

BSC

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ATTACHMENT A: Structural Support Steel Layout	5

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DISCLAIMER

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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 Receipt Facility (RF). The design of the steel framing system includes steel roof decking and structural steel beams.

2. REFERENCES

2.1 PROCEDURES/DIRECTIVES

- 2.1.1 EG-PRO-3DP-G04B-00037 Revision 8, ICN 0, *Calculations and Analyses*. Las Vegas, Nevada: Bechtel SAIC Company. ACC: [ENG.20070420.0002](#)
- 2.1.2 IT-PRO-011, Rev. 004, ICN 0. *Software Management*, Las Vegas, Nevada: Bechtel SAIC Company. ACC: DOC.20070319.0016.
- 2.1.3 Not used
- 2.1.4 ORD (Office of Repository Development) 2006. 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 BSC (Bechtel SAIC Company) 2006. *Seismic Analysis and Design Approach Document*. 000-30R-MGR0-02000-000-000. Las Vegas, Nevada: Bechtel SAIC Company. ACC: ENG.20061214.0008
- 2.2.4 BSC (Bechtel SAIC Company) 2007 *Receipt Facility Preliminary Layout Ground Floor Plan*. 200-P0K-RF00-10101-000-00A. Las Vegas, Nevada: Bechtel SAIC Company. ACC: [ENG.20070222.0002](#)
- 2.2.5 BSC (Bechtel SAIC Company) 2007. *Receipt Facility Preliminary Layout Second Floor Plan*. 200-P0K-RF00-10102-000-00A Las Vegas, Nevada: Bechtel SAIC Company. ACC: [ENG.20070222.0003](#)
- 2.2.6 BSC (Bechtel SAIC Company) 2007. *Receipt Facility Preliminary Layout Third Floor Plan*. 200-P0K-RF00-10103-000-00A Las Vegas, Nevada: Bechtel SAIC Company. ACC: [ENG.20070222.0004](#)

- 2.2.7 BSC (Bechtel SAIC Company) 2007. *Receipt Facility Preliminary Layout Section A* 200-P0K-RF00-10104-000-00A Las Vegas, Nevada: Bechtel SAIC Company. ACC: [ENG.20070222.0005](#)
- 2.2.8 BSC (Bechtel SAIC Company) 2007. *Receipt Facility Preliminary Layout Section B* 200-P0K-RF00-10105-000-00A. Las Vegas, Nevada: Bechtel SAIC Company. ACC: [ENG.20070222.0006](#)
- 2.2.9 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]
- 2.2.10 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]
- 2.2.11 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. ISBN 1-56424-000-2
- 2.2.12 BSC (Bechtel SAIC Company) 2006. *Receipt Facility (RF) Mass Properties* 200-SYC-RF00-00100-000-00A. Las Vegas, Nevada: Bechtel SAIC Company. ACC: [ENG.20061206.0001](#)
- 2.2.13 BSC (Bechtel SAIC Company) 2006. *RF Seismic Analysis* 200-SYC-RF00-00400-000-00A. Las Vegas, Nevada: Bechtel SAIC Company. ACC: [ENG.20070307.0003](#)
- 2.2.14 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]
- 2.2.15 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](#).
- 2.2.16 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](#).
- 2.2.17 Not used
- 2.2.18 ICC (International Code Council) 2003. *International Building Code 2000, with Errata to the 2000 International Building Code*. Falls Church, Virginia: International Code Council. TIC: [251054](#); [257198](#) ISBN 1-892395-25-8

- 2.2.19 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](#). ISBN 0784404453
- 2.2.20 AISC (American Institute of Steel Construction) 2005. *Steel Construction Manual*. 13th Edition. First Printing Chicago, Illinois: American Institute of Steel Construction. TIC: [258106](#). ISBN 1-56424-055-X
- 2.2.21 BSC (Bechtel SAIC Company) 2007. *Receipt Facility CTM Maintenance Crane Mechanical Equipment Envelope* 200-MJ0-HTC0-00101-000-00A. Las Vegas, Nevada: Bechtel SAIC Company. ACC: [ENG.20070228.0007](#)
- 2.2.22 BSC (Bechtel SAIC Company) 2007. *Receipt Facility Cask Handling Crane Mechanical Equipment Envelope* 200-MJ0-HM00-00101-000-00A. Las Vegas, Nevada: Bechtel SAIC Company. ACC: [ENG.20070228.0005](#)
- 2.2.23 BSC (Bechtel SAIC Company) 2007. *Receipt Facility Lid Bolting Room Crane Mechanical Equipment Envelope* 200-MJ0-HMC0-00101-000-00A. Las Vegas, Nevada: Bechtel SAIC Company. ACC: [ENG.20070228.0006](#)

2.3 DESIGN CONSTRAINTS

NONE

2.4 DESIGN OUTPUTS

Results of this calculation will be used as input to the structural steel drawings for the Receipt Facility (RF).

3. ASSUMPTIONS

3.1 ASSUMPTIONS REQUIRING VERIFICATION

- 3.1.1 The amplified slab acceleration for out-of-plane seismic loads is assumed as 2.0 times the slab acceleration obtained from the RF seismic analysis (Ref. 2.2.13).

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 seismic soil-structure interaction analysis was performed on the Canister Handling Facility (CHF) (Ref. 2.2.15) which developed in-structure response spectra at hard points on the walls. Using the 7% damped vertical response spectra given in Figure F-3 of Ref. 2.2.15, a ratio between the wall ZPA (Zero Period Acceleration) 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. 2.2.16). 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 in. floor with columns spaced at approx. 20 ft on centers the fundamental mode is approximately 25Hz 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:

$$response = (.5 * mass * ZPA) + (.5 * Ratio * mass * ZPA)$$

Where: Ratio = Acceleration @ 25 Hz / ZPA

Using the Response Ratio versus frequency plot described above the ratio for 25Hz was found to be 2.3. Using this value in the response equation above results in:

$$response = .5 * mass * ZPA + .5 * mass * 2.3 * ZPA$$

$$response = 1.65 * mass * ZPA$$

Where ZPA is the acceleration obtained from the RF seismic analysis (Ref. 2.2.13) at the floor level under consideration.

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

Where used: Sections 6.2, 6.5, 6.6.

- 3.1.2 The length of the bearing end of the beam used in the web crippling check is less than distance $d/2$ of the member.

Rationale– At this preliminary stage of the design process where the connection design has not yet been performed assuming a bearing width equal to or less than $\frac{1}{2}$ the depth of the member is appropriate as the actual bearing width will be approximate to this distance.

Where used: Sections 6.5 and 6.6.

3.2 ASSUMPTIONS NOT REQUIRING VERIFICATION

- 3.2.1 Steel beams, are designed as Type 2 construction per section Q1.2 (Ref. 2.2.9 & 2.2.10) 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. 2.2.11).

Where used: Sections 6.5 and 6.6.

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

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

Where used: Sections 6.5 and 6.6.

- 3.2.3 Decking will provide full lateral support to top flanges of beams during construction.

Rationale- The steel decking is connected to the steel beams with spot welds that are placed on the order of 12 to 18 inches. Compared to the spans of the beams this is a small enough length of the upper or compression flange to consider the beams as continuously supported.

Where used: Sections 4.3, 6.5, and 6.6.

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). The RF has been classified as a structure that is Important to Safety, ITS, in section 6.1.2 of the *Basis of Design for the TAD Canister-Based Repository Design Concept* document (Ref. 2.2.2). Therefore the approved version will be designated as QA:QA.

4.2 USE OF SOFTWARE

MathCAD Version 13 was utilized to perform design calculations. MathCAD was operated on a PC system running the Windows 2003 operating system. MathCAD as used in this calculation is considered as level 2 software usages as defined in IT-PRO-0011 (Ref. 2.1.2). MathCAD Version 13 is listed on the current controlled Software Report as well as the *Repository Project Management Automation Plan* (Ref. 2.1.4).

All MathCAD input values and equations are stated in the calculation. Checking of the MathCAD and calculations was done by using visual inspection and hand calculations to confirm the accuracy of the results.

4.3 DESCRIPTION OF CALCULATION APPROACH

The Receipt Facility (RF) framing plans in Attachment A were developed from the general arrangement sketches (Ref. 2.2.4, 2.2.5, 2.2.6, 2.2.7, and 2.2.8). 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).
- Determine allowable spans for steel floor decking using unshored construction, using the section properties outlined in the United Steel Deck Catalog (Ref. 2.2.14). Note: The steel decking is relied on only to support construction loads. The decking is not considered in service and extreme load combinations for the RF. Therefore this source is suitable for use in this calculation.
- 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:
 - Decking will provide full lateral support to top flanges of beams during construction (Assumption 3.2.3). Concrete slabs provide lateral support to the top flanges of beams, 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.
- Structural steel will not be used for the 4'-0" slabs located between Gridline 3 to 6, from E to C. 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. In addition there is insufficient head room below the 4'-0" slab to permit the use of supporting steel beams.
- Design structural steel beams, using Allowable Stress Design (ASD) provisions of ANSI N690 (Ref. 2.2.9 & 2.2.10). 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. Missile Impact due to Tornado loads was evaluated, and addressed in Section 6.7.
- Verify that deflections are acceptable (Ref. 2.2.9).
- The response of structural Steel roof framing to tornado missiles will be evaluated in a separate calculation.

- Demand/Capacity Ratio will be created in tabular format for every section of the calculation, and will be shown in Results, section 7.1.

5. LIST OF ATTACHMENTS

	Number of Pages
Attachment A. Structural Support Steel Layout.	5

6. BODY OF CALCULATION

6.1 STRUCTURAL PARAMETERS

The configuration of the Receipt Facility (RF), for the purpose of this analysis and design, is based on the Plant Design Sketches, Ref. 2.2.4 through 2.2.8. This building configuration forms the basis for layout of the structural steel framing.

The following loads are based on the Receipt Facility (RF) Mass Properties Calculations (Ref. 2.2.12)

6.2 ROOF LOADS

DL = 5000 lbs	Construction load, Ref. 2.2.1 Sect. 4.2.11.3.16 Does not Govern.
$DL_{eqroof} := 10\text{psf}$	Equipment dead load on roof
$DL_{framing} := 40\text{psf}$	Dead load of structural framing.
$DL_{roofing} := 55\text{psf}$	Dead load of roofing
$LL_{roof} := 40\text{psf}$	Roof live load Governs
Use 25% of LL for earthquake,	(Ref.2.2.3 Sec.10.3.1)
$LL_{const} := 50\text{psf}$	Construction live load for concrete placement (Ref. 2.2.1) (Sect.4.2.11.3.16)
Ash Load = $4/12(64) = 21\text{psf}$	(Ref. 2.2.1 Sect.4.2.11.3.4 & 6.1.11)

Snow Loads (S_n) Ref.2.2.1 sect. 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. 2.2.19)

$d_{sn} := 6\text{in}$	daily snowfall depth based on section 6.1.1 (Ref.2.2.1)
$\gamma_{sn} := 30\text{pcf}$	maximum snow density based on equation 7-4 (max density required for design), ASCE 7-98 (Ref. 2.2.19)
$P_g := d_{sn} \cdot \gamma_{sn}$ $P_g = 15.00\text{psf}$	Calculated ground snow load
$I := 1.2$	Highest value, Table 7-4, ASCE 7-98 (Ref. 2.2.19)

$P_f := I \cdot P_g$ $P_f = 18.00 \text{ psf}$ Flat roof snow load per section 7.3, ASCE 7-98
(Ref.2.2.19)

$S_n := P_f$ $S_n = 18.00 \text{ psf}$ Roof snow load

LLoad of 40psf will govern the load combinations for the Roof beams design.

Seismic:

Diaphragm Accelerations for DBGM-2 35' Upper Bound Alluvium, SRSS Combination

Diaphragm Level	East - West X - Acceleration		North - South Y - Acceleration		Vertical Z - Acceleration	
	ft /sec ²	g's	ft /sec ²	g's	ft /sec ²	g's
0' (Node 99)	16.4986	0.51	14.7483	0.46	19.0407	0.59
32' (Node 399)	22.9847	0.71	22.1327	0.69	21.7252	0.67
64' (Node 599)	29.7228	0.92	30.6657	0.95	23.1840	0.72
72' (Node 699)	31.2860	0.97	36.5482	1.14	23.1424	0.72
100' (Node 799)	46.5722	1.45	51.5880	1.60	25.5198	0.79

The above input is per Table 14, pg. 62, of Ref.2.2.13

Assumption: The amplified slab acceleration to be considered for out of plane seismic loads is **2.0** times the floor acceleration obtained from the **RF** seismic analysis per assumption 3.1.1.

6.3 MATERIAL PROPERTIES

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

$f'_c := 5000 \text{ psi}$ Concrete 28-day strength

$f_y := 60000 \text{ psi}$ Reinforcing steel yield strength, ASTM A706 Gr. 60

6.3.2 Structural Steel for ITS Structures Sect. 4.2.11.6.1 Table 4.2-1 Ref. 2.2.1

$F_{y50} := 50 \text{ ksi}$ W-Shape Yield Strength

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

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

$E := 29000\text{ksi}$ Steel Modulus of Elasticity

$w_c := 150\text{pcf}$ Unit weight of reinforced concrete

6.4 DETERMINE MAXIMUM SPANS FOR STEEL DECKING

Determine maximum allowable spans for 1'-6" slabs, using the data and methods (LRFD) outlined in United Steel Deck, Inc., Design Manual and Catalog of Products. Use 2 span minimum - **do not allow single-span deck installation.**

Use United Steel 3" LOK-FLOOR 16 gauge steel deck, 33ksi yield strength. The following properties are from the design manual: (pg.2 and 30, Ref.2.2.14)

$f_y := 33000\text{psi}$

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

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

$t := 0.0598\text{in}$ $A_s := 1.020\text{in}^2$ Thickness and cross-section area of decking.

$w_d := 3.5\text{psf}$ Unit weight of decking.

$R_b := 2540 \frac{\text{lbf}}{\text{ft}}$ Interior reaction allowable per foot. (Ref. 2.2.14 pg.40)

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

$\phi_b := 0.95$ Bending strength reduction factor.

During concrete placement, the worst case of a uniform live load or a linear live load at midspan is considered.

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

$P_c := 150\text{plf}$ Concentrated live load during concrete placement applied at midspan.(Ref.2.2.14 pg.17)

Calculate concrete weight for 18" thickness of slab above deck:

$w_{\text{conc}} := w_c \cdot \text{ft}$ Uniform weight of 1' thick concrete.

$$w_{c18} := w_{\text{conc}} \cdot \frac{18 + 1.5}{12} \quad w_{c18} = 244\text{psf}$$

Determine maximum span for negative moment, using a 2-span panel with both spans loaded, LFRD (See SDI Formulas for Construction Loads on pg. 17 of Ref.2.2.14)

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

$$M_{\text{maxneg}} := \phi_b \cdot f_y \cdot S_n \quad M_{\text{maxneg}} = 32761 \frac{\text{in} \cdot \text{lbf}}{\text{ft}}$$

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

Determine maximum span for positive moment, using a 2-span panel (**do not allow single span panels**), only 1 span loaded:

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

$$M_{\text{maxpos}} := \phi_b \cdot f_y \cdot S_p \quad M_{\text{maxpos}} = 32761 \frac{\text{in} \cdot \text{lbf}}{\text{ft}}$$

For 18" slab:

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

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

Check interior web crippling for 2-span panel, fully loaded, 5" bearing. ASD is used, with the 1/3 increase allowed for temporary loading per pg.18, Ref.2.2.14:

Reaction - interior

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

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

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

$$R_{\text{all}} = 3378.20 \frac{1}{\text{ft}} \text{ lbf}$$

$$L_{18\text{web}} := .80 \cdot \frac{R_{\text{all}}}{w_{c18} + LL_c + w_d}$$

$$L_{18\text{web}} = 9.09 \text{ ft}$$

Check web shear using double span:

$$V_w := 0.625 \cdot (1.6 \cdot w_{\text{conc}} + 1.4 \cdot LL_c + 1.2 \cdot w_d) \cdot L$$

$$\phi V_n = 6130.00 \frac{\text{lbf}}{\text{ft}}$$

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

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

$$L_{18V} = 21.13 \text{ ft}$$

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

AISI allowable: (Ref. 2.2.14 pg.18)

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

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

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

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

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

$$\phi V_n = 6130.00 \frac{\text{lbf}}{\text{ft}}$$

$$\left[\frac{0.125 \cdot (1.6 \cdot w_{c18} + 1.4 \cdot LL_c + 1.2 \cdot w_d) \cdot L_{18i}^2}{\phi M_n} \right]^2 + \left[\frac{0.625 (1.6 \cdot w_{c18} + 1.4 \cdot LL_c + 1.2 \cdot w_d) \cdot L_{18i}}{\phi V_n} \right]^2 = 1.0$$

$$C_{18m} := 0.125 \cdot \frac{1.6 \cdot w_{c18} + 1.4 \cdot LL_c + 1.2 \cdot w_d}{\phi M_n} \quad C_{18m} = 0.02 \frac{1}{\text{ft}^2}$$

$$C_{18v} := 0.625 \cdot \frac{1.6 \cdot w_{c18} + 1.4 \cdot LL_c + 1.2 \cdot w_d}{\phi V_n} \quad C_{18v} = 0.05 \frac{1}{\text{ft}}$$

$$(C_{18m} \cdot L_{18i}^2)^2 + (C_{18v} \cdot L_{18i})^2 = 1.0$$

C factors to simplify equation and solution.

$$L_{18i} := \frac{1}{2 \cdot C_{18m}} \cdot \left[(-2) \cdot C_{18v}^2 + 2 \cdot \left(C_{18v}^4 + 4 \cdot C_{18m}^2 \right)^{\frac{1}{2}} \right]^{\frac{1}{2}}$$

$L_{18i} = 6.68 \text{ ft}$ **Governs**

Check deflection; limit to the lesser of L/180 or 0.75": (Ref.2.2.14 pg.17&18)

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

$L_{18\Delta} = 11.18 \text{ ft}$

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

$L_{18\Delta 75} = 11.19 \text{ ft}$

Shortest allowable spans are based on bending/shear interaction. These are the longest clear spans allowable for decking, with a minimum of 2 continuous spans:

6.5ft. for 18" slab

With a flange width at about 9" this gives an allowable span between centerlines of beams at 7.25' Use 7'- 0" span for design.

6.5 DESIGN STRUCTURAL ROOF STEEL FRAMING

There are three areas below the roof slabs where the depth of the roof support beams could be impacted by the presence of cranes. The first area lies between column lines (C/Lines) C and E and C/Lines 3 and 6 where the roof beams have a clear span of 70'-0". This area has a roof elevation of 100'-0" (see Ref. 2.2.5). A 15 ton capacity CTM Maintenance Crane is located within this room. The elevation at the top of rails of the CTM maintenance crane is at 82'-3". The top of the trolley is 6'-0" above that and the top of a clearance envelope is at another 2'-0" above that (see dwg. 200-MJ0-HTC0-00101-000, Ref. 2.2.21) Thus the bottom of the roof support steel beams cannot be below elevation 90'-3". With a roof slab depth of 1'-9" this gives a clear space for the support beams of about 8'-0".

The second area lies between C/Lines C and E and 6 and 8 and has a roof elevation of 72'-0". The roof beams also have clear spans of 70'-0". Per dwg. 200-MJ0-HM00-00101-000 (see Ref. 2.2.22), the top of rail elevation for the 200 ton capacity crane located in this area is at 45'-0". The top of the trolley is 13'-3" above this, while the clearance envelope is another 2'-0" above this. Thus the bottom of the roof support beams cannot be lower than elevation 60'-3". With a roof slab depth of 1'-9" this gives a clear space for the roof support beams of about 10'-0".

The third and last area lies between C/Lines B and C and 3 and 4 and has a roof elevation of 64'-0". The roof support beams will have minimum clear spans of 39'-0". This area contains a 10 ton capacity crane whose top of rail elevation is 48'-9" (see dwg. 200-MJ0-HMC0-00101, Ref. 2.2.23). The top of the trolley is 4'-6" above this, while the clearance envelope is another 2'-0" above this, therefore, the bottom of the roof slabs cannot be below elevation 55'-3". With a 1'-9" slab this gives a clear space for the roof support beams of about 7'-0".

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

1. No composite action between the concrete slabs and supporting structural steel members is considered.
2. Decking provides full lateral support to top flanges of beams during construction. Concrete slabs provide lateral support of top flanges of beams during service.
3. The structural steel framing system provides all vertical support for the concrete slabs and superimposed loads in service. Other than spanning between beams, no credit is taken for self-support of the concrete slabs.
4. A992 steel for W-Sections.

$$F_{y50} = 50 \text{ ksi}$$

Using compact sections with continuous lateral support. (See Sect. Q1.5.1 of Ref. 2.2.9)

$$F_b := 0.66 \cdot F_{y50} \quad F_b = 33.00 \text{ ksi}$$

$$F_v := 0.40 \cdot F_{y50} \quad F_v = 20.00 \text{ ksi}$$

$$F_t := 0.60 \cdot F_{y50} \quad F_t = 30 \text{ ksi}$$

AISC N690 (Ref. 2.2.10 Table Q1.5.7.1) allows a 1.4 and 1.6 stress increase with seismic load combinations.

$$F_{be} := 1.6 \cdot F_b \quad F_{be} = 52.80 \text{ ksi}$$

$$F_{ve} := 1.4 \cdot F_v \quad F_{ve} = 28.00 \text{ ksi}$$

$$F_{te} := 1.6 \cdot F_t \quad F_{te} = 48 \text{ ksi}$$

Roof Loads (See Section 6.2)

Total Dead Load:

$$DL_{\text{framing}} = 40.00 \text{ psf} \quad \text{Includes decking weight.}$$

$$DL_{\text{eqroof}} = 10.00 \text{ psf}$$

$$DL_{\text{roofing}} := 55.00 \text{ psf}$$

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

$$DL_{\text{roof}} := DL_{\text{roofing}} + DL_{\text{framing}} + DL_{\text{eqroof}} + w_{c18} \quad DL_{\text{roof}} = 349 \text{ psf}$$

$$LL_{\text{roof}} = 40.00 \text{ psf} \quad \text{Governs over of the Live Loads, see Sect. 6.2, pg.11}$$

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 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.

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

4. In addition of the above, the deflection of each member will be checked against the deflection limit of $L/240$, which is based on section 1604.3.6 in Table 1604.3 of the IBC (Ref. 2.2.18)

Size Steel Beams Supporting Roofs Between Col.Line C/E and Col.L 2/8 @ EL. 72' and 100'

Clear spans between walls are 70'. Consider beam with A & B supports

$$L_{\text{spanAB}} := 70.0 \cdot \text{ft}$$

Maximum tributary width of slab is 7.0', as shown on the preliminary framing layout.

$$w_{\text{AB}} := 7.0 \text{ft} \cdot (LL_{\text{roof}} + DL_{\text{roof}}) \quad w_{\text{AB}} = 2.72 \text{ klf}$$

Use simple span:(See Assumptions 3.2.1, 3.2.2, 3.2.3)

$$M_{\text{AB}} := \frac{w_{\text{AB}} \cdot L_{\text{spanAB}}^2}{8} \quad M_{\text{AB}} = 1667 \text{ ft} \cdot \text{kip}$$

$$R_{\text{AB}} := \frac{w_{\text{AB}} \cdot L_{\text{spanAB}}}{2} \quad R_{\text{AB}} = 95.2 \text{ kip}$$

$$S_{\text{reqAB}} := \frac{M_{\text{AB}}}{F_b} \quad S_{\text{reqAB}} = 606.1 \text{ in}^3$$

W44X262 has the following properties:(Ref. 2.2.20 page 1-10 and 1-11)

$$S_{\text{W44x262}} := 1110 \text{ in}^3 \quad I_{\text{W44x262}} := 24100 \text{ in}^4 \quad t_{\text{fW44x262}} := 1.42 \cdot \text{in}$$

$$d_{\text{W44x262}} := 43.3 \text{ in} \quad t_{\text{wW44x262}} := 0.785 \text{ in} \quad N_{\text{W44x262}} := 15.0 \cdot \text{in} \quad (\text{See pg.7, Assumption 3.1.2})$$

$$f_{\text{vW44x262}} := \frac{R_{\text{AB}}}{d_{\text{W44x262}} \cdot t_{\text{wW44x262}}} \quad f_{\text{vW44x262}} = 2.80 \text{ ksi} \quad \text{OK}$$

Check W44X262 for extreme loads of $1.6S > LL + DL + E$ Ref. 2.2.1 Sect. 4.2.11.4.6(9).

The peak ground vertical acceleration is $SA = 2 \times 0.79g$ for the 2000-year seismic event, at 5% damping, and occurs at a period of 0.10 sec. at El. 72 & 100' (Conservatively) (Table in Sect. 6.2, pg.12 and Assumption 3.1.1)

$$SA_{100} := 2 \cdot 0.79 \quad SA_{100} = 1.58$$

$$F_{be} = 52.80 \text{ ksi} \quad F_{ve} = 28.00 \text{ ksi}$$

Use 25% LL during earthquake. (Ref.2.2.3 Sect.10.3.1)

$$W_{eAB} := 7.0\text{ft} \cdot [DL_{\text{roof}} + 0.25 \cdot LL_{\text{roof}} + SA_{100} \cdot (DL_{\text{roof}} + 0.25 \cdot LL_{\text{roof}})] \quad W_{eAB} = 6.48 \text{ klf}$$

$$M_{eAB} := \frac{W_{eAB} \cdot L_{\text{spanAB}}^2}{8} \quad M_{eAB} = 3968 \text{ ft} \cdot \text{kip}$$

$$R_{eAB} := \frac{W_{eAB} \cdot L_{\text{spanAB}}}{2} \quad R_{eAB} = 226.8 \text{ kip}$$

$$S_{\text{ereqAB}} := \frac{M_{eAB}}{F_{be}} \quad S_{\text{ereqAB}} = 901.9 \text{ in}^3 < 1110.0 \text{ in}^3 \quad \text{O'K}$$

$$\text{Demand/Capacity Ratio DC} \quad DC := \frac{S_{\text{ereqAB}}}{S_{W44x262}} \quad DC = 0.81$$

Web Crippling Check (Ref.2.2.9, Formula Q1.10-10)

$$R := 34t_{wW44x262}^2 \cdot \left[1 + 3 \cdot \left(\frac{N_{W44x262}}{d_{W44x262}} \right) \cdot \left(\frac{t_{wW44x262}}{t_{fW44x262}} \right)^{1.5} \right] \cdot \left[\sqrt{F_{y50} \cdot 1.6 \cdot \left(\frac{t_{fW44x262}}{t_{wW44x262}} \right)} \right] \cdot \frac{\sqrt{\text{kip}}}{\text{in}}$$

$$R = 359.71 \text{ kip} > 226.8 \text{ kip} \quad \text{O'K}$$

No Stiffeners Required

**Deflection Check
(DL only)**

Deflections of the long span beams are investigated for dead load and construction loads only (LL is much smaller than DL) to determine if camber should be added to offset construction loads deflections also composite beam action will reduce deflection between 33 and 50%.

$$E := 29000 \cdot \text{ksi}$$

$$W_{ABd} := 7.0\text{ft} \cdot (\text{DL}_{\text{roof}}) \quad W_{ABd} = 2.44 \text{ klf}$$

$$d_{\text{max}} := 5 \cdot W_{ABd} \cdot \frac{L_{\text{spanAB}}^4}{384E \cdot I_{W44x262}} \quad d_{\text{max}} = 1.89 \text{ in}$$

$$d_{\text{allow}} := \frac{70 \cdot 12 \cdot \text{in}}{240} \quad d_{\text{allow}} = 3.50 \text{ in} > d_{\text{max}} \quad \text{O'K} \quad (\text{Ref. 2.2.18 Table 1604.3})$$

**DEFLECTION CHECK:
Based on**

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:

The depth of fully stressed roof beams should not be less than $F_y/1000$ times the span.

$$F_y := 50 \quad \frac{F_y}{1000} = 0.0500$$

Beam depth Required:

$$d = 0.0500 \times 70\text{ft} \times 12" = 42" < W44X262 \quad \text{O'K}$$

Use W44X262 for Roof Beams with 70.0 ft. spans between Col.Line C/E and Col.Line 2/8 @ EL.72' and 100'

Size Steel Beams Supporting Roofs Between Col.Line A/B, B/C, A/C, E/F and Col.Line 3/4 and 6/7 @ EL. 64'

Clear spans between walls are 35' Consider beam having A & B supports.

$$L_{\text{spanAB}} := 35.0 \cdot \text{ft}$$

Maximum tributary width of slab is 7.0', as shown on the preliminary framing layout.

$$w_{\text{AB}} := 7.0 \text{ft} \cdot (LL_{\text{roof}} + DL_{\text{roof}}) \quad w_{\text{AB}} = 2.72 \text{ klf}$$

Use simple span:(Assumption 3.2.1, 3.2.2, 3.2.3)

$$M_{\text{AB}} := \frac{w_{\text{AB}} \cdot L_{\text{spanAB}}^2}{8} \quad M_{\text{AB}} = 417 \text{ ft} \cdot \text{kip}$$

$$R_{\text{AB}} := \frac{w_{\text{AB}} \cdot L_{\text{spanAB}}}{2} \quad R_{\text{AB}} = 47.6 \text{ kip}$$

$$S_{\text{reqAB}} := \frac{M_{\text{AB}}}{F_b} \quad S_{\text{reqAB}} = 151.5 \text{ in}^3$$

W30X108 has the following properties:

$$S_{\text{W30x108}} := 299 \text{ in}^3 \quad I_{\text{W30x108}} := 4470 \text{ in}^4 \quad t_{\text{W30x108}} := 0.760 \cdot \text{in}$$

$$d_{\text{W30x108}} := 29.83 \text{ in} \quad t_{\text{wW30x108}} := 0.545 \text{ in} \quad N_{\text{W30x108}} := 8.0 \text{ in} \quad (\text{pg.7, Assumption 3.1.2})$$

$$f_{\text{wW30x108}} := \frac{R_{\text{AB}}}{d_{\text{W30x108}} \cdot t_{\text{wW30x108}}} \quad f_{\text{wW30x108}} = 2.93 \text{ ksi} \quad \text{OK}$$

Check W30X108 for extreme loads of $1.6S > LL + DL + E$ Ref. 2.2.1 Sect. 4.2.11.4.6(9).

The peak ground vertical acceleration is $SA = 2.0 \times 0.72g$ for the 2000-year seismic event, at 5% damping, and occurs at a period of 0.10 sec. at El. 64' (See Table in Sect 6.2 pg.12, and Assumption 3.1.1)

$$SA := 2.0 \cdot 0.72 \quad SA = 1.44$$

$$F_{be} = 52.80 \text{ ksi} \quad F_{ve} = 28.00 \text{ ksi}$$

Use 25% LL during earthquake. (Ref.2.2.3 Sect.10.3.1)

$$W_{eAB} := 7.0 \text{ ft} \cdot [DL_{\text{roof}} + 0.25 \cdot LL_{\text{roof}} + SA \cdot (DL_{\text{roof}} + 0.25 \cdot LL_{\text{roof}})] \quad W_{eAB} = 6.13 \text{ klf}$$

$$M_{eAB} := \frac{W_{eAB} \cdot L_{\text{spanAB}}^2}{8} \quad M_{eAB} = 938 \text{ ft} \cdot \text{kip}$$

$$R_{eAB} := \frac{W_{eAB} \cdot L_{\text{spanAB}}}{2} \quad R_{eAB} = 107.2 \text{ kip}$$

$$S_{\text{ereqAB}} := \frac{M_{eAB}}{F_{be}} \quad S_{\text{ereqAB}} = 213.2 \text{ in}^3 < 299.0 \text{ in}^3 \quad \text{O'K}$$

Demand/Capacity Ratio DC

$$DC := \frac{S_{\text{ereqAB}}}{S_{W30x108}} \quad DC = 0.71$$

Web Crippling Check (Ref.2.2.9, Formula Q1.10-10)

$$R := 34 t_{wW30x108}^2 \cdot \left[1 + 3 \cdot \left(\frac{N_{W30x108}}{d_{W30x108}} \right) \cdot \left(\frac{t_{wW30x108}}{t_{fW30x108}} \right)^{1.5} \right] \cdot \left[\sqrt{F_{y50} \cdot 1.6 \cdot \left(\frac{t_{fW30x108}}{t_{wW30x108}} \right)} \right] \cdot \frac{\sqrt{\text{kip}}}{\text{in}}$$

$$R = 158.78 \text{ kip} > 107.2 \text{ kip} \quad \text{O'K}$$

No Stiffeners Required

**Deflection Check
(DL only)**

$$E := 29000 \cdot \text{ksi}$$

$$w_{ABd} := 7.0 \text{ft} \cdot (\text{DL}_{\text{roof}})$$

$$w_{ABd} = 2.44 \text{ klf}$$

$$d_{\text{max}} := 5 \cdot w_{ABd} \cdot \frac{L_{\text{spanAB}}^4}{384 E \cdot I_{W30 \times 108}}$$

$$d_{\text{max}} = 0.64 \text{ in}$$

$$d_{\text{allow}} := \frac{35 \cdot 12 \cdot \text{in}}{240}$$

$$d_{\text{allow}} = 1.75 \text{ in} > d_{\text{max}} \quad \text{O'K} \quad (\text{Ref. 2.2.18 Table 1604.3})$$

DEFLECTION CHECK :

Based on N690 criteria (Ref. 2.2.9 Sect.CQ1.13)

$$F_w := 50 \quad \frac{F_y}{1000} = 0.0500$$

Beam depth Required:

$$d = 0.0500 \times 35 \text{ft} \times 12" = 21" < W30 \times 108 \quad \text{O'K}$$

Use W30X108 for Roof Beams with 34'-8" and 35.0 ft. span between Col.Line A/C, E/F and Col.Line 6/7 also between Col.Line A/B and Col.Line 3/4 @ EL. 64'

Size Steel Beams Supporting Roofs Between Col.Line B/C, C/E, E/F and Col.Line 2/3, 3/4 @ EL. 64'

Clear spans between walls are 39' and 42' Consider beam having A & B supports.

$$L_{\text{spanAB}} := 42.0 \cdot \text{ft}$$

Maximum tributary width of slab is 7.0', as shown on the preliminary framing layout.

$$w_{\text{AB}} := 7.0 \text{ft} \cdot (\text{LL}_{\text{roof}} + \text{DL}_{\text{roof}}) \quad w_{\text{AB}} = 2.72 \text{ klf}$$

Use simple span: (Assumptions 3.2.1, 3.2.2, 3.2.3)

$$M_{\text{AB}} := \frac{w_{\text{AB}} \cdot L_{\text{spanAB}}^2}{8} \quad M_{\text{AB}} = 600 \text{ ft} \cdot \text{kip}$$

$$R_{\text{AB}} := \frac{w_{\text{AB}} \cdot L_{\text{spanAB}}}{2} \quad R_{\text{AB}} = 57.1 \text{ kip}$$

$$S_{\text{reqAB}} := \frac{M_{\text{AB}}}{F_b} \quad S_{\text{reqAB}} = 218.2 \text{ in}^3$$

W36X135 has the following properties:

$$S_{\text{W36x135}} := 439 \text{ in}^3 \quad I_{\text{W36x135}} := 7800 \text{ in}^4 \quad t_{\text{W36x135}} := 0.790 \text{ in}$$

$$d_{\text{W36x135}} := 35.55 \text{ in} \quad t_{\text{wW36x135}} := 0.600 \text{ in} \quad N_{\text{W36x135}} := 12.0 \text{ in} \quad (\text{See pg.7, Assumption 3.1.2})$$

$$f_{\text{vW36x135}} := \frac{R_{\text{AB}}}{d_{\text{W36x135}} \cdot t_{\text{wW36x135}}} \quad f_{\text{vW36x135}} = 2.68 \text{ ksi} \quad \text{OK}$$

Check W36X135 for extreme loads of $1.6S > LL + DL + E$ Ref. 2.1.1 Sect. 4.2.11.4.6(9).

The peak ground vertical acceleration is $SA = 2.0 \times 0.72g$ for the 2000-year seismic event, at 5% damping, and occurs at a period of 0.10 sec. at El. 64' (See Table in Sect.6.2 pg.12, and Assumption 3.1.1)

$$SA := 0.72 \cdot 2.0 \quad SA = 1.44$$

$$F_{be} = 52.80 \text{ ksi} \quad F_{ve} = 28.00 \text{ ksi}$$

Use 25% LL during earthquake. (Ref.2.2.3 Sect.10.3.1)

$$W_{eAB} := 7.0 \text{ ft} \cdot [DL_{\text{roof}} + 0.25 \cdot LL_{\text{roof}} + SA \cdot (DL_{\text{roof}} + 0.25 \cdot LL_{\text{roof}})] \quad W_{eAB} = 6.13 \text{ klf}$$

$$M_{eAB} := \frac{W_{eAB} \cdot L_{\text{spanAB}}^2}{8} \quad M_{eAB} = 1351 \text{ ft} \cdot \text{kip}$$

$$R_{eAB} := \frac{W_{eAB} \cdot L_{\text{spanAB}}}{2} \quad R_{eAB} = 128.7 \text{ kip}$$

$$S_{\text{ereqAB}} := \frac{M_{eAB}}{F_{be}} \quad S_{\text{ereqAB}} = 307.1 \text{ in}^3 < 439.0 \text{ in}^3 \quad \text{O'K}$$

$$\text{Demand/Capacity Ratio DC} \quad DC := \frac{S_{\text{ereqAB}}}{S_{W36x135}} \quad DC = 0.70$$

Web Crippling Check (Ref.2.2.9, Formula Q1.10-10)

$$R := 34 t_w W_{36x135}^2 \cdot \left[1 + 3 \cdot \left(\frac{N_{W36x135}}{d_{W36x135}} \right) \cdot \left(\frac{t_w W_{36x135}}{t_f W_{36x135}} \right)^{1.5} \right] \cdot \left[\sqrt{F_{y50} \cdot 1.6 \cdot \left(\frac{t_f W_{36x135}}{t_w W_{36x135}} \right)} \right] \cdot \frac{\sqrt{\text{kip}}}{\text{in}}$$

$$R = 209.82 \text{ kip} > 128.7 \text{ kip} \quad \text{O'K}$$

No Stiffeners Required

**Deflection Check
(DL only)**

$$E := 29000 \cdot \text{ksi}$$

$$w_{ABd} := 7.0 \text{ft} \cdot (\text{DL}_{\text{roof}}) \quad w_{ABd} = 2.44 \text{ klf}$$

$$d_{\text{max}} := 5 \cdot w_{ABd} \cdot \frac{L_{\text{spanAB}}^4}{384E \cdot I_{W36x135}} \quad d_{\text{max}} = 0.76 \text{ in}$$

$$d_{\text{allow}} := \frac{39 \cdot 12 \cdot \text{in}}{240} \quad d_{\text{allow}} = 1.95 \text{ in} > d_{\text{max}} \quad \text{O'K} \quad (\text{Ref. 2.2.18 Table 1604.3})$$

DEFLECTION CHECK :

Based on N690 criteria (Ref. 2.2.9 Sect.CQ1.13)

$$F_w := 50 \quad \frac{F_y}{1000} = 0.0500$$

Beam depth Required:

$$d = 0.0500 \times 42 \text{ft} \times 12" = 25.2" < W36X135 \quad \text{O'K}$$

Use W36X135 for Roof Beams with 39, 42 ft. span between Col.Line B/C, C/E, E/F and Col.Line 3/4, 2/3 @ EL. 64'

Size Steel Beams Supporting Roofs Between Col.Line A/C, E/F and Col.Line 7/8 @ EL. 64'

Clear spans between walls are 48.33' Consider beam having A & B supports

$$L_{\text{spanAB}} := 48.33 \cdot \text{ft}$$

Maximum tributary width of slab is 7.0', as shown on the preliminary framing layout.

$$w_{\text{AB}} := 7.0 \text{ft} \cdot (\text{LL}_{\text{roof}} + \text{DL}_{\text{roof}}) \quad w_{\text{AB}} = 2.72 \text{ klf}$$

Use simple span: (Assumption 3.2.1, 3.2.2, 3.2.3)

$$M_{\text{AB}} := \frac{w_{\text{AB}} \cdot L_{\text{spanAB}}^2}{8} \quad M_{\text{AB}} = 795 \text{ ft} \cdot \text{kip}$$

$$R_{\text{AB}} := \frac{w_{\text{AB}} \cdot L_{\text{spanAB}}}{2} \quad R_{\text{AB}} = 65.8 \text{ kip}$$

$$S_{\text{reqAB}} := \frac{M_{\text{AB}}}{F_b} \quad S_{\text{reqAB}} = 288.9 \text{ in}^3$$

W36X170 has the following properties:

$$S_{\text{W36x170}} := 580 \text{ in}^3 \quad I_{\text{W36x170}} := 10500 \text{ in}^4 \quad t_{\text{W36x170}} := 1.100 \text{ in}$$

$$d_{\text{W36x170}} := 36.17 \text{ in} \quad t_{\text{wW36x170}} := 0.680 \text{ in} \quad N_{\text{W36x170}} := 12.0 \text{ in} \quad (\text{See pg.7, Assumption 3.1.2})$$

$$f_{\text{wW36x170}} := \frac{R_{\text{AB}}}{d_{\text{W36x170}} \cdot t_{\text{wW36x170}}} \quad f_{\text{wW36x170}} = 2.67 \text{ ksi} \quad \text{OK}$$

Check W36X170 for extreme loads of $1.6S > LL + DL + E$ Ref. 2.2.1 Sect. 4.2.11.4.6(9).

The peak ground vertical acceleration is $SA = 2.0 \times 0.72g$ for the 2000-year seismic event, at 5% damping, and occurs at a period of 0.10 sec. at El. 64' (See Table in Section 6.2 pg.12 Assumption 3.1.1).

$$SA := 0.72 \cdot 2.0 \quad SA = 1.44$$

$$F_{be} = 52.80 \text{ ksi} \quad F_{ve} = 28.00 \text{ ksi}$$

Use 25% LL during earthquake. (Ref.2.2.3 Sect.10.3.1)

$$W_{eAB} := 7.0\text{ft} \cdot [DL_{\text{roof}} + 0.25 \cdot LL_{\text{roof}} + SA \cdot (DL_{\text{roof}} + 0.25 \cdot LL_{\text{roof}})] \quad W_{eAB} = 6.13 \text{ klf}$$

$$M_{eAB} := \frac{W_{eAB} \cdot L_{\text{spanAB}}^2}{8} \quad M_{eAB} = 1789 \text{ ft} \cdot \text{kip}$$

$$R_{eAB} := \frac{W_{eAB} \cdot L_{\text{spanAB}}}{2} \quad R_{eAB} = 148.1 \text{ kip}$$

$$S_{\text{ereqAB}} := \frac{M_{eAB}}{F_{be}} \quad S_{\text{ereqAB}} = 406.6 \text{ in}^3 < 580 \text{ in}^3 \quad \text{O'K}$$

Demand/Capacity Ratio DC

$$DC := \frac{S_{\text{ereqAB}}}{S_{W36x170}} \quad DC = 0.70$$

Web Crippling Check (Ref.2.2.9, Formula Q1.10-10)

$$R := 34t_{wW36x170}^2 \cdot \left[1 + 3 \cdot \left(\frac{N_{W36x170}}{d_{W36x170}} \right) \cdot \left(\frac{t_{wW36x170}}{t_{fW36x170}} \right)^{1.5} \right] \cdot \left[\sqrt{F_{y50} \cdot 1.6 \cdot \left(\frac{t_{fW36x170}}{t_{wW36x170}} \right)} \right] \cdot \frac{\sqrt{\text{kip}}}{\text{in}}$$

$$R = 265.37 \text{ kip} > 148.1 \text{ kip} \quad \text{O'K}$$

No Stiffeners Required

**Deflection Check
(DL only)**

$$E := 29000 \cdot \text{ksi}$$

$$W_{ABd} := 7.0\text{ft} \cdot (\text{DL}_{\text{roof}}) \quad W_{ABd} = 2.44 \text{ klf}$$

$$d_{\text{max}} := 5 \cdot W_{ABd} \cdot \frac{L_{\text{spanAB}}^4}{384E \cdot I_{W36 \times 170}} \quad d_{\text{max}} = 0.98 \text{ in}$$

$$d_{\text{allow}} := \frac{48.33 \cdot 12 \cdot \text{in}}{240} \quad d_{\text{allow}} = 2.42 \text{ in} > d_{\text{max}} \quad \text{O'K} \quad (\text{Ref. 2.2.18 Table 1604.3})$$

DEFLECTION CHECK :

Based on N690 criteria (Ref. 2.2.9 Sect CQ1.13)

$$F_w := 50 \quad \frac{F_y}{1000} = 0.0500$$

Beam depth Required:

$$d = 0.0500 \times 48.33\text{ft} \times 12" = 28.99" < W36 \times 170 \quad \text{O'K}$$

Use W36X170 for Roof Beams with 48.33 ft. span between Col.Line A/C, E/F and Col.Line 7/8 @ EL. 64'

Size Steel Beams Supporting Roofs Between Col.Line A/C, E/F and Col.Line 4/6 @ EL. 64'

Clear spans between walls are 55.0' Consider beam having A & B supports

$$L_{\text{spanAB}} := 55.0 \cdot \text{ft}$$

Maximum tributary width of slab is 7.0', as shown on the preliminary framing layout.

$$w_{\text{AB}} := 7.0 \cdot \text{ft} \cdot (\text{LL}_{\text{roof}} + \text{DL}_{\text{roof}}) \quad w_{\text{AB}} = 2.72 \text{ klf}$$

Use simple span: (Assumptions 3.2.1, 3.2.2, 3.2.3)

$$M_{\text{AB}} := \frac{w_{\text{AB}} \cdot L_{\text{spanAB}}^2}{8} \quad M_{\text{AB}} = 1029 \text{ ft} \cdot \text{kip}$$

$$R_{\text{AB}} := \frac{w_{\text{AB}} \cdot L_{\text{spanAB}}}{2} \quad R_{\text{AB}} = 74.8 \text{ kip}$$

$$S_{\text{reqAB}} := \frac{M_{\text{AB}}}{F_b} \quad S_{\text{reqAB}} = 374.2 \text{ in}^3$$

W36X210 has the following properties:

$$S_{\text{W36x210}} := 719 \text{ in}^3 \quad I_{\text{W36x210}} := 13200 \text{ in}^4 \quad t_{\text{fW36x210}} := 1.360 \text{ in}$$

$$d_{\text{W36x210}} := 36.69 \text{ in} \quad t_{\text{wW36x210}} := 0.830 \text{ in} \quad N_{\text{W36x210}} := 12.0 \text{ in} \quad (\text{See pg.7, Assumption 3.1.2})$$

$$f_{\text{vW36x210}} := \frac{R_{\text{AB}}}{d_{\text{W36x210}} \cdot t_{\text{wW36x210}}} \quad f_{\text{vW36x210}} = 2.46 \text{ ksi} \quad \text{OK}$$

Check W36X210 for extreme loads of $1.6S > LL + DL + E$ Ref. 2.2.1 Sect. 4.2.11.4.6(9).

The peak ground vertical acceleration is $SA = 2.0 \times 0.72g$ for the 2000-year seismic event, at 5% damping, and occurs at a period of 0.10 sec. at El. 64' (See Table in Sect.6.2 pg. 12 and Assumption 3.1.1)

$$SA := 0.72 \cdot 2.0 \quad SA = 1.44$$

$$F_{be} = 52.80 \text{ ksi} \quad F_{ve} = 28.00 \text{ ksi}$$

Use 25% LL during earthquake. (Ref. 2.2.3 Sect.10.3.1)

$$W_{eAB} := 7.0 \text{ ft} \cdot [DL_{\text{roof}} + 0.25LL_{\text{roof}} + SA \cdot (DL_{\text{roof}} + 0.25 \cdot LL_{\text{roof}})] \quad W_{eAB} = 6.13 \text{ klf}$$

$$M_{eAB} := \frac{W_{eAB} \cdot L_{\text{spanAB}}^2}{8} \quad M_{eAB} = 2317 \text{ ft} \cdot \text{kip}$$

$$R_{eAB} := \frac{W_{eAB} \cdot L_{\text{spanAB}}}{2} \quad R_{eAB} = 168.5 \text{ kip}$$

$$S_{\text{reqAB}} := \frac{M_{eAB}}{F_{be}} \quad S_{\text{reqAB}} = 526.6 \text{ in}^3 < 719.0 \text{ in}^3 \quad \text{O'K}$$

$$\text{Demand/Capacity Ratio DC} \quad DC := \frac{S_{\text{reqAB}}}{S_{W36x210}} \quad DC = 0.73$$

Web Crippling Check (Ref.2.2.9, Formula Q1.10-10)

$$R := 34t_{wW36x210}^2 \cdot \left[1 + 3 \cdot \left(\frac{N_{W36x210}}{d_{W36x210}} \right) \cdot \left(\frac{t_{wW36x210}}{t_{fW36x210}} \right)^{1.5} \right] \cdot \left[\sqrt{F_{y50} \cdot 1.6 \cdot \left(\frac{t_{fW36x210}}{t_{wW36x210}} \right)} \right] \cdot \frac{\sqrt{\text{kip}}}{\text{in}}$$

$$R = 393.62 \text{ kip} > 168.5 \text{ kip} \quad \text{O'K}$$

No Stiffeners Required

**Deflection Check
(DL only)**

$$E := 29000 \cdot \text{ksi}$$

$$W_{ABd} := 7.0 \text{ft} \cdot (\text{DL}_{\text{roof}})$$

$$W_{ABd} = 2.44 \text{ klf}$$

$$d_{\text{max}} := 5 \cdot W_{ABd} \cdot \frac{L_{\text{spanAB}}^4}{384E \cdot I_{W36x210}}$$

$$d_{\text{max}} = 1.31 \text{ in}$$

$$d_{\text{allow}} := \frac{55 \cdot 12 \cdot \text{in}}{240}$$

$$d_{\text{allow}} = 2.75 \text{ in}$$

$$> d_{\text{max}}$$

O'K (Ref. 2.2.18 Table 1604.3)

DEFLECTION CHECK :

Based on N690 criteria (Ref. 2.2.9 Sect.CQ1.13)

$$F_w := 50 \quad \frac{F_y}{1000} = 0.0500$$

Beam depth Required:

$$d = 0.0500 \times 48.33 \text{ft} \times 12" = 33" < W36X210 \quad \text{O'K}$$

Use W36X210 for Roof Beams with 55.0 ft. span between Col.Line A/C, E/F and Col.Line 4/6 @ EL. 64'

SUMMARY:

BEAM SIZES AT EACH ROOF LEVEL

EL. 100'

W44X262 pg.22

EL. 72'

W44X262 pg.22

EL. 64'

W30X108	Col.Lines A/B, A/C, E/F and 3/4, 6/7	pg. 25
W36X135	Col.Lines B/C, C/E, E/F and 2/3, 3/4	pg. 28
W36X170	Col.Lines A/C, E/F, and 7/8	pg. 31
W36x210	Col. Lines A/C, E/F, and 4/6	pg. 34

6.6 DESIGN FLOOR STRUCTURAL FRAMING

The steel floor deck can span a maximum of 7.0 ft with 18 in slab. Based on the building room plans, the maximum required c/c spacing between floor beams is 7.0 ft. the beam design is based on spacing.

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 is considered. The compression cords of the roof beams/girders are designed as partially composite with the roof slab.
2. Decking provides full lateral support to top flanges of beams during construction. Concrete slabs provide lateral support of top flanges of beams top cords during service.
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. Other than spanning between beams, no credit is taken for self-support of the concrete slabs.
4. A992 steel is use for W-Sections.

The following loads are based on the (RF) Mass Properties Calculations (Ref.2.2.12)

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:

The depth of fully stressed beams in floors should not be less than $F_y/800$ times the span.

For human comfort, the depth of steel beams supporting large open floor areas should not be less than 1/20 of the span.

$$F_y := 50 \quad \frac{F_y}{800} = 0.0625$$

FLOOR LOADS COMBINED

The weight of the 18" slab on 3" steel deck is:

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

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

Total Floor Dead Load:

$$DL_{eqfloor} := 100 \text{psf}$$

$$DL_{framing} := 40 \text{psf}$$

$$DL_{floor} := DL_{framing} + DL_{eqfloor} + w_{c18}$$

$$DL_{floor} = 384 \text{psf}$$

$$LL_{floor} := 100 \text{psf}$$

Contribution of LL to seismic mass:

$$LL_{efloor} := 0.25LL_{floor}$$

$$LL_{efloor} = 25 \text{psf}$$

Size Steel Beams Supporting Floors Between Col.Line C/E, E/F and Col.Line 2/3, 3/4 & 8/9 @ EL. 32'

Clear spans between walls are 39', 42' Consider beam having A & B supports on C/Lines 3 & 4

$$L_{spanAB} := 42.0 \cdot \text{ft} \quad (\text{Use span of 42' for both cases})$$

Maximum tributary width of slab is 7.0', as shown on the preliminary framing layout.

$$W_{AB} := 7.0 \text{ft} \cdot (DL_{\text{floor}} + LL_{\text{floor}}) \quad W_{AB} = 3.39 \text{ klf}$$

Use simple span: (Assumptions, 3.2.1, 3.2.2, 3.2.3)

$$M_{AB} := \frac{W_{AB} \cdot L_{spanAB}^2}{8} \quad M_{AB} = 747 \text{ ft} \cdot \text{kip}$$

$$R_{AB} := \frac{W_{AB} \cdot L_{spanAB}}{2} \quad R_{AB} = 71.1 \text{ kip}$$

$$S_{reqAB} := \frac{M_{AB}}{F_b} \quad S_{reqAB} = 271.5 \text{ in}^3$$

W36X150 has the following properties:

$$S_{W36x150} := 504 \text{ in}^3 \quad I_{W36x150} := 9040 \text{ in}^4 \quad t_{W36x150} := 0.940 \text{ in}$$

$$d_{W36x150} := 35.85 \text{ in} \quad t_{wW36x150} := 0.625 \text{ in} \quad N_{W36x150} := 12 \text{ in} \quad (\text{See pg.7, Assumption 3.1.2})$$

$$f_{vW36x150} := \frac{R_{AB}}{d_{W36x150} \cdot t_{wW36x150}} \quad f_{vW36x150} = 3.17 \text{ ksi} \quad \text{OK}$$

Check W36X150 for extreme loads of $1.6S > LL + DL + E$ Ref. 2.2.1 Sect. 4.2.11.4.6(9).

The peak ground vertical acceleration is $SA = 2.0 \times 0.67g$ for the 2000-year seismic event, at 5% damping, and occurs at a period of 0.10 sec. at El. 32' use for all elevations. (See Table in Sect. 6.2, pg.12 and Assumption 3.1.1)

$$SA := 0.67 \cdot 2.0 \quad SA = 1.34$$

$$F_{be} = 52.80 \text{ ksi} \quad F_{ve} = 28.00 \text{ ksi}$$

Use 25% LL during earthquake. (Ref.2.2.3 Sect. 10.3.1)

$$W_{eAB} := 7.0\text{ft} \cdot [DL_{\text{floor}} + 0.25LL_{\text{floor}} + SA \cdot (DL_{\text{floor}} + LL_{\text{efloor}})] \quad W_{eAB} = 6.70 \text{ klf}$$

$$M_{eAB} := \frac{W_{eAB} \cdot L_{\text{spanAB}}^2}{8} \quad M_{eAB} = 1476 \text{ ft} \cdot \text{kip}$$

$$R_{eAB} := \frac{W_{eAB} \cdot L_{\text{spanAB}}}{2} \quad R_{eAB} = 140.6 \text{ kip}$$

$$S_{\text{ereqAB}} := \frac{M_{eAB}}{F_{be}} \quad S_{\text{ereqAB}} = 335.5 \text{ in}^3 < 504.0 \text{ in}^3 \quad \text{O'K}$$

$$\text{Demand/Capacity Ratio DC} \quad DC := \frac{S_{\text{ereqAB}}}{S_{W36x150}} \quad DC = 0.67$$

Web Crippling Check (Ref.2.2.9, Formula Q1.10-10)

$$R := 34t_{wW36x150}^2 \left[1 + 3 \cdot \left(\frac{N_{W36x150}}{d_{W36x150}} \right) \cdot \left(\frac{t_{wW36x150}}{t_{fW36x150}} \right)^{1.5} \right] \cdot \left[\sqrt{F_{y50} \cdot 1.6 \cdot \left(\frac{t_{fW36x150}}{t_{wW36x150}} \right)} \right] \cdot \frac{\sqrt{\text{kip}}}{\text{in}}$$

$R = 225.00 \text{ kip} > 140.6 \text{ kip}$ O'K

No Stiffeners Required

**Deflection Check
(DL only)**

$E := 29000 \cdot \text{ksi}$

$w_{ABd} := 7.0 \text{ ft} \cdot (\text{DL}_{\text{floor}})$ $w_{ABd} = 2.69 \text{ klf}$

$d_{\text{max}} := 5 \cdot w_{ABd} \cdot \frac{L_{\text{spanAB}}^4}{384E \cdot I_{W36x150}}$ $d_{\text{max}} = 0.72 \text{ in}$

$d_{\text{allow}} := \frac{42.0 \cdot 12 \cdot \text{in}}{240}$ $d_{\text{allow}} = 2.10 \text{ in} > d_{\text{max}}$ O'K (Ref. 2.2.18 Table 1604.3)

DEFLECTION CHECK :

Based on N690 criteria (Ref. 2.2.9 Sect.CQ1.13)

$F_w := 50$ $\frac{F_y}{800} = 0.0625$

Beam depth Required:

$1/20 \times 42 \text{ ft} \times 12" = 25.2" < W36X150$ O'K

$d = 0.0625 \times 42 \text{ ft} \times 12" = 31.5" < W36X150$ O'K

Use W36X150 for Floor Beams with 42.0 ft. span between Col.Line C/E, E/F and Col.Line 2/3, 3/4 and 8/9 @ EL. 32'

**Size Steel Beams Supporting Floors Between Col.Line A/B, A/C, E/F and Col.Line
3/4 & 6/7 @ EL. 32'**

Clear spans between walls are 35.0' Consider beam having A & B supports

$$L_{\text{spanAB}} := 35.0 \cdot \text{ft}$$

Maximum tributary width of slab is 7.0', as shown on the preliminary framing layout.

$$w_{\text{AB}} := 7.0 \text{ft} \cdot (\text{DL}_{\text{floor}} + \text{LL}_{\text{floor}}) \quad w_{\text{AB}} = 3.39 \text{ klf}$$

Use simple span:(Assumptions 3.2.1, 3.2.2, 3.2.3)

$$M_{\text{AB}} := \frac{w_{\text{AB}} \cdot L_{\text{spanAB}}^2}{8} \quad M_{\text{AB}} = 519 \text{ ft} \cdot \text{kip}$$

$$R_{\text{AB}} := \frac{w_{\text{AB}} \cdot L_{\text{spanAB}}}{2} \quad R_{\text{AB}} = 59.3 \text{ kip}$$

$$S_{\text{reqAB}} := \frac{M_{\text{AB}}}{F_b} \quad S_{\text{reqAB}} = 188.6 \text{ in}^3$$

W30X116 has the following properties:

$$S_{W30x116} := 329 \text{ in}^3 \quad I_{W30x116} := 4930 \text{ in}^4 \quad t_{W30x116} := 0.850 \text{ in}$$

$$d_{W30x116} := 30.01 \text{ in} \quad t_{wW30x116} := 0.520 \text{ in} \quad N_{W30x116} := 12.0 \text{ in} \quad (\text{See pg.7, Assumption 3.1.2})$$

$$f_{vW30x116} := \frac{R_{AB}}{d_{W30x116} \cdot t_{wW30x116}} \quad f_{vW30x116} = 3.80 \text{ ksi} \quad \text{OK}$$

Check W30X116 for extreme loads of $1.6 > LL + DL + E$ Ref. 2.2.1 Sect. 4.2.11.4.6(9).

The peak ground vertical acceleration is $SA = 2.0 \times 0.67g$ for the 2000-year seismic event, at 5% damping, and occurs at a period of 0.10 sec. at El. 32' (See Table in Sect.6.2 pg.12, and Assumption 3.1.1)

$$SA := 0.67 \cdot 2.0 \quad SA = 1.34$$

$$F_{be} = 52.80 \text{ ksi} \quad F_{ve} = 28.00 \text{ ksi}$$

Use 25% LL during earthquake. (Ref. 2.2.3 Sect.10.3.1)

$$W_{eAB} := 7.0 \text{ ft} \cdot [DL_{\text{floor}} + 0.25LL_{\text{floor}} + SA \cdot (DL_{\text{floor}} + LL_{\text{efloor}})] \quad W_{eAB} = 6.70 \text{ klf}$$

$$M_{eAB} := \frac{W_{eAB} \cdot L_{\text{spanAB}}^2}{8} \quad M_{eAB} = 1025 \text{ ft} \cdot \text{kip}$$

$$R_{eAB} := \frac{W_{eAB} \cdot L_{\text{spanAB}}}{2} \quad R_{eAB} = 117.2 \text{ kip}$$

$$S_{\text{ereqAB}} := \frac{M_{eAB}}{F_{be}} \quad S_{\text{ereqAB}} = 233.0 \text{ in}^3 < 329.0 \text{ in}^3 \quad \text{O'K}$$

$$\text{Demand/Capacity Ratio DC} \quad DC := \frac{S_{\text{ereqAB}}}{S_{W30x116}} \quad DC = 0.71$$

Web Crippling Check (Ref.2.2.9, Formula Q1.10-10)

$$R_{ww} := 34t_{wW30x116}^2 \cdot \left[1 + 3 \cdot \left(\frac{N_{W30x116}}{d_{W30x116}} \right) \cdot \left(\frac{t_{wW30x116}}{t_{fW30x116}} \right)^{1.5} \right] \cdot \left[\sqrt{F_{y50} \cdot 1.6 \cdot \left(\frac{t_{fW30x116}}{t_{wW30x116}} \right)} \right] \cdot \frac{\sqrt{\text{kip}}}{\text{in}}$$

$$R = 165.48 \text{ kip} > 126.4 \text{ kip} \quad \text{O'K}$$

No Stiffeners Required

**Deflection Check
(DL only)**

$$E_{ww} := 29000 \cdot \text{ksi}$$

$$W_{ABd} := 7.0 \text{ ft} \cdot (\text{DL}_{\text{floor}}) \quad W_{ABd} = 2.69 \text{ klf}$$

$$d_{\text{max}} := 5 \cdot W_{ABd} \cdot \frac{L_{\text{spanAB}}^4}{384E \cdot I_{W30x116}} \quad d_{\text{max}} = 0.63 \text{ in}$$

$$d_{\text{allow}} := \frac{35.0 \cdot 12 \cdot \text{in}}{240} \quad d_{\text{allow}} = 1.75 \text{ in} > d_{\text{max}} \quad \text{O'K (Ref. 2.2.18 Table 1604.3)}$$

DEFLECTION CHECK :

Based on N690 criteria (Ref. 2.2.9 Sect.CQ1.13)

$$F_{ww} := 50 \quad \frac{F_y}{800} = 0.0625$$

Beam depth Required:

$$1/20 \times 35 \text{ ft} \times 12" = 21" \quad 26.25" < W30X116 \quad \text{O'K}$$

$$d = 0.0625 \times 35 \text{ ft} \times 12" = 26.25" < W30X116 \quad \text{O'K}$$

Use W30X116 for Floor Beams with 35.0 ft. span between Col.Line A/B, A/C, E/F and Col.Line 3/4 & 6/7 @ EL. 32'

Size Steel Beams Supporting Floors Between Col.Line A/C, E/F and Col.Line 4/6 @ EL. 32'

Clear spans between walls are 55.0' Consider beam having A & B supports on C/Lines 4 & 6

$$L_{\text{spanAB}} := 55.0 \cdot \text{ft}$$

Maximum tributary width of slab is 7.0', as shown on the preliminary framing layout.

$$w_{\text{AB}} := 7.0 \text{ft} \cdot (DL_{\text{floor}} + LL_{\text{floor}}) \quad w_{\text{AB}} = 3.39 \text{ klf}$$

Use simple span: (Assumptions 3.2.1, 3.2.2, 3.2.3)

$$M_{\text{AB}} := \frac{w_{\text{AB}} \cdot L_{\text{spanAB}}^2}{8} \quad M_{\text{AB}} = 1280 \text{ ft} \cdot \text{kip}$$

$$R_{\text{AB}} := \frac{w_{\text{AB}} \cdot L_{\text{spanAB}}}{2} \quad R_{\text{AB}} = 93.1 \text{ kip}$$

$$S_{\text{reqAB}} := \frac{M_{\text{AB}}}{F_b} \quad S_{\text{reqAB}} = 465.6 \text{ in}^3$$

W44X230 has the following properties:(Ref. 2.2.20 page 1-10 and 1-11)

$$S_{\text{W44x230}} := 971.0 \text{ in}^3 \quad I_{\text{W44x230}} := 20800 \text{ in}^4 \quad t_{\text{fW44x230}} := 1.220 \text{ in}$$

$$d_{\text{W44x230}} := 42.9 \text{ in} \quad t_{\text{wW44x230}} := 0.710 \text{ in} \quad N_{\text{W44x230}} := 15.0 \text{ in} \quad (\text{See pg.7, Assumption 3.1.2})$$

$$f_{\text{vW44x230}} := \frac{R_{\text{AB}}}{d_{\text{W44x230}} \cdot t_{\text{wW44x230}}} \quad f_{\text{vW44x230}} = 3.06 \text{ ksi} \quad \text{OK}$$

Check W44X230 for extreme loads of $1.6S > LL + DL + E$ Ref. 2.2.1 Sect. 4.2.11.4.6(9).

The peak ground vertical acceleration is $SA = 2.0 \times 0.67g$ for the 2000-year seismic event, at 5% damping, and occurs at a period of 0.10 sec. at El. 32' (See Table in Section 6.2, pg 12 and Assumption 3.1.1)

$$SA := 0.67 \cdot 2.0 \quad SA = 1.34$$

$$F_{be} = 52.80 \text{ ksi} \quad F_{ve} = 28.00 \text{ ksi}$$

Use 25% LL during earthquake. (Ref. 2.2.3 Sect.10.3.1)

$$W_{eAB} := 7.0 \text{ ft} \cdot [DL_{\text{floor}} + 0.25LL_{\text{floor}} + SA \cdot (DL_{\text{floor}} + LL_{\text{efloor}})] \quad W_{eAB} = 6.70 \text{ klf}$$

$$M_{eAB} := \frac{W_{eAB} \cdot L_{\text{spanAB}}^2}{8} \quad M_{eAB} = 2532 \text{ ft} \cdot \text{kip}$$

$$R_{eAB} := \frac{W_{eAB} \cdot L_{\text{spanAB}}}{2} \quad R_{eAB} = 184.1 \text{ kip}$$

$$S_{\text{reqAB}} := \frac{M_{eAB}}{F_{be}} \quad S_{\text{reqAB}} = 575.4 \text{ in}^3 < 971.0 \text{ in}^3 \quad \text{O'K}$$

$$\text{Demand/Capacity Ratio DC} \quad DC := \frac{S_{\text{reqAB}}}{S_{W44x230}} \quad DC = 0.59$$

Note: Allowable Section Size Controlled by Deflections, See next page.

Web Crippling Check (Ref.2.2.9, Formula Q1.10-10)

$$R := 34t_{wW44x230}^2 \left[1 + 3 \cdot \left(\frac{N_{W44x230}}{d_{W44x230}} \right) \cdot \left(\frac{t_{wW44x230}}{t_{fW44x230}} \right)^{1.5} \right] \left[\sqrt{F_{y50} \cdot 1.6 \cdot \left(\frac{t_{fW44x230}}{t_{wW44x230}} \right)} \right] \cdot \frac{\sqrt{\text{kip}}}{\text{in}}$$

$$R = 294.53 \text{ kip} > 184.1 \text{ kip} \quad \text{O'K}$$

No Stiffeners Required

**Deflection Check
(DL only)**

$$E := 29000 \cdot \text{ksi}$$

$$W_{ABd} := 7.0 \text{ ft} \cdot (\text{DL}_{\text{floor}})$$

$$W_{ABd} = 2.69 \text{ klf}$$

$$d_{\text{max}} := 5 \cdot W_{ABd} \cdot \frac{L_{\text{spanAB}}^4}{384E \cdot I_{W44x230}} \quad d_{\text{max}} = 0.92 \text{ in}$$

$$d_{\text{allow}} := \frac{55.0 \cdot 12 \cdot \text{in}}{240} \quad d_{\text{allow}} = 2.75 \text{ in} > d_{\text{max}} \quad \text{O'K} \quad (\text{Ref. 2.2.18 Table 1604.3})$$

DEFLECTION CHECK :

Based on N690 criteria (Ref. 2.2.9 Sect. CQ1.13)

$$F_w := 50 \quad \frac{F_y}{800} = 0.0625$$

Beam depth Required:

$$1/20 \times 55 \text{ ft} \times 12" = 33" < W44X230$$

$$d = 0.0625 \times 55 \text{ ft} \times 12" = 41.25" < W44X230 \quad \text{O'K}$$

Use W44X230 for Floor Beams with 55.0 ft. span between Col.Line A/C, E/F and Col.Line 4/6 @ EL. 32'

Size Steel Beams Supporting Floors Between Col.Line A/C, E/F and Col.Line 7/8, @ EL. 32'

Clear spans between walls are 48.33' Consider beam having A & B supports

$$L_{\text{spanAB}} := 48.33 \cdot \text{ft}$$

Maximum tributary width of slab is 7.0', as shown on the preliminary framing layout.

$$w_{\text{AB}} := 7.0 \text{ft} \cdot (\text{DL}_{\text{floor}} + \text{LL}_{\text{floor}}) \quad w_{\text{AB}} = 3.39 \text{ klf}$$

Use simple span: (Assumptions 3.2.1, 3.2.2, 3.2.3)

$$M_{\text{AB}} := \frac{w_{\text{AB}} \cdot L_{\text{spanAB}}^2}{8} \quad M_{\text{AB}} = 1280 \text{ ft} \cdot \text{kip}$$

$$R_{\text{AB}} := \frac{w_{\text{AB}} \cdot L_{\text{spanAB}}}{2} \quad R_{\text{AB}} = 81.8 \text{ kip}$$

$$S_{\text{reqAB}} := \frac{M_{\text{AB}}}{F_b} \quad S_{\text{reqAB}} = 359.5 \text{ in}^3$$

W36X160 has the following properties:

$$S_{\text{W36x160}} := 542 \text{ in}^3 \quad I_{\text{W36x160}} := 9750 \text{ in}^4 \quad t_{\text{fW36x160}} := 1.020 \text{ in}$$

$$d_{\text{W36x160}} := 36.01 \text{ in} \quad t_{\text{wW36x160}} := 0.650 \text{ in} \quad N_{\text{W36x160}} := 12.0 \text{ in} \quad (\text{See pg.7, Assumption 3.1.2})$$

$$f_{\text{wW36x160}} := \frac{R_{\text{AB}}}{d_{\text{W36x160}} \cdot t_{\text{wW36x160}}} \quad f_{\text{wW36x160}} = 3.50 \text{ ksi} \quad \text{OK}$$

Check W36X160 for extreme loads of $1.6S > LL + DL + E$ Ref. 2.2.1 Sect. 4.2.11.4.6(9)

The peak ground vertical acceleration is $SA = 2.0 \times 0.67g$ for the 2000-year seismic event, at 5% damping, and occurs at a period of 0.10 sec. at El. 32' (See Table in Sect. 6.2 pg.12 and Assumption 3.1.1)

$$SA := 0.67 \cdot 2.0 \quad SA = 1.34$$

$$F_{be} = 52.80 \text{ ksi} \quad F_{ve} = 28.00 \text{ ksi}$$

Use 25% LL during earthquake. (Ref.2.2.3 Sect 10.3.1)

$$w_{eAB} := 7.0 \text{ ft} \cdot [DL_{\text{floor}} + 0.25LL_{\text{floor}} + SA \cdot (DL_{\text{floor}} + LL_{\text{efloor}})] \quad w_{eAB} = 6.70 \text{ klf}$$

$$M_{eAB} := \frac{w_{eAB} \cdot L_{\text{spanAB}}^2}{8} \quad M_{eAB} = 1955 \text{ ft} \cdot \text{kip}$$

$$R_{eAB} := \frac{w_{eAB} \cdot L_{\text{spanAB}}}{2} \quad R_{eAB} = 161.8 \text{ kip}$$

$$S_{\text{reqAB}} := \frac{M_{eAB}}{F_{be}} \quad S_{\text{reqAB}} = 444.3 \text{ in}^3 < 542.0 \text{ in}^3 \quad \text{O'K}$$

Demand/Capacity Ratio DC $DC := \frac{S_{\text{reqAB}}}{S_{W36x160}} \quad DC = 0.82$

Web Crippling Check (Ref.2.2.9, Formula Q1.10-10)

$$R := 34t_{wW36x160}^2 \cdot \left[1 + 3 \cdot \left(\frac{N_{W36x160}}{d_{W36x160}} \right) \cdot \left(\frac{t_{wW36x160}}{t_{fW36x160}} \right)^{1.5} \right] \cdot \left[\sqrt{F_{y50} \cdot 1.6 \cdot \left(\frac{t_{fW36x160}}{t_{wW36x160}} \right)} \right] \cdot \frac{\sqrt{\text{kip}}}{\text{in}}$$

$$R = 242.81 \text{ kip} > 161.8 \text{ kip} \quad \text{O'K}$$

No Stiffeners Required.

**Deflection Check
(DL only)**

$$E := 29000 \cdot \text{ksi}$$

$$W_{ABd} := 7.0 \text{ ft} \cdot (\text{DL}_{\text{floor}}) \quad W_{ABd} = 2.69 \text{ klf}$$

$$d_{\text{max}} := 5 \cdot W_{ABd} \cdot \frac{L_{\text{spanAB}}^4}{384 E \cdot I_{W36x160}} \quad d_{\text{max}} = 1.17 \text{ in}$$

$$d_{\text{allow}} := \frac{48.33 \cdot 12 \cdot \text{in}}{240} \quad d_{\text{allow}} = 2.42 \text{ in} > d_{\text{max}} \quad \text{O'K} \quad (\text{Ref. 2.2.18 Table 1604.3})$$

DEFLECTION CHECK :

Based on N690 criteria (Ref. 2.2.9 Sect.CQ1.13)

$$F_w := 50 \quad \frac{F_y}{800} = 0.0625$$

Beam depth Required:

$$1/20 \times 48.33 \text{ ft} \times 12" = 29" < W36 \times 160 \quad \text{O'K}$$

$$d = 0.0625 \times 48.33 \text{ ft} \times 12" = 36.25" , d = 36.01 (W36 \times 160), \text{ say O'K}$$

Use W36X160 for Floor Beams with 48.33 ft. span between Col.Line A/C, E/F and Col.Line 7/8 @ EL. 32'

SUMMARY:

BEAM SIZES AT FLOOR EL 32'

W30X116	C/Line A/B, A/C, B/F and 3/4, 6/7	pg. 43
W36X150	C/Line C/E, E/F and 2/3, 3/4, 8/9	pg. 40
W44X230	C/Line A/C, E/F and 4/6	pg. 46
W36X160	C/Line A/C, E/F and 7/8	pg. 49

7.0 RESULTS AND CONCLUSIONS

7.1 RESULTS

The calculation results document the primary steel beam designs for supporting structural steel members for the roofs and floors of the Receipt Facility (RF) building. The roofs and floors structural members are adequate to withstand the dead, live and amplified slab acceleration loads during seismic events.

The results indicate that the depth of the steel beams are such that they will not encroach on the required clear space for the cranes to be used in the RF, as discussed in Section 6.5. The deepest member is 43.3in or about 3'-7" for a W44X262 member. Using the lowest available clear space of 7'-0" to the bottom of the slab, as identified in section 6.5, gives a remaining clear space of 3'-0", which is acceptable.

The steel framing member sections Demand/Capacity DC Ratios are tabulated below.

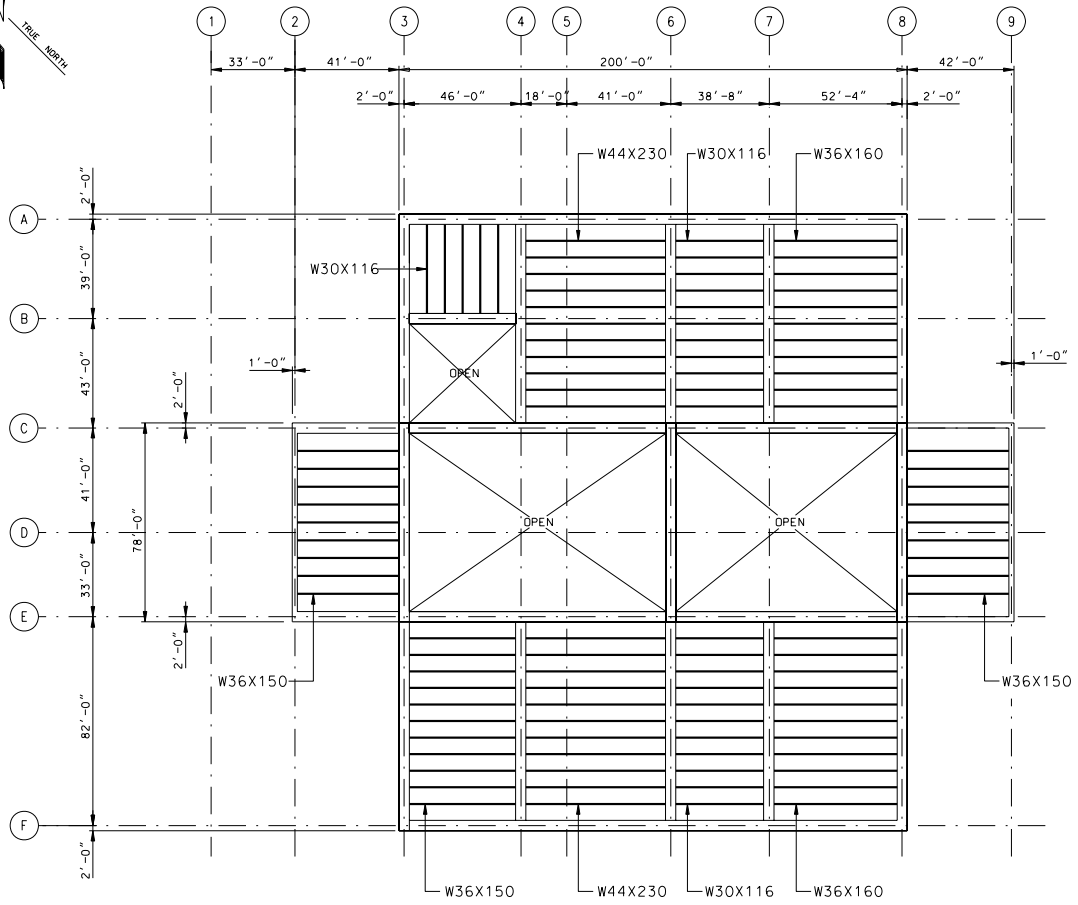
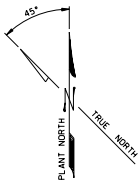
Steel Framing Sections and Demand:Capacity Ratios				
Slab Elevation	Area Roof/Floor	Beams		Page No.
		Section	DC Ratio	
100' R	C-E-2-8	W44x262	0.81	21
72' R	C-E-2-8	W44x262	0.81	21
64' R	A-C-E-F-6-7, A-B-3-4	W30x108	0.71	24
64' R	B-C-C-E-E-F-3-4-2-3	W36x135	0.70	27
64' R	A-C-A-F-7-8	W36x170	0.70	30
64' R	A-C-E-F-4-6	W36x210	0.73	33
32' F	C-E-E-F-2-3-3-4	W33x150	0.67	39
32' F	A-B-A-C-E-F-3-4-6-7	W30x116	0.71	42
32' F	A-C-E-F-4-6	W44x230	0.59	45
32' F	A-C-E-F-7-8	W36x160	0.82	48

7.2 CONCLUSIONS

The results of the calculation are reasonable and adequate for use in the structural design calculations and drawings being developed as part of the Tier 1 design for the (RF) building.

ATTACHMENT A
STRUCTURAL SUPPORT STEEL LAYOUT

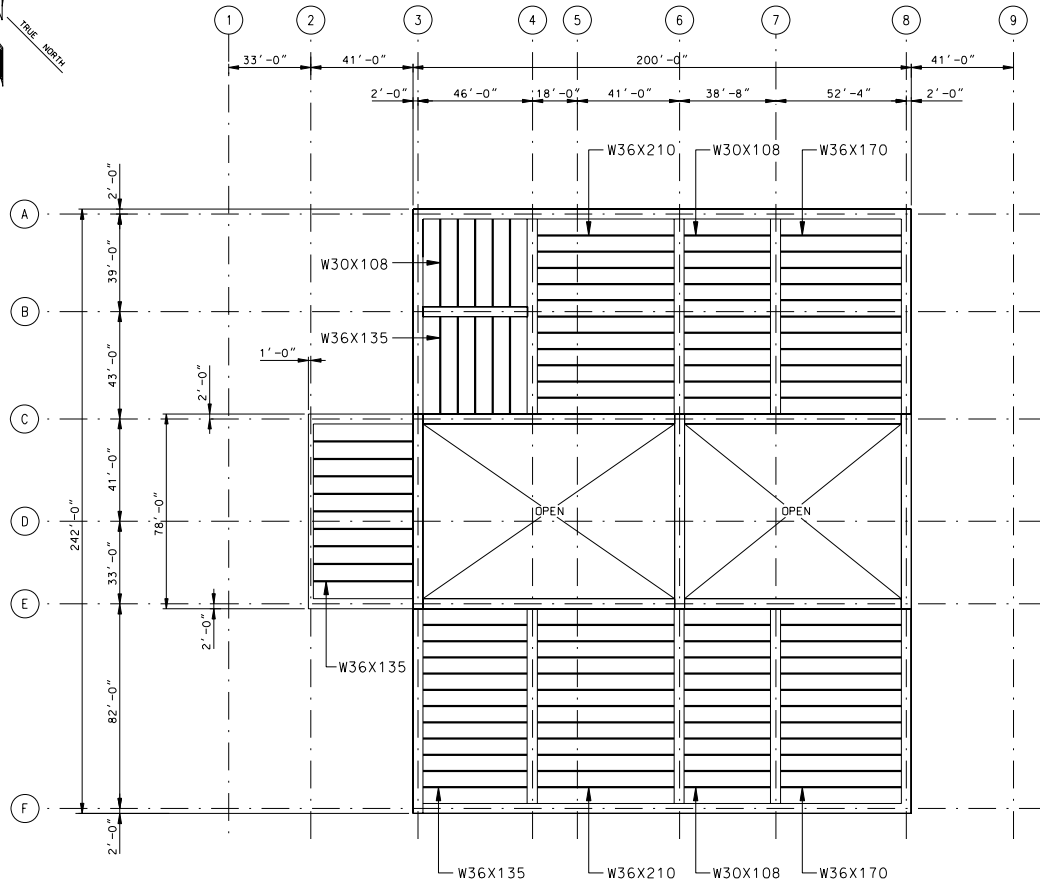
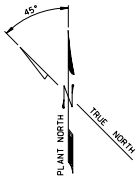
ATTACHMENT A



NOTE:
ALL STEEL FRAMING ARE
SPACED AT 7'-0" OC (MAX)

FLOOR STEEL FRAMING PLAN AT EL 32'-0"
TOS EL (-) 1'-9"

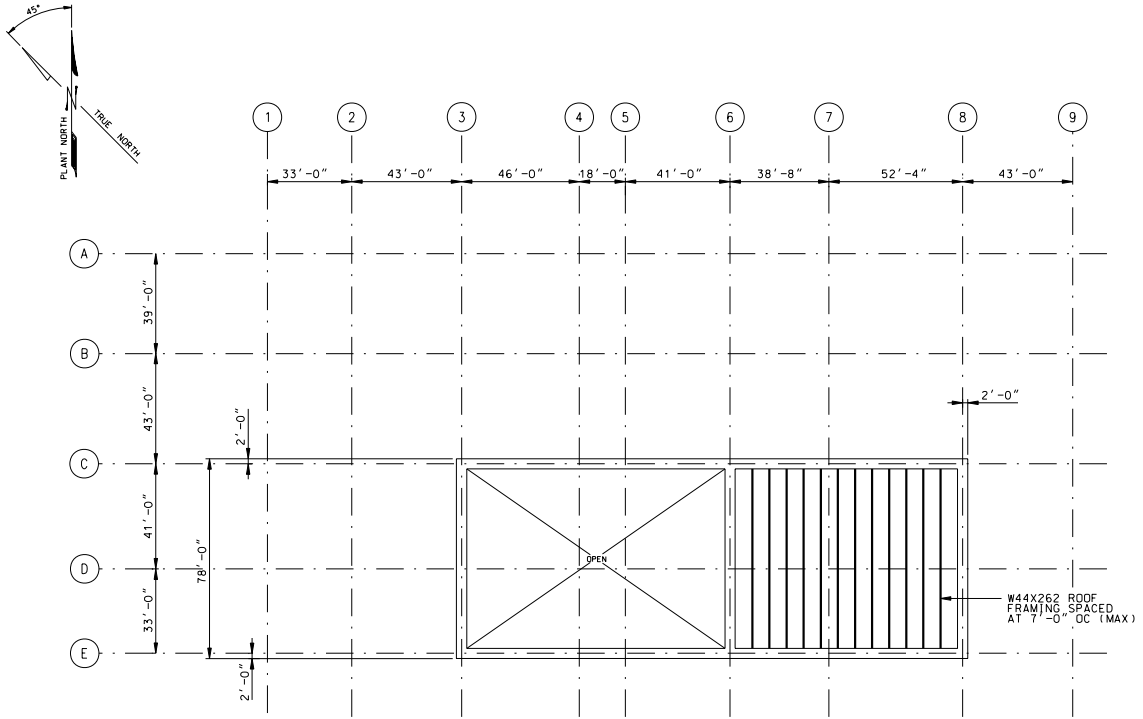
ATTACHMENT A



NOTE:
ALL STEEL FRAMING ARE
SPACED AT 7'-0" OC (MAX)

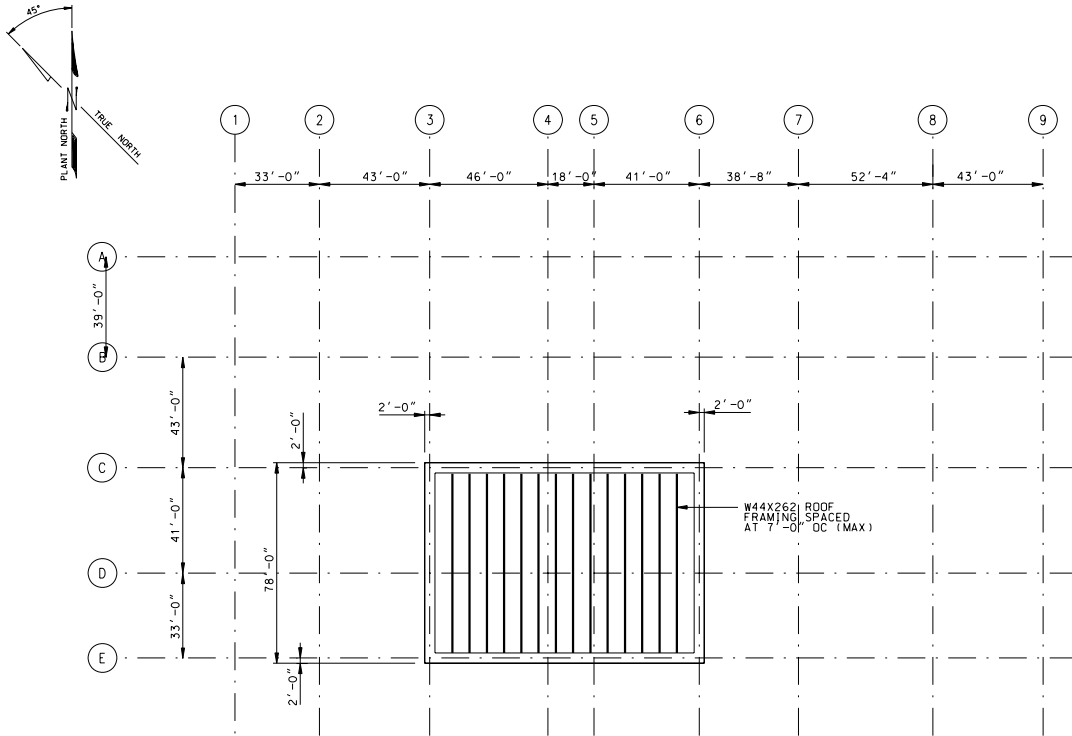
FLOOR STEEL FRAMING PLAN AT EL 64'-0"
TOS EL (-) 1'-9"

ATTACHMENT A



ROOF STEEL FRAMING PLAN AT EL 72'-0"
TOS EL (-) 1'-9"

ATTACHMENT A



ROOF STEEL FRAMING PLAN AT EL 100'-0"
TOS EL (-) 1'-9"