UNION STATION EXPANSION AND RESTORATION

WASHINGTON DC

FINAL THESIS REPORT: SIGNATURE EXPRESSION

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Spring 2009

Structural Option

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UNION STATION EXPANSION & RESTORATION

WASHINGTON DC

GENERAL BUILDING DATA:

LOCATION: BLOCK 720 ALONG H STREET
WASHINGTON DC

OCCUPANCY: MIXED USE

SIZE: 329,000 SQUARE FEET

HEIGHT: 5 STORIES ABOVE GRADE WITH A MAX HEIGHT OF 88 FEET AND 2 INCHES

COST: $23 MILLION

DELIVERY METHOD: DESIGN - BID - BUILD

PROJECT TEAM:

OWNER: UNION STATION REDEVELOPMENT CORPORATION

PRIME ARCHITECT: TIMOTHY HAAHS & ASSOCIATES

ASSOCIATE ARCHITECT: RTKL ASSOCIATES INC.

STRUCTURAL ENGINEER: TIMOTHY HAAHS & ASSOCIATES

CIVIL ENGINEER: SCHNABEL ENGINEERING

MEP ENGINEER: RTKL ASSOCIATES INC.

GENERAL CONTRACTOR: CLARK CONSTRUCTION

ARCHITECTURE:

- MIX USE WHICH INCLUDES AMTRAK STATION, MARC, WASHINGTON DC'S METRO, OFFICE SPACE, AND PARKING
- EXTERIOR FAÇADE ALONG WEST ELEVATION IS UNIFIED WITH PRECAST PANELS AND MIRRORS THE TRACKS TRAVELING THROUGH THE BUILDING
- NORTHWEST CORNER OF EXPANSION HAS A "CURTAIN" PERFORATED STAINLESS STEEL PANELS
- ROOF IS A 7" THICK P/T SLAB DUE TO PARKING AND RELIES ON 8 DRAINS TO PREVENT PONDING

STRUCTURAL SYSTEM:

- TWO WAY POST TENSION CAST-IN-PLACE FLOOR SYSTEM SUPPORTED BY 20 COLUMNS ON EACH FLOOR
- SEISMIC-FORCE-RESISTING FRAME SYSTEM IS ORDINARY REINFORCED CONCRETE MOMENT FRAMES
- CONCRETE PILES & COLUMNS REST ON SPREAD FOOTERS THAT SUPPORT THE STRUCUTRE FROM THE TRAIN TRACKS THAT TRAVEL BELOW

M.E.P. SYSTEMS:

- 1600 CFM AIR HADLING UNITS TO BE INSTALLED IN OFFICE SPACES, SECURITY AREA, AND MECHANICAL/ELECTRICAL ROOM
- 10 DIFFERENT LIGHTING FIXTURES USED THROUGHOUT EXPANSION
- 3 PHASE, 4 WIRE, 480 V NEW GENERATOR LOCATED ON GROUND FLOOR WITH POWER SUPPLIED FROM H STREET
- OFFICE SPACES, MEZZAINE LEVEL, AND ALL STAIR TOWERS HAVE SPRINKLER SYSTEMS INSTALLED WHILE PARKING AREAS WILL NOT BE SPRINKLED

ARCHITECTURAL ENGINEERING

JOSEPH W. WILCHER III
STRUCTURAL OPTION

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Dr. Memari

The entire AE faculty and staff

A special thanks to all friends, colleagues, peers, and family of the author.

The following thesis is dedicated to the author's parents, Joseph W. & Carolyn N. Wilcher. Thank you for the guidance and life lessons you have instilled upon me for over the last twenty-two plus years.

EXECUTIVE SUMMARY

In this final thesis report, the thesis proposal of creating a signature expression for the expansion to Union Station was carried out in full detailed. Using the design of the king post truss as the concept of the signature expression (located on the ground floor of the expansion), a full structural depth with an architectural and lighting study was accomplished where all three areas of architectural engineering focused on the trusses.

Changing the floor system from post-tension to composite steel was the first step in the structural depth portion of the thesis (starting on page 11). From there, multiple designs for the trusses were created by the author in order to determine a design that not only would be an expression in architectural, but as well in structural engineering (All of the design based on the look of the trusses can be found in the architectural breadth). Using standard truss analysis with the addition to using curved tension member as the brace members gave a unique way of looking at a truss in structural engineering. Two of the nine trusses are the focus within the body of the structural depth to show the process the author took in doing the structural calculations for each one.

Brace frames replaced the existing ordinary concrete moment frames as the new lateral system for the expansion to Union Station which are part of three of the nine trusses (refer to page 23). Each one of the trusses pin connections were analyzed as well as a heavy brace connection on one of the trusses. This was designated as the M.A.E. criteria for the thesis. Finally spot checks on the foundation were done to verify the trusses transferred the load from the upper floors down to the track level and then into the ground without any concerns in changing the existing foundation system.

As mentioned above, the design process of making the trusses look one of a kind is found in the architectural breadth portion of the thesis. On top of the design of the trusses, the author also looked at the vehicular circulation the busses need in order to maneuver and park under the trusses. Also, the waiting terminal on the ground floor was moved from its original location to help express the trusses in the expansion.

Within the lighting breadth of the thesis, LEDs were selected to highlight the trusses and full calculations for the Lumen Method were done in order to determine the amount of luminaries needed for one of the waiting terminals.

After each section of the report, a conclusion has been written to talk if the criteria goals for each section were meet (refer to page 10) and if not, the author talks about what could have been different in the process taken. All calculations for each of the breadths as well as the depth can be found in Appendixes A through M at the end of the report.

UNION STATION HISTORY

Located in the heart of Washington DC's commercial and manufacturing district, Union Station was built in the mid 1980's as one of first major public transportation buildings in the United States. One can purchase an Amtrak ticket to travel nationally on the railroad or ride a greyhound bus to travel to the desired location. The DC metro also travels through the station allowing local pedestrians and tourists to travel around our nation's capital.

Since the completion of Union Station, other renditions of the building have been designed and built in other major cities throughout the United States (Dallas, St. Louis, Los Angles, etc.). Each building was given a unique style of architecture to highlight the building in the city it resides in. For the original building of Union Station, a grand Glass Curtain wall is located along the west elevation of the building. This architectural feature allows guest and workers within the building to look at the sites of Washington DC while either riding up in the elevators or the escalators, taking the stairs, working in the office spaces, or sitting in the lobbies waiting to travel by means of one of the transportations offered.

Figure i: Union Station's Glass Curtain Wall

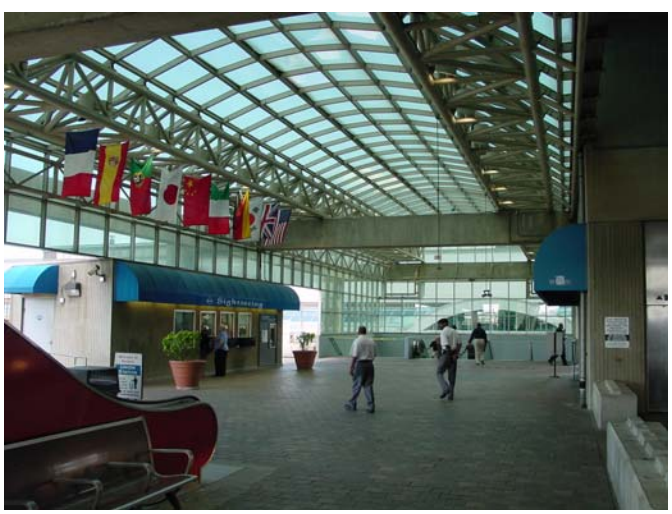


Figure ii: View Within Curtain Wall

EXISTING STRUCTURAL SYSTEM

Foundation:

Union Station's expansion main foundation system consists of concrete piles, which carry the load from the train track stations to the soil and supportive columns for all the levels above the track level. Each one rests upon a square footer that is either six feet or twelve feet in length and width, with a height of two feet.

All the piles are located between the eight locomotive rail ways that are part of Union Station. Maximum diameter size of the columns and the piles are 1 ½' and are spaced 22'-0" spanning in the north-south direction of the building between the railroads.

From the provided geotechnical report, the net soil bearing capacity for the site is 2000 PSF, which is considered weak for the soil. Fine to coarse sandy clay fill is the soil designation on the site for Union Station.

Existing Floor System:

Union Station's typical floor system is a two-way post-tension cast-in-place concrete slab with a thickness of 7". All the beams and girders are post-tension cast-in-place as well. In Union Station, the beams span a length of 63'-0". The girders located in the expansion, carry the load from the beams to the columns and have a typical span of 24'-4" throughout the expansion. The concrete compressive strength for the slabs, beams, and girders is f'_c = 5000 psi while the columns supporting the floors are cast-in-place with a compressive strength of 8000 psi. It is to be noted that the floor systems for the expansion and the existing structure for Union Station do not connect with each other.

For the Ground Level, a rigid 6 ½" concrete slab was used for majority of the floor. A composite design located along the west elevation was utilized to help reduce the weight within the weakest are of the site. A 5" light weight concrete slab over 1 ½" gage LOK-Floor was used which makes the ground floor total thickness to be 6 ½". Shear studs sized at ¾" x 4 ½" were used in the composite floor design. Typical member size for the beams is W27x84 which span 63'-0" and tie into a W33x118 girder. Each girder ties into the concrete columns that are part of the foundation system.

There are two typical bay sizes located in the expansion of Union Station, 63'-0" x 27'-6" and 63'-0" x 40'-0". Since the tracks running through Union Station were the major consideration in the design as well as the bus terminal, the use of long spans was concluded as the best approach for the design.

Lateral System:

Union Station's lateral load system is composed of ordinary reinforced concrete moment frames (See Figure 1 to the right). Lateral loads, as well as the gravity loads, reach the foundation of Union Station by first traveling through the beams, then carry through the girders which connect to the columns. From there, all loads travel down in the columns to the ground level and then the columns take all the loads into the foundation. Not all beams and girders take part of the lateral system in Union Station. The highlighted members within Figure 1 represent the beams and girders that act as part of the lateral system. Intermediate beams and girders are indicated as the black and white members within the figure.

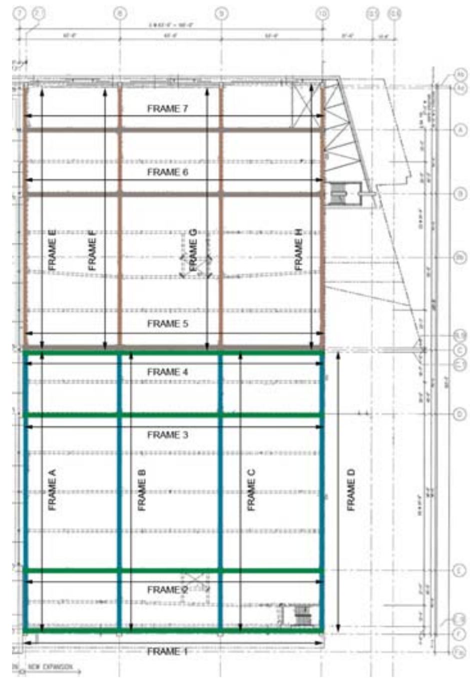


Figure 1: Moment Frames in Union Station Expansion

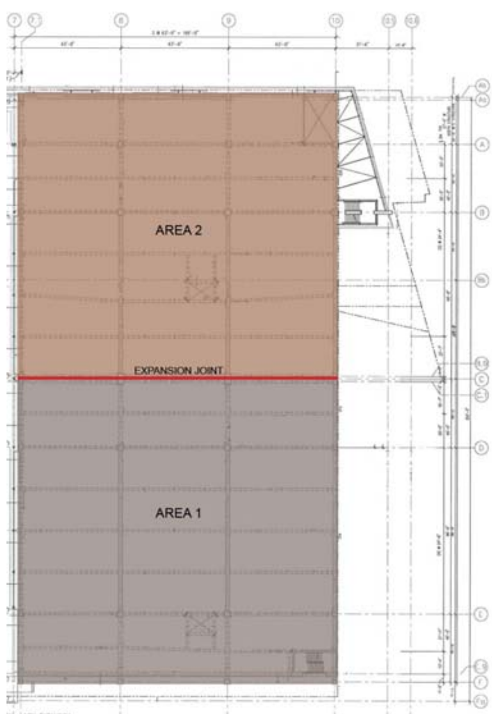


Figure 2: Expansion Joint within Expansion

An expansion joint was placed between column lines 7 and 7-1 is located between the existing structure and the expansion to Union Station (Refer to Figure Figure 2). As stated in the addendum, there is also an expansion joint within the expansion. This joint is used to create two separate structures that can move independent of each other due to forces acting upon the building.

PROBLEM STATEMENT

From Design Firm's View:

From the very start of the design for the expansion to Union Station, two major concerns for the building were used as a starting point. First, there had to be large open spaces with a minimum amount of columns for the track, ground, and mezzanine level. This is due to having a bus terminal located on the ground floor since and the owner wanted an open feeling for the mezzanine level. Second, the weight of the building should be at a minimum since the soil located on the site is considered poor. These two considerations lead to the use of the post-tension floor system and above average column sizes throughout the entire building.

From Author's View:

While the author agrees with the concerns the design firm came up with for the expansion to Union Station, another issue should have been addressed as well. While trying to create a building expansion that was cost-savings and fit within the two major concerns, there was no major attempt to create a signature expression for the expansion to Union Station. The author believes that even though the glass curtain wall of the existing structure stands out as an expression of architecture, the expansion to Union Station should have its own architecture feature since it is own building as well.

THESIS CRITERIA GOALS

Using the areas of concern from the design firm as well as the author's own point of view, the following criteria was established in order to complete the over goals of this thesis.

1.) Redesign floors mezzanine through third with a new structural system.
2.) Design a one of a kind transfer level that is located on the ground floor while incorporating the style of the king post truss.
    a. While the trusses act as the transfer system, create an architectural expression with the trusses by using different shapes and connections that show the trusses were solely made of the expansion to Union Station.
    b. Ignore the cost of how much the custom trusses and new floor system will cost since the author believes how important it is to have an architectural expression.
3.) Incorporate brace frames as the new lateral system for the expansion to Union Station.
4.) Verify the foundation of Union Station can support the new structure.
5.) Determine the vehicular circulation of the buses will not be affected by the truss designs.
6.) Incorporate the waiting/lobby area on the ground floor with the architecture of the trusses.
7.) Incorporate two new lighting layouts:
    a. Create a custom lighting scheme that will now only illuminate the trusses but highlight them to looking aesthetically pleasing.
    b. Replace the existing luminaries within the bus terminal with new, energy efficient ones.

All seven goals will be attempted by the author in order to give the expansion to Union Station not only to meet the goals of the owner, but to make the people who work and step into the expansion remember the one of a kind structural and architectural feature.

STRUCTURAL DEPTH

Location of Trusses:

Before any redesigning of the upper levels was started, the first task at hand was to determine where the transfer trusses would be placed on the ground floor. Keeping in mind there will be buses traveling and parking on the ground level, the trusses had to be placed where there would be minimal impact. The author concluded the best location for the trusses would be where the existing columns are located on the ground floor. Figure 3 below indicates where the king post trusses would be located (blue lines indicate the trusses while the red line represents the expansion joint).

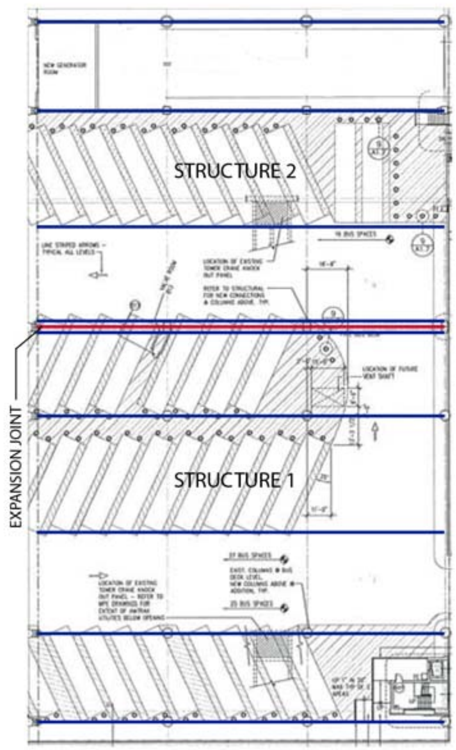


Figure 3: Location of Transfer Trusses on Ground Floor

Five trusses would be located within Structure 1 of the expansion to Union Station and four would be in Structure 2. Each truss would span the north-south length of the building which is 189'-0" and would be a height of 18'-0". Visual inspection of Figure 3 shows that some of the trusses will be located where buses must turn and park (Refer to the Architectural Breadth portion). Since the location of the king post trusses has been determined, the next step was to design a new structural floor system for the mezzanine level through the third floor.

Composite Steel Floor System [Preliminary Sizes]:

Since one of the major concerns for the expansion for Union Station was having large open floor plans on the ground and mezzanine level, the design team that created the building decided to keep the open plan on each level. On the upper floors however, a large open space is not necessarily required since there is only office space and parking. The author believes the use of a composite steel floor system is a valuable alternative structural gravity system to the post tension slab. A composite system not only can provide long spans, but also can reduce the slab thickness as well giving each level a higher floor to ceiling height.

Starting with the existing floor plans, a new column grid and beam layout for the gravity system was created. Figure 4 on the right shows a typical plan for levels one through three and the roof as well. In the north-south direction of the expansion, each column is spaced at 31'-6" while there are multiple spans in the east-west direction (49'-0" is the longest span for the east-west direction for both structures). Since the mezzanine level is shorter in length in the east-west direction, the only difference in the layout is the short span of 20'-0" located at the very top of structure two (Refer to Figure 5 for a visual representation). To view each typical floor with column markers, see Appendix A, Figures 1 & 2.

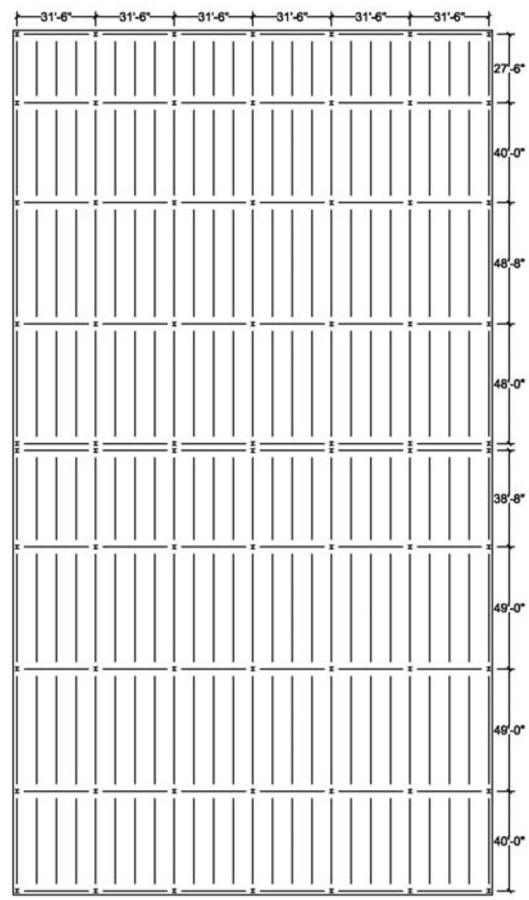


Figure 4: Composite System (Levels 1 -Roof)

The first step the author took in designing each beam and girder for the gravity system was determining the required loads for each floor per structure. Table 1 on page 13 of this report shows the dead and live loads used in accordance with ASCE 7-05. For this thesis project, no live load reductions were taken into account.
The author wanted to calculate the worst case scenario.

Each beam and girder for levels mezzanine to the roof was designed by hand using LRFD in conjunction with a calculation method learned in the advanced steel design course at The Pennsylvania State University. The bay size of 31'-6" x 49'-0" will be used as the example throughout this portion of this thesis as a guide to show the process of how the final members were selected.

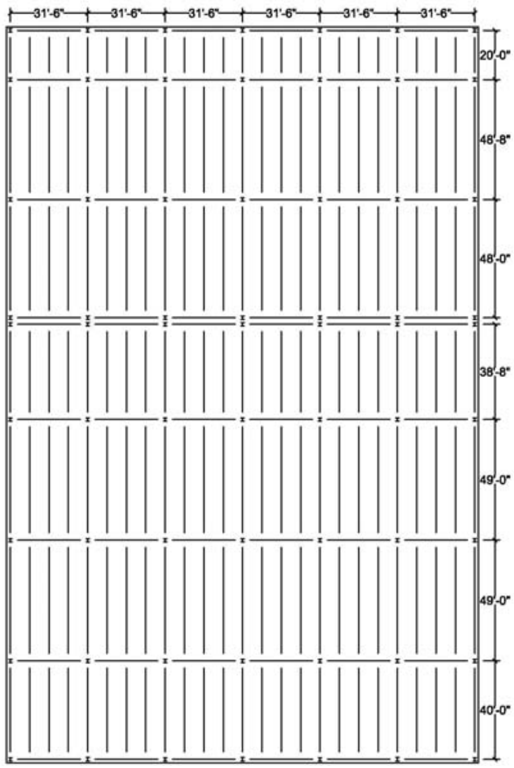


Figure 5: Composite System (Mezzanine Level)

DEAD LOADS						
Level	Roof	3	2	1	Mezzanine	Ground
Light Weight Concrete	110 pcf	110 pcf	110 pcf	110 pcf	110 pcf	110 pcf
Steel	490 pcf	490 pcf	490 pcf	490 pcf	490 pcf	-
M.E.P.	10 psf	10 psf	10 psf	10 psf	10 psf	10 psf
Finishes & Misc.	5 psf	5 psf	5 psf	5 psf	5 psf	5 psf

LIVE LOADS						
Level	Roof	3	2	1	Mezzanine	Ground
Landings	100 psf	100 psf	100 psf	100 psf	100 psf	100 psf
Lobbies	-	-	100 psf	100 psf	100 psf	100 psf
Mechanical	-	-	-	-	-	150 psf
Office	-	-	50 psf	50 psf	50 psf	-
Parking	50 psf	50 psf	50 psf	50 psf	-	-
Partition	-	-	10 psf	10 psf	10 psf	-
Stairs	100 psf	100 psf	100 psf	100 psf	100 psf	100 psf

Table 1: Gravity Loads from ASCE 7-05

Before the beams and girders were designed, a metal deck had to be selected for the composite steel floor system. Using the Vulcraft Steel Roof & Floor Deck catalog, a 2VLI16 metal deck with a 4.25" thick concrete slab was selected giving the total thickness to be 6 ¼" . Since original post-tension slab on the mezzanine to the roof was 7 ½", the floor thickness of the expansion of Union Station was increased by 1 ¼". Lightweight concrete was selected for the slab to help reduce the overall weight of the building and since the intermediate beams are spaced at 7'-10½" which is less than the maximum spacing of 12'-6" (To view the criteria designated to select the 2VLI16 metal deck can be found in Appendix B, Figure 1). The metal deck will span in the north-south direction of the expansion to Union Station, which is indicated by the arrow located on Figure 6.

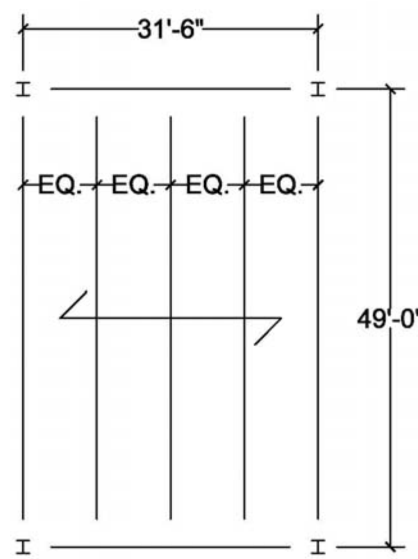


Figure 6: 31'-6" x 49'-0" Bay

Running in the 49'-0" direction of Figure 6 (See above) are the beams and the girders are the members that are 31'-6". The author selected members that would meet the construction, live load, and total deflection criteria set by the American Institute of Steel Construction (AISC). Using partial composite design, the number of shear studs required to transfer the loads from the concrete to the steel members was calculated as well by the requirements by AISC. Table 2 below shows the beam and girder member sizes calculated. To view the calculations for the beams and girders located in Table 2, see Appendix B, Calculations 1 through 10.

31'-6" x 49'-0" Interior Bay Beams: Strucutre 1 [Preliminary Calculations]					
Level	Roof	3rd	2nd	1st	Mezz.
Member	W24x55 <40>	W21x55 <24>	W21x55 <24>	W21x55 <24>	W21x55 <24>
31'-6" x 49'-0" Interior Bay Girders: Strucutre 1 [Preliminary Calculations]					
Level	Roof	3rd	2nd	1st	Mezz.
Member (G)	W24x94 <42>	W24x94 <42>	W24x94 <42>	W24x94 <42>	W24x94 <42>
Member (H)	W24x76 <46>	W24x76 <46>	W24x94 <42>	W24x94 <42>	W24x94 <42>

Table 2: Preliminary Typical Beam & Girder Sizes

Composite Steel Floor System [RAM]:

Using the same loads from Table 1, dimensions for bays in Figures 4 & 5 and now having the openings in the plans for the stairs and elevators, RAM Structural System was used to calculate the composite members to determine what sizes will be used. Figure 7 is the plan used for the roof, third, second, and first floor plan while Figure 8 is the plan for the mezzanine level. The location of the stairs and elevators are not the same as the existing expansion to Union Station. For more details on why the author moved some of their locations, refer to the Architecture Breadth portion of this thesis located on page 32.

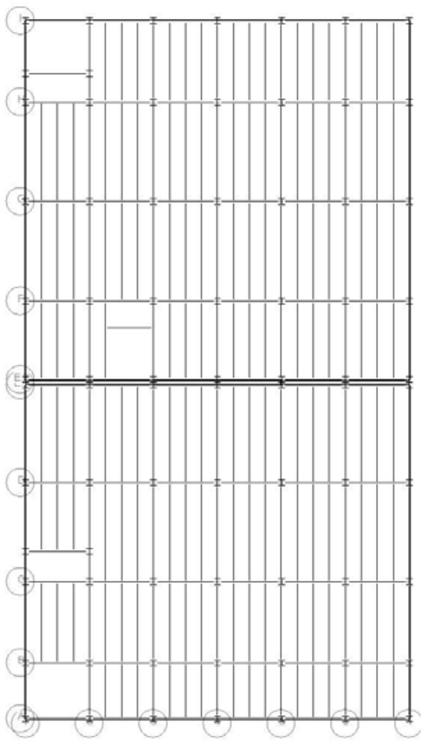


Figure 7: Roof, Third, Second, First Floor Plan

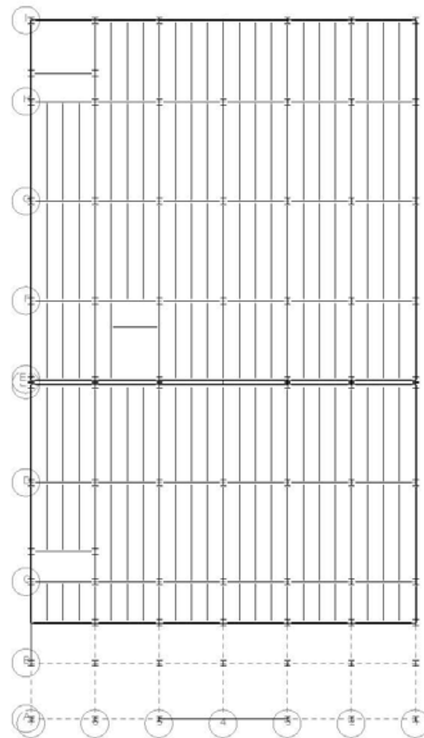


Figure 8: Mezzanine Floor Plan

After inputting all the loads, including a wall load of 35 psf, RAM Structural System was used to determine the member sizes and number of shear studs for both the composite beams and girders. Using the same interior as the one in the preliminary size section of the structural depth, Table 3 shows the sizes RAM determined were adequate for the expansion to Union Station.

31'-6" x 49'-0" Interior Bay Beams: Strucutre 1 [RAM]					
Level	Roof	3rd	2nd	1st	Mezz.
Member	W18x40 <53>	W18x40 <26>	W18x40 <40>	W18x40 <40>	W18x40 <40>
31'-6" x 49'-0" Interior Bay Girders: Strucutre 1 [RAM]					
Level	Roof	3rd	2nd	1st	Mezz.
Member (G)	W27x84 <48>	W27x84 <38>	W27x84 <38>	W27x84 <38>	W27x84 <38>
Member (H)	W24x76 <66>	W24x76 <34>	W24x76 <48>	W24x76 <48>	W24x76 <48>

Table 3: RAM Beam & Girder Sizes

Comparing the preliminary sizes to the ones calculated by RAM, one can see the beams used in RAM are smaller and lighter than the ones in the preliminary section. The reason for this is RAM used fully composite design instead of partial (as the author used in the hand calculations). Also, the RAM members have a larger camber than the members done by the author. The W18x40 beams have a camber of 2 ¼" while the W21x55 only have a ¼" camber. Since determining whether having a larger camber would cost more than a deeper and heavier beam was not part of this thesis, the author will use the beam sizes that were determined by RAM since they are smaller in depth and lighter in weight.

For the girders, the sizes are almost identical except the members in the G column line are deeper and heavier in RAM than the preliminary sizes. Since RAM could have another method of deterring the girders, the author will accept the values from RAM and use them as the final members for the expansion to Union Station.

Columns on Mezzanine through Third Floor:

After the beams and girders were designed in RAM, the columns that will transfer the gravity loads from each level had to be determined. RAM Structural System was used to calculate the member sizes for the columns. Looking at the same interior bay used as the example in this portion of this thesis (Figure 6 located on page 14), Table 4 on the following page shows the sizes of the columns used along grid lines G and H.

Column Line G				
Level	3rd	2nd	1st	Mezzanine
Member	W14x61	W14x82	W14x109	W14x132
Interaction	0.68	0.85	0.82	0.91
Column Line H				
Level	3rd	2nd	1st	Mezzanine
Member	W14x53	W14x74	W14x99	W14x176
Interaction	0.83	0.85	0.82	0.61

Table 4: Member Sizes along Grid Lines G & H

Location of Trusses [Additional Discussion]:

Once the composite steel gravity system was designed, the author went back to make sure where the original locations of the trusses were would line up with the proposed new column line. After investigating the floor plans, the trusses are directly below each column line of the composite steel system. Placing the trusses below the column makes the transfer system much more efficient. Figure 9 shows the trusses (blue hatching symbol) on top of the column line (black solid squares).

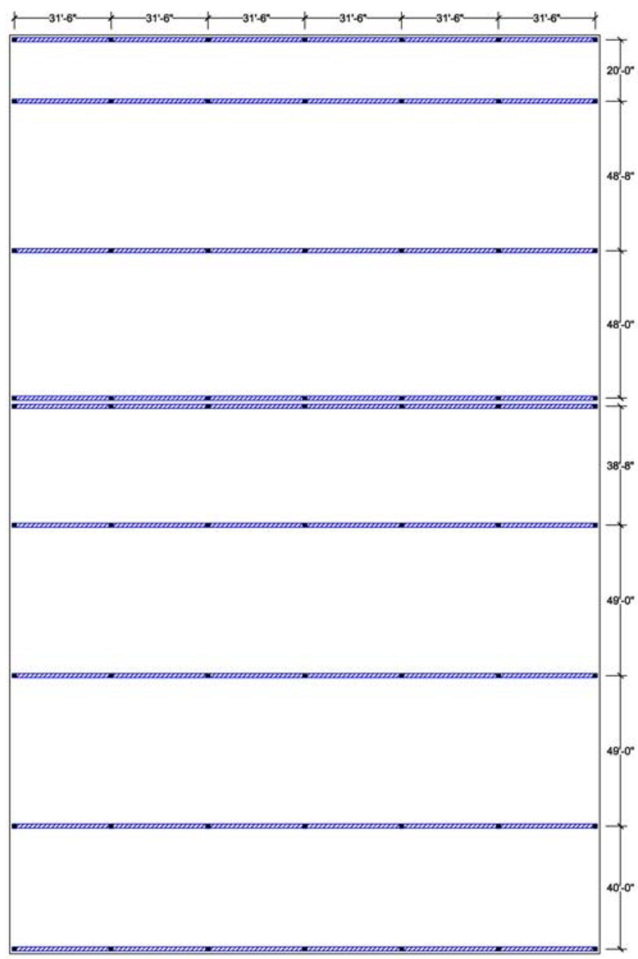


Figure 9: Column Line over Location of Trusses

Forces within Truss Members:

Before continuing on with the truss portion of this thesis, the author would like to make a statement to the reader. Since this portion of the thesis deals with the structural analysis of the trusses, all the architectural criteria the author used can be found within the Architectural Breadth portion. Also, since there is a total of nine trusses being designed for the expansion to Union Station, the author will use only two throughout this portion of the thesis because they are all similar to each other. It should be noted that all trusses were designed by the author. Truss 1 and 2, which is noted on Figure 10, will be the designated trusses used.

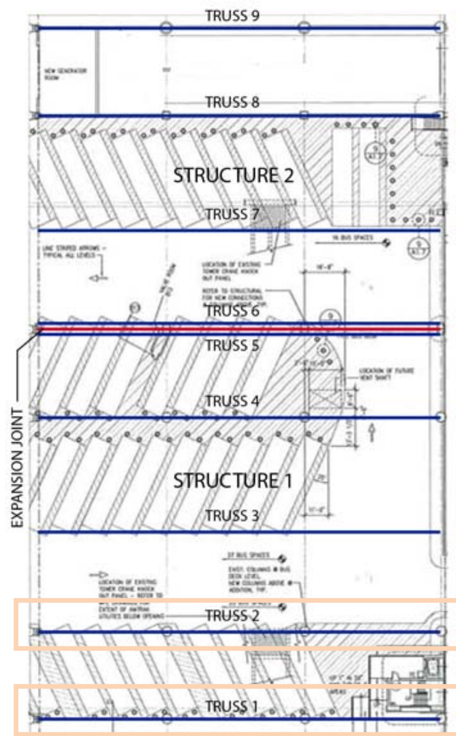


Figure 10: Location of Truss 2

To begin determining the forces within each truss member, the loads acting on the trusses from the four levels above the ground floor had to be resolved. Using RAM Structural, the point loads from Table 5 were figured from the columns on the mezzanine level. By inspection of the values from Table 5, the forces that are acting upon Truss 2 are significantly large. This makes sense because there are four levels the trusses must support and transfer the loads down to the track level then to the foundation.

Truss Point Loads Within Structure 1							
Truss 1							
Column Line	1	2	3	4	5	6	7
Loads (Kips)	337.90	578.80	578.80	578.80	578.80	307.83	72.64
Truss 2							
Column Line	1	2	3	4	5	6	7
Loads (Kips)	669.69	1125.06	1125.06	1125.06	1125.06	835.23	388.98
Truss 3							
Column Line	1	2	3	4	5	6	7
Loads (Kips)	739.67	1186.48	1186.48	1186.48	1186.48	1186.48	739.67
Truss 4							
Column Line	1	2	3	4	5	6	7
Loads (Kips)	900.71	2115.88	2115.88	2115.88	2003.01	2044.84	900.71
Truss 5							
Column Line	1	2	3	4	5	6	7
Loads (Kips)	300.22	501.36	501.36	501.36	302.57	381.97	300.22

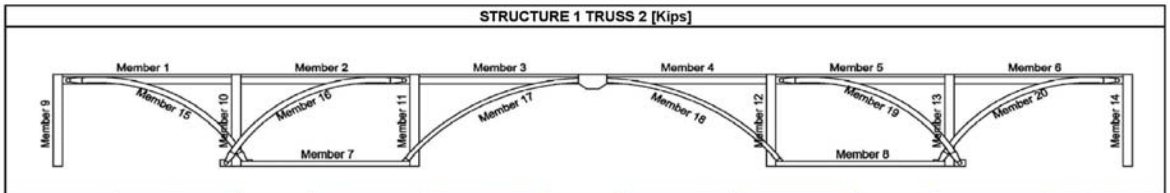
Truss Point Loads Within Structure 2							
Truss 6							
Column Line	1	2	3	4	5	6	7
Loads (Kips)	372.47	622.94	622.94	622.94	622.94	622.94	372.38
Truss 7							
Column Line	1	2	3	4	5	6	7
Loads (Kips)	728.85	1169.08	1169.08	1169.08	1169.08	1056.81	613.60
Truss 8							
Column Line	1	2	3	4	5	6	7
Loads (Kips)	667.22	1120.94	1120.94	1120.94	1120.94	761.28	320.18
Truss 9							
Column Line	1	2	3	4	5	6	7
Loads (Kips)	48.78	72.17	72.17	72.17	72.17	72.17	48.78

Table 5: Loads to Trusses From Above Levels

Using the loads from Table 5, a detailed spread sheet was used to determine the forces within each member for the trusses as well as the support reactions from the columns located on the track level. In addition to the spreadsheet, STAAD Pro was used to verify the forces in the members as well as the reaction values. Since the members for the trusses are unknown as well as the area, the author inputted a one square foot area in STAAD, the values came out to be within 1% of the spreadsheet calculations. To view the spreadsheet and STAAD Pro report of Truss 2, turn to Appendix C, Calculations 1 through 13.

Table 6 below shows the forces within each member of Truss 2. The reader should realize the image used is not the final image of the truss used, but as the design at the time when the loads were determined (See the Architectural Breadth of the thesis to learn more about the design of the trusses). After each load in Table 6 are the directions how the loads act within the members. Tension is represented as [T] and [C] means compression.

STRUCTURE 1 TRUSS 2 [Kips]

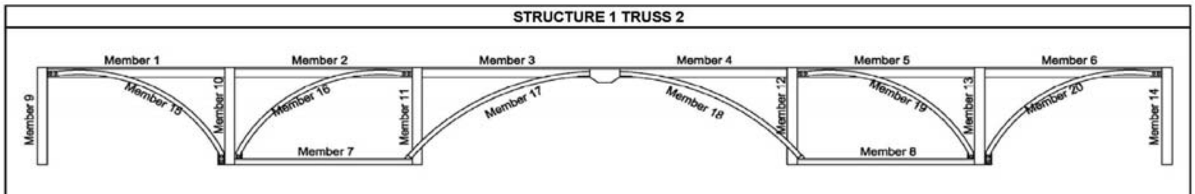


Member 1	860.13 [C]	Member 6	634.45 [C]	Member 11	1759.81 [C]	Member 16	1280.73 [T]
Member 2	860.13 [C]	Member 7	253.38 [T]	Member 12	1598.63 [C]	Member 17	1171.56 [C]
Member 3	253.38 [C]	Member 8	196.32 [T]	Member 13	835.23 [C]	Member 18	1093.17 [C]
Member 4	196.315 [C]	Member 9	1160.00 [C]	Member 14	750.64 [C]	Member 19	956.32 [T]
Member 5	634.45 [C]	Member 10	1125.06 [C]	Member 15	990.06 [T]	Member 20	730.29 [T]

Table 6: Loads in Truss 2 Members

Determination of Preliminary Member Sizes for Trusses:

STRUCTURE 1 TRUSS 2



Member 1	WT15x130.5	Member 6	WT15x130.5	Member 11	W14x176	Member 16	(2) HSS10x0.50
Member 2	WT15x130.5	Member 7	W16x31	Member 12	W14x176	Member 17	(2) HSS10x0.50
Member 3	WT15x130.5	Member 8	W16x31	Member 13	W14x176	Member 18	(2) HSS10x0.50
Member 4	WT15x130.5	Member 9	W14x176	Member 14	W14x176	Member 19	(2) HSS10x0.50
Member 5	WT15x130.5	Member 10	W14x176	Member 15	(2) HSS10x0.50	Member 20	(2) HSS10x0.50

Table 7: Preliminary Sizes for Truss 2

Following the criteria set by AISC, the author used the thirteenth edition of the steel to determine the preliminary sizes of the member for the king post trusses. Each column and top chord of each truss was selected using Part 4 of the manual by taking the un-braced length in the y-axis and making sure $\Phi P_n \geq P_u$. Since both the columns and top chords are in compression, Part 4 of the manual looks at members in compression. Both bottom chords were determined by using Part 5 of the manual since this part looks at members in tension. For the four curved bracing members in tension (Members 15, 16, 19, & 20), the preliminary sizes were selected from Part 1 of the manual by calculating the required I needed for the load then looking up a member that had a greater I. The two bracing members in compression were determined by the same method as the bracing members in tension. Table 7 shows the preliminary member sizes selected for Truss 2. A variety of shapes were selected for the trusses, which is explained in the Architectural Breadth of this thesis. To view the calculations for determining the preliminary sizes in Truss 2, refer to Appendix D, Calculations 1 through 7.

Curved Tension Members In Trusses:

Using curved tension members in the trusses, each one must be looked at to make sure that the moment created by the forces within the member will not cause the shape to go into a compression state. Taking the preliminary HSS sizes, the author created the curved members within STAAD. By making twenty-six increments along the radius of the arced shape as Figure 11 shows on the left, this allows to examine where the maximum moment will occur.

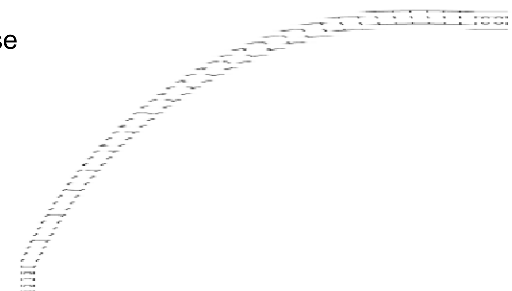


Figure 11: Segments Used For Tension Members

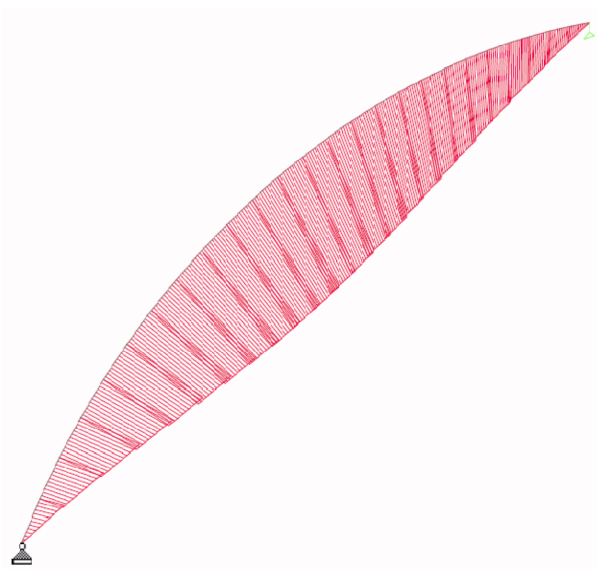


Figure 12: Moment within Tension Member 19

Once each section off the HSS members was modeled in STAAD as well as the forces causing the member to be in tension, an analysis was run on the worse tension member in Truss 2 which is Member 19. The reason for doing the worst case scenario is if the selected HSS member passes, then each tension chord in Truss 2 will pass as well. Figure 12 on the left shows the moment diagram created by the member after the analysis was done it STAAD. One can see how the moment diagram is the shape of a parabola acting in compression. This shows how the moment wants to cause the member to bend into a compression due to tension.

A closer inspection of the STAAD results also shows that the maximum moment occurs within the eleventh segment from the pin connection at the bottom of the member, which can be viewed in Appendix E. To determine the reaction force, R, the maximum moment was divided by the lever arm in the y-direction (Refer to Figure 13). This determined the force within the x-direction (R_x) and taking this value and dividing it by the angle created by the two ends points of the member, 30°, the value R is determined.

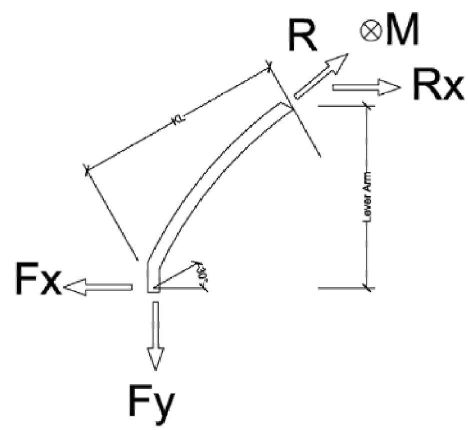
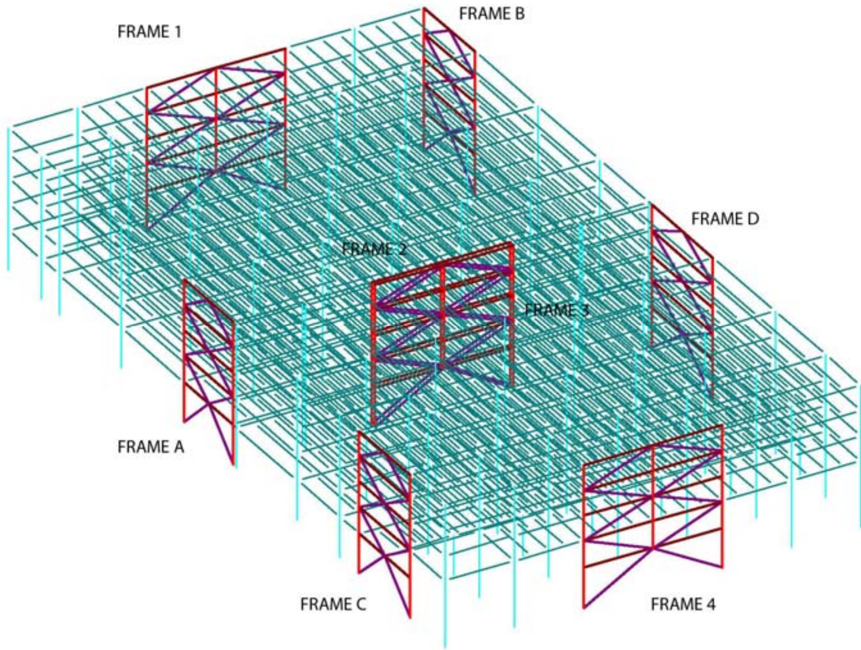


Figure 13: Determination of R

Going into the AISC Steel Manual and using Table 4-5, the ΦP_n can be determined by using the KL length where R is located. For Member 19 of Truss 2, the un-braced length is 23.42 ft and after interpolation within Table 4-5, ΦP_n comes out to be 343 Kips which is greater than 313 Kips for R. Therefore the preliminary size HSS10.0x0.500 can be used for the curved tension member throughout Truss 2. To view the calculations for ΦP_n, turn to Appendix E, Calculation 1.

Lateral Resisting System:



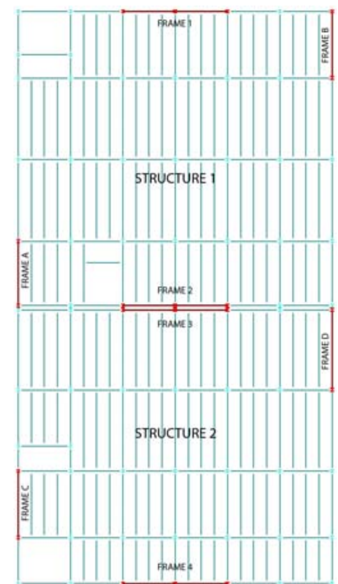


Figure 14: Isometric View of Lateral Resisting System

Figure 15: Plan View of Brace Frames

With a composite steel floor system being used instead of the post-tension system, a new lateral system was incorporated into the expansion of Union Station. Steel brace frames with a response modification factor (R) of 3.25 were selected to replace the existing concrete moment frames (R =3). A total of eight brace frames, four in each structure (two in the north-south direction as well as the east-west), were placed within the expansion. Figure 15 shows the frames in plan view and Figure 14 above shows an isometric view of the expansion to Union Station. From Figure 14, the bottom of the columns is where the ground floor is located. Therefore the ground level and the brace frames is shown in the view. One can then observe from Figure 10 (Page 18) that Frames 1, 2, and 3 are part of Trusses 1, 2, and 6. Each of the trusses with the brace frames as part of them as well as the remaining five frames had to be analyzed to determine if each frame can withstand the forces from wind and seismic. For this portion of the report, Frame 1 which is part of Truss 1 will be looked at in depth. All other Frame calculations can be found in Appendix F.

Selecting preliminary sizes that the author believed could withstand the lateral forces were modeled in RAM Frame and then the members that were undersized were replaced with members that met design requirements. After getting the member sizes for the frames, the author used SAP2000 to determine the required stiffness for each frame. Figure 16 shows the member sizes selected for Truss 1 and Frame 1. Within the truss, the two braces on the ground floor are W14x257. Since these members have to carry both gravity and lateral loads, this is the reason for having such heavy members. Going up the brace frame, the majority of the braces are W14x99. The author wanted to keep the same shape as much as possible throughout Frame 1 and all the others. Going down each level, the columns increase in weight to take more loads from the level above them.

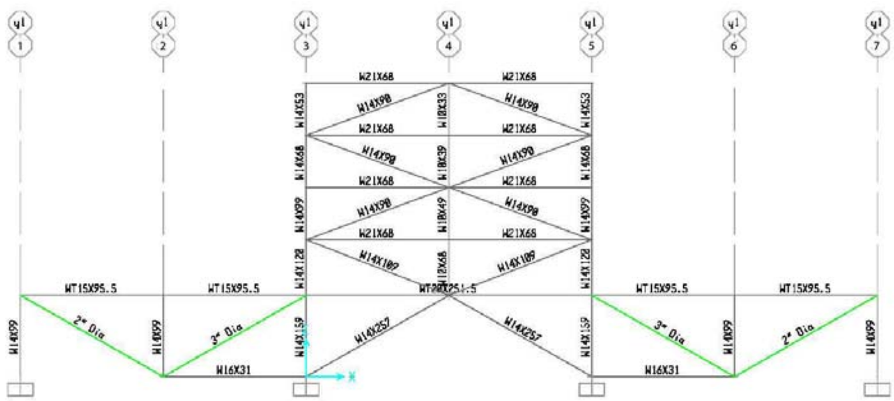


Figure 16: Member Sizes for Brae Frame 1

After the relative stiffness of each frame was determined, center of rigidity, direct, torsional, and net forces due to wind and seismic loads were calculated using RAM and also by hand as well. To understand what each of the previously mentioned definitions are, review technical report three written by the author. All calculations regarding the definitions are located within Appendix F.

Once the net forces due to wind and seismic were determined, each load for both forces was placed on each frame in SAP. Then each frame was analyzed one at a time to verify the serviceability of each frame. Tables 8 and 9 on the following page represent the allowable drift criteria for each floor and the entire expansion as well as the calculated drifts done by SAP. Looking at both tables, one can see that the seismic drift controls from the roof to the first level and the mezzanine level drift is controlled by the wind. These results are almost identical to what was happening in the expansion to Union Station when the ordinary concrete moment frames were being used. Since the response modification factor difference is 0.25 between the two systems, the values obtained are reasonable.

Controlling Wind Drift: Frame 1									
Story	Story Height (ft)	Story Drift (in)	Allowable Story Drift (in) D_{wind} = H/400			Total Drift (in)	Allowable Total Drift (in) D_{wind} = H/400		
Roof	11.500	0.041	<	0.345	Acceptable	0.509	<	1.94	Acceptable
3rd	11.500	0.049	<	0.345	Acceptable	0.468	<	1.595	Acceptable
2nd	11.500	0.140	<	0.345	Acceptable	0.419	<	1.25	Acceptable
1st	12.250	0.150	<	0.368	Acceptable	0.278	<	0.905	Acceptable
Mezzaine	17.917	0.129	<	0.538	Acceptable	0.129	<	0.5375	Acceptable

Table 8: Controlling Wind Drift for Frame 1

Controlling Seismic Drift: Frame 1									
Story	Story Height (ft)	Story Drift (in)	Allowable Story Drift (in) $D_{SEISMIC} = 0.020h_{sx}$			Total Drift (in)	Allowable Story Drift (in) $D_{SEISMIC} = 0.020h_{sx}$		
Roof	11.500	0.129	<	0.230	Acceptable	0.738	<	1.293	Acceptable
3rd	11.500	0.140	<	0.230	Acceptable	0.609	<	1.063	Acceptable
2nd	11.500	0.201	<	0.230	Acceptable	0.469	<	0.833	Acceptable
1st	12.250	0.189	<	0.245	Acceptable	0.269	<	0.603	Acceptable
Mezzaine	17.917	0.080	<	0.358	Acceptable	0.080	<	0.358	Acceptable

Table 9: Controlling Seismic Drift for Frame 1

Truss Connections [M.A.E. Criteria]:

Each of the nine trusses has twenty-six connections that are required for the selected geometry and all twenty-six connections are made up of three types; pin, heavy brace, and gusset plate. A majority of the connections are made up of pin connections from the braces within the truss as well as the top and bottom chords. The heavy brace connections are located on the three trusses that are part of the lateral system and the gusset plate connections are locate at the top in the middle of each truss. Figure 17 shows the location of each type of connections on the two different styles of trusses (Blue is Pin, Green is Heavy Braced, & Red is Gusset Plate). For this report, the typical pin connection for the tension members and the heavy brace members will be looked at.

Each of the pin connections for the tension members were designed based on the criteria set by AISC. Since there is a significant load within the members, each steel plate is A992. This is to keep the size and the thickness of each plate reasonable. For exterior trusses (Truss 1, 5, 6, & 9), a single bolt pin connections was used for the rods and a two bolt pin connection was used for the interior connections (Figures 18 & 19 show a visual of each plate). The reason for using two pins in the exterior trusses is the load on a single pin makes the diameter significantly large. Therefore using two smaller pins makes the connection look more aesthetically appealing. Dimensions a and b from Figures 18 & 19 are the minimum distance from the edge of the plate to prevent and failure from occurring. One can view the calculations for all the plates for each truss in Appendix H, Calculations 1 through 9 and Table 10 on page 26 summarizes the plates and pins used for Trusses 1 & 2.

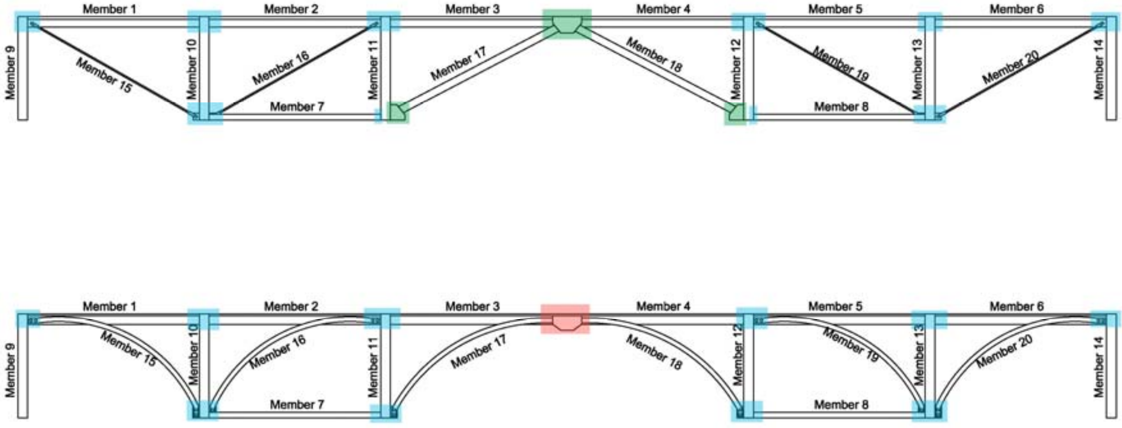


Figure 17: Types of Connections

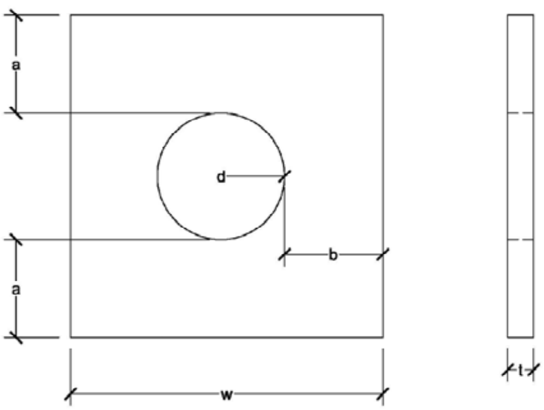


Figure 18: Single Pin Plate Design

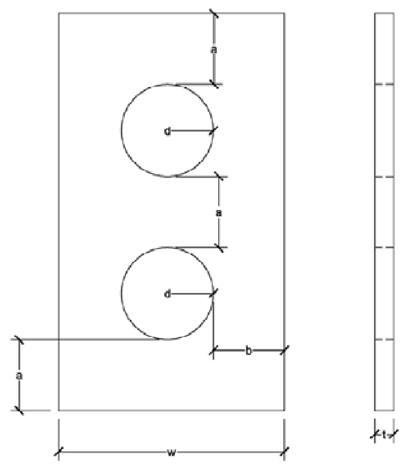


Figure 19: Double Pin Plate Design

Truss 1						Truss 2				
Member	15	16	19	20		Member	15	16	19	20
w (in)	10	10.5	10.5	10		w (in)	14	14.5	14	14
t (in)	1.125	1.25	1.25	1.125		t (in)	1.5	1.75	1.5	1.5
d_{pin} (in)	4	4	4	4		d_{pin} (in)	(2) 3	(2) 3	(2) 3	(2) 3

Table 10: Dimensions & Pin Sizes for Trusses 1 & 2

All pin plates will be connected to the columns web by using two welds along on the perimeter of the plate. Welds were used instead of bolts because the width of the plate should be at a minimum to prevent any trouble with the busses traveling near. Figure 20 on the left shows how the plates are connected to the columns of the trusses with. Each weld was designed by using Table 8-6 from AISC Steel Manual because the loads are coming in at an angle of sixty degrees off the vertical. Since the load is located at the center of the plate (Figure 21), there is no eccentricity from the load therefore the value of a within Table 8-6 is zero. Each plate weld calculation can be found in Appendix H, Calculations 10 through 18 and Table 11 below shows the size of the weld for Trusses 1 and 2.

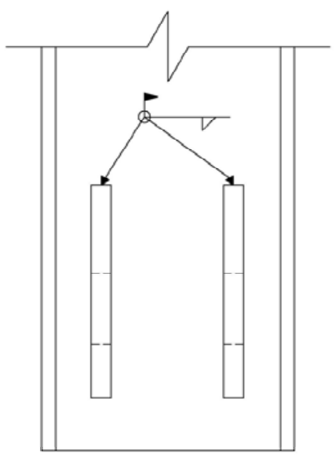


Figure 20: Weld Symbols for Plates

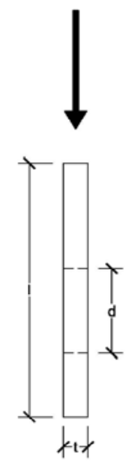


Figure 21: Location of Load

Truss 1						Truss 2				
Member	15	16	19	20		Member	15	16	19	20
Weld	Fillet	Fillet	Fillet	Fillet		Weld	Fillet	Fillet	Fillet	Fillet
Electrode	E70XX	E70XX	E70XX	E70XX		Electrode	E70XX	E70XX	E70XX	E70XX
t (in)	7/16	8/16	4/16	4/16		t (in)	9/16	11/16	9/16	7/16

Table 11: Weld Sizes for Trusses 1 & 2

All but two of the welds are fillet welds. Members 16 and 19 in Truss 4 have full penetration welds because the actual size of a fillet weld exceeds 1 inch and it is around the same cost for a fillet weld over 1 inch and a full penetration weld.

The second type of connection being looked at in this thesis is a heavy brace connection on Truss 1 which acts as part of the lateral system. To prevent any moment from occurring within the connection, the Uniform Force Method was used. Figure 22 shows the location as where the forces for the column and the beam would be located at on the plate. Since there is no beam required, due to the bottom chord at the location is a zero force member, and the connections is being attached to the column web, the only two forces acting are the shear in the column and the pull out force need in a beam. Therefore a WT7x41 was selected to handle to horizontal force from the connection.

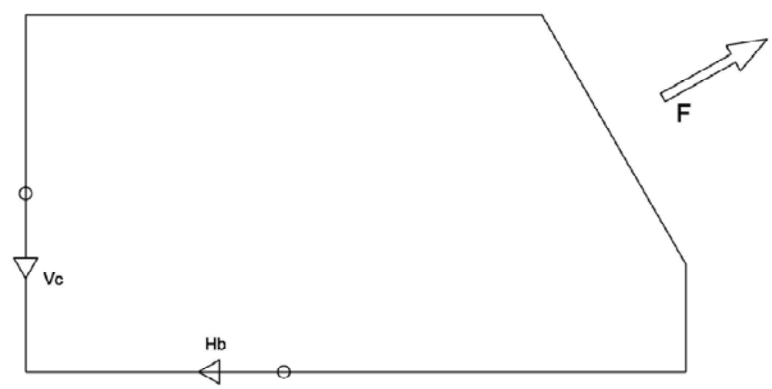


Figure 22: Column and Chord Force Distribution in Plate

All limit states for each portion of the connection were taken into detail for the heavy brace connection. On the following page of this thesis is a detailed drawing of what members, bolts, welds, and dimensions are required for brace member 17 for Truss 1. All calculations for this connection can be found in Appendix H, Calculations 19 through 22.

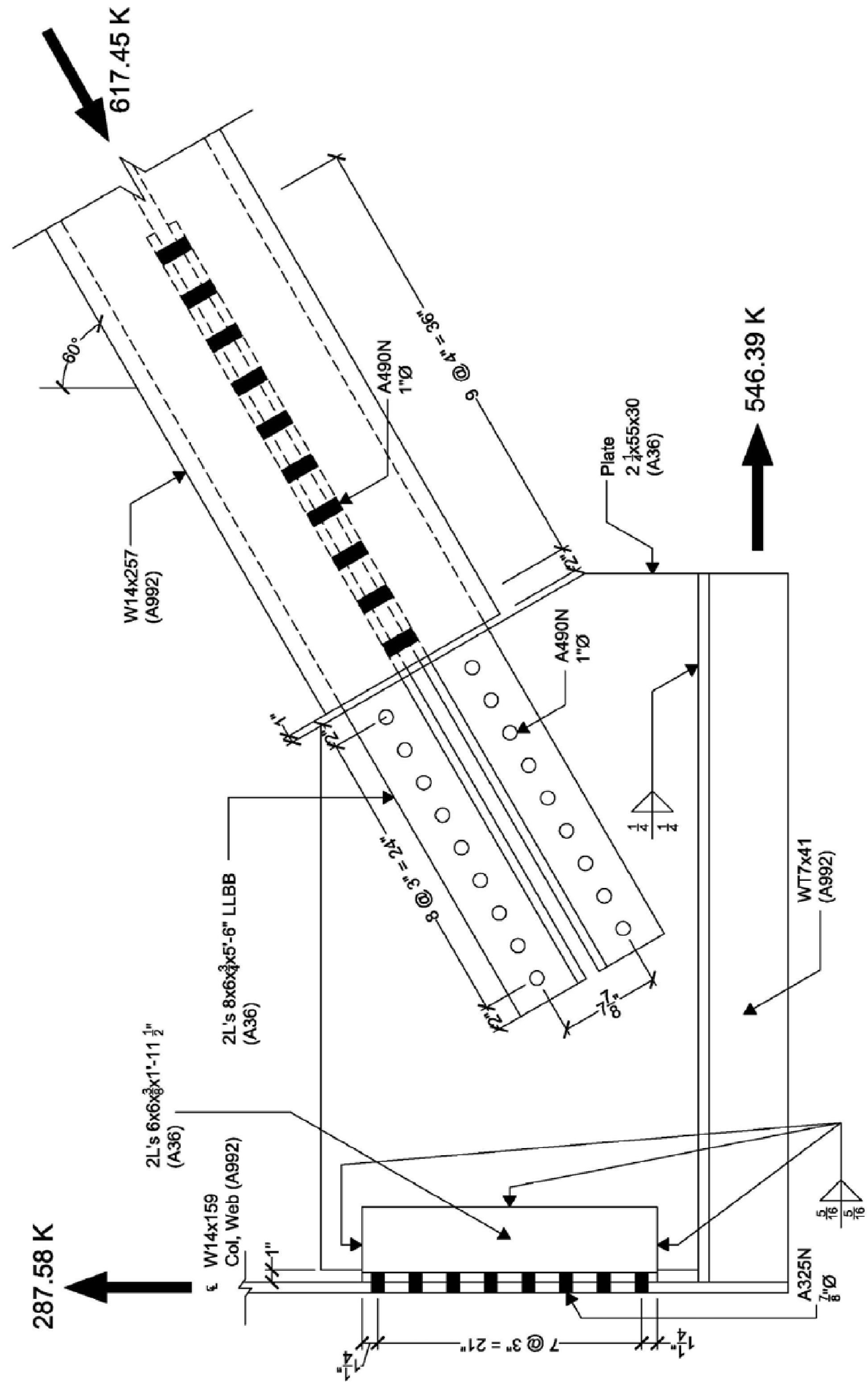
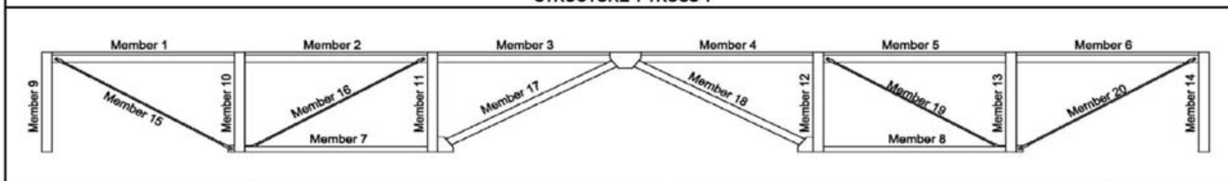


Figure 23: Detailed Connection of Heavy Brace Member

Final Members for Trusses:

After taking all the loads from the upper levels, figuring out the forces that act in all the members, performing lateral analysis, designing the connections, Table 12 and 13 shows the final members used for both Trusses 1 and 2. Appendix **I**, Tables 1 through 9 show all nine members with their final member sizes. As stated previously in this report, refer to the Architecture Breadth to understand why the two trusses were designed as they are.

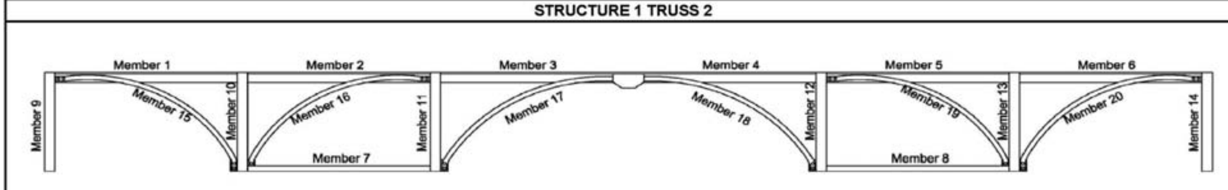
STRUCTURE 1 TRUSS 1



Member 1	WT15x95.5	Member 6	WT15x95.5	Member 11	W14x159	Member 16	(2) 3" Dia. Rod
Member 2	WT15x95.5	Member 7	W16x31	Member 12	W14x159	Member 17	W14x257
Member 3	WT20x251.5	Member 8	W16x31	Member 13	W14x99	Member 18	W14x257
Member 4	WT20x251.5	Member 9	W14x99	Member 14	W14x99	Member 19	(2) 3" Dia. Rod
Member 5	WT15x95.5	Member 10	W14x99	Member 15	(2) 2" Dia. Rod	Member 20	(2) 2" Dia. Rod

Table 12: Final Members for Truss 1

STRUCTURE 1 TRUSS 2



Member 1	WT15x130.5	Member 6	WT15x130.5	Member 11	W14x176	Member 16	(2) HSS10x0.50
Member 2	WT15x130.5	Member 7	W16x31	Member 12	W14x176	Member 17	(2) HSS10x0.50
Member 3	WT15x130.5	Member 8	W16x31	Member 13	W14x176	Member 18	(2) HSS10x0.50
Member 4	WT15x130.5	Member 9	W14x176	Member 14	W14x176	Member 19	(2) HSS10x0.50
Member 5	WT15x130.5	Member 10	W14x176	Member 15	(2) HSS10x0.50	Member 20	(2) HSS10x0.50

Table 13: Final Members for Truss 2

Foundation Verification:

Since the soil at the site for the expansion to Union Station is considered weak (Refer to Foundation Section of Existing Structural System), making sure the foundation can withstand the change from post-tension concrete to steel is important. The change of systems works as one can see from Table 14 on the following page. There is one area where there is a problem and that is located at Truss 4. The *b* designed succeeds the allowable area to put a square footer. One possibility to correct this is to have a different system as the foundation. Since the research and development of a new foundation system was not part of this thesis due to the time restriction the author had.

The second check for the foundation done was making sure overturning was not a problem. Table 15 on page 31 shows that overturning moment is not an issue to Union Station. The two checks done on the foundation verify that this new system can work on the site for the expansion to Union Station.

Spot Checks On Foundation: Structure 1				
Grid Line: Truss 1				
Member	9	11	12	14
P (Kips)	591.28	1210.00	1030.00	206.93
# Supporting Piles For Platfroms	4	4	4	4
Tons Per Pile	73.91	151.25	128.75	25.87
σ (psf)	2000.00	2000.00	2000.00	2000.00
$A_{required}$ (ft^2)	73.91	151.25	128.75	25.87
b (ft)	9	13	12	6
≤ 18 ft ?	Yes	Yes	Yes	Yes

Spot Checks On Foundation: Structure 2				
Grid Line: Truss 6				
Member	9	11	12	14
P (Kips)	641.03	1290.00	1290.00	641.03
# Supporting Piles For Platfroms	4	4	4	4
Tons Per Pile	80.13	161.25	161.25	80.13
σ (psf)	2000.00	2000.00	2000.00	2000.00
$A_{required}$ (ft^2)	80.13	161.25	161.25	80.13
b (ft)	9	13	13	9
≤ 18 ft ?	Yes	Yes	Yes	Yes

Spot Checks On Foundation: Structure 1				
Grid Line: Truss 2				
Member	9	11	12	14
P (Kips)	1160.00	2340.00	2140.00	750.64
# Supporting Piles For Platfroms	4	4	4	4
Tons Per Pile	145.00	292.50	267.50	93.83
σ (psf)	2000.00	2000.00	2000.00	2000.00
$A_{required}$ (ft^2)	145.00	292.50	267.50	93.83
b (ft)	13	18	17	10
≤ 18 ft ?	Yes	Yes	Yes	Yes

Spot Checks On Foundation: Structure 2				
Grid Line: Truss 7				
Member	9	11	12	14
P (Kips)	1230.00	2420.00	2350.00	1070.00
# Supporting Piles For Platfroms	4	4	4	4
Tons Per Pile	153.75	302.50	293.75	133.75
σ (psf)	2000.00	2000.00	2000.00	2000.00
$A_{required}$ (ft^2)	153.75	302.50	293.75	133.75
b (ft)	13	18	18	12
≤ 18 ft ?	Yes	Yes	Yes	Yes

Spot Checks On Foundation: Structure 1				
Grid Line: Truss 3				
Member	9	11	12	14
P (Kips)	1250.00	2460.00	2460.00	1250.00
# Supporting Piles For Platfroms	4	4	4	4
Tons Per Pile	156.25	307.50	307.50	156.25
σ (psf)	2000.00	2000.00	2000.00	2000.00
$A_{required}$ (ft^2)	156.25	307.50	307.50	156.25
b (ft)	13	18	18	13
≤ 18 ft ?	Yes	Yes	Yes	Yes

Spot Checks On Foundation: Structure 2				
Grid Line: Truss 8				
Member	9	11	12	14
P (Kips)	1160.00	2340.00	2090.00	650.31
# Supporting Piles For Platfroms	4	4	4	4
Tons Per Pile	145.00	292.50	261.25	81.29
σ (psf)	2000.00	2000.00	2000.00	2000.00
$A_{required}$ (ft^2)	145.00	292.50	261.25	81.29
b (ft)	13	18	17	10
≤ 18 ft ?	Yes	Yes	Yes	Yes

Spot Checks On Foundation: Structure 1				
Grid Line: Truss 4				
Member	9	11	12	14
P (Kips)	1820.00	4370.00	4220.00	1790.00
# Supporting Piles For Platfroms	4	4	4	4
Tons Per Pile	227.50	546.25	527.50	223.75
σ (psf)	2000.00	2000.00	2000.00	2000.00
$A_{required}$ (ft^2)	227.50	546.25	527.50	223.75
b (ft)	16	24	23	15
≤ 18 ft ?	Yes	No	No	Yes

Spot Checks On Foundation: Structure 2				
Grid Line: Truss 9				
Member	9	11	12	14
P (Kips)	79.82	149.39	149.39	79.82
# Supporting Piles For Platfroms	4	4	4	4
Tons Per Pile	9.98	18.67	18.67	9.98
σ (psf)	2000.00	2000.00	2000.00	2000.00
$A_{required}$ (ft^2)	9.98	18.67	18.67	9.98
b (ft)	4	5	5	4
≤ 18 ft ?	Yes	Yes	Yes	Yes

Spot Checks On Foundation: Structure 1				
Grid Line: Truss 5				
Member	9	11	12	14
P (Kips)	517.32	1040.00	769.39	461.84
# Supporting Piles For Platfroms	4	4	4	4
Tons Per Pile	64.67	130.00	96.17	57.73
σ (psf)	2000.00	2000.00	2000.00	2000.00
$A_{required}$ (ft^2)	64.67	130.00	96.17	57.73
b (ft)	9	12	10	8
≤ 18 ft ?	Yes	Yes	Yes	Yes

Table 14: Spot Checks on Foundation

Overturning Verification: Wind										
Structure 1						Structure 2				
Moment (ft-Kips)	L/2 (ft)	P (Kips)	W/2 (Kips)	Overturning Issues		Moment (ft-Kips)	L/2 (ft)	P (Kips)	W/2 (Kips)	Overturning Issues
45170	89.75	503.2869	27106.5	No		45235	83.5	541.7365	23324	No
Overturning Verification: Seismic										
Structure 1						Structure 2				
Moment (ft-Kips)	L/2 (ft)	P (Kips)	W/2 (Kips)	Overturning Issues		Moment (ft-Kips)	L/2 (ft)	P (Kips)	W/2 (Kips)	Overturning Issues
48422	89.75	539.5209	27106.5	No		46603	83.5	558.1198	23324	No

Table 15: Overturning Moment

Structural Depth Conclusion:

Designing a transfer system is no easy task for an engineer, especially when one decides to be creative and integrate the structure and architecture of the building. The hardest challenge was not to determine the loads from the above floor or figuring out the loads in the members or designing a new lateral system, but figuring out if the curved tension members selected could withstand the moment inside the member which wants to pull the member into a compression state. Making sure the foundation system could support the new structural system as well was a challenging task as well. Trying to keep in mind that the soil on the site is weak throughout the whole structural depth was at times hard.

After all the calculations and innovative structural designing, the author believes the thesis criteria goals 1 through 4 (refer to page 10) where accomplished. Switching from a post-tension concrete to a composite steel floor system with transfer trusses can satisfy not only the goals of the design firm, but the author's as well. Each truss shows how creative an engineer can be when working with certain boundaries to follow. The only main concern the author wishes there was more time for was the foundation system. If there was time, the author would try to create a new foundation system that would work around the train tracks and give extra support to the expansion of Union Station.

ARCHITECTURAL BREADTH

Architectural Design of King Post Trusses:

To create a signature expression for the expansion of Union Station, the author believed the best way to achieve this was to use the king post trusses that act as the transfer system. In order to make the trusses look appealing to the traveler's eye, a variety of members should be used and the author wanted to make the trusses look like no other truss someone has seen. This is to make the viewer look and ask themselves the questions about the trusses.

In the beginning of the design of the trusses, sketches were drawn up for parts of the trusses. The authors started off with just having one type of truss in the expansion. Having one truss kept the concept simple but intriguing. Shown below are concepts the author originally started off with. One can see from Figure 1 that the original thoughts of the top chord was to have a built up box shape with a WT member as the bottom portion with tension rods connecting into the web of the WT member. Using HSS rectangular members as columns were thought about since it would be different having them act as columns to carry massive loads. Figure 2 shows how the bottom chord could be rotated 90 degrees where the columns rest on the web of bottom chord and the original bracing members were going to be double angles. The original design of the trusses can be found on Figure 1 (Note that the truss is not to scale).

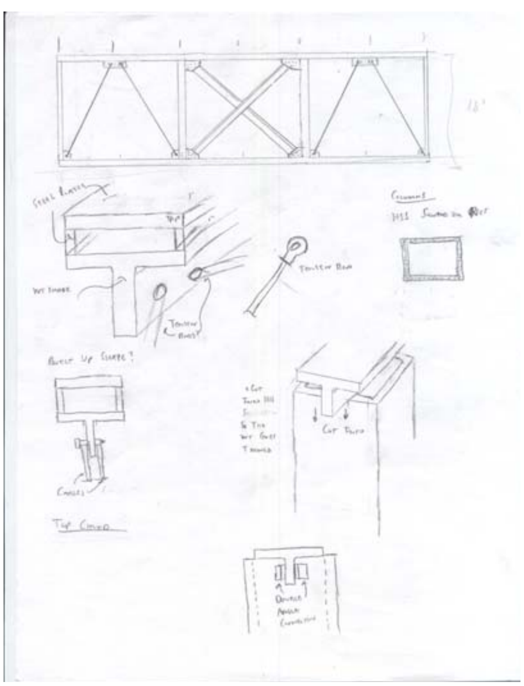


Figure 1: Top Chord & Column Concepts

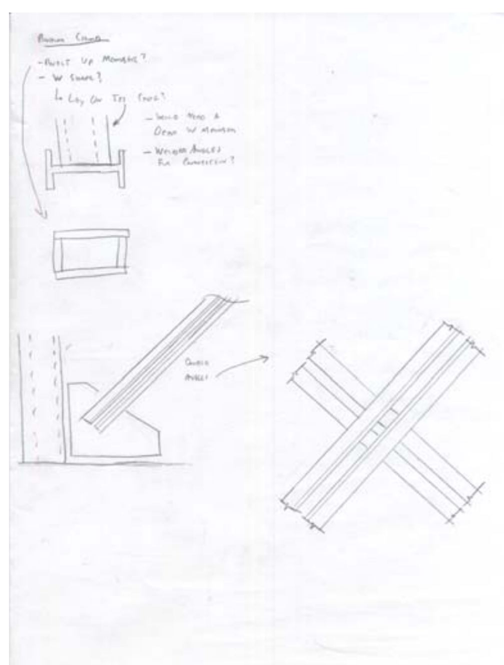


Figure 2: Bottom Chord & Brace Concepts

While the concepts and style of the truss is different than most trusses, the author felt there wasn't enough "signature" behind it. Going back and rethinking about some of the members to use, only WT members would be used for the top chords (no built up members). This is to give a more simple look to the top chords and no one would really see the built up member portion since the floor to ceiling height is 18'-0" high. Also three of the columns were removed in this design of the trusses because this would give long spans to the trusses which would help with traffic of the busses as well as make the trusses feel as if they were more related to the definition of a king post truss. Figure 3 below shows the second design of the truss for the expansion to Union Station. From here on out within this section, the truss figures are scaled correctly to the 18'-0" high by 189'-0" long.

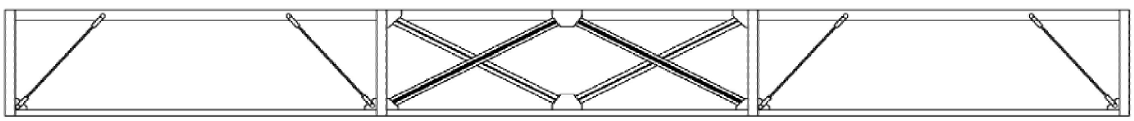


Figure 3: 2nd Design of Truss

Having long spans, tension rods, and an interesting diagonal bracing in the middle portion of the truss does make this design interesting to view. However, the author realized that problems could arise with the long spans from a structural engineering stand point. One problem is the weight from the floors above could cause a significant deflection which could lead to future problems. Another design was sketched up (Figure 4) and in this design, the three columns removed were placed back. Also the tension rods were inverted and now meet at the bottom of a truss because the author wanted to view the rods at a different perspective. Each tension rod was no longer attached to the web of the WT member at the top and a plate was used instead for the design.

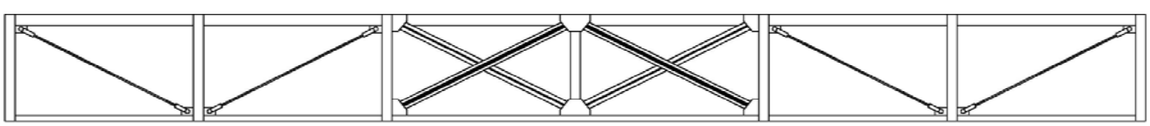


Figure 4: 3rd Design of Truss

Realizing that the tension rods look more appealing when they meet at the bottom of a column, the author finalized this portion of the trusses. When structural calculations were being done on the trusses (Refer to the structural depth portion of this report), there were three zero force members. Since those members serve no purpose, the author removed them from the truss and when that happened, the trusses became more intriguing to look at. Once this took place, the author decided to remove the center column and replace the double angles with a wide flange shape (Refer to Figure 5 below). All of the following changes started to make the truss look as if it were a signature expression for the expansion.

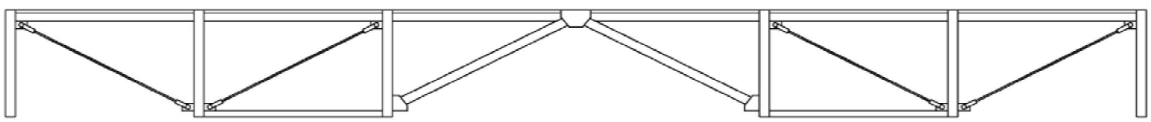


Figure 5: 4th Design of Truss

To finalize this truss, the tension rods were once again connected to the web of the WT members to make the connection seem simplified to the viewers' eye. Figure 6 shows the final design of the truss. Now that this truss was completed, the author realized that this truss would not work on the inside of the expansion since the busses must travel and park in their allowed areas. Therefore a new truss would be created for the interior and the truss already created will be used for the exterior of the building.

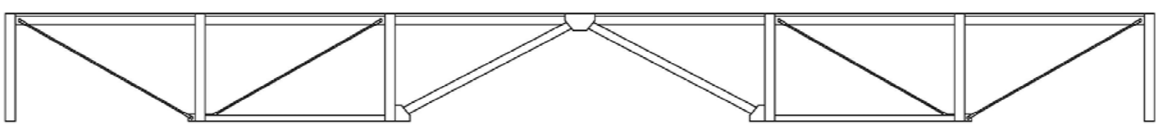


Figure 6: Final Exterior Truss Design

While thinking about what could be used in the interior truss to allow busses to travel under while making a signature expression, the idea of using curved members that create an arch would came to the author's attention. This gives the feeling of openness to the ground floor as well as drawing one's view to the trusses giving a sense of intrigue. HSS tubes were selected as the members for the curved for not only their strength they can carry but as another different steel member used in the trusses that already have multiple steel shapes within them. Figure 7 below shows the final design for the interior trusses.

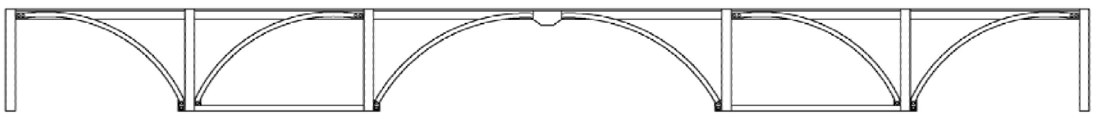


Figure 7: Final Interior Truss Design

Verification of Vehicular Circulation:

With the trusses placed at their desired locations on the ground floor, making sure the busses can travel and park to unload passengers is of high importance. From Figure 8 on the right, one can see the original circulation path used in the expansion to Union Station and where the trusses are located at. Since Trusses 5 and 6 are the exterior type due to being the ends of the two structures, this causes a problem with the original circulation. Due to the tension rods used as braces within the trusses, the busses will not be able to pass with the clearance height required as well as park under. Therefore the author proposes a change of the circulation on the ground floor and the location of the parking spaces as well.

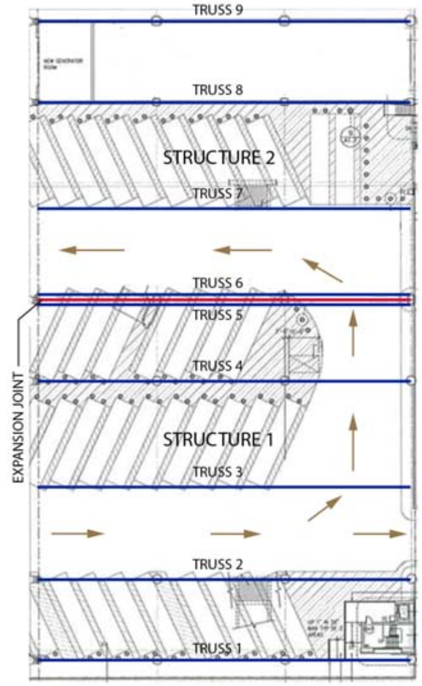


Figure 8: Existing Vehicular Circulation

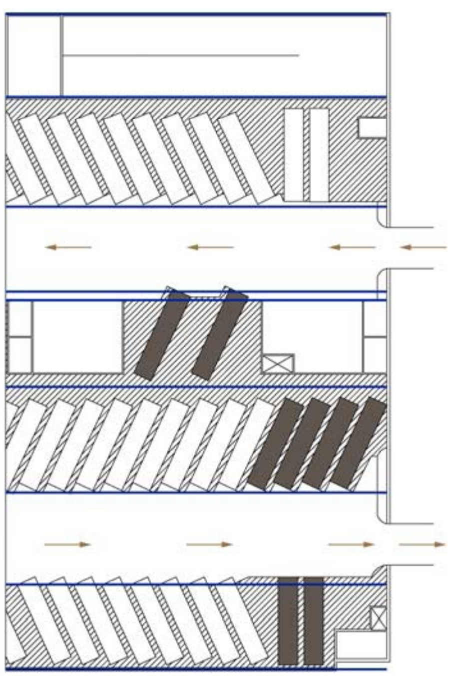


Figure 9: New Vehicular Circulation & Bus Locations

Figure 9 on the left indicates the new circulation for the expansion to Union Station. Instead of just having an exit to H-Street in Washington DC, an entrance was created as well which helps reduce the traffic at the main entrance of the entire building. Allowing an entrance in the expansion gives the busses the choice to reduce the trip around the building to their designated parking zone. As for the relocation of the busses that have issues with Trusses 5 & 6, the areas shaded gray in Figure 9 show where how six of the eight bus zones can be moved without causing major problems. At the very bottom right of the expansion, two of the buses were positioned where some of the waiting area is, but since more room was created between the trusses, the author decided to relocate the waiting terminals which are discussed on the following page.

Redistribution of Waiting Terminals:

Located next to the MEP Room of the expansion of Union Station (Upper left corner of Figure 8) is where all of the area for the waiting terminal is located. Now that the circulation and location for the busses has changed, the author decided to break up the one area into two parts that are now underneath the king post trusses (refer to Figure 10). This gives the trusses more of a signature expression while the crowd can notice them while they wait and stare at them through the glass walls that make up the new terminals (refer to Figure 11). The new floor plan not only helps with the expression of the trusses, but now draws travelers to want to stay inside the waiting terminals.

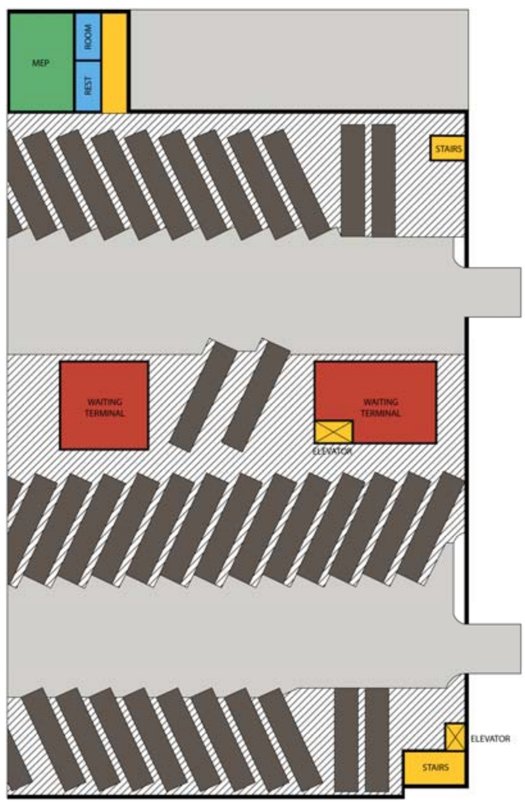


Figure 10: New Ground Floor Plan

Architectural Breadth Conclusion:

As stated in the previous page, the new floor plan of the expansion to Union Station optimizes the area while giving the signature expression of the trusses the author wanted to create. Having the waiting terminals under the trusses helps draw the attention to the trusses but also makes one to believe that this building was given a custom idea when in the design phase. If given more time for this thesis, the author would have liked to keep working on the architectural design of the trusses. While they are one of a kind for the building and do give off a signature look, there could have a better concept for the columns. One possibility could have been to have two different members act as the columns and join with a creative connection half way. To view renderings of the trusses and ground floor of the expansion to Union Station, Refer to the Renderings portion of this report located on page 44.

LIGHTING BREADTH

Selection of Luminaries to Highlight Trusses:

Acting as a signature expression for the expansion to Union Station, all the trusses should be illuminated to capture the grand impression each one gives off. Instead of using typical luminaries to highlight the trusses, the decision to use LEDs was determined by the author because not only do LEDs save energy and last for a long period of time, but they also gives off a high-performance illumination and beam quality to emphasis the structure being lit. The author selected the eW Graze Powercore Linear LED strip made by Philips. Typically the Philips eWs are used for exterior lighting to emphasis a façade or structure and since the trusses are part of the structure to Union Station, the LEDs fit the criteria where they are going to be used.

Each four foot section has forty-eight white LEDs inside that will give off five foot candles at a distance of eighteen feet (the height of the trusses). Each one of the trusses will have six of the four foot length LED strips per 31'-6" at the bottom of each one (Refer to Figure 1 to see layout). Note that the LEDs are not scaled to size in the width direction because the author wants the reader to be able to see how they will be spaced. Since the two bottom chords of all the trusses are rotate ninety degrees (resting on the web), the lights within that 31'-6" will be placed within the chord. Having indirect lighting will guide one's eyes from the ground to looking up and noticing the trusses within the expansion. Figure 2 below is a picture of the four foot strip of LEDs and to view the specifications for the lights, refer to Appendix J.

It should be noted to the reader that the LEDs will not be the main lighting system for the bus terminal area. Only will the LEDs serve the purpose of illuminating the trusses and another system shall be used to meet the requirements set forth by the IESNA for lighting the bus terminal.

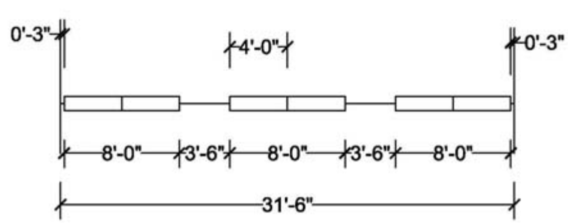


Figure 1: Typical 31'-6" LED Layout under Trusses

Figure 2: eW Graze Powercore 4' Strip

Illuminance Categories & Required Foot Candles:

Using Chapter 10 of the IESNA book, the required illuminance categories as well as foot candles were determined. Table 1 below shows each area with the requirements set forth by IESNA. The waiting terminals have a required fifty foot candles due to the fact that there will be ticket counters within the areas and according to Figure 2 in Appendix K, a minimum of fifty foot candles is required. All other remaining illuminance category requirements can be found from Figure 2 for the ground floor and Figure 1 in Appendix K shows the required foot candles for each illuminance category.

Ground Floor of Expansion to Union Station					
Area	Watiting Terminal	Restrooms	Bus Terminal	Elevators	Stairs
Category	E	B	A	B	B
Foot Candles	50	5	3	5	5

Table 1: Illuminance Categories & Foot Candles

Selection of Luminaries for Waiting Terminals:

Since the waiting terminal has been broken into two areas that are now located within the center of the expansion to Union Station, the author wanted to use different luminaries than the existing ones. The Avante recessed direct/indirect lighting luminarie was selected for each of the waiting terminals. Each luminarie consists of three T8 32 Watt lamps that create indirect light which is then reflected as direct lighting from the cover of the Avante luminarie. This luminarie is suggested using in areas where there is a work space that one has to concentrate on. Because there are ticket counters in the waiting terminals, this type of luminaire works sufficiently. Figure 3 on the right shows the design of the luminaire and the specifications can be found in Appendix L.

Figure 3: Avante 2x4 Luminaire

Lumen Method for Waiting Terminal:

From Chapter 9 of IESNA, the lumen method design approach was used to calculate the number of Avante 2x4 luminaries required for the waiting terminal on the left side of the ground floor plan (refer to Figure 10 on page 36 of the report). The cavity ratios used to determine the required ρ for the walls, floor, and ceiling were 80/60/30. These are the numbers typically used when designing for a room with its criteria. All light loss factors were determined as well based on the assumption the luminaries used are a category type VI with a clean environment and have a cleaning period of three months. The reason three months is used is staff may not clean the luminaries every week, but it is safe to assume around every three months a cleaning will take place.

After all calculations were done, the amount of luminaries required to light the 35'-0"x35'-0" waiting terminal is 10.55. Since a whole number is required, the author decided to select twelve luminaries as the number for the waiting terminal and this number falls within the ten percent tolerance allowed for the lumen method. Figure 4 below shows the waiting terminal with its relative ceiling grid, which has 2'x2' grids, and the location of the twelve Avante luminaries. One can notice that the author has spaced the luminaries evenly across the entire ceiling plan to evenly distribute the light being generated. To view all calculations, charts, and diagrams for the waiting terminal, refer to Appendix M.

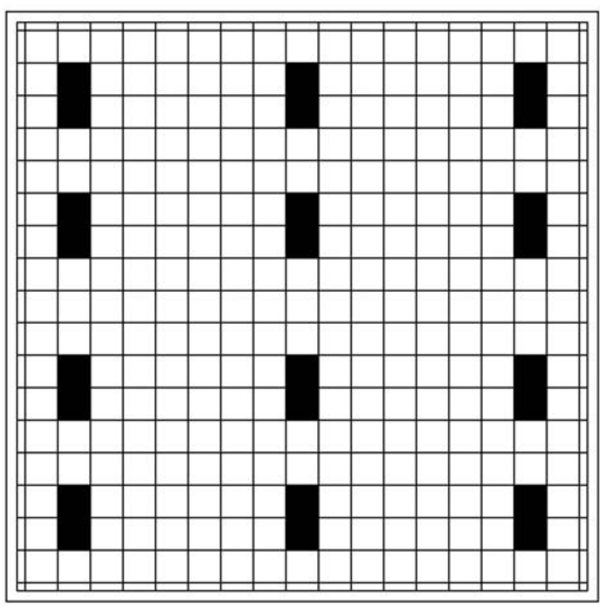


Figure 4: Lighting Layout for Waiting Terminal

Lighting Breadth Conclusion:

Giving the trusses a separate lighting system than the bus terminal creates a greater impact on how they are the signature expression for the expansion to Union Station. Using LEDs helps give more of a direct beam that shots up from the ground and invites one who is one the ground to look up and notice the trusses. Lighting up each truss gives the feeling of comfort to them as well. Instead of having a dark area where you cannot see what is happening, making it possible to see the connections and each member of the trusses make one feel safe when walking, waiting, or riding underneath the trusses.

Creating two separate waiting terminals and putting them under the trusses and giving them new lighting fixtures draws the travelers who are waiting to come to them and take a rest of this feet. Each of the new lightning schemes in the waiting terminals brighten up the center of the ground floor to the expansion of Union Station.

One can argue that the location of the LEDs on the ground could cause problem when the busses need to pass underneath and park. Recognizing this problem, the author suggests using strong plexus glass over the top of the LEDs to prevent them from being damaged by a moving vehicle.

CONCLUSION

All three topics discussed in this thesis; structural, architecture, and lighting were centered about the trusses. While other concerns were brought up in the two depths (i.e. moving the waiting terminal to a new location), they impacted on the concept of the trusses as well.

This signature expression does make a significant improvement to the expansion of Union Station and also meet all the criteria goals set forth by the author (Refer to page 10 to review the goals). As stated in the structural depth conclusion, the only concern the author has is with the foundation to the expansion. With more time, the author would have liked to try to redesign the foundation system so it would not be close to its limit.

One topic not mentioned in this thesis is the cost of creating this signature expression through trusses. The mezzanine level through the third floor's cost would not be a concern because those floors are switching from a post-tension floor slab to a composite steel system where the composite system is cheaper (Refer back to Technical Report II done by the author). However, the trusses would need significant time for steel to be erected as well as making sure all the tension members were ready for the loads from the floors above. Also, the welds the author requested for the plates on the columns would raise the cost since an extra set of specialized workers would be needed. The author still believes even though the cost of the expansion could increase and the schedule could take a little longer due to the trusses, the benefit of having this grand expression in the building would not only mean better business for the owner, but would give the occupants and travelers something to talk about while in the expansion to Union Station.

APPENDIX A: COMPOSITE STEEL STRUCTURAL FLOOR PLANS

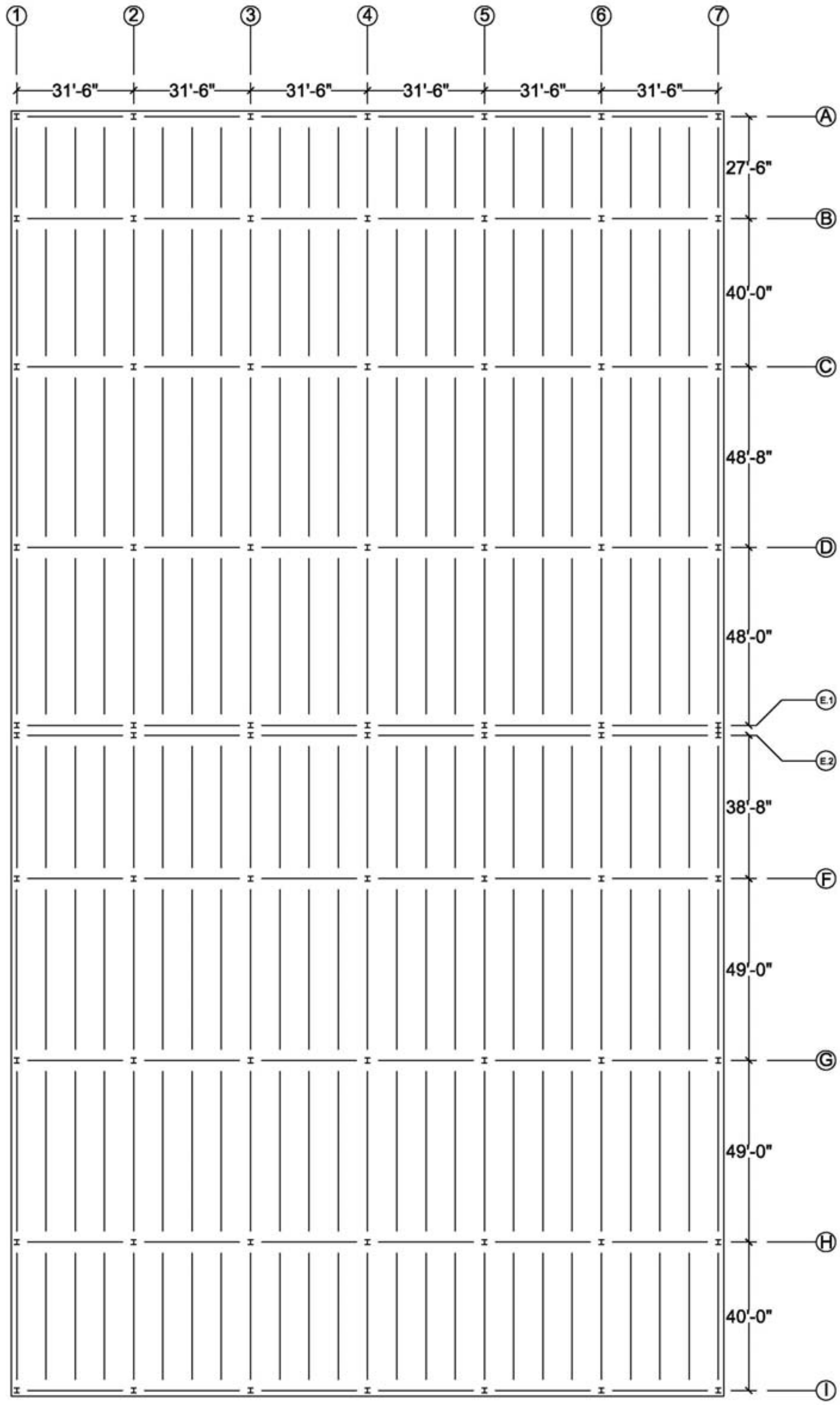


Figure 1: Preliminary Floor Plan for Levels 1st to Roof

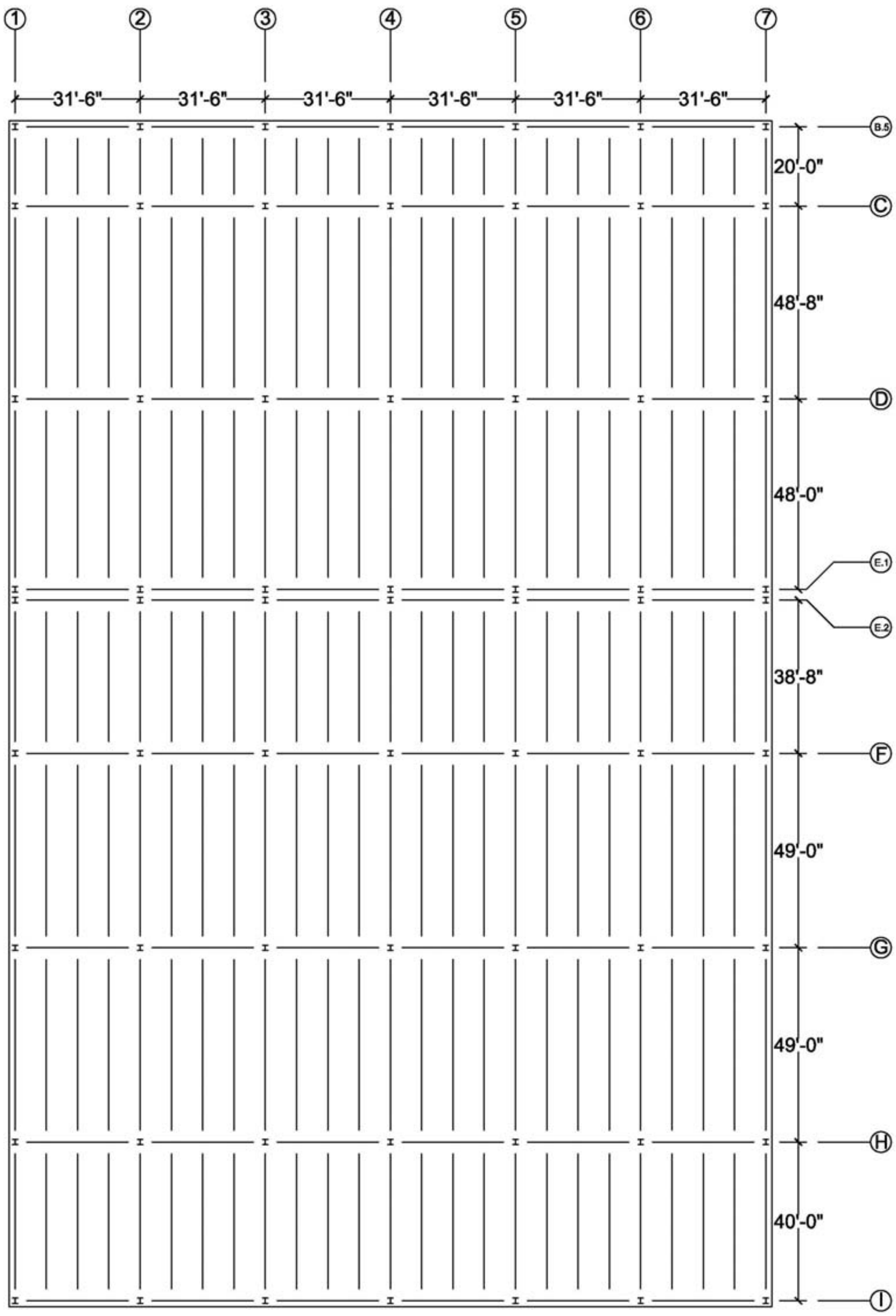


Figure 2: Preliminary Floor Plan for Mezzanine Level

APPENDIX B: PRELIMINARY COMPOSITE STEEL CALCULATIONS

COMPOSITE STEEL DECK	
Fire Rating (Hours)	2
Concrete (Type)	Lightweight
Concrete Unit Weight (pcf)	107 - 116
Concrete Thickness (in)	4.25
Loading Used (psf)	100
Superimposed Load (psf)	15
Total Load (psf)	115
Deck Thickness (in)	2
Total Floor Thickness (in)	6.25
Weight of Slab (psf)	51
Deck Type	2VLI16

Figure 1: Composite Steel Deck Calculations

Level	Roof	
Member (Grid Location)	49' Int.	
Slab Thickness (in)	5.25	
f'_c (psi)	3000	
Length (ft)	49	
Tributary Area (ft)	7.875	
Dead Load (psf)	0	
Super Imposed Load (psf)	15	
Live Load (psf)	50	
w_u (psf)	108	
M_u (ft-Kip)	254	

$a_{assumption}$ (in)	1.75
Y_2 (in)	3.5
Wide Flange Shape (Table 3-19)	W21x55
φM_n (ft-Kip)	642
$\varphi M_n \geq M_u$	Okay
Plastic Netural Axis Location	7

ΣQ_n (Kip) [Table 3-19]	203	
Q_n (Kip/Studs) [Table 3-21]	17.1	
beff (in) ≤	47.25	Controls
	147	
a (in)	1.685	
$a < a_{assumption}$	Conservative	
# Shear Studs	11.87	12.0
Total # Shear Studs	24.0	

Load Information	
Member Size	W21x55
φM_n (ft-Kip) [Table 3-2]	473
$w_{conc+deck}$ (psf)	42
w_{beam} (Kip/ft)	0.055
$w_{construction\ live}$ (psf)	20
w_{DL} (Kip/ft)	0.386
w_{LL} (Kip/ft)	0.158
w_u (Kip/ft)	0.71
M_u (ft-Kip)	215
$\varphi M_n \geq M_u$	Okay

Deflection During Construction	
L/360	1.633
w_{DL} (Kip/ft)	0.386
$I_{required}$ (in^4)	1056
I_{member} (in^4)	1140
$I_{member} > I_{required}$	Okay

Live Load Deflection	
w_{LL} (Kip/ft)	0.158
I_{LB} (in^4) [Table 3-20]	1770
Δ (in)	0.398
Δ < L/360	Okay

Total Deflection	
L/240	2.45
w_u (Kip/ft)	0.71
Δ (in)	1.99
Δ < L/240	Okay

Member	W21x55
Shear Studs	24.0

Calculation 1: Preliminary Roof Composite Steel Beams

Level	3rd Floor
Member (Grid Location)	49' Int.
Slab Thickness (in)	5.25
f'_c (psi)	3000
Length (ft)	49
Tributary Area (ft)	7.875
Dead Load (psf)	0
Super Imposed Load (psf)	15
Live Load (psf)	50
w_u (psf)	108
M_u (ft-Kip)	254

$a_{assumption}$ (in)	1.75
Y_2 (in)	3.5
Wide Flange Shape (Table 3-19)	W21x55
φM_n (ft-Kip)	642
$\varphi M_n \geq M_u$	Okay
Plastic Netural Axis Location	7

ΣQ_n (Kip) [Table 3-19]	203	
Q_n (Kip/Studs) [Table 3-21]	17.1	
beff (in) ≤	47.25	Controls
	147	
a (in)	1.685	
$a < a_{assumption}$	Conservative	
# Shear Studs	11.87	12.0
Total # Shear Studs	24.0	

Load Information	
Member Size	W21x55
φM_n (ft-Kip) [Table 3-2]	473
$w_{conc+deck}$ (psf)	42
w_{beam} (Kip/ft)	0.055
$w_{construction\ live}$ (psf)	20
w_{DL} (Kip/ft)	0.386
w_{LL} (Kip/ft)	0.158
w_u (Kip/ft)	0.71
M_u (ft-Kip)	215
$\varphi M_n \geq M_u$	Okay

Deflection During Construction	
L/360	1.633
w_{DL} (Kip/ft)	0.386
$I_{required}$ (in^4)	1056
I_{member} (in^4)	1140
$I_{member} > I_{required}$	Okay

Live Load Deflection	
w_{LL} (Kip/ft)	0.158
I_{LB} (in^4) [Table 3-20]	1770
Δ (in)	0.398
Δ < L/360	Okay

Total Deflection	
L/240	2.45
w_u (Kip/ft)	0.71
Δ (in)	1.99
Δ < L/240	Okay

Member	W21x55
Shear Studs	24.0

Calculation 2: Preliminary 3rd Floor Composite Beams

Level	2nd Level
Member (Grid Location)	49' Int.
Slab Thickness (in)	5.25
f'_c (psi)	3000
Length (ft)	49
Tributary Area (ft)	7.875
Dead Load (psf)	0
Super Imposed Load (psf)	15
Live Load (psf)	60
w_u (psf)	124
M_u (ft-Kip)	292

$a_{assumption}$ (in)	1.75
Y_2 (in)	3.5
Wide Flange Shape (Table 3-19)	W21x55
φM_n (ft-Kip)	642
$\varphi M_n \geq M_u$	Okay
Plastic Netural Axis Location	7

ΣQ_n (Kip) [Table 3-19]	203	
Q_n (Kip/Studs) [Table 3-21]	17.1	
beff (in) ≤	47.25	Controls
	147	
a (in)	1.685	
$a < a_{assumption}$	Conservative	
# Shear Studs	11.87	12.0
Total # Shear Studs	24.0	

Load Information	
Member Size	W21x55
φM_n (ft-Kip) [Table 3-2]	473
$w_{conc+deck}$ (psf)	42
w_{beam} (Kip/ft)	0.055
$w_{construction\ live}$ (psf)	20
w_{DL} (Kip/ft)	0.386
w_{LL} (Kip/ft)	0.158
w_u (Kip/ft)	0.71
M_u (ft-Kip)	215
$\varphi M_n \geq M_u$	Okay

Deflection During Construction	
L/360	1.633
w_{DL} (Kip/ft)	0.386
$I_{required}$ (in^4)	1056
I_{member} (in^4)	1140
$I_{member} > I_{required}$	Okay

Live Load Deflection	
w_{LL} (Kip/ft)	0.158
I_{LB} (in^4) [Table 3-20]	1770
Δ (in)	0.398
Δ < L/360	Okay

Total Deflection	
L/240	2.45
w_u (Kip/ft)	0.71
Δ (in)	1.99
Δ < L/240	Okay

Member	W21x55
Shear Studs	24.0

Calculation 3: Preliminary 2nd Floor Composite Beams

Level	1st Level	
Member (Grid Location)	27.5' Ext.	
Slab Thickness (in)	5.25	
f'c (psi)	3000	
Length (ft)	27.5	
Tributary Area (ft)	3.9375	
Dead Load (psf)	0	
Super Imposed Load (psf)	15	
Live Load (psf)	50	
w_u (psf)	108	
M_u (ft-Kip)	40	
$a_{assumption}$ (in)	1.25	
Y_2 (in)	4	
Wide Flange Shape (Table 3-19)	W12x16	
φM_n (ft-Kip)	113	
$\varphi M_n \geq M_u$	Okay	
Plastic Netural Axis Location	7	
ΣQ_n (Kip) [Table 3-19]	58.9	
Q_n (Kip/Studs) [Table 3-21]	17.1	
beff (in) ≤	23.625	Controls
	82.5	
a (in)	0.978	
a < $a_{assumption}$	Conservative	
# Shear Studs	3.44	4.0
Total # Shear Studs	8.0	

Load Information	
Member Size	W12x16
φM_n (ft-Kip) [Table 3-2]	75.4
$w_{conc+deck}$ (psf)	42
w_{beam} (Kip/ft)	0.016
$w_{construction\ live}$ (psf)	20
w_{DL} (Kip/ft)	0.181
w_{LL} (Kip/ft)	0.079
w_u (Kip/ft)	0.34
M_u (ft-Kip)	32
$\varphi M_n \geq M_u$	Okay

Deflection During Construction	
L/360	0.917
w_{DL} (Kip/ft)	0.181
$I_{required}$ (in^4)	88
I_{member} (in^4)	103
$I_{member} > I_{required}$	Okay

Live Load Deflection	
w_{LL} (Kip/ft)	0.079
I_{LB} (in^4) [Table 3-20]	197
Δ (in)	0.177
Δ < L/360	Okay

Total Deflection	
L/240	1.375
w_u (Kip/ft)	0.34
Δ (in)	0.85
Δ < L/240	Okay

Member	W12x16
Shear Studs	8.0

Calculation 4: Preliminary 1st Floor Composite Beams

Level	Mezzanine	
Member (Grid Location)	20' Ext.	
Slab Thickness (in)	5.25	
f'c (psi)	3000	
Length (ft)	20	
Tributary Area (ft)	3.9375	
Dead Load (psf)	0	
Super Imposed Load (psf)	15	
Live Load (psf)	60	
w_u (psf)	124	
M_u (ft-Kip)	24	
$a_{assumption}$ (in)	1.25	
Y_2 (in)	4	
Wide Flange Shape (Table 3-19)	W12x16	
φM_n (ft-Kip)	113	
$\varphi M_n \geq M_u$	Okay	
Plastic Netural Axis Location	7	
ΣQ_n (Kip) [Table 3-19]	58.9	
Q_n (Kip/Studs) [Table 3-21]	17.1	
beff (in) ≤	23.625	Controls
	60	
a (in)	0.978	
a < $a_{assumption}$	Conservative	
# Shear Studs	3.44	4.0
Total # Shear Studs	8.0	

Load Information	
Member Size	W12x16
φM_n (ft-Kip) [Table 3-2]	75.4
$w_{conc+deck}$ (psf)	42
w_{beam} (Kip/ft)	0.016
$w_{construction\ live}$ (psf)	20
w_{DL} (Kip/ft)	0.181
w_{LL} (Kip/ft)	0.079
w_u (Kip/ft)	0.34
M_u (ft-Kip)	17
$\varphi M_n \geq M_u$	Okay

Deflection During Construction	
L/360	0.667
w_{DL} (Kip/ft)	0.181
$I_{required}$ (in^4)	34
I_{member} (in^4)	103
$I_{member} > I_{required}$	Okay

Live Load Deflection	
w_{LL} (Kip/ft)	0.079
I_{LB} (in^4) [Table 3-20]	197
Δ (in)	0.050
Δ < L/360	Okay

Total Deflection	
L/240	1
w_u (Kip/ft)	0.34
Δ (in)	0.24
Δ < L/240	Okay

Member	W12x16
Shear Studs	8.0

Calculation 5: Preliminary Mezzanine Composite Beams

Level	Roof	
Grid Location	G	
Slab Thickness (in)	5.25	
f'_c (psi)	3000	
w_{u-Top} (psf)	108	
w_{u-Bot} (psf)	108	
Trib. Area of $Beam_{Top}$ (ft)	7.875	
Trib. Area of $Beam_{Bot}$ (ft)	7.875	
Beam $Length_{Top}$ (ft)	49	
Beam $Length_{Bot}$ (ft)	49	
Girder Length (ft)	31.5	
Gider Tributary Area (ft)	49	
P_{1-Top} (Kips)	20.83725	
P_{1-Bot} (Kips)	20.84	
$P_{1-Total}$ (Kips)	41.67	# 9 From Table 3-23
P_{2-Top} (Kips)	20.84	
P_{2-Bot} (Kips)	20.84	
$P_{2-Total}$ (Kips)	41.67	# 9 From Table 3-23
P_{3-Top} (Kips)	20.84	
P_{3-Bot} (Kips)	20.84	
$P_{3-Total}$ (Kips)	41.67	# 7 From Table 3-23
P_{Total} (Kips)	125.02	
R_A (Kips)	62.51	
R_B (Kips)	62.51	
a (ft)	10.5	
l/2 (ft)	15.75	
M_{u-1} (ft-Kip)	437.58	
M_{u-2} (ft-Kip)	656.37	
$M_{u-Total}$ (ft-Kip)	1093.96	

$a_{assumption}$ (in)	1.75	
Y_2 (in)	3.5	
Wide Flange Shape (Table 3-19)	W24x94	
ϕM_n (ft-Kip)	1280	
$\phi M_n \geq M_u$	Okay	
Plastic Netural Axis Location	7	

ΣQ_n (Kip) [Table 3-19]	346	
Q_n (Kip/Studs) [Table 3-21]	17.1	
beff (in) ≤	294	
	94.5	Controls
a (in)	1.436	
$a < a_{assumption}$	Conservative	
# Shear Studs	20.23	21.00
Total # Shear Studs	42.0	

Load Information	
Member Size	W24x94
ϕM_n (ft-Kip) [Table 3-2]	953
$w_{conc+deck}$ (psf)	42
w_{beam} (Kip/ft)	0.094
$w_{construction\ live}$ (psf)	20
w_{DL} (Kip/ft)	2.152
w_{LL} (Kip/ft)	0.980
w_u (Kip/ft)	4.15
M_u (ft-Kip)	515
$\phi M_n \geq M_u$	Okay

Deflection During Construction	
L/360	1.050
w_{DL} (Kip/ft)	2.152
$I_{required}$ (in^4)	1566
I_{member} (in^4)	2700
$I_{member} > I_{required}$	Okay

Live Load Deflection	
w_{LL} (Kip/ft)	0.980
I_{LB} (in^4) [Table 3-20]	4060
Δ (in)	0.184
Δ < L/360	Okay

Total Deflection	
L/240	1.575
w_u (Kip/ft)	4.15
Δ (in)	0.86
Δ < L/240	Okay

Member	W24x94
Shear Studs	42.0

Level	Roof	
Grid Location	H	
Slab Thickness (in)	5.25	
f'_c (psi)	3000	
w_{u-Top} (psf)	108	
w_{u-Bot} (psf)	108	
Trib. Area of $Beam_{Top}$ (ft)	7.875	
Trib. Area of $Beam_{Bot}$ (ft)	7.875	
Beam $Length_{Top}$ (ft)	49	
Beam $Length_{Bot}$ (ft)	40	
Girder Length (ft)	31.5	
Gider Tributary Area (ft)	44.5	
P_{1-Top} (Kips)	20.83725	
P_{1-Bot} (Kips)	17.01	
$P_{1-Total}$ (Kips)	37.85	# 9 From Table 3-23
P_{2-Top} (Kips)	20.84	
P_{2-Bot} (Kips)	17.01	
$P_{2-Total}$ (Kips)	37.85	# 9 From Table 3-23
P_{3-Top} (Kips)	20.84	
P_{3-Bot} (Kips)	17.01	
$P_{3-Total}$ (Kips)	37.85	# 7 From Table 3-23
P_{Total} (Kips)	113.54	
R_A (Kips)	56.77	
R_B (Kips)	56.77	
a (ft)	10.5	
l/2 (ft)	15.75	
M_{u-1} (ft-Kip)	397.40	
M_{u-2} (ft-Kip)	596.09	
$M_{u-Total}$ (ft-Kip)	993.49	

$a_{assumption}$ (in)	1.75	
Y_2 (in)	3.5	
Wide Flange Shape (Table 3-19)	W24x76	
ϕM_n (ft-Kip)	1080	
$\phi M_n \geq M_u$	Okay	
Plastic Netural Axis Location	6	

ΣQ_n (Kip) [Table 3-19]	393	
Q_n (Kip/Studs) [Table 3-21]	17.1	
beff (in) ≤	267	
	94.5	Controls
a (in)	1.631	
$a < a_{assumption}$	Conservative	
# Shear Studs	22.98	23.00
Total # Shear Studs	46.0	

Load Information	
Member Size	W24x76
ϕM_n (ft-Kip) [Table 3-2]	750
$w_{conc+deck}$ (psf)	42
w_{beam} (Kip/ft)	0.076
$w_{construction\ live}$ (psf)	20
w_{DL} (Kip/ft)	1.945
w_{LL} (Kip/ft)	0.890
w_u (Kip/ft)	3.76
M_u (ft-Kip)	466
$\phi M_n \geq M_u$	Okay

Deflection During Construction	
L/360	1.050
w_{DL} (Kip/ft)	1.945
$I_{required}$ (in^4)	1415
I_{member} (in^4)	2100
$I_{member} > I_{required}$	Okay

Live Load Deflection	
w_{LL} (Kip/ft)	0.890
I_{LB} (in^4) [Table 3-20]	3490
Δ (in)	0.195
Δ < L/360	Okay

Total Deflection	
L/240	1.575
w_u (Kip/ft)	3.76
Δ (in)	0.91
Δ < L/240	Okay

Member	W24x76
Shear Studs	46.0

Calculation 6: Preliminary Roof Composite Girders

Level	3rd Floor	
Grid Location	G	
Slab Thickness (in)	5.25	
f'_c (psi)	3000	
$w_{u\text{-}Top}$ (psf)	108	
$w_{u\text{-}Bot}$ (psf)	108	
Trib. Area of $Beam_{Top}$ (ft)	7.875	
Trib. Area of $Beam_{Bot}$ (ft)	7.875	
Beam $Length_{Top}$ (ft)	49	
Beam $Length_{Bot}$ (ft)	49	
Girder Length (ft)	31.5	
Gider Tributary Area (ft)	49	
$P_{1\text{-}Top}$ (Kips)	20.83725	
$P_{1\text{-}Bot}$ (Kips)	20.84	
$P_{1\text{-}Total}$ (Kips)	41.67	# 9 From Table 3-23
$P_{2\text{-}Top}$ (Kips)	20.84	
$P_{2\text{-}Bot}$ (Kips)	20.84	
$P_{2\text{-}Total}$ (Kips)	41.67	# 9 From Table 3-23
$P_{3\text{-}Top}$ (Kips)	20.84	
$P_{3\text{-}Bot}$ (Kips)	20.84	
$P_{3\text{-}Total}$ (Kips)	41.67	# 7 From Table 3-23
P_{Total} (Kips)	125.02	
R_A (Kips)	62.51	
R_B (Kips)	62.51	
a (ft)	10.5	
l/2 (ft)	15.75	
$M_{u\text{-}1}$ (ft-Kip)	437.58	
$M_{u\text{-}2}$ (ft-Kip)	656.37	
$M_{u\text{-}Total}$ (ft-Kip)	1093.96	

$a_{assumption}$ (in)	1.75
Y_2 (in)	3.5
Wide Flange Shape (Table 3-19)	W24x94
ϕM_n (ft-Kip)	1280
$\phi M_n \geq M_u$	Okay
Plastic Netural Axis Location	7

ΣQ_n (Kip) [Table 3-19]	346	
Q_n (Kip/Studs) [Table 3-21]	17.1	
beff (in) ≤	294	
	94.5	Controls
a (in)	1.436	
a < $a_{assumption}$	Conservative	
# Shear Studs	20.23	21.00
Total # Shear Studs	42.0	

Load Information	
Member Size	W24x94
ϕM_n (ft-Kip) [Table 3-2]	963
$w_{conc+deck}$ (psf)	42
w_{beam} (Kip/ft)	0.094
$w_{construction\ live}$ (psf)	20
w_{DL} (Kip/ft)	2.152
w_{LL} (Kip/ft)	0.980
w_u (Kip/ft)	4.15
M_u (ft-Kip)	515
$\phi M_n \geq M_u$	Okay

Deflection During Construction	
L/360	1.050
w_{DL} (Kip/ft)	2.152
$I_{required}$ (in^4)	1566
I_{member} (in^4)	2700
$I_{member} > I_{required}$	Okay

Live Load Deflection	
w_{LL} (Kip/ft)	0.980
I_{LB} (in^4) [Table 3-20]	4060
Δ (in)	0.184
Δ < L/360	Okay

Total Deflection	
L/240	1.575
w_u (Kip/ft)	4.15
Δ (in)	0.86
Δ < L/240	Okay

Member	W24x94
Shear Studs	42.0

Level	3rd Floor	
Grid Location	H	
Slab Thickness (in)	5.25	
f'_c (psi)	3000	
$w_{u\text{-}Top}$ (psf)	108	
$w_{u\text{-}Bot}$ (psf)	108	
Trib. Area of $Beam_{Top}$ (ft)	7.875	
Trib. Area of $Beam_{Bot}$ (ft)	7.875	
Beam $Length_{Top}$ (ft)	49	
Beam $Length_{Bot}$ (ft)	40	
Girder Length (ft)	31.5	
Gider Tributary Area (ft)	44.5	
$P_{1\text{-}Top}$ (Kips)	20.83725	
$P_{1\text{-}Bot}$ (Kips)	17.01	
$P_{1\text{-}Total}$ (Kips)	37.85	# 9 From Table 3-23
$P_{2\text{-}Top}$ (Kips)	20.84	
$P_{2\text{-}Bot}$ (Kips)	17.01	
$P_{2\text{-}Total}$ (Kips)	37.85	# 9 From Table 3-23
$P_{3\text{-}Top}$ (Kips)	20.84	
$P_{3\text{-}Bot}$ (Kips)	17.01	
$P_{3\text{-}Total}$ (Kips)	37.85	# 7 From Table 3-23
P_{Total} (Kips)	113.54	
R_A (Kips)	56.77	
R_B (Kips)	56.77	
a (ft)	10.5	
l/2 (ft)	15.75	
$M_{u\text{-}1}$ (ft-Kip)	397.40	
$M_{u\text{-}2}$ (ft-Kip)	596.09	
$M_{u\text{-}Total}$ (ft-Kip)	993.49	

$a_{assumption}$ (in)	1.75
Y_2 (in)	3.5
Wide Flange Shape (Table 3-19)	W24x76
ϕM_n (ft-Kip)	1080
$\phi M_n \geq M_u$	Okay
Plastic Netural Axis Location	6

ΣQ_n (Kip) [Table 3-19]	393	
Q_n (Kip/Studs) [Table 3-21]	17.1	
beff (in) ≤	267	
	94.5	Controls
a (in)	1.631	
a < $a_{assumption}$	Conservative	
# Shear Studs	22.98	23.00
Total # Shear Studs	46.0	

Load Information	
Member Size	W24x76
ϕM_n (ft-Kip) [Table 3-2]	750
$w_{conc+deck}$ (psf)	42
w_{beam} (Kip/ft)	0.076
$w_{construction\ live}$ (psf)	20
w_{DL} (Kip/ft)	1.945
w_{LL} (Kip/ft)	0.890
w_u (Kip/ft)	3.76
M_u (ft-Kip)	466
$\phi M_n \geq M_u$	Okay

Deflection During Construction	
L/360	1.050
w_{DL} (Kip/ft)	1.945
$I_{required}$ (in^4)	1415
I_{member} (in^4)	2100
$I_{member} > I_{required}$	Okay

Live Load Deflection	
w_{LL} (Kip/ft)	0.890
I_{LB} (in^4) [Table 3-20]	3490
Δ (in)	0.195
Δ < L/360	Okay

Total Deflection	
L/240	1.575
w_u (Kip/ft)	3.76
Δ (in)	0.91
Δ < L/240	Okay

Member	W24x76
Shear Studs	46.0

Calculation 7: Preliminary 3rd Floor Composite Girders

Level	2nd Floor	
Grid Location	G	
Slab Thickness (in)	5.25	
f'_c (psi)	3000	
$w_{u\text{-}Top}$ (psf)	124	
$w_{u\text{-}Bot}$ (psf)	124	
Trib. Area of Beam_{Top} (ft)	7.875	
Trib. Area of Beam_{Bot} (ft)	7.875	
Beam Length_{Top} (ft)	49	
Beam Length_{Bot} (ft)	49	
Girder Length (ft)	31.5	
Gider Tributary Area (ft)	49	
$P_{1\text{-}Top}$ (Kips)	23.92425	
$P_{1\text{-}Bot}$ (Kips)	23.92	
$P_{1\text{-}Total}$ (Kips)	47.85	# 9 From Table 3-23
$P_{2\text{-}Top}$ (Kips)	23.92	
$P_{2\text{-}Bot}$ (Kips)	23.92	
$P_{2\text{-}Total}$ (Kips)	47.85	# 9 From Table 3-23
$P_{3\text{-}Top}$ (Kips)	23.92	
$P_{3\text{-}Bot}$ (Kips)	23.92	
$P_{3\text{-}Total}$ (Kips)	47.85	# 7 From Table 3-23
P_{Total} (Kips)	143.55	
R_A (Kips)	71.77	
R_B (Kips)	71.77	
a (ft)	10.5	
l/2 (ft)	15.75	
$M_{u\text{-}1}$ (ft-Kip)	502.41	
$M_{u\text{-}2}$ (ft-Kip)	753.61	
$M_{u\text{-}Total}$ (ft-Kip)	1256.02	

$a_{assumption}$ (in)	1.75
Y_2 (in)	3.5
Wide Flange Shape (Table 3-19)	W24x94
φM_n (ft-Kip)	1280
$\varphi M_n \geq M_u$	Okay
Plastic Netural Axis Location	7

ΣQ_n (Kip) [Table 3-19]	346	
Q_n (Kip/Studs) [Table 3-21]	17.1	
beff (in) ≤	294	
	94.5	Controls
a (in)	1.436	
$a < a_{assumption}$	Conservative	
# Shear Studs	20.23	21.00
Total # Shear Studs	42.0	

Load Information	
Member Size	W24x94
φM_n (ft-Kip) [Table 3-2]	953
$w_{conc+deck}$ (psf)	42
w_{beam} (Kip/ft)	0.094
$w_{construction\ live}$ (psf)	20
w_{DL} (Kip/ft)	2.152
w_{LL} (Kip/ft)	0.980
w_u (Kip/ft)	4.15
M_u (ft-Kip)	515
$\varphi M_n \geq M_u$	Okay

Deflection During Construction	
L/360	1.050
w_{DL} (Kip/ft)	2.152
$I_{required}$ (in^4)	1566
I_{member} (in^4)	2700
$I_{member} > I_{required}$	Okay

Live Load Deflection	
w_{LL} (Kip/ft)	0.980
I_{LB} (in^4) [Table 3-20]	4060
Δ (in)	0.184
Δ < L/360	Okay

Total Deflection	
L/240	1.575
w_u (Kip/ft)	4.15
Δ (in)	0.86
Δ < L/240	Okay

Member	W24x94
Shear Studs	42.0

Level	2nd Floor	
Grid Location	H	
Slab Thickness (in)	5.25	
f'_c (psi)	3000	
$w_{u\text{-}Top}$ (psf)	124	
$w_{u\text{-}Bot}$ (psf)	124	
Trib. Area of Beam_{Top} (ft)	7.875	
Trib. Area of Beam_{Bot} (ft)	7.875	
Beam Length_{Top} (ft)	49	
Beam Length_{Bot} (ft)	40	
Girder Length (ft)	31.5	
Gider Tributary Area (ft)	44.5	
$P_{1\text{-}Top}$ (Kips)	23.92425	
$P_{1\text{-}Bot}$ (Kips)	19.53	
$P_{1\text{-}Total}$ (Kips)	43.45	# 9 From Table 3-23
$P_{2\text{-}Top}$ (Kips)	23.92	
$P_{2\text{-}Bot}$ (Kips)	19.53	
$P_{2\text{-}Total}$ (Kips)	43.45	# 9 From Table 3-23
$P_{3\text{-}Top}$ (Kips)	23.92	
$P_{3\text{-}Bot}$ (Kips)	19.53	
$P_{3\text{-}Total}$ (Kips)	43.45	# 7 From Table 3-23
P_{Total} (Kips)	130.36	
R_A (Kips)	65.18	
R_B (Kips)	65.18	
a (ft)	10.5	
l/2 (ft)	15.75	
$M_{u\text{-}1}$ (ft-Kip)	456.27	
$M_{u\text{-}2}$ (ft-Kip)	684.40	
$M_{u\text{-}Total}$ (ft-Kip)	1140.67	

$a_{assumption}$ (in)	1.75
Y_2 (in)	3.5
Wide Flange Shape (Table 3-19)	W24x94
φM_n (ft-Kip)	1270
$\varphi M_n \geq M_u$	Okay
Plastic Netural Axis Location	7

ΣQ_n (Kip) [Table 3-19]	346	
Q_n (Kip/Studs) [Table 3-21]	17.1	
beff (in) ≤	267	
	94.5	Controls
a (in)	1.436	
$a < a_{assumption}$	Conservative	
# Shear Studs	20.23	21.00
Total # Shear Studs	42.0	

Load Information	
Member Size	W24x94
φM_n (ft-Kip) [Table 3-2]	953
$w_{conc+deck}$ (psf)	42
w_{beam} (Kip/ft)	0.094
$w_{construction\ live}$ (psf)	20
w_{DL} (Kip/ft)	1.963
w_{LL} (Kip/ft)	0.890
w_u (Kip/ft)	3.78
M_u (ft-Kip)	469
$\varphi M_n \geq M_u$	Okay

Deflection During Construction	
L/360	1.050
w_{DL} (Kip/ft)	1.963
$I_{required}$ (in^4)	1428
I_{member} (in^4)	2700
$I_{member} > I_{required}$	Okay

Live Load Deflection	
w_{LL} (Kip/ft)	0.890
I_{LB} (in^4) [Table 3-20]	4060
Δ (in)	0.167
Δ < L/360	Okay

Total Deflection	
L/240	1.575
w_u (Kip/ft)	3.78
Δ (in)	0.78
Δ < L/240	Okay

Member	W24x94
Shear Studs	42.0

Calculation 8: Preliminary 2nd Floor Composite Girders

Level	1st Floor	
Grid Location	G	
Slab Thickness (in)	5.25	
f'_c (psi)	3000	
$w_{u\text{-}Top}$ (psf)	124	
$w_{u\text{-}Bot}$ (psf)	124	
Trib. Area of $Beam_{Top}$ (ft)	7.875	
Trib. Area of $Beam_{Bot}$ (ft)	7.875	
$Beam Length_{Top}$ (ft)	49	
$Beam Length_{Bot}$ (ft)	49	
Girder Length (ft)	31.5	
Gider Tributary Area (ft)	49	
$P_{1\text{-}Top}$ (Kips)	23.92425	
$P_{1\text{-}Bot}$ (Kips)	23.92	
$P_{1\text{-}Total}$ (Kips)	47.85	# 9 From Table 3-23
$P_{2\text{-}Top}$ (Kips)	23.92	
$P_{2\text{-}Bot}$ (Kips)	23.92	
$P_{2\text{-}Total}$ (Kips)	47.85	# 9 From Table 3-23
$P_{3\text{-}Top}$ (Kips)	23.92	
$P_{3\text{-}Bot}$ (Kips)	23.92	
$P_{3\text{-}Total}$ (Kips)	47.85	# 7 From Table 3-23
P_{Total} (Kips)	143.55	
R_A (Kips)	71.77	
R_B (Kips)	71.77	
a (ft)	10.5	
l/2 (ft)	15.75	
$M_{u\text{-}1}$ (ft-Kip)	502.41	
$M_{u\text{-}2}$ (ft-Kip)	753.61	
$M_{u\text{-}Total}$ (ft-Kip)	1256.02	
$a_{assumption}$ (in)	1.75	
Y_2 (in)	3.5	
Wide Flange Shape (Table 3-19)	W24x94	
ϕM_n (ft-Kip)	1280	
$\phi M_n \geq M_u$	Okay	
Plastic Netural Axis Location	7	
ΣQ_n (Kip) [Table 3-19]	348	
Q_n (Kip/Studs) [Table 3-21]	17.1	
beff (in) ≤	294	
	94.5	Controls
a (in)	1.436	
$a < a_{assumption}$	Conservative	
# Shear Studs	20.23	21.00
Total # Shear Studs	42.0	

Load Information	
Member Size	W24x94
ϕM_n (ft-Kip) [Table 3-2]	953
$W_{conc+deck}$ (psf)	42
W_{beam} (Kip/ft)	0.094
$W_{construction\ live}$ (psf)	20
W_{DL} (Kip/ft)	2.152
W_{LL} (Kip/ft)	0.980
W_u (Kip/ft)	4.15
M_u (ft-Kip)	515
$\phi M_n \geq M_u$	Okay
Deflection During Construction	
L/360	1.050
W_{DL} (Kip/ft)	2.152
$I_{required}$ (in^4)	1566
I_{member} (in^4)	2700
$I_{member} > I_{required}$	Okay
Live Load Deflection	
W_{LL} (Kip/ft)	0.980
I_{LB} (in^4) [Table 3-20]	4060
Δ (in)	0.184
Δ < L/360	Okay
Total Deflection	
L/240	1.575
W_u (Kip/ft)	4.15
Δ (in)	0.86
Δ < L/240	Okay
Member	W24x94
Shear Studs	42.0

Level	1st Floor	
Grid Location	H	
Slab Thickness (in)	5.25	
f'_c (psi)	3000	
$w_{u\text{-}Top}$ (psf)	124	
$w_{u\text{-}Bot}$ (psf)	124	
Trib. Area of $Beam_{Top}$ (ft)	7.875	
Trib. Area of $Beam_{Bot}$ (ft)	7.875	
$Beam Length_{Top}$ (ft)	49	
$Beam Length_{Bot}$ (ft)	40	
Girder Length (ft)	31.5	
Gider Tributary Area (ft)	44.5	
$P_{1\text{-}Top}$ (Kips)	23.92425	
$P_{1\text{-}Bot}$ (Kips)	19.53	
$P_{1\text{-}Total}$ (Kips)	43.45	# 9 From Table 3-23
$P_{2\text{-}Top}$ (Kips)	23.92	
$P_{2\text{-}Bot}$ (Kips)	19.53	
$P_{2\text{-}Total}$ (Kips)	43.45	# 9 From Table 3-23
$P_{3\text{-}Top}$ (Kips)	23.92	
$P_{3\text{-}Bot}$ (Kips)	19.53	
$P_{3\text{-}Total}$ (Kips)	43.45	# 7 From Table 3-23
P_{Total} (Kips)	130.36	
R_A (Kips)	65.18	
R_B (Kips)	65.18	
a (ft)	10.5	
l/2 (ft)	15.75	
$M_{u\text{-}1}$ (ft-Kip)	456.27	
$M_{u\text{-}2}$ (ft-Kip)	684.40	
$M_{u\text{-}Total}$ (ft-Kip)	1140.67	
$a_{assumption}$ (in)	1.75	
Y_2 (in)	3.5	
Wide Flange Shape (Table 3-19)	W24x94	
ϕM_n (ft-Kip)	1270	
$\phi M_n \geq M_u$	Okay	
Plastic Netural Axis Location	7	
ΣQ_n (Kip) [Table 3-19]	346	
Q_n (Kip/Studs) [Table 3-21]	17.1	
beff (in) ≤	267	
	94.5	Controls
a (in)	1.436	
$a < a_{assumption}$	Conservative	
# Shear Studs	20.23	21.00
Total # Shear Studs	42.0	

Load Information	
Member Size	W24x94
ϕM_n (ft-Kip) [Table 3-2]	953
$W_{conc+deck}$ (psf)	42
W_{beam} (Kip/ft)	0.094
$W_{construction\ live}$ (psf)	20
W_{DL} (Kip/ft)	1.963
W_{LL} (Kip/ft)	0.890
W_u (Kip/ft)	3.78
M_u (ft-Kip)	469
$\phi M_n \geq M_u$	Okay
Deflection During Construction	
L/360	1.050
W_{DL} (Kip/ft)	1.963
$I_{required}$ (in^4)	1428
I_{member} (in^4)	2700
$I_{member} > I_{required}$	Okay
Live Load Deflection	
W_{LL} (Kip/ft)	0.890
I_{LB} (in^4) [Table 3-20]	4060
Δ (in)	0.167
Δ < L/360	Okay
Total Deflection	
L/240	1.575
W_u (Kip/ft)	3.78
Δ (in)	0.78
Δ < L/240	Okay
Member	W24x94
Shear Studs	42.0

Calculation 9: Preliminary 1st Floor Composite Girders

Level	Mezzanine	
Grid Location	G	
Slab Thickness (in)	5.25	
f'_c (psi)	3000	
$w_{u\text{-}Top}$ (psf)	124	
$w_{u\text{-}Bot}$ (psf)	124	
Trib. Area of $Beam_{Top}$ (ft)	7.875	
Trib. Area of $Beam_{Bot}$ (ft)	7.875	
Beam $Length_{Top}$ (ft)	49	
Beam $Length_{Bot}$ (ft)	49	
Girder Length (ft)	31.5	
Gider Tributary Area (ft)	49	
$P_{1\text{-}Top}$ (Kips)	23.92425	
$P_{1\text{-}Bot}$ (Kips)	23.92	
$P_{1\text{-}Total}$ (Kips)	47.85	# 9 From Table 3-23
$P_{2\text{-}Top}$ (Kips)	23.92	
$P_{2\text{-}Bot}$ (Kips)	23.92	
$P_{2\text{-}Total}$ (Kips)	47.85	# 9 From Table 3-23
$P_{3\text{-}Top}$ (Kips)	23.92	
$P_{3\text{-}Bot}$ (Kips)	23.92	
$P_{3\text{-}Total}$ (Kips)	47.85	# 7 From Table 3-23
P_{Total} (Kips)	143.55	
R_A (Kips)	71.77	
R_B (Kips)	71.77	
a (ft)	10.5	
l/2 (ft)	15.75	
$M_{u\text{-}1}$ (ft-Kip)	502.41	
$M_{u\text{-}2}$ (ft-Kip)	753.61	
$M_{u\text{-}Total}$ (ft-Kip)	1256.02	

$a_{assumption}$ (in)	1.75
Y_2 (in)	3.5
Wide Flange Shape (Table 3-19)	W24x94
ϕM_n (ft-Kip)	1280
$\phi M_n \geq M_u$	Okay
Plastic Netural Axis Location	7

ΣQ_n (Kip) [Table 3-19]	346	
Q_n (Kip/Studs) [Table 3-21]	17.1	
beff (in) ≤	294	
	94.5	Controls
a (in)	1.436	
a < $a_{assumption}$	Conservative	
# Shear Studs	20.23	21.00
Total # Shear Studs	42.0	

Load Information	
Member Size	W24x94
ϕM_n (ft-Kip) [Table 3-2]	953
$w_{conc+deck}$ (psf)	42
w_{beam} (Kip/ft)	0.094
$w_{construction\ live}$ (psf)	20
w_{DL} (Kip/ft)	2.152
w_{LL} (Kip/ft)	0.980
w_u (Kip/ft)	4.15
M_u (ft-Kip)	515
$\phi M_n \geq M_u$	Okay

Deflection During Construction	
L/360	1.050
w_{DL} (Kip/ft)	2.152
$I_{required}$ (in^4)	1566
I_{member} (in^4)	2700
$I_{member} > I_{required}$	Okay

Live Load Deflection	
w_{LL} (Kip/ft)	0.980
I_{LB} (in^4) [Table 3-20]	4060
Δ (in)	0.184
Δ < L/360	Okay

Total Deflection	
L/240	1.575
w_u (Kip/ft)	4.15
Δ (in)	0.86
Δ < L/240	Okay

Member	W24x94
Shear Studs	42.0

Level	Mezzanine	
Grid Location	H	
Slab Thickness (in)	5.25	
f'_c (psi)	3000	
$w_{u\text{-}Top}$ (psf)	124	
$w_{u\text{-}Bot}$ (psf)	124	
Trib. Area of $Beam_{Top}$ (ft)	7.875	
Trib. Area of $Beam_{Bot}$ (ft)	7.875	
Beam $Length_{Top}$ (ft)	49	
Beam $Length_{Bot}$ (ft)	40	
Girder Length (ft)	31.5	
Gider Tributary Area (ft)	44.5	
$P_{1\text{-}Top}$ (Kips)	23.92425	
$P_{1\text{-}Bot}$ (Kips)	19.53	
$P_{1\text{-}Total}$ (Kips)	43.45	# 9 From Table 3-23
$P_{2\text{-}Top}$ (Kips)	23.92	
$P_{2\text{-}Bot}$ (Kips)	19.53	
$P_{2\text{-}Total}$ (Kips)	43.45	# 9 From Table 3-23
$P_{3\text{-}Top}$ (Kips)	23.92	
$P_{3\text{-}Bot}$ (Kips)	19.53	
$P_{3\text{-}Total}$ (Kips)	43.45	# 7 From Table 3-23
P_{Total} (Kips)	130.36	
R_A (Kips)	65.18	
R_B (Kips)	65.18	
a (ft)	10.5	
l/2 (ft)	15.75	
$M_{u\text{-}1}$ (ft-Kip)	456.27	
$M_{u\text{-}2}$ (ft-Kip)	684.40	
$M_{u\text{-}Total}$ (ft-Kip)	1140.67	

$a_{assumption}$ (in)	1.75
Y_2 (in)	3.5
Wide Flange Shape (Table 3-19)	W24x94
ϕM_n (ft-Kip)	1270
$\phi M_n \geq M_u$	Okay
Plastic Netural Axis Location	7

ΣQ_n (Kip) [Table 3-19]	346	
Q_n (Kip/Studs) [Table 3-21]	17.1	
beff (in) ≤	267	
	94.5	Controls
a (in)	1.436	
a < $a_{assumption}$	Conservative	
# Shear Studs	20.23	21.00
Total # Shear Studs	42.0	

Load Information	
Member Size	W24x94
ϕM_n (ft-Kip) [Table 3-2]	953
$w_{conc+deck}$ (psf)	42
w_{beam} (Kip/ft)	0.094
$w_{construction\ live}$ (psf)	20
w_{DL} (Kip/ft)	1.963
w_{LL} (Kip/ft)	0.890
w_u (Kip/ft)	3.78
M_u (ft-Kip)	469
$\phi M_n \geq M_u$	Okay

Deflection During Construction	
L/360	1.050
w_{DL} (Kip/ft)	1.963
$I_{required}$ (in^4)	1428
I_{member} (in^4)	2700
$I_{member} > I_{required}$	Okay

Live Load Deflection	
w_{LL} (Kip/ft)	0.890
I_{LB} (in^4) [Table 3-20]	4060
Δ (in)	0.167
Δ < L/360	Okay

Total Deflection	
L/240	1.575
w_u (Kip/ft)	3.78
Δ (in)	0.78
Δ < L/240	Okay

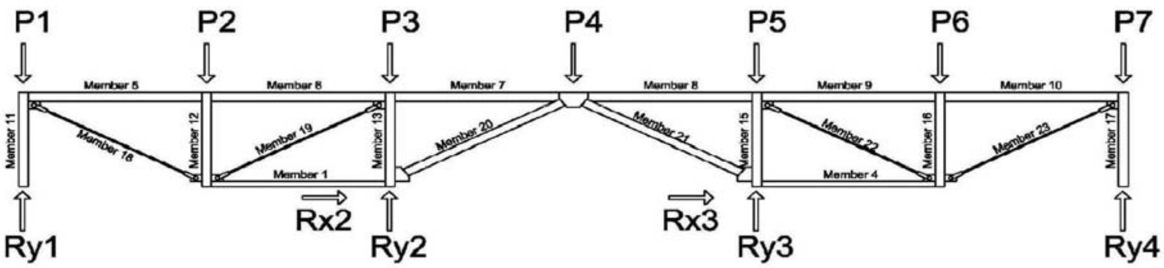
Member	W24x94
Shear Studs	42.0

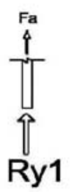
Calculation 10: Preliminary Mezzanine Composite Girders

APPENDIX C: LOADS WITHIN TRUSS 2 MEMBERS

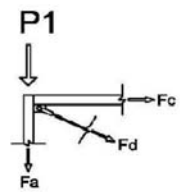
Structure1 Truss 2

P_1	669.69 Kips	R_{y1}	1160.00 Kips	R_{x1}	0 Kips
P_2	1125.06 Kips	R_{y2}	2340.00 Kips	R_{x2}	757.94 Kips
P_3	1125.06 Kips	R_{y3}	2140.00 Kips	R_{x3}	-757.94 Kips
P_4	1125.06 Kips	R_{y4}	750.64 Kips	R_{x4}	0 Kips
P_5	1125.06 Kips				
P_6	835.23 Kips				
P_7	388.98 Kips				

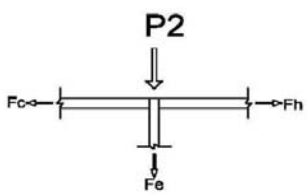




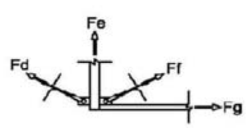
R_{y1} =	1160.00 Kips	
F_a =	-1160.00 Kips	(Compression)
ΣF_y =	0.00	



P_1 =	669.69 Kips	
F_a =	1160.00 Kips	(Compression)
F_{dy} =	490.31 Kips	
F_{dx} =	860.13 Kips	
F_d =	990.06 Kips	(Tension)
F_c =	-860.13 Kips	(Compression)
ΣF_x =	0.00	
ΣF_y =	0.00	

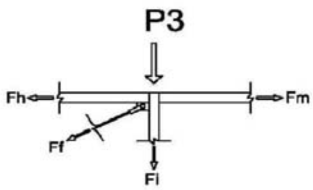


P_2 =	1125.06 Kips	
F_c =	860.13 Kips	(Compression)
F_h =	-860.13 Kips	(Compression)
F_e =	-1125.06 Kips	(Compression)
ΣF_x =	0.00	
ΣF_y =	0.00	

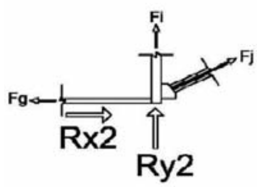


F_e =	1125.06 Kips	(Compression)
F_{dy} =	490.31 Kips	
F_{dx} =	860.13 Kips	
F_{fy} =	634.75 Kips	
F_{fx} =	1113.52 Kips	
F_f =	1281.73 Kips	(Tension)
F_g =	253.38 Kips	(Tension)
ΣF_x =	0.00	
ΣF_y =	0.00	

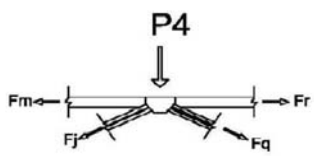
Calculations 1 through 4: Forces in Truss 2 Members



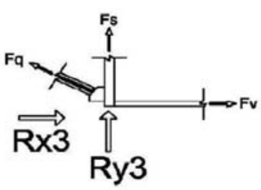
P_3 =	1125.06 Kips	
F_h =	860.13 Kips	(Compression)
F_{fy} =	634.75 Kips	
F_{fx} =	1113.52 Kips	
F_l =	-1759.81 Kips	(Compression)
F_m =	-253.38 Kips	(Compression)
ΣF_x =	0.00	
ΣF_y =	0.00	



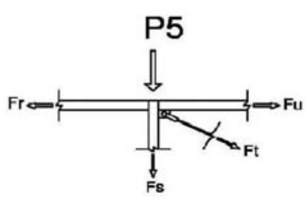
R_{y2} =	2340.00 Kips	
R_{x2} =	757.94 Kips	
F_g =	253.38 Kips	(Tension)
F_l =	-1759.81 Kips	(Compression)
F_{jy} =	-580.19 Kips	
F_{jx} =	-1017.80 Kips	
F_j =	-1171.56 Kips	(Compression)
F_k =	0.00 Kips	
ΣF_x =	0.00	
ΣF_y =	0.00	



P_4 =	1125.06 Kips
F_m =	-253.385 Kips
F_r =	-196.3155 Kips
F_{qy} =	-541.37 Kips
F_{qx} =	-949.7042 Kips
F_{jy} =	-580.19 Kips
F_{jx} =	-1017.803 Kips
ΣF_x =	0.0
ΣF_y =	0.50

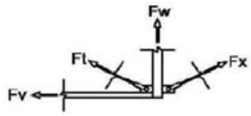


R_{y3} =	2140.00 Kips	
R_{x3} =	-757.94 Kips	
F_v =	196.32 Kips	(Compression)
F_s =	-1598.63 Kips	(Compression)
F_{qy} =	-541.37 Kips	
F_{qx} =	-949.70 Kips	
F_q =	-1093.17 Kips	(Compression)
ΣFx =	0.00	
ΣFy =	0.00	

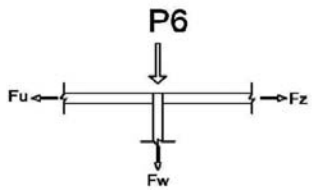


P_5 =	1125.06 Kips	
F_u =	634.45 Kips	(Compression)
F_{ty} =	473.57 Kips	
F_{tx} =	830.762 Kips	
F_s =	-1598.63 Kips	(Compression)
F_r =	-196.32 Kips	(Compression)
ΣF_x =	0.00	
ΣF_y =	0.00	

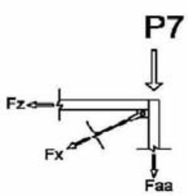
Calculations 5 through 9: Forces in Truss 2 Members



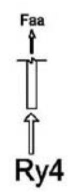
F_w =	835.23 Kips	(Compression)
F_{xy} =	361.66 Kips	
F_{xx} =	634.45 Kips	
F_{ty} =	473.57 Kips	
F_{tx} =	830.76 Kips	
F_t =	956.26 Kips	(Tension)
F_v =	196.32 Kips	(Tension)
ΣF_x =	0.00	
ΣF_y =	0.00	



P_6 =	835.23 Kips	
F_u =	-634.45 Kips	(Compression)
F_w =	-835.23 Kips	(Compression)
F_z =	634.45 Kips	(Compression)
ΣF_x =	0.00	
ΣF_y =	0.00	



P_7 =	388.98 Kips	
F_{aa} =	750.64 Kips	(Compression)
F_{xy} =	361.66 Kips	
F_{xx} =	634.45 Kips	
F_x =	730.29 Kips	(Tension)
F_z =	-634.45 Kips	(Compression)
ΣF_x =	0.00	
ΣF_y =	0.00	



R_{y4} =	750.64 Kips	
F_{aa} =	-750.64 Kips	(Compression)
ΣF_y =	0.00	

Calculations 10 through 13: Forces in Truss 2 Members

Joseph W. Wilcher III	Job No	Sheet No 1	Rev
Software licensed to PSUAE	Part		
Job Title	Ref		
	By	Date 09-Feb-09	Chd
Client	File Loads.std	Date/Time 10-Feb-2009 18:36	

Job Information

	Engineer	Checked	Approved
Name:			
Date:	09-Feb-09		

Structure Type	TRUSS

Number of Nodes	13	Highest Node	14
Number of Elements	21	Highest Beam	24

Number of Basic Load Cases	1
Number of Combination Load Cases	0

Included in this printout are data for:

All	The Whole Structure

Included in this printout are results for load cases:

Type	L/C	Name
Primary	1	LOAD

Reactions

		Horizontal	Vertical	Horizontal	Moment		
Node	L/C	FX (kip)	FY (kip)	FZ (kip)	MX (kip`in)	MY (kip`in)	MZ (kip`in)
1	1:LOAD	0.000	1.16E 3	0.000	0.000	0.000	0.000
3	1:LOAD	757.940	2.34E 3	0.000	0.000	0.000	0.000
5	1:LOAD	-757.940	2.14E 3	0.000	0.000	0.000	0.000
7	1:LOAD	0.000	750.641	0.000	0.000	0.000	0.000

Beam End Forces

Sign convention is as the action of the joint on the beam.

			Axial	Shear		Torsion	Bending	
Beam	Node	L/C	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip`in)	My (kip`in)	Mz (kip`in)
1	2	1:LOAD	257.135	0.000	0.000	0.000	0.000	0.000
	3	1:LOAD	-257.135	0.000	0.000	0.000	0.000	0.000
4	5	1:LOAD	195.840	0.000	0.000	0.000	0.000	0.000
	6	1:LOAD	-195.840	0.000	0.000	0.000	0.000	0.000
5	8	1:LOAD	855.860	0.000	0.000	0.000	0.000	0.000
	9	1:LOAD	-855.860	0.000	0.000	0.000	0.000	0.000
6	9	1:LOAD	855.860	0.000	0.000	0.000	0.000	0.000
	10	1:LOAD	-855.860	0.000	0.000	0.000	0.000	0.000
7	10	1:LOAD	-257.135	0.000	0.000	0.000	0.000	0.000
	11	1:LOAD	257.135	0.000	0.000	0.000	0.000	0.000

Print Time/Date: 10/02/2009 18:37 STAAD.Pro for Windows Release 2006 Print Run 1 of 2

Joseph W. Wilcher III

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Job Title	Job No	Sheet No 2	Rev
	Part		
	Ref		
	By	Date 09-Feb-09	Chd
Client	File Loads.std	Date/Time 10-Feb-2009 18:36	

Beam End Forces Cont...

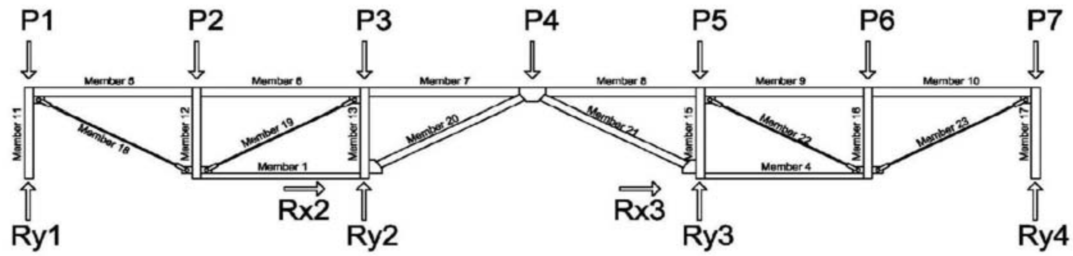
			Axial	Shear		Torsion	Bending	
Beam	Node	L/C	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip`in)	My (kip`in)	Mz (kip`in)
8	11	1:LOAD	-195.840	0.000	0.000	0.000	0.000	0.000
	12	1:LOAD	195.840	0.000	0.000	0.000	0.000	0.000
9	12	1:LOAD	632.906	0.000	0.000	0.000	0.000	0.000
	13	1:LOAD	-632.906	0.000	0.000	0.000	0.000	0.000
10	13	1:LOAD	632.906	0.000	0.000	0.000	0.000	0.000
	14	1:LOAD	-632.906	0.000	0.000	0.000	0.000	0.000
11	1	1:LOAD	1.16E 3	0.000	0.000	0.000	0.000	0.000
	8	1:LOAD	-1.16E 3	0.000	0.000	0.000	0.000	0.000
12	2	1:LOAD	1.13E 3	0.000	0.000	0.000	0.000	0.000
	9	1:LOAD	-1.13E 3	0.000	0.000	0.000	0.000	0.000
13	3	1:LOAD	1.76E 3	0.000	0.000	0.000	0.000	0.000
	10	1:LOAD	-1.76E 3	0.000	0.000	0.000	0.000	0.000
15	5	1:LOAD	1.6E 3	0.000	0.000	0.000	0.000	0.000
	12	1:LOAD	-1.6E 3	0.000	0.000	0.000	0.000	0.000
16	6	1:LOAD	835.230	0.000	0.000	0.000	0.000	0.000
	13	1:LOAD	-835.230	0.000	0.000	0.000	0.000	0.000
17	7	1:LOAD	750.641	0.000	0.000	0.000	0.000	0.000
	14	1:LOAD	-750.641	0.000	0.000	0.000	0.000	0.000
18	8	1:LOAD	-985.738	0.000	0.000	0.000	0.000	0.000
	2	1:LOAD	985.738	0.000	0.000	0.000	0.000	0.000
19	2	1:LOAD	-1.28E 3	0.000	0.000	0.000	0.000	0.000
	10	1:LOAD	1.28E 3	0.000	0.000	0.000	0.000	0.000
20	3	1:LOAD	1.17E 3	0.000	0.000	0.000	0.000	0.000
	11	1:LOAD	-1.17E 3	0.000	0.000	0.000	0.000	0.000
21	11	1:LOAD	1.1E 3	0.000	0.000	0.000	0.000	0.000
	5	1:LOAD	-1.1E 3	0.000	0.000	0.000	0.000	0.000
22	12	1:LOAD	-954.509	0.000	0.000	0.000	0.000	0.000
	6	1:LOAD	954.509	0.000	0.000	0.000	0.000	0.000
23	6	1:LOAD	-728.951	0.000	0.000	0.000	0.000	0.000
	14	1:LOAD	728.951	0.000	0.000	0.000	0.000	0.000
24	3	1:LOAD	0.000	0.000	0.000	0.000	0.000	0.000
	5	1:LOAD	0.000	0.000	0.000	0.000	0.000	0.000

Print Time/Date: 10/02/2009 18:37 STAAD.Pro for Windows Release 2006 Print Run 2 of 2

STAAD Output

APPENDIX D: PRELIMINARY TRUSS 2 MEMBER SIZES

Structure1 Truss 2



COLUMNS

Member	11		Member	12		Member	13	
P_u (Kips)	1160	Compression	P_u (Kips)	1125.06	Compression	P_u (Kips)	1759.81	Compression
Length (ft)	18		Length (ft)	18		Length (ft)	18	
Trial Member	W14x176		Trial Member	W14x176		Trial Member	W14x176	
Table 1-1 From Steel Manual			Table 1-1 From Steel Manual			Table 1-1 From Steel Manual		
A_g (in^2)	51.8		A_g (in^2)	51.8		A_g (in^2)	51.8	
r_x (in)	6.43		r_x (in)	6.43		r_x (in)	6.43	
r_y (in)	4.55		r_y (in)	4.55		r_y (in)	4.55	
Theoretical K [Table C-C2.2]	0.5		Theoretical K [Table C-C2.2]	0.5		Theoretical K [Table C-C2.2]	0.5	
KL/r_x	16.8		KL/r_x	16.8		KL/r_x	16.8	
KL/r_y	23.7	Controls	KL/r_y	23.7	Controls	KL/r_y	23.7	Controls
E (ksi)	29000		E (ksi)	29000		E (ksi)	29000	
F_y (ksi)	50		F_y (ksi)	50		F_y (ksi)	50	
$4.71*(E/F_y)^{1/2}$	113		$4.71*(E/F_y)^{1/2}$	113		$4.71*(E/F_y)^{1/2}$	113	
Use Fcr = 0.658^(Fy/Fe)*Fy			Use Fcr = 0.658^(Fy/Fe)*Fy			Use Fcr = 0.658^(Fy/Fe)*Fy		
F_e (ksi)	507		F_e (ksi)	507		F_e (ksi)	507	
F_{cr} (ksi)	48		F_{cr} (ksi)	48		F_{cr} (ksi)	48	
P_n (Kips)	2485		P_n (Kips)	2485		P_n (Kips)	2485	
ϕP_n (Kips)	2237		ϕP_n (Kips)	2237		ϕP_n (Kips)	2237	
Table 4-1 From Steel Manual			Table 4-1 From Steel Manual			Table 4-1 From Steel Manual		
$(KL)_y$ (ft)	18		$(KL)_y$ (ft)	18		$(KL)_y$ (ft)	18	
r_x/r_y	1.6		r_x/r_y	1.6		r_x/r_y	1.6	
ϕP_n (Kips)	1890		ϕP_n (Kips)	1890.0		ϕP_n (Kips)	1890.0	
$(KL)_{eff}$ (ft)	11.3	Okay	$(KL)_{eff}$ (ft)	11.3	Okay	$(KL)_{eff}$ (ft)	11.3	Okay
$\phi P_n \geq P_u$	Okay		$\phi P_n \geq P_u$	Okay		$\phi P_n \geq P_u$	Okay	
Member	W14x176		Member	W14x176		Member	W14x176	

Calculation 1: Preliminary Column Sizes for Truss 2

COLUMNS

Member	15		16		17	
P_u (Kips)	1598.63	Compression	835.23	Compression	750.64	Compression
Length (ft)	18		18		18	
Trial Member	W14x176		W14x176		W14x176	
Table 1-1 From Steel Manual						
A_g (in^2)	51.8		51.8		51.8	
r_x (in)	6.43		6.43		6.43	
r_y (in)	4.55		4.55		4.55	
Theoretical K [Table C-C2.2]	0.5		0.5		0.5	
KL/r_x	16.8		16.8		16.8	
KL/r_y	23.7	Controls	23.7	Controls	23.7	Controls
E (ksi)	29000		29000		29000	
F_y (ksi)	50		50		50	
$4.71*(E/F_y)^{1/2}$	113		113		113	
Use Fcr = 0.658^(Fy/Fe)*Fy						
F_e (ksi)	507		507		507	
F_{cr} (ksi)	48		48		48	
P_n (Kips)	2485		2485		2485	
φP_n (Kips)	2237		2237		2237	
Table 4-1 From Steel Manual						
$(KL)_y$ (ft)	18		18		18	
r_x/r_y	1.6		1.6		1.6	
φP_n (Kips)	1890		1890		1890	
$(KL)_{eff}$ (ft)	11.3	Okay	11.3	Okay	11.3	Okay
$\varphi P_n \geq P_u$	Okay		Okay		Okay	
Member	W14x176		W14x176		W14x176	

Calculation 2: Preliminary Column Sizes for Truss 2 Cont'd

BOTTOM CHORDS

Member	1	
P_u (Kips)	253.38	Tension
Length (ft)	31.5	
φ	0.9	
F_y (ksi)	50	
A_g (in^2)	5.63	
Table 5-1 From Steel Manual		
Member	W16x31	
A_g (in^2)	9.13	
φP_n (Kips)	411	Yielding
φP_n (Kips)	334	Rupture
$\varphi P_n \geq P_u$ (Yield)	Okay	
Member	W16x31	

Member	4	
P_u (Kips)	196.32	Tension
Length (ft)	31.5	
φ	0.9	
F_y (ksi)	50	
A_g (in^2)	4.36	
Table 5-1 From Steel Manual		
Member	W16x31	
A_g (in^2)	9.13	
φP_n (Kips)	441	Yielding
φP_n (Kips)	334	Rupture
$\varphi P_n \geq P_u$ (Yield)	Okay	
Member	W16x31	

Calculation 3: Preliminary Bottom Chord Sizes for Truss 2

TOP CHORDS

Member	5		6		7	
P_u (Kips)	860.13	Compression	860.13	Compression	253.38	Compression
Length (ft)	31.5		31.5		31.5	
Trial Member	WT15x130.5		WT15x130.5		WT15x130.5	
Table 1-1 From Steel Manual						
A_g (in^2)	38.4		38.4		38.4	
r_x (in)	4.46		4.46		4.46	
r_y (in)	3.53		3.53		3.53	
Theoretical K [Table C-C2.2]	0.5		0.5		0.5	
KL/r_x	42.4		42.4		42.4	
KL/r_y	53.5	Controls	53.5	Controls	53.5	Controls
E (ksi)	29000		29000		29000	
F_y (ksi)	50		50		50	
$4.71*(E/F_y)^{1/2}$	113		113		113	
Use Fcr = 0.658^(Fy/Fe)*Fy						
F_e (ksi)	100		100		100	
F_{cr} (ksi)	41		41		41	
P_n (Kips)	1557		1557		1557	
φP_n (Kips)	1401		1401		1401	
φP_n (Kips)	1261		1261		1261	
Table 4-7 From Steel Manual						
Member	WT15x130.5		WT15x130.5		WT15x130.5	
KL (ft)	31.5		31.5		31.5	
Axis	X-X		X-X		X-X	
r_x (in)	4.46		4.46		4.46	
r_y (in)	3.53		3.53		3.53	
φP_n (Kips)	1025		1025		1025	
$\varphi P_n \geq P_u$	Okay		Okay		Okay	
Member	WT15x130.5		WT15x130.5		WT15x130.5	

Calculation 4: Preliminary Top Chord Sizes for Truss 2

TOP CHORDS

Member	8	
P_u (Kips)	196.315	Compression
Length (ft)	31.5	
Trial Member	WT15x130.5	
Table 1-1 From Steel Manual		
A_g (in^2)	38.4	
r_x (in)	4.46	
r_y (in)	3.53	
Theoretical K [Table C-C2.2]	0.5	
KL/r_x	42.4	
KL/r_y	53.5	Controls
E (ksi)	29000	
F_y (ksi)	50	
$4.71*(E/F_y)^{1/2}$	113	
Use Fcr = 0.658^(Fy/Fe)*Fy		
F_e (ksi)	100	
F_{cr} (ksi)	41	
P_n (Kips)	1557	
φP_n (Kips)	1401	
φP_n (Kips)	1261	
Table 4-7 From Steel Manual		
Member	WT15x130.5	
KL (ft)	31.5	
Axis	X-X	
r_x (in)	4.46	
r_y (in)	3.53	
φP_n (Kips)	1025	
$\varphi P_n \geq P_u$	Okay	
Member	WT15x130.5	

Member	9	
P_u (Kips)	634.45	Compression
Length (ft)	31.5	
Trial Member	WT15x130.5	
Table 1-1 From Steel Manual		
A_g (in^2)	38.4	
r_x (in)	4.46	
r_y (in)	3.53	
Theoretical K [Table C-C2.2]	0.5	
KL/r_x	42.4	
KL/r_y	53.5	Controls
E (ksi)	29000	
F_y (ksi)	50	
$4.71*(E/F_y)^{1/2}$	113	
Use Fcr = 0.658^(Fy/Fe)*Fy		
F_e (ksi)	100	
F_{cr} (ksi)	41	
P_n (Kips)	1557	
φP_n (Kips)	1401	
φP_n (Kips)	1261	
Table 4-7 From Steel Manual		
Member	WT15x130.5	
KL (ft)	31.5	
Axis	X-X	
r_x (in)	4.46	
r_y (in)	3.53	
φP_n (Kips)	1025	
$\varphi P_n \geq P_u$	Okay	
Member	WT15x130.5	

Member	10	
P_u (Kips)	634.45	Compression
Length (ft)	31.5	
Trial Member	WT15x130.5	
Table 1-1 From Steel Manual		
A_g (in^2)	38.4	
r_x (in)	4.46	
r_y (in)	3.53	
Theoretical K [Table C-C2.2]	0.5	
KL/r_x	42.4	
KL/r_y	53.5	Controls
E (ksi)	29000	
F_y (ksi)	50	
$4.71*(E/F_y)^{1/2}$	113	
Use Fcr = 0.658^(Fy/Fe)*Fy		
F_e (ksi)	100	
F_{cr} (ksi)	41	
P_n (Kips)	1557	
φP_n (Kips)	1401	
φP_n (Kips)	1261	
Table 4-7 From Steel Manual		
Member	WT15x130.5	
KL (ft)	31.5	
Axis	X-X	
r_x (in)	4.46	
r_y (in)	3.53	
φP_n (Kips)	1025	
$\varphi P_n \geq P_u$	Okay	
Member	WT15x130.5	

Calculation 5: Preliminary Top Chord Sizes for Truss 2 Cont'd

BRACING (Tension)

Member	18	
P_u (Kips)	990.06	Tension
Length (ft)	36.3	
φ	0.9	
F_y (ksi)	42	
A_g (in^2)	26.19	
r (in)	2.89	
r_{design} (in)	3.00	
r_{final} (in)*	1.50	
*2 Members Will Be Used		
$I_{required}$ (in^4)	59	
Table 1-13 From Steel Manual		
Trial Member	HSS10x0.50	
A_g (in^2)	13.9	
r (in)	3.38	Okay
I_{member} (in^4)	159	Okay
Member	HSS10x0.50	

Member	19	
P_u (Kips)	1284.73	Tension
Length (ft)	36.3	
φ	0.9	
F_y (ksi)	42	
A_g (in^2)	33.99	
r (in)	3.29	
r_{design} (in)	4.00	
r_{final} (in)*	2.00	
*2 Members Will Be Used		
$I_{required}$ (in^4)	136	
Table 1-13 From Steel Manual		
Trial Member	HSS10x0.50	
A_g (in^2)	13.9	
r (in)	3.38	Okay
I_{member} (in^4)	159	Okay
Member	HSS10x0.50	

Member	22	
P_u (Kips)	956.26	Tension
Length (ft)	36.3	
φ	0.9	
F_y (ksi)	42	
A_g (in^2)	25.30	
r (in)	2.84	
r_{design} (in)	3.00	
r_{final} (in)*	1.50	
*2 Members Will Be Used		
$I_{required}$ (in^4)	57	
Table 1-13 From Steel Manual		
Trial Member	HSS10x0.50	
A_g (in^2)	13.9	
r (in)	3.38	Okay
I_{member} (in^4)	159	Okay
Member	HSS10x0.50	

Member	23	
P_u (Kips)	730.29	Tension
Length (ft)	36.3	
φ	0.9	
F_y (ksi)	42	
A_g (in^2)	19.32	
r (in)	2.48	
r_{design} (in)	3.00	
r_{final} (in)*	1.50	
*2 Members Will Be Used		
$I_{required}$ (in^4)	43	
Table 1-13 From Steel Manual		
Trial Member	HSS10x0.50	
A_g (in^2)	13.9	
r (in)	3.38	Okay
I_{member} (in^4)	159	Okay
Member	HSS10x0.50	

Calculation 6: Preliminary Tension Bracing Sizes for Truss 2

BRACING (Compression)

Member	20	
P_u (Kips)	1171.56	Compression
Length (ft)	36.3	
φ	0.9	
F_y (ksi)	42	
A_g (in^2)	30.99	
r (in)	3.14	
r_{design} (in)	4.00	
r_{final} (in)*	2.00	
*2 Members Will Be Used		
$I_{required}$ (in^4)	124	

Table 1-13 From Steel Manual		
Trial Member	HSS10x0.50	
A_g (in^2)	13.9	
r (in)	3.38	Okay
I_{member} (in^4)	159	Okay
Member	HSS10x0.50	

Member	21	
P_u (Kips)	1093.17	Compression
Length (ft)	36.3	
φ	0.9	
F_y (ksi)	42	
A_g (in^2)	28.92	
r (in)	3.03	
r_{design} (in)	4.00	
r_{final} (in)*	2.00	
*2 Members Will Be Used		
$I_{required}$ (in^4)	116	

Table 1-13 From Steel Manual		
Trial Member	HSS10x0.50	
A_g (in^2)	13.9	
r (in)	3.38	Okay
I_{member} (in^4)	159	Okay
Member	HSS10x0.50	

Calculation 7: Preliminary Compression Bracing Sizes for Truss 2

APPENDIX E: CURVED TENSION MEMBERS WITHIN TRUSS 2

Joseph W. Wilcher III	Job No	Sheet No 1	Rev
Software licensed to PSUAE	Part		
Job Title	Ref		
	By	Date 27-Feb-09	Chd
Client	File Truss 1, Strucutre 1, Mem	Date/Time 03-Mar-2009 20:36	

Job Information

	Engineer	Checked	Approved
Name:			
Date:	27-Feb-09		

Structure Type	SPACE FRAME

Number of Nodes	26	Highest Node	26
Number of Elements	25	Highest Beam	25

Number of Basic Load Cases	1
Number of Combination Load Cases	0

Included in this printout are data for:

All	The Whole Structure

Included in this printout are results for load cases:

Type	L/C	Name
Primary	1	LOAD

Beam Maximum Moments

Distances to maxima are given from beam end A.

Beam	Node A	Length (ft)	L/C		d (ft)	Max My (kip`ft)	d (ft)	Max Mz (kip`ft)
1	1	1.845	1:LOAD	Max -ve	0.000	0.000	0.000	0.000
				Max +ve	0.000	0.000	1.845	-650.586
2	2	1.796	1:LOAD	Max -ve	0.000	0.000		
				Max +ve	0.000	0.000	1.796	-1.22E 3
3	3	1.748	1:LOAD	Max -ve	0.000	0.000		
				Max +ve	0.000	0.000	1.748	-1.71E 3
4	4	1.695	1:LOAD	Max -ve	0.000	0.000		
				Max +ve	0.000	0.000	1.695	-2.11E 3
5	5	1.644	1:LOAD	Max -ve	0.000	0.000		
				Max +ve	0.000	0.000	1.644	-2.45E 3
6	6	1.599	1:LOAD	Max -ve	0.000	0.000		
				Max +ve	0.000	0.000	1.599	-2.71E 3
7	7	1.538	1:LOAD	Max -ve	0.000	0.000		
				Max +ve	0.000	0.000	1.538	-2.9E 3
8	8	1.496	1:LOAD	Max -ve	0.000	0.000		
				Max +ve	0.000	0.000	1.496	-3.04E 3
9	9	1.436	1:LOAD	Max -ve	0.000	0.000		
				Max +ve	0.000	0.000	1.436	-3.12E 3
10	10	1.401	1:LOAD	Max -ve	0.000	0.000		
				Max +ve	0.000	0.000	1.401	3.14E 3
11	11	1.342	1:LOAD	Max -ve	0.000	0.000		

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STADD Output for Curved Tension Members

Joseph W. Wilcher III

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Job No	Sheet No 2	Rev
Part		
Ref		
By	Date 27-Feb-09	Chd
File Truss 1, Strucutre 1, Mem	Date/Time 03-Mar-2009 20:36	

Job Title

Client

Beam Maximum Moments Cont...

Beam	Node A	Length (ft)	L/C		d (ft)	Max My (kip`ft)	d (ft)	Max Mz (kip`ft)
				Max +ve	0.000	0.000	0.000	-3.14E 3
12	12	1.301	1:LOAD	Max -ve	0.000	0.000		
				Max +ve	0.000	0.000	0.000	-3.13E 3
13	13	1.257	1:LOAD	Max -ve	0.000	0.000		
				Max +ve	0.000	0.000	0.000	-3.07E 3
14	14	1.222	1:LOAD	Max -ve	0.000	0.000		
				Max +ve	0.000	0.000	0.000	-2.98E 3
15	15	1.173	1:LOAD	Max -ve	0.000	0.000		
				Max +ve	0.000	0.000	0.000	-2.85E 3
16	16	1.147	1:LOAD	Max -ve	0.000	0.000		
				Max +ve	0.000	0.000	0.000	-2.69E 3
17	17	1.105	1:LOAD	Max -ve	0.000	0.000		
				Max +ve	0.000	0.000	0.000	-2.51E 3
18	18	1.086	1:LOAD	Max -ve	0.000	0.000		
				Max +ve	0.000	0.000	0.000	-2.3E 3
19	19	1.049	1:LOAD	Max -ve	0.000	0.000		
				Max +ve	0.000	0.000	0.000	-2.07E 3
20	20	1.023	1:LOAD	Max -ve	0.000	0.000		
				Max +ve	0.000	0.000	0.000	-1.82E 3
21	21	1.001	1:LOAD	Max -ve	0.000	0.000		
				Max +ve	0.000	0.000	0.000	-1.55E 3
22	22	0.980	1:LOAD	Max -ve	0.000	0.000		
				Max +ve	0.000	0.000	0.000	-1.27E 3
23	23	0.959	1:LOAD	Max -ve	0.000	0.000		
				Max +ve	0.000	0.000	0.000	-972.661
24	24	0.951	1:LOAD	Max -ve	0.000	0.000		
				Max +ve	0.000	0.000	0.000	-665.458
25	25	0.933	1:LOAD	Max -ve	0.000	0.000	0.933	0.000
				Max +ve	0.000	0.000	0.000	-338.251

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STADD Output for Curved Tension Members Cont'd

Curved Tension Member Analysis				
Structure 1 Truss 2			Moment From STAAD (ft-Kips)	3144.6
Member	19	Looking at 1 of the 2 Members	Lever Arm (ft)	11.604
F (Kips)	640.87		Angle (Degrees)	30
F_x (Kips)	556.76		R_x (Kips)	270.99
F_y (Kips)	317.38		R (Kips)	313

Preliminary Size	HSS10x0.5	
Table 4-5 In Steel Manual		
KL (ft)	23.42	
φP_n (Kips)	343	Okay
New Member	-	
φP_n (Kips)	-	

Final Member	HSS10.0x0.500

Calculation 1: Curved Tension Member Analysis for Truss 2

APPENDIX F: LATERAL RESISTING SYSTEM CALCULATIONS

Wind Parameters For Structure 1					
Basic Wind Speed (V)	Wind Exposure Category	Building Category	Importance Factor	Wind Directionality Factor (K_d)	Topographic Factor (K_{zt})
90 mph	B	III	1.15	0.85	1
Number of Stories	Building Height (ft)	N-S Building Length (ft)	E-W Building Length (ft)	L/B Along N-S Direction	L/B Along E-W Direction
5	88.167	179.50	189.00	0.950	1.05

Table 1: Wind Parameters for Structure 1

Wind Parameters For Strucutre 2					
Basic Wind Speed (V)	Wind Exposure Category	Building Category	Importance Factor	Wind Directionality Factor (K_d)	Topographic Factor (K_{zt})
90 mph	B	III	1.15	0.85	1
Number of Stories	Building Height (ft)	N-S Building Length (ft)	E-W Building Length (ft)	L/B Along N-S Direction	L/B Along E-W Direction
5	88.167	167.00	189.00	0.884	1.13

Table 2: Wind Parameters for Structure 2

Gust Factor: N-S Direction For Structure 1						
Stiffness	B (ft)	L (ft)	h (ft)	c	z (ft)	I_z
Rigid	189.00	179.50	88.167	0.3	64.667	0.268
l (ft)	ϵ	L_z (ft)	Q	g_Q	g_v	G
320	1/3.0	400	0.82	3.4	3.4	0.82

Gust Factor: E-W Direction For Structure 1						
Stiffness	B (ft)	L (ft)	h (ft)	c	z (ft)	I_z
Rigid	179.50	189.00	88.167	0.3	64.667	0.268
l (ft)	ϵ	L_z (ft)	Q	g_Q	g_v	G
320	1/3.0	400	0.82	3.4	3.4	0.82

Table 3: Gust Factors for Structure 1

Gust Factor: N-S Direction For Structure 2						
Stiffness	B (ft)	L (ft)	h (ft)	c	z (ft)	I_z
Rigid	189.00	167.00	88.167	0.3	64.667	0.268
l (ft)	∈	L_z (ft)	Q	g_Q	g_V	G
320	1/3.0	400	0.82	3.4	3.4	0.82

Gust Factor: E-W Direction Structure 2						
Stiffness	B (ft)	L (ft)	h (ft)	c	z (ft)	I_z
Rigid	167.00	189.00	88.167	0.3	64.667	0.268
l (ft)	∈	L_z (ft)	Q	g_Q	g_V	G
320	1/3.0	400	0.82	3.4	3.4	0.83

Table 4: Gust Factors for Structure 2

Wind Factors: N-S Direction For Strucutre 1				
C_p, Windward	C_p, Leeward	Gust Factor	GC_{pi}	GC_{pi}
0.8	-0.5	0.82	±0.18	±0.55

Wind Factors: E-W Direction For Structure 1				
C_p, Windward	C_p, Leeward	Gust Factor	GC_{pi}	GC_{pi}
0.8	-0.5	0.82	±0.18	±0.55

Table 5: Wind Factors for Structure 1

Wind Factors: N-S Direction For Structure 2				
C_p, Windward	C_p, Leeward	Gust Factor	GC_{pi}	GC_{pi}
0.8	-0.5	0.82	±0.18	±0.55

Wind Factors: E-W Direction Structure 2				
C_p, Windward	C_p, Leeward	Gust Factor	GC_{pi}	GC_{pi}
0.8	-0.5	0.83	±0.18	±0.55

Table 6: Wind Factors for Structure 2

Seismic Parameters For Structure 1													
S_s	S_1	Site Class	Occupancy Category	Importance Factor	Fa	Fv	S_{MS}	S_{M1}	S_{DS}	S_{D1}	Seismic Design Category	R	C_u
0.153	0.05	D	III	1.25	1.6	2.4	0.245	0.120	0.163	0.080	B	3.25	1.7
T_a	T	T_L	C_S	Roof Dead Load (psf)	Floor Dead Load (psf)	Snow Load (psf)	Wall Load (psf)	W_{roof} (Kips)	W_{floor} (Kips)	W_{Total} (Kips)	A (ft^2)	P (ft)	V (Kips)
0.787	1.34	8	0.0230	175	See Below	19	35	5937	48276	54213	33926	737	1248

Table 7: Seismic Parameters for Structure 1

Weight of Structure 1							
Level	Slab (psf)	Beams (psf)	Girders (psf)	Columns (psf)	Trusses (psf)	Floor Dead Load (psf)	W_{floor} (Kips)
Roof	51	40	84	-	-	175	5937
3rd Floor	51	40	84	61	-	236	8006
2nd Floor	51	40	84	82	-	257	8719
1st Floor	51	40	84	109	-	284	9635
Mezzazine	51	135	-	132	-	318	10788
Ground	81	-	-	-	247	328	11128
Track Level	-	-	-	-	-	-	-

Table 8: Weight of Structure 1

Seismic Parameters For Structure 2													
S_s	S_1	Site Class	Occupancy Category	Importance Factor	Fa	Fv	S_{MS}	S_{M1}	S_{DS}	S_{D1}	Seismic Design Category	R	C_u
0.153	0.05	D	III	1.25	1.6	2.4	0.245	0.120	0.163	0.080	B	3.25	1.7
T_a	T	T_L	C_S	Roof Dead Load (psf)	Floor Dead Load (psf)	Snow Load (psf)	Wall Load (psf)	W_{roof} (Kips)	W_{floor} (Kips)	W_{Total} (Kips)	A (ft^2)	P (ft)	V (Kips)
0.787	1.34	8	0.0230	167	See Below	19	35	5666	46648	52313	33926	737	1204

Table 9: Seismic Parameters for Structure 2

Weight of Structure 2							
Level	Slab (psf)	Beams (psf)	Girders (psf)	Columns (psf)	Trusses (psf)	Floor Dead Load (psf)	W_{floor} (Kips)
Roof	51	40	76	-	-	167	5666
3rd Floor	51	40	76	61	-	228	7735
2nd Floor	51	40	76	82	-	249	8447
1st Floor	51	40	76	99	-	266	9024
Mezzazine	51	135	-	132	-	318	10788
Ground	81	-	-	-	233	314	10653
Track Level	-	-	-	-	-	-	-

Table 9: Weight of Structure 2

Wind (North-South) For Structure 1										
Level	Height (Feet)	Tributary Area (Feet)	K_z	q_z (psf)	Windward (psf)	Leeward (psf)	Total (psf)	Story Force (Kips)	Story Shear (Kips)	Overturning Moment (Ft-Kips)
Roof	88.17	5.75	0.95	19.3	23.3	-18.5	41.8	43.1	43.1	247.8
3	76.67	11.5	0.92	18.6	22.5	-18.5	41.0	171.6	214.7	1730.3
2	65.17	11.5	0.87	17.6	14.8	-10.4	25.2	368.5	583.2	6318.5
1	53.67	11.875	0.82	16.6	13.9	-10.4	24.3	624.6	1207.8	16952.7
Mezzaine	41.42	15.1	0.77	15.6	13.1	-10.4	23.5	1015.0	2222.8	42854.0
Ground	23.5	10.96	0.65	13.2	11.0	-10.4	21.5	1015.0	2222.8	42854.0
Track Level	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1015.0	2222.8	42854.0

Wind (North-South) For Strucutre 1										
Level	Height (Feet)	Tributary Area (Feet)	K_z	q_z (psf)	Windward (psf)	Leeward (psf)	Total (psf)	Story Force (Kips)	Story Shear (Kips)	Overturning Moment (Ft-Kips)
Roof	88.17	5.75	0.95	19.3	23.3	-18.5	41.8	45.4	45.4	261.2
3	76.67	11.5	0.92	18.6	22.5	-18.5	41.1	180.9	226.3	1823.7
2	65.17	11.5	0.87	17.6	14.8	-10.4	25.2	388.4	614.7	6659.7
1	53.67	11.875	0.82	16.6	13.9	-10.4	24.4	658.3	1273.1	17868.4
Mezzaine	41.42	15.1	0.77	15.6	13.1	-10.4	23.5	1070.0	2343.0	45170.1
Ground	23.5	10.96	0.65	13.2	11.1	-10.4	21.5	1070.0	2343.0	45170.1
Track Level	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1070.0	2343.0	45170.1

Calculation 1: Shear & Moment Due to Wind Forces for Structure 1

Wind (North-South) For Structure 2										
Level	Height (Feet)	Tributary Area (Feet)	K_z	q_z (psf)	Windward (psf)	Leeward (psf)	Total (psf)	Story Force (Kips)	Story Shear (Kips)	Overturning Moment (Ft-Kips)
Roof	88.17	5.75	0.95	19.3	23.3	-18.5	41.8	40.1	40.1	230.5
3	76.67	11.5	0.92	18.6	22.5	-18.5	41.0	159.7	199.8	1609.8
2	65.17	11.5	0.87	17.6	14.8	-10.4	25.2	342.8	542.6	5878.5
1	53.67	11.875	0.82	16.6	13.9	-10.4	24.3	581.1	1123.7	15772.2
Mezzaine	41.42	15.1	0.77	15.6	13.1	-10.4	23.5	944.3	2068.0	39869.8
Ground	23.5	10.96	0.65	13.2	11.0	-10.4	21.5	944.3	2068.0	39869.8
Track Level	0.0	0.0	0.0	0.0	0.0	0.0	0.0	944.3	2068.0	39869.8

Wind (North-South) For Strucutre 2										
Level	Height (Feet)	Tributary Area (Feet)	K_z	q_z (psf)	Windward (psf)	Leeward (psf)	Total (psf)	Story Force (Kips)	Story Shear (Kips)	Overturning Moment (Ft-Kips)
Roof	88.17	5.75	0.95	19.3	23.3	-18.5	41.9	45.5	45.5	261.5
3	76.67	11.5	0.92	18.6	22.6	-18.5	41.1	181.1	226.6	1826.2
2	65.17	11.5	0.87	17.6	14.8	-10.5	25.3	388.9	615.6	6668.8
1	53.67	11.875	0.82	16.6	14.0	-10.5	24.4	659.3	1274.9	17893.4
Mezzaine	41.42	15.1	0.77	15.6	13.1	-10.5	23.6	1071.6	2346.5	45235.0
Ground	23.5	10.96	0.65	13.2	11.1	-10.5	21.5	1071.6	2346.5	45235.0
Track Level	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1071.6	2346.5	45235.0

Calculation 2: Shear & Moment Due to Wind Forces for Structure 2

Seismic For Structure 1					
Level	Height (ft)	Tributary Area (ft)	C_{VX}	F_x (Kips)	Overturning Moment (ft-Kips)
Roof	88.17	5.75	0.25	316	1815.4
3rd Floor	76.67	11.5	0.31	388	7677.9
2nd Floor	65.17	11.5	0.23	283	22928.6
1st Floor	53.67	11.875	0.15	181	30187.6
Mezzanine	41.42	15.1	0.06	80	48421.6
Ground	23.5	10.96	1.00	1248	48421.6
Track Level	0	0	1.00	1248	48421.6

Calculation 3: Shear & Moment Due to Seismic Forces for Structure 1

Seismic For Structure 2					
Level	Height (ft)	Tributary Area (ft)	C_{VX}	F_x (Kips)	Overturning Moment (ft-Kips)
Roof	88.17	5.75	0.25	300	1724.7
3rd Floor	76.67	11.5	0.31	378	7347.1
2nd Floor	65.17	11.5	0.23	277	22120.0
1st Floor	53.67	11.875	0.14	166	29055.3
Mezzazine	41.42	15.1	0.07	84	46603.0
Ground	23.5	10.96	1.00	1204	46603.0
Track Level	0	0	1.00	1204	46603.0

Calculation 4: Shear & Moment Due to Seismic Forces for Structure 2

Brace Frame 1: Structure 1		
Level	Displacement (in)	Stiffness (Kip/in)
Roof	0.0017	588.24
3	0.0013	769.23
2	0.0008	1250.00
1	0.0005	2000.00
Mezzanine	0.0001	10000.00
Total		2921.49

Brace Frame A: Structure 1		
Level	Displacement (in)	Stiffness (Kip/in)
Roof	0.0022	454.55
3	0.0016	625.00
2	0.0011	909.09
1	0.0006	1666.67
Mezzanine	0.0002	5000.00
Total		1731.06

Brace Frame 2: Structure 1		
Level	Displacement (In)	Stiffness (Kip/in)
Roof	0.0017	588.24
3	0.0013	769.23
2	0.0009	1111.11
1	0.0005	2000.00
Mezzanine	0.0001	10000.00
Total		2893.72

Brace Frame B: Structure 1		
Level	Displacement (in)	Stiffness (Kip/in)
Roof	0.0021	476.19
3	0.0015	666.67
2	0.001	1000.00
1	0.0006	1666.67
Mezzanine	0.0002	5000.00
Total		1761.90

Brace Frame 3: Structure 2		
Level	Displacement (in)	Stiffness (Kip/in)
Roof	0.0016	625.00
3	0.0012	833.33
2	0.0008	1250.00
1	0.0004	2500.00
Mezzanine	0.0001	10000.00
Total		3041.67

Brace Frame C: Structure 2		
Level	Displacement (in)	Stiffness (Kip/in)
Roof	0.0021	476.19
3	0.0016	625.00
2	0.001	1000.00
1	0.0006	1666.67
Mezzanine	0.0002	5000.00
Total		1753.57

Brace Frame 4: Structure 2		
Level	Displacement (In)	Stiffness (Kip/in)
Roof	0.0017	588.24
3	0.0013	769.23
2	0.0009	1111.11
1	0.0005	2000.00
Mezzanine	0.0002	5000.00
Total		1893.72

Brace Frame D: Structure 2		
Level	Displacement (in)	Stiffness (Kip/in)
Roof	0.0018	555.56
3	0.0014	714.29
2	0.0009	1111.11
1	0.0005	2000.00
Mezzanine	0.0002	5000.00
Total		1876.19

Calculation 5: Stiffness of Brace Frames

Story	Frame Stiffness: Structure 1				Center of Rigidity	
	1	2	A	B	X_R (ft)	Y_R (ft)
Roof	588.24	588.24	454.55	476.19	92.30	88.34
3	769.23	769.23	625.00	666.67	91.45	88.34
2	1250.00	1111.11	909.09	1000.00	90.00	83.14
1	2000.00	2000.00	1666.67	1666.67	94.50	88.34
Mezzanine	10000.00	10000.00	5000.00	5000.00	94.50	88.34

Story	Frame Stiffness Structure 2				Center of Rigidity	
	3	4	C	D	X_R (ft)	Y_R (ft)
Roof	625.00	588.24	476.19	555.56	87.23	79.60
3	833.33	769.23	625.00	714.29	88.20	78.80
2	1250.00	1111.11	1000.00	1111.11	89.53	77.26
1	2500.00	2000.00	1666.67	2000.00	85.91	72.96
Mezzanine	10000.00	5000.00	5000.00	5000.00	94.50	54.72

Calculation 5: Stiffness of Brace Frames

Torsional Moment Due To Wind: Structure 1						
Level	Story Force (Kips) North-South	Story Force (Kips) East-West	e_{N-S} (ft)	e_{E-W} (ft)	M_{N-S} (ft-kips)	M_{E-W} (ft-kips)
Roof	43.10	45.40	0.00	-2.20	0.00	-99.77
3	171.60	180.90	0.00	-3.05	0.00	-551.45
2	368.50	388.40	5.20	-4.50	1914.79	-1747.80
1	624.60	658.30	0.00	0.00	0.00	0.00
Mezzanine	1015.00	1070.00	0.00	0.00	0.00	0.00

Torsional Moment Due To Wind: Structure 2						
Level	Story Force (Kips) North-South	Story Force (Kips) East-West	e_{N-S} (ft)	e_{E-W} (ft)	M_{N-S} (ft-kips)	M_{E-W} (ft-kips)
Roof	40.10	45.50	2.49	-7.27	99.74	-330.75
3	159.70	181.10	3.28	-6.30	524.35	-1140.93
2	342.80	388.90	4.83	-4.97	1655.19	-1934.27
1	581.10	659.30	9.12	-8.59	5299.86	-5663.99
Mezzanine	944.30	1071.60	3.61	0.00	3411.44	0.00

Torsional Moment Due To Seismic: Structure 1					
Level	Story Force (Kips)	e_{N-S} (ft)	e_{E-W} (ft)	M_{N-S} (ft-kips)	M_{E-W} (ft-kips)
Roof	340.00	0.00	-2.20	0.00	-747.21
3	418.00	0.00	-3.05	0.00	-1274.23
2	304.00	5.20	-4.50	1579.64	-1368.00
1	195.00	0.00	0.00	0.00	0.00
Mezzanine	86.00	0.00	0.00	0.00	0.00

Torsional Moment Due To Seismic: Structure 2					
Level	Story Force (Kips)	e_{N-S} (ft)	e_{E-W} (ft)	M_{N-S} (ft-kips)	M_{E-W} (ft-kips)
Roof	300.00	2.49	-7.27	746.21	-2180.77
3	378.00	3.28	-6.30	1241.10	-2381.40
2	277.00	4.83	-4.97	1337.48	-1377.71
1	166.00	9.12	-8.59	1513.98	-1426.09
Mezzanine	84.00	3.61	0.00	303.46	0.00

Calculation 6: Torsional Moment Due to Wind & Seismic on Both Structures

Wind Direct Forces: Structure 1 (N-S Direction)				
Roof	K_i	P	ΣK_i	Force (Kips)
1	588.24	43.10	1176.47	21.55
2	588.24	43.10	1176.47	21.55
			Total	43.10
Wind Direct Forces: Structure 1 (E-W Direction)				
Roof	K_i	P	ΣK_i	Force (Kips)
A	454.55	45.40	930.74	22.17
B	476.19	45.40	930.74	23.23
			Total	45.40

Wind Direct Forces: Structure 2 (N-S Direction)				
Roof	K_i	P	ΣK_i	Force (Kips)
3	625.00	40.10	1213.24	20.66
4	588.24	40.10	1213.24	19.44
			Total	40.10
Wind Direct Forces: Structure 2 (E-W Direction)				
Roof	K_i	P	ΣK_i	Force (Kips)
C	476.19	45.50	1031.75	21.00
D	555.56	45.50	1031.75	24.50
			Total	45.50

Wind Direct Forces: Structure 1 (N-S Direction)				
3rd Floor	K_i	P	ΣK_i	Force (Kips)
1	769.23	171.60	1538.46	85.80
2	769.23	171.60	1538.46	85.80
			Total	171.60
Wind Direct Forces: Structure 1 (E-W Direction)				
3rd Floor	K_i	P	ΣK_i	Force (Kips)
A	625.00	180.90	1291.67	87.53
B	666.67	180.90	1291.67	93.37
			Total	180.90

Wind Direct Forces: Structure 2 (N-S Direction)				
3rd Floor	K_i	P	ΣK_i	Force (Kips)
3	833.33	159.70	1602.56	83.04
4	769.23	159.70	1602.56	76.66
			Total	159.70
Wind Direct Forces: Structure 2 (E-W Direction)				
3rd Floor	K_i	P	ΣK_i	Force (Kips)
C	625.00	181.10	1339.29	84.51
D	714.29	181.10	1339.29	96.59
			Total	181.10

Wind Direct Forces: Structure 1 (N-S Direction)				
2nd Floor	K_i	P	ΣK_i	Force (Kips)
1	1250.00	368.50	2361.11	195.09
2	1111.11	368.50	2361.11	173.41
			Total	368.50
Wind Direct Forces: Structure 1 (E-W Direction)				
2nd Floor	K_i	P	ΣK_i	Force (Kips)
A	909.09	388.40	1909.09	184.95
B	1000.00	388.40	1909.09	203.45
			Total	388.40

Wind Direct Forces: Structure 2 (N-S Direction)				
2nd Floor	K_i	P	ΣK_i	Force (Kips)
3	1250.00	342.80	2361.11	181.48
4	1111.11	342.80	2361.11	161.32
			Total	342.80
Wind Direct Forces: Structure 2 (E-W Direction)				
2nd Floor	K_i	P	ΣK_i	Force (Kips)
C	1000.00	388.90	2111.11	184.22
D	1111.11	388.90	2111.11	204.68
			Total	388.90

Wind Direct Forces: Structure 1 (N-S Direction)				
1st Floor	K_i	P	ΣK_i	Force (Kips)
1	2000.00	624.60	4000.00	312.30
2	2000.00	624.60	4000.00	312.30
			Total	624.60
Wind Direct Forces: Structure 1 (E-W Direction)				
1st Floor	K_i	P	ΣK_i	Force (Kips)
A	1666.67	658.30	3333.33	329.15
B	1666.67	658.30	3333.33	329.15
			Total	658.30

Wind Direct Forces: Structure 2 (N-S Direction)				
1st Floor	K_i	P	ΣK_i	Force (Kips)
3	2500.00	581.10	4500.00	322.83
4	2000.00	581.10	4500.00	258.27
			Total	581.10
Wind Direct Forces: Structure 2 (E-W Direction)				
1st Floor	K_i	P	ΣK_i	Force (Kips)
C	1666.67	659.30	3666.67	299.68
D	2000.00	659.30	3666.67	359.62
			Total	659.30

Wind Direct Forces: Structure 1 (N-S Direction)				
Mezzanine	K_i	P	ΣK_i	Force (Kips)
1	10000.00	1015.00	20000.00	507.50
2	10000.00	1015.00	20000.00	507.50
			Total	1015.00
Wind Direct Forces: Structure 1 (E-W Direction)				
Mezzanine	K_i	P	ΣK_i	Force (Kips)
A	5000.00	1070.00	10000.00	535.00
B	5000.00	1070.00	10000.00	535.00
			Total	1070.00

Wind Direct Forces: Structure 2 (N-S Direction)				
Mezzanine	K_i	P	ΣK_i	Force (Kips)
3	10000.00	944.30	15000.00	629.53
4	5000.00	944.30	15000.00	314.77
			Total	944.30
Wind Direct Forces: Structure 2 (E-W Direction)				
Mezzanine	K_i	P	ΣK_i	Force (Kips)
C	5000.00	1071.60	10000.00	535.80
D	5000.00	1071.60	10000.00	535.80
			Total	1071.60

Calculation 7: Direct Forces Due to Wind on Both Structures

Seismic Direct Forces: Structure 1 (N-S Direction)				
Roof	K_i	P	ΣK_i	Force (Kips)
1	588.24	340.00	1176.47	170.00
2	588.24	340.00	1176.47	170.00
			Total	340.00
Seismic Direct Forces: Structure 1 (E-W Direction)				
Roof	K_i	P	ΣK_i	Force (Kips)
A	454.55	340.00	930.74	166.05
B	476.19	340.00	930.74	173.95
			Total	340.00

Seismic Direct Forces: Structure 2 (N-S Direction)				
Roof	K_i	P	ΣK_i	Force (Kips)
3	625.00	300.00	1213.24	154.55
4	588.24	300.00	1213.24	145.45
			Total	300.00
Seismic Direct Forces: Structure 2 (E-W Direction)				
Roof	K_i	P	ΣK_i	Force (Kips)
C	476.19	300.00	1031.75	138.46
D	555.56	300.00	1031.75	161.54
			Total	300.00

Seismic Direct Forces: Structure 1 (N-S Direction)				
3rd Floor	K_i	P	ΣK_i	Force (Kips)
1	769.23	418.00	1538.46	209.00
2	769.23	418.00	1538.46	209.00
			Total	418.00
Seismic Direct Forces: Structure 1 (E-W Direction)				
3rd Floor	K_i	P	ΣK_i	Force (Kips)
A	625.00	418.00	1291.67	202.26
B	666.67	418.00	1291.67	215.74
			Total	418.00

Seismic Direct Forces: Structure 2 (N-S Direction)				
3rd Floor	K_i	P	ΣK_i	Force (Kips)
3	833.33	378.00	1602.56	196.56
4	769.23	378.00	1602.56	181.44
			Total	378.00
Seismic Direct Forces: Structure 2 (E-W Direction)				
3rd Floor	K_i	P	ΣK_i	Force (Kips)
C	625.00	378.00	1339.29	176.40
D	714.29	378.00	1339.29	201.60
			Total	378.00

Seismic Direct Forces: Structure 1 (N-S Direction)				
2nd Floor	K_i	P	ΣK_i	Force (Kips)
1	1250.00	304.00	2361.11	160.94
2	1111.11	304.00	2361.11	143.06
			Total	304.00
Seismic Direct Forces: Structure 1 (E-W Direction)				
2nd Floor	K_i	P	ΣK_i	Force (Kips)
A	909.09	304.00	1909.09	144.76
B	1000.00	304.00	1909.09	159.24
			Total	304.00

Seismic Direct Forces: Structure 2 (N-S Direction)				
2nd Floor	K_i	P	ΣK_i	Force (Kips)
3	1250.00	277.00	2361.11	146.65
4	1111.11	277.00	2361.11	130.35
			Total	277.00
Seismic Direct Forces: Structure 2 (E-W Direction)				
2nd Floor	K_i	P	ΣK_i	Force (Kips)
C	1000.00	277.00	2111.11	131.21
D	1111.11	277.00	2111.11	145.79
			Total	277.00

Seismic Direct Forces: Structure 1 (N-S Direction)				
1st Floor	K_i	P	ΣK_i	Force (Kips)
1	2000.00	195.00	4000.00	97.50
2	2000.00	195.00	4000.00	97.50
			Total	195.00
Seismic Direct Forces: Structure 1 (E-W Direction)				
1st Floor	K_i	P	ΣK_i	Force (Kips)
A	1666.67	195.00	3333.33	97.50
B	1666.67	195.00	3333.33	97.50
			Total	195.00

Seismic Direct Forces: Structure 2 (N-S Direction)				
1st Floor	K_i	P	ΣK_i	Force (Kips)
3	2500.00	166.00	4500.00	92.22
4	2000.00	166.00	4500.00	73.78
			Total	166.00
Seismic Direct Forces: Structure 2 (E-W Direction)				
1st Floor	K_i	P	ΣK_i	Force (Kips)
C	1666.67	166.00	3666.67	75.45
D	2000.00	166.00	3666.67	90.55
			Total	166.00

Seismic Direct Forces: Structure 1 (N-S Direction)				
Mezzanine	K_i	P	ΣK_i	Force (Kips)
1	10000.00	86.00	20000.00	43.00
2	10000.00	86.00	20000.00	43.00
			Total	86.00
Seismic Direct Forces: Structure 1 (E-W Direction)				
Mezzanine	K_i	P	ΣK_i	Force (Kips)
A	5000.00	86.00	10000.00	43.00
B	5000.00	86.00	10000.00	43.00
			Total	86.00

Seismic Direct Forces: Structure 2 (N-S Direction)				
Mezzanine	K_i	P	ΣK_i	Force (Kips)
3	10000.00	84.00	15000.00	56.00
4	5000.00	84.00	15000.00	28.00
			Total	84.00
Seismic Direct Forces: Structure 2 (E-W Direction)				
Mezzanine	K_i	P	ΣK_i	Force (Kips)
C	5000.00	84.00	10000.00	42.00
D	5000.00	84.00	10000.00	42.00
			Total	84.00

Calculation 8: Direct Forces Due to Seismic on Both Structures

Forces By Torsional Moment: N-S Direction (Structure 1)

Roof	K_i	d_i	Kd_i^2	Force (Kips)
1	588.24	-88.34	4590562	0.00
2	588.24	88.34	4590562	0.00
A	454.55	96.70	4250405	0.00
B	476.19	-92.30	4056805	0.00
		Total	17488334	0.00

Forces By Torsional Moment: N-S Direction (Structure 1)

3rd Floor	K_i	d_i	Kd_i^2	Force (Kips)
1	769.23	-88.34	6003043	0.000
2	769.23	88.33	6001684	0.000
A	625.00	97.55	5947502	0.000
B	666.67	-91.45	5575402	0.000
		Total	23527630	0.00

Forces By Torsional Moment: N-S Direction (Structure 1)

2nd Floor	K_i	d_i	Kd_i^2	Force (Kips)
1	1250.00	-83.14	8640325	-5.626
2	1111.11	93.53	9719845	5.626
A	909.09	99.00	8910000	4.872
B	1000.00	-90.00	8100000	-4.872
		Total	35370170	0.00

Forces By Torsional Moment: N-S Direction (Structure 1)

1st Floor	K_i	d_i	Kd_i^2	Force (Kips)
1	2000.00	-88.34	15607911	0.000
2	2000.00	88.33	15604378	0.000
A	1666.67	94.50	14883750	0.000
B	1666.67	-94.50	14883750	0.000
		Total	60979789	0.00

Forces By Torsional Moment: N-S Direction (Structure 1)

Mezzanine	K_i	d_i	Kd_i^2	Force (Kips)
1	10000.00	-88.34	78039556	0.000
2	10000.00	88.33	78021889	0.000
A	5000.00	94.50	44651250	0.000
B	5000.00	-94.50	44651250	0.000
		Total	245363945	0.00

Forces By Torsional Moment: E-W Direction (Structure 1)

Roof	K_i	d_i	Kd_i^2	Force (Kips)
1	588.24	-88.34	4590562	0.296
2	588.24	88.34	4590562	-0.296
A	454.55	96.70	4250405	-0.251
B	476.19	-92.30	4056805	0.251
		Total	17488334	0.00

Forces By Torsional Moment: E-W Direction (Structure 1)

3rd Floor	K_i	d_i	Kd_i^2	Force (Kips)
1	769.23	-88.34	6003043	1.593
2	769.23	88.33	6001684	-1.593
A	625.00	97.55	5947502	-1.429
B	666.67	-91.45	5575402	1.429
		Total	23527630	0.00

Forces By Torsional Moment: E-W Direction (Structure 1)

2nd Floor	K_i	d_i	Kd_i^2	Force (Kips)
1	1250.00	-83.14	8640325	5.135
2	1111.11	93.53	9719845	-5.135
A	909.09	99.00	8910000	-4.447
B	1000.00	-90.00	8100000	4.447
		Total	35370170	0.00

Forces By Torsional Moment: E-W Direction (Structure 1)

1st Floor	K_i	d_i	Kd_i^2	Force (Kips)
1	2000.00	-88.34	15607911	0.000
2	2000.00	88.33	15604378	0.000
A	1666.67	94.50	14883750	0.000
B	1666.67	-94.50	14883750	0.000
		Total	60979789	0.00

Forces By Torsional Moment: E-W Direction (Structure 1)

Mezzanine	K_i	d_i	Kd_i^2	Force (Kips)
1	10000.00	-88.34	78039556	0.000
2	10000.00	88.33	78021889	0.000
A	5000.00	94.50	44651250	0.000
B	5000.00	-94.50	44651250	0.000
		Total	245363945	0.00

Forces By Torsional Moment: N-S Direction (Structure 2)

Roof	K_i	d_i	Kd_i^2	Force (Kips)
3	625.00	-79.60	3960100	-0.286
4	588.24	84.57	4206810	0.286
C	476.19	101.77	4931968	0.279
D	555.56	-87.23	4227263	-0.279
		Total	17326141	0.00

Forces By Torsional Moment: N-S Direction (Structure 2)

3rd Floor	K_i	d_i	Kd_i^2	Force (Kips)
3	833.33	-78.80	5174533	-1.518
4	769.23	85.37	5605788	1.518
C	625.00	100.80	6350400	1.456
D	714.29	-88.20	5556600	-1.456
		Total	22687322	0.00

Forces By Torsional Moment: N-S Direction (Structure 2)

2nd Floor	K_i	d_i	Kd_i^2	Force (Kips)
3	1250.00	-77.26	7461385	-4.613
4	1111.11	86.91	8392030	4.612
C	1000.00	99.47	9894281	4.751
D	1111.11	-89.53	8906245	-4.751
		Total	34653940	0.00

Forces By Torsional Moment: N-S Direction (Structure 2)

1st Floor	K_i	d_i	Kd_i^2	Force (Kips)
3	2500.00	-72.96	13307904	-15.487
4	2000.00	91.21	16637434	15.488
C	1666.67	103.09	17712580	14.589
D	2000.00	-85.91	14761056	-14.589
		Total	62418974	0.00

Forces By Torsional Moment: N-S Direction (Structure 2)

Mezzanine	K_i	d_i	Kd_i^2	Force (Kips)
3	10000.00	-54.72	29942784	-10.421
4	5000.00	109.45	59893229	10.421
C	5000.00	94.50	44651250	8.998
D	5000.00	-94.50	44651250	-8.998
		Total	179138513	0.00

Forces By Torsional Moment: E-W Direction (Structure 2)

Roof	K_i	d_i	Kd_i^2	Force (Kips)
3	625.00	-79.60	3960100	-1.656
4	588.24	84.57	4206810	1.655
C	476.19	101.77	4931968	1.613
D	555.56	-87.23	4227263	-1.613
		Total	17326141	0.00

Forces By Torsional Moment: E-W Direction (Structure 2)

3rd Floor	K_i	d_i	Kd_i^2	Force (Kips)
3	833.33	-78.80	5174533	-6.891
4	769.23	85.37	5605788	6.891
C	625.00	100.80	6350400	6.611
D	714.29	-88.20	5556600	-6.611
		Total	22687322	0.00

Forces By Torsional Moment: E-W Direction (Structure 2)

2nd Floor	K_i	d_i	Kd_i^2	Force (Kips)
3	1250.00	-77.26	7461385	-16.995
4	1111.11	86.91	8392030	16.993
C	1000.00	99.47	9894281	17.504
D	1111.11	-89.53	8906245	-17.506
		Total	34653940	0.00

Forces By Torsional Moment: E-W Direction (Structure 2)

1st Floor	K_i	d_i	Kd_i^2	Force (Kips)
3	2500.00	-72.96	13307904	-33.683
4	2000.00	91.21	16637434	33.685
C	1666.67	103.09	17712580	31.728
D	2000.00	-85.91	14761056	-31.729
		Total	62418974	0.00

Forces By Torsional Moment: E-W Direction (Structure 2)

Mezzanine	K_i	d_i	Kd_i^2	Force (Kips)
3	10000.00	-54.72	29942784	-71.527
4	5000.00	109.45	59893229	71.532
C	5000.00	94.50	44651250	61.763
D	5000.00	-94.50	44651250	-61.763
		Total	179138513	0.00

Calculation 9: Torsional Forces on both Structures Due to Wind

Forces By Torsional Moment: N-S Direction (Structure 1)				
Roof	K_i	d_i	$K_i d_i^2$	Force (Kips)
1	588.24	-88.34	4590562	0.000
2	588.24	88.34	4590562	0.000
A	454.55	96.70	4250405	0.000
B	476.19	-92.30	4056805	0.000
		Total	17488334	0.00

Forces By Torsional Moment: N-S Direction (Structure 1)				
3rd Floor	K_i	d_i	$K_i d_i^2$	Force (Kips)
1	769.23	-88.34	6003043	0.000
2	769.23	88.33	6001684	0.000
A	625.00	97.55	5947502	0.000
B	666.67	-91.45	5575402	0.000
		Total	23527630	0.00

Forces By Torsional Moment: N-S Direction (Structure 1)				
2nd Floor	K_i	d_i	$K_i d_i^2$	Force (Kips)
1	1250.00	-83.14	8640325	-4.641
2	1111.11	93.53	9719845	4.641
A	909.09	99.00	8910000	4.019
B	1000.00	-90.00	8100000	-4.019
		Total	35370170	0.00

Forces By Torsional Moment: N-S Direction (Structure 1)				
1st Floor	K_i	d_i	$K_i d_i^2$	Force (Kips)
1	2000.00	-88.34	15607911	0.000
2	2000.00	88.33	15604378	0.000
A	1666.67	94.50	14883750	0.000
B	1666.67	-94.50	14883750	0.000
		Total	60979789	0.00

Forces By Torsional Moment: N-S Direction (Structure 1)				
Mezzanine	K_i	d_i	$K_i d_i^2$	Force (Kips)
1	10000.00	-88.34	78039556	0.000
2	10000.00	88.33	78021889	0.000
A	5000.00	94.50	44651250	0.000
B	5000.00	-94.50	44651250	0.000
		Total	245363945	0.00

Forces By Torsional Moment: E-W Direction (Structure 1)				
Roof	K_i	d_i	$K_i d_i^2$	Force (Kips)
1	588.24	-88.34	4590562	2.220
2	588.24	88.34	4590562	-2.220
A	454.55	96.70	4250405	-1.878
B	476.19	-92.30	4056805	1.878
		Total	17488334	0.00

Forces By Torsional Moment: E-W Direction (Structure 1)				
3rd Floor	K_i	d_i	$K_i d_i^2$	Force (Kips)
1	769.23	-88.34	6003043	3.680
2	769.23	88.33	6001684	-3.680
A	625.00	97.55	5947502	-3.302
B	666.67	-91.45	5575402	3.302
		Total	23527630	0.00

Forces By Torsional Moment: E-W Direction (Structure 1)				
2nd Floor	K_i	d_i	$K_i d_i^2$	Force (Kips)
1	1250.00	-83.14	8640325	4.019
2	1111.11	93.53	9719845	-4.019
A	909.09	99.00	8910000	-3.481
B	1000.00	-90.00	8100000	3.481
		Total	35370170	0.00

Forces By Torsional Moment: E-W Direction (Structure 1)				
1st Floor	K_i	d_i	$K_i d_i^2$	Force (Kips)
1	2000.00	-88.34	15607911	0.000
2	2000.00	88.33	15604378	0.000
A	1666.67	94.50	14883750	0.000
B	1666.67	-94.50	14883750	0.000
		Total	60979789	0.00

Forces By Torsional Moment: E-W Direction (Structure 1)				
Mezzanine	K_i	d_i	$K_i d_i^2$	Force (Kips)
1	10000.00	-88.34	78039556	0.000
2	10000.00	88.33	78021889	0.000
A	5000.00	94.50	44651250	0.000
B	5000.00	-94.50	44651250	0.000
		Total	245363945	0.00

Forces By Torsional Moment: N-S Direction (Structure 2)				
Roof	K_i	d_i	$K_i d_i^2$	Force (Kips)
3	625.00	-79.60	3960100	-2.143
4	588.24	84.57	4206810	2.142
C	476.19	101.77	4931968	2.087
D	555.56	-87.23	4227263	-2.087
		Total	17326141	0.00

Forces By Torsional Moment: N-S Direction (Structure 2)				
3rd Floor	K_i	d_i	$K_i d_i^2$	Force (Kips)
3	833.33	-78.80	5174533	-3.592
4	769.23	85.37	5605788	3.592
C	625.00	100.80	6350400	3.446
D	714.29	-88.20	5556600	-3.446
		Total	22687322	0.00

Forces By Torsional Moment: N-S Direction (Structure 2)				
2nd Floor	K_i	d_i	$K_i d_i^2$	Force (Kips)
3	1250.00	-77.26	7461385	-3.727
4	1111.11	86.91	8392030	3.727
C	1000.00	99.47	9894281	3.839
D	1111.11	-89.53	8906245	-3.839
		Total	34653940	0.00

Forces By Torsional Moment: N-S Direction (Structure 2)				
1st Floor	K_i	d_i	$K_i d_i^2$	Force (Kips)
3	2500.00	-72.96	13307904	-4.424
4	2000.00	91.21	16637434	4.424
C	1666.67	103.09	17712580	4.167
D	2000.00	-85.91	14761056	-4.168
		Total	62418974	0.00

Forces By Torsional Moment: N-S Direction (Structure 2)				
Mezzanine	K_i	d_i	$K_i d_i^2$	Force (Kips)
3	10000.00	-54.72	29942784	-0.927
4	5000.00	109.45	59893229	0.927
C	5000.00	94.50	44651250	0.800
D	5000.00	-94.50	44651250	-0.800
		Total	179138513	0.00

Forces By Torsional Moment: E-W Direction (Structure 2)				
Roof	K_i	d_i	$K_i d_i^2$	Force (Kips)
3	625.00	-79.60	3960100	6.262
4	588.24	84.57	4206810	-6.261
C	476.19	101.77	4931968	-6.100
D	555.56	-87.23	4227263	6.100
		Total	17326141	0.00

Forces By Torsional Moment: E-W Direction (Structure 2)				
3rd Floor	K_i	d_i	$K_i d_i^2$	Force (Kips)
3	833.33	-78.80	5174533	6.893
4	769.23	85.37	5605788	-6.893
C	625.00	100.80	6350400	-6.613
D	714.29	-88.20	5556600	6.613
		Total	22687322	0.00

Forces By Torsional Moment: E-W Direction (Structure 2)				
2nd Floor	K_i	d_i	$K_i d_i^2$	Force (Kips)
3	1250.00	-77.26	7461385	3.839
4	1111.11	86.91	8392030	-3.839
C	1000.00	99.47	9894281	-3.955
D	1111.11	-89.53	8906245	3.955
		Total	34653940	0.00

Forces By Torsional Moment: E-W Direction (Structure 2)				
1st Floor	K_i	d_i	$K_i d_i^2$	Force (Kips)
3	2500.00	-72.96	13307904	4.167
4	2000.00	91.21	16637434	-4.168
C	1666.67	103.09	17712580	-3.926
D	2000.00	-85.91	14761056	3.926
		Total	62418974	0.00

Forces By Torsional Moment: E-W Direction (Structure 2)				
Mezzanine	K_i	d_i	$K_i d_i^2$	Force (Kips)
3	10000.00	-54.72	29942784	0.000
4	5000.00	109.45	59893229	0.000
C	5000.00	94.50	44651250	0.000
D	5000.00	-94.50	44651250	0.000
		Total	179138513	0.00

Calculation 10: Torsional Forces on both Structures Due to Seismic

Net Force On Frames (N-S Direction): Structure 1			
Roof	Direct Force (Kips)	Torsional Force (Kips)	Net Force (Kips)
1	21.55	0.00	21.55
2	21.55	0.00	21.55
A	-	0.00	0.00
B	-	0.00	0.00

Net Force On Frames (N-S Direction): Structure 1			
3rd Floor	Direct Force (Kips)	Torsional Force (Kips)	Net Force (Kips)
1	85.80	0.00	85.80
2	85.80	0.00	85.80
A	-	0.00	0.00
B	-	0.00	0.00

Net Force On Frames (N-S Direction): Structure 1			
2nd Floor	Direct Force (Kips)	Torsional Force (Kips)	Net Force (Kips)
1	195.09	-5.63	200.71
2	173.41	5.63	179.04
A	-	4.87	4.87
B	-	-4.87	4.87

Net Force On Frames (N-S Direction): Structure 1			
1st Floor	Direct Force (Kips)	Torsional Force (Kips)	Net Force (Kips)
1	312.30	0.00	312.30
2	312.30	0.00	312.30
A	-	0.00	0.00
B	-	0.00	0.00

Net Force On Frames (N-S Direction): Structure 1			
Mezzanine	Direct Force (Kips)	Torsional Force (Kips)	Net Force (Kips)
1	507.50	0.00	507.50
2	507.50	0.00	507.50
A	-	0.00	0.00
B	-	0.00	0.00

Net Force On Frames (E-W Direction): Structure 1			
Roof	Direct Force (Kips)	Torsional Force (Kips)	Net Force (Kips)
1	-	0.30	0.30
2	-	-0.30	0.30
A	22.17	-0.25	22.42
B	23.23	0.25	23.48

Net Force On Frames (E-W Direction): Structure 1			
3rd Floor	Direct Force (Kips)	Torsional Force (Kips)	Net Force (Kips)
1	-	1.59	1.59
2	-	-1.59	1.59
A	87.53	-1.43	88.96
B	93.37	1.43	94.80

Net Force On Frames (E-W Direction): Structure 1			
2nd Floor	Direct Force (Kips)	Torsional Force (Kips)	Net Force (Kips)
1	-	5.14	5.14
2	-	-5.14	5.14
A	184.95	-4.45	189.40
B	203.45	4.45	207.89

Net Force On Frames (E-W Direction): Structure 1			
1st Floor	Direct Force (Kips)	Torsional Force (Kips)	Net Force (Kips)
1	-	0.00	0.00
2	-	0.00	0.00
A	329.15	0.00	329.15
B	329.15	0.00	329.15

Net Force On Frames (E-W Direction): Structure 1			
Mezzanine	Direct Force (Kips)	Torsional Force (Kips)	Net Force (Kips)
1	-	0.00	0.00
2	-	0.00	0.00
A	535.00	0.00	535.00
B	535.00	0.00	535.00

Net Force On Frames (N-S Direction): Structure 2			
Roof	Direct Force (Kips)	Torsional Force (Kips)	Net Force (Kips)
3	20.66	-0.29	20.94
4	19.44	0.29	19.73
C	-	0.28	0.28
D	-	-0.28	0.28

Net Force On Frames (N-S Direction): Structure 2			
3rd Floor	Direct Force (Kips)	Torsional Force (Kips)	Net Force (Kips)
3	83.04	-1.52	84.56
4	76.66	1.52	78.17
C	-	1.46	1.46
D	-	-1.46	1.46

Net Force On Frames (N-S Direction): Structure 2			
2nd Floor	Direct Force (Kips)	Torsional Force (Kips)	Net Force (Kips)
3	181.48	-4.61	186.10
4	161.32	4.61	165.93
C	-	4.75	4.75
D	-	-4.75	4.75

Net Force On Frames (N-S Direction): Structure 2			
1st Floor	Direct Force (Kips)	Torsional Force (Kips)	Net Force (Kips)
3	322.83	-15.49	338.32
4	258.27	15.49	273.76
C	-	14.59	14.59
D	-	-14.59	14.59

Net Force On Frames (N-S Direction): Structure 2			
Mezzanine	Direct Force (Kips)	Torsional Force (Kips)	Net Force (Kips)
3	629.53	-71.53	701.06
4	314.77	71.53	386.30
C	-	61.76	61.76
D	-	-61.76	61.76

Net Force On Frames (E-W Direction): Structure 2			
Roof	Direct Force (Kips)	Torsional Force (Kips)	Net Force (Kips)
3	-	-1.66	1.66
4	-	1.66	1.66
C	21.00	1.61	22.61
D	24.50	-1.61	26.11

Net Force On Frames (E-W Direction): Structure 2			
3rd Floor	Direct Force (Kips)	Torsional Force (Kips)	Net Force (Kips)
3	-	-6.89	6.89
4	-	6.89	6.89
C	84.51	6.61	91.12
D	96.59	-6.61	103.20

Net Force On Frames (E-W Direction): Structure 2			
2nd Floor	Direct Force (Kips)	Torsional Force (Kips)	Net Force (Kips)
3	-	-16.99	16.99
4	-	16.99	16.99
C	184.22	17.50	201.72
D	204.68	-17.51	222.19

Net Force On Frames (E-W Direction): Structure 2			
1st Floor	Direct Force (Kips)	Torsional Force (Kips)	Net Force (Kips)
3	-	-33.68	33.68
4	-	33.69	33.69
C	299.68	31.73	331.41
D	359.62	-31.73	391.35

Net Force On Frames (E-W Direction): Structure 2			
Mezzanine	Direct Force (Kips)	Torsional Force (Kips)	Net Force (Kips)
3	-	-71.53	71.53
4	-	71.53	71.53
C	535.80	61.76	597.56
D	535.80	-61.76	597.56

Calculation 11: Net Force on Both Structures Due to Wind

Net Force On Frames (N-S Direction): Structure 1			
Roof	Direct Force (Kips)	Torsional Force (Kips)	Net Force (Kips)
1	170.00	0.00	170.00
2	170.00	0.00	170.00
A	-	0.00	0.00
B	-	0.00	0.00

Net Force On Frames (N-S Direction): Structure 1			
3rd Floor	Direct Force (Kips)	Torsional Force (Kips)	Net Force (Kips)
1	209.00	0.00	209.00
2	209.00	0.00	209.00
A	-	0.00	0.00
B	-	0.00	0.00

Net Force On Frames (N-S Direction): Structure 1			
2nd Floor	Direct Force (Kips)	Torsional Force (Kips)	Net Force (Kips)
1	160.94	-4.64	165.58
2	143.06	4.64	147.70
A	-	4.02	4.02
B	-	-4.02	4.02

Net Force On Frames (N-S Direction): Structure 1			
1st Floor	Direct Force (Kips)	Torsional Force (Kips)	Net Force (Kips)
1	97.50	0.00	97.50
2	97.50	0.00	97.50
A	-	0.00	0.00
B	-	0.00	0.00

Net Force On Frames (N-S Direction): Structure 1			
Mezzanine	Direct Force (Kips)	Torsional Force (Kips)	Net Force (Kips)
1	43.00	0.00	43.00
2	43.00	0.00	43.00
A	-	0.00	0.00
B	-	0.00	0.00

Net Force On Frames (E-W Direction): Structure 1			
Roof	Direct Force (Kips)	Torsional Force (Kips)	Net Force (Kips)
1	-	2.22	2.22
2	-	-2.22	2.22
A	166.05	-1.88	167.92
B	173.95	1.88	175.83

Net Force On Frames (E-W Direction): Structure 1			
3rd Floor	Direct Force (Kips)	Torsional Force (Kips)	Net Force (Kips)
1	-	3.68	3.68
2	-	-3.68	3.68
A	202.26	-3.30	205.56
B	215.74	3.30	219.04

Net Force On Frames (E-W Direction): Structure 1			
2nd Floor	Direct Force (Kips)	Torsional Force (Kips)	Net Force (Kips)
1	-	4.02	4.02
2	-	-4.02	4.02
A	144.76	-3.48	148.24
B	159.24	3.48	162.72

Net Force On Frames (E-W Direction): Structure 1			
1st Floor	Direct Force (Kips)	Torsional Force (Kips)	Net Force (Kips)
1	-	0.00	0.00
2	-	0.00	0.00
A	97.50	0.00	97.50
B	97.50	0.00	97.50

Net Force On Frames (E-W Direction): Structure 1			
Mezzanine	Direct Force (Kips)	Torsional Force (Kips)	Net Force (Kips)
1	-	0.00	0.00
2	-	0.00	0.00
A	43.00	0.00	43.00
B	43.00	0.00	43.00

Net Force On Frames (N-S Direction): Structure 2			
Roof	Direct Force (Kips)	Torsional Force (Kips)	Net Force (Kips)
3	154.55	-2.14	156.69
4	145.45	2.14	147.60
C	-	2.09	2.09
D	-	-2.09	2.09

Net Force On Frames (N-S Direction): Structure 2			
3rd Floor	Direct Force (Kips)	Torsional Force (Kips)	Net Force (Kips)
3	196.56	-3.59	200.15
4	181.44	3.59	185.03
C	-	3.45	3.45
D	-	-3.45	3.45

Net Force On Frames (N-S Direction): Structure 2			
2nd Floor	Direct Force (Kips)	Torsional Force (Kips)	Net Force (Kips)
3	146.65	-3.73	150.37
4	130.35	3.73	134.08
C	-	3.84	3.84
D	-	-3.84	3.84

Net Force On Frames (N-S Direction): Structure 2			
1st Floor	Direct Force (Kips)	Torsional Force (Kips)	Net Force (Kips)
3	92.22	-4.42	96.65
4	73.78	4.42	78.20
C	-	4.17	4.17
D	-	-4.17	4.17

Net Force On Frames (N-S Direction): Structure 2			
Mezzanine	Direct Force (Kips)	Torsional Force (Kips)	Net Force (Kips)
3	56.00	-0.93	56.93
4	28.00	0.93	28.93
C	-	0.80	0.80
D	-	-0.80	0.80

Net Force On Frames (E-W Direction): Structure 2			
Roof	Direct Force (Kips)	Torsional Force (Kips)	Net Force (Kips)
3	-	6.26	6.26
4	-	-6.26	6.26
C	138.46	-6.10	144.56
D	161.54	6.10	167.64

Net Force On Frames (E-W Direction): Structure 2			
3rd Floor	Direct Force (Kips)	Torsional Force (Kips)	Net Force (Kips)
3	-	6.89	6.89
4	-	-6.89	6.89
C	176.40	-6.61	183.01
D	201.60	6.61	208.21

Net Force On Frames (E-W Direction): Structure 2			
2nd Floor	Direct Force (Kips)	Torsional Force (Kips)	Net Force (Kips)
3	-	3.84	3.84
4	-	-3.84	3.84
C	131.21	-3.95	135.17
D	145.79	3.95	149.74

Net Force On Frames (E-W Direction): Structure 2			
1st Floor	Direct Force (Kips)	Torsional Force (Kips)	Net Force (Kips)
3	-	4.17	4.17
4	-	-4.17	4.17
C	75.45	-3.93	79.38
D	90.55	3.93	94.47

Net Force On Frames (E-W Direction): Structure 2			
Mezzanine	Direct Force (Kips)	Torsional Force (Kips)	Net Force (Kips)
3	-	0.00	0.00
4	-	0.00	0.00
C	42.00	0.00	42.00
D	42.00	0.00	42.00

Calculation 12: Net Force on Both Structures Due to Seismic

APPENDIX G: DRIFT RESULTS

Controlling Wind Drift: Frame 1

Story	Story Height (ft)	Story Drift (in)	Allowable Story Drift (in) $D_{wind} = H/400$			Total Drift (in)	Allowable Total Drift (in) $D_{wind} = H/400$		
Roof	11.500	0.041	<	0.345	Acceptable	0.509	<	1.94	Acceptable
3rd	11.500	0.049	<	0.345	Acceptable	0.468	<	1.595	Acceptable
2nd	11.500	0.140	<	0.345	Acceptable	0.419	<	1.25	Acceptable
1st	12.250	0.150	<	0.368	Acceptable	0.278	<	0.905	Acceptable
Mezzanine	17.917	0.129	<	0.538	Acceptable	0.129	<	0.5375	Acceptable

Controlling Wind Drift: Frame 2

Story	Story Height (ft)	Story Drift (in)	Allowable Story Drift (in) $D_{wind} = H/400$			Total Drift (in)	Allowable Total Drift (in) $D_{wind} = H/400$		
Roof	11.500	0.042	<	0.345	Acceptable	0.514	<	1.94	Acceptable
3rd	11.500	0.050	<	0.345	Acceptable	0.472	<	1.595	Acceptable
2nd	11.500	0.136	<	0.345	Acceptable	0.421	<	1.25	Acceptable
1st	12.250	0.147	<	0.368	Acceptable	0.285	<	0.905	Acceptable
Mezzanine	17.917	0.139	<	0.538	Acceptable	0.139	<	0.5375	Acceptable

Controlling Wind Drift: Frame A

Story	Story Height (ft)	Story Drift (in)	Allowable Story Drift (in) $D_{wind} = H/400$			Total Drift (in)	Allowable Total Drift (in) $D_{wind} = H/400$		
Roof	11.500	0.079	<	0.345	Acceptable	0.681	<	1.94	Acceptable
3rd	11.500	0.094	<	0.345	Acceptable	0.602	<	1.595	Acceptable
2nd	11.500	0.161	<	0.345	Acceptable	0.508	<	1.25	Acceptable
1st	12.250	0.161	<	0.368	Acceptable	0.347	<	0.905	Acceptable
Mezzanine	17.917	0.186	<	0.538	Acceptable	0.186	<	0.5375	Acceptable

Controlling Wind Drift: Frame B

Story	Story Height (ft)	Story Drift (in)	Allowable Story Drift (in) $D_{wind} = H/400$			Total Drift (in)	Allowable Total Drift (in) $D_{wind} = H/400$		
Roof	11.500	0.075	<	0.345	Acceptable	0.674	<	1.94	Acceptable
3rd	11.500	0.089	<	0.345	Acceptable	0.599	<	1.595	Acceptable
2nd	11.500	0.161	<	0.345	Acceptable	0.510	<	1.25	Acceptable
1st	12.250	0.160	<	0.368	Acceptable	0.349	<	0.905	Acceptable
Mezzanine	17.917	0.188	<	0.538	Acceptable	0.188	<	0.5375	Acceptable

Table 1: Drift Results for Structure 1 Due to Wind

Controlling Wind Drift: Frame 3

Story	Story Height (ft)	Story Drift (in)	Allowable Story Drift (in) $D_{wind} = H/400$			Total Drift (in)	Allowable Total Drift (in) $D_{wind} = H/400$		
Roof	11.500	0.036	<	0.345	Acceptable	0.489	<	1.94	Acceptable
3rd	11.500	0.045	<	0.345	Acceptable	0.453	<	1.595	Acceptable
2nd	11.500	0.135	<	0.345	Acceptable	0.408	<	1.25	Acceptable
1st	12.250	0.146	<	0.368	Acceptable	0.273	<	0.905	Acceptable
Mezzanine	17.917	0.127	<	0.538	Acceptable	0.127	<	0.5375	Acceptable

Controlling Wind Drift: Frame 4

Story	Story Height (ft)	Story Drift (in)	Allowable Story Drift (in) $D_{wind} = H/400$			Total Drift (in)	Allowable Total Drift (in) $D_{wind} = H/400$		
Roof	11.500	0.034	<	0.345	Acceptable	0.477	<	1.94	Acceptable
3rd	11.500	0.041	<	0.345	Acceptable	0.443	<	1.595	Acceptable
2nd	11.500	0.119	<	0.345	Acceptable	0.402	<	1.25	Acceptable
1st	12.250	0.122	<	0.368	Acceptable	0.282	<	0.905	Acceptable
Mezzanine	17.917	0.160	<	0.538	Acceptable	0.160	<	0.5375	Acceptable

Controlling Wind Drift: Frame C

Story	Story Height (ft)	Story Drift (in)	Allowable Story Drift (in) $D_{wind} = H/400$			Total Drift (in)	Allowable Total Drift (in) $D_{wind} = H/400$		
Roof	11.500	0.075	<	0.345	Acceptable	0.681	<	1.94	Acceptable
3rd	11.500	0.090	<	0.345	Acceptable	0.606	<	1.595	Acceptable
2nd	11.500	0.161	<	0.345	Acceptable	0.517	<	1.25	Acceptable
1st	12.250	0.160	<	0.368	Acceptable	0.356	<	0.905	Acceptable
Mezzanine	17.917	0.196	<	0.538	Acceptable	0.196	<	0.5375	Acceptable

Controlling Wind Drift: Frame D

Story	Story Height (ft)	Story Drift (in)	Allowable Story Drift (in) $D_{wind} = H/400$			Total Drift (in)	Allowable Total Drift (in) $D_{wind} = H/400$		
Roof	11.500	0.060	<	0.345	Acceptable	0.677	<	1.94	Acceptable
3rd	11.500	0.079	<	0.345	Acceptable	0.617	<	1.595	Acceptable
2nd	11.500	0.166	<	0.345	Acceptable	0.538	<	1.25	Acceptable
1st	12.250	0.168	<	0.368	Acceptable	0.372	<	0.905	Acceptable
Mezzanine	17.917	0.204	<	0.538	Acceptable	0.204	<	0.5375	Acceptable

Table 2: Drift Results for Structure 2 Due to Wind

Controlling Seismic Drift: Frame 1

Story	Story Height (ft)	Story Drift (in)	Allowable Story Drift (in) $D_{SEISMIC} = 0.020h_{sx}$			Total Drift (in)	Allowable Story Drift (in) $D_{SEISMIC} = 0.020h_{sx}$		
Roof	11.500	0.129	<	0.230	Acceptable	0.738	<	1.293	Acceptable
3rd	11.500	0.140	<	0.230	Acceptable	0.609	<	1.063	Acceptable
2nd	11.500	0.201	<	0.230	Acceptable	0.469	<	0.833	Acceptable
1st	12.250	0.189	<	0.245	Acceptable	0.269	<	0.603	Acceptable
Mezzanine	17.917	0.080	<	0.358	Acceptable	0.080	<	0.358	Acceptable

Controlling Seismic Drift: Frame 2

Story	Story Height (ft)	Story Drift (in)	Allowable Story Drift (in) $D_{SEISMIC} = 0.020h_{sx}$			Total Drift (in)	Allowable Story Drift (in) $D_{SEISMIC} = 0.020h_{sx}$		
Roof	11.500	0.127	<	0.230	Acceptable	0.742	<	1.293	Acceptable
3rd	11.500	0.143	<	0.230	Acceptable	0.616	<	1.063	Acceptable
2nd	11.500	0.199	<	0.230	Acceptable	0.472	<	0.833	Acceptable
1st	12.250	0.189	<	0.245	Acceptable	0.273	<	0.603	Acceptable
Mezzanine	17.917	0.085	<	0.358	Acceptable	0.085	<	0.358	Acceptable

Controlling Seismic Drift: Frame A

Story	Story Height (ft)	Story Drift (in)	Allowable Story Drift (in) $D_{SEISMIC} = 0.020h_{sx}$			Total Drift (in)	Allowable Story Drift (in) $D_{SEISMIC} = 0.020h_{sx}$		
Roof	11.500	0.197	<	0.230	Acceptable	0.932	<	1.293	Acceptable
3rd	11.500	0.192	<	0.230	Acceptable	0.736	<	1.063	Acceptable
2nd	11.500	0.218	<	0.230	Acceptable	0.544	<	0.833	Acceptable
1st	12.250	0.217	<	0.245	Acceptable	0.326	<	0.603	Acceptable
Mezzanine	17.917	0.110	<	0.358	Acceptable	0.110	<	0.358	Acceptable

Controlling Seismic Drift: Frame B

Story	Story Height (ft)	Story Drift (in)	Allowable Story Drift (in) $D_{SEISMIC} = 0.020h_{sx}$			Total Drift (in)	Allowable Story Drift (in) $D_{SEISMIC} = 0.020h_{sx}$		
Roof	11.500	0.193	<	0.230	Acceptable	0.927	<	1.293	Acceptable
3rd	11.500	0.186	<	0.230	Acceptable	0.734	<	1.063	Acceptable
2nd	11.500	0.218	<	0.230	Acceptable	0.548	<	0.833	Acceptable
1st	12.250	0.216	<	0.245	Acceptable	0.330	<	0.603	Acceptable
Mezzanine	17.917	0.114	<	0.358	Acceptable	0.114	<	0.358	Acceptable

Table 3: Drift Results for Structure 1 Due to Seismic

Controlling Seismic Drift: Frame 3									
Story	Story Height (ft)	Story Drift (in)	Allowable Story Drift (in) $D_{SEISMIC} = 0.020h_{sx}$			Total Drift (in)	Allowable Story Drift (in) $D_{SEISMIC} = 0.020h_{sx}$		
Roof	11.500	0.110	<	0.230	Acceptable	0.653	<	1.293	Acceptable
3rd	11.500	0.126	<	0.230	Acceptable	0.543	<	1.063	Acceptable
2nd	11.500	0.183	<	0.230	Acceptable	0.418	<	0.833	Acceptable
1st	12.250	0.171	<	0.245	Acceptable	0.235	<	0.603	Acceptable
Mezzanine	17.917	0.064	<	0.358	Acceptable	0.064	<	0.358	Acceptable

Controlling Seismic Drift: Frame C									
Story	Story Height (ft)	Story Drift (in)	Allowable Story Drift (in) $D_{SEISMIC} = 0.020h_{sx}$			Total Drift (in)	Allowable Story Drift (in) $D_{SEISMIC} = 0.020h_{sx}$		
Roof	11.500	0.163	<	0.230	Acceptable	0.781	<	1.293	Acceptable
3rd	11.500	0.157	<	0.230	Acceptable	0.618	<	1.063	Acceptable
2nd	11.500	0.184	<	0.230	Acceptable	0.461	<	0.833	Acceptable
1st	12.250	0.182	<	0.245	Acceptable	0.277	<	0.603	Acceptable
Mezzanine	17.917	0.095	<	0.358	Acceptable	0.095	<	0.358	Acceptable

Controlling Seismic Drift: Frame 4									
Story	Story Height (ft)	Story Drift (in)	Allowable Story Drift (in) $D_{SEISMIC} = 0.020h_{sx}$			Total Drift (in)	Allowable Story Drift (in) $D_{SEISMIC} = 0.020h_{sx}$		
Roof	11.500	0.103	<	0.230	Acceptable	0.641	<	1.293	Acceptable
3rd	11.500	0.117	<	0.230	Acceptable	0.538	<	1.063	Acceptable
2nd	11.500	0.167	<	0.230	Acceptable	0.421	<	0.833	Acceptable
1st	12.250	0.153	<	0.245	Acceptable	0.253	<	0.603	Acceptable
Mezzanine	17.917	0.100	<	0.358	Acceptable	0.100	<	0.358	Acceptable

Controlling Seismic Drift: Frame D									
Story	Story Height (ft)	Story Drift (in)	Allowable Story Drift (in) $D_{SEISMIC} = 0.020h_{sx}$			Total Drift (in)	Allowable Story Drift (in) $D_{SEISMIC} = 0.020h_{sx}$		
Roof	11.500	0.149	<	0.230	Acceptable	0.774	<	1.293	Acceptable
3rd	11.500	0.147	<	0.230	Acceptable	0.625	<	1.063	Acceptable
2nd	11.500	0.189	<	0.230	Acceptable	0.478	<	0.833	Acceptable
1st	12.250	0.185	<	0.245	Acceptable	0.289	<	0.603	Acceptable
Mezzanine	17.917	0.104	<	0.358	Acceptable	0.104	<	0.358	Acceptable

Table 4: Drift Results for Structure 2 Due to Seismic

APPENDIX H: STEEL CONNECTIONS

Pin Connection Plate At Bottom of Truss 1 Structure 1

Member	15	
P_u (Kips)	255.82	[For 1 Member]
F_y (ksi)	50	
F_u (ksi)	65	
t (in)	1.125	
b_{eff} (in)	2.33	
	2.88	Controls
w (in)	10	
Tension Rupture		
φP_n (Kips)	315.9	Okay
Shear Rupture		
a (in)	3	
b (in)	3	
d (in)	4	
A_{sf} (in^2)	11.25	
φP_n (Kips)	438.75	Okay
Bearing On Pin		
A_{pb} (in^2)	4.5	
φP_n (Kips)	303.75	Okay
Tension Tielding		
φP_n (Kips)	506.25	Okay

10 in x 1.125 in pin-connected member with a 4 in pin

Pin Connection Plate At Bottom of Truss 1 Structure 1

Member	16	
P_u (Kips)	328.56	[For 1 Member]
F_y (ksi)	50	
F_u (ksi)	65	
t (in)	1.25	
b_{eff} (in)	2.70	
	3.13	Controls
w (in)	10.5	
Tension Rupture		
φP_n (Kips)	381.47	Okay
Shear Rupture		
a (in)	3.25	
b (in)	3.25	
d (in)	4	
A_{sf} (in^2)	13.125	
φP_n (Kips)	511.875	Okay
Bearing On Pin		
A_{pb} (in^2)	5	
φP_n (Kips)	337.5	Okay
Tension Tielding		
φP_n (Kips)	590.625	Okay

10.5 in x 1.25 in pin-connected member with a 4 in pin

Pin Connection Plate At Bottom of Truss 1 Structure 1

Member	19	
P_u (Kips)	175.21	[For 1 Member]
F_y (ksi)	50	
F_u (ksi)	65	
t (in)	1.25	
b_{eff} (in)	1.44	
	3.13	Controls
w (in)	10.5	
Tension Rupture		
φP_n (Kips)	381.47	Okay
Shear Rupture		
a (in)	3.25	
b (in)	3.25	
d (in)	4	
A_{sf} (in^2)	13.125	
φP_n (Kips)	511.875	Okay
Bearing On Pin		
A_{pb} (in^2)	5	
φP_n (Kips)	337.5	Okay
Tension Tielding		
φP_n (Kips)	590.625	Okay

10.5 in x 1.25 in pin-connected member with a 4 in pin

Pin Connection Plate At Bottom of Truss 1 Structure 1

Member	20	
P_u (Kips)	135.59	[For 1 Member]
F_y (ksi)	50	
F_u (ksi)	65	
t (in)	1.125	
b_{eff} (in)	1.24	
	2.88	Controls
w (in)	10	
Tension Rupture		
φP_n (Kips)	315.90	Okay
Shear Rupture		
a (in)	3	
b (in)	3	
d (in)	4	
A_{sf} (in^2)	11.25	
φP_n (Kips)	438.75	Okay
Bearing On Pin		
A_{pb} (in^2)	4.5	
φP_n (Kips)	303.75	Okay
Tension Tielding		
φP_n (Kips)	506.25	Okay

10 in x 1.125 in pin-connected member with a 4 in pin

Calculation 1: Pin & Plate Size for Trusses

Pin Connection Plate At Bottom of Truss 2 Structure 1

Member	15	
P_u (Kips)	495.03	[For 1 Member]
F_y (ksi)	50	
F_u (ksi)	65	
t (in)	1.5	
b_{eff} (in)	3.38	
	3.63	Controls
w (in)	14	
Tension Rupture		
ϕP_n (Kips)	530.89	Okay
Shear Rupture		
a (in)	4	
b (in)	4	
d (in)	6	
A_{sf} (in^2)	21	
ϕP_n (Kips)	819	Okay
Bearing On Pin		
A_{pb} (in^2)	9	
ϕP_n (Kips)	607.5	Okay
Tension Tielding		
ϕP_n (Kips)	945	Okay

14 in x 1.5 in pin-connected member with (2) 3 in pins

Pin Connection Plate At Bottom of Truss 2 Structure 1

Member	16	
P_u (Kips)	642.37	[For 1 Member]
F_y (ksi)	50	
F_u (ksi)	65	
t (in)	1.75	
b_{eff} (in)	3.76	
	4.13	Controls
w (in)	14.5	
Tension Rupture		
ϕP_n (Kips)	704.68	Okay
Shear Rupture		
a (in)	4.25	
b (in)	4.25	
d (in)	6	
A_{sf} (in^2)	25.375	
ϕP_n (Kips)	989.625	Okay
Bearing On Pin		
A_{pb} (in^2)	10.5	
ϕP_n (Kips)	708.75	Okay
Tension Tielding		
ϕP_n (Kips)	1141.875	Okay

14.5 in x 1.75 in pin-connected member with (2) 3 in pins

Pin Connection Plate At Bottom of Truss 2 Structure 1

Member	19	
P_u (Kips)	478.13	[For 1 Member]
F_y (ksi)	50	
F_u (ksi)	65	
t (in)	1.5	
b_{eff} (in)	3.27	
	3.63	Controls
w (in)	14	
Tension Rupture		
ϕP_n (Kips)	530.89	Okay
Shear Rupture		
a (in)	4	
b (in)	4	
d (in)	6	
A_{sf} (in^2)	21	
ϕP_n (Kips)	819	Okay
Bearing On Pin		
A_{pb} (in^2)	9	
ϕP_n (Kips)	607.5	Okay
Tension Tielding		
ϕP_n (Kips)	945	Okay

14 in x 1.5 in pin-connected member with (2) 3 in pins

Pin Connection Plate At Bottom of Truss 2 Structure 1

Member	20	
P_u (Kips)	365.15	[For 1 Member]
F_y (ksi)	50	
F_u (ksi)	65	
t (in)	1.5	
b_{eff} (in)	2.50	
	3.63	Controls
w (in)	14	
Tension Rupture		
ϕP_n (Kips)	530.89	Okay
Shear Rupture		
a (in)	4	
b (in)	4	
d (in)	6	
A_{sf} (in^2)	21	
ϕP_n (Kips)	819	Okay
Bearing On Pin		
A_{pb} (in^2)	9	
ϕP_n (Kips)	607.5	Okay
Tension Tielding		
ϕP_n (Kips)	945	Okay

14 in x 1.5 in pin-connected member with (2) 3 in pins

Calculation 2: Pin & Plate Size for Trusses Cont'd

Pin Connection Plate At Bottom of Truss 3 Structure 1

Member	15	
P_u (Kips)	515.245	[For 1 Member]
F_y (ksi)	50	
F_u (ksi)	65	
t (in)	1.75	
b_{eff} (in)	3.02	
	4.13	Controls
w (in)	14.5	
Tension Rupture		
φP_n (Kips)	704.68	Okay
Shear Rupture		
a (in)	4.25	
b (in)	4.25	
d (in)	6	
A_{sf} (in^2)	25.375	
φP_n (Kips)	989.625	Okay
Bearing On Pin		
A_{pb} (in^2)	10.5	
φP_n (Kips)	708.75	Okay
Tension Tielding		
φP_n (Kips)	1141.875	Okay

14.5 in x 1.75 in pin-connected member with (2) 3 in pins

Pin Connection Plate At Bottom of Truss 3 Structure 1

Member	16	
P_u (Kips)	682.66	[For 1 Member]
F_y (ksi)	50	
F_u (ksi)	65	
t (in)	1.75	
b_{eff} (in)	4.00	
	4.13	Controls
w (in)	14.5	
Tension Rupture		
φP_n (Kips)	704.68	Okay
Shear Rupture		
a (in)	4.25	
b (in)	4.25	
d (in)	6	
A_{sf} (in^2)	25.375	
φP_n (Kips)	989.625	Okay
Bearing On Pin		
A_{pb} (in^2)	10.5	
φP_n (Kips)	708.75	Okay
Tension Tielding		
φP_n (Kips)	1141.875	Okay

14.5 in x 1.75 in pin-connected member with (2) 3 in pins

Pin Connection Plate At Bottom of Truss 3 Structure 1

Member	19	
P_u (Kips)	682.66	[For 1 Member]
F_y (ksi)	50	
F_u (ksi)	65	
t (in)	1.75	
b_{eff} (in)	4.00	
	4.13	Controls
w (in)	14.5	
Tension Rupture		
φP_n (Kips)	704.68	Okay
Shear Rupture		
a (in)	4.25	
b (in)	4.25	
d (in)	6	
A_{sf} (in^2)	25.375	
φP_n (Kips)	989.625	Okay
Bearing On Pin		
A_{pb} (in^2)	10.5	
φP_n (Kips)	708.75	Okay
Tension Tielding		
φP_n (Kips)	1141.875	Okay

14.5 in x 1.75 in pin-connected member with (2) 3 in pins

Pin Connection Plate At Bottom of Truss 3 Structure 1

Member	20	
P_u (Kips)	515.245	[For 1 Member]
F_y (ksi)	50	
F_u (ksi)	65	
t (in)	1.75	
b_{eff} (in)	3.02	
	4.13	Controls
w (in)	14.5	
Tension Rupture		
φP_n (Kips)	704.68	Okay
Shear Rupture		
a (in)	4.25	
b (in)	4.25	
d (in)	6	
A_{sf} (in^2)	25.375	
φP_n (Kips)	989.625	Okay
Bearing On Pin		
A_{pb} (in^2)	10.5	
φP_n (Kips)	708.75	Okay
Tension Tielding		
φP_n (Kips)	1141.875	Okay

14.5 in x 1.75 in pin-connected member with (2) 3 in pins

Calculation 3: Pin & Plate Size for Trusses Cont'd

Pin Connection Plate At Bottom of Truss 4 Structure
1

Member	15	
P_u (Kips)	928.145	[For 1 Member]
F_y (ksi)	50	
F_u (ksi)	65	
t (in)	2.125	
b_{eff} (in)	4.48	
	4.88	Controls
w (in)	18	
Tension Rupture		
φP_n (Kips)	1011.08	Okay
Shear Rupture		
a (in)	5	
b (in)	5	
d (in)	8	
A_{sf} (in^2)	38.25	
φP_n (Kips)	1491.75	Okay
Bearing On Pin		
A_{pb} (in^2)	17	
φP_n (Kips)	1147.5	Okay
Tension Tielding		
φP_n (Kips)	1721.25	Okay

18 in x 2.125 in pin-connected member with (2) 4 in pins

Pin Connection Plate At Bottom of Truss 4 Structure
1

Member	16	
P_u (Kips)	1208.12	[For 1 Member]
F_y (ksi)	50	
F_u (ksi)	65	
t (in)	2.375	
b_{eff} (in)	5.22	
	5.38	Controls
w (in)	19	
Tension Rupture		
φP_n (Kips)	1245.81	Okay
Shear Rupture		
a (in)	5.5	
b (in)	5.5	
d (in)	8	
A_{sf} (in^2)	45.125	
φP_n (Kips)	1759.875	Okay
Bearing On Pin		
A_{pb} (in^2)	19	
φP_n (Kips)	1282.5	Okay
Tension Tielding		
φP_n (Kips)	2030.625	Okay

19 in x 2.375 in pin-connected member with (2) 4 in pins

Pin Connection Plate At Bottom of Truss 4 Structure
1

Member	19	
P_u (Kips)	1238.41	[For 1 Member]
F_y (ksi)	50	
F_u (ksi)	65	
t (in)	2.375	
b_{eff} (in)	5.35	
	5.38	Controls
w (in)	19	
Tension Rupture		
φP_n (Kips)	1245.81	Okay
Shear Rupture		
a (in)	5.5	
b (in)	5.5	
d (in)	8	
A_{sf} (in^2)	45.125	
φP_n (Kips)	1759.875	Okay
Bearing On Pin		
A_{pb} (in^2)	19	
φP_n (Kips)	1282.5	Okay
Tension Tielding		
φP_n (Kips)	2030.625	Okay

19 in x 2.375 in pin-connected member with (2) 4 in pins

Pin Connection Plate At Bottom of Truss 4 Structure
1

Member	20	
P_u (Kips)	897.855	[For 1 Member]
F_y (ksi)	50	
F_u (ksi)	65	
t (in)	2.125	
b_{eff} (in)	4.33	
	4.88	Controls
w (in)	18	
Tension Rupture		
φP_n (Kips)	1011.08	Okay
Shear Rupture		
a (in)	5	
b (in)	5	
d (in)	8	
A_{sf} (in^2)	38.25	
φP_n (Kips)	1491.75	Okay
Bearing On Pin		
A_{pb} (in^2)	17	
φP_n (Kips)	1147.5	Okay
Tension Tielding		
φP_n (Kips)	1721.25	Okay

18 in x 2.125 in pin-connected member with (2) 4 in pins

Calculation 4: Pin & Plate Size for Trusses Cont'd

Pin Connection Plate At Bottom of Truss 5 Structure 1		
Member	15	
P_u (Kips)	219.4	[For 1 Member]
F_y (ksi)	50	
F_u (ksi)	65	
t (in)	1.125	
b_{eff} (in)	2.00	
	2.88	Controls
w (in)	10	
Tension Rupture		
φP_n (Kips)	315.9	Okay
Shear Rupture		
a (in)	3	
b (in)	3	
d (in)	4	
A_{sf} (in^2)	11.25	
φP_n (Kips)	438.75	Okay
Bearing On Pin		
A_{pb} (in^2)	4.5	
φP_n (Kips)	303.75	Okay
Tension Tielding		
φP_n (Kips)	506.25	Okay
10 in x 1.125 in pin-connected member with a 4 in pin		

Pin Connection Plate At Bottom of Truss 5 Structure 1		
Member	16	
P_u (Kips)	287.00	[For 1 Member]
F_y (ksi)	50	
F_u (ksi)	65	
t (in)	1.25	
b_{eff} (in)	2.35	
	3.13	Controls
w (in)	10.5	
Tension Rupture		
φP_n (Kips)	381.47	Okay
Shear Rupture		
a (in)	3.25	
b (in)	3.25	
d (in)	4	
A_{sf} (in^2)	13.125	
φP_n (Kips)	511.875	Okay
Bearing On Pin		
A_{pb} (in^2)	5	
φP_n (Kips)	337.5	Okay
Tension Tielding		
φP_n (Kips)	590.625	Okay
10.5 in x 1.25 in pin-connected member with a 4 in pin		

Pin Connection Plate At Bottom of Truss 5 Structure 1		
Member	19	
P_u (Kips)	222.48	[For 1 Member]
F_y (ksi)	50	
F_u (ksi)	65	
t (in)	1.25	
b_{eff} (in)	1.83	
	3.13	Controls
w (in)	10.5	
Tension Rupture		
φP_n (Kips)	381.47	Okay
Shear Rupture		
a (in)	3.25	
b (in)	3.25	
d (in)	4	
A_{sf} (in^2)	13.125	
φP_n (Kips)	511.875	Okay
Bearing On Pin		
A_{pb} (in^2)	5	
φP_n (Kips)	337.5	Okay
Tension Tielding		
φP_n (Kips)	590.625	Okay
10.5 in x 1.25 in pin-connected member with a 4 in pin		

Pin Connection Plate At Bottom of Truss 5 Structure 1		
Member	20	
P_u (Kips)	163.17	[For 1 Member]
F_y (ksi)	50	
F_u (ksi)	65	
t (in)	1.125	
b_{eff} (in)	1.49	
	2.88	Controls
w (in)	10.5	
Tension Rupture		
φP_n (Kips)	315.90	Okay
Shear Rupture		
a (in)	3.25	
b (in)	3.25	
d (in)	4	
A_{sf} (in^2)	11.8125	
φP_n (Kips)	460.6875	Okay
Bearing On Pin		
A_{pb} (in^2)	4.5	
φP_n (Kips)	303.75	Okay
Tension Tielding		
φP_n (Kips)	531.5625	Okay
10.5 in x 1.125 in pin-connected member with a 4 in pin		

Calculation 5: Pin & Plate Size for Trusses Cont'd

Pin Connection Plate At Bottom of Truss 6 Structure 2

Member	15	
P_u (Kips)	271.15	[For 1 Member]
F_y (ksi)	50	
F_u (ksi)	65	
t (in)	1.125	
b_{eff} (in)	2.47	
	2.88	Controls
w (in)	10	
Tension Rupture		
φP_n (Kips)	315.9	Okay
Shear Rupture		
a (in)	3	
b (in)	3	
d (in)	4	
A_{sf} (in^2)	11.25	
φP_n (Kips)	438.75	Okay
Bearing On Pin		
A_{pb} (in^2)	4.5	
φP_n (Kips)	303.75	Okay
Tension Tielding		
φP_n (Kips)	506.25	Okay

10 in x 1.125 in pin-connected member with a 4 in pin

Pin Connection Plate At Bottom of Truss 6 Structure 2

Member	16	
P_u (Kips)	357.79	[For 1 Member]
F_y (ksi)	50	
F_u (ksi)	65	
t (in)	1.375	
b_{eff} (in)	2.67	
	3.38	Controls
w (in)	10.5	
Tension Rupture		
φP_n (Kips)	453.13	Okay
Shear Rupture		
a (in)	3.25	
b (in)	3.25	
d (in)	4	
A_{sf} (in^2)	14.4375	
φP_n (Kips)	563.0625	Okay
Bearing On Pin		
A_{pb} (in^2)	5.5	
φP_n (Kips)	371.25	Okay
Tension Tielding		
φP_n (Kips)	649.6875	Okay

10.5 in x 1.375 in pin-connected member with a 4 in pin

Pin Connection Plate At Bottom of Truss 6 Structure 2

Member	19	
P_u (Kips)	357.79	[For 1 Member]
F_y (ksi)	50	
F_u (ksi)	65	
t (in)	1.375	
b_{eff} (in)	2.67	
	3.38	Controls
w (in)	10.5	
Tension Rupture		
φP_n (Kips)	453.13	Okay
Shear Rupture		
a (in)	3.25	
b (in)	3.25	
d (in)	4	
A_{sf} (in^2)	14.4375	
φP_n (Kips)	563.0625	Okay
Bearing On Pin		
A_{pb} (in^2)	5.5	
φP_n (Kips)	371.25	Okay
Tension Tielding		
φP_n (Kips)	649.6875	Okay

10.5 in x 1.375 in pin-connected member with a 4 in pin

Pin Connection Plate At Bottom of Truss 6 Structure 2

Member	20	
P_u (Kips)	271.15	[For 1 Member]
F_y (ksi)	50	
F_u (ksi)	65	
t (in)	1.125	
b_{eff} (in)	2.47	
	2.88	Controls
w (in)	10	
Tension Rupture		
φP_n (Kips)	315.90	Okay
Shear Rupture		
a (in)	3	
b (in)	3	
d (in)	4	
A_{sf} (in^2)	11.25	
φP_n (Kips)	438.75	Okay
Bearing On Pin		
A_{pb} (in^2)	4.5	
φP_n (Kips)	303.75	Okay
Tension Tielding		
φP_n (Kips)	506.25	Okay

10 in x 1.125 in pin-connected member with a 4 in pin

Calculation 6: Pin & Plate Size for Trusses Cont'd

Pin Connection Plate At Bottom of Truss 7 Structure 2

Member	15	
P_u (Kips)	505.975	[For 1 Member]
F_y (ksi)	50	
F_u (ksi)	65	
t (in)	1.75	
b_{eff} (in)	2.97	
	4.13	Controls
w (in)	14.5	
Tension Rupture		
ϕP_n (Kips)	704.68	Okay
Shear Rupture		
a (in)	4.25	
b (in)	4.25	
d (in)	6	
A_{sf} (in^2)	25.375	
ϕP_n (Kips)	989.625	Okay
Bearing On Pin		
A_{pb} (in^2)	10.5	
ϕP_n (Kips)	708.75	Okay
Tension Tielding		
ϕP_n (Kips)	1141.875	Okay

14.5 in x 1.75 in pin-connected member with (2) 3 in pins

Pin Connection Plate At Bottom of Truss 7 Structure 2

Member	16	
P_u (Kips)	692.37	[For 1 Member]
F_y (ksi)	50	
F_u (ksi)	65	
t (in)	1.75	
b_{eff} (in)	4.06	
	4.13	Controls
w (in)	14.5	
Tension Rupture		
ϕP_n (Kips)	704.68	Okay
Shear Rupture		
a (in)	4.25	
b (in)	4.25	
d (in)	6	
A_{sf} (in^2)	25.375	
ϕP_n (Kips)	989.625	Okay
Bearing On Pin		
A_{pb} (in^2)	10.5	
ϕP_n (Kips)	708.75	Okay
Tension Tielding		
ϕP_n (Kips)	1141.875	Okay

14.5 in x 1.75 in pin-connected member with (2) 3 in pins

Pin Connection Plate At Bottom of Truss 7 Structure 2

Member	19	
P_u (Kips)	606.20	[For 1 Member]
F_y (ksi)	50	
F_u (ksi)	65	
t (in)	1.75	
b_{eff} (in)	3.55	
	4.13	Controls
w (in)	14.5	
Tension Rupture		
ϕP_n (Kips)	704.68	Okay
Shear Rupture		
a (in)	4.25	
b (in)	4.25	
d (in)	6	
A_{sf} (in^2)	25.375	
ϕP_n (Kips)	989.625	Okay
Bearing On Pin		
A_{pb} (in^2)	10.5	
ϕP_n (Kips)	708.75	Okay
Tension Tielding		
ϕP_n (Kips)	1141.875	Okay

14.5 in x 1.75 in pin-connected member with (2) 3 in pins

Pin Connection Plate At Bottom of Truss 7 Structure 2

Member	20	
P_u (Kips)	460.795	[For 1 Member]
F_y (ksi)	50	
F_u (ksi)	65	
t (in)	1.75	
b_{eff} (in)	2.70	
	4.13	Controls
w (in)	14.5	
Tension Rupture		
ϕP_n (Kips)	704.68	Okay
Shear Rupture		
a (in)	4.25	
b (in)	4.25	
d (in)	6	
A_{sf} (in^2)	25.375	
ϕP_n (Kips)	989.625	Okay
Bearing On Pin		
A_{pb} (in^2)	10.5	
ϕP_n (Kips)	708.75	Okay
Tension Tielding		
ϕP_n (Kips)	1141.875	Okay

14.5 in x 1.75 in pin-connected member with (2) 3 in pins

Calculation 7: Pin & Plate Size for Trusses Cont'd

Pin Connection Plate At Bottom of Truss 8 Structure 2

Member	15	
P_u (Kips)	497.525	[For 1 Member]
F_y (ksi)	50	
F_u (ksi)	65	
t (in)	1.75	
b_{eff} (in)	2.92	
	4.13	Controls
w (in)	14.5	
Tension Rupture		
φP_n (Kips)	704.68	Okay
Shear Rupture		
a (in)	4.25	
b (in)	4.25	
d (in)	6	
A_{sf} (in^2)	25.375	
φP_n (Kips)	989.625	Okay
Bearing On Pin		
A_{pb} (in^2)	10.5	
φP_n (Kips)	708.75	Okay
Tension Tielding		
φP_n (Kips)	1141.875	Okay
14.5 in x 1.75 in pin-connected member with (2) 3 in pins		

Pin Connection Plate At Bottom of Truss 8 Structure 2

Member	16	
P_u (Kips)	634.21	[For 1 Member]
F_y (ksi)	50	
F_u (ksi)	65	
t (in)	1.75	
b_{eff} (in)	3.72	
	4.13	Controls
w (in)	14.5	
Tension Rupture		
φP_n (Kips)	704.68	Okay
Shear Rupture		
a (in)	4.25	
b (in)	4.25	
d (in)	6	
A_{sf} (in^2)	25.375	
φP_n (Kips)	989.625	Okay
Bearing On Pin		
A_{pb} (in^2)	10.5	
φP_n (Kips)	708.75	Okay
Tension Tielding		
φP_n (Kips)	1141.875	Okay
14.5 in x 1.75 in pin-connected member with (2) 3 in pins		

Pin Connection Plate At Bottom of Truss 8 Structure 2

Member	19	
P_u (Kips)	435.30	[For 1 Member]
F_y (ksi)	50	
F_u (ksi)	65	
t (in)	1.5	
b_{eff} (in)	2.98	
	3.63	Controls
w (in)	14	
Tension Rupture		
φP_n (Kips)	530.89	Okay
Shear Rupture		
a (in)	4	
b (in)	4	
d (in)	6	
A_{sf} (in^2)	21	
φP_n (Kips)	819	Okay
Bearing On Pin		
A_{pb} (in^2)	9	
φP_n (Kips)	607.5	Okay
Tension Tielding		
φP_n (Kips)	945	Okay
14 in x 1.5 in pin-connected member with (2) 3 in pins		

Pin Connection Plate At Bottom of Truss 8 Structure 2

Member	20	
P_u (Kips)	333.32	[For 1 Member]
F_y (ksi)	50	
F_u (ksi)	65	
t (in)	1.5	
b_{eff} (in)	2.28	
	3.63	Controls
w (in)	14	
Tension Rupture		
φP_n (Kips)	530.89	Okay
Shear Rupture		
a (in)	4	
b (in)	4	
d (in)	6	
A_{sf} (in^2)	21	
φP_n (Kips)	819	Okay
Bearing On Pin		
A_{pb} (in^2)	9	
φP_n (Kips)	607.5	Okay
Tension Tielding		
φP_n (Kips)	945	Okay
14 in x 1.5 in pin-connected member with (2) 3 in pins		

Calculation 8: Pin & Plate Size for Trusses Cont'd

Pin Connection Plate At Bottom of Truss 9 Structure 2

Member	15	
P_u (Kips)	31.335	[For 1 Member]
F_y (ksi)	50	
F_u (ksi)	65	
t (in)	0.75	
b_{eff} (in)	0.43	
	2.13	Controls
w (in)	9	
Tension Rupture		
φP_n (Kips)	155.75625	Okay
Shear Rupture		
a (in)	2.5	
b (in)	2.5	
d (in)	4	
A_{sf} (in^2)	6.75	
φP_n (Kips)	263.25	Okay
Bearing On Pin		
A_{pb} (in^2)	3	
φP_n (Kips)	202.5	Okay
Tension Tielding		
φP_n (Kips)	303.75	Okay

9 in x 0.75 in pin-connected member with a 4 in pin

Pin Connection Plate At Bottom of Truss 9 Structure 2

Member	16	
P_u (Kips)	41.53	[For 1 Member]
F_y (ksi)	50	
F_u (ksi)	65	
t (in)	0.75	
b_{eff} (in)	0.57	
	2.13	Controls
w (in)	9	
Tension Rupture		
φP_n (Kips)	155.76	Okay
Shear Rupture		
a (in)	2.5	
b (in)	2.5	
d (in)	4	
A_{sf} (in^2)	6.75	
φP_n (Kips)	263.25	Okay
Bearing On Pin		
A_{pb} (in^2)	3	
φP_n (Kips)	202.5	Okay
Tension Tielding		
φP_n (Kips)	303.75	Okay

9 in x 0.75 in pin-connected member with a 4 in pin

Pin Connection Plate At Bottom of Truss 9 Structure 2

Member	19	
P_u (Kips)	41.53	[For 1 Member]
F_y (ksi)	50	
F_u (ksi)	65	
t (in)	0.75	
b_{eff} (in)	0.57	
	2.13	Controls
w (in)	9	
Tension Rupture		
φP_n (Kips)	155.76	Okay
Shear Rupture		
a (in)	2.5	
b (in)	2.5	
d (in)	4	
A_{sf} (in^2)	6.75	
φP_n (Kips)	263.25	Okay
Bearing On Pin		
A_{pb} (in^2)	3	
φP_n (Kips)	202.5	Okay
Tension Tielding		
φP_n (Kips)	303.75	Okay

9 in x 0.75 in pin-connected member with a 4 in pin

Pin Connection Plate At Bottom of Truss 9 Structure 2

Member	20	
P_u (Kips)	31.34	[For 1 Member]
F_y (ksi)	50	
F_u (ksi)	65	
t (in)	0.75	
b_{eff} (in)	0.43	
	2.13	Controls
w (in)	9	
Tension Rupture		
φP_n (Kips)	155.76	Okay
Shear Rupture		
a (in)	2.5	
b (in)	2.5	
d (in)	4	
A_{sf} (in^2)	6.75	
φP_n (Kips)	263.25	Okay
Bearing On Pin		
A_{pb} (in^2)	3	
φP_n (Kips)	202.5	Okay
Tension Tielding		
φP_n (Kips)	303.75	Okay

9 in x 0.75 in pin-connected member with a 4 in pin

Calculation 9: Pin & Plate Size for Trusses Cont'd

Pin Plate Connections To Columns [Truss 1 Structure 1]

Member	15
Column	10
R_u (Kips)	255.82
t_p (in)	1.125

a	k = 0.1	k = 0.113	k = 0.2
0	5.58	5.635	6.02

Table 8-6 From Steel Manual	
Angle (deg)	60
l (in)	10
kl (in)	1.125
k (in)	0.113
xl (in)	0
al (in)	0
a	0
C	5.635
C_1	1
φ	0.75
$D_{required}$	6.05
D_{used}	7 (1/16")

Fillet Weld Limitations	
Max Size: $t_p \geq 1/4"$	
t_w (in 1/16")	17
$D_{used} \leq t_w$	Okay
Min Length: $L_{weld} \geq 4t_w$	
Lweld (in)	10
$4t_w$ (in)	4.25
$L_{weld} \geq 4t_w$	Okay

Use a 7/16" E70XX Fillet Weld Around Plate Perimeter

Pin Plate Connections To Columns [Truss 1 Structure 1]

Member	16
Column	10
R_u (Kips)	328.56
t_p (in)	1.25

a	k = 0.1	k = 0.119	k = 0.2
0	5.58	5.664	6.02

Table 8-6 From Steel Manual	
Angle (deg)	60
l (in)	10.5
kl (in)	1.25
k (in)	0.119
xl (in)	0
al (in)	0
a	0
C	5.664
C_1	1
φ	0.75
$D_{required}$	7.37
D_{used}	8 (1/16")

Fillet Weld Limitations	
Max Size: $t_p \geq 1/4"$	
t_w (in 1/16")	19
$D_{used} \leq t_w$	Okay
Min Length: $L_{weld} \geq 4t_w$	
Lweld (in)	10.5
$4t_w$ (in)	4.75
$L_{weld} \geq 4t_w$	Okay

Use a 8/16" E70XX Fillet Weld Around Plate Perimeter

Pin Plate Connections To Columns [Truss 1 Structure 1]

Member	19
Column	13
R_u (Kips)	175.21
t_p (in)	1.25

a	k = 0.1	k = 0.119	k = 0.2
0	5.58	5.664	6.02

Table 8-6 From Steel Manual	
Angle (deg)	60
l (in)	10.5
kl (in)	1.25
k (in)	0.119
xl (in)	0
al (in)	0
a	0
C	5.664
C_1	1
φ	0.75
$D_{required}$	3.93
D_{used}	4 (1/16")

Fillet Weld Limitations	
Max Size: $t_p \geq 1/4"$	
t_w (in 1/16")	19
$D_{used} \leq t_w$	Okay
Min Length: $L_{weld} \geq 4t_w$	
Lweld (in)	10.5
$4t_w$ (in)	4.75
$L_{weld} \geq 4t_w$	Okay

Use a 4/16" E70XX Fillet Weld Around Plate Perimeter

Pin Plate Connections To Columns [Truss 1 Structure 1]

Member	20
Column	13
R_u (Kips)	135.59
t_p (in)	1.125

a	k = 0.1	k = 0.113	k = 0.2
0	5.58	5.635	6.02

Table 8-6 From Steel Manual	
Angle (deg)	60
l (in)	10
kl (in)	1.125
k (in)	0.113
xl (in)	0
al (in)	0
a	0
C	5.635
C_1	1
φ	0.75
$D_{required}$	3.21
D_{used}	4 (1/16")

Fillet Weld Limitations	
Max Size: $t_p \geq 1/4"$	
t_w (in 1/16")	17
$D_{used} \leq t_w$	Okay
Min Length: $L_{weld} \geq 4t_w$	
Lweld (in)	10
$4t_w$ (in)	4.25
$L_{weld} \geq 4t_w$	Okay

Use a 4/16" E70XX Fillet Weld Around Plate Perimeter

Calculation 10: Plate Connection to Column

Pin Plate Connections To Columns [Truss 2 Structure 1]

Member	15	
Column	10	
R_u (Kips)	495.03	
t_p (in)	1.5	
Table 8-6 From Steel Manual		
Angle (deg)	60	
l (in)	14	
kl (in)	1.5	
k (in)	0.107	
xl (in)	0	
al (in)	0	
a	0	
C	5.611	
C_1	1	
φ	0.75	
$D_{required}$	8.40	1/16"
D_{used}	9	
Fillet Weld Limitations		
Max Size: $t_p \geq 1/4"$		
t_w (in 1/16")	23	
$D_{used} \leq t_w$	Okay	
Min Length: $L_{weld} \geq 4t_w$		
Lweld (in)	14	
$4t_w$ (in)	5.75	
$L_{weld} \geq 4t_w$	Okay	

	k		
a	0.1	0.107	0.2
0	5.58	5.611	6.02

Use a 9/16" E70XX Fillet Weld Around Plate Perimeter

Pin Plate Connections To Columns [Truss 2 Structure 1]

Member	16	
Column	10	
R_u (Kips)	642.37	
t_p (in)	1.75	
Table 8-6 From Steel Manual		
Angle (deg)	60	
l (in)	14.5	
kl (in)	1.75	
k (in)	0.121	
xl (in)	0	
al (in)	0	
a	0	
C	5.671	
C_1	1	
φ	0.75	
$D_{required}$	10.42	1/16"
D_{used}	11	
Fillet Weld Limitations		
Max Size: $t_p \geq 1/4"$		
t_w (in 1/16")	27	
$D_{used} \leq t_w$	Okay	
Min Length: $L_{weld} \geq 4t_w$		
Lweld (in)	14.5	
$4t_w$ (in)	6.75	
$L_{weld} \geq 4t_w$	Okay	

	k		
a	0.1	0.121	0.2
0	5.58	5.671	6.02

Use a 11/16" E70XX Fillet Weld Around Plate Perimeter

Pin Plate Connections To Columns [Truss 2 Structure 1]

Member	19	
Column	13	
R_u (Kips)	478.13	
t_p (in)	1.5	
Table 8-6 From Steel Manual		
Angle (deg)	60	
l (in)	14	
kl (in)	1.5	
k (in)	0.107	
xl (in)	0	
al (in)	0	
a	0	
C	5.611	
C_1	1	
φ	0.75	
$D_{required}$	8.11	1/16"
D_{used}	9	
Fillet Weld Limitations		
Max Size: $t_p \geq 1/4"$		
t_w (in 1/16")	23	
$D_{used} \leq t_w$	Okay	
Min Length: $L_{weld} \geq 4t_w$		
Lweld (in)	14	
$4t_w$ (in)	5.75	
$L_{weld} \geq 4t_w$	Okay	

	k		
a	0.1	0.107	0.2
0	5.58	5.611	6.02

Use a 9/16" E70XX Fillet Weld Around Plate Perimeter

Pin Plate Connections To Columns [Truss 2 Structure 1]

Member	20	
Column	13	
R_u (Kips)	365.15	
t_p (in)	1.5	
Table 8-6 From Steel Manual		
Angle (deg)	60	
l (in)	14	
kl (in)	1.5	
k (in)	0.107	
xl (in)	0	
al (in)	0	
a	0	
C	5.611	
C_1	1	
φ	0.75	
$D_{required}$	6.20	1/16"
D_{used}	7	
Fillet Weld Limitations		
Max Size: $t_p \geq 1/4"$		
t_w (in 1/16")	23	
$D_{used} \leq t_w$	Okay	
Min Length: $L_{weld} \geq 4t_w$		
Lweld (in)	14	
$4t_w$ (in)	5.75	
$L_{weld} \geq 4t_w$	Okay	

	k		
a	0.1	0.107	0.2
0	5.58	5.611	6.02

Use a 7/16" E70XX Fillet Weld Around Plate Perimeter

Calculation 11: Plate Connection to Column Cont'd

Pin Plate Connections To Columns [Truss 3 Structure 1]

Member	15
Column	10
R_u (Kips)	515.25
t_p (in)	1.75
Table 8-6 From Steel Manual	
Angle (deg)	60
l (in)	14.5
kl (in)	1.75
k (in)	0.121
xl (in)	0
al (in)	0
a	0
C	5.671
C_1	1
φ	0.75
$D_{required}$	8.35
D_{used}	9
Fillet Weld Limitations	
Max Size: $t_p \geq 1/4"$	
t_w (in 1/16")	27
$D_{used} \leq t_w$	Okay
Min Length: $L_{weld} \geq 4t_w$	
Lweld (in)	14.5
$4t_w$ (in)	6.75
$L_{weld} \geq 4t_w$	Okay

1/16"

a	k		
	0.1	0.121	0.2
0	5.58	5.671	6.02

Use a 9/16" E70XX Fillet Weld Around Plate Perimeter

Pin Plate Connections To Columns [Truss 3 Structure 1]

Member	16
Column	10
R_u (Kips)	682.66
t_p (in)	1.75
Table 8-6 From Steel Manual	
Angle (deg)	60
l (in)	14.5
kl (in)	1.75
k (in)	0.121
xl (in)	0
al (in)	0
a	0
C	5.671
C_1	1
φ	0.75
$D_{required}$	11.07
D_{used}	12
Fillet Weld Limitations	
Max Size: $t_p \geq 1/4"$	
t_w (in 1/16")	27
$D_{used} \leq t_w$	Okay
Min Length: $L_{weld} \geq 4t_w$	
Lweld (in)	14.5
$4t_w$ (in)	6.75
$L_{weld} \geq 4t_w$	Okay

1/16"

a	k		
	0.1	0.121	0.2
0	5.58	5.671	6.02

Use a 12/16" E70XX Fillet Weld Around Plate Perimeter

Pin Plate Connections To Columns [Truss 3 Structure 1]

Member	19
Column	13
R_u (Kips)	682.66
t_p (in)	1.75
Table 8-6 From Steel Manual	
Angle (deg)	60
l (in)	14.5
kl (in)	1.75
k (in)	0.121
xl (in)	0
al (in)	0
a	0
C	5.671
C_1	1
φ	0.75
$D_{required}$	11.07
D_{used}	12
Fillet Weld Limitations	
Max Size: $t_p \geq 1/4"$	
t_w (in 1/16")	27
$D_{used} \leq t_w$	Okay
Min Length: $L_{weld} \geq 4t_w$	
Lweld (in)	14.5
$4t_w$ (in)	6.75
$L_{weld} \geq 4t_w$	Okay

1/16"

a	k		
	0.1	0.121	0.2
0	5.58	5.671	6.02

Use a 12/16" E70XX Fillet Weld Around Plate Perimeter

Pin Plate Connections To Columns [Truss 3 Structure 1]

Member	20
Column	13
R_u (Kips)	515.25
t_p (in)	1.75
Table 8-6 From Steel Manual	
Angle (deg)	60
l (in)	14.5
kl (in)	1.75
k (in)	0.121
xl (in)	0
al (in)	0
a	0
C	5.671
C_1	1
φ	0.75
$D_{required}$	8.35
D_{used}	9
Fillet Weld Limitations	
Max Size: $t_p \geq 1/4"$	
t_w (in 1/16")	27
$D_{used} \leq t_w$	Okay
Min Length: $L_{weld} \geq 4t_w$	
Lweld (in)	14.5
$4t_w$ (in)	6.75
$L_{weld} \geq 4t_w$	Okay

1/16"

a	k		
	0.1	0.121	0.2
0	5.58	5.671	6.02

Use a 9/16" E70XX Fillet Weld Around Plate Perimeter

Calculation 12: Plate Connection to Column Cont'd

Pin Plate Connections To Columns [Truss 4 Structure 1]

Member	15	
Column	10	
R_u (Kips)	928.15	
t_p (in)	2.125	
Table 8-6 From Steel Manual		
Angle (deg)	60	
l (in)	18	
kl (in)	2.125	
k (in)	0.118	
xl (in)	0	
al (in)	0	
a	0	
C	5.659	
C_1	1	
φ	0.75	
$D_{required}$	12.15	1/16"
D_{used}	13	
Fillet Weld Limitations		
Max Size: $t_p \geq 1/4"$		
t_w (in 1/16")	33	
$D_{used} \leq t_w$	Okay	
Min Length: $L_{weld} \geq 4t_w$		
Lweld (in)	18	
$4t_w$ (in)	8.25	
$L_{weld} \geq 4t_w$	Okay	

a	k		
a	0.1	0.118	0.2
0	5.58	5.659	6.02

Use a 13/16" E70XX Fillet Weld Around Plate Perimeter

Pin Plate Connections To Columns [Truss 4 Structure 1]

Member	16	
Column	10	
R_u (Kips)	1208.12	
t_p (in)	2.375	
Table 8-6 From Steel Manual		
Angle (deg)	60	
l (in)	19	
kl (in)	2.375	
k (in)	0.125	
xl (in)	0	
al (in)	0	
a	0	
C	5.690	
C_1	1	
φ	0.75	
$D_{required}$	14.90	1/16"
D_{used}	15	
Fillet Weld Limitations		
Max Size: $t_p \geq 1/4"$		
t_w (in 1/16")	37	
$D_{used} \leq t_w$	Okay	
Min Length: $L_{weld} \geq 4t_w$		
Lweld (in)	19	
$4t_w$ (in)	9.25	
$L_{weld} \geq 4t_w$	Okay	

a	k		
a	0.1	0.125	0.2
0	5.58	5.690	6.02

Use a Full Penitration Weld

Pin Plate Connections To Columns [Truss 4 Structure 1]

Member	19	
Column	13	
R_u (Kips)	1238.42	
t_p (in)	2.375	
Table 8-6 From Steel Manual		
Angle (deg)	60	
l (in)	19	
kl (in)	2.375	
k (in)	0.125	
xl (in)	0	
al (in)	0	
a	0	
C	5.690	
C_1	1	
φ	0.75	
$D_{required}$	15.27	1/16"
D_{used}	16	
Fillet Weld Limitations		
Max Size: $t_p \geq 1/4"$		
t_w (in 1/16")	37	
$D_{used} \leq t_w$	Okay	
Min Length: $L_{weld} \geq 4t_w$		
Lweld (in)	19	
$4t_w$ (in)	9.25	
$L_{weld} \geq 4t_w$	Okay	

a	k		
a	0.1	0.125	0.2
0	5.58	5.690	6.02

Use a Full Penitration Weld

Pin Plate Connections To Columns [Truss 4 Structure 1]

Member	20	
Column	13	
R_u (Kips)	897.86	
t_p (in)	2.125	
Table 8-6 From Steel Manual		
Angle (deg)	60	
l (in)	18	
kl (in)	2.125	
k (in)	0.118	
xl (in)	0	
al (in)	0	
a	0	
C	5.659	
C_1	1	
φ	0.75	
$D_{required}$	11.75	1/16"
D_{used}	12	
Fillet Weld Limitations		
Max Size: $t_p \geq 1/4"$		
t_w (in 1/16")	33	
$D_{used} \leq t_w$	Okay	
Min Length: $L_{weld} \geq 4t_w$		
Lweld (in)	18	
$4t_w$ (in)	8.25	
$L_{weld} \geq 4t_w$	Okay	

a	k		
a	0.1	0.118	0.2
0	5.58	5.659	6.02

Use a 12/16" E70XX Fillet Weld Around Plate Perimeter

Calculation 13: Plate Connection to Column Cont'd

Pin Plate Connections To Columns [Truss 5 Structure 1]

Member	15			k		
Column	10	a	0.1	0.113	0.2	
R_u (Kips)	219.4	0	5.58	5.635	6.02	
t_p (in)	1.125					
Table 8-6 From Steel Manual						
Angle (deg)	60					
l (in)	10					
kl (in)	1.125					
k (in)	0.113					
xl (in)	0					
al (in)	0					
a	0					
C	5.635					
C_1	1					
φ	0.75					
$D_{required}$	5.19	1/16"				
D_{used}	6					
Fillet Weld Limitations						
Max Size: $t_p \geq 1/4"$						
t_w (in 1/16")	17					
$D_{used} \leq t_w$	Okay					
Min Length: $L_{weld} \geq 4t_w$						
Lweld (in)	10					
$4t_w$ (in)	4.25					
$L_{weld} \geq 4t_w$	Okay					

Use a 6/16" E70XX Fillet Weld Around Plate Perimeter

Pin Plate Connections To Columns [Truss 5 Structure 1]

Member	16			k	
Column	10	a	0.1	0.119	0.2
R_u (Kips)	287	0	5.58	5.664	6.02
t_p (in)	1.25				
Table 8-6 From Steel Manual					
Angle (deg)	60				
l (in)	10.5				
kl (in)	1.25				
k (in)	0.119				
xl (in)	0				
al (in)	0				
a	0				
C	5.664				
C_1	1				
φ	0.75				
$D_{required}$	6.43	1/16"			
D_{used}	7				
Fillet Weld Limitations					
Max Size: $t_p \geq 1/4"$					
t_w (in 1/16")	19				
$D_{used} \leq t_w$	Okay				
Min Length: $L_{weld} \geq 4t_w$					
Lweld (in)	10.5				
$4t_w$ (in)	4.75				
$L_{weld} \geq 4t_w$	Okay				

Use a 7/16" E70XX Fillet Weld Around Plate Perimeter

Pin Plate Connections To Columns [Truss 5 Structure 1]

Member	19			k	
Column	13	a	0.1	0.119	0.2
R_u (Kips)	222.48	0	5.58	5.664	6.02
t_p (in)	1.25				
Table 8-6 From Steel Manual					
Angle (deg)	60				
l (in)	10.5				
kl (in)	1.25				
k (in)	0.119				
xl (in)	0				
al (in)	0				
a	0				
C	5.664				
C_1	1				
φ	0.75				
$D_{required}$	4.99	1/16"			
D_{used}	5				
Fillet Weld Limitations					
Max Size: $t_p \geq 1/4"$					
t_w (in 1/16")	19				
$D_{used} \leq t_w$	Okay				
Min Length: $L_{weld} \geq 4t_w$					
Lweld (in)	10.5				
$4t_w$ (in)	4.75				
$L_{weld} \geq 4t_w$	Okay				

Use a 6/16" E70XX Fillet Weld Around Plate Perimeter

Pin Plate Connections To Columns [Truss 5 Structure 1]

Member	20			k	
Column	13	a	0.1	0.107	0.2
R_u (Kips)	163.17	0	5.58	5.611	6.02
t_p (in)	1.125				
Table 8-6 From Steel Manual					
Angle (deg)	60				
l (in)	10.5				
kl (in)	1.125				
k (in)	0.107				
xl (in)	0				
al (in)	0				
a	0				
C	5.611				
C_1	1				
φ	0.75				
$D_{required}$	3.69	1/16"			
D_{used}	4				
Fillet Weld Limitations					
Max Size: $t_p \geq 1/4"$					
t_w (in 1/16")	17				
$D_{used} \leq t_w$	Okay				
Min Length: $L_{weld} \geq 4t_w$					
Lweld (in)	10.5				
$4t_w$ (in)	4.25				
$L_{weld} \geq 4t_w$	Okay				

Use a 4/16" E70XX Fillet Weld Around Plate Perimeter

Calculation 14: Plate Connection to Column Cont'd

Pin Plate Connections To Columns [Truss 6 Structure 2]

Member	15
Column	10
R_u (Kips)	271.15
t_p (in)	1.125
Table 8-6 From Steel Manual	
Angle (deg)	60
l (in)	10
kl (in)	1.125
k (in)	0.113
xl (in)	0
al (in)	0
a	0
C	5.635
C_1	1
φ	0.75
$D_{required}$	6.42
D_{used}	7
Fillet Weld Limitations	
Max Size: $t_p \geq 1/4"$	
t_w (in 1/16")	17
$D_{used} \leq t_w$	Okay
Min Length: $L_{weld} \geq 4t_w$	
Lweld (in)	10
$4t_w$ (in)	4.25
$L_{weld} \geq 4t_w$	Okay

1/16"

	k		
a	0.1	0.113	0.2
0	5.58	5.635	6.02

Use a 7/16" E70XX Fillet Weld Around Plate Perimeter

Pin Plate Connections To Columns [Truss 6 Structure 2]

Member	16
Column	10
R_u (Kips)	357.79
t_p (in)	1.375
Table 8-6 From Steel Manual	
Angle (deg)	60
l (in)	10.5
kl (in)	1.375
k (in)	0.131
xl (in)	0
al (in)	0
a	0
C	5.716
C_1	1
φ	0.75
$D_{required}$	7.95
D_{used}	8
Fillet Weld Limitations	
Max Size: $t_p \geq 1/4"$	
t_w (in 1/16")	21
$D_{used} \leq t_w$	Okay
Min Length: $L_{weld} \geq 4t_w$	
Lweld (in)	10.5
$4t_w$ (in)	5.25
$L_{weld} \geq 4t_w$	Okay

1/16"

	k		
a	0.1	0.131	0.2
0	5.58	5.716	6.02

Use a 8/16" E70XX Fillet Weld Around Plate Perimeter

Pin Plate Connections To Columns [Truss 6 Structure 2]

Member	19
Column	13
R_u (Kips)	357.79
t_p (in)	1.375
Table 8-6 From Steel Manual	
Angle (deg)	60
l (in)	10.5
kl (in)	1.375
k (in)	0.131
xl (in)	0
al (in)	0
a	0
C	5.716
C_1	1
φ	0.75
$D_{required}$	7.95
D_{used}	8
Fillet Weld Limitations	
Max Size: $t_p \geq 1/4"$	
t_w (in 1/16")	21
$D_{used} \leq t_w$	Okay
Min Length: $L_{weld} \geq 4t_w$	
Lweld (in)	10.5
$4t_w$ (in)	5.25
$L_{weld} \geq 4t_w$	Okay

1/16"

	k		
a	0.1	0.131	0.2
0	5.58	5.716	6.02

Use a 8/16" E70XX Fillet Weld Around Plate Perimeter

Pin Plate Connections To Columns [Truss 6 Structure 2]

Member	20
Column	13
R_u (Kips)	271.15
t_p (in)	1.125
Table 8-6 From Steel Manual	
Angle (deg)	60
l (in)	10
kl (in)	1.125
k (in)	0.113
xl (in)	0
al (in)	0
a	0
C	5.635
C_1	1
φ	0.75
$D_{required}$	6.42
D_{used}	7
Fillet Weld Limitations	
Max Size: $t_p \geq 1/4"$	
t_w (in 1/16")	17
$D_{used} \leq t_w$	Okay
Min Length: $L_{weld} \geq 4t_w$	
Lweld (in)	10
$4t_w$ (in)	4.25
$L_{weld} \geq 4t_w$	Okay

1/16"

	k		
a	0.1	0.113	0.2
0	5.58	5.635	6.02

Use a 8/16" E70XX Fillet Weld Around Plate Perimeter

Calculation 15: Plate Connection to Column Cont'd

Pin Plate Connections To Columns [Truss 7 Structure 2]

Member	15	
Column	10	
R_u (Kips)	505.975	
t_p (in)	1.75	
Table 8-6 From Steel Manual		
Angle (deg)	60	
l (in)	14.5	
kl (in)	1.75	
k (in)	0.121	
xl (in)	0	
al (in)	0	
a	0	
C	5.671	
C_1	1	
φ	0.75	
$D_{required}$	8.20	1/16"
D_{used}	9	
Fillet Weld Limitations		
Max Size: $t_p \geq 1/4"$		
t_w (in 1/16")	27	
$D_{used} \leq t_w$	Okay	
Min Length: $L_{weld} \geq 4t_w$		
Lweld (in)	14.5	
$4t_w$ (in)	6.75	
$L_{weld} \geq 4t_w$	Okay	

a	k		
a	0.1	0.121	0.2
0	5.58	5.671	6.02

Use a 9/16" E70XX Fillet Weld Around Plate Perimeter

Pin Plate Connections To Columns [Truss 7 Structure 2]

Member	16	
Column	10	
R_u (Kips)	692.37	
t_p (in)	1.75	
Table 8-6 From Steel Manual		
Angle (deg)	60	
l (in)	14.5	
kl (in)	1.75	
k (in)	0.121	
xl (in)	0	
al (in)	0	
a	0	
C	5.671	
C_1	1	
φ	0.75	
$D_{required}$	11.23	1/16"
D_{used}	12	
Fillet Weld Limitations		
Max Size: $t_p \geq 1/4"$		
t_w (in 1/16")	27	
$D_{used} \leq t_w$	Okay	
Min Length: $L_{weld} \geq 4t_w$		
Lweld (in)	14.5	
$4t_w$ (in)	6.75	
$L_{weld} \geq 4t_w$	Okay	

a	k		
a	0.1	0.121	0.2
0	5.58	5.671	6.02

Use a 12/16" E70XX Fillet Weld Around Plate Perimeter

Pin Plate Connections To Columns [Truss 7 Structure 2]

Member	19	
Column	13	
R_u (Kips)	606.2	
t_p (in)	1.75	
Table 8-6 From Steel Manual		
Angle (deg)	60	
l (in)	14.5	
kl (in)	1.75	
k (in)	0.121	
xl (in)	0	
al (in)	0	
a	0	
C	5.671	
C_1	1	
φ	0.75	
$D_{required}$	9.83	1/16"
D_{used}	10	
Fillet Weld Limitations		
Max Size: $t_p \geq 1/4"$		
t_w (in 1/16")	27	
$D_{used} \leq t_w$	Okay	
Min Length: $L_{weld} \geq 4t_w$		
Lweld (in)	14.5	
$4t_w$ (in)	6.75	
$L_{weld} \geq 4t_w$	Okay	

a	k		
a	0.1	0.121	0.2
0	5.58	5.671	6.02

Use a 10/16" E70XX Fillet Weld Around Plate Perimeter

Pin Plate Connections To Columns [Truss 7 Structure 2]

Member	20	
Column	13	
R_u (Kips)	460.795	
t_p (in)	1.75	
Table 8-6 From Steel Manual		
Angle (deg)	60	
l (in)	14.5	
kl (in)	1.75	
k (in)	0.121	
xl (in)	0	
al (in)	0	
a	0	
C	5.671	
C_1	1	
φ	0.75	
$D_{required}$	7.47	1/16"
D_{used}	8	
Fillet Weld Limitations		
Max Size: $t_p \geq 1/4"$		
t_w (in 1/16")	27	
$D_{used} \leq t_w$	Okay	
Min Length: $L_{weld} \geq 4t_w$		
Lweld (in)	14.5	
$4t_w$ (in)	6.75	
$L_{weld} \geq 4t_w$	Okay	

a	k		
a	0.1	0.121	0.2
0	5.58	5.671	6.02

Use a 8/16" E70XX Fillet Weld Around Plate Perimeter

Calculation 16: Plate Connection to Column Cont'd

Pin Plate Connections To Columns [Truss 8 Structure 2]

Member	15
Column	10
R_u (Kips)	497.525
t_p (in)	1.75
Table 8-6 From Steel Manual	
Angle (deg)	60
l (in)	14.5
kl (in)	1.75
k (in)	0.121
xl (in)	0
al (in)	0
a	0
C	5.671
C_1	1
φ	0.75
$D_{required}$	8.07
D_{used}	9
Fillet Weld Limitations	
Max Size: $t_p \geq 1/4"$	
t_w (in 1/16")	27
$D_{used} \leq t_w$	Okay
Min Length: $L_{weld} \geq 4t_w$	
Lweld (in)	14.5
$4t_w$ (in)	6.75
$L_{weld} \geq 4t_w$	Okay

1/16"

a	k		
	0.1	0.121	0.2
0	5.58	5.671	6.02

Use a 9/16" E70XX Fillet Weld Around Plate Perimeter

Pin Plate Connections To Columns [Truss 8 Structure 2]

Member	16
Column	10
R_u (Kips)	634.21
t_p (in)	1.75
Table 8-6 From Steel Manual	
Angle (deg)	60
l (in)	14.5
kl (in)	1.75
k (in)	0.121
xl (in)	0
al (in)	0
a	0
C	5.671
C_1	1
φ	0.75
$D_{required}$	10.28
D_{used}	11
Fillet Weld Limitations	
Max Size: $t_p \geq 1/4"$	
t_w (in 1/16")	27
$D_{used} \leq t_w$	Okay
Min Length: $L_{weld} \geq 4t_w$	
Lweld (in)	14.5
$4t_w$ (in)	6.75
$L_{weld} \geq 4t_w$	Okay

1/16"

a	k		
	0.1	0.121	0.2
0	5.58	5.671	6.02

Use a 11/16" E70XX Fillet Weld Around Plate Perimeter

Pin Plate Connections To Columns [Truss 8 Structure 2]

Member	19
Column	13
R_u (Kips)	435.3
t_p (in)	1.5
Table 8-6 From Steel Manual	
Angle (deg)	60
l (in)	14
kl (in)	1.5
k (in)	0.107
xl (in)	0
al (in)	0
a	0
C	5.611
C_1	1
φ	0.75
$D_{required}$	7.39
D_{used}	8
Fillet Weld Limitations	
Max Size: $t_p \geq 1/4"$	
t_w (in 1/16")	23
$D_{used} \leq t_w$	Okay
Min Length: $L_{weld} \geq 4t_w$	
Lweld (in)	14
$4t_w$ (in)	5.75
$L_{weld} \geq 4t_w$	Okay

1/16"

a	k		
	0.1	0.107	0.2
0	5.58	5.611	6.02

Use a 8/16" E70XX Fillet Weld Around Plate Perimeter

Pin Plate Connections To Columns [Truss 8 Structure 2]

Member	20
Column	13
R_u (Kips)	333.32
t_p (in)	1.5
Table 8-6 From Steel Manual	
Angle (deg)	60
l (in)	14
kl (in)	1.5
k (in)	0.107
xl (in)	0
al (in)	0
a	0
C	5.611
C_1	1
φ	0.75
$D_{required}$	5.66
D_{used}	6
Fillet Weld Limitations	
Max Size: $t_p \geq 1/4"$	
t_w (in 1/16")	23
$D_{used} \leq t_w$	Okay
Min Length: $L_{weld} \geq 4t_w$	
Lweld (in)	14
$4t_w$ (in)	5.75
$L_{weld} \geq 4t_w$	Okay

1/16"

a	k		
	0.1	0.107	0.2
0	5.58	5.611	6.02

Use a 6/16" E70XX Fillet Weld Around Plate Perimeter

Calculation 17: Plate Connection to Column Cont'd

Pin Plate Connections To Columns [Truss 9 Structure 2]

Member	15	
Column	10	
R_u (Kips)	31.335	
t_p (in)	0.75	
Table 8-6 From Steel Manual		
Angle (deg)	60	
l (in)	9	
kl (in)	0.75	
k (in)	0.083	
xl (in)	0	
al (in)	0	
a	0	
C	5.508	
C_1	1	
φ	0.75	
$D_{required}$	0.84	1/16"
D_{used}	1	
Fillet Weld Limitations		
Max Size: $t_p \geq 1/4"$		
t_w (in 1/16")	11	
$D_{used} \leq t_w$	Okay	
Min Length: $L_{weld} \geq 4t_w$		
Lweld (in)	9	
$4t_w$ (in)	2.75	
$L_{weld} \geq 4t_w$	Okay	

	k		
a	0	0.083	0.1
0	5.15	5.508	5.58

Use a 1/16" E70XX Fillet Weld Around Plate Perimeter

Pin Plate Connections To Columns [Truss 9 Structure 2]

Member	16	
Column	10	
R_u (Kips)	41.53	
t_p (in)	0.75	
Table 8-6 From Steel Manual		
Angle (deg)	60	
l (in)	9	
kl (in)	0.75	
k (in)	0.083	
xl (in)	0	
al (in)	0	
a	0	
C	5.508	
C_1	1	
φ	0.75	
$D_{required}$	1.12	1/16"
D_{used}	2	
Fillet Weld Limitations		
Max Size: $t_p \geq 1/4"$		
t_w (in 1/16")	11	
$D_{used} \leq t_w$	Okay	
Min Length: $L_{weld} \geq 4t_w$		
Lweld (in)	9	
$4t_w$ (in)	2.75	
$L_{weld} \geq 4t_w$	Okay	

	k		
a	0	0.083	0.1
0	5.15	5.508	5.58

Use a 2/16" E70XX Fillet Weld Around Plate Perimeter

Pin Plate Connections To Columns [Truss 9 Structure 2]

Member	19	
Column	13	
R_u (Kips)	41.53	
t_p (in)	0.75	
Table 8-6 From Steel Manual		
Angle (deg)	60	
l (in)	9	
kl (in)	0.75	
k (in)	0.083	
xl (in)	0	
al (in)	0	
a	0	
C	5.508	
C_1	1	
φ	0.75	
$D_{required}$	1.12	1/16"
D_{used}	2	
Fillet Weld Limitations		
Max Size: $t_p \geq 1/4"$		
t_w (in 1/16")	11	
$D_{used} \leq t_w$	Okay	
Min Length: $L_{weld} \geq 4t_w$		
Lweld (in)	9	
$4t_w$ (in)	2.75	
$L_{weld} \geq 4t_w$	Okay	

	k		
a	0	0.083	0.1
0	5.15	5.508	5.58

Use a 2/16" E70XX Fillet Weld Around Plate Perimeter

Pin Plate Connections To Columns [Truss 9 Structure 2]

Member	20	
Column	13	
R_u (Kips)	31.335	
t_p (in)	0.75	
Table 8-6 From Steel Manual		
Angle (deg)	60	
l (in)	9	
kl (in)	0.75	
k (in)	0.083	
xl (in)	0	
al (in)	0	
a	0	
C	5.508	
C_1	1	
φ	0.75	
$D_{required}$	0.84	1/16"
D_{used}	1	
Fillet Weld Limitations		
Max Size: $t_p \geq 1/4"$		
t_w (in 1/16")	11	
$D_{used} \leq t_w$	Okay	
Min Length: $L_{weld} \geq 4t_w$		
Lweld (in)	9	
$4t_w$ (in)	2.75	
$L_{weld} \geq 4t_w$	Okay	

	k		
a	0	0.083	0.1
0	5.15	5.508	5.58

Use a 1/16" E70XX Fillet Weld Around Plate Perimeter

Calculation 18: Plate Connection to Column Cont'd

Gusset Plate Info	
h (in)	30
w (in)	55
t (in)	2.25
F_y (ksi)	36
F_u (ksi)	58
l (in)	35.625

Bottom Chord Info	
Member	W16x31
t_w (in)	0.275
t_f (in)	0.44
F_y (ksi)	50
F_u (ksi)	65
d (in)	18.5

Brace Member Info	
Member	W14x257
t_w (in)	1.18
t_f (in)	1.89
F_y (ksi)	50
F_u (ksi)	65
A_g (in^2)	75.6
Bolt Type	A490N
φ (in)	1
φ R_n (Kips)	35.3
x (in)	1.75
T	10

Tables 1 through 3: Heavy Brace Member Information

Angles: Gusset Plate to Brace	
Members	2L8x6x3/4 LLBB
L (in)	Refer to Drawing
t (in)	0.75
F_y (ksi)	36
F_u (ksi)	58
Bolt Type	A490N
φ (in)	1
φ R_n (Kips)	35.3
Weld Size	-
A_g (in^2)	19.88

Angles: Gusset Plate to Column	
Members	2L6x6x3/8
L (in)	23.5
t (in)	0.75
F_y (ksi)	36
F_u (ksi)	58
Bolt Type	A325N
φ (in)	7/8
φ R_n (Kips)	21.6
A_b (in^2)	0.601
Weld Size (in)	5/16

Tables 4 & 5: Heavy Brace Member Information Cont'd

Determine Load Distrabution											
a (in)	$w_{GP}/2$ (in)	α (in)	d_b (in)	e_b (in)	e_c (in)	β = $h_{GP}/2$ (in)	Φ (Deg)	tan Φ	α-βtanΦ	$e_b$tanΦ-e_c	r (in)
1	27.5	28.5	0	0	0	15	62	1.87824	0.00	0.00	32.21
									No Moments Exist		
P_u (Kips)	V_c (Kips)	H_c (Kips)	V_b (Kips)	H_b (Kips)							
617.45	287.58	0.00	0.00	546.39							

Calculation 19: Uniform Force Method

Brace Member to Double Angle Connection

Wide Flange Limit States

Tension Yielding			Tension Rupture			Block Shear		
A_g (in^2)	75.6		A_n (in^2)	74.2725		a (in)	2	
P_u (Kips)	617.45		x (in)	1.75		L (in)	38	
φR_n (Kips)	3402	Okay	L (in)	36	10 Bolts	T/2 (in)	5	
			U	0.951388889		t_w (in)	1.18	
			A_e (in^2)	70.66203125		A_{gv} (in^2)	44.84	
				64.26	Controls	A_{nv} (in^2)	38.20	
			φR_n (Kips)	3132.675	Okay	A_{nt} (in^2)	4.34	
						$0.6F_uA_{nv}$	1489.90	
						$0.6F_yA_{gv}$	1345.20	Controls
						$U_{bs}F_uA_{nt}$	281.86	
						φR_n (Kips)	1220.29	Okay

Bolts (Shear, Bearing, & Tearout)

Bolt Shear			Bearing of Wide Flange			Bearing On Angles		
φR_n (Kips)	35.3	1 Bolt	$\varphi 2.4F_ut_wd_b$	138.06	1 Bolt	$\varphi 2.4F_utd_b$	156.6	1 Bolt
Tearout Flange Web (1)			Tearout Flange Other (2-10)			Tearout Angles Edge (10)		
L_c (in)	1.46875		L_c (in)	2.9375		L_c (in)	1.46875	
$\varphi 1.2F_uL_ct_w$	101.3878	1 Bolt	$\varphi 1.2F_uL_ct_w$	202.775625	1 Bolt	$\varphi 1.2F_uL_ct$	115.003125	1 Bolt
Tearout Angles Other (1-9)								
L_c (in)	2.9375							
$\varphi 1.2F_uL_ct$	230.0063	1 Bolt						
Bolt 1	35.3	138.06	156.6	101.39	230.01			
Bolt 2	35.3	138.06	156.6	202.78	230.01			
Bolt 3	35.3	138.06	156.6	202.78	230.01			
Bolt 4	35.3	138.06	156.6	202.78	230.01			
Bolt 5	35.3	138.06	156.6	202.78	230.01			
Bolt 6	35.3	138.06	156.6	202.78	230.01			
Bolt 7	35.3	138.06	156.6	202.78	230.01			
Bolt 8	35.3	138.06	156.6	202.78	230.01			
Bolt 9	35.3	138.06	156.6	202.78	230.01			
Bolt 10	35.3	138.06	156.6	202.78	115.00			
φR_n (Kips)	706	Okay						

Double Angle Limit States

Tension Yielding			Tension Rupture			Block Shear		
A_g (in^2)	19.88	Both	A_n (in^2)	19.03625		a (in)	2	
P_u (Kips)	617.45		x (in)	2.55		L (in)	38	
φR_n (Kips)	715.68	Okay	L (in)	36	10 Bolts	LL/2 (in)	5.5	
			U	0.929166667		t (in)	0.75	
			A_e (in^2)	17.68784896		A_{gv} (in^2)	28.50	
				16.898	Controls	A_{nv} (in^2)	23.86	
			φR_n (Kips)	735.063	Okay	A_{nt} (in^2)	5.08	
						$0.6F_uA_{nv}$	830.31	
						$0.6F_yA_{gv}$	615.60	Controls
						$U_{bs}F_uA_{nt}$	294.53	
						φR_n (Kips)	682.60	Okay

Calculation 20: Brace Member to Double Angle Connection Limit States

Double Angle Connection From Brace Member to Gusset Plate								
Double Angle Limit States								
Tension Yielding			Tension Rupture			Block Shear		
A_g (in^2)	19.88	Both	A_n (in^2)	18.1925		a (in)	3	
P_u (Kips)	617.45		x (in)	2.55		L (in)	27	
φR_n (Kips)	715.68	Okay	L (in)	24	9	SL/2 (in)	2.75	
			U	0.89375	Use 0.9	t (in)	0.75	
			A_e (in^2)	16.37325	Controls	A_{gv} (in^2)	40.50	
				16.898		A_{nv} (in^2)	22.36	
			φR_n (Kips)	735.063	Okay	A_{nt} (in^2)	1.78	
						$0.6F_uA_{nv}$	778.11	Controls
						$0.6F_yA_{gv}$	874.80	
						$U_{bs}F_uA_{nt}$	103.31	
						φR_n (Kips)	661.06	Okay
Bolts (Shear, Bearing, & Tearout)								
Bolt Shear			Bearing On Angles			Bearing On Gusset Plate		
φR_n (Kips)	35.3	1 Bolt	$\varphi 2.4F_utd_b$	156.6	1 Bolt	ta < tp : Only Check Angles		
Tearout Angle (1)			Tearout Angle Other (2-9)			Tearout Gusset Edge (9)		
L_c (in)	1.46875		L_c (in)	1.9375		L_c (in)	1.46875	
$\varphi 1.2F_uL_ct$	57.50156		$\varphi 1.2F_uL_ct_w$	75.85313		$\varphi 1.2F_uL_ct_w$	172.5047	
Tearout Gusset Other (1-8)								
L_c (in)	1.9375							
$\varphi 1.2F_uL_ct$	227.5594							
Bolt 1	35.3	156.6	-	57.50	227.56			
Bolt 2	35.3	156.6	-	75.85	227.56			
Bolt 3	35.3	156.6	-	75.85	227.56			
Bolt 4	35.3	156.6	-	75.85	227.56			
Bolt 5	35.3	156.6	-	75.85	227.56			
Bolt 6	35.3	156.6	-	75.85	227.56			
Bolt 7	35.3	156.6	-	75.85	227.56			
Bolt 8	35.3	156.6	-	75.85	227.56			
Bolt 9	35.3	156.6	-	75.85	172.50			
φR_n (Kips)	635.4	Okay						
Gusset Plate Limit States								
Tension Yielding			Tension Rupture			Block Shear		
A_g (in^2)	80.15625		A_n (in^2)	30.5625		A_{gv} (in^2)	29.53125	
P_u (Kips)	617.45		P_u (Kips)	617.45		A_{nv} (in^2)	16.24219	
φR_n (Kips)	4184.156	Okay	φR_n (Kips)	1329.469	Okay	A_{nt} (in^2)	4.640625	
						$0.6F_uA_{nv}$	565.2281	Controls
						$0.6F_yA_{gv}$	637.875	
						$U_{bs}F_uA_{nt}$	269.1563	
						φR_n (Kips)	625.7883	Okay

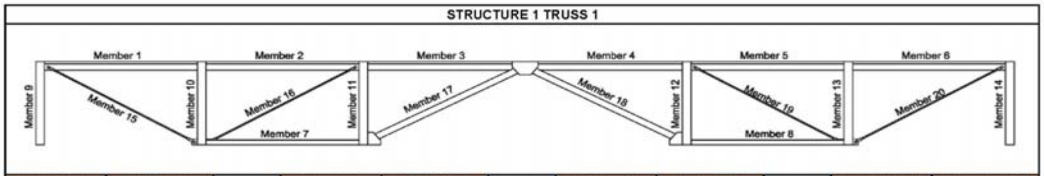
Calculation 21: Brace Double Angles to Gusset Plate Limit States

Gusset Plate to Column Connection								
Shear Stresses In Bolts			Avaible Tensile Strength Per Bolt			Calculate r_{ut}		
f_v (ksi)	29.9059		F_t (ksi)	50.10	< 90 ksi	r_{ut} (Kips)	0	No Prying
			φr_n (Kips)	22.58357426				
Weld Limit States								
Using Table 10-2 In Manual								
n	8		L (in)	23.5		Weld Size (in)	5/16	
P_u (Kips)	287.58							
φR_n (Kips)	338	Okay						
Double Angle Limit States								
Shear Yield			Shear Rupture			Block Shear		
A_{gv} (in^2)	17.625		Anv (in2)	11.625		A_{gv} (in^2)	35.25	
φR_n (Kips)	685.26	Okay	φR_n (Kips)	303.4125	Okay	A_{nv} (in^2)	24.00	
						A_{nt} (in^2)	3.86	
						$0.6F_uA_{nv}$	835.20	
						$0.6F_yA_{gv}$	761.40	Controls
						$U_{bs}F_uA_{nt}$	223.62	
						φR_n (Kips)	738.76	Okay
Bolts (Shear, Bearing, & Tearout)								
Bolt Shear			Bearing on Angles			Bearing On Column		
φR_n (Kips)	21.6	1 Bolt	$\varphi 2.4F_utd_b$	68.5125	1 Bolt	ta < tc : Only Check Angles		
Tearout Angle (1)			Tearout Angle Other (2-8)			Tearout Column Edge (8)		
L_c (in)	1.03125	1 Bolt	L_c (in)	2.0625	1 Bolt	L_c (in)	1.03125	1 Bolt
$\varphi 1.2F_uL_ct$	40.37344		$\varphi 1.2F_uL_ct$	80.746875		$\varphi 1.2F_uL_ct$	45.2460938	
Tearout Column Other (1-7)								
L_c (in)	2.0625	1 Bolt						
$\varphi 1.2F_uL_ct$	90.49219							
Bolt 1	21.6	68.51	-	40.37	90.49			
Bolt 2	21.6	68.51	-	80.75	90.49			
Bolt 3	21.6	68.51	-	80.75	90.49			
Bolt 4	21.6	68.51	-	80.75	90.49			
Bolt 5	21.6	68.51	-	80.75	90.49			
Bolt 6	21.6	68.51	-	80.75	90.49			
Bolt 7	21.6	68.51	-	80.75	45.25			
φR_n (Kips)	302.4	Okay						

Calculation 22: Gusset Plate to Column Limit States

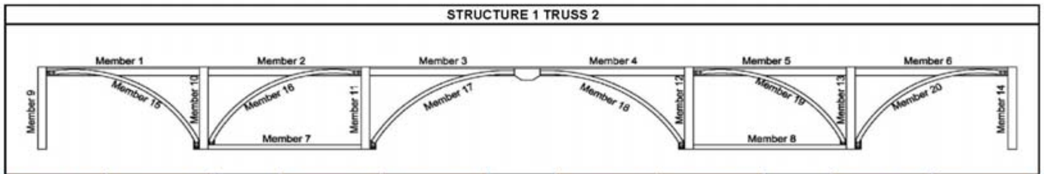
APPENDIX I: FINAL MEMBER SIZES FOR TRUSSES

STRUCTURE 1 TRUSS 1



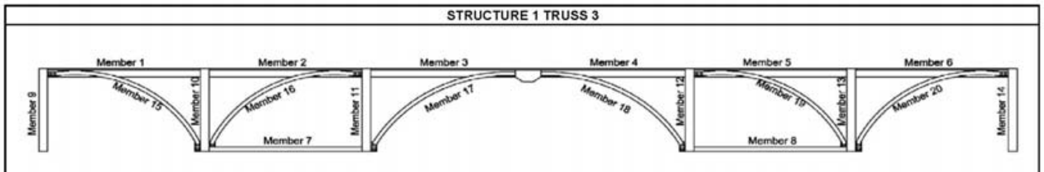
Member 1	WT15x95.5	Member 6	WT15x95.5	Member 11	W14x159	Member 16	(2) 3" Dia. Rod
Member 2	WT15x95.5	Member 7	W16x31	Member 12	W14x159	Member 17	W14x257
Member 3	WT20x251.5	Member 8	W16x31	Member 13	W14x99	Member 18	W14x257
Member 4	WT20x251.5	Member 9	W14x99	Member 14	W14x99	Member 19	(2) 3" Dia. Rod
Member 5	WT15x95.5	Member 10	W14x99	Member 15	(2) 2" Dia. Rod	Member 20	(2) 2" Dia. Rod

STRUCTURE 1 TRUSS 2



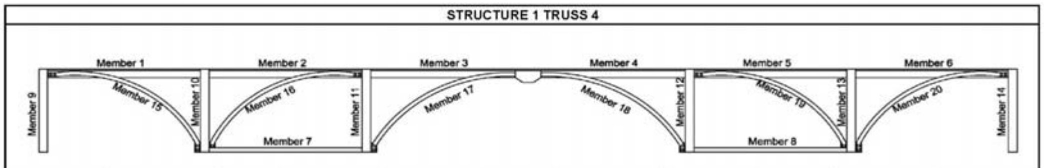
Member 1	WT15x130.5	Member 6	WT15x130.5	Member 11	W14x176	Member 16	(2) HSS10x0.50
Member 2	WT15x130.5	Member 7	W16x31	Member 12	W14x176	Member 17	(2) HSS10x0.50
Member 3	WT15x130.5	Member 8	W16x31	Member 13	W14x176	Member 18	(2) HSS10x0.50
Member 4	WT15x130.5	Member 9	W14x176	Member 14	W14x176	Member 19	(2) HSS10x0.50
Member 5	WT15x130.5	Member 10	W14x176	Member 15	(2) HSS10x0.50	Member 20	(2) HSS10x0.50

STRUCTURE 1 TRUSS 3



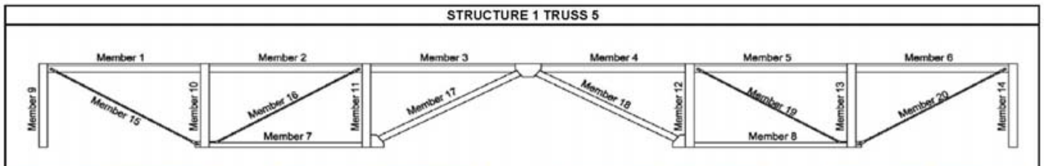
Member 1	WT15x146	Member 6	WT15x146	Member 11	W14x193	Member 16	(2) HSS10x0.50
Member 2	WT15x146	Member 7	W16x31	Member 12	W14x193	Member 17	(2) HSS10x0.50
Member 3	WT15x146	Member 8	W16x31	Member 13	W14x193	Member 18	(2) HSS10x0.50
Member 4	WT15x146	Member 9	W14x193	Member 14	W14x193	Member 19	(2) HSS10x0.50
Member 5	WT15x146	Member 10	W14x193	Member 15	(2) HSS10x0.50	Member 20	(2) HSS10x0.50

STRUCTURE 1 TRUSS 4



Member 1	WT15x195.5	Member 6	WT15x195.5	Member 11	W14x342	Member 16	HSS14x0.50
Member 2	WT15x195.5	Member 7	W16x50	Member 12	W14x342	Member 17	HSS14x0.50
Member 3	WT15x195.5	Member 8	W16x50	Member 13	W14x342	Member 18	HSS14x0.50
Member 4	WT15x195.5	Member 9	W14x342	Member 14	W14x342	Member 19	HSS14x0.50
Member 5	WT15x195.5	Member 10	W14x342	Member 15	HSS14x0.50	Member 20	HSS14x0.50

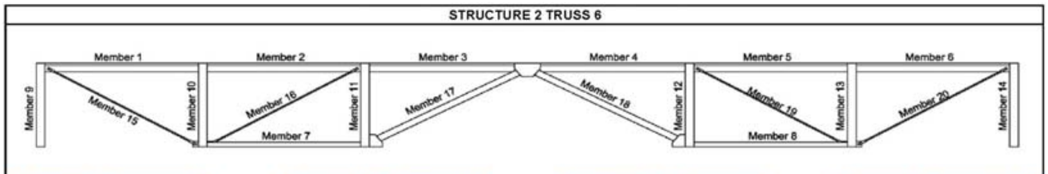
STRUCTURE 1 TRUSS 5



Member 1	WT15x95.5	Member 6	WT15x95.5	Member 11	W14x132	Member 16	(2) 3" Dia. Rod
Member 2	WT15x95.5	Member 7	W16x31	Member 12	W14x132	Member 17	W14x233
Member 3	WT20x251.5	Member 8	W16x31	Member 13	W14x99	Member 18	W14x233
Member 4	WT20x251.5	Member 9	W14x99	Member 14	W14x99	Member 19	(2) 3" Dia. Rod
Member 5	WT15x95.5	Member 10	W14x99	Member 15	(2) 2" Dia. Rod	Member 20	(2) 2" Dia. Rod

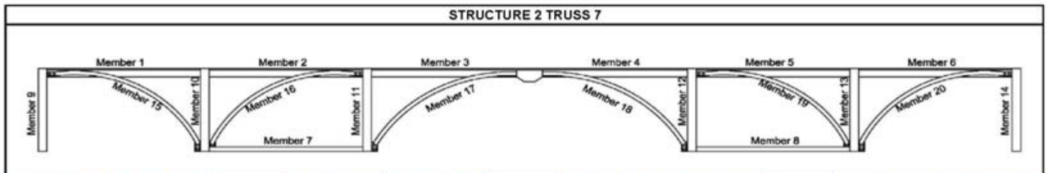
Tables 1 through 5: Final Member Sizes for Trusses

STRUCTURE 2 TRUSS 6



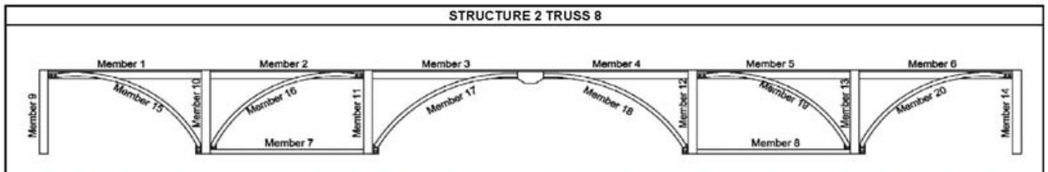
Member 1	WT15x95.5	Member 6	WT15x95.5	Member 11	W14x193	Member 16	(2) 3" Dia. Rod
Member 2	WT15x95.5	Member 7	W16x31	Member 12	W14x193	Member 17	W14x311
Member 3	WT18x400	Member 8	W16x31	Member 13	W14x99	Member 18	W14x311
Member 4	WT18x400	Member 9	W14x99	Member 14	W14x99	Member 19	(2) 3" Dia. Rod
Member 5	WT15x95.5	Member 10	W14x99	Member 15	(2) 2" Dia. Rod	Member 20	(2) 2" Dia. Rod

STRUCTURE 2 TRUSS 7



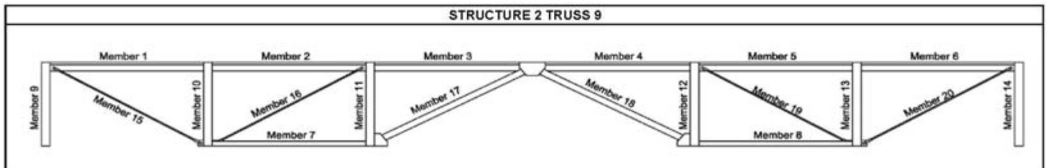
Member 1	WT15x146	Member 6	WT15x146	Member 11	W14x193	Member 16	HSS10x0.50
Member 2	WT15x146	Member 7	W16x31	Member 12	W14x193	Member 17	HSS10x0.50
Member 3	WT15x146	Member 8	W16x31	Member 13	W14x193	Member 18	HSS10x0.50
Member 4	WT15x146	Member 9	W14x193	Member 14	W14x193	Member 19	HSS10x0.50
Member 5	WT15x146	Member 10	W14x193	Member 15	HSS10x0.50	Member 20	HSS10x0.50

STRUCTURE 2 TRUSS 8



Member 1	WT15x146	Member 6	WT15x146	Member 11	W14x193	Member 16	HSS10x0.50
Member 2	WT15x146	Member 7	W16x31	Member 12	W14x193	Member 17	HSS10x0.50
Member 3	WT15x146	Member 8	W16x31	Member 13	W14x193	Member 18	HSS10x0.50
Member 4	WT15x146	Member 9	W14x193	Member 14	W14x193	Member 19	HSS10x0.50
Member 5	WT15x146	Member 10	W14x193	Member 15	HSS10x0.50	Member 20	HSS10x0.50

STRUCTURE 2 TRUSS 9



Member 1	WT15x95.5	Member 6	WT15x95.5	Member 11	W14x99	Member 16	(2) 1" Dia. Rod
Member 2	WT15x95.5	Member 7	W16x31	Member 12	W14x99	Member 17	W14x145
Member 3	WT15x95.5	Member 8	W16x31	Member 13	W14x99	Member 18	W14x145
Member 4	WT15x95.5	Member 9	W14x99	Member 14	W14x99	Member 19	(2) 1" Dia. Rod
Member 5	WT15x95.5	Member 10	W14x99	Member 15	(2) 1" Dia. Rod	Member 20	(2) 1" Dia. Rod

Tables 6 through 9: Final Member Sizes for Trusses Cont'd

APPENDIX J: LED SPECIFICATIONS

Date: ____________________ Type: ____________________

Firm Name: ____

Project: ____

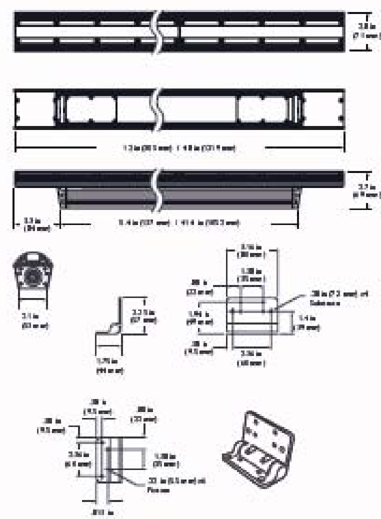
eW Graze Powercore

2700 K, 10° x 60° Lens

Linear, white LED surface light for wall washing and grazing

eW® Graze Powercore is a linear lighting fixture optimized for surface grazing and wall-washing applications requiring high-quality white light. Featuring Powercore® technology, eW Graze Powercore processes power directly from line voltage, eliminating the need for low-voltage, external power supplies. Available in 2700 K or 4000 K color temperatures, eW Graze Powercore offers superior illumination quality and dramatic energy savings for new installations and retrofit upgrades. Combining a space-efficient, low-profile aluminum housing and flexible mounting options allows for discrete placement within a wide range of compact architectural details

- Tailor light output to specific applications — eW Graze Powercore is available in 1 ft and 4 ft exterior-rated housings, with 10° x 60° and 30° x 60° beam angle options.
- High-performance illumination and beam quality — eW Graze Powercore offers superior beam quality for striation-free saturation as close as 6 in (152 mm) from fixture placement. With a 60° horizontal beam angle, eW Graze Powercore accommodates end-to-end or incremental placement without visible light scalloping between fixtures.
- Supports new applications for white light— Long-life LEDs (50,000 hours at 50% lumen maintenance) significantly reduce or eliminate maintenance problems, allowing the use of white lighting in spaces where bulb maintenance may be limited or unfeasible.
- Universal power input range — eW Graze Powercore accepts line voltage input of 100, 120, 220 – 240, and 277 VAC.
- Versatile installation options — Constant torque, locking hinges offer simple position control from various angles, without special tools. The low-profile extruded aluminum housing accommodates installation within wide-ranging architectural niches.
- "Cool lighting" functionality — eW Graze Powercore fixtures do not heat illuminated surfaces, discharge infrared radiation or emit ultraviolet light.
- Dimming capable — Patented DIMand™ technology offers smooth dimming capability with standard ELV-type dimmers.
- Trouble-free, code-compliant installation — IP66, UL wet location ratings. UL / cUL, FCC, CE, RoHS, WEEE certified.



For detailed product information, please refer to the eW Blast Powercore Product Guide at www.colorkinetics.com/ls/essentialwhite/ewgraze/

PHILIPS

Specifications

Due to continuous improvements and innovations, specifications may change without notice.

Item	Specification	1 ft	4 ft
Output	Beam Angle	10° x 60°	
	Color Temperature	2700 K (+375 / -300)	
	Lumens†	404	1616
	Efficacy (Lm/W)	27.9	
	Mixing Distance	6 in (152 mm) to uniform beam saturation	
	Lumen Maintenance‡	100,000+ hours L70 @ 25° C 50,000 hours L70 @ 50° C	
Electrical	Input Voltage	100 / 120 / 220 – 240 / 277 VAC	
	Power Consumption	14.5 W maximum at full output, steady state	58.0 W maximum at full output, steady state
Control		Commercially available ELV control dimmers	
Physical	Dimensions (Height x Width x Depth)	2.7 x 12 x 2.8 in (69 x 305 x 71 mm)	2.7 x 48 x 2.8 in (69 x 1219 x 71 mm)
	Weight	2.7 lb (1.2 kg)	10.8 lb (4.9 kg)
	Housing	Extruded anodized aluminum	
	Lens	Clear polycarbonate	
	Fixture Connectors	Integral male / female waterproof connectors	
	Mounting	Multipositional constant torque locking hinges	
	Temperature	-40° – 122° F (-40° – 50° C) Operating -4° – 122° F (-20° – 50° C) Startup	
	Humidity	0 – 95%, non-condensing	
	Fixture Run Lengths*	88 – 110 VAC 97 – 120 VAC 180 – 220 VAC 197 – 240 VAC	*Configuration: 1 ft (305 mm) fixtures installed end-to-end, 20A circuit, standard 50 ft (15.2 m) Leader Cable*
Certification and Safety	Certification	UL / cUL, FCC Class A, CE, RoHS, WEEE	
	LED Class	Class 2 LED product	
	Environment	Dry / Damp / Wet Location, IP66	

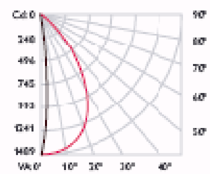
† Lumen measurement complies with IES LM 79-08.
‡ L_{70} = 70% maintenance of lumen output. (When light output drops below 70% of initial output.)
* These figures, provided as a guideline, are accurate for this configuration only. Changing the configuration can affect the fixture run lengths.

OPTIBIN | POWERCORE | DIMAND

Photometrics

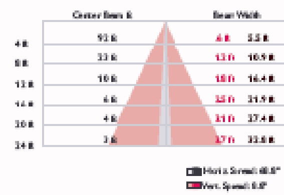
2700 K, 1 ft, 10° x 60° lens

Polar Candela Distribution



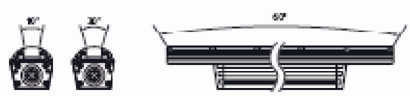
	0	22.5	44	67.5	90
0	1470	1470	1470	1470	1470
5	817	879	880	1238	1470
15	90	98	131	460	1304
25	37	43	61	138	1140
35	22	25	36	67	716
45	11	12	25	43	271
55	4	6	12	27	70
65	1	2	6	13	25
75	1	1	1	5	9
85	1	0	0	1	1
90	0	0	0	0	0

Illuminance at Distance



Power Consumption	14.5 W
Lumens	404
Efficacy	27.9 Lm/W

For lux multiply fc by 10.7



Fixtures

Item	Beam Angle	Voltage	Size	Item Number	Philips 12NC
eW Graze Powercore, 2700 K	10° x 60°	120 VAC	1 ft	523-000030-00	910503700276
			4 ft	523-000030-02	910503700278
		277 VAC	1 ft	523-000030-08	910503700284
			4 ft	523-000030-10	910503700286
		220 – 240 VAC	1 ft	523-000030-16	910503700292
			4 ft	523-000030-18	910503700294
		100 VAC	1 ft	523-000030-24	910503700300
			4 ft	523-000030-26	910503700302

Use Item Number when ordering in North America.

Accessories

Item	Type	Size	Item Number	Philips 12NC
Leader Cable	UL / cUL	50 ft (15.2 m)	108-000041-00	910503700320
	CE		108-000041-01	910503700320
Jumper Cable	UL / cUL	End-to-End	108-000039-00	910503700314
		1 ft (305 mm)	108-000039-01	910503700315
		5 ft (1.5 m)	108-000039-02	910503700316
	CE	End-to-End	108-000040-00	910503700317
		1 ft (305 mm)	108-000040-01	910503700318
		5 ft (1.5 m)	108-000040-02	910503700319

PHILIPS

Philips Color Kinetics
3 Burlington Woods Drive
Burlington, Massachusetts 01803 USA
Tel 888.FullRGB
Tel 617.423.9999
Fax 617.423.9998
www.colorkinetics.com

DAS-000009-01 R01 02-09

APPENDIX K: IESNA REQUIREMENTS

in enclosures that isolate ballast vibrations, or electronic ballasts.

ILLUMINANCE SELECTION

In 1979, the IESNA established an illuminance selection procedure, which was published in the 6th, 7th, and 8th editions of its *Lighting Handbook*. The philosophy of that procedure was to enable the lighting designer to select illuminances based on a knowledge of space and occupant characteristics as well as the task and worker characteristics.

The philosophy of that procedure has been embraced again in this edition, but the procedure has been modified and simplified to place visual performance and therefore illuminance selection more in balance with the other important lighting design criteria presented in this chapter and discussed throughout this edition of the *IESNA Lighting Handbook*. Specifically, the recommended illuminances provided in the Design Guide are based on the Society's judgment of best practice for "typical" applications. Every situation is unique so, naturally, typical conditions may not be appropriate for a specific application. As a professional, the lighting designer should have a better understanding of the particular space and the needs of the occupants and clients than what can be presented in a recommended illuminance value for a typical space.

Illuminance Recommendations

In 1979, the IESNA established nine illuminance categories, "A," the lowest set of recommended illuminances, through "I," the highest set. Each of the nine categories had general descriptions of the visual task, irrespective of the application. Generally, the same approach has been employed in this edition of the *IESNA Lighting Handbook* to help lighting designers establish the best task illuminance. However, four important modifications have been adopted.

- The recommended illuminances are no longer provided without reference to a specific application. Every application in the Design Guide has a specific recommended illuminance (horizontal, vertical, or both) representing best practice for a typical application.
- The nine illuminance selection categories established earlier by the IESNA have been reduced to seven categories and organized into three sets of visual tasks (orientation and simple, common, and special). These groupings provide additional clarity to the category descriptions (Figure 10-9).
- Additional precision has been given to the task descriptions in each category. In the previous three editions it was impossible for the lighting designer to unambiguously ascertain what constituted, for example, "low contrast" or "small size." Specific ranges of contrast and size have been established for this edition (Figures 10-10 and 10-11).
- Recommended illuminances increase roughly logarithmically with increasing task difficulty by combined changes in task contrast and task size, as defined in Figure 10-10. These recommendations are guided by both the scientific literature and practical experience.

Figure 10-9. Determination of Illuminance Categories*

Orientation and simple visual tasks. Visual performance is largely unimportant. These tasks are found in public spaces where reading and visual inspection are only occasionally performed. Higher levels are recommended for tasks where visual performance is occasionally important.

A	Public spaces	30 lx (3 fc)
B	Simple orientation for short visits	50 lx (5 fc)
C	Working spaces where simple visual tasks are performed	100 lx (10 fc)

Common visual tasks. Visual performance is important. These tasks are found in commercial, industrial and residential applications. Recommended illuminance levels differ because of the characteristics of the visual task being illuminated. Higher levels are recommended for visual tasks with critical elements of low contrast or small size.

D	Performance of visual tasks of high contrast and large size	300 lx (30 fc)
E	Performance of visual tasks of high contrast and small size, or visual tasks of low contrast and large size	500 lx (50 fc)
F	Performance of visual tasks of low contrast and small size	1000 lx (100 fc)

Special visual tasks. Visual performance is of critical importance. These tasks are very specialized, including those with very small or very low contrast critical elements. Recommended illuminance levels should be achieved with supplementary task lighting. Higher recommended levels are often achieved by moving the light source closer to the task.

G	Performance of visual tasks near threshold	3000 to 10,000 lx (300 to 1000 fc)

* Expected accuracy in illuminance calculations are given in Chapter 9, Lighting Calculations. To account for both uncertainty in photometric measurements and uncertainty in space reflections, measured illuminances should be with ± 10% of the recommended value. It should be noted, however, that the final illuminance may deviate from these recommended values due to other lighting design criteria.

High illuminances can partially compensate for small size and low contrast to maintain high levels of visual performance. Changes in visual performance as a function of task contrast and size, background reflectance, and observer age can be calculated precisely.[15] For well-controlled situations, this procedure can be a useful predictive tool. However, performance at a visual task depends on many uncontrolled vi-

Figure 1

Transportation-6

IESNA Lighting Design Guide

Very Important | Important | Somewhat important | Blank = Not important or not applicable

V. TRANSPORTATION[a] LOCATIONS AND TASKS / Design Issues	Appearance of Space and Luminaires	Color Appearance (and Color Contrast)	Daylighting Integration and Control	Direct Glare	Flicker (and Strobe)	Light Distribution on Surfaces	Light Distribution on Task Plane (Uniformity)	Luminances of Room Surfaces	Modeling of Faces or Objects	Point(s) of Interest	Reflected Glare	Shadows	Source/Task/Eye Geometry	Sparkle/Desirable Reflected Highlights	Surface Characteristics	System Control and Flexibility	Special Considerations	Notes on Special Considerations	Illuminance (Horizontal)	Category or Value (lux)[b]	Illuminance (Vertical)	Category or Value (lux)[b]	Notes on Illuminance — see end of section	Reference Chapter(s)
Workshops																								
(see Machining in Section II, Industrial)																								
General																				D		B		
On bench top																		(6)		E		C		
Machine shop																		(6)		G		E		
Cargo holds																								
Safety																				A		A		
During cargo handling																				D		B		
Passageways and trunks																				A		A		
Transportation Terminals																								
Waiting room and lounge																				B		A		Ch. 23
Ticket counters																				E		D		
Baggage checking																				D		B		
Rest rooms																				B		D		
Concourse																				A		A		
Boarding area																				B		B		

Notes:

(a) Design issues are listed for room or space. Refer to specfic task under Reading or Graphic Design and Materials, for example, for task design issues and illuminance

(b) Low illuminances (less than 30 lux) are given in lux; values greater than 30 lux are given in letter categories.

Figure 2

APPENDIX L: AVANTE 2x4 SPECIFICATIONS

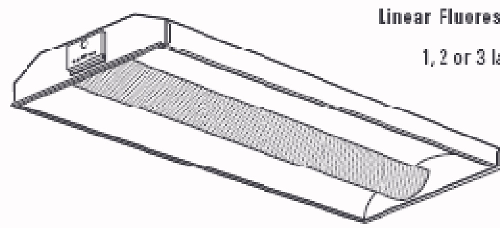
LITHONIA LIGHTING®

Catalog Number	
Notes	Type

Avante®
Recessed Direct/Indirect Lighting

2AV 2'x4'

Linear Fluorescent
T8
1, 2 or 3 lamps



FEATURES & SPECIFICATIONS

INTENDED USE

The Avante 2x4 is a general lighting luminaire for large spaces including open offices, circulation areas, classrooms, libraries, cafeterias, airport ticketing and wait areas, and numerous other commercial applications. Static or air functions available.

CONSTRUCTION

Housing is gloss white enamel on cold rolled steel. All edges hemmed or rounded.

All shieldings pivot on light traps and swing down for easy lamp access.

Molded light traps prevent light leaks between shielding and endplates.

All air and screw slot units supplied with screw-on tee bar clips. Ballast access is from below.

OPTICAL SYSTEM

Twin matte white polyester powder paint finished reflectors provide uniform light distribution. Optional low brightness diffuse aluminum stepped reflectors available.

All diffusers control direct light distribution and glare by shielding lamps from direct view.

Metal diffuser staggered round holes (MDR) 52% open perforated metal with .075" diameter holes backed with white acrylic diffuser.

Straight blade louver (SBL) sides of perforated metal with staggered round holes and solid blade louvered center. Sides and louver backed with white acrylic diffuser.

Metal diffuser aligned mini slots (MDM) 48% open perforated metal backed with white acrylic diffuser.

Acrylic diffuser prismatic lens (ADP) extruded acrylic lens backed with white acrylic diffuser.

Metal diffuser with center slots (MDC). 52% open metal, .075" diameter holes with 1" wide solid center. Slotted with 1/2"x2" open slots. Diffuser is backed with white acrylic overlay.

ELECTRICAL SYSTEM

All ballasts supplied are class P, thermally protected, resetting, HPF, non-PCB, UL Listed, CSA Certified. Ballasts are sound rated A. Standard combinations conform to UL 935.

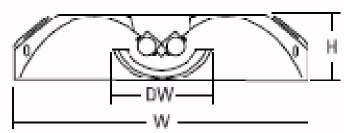
INSTALLATION

Trims available for standard 1" and 9/16" tee bar or screw slot grids.

Fixtures can be row mounted end-to-end.

Specifications

Length: 48" (1219)
Width: 24" (602)
Diffuser Width: 8" (203)
Depth: 5-1/2" (140)



All dimensions are inches (millimeters).

Drywall ceiling adapters available.

LISTING

UL Listed to US and Canadian safety standards. Chicago Plenum approved and NYC approved (see Options).

Specifications are subject to change without notice.

Avante is covered by one or more of the following patents: 5,918,829; 399,586; 411,641; 413,402; 2,212,513; 67513.

ORDERING INFORMATION

For shortest lead times, configure product using **standard options (shown in bold).**

Example: 2AV G 2 32 MDR MVOLT GEB10IS

2AV __ __ __ 32 __ __ __

Series
2AV **2' wide**

Trim type
G **Grid trim**
ST Screw slot

Air function
(blank) **Static (no air function)**
A Air return/supply

Number of lamps
1
2
3
Not included.

Lamp type
32 **32W T8 (48")**

Diffuser
MDR **Metal diffuser, round holes**
SBL Straight blade louver, round holes
MDM Metal diffuser, mini slots
ADP Acrylic diffuser, linear prismatic lens
MDC Metal diffuser, round holes with large center slots[1]
Others available.

Voltage
MVOLT[2]
347
Others available.

Options
GEB10IS **Electronic ballast, ≤ 10% THD, instant start**
GEB10PS Electronic ballast, ≤ 10% THD, program start
ALG Acrylic litter guard[1]
EL14 **Emergency battery pack (nominal 1200 lumens, see Life Safety section)**
GLR Internal fast-blow fuse[3]
LP_ Lamped. Specify lamp type and color
PWS1836 **6' prewire, 3/8" dia, 18-gauge, 3 wires**
NYC New York City approved
CP Chicago Plenum approved
APB Air pattern control blades[1]
Reflector option
ASR Aluminum stepped reflector

Accessories

Order as separate catalog number.

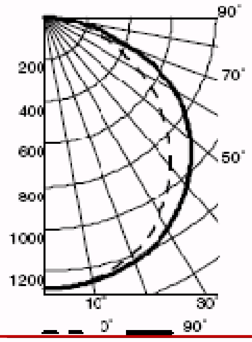
DGA24 Drywall ceiling adapter, unit installation. Use G trim plus DGA accessory for fixture trim flange and fixture support in plaster or plasterboard ceilings.

NOTES:
1 Refer to options and accessories section for more detailed information.
2 MVOLT (120-277 volt).
3 Must specify voltage, 120 or 277.

Fluorescent **Sheet #: 2AV 2X4 T8** ARCH-280

2AV 2x4 Direct/Indirect Lighting

2AV G 2 32 MDR, (2) 32W T8 lamps, 2850 lumens per lamp, s/m 1.2 (along) 1.3 (across), test no. LTL 10121



CP Summary / **Coefficients of Utilization**

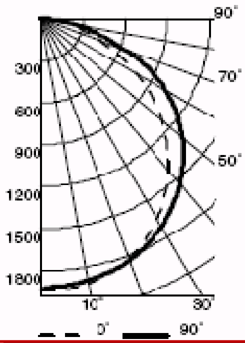
CP Summary	0°	90	pf (20%) pc → / pw ↓ (RCR)	80% 70%	80% 50%	80% 30%	70% 50%	70% 30%	70% 10%	50% 50%	50% 30%	50% 10%
0°	1280	1280	0	81	81	81	79	79	79	76	76	76
5°	1279	1277	1	74	71	68	69	66	64	66	64	62
15°	1233	1253	2	67	61	57	60	56	52	58	54	51
25°	1141	1194	3	61	54	48	53	47	43	50	46	42
35°	1005	1101	4	56	47	41	46	41	36	45	40	36
45°	828	969	5	51	42	36	41	36	31	40	35	31
55°	623	806	6	47	38	32	37	31	27	36	31	27
65°	406	590	7	44	34	28	34	28	24	33	27	24
75°	211	291	8	41	31	25	31	25	21	30	25	21
85°	46	64	9	38	29	23	28	23	19	27	22	19
90	0	0	10	36	26	21	26	21	17	25	21	17

Zonal Lumen Summary

Zone	Lumens	% Lamp	% Fixture
0° - 30°	1010	17.7	26.0
0° - 40°	1669	29.3	43.0
0° - 60°	3015	52.9	77.7
0° - 90°	3882	68.1	100.0
90° - 180°	0	0.0	0.0
0° - 180°	3882	68.1	100.0

Efficiency: 68.1%

2AV G 3 32 MDR, (3) 32W T8 lamps, 2850 lumens per lamp, s/m 1.3 (along) 1.3 (across), test no. LTL 10155



CP Summary / **Coefficients of Utilization**

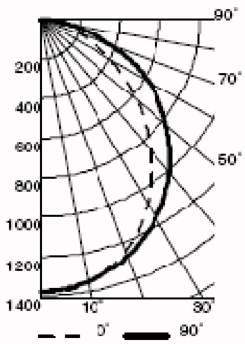
CP Summary	0°	90	pf (20%) pc → / pw ↓ (RCR)	80% 70%	80% 50%	80% 30%	70% 50%	70% 30%	70% 10%	50% 50%	50% 30%	50% 10%
0°	1934	1934	0	81	81	81	79	79	79	76	76	76
5°	1941	1913	1	74	71	68	69	67	64	66	64	62
15°	1880	1871	2	67	62	57	60	56	52	58	54	51
25°	1747	1781	3	61	54	48	53	47	43	51	46	42
35°	1539	1632	4	56	48	41	47	41	37	45	40	36
45°	1275	1435	5	51	42	36	42	36	31	40	35	31
55°	956	1179	6	47	38	32	37	32	27	36	31	27
65°	611	840	7	44	34	28	34	28	24	33	28	24
75°	298	431	8	41	31	26	31	25	21	30	25	21
85°	66	90	9	38	29	23	28	23	19	28	23	19
90	0	0	10	36	27	21	26	21	17	26	21	17

Zonal Lumen Summary

Zone	Lumens	% Lamp	% Fixture
0° - 30°	1529	17.9	26.2
0° - 40°	2527	29.6	43.3
0° - 60°	4552	53.2	78.0
0° - 90°	5837	68.3	100.0
90° - 180°	0	0.0	0.0
0° - 180°	5837	68.3	100.0

Efficiency: 68.3%

2AV G 3 32 MDR ASR, (3) 32W T8 lamps, 2850 lumens per lamp, s/m 1.2 (along) 1.3 (across), test no. LTL 10120



CP Summary / **Coefficients of Utilization**

CP Summary	0°	90	pf (20%) pc → / pw ↓ (RCR)	80% 70%	80% 50%	80% 30%	70% 50%	70% 30%	70% 10%	50% 50%	50% 30%	50% 10%
0°	1377	1377	0	50	50	50	49	49	49	47	47	47
5°	1380	1370	1	46	44	42	43	41	40	41	40	39
15°	1309	1317	2	42	39	36	38	35	33	36	34	32
25°	1171	1223	3	38	34	31	33	30	28	32	29	27
35°	974	1090	4	35	30	26	30	26	24	29	26	23
45°	745	920	5	32	27	23	26	23	20	26	23	20
55°	511	728	6	30	24	21	24	20	18	23	20	18
65°	297	514	7	28	22	18	22	18	16	21	18	16
75°	134	254	8	26	20	17	20	17	14	19	16	14
85°	33	54	9	24	19	15	18	15	13	18	15	13
90	0	0	10	23	17	14	17	14	12	16	14	12

Zonal Lumen Summary

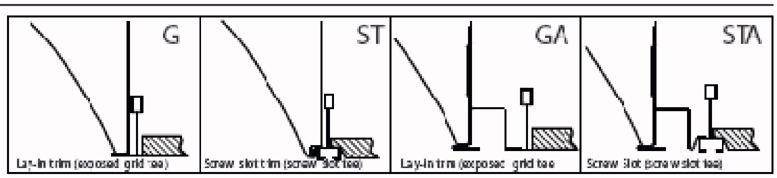
Zone	Lumens	% Lamp	% Fixture
0° - 30°	1061	12.3	20.2
0° - 40°	1697	19.9	47.1
0° - 60°	2907	34.0	80.6
0° - 90°	3606	42.2	100.0
90° - 180°	0	0.0	0.0
0° - 180°	3606	42.2	100.0

Efficiency: 42.2%

For additional photometric information, visit our website at 'lithonia.com'.

MOUNTING DATA

Ceiling Type	Appropriate Trim Type
Exposed grid tee (1" and 9/16")	G
Concealed grid tee	G
Screw slot	ST
Plaster or plasterboard	G*



*DGA accessory available to provide ceiling trim flange and fixture support for plaster or plasterboard ceiling. Recommended rough-in dimensions for DGA installation is 31-3/4" x 48-3/4" (Tolerance is +1/8", -0").

LITHONIA LIGHTING®
An Acuity Brands Company

Sheet #: 2AV 2X4 T8

Lithonia Lighting
Fluorescent
One Lithonia Way, Conyers, GA 30012
Phone: 800-858-7763 Fax: 770-922-6789
www.lithonia.com

APPENDIX M: WAITING TERMINAL CALCULATIONS

Lumen Method For Waiting Terminal 1		
Compute Cavity Ratios		
RCR	1.86	
CCR	0	
FCR	1.00	
ρ_{cc}		
Base Reflectance	80	
Wall Reflectance	60	
ρ_{cc}	1.8	62
	1.86	61
	2	60
ρ_{fc}		
Base Reflectance	30	
Wall Reflectance	60	
ρ_{fc}	1	29
	1.00	29
	1.2	28
ρ_w		
Wall Area (ft^2)	420	
Window Area (ft^2)	980	
ρ_w	0.32	
CU		
RCR	1.17	
CU @ 61/32/20	63	
Correction Factor	67	

Light Loss Factors	
1) LLD	
Mean Lumens	2802
Intial Lumens	2950
LLD	0.949831
2) LDD	
Category	VI
Cleaning Period	3
LDD	0.97
3) RSDD	
RSDD	0.9
4) BF	
BF	1.18
LLF	0.978458
# Luminares	10.55712

Calculation 1: Lumen Method for Waiting Terminal 1

CU @ 72/28/20

	80		
	50	32	30
1	71		68
1.17	69.30	66.447	66.13
2	61		57

CU

61
63

	70		
	50	32	30
1	69		66
1.17	67.47	64.617	64.30
2	60		56

Calculation 2: Determining CU

GENERAL INFORMATION

Project identification: Union Station Expansion : Waiting Terminal
(Give name of area and/or building and room number)

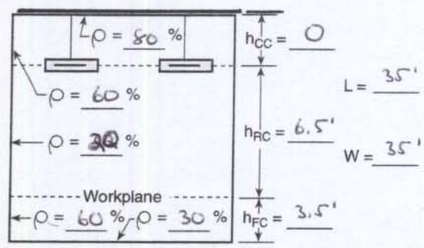
Average maintained illuminance for design: __ lux or 50 footcandles

Lamp data:
Type and color: T8 White
Number per luminaire: 3
Total lumens per luminaire: 2950(3) = 8850

Luminaire data:
Manufacturer: Avante [Lithonia]
Catalog number: 2AV-6-3-32-MDR-MVOLT-GEB10IS

SELECTION OF COEFFICIENT OF UTILIZATION

Step 1: Fill in sketch at right



Step 2: Determine Cavity Ratios

Room Cavity Ratio, RCR = 1.86
Ceiling Cavity Ratio, CCR = 0
Floor Cavity Ratio, FCR = 1.00

Step 3: Obtain Effective Ceiling Cavity Reflectance (ρ_{CC}) ρ_{CC} = 61

Step 4: Obtain Effective Floor Cavity Reflectance (ρ_{FC}) ρ_{FC} = 29

Step 5: Obtain Coefficient of Utilization (CU) from Manufacturer's Data CU = 67

SELECTION OF LIGHT LOSS FACTORS

Nonrecoverable		Recoverable	
Luminaire ambient temperature	—	Room surface dirt depreciation RSDD	0.9
Voltage to luminaire	—	Lamp lumen depreciation LLD	0.95
Ballast factor	1.18	Lamp burnouts factor LBO	1.0
Luminaire surface depreciation	—	Luminaire dirt depreciation LDD	0.47

Total light loss factor, LLF (product of individual factors above) = 0.478

CALCULATIONS

(Average Maintained Illuminance)

$$\text{Number of Luminaires} = \frac{(\text{Illuminance}) \times (\text{Area})}{(\text{Lumens per Luminaire}) \times (\text{CU}) \times (\text{LLF})}$$

$$= \frac{50(35)(35)}{(3)(2950)(0.67)(0.478)} = 10.56$$

$$\text{Illuminance} = \frac{(\text{Number of Luminaires}) \times (\text{Lumens per Luminaire}) \times (\text{CU}) \times (\text{LLF})}{(\text{Area})}$$

$$= \frac{12(3)(2950)(0.67)(0.478)}{(35)(35)} = 57\ \text{Fc}$$

Calculated by: Joseph W. Wilcher III Date: 3/17/09

Figure 9-25. Average illuminance calculation sheet.

Figure 9-27. Percent Effective Ceiling or Floor Cavity Reflectances for Various Reflectance Combinations*

Percent Base† Reflectance: 90

Cavity Ratio \ Percent Wall Reflectance	90	80	70	60	50	40	30	20	10	0
0.2	89	88	88	87	86	85	85	84	84	82
0.4	88	87	86	85	84	83	81	80	79	76
0.6	87	86	84	82	80	79	77	76	74	73
0.8	87	85	82	80	77	75	73	71	69	67
1.0	86	83	80	77	75	72	69	66	64	62
1.2	85	82	78	75	72	69	66	63	60	57
1.4	85	80	77	73	69	65	62	59	57	52
1.6	84	79	75	71	67	63	59	56	53	50
1.8	83	78	73	69	64	60	56	53	50	48
2.0	83	77	72	67	62	56	53	50	47	43
2.2	82	76	70	65	59	54	50	47	44	40
2.4	82	75	69	64	58	53	48	45	41	37
2.6	81	74	67	62	56	51	46	42	38	35
2.8	81	73	66	60	54	49	44	40	36	34
3.0	80	72	64	58	52	47	42	38	34	30
3.2	79	71	63	56	50	45	40	36	32	28
3.4	79	70	62	54	48	43	38	34	30	27
3.6	78	69	61	53	47	42	36	32	28	25
3.8	78	69	60	51	45	40	35	31	27	23
4.0	77	69	58	51	44	39	33	29	25	22
4.2	77	62	57	50	43	37	32	28	24	21
4.4	76	61	56	49	42	36	31	27	23	20
4.6	76	60	55	47	40	35	30	26	22	19
4.8	75	59	54	46	39	34	28	25	21	18
5.0	75	59	53	45	38	33	28	24	20	16
6.0	73	61	49	41	34	29	24	20	16	11
7.0	70	58	45	38	30	27	21	18	14	08
8.0	68	55	42	35	27	23	18	15	12	06
9.0	66	52	38	31	25	21	16	14	11	05
10.0	65	51	36	29	22	19	15	11	09	04

Percent Base† Reflectance: 80

Cavity Ratio \ Percent Wall Reflectance	90	80	70	60	50	40	30	20	10	0
0.2	79	78	78	77	77	76	76	75	74	72
0.4	79	77	76	75	74	73	72	71	70	68
0.6	78	76	75	73	71	70	68	66	65	63
0.8	78	75	73	71	69	67	65	63	61	57
1.0	77	74	72	69	67	65	62	60	57	55
1.2	76	73	70	67	64	61	58	55	53	51
1.4	76	72	68	65	62	59	55	53	50	48
1.6	75	71	67	63	60	57	53	50	47	44
1.8	75	70	66	62	58	54	50	47	44	41
2.0	74	69	64	60	56	52	48	45	41	38
2.2	74	68	63	58	54	49	45	42	38	35
2.4	73	67	61	56	52	47	43	40	36	33
2.6	73	66	60	55	50	45	41	38	34	31
2.8	73	65	59	53	48	43	39	36	32	29
3.0	72	65	58	52	47	42	37	34	30	27
3.2	72	65	57	51	45	40	35	33	28	25
3.4	71	64	56	49	44	39	34	32	27	24
3.6	71	63	54	48	43	38	32	30	25	23
3.8	70	62	53	47	41	36	31	28	24	22
4.0	70	61	53	46	40	35	30	26	22	20
4.2	69	60	52	45	39	34	29	25	21	18
4.4	69	60	51	44	38	33	28	24	20	17
4.6	69	59	50	43	37	32	27	23	19	15
4.8	68	58	49	42	36	31	26	22	18	14
5.0	68	58	48	41	35	30	25	21	18	14
6.0	66	55	44	38	31	27	22	19	15	10
7.0	64	53	41	35	28	24	19	16	12	07
8.0	62	50	38	32	25	21	17	14	11	05
9.0	61	49	36	30	23	19	15	13	10	04
10.0	59	46	33	27	21	18	14	11	08	03

Percent Base† Reflectance: 70

Cavity Ratio \ Percent Wall Reflectance	90	80	70	60	50	40	30	20	10	0
0.2	70	69	68	68	67	67	66	66	65	64
0.4	69	68	67	66	65	64	63	62	61	58
0.6	69	67	65	64	63	61	59	58	57	54
0.8	68	66	64	62	60	58	56	55	53	50
1.0	68	65	62	60	58	55	53	52	50	47
1.2	67	64	61	59	57	54	50	48	46	44
1.4	67	63	60	58	55	51	47	45	44	41
1.6	67	62	59	56	53	47	45	43	41	38
1.8	66	61	58	54	51	46	42	40	38	35
2.0	66	60	56	52	49	45	40	38	36	33
2.2	66	60	55	51	48	43	38	36	34	32
2.4	65	60	54	50	46	41	37	35	32	30
2.6	65	59	54	49	45	40	35	33	30	28
2.8	65	59	53	48	43	38	33	30	28	26
3.0	64	58	52	47	42	37	32	29	27	24
3.2	64	58	51	46	40	36	31	28	25	23
3.4	64	57	50	45	39	35	29	27	24	22
3.6	63	56	49	44	38	33	28	25	22	20
3.8	63	56	49	43	37	32	27	24	21	19
4.0	63	55	48	42	36	31	26	23	20	17
4.2	62	55	47	41	35	30	25	22	19	16
4.4	62	54	46	40	34	29	24	21	18	15
4.6	62	53	45	39	33	28	24	21	17	14
4.8	62	53	45	38	32	27	23	20	16	13
5.0	61	52	44	36	31	26	22	19	16	12
6.0	60	51	41	35	28	24	19	16	13	09
7.0	58	43	38	32	26	22	17	14	11	06
8.0	57	46	35	29	23	19	15	13	10	05
9.0	56	45	33	27	21	18	14	12	09	04
10.0	55	43	31	25	19	16	12	10	08	03

Percent Base† Reflectance: 60

Cavity Ratio \ Percent Wall Reflectance	90	80	70	60	50	40	30	20	10	0
0.2	60	59	59	59	58	57	56	56	55	53
0.4	60	59	59	58	57	55	54	53	52	50
0.6	60	58	57	56	55	53	51	51	50	46
0.8	59	57	56	55	54	51	48	47	46	43
1.0	59	57	55	53	51	48	45	44	43	41
1.2	59	56	54	51	49	46	44	42	40	38
1.4	59	56	53	49	47	44	41	39	38	36
1.6	59	55	52	48	45	42	39	37	35	33
1.8	58	55	51	47	44	40	37	35	33	31
2.0	58	54	50	46	43	39	35	33	31	29
2.2	58	53	49	45	42	37	34	31	29	28
2.4	58	53	48	44	41	36	32	30	27	26
2.6	58	53	48	43	39	35	31	28	26	24
2.8	58	53	47	43	38	34	29	27	24	22
3.0	57	52	46	42	37	32	28	25	23	20
3.2	57	51	45	41	36	31	27	23	22	18
3.4	57	51	45	40	35	30	26	23	20	17
3.6	57	50	44	39	34	29	25	22	19	16
3.8	57	50	43	38	33	29	24	21	19	15
4.0	57	49	42	37	32	28	23	20	18	14
4.2	56	49	42	37	32	27	22	19	17	14
4.4	56	49	42	36	31	27	22	19	16	13
4.6	56	49	41	35	30	26	21	18	16	13
4.8	56	48	41	34	29	25	21	18	15	12
5.0	56	48	40	34	28	24	20	17	14	11
6.0	55	45	37	31	25	21	17	14	11	07
7.0	54	43	35	30	24	20	15	12	09	05
8.0	53	42	33	28	22	18	14	11	08	04
9.0	52	40	31	26	20	16	12	10	07	03
10.0	51	39	29	24	18	15	11	09	07	02

Percent Base† Reflectance: 50

Cavity Ratio \ Percent Wall Reflectance	90	80	70	60	50	40	30	20	10	0
0.2	50	50	49	49	48	48	47	46	46	44
0.4	50	49	48	48	47	46	45	45	44	42
0.6	50	48	47	46	45	44	43	42	41	38
0.8	50	48	47	45	44	42	40	39	38	36
1.0	50	48	46	44	43	41	38	37	36	34
1.2	50	47	45	43	41	39	36	35	34	29
1.4	50	47	45	42	40	38	35	34	35	27
1.6	50	47	44	41	39	36	33	32	30	26
1.8	50	46	43	40	38	35	31	30	28	25
2.0	50	46	43	40	37	34	30	28	26	24
2.2	50	46	42	38	36	33	29	27	24	22
2.4	50	46	42	37	35	31	27	25	23	21
2.6	50	46	41	37	34	30	26	23	21	20
2.8	50	46	41	36	33	29	25	22	20	19
3.0	50	45	40	36	32	28	24	21	19	17
3.2	50	44	39	35	31	27	23	20	18	16
3.4	50	44	39	35	30	26	22	19	17	15
3.6	50	44	39	34	29	25	21	18	16	14
3.8	50	44	38	34	29	25	21	17	15	13
4.0	50	44	38	33	28	24	20	17	15	12
4.2	50	43	37	32	28	24	20	17	14	12
4.4	50	43	37	32	27	23	19	16	13	11
4.6	50	43	36	31	26	22	18	15	13	10
4.8	50	43	36	31	26	22	18	15	12	09
5.0	50	42	35	30	25	21	17	14	12	09
6.0	50	42	34	29	23	19	15	13	10	06
7.0	49	41	32	27	21	18	14	11	08	05
8.0	49	40	30	25	19	16	12	10	07	03
9.0	48	39	29	24	18	15	11	09	07	03
10.0	47	37	27	22	17	14	10	08	06	02

* Values in this table are based on a length to width ratio of 1.6.
†Ceiling, floor, or floor of cavity.

Figure 9-27. *Continued**

Percent Base† Reflectance	40									
Percent Wall Reflectance	90	80	70	60	50	40	30	20	10	0
Cavity Ratio										
0.2	40	40	39	39	39	38	38	37	36	36
0.4	41	40	39	39	38	37	36	35	34	34
0.6	41	40	39	38	37	36	34	33	32	31
0.8	41	40	38	37	36	35	33	32	31	29
1.0	42	40	38	37	35	33	32	31	29	27
1.2	42	40	38	36	34	32	30	29	27	25
1.4	42	39	37	35	33	31	29	27	25	23
1.6	42	39	37	35	32	30	27	25	23	22
1.8	42	39	36	34	31	29	26	24	22	21
2.0	42	39	36	34	31	28	25	23	21	19
2.2	42	39	36	33	30	27	24	22	19	18
2.4	43	39	35	33	29	27	24	21	18	17
2.6	43	39	35	32	29	26	23	20	17	15
2.8	43	39	35	32	28	25	22	19	16	14
3.0	43	39	35	31	27	24	21	18	16	13
3.2	43	39	35	31	27	23	20	71	15	13
3.4	43	39	34	30	26	23	20	17	14	12
3.6	44	39	34	30	26	22	19	16	14	11
3.8	44	38	33	29	25	22	18	16	13	10
4.0	44	38	33	29	25	21	18	15	12	10
4.2	44	38	33	29	24	21	17	15	12	10
4.4	44	38	33	28	24	20	17	14	11	09
4.6	44	38	32	28	23	19	16	14	11	08
4.8	44	38	32	27	22	19	16	13	10	08
5.0	45	38	31	27	22	19	15	13	10	07
6.0	44	37	30	25	20	17	13	11	08	05
7.0	44	36	29	24	19	16	12	10	07	04
8.0	44	35	28	23	18	15	11	09	06	03
9.0	44	35	26	21	16	13	10	08	05	02
10.0	43	34	25	20	15	12	08	07	05	02

Percent Base† Reflectance	30									
Percent Wall Reflectance	90	80	70	60	50	40	30	20	10	0
Cavity Ratio										
0.2	31	31	30	30	29	29	29	28	28	27
0.4	31	31	30	30	29	28	28	27	26	25
0.6	32	31	30	29	28	27	26	26	25	23
0.8	32	31	30	29	28	26	25	25	23	22
1.0	33	32	30	29	27	25	24	23	22	20
1.2	33	32	30	28	27	25	23	22	21	19
1.4	34	32	30	28	26	24	22	21	19	18
1.6	34	33	29	27	25	23	22	20	18	17
1.8	35	33	29	27	25	23	21	19	17	16
2.0	35	33	29	26	24	22	20	18	16	14
2.2	36	32	29	26	24	22	19	17	15	13
2.4	36	32	29	26	24	22	19	16	14	12
2.6	36	32	29	25	23	21	18	16	14	12
2.8	37	33	29	25	23	21	17	15	13	11
3.0	37	33	29	25	22	20	17	15	12	10
3.2	37	33	29	25	22	19	16	14	12	10
3.4	37	33	29	25	22	19	16	14	11	09
3.6	38	33	29	24	21	18	15	13	10	09
3.8	38	33	28	24	21	18	15	13	10	08
4.0	38	33	28	24	21	18	14	12	09	07
4.2	38	33	28	24	20	17	14	12	09	07
4.4	39	33	28	24	20	17	14	11	09	06
4.6	39	33	28	24	20	17	13	10	08	06
4.8	39	33	28	24	20	17	13	10	08	05
5.0	39	33	28	24	19	16	13	10	08	05
6.0	39	33	27	23	18	15	11	09	06	04
7.0	40	33	26	22	17	14	10	08	05	03
8.0	40	33	26	21	16	13	09	07	04	02
9.0	40	33	25	20	15	12	09	07	04	02
10.0	40	32	24	19	14	11	08	06	03	01

Percent Base† Reflectance	20									
Percent Wall Reflectance	90	80	70	60	50	40	30	20	10	0
Cavity Ratio										
0.2	21	20	20	20	20	20	19	19	19	17
0.4	22	21	20	20	20	19	19	18	18	16
0.6	23	21	21	20	19	19	18	18	17	15
0.8	24	22	21	20	19	19	18	17	16	14
1.0	25	23	22	20	19	18	17	16	15	13
1.2	25	23	22	20	19	17	17	16	14	12
1.4	26	24	22	20	18	17	16	15	13	12
1.6	26	24	22	20	18	17	16	15	13	11
1.8	27	25	23	20	18	17	15	14	12	10
2.0	28	25	23	20	18	16	15	13	11	09
2.2	28	25	23	20	18	16	14	12	10	09
2.4	29	26	23	20	18	16	14	12	10	08
2.6	29	26	23	20	18	16	14	11	09	08
2.8	30	27	23	20	18	15	13	11	09	07
3.0	30	27	23	20	17	15	13	11	09	07
3.2	31	27	23	20	17	15	12	11	09	06
3.4	31	27	23	20	17	15	12	10	08	06
3.6	32	27	23	20	17	15	12	10	08	05
3.8	32	28	23	20	17	15	12	10	07	05
4.0	33	28	23	20	17	14	11	09	07	05
4.2	33	28	23	20	17	14	11	09	07	04
4.4	34	28	24	20	17	14	11	09	07	04
4.6	34	29	24	20	17	14	11	09	07	04
4.8	35	29	24	20	17	13	10	08	06	04
5.0	35	29	24	20	16	13	10	08	06	04
6.0	36	30	24	20	16	13	10	08	05	02
7.0	36	30	24	20	15	12	03	07	04	02
8.0	37	30	23	19	15	12	08	06	03	01
9.0	37	29	23	19	14	11	08	06	03	01
10.0	37	29	22	18	13	10	07	05	03	01

Percent Base† Reflectance	10									
Percent Wall Reflectance	90	80	70	60	50	40	30	20	10	0
Cavity Ratio										
0.2	11	11	11	10	10	10	10	09	09	09
0.4	12	11	11	11	11	10	10	09	09	08
0.6	13	13	12	11	11	10	10	09	08	08
0.8	15	14	13	12	11	10	10	09	08	07
1.0	16	14	13	12	12	11	10	09	08	07
1.2	17	15	14	13	12	11	10	09	07	06
1.4	18	16	14	13	12	11	10	09	07	06
1.6	19	17	15	14	12	11	09	08	07	06
1.8	19	17	15	14	13	11	09	08	06	05
2.0	20	18	16	14	13	11	09	08	06	05
2.2	21	19	16	14	13	11	09	07	06	05
2.4	22	19	17	15	13	11	09	07	06	05
2.6	23	20	17	15	13	11	09	07	06	04
2.8	23	20	18	16	13	11	09	07	05	03
3.0	24	21	18	16	13	11	09	07	05	03
3.2	25	21	18	16	13	11	09	07	05	03
3.4	26	22	18	16	13	11	09	07	05	03
3.6	26	22	19	16	13	11	09	06	04	03
3.8	27	23	19	17	14	11	09	06	04	02
4.0	27	23	20	17	14	11	09	06	04	02
4.2	28	24	20	17	14	11	09	06	04	02
4.4	28	24	20	17	14	11	08	06	04	02
4.6	29	25	20	17	14	11	08	06	04	02
4.8	29	25	20	17	14	11	08	06	04	02
5.0	30	25	20	17	14	11	08	06	04	02
6.0	31	26	21	18	14	11	08	06	03	01
7.0	32	27	21	17	13	11	08	06	03	01
8.0	33	27	21	17	13	10	07	05	03	01
9.0	34	28	21	17	13	10	07	05	02	01
10.0	34	28	21	17	12	10	07	05	02	01

Percent Base† Reflectance	0									
Percent Wall Reflectance	90	80	70	60	50	40	30	20	10	0
Cavity Ratio										
0.2	02	02	02	01	01	01	01	00	00	0
0.4	04	03	03	02	02	02	01	01	00	0
0.6	05	05	04	03	03	02	02	01	01	0
0.8	07	06	05	04	04	03	02	02	01	0
1.0	08	07	06	05	04	03	02	02	01	0
1.2	10	08	07	06	05	04	03	02	01	0
1.4	11	09	08	07	06	04	03	02	01	0
1.6	12	10	09	07	06	05	03	02	01	0
1.8	13	11	09	08	07	05	04	03	01	0
2.0	14	12	10	09	07	05	04	03	01	0
2.2	15	13	11	09	07	06	04	03	01	0
2.4	16	13	11	09	08	06	04	03	01	0
2.6	17	14	12	10	08	06	05	03	02	0
2.8	17	15	13	10	08	07	05	03	02	0
3.0	18	16	13	11	09	07	05	03	02	0
3.2	19	16	14	11	09	07	05	03	02	0
3.4	20	17	14	12	09	07	05	03	02	0
3.6	20	17	15	12	10	08	05	04	02	0
3.8	21	18	15	12	10	08	05	04	02	0
4.0	22	18	15	13	10	08	05	04	02	0
4.2	22	19	16	13	10	08	06	04	02	0
4.4	23	19	16	13	10	08	06	04	02	0
4.6	23	20	17	13	11	08	06	04	02	0
4.8	24	20	17	14	11	08	06	04	02	0
5.0	25	21	17	14	11	08	06	04	02	0
6.0	27	23	18	15	12	09	06	04	02	0
7.0	28	24	19	15	12	09	06	04	02	0
8.0	30	25	20	15	12	09	06	04	02	0
9.0	31	25	20	15	12	09	06	04	02	0
10.0	31	25	20	15	12	09	06	04	02	0

* Values in this table are based on a length to width ratio of 1.6.
† Ceiling, floor, or floor of cavity.