BSC

Design Calculation or Analysis Cover Sheet

Complete only applicable items.

1. QA: QA
2. Page 1

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Attachments		Total Number of Pages
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Attachment C.	WHF Pool Room Truss Free Body Diagram	2

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)
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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.

CONTENTS

ACRONYMS

BDBGM	Beyond Design Basis Ground Motion
CL	Column Line
DC	Demand to Capacity Ratio
El.	Elevation
ITS	Important to Safety
PDC	Project Design Criteria (Ref. 2.2.1)
SADA	Seismic Analysis and Design Approach (Ref. 2.2.3)
SPA	Spaces
UNO	Unless Noted Otherwise
WHF	Wet Handling Facility

1. PURPOSE

The purpose of this calculation is to perform a preliminary design of the structural steel framing that supports the reinforced concrete floor and roof slabs of the Wet Handling Facility (WHF). The preliminary design of the steel framing system includes steel floor decking and structural steel beams, girders, trusses, and columns.

2. REFERENCES

2.1 PROCEDURES/DIRECTIVES

2.1.1 BSC 2007. EG-PRO-3DP-G04B-00037, Rev. 8, *Calculations and Analyses.* Las Vegas, Nevada. Bechtel SAIC Company. ACC: ENG.20070420.0002.

2.1.2 BSC (Bechtel SAIC Company) 2007. IT-PRO-0011 Rev.4, ICN0. *Software Management.* Las Vegas, Nevada, Bechtel SAIC Company. ACC: DOC20070319.0016.

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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-MGR0-00300-000-080-000. Las Vegas, Nevada: Bechtel SAIC Company. ACC: ENG.20061023.0002.

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2.2.8 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]

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2.2.10 AISC (American Institute of Steel Construction) 2005. *Steel Construction Manual.* 13th Edition. Chicago, Illinois: American Institute of Steel Construction. TIC: 258106. [DIRS 176291]

2.2.11 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]

2.2.12 United Steel Deck 2006. United Steel Deck, Steel Decks for Floors and Roofs, Design Manual and Catalog of Products. Catalog #303-16. Summit, New Jersey: United Steel Deck. TIC: 259011. [DIRS 178703]

2.2.13 Not used.

2.2.14 Not used.

2.2.15 Not used.

2.2.16 Not used.

2.2.17 Not used.

2.2.18 Not used.

2.2.19 BSC (Bechtel SAIC Company) 2007, *Wet Handling Facility (WHF) Mass Properties* 050-SYC-WH00-00300-000-00B. Las Vegas, Nevada: Bechtel SAIC Company. ACC: ENG.20070326.0001.

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2.2.25 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]

2.2.26 BSC (Bechtel SAIC Company) 2007. *Drawing Change Notice: Wet Handling Facility Cask Handling Crane Mechanical Equipment Envelope* 050-MJ0-HM00-00101-000-00A-DCN002. Las Vegas, Nevada: Bechtel SAIC Company. ACC: ENG.20070308.0020.

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 detailed design of the Wet Handling Facility (WHF). Currently, document numbers have not been assigned for these drawings.

3. ASSUMPTIONS

3.1 ASSUMPTIONS REQUIRING VERIFICATION

3.1.1 Building Dimensions

The WHF plans and sections from the plant design model as of September 20, 2006 (Ref. 2.2.4, 2.2.5, 2.2.6, and 2.2.7) are the basis of the structural steel framing layout. Those WHF plans and sections have been superseded by Ref. 2.2.21, 2.2.22, 2.2.23, and 2.2.24; however, the September 20, 2006 sketches are used in this calculation with the exceptions noted in Assumptions 3.1.12, 3.1.13, and 3.1.14.

Rationale: With the exceptions noted in subsequent assumptions, there are no changes that have a significant impact on the WHF structural steel framing. This assumption will be verified in Tier 2 design. This assumption is being tracked in CalcTrac.

This assumption is used in Sections 6.2 and 6.3.

3.1.2 Structural Steel Framing Dead Loads

Floors at El 20' and 40'		40 psf
Roofs at El. 80' and 100'		60 psf
Roof supported by trusses	Additional	40 psf

Rationale: Structural steel represents a small fraction of the total mass of the WHF structure. Actual steel weights will be used as the design matures in the detailed design phase of the project. The 40 psf floor load is consistent with the WHF Mass Properties Calculation, Ref. 2.2.19. An additional amount of 20 psf (60 psf total) was added for the roof loads to account for the heavy girders required for the long spans. For the areas supported by heavy trusses, 40 psf additional steel dead load is used (total 100 psf). This assumption will be verified in Tier 2 design. This assumption is being tracked in CalcTrac.

This assumption is used in Sections 6.2 and 6.3.

3.1.3 Equipment Dead Loads

Floors at El 20' and 40'	100 psf
Roofs at El. 80' and 100'	10 psf

Equipment dead loads include HVAC equipment, electrical equipment, cable trays, piping, etc.

Rationale: The 10 psf and 100 psf dead loads are conservative assumptions for this type of structure. Actual equipment weights will be used as the design matures in the detailed design phase of the project. These assumed dead loads are consistent with the WHF Mass Properties Calculation, Ref. 2.2.19. This assumption will be verified in Tier 2 design. This assumption is being tracked in CalcTrac.

This assumption is used in Sections 6.2 and 6.3.

3.1.4 Roofing Dead Load

Roof El. 80' & 100' 55 psf.

Rationale: This is a reasonable assumption that allows for a lightweight concrete fill material to be applied over the concrete slab with an average thickness of 6 inches as well as membrane roofing material. This assumed dead load is consistent with the WHF Mass Properties Calculation, Ref. 2.2.19. This assumption will be verified in Tier 2 design. This assumption is being tracked in CalcTrac.

This assumption is used in Sections 6.2 and 6.3.

3.1.5 Live Load

Floors at El 20' and 40' 100 psf

Roofs at El. 80' and 100' 40 psf

Twenty five percent (25 %) of the live loads (25 psf and 10 psf) of these loads is included in the seismic mass for determining seismic loads in the applicable load combinations.

Rationale: These assumed live loads are consistent with the WHF Mass Properties Calculation, Ref. 2.2.19. Addition of 25% of live load to the seismic mass is consistent with Section 7.2.1 of Ref. 2.2.3. This assumption will be verified in Tier 2 design. This assumption is being tracked in CalcTrac.

This assumption is used in Sections 6.2 and 6.3.

3.1.6 Truss Depth

Up to 8'-6" deep trusses can be used to support the roof slab of the Pool Room.

Rationale: The rail height of the 200-ton crane has been lowered from El. 58'-0" to El. 53'-3", per Ref. 2.2.26. The height of the crane is 14'-0", and an additional 2'-0" clearance is required above the crane. The lowest elevation of the bottom of steel is 69'-3". The El. 80' roof slab is 2'-0" thick, with 3" additional for decking. This allows 8'-6" between the top of the crane clearance envelope and the bottom of the roof slab deck. This assumption will be verified in Tier 2 design. This assumption is being tracked in CalcTrac.

This assumption is used in Section 6.2.

3.1.7 Amplified Slab Acceleration

The amplified slab acceleration to be considered for out of plane seismic loads is 2 times the floor vertical acceleration obtained from the WHF Seismic Analysis (Ref 2.2.20).

Rationale: The Tier-1 Seismic Analysis models do not include the effects of vertical floor flexibility, i.e. the floors are considered as rigid diaphragms. A multiplication factor of 2.0 over the vertical acceleration at the supporting walls to account for amplification due to slab flexibility is assumed to be a reasonable estimate. This assumption will be verified in Tier 2 seismic analysis where floor slab flexibility effects will be considered. This assumption is being tracked in CalcTrac.

This assumption is used in Section 6.2.

3.1.8 Truss Tension Cord Bracing

Truss bottom cords are braced laterally at each panel point.

Rationale: This defines the unbraced length for truss vertical compression members. This assumption will be verified in Tier 2 design. This assumption is being tracked in CalcTrac.

This assumption is used in Section 6.2.

3.1.9 Not used.

3.1.10 Blocking

Lateral bracing for the bottom flanges of the steel roof and floor beams is not designed in this calculation, but will be provided in detailed design as required for uplift forces.

Rationale: During a seismic event, net uplift forces could be induced in the floor and roof slabs, resulting in compression in the bottom flanges of the supporting steel beams. Blocking provides lateral support so that the required moment capacity can be developed in the beams. No bottom flange bracing is provided in this preliminary design. This assumption will be verified in Tier 2 design. This assumption is being tracked in CalcTrac.

This assumption is used in Section 6.2

3.1.11 Not used.

3.1.12 Crane Maintenance Area Slab

The Crane Maintenance Area slab at CL C/D - CL 2/3 is an 18” thick slab at El. 40’.

Rationale: This is based on the most current Plant Design Model, as reflected in Ref. 2.2.22 and 2.2.23. Since this change has a significant impact on the steel framing design, the changed configuration is used for this calculation. This assumption will be verified in Tier 2 design. This assumption is being tracked in CalcTrac.

This assumption is used in Section 6.2

3.1.13 Pool Equipment Walls

The walls in the pool equipment rooms (CL C/D – CL 2/3) are as shown on Ref. 2.2.21.

Rationale: This is based on the most current Plant Design Model, as reflected in Ref. 2.2.21. Since this change has a significant impact on the steel framing design, the changed configuration is used for this calculation. This assumption will be verified in Tier 2 design. This assumption is being tracked in CalcTrac.

This assumption is used in Section 6.2

3.1.14 Mezzanine Slab

The Mezzanine slab at CL C/D – CL 2/3 is a 2' thick slab at El. 20'.

Rationale: This is based on the most current Plant Design Model, as reflected in Ref. 2.2.22 and 2.2.23. Since this change has a significant impact on the steel framing design, the changed configuration is used for this calculation. This assumption will be verified in Tier 2 design. This assumption is being tracked in CalcTrac.

This assumption is used in Section 6.2

3.1.15 Decking Continuity

Steel decking supporting floor and roof slabs during construction will be laid out to provided a minimum of 2 continuous spans.

Rationale: Providing 2 continuous spans allows greater spans for decking and thus greater spacing between supporting beams than single span decking. General industry practice is to require a minimum of 3 or 2 continuous spans for this reason, as well as for safety during installation. This assumption will be verified in Tier 2 design. This assumption is being tracked in CalcTrac.

This assumption is used in Section 6.1

3.1.16 Beam Support

Structural steel beams and girders are simply supported by brackets attached to wall embedments, and therefore span the clear distance between walls.

Rationale: This is a standard means of supporting beams and girders. This assumption will be verified in Tier 2 design. This assumption is being tracked in CalcTrac.

This assumption is used in Section 6.1

3.2 ASSUMPTIONS NOT REQUIRING VERIFICATION

3.2.1 Construction

Steel beams, girders, and trusses are designed as Type 2 construction per section Q1.2 (Ref. 2.2.9) 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 are 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 that 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. 2.2.11).

3.2.2 No Composite Action

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.

3.2.3 Lateral Support

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

Rationale: Standard engineering practice.

3.2.4 Not used.

3.2.5 Live Load in Earthquake

Full100% Live Load is used in load combinations that include earthquake load.

Rationale: The SADA Document (Ref. 2.2.3) requires that only 25% of live load be used in load combinations that include earthquake load. Therefore, this is a conservative bounding assumption.

3.2.6 Demand to Capacity Ratio

The Demand Capacity (DC) ratio for steel framing and roof trusses is limited to 0.70.

Rationale: This reduced DC Ratio is applied to provide a margin for the Beyond Design Basis Ground Motion (BDBGM) earthquake.

4. METHODOLOGY

4.1 QUALITY ASSURANCE

This calculation was prepared in accordance with procedure EG-PRO-3DP-G04B-00037 Rev. 08, *Calculations and Analysis* (Ref.2.1.1). The *Basis of Design for the TAD Canister-Based Repository Design Concept,* classifies the WHF structure as ITS (Section 5.1.2 Ref. 2.2.2). Therefore the approved record designation of this calculation is designated QA:QA.

4.2 USE OF SOFTWARE

Word 2003, part of the Microsoft Office 2003 suite of programs, was used in preparation of this document. Microsoft Office 2003 usage is classified as Level 2 as defined in IT-PRO-0011, Software Management, (Ref 2.1.2). Microsoft Office 2003 is listed on the current Software Report. Microsoft Office software is also listed in 000-PLN-MGR0-00200-000, Repository Project Management Automation Plan, (Ref. 2.1.3).

Excel 2003, part of the Microsoft Office 2003 suite, was used in this calculation to perform mathematical computations. Microsoft Office 2003 usage is classified as Level 2 as defined in IT-PRO-0011, Software Management, (Ref 2.1.2). Microsoft Office 2003 is listed on the current Software Report. Microsoft Office software is also listed in 000-PLN-MGR0-00200-000, Repository Project Management Automation Plan, (Ref. 2.1.3). Verification of the Excel computations in this calculation was done using a hand calculator.

MathCad13 was utilized to perform mathematical computations in this calculation. MathCad13 usage is classified as Level 2 as defined in IT-PRO-0011, Software Management,, (Ref 2.1.2). MathCad13 is listed on the current Software Report. The MathCad software is also listed in 000-PLN-MGR0-00200-000, Repository Project Management Automation Plan, (Ref. 2.1.3). Verification of the MathCad13 computations in this calculation was done using a hand calculator.

The software mentioned above was run in a Windows XP Professional operating system on a Dell Optiplex GX620 Pentium D desktop personal computer.

Computations performed using Excel and MathCad13 are based on actual numbers stored in the programs. The numbers shown on the calculations have been rounded off.

4.3 DESIGN APPROACH

The WHF framing plans shown in Attachment A are based on the general arrangement sketches (Ref. 2.2.4 through 2.2.7), as discussed in Assumption 3.1.1 (Note also Assumptions 3.1.12, 3.1.13, and 3.1.14). 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. 2.2.3), and the WHF Mass Properties Calculation (Ref. 2.2.19).

- Determine allowable spans for steel floor decking assuming unshored construction, using the methodology outlined in the steel deck manual of Ref. 2.2.12. This methodology is based on American Iron and Steel Institute (AISI) Specification for the Design of Cold Formed Steel Structural Members. Note that 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 normal and extreme load combinations for the WHF.

- Determine efficient floor and roof framing layout based on maximum center to center beam spacing from the results of the decking calculation described above. Beams and girders are simply supported and span the entire clear distance between supporting walls (Assumption 3.1.16).

- Structural steel will not be used for the El. 32’-0” 4’-thick slabs located between CL A/B and CL 4/7. 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 2.2.11) in accordance with ANSI/AISC N690 (Ref. 2.2.9). The structural steel framing system provides all vertical support for the concrete slabs for all applicable normal and extreme load combinations. The steel framing is designed for construction, normal, extreme load combinations and will remain as permanent framing in the building structural system.

- Verify that deflection criteria are met (Ref. 2.2.9).

- Calculate Demand to Capacity Ratios of structural steel members.

- The response of structural steel roof framing members to tornado missiles will be evaluated in a separate calculation.

5. LIST OF ATTACHMENTS

6. BODY OF CALCULATION

6.1 STEEL FLOOR DECK SPANS

Determine maximum allowable spans for 2' and 1'-6" thick slabs, using the data and methods outlined in Ref. 2.2.12. Use 2 span minimum (Assumption 3.1.15).

6.1.1 Steel Deck Properties

Use 3" deep 16 gauge steel deck, F_y= 33 ksi. The following properties are from Ref. 2.2.12 Pg. 30, 16 Gauge:

$f_{ydeck} := 33000psi$

$S_p := 1.045\frac{in^3}{ft}$ $S_n := 1.045\frac{in^3}{ft}$ Section moduli for deck, +/- moment.

$I := 1.666\frac{in^4}{ft}$ Moment of inertia for decking.

$t := 0.0598in$ $A_s := 1.020in^2$ Thickness and crossectional area of decking.

$w_d := 3.5psf$ Unit weight of decking.

$R_b := 2540\frac{lbf}{ft}$ Interior reaction allowable per foot.

$\phi V_n := 6130\frac{lbf}{ft}$ Design shear strength per foot.

$\phi_b := 0.95$ Bending strength reduction factor Pg. 17, Ref. 2.2.12.

6.1.2 Material Properties

Sect. 4.2.11.6.6 Ref. 2.2.1

$E := 29000ksi$ Steel Modulus of Elasticity

$w_c := 150pcf$ Unit weight of reinforced concrete

6.1.3 Loads

The steel decking is relied on only to carry construction loads. The deck is not considered as reinforcement for service or extreme load combinations. Thus, the deck is NOT ITS.

During concrete placement, the worst case of a uniform live load or a linear live load at midspan is considered (Ref. 2.2.12 and 2.2.1).

$LL_c := 50\text{psf}$ — Uniform live load during concrete placement (Ref. 2.2.1 Sect. 4.2.11.3.16).

$P_c := 150\text{plf}$ — Linear live load during concrete placement applied at midspan (Ref. 2.2.12 Pg. 17).

Calculate concrete weight for 18" and 24" thickness of slabs above deck:

$$w_{c18} := w_c \cdot \frac{18\text{in} + 1.5\text{in}}{12 \cdot \frac{\text{in}}{\text{ft}}} \qquad w_{c18} = 244\,\text{psf}$$

$$w_{c24} := w_c \cdot \frac{24\text{in} + 1.5\text{in}}{12 \frac{\text{in}}{\text{ft}}} \qquad w_{c24} = 319\,\text{psf}$$

6.1.4 Negative Moment

Determine maximum span for negative moment, using a 2-span panel with both spans loaded (Ref. 2.2.12 Pg. 18):

$$M_{neg} := 0.125 \cdot L^2 \cdot \left(1.6 \cdot w_{conc} + 1.4 \cdot LL_c + 1.2 \cdot w_d\right)$$

$$M_{maxneg} := \phi_b \cdot f_{ydeck} \cdot S_n \qquad M_{maxneg} = 32761 \frac{\text{in} \cdot \text{lbf}}{\text{ft}}$$

$$L_{18neg} := \sqrt{\frac{M_{maxneg}}{0.125 \cdot \left(1.6 \cdot w_{c18} + 1.4 LL_c + 1.2 \cdot w_d\right)}} \qquad L_{18neg} = 6.86\ \text{ft}$$

$$L_{24neg} := \sqrt{\frac{M_{maxneg}}{0.125 \cdot \left(1.6 \cdot w_{c24} + 1.4 LL_c + 1.2 \cdot w_d\right)}} \qquad L_{24neg} = 6.11\ \text{ft}$$

6.1.5 Positive Moment

Determine maximum span for positive moment, using a 2-span panel, only 1 span loaded (Ref. 2.2.12 Pgs. 17 & 18), using the linear concentrated construction load (P_c) along midspan:

$$M_{pos} := 0.203L\cdot(1.4\cdot P_c) + 0.096L^2\cdot(1.6\cdot w_{conc} + 1.2\cdot w_d)$$

$$M_{maxpos} := \phi_b\cdot f_{ydeck}\cdot S_p \qquad M_{maxpos} = 32761\,\frac{in\cdot lbf}{ft}$$

<u>For 18" slab</u>:

$$0.203L_{18pos}\cdot(1.4\cdot P_c) + 0.096L_{18pos}{}^2\cdot(1.6\cdot w_{c18} + 1.2\cdot w_d) = M_{maxpos}$$

$$L_{18pos} = 7.95\,ft$$

Check calc - since the equation is solved internally by the MathCAD software, the following is a validation of the results:

$$M_{18pos} := 0.203L_{18pos}\cdot(1.4\cdot P_c) + 0.096L_{18pos}{}^2\cdot(1.6\cdot w_{c18} + 1.2\cdot w_d)$$

$M_{18pos} = 32761\,\frac{in\cdot lbf}{ft}$ This equals $M_{max,}$ therefore, OK

<u>For 24" slab:</u>

$$0.203L_{24pos}\cdot(1.4\cdot P_c) + 0.096L_{24pos}{}^2\cdot(1.6\cdot w_{c24} + 1.2\cdot w_d) = M_{maxpos}$$

$$L_{24pos} = 7.02\,ft$$

Check calc - since the equation is solved internally by the MathCAD software, the following is a validation of the results:

$$M_{24pos} := 0.203L_{24pos}\cdot(1.4\cdot P_c) + 0.096L_{24pos}{}^2\cdot(1.6\cdot w_{c24} + 1.2\cdot w_d)$$

$M_{24pos} = 32761\,\frac{in\cdot lbf}{ft}$ This equals $M_{max,}$ therefore, OK

6.1.6 Web Crippling

Check interior web cripling for 2-span panel, fully loaded, 5" bearing, with 1/3 increase is allowed for temporary loading (Ref. 2.2.12 Pg. 18):

Reaction - interior

$R_b = 2540 \frac{lbf}{ft}$ Allowable interior reaction per ft.

$R_i := 1.25 \cdot L_{web} \cdot \left(w_{conc} + LL_c + w_d\right)$

$R_{all} := R_b \cdot 1.33$

For 18" slab:

$R_{all} = 1.25 \cdot L_{18web} \cdot \left(w_{c18} + LL_c + w_d\right)$

$L_{18web} := 0.80 \cdot \frac{R_{all}}{w_{c18} + LL_c + w_d}$

$L_{18web} = 9.09\ ft$

For 24" slab:

$R_{all} = 1.25 \cdot L_{24web} \cdot \left(w_{c24} + LL_c + w_d\right)$

$L_{24web} := 0.80 \cdot \frac{R_{all}}{w_{c24} + LL_c + w_d}$

$L_{24web} = 7.26\ ft$

6.1.7 Web Shear

Check web shear using double span (Ref. 2.2.12 Pg. 18):

$V := 0.625 \cdot \left(1.6 \cdot w_{conc} + 1.4 \cdot LL_c + 1.2 \cdot w_d\right) \cdot L$

$\phi V_n = 6130 \frac{lbf}{ft}$ Allowable web shear per ft.

For 18" slab:

$$\phi V_n = 0.625\cdot\left(1.6\cdot w_{c18} + 1.4\cdot LL_c + 1.2\cdot w_d\right)\cdot L_{18V}$$

$$L_{18V} := 8.0\cdot\frac{\phi V_n}{8.0\cdot w_{c18} + 7.\cdot LL_c + 6.\cdot w_d}$$

$$L_{18V} = 21.1\ \text{ft}$$

For 24" slab:

$$\phi V_n = 0.625\cdot\left(1.6\cdot w_{c24} + 1.4\cdot LL_c + 1.2\cdot w_d\right)\cdot L_{24V}$$

$$L_{24V} := 8.\cdot\frac{\phi V_n}{8.\cdot w_{c24} + 7.\cdot LL_c + 6.\cdot w_d}$$

$$L_{24V} = 16.8\ \text{ft}$$

6.1.8 Shear / Bending Interaction

Check shear and bending interaction at the interior support, using 2-span configuration.

AISI allowable (Ref. 2.2.12 Pg. 18):

$$\left(\frac{M_{applied}}{\phi M_n}\right)^2 + \left(\frac{V_{applied}}{\phi V_n}\right)^2 \le 1.0$$

For 18" slab:

$$M_{18applied} = 0.125\cdot\left(1.6\cdot w_{c18} + 1.4\cdot LL_c + 1.2\cdot w_d\right)\cdot {L_{18i}}^2$$

To simplify the solution, let

$$C_{18M} := 0.125\cdot\left(1.6\cdot w_{c18} + 1.4\cdot LL_c + 1.2\cdot w_d\right) \qquad C_{18M} = 58.0\,\text{psf}$$

$$\phi M_n := M_{maxneg}$$

$$\phi M_n = 32761\,\frac{\text{in}\cdot\text{lbf}}{\text{ft}}$$

$$V_{18applied} = 0.625\left(1.6 \cdot w_{c18} + 1.4 \cdot LL_c + 1.2 \cdot w_d\right) \cdot L_{18i}$$

Let $C_{18V} := 0.625\left(1.6 \cdot w_{c18} + 1.4 \cdot LL_c + 1.2 \cdot w_d\right)$ $\quad C_{18V} = 290\,psf$

$$\phi V_n = 6130 \frac{lbf}{ft}$$

$$\left(\frac{C_{18M} \cdot L_{18i}^{\,2}}{\phi M_n}\right)^2 + \left(\frac{C_{18V} \cdot L_{18i}}{\phi V_n}\right)^2 = 1.0$$

The positive real soultion determined by MathCAD:

$$L_{18i} := \frac{1}{2 \cdot C_{18M} \cdot \phi V_n} \cdot \left[-\left[2 \cdot C_{18V}^{\,2} \cdot \phi M_n - 2 \cdot \left(C_{18V}^{\,4} \cdot \phi M_n^{\,2} + 4 \cdot C_{18M}^{\,2} \cdot \phi V_n^{\,4} \right)^{\frac{1}{2}} \right] \cdot \phi M_n \right]^{\frac{1}{2}}$$

$L_{18i} = 6.68\ ft$ **For 18" slab**

<u>For 24" slab:</u>

$$M_{24applied} = 0.125 \cdot \left(1.6 \cdot w_{c24} + 1.4 \cdot LL_c + 1.2 \cdot w_d\right) \cdot L_{24i}^{\,2}$$

To simplify the solution, let

$C_{24M} := 0.125 \cdot \left(1.6 \cdot w_{c24} + 1.4 \cdot LL_c + 1.2 \cdot w_d\right)$ $\quad C_{24M} = 73.0\,psf$

$$\phi M_n := M_{maxneg}$$

$$\phi M_n = 32761 \frac{in \cdot lbf}{ft}$$

$$V_{24applied} = 0.625\left(1.6 \cdot w_{c24} + 1.4 \cdot LL_c + 1.2 \cdot w_d\right) \cdot L_{24i}$$

Let $C_{24V} := 0.625\left(1.6 \cdot w_{c24} + 1.4 \cdot LL_c + 1.2 \cdot w_d\right)$ $\quad C_{24V} = 365\,psf$

$$\phi V_n = 6130 \frac{lbf}{ft}$$

To determine maximum span, let

$$\left(\frac{C_{24M} \cdot L_{24i}^{\ 2}}{\phi M_n}\right)^2 + \left(\frac{C_{24V} \cdot L_{24i}}{\phi V_n}\right)^2 = 1.0$$

Solving for L_{24i}

$$L_{24i} := \frac{1}{2 \cdot C_{24M} \cdot \phi V_n} \cdot 2^{\frac{1}{2}} \cdot \left[\left[\left(-C_{24V}^{\ 2}\right) \cdot \phi M_n + \left(C_{24V}^{\ 4} \cdot \phi M_n^{\ 2} + 4 \cdot C_{24M}^{\ 2} \cdot \phi V_n^{\ 4}\right)^{\frac{1}{2}}\right] \cdot \phi M_n\right]^{\frac{1}{2}}$$

$L_{24i} = 5.92\ ft$ **For 24" slab**

6.1.9 Deflection

Check deflection; limit to the lesser of L/180 or 0.75" (Ref. 2.2.12 Pg. 18):

For 18" slab:

$$\frac{L_{18\Delta}}{180} = 0.0054 \cdot \frac{\left(w_{c18} + w_d\right) \cdot L_{18\Delta}^{\ 4}}{E \cdot I}$$

$$L_{18\Delta} := \frac{5}{9 \cdot \left(w_{c18} + w_d\right)} \cdot 6^{\frac{1}{3}} \cdot \left[E \cdot I \cdot \left(w_{c18} + w_d\right)^2\right]^{\frac{1}{3}}$$

$L_{18\Delta} = 11.2\ ft$

$$0.75\text{in} = 0.0054\cdot\frac{\left(w_{c18}+w_d\right)\cdot L_{18\Delta}^{\ 4}}{E\cdot I}$$

$$L_{18\Delta75} := \frac{5}{3\cdot w_{c18}+3\cdot w_d}\cdot 3^{\frac{1}{2}}\cdot\left[\left(w_{c18}+w_d\right)\cdot 2^{\frac{1}{2}}\cdot\left[\left(w_{c18}+w_d\right)\cdot\text{in}\cdot E\cdot I\right]^{\frac{1}{2}}\right]^{\frac{1}{2}}$$

$$L_{18\Delta75} = 11.2\ \text{ft}$$

<u>For 24" slab</u>:

$$\frac{L_{24\Delta}}{180} = 0.0054\cdot\frac{\left(w_{c24}+w_d\right)\cdot L_{24\Delta}^{\ 4}}{E\cdot I}$$

$$L_{24\Delta} := \frac{5}{9\cdot\left(w_{c24}+w_d\right)}\cdot 6^{\frac{1}{3}}\cdot\left[E\cdot I\cdot\left(w_{c24}+w_d\right)^2\right]^{\frac{1}{3}}$$

$$L_{24\Delta} = 10.2\ \text{ft}$$

$$0.75\text{in} = 0.0054\cdot\frac{\left(w_{c24}+w_d\right)\cdot L_{24\Delta}^{\ 4}}{E\cdot I}$$

$$L_{24\Delta75} := \frac{5}{3\cdot w_{c24}+3\cdot w_d}\cdot 3^{\frac{1}{2}}\cdot\left[\left(w_{c24}+w_d\right)\cdot 2^{\frac{1}{2}}\cdot\left[\left(w_{c24}+w_d\right)\cdot\text{in}\cdot E\cdot I\right]^{\frac{1}{2}}\right]^{\frac{1}{2}}$$

$$L_{24\Delta75} = 10.5\ \text{ft}$$

6.1.10 Summary - Maximum Deck Span

The results of the calculation are summarized below. The shortest allowable spans are based on bending/shear interaction (Sect. 6.1.8). These are the longest clear spans allowable for 3" deep, 16 gauge metal decking (S_{min}=1.045 in^3/ft, I_{min}=1.666 in^4/ft), with a minimum of 2 continuous spans:

5.92 ft. for 24" slab

6.68 ft. for 18" slab

Maximum Deck Spans for Limit States (ft)			
Sect.	Limit	24" Slab	18" Slab
6.1.4	Negative Moment	6.11	6.86
6.1.5	Positive Moment	7.02	7.95
6.1.6	Web Crippling	7.26	9.09
6.1.7	Web Shear	16.8	21.1
6.1.8	Shear/Bending Interaction	5.92	6.68
6.1.9	Deflection	10.2	11.2

6.2 DESIGN OF STRUCTURAL STEEL FRAMING

The preliminary framing layout shown in Attachment A is based on a maximum c/c beam spacing of 6.6 ft. for 24" slabs (5.77' span for steel deck, plus a nominal flange width of 0.83' for the supporting beams).

The design of the structural framing is based on the following:

1. No composite action between the concrete slabs and supporting structural steel beams, girders, or trusses is considered.
2. Decking provides full lateral support to top flanges of beams during construction (Assumption 3.2.3).
3. The structural steel framing system provides all vertical support for the concrete slabs and superimposed loads for all applicable service and extreme load combinations.
4. Steel framing is designed as Type 2 per Q1.2, Ref. 2.2.9 (Assumption 3.2.1).
5. Selection of steel members is from Part 1 of Ref. 2.2.10.

6.2.1 MATERIAL PROPERTIES

Concrete and Reinforcement for ITS Structures — Sect. 4.2.11.6.2 Ref. 2.2.1

$f'_c := 5000psi$ — Concrete 28-day strength

$f_y := 60000psi$ — Reinforcing steel yield strength, ASTM A706 Gr. 60

Structural Steel for ITS Structures — Sect. 4.2.11.6.1 Ref. 2.2.1

$F_{y50} := 50ksi$ — W-Shape Yield Strength, ASTM A992

$F_{u50} := 65ksi$ — W-Shape Tensile Strength

$F_{y36} := 36ksi$ — A36 Yield Strength (for plates and other non-W sections), ASTM A36

$F_{u36} := 58ksi$ — A36 Tensile Strength

Structural Analysis/Design Material Properties — Sect. 4.2.11.6.6 Ref. 2.2.1

$E := 29000ksi$ — Steel Modulus of Elasticity

$w_c := 150pcf$ — Unit weight of reinforced concrete

6.2.2 ALLOWABLE STRESSES

Sect. Q1.5.1 Ref. 2.2.9

Using compact sections with continuous lateral support:

$F_b := 0.66 \cdot F_{y50}$ $\quad F_b = 33.0\,\text{ksi}$

$F_v := 0.40 \cdot F_{y50}$ $\quad F_v = 20.0\,\text{ksi}$

$F_t := 0.60 \cdot F_{y50}$ $\quad F_t = 30.0\,\text{ksi}$

AISC N690 (Table Q1.5.7.1 Ref. 2.2.9) allows a 1.6 stress increase with Extreme and Abnormal Severe load combinations (1.4 for shear and 1.5 for axial compression Ref. 2.2.8):

$F_{be} := 1.6 \cdot F_b$ $\quad F_{be} = 52.8\,\text{ksi}$

$F_{ve} := 1.4 \cdot F_v$ $\quad F_{ve} = 28.0\,\text{ksi}$

$F_{te} := 1.6 \cdot F_t$ $\quad F_{te} = 48.0\,\text{ksi}$

$F_{ae} = 1.5\ F_a$ $\quad F_a$ is determined by KL/r

A target DC ratio of 0.70 is to be applied to steel framing design to provide margin for the BDBGM event as discussed in Assumption 3.2.6. Therefore, the allowable stresses become:

$F_{bb} := 0.70 \cdot 1.6 \cdot F_b$ $\quad F_{bb} = 37.0\,\text{ksi}$

$F_{vb} := 0.70 \cdot 1.4 \cdot F_v$ $\quad F_{vb} = 19.6\,\text{ksi}$

$F_{tb} := 0.70 \cdot 1.6 \cdot F_t$ $\quad F_{tb} = 33.6\,\text{ksi}$

$F_{ab} = 0.70 \cdot 1.5\ F_a$ $\quad F_a$ is determined by KL/r

6.2.3 LOADS

Applicable loads are listed below. Lateral Earth Pressure (H), Thermal Loads (T_o and T_a), Fluid Load (F), Operating Pipe Reactions (Ro), and Flood Load (Fa) (Sect. 4.2.11.3 Ref. 2.2.1) are not applicable to the WHF floor and roof slabs. Wind Load (W) and Tornado Wind Loads, (W_t), produce negative pressure on the roof. Since the upward pressure opposes gravity loads, tornado wind and differential pressure loads are conservatively disregarded.

Dead Loads:

$DL_{eqfloor} := 100psf$	Equipment dead load on floor (Assumption 3.1.3)
$DL_{eqroof} := 10psf$	Equipment dead load on roof (Assumption 3.1.3)
$DL_{framing} := 40psf$	Dead load of structural framing (Assumption 3.1.2)
$DL_{framingroof} := 60psf$	Dead load of structural framing (Assumption 3.1.2)
$DL_{truss} := 40psf$	Additional dead load of trusses (Assumption 3.1.2)
$DL_{roofing} := 55psf$	Dead load of roofing (Assumption 3.1.4)

Calculate dead load of concrete slabs:

$$w_{c24} := \frac{24in + 1.5in}{12\frac{in}{ft}} w_c$$

Weight of 24" concrete slab on 3" metal deck

$$w_{c24} = 319\,psf$$

The crane maintenance slab at El. 40 ft. between CL B and CL C is 18 in. thick:

$$w_{c18} := \frac{18in + 1.5in}{12\frac{in}{ft}} w_c$$

Weight of 18" concrete slab on 3" metal deck

$$w_{c18} = 244\,psf$$

Total Floor Dead Load:

$$DL_{floor} := DL_{framing} + DL_{eqfloor} + w_{c24} \qquad DL_{floor} = 459\,psf$$

$$DL_{18floor} := DL_{framing} + DL_{eqfloor} + w_{c18} \qquad DL_{18floor} = 384\,psf$$

Total Roof Dead Load:

$DL_{roof} := DL_{framingroof} + DL_{roofing} + DL_{eqroof} + w_{c24}$ Roofs El. 80' & 100'

$DL_{roof} = 444\,psf$

$DL_{rooftr} := DL_{roof} + DL_{truss}$ Pool Room Truss Roof El. 80'

$DL_{rooftr} = 484\,psf$

Live Loads:

$LL_{floor} := 100psf$ Floor live load (Assumption 3.1.5)

$LL_{roof} := 40psf$ Roof live load (Assumption 3.1.5)

$LL_{const} := 50psf$ Construction live load for concrete placement on deck (Sect. 4.2.11.3.16 Ref. 2.2.1)

$LL_{constP} := 5kip$ Construction concentrated live load for beams, not cumulative (Sect. 4.2.11.3.16 Ref. 2.2.1)

Ash Load:

$\gamma_A := 63pcf$ Density of ash (Sect. 6.1.11 Ref. 2.2.1)

$d_A := 4in$ Depth of ash (Sect. 6.1.11 Ref. 2.2.1)

$A := \gamma_A \cdot d_A$ Ash load

$A = 21\,psf$

Snow Load:

$\gamma_S := 30pcf$ Maximum snow density (Eq. 7-4 Ref. 2.2.25)

$d_S := 6.6in$ Maximum monthly snowfall (Sect. 6.1.1 Ref. 2.2.1)

$p_g := \gamma_S \cdot d_S$ Ground snow load

$p_g = 16\,psf$

$I := 1.2$ Maximum Importance Factor (Table 7-4 Ref. 2.2.25)

$S_n := I \cdot p_g$ Flat roof snow load (Sect. 7.3 Ref. 2.2.25)

$S_n = 20\,psf$

Earthquake Loads:

The following vertical seismic accelerations are from Table 18 of Ref. 2.2.20. To account for the effects of amplified slab acceleration, a factor of 2 times the vertical acceleration is applied per Assumption 3.1.7.

Elevation (ft.)	Vertical Acceleration (g)	Amplified Vertical Acceleration (g)	
40	$S_{z40} := 0.660$	$SA_{z40} := 2 \cdot S_{z40}$	$SA_{z40} = 1.32$
80	$S_{z80} := 0.707$	$SA_{z80} := 2 \cdot S_{z80}$	$SA_{z80} = 1.41$
100	$S_{z100} := 0.880$	$SA_{z100} := 2 \cdot S_{z100}$	$SA_{z100} = 1.76$

Seismic mass consists of full DL + 25% of LL Sect. 7.2.1 Ref.2.2.3

$E_{floor} := SA_{z40} \cdot \left(DL_{floor} + 0.25 \cdot LL_{floor}\right)$ Earthquake vertical load on 24" floors at El. 40 and 20 ft. (conservative for 20 ft.)

$E_{floor} = 639\,psf$

$E_{18floor} := SA_{z40} \cdot \left(DL_{18floor} + 0.25 \cdot LL_{floor}\right)$ Earthquake vertical load on 18" floor at El. 40 ft.

$E_{18floor} = 540\,psf$

$E_{80roof} := SA_{z80} \cdot \left(DL_{roof} + 0.25 \cdot LL_{roof}\right)$ Earthquake vertical load on El. 80 ft. roof.

$E_{80roof} = 642\,psf$

$E_{80rooftr} := SA_{z80} \cdot \left(DL_{rooftr} + 0.25 \cdot LL_{roof}\right)$ Earthquake vertical load on El. 80 ft. roof, w/ add'l truss load.

$E_{80rooftr} = 698\,psf$

$E_{100roof} := SA_{z100} \cdot \left(DL_{roof} + 0.25 \cdot LL_{roof}\right)$ Earthquake vertical load on El. 100 ft. roof.

$E_{100roof} = 799\,psf$

6.2.4 LOAD COMBINATIONS

The following load combinations are derived from the *Project Design Criteria*, Ref. 2.2.1, Sect. 4.2.11.4.6. As discussed above, many of the loads are not applicable. Wind and tornado wind loads on the roof oppose the gravity loads (i.e., uplift), and are therefore disregarded. Since roof live load, L_r at 40 psf, exceeds both A and S_n, both of those loads are disregarded. None of the combinations include A or S_n combined with L_r. Therefore, Load Combination 1, D + L + Lr, encompasses the worst case combination for normal loads; and Load Combination 9, D + L + L_r + E, encompasses the worst case load combination for extreme loads where stress limits are increased (See Sect. 6.2.2).

N and S are used to denote Normal and Extreme load combinations, respectively. The net uplift loads are shown for the extreme load combinations where vertical seismic acceleration with amplification exceeds 1 g. The uplift for the roof slab at El. 80 ft. is used to compute stress reversal in the truss members. The other net uplift forces are shown for information only. Since they are less than the extreme downward load combinations, the steel beams designed for extreme loads combinations can accommodate uplift, provided lateral support is given to the bottom beams flanges as needed. See assumption 3.1.10. Bottom flange bracing will be designed in the detailed design phase if Tier 2 analysis shows uplift. See Assumption 3.2.5 regarding the use of 100% Live Load in the load combination that includes earthquake.

$N_r := DL_{roof} + LL_{roof}$	$N_r = 484\,psf$	Normal roof
$N_{rtr} := DL_{rooftr} + LL_{roof}$	$N_{rtr} = 524\,psf$	Normal truss
$N_{fl} := DL_{floor} + LL_{floor}$	$N_{fl} = 559\,psf$	Normal floor
$N_{18fl} := DL_{18floor} + LL_{floor}$	$N_{18fl} = 484\,psf$	Normal 18" floor
$S_{100r} := DL_{roof} + LL_{roof} + E_{100roof}$	$S_{100r} = 1282\,psf$	Extreme 100' roof
$S_{100rNEG} := DL_{roof} + LL_{roof} - E_{100roof}$	$S_{100rNEG} = -315\,psf$	Extreme 100' roof
$S_{80r} := DL_{roof} + LL_{roof} + E_{80roof}$	$S_{80r} = 1125\,psf$	Extreme 80' roof
$S_{80rNEG} := DL_{roof} + LL_{roof} - E_{80roof}$	$S_{80rNEG} = -158\,psf$	Extreme 80' roof
$S_{80rtr} := DL_{rooftr} + LL_{roof} + E_{80rooftr}$	$S_{80rtr} = 1222\,psf$	Extreme at truss
$S_{80rtrNEG} := DL_{rooftr} + LL_{roof} - E_{80rooftr}$	$S_{80rtrNEG} = -174\,psf$	Extreme at truss
$S_{fl} := DL_{floor} + LL_{floor} + E_{floor}$	$S_{fl} = 1197\,psf$	Extreme 24" floor
$S_{flNEG} := DL_{floor} + LL_{floor} - E_{floor}$	$S_{flNEG} = -79.8\,psf$	Extreme 24" floor
$S_{18fl} := DL_{18floor} + LL_{floor} + E_{18floor}$	$S_{18fl} = 1023\,psf$	Extreme 18" floor

The extreme load combinations will control the design if the ratio of the extreme load (S) to the normal load (N) exceeds 1.6, the *Stress Limit Coefficient* provided by AISC N690 Table Q1.5.7.1, Ref. 2.2.9 for extreme load combinations.

$$\frac{S_{100r}}{N_r} = 2.7 \qquad \frac{S_{80r}}{N_r} = 2.3 \qquad \frac{S_{80rtr}}{N_{rtr}} = 2.3 \qquad \frac{S_{fl}}{N_{fl}} = 2.1 \qquad \frac{S_{18fl}}{N_{18fl}} = 2.1$$

In all cases the ratio exceeds 1.6; therefore, the extreme load combinations govern. Design for the extreme load combinations envelopes the normal load combinations.

6.2.5 ROOF TRUSS DESIGN - POOL ROOM

The Pool Room roof spans 100 ft. between the north and south walls. The total length of the room is 262 ft. The top of crane rail is at El. 53'-3" (Ref.2.2.26). The crane height is 14 ft, and 2 ft. clearance is required between the top of crane and bottom of truss steel. Thus, the elevation for the bottom of truss steel is 53.25' + 14' + 2' = 69.25'.

The top of concrete of the roof is at El. 80 ft. The 24" slab on 3" deck totals 2'-3" thickness. The bottom of the slab is at El. 80' - 2.25' = El. 77.75 ft. The truss depth (out to out) is therefore limited to 8.5 ft. maximum. A truss depth of 8.25 ft. is used for design to allow for deflection and construction tolerances. See Assumption 3.1.6.

The 3" deck will have a clear span of 5.77 ft. (less than 5.92' maximum, See Sect. 6.1.10). The supporting beam flange width can be added to obtain the c/c spacing of support beams. A W12x53 beam has a flange width of 10". Adding the flange width to the 5.77' deck clear span gives a c/c beam spacing of 6.60 ft. With 100 ft. between the wall faces, the truss will span slightly less than 99 ft., being supported by corbels or embedments. The 99 ft. truss span / 15 spaces gives 6.6 ft. between support purlins. This is the spacing that is used for the truss panel points. There are 14 panel points where roof loads are applied to the truss. The ends of the decking at the walls are supported directly by the walls.

It has been determined that 15 trusses at a c/c spacing of 16.4 ft. will be used. See Attachments A and C.

Truss Loads for Extreme Load Combination: D + L + E

Determine the concentrated load, P_p, to be applied at each panel point of the truss:

$W := 6.60\text{ft}$ Width of slab strip = beam spacing

$L := 16.4\text{ft}$ Length of slab strip = truss spacing

$P_p := (W \cdot L) \cdot S_{80rtr}$ $P_p = 132\,\text{kip}$

There are 14 panel points where roof loads are applied. Thus, the end reactions are:

$R_{truss} := \frac{14 \cdot P_p}{2}$ $\quad\quad R_{truss} = 926\,kip$

To determine the effective depth, d_{truss}, 19" deep top and bottom chords are considered:

$d_{truss} := 8.25ft - 19in$ $\quad\quad d_{truss} = 6.67\ ft$

The spacing of panel points, s_p, is:

$s_p := \frac{99ft}{15}$ $\quad\quad s_p = 6.60\ ft$

6.2.5.1 Design Truss Diagonal Members - Extreme Loads

See Attachment C for the truss free body diagram. The vertical and horizontal components of the diagonal truss vectors are:

$h_d := \frac{s_p}{\sqrt{{s_p}^2 + {d_{truss}}^2}}$ $\quad\quad h_d = 0.7035$

$v_d := \frac{d_{truss}}{\sqrt{{s_p}^2 + {d_{truss}}^2}}$ $\quad\quad v_d = 0.7107$

The tension in the diagonals closest to the walls is thus:

$T_{d1} := \frac{R_{truss}}{v_d}$ $\quad\quad T_{d1} = 1303\,kip$

Crossectional area required using W-Shape or WT is (F_y=50 ksi):

$F_{tb} = 33.6\,ksi$ $\quad\quad$ Allowable tensile stress in truss member w/ 0.70 DC Ratio

$A_{sd1} := \frac{T_{d1}}{F_{tb}}$ $\quad\quad A_{sd1} = 38.8\,in^2$ $\quad\quad$ 2 WT6X68 A_s=40.0 in^2

The maximum tensile force in the other diagonal members is calculated in a similar manner, deducting 1 panel point load (P_p) for each incremental panel progressing toward the center of the truss. Numbering the diagonals as d1 (as above), d2, d3, ..., the tensile force and required steel areas are determined as follows:

$$T_{d2} := \frac{R_{truss} - 1 \cdot P_p}{v_d} \qquad T_{d2} = 1117\,\text{kip}$$

$$A_{sd2} := \frac{T_{d2}}{F_{tb}} \qquad A_{sd2} = 33.2\,\text{in}^2$$

$$T_{d3} := \frac{R_{truss} - 2 \cdot P_p}{v_d} \qquad T_{d3} = 931\,\text{kip}$$

$$A_{sd3} := \frac{T_{d3}}{F_{tb}} \qquad A_{sd3} = 27.7\,\text{in}^2$$

The force and required steel area for each diagonal is calculated in a similar manner and tabulated below. To simplify the truss configuration, all members are the same.

The Demand:Capacity Ratios for the truss diagonals are calculated as follows:

$$A_{s2WT6x68} := 40.0\text{in}^2 \qquad F_{te} = 48.0\,\text{ksi}$$

$$f_{t2WT6x68} := \frac{T_{d1}}{A_{s2WT6x68}} \qquad f_{t2WT6x68} = 32.6\,\text{ksi}$$

$$DC_{2WT6x68} := \frac{f_{t2WT6x68}}{F_{te}} \qquad DC_{2WT6x68} = 0.68$$

The DC Ratios are calculated in the same manner for the other truss diagonals and tabulated below:

Roof Truss Diagonals Forces and DC Ratios - Extreme Load						
Panel	Force (kips)	Min Area (in^2)	WT Sections (back-to-back)	Actual Area	f_t (ksi)	DC Ratio
1	1303	38.8	2 WT 6X68	40.0	32.6	0.68
2	1117	33.2	2 WT 6X68	40.0	27.9	0.58
3	931	27.7	2 WT 6X68	40.0	23.3	0.48
4	744	22.2	2 WT 6X68	40.0	18.6	0.39
5	558	16.6	2 WT 6X68	40.0	14.0	0.29
6	372	11.1	2 WT 6X68	40.0	9.3	0.19
7	186	5.5	2 WT 6X68	40.0	4.7	0.10
8	0	n/a	2 WT 6X68	n/a	n/a	n/a

6.2.5.2 Uplift for Truss Diagonal Members - Extreme Loads

Since the amplified vertical acceleration can result in a net uplift load, the diagonal members can go into compression. Anchorage of trusses will be confirmed during detailed design.

$S_{80rtrNEG} = -174\,psf$ Uniform uplift for extreme load comb., Sect. 6.2.4

$P_{pNEG} := (W \cdot L) \cdot S_{80rtrNEG}$ $P_{pNEG} = -18.9\,kip$ Max. uplift at panel point

$R_{trussNEG} := \dfrac{14 \cdot P_{pNEG}}{2}$ $R_{trussNEG} = -132\,kip$ Max. uplift reaction

The compressive forces in the diagonal members are calculated in the same manner as shown above, using the uplift panel point and reaction loads. The resulting forces are listed below.

The maximum unbraced length of the diagonals is the distance between working points minus 2 ft., to account for truss cords and gussets. See Attachment B. Therefore;

$L_d := \sqrt{s_p^{\,2} + d_{truss}^{\,2}} - 2ft$ $L_d = 88.6\,in$ Unbraced length of diagonal

$K := 1.0$ Effective length factor

$r_{WT6X68} := 1.59in$ Minimum radius of gyration for WT6X68

$$\frac{K \cdot L_d}{r_{WT6X68}} = 56$$

$F_{aWT6X68} := 23.39\text{ksi}$ Allowable axial stress per Table Q3-50 Ref. 2.2.9

$F_{aeWT6X68} := 1.5 \cdot 0.7 F_{aWT6X68}$ Allowable axial stress for extreme load condition Table Q1.5.7.1 Ref. 2.2.9, with 0.7 D/C ratio

$F_{aeWT6X68} = 24.6\,\text{ksi}$

The compression in the diagonals closest to the walls is thus:

$C_{d1} := \dfrac{R_{trussNEG}}{v_d}$ $C_{d1} = -186\,\text{kip}$

$A_{sWT6X68} := 20.0\text{in}^2$

$f_{aeWT6X68} := \dfrac{-C_{d1}}{2 \cdot A_{sWT6X68}}$ $f_{aeWT6X68} = 4.65\,\text{ksi}$is < $F_{aeWT6X68} = 24.6\,\text{ksi}$

OK

The results for all diagonal members are summarized below:

Roof Truss Diagonals - Extreme Uplift Load									
Panel	Force (kips)	Force per Member (kips)	WT Sections (back-to-back)	Radius of Gyration (in)	KL/r	F_{ae} (ksi)	A_s (in^2)	f_{ae} (ksi)	
1	-186	-93.0	2 WT 6X68	1.59	56	24.6	20.0	4.65	OK
2	-159	-79.7	2 WT 6X68	1.59	56	24.6	20.0	3.98	OK
3	-133	-66.4	2 WT 6X68	1.59	56	24.6	20.0	3.32	OK
4	-106	-53.1	2 WT 6X68	1.59	56	24.6	20.0	2.66	OK
5	-80	-39.8	2 WT 6X68	1.59	56	24.6	20.0	1.99	OK
6	-53	-26.6	2 WT 6X68	1.59	56	24.6	20.0	1.33	OK
7	-27	-13.3	2 WT 6X68	1.59	56	24.6	20.0	0.66	OK
8	0	0.0	2 WT 6X68	1.59	56	24.6	20.0	0.00	OK

For all diagonal members, axial stress is well below allowable for Extreme uplift load.

6.2.5.3 Design Truss Vertical Members - Extreme Loads

Each vertical member will support an axial compressive force equal to the vertical component of the force in the diagonal member that is on the support-side of the vertical member, i.e., the diagonal that connects to the bottom of the vertical member. The trusses are assumed to be bottom-supported, thus the end verticals will support the total reaction. See Attachment C. The compressive force in each vertical member is calculated as follows:

$C_{v1} := R_{truss}$ $\quad C_{v1} = 926\,kip$

$C_{v2} := R_{truss}$ $\quad C_{v2} = 926\,kip$

$C_{v3} := R_{truss} - 1 \cdot P_p$ $\quad C_{v3} = 794\,kip$

$C_{v4} := R_{truss} - 2 \cdot P_p$ $\quad C_{v4} = 661\,kip$

$C_{v5} := R_{truss} - 3 \cdot P_p$ $\quad C_{v5} = 529\,kip$

$C_{v6} := R_{truss} - 4 \cdot P_p$ $\quad C_{v6} = 397\,kip$

$C_{v7} := R_{truss} - 5 \cdot P_p$ $\quad C_{v7} = 265\,kip$

$C_{v8} := R_{truss} - 6 \cdot P_p$ $\quad C_{v8} = 132\,kip$

Although the rotation at the top of the vertical member will be somewhat restrained by the concrete slab, a K value of 1.0 is conservatively assumed. The bottom panel points must be braced (Assumption 3.1.8).

$K := 1.0$

The length is less than 6.7 ft. between centers of the top and bottom truss chords. Using 6.7 ft., the effective length is:

$L_{vert} := d_{truss}$ $\quad L_e := K \cdot L_{vert}$ $\quad L_e = 6.7\ ft$

W12 sections will be considered for vertical members. Weak axis radius of gyration values range from ~3.0 in to ~4.0 in for the reasonable range of W12 sections to be considered. The range of KL/r values is:

$r_{min} := 3.0in$ $\quad r_{max} := 4.0in$

$\frac{K \cdot L_{vert}}{r_{min}} = 27$ $F_{amin} := 27.52ksi$ Table Q3-50, Ref. 2.1.10

$\frac{K \cdot L_{vert}}{r_{max}} = 20$ $F_{amax} := 28.30ksi$

Use 28ksi for preliminary sizing, and verify:

$F_a := 28ksi$ $F_{ab} := 0.70 \cdot 1.5 \cdot F_a$ $F_{ab} = 29.4\,ksi$

For members V1 - V8 (V1 & V2 have equal loads):

$A_{sv1} := \frac{C_{v1}}{F_{ab}}$ $A_{sv1} = 31.5\,in^2$

Try W12X120 $A_{sW12X120} := 35.3in^2$

$r_{W12X120} := 3.16in$ $\frac{K \cdot L_e}{r_{W12X120}} = 25$ $F_{aW12X120} := 27.75 \cdot ksi$

$F_{aeW12X120} := 0.70 \cdot 1.5 \cdot F_{aW12X120}$ $F_{aeW12X120} = 29.1\,ksi$

$f_{aeW12X120} := \frac{C_{v1}}{A_{sW12X120}}$ $f_{aeW12X120} = 26.2\,ksi$ **OK - use W12X120**

Use the same section for all vertical members to simplify truss configuration. It is not necessary to check uplift in vertical members. Uplift would put the members in tension.

The Demand:Capactiy Ratios for the truss verticals are calculated as follows:

$DC_{W12x120} := \frac{f_{aeW12X120}}{1.5 \cdot F_{aW12X120}}$ $DC_{W12x120} = 0.63$

The DC ratios for the other vertical members are calculated in the same manner and tabulated below:

Truss Verticals Forces & DC Ratios					
Panel	Compression (Kips)	Section	Steel Area (in^2)	f_{ae} (ksi)	DC Ratio
1	926	W12x120	35.3	26.2	0.63
2	926	W12x120	35.3	26.2	0.63
3	794	W12x120	35.3	22.5	0.54
4	661	W12x120	35.3	18.7	0.45
5	529	W12x120	35.3	15.0	0.36
6	397	W12x120	35.3	11.2	0.27
7	265	W12x120	35.3	7.5	0.18
8	132	W12x120	35.3	3.7	0.09

6.2.5.4 Design Truss Bottom Chord Members - Extreme Loads

Maximum tension is in the center panel. The tension force is determined by summing the moments about upper panel point 8 (See Attachment C).

$$\Sigma M_8 := R_{truss} \cdot (7 s_p) - P_p \cdot (1 \cdot s_p + 2 \cdot s_p + 3.s_p + 4 \cdot s_p + 5 \cdot s_p + 6 \cdot s_p)$$

$$\Sigma M_8 = 2.44 \times 10^4 \, kip \cdot ft$$

$$T_8 := \frac{\Sigma M_8}{d_{truss}} \qquad T_8 = 3666 \, kip \qquad F_{tb} = 33.6 \, ksi$$

$$A_{st8} := \frac{T_8}{F_{tb}} \qquad A_{st8} = 109 \, in^2$$

W14x370 has A_s = 109 in^2. **<u>Use W14x370</u>**

The tensile forces in the remaining sections of the bottom chord are calculated in the same manner and summarized below.

The Demand:Capacity Ratios for the bottom chord are calculated as follows:

$$A_{sW14x370} := 109 in^2 \qquad F_{te} = 48.0 \, ksi$$

$$f_{tW14x370} := \frac{T_8}{A_{sW14x370}} \qquad f_{tW14x370} = 33.6 \, ksi$$

$DC_{W14x370} := \frac{f_{tW14x370}}{F_{te}}$ $\qquad DC_{W14x370} = 0.70$

DC Ratios for the other sections of the bottom chord are calculated in the same manner.

Truss Bottom Chord Forces & DC Ratios					
Panel	Tension (Kip)	Section	Steel Area (in^2)	f_t (ksi)	DC Ratio
1	0	W14x370	109	n/a	n/a
2	917	W14x370	109	8.4	0.18
3	1702	W14x370	109	15.6	0.33
4	2357	W14x370	109	21.6	0.45
5	2881	W14x370	109	26.4	0.55
6	3273	W14x370	109	30.0	0.63
7	3535	W14x370	109	32.4	0.68
8	3666	W14x370	109	33.6	0.70

Check Uplift - uplift will cause compression in the bottom chord, equal to the ratio of uplift load to downward load.

$C_8 := T_8 \cdot \frac{S_{80rtrNEG}}{S_{80rtr}}$ $\qquad C_8 = -523\,kip$ $\qquad$ Compression at center of bottom chord

$L := s_p$ $\qquad L = 6.60\ ft$ $\qquad$ Unbraced length

$r_{W14x370} := 4.27in$ $\qquad$ Radius of gyration - weak axis

$\frac{K \cdot L}{r_{W14x370}} = 19$ $\qquad$ Slenderness ratio

$F_a := 28.40ksi$ $\qquad$ Allowable axial stress per Table Q3-50 Ref. 2.2.9

$F_{ae} := 0.7 \cdot 1.5 \cdot F_a$ $\qquad F_{ae} = 29.8\,ksi$ $\qquad$ Allowable axial stress for extreme load condition Table Q1.5.7.1 Ref. 2.2.9

$A_{sW14x370} := 109in^2$

$f_{aeW14x370} := \frac{C_8}{A_{sW14x370}}$ $\qquad f_{aeW14x370} = -4.8\,ksi$ is < $F_{ae} = 29.8\,ksi$

OK for uplift

6.2.5.5 Design Truss Top Chord - Extreme Loads

The maximum compressive force at midspan is determined by summing the moments about the lower panel point 8 (See Attachment C):

$$\Sigma M_8 := R_{truss} \cdot (7 \cdot s_p) - P_p \cdot (1 \cdot s_p + 2 \cdot s_p + 3 \cdot s_p + 4 \cdot s_p + 5 \cdot s_p + 6 \cdot s_p)$$

$$\Sigma M_8 = 2.44 \times 10^4 \, kip \cdot ft$$

$$C_8 := \frac{\Sigma M_8}{d_{truss}} \qquad C_8 = 3666 \, kip$$

This is equal and opposite of T_8 as expected.

The compressive forces in the remaining chord sections are calculated in the same manner and are tabulated below.

Since the steel member has continuous lateral support in both directions, provided by the concrete slab, KL/r is taken as 1. The corresponding allowable compressive stress is:

$$F_a := 29.94ksi$$

Table Q3-50, Ref. 2.1.10

$$F_{ab} := 0.70 \cdot 1.5 \cdot F_a \qquad F_{ab} = 31.4 \, ksi$$

$$A_{sc} := \frac{C_8}{F_{ab}} \qquad A_{sc} = 117 \, in^2$$

W14x398 has $A_s = 117$ in^2

Use W14x398

The Demand:Capacity Ratios for the top truss chord members are calculated as follows:

$$A_{sW14x398} := 117 in^2$$

$$f_{aeW14x398} := \frac{C_8}{A_{sW14x398}} \qquad f_{aeW14x398} = 31.3 \, ksi$$

$$DC_{W14x398} := \frac{f_{aeW14x398}}{1.5 \cdot F_a} \qquad DC_{W14x398} = 0.70$$

The DC Ratios for the remaining sections of the top truss chord are calculated in the same manner and tabulated below:

Truss Top Chord Forces & DC Ratios					
Panel	Comp. (Kip)	Section	Steel Area (in^2)	f_{ae} (ksi)	D/C Ratio
1	925	W14x398	117	7.9	0.18
2	1708	W14x398	117	14.6	0.33
3	2361	W14x398	117	20.2	0.45
4	2883	W14x398	117	24.6	0.55
5	3275	W14x398	117	28.0	0.62
6	3536	W14x398	117	30.2	0.67
7	3666	W14x398	117	31.3	0.70
8	3666	W14x398	117	31.3	0.70

6.2.5.6 Summary - Truss Member Forces for Extreme Load Combination

Forces in the remaining top and bottom cord members are readily obtained by adding the horizontal component of the force in diagonal member (h_d= 0.7035) to the the force in the adjacent horizontal member. Truss member forces are summarized in the following table:

Truss Member Forces - Extreme Load Combination								
Panel	Diagonal		Outside Vertical		Bottom Chord		Top Chord	
	T (Kip)	DC	C (Kip)	DC	T (Kip)	DC	C (Kip)	DC
1	1303	0.68	926	0.63	0	n/a	925	0.18
2	1117	0.58	926	0.63	917	0.18	1708	0.33
3	931	0.48	794	0.54	1702	0.33	2361	0.45
4	744	0.39	661	0.45	2357	0.45	2883	0.55
5	558	0.29	529	0.36	2881	0.55	3275	0.62
6	372	0.19	397	0.27	3273	0.63	3536	0.67
7	186	0.10	265	0.18	3535	0.68	3666	0.70
8	0	n/a	132	0.09	3666	0.70	3666	0.70

6.2.6 ROOF FRAMING DESIGN - EXTREME LOAD COMBINATION
(see Page A-2 for framing plan)

6.2.6.1 Pool Room Roof Beams - CL B-C-1-7

The truss spacing is 16.4 ft. Beams are spaced at 6.60 ft., at the truss panel points.

$W = 6.60\,ft$ $\quad L := 16.4ft$

$S_{80r} = 1125\,psf$ Extreme load combination @ El. 80 ft. (Sect. 6.2.4)

$F_{bb} = 37.0\,ksi$ Allowable bending stress for extreme load comb.

The maximum moment is:

$$M_{rCD} := \frac{W \cdot S_{80r} \cdot L^2}{8} \qquad M_{rCD} = 249.7\,kip \cdot ft$$

Required Section Modulus:

$$S_{CDreq} := \frac{M_{rCD}}{F_{bb}} \qquad S_{CDreq} = 81.1\,in^3 \qquad S_{xW12x72} := 97.4in^3$$

Demand Capacity Ratio:

$$DC_{W12x72} := \frac{M_{rCD}}{S_{xW12x72} \cdot F_{be}} \qquad DC_{W12x72} = 0.58$$

The maximum shear (reaction) is:

$$R_{rCD} := \frac{S_{80r} \cdot L \cdot W}{2} \qquad R_{rCD} = 60.9\,kip$$

$d_{W12x72} := 12.3in$ $\quad t_{wW12x72} := 0.430in$

$$f_{vrCD} := \frac{R_{rCD}}{d_{W12x72} \cdot t_{wW12x72}} \qquad f_{vrCD} = 11.5\,ksi \quad \text{is} < \quad F_{vb} = 19.6\,ksi$$

Moment for Construction Loads - The 5 kip concentrated load is placed at midspan:

$$M_{constCD} := \frac{W \cdot \left(DL_{framingroof} + w_{c24} + LL_{const}\right) \cdot L^2}{8} + \frac{LL_{constP} \cdot L}{4}$$

$M_{constCD} = 116\,ft \cdot kip$

$$S_{constCD} := \frac{M_{constCD}}{F_b} \qquad S_{constCD} = 42.0\,in^3$$

W12x72 has S_x= 97.4 in^3. **OK - Use W12X72**

Note: The construction load case is not checked in the following calculations because it is obvious that the construction load case does not control.

6.2.6.2 Roof Beams CL A-B-1-2, A-B-3-4, C-D-1-2, C-D-3-6

Girders divide the 49 ft. spans across the rooms by 2. As shown in Sect. 6.1, the maximum deck span for a 24" slab is 5.9 ft. Considering a minimum 0.5 ft. flange width, the maximum C/C beam spacing is 6.4 ft. Using that maximum span to layout the roof beams on the building plan, the maximum c/c spacing of the roof and floor beams in the 24" slab areas is 6.25 ft. (See Attachment A).

In these areas, girders divide the distance between the walls in half. Therefore, the length of the beams is:

$L_{rA} := \dfrac{49\text{ft}}{2}$ $\quad L_{rA} = 24.5\ \text{ft}$

$W_{rA} := \dfrac{50\text{ft}}{8}$ $\quad W_{rA} = 6.25\ \text{ft}$ Beam spacing - see layout sketches in Attachment A

$S_{80r} = 1125\,\text{psf}$ Extreme roof load @ El. 80' (Sect. 6.2.4)

$R_{rA} := \dfrac{W_{rA}\cdot S_{80r}\cdot L_{rA}}{2}$ $\quad R_{rA} = 86.2\,\text{kip}$

$M_{rA} := \dfrac{W_{rA}\cdot S_{80r}\cdot L_{rA}^{2}}{8}$ $\quad M_{rA} = 528\,\text{kip}\cdot\text{ft}$

$S_{reqrA} := \dfrac{M_{rA}}{F_{bb}}$ $\quad S_{reqrA} = 171\,\text{in}^3$ $\quad S_{xW24x76} := 176\text{in}^3$

Demand Capacity Ratio:

$DC_{W24x76} := \dfrac{M_{rA}}{S_{xW24x76}\cdot F_{be}}$ $\quad DC_{W24x76} = 0.68$

Shear:

$d_{W24x76} := 23.9\text{in}$ $\quad t_{wW24x76} := 0.440\cdot\text{in}$

$f_{vW24x76} := \dfrac{R_{rA}}{d_{W24x76}\cdot t_{wW24x76}}$ $\quad f_{vW24x76} = 8.19\,\text{ksi}$ is < $F_{vb} = 19.6\,\text{ksi}$

OK - Use W24x76

6.2.6.3 Roof Beams CL A-B-2-3, C-D-2-3, C-D-6-7

Columns cannot be accommodated in these areas. The beams are designed to span the entire 49 ft. distance between walls.

$L_{rB} := 49ft$

$W_{rB} := \frac{50ft}{8}$ $W_{rB} = 6.25\ ft$ Beam spacing

$S_{80r} = 1125\,psf$ Extreme roof load @ El. 80' (Sect. 6.2.4)

$R_{rB} := \frac{W_{rB} \cdot S_{80r} \cdot L_{rB}}{2}$ $R_{rB} = 172\,kip$

$M_{rB} := \frac{W_{rB} \cdot S_{80r} \cdot L_{rB}^{2}}{8}$ $M_{rB} = 2111\,kip \cdot ft$

$S_{reqrB} := \frac{M_{rB}}{F_{bb}}$ $S_{reqrB} = 685\,in^{3}$

$S_{xW36x210} := 719in^{3}$

Demand Capacity Ratio:

$DC_{W36x210} := \frac{M_{rB}}{S_{xW36x210} \cdot F_{be}}$ $DC_{W36x210} = 0.67$

Shear:

$d_{W36x210} := 36.7in$ $t_{wW36x210} := .830in$

$f_{vW36x210} := \frac{R_{rB}}{d_{W36x210} \cdot t_{wW36x210}}$ $f_{vW36x210} = 5.66\,ksi$ is < $F_{vb} = 19.6\,ksi$

OK - Use W36x210

6.2.6.4 Roof Beams CL A-B-4-7 (Roof @ El. 100')

For the reason stated in the previous section, roof beams are designed to span the entire distance between walls. The room length is 114 ft., requiring 17 beams spaced at 6.33 ft. c/c.

$L_{rC} := 49\text{ft}$

$W_{rC} := \dfrac{114\text{ft}}{18}$ $\quad W_{rC} = 6.33\ \text{ft}$ Beam spacing

$S_{100r} = 1282\,\text{psf}$ Extreme roof load @ El. 100' (Sect. 6.2.4)

$R_{rC} := \dfrac{W_{rC} \cdot S_{100r} \cdot L_{rC}}{2}$ $\quad R_{rC} = 199\,\text{kip}$

$M_{rC} := \dfrac{W_{rC} \cdot S_{100r} \cdot L_{rC}^{\ 2}}{8}$ $\quad M_{rC} = 2437\,\text{kip}\cdot\text{ft}$

$S_{reqrC} := \dfrac{M_{rC}}{F_{bb}}$ $\quad S_{reqrC} = 791\,\text{in}^3$

$S_{xW36x232} := 809\text{in}^3$

Demand Capacity Ratio:

$DC_{W36x232} := \dfrac{M_{rC}}{S_{xW36x232} \cdot F_{be}}$ $\quad DC_{W36x232} = 0.68$

Shear:

$d_{W36x232} := 37.1\text{in}$ $\quad t_{wW36x232} := 0.870\text{in}$

$f_{vW36x232} := \dfrac{R_{rC}}{d_{W36x232} \cdot t_{wW36x232}}$ $\quad f_{vW36x232} = 6.16\,\text{ksi}$ is < $\quad F_{vb} = 19.6\,\text{ksi}$

OK - Use W36x232

6.2.6.5 Roof Girders CL A-B-1-2, C-D-1-2

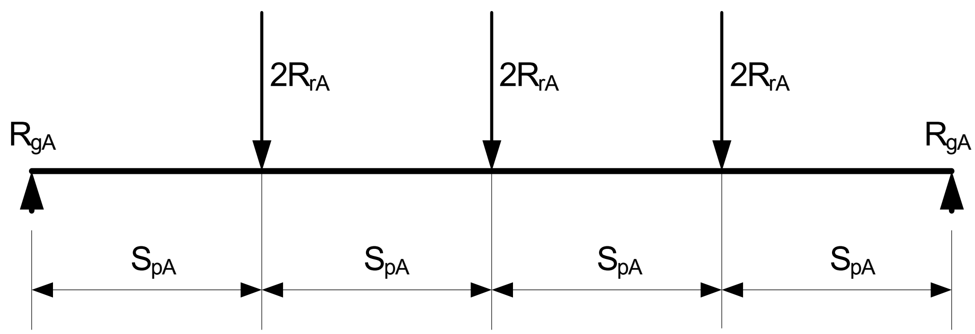
Each of the 2 rooms is 50 ft. in length, divided in half by a column. Seven lines of roof beams are spaced equally.

$L_{gA} := 25ft$ $\quad s_{pA} := \frac{50ft}{8}$ $\quad s_{pA} = 6.25\ ft$

The end reaction of each beam, as calculated in 6.2.6.2:

$R_{rA} = 86.2\,kip$

Six beams frame into each girder, 3 each side. The remaining 2 beams connect directly to the column. Loading is symmetrical.



$R_{gA} := \frac{2 \cdot R_{rA} \cdot 3}{2}$ $\quad R_{gA} = 258\,kip$

$M_{gA} := R_{gA} \cdot 2 \cdot s_{pA} - 2 \cdot R_{rA} \cdot s_{pA}$ $\quad M_{gA} = 2154\,kip \cdot ft$

$S_{gAreq} := \frac{M_{gA}}{F_{bb}}$ $\quad S_{gAreq} = 699\,in^3$

$S_{xW36x210} := 719in^3$

Demand Capacity Ratio:

$DC_{W36x210} := \frac{M_{gA}}{S_{xW36x210} \cdot F_{be}}$ $\quad DC_{W36x210} = 0.68$

Shear:

$d_{W36x210} = 36.7\,in$ $\quad t_{wW36x210} = 0.830\,in$

$f_{vW36x210} := \frac{R_{gA}}{d_{W36x210} \cdot t_{wW36x210}}$ $\quad f_{vW36x210} = 8.5\,ksi$ is < $F_{vb} = 19.6\,ksi$

OK - Use W36x210

6.2.6.6 Roof Columns CL A-B-1-2, C-D-1-2

Both rooms have a column in the center. Two beams and 2 girders frame into each column.

The ceiling height is 37.75 ft from floor to bottom of roof deck. Girders and beams frame into the column providing lateral restraint and reducing the unbraced length. Reduce the effective length by the beam depth (24") less 3", for a net of 1.75 ft. The unbraced length is thus 36.0 ft.

$K = 1.0$ Effective length factor

$L_{colA} := 36.0ft$

$P_{colA} := 2 \cdot R_{gA} + 2 \cdot R_{rA}$ $P_{colA} = 689\,kip$

Assume a weak axis radius of gyration:

$r_{yass} := 4.0in$

$$\frac{K \cdot L_{colA}}{r_{yass}} = 108$$

$F_{aass} := 12.80ksi$ Table Q3-50 Ref. 2.2.9

$$A_{sreq} := \frac{P_{colA}}{0.70 \cdot 1.5F_{aass}} \qquad A_{sreq} = 51.3\,in^2$$

$A_{sW14x176} := 51.8in^2$ $r_{yW14x176} := 4.02in$

$$\frac{K \cdot L_{colA}}{r_{yW14x176}} = 107$$ Table Q3-50 Ref. 2.2.9

$F_{aeW14x176} := 1.5 \cdot 13.04ksi$ $F_{aeW14x176} = 19.6\,ksi$

$$f_{aeW14x176} := \frac{P_{colA}}{A_{sW14x176}} \qquad f_{aeW14x176} = 13.3\,ksi$$

Demand Capacity Ratio

$$DC_{W14x176} := \frac{f_{aeW14x176}}{F_{aeW14x176}} \qquad DC_{W14x176} = 0.68$$

OK - Use W14x176

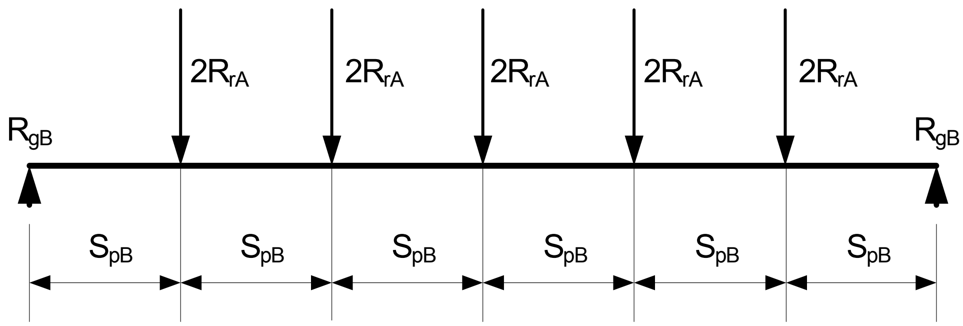
6.2.6.7 Roof Girders CL A-B-3-4, C-D-3-4

Both of the rooms are 36 ft. wide. Five roof beams on each side of the girder are equally spaced. There is not an intermediate column.

$L_{gB} := 36.0ft$ $s_{pB} := \frac{L_{gB}}{6}$ $s_{pB} = 6.00\ ft$

The end reaction of each beam, as calculated in Sect 6.2.6.2:

$R_{rA} = 86.2\,kip$



Ten beams frame into each girder, 5 each side. Since the girder is symmetrically loaded, the end reactions are:

$R_{gB} := \frac{10 \cdot R_{rA}}{2}$ $R_{gB} = 431\,kip$

By inspection of the free body, the maximum moment occurs at the center.

$M_{gB} := R_{gB} \cdot 3s_{pB} - 2 \cdot R_{rA} \cdot \left(1 \cdot s_{pB} + 2 \cdot s_{pB}\right)$ $M_{gB} = 4653\,kip \cdot ft$

$S_{gBreq} := \frac{M_{gB}}{F_{bb}}$ $S_{gBreq} = 1511\,in^3$

$S_{xW36x441} := 1650in^3$

Demand Capacity Ratio:

$DC_{W36x441} := \frac{M_{gB}}{F_{be} \cdot S_{xW36x441}}$ $DC_{W36x441} = 0.64$

Shear:

$d_{W36x441} := 38.9in$ $t_{wW36x441} := 1.36in$

$f_{vW36x441} := \frac{R_{gB}}{d_{W36x441} \cdot t_{wW36x441}}$ $f_{vW36x441} = 8.1\,ksi$ is < $F_{vb} = 19.6\,ksi$

OK - Use W36x441

6.2.6.8 Roof Girders CL C-D-4-6

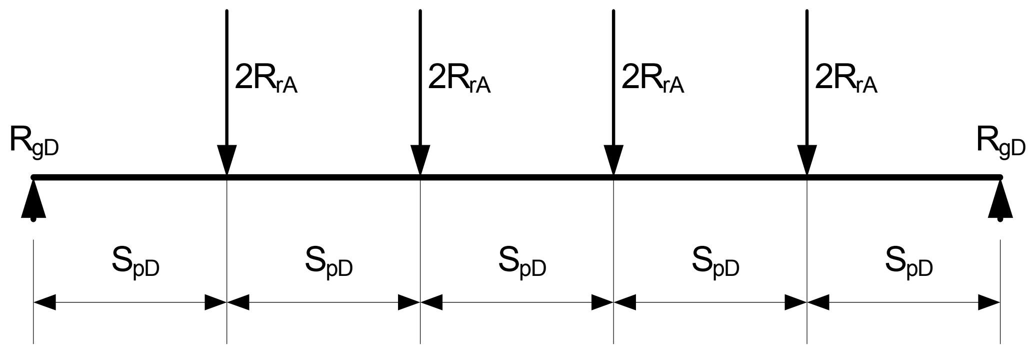
This room is 60 ft. in length, divided in half by a column. One roof beam connects directly to the column from each side. Four roof beams, equally spaced, frame into each side of each girder. There is a total of 18 roof beams in this room.

$L_{gD} := 30ft$ $\quad s_{pD} := \frac{L_{gD}}{5}$ $\quad s_{pD} = 6.0\ ft$

The end reaction of each beam, as calculated in Sect. 6.2.6.2:

$R_{rA} = 86.2\,kip$

Eight beams frame into each girder, 4 each side. The loading is symmetrical. The girder reactions are:



$R_{gD} := \frac{8 \cdot R_{rA}}{2}$ $\quad R_{gD} = 345\,kip$

By inspection, the maximum moment is constant between the 2 center connections.

$M_{gD} := R_{gD} \cdot 2 s_{pD} - 2 \cdot R_{rA} \cdot s_{pD}$ $\quad M_{gD} = 3102\,kip \cdot ft$

$S_{gDreq} := \frac{M_{gD}}{F_{bb}}$ $\quad S_{gDreq} = 1007\,in^3$

$S_{xW36x282} := 1050 in^3$

Demand Capacity Ratio:

$DC_{W36x282} := \frac{M_{gD}}{F_{be} \cdot S_{xW36x282}}$ $\quad DC_{W36x282} = 0.67$

Shear:

$d_{W36x282} := 37.1 in$ $\quad t_{wW36x282} := 0.885 in$

$f_{vW36x282} := \frac{R_{gD}}{d_{W36x282} \cdot t_{wW36x282}}$ $\quad f_{vW36x282} = 10.5\,ksi$ is < $F_{vb} = 19.6\,ksi$

OK - Use W36x282

6.2.6.9 Roof Column CL C-D-4-6

The room has a column in the center. The ceiling height is 37.75 ft from floor to bottom of roof deck. Since the roof girders and beams connect to the column in both planes, the column unbraced length is reduced by the depth of the roof beams. Deducting 1.75 ft. for consideration of support offered by the connecting beams, the unbraced length is 36.00 ft.

$K = 1.0$ $L_{colD} := 36.0ft$

$P_{colD} := 2 \cdot R_{gD} + 2 \cdot R_{rA}$ $P_{colD} = 862\,kip$

Assume a weak axis radius of gyration:

$r_{yass} := 4.0in$

$$\frac{K \cdot L_{colD}}{r_{yass}} = 108$$

$F_{aass} := 12.80ksi$ Table Q3-50 Ref. 2.2.9

$A_{sreq} := \frac{P_{colD}}{0.70 \cdot 1.5F_{aass}}$ $A_{sreq} = 64.1\,in^2$

$A_{sW14x233} := 68.5in^2$ $r_{yW14x233} := 4.10in$

$$\frac{K \cdot L_{colD}}{r_{yW14x233}} = 105$$

$F_{aeW14x233} := 1.5 \cdot 13.53ksi$ $F_{aeW14x233} = 20.3\,ksi$

$f_{aeW14x233} := \frac{P_{colD}}{A_{sW14x233}}$ $f_{aeW14x233} = 12.6\,ksi$

Demand Capacity Ratio

$DC_{W14x233} := \frac{f_{aeW14x233}}{F_{aeW14x233}}$ $DC_{W14x233} = 0.62$

OK - Use W14x233

6.2.7 FLOOR FRAMING DESIGN - EXTREME LOAD COMBINATION

(See Page A-3 for floor framing plan)

As calculated in Sect. 6.2.4, the extreme load combinations for the floors are:

$S_{fl} = 1197\,psf$ Typical 24" floor slabs

$S_{18fl} = 1023\,psf$ Crane Maintenance Area 18" slab

6.2.7.1 Floor Beams CL A-B-1-2, A-B-3-4, C-D-1-2, C-D-3-6

See Attachment A, Page A-3. In these areas, girders divide the distance between the walls in half. Therefore, the length of the beams is:

$L_{fA} := \frac{49ft}{2}$ $L_{fA} = 24.5\ ft$

$W_{fA} := \frac{50ft}{8}$ $W_{fA} = 6.25\ ft$ Beam spacing (max.)

$R_{fA} := \frac{W_{fA} \cdot S_{fl} \cdot L_{fA}}{2}$ $R_{fA} = 91.7\,kip$

$M_{fA} := \frac{W_{fA} \cdot S_{fl} \cdot L_{fA}^2}{8}$ $M_{fA} = 561\,kip \cdot ft$

$S_{reqfA} := \frac{M_{fA}}{F_{bb}}$ $S_{reqfA} = 182\,in^3$

$S_{xW24x84} := 196in^3$

Demand Capacity Ratio:

$DC_{W24x84} := \frac{M_{fA}}{F_{be} \cdot S_{xW24x84}}$ $DC_{W24x84} = 0.65$

Shear:

$d_{W24x84} := 24.1in$ $t_{wW24x84} := 0.470in$

$f_{vW24x84} := \frac{R_{fA}}{d_{W24x84} \cdot t_{wW24x84}}$ $f_{vW24x84} = 8.1\,ksi$ is < $F_{vb} = 19.6\,ksi$

OK - Use W24x84

6.2.7.2 Floor Beams CL A-B-2-3, C-D-2-3

Columns cannot be accommodated in these areas. The beams are designed to span the entire 49 ft. distance between walls.

$L_{fB} := 49ft$

$W_{fB} := \dfrac{50ft}{8}$ $\qquad W_{fB} = 6.25\ ft$ $\qquad$ Beam spacing

$S_{fl} = 1197\,psf$ $\qquad$ Extreme floor load

$R_{fB} := \dfrac{W_{fB} \cdot S_{fl} \cdot L_{fB}}{2}$ $\qquad R_{fB} = 183\,kip$

$M_{fB} := \dfrac{W_{fB} \cdot S_{fl} \cdot L_{fB}^{\,2}}{8}$ $\qquad M_{fB} = 2246\,kip \cdot ft$

$S_{reqfB} := \dfrac{M_{fB}}{F_{bb}}$ $\qquad S_{reqfB} = 729\,in^3$

$S_{xW36x232} := 809in^3$

Demand Capacity Ratio:

$DC_{W36x232} := \dfrac{M_{fB}}{F_{be} \cdot S_{xW36x232}}$ $\qquad DC_{W36x232} = 0.63$

Shear:

$d_{W36x232} = 37.1\,in$ $\qquad t_{wW36x232} = 0.870\,in$

$f_{vW36x232} := \dfrac{R_{fB}}{d_{W36x232} \cdot t_{wW36x232}}$ $\qquad f_{vW36x232} = 5.68\,ksi$ $\qquad$ is < $\quad F_{vb} = 19.6\,ksi$

OK - Use W36x232

6.2.7.3 Floor Girders CL A-B-1-2, C-D-1-2

Both rooms 50 ft. in length, divided in half by a column.

Seven lines of beams are spaced equally across the room. Three beams connect to each side of each girder, and 1 beam connects directly into each side of the column. The free body diagram is the same as for the roof directly above (Sect. 6.2.6.5).

$$L_{gA} := 25ft$$

$$s_{pA} := \frac{50ft}{8} \qquad s_{pA} = 6.25\ ft$$

The end reaction of each beam, as calculated in Sect. 6.2.7.1:

$$R_{fA} = 91.7\,kip$$

$$R_{fgA} := \frac{2 \cdot R_{fA} \cdot 3}{2} \qquad R_{fgA} = 275\,kip$$

$$M_{fgA} := R_{fgA} \cdot 2 \cdot s_{pA} - 2 \cdot R_{fA} \cdot s_{pA} \qquad M_{fgA} = 2292\,kip \cdot ft$$

$$S_{fgAreq} := \frac{M_{fgA}}{F_{bb}} \qquad S_{fgAreq} = 744\,in^3$$

$$S_{xW36x232} := 809in^3$$

Demand Capacity Ratio:

$$DC_{W36x232} := \frac{M_{fgA}}{F_{be} \cdot S_{xW36x232}} \qquad DC_{W36x232} = 0.64$$

Shear:

$$d_{W36x232} := 37.1in \qquad t_{wW36x232} := 0.870in$$

$$f_{vW36x232} := \frac{R_{fgA}}{d_{W36x232} \cdot t_{wW36x232}} \qquad f_{vW36x232} = 8.5\,ksi \quad \text{is} < \quad F_{vb} = 19.6\,ksi$$

<u>OK - Use W36x232</u>

6.2.7.4 Floor Columns CL A-B-1-2, C-D-1-2

Both rooms have columns in the center. The column must support the load from the roof colum
as well as the floor load. Two girders and 2 beams frame into each column.

The ceiling height below is 37.75 ft. from ground floor to bottom of 2nd floor deck. Since the beams and girders provide support to the column in both planes, the effective length of the columns is reduced by 1.75 ft., for a net effective length of 36.0 ft.

$K = 1.0$ $\qquad$ $L_{colAA} := 36.0ft$

$P_{colA} = 689\,kip$ $\qquad$ Column load from roof , Sect. 6.2.6.6

$R_{fgA} = 275\,kip$ $\qquad$ Floor girder reaction, Sect. 6.2.7.3

$R_{fA} = 91.7\,kip$ $\qquad$ Floor beam reaction, Sect. 6.2.7.1

$P_{colAA} := P_{colA} + 2 \cdot R_{fgA} + 2 \cdot R_{fA}$ $\qquad$ $P_{colAA} = 1423\,kip$

Assume a weak axis radius of gyration:

$r_{yass} := 4.0in$ $\qquad$ $\dfrac{K \cdot L_{colAA}}{r_{yass}} = 108$

$F_{aass} := 12.80ksi$ $\qquad$ Table Q3-50 Ref. 2.2.9

$A_{sreq} := \dfrac{P_{colAA}}{0.70 \cdot 1.5 F_{aass}}$ $\qquad$ $A_{sreq} = 106\,in^2$

$A_{sW14x370} := 109in^2$ $\qquad$ $r_{yW14x370} := 4.27in$

$\dfrac{K \cdot L_{colAA}}{r_{yW14x370}} = 101$

$F_{aeW14x370} := 1.5 \cdot 14.47ksi$ $\qquad$ Table Q3-50 Ref. 2.2.9

$F_{aeW14x370} = 21.7\,ksi$

$f_{aeW14x370} := \dfrac{P_{colAA}}{A_{sW14x370}}$ $\qquad$ $f_{aeW14x370} = 13.1\,ksi$

Demand Capacity Ratio

$DC_{W14x370} := \dfrac{f_{aeW14x370}}{F_{aeW14x370}}$ $\qquad$ $DC_{W14x370} = 0.60$

OK - Use W14x370

6.2.7.5 Floor Girders CL A-B-3-4, C-D-3-4

Both of the rooms are 36 ft. wide. Five floor beams on each side of the girder are equally spaced. There is not an intermediate column.

$L_{gB} := 36ft$

$s_{pB} := \frac{L_{gB}}{6}$ $\qquad s_{pB} = 6.0\ ft$

The end reaction of each beam, as calculated in Sect. 6.2.7.1:

$R_{fA} = 91.7\,kip$

Ten beams frame into each girder, 5 per side. The free body diagram is the same as for the roof areas directly above, Sect. 6.2.6.7. Since the girder is symmetrically loaded, the end reactions are:

$R_{fgB} := \frac{10 \cdot R_{fA}}{2}$ $\qquad R_{fgB} = 458\,kip$ $\qquad$ Girder reaction

The maximum moment occurs at the center beam connection.

$M_{fgB} := R_{fgB} \cdot 3s_{pB} - 2 \cdot R_{fA} \cdot \left(1s_{pB} + 2 \cdot s_{pB}\right)$ $\qquad M_{fgB} = 4950\,kip \cdot ft$

$S_{fgBreq} := \frac{M_{fgB}}{F_{bb}}$ $\qquad S_{fgBreq} = 1607\,in^3$

$S_{xW36x441} := 1650in^3$

Demand Capacity Ratio:

$DC_{W36x441} := \frac{M_{fgB}}{F_{be} \cdot S_{xW36x441}}$ $\qquad DC_{W36x441} = 0.68$

Shear:

$d_{W36x441} := 38.9in$ $\qquad t_{wW36x441} := 1.36in$

$f_{vW36x441} := \frac{R_{fgB}}{d_{W36x441} \cdot t_{wW36x441}}$ $\qquad f_{vW36x441} = 8.7\,ksi$ is < $F_{vb} = 19.6\,ksi$

<u>OK - Use W36x441</u>

6.2.7.6 Floor Girder CL C-D-4-6

This room is 60 ft. in length, divided in half by a column. See Attachment A.

One floor beam connects directly to the column from each side. Four floor beams, equally spaced, frame into each side of each girder. There is a total of 18 floor beams in this room.

$L_{gD} := 30ft$

$s_{pD} := \frac{L_{gD}}{5}$ $\qquad$ $s_{pD} = 6.00\ ft$

The end reaction of each beam, as calculated in Sect. 6.2.7.1:

$R_{fA} = 91.7\,kip$

Eight beams frame into each girder, 4 per side. The loading is symmetrical. The free body diagram is the same as for the roof area above, Sect. 6.2.6.8. The girder reactions are:

$R_{fgDD} := \frac{8 \cdot R_{fA}}{2}$ $\qquad$ $R_{fgDD} = 367\,kip$

The maximum moment at the center of the girder is:

$M_{fgDD} := R_{fgDD} \cdot 2s_{pD} - 2 \cdot R_{fA} \cdot s_{pD}$ $M_{fgDD} = 3300\,kip \cdot ft$

$S_{fgDDreq} := \frac{M_{fgDD}}{F_{bb}}$ $\qquad$ $S_{fgDDreq} = 1071\,in^3$

$S_{xW36x302} := 1130in^3$

Demand Capacity Ratio:

$DC_{W36x302} := \frac{M_{fgDD}}{F_{be} \cdot S_{xW36x302}}$ $\qquad$ $DC_{W36x302} = 0.66$

Shear:

$d_{W36x302} := 37.3in$ $\qquad$ $t_{wW36x302} := 0.945in$

$f_{vW36x302} := \frac{R_{fgDD}}{d_{W36x302} \cdot t_{wW36x302}}$ $\qquad$ $f_{vW36x302} = 10.4\,ksi$ is < $\quad F_{vb} = 19.6\,ksi$

OK - Use W36x302

6.2.7.7 Floor Column CL C-D-4-6

The room has a column in the center. The ceiling height is 37.75 ft from floor to bottom of floor deck. Since the roof girders and beams connect to the column in both planes, the column unbraced length is reduced by 1.75 ft. Thus, the effective length is 36.0 ft.

$K = 1.0$ $\quad$ $L_{colDD} := 36.0ft$

$P_{colD} = 862\,kip$ $\quad$ Load from roof column above, Sect. 6.2.6.9

$R_{fgDD} = 367\,kip$ $\quad$ Girder reaction, Sect. 6.2.7.6

$R_{fA} = 91.7\,kip$ $\quad$ Floor beam reaction, Sect. 6.2.7.1

$P_{colDD} := P_{colD} + 2\cdot R_{fgDD} + 2\cdot R_{fA}$ $\quad$ $P_{colDD} = 1778\,kip$

Assume a weak axis radius of gyration:

$r_{yass} := 4.00in$

$$\frac{K\cdot L_{colDD}}{r_{yass}} = 108$$

$F_{aass} := 12.80ksi$

$$A_{sreq} := \frac{P_{colDD}}{0.70\cdot 1.5F_{aass}} \qquad A_{sreq} = 132\,in^2$$

$A_{sW14x426} := 125in^2$ $\quad$ $r_{yW14x426} := 4.34in$

$$\frac{K\cdot L_{colDD}}{r_{yW14x426}} = 100$$

$F_{aeW14x426} := 1.5\cdot 14.71ksi$ $\quad$ $F_{aeW14x426} = 22.1\,ksi$

$$f_{aeW14x426} := \frac{P_{colDD}}{A_{sW14x426}} \qquad f_{aeW14x426} = 14.2\,ksi$$

Demand Capacity Ratio

$$DC_{W14x426} := \frac{f_{aeW14x426}}{F_{aeW14x426}} \qquad DC_{W14x426} = 0.64$$

<u>OK - Use W14x426</u>

6.2.7.8 Floor Beams CL B-C-1-2

In this area the slab at El. 40 ft. is 18 in. thick. Per Sect. 6.1, the steel deck can span 6.68 ft. for an 18 in. slab. Assuming a 6 in. flange of the support beams, the maximum beam spacing is 7.2 ft. No columns are permitted below the floor because an overhead crane operates in the area.

The room is 100 ft. long by 50 ft. wide. Fourteen beams spaced at 6.67 ft. will span the room, as shown on the layout sketch.

$L_{fB} := 50.0ft$

$W_{fB} := \frac{100ft}{15}$ $\quad W_{fB} = 6.67\ ft$ $\quad$ Beam spacing

$S_{18fl} = 1023\,psf$ $\quad$ Extreme floor uniform load Sect. 6.2.4

$R_{fB} := \frac{W_{fB} \cdot S_{18fl} \cdot L_{fB}}{2}$ $\quad R_{fB} = 171\,kip$

$M_{fB} := \frac{W_{fB} \cdot S_{18fl} \cdot L_{fB}^2}{8}$ $\quad M_{fB} = 2132\,kip \cdot ft$

$S_{reqfB} := \frac{M_{fB}}{F_{bb}}$ $\quad S_{reqfB} = 692\,in^3$

$S_{xW36x210} := 719in^3$

Demand Capacity Ratio:

$DC_{W36x210} := \frac{M_{fB}}{F_{be} \cdot S_{xW36x210}}$ $\quad DC_{W36x210} = 0.67$

Shear:

$d_{W36x210} = 36.7\,in$ $\quad t_{wW36x210} = 0.830\,in$

$f_{vW36x210} := \frac{R_{fB}}{d_{W36x210} \cdot t_{wW36x210}}$ $\quad f_{vW36x210} = 5.6\,ksi$ $\quad$ is < $\quad F_{vb} = 19.6\,ksi$

OK - Use W36x210

6.2.7.9 Mezzanine Floor (El. 20 ft) Beams CL B-C-1-2
(See Page A-4 for floor framing plan)

The 24" mezzanine floor is supported by a combination of bearing walls and beams. As shown of the attached layout sketches, the beams' spans range from 8 ft. to 24 ft. The maximum span for floor deck is 6 ft. in this area.

$S_{fl} = 1197\,\text{psf}$ Extreme Load Combination for 2' slab (Sect. 6.2.4)

<u>Long span beams:</u>

$L_{fF} := 24.0\text{ft}$ $W_{fF} := 6.0\text{ft}$

$$R_{fF} := \frac{W_{fF} \cdot S_{fl} \cdot L_{fF}}{2} \qquad R_{fF} = 86.2\,\text{kip}$$

$$M_{fF} := \frac{W_{fF} \cdot S_{fl} \cdot L_{fF}^{\;2}}{8} \qquad M_{fF} = 517\,\text{kip} \cdot \text{ft}$$

$$S_{reqfF} := \frac{M_{fF}}{F_{bb}} \qquad S_{reqfF} = 168\,\text{in}^3 \qquad S_{xW24x76} := 176\text{in}^3$$

Demand Capacity Ratio:

$$DC_{W24x76} := \frac{M_{fF}}{F_{be} \cdot S_{xW24x76}} \qquad DC_{W24x76} = 0.67$$

Shear:

$d_{W24x76} = 23.9\,\text{in}$ $t_{wW24x76} = 0.440\,\text{in}$

$$f_{vW24x76} := \frac{R_{fF}}{d_{W24x76} \cdot t_{wW24x76}} \qquad f_{vW24x76} = 8.20\,\text{ksi} \text{ is} < \quad F_{vb} = 19.6\,\text{ksi}$$

<u>OK - Use W24x76 for Long Beams</u>

Short span beams:

$L_{fG} := 8.0ft$ $W_{fG} := 6.0ft$

$R_{fG} := \frac{W_{fG} \cdot S_{fl} \cdot L_{fG}}{2}$ $R_{fG} = 28.7 kip$

$M_{fG} := \frac{W_{fG} \cdot S_{fl} \cdot L_{fG}^2}{8}$ $M_{fG} = 57 kip \cdot ft$

$S_{reqfG} := \frac{M_{fG}}{F_{bb}}$ $S_{reqfG} = 18.7 in^3$ $S_{xW10x19} := 18.8in^3$

Demand Capacity Ratio:

$DC_{W10x19} := \frac{M_{fG}}{F_{be} \cdot S_{xW10x19}}$ $DC_{W10x19} = 0.69$

Shear:

$d_{W10x19} := 10.2in$ $t_{wW10x19} := 0.250in$

$f_{vW10x19} := \frac{R_{fG}}{d_{W10x19} \cdot t_{wW10x19}}$ $f_{vW10x19} = 11.3 ksi$ is < $F_{vb} = 19.6 ksi$

OK - Use W10x19 for Short Beams

Short Girders: There are several short girders that span 10.5 ft. on the east side of the room. The highest load case is the short girder that supports end reactions of a 20.25 ft. beam and a 21.5 ft. beam (accounting for wall thickness). Assume that the load is applied at the middle of the girder (worst case).

$P_{fH} := S_{fl} \cdot \frac{10.5 \cdot ft}{2} \cdot \left(\frac{20.25 \cdot ft + 21.50 \cdot ft}{2}\right)$ Beam reaction loads applied at center

$P_{fH} = 131 kip$

$M_{fH} := P_{fH} \cdot \frac{10.5ft}{4}$ $M_{fH} = 344 kip \cdot ft$

$$S_{reqH} := \frac{M_{fH}}{F_{bb}} \qquad S_{reqH} = 112\,in^3$$

Use W24x76 with S_x= 176 in^3 to facilitate connections and to match other steel

$$S_{xW24x76} := 176in^3$$

Demand Capacity Ratio:

$$DC_{W24x76} := \frac{M_{fH}}{F_{be} \cdot S_{xW24x76}} \qquad DC_{W24x76} = 0.44$$

Shear:

$$f_{vW24x76} := \frac{P_{fH}}{2 \cdot d_{W24x76} \cdot t_{wW24x76}} \qquad f_{vW24x76} = 6.24\,ksi \quad \text{is <} \quad F_{vb} = 19.6\,ksi$$

Use W24x76 for Short Girders

Opening Framing: There is a 4 ft. by 20 ft. opening in the slab as shown in Attachment A. The east and west header beams support the load from the north header. The reactions for the north header are each 1/4 of the 20'-0" by 12'-6" supported floor area.

$$R_{NH} := S_{fl} \cdot \frac{20ft \cdot 12.5ft}{4} \qquad R_{NH} = 74.8\,kip$$

The reactions are applied at 6'-0" from the ends of the east and west header beams. The length of the east and west headers is 18'-6". The reaction at the south end of the east and west headers is:

$$L_{EWH} := 18.5ft$$

$$R_{EWH} := R_{NH} \cdot \frac{L_{EWH} - 6.0ft}{L_{EWH}} \qquad R_{EWH} = 50.6\,kip$$

Maximum moment is at the connection of the north header:

$$M_{EWH} := R_{EWH} \cdot 6.0ft \qquad M_{EWH} = 303\,kip \cdot ft$$

Add the moment from the slab supported by the header from the side opposite the opening, i.e., a 3' strip of the slab. This is 1/2 of M_fF calculated above for the 24' span beams. This is conservative since the maximum moments do not actually occur at the same point on the beam.

$M_{fF} = 517\,kip\cdot ft$ Sect. 6.2.7.9

$$S_{reqEWH} := \frac{M_{EWH} + \frac{M_{fF}}{2}}{F_{bb}} \qquad S_{reqEWH} = 182\,in^3$$

$$S_{xW24x84} := 196in^3$$

Demand Capacity Ratio:

$$DC_{W24x84} := \frac{M_{EWH} + \frac{M_{fF}}{2}}{F_{be}\cdot S_{xW24x84}} \qquad DC_{W24x84} = 0.65$$

Shear:

$d_{W24x84} = 24.1\,in$ $t_{wW24x84} = 0.470\,in$

$$f_{vW24x84} := \frac{\frac{R_{fF}}{2} + R_{EWH}}{d_{W24x84}\cdot t_{wW24x84}} \qquad f_{vW24x84} = 8.3\,ksi \quad \text{is <} \quad F_{vb} = 19.6\,ksi$$

OK - Use W24x84 for Header Beams

6.2.8 DEFLECTIONS

The *Project Design Criteria Document* (Sect. 4.2.11.4.8 Ref. 2.2.1) states that deflections in structural steel members shall be in accordance with ANSI/AISC N690 Section Q1.13 and Commentary CQ1.13 (Ref. 2.2.9). However, N690 gives only guidelines for deflection limits:

1. The depth of fully stressed beams and girders in floors should not be less than $F_y/800$ times the span.
2. The depth of fully stressed roof purlins should not be less than $F_y/1000$ times the span.
3. For human comfort, the depth of steel beams supporting large open floor areas should not be less than 1/20 of the span.

$$F_y := 50 \qquad \frac{F_y}{800} = 0.0625 \qquad \frac{F_y}{1000} = 0.0500$$

The roof and floor beams maximum span for each beam depth, and ANSI/AISC N690 deflection criteria (Ref. 2.2.9 CQ1.13) are presented in the following table. The deflection criteria is for fully stressed members. Only during the extreme load cases might these members be fully stressed. Thus, the deflections during normal service are well below the guidelines of ANSI/AISC N690.

Beam / Girder Type	Span (ft)	Fy/800 x Span (in)	Span/20 (in)	Fy/1000 x Span (in)	Beam/ Girder Depth	
Roof	16.4	n/a	n/a	9.8	12.3	OK
Roof	24.5	n/a	n/a	14.7	23.9	OK
Roof	50.0	n/a	n/a	30.0	36.7	OK
Floor	24.5	18.4	14.7	n/a	24.1	OK
Floor	49.0	36.8	29.4	n/a	36.7	0.2%<

The W36 beams with 49 ft. span have approximately 0.2% less depth than recommended by the $F_y/800$ guideline. However, the guideline is for "fully stressed" members. During normal service loading, these members are not near "fully stressed." Additionally, live loads are only a small part of the normal load combinations. Deflection limits are generally based on live loads, as dead loads can be offset by cambering the beams if necessary.

7. RESULTS AND CONCLUSIONS

7.1 RESULTS

Results of this calculation are the layout and sizing of structural steel framing members, shown in Attachments A and B. This forms the basis for the preliminary structural steel framing drawings for the WHF, and the input to detailed design and the Tier 2 structural model.

The steel framing and truss member sections and DC Ratios are tabulated below. The DC Ratio limit is 0.70 for steel framing and truss members (Assumption 3.2.6).

Steel Framing Sections and DC Ratios							
Slab Elevation	Area	Beams		Girders		Columns	
		Section	D:C Ratio	Section	D:C Ratio	Section	D:C Ratio
100	A-B-4-7	W36x232	0.68				
80	A-B-1-2	W24x76	0.68	W36x210	0.68	W14x176	0.68
80	A-B-2-3	W36x210	0.67				
80	A-B-3-4	W24x76	0.68	W36x441	0.64		
80	B-C-1-7	W12x72	0.58	Trusses			
80	C-D-1-2	W24x76	0.68	W36x210	0.68	W14x176	0.68
80	C-D-2-3	W36x210	0.67				
80	C-D-3-4	W24x76	0.68	W36x441	0.64		
80	C-D-4-6	W24x76	0.68	W36x282	0.67	W14x233	0.62
80	C-D-6-7	W36x210	0.67				
40	A-B-1-2	W24x84	0.65	W36x232	0.64	W14x370	0.60
40	A-B-2-3	W36x232	0.63				
40	A-B-3-4	W24x84	0.65	W36x441	0.68		
40	B-C-1-2	W36x210	0.67				
40	C-D-1-2	W24x84	0.65	W36x232	0.64	W14x370	0.60
40	C-D-2-3	W36x232	0.63				
40	C-D-3-4	W24x84	0.65	W36x441	0.68		
40	C-D-4-6	W24x84	0.65	W36x302	0.66	W14x426	0.64
20	B-C-1-2	W24x76	0.67				
20	B-C-1-2	W10x19	0.69	W24x76	0.44		

Truss Member Sections and Demand: Capacity Ratios								
Panel	Diagonals		OutsideVerticals		Bottom Chord		Top Chord	
	Section	D:C	Section	D:C	Section	D:C	Section	D:C
1	2-WT6x68	0.68	W12x120	0.63	W14x370	n/a	W14x398	0.18
2	2-WT6x68	0.58	W12x120	0.63	W14x370	0.18	W14x398	0.33
3	2-WT6x68	0.48	W12x120	0.54	W14x370	0.33	W14x398	0.45
4	2-WT6x68	0.39	W12x120	0.45	W14x370	0.45	W14x398	0.55
5	2-WT6x68	0.29	W12x120	0.36	W14x370	0.55	W14x398	0.62
6	2-WT6x68	0.19	W12x120	0.27	W14x370	0.63	W14x398	0.67
7	2-WT6x68	0.10	W12x120	0.18	W14x370	0.68	W14x398	0.70
8	2-WT6x68	n/a	W12x120	0.09	W14x370	0.70	W14x398	0.70

7.2 CONCLUSIONS

The structural steel framing design, as shown in Attachment A, is adequate for the load combinations given in the Project Design Criteria, Ref. 2.2.1, and governing codes and standards.

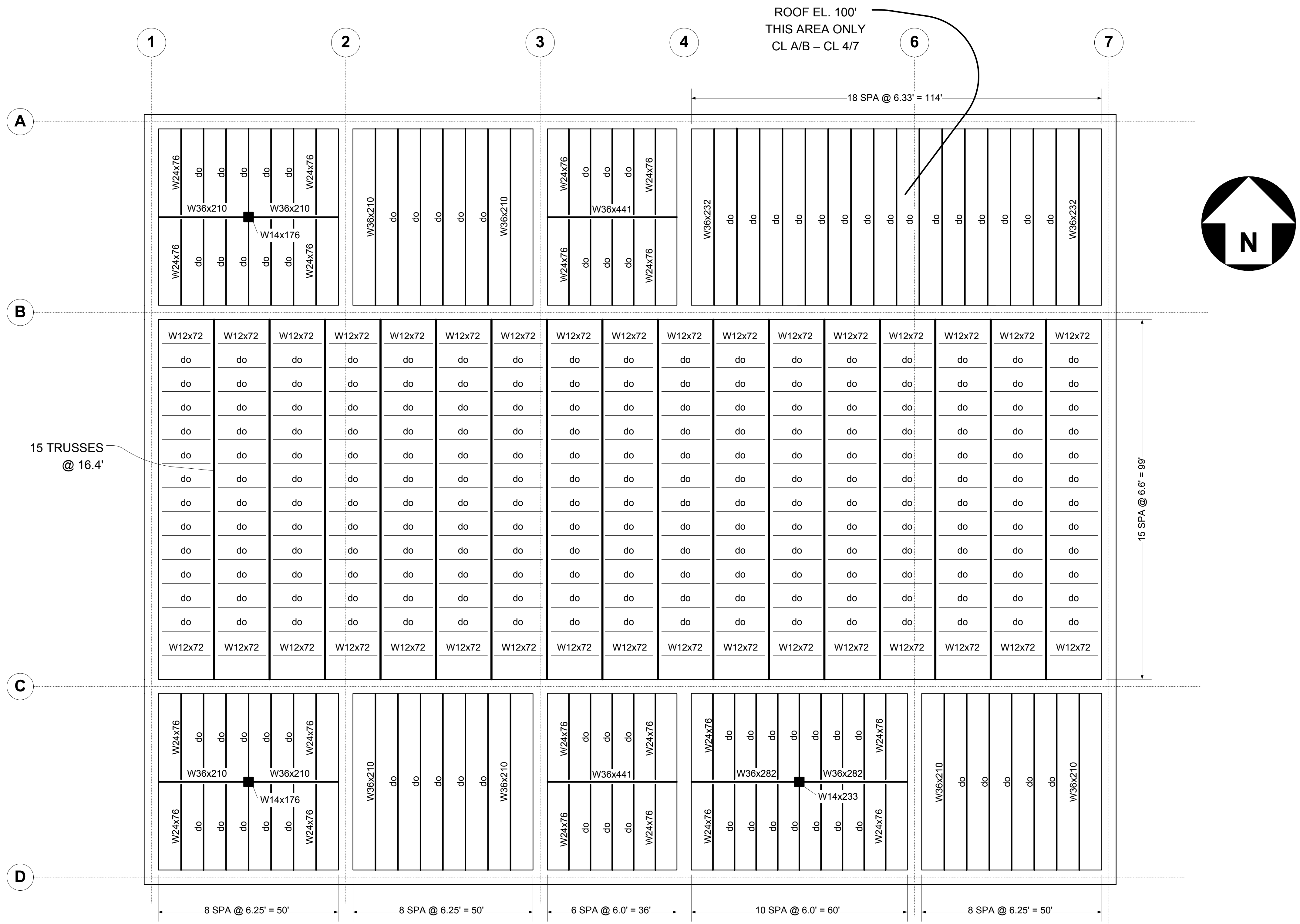
The outputs of this calculation are reasonable compared to the inputs.

The results are suitable for the intended use.

ATTACHMENT A

WET HANDLING FACILITY

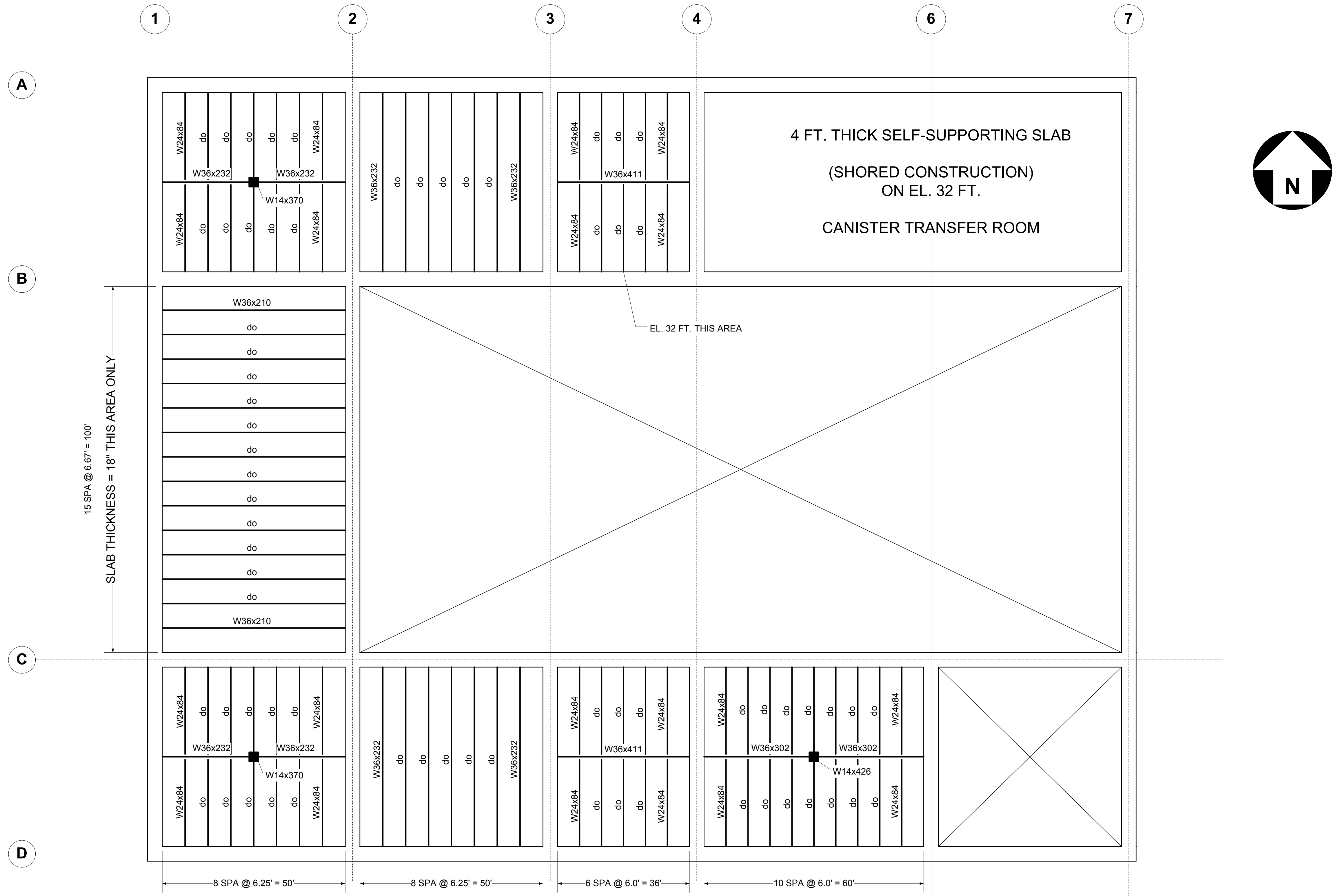
STEEL FRAMING PLANS



ROOF FRAMING PLAN
EL. 80' UNLESS NOTED OTHERWISE

WET HANDLING FACILITY

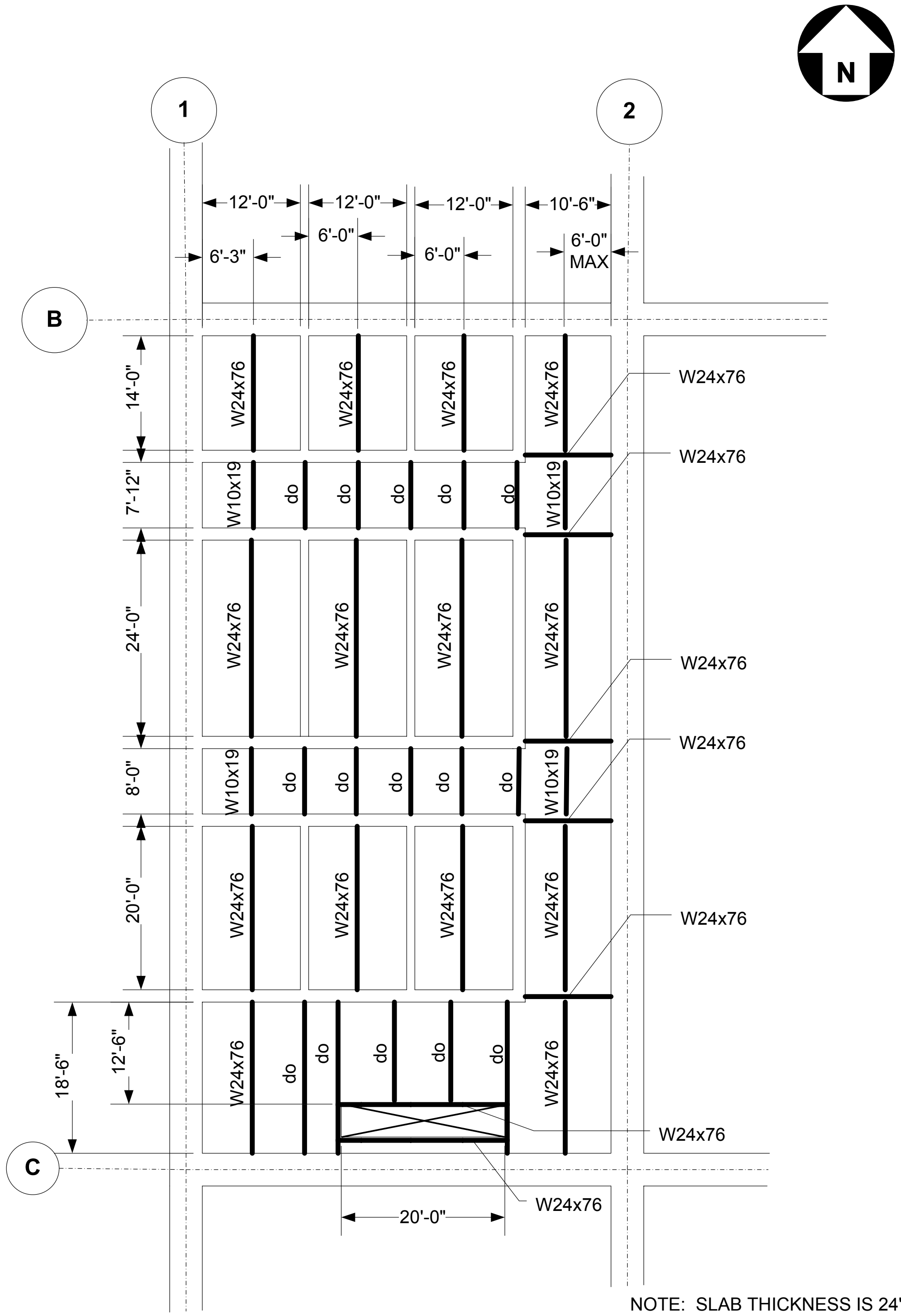
NOTE: ALL ROOF SLABS ARE 24" THICK



EL. 40 FT. (UNO) FLOOR FRAMING PLAN

WET HANDLING FACILITY

NOTE: ALL SLABS ARE 24" THICK UNLESS NOTED OTHERWISE

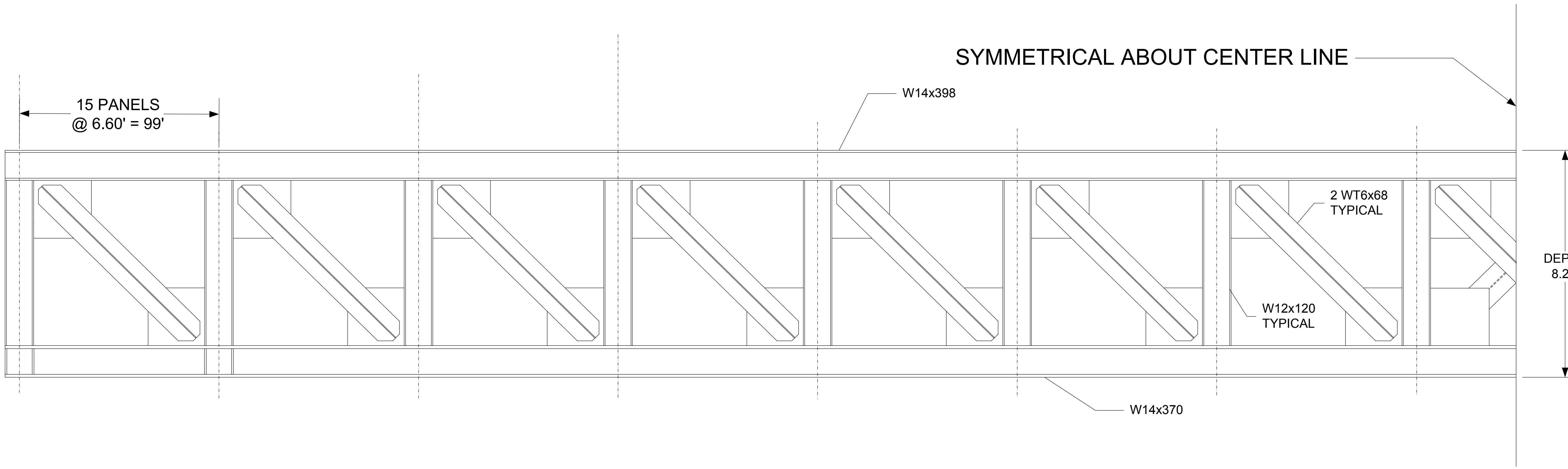


EL. 20 FT. MEZZANINE FRAMING PLAN
WET HANDLING FACILITY

ATTACHMENT B

WET HANDLING FACILITY

POOL ROOM ROOF TRUSS

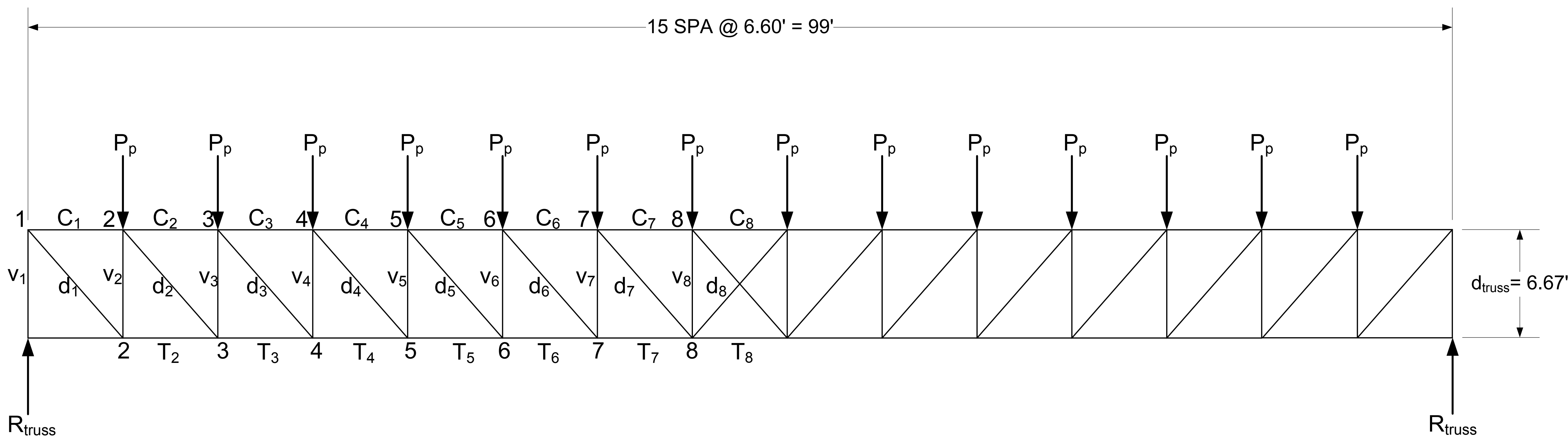


WET HANDLING FACILITY – POOL ROOM ROOF TRUSS

ATTACHMENT C

WHF POOL ROOM TRUSS

FREE BODY DIAGRAM



WHF POOL ROOM TRUSS FREE BODY DIAGRAM

BSC

Calculation/Analysis Change Notice

Complete only applicable items.

1. QA: QA
2. Page 1 of 1

3. Document Identifier:	4. Rev.:	5. CACN:
050-SSC-WH00-00100-000	00A	001

6. Title:
Wet Handling Facility Structural Steel Framing

7. Reason for Change:
To correct a typographical error. A W36X411 girder section is incorrectly shown on the sketch of Attachment A, Page A-3. The correct section is W36X441. The W36X441 section is correctly shown in the calculation Section 6.2.7.5 and in the summary table in Section 7.1. However, the "441" was incorrectly transcribed as "411" onto the sketch. A W36X411 section does not exist.

Revising this calculation with the correct girder size addresses the inconsistencies identified in CR 10950.

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:

See Attachment A, Page A-3.

Change "W36X411" to "W36X441" in 2 places.

11. REVIEWS AND APPROVAL

Printed Name	Signature	Date
11a. Originator: John Rast	[signature]	7/30/2007
11b. Checker: Ravinder Sanan	RK Sanan	7/30/07
11c. EGS: Michael Ruben/ Thomas Frankert	7/30/07 Michael S Ruben / T Frankert	7/30/07
11d. DEM: Raj Rajagopal	R Rajagopal	7/30/07
11e. Design Authority: Barbara Rusinko	B Rusinko	7/31/07