frac{e1_{G3}\cdot(S1_a\cdot trib_{G3}\cdot L_{G3})\cdot L_{G3}^3}{E_s\cdot I} + \left[\frac{e2_{G3}\cdot(S1_a\cdot L1\cdot L_{G1})\cdot L_{G3}^3}{E_s\cdot I}\right]$ $\Delta_{G3} = 0.861\,in$ max (D + L) girder deflection

$\Delta_{allow} := \frac{L_{G3}}{240}$ $\Delta_{allow} = 1.9\,in$ allowable girder deflection (Ref. 2.2.4)

$\frac{\Delta_{G3}}{\Delta_{allow}} = 0.45$ deflection demand/capacity ratio

$shear_check := if(f_v \le F_v, "OK", "N.G!")$ shear_check = "OK"

$moment_check := if(M_{G3} \le M_r, "OK", "N.G!")$ moment_check = "OK"

$deflection_check := if(\Delta_{G3} \le \Delta_{allow}, "OK", "N.G!")$ deflection_check = "OK"

$depth_check1 := if\left(d \ge \frac{F_y\cdot L_{G3}}{1000ksi}, "OK", "N.G!"\right)$ ANSI/AISC N690-1994 section Q1.13, CQ1.13 (Ref. 2.2.6) depth_check1 = "OK"

$depth_check2 := if\left(d \ge \frac{L_{G3}}{20}, "OK", "N.G!"\right)$ ANSI/AISC N690-1994 section Q1.13, CQ1.13 (Ref. 2.2.6) depth_check2 = "OK"

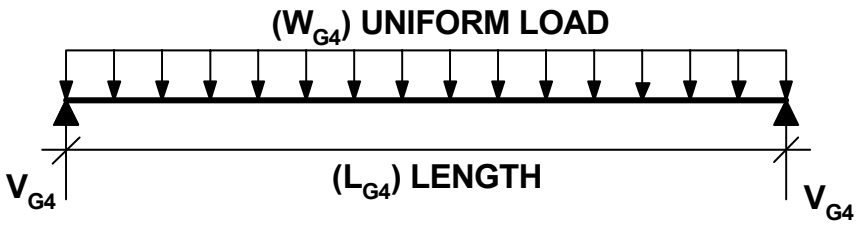
ROOF GIRDER G3 USE: G3 = "W36x359"

ROOF GIRDER G4

ROOF ELEV. 64'-0" - Girder Design between Gridline 5 to 6, from D to E, and G to H.
Nominal concrete slab thickness = 1'-6" (Attachment A-2)

BEAM PROPERTIES per AISC 1997 (Ref. 2.2.5)

$G4 := "W36x300"$ $M_r := 3050k\cdot ft$ $d := 36.74in$ $t_w := 0.945in$ $I := 20300in^4$ $t_f := 1.68in$



5 concentrated loads equally spaced; consider as uniform load. Typical Beam loading diagram, Part 2, Beam Diagrams and Formulas, AISC 1997 (Ref. 2.2.5)

$L_{G4} := 36ft$ $trib_{G4} := 26ft$ Max girder span/ tributary loading

$\omega_{G4} := BMax_{rf} \cdot trib_{G4}$ $\omega_{G4} = 16.09\,klf$ Uniform girder loading for flexure

$\omega_{G4v} := VMax_{rf} \cdot trib_{G4}$ $\omega_{G4v} = 18.4\,klf$ Uniform girder loading for shear

$\omega_{G4const} := S19 \cdot trib_{G4}$ $\omega_{G4const} = 9.59\,klf$ Uniform girder construction loading

$a_{G4} := 0.125$

$c_{G4} := 0.5$

$e_{G4} := 0.013$

Coefficients for uniform load on beam, AISC Beam Diagrams and Formulas; Table of concentrated load equivalents, Case oo (Ref. 2.2.5)

$M_{g4} := a_{G4} \cdot (\omega_{G4} \cdot L_{G4}) \cdot L_{G4}$ $M_{g4} = 2606.2\,k \cdot ft$ Girder moment

$M_{G4const} := a_{G4} \cdot (\omega_{G4const} \cdot L_{G4}) \cdot L_{G4} + \frac{C_p \cdot L_{G4}}{4}$ $M_{G4const} = 1598.2\,k \cdot ft$ Girder moment w/ construction loading

$V_{g4} := c_{G4} \cdot (\omega_{G4v} \cdot L_{G4})$ $V_{g4} = 330.9\,kip$ Girder shear

$V_{g4const} := c_{G4} \cdot (\omega_{G4const} \cdot L_{G4}) + C_p$ $V_{g4const} = 177.6\,kip$ Girder shear w/ construction loading

$M_{G4} := \max(M_{g4}, M_{G4const})$ $M_{G4} = 2606.2\,k \cdot ft$ max girder beam moment

$V_{G4} := \max(V_{g4}, V_{g4const})$ $V_{G4} = 330.9\,kip$ max girder beam shear

Allowable shear stress per ANSI/AISC N690-1994 section Q1.5.1.2.1 (Ref. 2.2.6)

$F_v := 0.4 \cdot F_y$ $F_v = 20\,ksi$ allowable shear stress

$f_v := \frac{V_{G4}}{d \cdot t_w}$ $f_v = 9.5\,ksi$ calculated shear stress

$DC_f := \frac{M_{G4}}{M_r}$ $DC_f = 0.85$ flexure demand/capacity ratio

$DC_v := \frac{f_v}{F_v}$ $DC_v = 0.48$ shear demand/capacity ratio

$\Delta_{G4} := \frac{e_{G4} \cdot (S1_a \cdot L2 \cdot L_{G4}) \cdot L_{G4}^3}{E_s \cdot I}$ $\Delta_{G4} = 0.656\,in$ max (D + L) girder deflection

$\Delta_{allow} := \frac{L_{G4}}{240}$ $\Delta_{allow} = 1.8\,in$ allowable girder deflection (Ref. 2.2.4)

$\frac{\Delta_{G4}}{\Delta_{allow}} = 0.36$ deflection demand/capacity ratio

$shear_check := if\left(f_v \le F_v, "OK", "N.G!"\right)$ shear_check = "OK"

$moment_check := if\left(M_{G4} \le M_r, "OK", "N.G!"\right)$ moment_check = "OK"

$deflection_check := if\left(\Delta_{G4} \le \Delta_{allow}, "OK", "N.G!"\right)$ deflection_check = "OK"

$depth_check1 := if\left(d \ge \frac{F_y \cdot L_{G4}}{1000 \cdot ksi}, "OK", "N.G!"\right)$ ANSI/AISC N690-1994 section Q1.13, CQ1.13 (Ref. 2.2.6) depth_check1 = "OK"

$depth_check2 := if\left(d \ge \frac{L_{G4}}{20}, "OK", "N.G!"\right)$ ANSI/AISC N690-1994 section Q1.13, CQ1.13 (Ref. 2.2.6) depth_check2 = "OK"

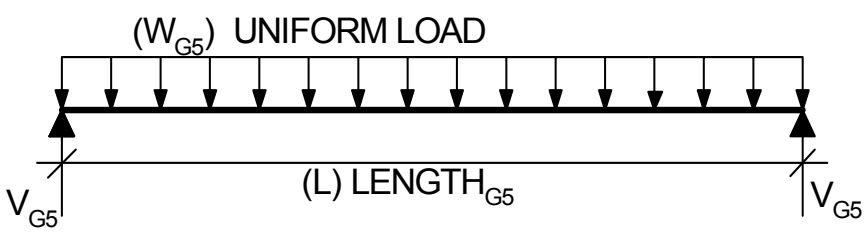
ROOF GIRDER G4 USE: G4 = "W36x300"

ROOF GIRDER G5

ROOF ELEV. 64'-0" - Girder Design between Gridline 5 to 6, from E to G.
Nominal concrete slab thickness = 2'-9" (Attachment A-2) assumption 3.1.10

BEAM PROPERTIES per AISC 1997 (Ref. 2.2.5)

$G5 := "W36x393"$ $M_r := 3990k \cdot ft$ $d := 37.80in$ $t_w := 1.220in$ $I := 27500in^4$ $t_f := 2.200in$



7 concentrated loads equally spaced; consider as uniform load. Typical Beam loading diagram, Part 2, Beam Diagrams and Formulas, AISC 1997 (Ref. 2.2.5)

$L_{G5} := 36ft$ $trib_{G5} := 22.5ft$ Max girder span/ tributary loading

$\omega_{G5} := BMax_{rf3} \cdot trib_{G5}$ $\omega_{G5} = 20.88\,klf$ Uniform girder loading for flexure

$\omega_{G5v} := VMax_{rf3} \cdot trib_{G5}$ $\omega_{G5v} = 23.9\,klf$ Uniform girder loading for shear

$\omega_{G5const} := S20 \cdot trib_{G5}$ $\omega_{G5const} = 12.52\,klf$ Uniform girder construction loading

$a_{G5} := 0.125$

$c_{G5} := 0.5$

$e_{G5} := 0.013$

Coefficients for uniform load on beam, AISC Beam Diagrams and Formulas; Table of concentrated load equivalents, Case oo (Ref. 2.2.5)

$M_{g5} := a_{G5} \cdot \left(\omega_{G5} \cdot L_{G5}\right) \cdot L_{G5}$ $M_{g5} = 3383\,k \cdot ft$ Girder moment

$M_{G5const} := a_{G5} \cdot \left(\omega_{G5const} \cdot L_{G5}\right) \cdot L_{G5} + \frac{C_p \cdot L_{G5}}{4}$ $M_{G5const} = 2072.5\,k \cdot ft$ Girder moment w/ construction loading

$V_{g5} := c_{G5} \cdot \left(\omega_{G5v} \cdot L_{G5}\right)$ $V_{g5} = 429.6\,kip$ Girder shear

$V_{g5const} := c_{G5} \cdot \left(\omega_{G5const} \cdot L_{G5}\right) + C_p$ $V_{g5const} = 230.3\,kip$ Girder shear w/ construction loading

$M_{G5} := max\left(M_{g5}, M_{G4const}\right)$ $M_{G5} = 3383\,k \cdot ft$ max girder beam moment

$V_{G5} := max\left(V_{g5}, V_{g5const}\right)$ $V_{G5} = 429.6\,kip$ max girder beam shear

Allowable shear stress per ANSI/AISC N690-1994 section Q1.5.1.2.1 (Ref. 2.2.6)

$F_v := 0.4 \cdot F_y$ $\quad F_v = 20\,ksi$ allowable shear stress

$f_v := \frac{V_{G5}}{d \cdot t_w}$ $\quad f_v = 9.3\,ksi$ calculated shear stress

$DC_f := \frac{M_{G5}}{M_r}$ $\quad DC_f = 0.85$ flexure demand/capacity ratio

$DC_v := \frac{f_v}{F_v}$ $\quad DC_v = 0.47$ shear demand/capacity ratio

$\Delta_{G5} := \frac{e_{G5} \cdot (S1_d \cdot L3 \cdot L_{G5}) \cdot L_{G5}^3}{E_s \cdot I}$ $\quad \Delta_{G5} = 0.619\,in$ max (D + L) girder deflection

$\Delta_{allow} := \frac{L_{G5}}{240}$ $\quad \Delta_{allow} = 1.8\,in$ allowable girder deflection (Ref. 2.2.4)

$\frac{\Delta_{G5}}{\Delta_{allow}} = 0.34$ deflection demand/capacity ratio

$shear_check := if(f_v \le F_v, "OK", "N.G!")$ $\quad$ shear_check = "OK"

$moment_check := if(M_{G5} \le M_r, "OK", "N.G!")$ $\quad$ moment_check = "OK"

$deflection_check := if(\Delta_{G5} \le \Delta_{allow}, "OK", "N.G!")$ $\quad$ deflection_check = "OK"

$depth_check1 := if\left(d \ge \frac{F_y \cdot L_{G5}}{1000 \cdot ksi}, "OK", "N.G!"\right)$ ANSI/AISC N690-1994 section Q1.13, CQ1.13 (Ref. 2.2.6) $\quad$ depth_check1 = "OK"

$depth_check2 := if\left(d \ge \frac{L_{G5}}{20}, "OK", "N.G!"\right)$ ANSI/AISC N690-1994 section Q1.13, CQ1.13 (Ref. 2.2.6) $\quad$ depth_check2 = "OK"

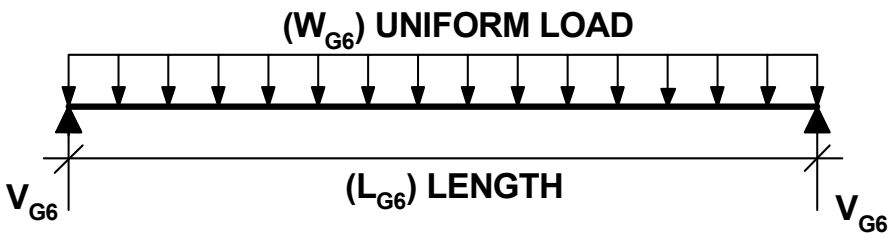
ROOF GIRDER G5 USE: $\quad$ G5 = "W36x393"

ROOF GIRDER G6

ROOF ELEV. 32'-0" - Girder Design between Gridline 1 and 2, from E to G at Exit Vestibule.
Nominal concrete slab thickness = 1'-6" (Attachment A-1). Girder G6 envelopes roof girder at entrance vestibule between gridline 12 and 13, from E to G. Therefore, girder design between Grid 1 & 2 will be used at entrance vestibule as well.

BEAM PROPERTIES per AISC 1997 (Ref. 2.2.5)

$G6 := \text{"W36X359"}$ $M_r := 3630k\cdot ft$ $d := 37.40in$ $t_w := 1.12in$ $I := 24800in^4$ $t_f := 2.01in$



6 concentrated loads equally spaced; consider as uniform load. Typical Beam loading diagram, Part 2, Beam Diagrams and Formulas, AISC 1997 (Ref. 2.2.5)

$L_{G6} := 46ft$	$trib_{G6} := 18.8ft$	Max girder span and tributary loading
$\omega_{G6} := BMax_{rf}\cdot trib_{G6}$	$\omega_{G6} = 11.63\,klf$	Uniform girder loading for flexure
$\omega_{G6v} := VMax_{rf}\cdot trib_{G6}$	$\omega_{G6v} = 13.3\,klf$	Uniform girder loading for shear
$\omega_{G6const} := S19\cdot trib_{G6}$	$\omega_{G6const} = 6.93\,klf$	Uniform girder construction loading
$a_{G6} := 0.125$ $c_{G6} := 0.5$ $e_{G6} := 0.013$		Coefficients for uniform load on beam, AISC Beam Diagrams and Formulas; Table of concentrated load equivalents, Case oo (Ref. 2.2.5)
$M_{g6} := a_{G6}\cdot(\omega_{G6}\cdot L_{G6})\cdot L_{G6}$	$M_{g6} = 3076.8\,k\cdot ft$	Girder moment
$M_{G6const} := a_{G6}\cdot(\omega_{G6const}\cdot L_{G6})\cdot L_{G6} + \frac{C_p\cdot L_{G6}}{4}$	$M_{G6const} = 1891.1\,k\cdot ft$	Girder moment w/ construction loading
$V_{g6} := c_{G6}\cdot(\omega_{G6v}\cdot L_{G6})$	$V_{g6} = 305.8\,kip$	Girder shear
$V_{g6const} := c_{G6}\cdot(\omega_{G6const}\cdot L_{G6}) + C_p$	$V_{g6const} = 164.4\,kip$	Girder shear w/ construction loading
$M_{G6} := \max(M_{g6}, M_{G6const})$	$M_{G6} = 3076.8\,k\cdot ft$	max girder beam moment
$V_{G6} := \max(V_{g6}, V_{g6const})$	$V_{G6} = 305.8\,kip$	max girder beam shear

Allowable shear stress per ANSI/AISC N690-1994 section Q1.5.1.2.1 (Ref. 2.2.6)

$F_v := 0.4\cdot F_y$	$F_v = 20\,ksi$	allowable shear stress
$f_v := \frac{V_{G6}}{d\cdot t_w}$	$f_v = 7.3\,ksi$	calculated shear stress
$DC_f := \frac{M_{G6}}{M_r}$	$DC_f = 0.85$	flexure demand/capacity ratio
$DC_v := \frac{f_v}{F_v}$	$DC_v = 0.36$	shear demand/capacity ratio

$\Delta_{G6} := \dfrac{e_{G6}\cdot(S1_a\cdot trib_{G6}\cdot L_{G6})\cdot L_{G6}^3}{E_s\cdot I}$ $\Delta_{G6} = 1.035\,in$ max (D + L) girder deflection

$\Delta_{allow} := \dfrac{L_{G6}}{240}$ $\Delta_{allow} = 2.3\,in$ allowable girder deflection (Ref. 2.2.4) $\dfrac{\Delta_{G6}}{\Delta_{allow}} = 0.45$ deflection demand/capacity ratio

$shear_check := if(f_v \le F_v, "OK", "N.G!")$ shear_check = "OK"

$moment_check := if(M_{G6} \le M_r, "OK", "N.G!")$ moment_check = "OK"

$deflection_check := if(\Delta_{G6} \le \Delta_{allow}, "OK", "N.G!")$ deflection_check = "OK"

$depth_check1 := if\left(d \ge \dfrac{F_y\cdot L_{G6}}{800ksi}, "OK", "N.G!"\right)$ ANSI/AISC N690-1994 section Q1.13, CQ1.13 (Ref. 2.2.6) depth_check1 = "OK"

$depth_check2 := if\left(d \ge \dfrac{L_{G6}}{20}, "OK", "N.G!"\right)$ ANSI/AISC N690-1994 section Q1.13, CQ1.13 (Ref. 2.2.6) depth_check2 = "OK"

ROOF GIRDER G6 USE: G6 = "W36X359"

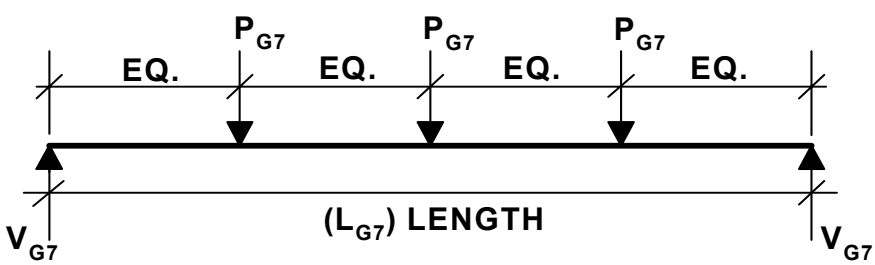
6.4.2 FLOOR GIRDER CALCULATION, Ref. 2.2.5, Ref. 2.2.6

FLOOR GIRDER G7

FLOOR ELEV. 32'-0" - Girder Design between Gridline 2 to 5, and 6 to 12, from D to E and G to H.
Nominal concrete slab thickness = 1'-6" (Attachment A-1)

BEAM PROPERTIES per AISC 1997 (Ref. 2.2.5)

$G7 := "W33X118"$ $M_r := 987k\cdot ft$ $d := 32.86in$ $t_w := 0.55in$ $I := 5900in^4$ $t_f := 0.740in$



3 equally spaced concentrated loads; Girder loading diagram, Part 2, Beam Diagrams and Formulas, AISC 1997 (Ref. 2.2.5)

P_{G7} reaction from beam B8 (each side of girder)

$L_{G7} := 26ft$ Max girder span

$P_{G7} := 2\cdot\omega_{b8}\cdot\dfrac{L8}{2}$ $P_{G7} = 58.9\,kip$ Concentrated beam load for bending (see beam B8 design)

$P_{G7v} := 2\omega_{b8v}\cdot\dfrac{L8}{2}$ $P_{G7v} = 67.3\,kip$ Concentrated beam load for shear

$P_{G7const} := 2\cdot\left(\omega_{b8_const}\cdot\dfrac{L8}{2}\right)$ $P_{G7const} = 34.8\,kip$ Concentrated beam construction load

$a_{G7} := 0.5$

$c_{G7} := 1.5$

$e_{G7} := 0.0495$

Coefficients for 3 equally spaced concentrated loads on beam, AISC Beam Diagrams and Formulas; Table of concentrated load equivalents, Case 4 (Ref. 2.2.5).

$M_{g7} := a_{G7} \cdot P_{G7} \cdot L_{G7}$ $\quad M_{g7} = 765.7\,k \cdot ft$ Girder moment

$M_{g7const} := \left(a_{G7} \cdot P_{G7const} \cdot L_{G7}\right) + \frac{C_p \cdot L_{G7}}{4}$ $\quad M_{g7const} = 484.3\,k \cdot ft$ Girder moment w/ construction loading

$V_{g7} := c_{G7} \cdot P_{G7v}$ $\quad V_{g7} = 101\,kip$ Girder shear

$V_{g7const} := c_{G7} \cdot P_{G7const} + C_p$ $\quad V_{g7const} = 57.1\,kip$ Girder shear w/ construction loading

$M_{G7} := \max\left(M_{g7}, M_{g7const}\right)$ $\quad M_{G7} = 765.7\,k \cdot ft$ max girder beam moment

$V_{G7} := \max\left(V_{g7}, V_{g7const}\right)$ $\quad V_{G7} = 101\,kip$ max girder beam shear

Allowable shear stress per ANSI/AISC N690-1994 section Q1.5.1.2.1 (Ref. 2.2.6)

$F_v := 0.4 \cdot F_y$ $\quad F_v = 20\,ksi$ allowable shear stress

$f_v := \frac{V_{G7}}{d \cdot t_w}$ $\quad f_v = 5.6\,ksi$ calculated shear stress

$DC_f := \frac{M_{G7}}{M_r}$ $\quad DC_f = 0.78$ flexure demand/capacity ratio

$DC_v := \frac{f_v}{F_v}$ $\quad DC_v = 0.28$ shear demand/capacity ratio

$\Delta_{G7} := \frac{e_{G7} \cdot \left(S1_c \cdot trib_{b8} \cdot L8\right) \cdot L_{G7}^3}{E_s \cdot I}$ $\quad \Delta_{G7} = 0.376\,in$ max (D + L) girder deflection

$\Delta_{allow} := \frac{L_{G7}}{240}$ $\quad \Delta_{allow} = 1.3\,in$ allowable girder deflection (Ref. 2.2.4)

$\frac{\Delta_{G7}}{\Delta_{allow}} = 0.29$ deflection demand/capacity ratio

$shear_check := if\left(f_v \le F_v, "OK", "N.G!"\right)$ $\quad shear_check = "OK"$

$moment_check := if\left(M_{G7} \le M_r, "OK", "N.G!"\right)$ $\quad moment_check = "OK"$

$deflection_check := if\left(\Delta_{G7} \le \Delta_{allow}, "OK", "N.G!"\right)$ $\quad deflection_check = "OK"$

$depth_check1 := if\left(d \ge \frac{F_y \cdot L_{G7}}{800ksi}, "OK", "N.G!"\right)$ ANSI/AISC N690-1994 section Q1.13, CQ1.13 (Ref. 2.2.6) $\quad depth_check1 = "OK"$

$depth_check2 := if\left(d \ge \frac{L_{G7}}{20}, "OK", "N.G!"\right)$ ANSI/AISC N690-1994 section Q1.13, CQ1.13 (Ref. 2.2.6) $\quad depth_check2 = "OK"$

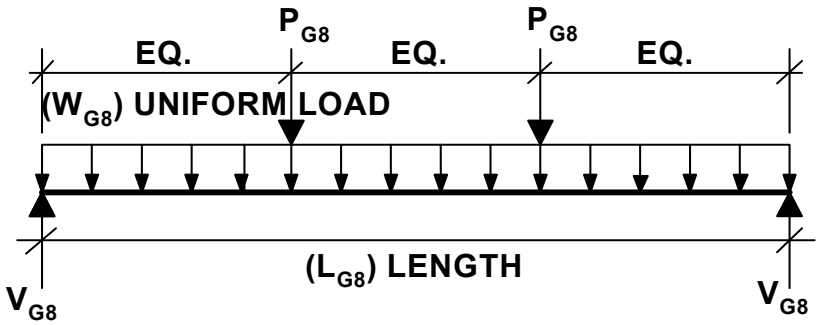
FLOOR GIRDER G7 USE: $\quad G7 = "W33X118"$

FLOOR GIRDER G8

FLOOR ELEV. 32'-0" - Girder Design between Gridlines 3 to 5, and 10 to 12, from D to E and G to H.
Nominal concrete slab thickness = 1'-6" (Attachment A-1)

BEAM PROPERTIES per AISC1997 (Ref. 2.2.5)

$G8 := \text{"W36x393"}$ $M_r := 3990k\cdot ft$ $d := 37.80in$ $t_w := 1.22in$ $I := 27500in^4$ $t_f := 2.20in$



Girder loading diagram, Part 2, Beam Diagrams and Formulas, AISC 1997 (Ref. 2.2.5)

P_{G8} reactions from girder G7 (each side of girder)

$L_{G8} := 38ft$ $trib_{G8} := 6.5ft$ Max girder span

$\omega_{G8} := BMax_{fl}\cdot trib_{G8}$ $\omega_{G8} = 4.1\,klf$ Uniform girder loading for flexure

$\omega_{G8v} := VMax_{fl}\cdot trib_{G8}$ $\omega_{G8v} = 4.6\,klf$ Uniform girder loading for shear

$\omega_{G8const} := S19\cdot trib_{G8}$ $\omega_{G8const} = 2.4\,klf$ Uniform girder construction loading

$P_{G8} := 2\cdot c_{G7}\cdot P_{G7}$ $P_{G8} = 176.7\,kip$ Concentrated beam load for flexure (see girder G7 design)

$P_{G8v} := 2\cdot c_{G7}\cdot P_{G7v}$ $P_{G8v} = 201.9\,kip$ Concentrated beam load for shear

$P_{G8const} := 2(c_{G7}\cdot P_{G7const})$ $P_{G8const} = 104.3\,kip$ Concentrated beam construction load

$a1_{G8} := 0.125$
$c1_{G8} := 0.5$
$e1_{G8} := 0.013$

Coefficients for uniform load on beam, AISC Beam Diagrams and Formulas; Table of concentrated load equivalents, Case oo (Ref. 2.2.5)

$a2_{G8} := 0.3333$
$c2_{G8} := 1.0$
$e2_{G8} := 0.0355$

Coefficients for 2-concentrated loads on beam, AISC Beam Diagrams and Formulas; Table of concentrated load equivalents, Case 3 (Ref. 2.2.5)

$M_{g8} := a1_{G8}\cdot(\omega_{G8}\cdot L_{G8})\cdot L_{G8} + a2_{G8}\cdot P_{G8}\cdot L_{G8}$ $M_{g8} = 2971.2\,k\cdot ft$ Girder moment

$M_{G8const} := a1_{G8}\cdot(\omega_{G8const}\cdot L_{G8})\cdot L_{G8} + (a2_{G8}\cdot P_{G8const}\cdot L_{G8}) + \frac{C_p\cdot L_{G8}}{4}$

$M_{G8const} = 1800.7\,k\cdot ft$ Girder moment w/ construction loading

$V_{g8} := c1_{G8}\cdot(\omega_{G8v}\cdot L_{G8}) + c2_{G8}\cdot P_{G8v}$ $V_{g8} = 290.1\,kip$ Girder shear

$V_{g8const} := c1_{G8}\cdot(\omega_{G8const}\cdot L_{G8}) + (c2_{G8}\cdot P_{G8const}) + C_p$ $V_{g8const} = 154.8\,kip$ Girder shear w/ construction loading

$M_{G8} := \max(M_{g8}, M_{G8const})$ $M_{G8} = 2971.2\,k\cdot ft$ max girder beam moment

$V_{G8} := \max(V_{g8}, V_{g8const})$ $V_{G8} = 290.1\,kip$ max girder beam shear

Allowable shear stress per ANSI/AISC N690-1994 section Q1.5.1.2.1 (Ref. 2.2.6)

$F_v := 0.4 \cdot F_y$ $F_v = 20\,ksi$ allowable shear stress

$f_v := \frac{V_{G8}}{d \cdot t_w}$ $f_v = 6.3\,ksi$ calculated shear stress

$DC_f := \frac{M_{G8}}{M_r}$ $DC_f = 0.74$ flexure demand/capacity ratio

$DC_v := \frac{f_v}{F_v}$ $DC_v = 0.31$ shear demand/capacity ratio

$\Delta_{G8} := \frac{e1_{G8} \cdot (S1_c \cdot trib_{G8} \cdot L_{G8}) \cdot L_{G8}^3}{E_s \cdot I} + \left[\frac{e2_{G8} \cdot (S1_c \cdot L_{G7} \cdot L8) \cdot L_{G8}^3}{E_s \cdot I}\right]$ $\Delta_{G8} = 0.895\,in$ max (D + L) girder deflection

$\Delta_{allow} := \frac{L_{G8}}{240}$ $\Delta_{allow} = 1.9\,in$ allowable girder deflection (Ref. 2.2.4) $\frac{\Delta_{G8}}{\Delta_{allow}} = 0.47$ deflection demand/capacity ratio

$shear_check := if(f_v \le F_v, "OK", "N.G!")$ shear_check = "OK"

$moment_check := if(M_{G8} \le M_r, "OK", "N.G!")$ moment_check = "OK"

$deflection_check := if(\Delta_{G8} \le \Delta_{allow}, "OK", "N.G!")$ deflection_check = "OK"

$depth_check1 := if\left(d \ge \frac{F_y \cdot L_{G8}}{800ksi}, "OK", "N.G!"\right)$ ANSI/AISC N690-1994 section Q1.13, CQ1.13 (Ref. 2.2.6) depth_check1 = "OK"

$depth_check2 := if\left(d \ge \frac{L_{G8}}{20}, "OK", "N.G!"\right)$ ANSI/AISC N690-1994 section Q1.13, CQ1.13 (Ref. 2.2.6) depth_check2 = ▮

FLOOR GIRDER G8 USE: G8 = "W36x393"

FLOOR GIRDER G9 with forklift loading

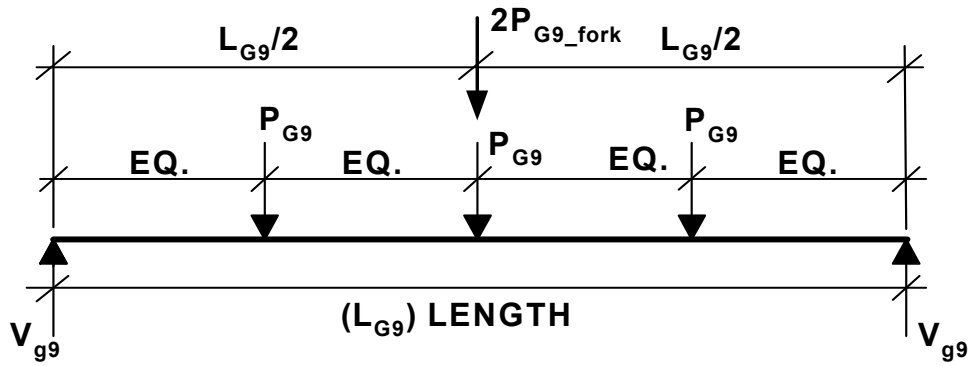
FLOOR ELEV. 32'-0" - Girder Design between Gridline 5 to 6, from D to E and G to H.
Nominal concrete slab thickness = 1'-6" (Attachment A-1)

BEAM PROPERTIES per AISC 1997 (Ref. 2.2.5)

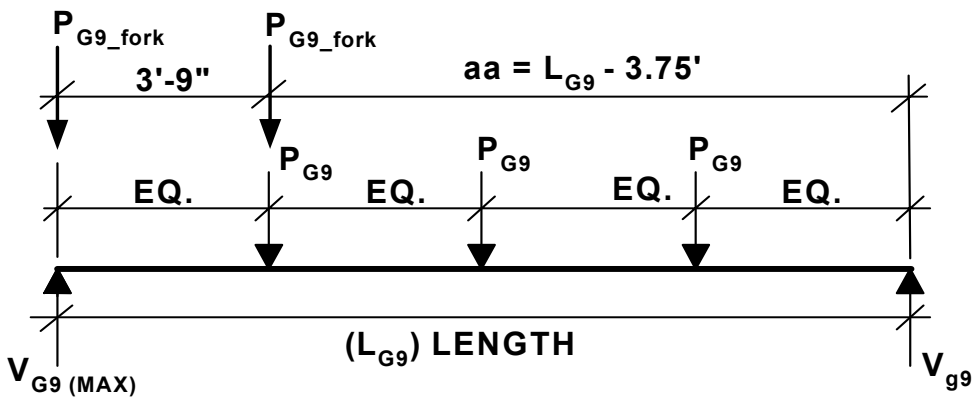
$G9 := "W33X130"$ $M_r := 1120k \cdot ft$ $d := 33.09in$ $t_w := 0.58in$ $I := 6710in^4$ $t_f := 0.855in$

Girder loading diagram, Part 2, Beam Diagrams and Formulas, AISC 1997 (Ref. 2.2.5)

P_{G9} reactions from beam B9 (each side of girder), P_{G9_fork} reactions from forklift wheel load (Assumption 3.1.2)
$2P_{G9_fork}$ includes loading from both wheels



MAX FLEXURE LOADING



MAX SHEAR LOADING

$L_{G9} := 26ft$		Max girder span
$P_{G9} := 2 \cdot \omega_{b9} \cdot \frac{L9}{2}$	$P_{G9} = 48.7\,kip$	Concentrated beam load for bending (see beam B9 design)
$P_{G9v} := 2\omega_{b9v} \cdot \frac{L9}{2}$	$P_{G9v} = 55.7\,kip$	Concentrated beam load for shear
$P_{G9const} := 2 \cdot \left(\omega_{b9_const} \cdot \frac{L9}{2}\right)$	$P_{G9const} = 28.8\,kip$	Concentrated beam construction load
$P_{G9_fork} := L_p$	$P_{G9_fork} = 16\,kip$	Concentrated beam load from forklift wheel load spaced at 3'-9" o.c. (Assumption 3.1.2)
$aa := L_{G9} - 3.75ft$	$aa = 22.3\ ft$	length from girder end to forklift wheel load
$a_{G9} := 0.5$ $c_{G9} := 1.5$ $e_{G9} := 0.0495$		Coefficients for 3 equally spaced concentrated loads on beam, AISC Beam Diagrams and Formulas; Table of concentrated load equivalents, Case 4 (Ref. 2.2.5.)
$M_{g9} := a_{G9} \cdot P_{G9} \cdot L_{G9} + \frac{\left(2 \cdot P_{G9_fork}\right) \cdot L_{G9}}{4}$	$M_{g9} = 841.7\,k \cdot ft$	Girder moment
$M_{g9const} := \left(a_{G9} \cdot P_{G9const} \cdot L_{G9}\right) + \frac{C_p \cdot L_{G9}}{4}$	$M_{g9const} = 406.4\,k \cdot ft$	Girder moment w/ construction loading

$V_{g9} := c_{G9} \cdot P_{G9v} + P_{G9_fork} + \frac{P_{G9_fork} \cdot aa}{L_{G9}}$ $V_{g9} = 113.3\,kip$ Girder shear

$V_{g9const} := c_{G9} \cdot P_{G9const} + C_p$ $V_{g9const} = 48.1\,kip$ Girder shear w/ construction loading

$M_{G9} := \max(M_{g9}, M_{g9const})$ $M_{G9} = 841.7\,k \cdot ft$ max girder beam moment

$V_{G9} := \max(V_{g9}, V_{g9const})$ $V_{G9} = 113.3\,kip$ max girder beam shear

Allowable shear stress per ANSI/AISC N690-1994 section Q1.5.1.2.1 (Ref. 2.2.6)

$F_v := 0.4 \cdot F_y$ $F_v = 20\,ksi$ allowable shear stress

$f_v := \frac{V_{G9}}{d \cdot t_w}$ $f_v = 5.9\,ksi$ calculated shear stress

$DC_f := \frac{M_{G9}}{M_r}$ $DC_f = 0.75$ flexure demand/capacity ratio

$DC_v := \frac{f_v}{F_v}$ $DC_v = 0.3$ shear demand/capacity ratio

$\Delta_{G9} := \left[\frac{e_{G9} \cdot (S1_c \cdot trib_{b9} \cdot L9) \cdot L_{G9}^3}{E_s \cdot I}\right] + \frac{(2 \cdot P_{G9_fork}) \cdot L_{G9}^3}{48 \cdot E_s \cdot I}$ $\Delta_{G9} = 0.377\,in$ max (D + L) girder deflection

$\Delta_{allow} := \frac{L_{G9}}{240}$ $\Delta_{allow} = 1.3\,in$ allowable girder deflection (Ref. 2.2.4)

$\frac{\Delta_{G9}}{\Delta_{allow}} = 0.29$ deflection demand/capacity ratio

$shear_check := if(f_v \le F_v, "OK", "N.G!")$ shear_check = "OK"

$moment_check := if(M_{G9} \le M_r, "OK", "N.G!")$ moment_check = "OK"

$deflection_check := if(\Delta_{G9} \le \Delta_{allow}, "OK", "N.G!")$ deflection_check = "OK"

$depth_check1 := if\left(d \ge \frac{F_y \cdot L_{G9}}{800ksi}, "OK", "N.G!"\right)$ ANSI/AISC N690-1994 section Q1.13, CQ1.13 (Ref. 2.2.6) depth_check1 = "OK"

$depth_check2 := if\left(d \ge \frac{L_{G9}}{20}, "OK", "N.G!"\right)$ ANSI/AISC N690-1994 section Q1.13, CQ1.13 (Ref. 2.2.6) depth_check2 = "OK"

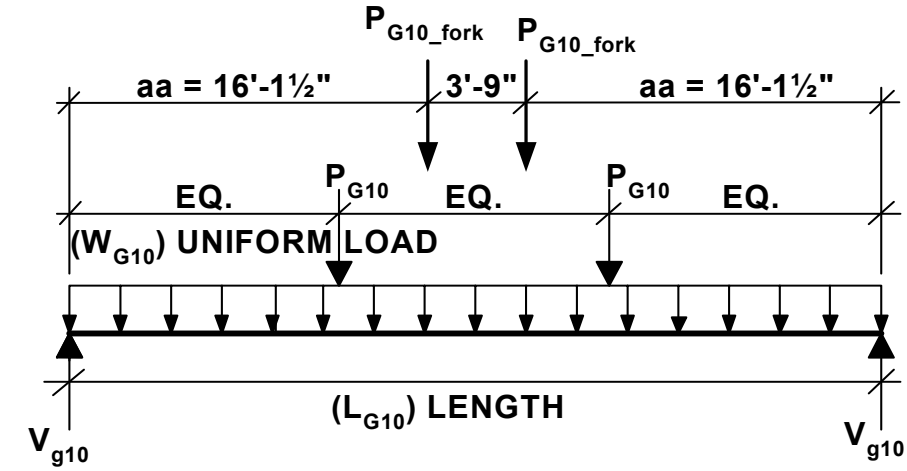
FLOOR GIRDER G9 USE: G9 = "W33X130"

FLOOR GIRDER G10 with forklift loads

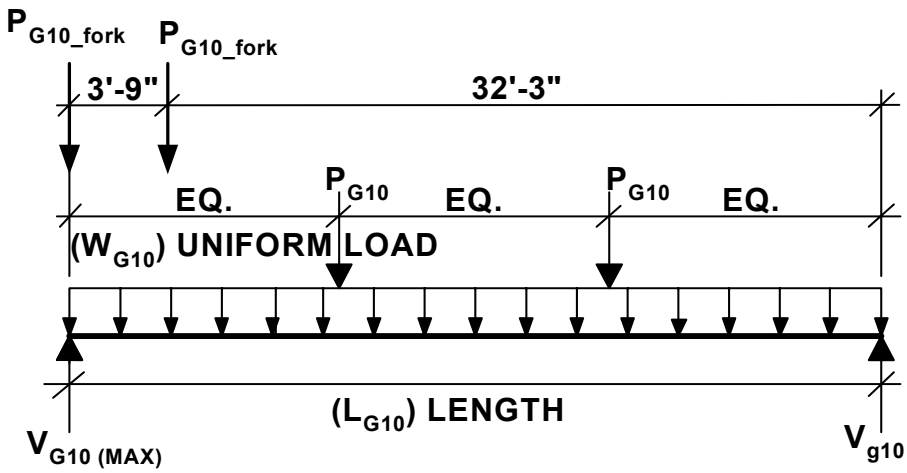
FLOOR ELEV. 32'-0" - Girder Design between Gridline 5to 6, from D to E and G to H.
Nominal concrete slab thickness = 1'-6" (Attachment A-1)

BEAM PROPERTIES per AISC 1997 (Ref. 2.2.5)

$G10 := "W36x359"$ $M_r := 3630k \cdot ft$ $d := 37.40in$ $t_w := 1.12in$ $I := 24800in^4$ $t_f := 2.01in$



MAX FLEXURE LOADING



MAX SHEAR LOADING

Girder loading diagram, Part 2, Beam Diagrams and Formulas, AISC 1997 (Ref. 2.2.5)

P_{G10} reactions from girder G9 (each side of girder)
P_{G10_fork} reactions from forklift wheel load (Assumption 3.1.2)

$L_{G10} := 36ft$	$trib_{G10} := 6.5ft$	Max girder span
$\omega_{G10} := BMax_{fl} \cdot trib_{G10}$	$\omega_{G10} = 4.06\,klf$	Uniform girder loading for flexure
$\omega_{G10v} := VMax_{fl} \cdot trib_{G10}$	$\omega_{G10v} = 4.6\,klf$	Uniform girder loading for shear
$\omega_{G10const} := S19 \cdot trib_{G10}$	$\omega_{G10const} = 2.4\,klf$	Uniform girder construction loading
$P_{G10} := 2 \cdot c_{G9} \cdot P_{G9}$	$P_{G10} = 146.2\,kip$	Concentrated beam load for flexure (see girder G9 design)
$P_{G10v} := 2 \cdot c_{G9} \cdot P_{G9v}$	$P_{G10v} = 167.1\,kip$	Concentrated beam load for shear
$P_{G10const} := 2(c_{G9} \cdot P_{G9const})$	$P_{G10const} = 86.3\,kip$	Concentrated beam construction load
$P_{G10_fork} := L_p$	$P_{G10_fork} = 16\,kip$	Concentrated beam load from forklift wheel load spaced at 3'-9" o.c. (Assumption 3.1.2)

$aa := \frac{L_{G10} - 3.75ft}{2}$ $aa = 16.1\ ft$ length from girder end to forklift wheel load

$a1_{G10} := 0.125$
$c1_{G10} := 0.5$
$e1_{G10} := 0.013$

Coefficients for uniform load on beam AISC Beam Diagrams and Formulas; Table of concentrated load equivalents, Case *oo (*Ref. 2.2.5.)

$a2_{G10} := 0.3333$
$c2_{G10} := 1.0$
$e2_{G10} := 0.0355$

Coefficients for 2 equally spaced concentrated loads on beam, AISC Beam Diagrams and Formulas; Table of concentrated load equivalents, Case 3 (Ref. 2.2.5.)

$M_{g10} := a1_{G10} \cdot (\omega_{G10} \cdot L_{G10}) \cdot L_{G10} + a2_{G10} \cdot P_{G10} \cdot L_{G10} + P_{G10_fork} \cdot aa$ $M_{g10} = 2670.7\,k \cdot ft$ Girder moment

$M_{G10const} := a1_{G10} \cdot (\omega_{G10const} \cdot L_{G10}) \cdot L_{G10} + (a2_{G10} \cdot P_{G10const} \cdot L_{G10}) + \frac{C_p \cdot L_{G10}}{4}$ Girder moment w/ construction loading

$M_{G10const} = 1468.6\,k \cdot ft$

$V_{g10} := c1_{G10} \cdot (\omega_{G10v} \cdot L_{G10}) + c2_{G10} \cdot P_{G10v} + P_{G10_fork} + \frac{P_{G10_fork} \cdot (L_{G10} - 3.75ft)}{L_{G10}}$ Girder shear

$V_{g10} = 281\,kip$

$V_{g10const} := c1_{G10} \cdot (\omega_{G10const} \cdot L_{G10}) + (c2_{G10} \cdot P_{G10const}) + C_p$ $V_{g10const} = 134.4\,kip$ Girder shear w/ construction loading

$M_{G10} := \max(M_{g10}, M_{G10const})$ $M_{G10} = 2670.7\,k \cdot ft$ max girder beam moment

$V_{G10} := \max(V_{g10}, V_{g10const})$ $V_{G10} = 281\,kip$ max girder beam shear

Allowable shear stress per ANSI/AISC N690-1994 section Q1.5.1.2.1 (Ref. 2.2.6)

$F_v := 0.4 \cdot F_y$ $F_v = 20\,ksi$ allowable shear stress

$f_v := \frac{V_{G10}}{d \cdot t_w}$ $f_v = 6.7\,ksi$ calculated shear stress

$DC_f := \frac{M_{G10}}{M_r}$ $DC_f = 0.74$ flexure demand/capacity ratio

$DC_v := \frac{f_v}{F_v}$ $DC_v = 0.34$ shear demand/capacity ratio

$$\Delta_{G10} := \frac{e1_{G10} \cdot (S1_c \cdot trib_{G10} \cdot L_{G10}) \cdot L_{G10}^3}{E_s \cdot I} + \left[\frac{e2_{G10} \cdot (S1_c \cdot L_{G9} \cdot L9) \cdot L_{G10}^3}{E_s \cdot I}\right] + \left[\left[\frac{P_{G10_fork} \cdot (aa)}{24 \cdot E_s \cdot I}\right] \cdot \left(3 \cdot L_{G10}^2 - 4 \cdot aa^2\right)\right]$$

$\Delta_{G10} = 0.792\,in$ max (D + L) girder deflection

$\Delta_{allow} := \frac{L_{G10}}{240}$ $\Delta_{allow} = 1.8\,in$ allowable girder deflection (Ref. 2.2.4)

$\frac{\Delta_{G10}}{\Delta_{allow}} = 0.44$ deflection demand/capacity ratio

$shear_check := if\left(f_v \le F_v, "OK", "N.G!"\right)$ shear_check = "OK"

$moment_check := if\left(M_{G10} \le M_r, "OK", "N.G!"\right)$ moment_check = "OK"

$deflection_check := if\left(\Delta_{G10} \le \Delta_{allow}, "OK", "N.G!"\right)$ deflection_check = "OK"

$depth_check1 := if\left(d \ge \frac{F_y \cdot L_{G10}}{800ksi}, "OK", "N.G!"\right)$ ANSI/AISC N690-1994 section Q1.13, CQ1.13 (Ref. 2.2.6) depth_check1 = "OK"

$depth_check2 := if\left(d \ge \frac{L_{G10}}{20}, "OK", "N.G!"\right)$ ANSI/AISC N690-1994 section Q1.13, CQ1.13 (Ref. 2.2.6) depth_check2 = "OK"

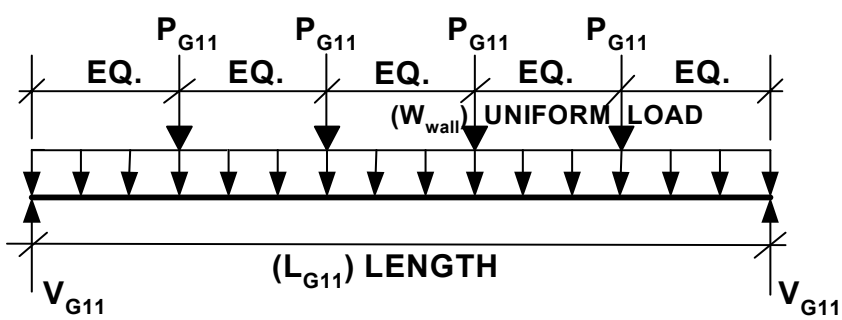
FLOOR GIRDER G10 USE: G10 = "W36x359"

FLOOR GIRDER G11

FLOOR ELEV. 32'-0" - Girder Design at Gridline 10, from E to G.
Girder G11 designed for construction load only (Assumption 3.1.7)
Nominal concrete slab thickness = 1'-6" (Attachment A-1)

BEAM PROPERTIES per AISC 1997 (Ref. 2.2.5)

$G11 := "W30X235"$ $M_r := 2050k \cdot ft$ $d := 31.3in$ $t_w := 0.830in$ $I := 11700in^4$ $t_f := 1.50in$



Girder loading diagram, Part 2, Beam Diagrams and Formulas, AISC 1997 (Ref. 2.2.5)

P_{G11} reaction from beam B10 (one side of girder)

$L_{G11} := 32ft$ Max girder span

$\omega_{wall} := \gamma_c \cdot 2 \cdot ft \cdot (25 \cdot ft)$ $\omega_{wall} = 7.5\,klf$ uniform load from 2'-0" thick concrete wall 1st construction pour (Assumption 3.1.7)

$P_{G11const} := \omega_{b10_const} \cdot \frac{L10}{2}$ $P_{G11const} = 15.3\,kip$ Concentrated beam construction load (see beam B10 design)

$a1_{G11} := 0.125$
$c1_{G11} := 0.5$
$e1_{G11} := 0.013$

Coefficients for uniform load on beam AISC Beam Diagrams and Formulas; Table of concentrated load equivalents, Case *oo* (Ref. 2.2.5).

$a2_{G11} := 0.6$
$c2_{G11} := 2.0$
$e2_{G11} := 0.0630$

Coefficients for 4 equally spaced concentrated loads on beam, AISC Beam Diagrams and Formulas; Table of concentrated load equivalents, Case 5 (Ref. 2.2.5).

$M_{G11const} := a1_{G11} \cdot (\omega_{wall}) \cdot L_{G11}^2 + a2_{G11} \cdot P_{G11const} \cdot L_{G11} + \frac{C_p \cdot L_{G11}}{4}$

$M_{G11const} = 1294.5\,k \cdot ft$

Girder moment w/ construction loading

$V_{G11const} := c1_{G11} \cdot (\omega_{wall}) \cdot L_{G11} + c2_{G11} \cdot P_{G11const} + C_p$

$V_{G11const} = 155.7\,kip$

Girder shear w/ construction loading

$M_{G11} := M_{G11const}$ $M_{G11} = 1294.5\,k \cdot ft$ max girder beam moment

$V_{G11} := V_{G11const}$ $V_{G11} = 155.7\,kip$ max girder beam shear

Allowable shear stress per ANSI/AISC N690-1994 section Q1.5.1.2.1 (Ref. 2.2.6)

$F_v := 0.4 \cdot F_y$ $F_v = 20\,ksi$ allowable shear stress

$f_v := \frac{V_{G11}}{d \cdot t_w}$ $f_v = 6\,ksi$ calculated shear stress

$DC_f := \frac{M_{G11}}{M_r}$ $DC_f = 0.63$ flexure demand/capacity ratio

$DC_v := \frac{f_v}{F_v}$ $DC_v = 0.3$ shear demand/capacity ratio

$\Delta_{G11const} := \frac{e1_{G11} \cdot (\omega_{wall}) \cdot L_{G11}^4}{E_s \cdot I} + \left(\frac{e2_{G11} \cdot P_{G11const} \cdot L_{G11}^3}{E_s \cdot I}\right) + \frac{C_p \cdot L_{G11}^3}{48 \cdot E_s \cdot I}$

$\Delta_{G11const} = 0.699\,in$ max construction girder deflection

$\Delta_{allow} := \frac{L_{G11}}{240}$ $\Delta_{allow} = 1.6\,in$ allowable girder deflection (Ref. 2.2.4)

$\frac{\Delta_{G11const}}{\Delta_{allow}} = 0.44$ deflection demand/capacity ratio

$shear_check := if(f_v \le F_v, "OK", "N.G!")$ shear_check = "OK"

$moment_check := if(M_{G11} \le M_r, "OK", "N.G!")$ moment_check = "OK"

$deflection_check := if(\Delta_{G11const} \le \Delta_{allow}, "OK", "N.G!")$ deflection_check = "OK"

$depth_check1 := if\left(d \ge \frac{F_y \cdot L_{G11}}{800 \cdot ksi}, "OK", "N.G!"\right)$ ANSI/AISC N690-1994 section Q1.13, CQ1.13 (Ref. 2.2.6) depth_check1 = "OK"

$depth_check2 := if\left(d \ge \frac{L_{G11}}{20}, "OK", "N.G!"\right)$ ANSI/AISC N690-1994 section Q1.13, CQ1.13 (Ref. 2.2.6) depth_check2 = "OK"

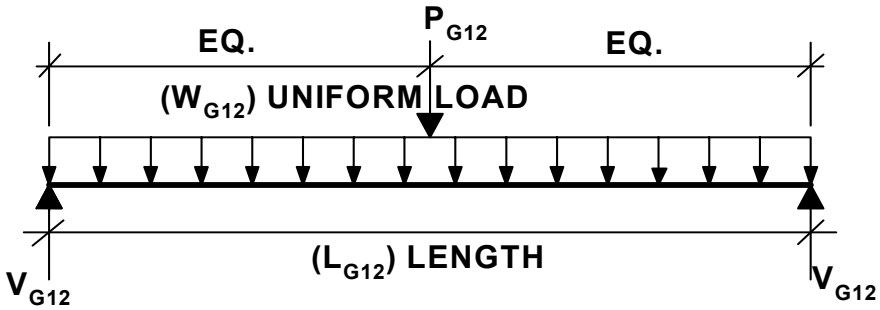
FLOOR GIRDER G11 USE: G11 = "W30X235"

FLOOR GIRDER G12

FLOOR ELEV. 32'-0" - Girder Design between Gridline 2 to 4, and 6 to 9, from D to E and G to H.
Nominal concrete slab thickness = 1'-6" (Attachment A-1)

BEAM PROPERTIES per AISC 1997 (Ref. 2.2.5)

$G12 := \text{"W36x232"}_r \cdot \quad d := 37.12in \quad t_w := 0.87in \quad I := 15000in^4 \quad t_f := 1.57in$



Girder loading diagram, Part 2, Beam Diagrams and Formulas, AISC 1997 (Ref. 2.2.5)

P_{G12} reaction from girder G7 (each side of girder)

$L_{G12} := 29ft$ $\quad trib_{G12} := 6.5ft$ — Max girder span

$\omega_{G12} := BMax_{fl} \cdot trib_{G12}$ $\quad \omega_{G12} = 4.06\,klf$ — Uniform girder loading for flexure

$\omega_{G12v} := VMax_{fl} \cdot trib_{G12}$ $\quad \omega_{G12v} = 4.6\,klf$ — Uniform girder loading for shear

$\omega_{G12const} := S19 \cdot trib_{G12}$ $\quad \omega_{G12const} = 2.4\,klf$ — Uniform girder construction loading

$P_{G12} := 2 \cdot c_{G7} \cdot P_{G7}$ $\quad P_{G12} = 176.7\,kip$ — Concentrated beam load for flexure (see girder G7 design)

$P_{G12v} := 2 \cdot c_{G7} \cdot P_{G7v}$ $\quad P_{G12v} = 201.9\,kip$ — Concentrated beam load for shear

$P_{G12const} := 2c_{G7} \cdot P_{G7const}$ $\quad P_{G12const} = 104.3\,kip$ — Concentrated beam construction load

$a1_{G12} := 0.125$
$c1_{G12} := 0.5$
$e1_{G12} := 0.013$

Coefficients for uniform load on beam, AISC Beam Diagrams and Formulas; Table of concentrated load equivalents, Case oo (Ref. 2.2.5)

$a2_{G12} := 0.25$
$c2_{G12} := 0.5$
$e2_{G12} := 0.0208$

Coefficients for single concentrated load on beam, AISC Beam Diagrams and Formulas; Table of concentrated load equivalents, Case 2 (Ref. 2.2.5)

$M_{g12} := a1_{G12} \cdot (\omega_{G12} \cdot L_{G12}) \cdot L_{G12} + a2_{G12} \cdot P_{G12} \cdot L_{G12}$ $\quad M_{g12} = 1708.1\,k \cdot ft$ — Girder moment

$M_{G12const} := a1_{G12} \cdot (\omega_{G12const} \cdot L_{G12}) \cdot L_{G12} + (a2_{G12} \cdot P_{G12const} \cdot L_{G12}) + \frac{C_p \cdot L_{G12}}{4}$ — Girder moment w/ construction loading

$M_{G12const} = 1044.1\,k \cdot ft$

$V_{g12} := c1_{G12} \cdot (\omega_{G12v} \cdot L_{G12}) + c2_{G12} \cdot P_{G12v}$ $\quad V_{g12} = 168.3\,kip$ — Girder shear

$V_{g12const} := c1_{G12} \cdot (\omega_{G12const} \cdot L_{G12}) + (c2_{G12} \cdot P_{G12const}) + C_p$ $\quad V_{g12const} = 91.9\,kip$ — Girder shear w/ construction loading

$M_{G12} := \max(M_{g12}, M_{G12const})$ $M_{G12} = 1708.1\,k\cdot ft$ max girder beam moment

$V_{G12} := \max(V_{g12}, V_{g12const})$ $V_{G12} = 168.3\,kip$ max girder beam shear

Allowable shear stress per ANSI/AISC N690-1994 section Q1.5.1.2.1 (Ref. 2.2.6)

$F_v := 0.4\cdot F_y$ $F_v = 20\,ksi$ allowable shear stress

$f_v := \frac{V_{G12}}{d\cdot t_w}$ $f_v = 5.2\,ksi$ calculated shear stress

$DC_f := \frac{M_{G12}}{M_r}$ $DC_f = 0.77$ flexure demand/capacity ratio

$DC_v := \frac{f_v}{F_v}$ $DC_v = 0.26$ shear demand/capacity ratio

$\Delta_{G12} := \frac{e1_{G12}\cdot(\omega_{G12}\cdot L_{G12})\cdot L_{G12}^3}{E_s\cdot I} + \left(\frac{e2_{G12}\cdot P_{G12}\cdot L_{G12}^3}{E_s\cdot I}\right)$ $\Delta_{G12} = 0.504\,in$ girder deflection

$\Delta_{allow} := \frac{L_{G12}}{240}$ $\Delta_{allow} = 1.45\,in$ allowable girder deflection

$\frac{\Delta_{G12}}{\Delta_{allow}} = 0.32$ deflection demand/capacity ratio

$shear_check := if(f_v \le F_v, "OK", "N.G!")$ shear_check = "OK"

$moment_check := if(M_{G12} \le M_r, "OK", "N.G!")$ moment_check = "OK"

$deflection_check := if(\Delta_{G12} \le \Delta_{allow}, "OK", "N.G!")$ deflection_check = "OK"

$depth_check1 := if\left(d \ge \frac{F_y\cdot L_{G12}}{800ksi}, "OK", "N.G!"\right)$ ANSI/AISC N690-1994 section Q1.13, CQ1.13 (Ref. 2.2.6) depth_check1 = "OK"

$depth_check2 := if\left(d \ge \frac{L_{G12}}{20}, "OK", "N.G!"\right)$ ANSI/AISC N690-1994 section Q1.13, CQ1.13 (Ref. 2.2.6) depth_check2 = "OK"

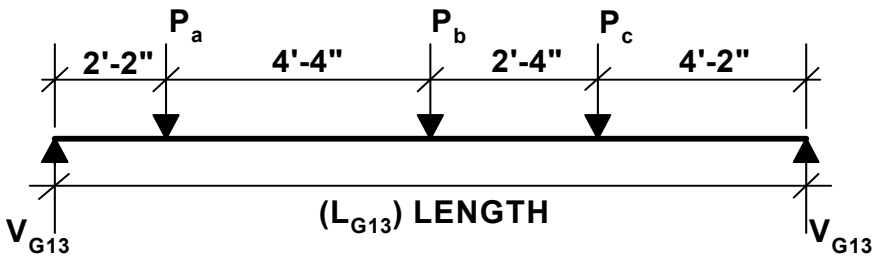
FLOOR GIRDER G12 USE: G12 = "W36x232"

FLOOR GIRDER G13

FLOOR ELEV. 32'-0" - Girder Design at opening between Gridline 8 to 9, from D to E and G to H.
Nominal concrete slab thickness = 1'-6" (Attachment A-1)

BEAM PROPERTIES per AISC 1997 (Ref. 2.2.5)

$G13 := \text{"W18x35"}$ $M_r := 158k\cdot ft$ $d := 17.70in$ $t_w := 0.30in$ $I := 510in^4$ $t_f := 0.425in$



Girder loading diagram, Part 2, Beam Diagrams and Formulas, AISC 1997 (Ref. 2.2.5)

$L_{G13} := 13ft$ Max girder span

$P_a := BMax_{fl}\cdot\left(\frac{2.167ft + 1ft}{2}\right)\cdot\left(\frac{6.33ft}{2}\right)$ $P_a = 3.13\,kip$ Concentrated beam load for flexure

$P_b := BMax_{fl}\cdot\left(\frac{L_{G13}}{2}\right)\cdot\left(\frac{L11}{2}\right)$ $P_b = 15.6\,kip$ Concentrated beam load for flexure

$P_c := BMax_{fl}\cdot\left(\frac{4.167ft + 1ft}{2}\right)\cdot\left(\frac{6.33ft}{2}\right)$ $P_c = 5.11\,kip$ Concentrated beam load for flexure

$P_{av} := VMax_{fl}\cdot\left(\frac{2.167ft + 1ft}{2}\right)\cdot\left(\frac{6.33ft}{2}\right)$ $P_{av} = 3.58\,kip$ Concentrated beam load for shear

$P_{bv} := VMax_{fl}\cdot\left(\frac{L_{G13}}{2}\right)\cdot\left(\frac{L11}{2}\right)$ $P_{bv} = 17.8\,kip$ Concentrated beam load for shear

$P_{cv} := VMax_{fl}\cdot\left(\frac{4.167ft + 1ft}{2}\right)\cdot\left(\frac{6.33ft}{2}\right)$ $P_{cv} = 5.84\,kip$ Concentrated beam load for shear

$P_{a_const} := S19\cdot\left(\frac{2.167ft + 1ft}{2}\right)\cdot\left(\frac{6.33ft}{2}\right)$ $P_{a_const} = \blacksquare\,kip$ Construction beam load

$P_{b_const} := S19\cdot\left(\frac{L_{G13}}{2}\right)\cdot\left(\frac{L11}{2}\right) + C_p$ $P_{b_const} = 14.2\,kip$ Construction beam load

$P_{c_const} := S19\cdot\left(\frac{4.167ft + 1ft}{2}\right)\cdot\left(\frac{6.33ft}{2}\right)$ $P_{c_const} = 3.02\,kip$ Construction beam load

$$M_{g13} := \left[\frac{6.5\cdot ft\cdot\left[P_a\cdot(10.833\cdot ft) + P_b\cdot(6.5\cdot ft) + P_c\cdot(4.167\cdot ft)\right]}{L_{G13}}\right] - P_a\cdot(4.333ft) \qquad M_{g13} = 64.7\,ft\,kip$$ Girder moment

$$M_{G13const} := \left[\frac{6.5\cdot ft\cdot\left[P_{a_const}\cdot(10.833\cdot ft) + P_{b_const}\cdot(6.5\cdot ft) + P_{c_const}\cdot(4.167\cdot ft)\right]}{L_{G13}}\right] - P_{a_const}\cdot(4.333ft)$$

$M_{G13const} = 54.4\,k\cdot ft$ Girder Construction moment

$V_{g13} := \frac{P_a \cdot (10.833 \cdot ft) + P_b \cdot (6.5 \cdot ft) + P_c \cdot (4.167 \cdot ft)}{L_{G13}}$ $V_{g13} = 12\,kip$ Girder shear

$V_{g13const} := \frac{P_{a_const} \cdot (10.833 \cdot ft) + P_{b_const} \cdot (6.5 \cdot ft) + P_{c_const} \cdot (4.167 \cdot ft)}{L_{G13}}$ $V_{g13const} = 9.6\,kip$ Girder construction shear

$M_{G13} := \max(M_{g13}, M_{G13const})$ $M_{G13} = 64.7\,k \cdot ft$ max girder beam moment

$V_{G13} := \max(V_{g13}, V_{g13const})$ $V_{G13} = 12\,kip$ max girder beam shear

Allowable shear stress per ANSI/AISC N690-1994 section Q1.5.1.2.1 (Ref. 2.2.6)

$F_v := 0.4 \cdot F_y$ $F_v = 20\,ksi$ allowable shear stress

$f_v := \frac{V_{G13}}{d \cdot t_w}$ $f_v = 2.3\,ksi$ calculated shear stress

$DC_f := \frac{M_{G13}}{M_r}$ $DC_f = 0.41$ flexure demand/capacity ratio

$DC_v := \frac{f_v}{F_v}$ $DC_v = 0.11$ shear demand/capacity ratio

$P_{tot} := \left(\frac{S1_c}{BMax_{fl}}\right) \cdot (P_a + P_b + P_c)$ $P_{tot} = 17.3\,kip$ total (D + L) loading

$X_{dist} := \frac{P_a \cdot 2.167ft + P_b \cdot 6.5ft + P_c \cdot 8.833ft}{P_a + P_b + P_c}$ $X_{dist} = 6.4\,ft$ centroid of loading

$\Delta_{G13} := \frac{P_{tot} \cdot X_{dist}^2 \cdot (L_{G13} - X_{dist})^2}{3 \cdot E_s \cdot I \cdot L_{G13}}$ $\Delta_{G13} = 0.09\,in$ girder (D + L) deflection

$\Delta_{allow} := \frac{L_{G13}}{240}$ $\Delta_{allow} = 0.65\,in$ allowable girder deflection

$\frac{\Delta_{G13}}{\Delta_{allow}} = 0.14$ deflection demand/capacity ratio

$shear_check := if(f_v \le F_v, "OK", "N.G!")$ shear_check = "OK"

$moment_check := if(M_{G13} \le M_r, "OK", "N.G!")$ moment_check = "OK"

$deflection_check := if(\Delta_{G13} \le \Delta_{allow}, "OK", "N.G!")$ deflection_check = "OK"

$depth_check1 := if\left(d \ge \frac{F_y \cdot L_{G13}}{800ksi}, "OK", "N.G!"\right)$ ANSI/AISC N690-1994 section Q1.13, CQ1.13 (Ref. 2.2.6) depth_check1 = "OK"

$depth_check2 := if\left(d \ge \frac{L_{G13}}{20}, "OK", "N.G!"\right)$ ANSI/AISC N690-1994 section Q1.13, CQ1.13 (Ref. 2.2.6) depth_check2 = "OK"

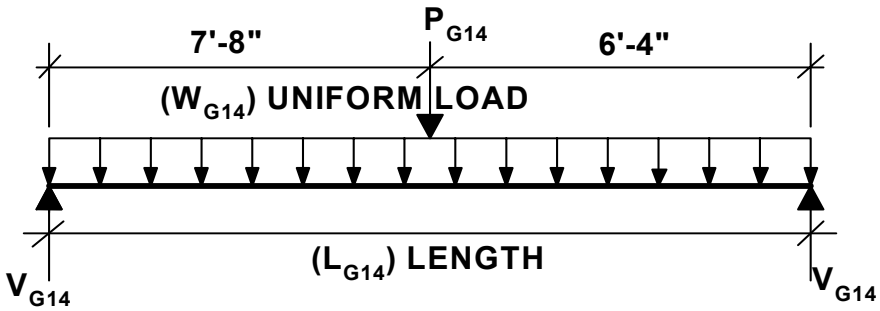
FLOOR GIRDER G13 USE: G13 = "W18x35"

FLOOR GIRDER G14

FLOOR ELEV. 32'-0" - Girder Design between Gridline 2 to 4, and 6 to 9, from D to E and G to H.
Nominal concrete slab thickness = 1'-6" (Attachment A-1)

BEAM PROPERTIES per AISC 1997 (Ref. 2.2.5)

$G14 := \text{"W21x44"}$ $M_r := 224k\cdot ft$ $d := 20.66in$ $t_w := 0.35in$ $I := 843in^4$ $t_f := 0.45in$



Girder loading diagram, Part 2, Beam Diagrams and Formulas, AISC 1997 (Ref. 2.2.5)

P_{G14} reaction from girder G13 (one side of girder)

$L_{G14} := 14ft$	$trib_{G14} := 6.5ft$	Max girder span
$\omega_{G14} := BMax_{fl}\cdot trib_{G14}$	$\omega_{G14} = 4.06\,klf$	Uniform girder loading for flexure
$\omega_{G14v} := VMax_{fl}\cdot trib_{G14}$	$\omega_{G14v} = 4.6\,klf$	Uniform girder loading for shear
$\omega_{G14const} := S19\cdot trib_{G14}$	$\omega_{G14const} = 2.4\,klf$	Uniform girder construction loading
$P_{G14} := V_{g13}$	$P_{G14} = 12\,kip$	Concentrated beam load for flexure (see girder G13 design)
$P_{G14v} := \left(\frac{1.6}{1.4}\right)\cdot V_{g13}$	$P_{G14v} = 13.8\,kip$	Concentrated beam load for shear
$P_{G14const} := V_{g13const}$	$P_{G14const} = 9.6\,kip$	Concentrated beam construction load

$$M_{g14} := \frac{\omega_{G14}\cdot(7.667\cdot ft)}{2}\cdot\left(L_{G14} - 7.667ft\right) + \frac{P_{G14}\cdot(7.667\cdot ft)\cdot(6.333\cdot ft)}{L_{G14}}$$

$M_{g14} = 140.4\,k\cdot ft$ Girder moment

$$M_{g14const} := \frac{\omega_{G14const}\cdot(7.667\cdot ft)}{2}\cdot\left(L_{G14} - 7.667ft\right) + \frac{P_{G14const}\cdot(7.667\cdot ft)\cdot(6.333\cdot ft)}{L_{G14}}$$

Girder moment w/ construction loading

$M_{g14const} = 91.5\,k\cdot ft$

$$V_{g14} := \frac{\omega_{G14}\cdot L_{G14}}{2} + \frac{P_{G14}\cdot(7.667ft)}{L_{G14}}$$

$V_{g14} = 35\,kip$ reaction load for Girder, see G15

$$V_{g14v} := \frac{\omega_{G14v}\cdot L_{G14}}{2} + \frac{P_{G14v}\cdot(7.667ft)}{L_{G14}}$$

$V_{g14v} = 40\,kip$ Girder shear

$V_{g14const} := \frac{\omega_{G14const} \cdot L_{G14}}{2} + \frac{P_{G14const} \cdot (7.667ft)}{L_{G14}} + C_p$ $V_{g14const} = 27\,kip$ Girder shear w/ construction loading

$M_{G14} := \max(M_{g14}, M_{g14const})$ $M_{G14} = 140.4\,k \cdot ft$ max girder beam moment

$V_{G14} := \max(V_{g14v}, V_{g14const})$ $V_{G14} = 40\,kip$ max girder beam shear

Allowable shear stress per ANSI/AISC N690-1994 section Q1.5.1.2.1 (Ref. 2.2.6)

$F_v := 0.4 \cdot F_y$ $F_v = 20\,ksi$ allowable shear stress

$f_v := \frac{V_{G14}}{d \cdot t_w}$ $f_v = 5.5\,ksi$ calculated shear stress

$DC_f := \frac{M_{G14}}{M_r}$ $DC_f = 0.63$ flexure demand/capacity ratio

$DC_v := \frac{f_v}{F_v}$ $DC_v = 0.28$ shear demand/capacity ratio

max deflection at $x := \sqrt{\frac{7.667 \cdot [7.667 + 2 \cdot (6.333)]}{3}}\,ft$ $x = 7.21\ ft$ $a := 7.667ft$ distance to load from left
$b := 6.333ft$ distance to load from right

$\Delta_{G14} := \frac{\omega_{G14} \cdot x}{24 \cdot E_s \cdot I} \cdot \left(L_{G14}^3 - 2 \cdot L_{G14} \cdot x^2 + x^3\right) + \left[\frac{P_{G14} \cdot a \cdot b \cdot (a + 2 \cdot b) \cdot \sqrt{3a(a + 2b)}}{27 E_s \cdot I \cdot L_{G14}}\right]$ $\Delta_{G14} = 0.192\,in$ girder deflection

$\Delta_{allow} := \frac{L_{G14}}{240}$ $\Delta_{allow} = 0.7\,in$ allowable girder deflection $\frac{\Delta_{G14}}{\Delta_{allow}} = 0.27$ deflection demand/capacity ratio

$shear_check := if(f_v \le F_v, "OK", "N.G!")$ shear_check = "OK"

$moment_check := if(M_{G14} \le M_r, "OK", "N.G!")$ moment_check = "OK"

$deflection_check := if(\Delta_{G14} \le \Delta_{allow}, "OK", "N.G!")$ deflection_check = "OK"

$depth_check1 := if\left(d \ge \frac{F_y \cdot L_{G14}}{800ksi}, "OK", "N.G!"\right)$ ANSI/AISC N690-1994 section Q1.13, CQ1.13 (Ref. 2.2.6) depth_check1 = "OK"

$depth_check2 := if\left(d \ge \frac{L_{G14}}{20}, "OK", "N.G!"\right)$ ANSI/AISC N690-1994 section Q1.13, CQ1.13 (Ref. 2.2.6) depth_check2 = "OK"

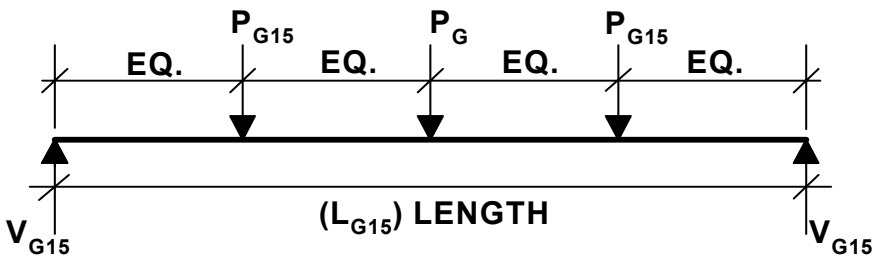
FLOOR GIRDER G14 USE: G14 = "W21x44"

FLOOR GIRDER G15

FLOOR ELEV. 32'-0" - Girder Design between Gridline 8 to 9, from D to E and G to H.
Nominal concrete slab thickness = 1'-6" (Attachment A-1)

BEAM PROPERTIES per AISC 1997 (Ref. 2.2.5)

$G15 := \text{"W33X118"}$ $M_r := 987k\cdot ft$ $d := 32.86in$ $t_w := 0.55in$ $I := 5900in^4$ $t_f := 0.740in$



3 equally spaced concentrated loads; Girder loading diagram, Part 2, Beam Diagrams and Formulas, AISC 1997 (Ref. 2.2.5)

P_{G15} reaction from beam B8 and G14
P_G reaction from beam B8 and B11

$L_{G15} := 26ft$ — Max girder span

$P_{G15} := \left(\omega_{b8}\cdot\frac{L8}{2}\right) + V_{g14}$ $P_{G15} = 64.5\,kip$ — Concentrated beam load for bending (see beam B8 and G14 design)

$P_{G15v} := \left(\omega_{b8v}\cdot\frac{L8}{2}\right) + V_{g14v}$ $P_{G15v} = 73.7\,kip$ — Concentrated beam load for shear

$P_{G15const} := \left(\omega_{b8_const}\cdot\frac{L8}{2}\right) + V_{g14const}$ $P_{G15const} = 44.4\,kip$ — Concentrated beam construction load

$P_G := \left(\omega_{b8}\cdot\frac{L8}{2}\right) + V_{b11}$ $P_G = 47.3\,kip$ — Concentrated beam load for bending (see beam B8 and B11 design)

$P_{Gv} := \left(\omega_{b8v}\cdot\frac{L8}{2}\right) + V_{b11v}$ $P_{Gv} = 51.5\,kip$ — Concentrated beam load for shear

$P_{Gconst} := \left(\omega_{b8_const}\cdot\frac{L8}{2}\right) + V_{b11_const}$ $P_{G15const} = 44.4\,kip$ — Concentrated beam construction load

$M_{g15} := \left(\frac{P_G\cdot L_{G15}}{4}\right) + P_{G15}\cdot 6.5ft$ $M_{g15} = 726.2\,k\cdot ft$ — Girder moment

$M_{g15const} := \left(\frac{P_{G15const}\cdot L_{G15}}{4}\right) + P_{Gconst}\cdot 6.5ft$

$M_{g15const} = 493.9\,k\cdot ft$ — Girder moment w/ construction loading

$V_{g15} := P_{G15v} + \frac{P_{Gv}}{2}$ $V_{g15} = 99.4\,kip$ — Girder shear

$V_{g15const} := P_{G15const} + \frac{P_{Gconst}}{2} + C_p$ $V_{g15const} = 65.2\,kip$ — Girder shear w/ construction loading

$M_{G15} := \max(M_{g15}, M_{g15const})$ $M_{G15} = 726.2\,k\cdot ft$ — max girder beam moment

$V_{G15} := \max(V_{g15}, V_{g15const})$ $V_{G15} = 99.4\,kip$ max girder beam shear

Allowable shear stress per ANSI/AISC N690-1994 section Q1.5.1.2.1 (Ref. 2.2.6)

$F_v := 0.4 \cdot F_y$ $F_v = 20\,ksi$ allowable shear stress

$f_v := \frac{V_{G15}}{d \cdot t_w}$ $f_v = 5.5\,ksi$ calculated shear stress

$DC_f := \frac{M_{G15}}{M_r}$ $DC_f = 0.74$ flexure demand/capacity ratio

$DC_v := \frac{f_v}{F_v}$ $DC_v = 0.28$ shear demand/capacity ratio

$\Delta_{G15} := \frac{S1_c}{BMax_{fl}} \cdot \left[\left(\frac{P_{G15} \cdot 6.5 \cdot ft}{24 \cdot E_s \cdot I} \right) \cdot \left[3 \cdot L_{G15}^2 - 4 \cdot (6.5ft)^2 \right] + \frac{P_G \cdot L_{G15}^3}{48 E_s \cdot I} \right]$ $\Delta_{G15} = 0.365\,in$ max (D + L) girder deflection

$\Delta_{allow} := \frac{L_{G15}}{240}$ $\Delta_{allow} = 1.3\,in$ allowable girder deflection (Ref. 2.2.4)

$\frac{\Delta_{G15}}{\Delta_{allow}} = 0.28$ deflection demand/capacity ratio

$shear_check := if(f_v \le F_v, "OK", "N.G!")$ shear_check = "OK"

$moment_check := if(M_{G15} \le M_r, "OK", "N.G!")$ moment_check = "OK"

$deflection_check := if(\Delta_{G15} \le \Delta_{allow}, "OK", "N.G!")$ deflection_check = "OK"

$depth_check1 := if\left(d \ge \frac{F_y \cdot L_{G15}}{800ksi}, "OK", "N.G!"\right)$ ANSI/AISC N690-1994 section Q1.13, CQ1.13 (Ref. 2.2.6) depth_check1 = "OK"

$depth_check2 := if\left(d \ge \frac{L_{G15}}{20}, "OK", "N.G!"\right)$ ANSI/AISC N690-1994 section Q1.13, CQ1.13 (Ref. 2.2.6) depth_check2 = "OK"

FLOOR GIRDER G15 USE: G15 = "W33X118"

6.5 INTERIOR COLUMN CALCULATIONS, Ref. 2.2.5, & 2.2.6

INTERIOR COLUMNS (COLUMNS SIZES SHOWN ON PAGE 67)

AISC N690 (Table Q1.5.7.1 Ref. 2.2.6) allows a 1.5 stress increase with Extreme and Abnormal Severe load combinations for members with axial compression.Column loads are based on maximum shear values with a 1.4 factor. To adjust maximum loading for members with axial compression multiply maximum roof loading by 0.9333: (1.4/1.5 x $VMax_{rf}$ = 0.9333 x $VMax_{rf}$).

COLUMN C1 $C1 := \text{"W14x257"}$

COLUMNS AT GRIDLINES D.3-2.5, D.7-2.5, G.3-2.5, AND G.7-2.5, Elevation from 0'-0" to 64'-0". Supports second floor and roof framing members. (Attachment A-1, & A-2)

$H_{rf_c1} := \left[64\cdot ft - 1.5\cdot ft - \left(\frac{3.0\cdot ft}{2}\right)\right] - 32ft$ $H_{rf_c1} = 29\ ft$ Unbraced Column height from EL. 32'-0" to CL of W36 girder (G2) at roof (+64'-0")

$H_{fl_c1} := \left(32ft - 1.5ft - \frac{3.0ft}{2}\right)$ $H_{fl_c1} = 29\ ft$ Unbraced Column height from EL. 0'-0" to CL of W36 girder (G12) at floor (+32'-0")

$P_{rf_c1} := 2\cdot\left(\frac{1.4}{1.5}\right)\cdot\left(V_{G1} + V_{G2}\right)$ $P_{rf_c1} = 497.6\,kip$ max column load between 2nd floor and roof

$P_{fl_c1} := 2\cdot\left(\frac{1.4}{1.5}\right)\left(V_{G1} + V_{G2} + V_{G7} + V_{G12}\right)$ $P_{fl_c1} = 1000.3\,kip$ max column load between 1st and 2nd floor

$P_{allow_rf} := 1326kip$ allow column load between 2nd floor & roof H_{rf} = 30 ft. AISC ASD Column load tables (Ref. 2.2.5). Load tables are based on equations E2-1, and E2-2 which are equivalent to AISC N690 equations Q1.5-1, and Q1.5-2 respectively (Ref. 2.2.6)

$P_{allow_fl} := 1326kip$ allow column load between 1st & 2nd floor H_{rf} = 30 ft. AISC ASD Column load tables (Ref. 2.2.5)

$DC_p := \frac{P_{fl_c1}}{P_{allow_fl}}$ $DC_p = 0.75$ Axial load demand/capacity ratio

$column_check_roof := if\left(P_{allow_rf} \geq P_{rf_c1}, \text{"OK"}, \text{"N.G!"}\right)$ $column_check_roof = \text{"OK"}$

$column_check_floor := if\left(P_{allow_fl} \geq P_{fl_c1}, \text{"OK"}, \text{"N.G!"}\right)$ $column_check_floor = \text{"OK"}$

COLUMN C1 USE: $C1 = \text{"W14x257"}$

COLUMN C2 $C2 := \text{"W14x145"}$

COLUMNS AT GRIDLINES D.3-6.8, D.7-6.8, G.3-6.8, AND G.7-6.8, Elevation from 0'-0" to 32'-0". Support floor framing members. (Attachment A-1)

$H_{fl_c2} := \left(32ft - 1.5ft - \frac{3.0ft}{2}\right)$ $H_{fl_c2} = 29\ ft$ Unbraced Column height from EL. 0'-0" to CL of W36 girder (G12) at floor (+32'-0")

$P_{fl_c2} := 2\cdot\left(\frac{1.4}{1.5}\right)\cdot\left(V_{G7} + V_{G12}\right)$ $P_{fl_c2} = 502.6\,kip$ max column load between 1st and 2nd floor

$P_{allow_fl} := 719kip$ allow column load between 1st and 2nd floor H_{rf} = 30 ft. AISC ASD Column load tables (Ref. 2.2.5). Load tables are based on equations E2-1, and E2-2 which are equivalent to AISC N690 equations Q1.5-1, and Q1.5-2 respectively (Ref. 2.2.6)

$DC_p := \frac{P_{fl_c2}}{P_{allow_fl}}$ $DC_p = 0.7$ Axial load demand/capacity ratio

$column_check_floor := if\left(P_{allow_fl} \ge P_{fl_c2}, \text{"OK"}, \text{"N.G!"}\right)$ $column_check_floor = \text{"OK"}$

COLUMN C2 USE: $C2 = \text{"W14x145"}$

COLUMN C3 $C3 := \text{"W14x233"}$

COLUMNS AT GRIDLINES D.3-10, D.7-10, G.3-10, AND G.7-10, Elevation from 0'-0" to 64'-0". Supports second floor and roof framing members. (Attachment A-1, & A-2)

$H_{rf_c3} := \left[64\cdot ft - 1.5\cdot ft - \left(\frac{3.0\cdot ft}{2}\right)\right] - 32ft$ $H_{rf_c3} = 29\ ft$ Unbraced Column height from EL. 32'-0" to CL of W36 girder (G3) at roof (+64'-0")

$H_{fl_c3} := \left(32ft - 1.5ft - \frac{3.0ft}{2}\right)$ $H_{fl_c3} = 29\ ft$ Unbraced Column height from EL. 0'-0" to CL of W36 girder (G8) at floor (+32'-0")

$P_{rf_c3} := \left(\frac{1.4}{1.5}\right)\cdot\left(V_{b1} + 2V_{G1} + V_{G3}\right)$ $P_{rf_c3} = 485.8\,kip$ max column load between 2nd floor and roof

$P_{fl_c3} := P_{rf_c3} + \left(\frac{1.4}{1.5}\right)\cdot\left(V_{b8} + 2\cdot V_{G7} + V_{G8}\right)$

$P_{fl_c3} = 976.5\,kip$ max column load between 1st and 2nd floor

$P_{allow_rf} := 1192kip$ allow column load between 2nd floor & roof H_{rf} = 30 ft. AISC ASD Column load tables (Ref. 2.2.5). Load tables are based on equations E2-1, and E2-2 which are equivalent to AISC N690 equations Q1.5-1, and Q1.5-2 respectively (Ref. 2.2.6)

$P_{allow_fl} := 1192kip$ allow column load between 1st & 2nd floor H_{rf} = 30 ft. AISC ASD Column load tables (Ref. 2.2.5)

$DC_p := \frac{P_{fl_c3}}{P_{allow_fl}}$ $DC_p = 0.82$ Axial load demand/capacity ratio

$column_check_roof := if\left(P_{allow_rf} \ge P_{rf_c3}, \text{"OK"}, \text{"N.G!"}\right)$ $column_check_roof = \text{"OK"}$

$column_check_floor := if\left(P_{allow_fl} \ge P_{fl_c3}, \text{"OK"}, \text{"N.G!"}\right)$ $column_check_floor = \text{"OK"}$

COLUMN C3 USE: $C3 = \text{"W14x233"}$

COLUMN C4 $C4 := \text{"W14x311"}$

COLUMNS AT GRIDLINES E.3-10, F-10, AND F.7-10, Elevation from 0'-0" to 32'-0". Supports second floor and roof framing members. (Attachment A-1)

$H_{fl_c4} := \left(32ft - 1.5ft - \frac{3.0ft}{2}\right)$ $H_{fl_c4} = 29\ ft$ Unbraced Column height from EL. 0'-0" to CL of W36 girder (G11) at floor (+32'-0")

$trib_{rf} := \left(\frac{12 \cdot ft}{2} + \frac{17.75 \cdot ft}{2}\right) \cdot \left(\frac{32ft}{2} + \frac{32ft}{2}\right)$ $trib_{rf} = 476\ ft^2$ roof tributary area

$trib_{flr} := \left(\frac{13 \cdot ft}{2}\right) \cdot \left(\frac{32 \cdot ft}{2} + \frac{32 \cdot ft}{2}\right)$ $trib_{flr} = 208\ ft^2$ floor tributary area

$P_{roof} := VMax_{rf} \cdot trib_{rf}$ $P_{roof} = 336.6\,kip$ Axial roof load for shear (elev. +72.00 ft)

$P_{floor} := VMax_{fl} \cdot trib_{flr}$ $P_{floor} = 148.6\,kip$ Axial roof load for flexure (elev. +32.00 ft)

$P_{wall} := \gamma_c \cdot 2 \cdot ft \cdot (72ft - 32ft) \cdot 32ft$ $P_{wall} = 384\,kip$ Axial load from 2'-0" thick concrete wall full height from floor to roof

$E_{wall} := \left(\frac{P_{wall} \cdot S_{aV} \cdot Amp}{1.4}\right)$ $E_{wall} = 449.8\,kip$ wall seismic axial load

$P_{fl_c4} := \left(\frac{1.4}{1.5}\right) \cdot \left(P_{roof} + P_{floor} + P_{wall} + E_{wall}\right)$

$P_{fl_c4} = 1231.1\,kip$ max column load between 1st and 2nd floor

$P_{allow_fl} := 1631kip$ allow column load between 1st & 2nd floor H_{rf} = 30 ft. AISC ASD Column load tables (Ref. 2.2.5). Load tables are based on equations E2-1, and E2-2 which are equivalent to AISC N690 equations Q1.5-1, and Q1.5-2 respectively (Ref. 2.2.6)

$DC_p := \frac{P_{fl_c4}}{P_{allow_fl}}$ $DC_p = 0.75$ Axial load demand/capacity ratio

$column_check_floor := if\left(P_{allow_fl} \geq P_{fl_c4}, \text{"OK"}, \text{"N.G!"}\right)$ $column_check_floor = \text{"OK"}$

COLUMN C4 USE: $C4 = \text{"W14x311"}$

COLUMN C5 $C5 := \text{"W14x176"}$

COLUMNS AT GRIDLINES D.3-11, D.7-11, G.3-11, AND G.7-11, Elevation from 32'-0" to 64'-0". Supports roof framing members. (Attachment A-2)

$H_{rf_c5} := \left[64 \cdot ft - 1.5 \cdot ft - \left(\frac{3.0 \cdot ft}{2}\right)\right] - 32ft$ $H_{rf_c5} = 29\ ft$ Unbraced Column height from EL. 32'-0" to CL of W36 girder (G3) at roof (+64'-0")

$P_{rf_c5} := 2 \cdot \left(\frac{1.4}{1.5}\right) \cdot \left(V_{G1} + V_{G3}\right)$ $P_{rf_c5} = 722.9\,kip$ max column load between 2nd floor and roof

$P_{allow_rf} := 882kip$

allow column load between 2nd floor & roof H_{rf} = 30 ft. AISC ASD Column load tables (Ref. 2.2.5). Load tables are based on equations E2-1, and E2-2 which are equivalent to AISC N690 equations Q1.5-1, and Q1.5-2 respectively (Ref. 2.2.6)

$DC_p := \frac{P_{rf_c5}}{P_{allow_rf}}$ $DC_p = 0.82$ Axial load demand/capacity ratio

$column_check_roof := if\left(P_{allow_rf} \geq P_{rf_c5}, "OK", "N.G!"\right)$

column_check_roof = "OK"

COLUMN C5 USE: C5 = "W14x176"

FRAMING SUMMARY

ROOF BEAMS	ROOF GIRDERS	FLOOR BEAMS	FLOOR GIRDERS	COLUMNS
B1 = "W16X31"	G1 = "W30X116"	B8 = "W18X35"	G7 = "W33X118"	C1 = "W14x257"
B2 = "W24X68"	G2 = "W36x210"	B9 = "W18X40"	G8 = "W36x393"	C2 = "W14x145"
B3 = "W24X55"	G3 = "W36x359"	B10 = ▮	G9 = "W33X130"	C3 = "W14x233"
B4 = "W18X40"	G4 = "W36x300"	B11 = "W18X35"	G10 = "W36x359"	C4 = "W14x311"
B5 = "W12X30"	G5 = "W36x393"		G11 = "W30X235"	C5 = "W14x176"
B6 = "W18X55"	G6 = "W36X359"		G12 = "W36x232"	
B7 = "W21X44"			G13 = "W18x35"	
			G14 = "W21x44"	
			G15 = "W33X118"	

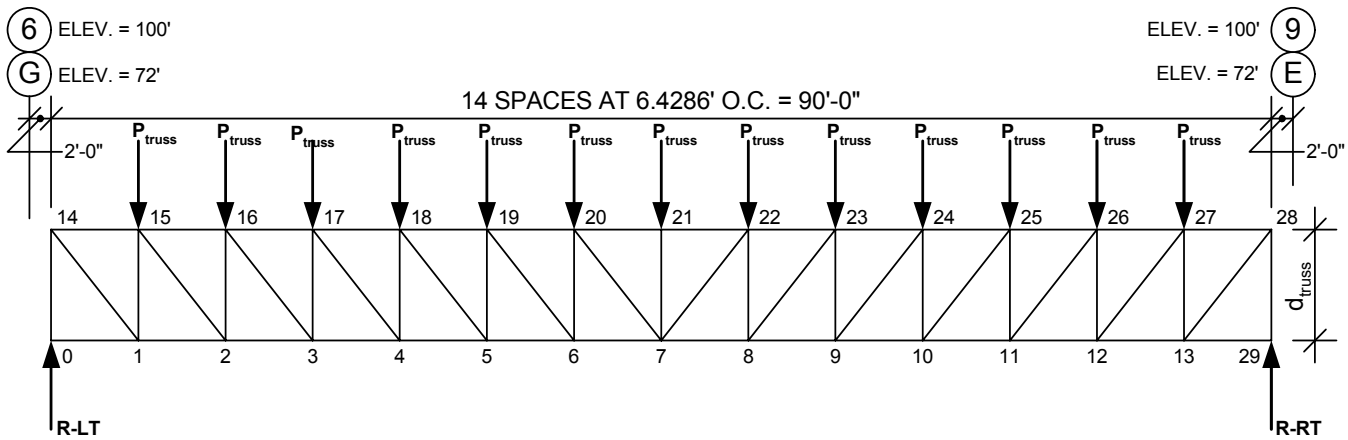
NOTE: SEE RESULTS SECTION 7.1 FOR SHEAR AND FLEXURE DEMAND/CAPACITY RATIOS

6.6 NOT USED

6.7 TRUSS T1 CALCULATION, Ref. 2.2.5, & 2.2.6

TRUSS T1 CALCULATION (Total of 14 T1 Trusses required)

ROOF ELEV. 72'-0" - Truss Design between Gridline G to E, from 10 to 12, and
ROOF ELEV. 100'-0" - Truss Design between Gridline 6 to 9, from D to H.
Nominal concrete slab thickness = 1'-6" (Attachment A-3)



TRUSS T1 P_{truss} loading from beam B6 (each side of truss) Elev. +100'-0", and B4 at Elev. +72'-0" (each side of truss). Truss at elevation 100' is analyzed and designed here. The truss design at +72'-0" is conservative because the beam reaction from B6 is greater than B4, (B6 = 47.6k) > (B4 = 39.2k).

$L_{truss} := 90ft$ truss length

$spacing := \frac{L_{truss}}{14}$ $spacing = 6.4\,ft$ panel point spacing

$trib_{t1} := 21.1667ft$ tributary loading (T1 Grids 6-9/D-H)

TOP & BOTTOM CHORD PROPERTIES

$TC := "W14X342"$ $BC := "W14X342"$

$A := 101.0in^2$ $d := 17.54in$ $t_w := 1.540in$ $t_f := 2.470in$ $b_f := 16.360in$

$I_x := 4900in^4$ $S_x := 559in^3$ $r_x := 6.98in$

$I_y := 1810in^4$ $S_y := 221in^3$ $r_y := 4.24in$

WEB MEMBER PROPERTIES

$WEB := "W12X170"$

$A_{web} := 50.0in^2$ $d_{web} := 14.03in$ $t_{wweb} := 0.960in$ $t_{fweb} := 1.560in$ $b_{fweb} := 12.570in$

$I_{xweb} := 1650in^4$ $S_{xweb} := 235in^3$ $r_{xweb} := 5.74in$

$I_{yweb} := 517in^4$ $S_{yweb} := 82.3in^3$ $r_{yweb} := 3.22in$

$d_{truss} := 78.46in$ — truss depth center of top chord to center of bottom chord (overall depth D = 8'-0")

$h := \sqrt{spacing^2 + d_{truss}^2}$ $\quad h = 9.2\,ft$ — diagonal web length

$D := d_{truss} + 0.5\cdot(d + d)$ $\quad D = 8.00\,ft$ — maximum outer dimensions of truss

AISC N690 (Table Q1.5.7.1 Ref. 2.2.6) allows a 1.6 stress increase with Extreme and Abnormal Severe load combinations (1.4 for shear and 1.5 for axial compression Ref. 2.2.7). To adjust maximum loading for members with axial compression multiply maximum roof loading by 1.0667: (1.6/1.5 x $BMax_{rf}$ = 1.0667 x $BMax_{rf}$).

$P_{truss} := \left(\frac{1.6}{1.5}\right)\cdot\left(BMax_{rf}\cdot trib_{t1} + D_{gird}\right)\cdot spacing$ $\quad P_{truss} = 96.7\,kip$ — panel joint loading for flexure
D_{gird} = assumed self wt.
(Assumption 3.1.4)
(see beam B6 design)

$P_{truss_const} := \left(S19\cdot trib_{t1} + D_{gird}\right)\cdot spacing$ $\quad P_{truss_const} = 56.6\,kip$ — panel joint construction loading

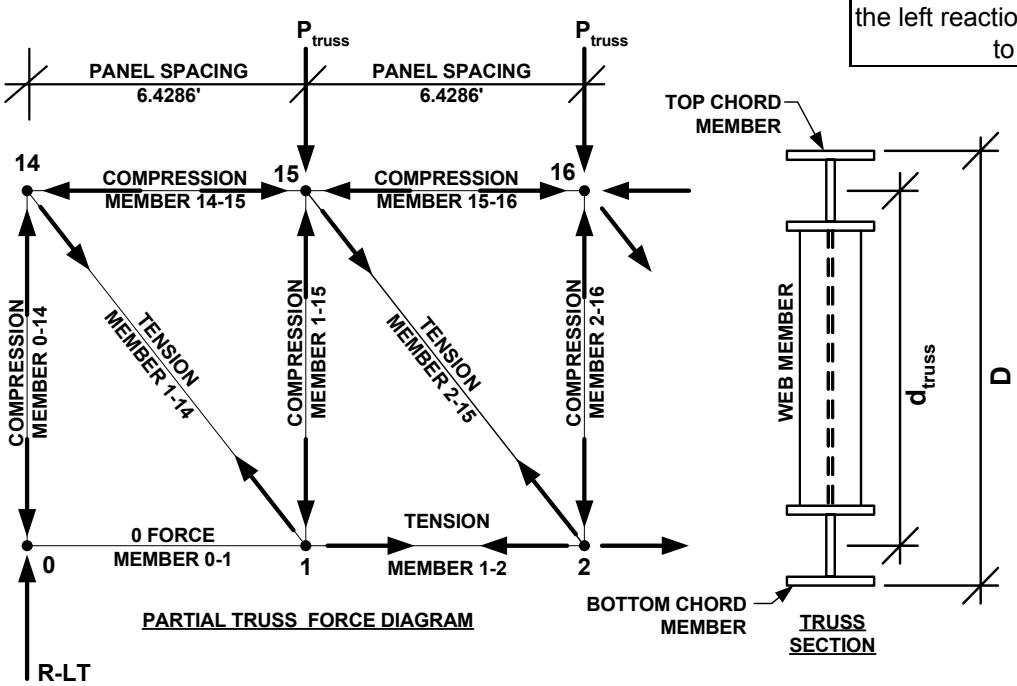
$P_{truss_v} := \left(VMax_{rf}\cdot trib_{t1} + D_{gird}\right)\cdot spacing$ $\quad P_{truss_v} = 102.7\,kip$ — panel joint loading for shear

$R_{LT} := P_{truss}\cdot 6.5$ $\quad R_{LT} = 628.3\,kip$ — truss left end reaction from flexure loading

$R_{LT_const} := P_{truss_const}\cdot 6.5$ $\quad R_{LT_const} = 367.9\,kip$ — truss left end reaction from construction loading

$R_{LT_v} := P_{truss_v}\cdot 6.5$ $\quad R_{LT_v} = 667.2\,kip$ — truss left end reaction from shear loading

Member axial forces are calculated by the joint equilibrium approach starting at the left reaction node 0 and then progressing to the center of truss



MEMBER AXIAL FORCES

$M_{0_14} := R_{LT}$ $M_{0_14} = 628.3\,kip$ compression

$M_{1_14} := \dfrac{R_{LT}}{\dfrac{d_{truss}}{h}}$ $M_{1_14} = 881.2\,kip$ tension

$M_{14_15} := R_{LT} \cdot \dfrac{spacing}{d_{truss}}$ $M_{14_15} = 617.8\,kip$ compression

$M_{1_15} := M_{1_14} \cdot \left(\dfrac{d_{truss}}{h}\right)$ $M_{1_15} = 628.3\,kip$ compression

$M_{1_2} := M_{1_14} \cdot \left(\dfrac{spacing}{h}\right)$ $M_{1_2} = 617.8\,kip$ tension

$M_{2_15} := \dfrac{M_{1_15} - P_{truss}}{\dfrac{d_{truss}}{h}}$ $M_{2_15} = 745.6\,kip$ tension

$M_{15_16} := M_{14_15} + M_{2_15} \cdot \left(\dfrac{spacing}{h}\right)$ $M_{15_16} = 1140.5\,kip$ compression

$M_{2_3} := M_{1_2} + M_{2_15} \cdot \left(\dfrac{spacing}{h}\right)$ $M_{2_3} = 1140.5\,kip$ tension

$M_{2_16} := M_{2_15} \cdot \left(\dfrac{d_{truss}}{h}\right)$ $M_{2_16} = 531.7\,kip$ compression

$M_{3_16} := \dfrac{M_{2_16} - P_{truss}}{\dfrac{d_{truss}}{h}}$ $M_{3_16} = 610\,kip$ tension

$M_{16_17} := M_{15_16} + M_{3_16} \cdot \left(\dfrac{spacing}{h}\right)$ $M_{16_17} = 1568.2\,kip$ compression

$M_{3_4} := M_{2_3} + M_{3_16} \cdot \left(\dfrac{spacing}{h}\right)$ $M_{3_4} = 1568.2\,kip$ tension

$M_{3_17} := M_{3_16} \cdot \left(\dfrac{d_{truss}}{h}\right)$ $M_{3_17} = 435\,kip$ compression

$M_{4_17} := \dfrac{M_{3_17} - P_{truss}}{\dfrac{d_{truss}}{h}}$ $M_{4_17} = 474.5\,kip$ tension

$M_{17_18} := M_{16_17} + M_{4_17} \cdot \left(\dfrac{spacing}{h}\right)$ $M_{17_18} = 1900.834\,kip$ compression

$M_{4_5} := M_{3_4} + M_{4_17} \cdot \left(\dfrac{spacing}{h}\right)$ $M_{4_5} = 1900.834\,kip$ tension

$M_{4_18} := M_{4_17} \cdot \left(\dfrac{d_{truss}}{h}\right)$ $M_{4_18} = 338.3\,kip$ compression

$M_{5_18} := \dfrac{M_{4_18} - P_{truss}}{\dfrac{d_{truss}}{h}}$ $M_{5_18} = 338.9\,kip$ tension

$M_{18_19} := M_{17_18} + M_{5_18} \cdot \left(\dfrac{spacing}{h}\right)$ $M_{18_19} = 2138.438\,kip$ compression

$M_{5_6} := M_{4_5} + M_{5_18} \cdot \left(\dfrac{spacing}{h}\right)$ $M_{5_6} = 2138.44\,kip$ tension

$M_{5_19} := M_{5_18} \cdot \left(\dfrac{d_{truss}}{h}\right)$ $M_{5_19} = 241.7\,kip$ compression

$M_{6_19} := \dfrac{M_{5_19} - P_{truss}}{\dfrac{d_{truss}}{h}}$ $M_{6_19} = 203.342\,kip$ tension

$M_{19_20} := M_{18_19} + M_{6_19} \cdot \left(\dfrac{spacing}{h}\right)$ $M_{19_20} = 2281\,kip$ compression

$M_{6_7} := M_{5_6} + M_{6_19} \cdot \left(\dfrac{spacing}{h}\right)$ $M_{6_7} = 2281\,kip$ tension

$M_{6_20} := M_{6_19} \cdot \left(\dfrac{d_{truss}}{h}\right)$ $M_{6_20} = 145\,kip$ compression

$M_{7_20} := \dfrac{M_{6_20} - P_{truss}}{\dfrac{d_{truss}}{h}}$ $M_{7_20} = 67.8\,kip$ tension

$M_{20_21} := M_{19_20} + M_{7_20} \cdot \left(\dfrac{spacing}{h}\right)$ $M_{20_21} = 2328.52\,kip$ compression

$M_{7_21} := P_{truss}$ $M_{7_21} = 96.7\,kip$ compression

Design top and bottom chord with 5000 lb occasional construction load at midpoint for bending. This is a conservative design as member axial force would be less using uniform construction loading (load combination S19). Design tension and compression webs for pure tension and compression.

To account for load reversal from Beyond Design Basis Ground Motion BDBGM design web elements for the greatest compression or tension force and the maximum web member length. For top and bottom chord members, use the same member size.

$WEB_{comp} := \max(M_{0_14}, M_{1_15}, M_{2_16}, M_{3_17}, M_{4_18}, M_{5_19}, M_{6_20}, M_{7_21})$ $WEB_{comp} = 628.32\,kip$

$WEB_{ten} := \max(M_{1_14}, M_{2_15}, M_{3_16}, M_{4_17}, M_{5_18}, M_{6_19}, M_{7_20})$ $WEB_{ten} = 881.2\,kip$

$WEB_{max} := \max(WEB_{comp}, WEB_{ten})$ $WEB_{max} = 881.2\,kip$

WEB COMPRESSION

$WEB_{max} = 881.2\,kip$

$L_{c_web} := d_{truss}$ $L_{c_web} = 6.5\ ft$ column load tables are based on Eqn. Q1.5-1 elastic range and Eqn. Q1.5-2 inelastic range (Ref. 2.2.6)

$h = 9.2\ ft$

$L_{max} := \max(L_{c_web}, h)$ $L_{max} = 9.2\ ft$ Use 10'-0"

$C_{web_allow} := 1310kip$ allowable column compressive force per AISC column tables page 3-30 (Ref. 2.2.5)

$comp_web_check := if(C_{web_allow} \geq WEB_{max}, "OK", "N.G!")$ $comp_web_check = "OK"$

$DC_{c_web} := \frac{WEB_{max}}{C_{web_allow}}$ $DC_{c_web} = 0.67$ web compression member demand/capacity ratio

WEB TENSION

$WEB_{max} = 881.2\,kip$

$F_t := \min(0.45 \cdot F_y, 0.5 \cdot F_u)$ $F_t = 22.5\,ksi$ Section Q1.5.1.1 (Ref. 2.2.6) pin connected member

$A_{req} := \frac{WEB_{max}}{F_t}$ $A_{req} = 39.2\,in^2$

$T_{web_allow} := F_t \cdot A_{web}$ $T_{web_allow} = 1125\,kip$

$tension_web_check := if(T_{web_allow} \geq WEB_{max}, "OK", "N.G!")$ $tension_web_check = "OK"$

$DC_{t_web} := \frac{WEB_{max}}{T_{web_allow}}$ $DC_{t_web} = 0.78$ web tension member demand/capacity ratio

TOP CHORD

$L_{tc} := spacing$ $L_{tc} = 77.1\,in$

$C_{top} := \max(M_{14_15}, M_{15_16}, M_{16_17}, M_{17_18}, M_{18_19}, M_{19_20}, M_{20_21})$ $C_{top} = 2328.52\,kip$

$M_{tc} := \frac{C_p \cdot L_{tc}}{4}$ $M_{tc} = 96.4\,k \cdot in$

Section Q1.5.1.4.1 (Ref. 2.2.6)

$compact_check := if\left[\left(\frac{b_f}{2 \cdot t_f}\right) \leq \left(\frac{65}{\sqrt{F_y \cdot ksi^{-1}}}\right), "compact", "non\text{-}compact"\right]$ $compact_check = "compact"$

$F_b := if[(compact_check = "compact"), 0.66 \cdot F_y, 0.6F_y]$ $F_b = 33\,ksi$

$f_b := \frac{M_{tc}}{S_x}$ $f_b = 0.2\,ksi$

$K := 1.0$ $r := \min(r_x, r_y)$ $r = 4.2\,in$ $\frac{K \cdot L_{tc}}{r} = 18.2$

$C_c := \sqrt{\frac{2 \cdot \pi^2 \cdot E_s}{F_y}}$ $C_c = 107$

$$F_a := \text{if}\left[\frac{K \cdot L_{tc}}{r} < C_c, \frac{\left[1 - \frac{\left(\frac{K \cdot L_{tc}}{r}\right)^2}{2 \cdot C_c^2}\right] \cdot F_y}{\frac{5}{3} + \frac{3 \cdot \left(\frac{K \cdot L_{tc}}{r}\right)}{8 \cdot C_c} - \frac{\left(\frac{K \cdot L_{tc}}{r}\right)^3}{8 \cdot C_c^3}}, \frac{12 \cdot \pi^2 \cdot E_s}{23 \cdot \left(\frac{K \cdot L_{tc}}{r}\right)^2}\right]$$

Eqn. Q1.5-1 & Q1.5-2 (Ref. 2.2.6)

$F_a = 28.49\,ksi$

$P_{allow} := F_a \cdot A$

$P_{allow} = 2877.18\,kip$

$f_a := \frac{C_{top}}{A}$ $f_a = 23.1\,ksi$

$F'_{ex} := \frac{\pi^2 \cdot E_s}{\left(\frac{K \cdot L_{tc}}{r_x}\right)^2}$ $F'_{ex} = 2343.2\,ksi$

Eqn (4-1) ANSI/AISC 341-02 Ref. (2.2.16)

$C_m := 0.85$

$\frac{f_a}{F_a} = 0.809$ demand/capacity axial

$\frac{C_m \cdot f_b}{\left(1 - \frac{f_a}{F'_{ex}}\right) \cdot F_b} = 0.004$ demand/capacity bending

$H_{1_1} := \frac{f_a}{F_a} + \frac{C_m \cdot f_b}{\left(1 - \frac{f_a}{F'_{ex}}\right) \cdot F_b}$ $H_{1_1} = 0.814$

AISC eqn. Q1.6-1a combined axial compression and bending (Ref. 2.2.6)

$H_{1_2} := \frac{f_a}{0.6 \cdot F_y} + \frac{f_b}{F_b}$ $H_{1_2} = 0.774$

AISC eqn. Q1.6-1b combined axial compression and bending (Ref. 2.2.6)

Top_chord_stress_check := if(H1_1 ≤ 1.0, if(H1_2 ≤ 1.0, "OK", "N.G!"), "N.G!") Top_chord_stress_check = "OK"

CHECK TOP CHORD SHEAR

$F_v := 0.4 \cdot F_y$ $F_v = 20\,ksi$

$f_v := \frac{P_{truss_v}}{d \cdot t_w}$ $f_v = 3.8\,ksi$

shear_check := if(Fv ≥ fv, "OK", "N.G!") shear_check = "OK"

BOTTOM CHORD

$L_{bc} := spacing$ $L_{bc} = 6.429\ ft$

$C_{bott} := \max(M_{1_2}, M_{2_3}, M_{3_4}, M_{4_5}, M_{5_6}, M_{6_7})$ $C_{bott} = 2281\ kip$

$M_{bc} := \frac{C_p \cdot L_{bc}}{4}$ $M_{bc} = 96.4\ k \cdot in$

$F_t := \min(0.60 \cdot F_y, 0.5 \cdot F_u)$ $F_t = 30\ ksi$ Section Q1.5.1.1 (Ref. 2.2.6) non-pin connected member

$A_{req_bc} := \frac{C_{bott}}{F_t}$ $A_{req_bc} = 76\ in^2$

$compact_check := if\left[\left(\frac{b_f}{2 \cdot t_f}\right) \le \left(\frac{65}{\sqrt{F_y \cdot ksi^{-1}}}\right), "compact", "non\text{-}compact"\right]$ Section Q1.5.1.4.1 (Ref 2.2.6)

$compact_check = "compact"$

$F_b := if[(compact_check = "compact"), 0.66 \cdot F_y, 0.6F_y]$ $F_b = 33\ ksi$

$f_b := \frac{M_{bc}}{S_x}$ $S_x = 559\ in^3$ $f_b = 0.2\ ksi$

$f_t := \frac{C_{bott}}{A}$ $f_t = 22.6\ ksi$

$\frac{f_t}{F_t} = 0.753$ demand/capacity axial tension $\frac{f_b}{F_b} = 0.005$ demand/capacity bending

$H_{2_1} := \frac{f_t}{F_t} + \frac{f_b}{F_b}$ $H_{2_1} = 0.758$ AISC eqn. Q1.6-3 combined axial tension and bending (Ref. 2.2.6)

$Bottom_chord_stress_check := if(H_{2_1} \le 1.0, "OK", "N.G!")$ $(Bottom_chord_stress_check) = "OK"$

Check truss deflection using method of virtual work, Section 1, Structural Analysis, Equation (28) (Ref. 2.2.12)

$$\Delta := \sum_k \frac{n_k \cdot N_k \cdot L_k}{A_k \cdot E_k}$$

n_k = internal virtual axial force in a truss member caused by the external virtual unit load
N_k = internal force in a truss member caused by the real loads
L_k = length of member
A_k = cross sectional area of member
E_k = modulus of elasticity of member

MEMBER VIRTURAL FORCES (1.0 kip load applied at the midpoint of the truss span)

$p_{truss} := 1.0k$ $r_{LT} := 0.5k$ $m_{0_14} := r_{LT}$ $m_{0_14} = 0.5\ kip$ compression

$m_{1_14} := \dfrac{r_{LT}}{\dfrac{d_{truss}}{h}}$ $m_{1_14} = 0.7\,kip$ tension

$m_{14_15} := r_{LT} \cdot \dfrac{spacing}{d_{truss}}$ $m_{14_15} = 0.5\,kip$ compression

$m_{1_15} := m_{1_14} \cdot \left(\dfrac{d_{truss}}{h}\right)$ $m_{1_15} = 0.5\,kip$ compression

$m_{1_2} := m_{1_14} \cdot \left(\dfrac{spacing}{h}\right)$ $m_{1_2} = 0.5\,kip$ tension

$m_{2_15} := \dfrac{m_{1_15}}{\dfrac{d_{truss}}{h}}$ $m_{2_15} = 0.7\,kip$ tension

$m_{15_16} := m_{14_15} + m_{2_15} \cdot \left(\dfrac{spacing}{h}\right)$ $m_{15_16} = 0.983\,kip$ compression

$m_{2_3} := m_{1_2} + m_{2_15} \cdot \left(\dfrac{spacing}{h}\right)$ $m_{2_3} = 1\,kip$ tension

$m_{2_16} := m_{2_15} \cdot \left(\dfrac{d_{truss}}{h}\right)$ $m_{2_16} = 0.5\,kip$ compression

$m_{3_16} := \dfrac{m_{2_16}}{\dfrac{d_{truss}}{h}}$ $m_{3_16} = 0.7\,kip$ tension

$m_{16_17} := m_{15_16} + m_{3_16} \cdot \left(\dfrac{spacing}{h}\right)$ $m_{16_17} = 1.5\,kip$ compression

$m_{3_4} := m_{2_3} + m_{3_16} \cdot \left(\dfrac{spacing}{h}\right)$ $m_{3_4} = 1.5\,kip$ tension

$m_{3_17} := m_{3_16} \cdot \left(\dfrac{d_{truss}}{h}\right)$ $m_{3_17} = 0.5\,kip$ compression

$m_{4_17} := \dfrac{m_{3_17}}{\dfrac{d_{truss}}{h}}$ $m_{4_17} = 0.7\,kip$ tension

$m_{17_18} := m_{16_17} + m_{4_17} \cdot \left(\dfrac{spacing}{h}\right)$ $m_{17_18} = 1.966\,kip$ compression

$m_{4_5} := m_{3_4} + m_{4_17} \cdot \left(\dfrac{spacing}{h}\right)$ $m_{4_5} = 1.966\,kip$ tension

$m_{4_18} := m_{4_17} \cdot \left(\dfrac{d_{truss}}{h}\right)$ $m_{4_18} = 0.5\,kip$ compression

$m_{5_18} := \dfrac{m_{4_18}}{\dfrac{d_{truss}}{h}}$ $m_{5_18} = 0.7\,kip$ tension

$m_{18_19} := m_{17_18} + m_{5_18} \cdot \left(\frac{spacing}{h}\right)$ $m_{18_19} = \blacksquare\ kip$ compression

$m_{5_6} := m_{4_5} + m_{5_18} \cdot \left(\frac{spacing}{h}\right)$ $m_{5_6} = 2.458\,kip$ tension

$m_{5_19} := m_{5_18} \cdot \left(\frac{d_{truss}}{h}\right)$ $m_{5_19} = 0.5\,kip$ compression

$m_{6_19} := \frac{m_{5_19}}{\frac{d_{truss}}{h}}$ $m_{6_19} = 0.701\,kip$ tension

$m_{19_20} := m_{18_19} + m_{6_19} \cdot \left(\frac{spacing}{h}\right)$ $m_{19_20} = 2.95\,kip$ compression

$m_{6_7} := m_{5_6} + m_{6_19} \cdot \left(\frac{spacing}{h}\right)$ $m_{6_7} = 2.95\,kip$ tension

$m_{6_20} := m_{6_19} \cdot \left(\frac{d_{truss}}{h}\right)$ $m_{6_20} = 0.5\,kip$ compression

$m_{7_20} := \frac{m_{6_20}}{\frac{d_{truss}}{h}}$ $m_{7_20} = 0.7\,kip$ tension

$m_{20_21} := m_{19_20} + m_{7_20} \cdot \left(\frac{spacing}{h}\right)$ $m_{20_21} = 3.44\,kip$ compression

$m_{7_21} := p_{truss}$ $m_{7_21} = 1\,kip$ tension

$\Delta_{1_2} := \frac{M_{1_2} \cdot m_{1_2} \cdot spacing}{A \cdot E_s \cdot kip}$ $\Delta_{1_2} = 0.008\,in$ $\Delta_{14_15} := \frac{M_{14_15} \cdot m_{14_15} \cdot spacing}{A \cdot E_s \cdot kip}$ $\Delta_{14_15} = 0.008\,in$

$\Delta_{2_3} := \frac{M_{2_3} \cdot m_{2_3} \cdot spacing}{A \cdot E_s \cdot kip}$ $\Delta_{2_3} = 0.03\,in$ $\Delta_{15_16} := \frac{M_{15_16} \cdot m_{15_16} \cdot spacing}{A \cdot E_s \cdot kip}$ $\Delta_{15_16} = 0.03\,in$

$\Delta_{3_4} := \frac{M_{3_4} \cdot m_{3_4} \cdot spacing}{A \cdot E_s \cdot kip}$ $\Delta_{3_4} = 0.061\,in$ $\Delta_{16_17} := \frac{M_{16_17} \cdot m_{16_17} \cdot spacing}{A \cdot E_s \cdot kip}$ $\Delta_{16_17} = 0.061\,in$

$\Delta_{4_5} := \frac{M_{4_5} \cdot m_{4_5} \cdot spacing}{A \cdot E_s \cdot kip}$ $\Delta_{4_5} = 0.098\,in$ $\Delta_{17_18} := \frac{M_{17_18} \cdot m_{17_18} \cdot spacing}{A \cdot E_s \cdot kip}$ $\Delta_{17_18} = 0.098\,in$

$\Delta_{5_6} := \frac{M_{5_6} \cdot m_{5_6} \cdot spacing}{A \cdot E_s \cdot kip}$ $\Delta_{5_6} = 0.138\,in$ $\Delta_{18_19} := \frac{M_{18_19} \cdot m_{18_19} \cdot spacing}{A \cdot E_s \cdot kip}$ $\Delta_{18_19} = 0.138\,in$

$\Delta_{6_7} := \frac{M_{6_7} \cdot m_{6_7} \cdot spacing}{A \cdot E_s \cdot kip}$ $\Delta_{6_7} = 0.177\,in$ $\Delta_{19_20} := \frac{M_{19_20} \cdot m_{19_20} \cdot spacing}{A \cdot E_s \cdot kip}$ $\Delta_{19_20} = 0.177\,in$

$\Delta_{1_14} := \frac{M_{1_14} \cdot m_{1_14} \cdot h}{A_{web} \cdot E_s \cdot kip}$ $\Delta_{1_14} = 0.047\,in$ $\Delta_{20_21} := \frac{M_{20_21} \cdot m_{20_21} \cdot spacing}{A \cdot E_s \cdot kip}$ $\Delta_{20_21} = 0.211\,in$

$\Delta_{2_15} := \frac{M_{2_15} \cdot m_{2_15} \cdot h}{A_{web} \cdot E_s \cdot kip}$ $\Delta_{2_15} = 0.04\,in$ $\Delta_{1_15} := \frac{M_{1_15} \cdot m_{1_15} \cdot d_{truss}}{A_{web} \cdot E_s \cdot kip}$ $\Delta_{1_15} = 0.017\,in$

$\Delta_{3_16} := \frac{M_{3_16} \cdot m_{3_16} \cdot h}{A_{web} \cdot E_s \cdot kip}$ $\Delta_{3_16} = 0.032\,in$ $\Delta_{2_16} := \frac{M_{2_16} \cdot m_{2_16} \cdot d_{truss}}{A_{web} \cdot E_s \cdot kip}$ $\Delta_{2_16} = 0.014\,in$

$\Delta_{4_17} := \frac{M_{4_17} \cdot m_{4_17} \cdot h}{A_{web} \cdot E_s \cdot kip}$ $\Delta_{4_17} = 0.025\,in$ $\Delta_{3_17} := \frac{M_{3_17} \cdot m_{3_17} \cdot d_{truss}}{A_{web} \cdot E_s \cdot kip}$ $\Delta_{3_17} = 0.012\,in$

$\Delta_{5_18} := \frac{M_{5_18} \cdot m_{5_18} \cdot h}{A_{web} \cdot E_s \cdot kip}$ $\Delta_{5_18} = 0.018\,in$ $\Delta_{4_18} := \frac{M_{4_18} \cdot m_{4_18} \cdot d_{truss}}{A_{web} \cdot E_s \cdot kip}$ $\Delta_{4_18} = 0.009\,in$

$\Delta_{6_19} := \frac{M_{6_19} \cdot m_{6_19} \cdot h}{A_{web} \cdot E_s \cdot kip}$ $\Delta_{6_19} = 0.011\,in$ $\Delta_{5_19} := \frac{M_{5_19} \cdot m_{5_19} \cdot d_{truss}}{A_{web} \cdot E_s \cdot kip}$ $\Delta_{5_19} = 0.007\,in$

$\Delta_{7_20} := \frac{M_{7_20} \cdot m_{7_20} \cdot h}{A_{web} \cdot E_s \cdot kip}$ $\Delta_{7_20} = 0.004\,in$ $\Delta_{6_20} := \frac{M_{6_20} \cdot m_{6_20} \cdot d_{truss}}{A_{web} \cdot E_s \cdot kip}$ $\Delta_{6_20} = 0.004\,in$

$\Delta_{0_14} := \frac{M_{0_14} \cdot m_{0_14} \cdot d_{truss}}{A_{web} \cdot E_s \cdot kip}$ $\Delta_{0_14} = 0.017\,in$ $\Delta_{7_21} := \frac{0.5M_{7_21} \cdot m_{7_21} \cdot d_{truss}}{A_{web} \cdot E_s \cdot kip}$ $\Delta_{7_21} = 0.003\,in$

$\Delta_{t_chord} := \Delta_{14_15} + \Delta_{15_16} + \Delta_{16_17} + \Delta_{17_18} + \Delta_{18_19} + \Delta_{19_20} + \Delta_{20_21}$

$\Delta_{b_chord} := \Delta_{1_2} + \Delta_{2_3} + \Delta_{3_4} + \Delta_{4_5} + \Delta_{5_6} + \Delta_{6_7}$

$\Delta_{web1} := \Delta_{1_14} + \Delta_{2_15} + \Delta_{3_16} + \Delta_{4_17} + \Delta_{5_18} + \Delta_{6_19} + \Delta_{7_20}$

$\Delta_{web2} := \Delta_{0_14} + \Delta_{1_15} + \Delta_{2_16} + \Delta_{3_17} + \Delta_{4_18} + \Delta_{5_19} + \Delta_{6_20} + \Delta_{7_21}$

$\Delta_{tot} := 2 \cdot (\Delta_{t_chord} + \Delta_{b_chord} + \Delta_{web1} + \Delta_{web2})$ $\Delta_{tot} = 2.99\,in$

$\Delta_{all} := \frac{L_{truss}}{240}$ $\Delta_{all} = 4.5\,in$ (Ref. 2.2.4)

$deflection_check := if(\Delta_{tot} \le \Delta_{all}, "OK", "N.G!")$ $deflection_check = "OK"$

TRUSS T1

$TC = "W14X342"$	Truss top chord	$spacing = 6.4286\,ft$	Panel point spacing center to center
$BC = "W14X342"$	Truss bottom chord	$d_{truss} = 78.5\,in$	Truss depth; center to center spacing
$WEB = "W12X170"$	Diagonal web member	$D = 8\,ft$	Truss depth maximum dimension
$WEB = "W12X170"$	Vertical web member	$\Delta_{tot} = 2.99\,in$	Truss deflection
$L_{truss} = 90\,ft$	Total truss length		

NOTE: To reduce deflection from construction loading camber in the truss will be equal to dead load deflection to be determined in final design.

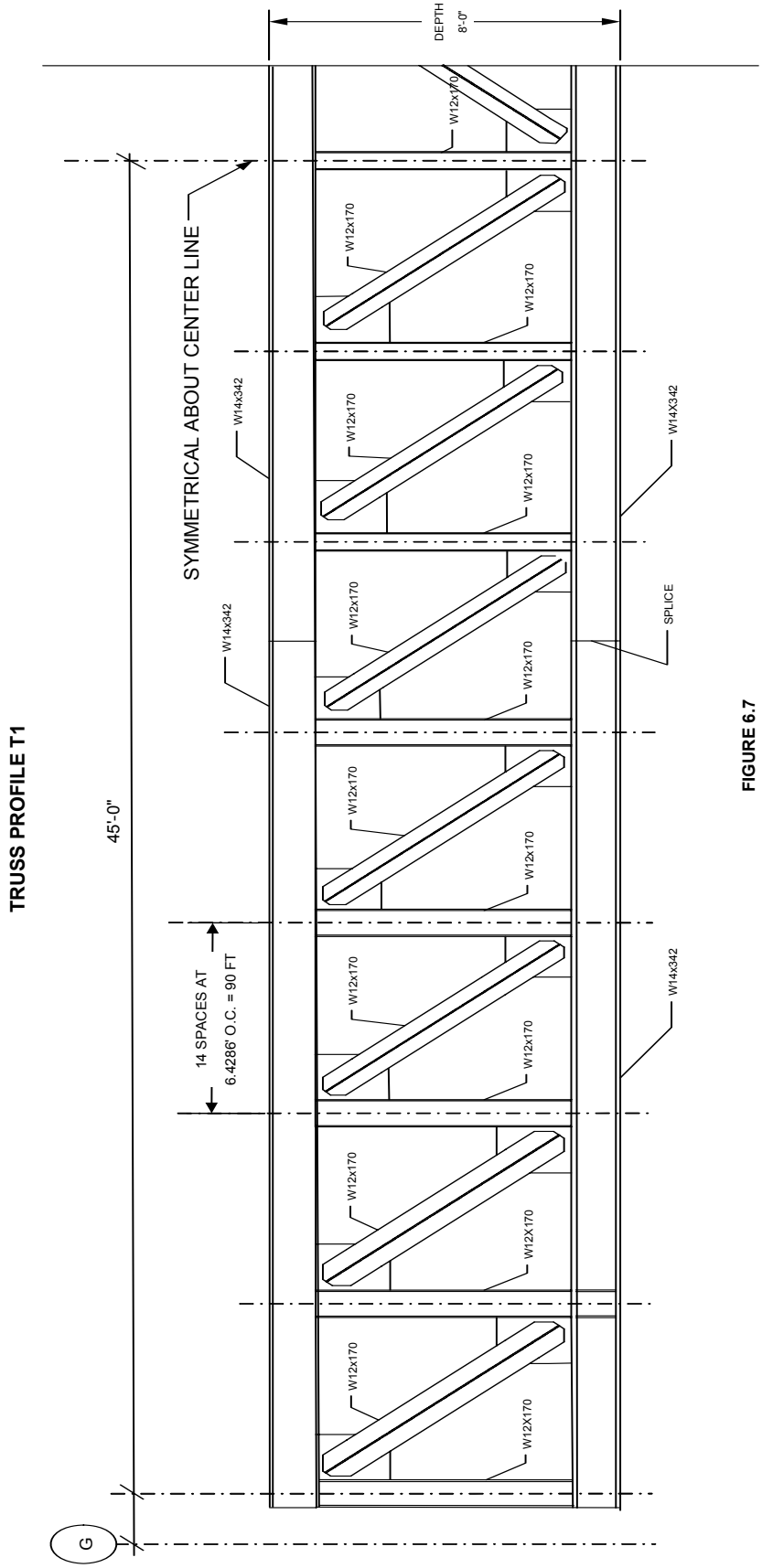


FIGURE 6.7

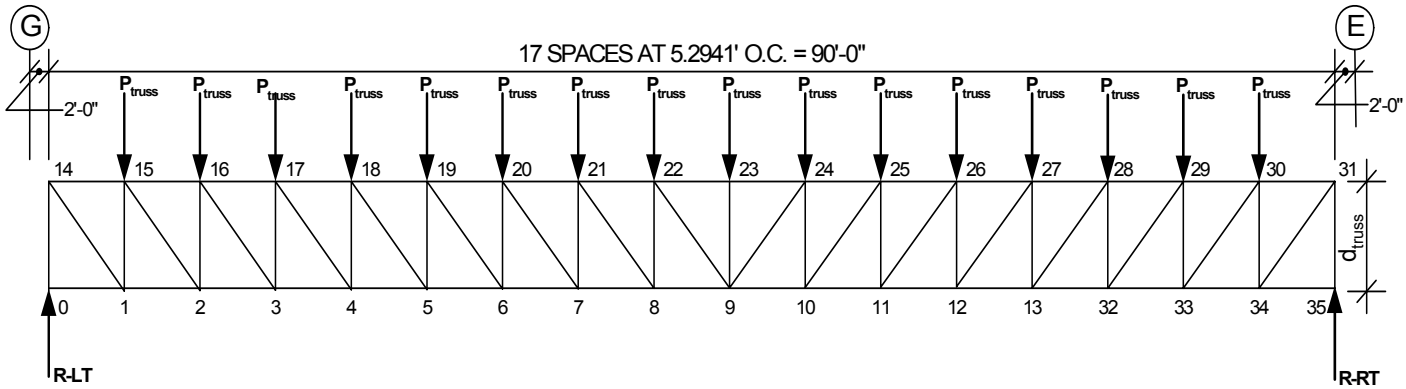
CANISTER RECEIPT AND CLOSURE FACILITY – ROOF TRUSS T1

NOTE: THIS TRUSS REPRESENTS A GRAPHICAL LAYOUT OF MEMBER SIZES, DIMENSIONS, AND PANEL POINT SPACING. ACTUAL DETAILED CONNECTIONS, SPLICES, AND LOCATION WILL BE DESIGNED IN CONFIRMED CALCULATION

6.8 TRUSS T2 CALCULATION, Ref. 2.2.5, & 2.2.6

TRUSS T2 CALCULATION (Total of 6 T2 trusses required)

ROOF ELEV. 64'-0" - Truss Design between Gridline G to E, from 2 to 4.
Nominal concrete slab thickness = 2'-9" (Attachment A-2)



P_{truss} loading from beam B5 (each side of girder)

TRUSS T2

$L_{truss} := 90ft$ truss length

$spacing := \frac{L_{truss}}{17}$ $spacing = 5.294\,ft$ panel point spacing

$trib_{t2} := 12ft$ tributary loading (T2 Grids 2-4/E-G)

TOP CHORD PROPERTIES

TC := "W14X342" BC := "W14X342"

$A := 101.0in^2$ $d := 17.54in$ $t_w := 1.540in$ $t_f := 2.470in$ $b_f := 16.360in$

$I_x := 4900in^4$ $S_x := 559in^3$ $r_x := 6.98in$

$I_y := 1810in^4$ $S_y := 221in^3$ $r_y := 4.24in$

WEB MEMBER PROPERTIES

WEB := "W12X170"

$A_{web} := 50.0in^2$ $d_{web} := 14.03in$ $t_{wweb} := 0.960in$ $t_{fweb} := 1.560in$ $b_{fweb} := 12.570in$

$I_{xweb} := 1650in^4$ $S_{xweb} := 235in^3$ $r_{xweb} := 5.74in$

$I_{yweb} := 517in^4$ $S_{yweb} := 82.3in^3$ $r_{yweb} := 3.22in$

$d_{truss} := 78.46in$ truss depth center of top chord to center of bottom chord

$h := \sqrt{spacing^2 + d_{truss}^2}$ $h = 8.4\,ft$ diagonal web length

$D := d_{truss} + 0.5\cdot(d + d)$ $D = 8.00\,ft$ maximum outer dimensions of truss

AISC N690 (Table Q1.5.7.1 Ref. 2.2.6) allows a 1.6 stress increase with Extreme and Abnormal Severe load combinations (1.4 for shear and 1.5 for axial compression Ref. 2.2.7). To adjust maximum loading for members with axial compression multiply maximum roof loading by 1.0667: (1.6/1.5 x $BMax_{rf}$ = 1.0667 x $BMax_{rf}$).

$P_{truss} := \left(\frac{1.6}{1.5}\right)\cdot\left(BMax_{rf3}\cdot trib_{t2} + D_{gird}\right)\cdot spacing$ $\quad P_{truss} = 68.5\,kip$ — panel joint loading for flexure
D_{gird} = assumed self wt.
(Assumption 3.1.4)
(see beam B5 design)

$P_{truss_const} := \left(S20\cdot trib_{t2} + D_{gird}\right)\cdot spacing$ $\quad P_{truss_const} = 40.6\,kip$ — panel joint construction loading

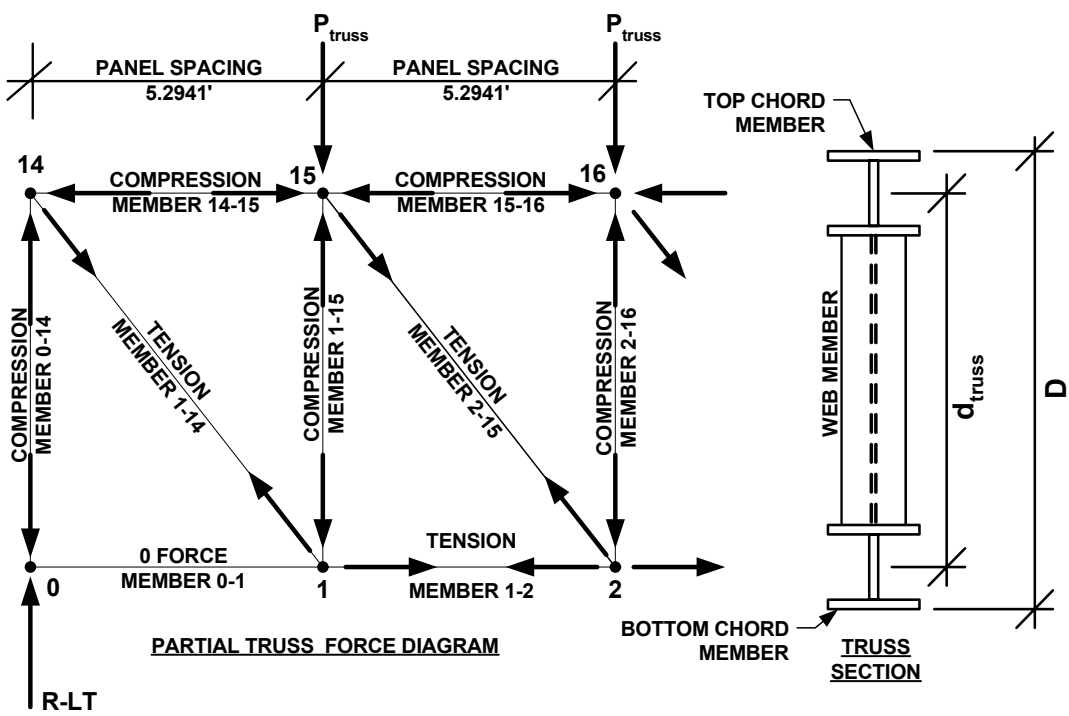
$P_{truss_v} := \left(VMax_{rf3}\cdot trib_{t2} + D_{gird}\right)\cdot spacing$ $\quad P_{truss_v} = 72.7\,kip$ — panel joint loading for shear

$R_{LT} := P_{truss}\cdot 8$ $\quad R_{LT} = 548.3\,kip$ — truss left end reaction from flexure loading

$R_{LT_const} := P_{truss_const}\cdot 8$ $\quad R_{LT_const} = 325.1\,kip$ — truss left end reaction from construction loading

$R_{LT_v} := P_{truss_v}\cdot 8$ $\quad R_{LT_v} = 581.4\,kip$ — truss left end reaction from shear loading

Member axial forces are calculated by the joint equilibrium approach starting at the left reaction node 0 and then progressing to the center of truss



MEMBER AXIAL FORCES

$M_{0_14} := R_{LT}$ $\quad M_{0_14} = 548.3\,kip$ — compression

$M_{1_14} := \dfrac{R_{LT}}{\dfrac{d_{truss}}{h}}$ $M_{1_14} = 705.5\,kip$ tension

$M_{14_15} := R_{LT} \cdot \dfrac{spacing}{d_{truss}}$ $M_{14_15} = 444\,kip$ compression

$M_{1_15} := M_{1_14} \cdot \left(\dfrac{d_{truss}}{h}\right)$ $M_{1_15} = 548.3\,kip$ compression

$M_{1_2} := M_{1_14} \cdot \left(\dfrac{spacing}{h}\right)$ $M_{1_2} = 444\,kip$ tension

$M_{2_15} := \dfrac{M_{1_15} - P_{truss}}{\dfrac{d_{truss}}{h}}$ $M_{2_15} = 617.3\,kip$ tension

$M_{15_16} := M_{14_15} + M_{2_15} \cdot \left(\dfrac{spacing}{h}\right)$ $M_{15_16} = \blacksquare\,kip$ compression

$M_{2_3} := M_{1_2} + M_{2_15} \cdot \left(\dfrac{spacing}{h}\right)$ $M_{2_3} = 832.5\,kip$ tension

$M_{2_16} := M_{2_15} \cdot \left(\dfrac{d_{truss}}{h}\right)$ $M_{2_16} = 479.8\,kip$ compression

$M_{3_16} := \dfrac{M_{2_16} - P_{truss}}{\dfrac{d_{truss}}{h}}$ $M_{3_16} = 529.2\,kip$ tension

$M_{16_17} := M_{15_16} + M_{3_16} \cdot \left(\dfrac{spacing}{h}\right)$ $M_{16_17} = 1165.5\,kip$ compression

$M_{3_4} := M_{2_3} + M_{3_16} \cdot \left(\dfrac{spacing}{h}\right)$ $M_{3_4} = 1165.5\,kip$ tension

$M_{3_17} := M_{3_16} \cdot \left(\dfrac{d_{truss}}{h}\right)$ $M_{3_17} = 411.2\,kip$ compression

$M_{4_17} := \dfrac{M_{3_17} - P_{truss}}{\dfrac{d_{truss}}{h}}$ $M_{4_17} = 441\,kip$ tension

$M_{17_18} := M_{16_17} + M_{4_17} \cdot \left(\dfrac{spacing}{h}\right)$ $M_{17_18} = 1442.95\,kip$ compression

$M_{4_5} := M_{3_4} + M_{4_17} \cdot \left(\dfrac{spacing}{h}\right)$ $M_{4_5} = 1442.95\,kip$ tension

$M_{4_18} := M_{4_17} \cdot \left(\frac{d_{truss}}{h}\right)$ $M_{4_18} = 342.7\,kip$ compression

$M_{5_18} := \frac{M_{4_18} - P_{truss}}{\frac{d_{truss}}{h}}$ $M_{5_18} = 352.8\,kip$ tension

$M_{18_19} := M_{17_18} + M_{5_18} \cdot \left(\frac{spacing}{h}\right)$ $M_{18_19} = 1664.94\,kip$ compression

$M_{5_6} := M_{4_5} + M_{5_18} \cdot \left(\frac{spacing}{h}\right)$ $M_{5_6} = 1664.94\,kip$ tension

$M_{5_19} := M_{5_18} \cdot \left(\frac{d_{truss}}{h}\right)$ $M_{5_19} = 274.2\,kip$ compression

$M_{6_19} := \frac{M_{5_19} - P_{truss}}{\frac{d_{truss}}{h}}$ $M_{6_19} = 264.6\,kip$ tension

$M_{19_20} := M_{18_19} + M_{6_19} \cdot \left(\frac{spacing}{h}\right)$ $M_{19_20} = 1831.44\,kip$ compression

$M_{6_7} := M_{5_6} + M_{6_19} \cdot \left(\frac{spacing}{h}\right)$ $M_{6_7} = 1831.44\,kip$ tension

$M_{6_20} := M_{6_19} \cdot \left(\frac{d_{truss}}{h}\right)$ $M_{6_20} = 205.6\,kip$ compression

$M_{7_20} := \frac{M_{6_20} - P_{truss}}{\frac{d_{truss}}{h}}$ $M_{7_20} = 176.4\,kip$ tension

$M_{20_21} := M_{19_20} + M_{7_20} \cdot \left(\frac{spacing}{h}\right)$ $M_{20_21} = 1942.43\,kip$ compression

$M_{7_8} := M_{6_7} + M_{7_20} \cdot \left(\frac{spacing}{h}\right)$ $M_{7_8} = 1942.43\,kip$ tension

$M_{7_21} := M_{7_20} \cdot \left(\frac{d_{truss}}{h}\right)$ $M_{7_21} = 137.08\,kip$ compression

$M_{8_21} := \frac{M_{7_21} - P_{truss}}{\frac{d_{truss}}{h}}$ $M_{8_21} = 88.2\,kip$ tension

$M_{21_22} := M_{20_21} + M_{8_21} \cdot \left(\frac{spacing}{h}\right)$ $M_{21_22} = 1997.93\,kip$ compression

$M_{8_9} := M_{7_8} + M_{8_21} \cdot \left(\frac{spacing}{h}\right)$ $M_{8_9} = 1997.93\,kip$ tension

$M_{8_22} := M_{8_21} \cdot \left(\frac{d_{truss}}{h}\right)$ $M_{8_22} = 68.54\,kip$ compression

$M_{9_22} := \frac{M_{8_22} - P_{truss}}{\frac{d_{truss}}{h}}$ $M_{9_22} = -0.00\,kip$ 0 force member

$M_{22_23} := M_{21_22}$ $M_{22_23} = 1997.9\,kip$ tension

Design top and bottom chord with 5000 lb construction load at midpoint for bending. This is a conservative design as member axial force would be less using uniform construction loading (load combination S20). Design tension and compression webs for pure tension and compression.

To account for load reversal from Beyond Design Basis Ground Motion BDBGM design web elements for the greatest compression or tension force and the maximum web member length. For top and bottom chord members, use the same member size.

$WEB_{comp} := \max(M_{0_14}, M_{1_15}, M_{2_16}, M_{3_17}, M_{4_18}, M_{5_19}, M_{6_20}, M_{7_21}, M_{8_22})$ $WEB_{comp} = 548.33\,kip$

$WEB_{ten} := \max(M_{1_14}, M_{2_15}, M_{3_16}, M_{4_17}, M_{5_18}, M_{6_19}, M_{7_20}, M_{8_21}, M_{9_22})$ $WEB_{ten} = 705.5\,kip$

$WEB_{max} := \max(WEB_{comp}, WEB_{ten})$ $WEB_{max} = 705.5\,kip$

WEB COMPRESSION

$WEB_{max} = 705.5\,kip$

$h = 8.4\,ft$

$L_{c_web} := d_{truss}$

$L_{max} := \max(L_{c_web}, h)$ $L_{max} = 8.4\,ft$ Use 9'-0"

$C_{web} := \text{"W12x170"}$ column load tables are based on Eqn. Q1.5-1 elastic range and Eqn. Q1.5-2 inelastic range (Ref. 2.2.6)

$C_{web_allow} := 1335kip$ allowable column compressive force per AISC column tables page 3-30 (Ref. 2.2.5)

$comp_web_check := \text{if}(C_{web_allow} \geq WEB_{max}, \text{"OK"}, \text{"N.G!"})$ $comp_web_check = \text{"OK"}$

$\frac{WEB_{max}}{C_{web_allow}} = 0.53$ axial compression demand/capacity ratio

WEB TENSION

$WEB_{max} = 705.5\,kip$

$F_t := \min(0.45 \cdot F_y, 0.5 \cdot F_u)$ $F_t = 22.5\,ksi$ Section Q1.5.1.1 (Ref. 2.2.6) pin connected member

$A_{req} := \frac{WEB_{max}}{F_t}$ $A_{req} = 31.4\,in^2$

$T_{web_allow} := F_t \cdot A_{web}$ $T_{web_allow} = 1125\,kip$

$tension_web_check := if\left(T_{web_allow} \geq WEB_{max}, "OK", "N.G!"\right)$ $tension_web_check = "OK"$

$\frac{WEB_{max}}{T_{web_allow}} = 0.63$

axial tension demand/capacity ratio

TOP CHORD

$L_{tc} := spacing$ $L_{tc} = 63.5\,in$

$C_{top} := max\left(M_{14_15}, M_{15_16}, M_{16_17}, M_{17_18}, M_{18_19}, M_{19_20}, M_{20_21}, M_{21_22}\right)$ $C_{top} = 1997.93\,kip$

$M_{tc} := \frac{C_p \cdot L_{tc}}{4}$ $M_{tc} = 79.4\,k \cdot in$

Section Q1.5.1.4.1 (Ref 2.2.6)

$compact_check := if\left[\left(\frac{b_f}{2 \cdot t_f}\right) \leq \left(\frac{65}{\sqrt{F_y \cdot ksi^{-1}}}\right), "compact", "non\text{-}compact"\right]$ $compact_check = "compact"$

$F_b := if\left[(compact_check = "compact"), 0.66 \cdot F_y, 0.6 F_y\right]$ $F_b = 33\,ksi$

$f_b := \frac{M_{tc}}{S_x}$ $f_b = 0.1\,ksi$

$K := 1.0$ $r := min\left(r_x, r_y\right)$ $r = 4.2\,in$ $\frac{K \cdot L_{tc}}{r} = 15$

$C_c := \sqrt{\frac{2 \cdot \pi^2 \cdot E_s}{F_y}}$ $C_c = 107$

Eqn. Q1.5-1 and Q1.5-2 (Ref. 2.2.6)

$$F_a := if\left[\frac{K \cdot L_{tc}}{r} < C_c, \frac{\left[1 - \frac{\left(\frac{K \cdot L_{tc}}{r}\right)^2}{2 \cdot C_c^{\,2}}\right] \cdot F_y}{\left[\frac{5}{3} + \frac{3 \cdot \left(\frac{K \cdot L_{tc}}{r}\right)}{8 \cdot C_c} - \frac{\left(\frac{K \cdot L_{tc}}{r}\right)^3}{8 \cdot C_c^{\,3}}\right]}, \frac{12 \cdot \pi^2 \cdot E_s}{23 \cdot \left(\frac{K \cdot L_{tc}}{r}\right)^2}\right]$$

$F_a = ▮\,ksi$

$P_{allow} := F_a \cdot A$

$P_{allow} = 2909.23\,kip$

$f_a := \frac{C_{top}}{A}$

$F'_{ex} := \dfrac{\pi^2 \cdot E_s}{\left(\dfrac{K \cdot L_{tc}}{r_x}\right)^2}$ $F'_{ex} = 3455.1\,ksi$ Eqn (4-1) ANSI/AISC 341-02 Ref. (2.2.16)

$C_m := 0.85$

$\dfrac{f_a}{F_a} = 0.687$ demand/capacity axial

$\dfrac{C_m \cdot f_b}{\left(1 - \dfrac{f_a}{F'_{ex}}\right) \cdot F_b} = 0.004$ demand/capacity bending

$H_{1_1} := \dfrac{f_a}{F_a} + \dfrac{C_m \cdot f_b}{\left(1 - \dfrac{f_a}{F'_{ex}}\right) \cdot F_b}$ $H_{1_1} = 0.69$ AISC eqn. Q1.6-1a combined axial compression and bending (Ref. 2.2.6)

$H_{1_2} := \dfrac{f_a}{0.6 \cdot F_y} + \dfrac{f_b}{F_b}$ $H_{1_2} = 0.664$ AISC eqn. Q1.6-1b combined axial compression and bending (Ref. 2.2.6)

$Top_chord_stress_check := if\left(H_{1_1} \le 1.0, if\left(H_{1_2} \le 1.0, "OK", "N.G!"\right), "N.G!"\right)$ $Top_chord_stress_check = "OK"$

CHECK TOP CHORD SHEAR

$F_v := 0.4 \cdot F_y$ $F_v = 20\,ksi$

$f_v := \dfrac{P_{truss_v}}{d \cdot t_w}$ $f_v = 2.7\,ksi$

$shear_check := if\left(F_v \ge f_v, "OK", "N.G!"\right)$ $shear_check = "OK"$

BOTTOM CHORD

$L_{bc} := spacing$

$C_{bott} := max\left(M_{1_2}, M_{2_3}, M_{3_4}, M_{4_5}, M_{5_6}, M_{6_7}, M_{7_8}, M_{8_9}\right)$ $C_{bott} = 1997.93\,kip$

$M_{bc} := \dfrac{C_p \cdot L_{bc}}{4}$ $M_{bc} = 79.4\,k \cdot in$

$F_t := min\left(0.6 \cdot F_y, 0.5 \cdot F_u\right)$ $F_t = 30\,ksi$ Section Q1.5.1.1 (Ref. 2.2.6) non-pin connected member

$A_{req_bc} := \dfrac{C_{bott}}{F_t}$ $A_{req_bc} = 66.6\,in^2$

Section Q1.5.1.4.1 (Ref 2.2.6)

$compact_check := if\left[\left(\dfrac{b_f}{2 \cdot t_f}\right) \le \left(\dfrac{65}{\sqrt{F_y \cdot ksi^{-1}}}\right), "compact", "non\text{-}compact"\right]$ $compact_check = "compact"$

$F_b := if\left[(compact_check = "compact"), 0.66 \cdot F_y, 0.6 F_y\right]$ $F_b = 33\,ksi$

$f_b := \dfrac{M_{bc}}{S_x}$ $f_b = 0.1\,ksi$

$f_t := \frac{C_{bott}}{A}$ $f_t = 19.8\,ksi$

$\frac{f_a}{F_t} = 0.659$ demand/capacity axial tension $\frac{f_b}{F_b} = 0.004$ demand/capacity bending

$H_{2_1} := \frac{f_a}{F_t} + \frac{f_b}{F_b}$ $H_{2_1} = 0.664$ AISC eqn. Q1.6-3 combined axial tension and bending (Ref. 2.2.6)

$Bottom_chord_stress_check := if\left(H_{2_1} \le 1.0, "OK", "N.G!"\right)$ Bottom_chord_stress_check = "OK"

Check truss deflection using method of virtual work, Section 1, Structural Analysis, Equation (28) (Ref. 2.2.12)

$$\Delta := \sum_k \frac{n_k \cdot N_k \cdot L_k}{A_k \cdot E_k}$$

n_k = internal virtual axial force in a truss member caused by the external virtual unit load
N_k = internal force in a truss member caused by the real loads
L_k = length of member
A_k = cross sectional area of member
E_k = modulus of elasticity of member

MEMBER VIRTURAL FORCES (1.0 kip load applied at the midpoint of the truss span)

$p_{truss} := 1.0k$ virtual load at center of truss

$r_{LT} := 0.5k$ virtual reaction of truss

$m_{0_14} := r_{LT}$ $m_{0_14} = 0.5\,kip$ compression

$m_{1_14} := \frac{r_{LT}}{\frac{d_{truss}}{h}}$ $m_{1_14} = 0.6\,kip$ tension

$m_{14_15} := r_{LT} \cdot \frac{spacing}{d_{truss}}$ $m_{14_15} = 0.4\,kip$ compression

$m_{1_15} := m_{1_14} \cdot \left(\frac{d_{truss}}{h}\right)$ $m_{1_15} = 0.5\,kip$ compression

$m_{1_2} := m_{1_14} \cdot \left(\frac{spacing}{h}\right)$ $m_{1_2} = 0.4\,kip$ tension

$m_{2_15} := \frac{m_{1_15}}{\frac{d_{truss}}{h}}$ $m_{2_15} = 0.6\,kip$ tension

$m_{15_16} := \left[m_{14_15} + m_{2_15} \cdot \left(\frac{spacing}{h}\right)\right]$ $m_{15_16} = 0.81\,kip$ compression

$m_{2_3} := m_{1_2} + m_{2_15} \cdot \left(\frac{spacing}{h}\right)$ $m_{2_3} = 0.81\,kip$ tension

$m_{2_16} := m_{2_15} \cdot \left(\frac{d_{truss}}{h}\right)$ $m_{2_16} = 0.5\,kip$ compression

$m_{3_16} := \frac{m_{2_16}}{\frac{d_{truss}}{h}}$ $m_{3_16} = 0.6\,kip$ tension

$m_{16_17} := m_{15_16} + m_{3_16} \cdot \left(\frac{spacing}{h}\right)$ $m_{16_17} = 1.2\,kip$ compression

$m_{3_4} := m_{2_3} + m_{3_16} \cdot \left(\frac{spacing}{h}\right)$ $m_{3_4} = 1.2\,kip$ tension

$m_{3_17} := m_{3_16} \cdot \left(\frac{d_{truss}}{h}\right)$ $m_{3_17} = 0.5\,kip$ compression

$m_{4_17} := \frac{m_{3_17}}{\frac{d_{truss}}{h}}$ $m_{4_17} = 0.6\,kip$ tension

$m_{17_18} := m_{16_17} + m_{4_17} \cdot \left(\frac{spacing}{h}\right)$ $m_{17_18} = 1.62\,kip$ compression

$m_{4_5} := m_{3_4} + m_{4_17} \cdot \left(\frac{spacing}{h}\right)$ $m_{4_5} = 1.62\,kip$ tension

$m_{4_18} := m_{4_17} \cdot \left(\frac{d_{truss}}{h}\right)$ $m_{4_18} = 0.5\,kip$ compression

$m_{5_18} := \frac{m_{4_18}}{\frac{d_{truss}}{h}}$ $m_{5_18} = 0.6\,kip$ tension

$m_{18_19} := m_{17_18} + m_{5_18} \cdot \left(\frac{spacing}{h}\right)$ $m_{18_19} = 2.02\,kip$ compression

$m_{5_6} := m_{4_5} + m_{5_18} \cdot \left(\frac{spacing}{h}\right)$ $m_{5_6} = 2.02\,kip$ tension

$m_{5_19} := m_{5_18} \cdot \left(\frac{d_{truss}}{h}\right)$ $m_{5_19} = 0.5\,kip$ compression

$m_{6_19} := \frac{m_{5_19}}{\frac{d_{truss}}{h}}$ $m_{6_19} = 0.6\,kip$ tension

$m_{19_20} := m_{18_19} + m_{6_19} \cdot \left(\frac{spacing}{h}\right)$ $m_{19_20} = 2.43\,kip$ compression

$m_{6_7} := m_{5_6} + m_{6_19} \cdot \left(\frac{spacing}{h}\right)$ $m_{6_7} = 2.43\,kip$ tension

$m_{6_20} := m_{6_19} \cdot \left(\frac{d_{truss}}{h}\right)$ $m_{6_20} = 0.5\,kip$ compression

$m_{7_20} := \frac{m_{6_20}}{\frac{d_{truss}}{h}}$ $m_{7_20} = 0.6\,kip$ tension

$m_{20_21} := m_{19_20} + m_{7_20} \cdot \left(\frac{spacing}{h}\right)$ $m_{20_21} = 2.83\,kip$ compression

$m_{7_8} := m_{6_7} + m_{7_20} \cdot \left(\frac{spacing}{h}\right)$ $m_{7_8} = \blacksquare\,kip$ tension

$m_{7_21} := m_{7_20} \cdot \left(\frac{d_{truss}}{h}\right)$ $m_{7_21} = 0.5\,kip$ compression

$m_{8_21} := \frac{m_{7_21}}{\frac{d_{truss}}{h}}$ $m_{8_21} = 0.6\,kip$ tension

$m_{21_22} := m_{20_21} + m_{8_21} \cdot \left(\frac{spacing}{h}\right)$ $m_{21_22} = 3.24\,kip$ compression

$m_{8_9} := m_{7_8} + m_{8_21} \cdot \left(\frac{spacing}{h}\right)$ $m_{8_9} = 3.24\,kip$ tension

$m_{8_22} := m_{8_21} \cdot \left(\frac{d_{truss}}{h}\right)$ $m_{8_22} = 0.5\,kip$ compression

$m_{22_23} := m_{21_22}$ $m_{21_22} = 3.2\,kip$

$\Delta_{1_2} := \frac{M_{1_2} \cdot m_{1_2} \cdot spacing}{A \cdot E_s \cdot kip}$ $\Delta_{1_2} = 0.004\,in$ $\Delta_{14_15} := \frac{M_{14_15} \cdot m_{14_15} \cdot spacing}{A \cdot E_s \cdot kip}$ $\Delta_{14_15} = 0.004\,in$

$\Delta_{2_3} := \frac{M_{2_3} \cdot m_{2_3} \cdot spacing}{A \cdot E_s \cdot kip}$ $\Delta_{2_3} = 0.015\,in$ $\Delta_{15_16} := \frac{M_{15_16} \cdot m_{15_16} \cdot spacing}{A \cdot E_s \cdot kip}$ $\Delta_{15_16} = 0.015\,in$

$\Delta_{3_4} := \frac{M_{3_4} \cdot m_{3_4} \cdot spacing}{A \cdot E_s \cdot kip}$ $\Delta_{3_4} = 0.031\,in$ $\Delta_{16_17} := \frac{M_{16_17} \cdot m_{16_17} \cdot spacing}{A \cdot E_s \cdot kip}$ $\Delta_{16_17} = 0.031\,in$

$\Delta_{4_5} := \frac{M_{4_5} \cdot m_{4_5} \cdot spacing}{A \cdot E_s \cdot kip}$ $\Delta_{4_5} = 0.051\,in$ $\Delta_{17_18} := \frac{M_{17_18} \cdot m_{17_18} \cdot spacing}{A \cdot E_s \cdot kip}$ $\Delta_{17_18} = 0.051\,in$

$\Delta_{5_6} := \frac{M_{5_6} \cdot m_{5_6} \cdot spacing}{A \cdot E_s \cdot kip}$ $\Delta_{5_6} = 0.073\,in$ $\Delta_{18_19} := \frac{M_{18_19} \cdot m_{18_19} \cdot spacing}{A \cdot E_s \cdot kip}$ $\Delta_{18_19} = 0.073\,in$

$\Delta_{6_7} := \frac{M_{6_7} \cdot m_{6_7} \cdot spacing}{A \cdot E_s \cdot kip}$ $\Delta_{6_7} = 0.096\,in$

$\Delta_{7_8} := \frac{M_{7_8} \cdot m_{7_8} \cdot spacing}{A \cdot E_s \cdot kip}$ $\Delta_{7_8} = 0.119\,in$

$\Delta_{8_9} := \frac{0.5M_{8_9} \cdot m_{8_9} \cdot spacing}{A \cdot E_s \cdot kip}$ $\Delta_{8_9} = 0.07\,in$

$\Delta_{19_20} := \frac{M_{19_20} \cdot m_{19_20} \cdot spacing}{A \cdot E_s \cdot kip}$ $\Delta_{19_20} = 0.096\,in$

$\Delta_{20_21} := \frac{M_{20_21} \cdot m_{20_21} \cdot spacing}{A \cdot E_s \cdot kip}$ $\Delta_{20_21} = 0.119\,in$

$\Delta_{21_22} := \frac{M_{21_22} \cdot m_{21_22} \cdot spacing}{A \cdot E_s \cdot kip}$ $\Delta_{21_22} = 0.14\,in$

$\Delta_{22_23} := \frac{0.5M_{22_23} \cdot m_{22_23} \cdot spacing}{A \cdot E_s \cdot kip}$ $\Delta_{22_23} = 0.07\,in$

$\Delta_{1_14} := \frac{M_{1_14} \cdot m_{1_14} \cdot h}{A_{web} \cdot E_s \cdot kip}$ $\Delta_{1_14} = 0.032\,in$

$\Delta_{2_15} := \frac{M_{2_15} \cdot m_{2_15} \cdot h}{A_{web} \cdot E_s \cdot kip}$ $\Delta_{2_15} = 0.028\,in$

$\Delta_{3_16} := \frac{M_{3_16} \cdot m_{3_16} \cdot h}{A_{web} \cdot E_s \cdot kip}$ $\Delta_{3_16} = 0.024\,in$

$\Delta_{4_17} := \frac{M_{4_17} \cdot m_{4_17} \cdot h}{A_{web} \cdot E_s \cdot kip}$ $\Delta_{4_17} = 0.02\,in$

$\Delta_{5_18} := \frac{M_{5_18} \cdot m_{5_18} \cdot h}{A_{web} \cdot E_s \cdot kip}$ $\Delta_{5_18} = 0.016\,in$

$\Delta_{6_19} := \frac{M_{6_19} \cdot m_{6_19} \cdot h}{A_{web} \cdot E_s \cdot kip}$ $\Delta_{6_19} = 0.012\,in$

$\Delta_{7_20} := \frac{M_{7_20} \cdot m_{7_20} \cdot h}{A_{web} \cdot E_s \cdot kip}$ $\Delta_{7_20} = 0.008\,in$

$\Delta_{8_21} := \frac{M_{8_21} \cdot m_{8_21} \cdot h}{A_{web} \cdot E_s \cdot kip}$ $\Delta_{8_21} = 0.004\,in$

$\Delta_{0_14} := \frac{M_{0_14} \cdot m_{0_14} \cdot d_{truss}}{A_{web} \cdot E_s \cdot kip}$ $\Delta_{0_14} = 0.015\,in$

$\Delta_{20_21} := \frac{M_{20_21} \cdot m_{20_21} \cdot spacing}{A \cdot E_s \cdot kip}$ $\Delta_{20_21} = 0.119\,in$

$\Delta_{1_15} := \frac{M_{1_15} \cdot m_{1_15} \cdot d_{truss}}{A_{web} \cdot E_s \cdot kip}$ $\Delta_{1_15} = 0.015\,in$

$\Delta_{2_16} := \frac{M_{2_16} \cdot m_{2_16} \cdot d_{truss}}{A_{web} \cdot E_s \cdot kip}$ $\Delta_{2_16} = 0.013\,in$

$\Delta_{3_17} := \frac{M_{3_17} \cdot m_{3_17} \cdot d_{truss}}{A_{web} \cdot E_s \cdot kip}$ $\Delta_{3_17} = 0.011\,in$

$\Delta_{4_18} := \frac{M_{4_18} \cdot m_{4_18} \cdot d_{truss}}{A_{web} \cdot E_s \cdot kip}$ $\Delta_{4_18} = 0.009\,in$

$\Delta_{5_19} := \frac{M_{5_19} \cdot m_{5_19} \cdot d_{truss}}{A_{web} \cdot E_s \cdot kip}$ $\Delta_{5_19} = 0.007\,in$

$\Delta_{6_20} := \frac{M_{6_20} \cdot m_{6_20} \cdot d_{truss}}{A_{web} \cdot E_s \cdot kip}$ $\Delta_{6_20} = 0.006\,in$

$\Delta_{7_21} := \frac{M_{7_21} \cdot m_{7_21} \cdot d_{truss}}{A_{web} \cdot E_s \cdot kip}$ $\Delta_{7_21} = 0.004\,in$

$\Delta_{8_22} := \frac{M_{8_22} \cdot m_{8_22} \cdot d_{truss}}{A_{web} \cdot E_s \cdot kip}$ $\Delta_{8_22} = 0.002\,in$

$\Delta_{tchord} := \Delta_{1_2} + \Delta_{2_3} + \Delta_{3_4} + \Delta_{4_5} + \Delta_{5_6} + \Delta_{6_7} + \Delta_{7_8} + \Delta_{8_9}$

$\Delta_{bchord} := \Delta_{14_15} + \Delta_{15_16} + \Delta_{16_17} + \Delta_{17_18} + \Delta_{18_19} + \Delta_{19_20} + \Delta_{20_21} + \Delta_{21_22} + \Delta_{22_23}$

$\Delta_{web1} := \Delta_{1_14} + \Delta_{2_15} + \Delta_{3_16} + \Delta_{4_17} + \Delta_{5_18} + \Delta_{6_19} + \Delta_{7_20} + \Delta_{8_21}$

$\Delta_{web2} := \Delta_{0_14} + \Delta_{1_15} + \Delta_{2_16} + \Delta_{3_17} + \Delta_{4_18} + \Delta_{5_19} + \Delta_{6_20} + \Delta_{7_21} + \Delta_{8_22}$

$\Delta_{tot} := 2 \cdot \left(\Delta_{tchord} + \Delta_{bchord} + \Delta_{web1} + \Delta_{web2}\right)$ $\Delta_{tot} = 2.565\,in$

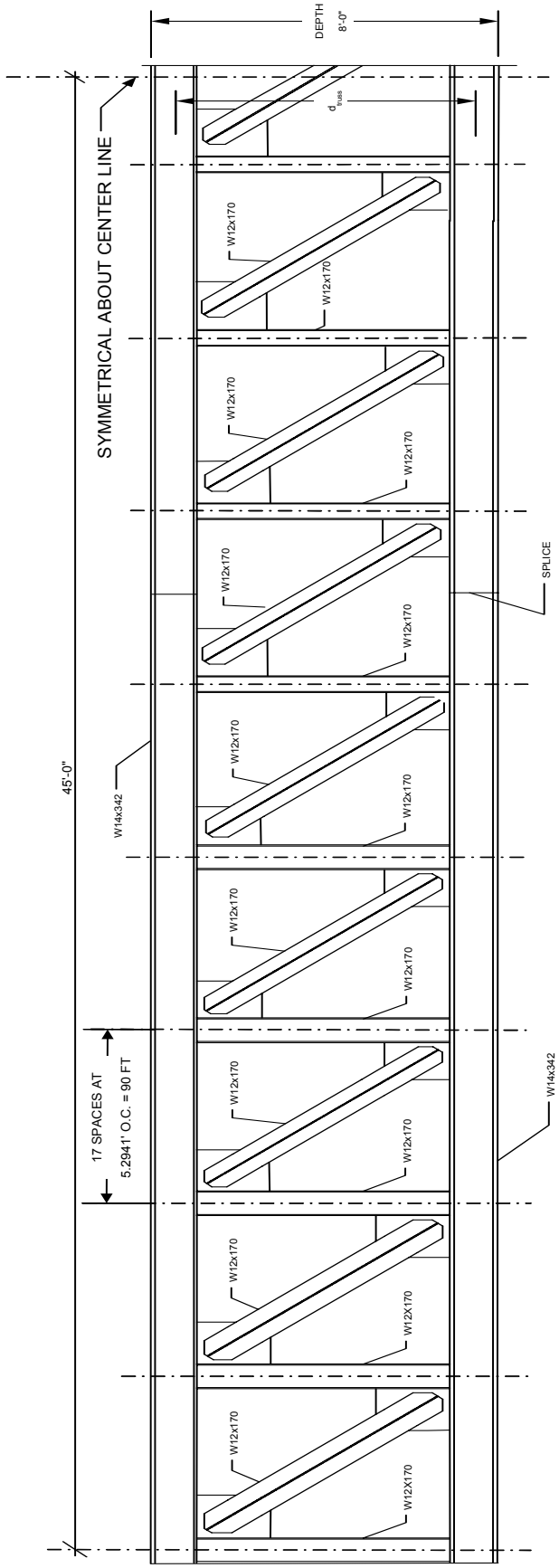
$\Delta_{all} := \frac{L_{truss}}{240}$ $\Delta_{all} = 4.5\,in$ (Ref. 2.2.4)

deflection_check := if$\left(\Delta_{tot} \le \Delta_{all}, \text{"OK"}, \text{"N.G!"}\right)$ deflection_check = "OK"

TRUSS T2

TC = "W14X342"	Truss top chord	spacing = 5.2941 ft	Panel point spacing center to center
BC = "W14X342"	Truss bottom chord	d_{truss} = 78.5 in	Truss depth; center to center spacing
WEB = "W12X170"	Diagonal web member	D = 8 ft	Truss depth; maximum dimension
WEB = "W12X170"	Vertical web member	Δ_{tot} = 2.565 in	Truss deflection
L_{truss} = ▮ ft	Total truss length		

NOTE: To reduce deflection from construction loading camber in the truss will be equal to dead load deflection to be determined in final design.



TRUSS T2 PROFILE

FIGURE 6.8

CANISTER RECEIPT AND CLOSURE FACILITY – ROOF TRUSS T2

NOTE: THIS TRUSS REPRESENTS A GRAPHICAL LAYOUT OF MEMBER SIZES, DIMENSIONS, AND PANEL POINT SPACING. ACTUAL DETAILED CONNECTIONS, SPLICES, AND LOCATION WILL BE DESIGNED IN CONFIRMED CALCULATION

7. RESULTS AND CONCLUSIONS

7.1 RESULTS

Results from this calculation will be used to support the interior steel framing drawings at the second floor and roof levels of the Canister Receipt and Closure Facility (CRCF).

The primary outputs from this calculation are:

TABLE 2: BEAM AND GIRDER FRAMING SUMMARY

BEAM & GIRDER FRAMING SUMMARY			DEMAND/CAPACITY RATIO		
ROOF BEAM	**SIZE**	**ELEVATION (FT)**	**D/C Flexure**	**D/C Shear**	**D/C Deflection**
B1	W16x31	64	0.81	0.38	0.32
B2	W24x68	64	0.74	0.28	0.35
B3	W24x55	64	0.84	0.29	0.34
B4	W18x40	72	0.84	0.36	0.36
B5	W12x30	64	0.84	0.53	0.35
B6	W18x55	100	0.83	0.34	0.42
B7	W21x44	34 & 36	0.81	0.31	0.32
FLOOR BEAM	**SIZE**	**ELEVATION (FT)**	**D/C Flexure**	**D/C Shear**	**D/C Deflection**
B8	W18x35	32	0.68	0.32	0.27
B9	W18x40	32	0.74	0.49	0.29
B10	W10x39	32	0.73	0.48	0.47
B11	W18x35	32	0.19	0.17	0.04
ROOF GIRDER	**SIZE**	**ELEVATION (FT)**	**D/C Flexure**	**D/C Shear**	**D/C Deflection**
G1	W30x116	64	0.84	0.29	0.30
G2	W36x210	64	0.85	0.27	0.31
G3	W36x359	64	0.81	0.34	0.45
G4	W36x300	64	0.85	0.48	0.36
G5	W36x393	64	0.85	0.47	0.34
G6	W36x359	34 & 36	0.85	0.36	0.45
FLOOR GIRDER	**SIZE**	**ELEVATION (FT)**	**D/C Flexure**	**D/C Shear**	**D/C Deflection**
G7	W33x118	32	0.78	0.28	0.29
G8	W36x393	32	0.74	0.31	0.47
G9	W33x130	32	0.75	0.30	0.29
G10	W36x359	32	0.74	0.34	0.44
G11	W30x235	32	0.63	0.30	0.44
G12	W36x232	32	0.77	0.26	0.32
G13	W18x35	32	0.41	0.11	0.14
G14	W21x44	32	0.63	0.28	0.27
G15	W33X118	32	0.74	0.28	0.28

TABLE 3: TRUSS FRAMING SUMMARY

TRUSS FRAMING SUMMARY		
TRUSS DETAILS	TRUSS T1	TRUSS T2
LENGTH (FT)	90	90
NO. OF PANELS	14	17
PANEL SPACING (FT)	6.4286	5.2941
TOP CHORD	W14x342	W14x342
BOTTOM CHORD	W14x342	W14x342
VERTICAL WEB	W12x170	W12x170
DIAGONAL WEB	W12x170	W12x170
CENTER TO CENTER TRUSS DEPTH (IN)	78.46	78.46
TOTAL DEPTH (IN)	96	96
TRUSS DEFLECTION (IN)	2.99	2.57
ELEVATION (FT)	72 & 100	64

TABLE 4: TRUSS DEMAND/CAPACITY RATIO

TRUSS DEMAND/CAPACITY RATIO			
MEMBER TRUSS T1	D/C AXIAL	D/C BENDING	INTERACTION
TOP CHORD	0.809	0.004	0.814
BOTTOM CHORD	0.753	0.005	0.758
COMPRESSION WEB	0.67	N/A	N/A
TENSION WEB	0.78	N/A	N/A
MEMBER TRUSS T2	D/C AXIAL	D/C BENDING	INTERACTION
TOP CHORD	0.687	0.004	0.691
BOTTOM CHORD	0.659	0.004	0.664
COMPRESSION WEB	0.530	N/A	N/A
TENSION WEB	0.630	N/A	N/A

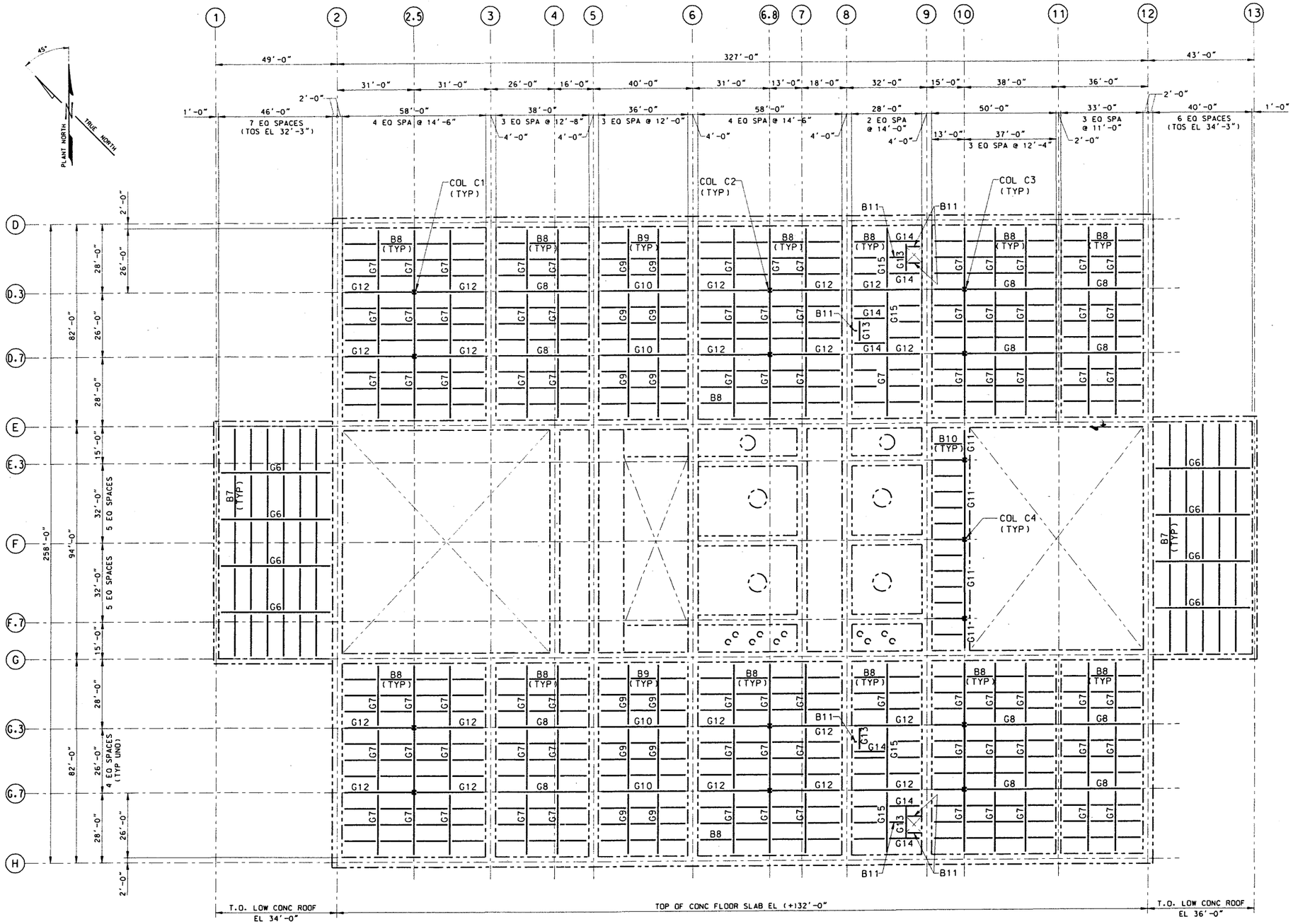
TABLE 5: COLUMN FRAMING SUMMARY

COLUMNS			DEMAND/CAPACITY RATIO
MARK	SIZE	GRID LOCATION	D/C AXIAL
C1	W14x257	D.3-2.5, D.7-2.5, G.3-2.5, G.7-2.5	0.75
C2	W14x145	D.3-6.8, D.7-6.8, G.3-6.8, G.7-6.8	0.70
C3	W14x233	D.3-10, D.7-10, G.3-10, G.7-10	0.82
C4	W14x311	E.3-10, F-10, F.7-10	0.75
C5	W14x176	D.3-11, D.7-11, G.3-11, G.7-11	0.82

7.2 CONCLUSIONS

The results of the calculation shown in the table demonstrate that the structural steel members for the *Canister Receipt and Closure Facility (CRCF)* are adequate for the site specific governing load combinations in accordance with required design codes. Structural steel members are adequately designed to meet safety requirements of the *Canister Receipt and Closure Facility (CRCF)*. The final structural design will include all revised loads from the Tier II analysis to demonstrate the safety requirements of the structure.

The results of this calculation are adequate for the Tier 1 committed design, in support of the License Application.

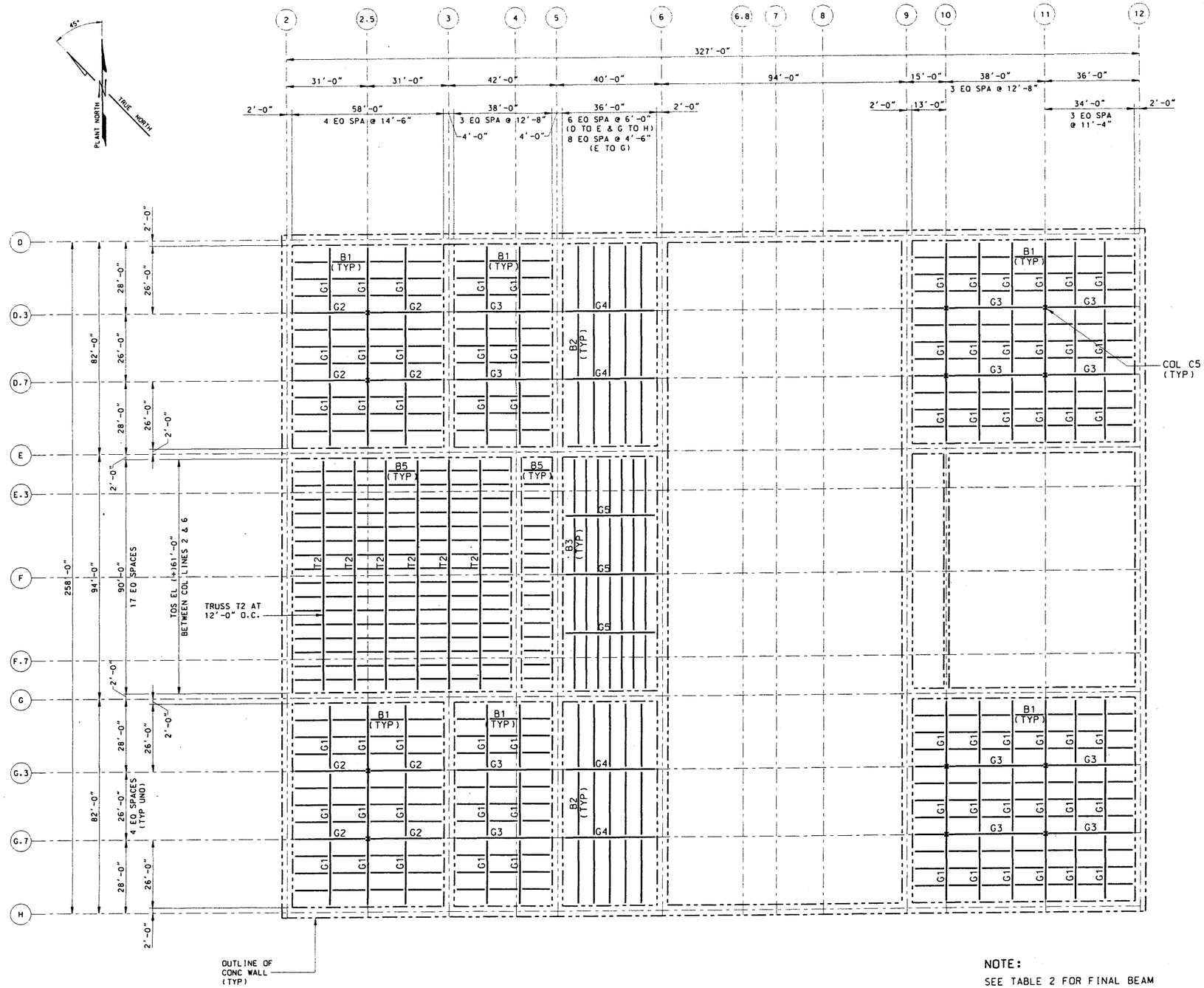


NOTE:

SEE TABLE 2 FOR FINAL BEAM AND GIRDER SELECTION.

SEE TABLE 5 FOR FINAL COLUMN SELECTION

SECOND FLOOR FRAMING PLAN AT TOS EL (+)30'-3 " (TYP UNO)
AND LOW ROOF FRAMING AT TOS EL (+)32'-3" & (+)34'-3"



ROOF FRAMING PLAN AT TOS EL (+)62'-3" (TYP UNO)

TOP OF CONC ROOF SLAB AT EL (+)64'-0"

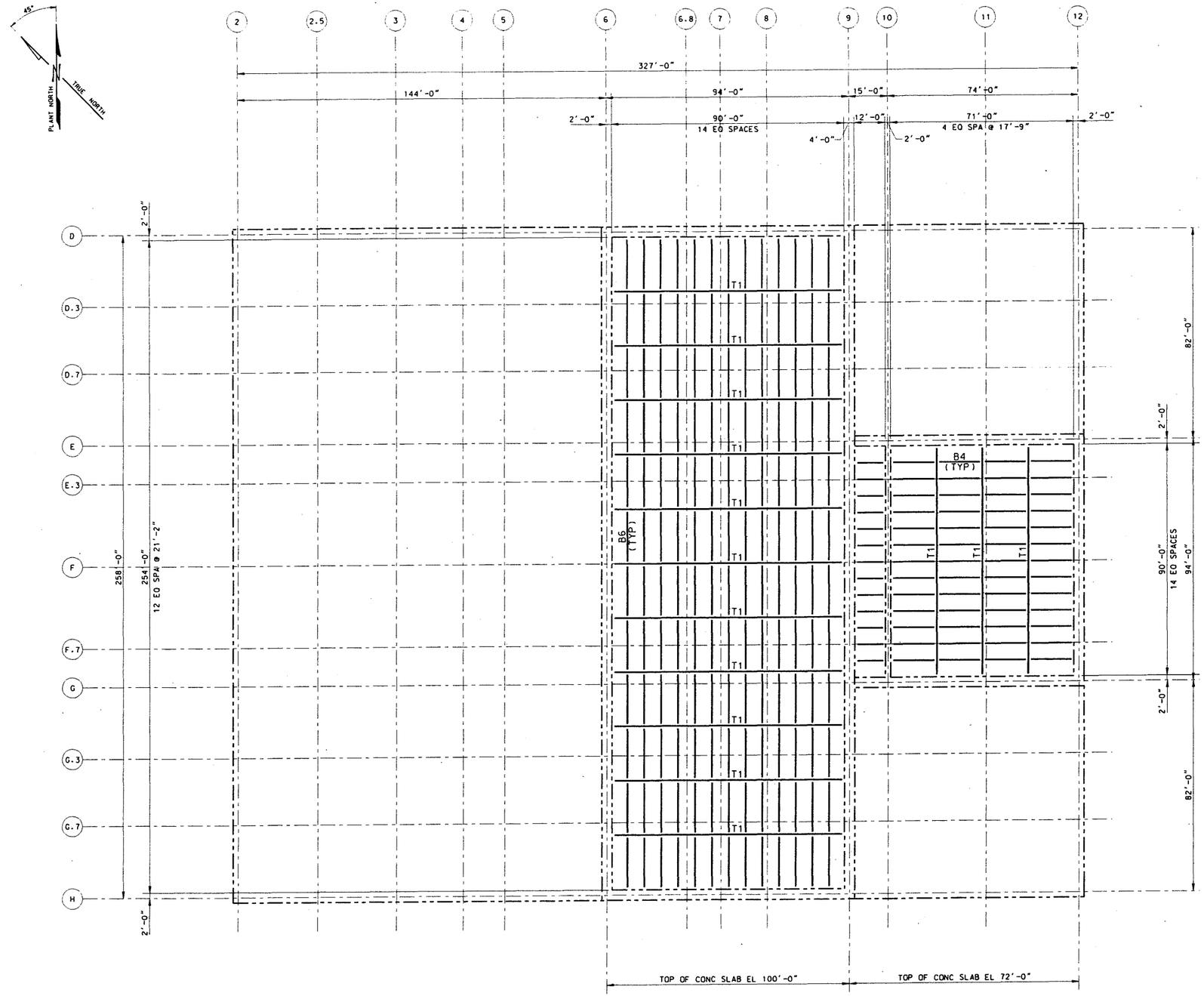
NOTE:

SEE TABLE 2 FOR FINAL BEAM AND GIRDER SELECTION.

SEE TABLE 3 FOR FINAL TRUSS DESIGN

SEE TABLE 5 FOR FINAL COLUMN SELECTION

DOCUMENT IDENTIFIER
060-SSC-CR00-00300-000-00B



NOTE:

SEE TABLE 2 FOR FINAL BEAM AND GIRDER SELECTION.

SEE TABLE 3 FOR FINAL TRUSS DESIGN

ROOF FRAMING PLAN AT TOS EL (+)98'-3" AND EL (+)70'-3"

TOP OF CONC ROOF SLAB AT EL (+)100'-0" AND EL (+)72'-0" AS NOTED

BSC

Calculation/Analysis Change Notice

Complete only applicable items.

1. QA: QA SKG 5/14/08
2. Page 1 of 1

3. Document Identifier:	4. Rev.:	5. CACN:
060-SSC-CR00-00300-000	00B	001

6. Title:
CRCF Interior Structural Steel Design

ENG.20080516.0001

7. Reason for Change:
While evaluating extended conditions for CR 11896 it was discovered that reference 2.2.7 which is listed as not used has been referenced in the body of the calculation.

8. Supersedes Change Notice: ☐ Yes If, Yes, CACN No.: _____ ☒ No

9. Change Impact:

Inputs Changed:	☐ Yes	☒ No	Results Impacted:	☐ Yes	☒ No
Assumptions Changed:	☐ Yes	☒ No	Design Impacted:	☐ Yes	☒ No

10. Description of Change:
Revise reference 2.2.7 to:

2.2.7 See Reference 2.2.6 and 2.2.19

11. REVIEWS AND APPROVAL

Printed Name	Signature	Date
11a. Originator: Surendra Goel	[signature]	5/14/08
11b. Checker: Gopal Rao	[signature]	5/14/08
11c. EGS: Thomas Frankert	[signature]	5/15/08
11d. DEM: Raj Rajagopal	[signature]	5/15/08
11e. Design Authority: Barbara Rusinko	[signature]	5/16/08