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Caltrain bridging structure and commercial buildings

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Caltrain Bridging Structure and Commercial Buildings

Senior Design Project Report
by
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Spring 2013

Department of Civil Engineering
Santa Clara University

SANTA CLARA UNIVERSITY

Department of Civil Engineering

I hereby recommend that the
SENIOR DESIGN PROJECT REPORT
prepared under my supervision by

Anthony Navarrete
&
Guadalupe Gonzalez

entitled

CALTRAIN BRIDGING STRUCTURE AND COMMERCIAL BUILDINGS

be accepted in partial fulfillment of the requirements for
the degree of

BACHELOR OF SCIENCE IN CIVIL ENGINEERING

______________________________________ ____________
Advisor Date

______________________________________ ____________
Chairman of Department Date

CALTRAIN BRIDGING STRUCTURE AND COMMERCIAL BUILDINGS

by

Anthony Navarrete
&
Guadalupe Gonzalez

SENIOR DESIGN PROJECT

submitted to
the Department of Civil Engineering

of

SANTA CLARA UNIVERSITY

in partial fulfillment of the requirements
for the degree of
Bachelor of Science in Civil Engineering

Santa Clara, California

Spring 2013

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CALTRAIN BRIDGING STRUCTURE AND COMMERCIAL BUILDINGS

Anthony Navarrete and Guadalupe Gonzalez

Department of Civil Engineering
Santa Clara University, Spring 2013

ABSTRACT

The proposed project involves the design of three tower structures connecting a pedestrian bridge to help provide commuters, bicyclists, and pedestrians access to the east and west sides of the Santa Clara Caltrain rail system. Since the new San Jose Earthquakes stadium is currently being constructed on the east side of the tracks adjacent to Coleman Avenue, high business potential for a possible multi-use retail development exists. In addition to the three towers, three commercial structures will be designed in order to satisfy the need for a retail development. This project will serve as an attraction that will help attract commuters, travelers, and soccer fans. The combined development of the towers and the commercial structures will also allow commuters to access the San Jose Airport.

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1.0 <u>INTRODUCTION AND BACKGROUND</u>

Adjacent to the San Jose Airport off Coleman Avenue are three vacant parcels, one of which is currently under development. The new San Jose Earthquakes stadium has broken ground and is currently under construction on the southern most lot. In order to access this site from the Santa Clara University campus, one must take either West Hedding Street, a three mile detour, or De La Cruz Boulevard, a two mile detour. However, pedestrians and bicyclists have no access through the De La Cruz Boulevard route. The presence of the Caltrain rail system limits students and other citizens around the Santa Clara campus to take one of these two routes. Figure 1 shows the vicinity map of the vacant area of interest.

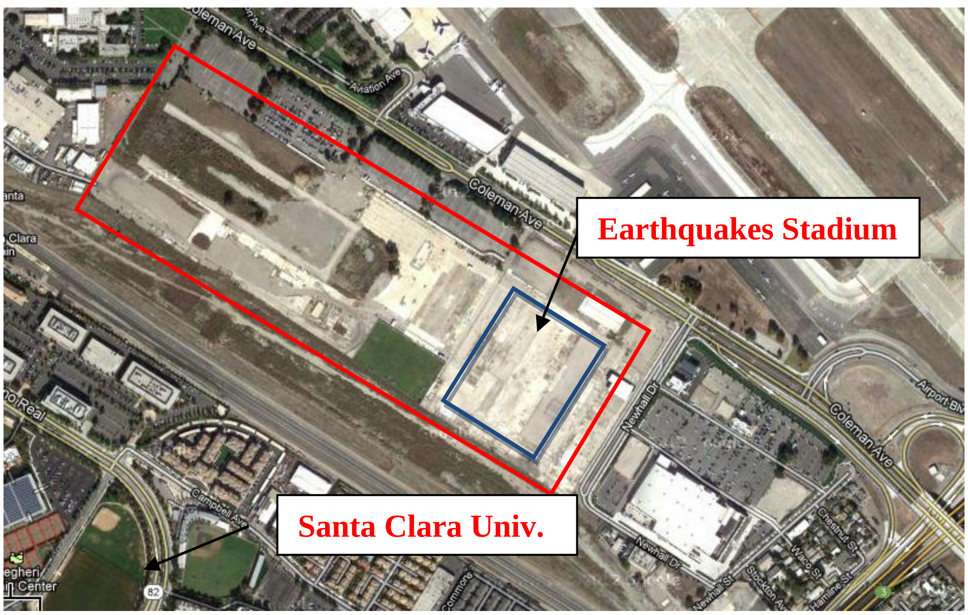

Figure 1: Aerial view of vacant lot

The Caltrain rail system runs as far south as Gilroy and as far north as downtown San Francisco. Some of the most popular attractions where the Caltrain makes stops are the HP Pavillion, the San Francisco Airport, and AT&T Park. Once the Earthquakes stadium is fully constructed, it will be located directly across the Santa Clara Transit Center. Yet, with the presence of such a major attraction, there is currently no access to the stadium site from the transit station. Again, the only ways to access the site are through the aforementioned routes. There is also no access to the San Jose Airport from the transit center. Caltrain riders must currently stop at the Santa Clara station and take a Valley Transportation Authority (VTA) shuttle bus in order to reach the airport.

The area surrounding the Caltrain station and the proposed site consists mostly of residential and industrial development, with a small presence of commercial and mixed-use facilities. Nearby commercial development includes Costco, approximately a half mile north, and a shopping center approximately one mile south which contains: Target, Marshalls, Michaels, and various restaurants. Another popular destination available for the Santa Clara community is Santana Row and the Valley Fair Shopping Center, which are three miles away from the university campus. Although these are readily available options for the Santa Clara and San Jose areas, the Caltrain station does not stop near these sites, making them difficult for pedestrians, bicyclists, and commuters to reach by mode of mass transit.

1.1 PROJECT NEED

There is currently a need for a structural development that would bridge the gap between the east and west sides of the Santa Clara Caltrain rail system. Such a development would allow for easy access to opposite sides, connecting the Santa Clara Transit Center to the San Jose Airport and Earthquakes stadium as a result. By connecting the Santa Clara Transit Center to the airport and stadium, the need for a VTA shuttle from the transit center would be eliminated, resulting in the reduction of vehicle gas mileage and carbon emissions.

However, a bridging structure is not all that is needed for the area surrounding the transit center. When researching plans for the vacant land, it was discovered in the Draft

Environmental Impact Report (EIR) for the stadium that a planned development of residential complexes was proposed adjacent to the new stadium. Some agencies that were opposed to that development were VTA and the Public Utilities Commission. Both expressed their concerns in the EIR about the lack of transit supportive facilities and access to the Earthquakes stadium. A commercial development on the remaining vacant area would fulfill the need for an entertainment center near the transit center while also creating a point of interest along the Caltrain rail system, becoming a transit supportive facility in effect.

2.0 SUMMARY OF ALTERNATIVE ANALYSIS

The original intent of the project was to provide access to pedestrians and bicyclists between the west and the east sides of the Santa Clara Caltrain tracks. As the project evolved, a priority that arose was to implement a plan that would increase the potential of all of the surrounding businesses and also fulfill the need for an entertainment center. Several alternative solutions were taken into consideration.

A simple pedestrian bridge would be sufficient to bridge the gap between the east and west sides of the tracks and provide access to those using mass transit. It would also be of use to access the new Earthquakes stadium or to provide a faster way of accessing the San Jose Airport. The cost of this pedestrian bridge would be very minimal, as it would be built out of HSS steel sections, and would resemble pedestrian bridges seen across the Bay Area. An example of the pedestrian bridge that could be implemented was the bridge in the Mountain View Caltrain station, which can be seen in Figure 2. This proposed solution would grant access and would provide a fast solution to the aforementioned needs.

Figure 2: Mountain View Caltrain pedestrian bridge

Another solution discussed was the use of a tunnel system. The tunnel system would allow pedestrians to cross the tracks from El Camino Real to the Coleman Avenue side. Currently, the Santa Clara Caltrain has a tunnel system, which allows pedestrians going north to access the north terminal. However, these tunnels that exist are relatively short in length. The tunnel that would be proposed would be over five hundred feet in length. In terms of serviceability, this proposed idea was considered to be dangerous in nature. The length of the tunnel would present security issues. Also, aesthetically, it did not appeal towards tourists and mass crowds. One of the purposes of having this access to the east side of the tracks was to increase business potential around the area. Having a tunnel would not be beneficial to existing and future businesses.

The two alternative solutions were analyzed and were found to be relatively cost effective and would not be very disturbing to the surrounding businesses during construction. However, although they are cost effective, they do not provide the Santa Clara community with an entertainment center and do not appeal to visitors.

2.1 FINAL SOLUTION

In the end, the most appropriate and beneficial design was determined to be three tower structures that support a pedestrian bridge. The pedestrian bridge will span from the west end of the current Caltrain station to the east side adjacent to Coleman Avenue, crossing over the active rail system. Running trains will funnel through the structures from both northbound and southbound directions. In conjunction with the towers, three commercial structures will also be designed in order to fulfill the need for a mixed use, transit supportive facility. Figure 3 illustrates the approximate location of these structures.

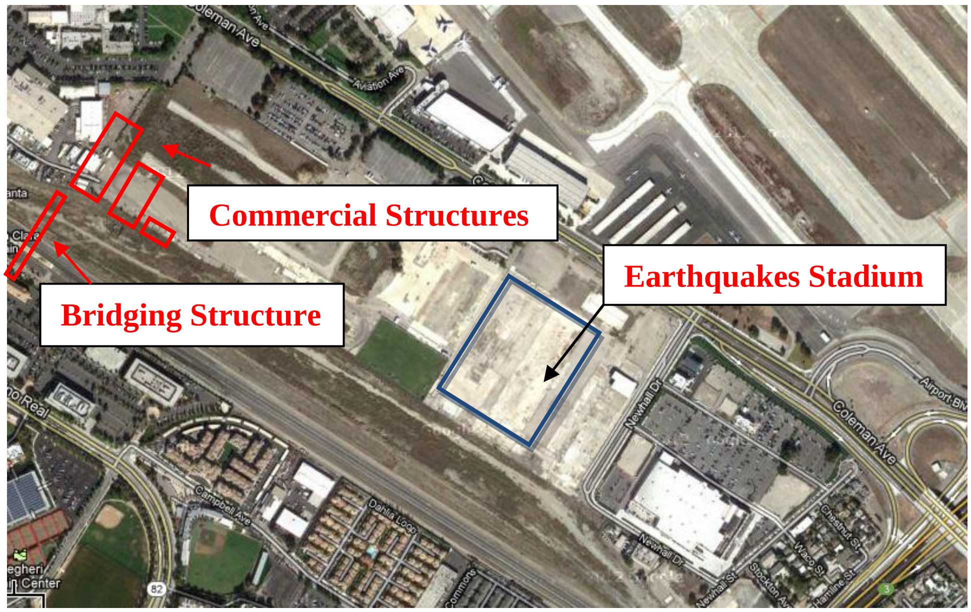

Figure 3: Aerial view of general project outline

The commercial structures will serve as a staging area for commuters trying to access the San Jose Airport and will consist of restaurants, bars, retail stores, and various other entertainment establishments. Train commuters, pedestrians, and bicyclists will be able to reach the new entertainment center from the transit center via the tower structures. They will eliminate the need for bicyclists and pedestrians to take the W. Hedding Street route by providing a short 0.3 mile access route. This proposal will also cater to the future development of the California High Speed Rail and Bart station, which will see extensions run adjacent to the current Caltrain tracks and proposed site in the near future. Figure 4 shows a model of the proposed towers and commercial structures.

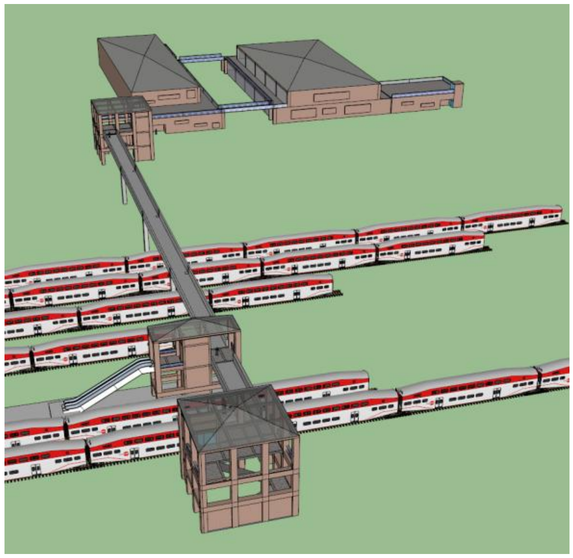

Figure 4: Proposed project looking east (not to scale)

The project will also draw significant commercial business opportunities for the neighboring communities. The Caltrain station could experience an increase in ridership because of its proximity to the airport. Commuters can board shuttles from a shuttle system on the east side of the tracks near Coleman Avenue incorporated into the commercial structures. This shuttle system will be a swift and convenient alternative to the current system provided by VTA through route ten from the Caltrain station, which must travel north on El Camino Real, over the De La Cruz Boulevard bridge to Coleman Avenue, and finally into Airport Boulevard. As a result, the mileage and carbon emissions experienced previously will be reduced or eliminated. A target for this project is to modify the current station so that it functions similarly to the Millbrae Caltrain/BART connection, which routes commuters directly into the San Francisco Airport.

This development will benefit all the neighboring communities. The new San Jose Earthquakes soccer stadium is currently under construction at the corner of Coleman Avenue and Newhall Drive, across from the San Jose Airport. The proposed structure will run and end adjacent to this new stadium, making it easily accessible for not only the neighboring community in the South Bay, but up to the North Bay as well. Since soccer games are major events, the potential for commercial business is soaring. Before and after these major events, attendees will be able to eat, shop, and interact comfortably through the use of the proposed facility. This soccer stadium also has the potential to increase Caltrain business in the ways that the San Jose Sharks' HP Pavilion and San Francisco Giants' AT&T Park do.

2.2 DESIGN SCOPE

Given the large scope of the project, only certain structural design processes were able to be accomplished. Because of intricate details, the steel bridge, structural connections, and foundations were excluded from the design scope. Included within the project are the structural design of the three towers and the three commercial structures. However, foundations and bridge interactions were taken into consideration because of their significant impact on design

3.0 APPLICABLE DESIGN CRITERIA AND STANDARDS

Because the San Jose and Santa Clara boundary line runs through the parcel of interest, contacting both jurisdictions was necessary in order to determine if any planning or building issues would apply. The Santa Clara jurisdiction stated that a structure could not be located on the physical boundary line. As a result, it was required that the structural layout avoid encroaching the boundary. Also, because of the presence of the city boundary line, both cities of San Jose and Santa Clara were contacted in order to determine the design wind speed. Santa Clara provided a design wind speed of 75 mph while San Jose claimed a wind speed of 85 mph. For conservative purposes, an 85 mph wind speed was used for lateral design.

Given the project's close proximity to the San Jose Airport, height restrictions for airport clearance needed to be taken into consideration. The height limit around the Coleman Ave. area is 210 ft. Finally, vertical rail clearances needed to be taken into account. The 2011 Highway Design Manual states a minimum rail clearance of 26 ft. for an electric rail system. Although only the Caltrain and Amtrak train systems run through the area, BART and California High Speed Rail, electric systems, are expected to run through in the near future.

4.0 DESCRIPTION OF DESIGNED FACILITY

The three towers serve as entry and exit points for those using the bridging structure. Towers one and three are identical for ease of construction. Towers one and three have a 45 ft. by 45 ft. footprint and a height of 50 ft. The bridge will be connecting the towers at an elevation of 30 ft, which will serve as a clearance for the Caltrain and other future systems. A 5 ft. mechanical level is included in the elevator framing to allow for a pulley system. Tower two is different in design because it needed to be incorporated with the newly remodeled Santa Clara Caltrain; as a result, the footprint could only be 15 ft. by 45 ft. Figure 5 details the south elevation of the tower structures.

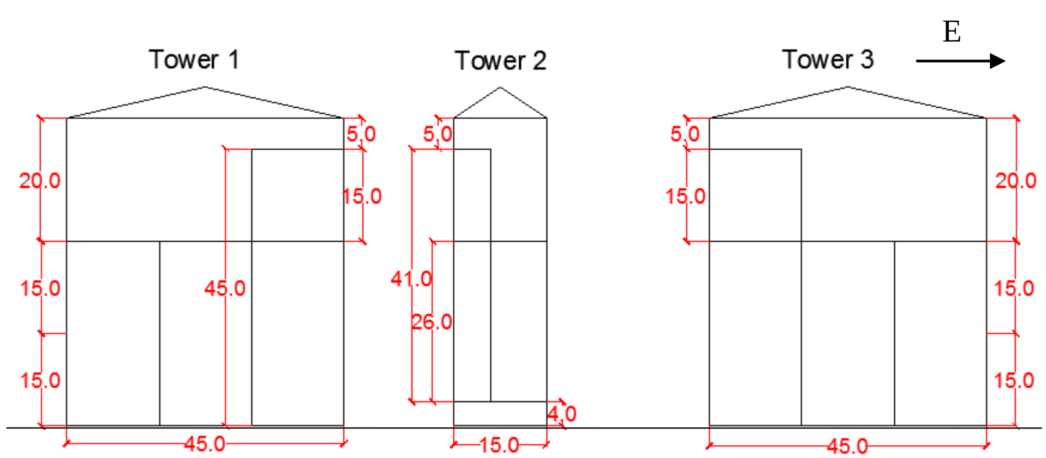

Figure 5: Tower structures - south elevation

A model of the south elevation of towers one and two can be seen in Figure 6 below. As mentioned previously, tower three is identical to tower one.

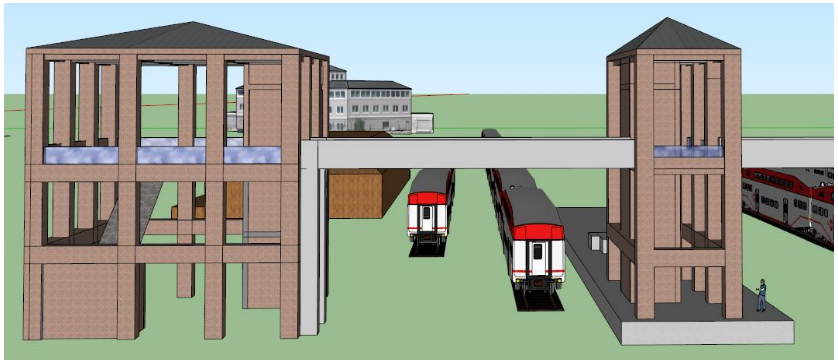

Figure 6: Model view of tower structures – south elevation

The three commercial structures were designed to be in close proximity to one another. All three commercial structures have story heights of 17 ft. and commercial structures A and B have 12 ft. walkways. Commercial structure A has a 34 ft. height at the bottom of the roof, an 82 ft. width, and a 27 ft. elevator. A 40 ft. walkway connects commercial structures A and B. Commercial structure B has a 124 ft. width. Structure C has only one story for a total height of 17 ft. and a width of 80 ft. Figure 7 details the west elevation of the commercial structures. More detailed AutoCAD drawings can be found in Appendix I.

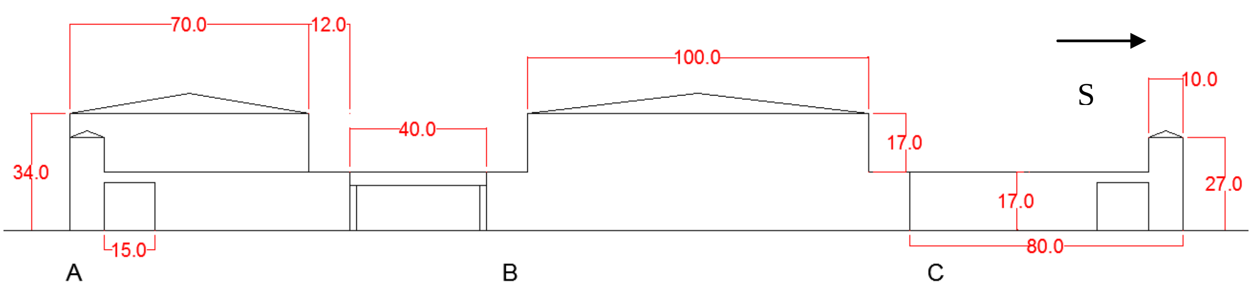

Figure 7: Commercial structures - west elevation

A model view of the west elevation of the commercial structures can be seen in Figure 8 below.

Figure 8: Model view of commercial structures – west elevation

5.0 DESIGN LOADS

All design loads were taken out of the 2010 Minimum Design Loads for Buildings and Other Structure (ASCE 1-10).

5.1 DEAD LOADS

The dead loads for the towers and the commercial structures were determined by contacting the jurisdiction to see what typical loads were for related structures in the area. Tables C3-1 and C3-2 from ASCE 7-10 were also referred to for approximate weights of construction materials such as Spanish clay tile, metal decking, and concrete decking. The roof dead loads were determined to be 20 psf while the floor dead loads were 80 psf. For tower two, loads due to the bearing connection of the bridge also had to be taken into consideration. A site visit to the Mountain View Caltrain station was conducted in order to record approximate sizes of the pedestrian bridge that crosses the rail system. Using these dimensions, a bridge model was created through Visual Analysis in order to determine an approximate bridge dead weight. Pioneer Bridges was also contacted. They provided typical weights of bridges used for similar applications. Included in the weight of the bridge was a 5 inch concrete deck, which was an estimate also provided by Pioneer Bridges. Weights due to the elevator mechanical equipment and escalator also had to be taken into consideration. These weights were provided by American Elevator Company.

5.2 LIVE LOADS

Live Loads were determined using Table 4-1 from ASCE 7-10 Minimum Design Loads. For a commercial retail space, the minimum design roof live load is 25 psf. The minimum design floor load is 100 psf. For conservative purposes, live load reduction was not considered.

5.3 WIND LOADS

Although Figure 26.5-1B from ASCE 7-10 states that the minimum design wind speeds for Occupancy Category II in California is 115 mph, it was deemed necessary to contact the jurisdictions for the implemented wind speed. After contacting the Santa Clara and San Jose jurisdictions, it was determined that the exposure category for the project was exposure B. Wind pressures for the towers and commercial structures were found using ASCE 7-10 Minimum Design Loads. The towers and the commercial structures were treated as rigid structures because their fundamental frequency was greater than one. Because the towers are not expected to have 80% total closure on all sides nor 80% total opening on all sides, the towers were treated as partially enclosed structures. For the retail development, the structures were treated as fully enclosed. For conservative estimations, the maximum net pressure experienced for each structure was uniformly distributed across the applicable face. However, the jurisdictions did notify that wind design does not govern over seismic design. As a result, for lateral design, it was assumed that the worst case wind situation would not occur at the same time as the worst case seismic event. Wind load calculations and results can be found in Appendix D.

5.3 SEISMIC LOADS

Earthquake analysis was performed using ASCE 7-10 Minimum Design Loads. Using the United States Geological Survey (USGS), the (Ss) and (S_1) values were determined for our structures. After obtaining the boring log from Santa Clara Bannan Engineering building, the site class was determined to be site class D. The importance factor for a category II building was one and the response modification factor (R), over strength factor (Ω), and deflection amplification factors (C_d) were all determined based on a special moment frame design using Table 12.2-1 in ASCE 7-10. These values were used in conjunction with the ASCE 7-10 manual to determine fundamental periods of our structures (T). The (Cs) values were determined using the fundamental period. After determining the total seismic base shear of the structures, forces at each level were determined based on the level load. Seismic loads for individual structures can be found in Appendix E.

6.0 GENERAL GRAVITY DESIGN

All simple beams were analyzed with uniformly distributed gravity loads. The stiffness requirement was determined using the maximum allowable deflection, $\Delta_{LIM,}$ due to the live load. The deflection limit was taken as L/360 for the floors, and L/240 for the roof ; where L is the length of the beam in inches. All members were designed using W-sections and were checked to comply with the deflection limits, lateral torsional buckling, shear, and yielding strength requirements. Girders were analyzed using point loads due to the beams framing into the members. They too were checked to comply with the strength requirements previously mentioned. To maintain consistent column sizes throughout the structures, gravity column sizes were governed by the column sizes implemented in the moment resisting frame. Beam and girder calculations can be found in Appendix B.

6.1 TOWER GRAVITY DESIGN

The tower gravity design was governed by the fact that columns could not run through the middle of the structure because they would obstruct the desired open spacing. As a result, in towers one and three, a 45 ft. girder runs across the center of the structures, which presented uneven loadings across the span of the member. Figure 9 presents a cut-out of towers one and three which indicates the location of the 45 ft. girder. Some beam and girder sizes included in the second story of towers one and two are W18x76, W18x50, and for the 45 ft. member, W24x192.

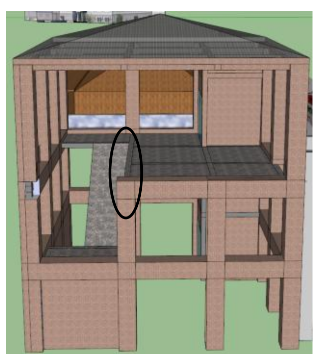

Figure 9: Second story plan view– towers one and three

Unlike most of the beams implemented in towers one and three, the beams supporting the pedestrian bridge in tower two saw a significant increase in load. The reaction seen at each support was 35 kips, which can be seen in Figure 10. This resulted in an increase of 2.33 klf along the length of the 15 ft. supporting member.

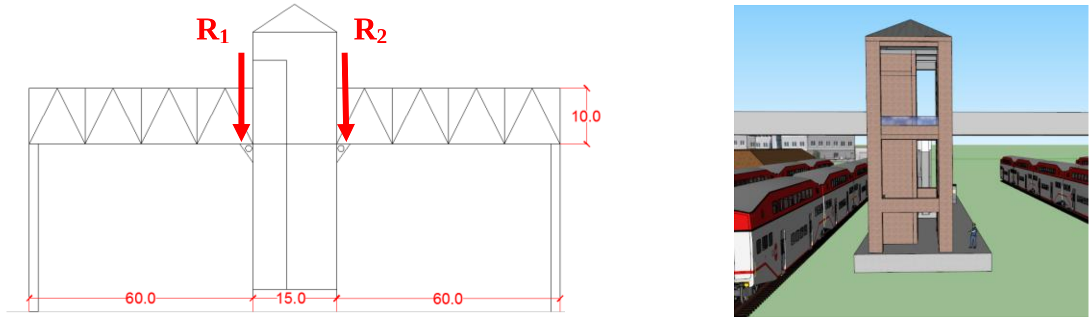

Figure 10: Tower two bridge supports

A feature all three towers share in common is the implementation of a pulley supported elevator system. This required a 5 ft. clearance between the top of the elevator shaft and the tower ceiling. A slab, supporting the pulley mechanical equipment, rests on the members and distributes half its load to the center supporting beam.

6.2 <u>COMMERCIAL STRUCTURES GRAVITY DESIGN</u>

Because prefabricated trusses allow for more serviceable construction, the initial gravity design of the commercial structures included truss joists on the second story and the roof. However, the loads seen on the second story were too large for the prefabricated truss sizes that were being considered. As a result, trusses were implemented only on the roof due to the relatively small loads experienced. Trusses allow for easy installation.

For truss specifications and strength capacities, Vulcraft Group was consulted. Using their product catalog, a truss model was selected. The trusses were spaced at 5 ft. with lengths that varied from 25 ft. to 35 ft. The required truss strength was an LRFD load of 310 plf, resulting in the implementation of the 14K3 and 18K7 joists from the K-series. Each had strength capacities of 339 plf and 367 plf and depths of 14 in. and 18 in. respectively. This allows for mechanical and electrical run-through as well as fast installation. The K-series catalog with LRFD strength capacities can be found in Appendix G. A typical Vulcraft joist detail can be seen in Figure 11 below and a plan view of the joist system can be seen in the detailed drawings of commercial structures A and B in Appendix I.

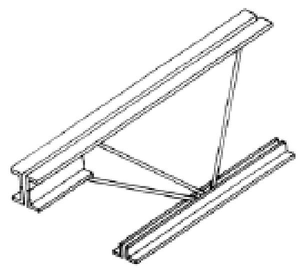

Figure 11: Vulcraft K-series joist

No unusual loading patterns were experienced throughout the rest of the commercial structures. The smallest and largest sizes implemented in the commercial buildings were W12x26 and W18x130 respectively.

7.0 GENERAL LATERAL DESIGN

Because steel ordinary moment frames are only permitted for single story structures and intermediate moment frames for heights 35 ft. or less in site class D, steel special moment frames were implemented in all structures. Since the project is located in a high seismic region, only seismic forces were used for lateral design.

Preliminary sizes for the moment frames were done using force and stiffness equations. The maximum inelastic interstory drift allowed was 2.5% of the story height. Given this maximum inelastic response, the maximum allowable elastic drift was determined using equation 12.12-1 from ASCE 7-10. Using the maximum allowable interstory elastic drift, a preliminary sized member was chosen using a required moment of inertia based on the stifness of a given frame. The moment of intertia was isolated from the equation. The force acted on the frame was found by dividing the earthquake shear forces by the number of resisting frames in the plane of the force. Using the discovered moment of inertia, a preliminary sized member was chosen out of the AISC Steel Construction Manual and modeled in Visual Analysis. After introducing lateral and gravity loads into the model frame, column and beam sizes were sized up until adequate strength, deflection, and drift limits were satisfied

Because moment frames were implemented for the lateral resistance, significant drift is expected to occur. Because various structures interact with each other, such as commercial building B and C, the use of seismic joints was vital to allow for sufficient drift without structural damage. Using the maximum allowable inelastic drift, 2.5% of the story height, seismic spacing and joints were determined through the use of equation 12.12-2 from ASCE 7-10. Equation 12.12-2 takes the sum of the squares of the maximum inelastic interstory drifts. This spacing was used to size the expansion joints, which were found by consulting EMSEAL Joint Systems. The largest required size was 7.5”. Lateral calculations, column sizing, and maximum inelastic drifts can be found in Appendix E.

7.1 TOWER LATERAL DESIGN

Given the towers’ limited space and open interior area, certain members experienced strong and weak axis bending. In designing the lateral system for the towers, a challenge was encountered for tower two. The design for tower two was restricted by the fact that the newly remodeled Santa Clara Caltrain was to be incorporated. The incorporation did not allow for the bridge to have its own foundation. Two bridge sections had to rest at level two of tower two and introduced some dead load to the structure. The total dead load for the bridge was estimated at 70 kips. The 70 kip dead load of the bridging structure introduced two 35 kip reactions in the south face of tower two.

A bearing load connection was used to transfer very minimal lateral forces. However, because steel rests on steel, the surface area will allow for some lateral load transfer. A steel on steel static friction coefficient was used for the lateral load transfer. AISC 2011 suggests two different values for static friction depending on the finish of the connection. For conservative purposes, class B surfaces were used. Class B has a static friction of 0.5 (AISC 16.1-126). The static friction was assumed to allow some lateral load transfer from the bridging structure. In a seismic event, the reactions of 35 kips transferred a lateral load due to the static friction of 17.5 kips. Figure 12 illustrates the lateral force created by the simple bearing connection of the pedestrian bridge onto tower two. For conservative design, worst case scenario was assumed, which saw the lateral forces due to the pedestrian bridge act in the same direction, creating twice the resultant.

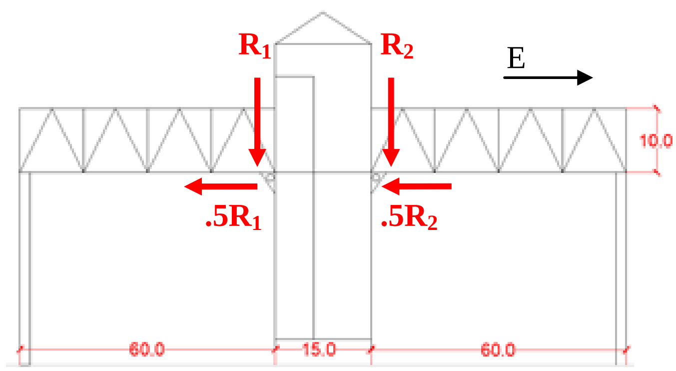

Figure 12: Tower two lateral reactions – south elevation

The two frames supporting the bridge were designed to take the entire lateral load due to the bridge. By separating the two south frames from the two north frames, seen in Figure 13, torsional effects due to the bridge were minimalized.

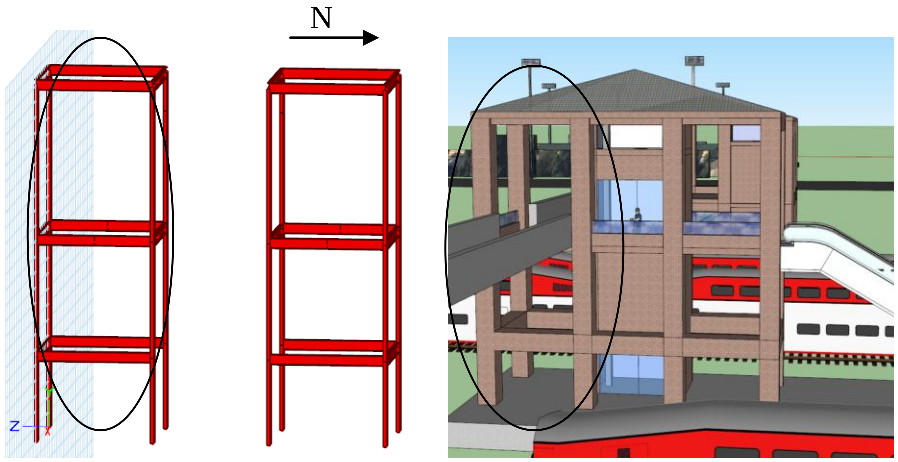

Figure 13: Tower two lateral resistance – east elevation

Since the pedestrian bridge is supported on its own foundation when it reaches towers one and three, seismic joints were necessary to allow for drift between the two towers and the pedestrian bridge. A 6.4 in. seismic joint is required in the locations illustrated in Figure 14.

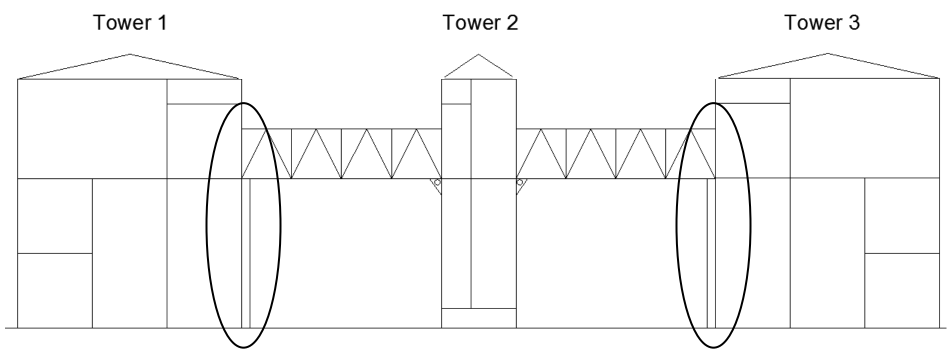

Figure 14: Location of tower seismic joints – south elevation

7.2 COMMERCIAL STRUCTURES LATERAL DESIGN

Unlike the towers, the lateral system for the commercial structures was separated into north-south resistances and east-west resistances, allowing the columns to be oriented along their strong axis and preventing weak axis bending. Figure 15 illustrates the separation of the lateral resistance for commercial structure A.

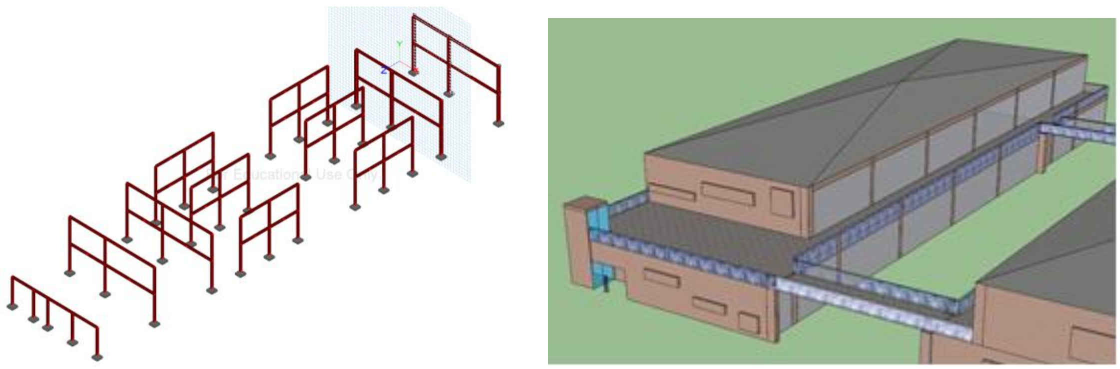

Figure 15: Commercial structure A lateral resistance

Commercial buildings A and C have 27 ft. elevators that span 10 ft. above the first floor walkway. This was necessary to access their respective second floor walkways, which can be seen in Figure 16. As a result, the upper 10 ft. of the elevators are vulnerable to lateral forces that cannot be withstood by the large commercial structure itself. In order to provide sufficient lateral strength, the elevators were designed as separate structures in order to implement moment frames and allow for some lateral drift.

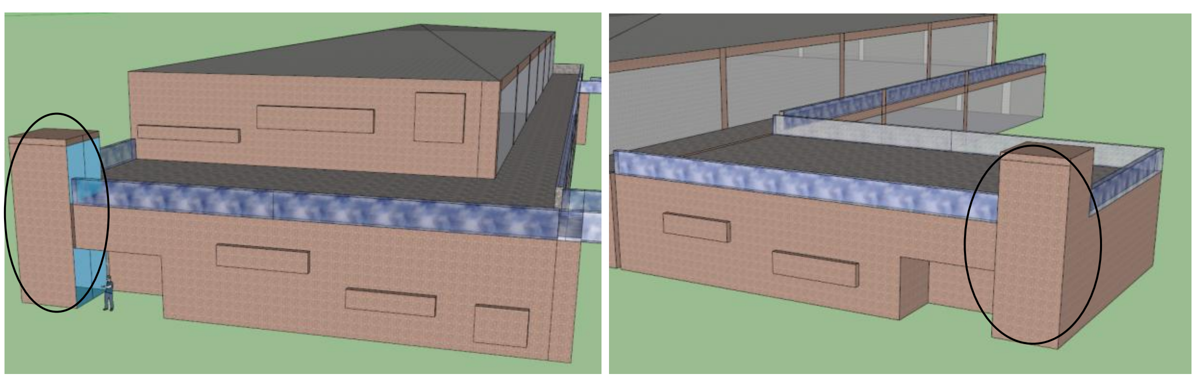

Figure 16: Commercial elevator A (left) and commercial elevator C (right)

Figure 17 demonstrates where seismic joint are necessary to accommodate for lateral drift. The pedestrian walkway connecting commercial buildings A and B was treated as a separate structure. 7.1 in. seismic joints are required at these locations.

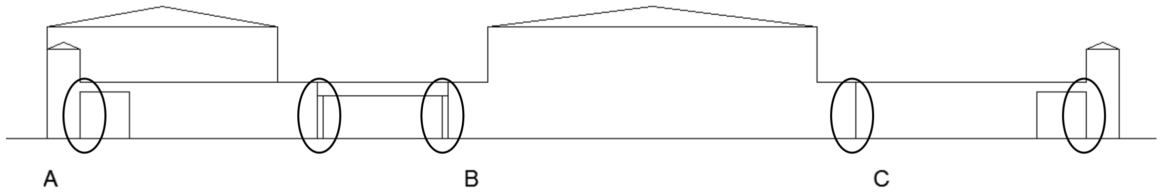

Figure 17: Location of commercial structure seismic joints – west elevation

8.0 SOIL CONDITIONS AND FOUNDATION

Because a soils report for the project site was not available, other means of obtaining soil records needed to be taken. Given Santa Clara University's close proximity to the project site, the soils report for the Bannan Engineering building was used to approximate soil conditions. Assuming the same site conditions applied, foundation recommendations were based off the boring logs for the engineering building. The presence of expansive and liquefiable soils presented the possibility of two site classes, D and E. Site class D was determined by the fact that the compressive strength for the clay was far greater than the 500 psf threshold and the clay layer was less than 10 ft. in thickness. The moisture content, however, was 40 %, which would trigger site class E if all the previously mentioned soil conditions also fell under site class E. ASCE 7-10 was referred to determine site class.

The boring logs show that there is a clay layer at 6 ft. and a sand layer at 20 ft. Naturally, this presents settlement and liquefactions issues respectively. The logs of boring can be found in Appendix F. On the basis of which, it was recommended that the engineering building be built on shallow spread footing placed in engineered fill. Given the commercial building's open area, the same recommendations were applicable. However, given the limited space of the towers within the railway lines, shallow footings were not feasible. Accordingly, pile foundations were recommended.

Piles will not only reduce periodic vibrations from the rails and increase stability, but they also allow for ease of constructability within the confines of the tracks. By the use of a mechanical auger, piles can be drilled into the soil without making significant disturbances to the soil or the rails.

9.0 COST ANALYSIS

A material cost estimate was performed for the proposed development. The total estimated material cost is approximately \$3.2 million. Table 1 shows the cost of individual materials and a complete breakdown of specific building material costs can be found in Appendix H.

Total Cost	
Slab on metal deck	\$166,000.00
Slab on grade	\$162,000.00
Steel	\$2,900,000.00
Total	\$3,228,000.00

Table 1: Total Material Cost

In order to be sustainable and promote green building, quotes from local providers were taken in order to decrease gas emissions due to transportation. For concrete, Central Concrete provided a quote for normal weight concrete of \$150 per cubic yard of concrete. For steel, Schuff Steel provided gave a quote of \$1.75 per pound of steel. The quantities for the slab on metal deck were increased by 10% because of the fact that deflection of the metal deck is expected to occur. In turn, this increases the quantity of concrete needed to meet required floor elevation. For complete takeoff, see Appendix H.

10.0 RISK ASSESSMENT

There were many assumptions that took place during the design of the three tower structures and three commercial buildings. Being aware of uncertainties, worse case scenarios were assumed for many structural cases. An example, as mentioned earlier, would be the beam design or wind design. For beam design, the heaviest loaded beam governed design. For wind, the maximum wind pressure was uniformly distributed. The reason for doing so is to compensate for the lack of professional knowledge and experience in certain situations.

11.0 CONCLUSION

This is a project that would be very beneficial to not only the Santa Clara community, but the Bay Area as well. The proposed development would encourage the use of public transportation, and would serve as an example in developing sustainable mass transit. Since the proposed project also includes a commercial development, the facilities would provide a great business potential. The commercial structures would also serve the entertainment needs of the Santa Clara community by housing restaurants, movie theatres, and restaurants.

In exploring different solutions and alternatives for the site, much knowledge was acquired in the process of developing a city project. Working with different jurisdictions and organizations made it apparent that much planning and communication is needed for a successful project that benefits everyone. The designing and analysis of different loading scenarios was also a challenge but allowed for much industry exposure. Navigating through codes and manuals such as the ASCE 7-10 were just small parts of the design process.

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APPENDIX A

DESIGN VALUES AND STANDARDS

Project:	Caltrain Bridging Structure and Commercial Buildings Coleman Ave., San Jose and Santa Clara, CA
Designers:	Anthony Navarrete Guadalupe Gonzalez
Project Number:	CENG 193 – Spring 2013
Jurisdictions:	State of California, Cities of Santa Clara and San Jose
Codes, Specifications, and Standards:	Department of Transportation 2011 Highway Design Manual, 2011 ASCE 7-10 Minimum Design Loads, 2010 California Building Code, AISC 14th edition
Software Used:	2011 Microsoft Excel, Visual Analysis 6.0

Basic Loads:

1. Gravity Loads:

Roof Live	25 PSF
Roof Dead	20 PSF
-Metal Deck	4 PSF
-Spanish Clay Tile	12 PSF
Floor Live	100 PSF
Floor Dead	
-Metal Deck	4 PSF
-Concrete Deck	45 PSF
-Miscellaneous	30 PSF
-Elevator	7 kips
-Escalator	6 kips
-Bridge	32 kips

2. Deflection Limits:

Roof Deflection	L/240
Floor Deflection	L/360

3. Lateral Loads:

Wind Criteria:	
Wind Speed	85 MPH
Wind Exposure	B
Category	II
Seismic Criteria	
Zone	4
Site Class	D
Drift	0.025h
R	6
Ω	3
C_d	5.5

Soils: Per Bannan Engineering soils report.

Materials:

Structural Steel	
W-sections	A992
HSS-sections	
Vulcraft prefabricated trusses	
Decking	
Concrete Deck	

APPENDIX B

BEAM/GIRDER CALCULATIONS

TOWER 1/3 FIRST FLOOR

Beam Design GL Q,X,13-2,15-2

Design Requirements	
Max. Deflection (in)	0.75
Floor Live Load, w_L (psf)	100
Floor Dead Load, w_D (psf)	125
Beam Length (ft)	15
Tributary Area (ft)	15
Total Dead Load (klf)	2.25
Total Live Load (klf)	2.4
Total Load (Klf)	4.65
Required Moment, M_r (kip-ft)	130.7813
Required Shear, V_r (kips)	34.875
Shear with Self Weight	35.25

$I_{req'd}$ (in^4)	243.52
W18X50	800

Nominal Weight	Area, A	Depth, d	Web		Flange		Compact Section Criteria		Axis X-X				Axis Y-Y				r_{ts}	h_o	Torsional Properties	
			Thickness, t_w	$t_w/2$	Width, b_f	Thickness, t_f			I	S	r	Z	I	S	r	Z			J	C_w
(lb/ft)	(in^2)	(in)	(in)	(in)	(in)	(in)	$b_f/2t_f$	h/t_w	(in^4)	(in^3)	(in)	(in^3)	(in^4)	(in^3)	(in)	(in^3)	(in)	(in)	(in^4)	(in^6)
50	14.7	18	0.355	0.1775	7.5	0.57	6.57	45.2	800	88.9	7.38	101	40.1	10.7	16.6	50	1.98	17.4	1.24	3040

Flexure

Flange Slenderness		Web Slenderness	
λ_p	9.15	λ_p	90.55
λ_r	24.08	λ_r	137.27
b/t	6.57	h/t_w	45.20

Unbraced Length	
L_p	5.83
L_r	16.90
L_b	15.00

Yielding	
M_p (kip-in)	5050.00
Cb	1.14
M_p (kip-ft)	420.83

LTB		
	If $L_p < L_b < L_r$	If $L_b > L_r$
Mn (kip-ft)	326.16	3495.12
Values must be < M_p		

Shear

	h/t_w	45.2
Case 1:	$2.24\sqrt{E/F_y}$	53.95
Case 2:		
Case 3:		

If $h/t_w < 2.24\sqrt{E/F_y}$	
ϕ_v	1
C_v	1
$\phi_v V_n$	191.7

If $h/t_w > 2.24\sqrt{E/Fy}$	
$1.10\sqrt{k_v E/Fy}$	59.24
$1.37\sqrt{k_v E/Fy}$	73.78

If $h/t_w < 1.10\sqrt{k_v E/Fy}$	
C_v	1
ϕ_v	0.9
$\phi_v V_n$	172.53

If $1.10\sqrt{k_v E/F_y} < h/t_w < 1.37\sqrt{k_v E/F_y}$	
C_v	1.31
ϕ_v	0.90
$\phi_v V_n$	226.11

If $h/t_w > 1.37\sqrt{k_v E/Fy}$	
C_v	2.14
ϕ_v	0.90
$\phi_v V_n$	369.80

$\phi_v V_n$	191.70
$\phi_b M_n$	293.54

Check for Self-Weight

M_r (kip-ft)	132.19	<	293.54
V_r (kips)	35.25	<	191.70

TOWER 1/3 SECOND FLOOR

Beam Design GL 14,15

Design Requirements	
Max. Deflection (in)	0.75
Floor Live Load, w_L (psf)	100
Floor Dead Load, w_D (psf)	125
Beam Length (ft)	15
Tributary Area (ft)	15
Total Dead Load (klf)	2.25
Total Live Load (klf)	2.4
Total Load (Klf)	4.65
Required Moment, M_r (kip-ft)	130.7813
Required Shear, V_r (kips)	34.875
Shear with Self Weight	35.25

$I_{req'd}$ (in^4)	243.52
W18X50	800

Nominal Weight	Area, A	Depth, d	Web		Flange		Compact Section Criteria		Axis X-X				Axis Y-Y				r_{ts}	h_o	Torsional Properties	
			Thickness, t_w	$t_w/2$	Width, b_f	Thickness, t_f			I	S	r	Z	I	S	r	Z			J	C_w
(lb/ft)	(in^2)	(in)	(in)	(in)	(in)	(in)	$b_f/2t_f$	h/t_w	(in^4)	(in^3)	(in)	(in^3)	(in^4)	(in^3)	(in)	(in^3)	(in)	(in)	(in^4)	(in^6)
50	14.7	18	0.355	0.1775	7.5	0.57	6.57	45.2	800	88.9	7.38	101	40.1	10.7	16.6	50	1.98	17.4	1.24	3040

Flexure

Flange Slenderness		Web Slenderness	
λ_p	9.15	λ_p	90.55
λ_r	24.08	λ_r	137.27
b/t	6.57	h/t_w	45.20

Unbraced Length	
L_p	5.83
L_r	16.90
L_b	15.00

Yielding	
M_p (kip-in)	5050.00
Cb	1.14
M_p (kip-ft)	420.83

LTB		
	If $L_p < L_b < L_r$	If $L_b > L_r$
Mn (kip-ft)	326.16	3495.12
Values must be $< M_p$		

Shear

	h/t_w	45.2
Case 1:	$2.24\|(E/F_y)$	53.95
Case 2:		
Case 3:		

If $h/t_w < 2.24\|(E/F_y)$	
ϕ_v	1
C_v	1
$\phi_v V_n$	191.7

If $h/t_w > 2.24\|(E/Fy)$	
$1.10\|k_v E/Fy$	59.24
$1.37\|k_v E/Fy$	73.78

If $h/t_w < 1.10\|(k_v E/Fy)$	
C_v	1
ϕ_v	0.9
$\phi_v V_n$	172.53

If $1.10\|k_v E/F_y < h/t_w < 1.37\|k_v E/F_y$	
C_v	1.31
ϕ_v	0.90
$\phi_v V_n$	226.11

If $h/t_w > 1.37\|(k_v E/Fy)$	
C_v	2.14
ϕ_v	0.90
$\phi_v V_n$	369.80

$\phi_v V_n$	191.70
$\phi_b M_n$	293.54

Check for Self-Weight

M_r (kip-ft)	132.19	<	293.54
V_r (kips)	35.25	<	191.70

Beam Design GL 13.1

Design Requirements	
Max. Deflection (in)	0.75
Floor Live Load, w_L (psf)	100
Floor Dead Load, w_D (psf)	125
Beam Length (ft)	15
Tributary Area (ft)	27.5
Total Dead Load (klf)	4.125
Total Live Load (klf)	4.4
Total Load (Klf)	8.525
Required Moment, M_r (kip-ft)	239.7656
Required Shear, V_r (kips)	63.9375
Shear with Self Weight	64.3125

$I_{req'd}$ (in^4)	446.46
W18X50	800

Nominal Weight	Area, A	Depth, d	Web		Flange		Compact Section Criteria		Axis X-X				Axis Y-Y				r_{ts}	h_o	Torsional Properties	
			Thickness, t_w	$t_w/2$	Width, b_f	Thickness, t_f			I	S	r	Z	I	S	r	Z			J	C_w
(lb/ft)	(in^2)	(in)	(in)	(in)	(in)	(in)	$b_f/2t_f$	h/t_w	(in^4)	(in^3)	(in)	(in^3)	(in^4)	(in^3)	(in)	(in^3)	(in)	(in)	(in^4)	(in^6)
50	14.7	18	0.355	0.1775	7.5	0.57	6.57	45.2	800	88.9	7.38	101	40.1	10.7	16.6	50	1.98	17.4	1.24	3040

Flexure

Flange Slenderness		Web Slenderness	
λ_p	9.15	λ_p	90.55
λ_r	24.08	λ_r	137.27
b/t	6.57	h/t_w	45.20

Unbraced Length	
L_p	5.83
L_r	16.90
L_b	15.00

Yielding	
M_p (kip-in)	5050.00
Cb	1.14
M_p (kip-ft)	420.83

LTB		
	If $L_p < L_b < L_r$	If $L_b > L_r$
Mn (kip-ft)	326.16	3495.12
Values must be < M_p		

Shear

	h/t_w	45.2
Case 1:	$2.24\sqrt{(E/F_y)}$	53.95
Case 2:		
Case 3:		

If $h/t_w < 2.24\sqrt{(E/F_y)}$	
ϕ_v	1
C_v	1
$\phi_v V_n$	191.7

If $h/t_w > 2.24\sqrt{(E/Fy)}$	
$1.10\sqrt{k_v E/Fy}$	59.24
$1.37\sqrt{k_v E/Fy}$	73.78

If $h/t_w < 1.10\sqrt{(k_v E/Fy)}$	
C_v	1
ϕ_v	0.9
$\phi_v V_n$	172.53

If $1.10\sqrt{k_v E/F_y} < h/t_w < 1.37\sqrt{k_v E/F_y}$	
C_v	1.31
ϕ_v	0.90
$\phi_v V_n$	226.11

If $h/t_w > 1.37\sqrt{(k_v E/Fy)}$	
C_v	2.14
ϕ_v	0.90
$\phi_v V_n$	369.80

$\phi_v V_n$	191.70
$\phi_b M_n$	293.54

Check for Self-Weight

M_r (kip-ft)	241.17	<	293.54
V_r (kips)	64.31	<	191.70

Girder Design GL X,W,R,Q

Design Requirements	
Max. Deflection (in)	2.25
Beam Length (ft)	45
Required Moment, Mr (kip-ft)	935.064
Point Load P1 (kips)	62.4
Point Load P2 (kips)	28.8

$I_{req'd} (in^4)$	5421.11
W24X192	6260

Nominal Weight	Area, A	Depth, d	Web		Flange		Compact Section Criteria		Axis X-X				Axis Y-Y				r_{ts}	h_o	Torsional Properties	
			Thickness, t_w	$t_w/2$	Width, b_f	Thickness, t_f			I	S	r	Z	I	S	r	Z			J	C_w
(lb/ft)	(in^2)	(in)	(in)	(in)	(in)	(in)	$b_f/2t_f$	h/t_w	(in^4)	(in^3)	(in)	(in^3)	(in^4)	(in^3)	(in)	(in^3)	(in)	(in)	(in^4)	(in^6)
192	56.5	25.5	0.81	0.405	13	1.46	4.43	26.6	6260	491	10.5	559	530	81.8	126	192	3.6	24	30.8	76300

Flexure

Flange Slenderness		Web Slenderness	
λ_p	9.15	λ_p	90.55
λ_r	24.08	λ_r	137.27
b/t	4.43	h/t_w	26.60

Unbraced Length	
L_p	10.80
L_r	39.70
L_b	15.00

Yielding	
M_p (kip-in)	27950.00
Cb	1.67
M_p (kip-ft)	2329.17

LTB		
	If $L_p < L_b < L_r$	If Lb > Lr
Mn (kip-ft)	3671.99	93781.08
Values must be < M_p		

$\phi_b M_n$	2096.25

Shear

	h/t_w	26.60
Case 1:	$2.24\sqrt{(E/F_y)}$	53.95
Case 2:		
Case 3:		

If $h/t_w < 2.24\sqrt{(E/F_y)}$	
ϕ_v	1.00
C_v	1.00
$\phi_v V_n$	619.65

If $h/t_w > 2.24\sqrt{(E/Fy)}$	
$1.10\sqrt{k_v E/Fy}$	59.24
$1.37\sqrt{k_v E/Fy}$	73.78

If $h/t_w < 1.10\sqrt{(k_v E/Fy)}$	
C_v	1
ϕ_v	0.9
$\phi_v V_n$	557.69

If $1.10\sqrt{k_v E/F_y} < h/t_w < 1.37\sqrt{k_v E/F_y}$	
C_v	2.23
ϕ_v	0.90
$\phi_v V_n$	1241.94

If $h/t_w > 1.37\sqrt{(k_v E/Fy)}$	
C_v	6.19
ϕ_v	0.90
$\phi_v V_n$	3451.44

$\phi_v V_n$	619.65

Check for Self-Weight

M_r (kip-ft)	983.66	<	2096.25
V_r (kips)	31.20	<	619.65

Girder Design Perimeter Moment Frame

Design Requirements	
Max. Deflection (in)	0.75
Floor Live Load, w_L (psf)	100
Floor Dead Load, w_D (psf)	125
Beam Length (ft)	15
Tributary Area (ft)	7.5
Total Dead Load (klf)	1.125
Total Live Load (klf)	1.2
Total Load (Klf)	2.325
Required Moment, M_r (kip-ft)	65.39063
Required Shear, V_r (kips)	17.4375
Shear with Self Weight	18.0075

$I_{req'd}$ (in^4)	121.76
W18X76	1330

Nominal Weight	Area, A	Depth, d	Web		Flange		Compact Section Criteria		Axis X-X				Axis Y-Y				r_{ts}	h_o	Torsional Properties	
			Thickness, t_w	$t_w/2$	Width, b_f	Thickness, t_f			I	S	r	Z	I	S	r	Z			J	C_w
(lb/ft)	(in^2)	(in)	(in)	(in)	(in)	(in)	$b_f/2t_f$	h/t_w	(in^4)	(in^3)	(in)	(in^3)	(in^4)	(in^3)	(in)	(in^3)	(in)	(in)	(in^4)	(in^6)
76	22.3	18.2	0.425	0.2125	11	0.68	8.11	37.8	1330	146	7.73	163	152	27.6	42.2	76	3.02	17.5	2.83	11700

Flexure

Flange Slenderness		Web Slenderness	
λ_p	9.15	λ_p	90.55
λ_r	24.08	λ_r	137.27
b/t	8.11	h/t_w	37.80

Unbraced Length	
L_p	5.83
L_r	16.90
L_b	15.00

Yielding	
M_p (kip-in)	8150.00
Cb	1.14
M_p (kip-ft)	679.17

LTB		
	If $L_p < L_b < L_r$	If $L_b > L_r$
Mn (kip-ft)	533.31	13353.54
Values must be < M_p		

Shear

	h/t_w	37.8
Case 1:	$2.24\|(E/F_y)$	53.95
Case 2:		
Case 3:		

If $h/t_w < 2.24\|(E/F_y)$	
ϕ_v	1
C_v	1
$\phi_v V_n$	232.05

If $h/t_w > 2.24\|(E/Fy)$	
$1.10\|k_v E/Fy$	59.24
$1.37\|k_v E/Fy$	73.78

If $h/t_w < 1.10\|(k_v E/Fy)$	
C_v	1
ϕ_v	0.9
$\phi_v V_n$	208.85

If $1.10\|k_v E/F_y < h/t_w < 1.37\|k_v E/F_y$	
C_v	1.57
ϕ_v	0.90
$\phi_v V_n$	327.28

If $h/t_w > 1.37\|(k_v E/Fy)$	
C_v	3.06
ϕ_v	0.90
$\phi_v V_n$	640.05

$\phi_v V_n$	232.05
$\phi_b M_n$	479.98

Check for Self-Weight

M_r (kip-ft)	67.53	<	479.98
V_r (kips)	18.01	<	232.05

TOWER 1/3 MECHANICAL LEVEL

Beam Design GL 13-05, 15-1

Design Requirements	
Max. Deflection (in)	0.75
Floor Live Load, w_L (psf)	300
Floor Dead Load, w_D (psf)	120
Beam Length (ft)	15
Tributary Area (ft)	15
Total Dead Load (klf)	2.16
Total Live Load (klf)	7.2
Total Load (Klf)	9.36
Required Moment, M_r (kip-ft)	263.25
Required Shear, V_r (kips)	70.2
Shear with Self Weight	70.6875

$I_{req'd}$ (in^4)	490.19
W12X65	533

Nominal Weight	Area, A	Depth, d	Web		Flange		Compact Section Criteria		Axis X-X				Axis Y-Y				r_{ts}	h_o	Torsional Properties	
			Thickness, t_w	$t_w/2$	Width, b_f	Thickness, t_f			I	S	r	Z	I	S	r	Z			J	C_w
(lb/ft)	(in^2)	(in)	(in)	(in)	(in)	(in)	$b_f/2t_f$	h/t_w	(in^4)	(in^3)	(in)	(in^3)	(in^4)	(in^3)	(in)	(in^3)	(in)	(in)	(in^4)	(in^6)
65	19.1	12.1	0.39	0.195	12	0.605	9.92	24.9	533	87.9	5.28	96.8	174	29.1	44.1	65	3.38	11.5	2.18	5780

Flexure

Flange Slenderness		Web Slenderness	
λ_p	9.15	λ_p	90.55
λ_r	24.08	λ_r	137.27
b/t	9.92	h/t_w	24.90

Unbraced Length	
L_p	11.90
L_r	35.10
L_b	15.00

Yielding	
M_p (kip-in)	4840.00
Cb	1.14
M_p (kip-ft)	403.33

LTB		
	If $L_p < L_b < L_r$	If $L_b > L_r$
Mn (kip-ft)	436.02	10070.52
Values must be < M_p		

Shear

	h/t_w	24.9
Case 1:	$2.24\sqrt{(E/F_y)}$	53.95
Case 2:		
Case 3:		

If $h/t_w < 2.24\sqrt{(E/F_y)}$	
ϕ_v	1
C_v	1
$\phi_v V_n$	141.57

If $h/t_w > 2.24\sqrt{(E/Fy)}$	
$1.10\sqrt{k_v E/Fy}$	59.24
$1.37\sqrt{k_v E/Fy}$	73.78

If $h/t_w < 1.10\sqrt{(k_v E/Fy)}$	
C_v	1
ϕ_v	0.9
$\phi_v V_n$	127.41

If $1.10\sqrt{k_v E/F_y} < h/t_w < 1.37\sqrt{k_v E/F_y}$	
C_v	2.38
ϕ_v	0.90
$\phi_v V_n$	303.11

If $h/t_w > 1.37\sqrt{(k_v E/Fy)}$	
C_v	7.06
ϕ_v	0.90
$\phi_v V_n$	899.89

$\phi_v V_n$	141.57
$\phi_b M_n$	392.42

Check for Self-Weight

M_r (kip-ft)	265.08	<	392.42
V_r (kips)	70.69	<	141.57

Girder Design GL R,S,V,W

Design Requirements	
Max. Deflection (in)	1
Beam Length (ft)	20
Required Moment, Mr (kip-ft)	355
Point Load P1 (kips)	71

$I_{req'd}$ (in^4)	705.10
W12X96	833

Nominal Weight	Area, A	Depth, d	Web: Thickness, t_w	Web: $t_w/2$	Flange: Width, b_f	Flange: Thickness, t_f	Compact Section Criteria: $b_f/2t_f$	Compact Section Criteria: h/t_w	Axis X-X: I	Axis X-X: S	Axis X-X: r	Axis X-X: Z	Axis Y-Y: I	Axis Y-Y: S	Axis Y-Y: r	Axis Y-Y: Z	r_{ts}	h_o	Torsional Properties: J	Torsional Properties: C_w
(lb/ft)	(in^2)	(in)	(in)	(in)	(in)	(in)			(in^4)	(in^3)	(in)	(in^3)	(in^4)	(in^3)	(in)	(in^3)	(in)	(in)	(in^4)	(in^6)
96	28.2	12.7	0.55	0.275	12.2	0.9	6.76	17.7	833	131	5.44	147	270	44.4	67.5	96	3.49	11.8	6.85	9410

Flexure

Flange Slenderness		Web Slenderness	
λ_p	9.15	λ_p	90.55
λ_r	24.08	λ_r	137.27
b/t	6.76	h/t_w	17.70

Unbraced Length	
L_p	2.66
L_r	23.80
L_b	7.50

Yielding	
M_p (kip-in)	7350.00
Cb	1.67
M_p (kip-ft)	612.50

LTB		
	If $L_p < L_b < L_r$	If Lb > Lr
Mn (kip-ft)	934.78	94061.28
Values must be < M_p		

$\phi_b M_n$	551.25

Shear

	h/t_w	17.70
Case 1:	$2.24\sqrt{(E/F_y)}$	53.95
Case 2:		
Case 3:		

If $h/t_w < 2.24\sqrt{(E/F_y)}$	
ϕ_v	1.00
C_v	1.00
$\phi_v V_n$	209.55

If $h/t_w > 2.24\sqrt{(E/Fy)}$	
$1.10\sqrt{k_v E/Fy}$	59.24
$1.37\sqrt{k_v E/Fy}$	73.78

If $h/t_w < 1.10\sqrt{(k_v E/Fy)}$	
C_v	1
ϕ_v	0.9
$\phi_v V_n$	188.60

If $1.10\sqrt{k_v E/F_y} < h/t_w < 1.37\sqrt{k_v E/F_y}$	
C_v	3.35
ϕ_v	0.90
$\phi_v V_n$	631.17

If $h/t_w > 1.37\sqrt{(k_v E/Fy)}$	
C_v	13.98
ϕ_v	0.90
$\phi_v V_n$	2636.08

$\phi_v V_n$	209.55

Check for Self-Weight

M_r (kip-ft)	359.80	<	551.25
V_r (kips)	35.50	<	209.55

TOWER 1/3 ROOF

Beam Design GL Q,R,X,W

Design Requirements	
Max. Deflection (in)	0.75
Floor Live Load, w_L (psf)	20
Floor Dead Load, w_D (psf)	25
Beam Length (ft)	15
Tributary Area (ft)	15
Total Dead Load (klf)	0.45
Total Live Load (klf)	0.48
Total Load (Klf)	0.93
Required Moment, M_r (kip-ft)	26.15625
Required Shear, V_r (kips)	6.975
Shear with Self Weight	7.2

$I_{req'd}$ (in^4)	48.70
W12X30	238

Nominal Weight	Area, A	Depth, d	Web		Flange		Compact Section Criteria		Axis X-X				Axis Y-Y				r_{ts}	h_o	Torsional Properties	
			Thickness, t_w	$t_w/2$	Width, b_f	Thickness, t_f			I	S	r	Z	I	S	r	Z			J	C_w
(lb/ft)	(in^2)	(in)	(in)	(in)	(in)	(in)	$b_f/2t_f$	h/t_w	(in^4)	(in^3)	(in)	(in^3)	(in^4)	(in^3)	(in)	(in^3)	(in)	(in)	(in^4)	(in^6)
30	8.79	12.3	0.26	0.13	6.52	0.44	7.41	41.8	238	38.6	5.21	43.1	20.3	6.24	9.56	30	1.77	11.9	0.457	720

Flexure

Flange Slenderness		Web Slenderness	
λ_p	9.15	λ_p	90.55
λ_r	24.08	λ_r	137.27
b/t	7.41	h/t_w	41.80

Unbraced Length	
L_p	5.37
L_r	15.60
L_b	15.00

Yielding	
M_p (kip-in)	2155.00
Cb	1.14
M_p (kip-ft)	179.58

LTB		
	If $L_p < L_b < L_r$	If $L_b > L_r$
Mn (kip-ft)	132.40	1212.73
Values must be < M_p		

Shear

	h/t_w	41.8
Case 1:	$2.24\sqrt{(E/F_y)}$	53.95
Case 2:		
Case 3:		

If $h/t_w < 2.24\sqrt{(E/F_y)}$	
ϕ_v	1
C_v	1
$\phi_v V_n$	95.94

If $h/t_w > 2.24\sqrt{(E/Fy)}$	
$1.10\sqrt{k_v E/Fy}$	59.24
$1.37\sqrt{k_v E/Fy}$	73.78

If $h/t_w < 1.10\sqrt{(k_v E/Fy)}$	
C_v	1
ϕ_v	0.9
$\phi_v V_n$	86.35

If $1.10\sqrt{k_v E/F_y} < h/t_w < 1.37\sqrt{k_v E/F_y}$	
C_v	1.42
ϕ_v	0.90
$\phi_v V_n$	122.37

If $h/t_w > 1.37\sqrt{(k_v E/Fy)}$	
C_v	2.51
ϕ_v	0.90
$\phi_v V_n$	216.40

$\phi_v V_n$	95.94
$\phi_b M_n$	119.16

Check for Self-Weight

M_r (kip-ft)	27.00	<	119.16
V_r (kips)	7.20	<	95.94

Girder Design GL 14,15

Design Requirements	
Max. Deflection (in)	2.25
Beam Length (ft)	45
Required Moment, Mr (kip-ft)	214.8849
Point Load P1 (kips)	14.34
Point Load P2 (kips)	14.34

$I_{req'd}$ (in^4)	1245.81

W24X68	1830

Nominal Weight	Area, A	Depth, d	Web		Flange		Compact Section Criteria		Axis X-X				Axis Y-Y				r_{ts}	h_o	Torsional Properties	
			Thickness, t_w	$t_w/2$	Width, b_f	Thickness, t_f			I	S	r	Z	I	S	r	Z			J	C_w
(lb/ft)	(in^2)	(in)	(in)	(in)	(in)	(in)	$b_f/2t_f$	h/t_w	(in^4)	(in^3)	(in)	(in^3)	(in^4)	(in^3)	(in)	(in^3)	(in)	(in)	(in^4)	(in^6)
68	20.1	23.7	0.415	0.2075	8.97	0.585	7.66	52	1830	154	9.55	177	70.4	15.7	24.5	68	2.3	23.1	1.87	9430

Flexure

Flange Slenderness		Web Slenderness	
λ_p	9.15	λ_p	90.55
λ_r	24.08	λ_r	137.27
b/t	7.66	h/t_w	52.00

Unbraced Length	
L_p	6.61
L_r	18.90
L_b	15.00

Yielding	
M_p (kip-in)	8850.00
Cb	1.67
M_p (kip-ft)	737.50

LTB		
	If $L_p < L_b < L_r$	If Lb > Lr
Mn (kip-ft)	902.91	12006.19
Values must be < M_p		

$\phi_b M_n$	663.75

Shear

	h/t_w	52.00
Case 1:	$2.24\sqrt{(E/F_y)}$	53.95
Case 2:		
Case 3:		

If $h/t_w < 2.24\sqrt{(E/F_y)}$	
ϕ_v	1.00
C_v	1.00
$\phi_v V_n$	295.07

If $h/t_w > 2.24\sqrt{(E/Fy)}$	
$1.10\sqrt{k_v E/Fy}$	59.24
$1.37\sqrt{k_v E/Fy}$	73.78

If $h/t_w < 1.10\sqrt{(k_v E/Fy)}$	
C_v	1
ϕ_v	0.9
$\phi_v V_n$	265.56

If $1.10\sqrt{k_v E/F_y} < h/t_w < 1.37\sqrt{k_v E/F_y}$	
C_v	1.14
ϕ_v	0.90
$\phi_v V_n$	302.52

If $h/t_w > 1.37\sqrt{(k_v E/Fy)}$	
C_v	1.62
ϕ_v	0.90
$\phi_v V_n$	430.06

$\phi_v V_n$	295.07

Check for Self-Weight

M_r (kip-ft)	232.10	<	663.75
V_r (kips)	7.17	<	295.07

Girder Design Perimeter

Design Requirements	
Max. Deflection (in)	0.75
Floor Live Load, w_L (psf)	20
Floor Dead Load, w_D (psf)	25
Beam Length (ft)	15
Tributary Area (ft)	7.5
Total Dead Load (klf)	0.225
Total Live Load (klf)	0.24
Total Load (Klf)	0.465
Required Moment, M_r (kip-ft)	13.07813
Required Shear, V_r (kips)	3.4875
Shear with Self Weight	4.0575

$I_{req'd}$ (in^4)	24.35
W18X76	1330

Nominal Weight	Area, A	Depth, d	Web		Flange		Compact Section Criteria		Axis X-X				Axis Y-Y				r_{ts}	h_o	Torsional Properties	
			Thickness, t_w	$t_w/2$	Width, b_f	Thickness, t_f			I	S	r	Z	I	S	r	Z			J	C_w
(lb/ft)	(in^2)	(in)	(in)	(in)	(in)	(in)	$b_f/2t_f$	h/t_w	(in^4)	(in^3)	(in)	(in^3)	(in^4)	(in^3)	(in)	(in^3)	(in)	(in)	(in^4)	(in^6)
76	22.3	18.2	0.425	0.2125	11	0.68	8.11	37.8	1330	146	7.73	163	152	27.6	42.2	76	3.02	17.5	2.83	11700

Flexure

Flange Slenderness		Web Slenderness	
λ_p	9.15	λ_p	90.55
λ_r	24.08	λ_r	137.27
b/t	8.11	h/t_w	37.80

Unbraced Length	
L_p	5.37
L_r	15.60
L_b	15.00

Yielding	
M_p (kip-in)	8150.00
Cb	1.14
M_p (kip-ft)	679.17

LTB		
	If $L_p < L_b < L_r$	If $L_b > L_r$
Mn (kip-ft)	500.79	13353.54
Values must be < M_p		

Shear

	h/t_w	37.8
Case 1:	$2.24\|(E/F_y)$	53.95
Case 2:		
Case 3:		

If $h/t_w < 2.24\|(E/F_y)$	
ϕ_v	1
C_v	1
$\phi_v V_n$	232.05

If $h/t_w > 2.24\|(E/Fy)$	
$1.10\|k_v E/Fy$	59.24
$1.37\|k_v E/Fy$	73.78

If $h/t_w < 1.10\|(k_v E/Fy)$	
C_v	1
ϕ_v	0.9
$\phi_v V_n$	208.85

If $1.10\|k_v E/F_y < h/t_w < 1.37\|k_v E/F_y$	
C_v	1.57
ϕ_v	0.90
$\phi_v V_n$	327.28

If $h/t_w > 1.37\|(k_v E/Fy)$	
C_v	3.06
ϕ_v	0.90
$\phi_v V_n$	640.05

$\phi_v V_n$	232.05
$\phi_b M_n$	450.71

Check for Self-Weight

M_r (kip-ft)	15.22	<	450.71
V_r (kips)	4.06	<	232.05

TOWER 2 SECOND FLOOR

Beam Design GL 14,15

Design Requirements	
Max. Deflection (in)	0.75
Floor Live Load, w_L (psf)	100
Floor Dead Load, w_D (psf)	125
Beam Length (ft)	15
Tributary Area (ft)	15
Total Dead Load (klf)	2.25
Total Live Load (klf)	2.4
Total Load (Klf)	4.65
Required Moment, M_r (kip-ft)	130.7813
Required Shear, V_r (kips)	34.875
Shear with Self Weight	35.25

$I_{req'd}$ (in^4)	243.52

W18X50	800

Nominal Weight	Area, A	Depth, d	Web		Flange		Compact Section Criteria		Axis X-X				Axis Y-Y				r_{ts}	h_o	Torsional Properties	
			Thickness, t_w	$t_w/2$	Width, b_f	Thickness, t_f			I	S	r	Z	I	S	r	Z			J	C_w
(lb/ft)	(in^2)	(in)	(in)	(in)	(in)	(in)	$b_f/2t_f$	h/t_w	(in^4)	(in^3)	(in)	(in^3)	(in^4)	(in^3)	(in)	(in^3)	(in)	(in)	(in^4)	(in^6)
50	14.7	18	0.355	0.1775	7.5	0.57	6.57	45.2	800	88.9	7.38	101	40.1	10.7	16.6	50	1.98	17.4	1.24	3040

Flexure

Flange Slenderness		Web Slenderness	
λ_p	9.15	λ_p	90.55
λ_r	24.08	λ_r	137.27
b/t	6.57	h/t_w	45.20

Unbraced Length	
L_p	5.83
L_r	16.90
L_b	15.00

Yielding	
M_p (kip-in)	5050.00
Cb	1.14
M_p (kip-ft)	420.83

LTB		
	If $L_p < L_b < L_r$	If $L_b > L_r$
Mn (kip-ft)	326.16	3495.12
Values must be < M_p		

Shear

	h/t_w	45.2
Case 1:	$2.24\|(E/F_y)$	53.95
Case 2:		
Case 3:		

If $h/t_w < 2.24\|(E/F_y)$	
ϕ_v	1
C_v	1
$\phi_v V_n$	191.7

If $h/t_w > 2.24\|(E/Fy)$	
$1.10\|k_v E/Fy$	59.24
$1.37\|k_v E/Fy$	73.78

If $h/t_w < 1.10\|(k_v E/Fy)$	
C_v	1
ϕ_v	0.9
$\phi_v V_n$	172.53

If $1.10\|k_v E/F_y < h/t_w < 1.37\|k_v E/F_y$	
C_v	1.31
ϕ_v	0.90
$\phi_v V_n$	226.11

If $h/t_w > 1.37\|(k_v E/Fy)$	
C_v	2.14
ϕ_v	0.90
$\phi_v V_n$	369.80

$\phi_v V_n$	191.70
$\phi_b M_n$	293.54

Check for Self-Weight

M_r (kip-ft)	132.19	<	293.54
V_r (kips)	35.25	<	191.70

Girder Design Perimeter

Design Requirements	
Max. Deflection (in)	0.75
Floor Live Load, w_L (psf)	300
Floor Dead Load, w_D (psf)	120
Beam Length (ft)	15
Tributary Area (ft)	7.5
Total Dead Load (klf)	1.08
Total Live Load (klf)	3.6
Total Load (Klf)	7.31
Required Moment, M_r (kip-ft)	205.6641
Required Shear, V_r (kips)	54.84375
Shear with Self Weight	55.41375
Bridge Weight (kips)	42
Bridge Load + Concrete (kips)	39.49
Addition klf due to bridge	2.63

$I_{req'd}$ (in^4)	245.09
W18X76	1330

Nominal Weight	Area, A	Depth, d	Web		Flange		Compact Section Criteria		Axis X-X				Axis Y-Y				r_{ts}	h_o	Torsional Properties	
			Thickness, t_w	$t_w/2$	Width, b_f	Thickness, t_f			I	S	r	Z	I	S	r	Z			J	C_w
(lb/ft)	(in^2)	(in)	(in)	(in)	(in)	(in)	$b_f/2t_f$	h/t_w	(in^4)	(in^3)	(in)	(in^3)	(in^4)	(in^3)	(in)	(in^3)	(in)	(in)	(in^4)	(in^6)
76	22.3	18.2	0.425	0.2125	11	0.68	8.11	37.8	1330	146	7.73	163	152	27.6	42.2	76	3.02	17.5	2.83	11700

Flexure

Flange Slenderness		Web Slenderness	
λ_p	9.15	λ_p	90.55
λ_r	24.08	λ_r	137.27
b/t	8.11	h/t_w	37.80

Unbraced Length	
L_p	11.90
L_r	35.10
L_b	15.00

Yielding	
M_p (kip-in)	8150.00
Cb	1.14
M_p (kip-ft)	679.17

LTB		
	If $L_p < L_b < L_r$	If $L_b > L_r$
Mn (kip-ft)	733.31	13353.54
Values must be < M_p		

Shear

	h/t_w	37.8
Case 1:	$2.24\|(E/F_y)$	53.95
Case 2:		
Case 3:		

If $h/t_w < 2.24\|(E/F_y)$	
ϕ_v	1
C_v	1
$\phi_v V_n$	232.05

If $h/t_w > 2.24\|(E/Fy)$	
$1.10\|k_v E/Fy$	59.24
$1.37\|k_v E/Fy$	73.78

If $h/t_w < 1.10\|(k_v E/Fy)$	
C_v	1
ϕ_v	0.9
$\phi_v V_n$	208.85

If $1.10\|k_v E/F_y < h/t_w < 1.37\|k_v E/F_y$	
C_v	1.57
ϕ_v	0.90
$\phi_v V_n$	327.28

If $h/t_w > 1.37\|(k_v E/Fy)$	
C_v	3.06
ϕ_v	0.90
$\phi_v V_n$	640.05

$\phi_v V_n$	232.05
$\phi_b M_n$	659.98

Check for Self-Weight

M_r (kip-ft)	207.80	<	659.98
V_r (kips)	55.41	<	232.05

TOWER 2 ROOF

Beam Design GL 14,15

Design Requirements	
Max. Deflection (in)	0.75
Floor Live Load, w_L (psf)	20
Floor Dead Load, w_D (psf)	25
Beam Length (ft)	15
Tributary Area (ft)	15
Total Dead Load (klf)	0.45
Total Live Load (klf)	0.48
Total Load (Klf)	0.93
Required Moment, M_r (kip-ft)	26.15625
Required Shear, V_r (kips)	6.975
Shear with Self Weight	7.2

$I_{req'd}$ (in^4)	48.70
W12X30	238

Nominal Weight	Area, A	Depth, d	Web		Flange		Compact Section Criteria		Axis X-X				Axis Y-Y				r_{ts}	h_o	Torsional Properties	
			Thickness, t_w	$t_w/2$	Width, b_f	Thickness, t_f			I	S	r	Z	I	S	r	Z			J	C_w
(lb/ft)	(in^2)	(in)	(in)	(in)	(in)	(in)	$b_f/2t_f$	h/t_w	(in^4)	(in^3)	(in)	(in^3)	(in^4)	(in^3)	(in)	(in^3)	(in)	(in)	(in^4)	(in^6)
30	8.79	12.3	0.26	0.13	6.52	0.44	7.41	41.8	238	38.6	5.21	43.1	20.3	6.24	9.56	30	1.77	11.9	0.457	720

Flexure

Flange Slenderness		Web Slenderness	
λ_p	9.15	λ_p	90.55
λ_r	24.08	λ_r	137.27
b/t	7.41	h/t_w	41.80

Unbraced Length	
L_p	5.37
L_r	15.60
L_b	15.00

Yielding	
M_p (kip-in)	2155.00
Cb	1.14
M_p (kip-ft)	179.58

LTB		
	If $L_p < L_b < L_r$	If $L_b > L_r$
Mn (kip-ft)	132.40	1212.73
Values must be < M_p		

Shear

	h/t_w	41.8
Case 1:	$2.24\|(E/F_y)$	53.95
Case 2:		
Case 3:		

If $h/t_w < 2.24\|(E/F_y)$	
ϕ_v	1
C_v	1
$\phi_v V_n$	95.94

If $h/t_w > 2.24\|(E/Fy)$	
$1.10\|k_v E/Fy$	59.24
$1.37\|k_v E/Fy$	73.78

If $h/t_w < 1.10\|(k_v E/Fy)$	
C_v	1
ϕ_v	0.9
$\phi_v V_n$	86.35

If $1.10\|k_v E/F_y < h/t_w < 1.37\|k_v E/F_y$	
C_v	1.42
ϕ_v	0.90
$\phi_v V_n$	122.37

If $h/t_w > 1.37\|(k_v E/Fy)$	
C_v	2.51
ϕ_v	0.90
$\phi_v V_n$	216.40

$\phi_v V_n$	95.94
$\phi_b M_n$	119.16

Check for Self-Weight

M_r (kip-ft)	27.00	<	119.16
V_r (kips)	7.20	<	95.94

Girder Design Perimeter

Design Requirements	
Max. Deflection (in)	0.75
Floor Live Load, w_L (psf)	20
Floor Dead Load, w_D (psf)	25
Beam Length (ft)	15
Tributary Area (ft)	7.5
Total Dead Load (klf)	0.225
Total Live Load (klf)	0.24
Total Load (Klf)	0.465
Required Moment, M_r (kip-ft)	13.07813
Required Shear, V_r (kips)	3.4875
Shear with Self Weight	4.0575

$I_{req'd}$ (in^4)	24.35

W18X76	1330

Nominal Weight	Area, A	Depth, d	Web		Flange		Compact Section Criteria		Axis X-X				Axis Y-Y				r_{ts}	h_o	Torsional Properties	
			Thickness, t_w	$t_w/2$	Width, b_f	Thickness, t_f			I	S	r	Z	I	S	r	Z			J	C_w
(lb/ft)	(in^2)	(in)	(in)	(in)	(in)	(in)	$b_f/2t_f$	h/t_w	(in^4)	(in^3)	(in)	(in^3)	(in^4)	(in^3)	(in)	(in^3)	(in)	(in)	(in^4)	(in^6)
76	22.3	18.2	0.425	0.2125	11	0.68	8.11	37.8	1330	146	7.73	163	152	27.6	42.2	76	3.02	17.5	2.83	11700

Flexure

Flange Slenderness		Web Slenderness	
λ_p	9.15	λ_p	90.55
λ_r	24.08	λ_r	137.27
b/t	8.11	h/t_w	37.80

Unbraced Length	
L_p	5.37
L_r	15.60
L_b	15.00

Yielding	
M_p (kip-in)	8150.00
Cb	1.14
M_p (kip-ft)	679.17

LTB		
	If $L_p < L_b < L_r$	If $L_b > L_r$
Mn (kip-ft)	500.79	13353.54
Values must be $< M_p$		

Shear

	h/t_w	37.8
Case 1:	$2.24\sqrt{(E/F_y)}$	53.95
Case 2:		
Case 3:		

If $h/t_w < 2.24\sqrt{(E/F_y)}$	
ϕ_v	1
C_v	1
$\phi_v V_n$	232.05

If $h/t_w > 2.24\sqrt{(E/Fy)}$	
$1.10\sqrt{k_v E/Fy}$	59.24
$1.37\sqrt{k_v E/Fy}$	73.78

If $h/t_w < 1.10\sqrt{(k_v E/Fy)}$	
C_v	1
ϕ_v	0.9
$\phi_v V_n$	208.85

If $1.10\sqrt{k_v E/F_y} < h/t_w < 1.37\sqrt{k_v E/F_y}$	
C_v	1.57
ϕ_v	0.90
$\phi_v V_n$	327.28

If $h/t_w > 1.37\sqrt{(k_v E/Fy)}$	
C_v	3.06
ϕ_v	0.90
$\phi_v V_n$	640.05

$\phi_v V_n$	232.05
$\phi_b M_n$	450.71

Check for Self-Weight

M_r (kip-ft)	15.22	<	450.71
V_r (kips)	4.06	<	232.05

COMMERCIAL BUILDING A – FIRST FLOOR

Beam Design Bay B-C

Design Requirements	
Max. Deflection (in)	1.25
Floor Live Load, w_L (psf)	100
Floor Dead Load, w_D (psf)	80
Beam Length (ft)	25
Tributary Area (ft)	10
Total Dead Load (klf)	0.96
Total Live Load (klf)	1.6
Total Load (Klf)	2.56
Required Moment, M_r (kip-ft)	200.00
Required Shear, V_r (kips)	32
Shear with Self Weight	32.95

$I_{req'd}$ (in^4)	620.69
W18X76	1330

Nominal Weight	Area, A	Depth, d	Web		Flange		Compact Section Criteria		Axis X-X				Axis Y-Y				r_{ts}	h_o	Torsional Properties	
			Thickness, t_w	$t_w/2$	Width, b_f	Thickness, t_f			I	S	r	Z	I	S	r	Z			J	C_w
(lb/ft)	(in^2)	(in)	(in)	(in)	(in)	(in)	$b_f/2t_f$	h/t_w	(in^4)	(in^3)	(in)	(in^3)	(in^4)	(in^3)	(in)	(in^3)	(in)	(in)	(in^4)	(in^6)
76	22.3	18.2	0.425	0.2125	11	0.68	8.11	37.8	1330	146	7.73	163	152	27.6	42.2	76	3.02	17.5	2.83	11700

Flexure

Flange Slenderness		Web Slenderness	
λ_p	9.15	λ_p	90.55
λ_r	24.08	λ_r	137.27
b/t	8.11	h/t_w	37.80

Unbraced Length	
L_p	9.22
L_r	27.10
L_b	25

Yielding	
M_p (kip-in)	8150
Cb	1.14
M_p (kip-ft)	679.17

LTB		
	If $L_p < L_b < L_r$	If $L_b > L_r$
Mn (kip-ft)	517.71	4807.27
Values must be < M_p		

Shear

	h/t_w	37.80
Case 1:	$2.24\|(E/F_y)$	53.95
Case 2:		
Case 3:		

If $h/t_w < 2.24\|(E/F_y)$	
ϕ_v	1.00
C_v	1.00
$\phi_v V_n$	232.05

If $h/t_w > 2.24\|(E/Fy)$	
$1.10\|k_v E/Fy$	59.24
$1.37\|k_v E/Fy$	73.78

If $h/t_w < 1.10\|(k_v E/Fy)$	
C_v	1
ϕ_v	0.9
$\phi_v V_n$	208.85

If $1.10\|k_v E/F_y < h/t_w < 1.37\|k_v E/F_y$	
C_v	1.57
ϕ_v	0.90
$\phi_v V_n$	327.28

If $h/t_w > 1.37\|(k_v E/Fy)$	
C_v	3.06
ϕ_v	0.90
$\phi_v V_n$	640.05

$\phi_v V_n$	232.05
$\phi_b M_n$	465.94

Check for Self-Weight

M_r (kip-ft)	205.94	<	465.94
V_r (kips)	32.95	<	232.05

Beam Design Bays C-D

Design Requirements	
Max. Deflection (in)	1.75
Floor Live Load, w_L (psf)	100
Floor Dead Load, w_D (psf)	80
Beam Length (ft)	35
Tributary Area (ft)	10
Total Dead Load (klf)	0.96
Total Live Load (klf)	1.6
Total Load (Klf)	2.56
Required Moment, M_r (kip-ft)	392.00
Required Shear, V_r (kips)	44.8
Shear with Self Weight	47.08

$I_{req'd}$ (in^4)	1703.17

W18X130	2460

Nominal Weight	Area, A	Depth, d	Web		Flange		Compact Section Criteria		Axis X-X				Axis Y-Y				r_{ts}	h_o	Torsional Properties	
			Thickness, t_w	$t_w/2$	Width, b_f	Thickness, t_f			I	S	r	Z	I	S	r	Z			J	C_w
(lb/ft)	(in^2)	(in)	(in)	(in)	(in)	(in)	$b_f/2t_f$	h/t_w	(in^4)	(in^3)	(in)	(in^3)	(in^4)	(in^3)	(in)	(in^3)	(in)	(in)	(in^4)	(in^6)
130	38.3	19.3	0.67	0.335	11.2	1.2	4.65	23.9	2460	256	8.03	290	278	49.9	76.7	130	3.13	18.1	14.5	22700

Flexure

Flange Slenderness		Web Slenderness	
λ_p	9.15	λ_p	90.55
λ_r	24.08	λ_r	137.27
b/t	4.65	h/t_w	23.9

Unbraced Length	
L_p	9.54
L_r	36.6
L_b	35

Yielding	
M_p (kip-in)	14500
Cb	1.14
M_p (kip-ft)	1208.33

LTB		
	If $L_p < L_b < L_r$	If $L_b > L_r$
Mn (kip-ft)	879.50	4619.60
Values must be < M_p		

Shear

	h/t_w	23.90
Case 1:	$2.24\sqrt{(E/F_y)}$	53.95
Case 2:		
Case 3:		

If $h/t_w < 2.24\sqrt{(E/F_y)}$	
ϕ_v	1.00
C_v	1.00
$\phi_v V_n$	387.93

If $h/t_w > 2.24\sqrt{(E/Fy)}$	
$1.10\sqrt{k_v E/Fy}$	59.24
$1.37\sqrt{k_v E/Fy}$	73.78

If $h/t_w < 1.10\sqrt{(k_v E/Fy)}$	
C_v	1.00
ϕ_v	0.90
$\phi_v V_n$	349.14

If $1.10\sqrt{k_v E/F_y} < h/t_w < 1.37\sqrt{k_v E/F_y}$	
C_v	2.48
ϕ_v	0.90
$\phi_v V_n$	865.35

If $h/t_w > 1.37\sqrt{(k_v E/Fy)}$	
C_v	7.67
ϕ_v	0.90
$\phi_v V_n$	2676.55

$\phi_v V_n$	387.93
$\phi_b M_n$	791.55

Check for Self-Weight

M_r (kip-ft)	411.91	<	791.55
V_r (kips)	47.08	<	387.93

Beam Design Bays D-E

Design Requirements	
Max. Deflection (in)	0.6
Floor Live Load, w_L (psf)	100
Floor Dead Load, w_D (psf)	80
Beam Length (ft)	12
Tributary Area (ft)	10
Total Dead Load (klf)	0.96
Total Live Load (klf)	1.6
Total Load (Klf)	2.56
Required Moment, M_r (kip-ft)	46.08
Required Shear, V_r (kips)	15.36
Shear with Self Weight	15.52

$I_{req'd}$ (in^4)	68.64
W12X26	204

Nominal Weight	Area, A	Depth, d	Web		Flange		Compact Section Criteria		Axis X-X				Axis Y-Y				r_{ts}	h_o	Torsional Properties	
			Thickness, t_w	$t_w/2$	Width, b_f	Thickness, t_f			I	S	r	Z	I	S	r	Z			J	C_w
(lb/ft)	(in^2)	(in)	(in)	(in)	(in)	(in)	$b_f/2t_f$	h/t_w	(in^4)	(in^3)	(in)	(in^3)	(in^4)	(in^3)	(in)	(in^3)	(in)	(in)	(in^4)	(in^6)
26	7.65	12.2	0.23	0.115	6.49	0.38	8.54	47.2	204	33.4	5.17	37.2	17.3	5.34	8.17	26	1.75	11.8	0.3	607

Flexure

Flange Slenderness		Web Slenderness	
λ_p	9.15	λ_p	90.55
λ_r	24.08	λ_r	137.27
b/t	8.54	h/t_w	47.2

Unbraced Length	
L_p	5.33
L_r	14.9
L_b	12

Yielding	
M_p (kip-in)	1860
Cb	1.14
M_p (kip-ft)	155.00

LTB		
	If $L_p < L_b < L_r$	If $L_b > L_r$
Mn (kip-ft)	130.53	1602.77
Values must be < M_p		

Shear

	h/t_w	47.20
Case 1:	2.24|(E/F_y)	53.95
Case 2:		
Case 3:		

If h/t_w < 2.24|(E/F_y)	
ϕ_v	1.00
C_v	1.00
$\phi_v V_n$	84.18

If h/t_w > 2.24|(E/Fy)	
1.10|k_vE/Fy	59.24
1.37|k_vE/Fy	73.78

If h/t_w < 1.10|(k_vE/Fy)	
C_v	1.00
ϕ_v	0.90
$\phi_v V_n$	75.76

If 1.10|k_vE/F_y < h/t_w < 1.37|k_vE/F_y	
C_v	1.26
ϕ_v	0.90
$\phi_v V_n$	95.08

If h/t_w > 1.37|(k_vE/Fy)	
C_v	1.97
ϕ_v	0.90
$\phi_v V_n$	148.92

$\phi_v V_n$	84.18
$\phi_b M_n$	117.48

Check for Self-Weight

M_r (kip-ft)	46.55	<	117.48
V_r (kips)	15.52	<	84.18

Girder Design GL A and E/ 0-24

Design Requirements	
Max. Deflection (in)	1
Beam Length (ft)	20
Required Moment, Mr (kip-ft)	93.8
Point Load P1 (kips)	18.76

$I_{req'd}$ (in^4)	186.31
W12X50	391

Nominal Weight	Area, A	Depth, d	Web		Flange		Compact Section Criteria		Axis X-X				Axis Y-Y				r_{ts}	h_o	Torsional Properties	
			Thickness, t_w	$t_w/2$	Width, b_f	Thickness, t_f			I	S	r	Z	I	S	r	Z			J	C_w
(lb/ft)	(in^2)	(in)	(in)	(in)	(in)	(in)	$b_f/2t_f$	h/t_w	(in^4)	(in^3)	(in)	(in^3)	(in^4)	(in^3)	(in)	(in^3)	(in)	(in)	(in^4)	(in^6)
50	14.6	12.2	0.37	0.185	8.08	0.64	6.31	26.8	391	64.2	5.18	71.9	56.3	13.9	21.3	50	2.25	11.6	1.71	1880

Flexure

Flange Slenderness		Web Slenderness	
λ_p	9.15	λ_p	90.55
λ_r	24.08	λ_r	137.27
b/t	6.31	h/t_w	26.80

Unbraced Length	
L_p	2.66
L_r	23.80
L_b	10.00

Yielding	
M_p (kip-in)	3595.00
Cb	1.67
M_p (kip-ft)	299.58

LTB		
	If $L_p < L_b < L_r$	If Lb > Lr
Mn (kip-ft)	435.17	10777.33
Values must be < M_p		

$\phi_b M_n$	269.63

Shear

	h/t_w	26.80
Case 1:	$2.24\|(E/F_y)$	53.95
Case 2:		
Case 3:		

If $h/t_w < 2.24\|(E/F_y)$	
ϕ_v	1.00
C_v	1.00
$\phi_v V_n$	135.42

If $h/t_w > 2.24\|(E/Fy)$	
$1.10\|k_v E/Fy$	59.24
$1.37\|k_v E/Fy$	73.78

If $h/t_w < 1.10\|(k_v E/Fy)$	
C_v	1
ϕ_v	0.9
$\phi_v V_n$	121.88

If $1.10\|k_v E/F_y < h/t_w < 1.37\|k_v E/F_y$	
C_v	2.21
ϕ_v	0.90
$\phi_v V_n$	269.39

If $h/t_w > 1.37\|(k_v E/Fy)$	
C_v	6.10
ϕ_v	0.90
$\phi_v V_n$	743.07

$\phi_v V_n$	135.42

Note: Use Yeilding

Check for Self-Weight

M_r (kip-ft)	96.30	<	269.63
V_r (kips)	9.38	<	135.42

Girder Design GL BCD/ 8-25 and 25-28

Design Requirements	
Max. Deflection (in)	1
Beam Length (ft)	20
Required Moment, Mr (kip-ft)	480
Point Load P1 (kips)	96

$I_{req'd}$ (in^4)	953.38

W18X65	1070

Nominal Weight	Area, A	Depth, d	Web		Flange		Compact Section Criteria		Axis X-X				Axis Y-Y				r_{ts}	h_o	Torsional Properties	
			Thickness, t_w	$t_w/2$	Width, b_f	Thickness, t_f			I	S	r	Z	I	S	r	Z			J	C_w
(lb/ft)	(in^2)	(in)	(in)	(in)	(in)	(in)	$b_f/2t_f$	h/t_w	(in^4)	(in^3)	(in)	(in^3)	(in^4)	(in^3)	(in)	(in^3)	(in)	(in)	(in^4)	(in^6)
65	19.1	18.4	0.45	0.225	7.59	0.75	5.06	35.7	1070	117	7.49	133	54.8	14.4	22.5	65	2.03	17.7	2.73	4240

Flexure

Flange Slenderness		Web Slenderness	
λ_p	9.15	λ_p	90.55
λ_r	24.08	λ_r	137.27
b/t	5.06	h/t_w	35.70

Unbraced Length	
L_p	5.97
L_r	18.80
L_b	10.00

Yielding	
M_p (kip-in)	6650.00
Cb	1.67
M_p (kip-ft)	554.17

LTB		
	If $L_p < L_b < L_r$	If Lb > Lr
Mn (kip-ft)	813.77	15987.82
Values must be < M_p		

$\phi_b M_n$	498.75

Shear

	h/t_w	35.70
Case 1:	$2.24\sqrt{E/F_y}$	53.95
Case 2:		
Case 3:		

If $h/t_w < 2.24\sqrt{E/F_y}$	
ϕ_v	1.00
C_v	1.00
$\phi_v V_n$	248.40

If $h/t_w > 2.24\sqrt{E/Fy}$	
$1.10\sqrt{k_v E/Fy}$	59.24
$1.37\sqrt{k_v E/Fy}$	73.78

If $h/t_w < 1.10\sqrt{k_v E/Fy}$	
C_v	1
ϕ_v	0.9
$\phi_v V_n$	223.56

If $1.10\sqrt{k_v E/F_y} < h/t_w < 1.37\sqrt{k_v E/F_y}$	
C_v	1.66
ϕ_v	0.90
$\phi_v V_n$	370.95

If $h/t_w > 1.37\sqrt{k_v E/Fy}$	
C_v	3.44
ϕ_v	0.90
$\phi_v V_n$	768.13

$\phi_v V_n$	248.40

Check for Self-Weight

M_r (kip-ft)	483.25	<	498.75
V_r (kips)	48.00	<	248.40

Girder Design GL BCD/ 0-3, 5-7, 9-14

Design Requirements	
Max. Deflection (in)	1
Beam Length (ft)	20
Required Moment, Mr (kip-ft)	480
Point Load P1 (kips)	96

$I_{req'd}$ (in^4)	953.38
W18X65	1070

Nominal Weight	Area, A	Depth, d	Web: Thickness, t_w	Web: $t_w/2$	Flange: Width, b_f	Flange: Thickness, t_f	Compact Section Criteria: $b_f/2t_f$	Compact Section Criteria: h/t_w	Axis X-X: I	Axis X-X: S	Axis X-X: r	Axis X-X: Z	Axis Y-Y: I	Axis Y-Y: S	Axis Y-Y: r	Axis Y-Y: Z	r_{ts}	h_o	Torsional Properties: J	Torsional Properties: C_w
(lb/ft)	(in^2)	(in)	(in)	(in)	(in)	(in)			(in^4)	(in^3)	(in)	(in^3)	(in^4)	(in^3)	(in)	(in^3)	(in)	(in)	(in^4)	(in^6)
65	19.1	18.4	0.45	0.225	7.59	0.75	5.06	35.7	1070	117	7.49	133	54.8	14.4	22.5	65	2.03	17.7	2.73	4240

Flexure

Flange Slenderness		Web Slenderness	
λ_p	9.15	λ_p	90.55
λ_r	24.08	λ_r	137.27
b/t	5.06	h/t_w	35.70

Unbraced Length	
L_p	5.97
L_r	18.80
L_b	10.00

Yielding	
M_p (kip-in)	6650.00
Cb	1.67
M_p (kip-ft)	554.17

LTB		
	If $L_p < L_b < L_r$	If Lb > Lr
Mn (kip-ft)	813.77	15987.82
Values must be < M_p		

$\phi_b M_n$	498.75

Shear

	h/t_w	35.70
Case 1:	2.24|(E/F_y)|	53.95
Case 2:		
Case 3:		

If h/t_w < 2.24|(E/F_y)|	
ϕ_v	1.00
C_v	1.00
$\phi_v V_n$	248.40

If h/t_w > 2.24|(E/Fy)|	
1.10|k_vE/Fy	59.24
1.37|k_vE/Fy	73.78

If h/t_w < 1.10|(k_vE/Fy)|	
C_v	1
ϕ_v	0.9
$\phi_v V_n$	223.56

If 1.10|k_vE/F_y < h/t_w < 1.37|k_vE/F_y	
C_v	1.66
ϕ_v	0.90
$\phi_v V_n$	370.95

If h/t_w > 1.37|(k_vE/Fy)|	
C_v	3.44
ϕ_v	0.90
$\phi_v V_n$	768.13

$\phi_v V_n$	248.40

Check for Self-Weight

M_r (kip-ft)	483.25	<	498.75
V_r (kips)	48.00	<	248.40

COMMERCIAL BUILDING A – ROOF

Beam Design Bay A-B / 0-12

Design Requirements	
Max. Deflection (in)	0.5
Floor Live Load, w_L (psf)	20
Floor Dead Load, w_D (psf)	25
Beam Length (ft)	10
Tributary Area (ft)	10
Total Dead Load (klf)	0.3
Total Live Load (klf)	0.32
Total Load (Klf)	0.62
Required Moment, M_r (kip-ft)	7.75
Required Shear, V_r (kips)	3.1
Shear with Self Weight	3.23

$I_{req'd}$ (in^4)	9.62

W12X26	204

Nominal Weight	Area, A	Depth, d	Web		Flange		Compact Section Criteria		Axis X-X				Axis Y-Y				r_{ts}	h_o	Torsional Properties	
			Thickness, t_w	$t_w/2$	Width, b_f	Thickness, t_f			I	S	r	Z	I	S	r	Z			J	C_w
(lb/ft)	(in^2)	(in)	(in)	(in)	(in)	(in)	$b_f/2t_f$	h/t_w	(in^4)	(in^3)	(in)	(in^3)	(in^4)	(in^3)	(in)	(in^3)	(in)	(in)	(in^4)	(in^6)
26	7.65	12.2	0.23	0.115	6.49	0.38	8.54	47.2	204	33.4	5.17	37.2	17.3	5.34	8.17	26	1.75	11.8	0.3	607

Flexure

Flange Slenderness		Web Slenderness	
λ_p	9.15	λ_p	90.55
λ_r	24.08	λ_r	137.27
b/t	8.54	h/t_w	47.20

Unbraced Length	
L_p	5.33
L_r	14.90
L_b	10.00

Yielding	
M_p (kip-in)	1860.00
Cb	1.14
M_p (kip-ft)	155.00

LTB		
	If $L_p < L_b < L_r$	If $L_b > L_r$
Mn (kip-ft)	144.20	2307.99
Values must be < M_p		

Shear

	h/t_w	47.2
Case 1:	$2.24\|(E/F_y)$	53.95
Case 2:		
Case 3:		

If $h/t_w < 2.24\|(E/F_y)$	
ϕ_v	1
C_v	1
$\phi_v V_n$	84.18

If $h/t_w > 2.24\|(E/Fy)$	
$1.10\|k_v E/Fy$	59.24
$1.37\|k_v E/Fy$	73.78

If $h/t_w < 1.10\|(k_v E/Fy)$	
C_v	1
ϕ_v	0.9
$\phi_v V_n$	75.76

If $1.10\|k_v E/F_y < h/t_w < 1.37\|k_v E/F_y$	
C_v	1.26
ϕ_v	0.90
$\phi_v V_n$	95.08

If $h/t_w > 1.37\|(k_v E/Fy)$	
C_v	1.97
ϕ_v	0.90
$\phi_v V_n$	148.92

$\phi_v V_n$	84.18
$\phi_b M_n$	129.78

Check for Self-Weight

M_r (kip-ft)	8.08	<	129.78
V_r (kips)	3.23	<	84.18

Girder Design GL A/ 0-12

Design Requirements	
Max. Deflection (in)	1
Beam Length (ft)	20
Required Moment, Mr (kip-ft)	16.15
Point Load P1 (kips)	3.23

$I_{req'd}$ (in^4)	32.08
W12X50	391

Nominal Weight	Area, A	Depth, d	Web		Flange		Compact Section Criteria		Axis X-X				Axis Y-Y				r_{ts}	h_o	Torsional Properties	
			Thickness, t_w	$t_w/2$	Width, b_f	Thickness, t_f			I	S	r	Z	I	S	r	Z			J	C_w
(lb/ft)	(in^2)	(in)	(in)	(in)	(in)	(in)	$b_f/2t_f$	h/t_w	(in^4)	(in^3)	(in)	(in^3)	(in^4)	(in^3)	(in)	(in^3)	(in)	(in)	(in^4)	(in^6)
50	14.6	12.2	0.37	0.185	8.08	0.64	6.31	26.8	391	64.2	5.18	71.9	56.3	13.9	21.3	50	2.25	11.6	1.71	1880

Flexure

Flange Slenderness		Web Slenderness	
λ_p	9.15	λ_p	90.55
λ_r	24.08	λ_r	137.27
b/t	6.31	h/t_w	26.80

Unbraced Length	
L_p	2.66
L_r	23.80
L_b	10.00

Yielding	
M_p (kip-in)	3595.00
Cb	1.67
M_p (kip-ft)	299.58

LTB		
	If $L_p < L_b < L_r$	If Lb > Lr
Mn (kip-ft)	435.17	10777.33
Values must be < M_p		

$\phi_b M_n$	269.63

Shear

	h/t_w	26.80
Case 1:	$2.24\|(E/F_y)$	53.95
Case 2:		
Case 3:		

If $h/t_w < 2.24\|(E/F_y)$	
ϕ_v	1.00
C_v	1.00
$\phi_v V_n$	135.42

If $h/t_w > 2.24\|(E/Fy)$	
$1.10\|k_vE/Fy$	59.24
$1.37\|k_vE/Fy$	73.78

If $h/t_w < 1.10\|(k_vE/Fy)$	
C_v	1
ϕ_v	0.9
$\phi_v V_n$	121.88

If $1.10\|k_vE/F_y < h/t_w < 1.37\|k_vE/F_y$	
C_v	2.21
ϕ_v	0.90
$\phi_v V_n$	269.39

If $h/t_w > 1.37\|(k_vE/Fy)$	
C_v	6.10
ϕ_v	0.90
$\phi_v V_n$	743.07

$\phi_v V_n$	135.42

Check for Self-Weight

M_r (kip-ft)	18.65	<	269.63
V_r (kips)	1.62	<	135.42

Girder Design GL B,C,D/ 0-12

Design Requirements	
Inputs	
Beam Length (ft)	20
Point Load P1 (kips)	13.68
Point Load P2 (kips)	13.68
Point Load P3 (kips)	13.68
Required Moment, Mr (kip-ft)	136.8
Max. Deflection all (in)	1
Lp	6.92
Lr	23.8
Lb	5
$I_{req'd}$ (in^4)	326.06

W12X50	391

Nominal Weight	Area, A	Depth, d	Web		Flange		Compact Section Criteria		Axis X-X				Axis Y-Y				r_{ts}	h_o	Torsional Properties	
			Thickness, t_w	$t_w/2$	Width, b_f	Thickness, t_f			I	S	r	Z	I	S	r	Z			J	C_w
(lb/ft)	(in^2)	(in)	(in)	(in)	(in)	(in)	$b_f/2t_f$	h/t_w	(in^4)	(in^3)	(in)	(in^3)	(in^4)	(in^3)	(in)	(in^3)	(in)	(in)	(in^4)	(in^6)
50	14.6	12.2	0.37	0.185	8.08	0.64	6.31	26.8	391	64.2	5.18	71.9	56.3	13.9	21.3	50	2.25	11.6	1.71	1880

Flexure

Flange Slenderness		Web Slenderness	
λ_p	9.15	λ_p	90.55
λ_r	24.08	λ_r	137.27
b/t	6.31	h/t_w	26.8

Yielding	
M_p (kip-in)	3235.50
Cb	1.11
M_p (kip-ft)	269.63

LTB		
	If $L_p < L_b < L_r$	If Lb > Lr
Mn (kip-ft)	309.68	28653.51
Values must be < M_p		

Shear

Case 1:	h/t_w	26.8
Case 2:	$2.24\|(E/F_y)$	53.95
Case 3:		

If $h/t_w < 2.24\|(E/F_y)$	
ϕ_v	1
C_v	1
$\phi_v V_n$	135.42

If $h/t_w > 2.24\|(E/Fy)$	
$1.10\|k_vE/Fy$	59.24
$1.37\|k_vE/Fy$	73.78

If $h/t_w < 1.10\|(k_vE/Fy)$	
C_v	1.00
ϕ_v	0.90
$\phi_v V_n$	121.88

If $1.10\|k_vE/F_y < h/t_w < 1.37\|k_vE/F_y$	
C_v	2.21
ϕ_v	0.90
$\phi_v V_n$	269.39

If $h/t_w > 1.37\|(k_vE/Fy)$	
C_v	6.10
ϕ_v	0.90
$\phi_v V_n$	743.07

$\phi_b M_n$	242.66
$\phi_v V_n$	135.42

Check for Self-Weight

M_r (kip-ft)	139.30	<	242.66
V_r (kips)	6.84	<	135.42

COMMERCIAL BUILDING B – FIRST FLOOR

Beam Design Bays B-F GL 0-11

Design Requirements	
Max. Deflection (in)	1.25
Floor Live Load, w_L (psf)	100
Floor Dead Load, w_D (psf)	80
Beam Length (ft)	25
Tributary Area (ft)	10
Total Dead Load (klf)	0.96
Total Live Load (klf)	1.6
Total Load (Klf)	2.56
Required Moment, M_r (kip-ft)	200.00
Required Shear, V_r (kips)	32
Shear with Self Weight	32.95

$I_{req'd}$ (in^4)	620.69
W18X76	1330

Nominal Weight	Area, A	Depth, d	Web		Flange		Compact Section Criteria		Axis X-X				Axis Y-Y				r_{ts}	h_o	Torsional Properties	
			Thickness, t_w	$t_w/2$	Width, b_f	Thickness, t_f			I	S	r	Z	I	S	r	Z			J	C_w
(lb/ft)	(in^2)	(in)	(in)	(in)	(in)	(in)	$b_f/2t_f$	h/t_w	(in^4)	(in^3)	(in)	(in^3)	(in^4)	(in^3)	(in)	(in^3)	(in)	(in)	(in^4)	(in^6)
76	22.3	18.2	0.425	0.2125	11	0.68	8.11	37.8	1330	146	7.73	163	152	27.6	42.2	76	3.02	17.5	2.83	11700

Flexure

Flange Slenderness		Web Slenderness	
λ_p	9.15	λ_p	90.55
λ_r	24.08	λ_r	137.27
b/t	8.11	h/t_w	37.8

Unbraced Length	
L_p	9.22
L_r	27.1
L_b	25

Yielding	
M_p (kip-in)	8150
Cb	1.14
M_p (kip-ft)	679.17

LTB		
	If $L_p < L_b < L_r$	If $L_b > L_r$
Mn (kip-ft)	517.71	4807.27
Values must be < M_p		

Shear

	h/t_w	37.80
Case 1:	$2.24\|(E/F_y)$	53.95
Case 2:		
Case 3:		

If $h/t_w < 2.24\|(E/F_y)$	
ϕ_v	1.00
C_v	1.00
$\phi_v V_n$	232.05

If $h/t_w > 2.24\|(E/Fy)$	
$1.10\|k_v E/Fy$	59.24
$1.37\|k_v E/Fy$	73.78

If $h/t_w < 1.10\|(k_v E/Fy)$	
C_v	1.00
ϕ_v	0.90
$\phi_v V_n$	208.85

If $1.10\|k_v E/F_y < h/t_w < 1.37\|k_v E/F_y$	
C_v	1.57
ϕ_v	0.90
$\phi_v V_n$	327.28

If $h/t_w > 1.37\|(k_v E/Fy)$	
C_v	3.06
ϕ_v	0.90
$\phi_v V_n$	640.05

$\phi_v V_n$	232.05
$\phi_b M_n$	465.94

Check for Self-Weight

M_r (kip-ft)	205.94	<	465.94
V_r (kips)	32.95	<	232.05

Beam Design GL A,0,G (Perimeter)

Design Requirements	
Max. Deflection (in)	0.6
Floor Live Load, w_L (psf)	100
Floor Dead Load, w_D (psf)	80
Beam Length (ft)	12
Tributary Area (ft)	12
Total Dead Load (klf)	1.152
Total Live Load (klf)	1.92
Total Load (Klf)	3.072
Required Moment, M_r (kip-ft)	55.30
Required Shear, V_r (kips)	18.432
Shear with Self Weight	18.59

$I_{req'd}$ (in^4)	82.37
W12X26	204

Nominal Weight	Area, A	Depth, d	Web		Flange		Compact Section Criteria		Axis X-X				Axis Y-Y				r_{ts}	h_o	Torsional Properties	
			Thickness, t_w	$t_w/2$	Width, b_f	Thickness, t_f			I	S	r	Z	I	S	r	Z			J	C_w
(lb/ft)	(in^2)	(in)	(in)	(in)	(in)	(in)	$b_f/2t_f$	h/t_w	(in^4)	(in^3)	(in)	(in^3)	(in^4)	(in^3)	(in)	(in^3)	(in)	(in)	(in^4)	(in^6)
26	7.65	12.2	0.23	0.115	6.49	0.38	8.54	47.2	204	33.4	5.17	37.2	17.3	5.34	8.17	26	1.75	11.8	0.3	607

Flexure

Flange Slenderness		Web Slenderness	
λ_p	9.15	λ_p	90.55
λ_r	24.08	λ_r	137.27
b/t	8.54	h/t_w	47.2

Unbraced Length	
L_p	5.33
L_r	14.9
L_b	12

Yielding	
M_p (kip-in)	1860
Cb	1.14
M_p (kip-ft)	155.00

LTB		
	If $L_p < L_b < L_r$	If $L_b > L_r$
Mn (kip-ft)	130.53	1602.77
Values must be < M_p		

Shear

	h/t_w	47.20
Case 1:	$2.24\sqrt{(E/F_y)}$	53.95
Case 2:		
Case 3:		

If $h/t_w < 2.24\sqrt{(E/F_y)}$	
ϕ_v	1.00
C_v	1.00
$\phi_v V_n$	84.18

If $h/t_w > 2.24\sqrt{(E/Fy)}$	
$1.10\sqrt{k_v E/Fy}$	59.24
$1.37\sqrt{k_v E/Fy}$	73.78

If $h/t_w < 1.10\sqrt{(k_v E/Fy)}$	
C_v	1.00
ϕ_v	0.90
$\phi_v V_n$	75.76

If $1.10\sqrt{k_v E/F_y} < h/t_w < 1.37\sqrt{k_v E/F_y}$	
C_v	1.26
ϕ_v	0.90
$\phi_v V_n$	95.08

If $h/t_w > 1.37\sqrt{(k_v E/Fy)}$	
C_v	1.97
ϕ_v	0.90
$\phi_v V_n$	148.92

$\phi_v V_n$	84.18
$\phi_b M_n$	117.48

Check for Self-Weight

M_r (kip-ft)	55.76	<	117.48
V_r (kips)	18.59	<	84.18

COMMERCIAL BUILDING B – ROOF

Girder Design GL B-F/ 8-16

Design Requirements	
Inputs	
Beam Length (ft)	25
Point Load P1 (kips)	6.38
Point Load P2 (kips)	6.38
Point Load P3 (kips)	6.38
Required Moment, Mr (kip-ft)	79.6875
Max. Deflection all (in)	1.25
Lp	6.92
Lr	23.8
Lb	5
$I_{req'd}$ (in^4)	237.41

W12X50	391

Nominal Weight	Area, A	Depth, d	Web		Flange		Compact Section Criteria		Axis X-X				Axis Y-Y				r_{ts}	h_o	Torsional Properties	
			Thickness, t_w	$t_w/2$	Width, b_f	Thickness, t_f			I	S	r	Z	I	S	r	Z			J	C_w
(lb/ft)	(in^2)	(in)	(in)	(in)	(in)	(in)	$b_f/2t_f$	h/t_w	(in^4)	(in^3)	(in)	(in^3)	(in^4)	(in^3)	(in)	(in^3)	(in)	(in)	(in^4)	(in^6)
50	14.6	12.2	0.37	0.185	8.08	0.64	6.31	26.8	391	64.2	5.18	71.9	56.3	13.9	21.3	50	2.25	11.6	1.71	1880

Flexure

Flange Slenderness		Web Slenderness	
λ_p	9.15	λ_p	90.55
λ_r	24.08	λ_r	137.27
b/t	6.31	h/t_w	26.8

Yielding	
M_p (kip-in)	3595.00
Cb	1.11
M_p (kip-ft)	299.58

LTB		
	If $L_p < L_b < L_r$	If Lb > Lr
Mn (kip-ft)	346.72	28653.51
Values must be < M_p		

Shear

Case 1:	h/t_w	26.8
Case 2:	$2.24\|(E/F_y)\|$	53.95
Case 3:		

If $h/t_w < 2.24\|(E/F_y)\|$	
ϕ_v	1
C_v	1
$\phi_v V_n$	135.42

If $h/t_w > 2.24\|(E/Fy)\|$	
$1.10\|k_v E/Fy$	59.24
$1.37\|k_v E/Fy$	73.78

If $h/t_w < 1.10\|(k_v E/Fy)\|$	
C_v	1.00
ϕ_v	0.90
$\phi_v V_n$	121.88

If $1.10\|k_v E/F_y < h/t_w < 1.37\|k_v E/F_y$	
C_v	2.21
ϕ_v	0.90
$\phi_v V_n$	269.39

If $h/t_w > 1.37\|(k_v E/Fy)\|$	
C_v	6.10
ϕ_v	0.90
$\phi_v V_n$	743.07

$\phi_b M_n$	269.63
$\phi_v V_n$	135.42

Check for Self-Weight

M_r (kip-ft)	83.59	<	269.63
V_r (kips)	3.19	<	135.42

COMMERCIAL BUILDING C

Beam Design Building C

Design Requirements	
Max. Deflection (in)	1.25
Floor Live Load, w_L (psf)	100
Floor Dead Load, w_D (psf)	80
Beam Length (ft)	25
Tributary Area (ft)	10
Total Dead Load (klf)	0.96
Total Live Load (klf)	1.6
Total Load (Klf)	2.56
Required Moment, M_r (kip-ft)	200.00
Required Shear, V_r (kips)	32
Shear with Self Weight	32.95

$I_{req'd}$ (in^4)	620.69
W18X76	1330

Nominal Weight	Area, A	Depth, d	Web		Flange		Compact Section Criteria		Axis X-X				Axis Y-Y				r_{ts}	h_o	Torsional Properties	
			Thickness, t_w	$t_w/2$	Width, b_f	Thickness, t_f			I	S	r	Z	I	S	r	Z			J	C_w
(lb/ft)	(in^2)	(in)	(in)	(in)	(in)	(in)	$b_f/2t_f$	h/t_w	(in^4)	(in^3)	(in)	(in^3)	(in^4)	(in^3)	(in)	(in^3)	(in)	(in)	(in^4)	(in^6)
76	22.3	18.2	0.425	0.2125	11	0.68	8.11	37.8	1330	146	7.73	163	152	27.6	42.2	76	3.02	17.5	2.83	11700

Flexure

Flange Slenderness		Web Slenderness	
λ_p	9.15	λ_p	90.55
λ_r	24.08	λ_r	137.27
b/t	8.11	h/t_w	37.8

Unbraced Length	
L_p	9.22
L_r	27.1
L_b	25

Yielding	
M_p (kip-in)	8150
Cb	1.14
M_p (kip-ft)	679.17

LTB		
	If $L_p < L_b < L_r$	If $L_b > L_r$
Mn (kip-ft)	517.71	4807.27
Values must be < M_p		

Shear

	h/t_w	37.80
Case 1:	$2.24\|(E/F_y)$	53.95
Case 2:		
Case 3:		

If $h/t_w < 2.24\|(E/F_y)$	
ϕ_v	1.00
C_v	1.00
$\phi_v V_n$	232.05

If $h/t_w > 2.24\|(E/Fy)$	
$1.10\|k_v E/Fy$	59.24
$1.37\|k_v E/Fy$	73.78

If $h/t_w < 1.10\|(k_v E/Fy)$	
C_v	1.00
ϕ_v	0.90
$\phi_v V_n$	208.85

If $1.10\|k_v E/F_y < h/t_w < 1.37\|k_v E/F_y$	
C_v	1.57
ϕ_v	0.90
$\phi_v V_n$	327.28

If $h/t_w > 1.37\|(k_v E/Fy)$	
C_v	3.06
ϕ_v	0.90
$\phi_v V_n$	640.05

$\phi_v V_n$	232.05
$\phi_b M_n$	465.94

Check for Self-Weight

M_r (kip-ft)	205.94	<	465.94
V_r (kips)	32.95	<	232.05

Girder Design GL A-D

Design Requirements	
Max. Deflection (in)	1.25
Beam Length (ft)	25
Required Moment, Mr (kip-ft)	496.25
Point Load P1 (kips)	79.4
Point Load P2 (kips)	79.4

$I_{req'd}$ (in^4)	2129.0152
W18X130	2460

Nominal Weight	Area, A	Depth, d	Web		Flange		Compact Section Criteria		Axis X-X				Axis Y-Y				r_{ts}	h_o	Torsional Properties	
			Thickness, t_w	$t_w/2$	Width, b_f	Thickness, t_f			I	S	r	Z	I	S	r	Z			J	C_w
(lb/ft)	(in^2)	(in)	(in)	(in)	(in)	(in)	$b_f/2t_f$	h/t_w	(in^4)	(in^3)	(in)	(in^3)	(in^4)	(in^3)	(in)	(in^3)	(in)	(in)	(in^4)	(in^6)
130	38.3	19.3	0.67	0.335	11.2	1.2	4.65	23.9	2460	256	8.03	290	278	49.9	76.7	130	3.13	18.1	14.5	22700

Flexure

Flange Slenderness		Web Slenderness	
λ_p	9.15	λ_p	90.55
λ_r	24.08	λ_r	137.27
b/t	4.65	h/t_w	23.90

Unbraced Length	
L_p	9.54
L_r	36.60
L_b	7.50

Yielding	
M_p (kip-in)	14500.00
Cb	1.67
M_p (kip-ft)	1208.33

LTB		
	If $L_p < L_b < L_r$	If Lb > Lr
Mn (kip-ft)	2076.04	147848.66
Values must be < M_p		

$\phi_b M_n$	1087.50

Shear

	h/t_w	23.90
Case 1:	$2.24\sqrt{(E/F_y)}$	53.95
Case 2:		
Case 3:		

If $h/t_w < 2.24\sqrt{(E/F_y)}$	
ϕ_v	1.00
C_v	1.00
$\phi_v V_n$	387.93

If $h/t_w > 2.24\sqrt{(E/Fy)}$	
$1.10\sqrt{k_v E/Fy}$	59.24
$1.37\sqrt{k_v E/Fy}$	73.78

If $h/t_w < 1.10\sqrt{(k_v E/Fy)}$	
C_v	1
ϕ_v	0.9
$\phi_v V_n$	349.14

If $1.10\sqrt{k_v E/F_y} < h/t_w < 1.37\sqrt{k_v E/F_y}$	
C_v	2.48
ϕ_v	0.90
$\phi_v V_n$	865.35

If $h/t_w > 1.37\sqrt{(k_v E/Fy)}$	
C_v	7.67
ϕ_v	0.90
$\phi_v V_n$	2676.55

$\phi_v V_n$	387.93

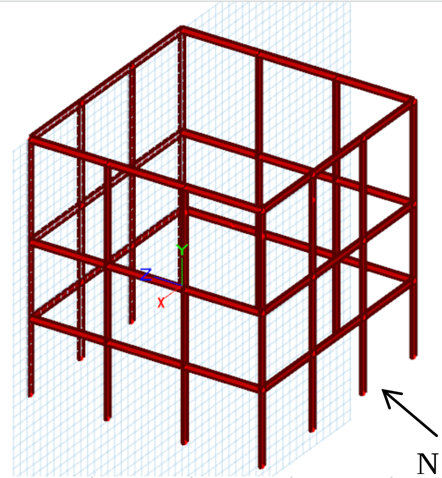
Check for Self-Weight

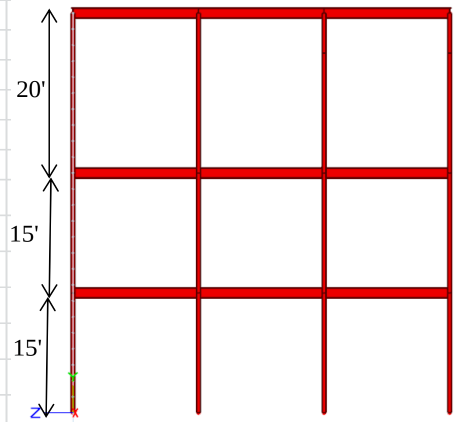
M_r (kip-ft)	506.41	<	1087.50
V_r (kips)	39.70	<	387.93

APPENDIX C

LATERAL DESIGN CALCULATIONS

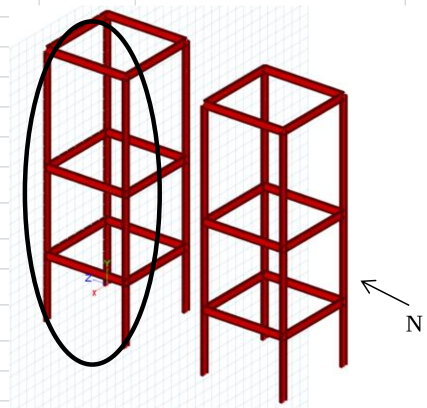
Tower 1/3			
N/S = E/W			
1st Story			
Allowable Story Drift (in)	4.5		
Story Height (ft)	15		
Story Height (in)	180		
$\Delta_{inelastic}$ (in)	4.5		
I_e	1		
Cd	5.5		
$\delta_{elastic}$ (in)	0.818	<<< Design Deflection	
F_1 (k)	18.27	9.135	
k=F/Δ (k/in)	22.33		
$k=24EI/L^3$ (k/in)		<<< 4 Columns	
$I=KL^3/24E$ (in^4)	187.11		
Choose Member	W21x182		
Deflection			
δ1 (in)	0.573	<.818	Ok
2nd Story			
Allowable Story Drift (in)	4.5		
Story Height (ft)	15		
Story Height (in)	180		
$\Delta_{inelastic}$ (in)	4.5		
I_e	1		
Cd	5.5		
$\delta_{elastic}$ (in)	0.818	<<< Design Deflection	
F_2 (k)	17.99	8.995	
F=kΔ (k)			
k=F/Δ (k/in)	21.99		
$k=24EI/L^3$ (k/in)		<<< 4 Columns	
$I=KL^3/24E$ (in^4)	184.2424		
Choose Member	W21x182		
Deflection			
δ2 (in)	0.509	<.818	Ok
3rd Story			
Allowable Story Drift (in)	6		
Story Height (ft)	20		
Story Height (in)	240		
$\Delta_{inelastic}$ (in)	6		
I_e	1		
Cd	5.5		
$\delta_{elastic}$ (in)	1.091	<<< Design Deflection	
F_3 (k)	13.75	6.875	
F=kΔ (k)			
k=F/Δ (k/in)	12.604		
$k=24EI/L^3$ (k/in)		<<< 4 Columns	
$I=KL^3/24E$ (in^4)	250.3448	<<<Governs	
Choose Member	W21x182		
Deflection			
δ3 (in)	1.035	<1.091	Ok

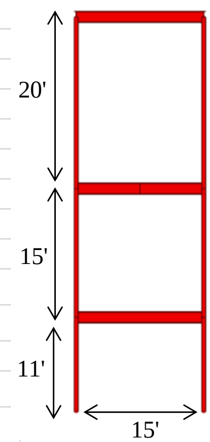




Tower 2 N/S – E/W lateral frame

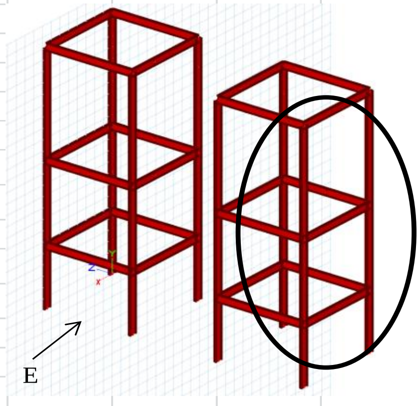
Tower 2			
N/S			
1st Story			
Allowable Story Drift (in)	3.3		
Story Height (ft)	11		
Story Height (in)	132		
$\Delta_{inelastic}$ (in)	3.3		
I_e	1		
Cd	5.5		
$\delta_{elastic}$ (in)	0.600	<<< Design Deflection	
F_1 (k)	26.87	6.7175	
$F=k\Delta$ (k)			
$k=F/\Delta$ (k/in)	11.20		
$k=6EI/L^3$ (k/in)		<<< 2 Columns	
$I=KL^3/6E$ (in^4)	147.99		
Choose Member	W12x210		
Deflection			
δ1 (in)	0.144	<.6	Ok
2nd Story			
Allowable Story Drift (in)	4.5		
Story Height (ft)	15		
Story Height (in)	180		
$\Delta_{inelastic}$ (in)	4.5		
I_e	1		
Cd	5.5		
$\delta_{elastic}$ (in)	0.818	<<< Design Deflection	
F_2 (k)	26.64	6.66	
$F=k\Delta$ (k)			
$k=F/\Delta$ (k/in)	8.140		
$k=6EI/L^3$ (k/in)		<<< 2 Columns	
$I=KL^3/6E$ (in^4)	272.83		
Choose Member	W12x210		
Deflection			
δ2 (in)	0.276	<.818	Ok
3rd Story			
Allowable Story Drift (in)	6		
Story Height (ft)	20		
Story Height (in)	240		
$\Delta_{inelastic}$ (in)	6		
I_e	1		
Cd	5.5		
$\delta_{elastic}$ (in)	1.091	<<< Design Deflection	
F_3 (k)	12.97	3.2425	
$F=k\Delta$ (k)			
$k=F/\Delta$ (k/in)	2.97		
$k=6EI/L^3$ (k/in)		<<< 2 Columns	
$I=KL^3/6E$ (in^4)	236.14	<<<Governs	
Choose Member	W12x210		
Deflection			
δ3 (in)	0.404	<1.091	Ok

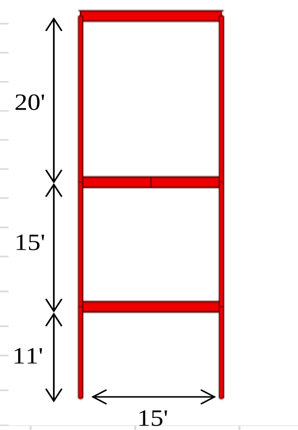




Tower 2 N/S lateral frame

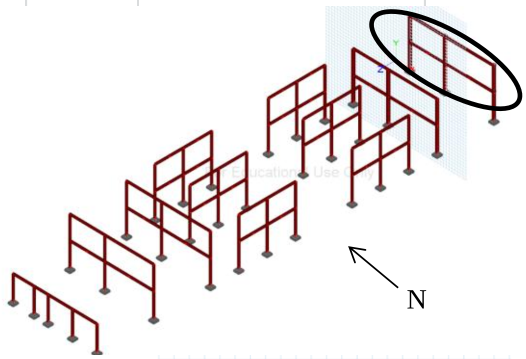
Tower 2			
E/W			
1st Story			
Allowable Story Drift (in)	3.3		
Story Height (ft)	11		
Story Height (in)	132		
$\Delta_{inelastic}$ (in)	3.3		
I_e	1		
Cd	5.5		
$\delta_{elastic}$ (in)	0.600	<<< Design Deflection	
F_1 (k)	96.87	48.435	
F=kΔ (k)			
k=F/Δ (k/in)	80.725		
$k=6EI/L^3$ (k/in)		<<< 2 Columns	
$I=KL^3/6E$ (in^4)	1067.04		
Choose Member	W12x210		
Deflection			
δ1 (in)	0.371	<.6	Ok
2nd Story			
Allowable Story Drift (in)	4.5		
Story Height (ft)	15		
Story Height (in)	180		
$\Delta_{inelastic}$ (in)	4.5		
I_e	1		
Cd	5.5		
$\delta_{elastic}$ (in)	0.818	<<< Design Deflection	
F_2 (k)	96.64	48.32	
F=kΔ (k)			
k=F/Δ (k/in)	59.058		
$k=6EI/L^3$ (k/in)		<<< 2 Columns	
$I=KL^3/6E$ (in^4)	1979.5		
Choose Member	W12x210		
Deflection			
δ2 (in)	0.708	<.818	Ok
3rd Story			
Allowable Story Drift (in)	6		
Story Height (ft)	20		
Story Height (in)	240		
$\Delta_{inelastic}$ (in)	6		
I_e	1		
Cd	5.5		
$\delta_{elastic}$ (in)	1.091	<<< Design Deflection	
F_3 (k)	12.97	6.485	
F=kΔ (k)			
k=F/Δ (k/in)	11.889		
$k=6EI/L^3$ (k/in)		<<< 2 Columns	
$I=KL^3/6E$ (in^4)	944.5738	<<<Governs	
Choose Member	W12x210		
Deflection			
δ3 (in)	0.792	<1.091	Ok

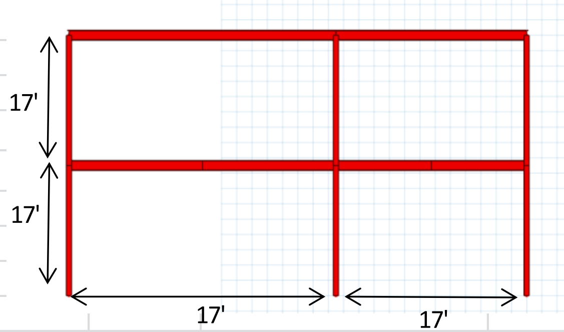




Tower 2 E/W lateral frame

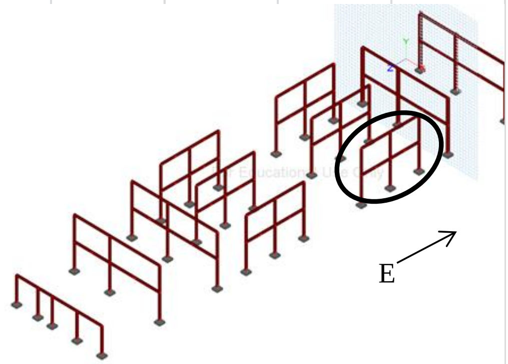
Building A			
N/S			
1st Story			
Allowable Story Drift (in)	5.1		
Story Height (ft)	17		
Story Height (in)	204		
$\Delta_{inelastic}$ (in)	5.1		
I_e	1		
Cd	5.5		
$\delta_{elastic}$ (in)	0.927	<<< Design Deflection	
F_1 (k)	103.6		
$F=k\Delta$ (k)			
$k=F/\Delta$ (k/in)	111.7255		
$k=18EI/L^3$ (k/in)		<<< 3 Columns	
$I=KL^3/18E$ (in^4)	1817.07		
Choose Member	W24x104		
Deflection			
δ1 (in)	0.8	<.927	Ok
2nd Story			
Allowable Story Drift (in)	5.1		
Story Height (ft)	17		
Story Height (in)	204		
$\Delta_{inelastic}$ (in)	5.1		
I_e	1		
Cd	5.5		
$\delta_{elastic}$ (in)	0.927	<<< Design Deflection	
F_2 (k)	56.11		
$F=k\Delta$ (k)			
$k=F/\Delta$ (k/in)	60.511		
$k=18EI/L^3$ (k/in)		<<< 3 Columns	
$I=KL^3/18E$ (in^4)	984.131		
Choose Member	W24x104		
Deflection			
δ2 (in)	0.85	<.927	Ok

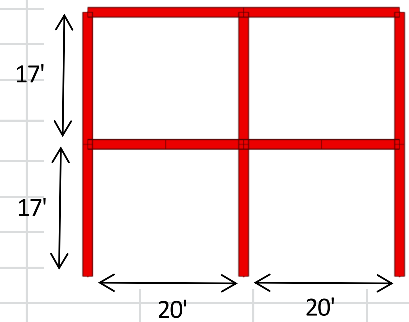




Building A N/S lateral frame

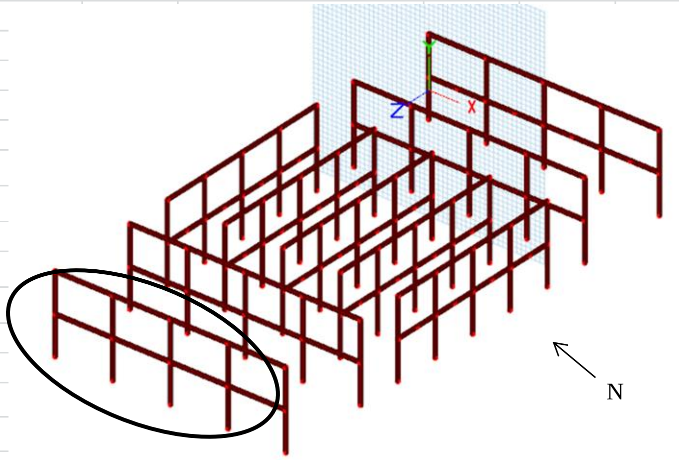
Building A			
E/W			
1st Story			
Allowable Story Drift (in)	5.1		
Story Height (ft)	17		
Story Height (in)	204		
$\Delta_{inelastic}$ (in)	5.1		
I_e	1		
Cd	5.5		
$\delta_{elastic}$ (in)	0.927	<<< Design Deflection	
F_1 (k)	76.98		
F=kΔ (k)			
k=F/Δ (k/in)	83.01765		
$k=18EI/L^3$ (k/in)		<<< 3 Columns	
$I=KL^3/18E$ (in^4)	1350.18		
Choose Member	W24x103		
Deflection			
δ1 (in)	0.501	<.927	Ok
2nd Story			
Allowable Story Drift (in)	5.1		
Story Height (ft)	17		
Story Height (in)	204		
$\Delta_{inelastic}$ (in)	5.1		
I_e	1		
Cd	5.5		
$\delta_{elastic}$ (in)	0.927	<<< Design Deflection	
F_2 (k)	37.4		
F=kΔ (k)			
k=F/Δ (k/in)	40.333		
$k=18EI/L^3$ (k/in)		<<< 3 Columns	
$I=KL^3/18E$ (in^4)	655.970		
Choose Member	W24x103		
Deflection			
δ2 (in)	0.421	<.927	Ok

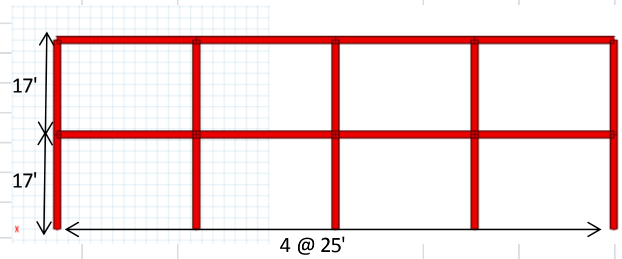




Building A E/W lateral frame

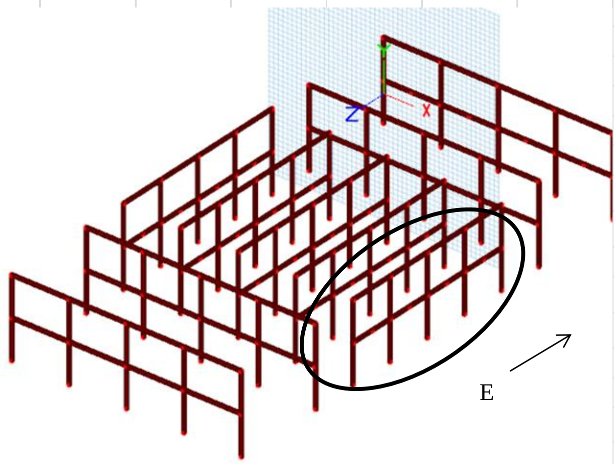
Building B			
N/S			
1st Story			
Allowable Story Drift (in)	5.1		
Story Height (ft)	17		
Story Height (in)	204		
$\Delta_{inelastic}$ (in)	5.1		
I_e	1		
Cd	5.5		
$\delta_{elastic}$ (in)	0.927	<<< Design Deflection	
F_1 (k)	100.43		
F=kΔ (k)			
k=F/Δ (k/in)	108.3069		
$k=30EI/L^3$ (k/in)		<<< 5 Columns	
$I=KL^3/30E$ (in^4)	1056.88		
Choose Member	W14x132		
Deflection			
δ1 (in)	0.745	<.927	Ok
2nd Story			
Allowable Story Drift (in)	5.1		
Story Height (ft)	17		
Story Height (in)	204		
$\Delta_{inelastic}$ (in)	5.1		
I_e	1		
Cd	5.5		
$\delta_{elastic}$ (in)	0.927	<<< Design Deflection	
F_2 (k)	48.41		
F=kΔ (k)			
k=F/Δ (k/in)	52.207		
$k=30EI/L^3$ (k/in)		<<< 4 Columns	
$I=KL^3/30E$ (in^4)	509.447		
Choose Member	W14x132		
Deflection			
δ2 (in)	0.537	<.927	Ok

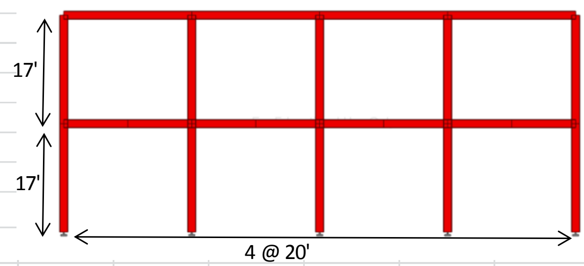




Building B N/S lateral frame

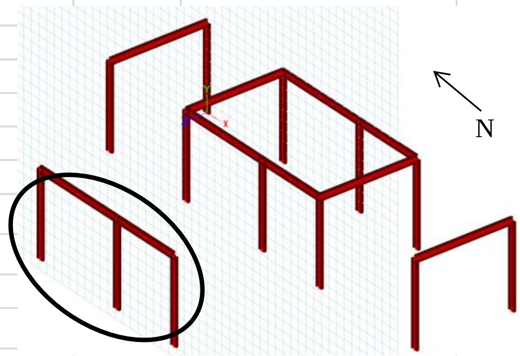
Building B			
E/W			
1st Story			
Allowable Story Drift (in)	5.1		
Story Height (ft)	17		
Story Height (in)	204		
$\Delta_{inelastic}$ (in)	5.1		
I_e	1		
Cd	5.5		
$\delta_{elastic}$ (in)	0.927	<<< Design Deflection	
F_1 (k)	80.35		
F=kΔ (k)			
k=F/Δ (k/in)	86.65196		
k=30EI/L^3 (k/in)		<<< 5 Columns	
I=KL^3/30E (in^4)	845.57		
Choose Member	W24x94		
Deflection			
δ1 (in)	0.78	<.927	Ok
2nd Story			
Allowable Story Drift (in)	5.1		
Story Height (ft)	17		
Story Height (in)	204		
$\Delta_{inelastic}$ (in)	5.1		
I_e	1		
Cd	5.5		
$\delta_{elastic}$ (in)	0.927	<<< Design Deflection	
F_2 (k)	38.73		
F=kΔ (k)			
k=F/Δ (k/in)	41.768		
k=30EI/L^3 (k/in)		<<< 4 Columns	
I=KL^3/30E (in^4)	407.578		
Choose Member	W24x94		
Deflection			
δ2 (in)	0.58	<.927	Ok

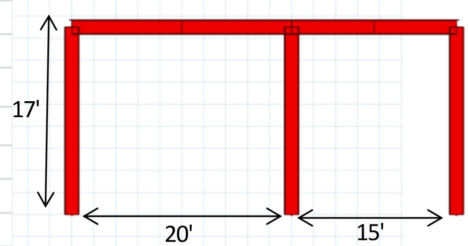




Building B E/W lateral frame

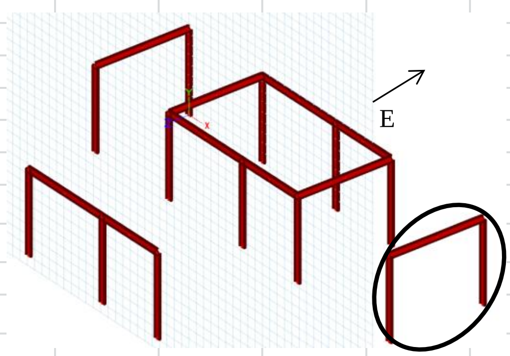
Building C			
N/S			
1st Story			
Allowable Story Drift (in)	5.1		
Story Height (ft)	17		
Story Height (in)	204		
$\Delta_{inelastic}$ (in)	5.1		
I_e	1		
Cd	5.5		
$\delta_{elastic}$ (in)	0.927	<<< Design Deflection	
F_1 (k)	52.09	17.36	
F=kΔ (k)			
k=F/Δ (k/in)	18.72516		
$k=12EI/L^3$ (k/in)		<<< 3 Columns	
$I=KL^3/12E$ (in^4)	456.81		
Choose Member	W12x87		
Deflection			
δ1 (in)	0.283	<.927	Ok

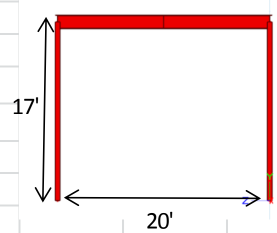




Building C N/S lateral frame

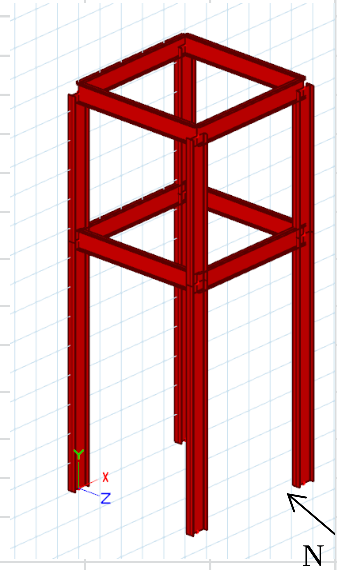
Building C			
E/W			
1st Story			
Allowable Story Drift (in)	5.1		
Story Height (ft)	17		
Story Height (in)	204		
$\Delta_{inelastic}$ (in)	5.1		
I_e	1		
Cd	5.5		
$\delta_{elastic}$ (in)	0.927	<<< Design Deflection	
F_1 (k)	52.09	13.02	
F=kΔ (k)			
k=F/Δ (k/in)	14.04387		
$k=6EI/L^3$ (k/in)		<<< 2 Columns	
$I=KL^3/6E$ (in^4)	685.22		
Choose Member	W12x87		
Deflection			
δ1 (in)	0.7	<.927	Ok

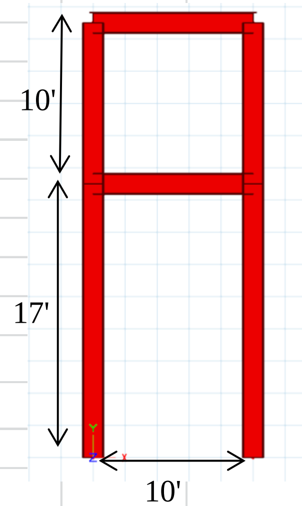




Building C E/W lateral frame

Elevator			
E/W = N/S			
1st Story			
Allowable Story Drift (in)	5.1		
Story Height (ft)	17		
Story Height (in)	204		
$\Delta_{inelastic}$ (in)	5.1		
I_e	1		
Cd	5.5		
$\delta_{elastic}$ (in)	0.927	<<< Design Deflection	
F_1 (k)	1.07	0.535	
F=kΔ (k)			
k=F/Δ (k/in)	1.153922		
$k=6EI/L^3$ (k/in)		<<< 2 Columns	
$I=KL^3/6E$ (in^4)	56.30119		
Choose Member	W12x22		
Deflection			
δ1 (in)	0.834	0.927	Ok
2nd Story			
Allowable Story Drift (in)	3		
Story Height (ft)	10		
Story Height (in)	120		
$\Delta_{inelastic}$ (in)	3		
I_e	1		
Cd	5.5		
$\Delta_{elastic}$ (in)	0.545	<<< Design Deflection	
F_2 (k)	0.8199	0.40995	
F=kΔ (k)			
k=F/Δ (k/in)	1.50		
$k=6EI/L^3$ (k/in)		<<< 2 Columns	
$I=KL^3/6E$ (in^4)	14.92783		
Choose Member	W12x22		
Deflection			
δ2 (in)	0.465	<.545	Ok



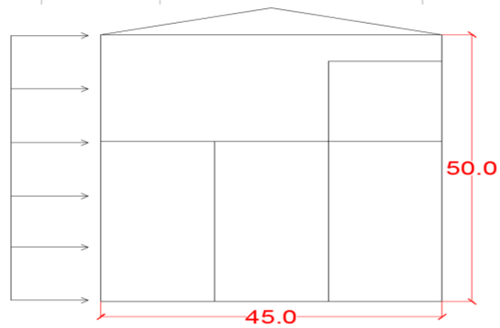


Elevator N/S – E/W lateral frame

APPENDIX D

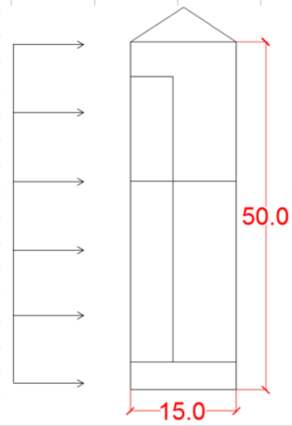
WIND CALCULATIONS

E/W = N/S			
Tower 1/3 Wind			
Risk Category II			
V (mph)	85		
Kd (Directionality Factor)	0.85		
Exposure	B		
Kzt (Topographic Effect)	1		
G (Gust Factor)	0.85		
Partially Enclosed			
GCpi	0.55		
	-0.55		
Kz (Velocity Pressure Coefficient)	0.814		
qz @ 52' (Velocity Pressure)	12.797		
qh=qz (Partially Enclosed)			



>Total pressure = external pressure - internal pressure			
>p=qGCp - qi(GCpi)			
>Classify as Rigid (Low Rise, ASCE Ch. 26.2)			
>Gable Roof Structure			
Wall Pressure Coefficents			
Cp (Windward)	0.8	<<<<<	Figure 27.4-1
L	45		
B	45		
L/B	1.00		
Cp (Windward)	0.8	<<<<<	Figure 27.4-1
Cp (Leeward)	-0.500	<<<<<	Figure 27.4-1
Cp (Side)	-0.7	<<<<<	Figure 27.4-1
q = qz			
Windward Pressure			
P (psf) (Pos. GCpi)	1.66		
P (psf) (Neg. GCpi)	15.74	<< governs	
Leeward Pressure			
P (psf) (Pos. GCpi)	-12.48	<< governs	
P (psf) (Neg. GCpi)	1.60		
Sidewall Pressure			
P (psf) (Pos. GCpi)	-14.653	<< governs	
P (psf) (Neg. GCpi)	-0.576		
Roof Pressure Coefficients		Assume (15) Degrees	
h	52		
l	45		
h/l	1.16		
Interpolate			
Cp1 (Windward)	-1		
Cp2 (Windward)	-0.18		
Cp (Leeward)	-0.6		
Windward Roof Pressure			
P (psf) (Pos. GCpi)	-17.92	<<governs	
P (psf) (Neg. GCpi)	-3.84		
Leeward Roof Pressure			
P (psf) (Pos. GCpi)	-13.57	<<governs	
P (psf) (Neg. GCpi)	0.51		
16 PSF MINIMUM!!			
Net Pressure	28.22		

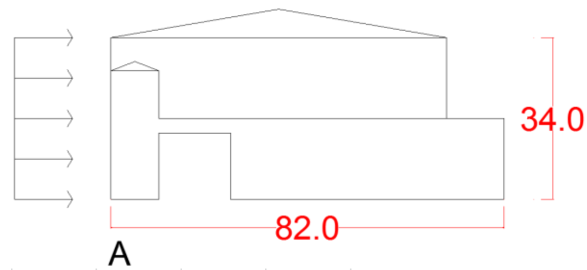
Towers 1 and 3 wind calculations



E/W		
Tower 2 Wind		
Risk Category II		
V (mph)	85	
Kd (Directionality Factor)	0.85	
Exposure	B	
Kzt (Topographic Effect)	1	
G (Gust Factor)	0.85	
Partially Enclosed		
GCpi	0.55	
	-0.55	
Kz (Velocity Pressure Coefficient)	0.814	
qz @ 51' (Velocity Pressure)	12.797	
qh=qz (Partially Enclosed)		
>Total pressure = external pressure - internal pressure		
>p=qGCp - qi(GCpi)		
>Classify as Rigid (Low Rise, ASCE Ch. 26.2)		
>Gable Roof Structure		
Wall Pressure Coefficents		
Cp (Windward)	0.8	<<<<< Figure 27.4-1
L	15	
B	45	
L/B	0.33	
Cp (Windward)	0.8	<<<<< Figure 27.4-1
Cp (Leeward)	-0.500	<<<<< Figure 27.4-1
Cp (Side)	-0.7	<<<<< Figure 27.4-1
q = qz		
Windward Pressure		
P (psf) (Pos. GCpi)	1.66	
P (psf) (Neg. GCpi)	15.74	<< governs
Leeward Pressure		
P (psf) (Pos. GCpi)	-12.48	<< governs
P (psf) (Neg. GCpi)	1.60	
Sidewall Pressure		
P (psf) (Pos. GCpi)	-14.653	<< governs
P (psf) (Neg. GCpi)	-0.576	
Roof Pressure Coefficients		Assume (15) Degrees
h	51	
l	15	
h/l	3.40	
Interpolate		
Cp1 (Windward)	-1	
Cp2 (Windward)	-0.18	
Cp (Leeward)	-0.6	
Windward Roof Pressure		
P (psf) (Pos. GCpi)	-17.92	<<governs
P (psf) (Neg. GCpi)	-3.84	
Leeward Roof Pressure		
P (psf) (Pos. GCpi)	-13.57	<<governs
P (psf) (Neg. GCpi)	0.51	
16 PSF MINIMUM!!		
Net Pressure	28.22	

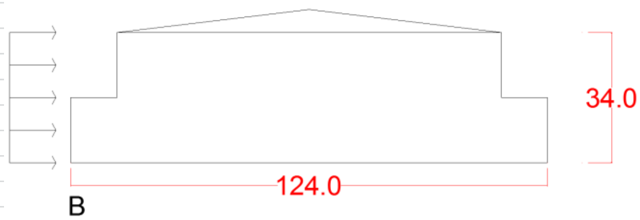
Tower 2 wind calculations

N/S			
Building A Wind			
Risk Category II			
V (mph)	85		
Kd (Directionality Factor)	0.85	<<<<<	Figure 26.6-1
Exposure	B	<<<<<	Jurisdiction
Kzt (Topographic Effect)	1	<<<<<	Figure 26.8-1
G (Gust Factor)	0.85		
Fully Enclosed			
GCpi	0.18	<<<<<	Figure 26.11-1
	-0.18		
Kz (Velocity Pressure Coefficient)	0.742	<<<<<	Figure 27.3-1
Kh (Velocity Pressure Coefficient)	0.724	<<<<<	Figure 27.3-1
qz @ 37' (Velocity Pressure)	11.665		
qh @ 34' (Velocity Pressure)	11.382		
Wall Pressure Coefficents			
Cp (Windward)	0.8	<<<<<	Figure 27.4-1
L	82		
B	240		
L/B	0.34		
Cp (Windward)	0.8	<<<<<	Figure 27.4-1
Cp (Leeward)	-0.500	<<<<<	Figure 27.4-1
Cp (Side)	-0.7	<<<<<	Figure 27.4-1
q = qz			
Windward Pressure			
P (psf) (Pos. GCpi)	5.88		
P (psf) (Neg. GCpi)	9.98		
Leeward Pressure			
P (psf) (Pos. GCpi)	-6.89		
P (psf) (Neg. GCpi)	-2.79		
Sidewall Pressure			
P (psf) (Pos. GCpi)	-8.821		
P (psf) (Neg. GCpi)	-4.724		
Roof Pressure Coefficients		Assume (10) Degrees	
h	37		
l	82		
h/l	0.45		
Interpolate			
Cp1 (Windward)	-0.86		
Cp2 (Windward)	-0.18		
Cp (Leeward)	-0.46		
Windward Roof Pressure			
P (psf) (Pos. GCpi)	-10.58		
P (psf) (Neg. GCpi)	-6.48		
Leeward Roof Pressure			
P (psf) (Pos. GCpi)	-6.61		
P (psf) (Neg. GCpi)	-2.51		
16 PSF MINIMUM!!			
Net Pressure	16.87		



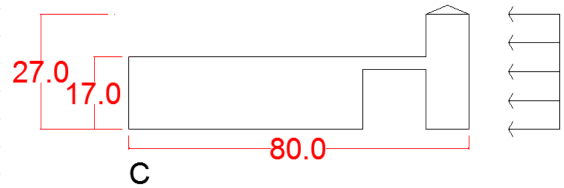
Commercial building A wind calculations

N/S			
Building A Wind			
Risk Category II			
V (mph)	85		
Kd (Directionality Factor)	0.85	<<<<<	Figure 26.6-1
Exposure	B		
Kzt (Topographic Effect)	1	<<<<<	Figure 26.8-1
G (Gust Factor)	0.85		
Fully Enclosed			
GCpi	0.18	<<<<<	Figure 26.11-1
	-0.18		
Kz	0.742	<<<<<	Figure 27.3-1
Kh	0.724	<<<<<	Figure 27.3-1
qz @37' (Velocity Pressure)	11.665		
qh @ 34 ' (Velocity Pressure)	11.382		
Wall Pressure Coefficents			
Cp (Windward)	0.8	<<<<<	Figure 27.4-1
L	82		
B	240		
L/B	0.34		
Cp (Windward)	0.8	<<<<<	Figure 27.4-1
Cp (Leeward)	-0.500	<<<<<	Figure 27.4-1
Cp (Side)	-0.7	<<<<<	Figure 27.4-1
q = qz			
Windward Pressure			
P (psf) (Pos. GCpi)	5.88		
P (psf) (Neg. GCpi)	9.98	<< governs	
Leeward Pressure			
P (psf) (Pos. GCpi)	-6.89	<< governs	
P (psf) (Neg. GCpi)	-2.79		
Sidewall Pressure			
P (psf) (Pos. GCpi)	-8.821	<< governs	
P (psf) (Neg. GCpi)	-4.724		
Roof Pressure Coefficients		Assume (10) Degrees	
h	37		
l	124		
h/l	0.30		
Interpolate			
Cp1 (Windward)	-0.74		
Cp2 (Windward)	-0.18		
Cp (Leeward)	-0.34	<<interpolate	
Windward Roof Pressure			
P (psf) (Pos. GCpi)	-9.39	<<governs	
P (psf) (Neg. GCpi)	-5.29		
Leeward Roof Pressure			
P (psf) (Pos. GCpi)	-5.42	<<governs	
P (psf) (Neg. GCpi)	-1.32		
16 PSF MINIMUM!!			
Net Pressure (psf)	16.87		



Commercial building B wind calculations

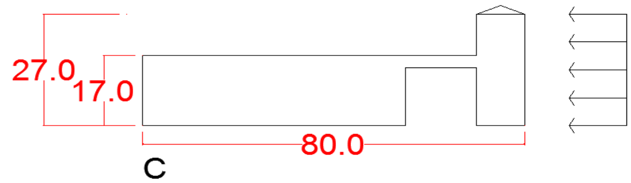
E/W			
Building C Wind			
Risk Category II			
V (mph)	85		
Kd (Directionality Factor)	0.85	<<<<<	Figure 26.6-1
Exposure	B		
Kzt (Topographic Effect)	1	<<<<<	Figure 26.8-1
G (Gust Factor)	0.85		
Fully Enclosed			
GCpi	0.18	<<<<<	Figure 26.11-1
	-0.18		
Kz (Velocity Pressure Coefficient)	0.59	<<<<<	Figure 27.3-1
Kh (Velocity Pressure Coefficient)	0.59	<<<<<	Figure 27.3-1
qh @ 17' (Velocity Pressure)	9.276		
qh=qz			
>Total pressure = external pressure - internal pressure			
>p=qGCp - qi(GCpi)			
>*No North/South Wind			
>Rigid (Low Rise, ASCE Ch. 26.2)			
>Gable Roof Structure			
Wall Pressure Coefficents			
Cp (Windward)	0.8	<<<<<	Figure 27.4-1
L	50		
B	80		
L/B	0.63		
Cp (Windward)	0.8	<<<<<	Figure 27.4-1
Cp (Leeward)	-0.500	<<<<<	Figure 27.4-1
Cp (Side)	-0.7	<<<<<	Figure 27.4-1
q = qz			
Windward Pressure			
P (psf) (Pos. GCpi)	4.64		
P (psf) (Neg. GCpi)	7.98	<< governs	
Leeward Pressure			
P (psf) (Pos. GCpi)	-5.61	<< governs	
P (psf) (Neg. GCpi)	-2.27		
Sidewall Pressure			
P (psf) (Pos. GCpi)	-7.189	<< governs	
P (psf) (Neg. GCpi)	-3.849		
16 PSF MINIMUM!!			
Net Pressure (psf)	13.59		



* Elevator Wind Governs

* Use Elevator Pressure for Design. Refer to Elevator Wind Design

Commercial building C wind calculations



N/S			
Elevators			
Risk Category II			
V (mph)	85		
Kd (Directionality Factor)	0.85		
Exposure	B		
Kzt (Topographic Effect)	1		
G (Gust Factor)	0.85		
Fully Enclosed			
GCpi	0.18		
	-0.18		
Kz (Velocity Pressure Coefficient)	0.676		
qz @ 27' (Velocity Pressure)	10.628		
qh @ 27			
>Total pressure = external pressure - internal pressure			
>p=qGCp - qi(GCpi)			
>Classify as Rigid (Low Rise, ASCE Ch. 26.2)			
>Gable Roof Structure			
Wall Pressure Coefficents			
Cp (Windward)	0.8	<<<<<	Figure 27.4-1
L	10		
B	10		
L/B	1.00		
Cp (Windward)	0.8	<<<<<	Figure 27.4-1
Cp (Leeward)	-0.500	<<<<<	Figure 27.4-1
Cp (Side)	-0.7	<<<<<	Figure 27.4-1
q = qz			
Windward Pressure			
P (psf) (Pos. GCpi)	5.31		
P (psf) (Neg. GCpi)	9.14	<< governs	
Leeward Pressure			
P (psf) (Pos. GCpi)	-6.43	<< governs	
P (psf) (Neg. GCpi)	-2.60		
Sidewall Pressure			
P (psf) (Pos. GCpi)	-8.237	<< governs	
P (psf) (Neg. GCpi)	-4.411		
Roof Pressure Coefficients		Assume (15) Degrees	
h	27		
l	10		
h/l	2.70		
Interpolate			
Cp1 (Windward)	-1		
Cp2 (Windward)	-0.18		
Cp (Leeward)	-0.6		
Windward Roof Pressure			
P (psf) (Pos. GCpi)	-10.95	<<governs	
P (psf) (Neg. GCpi)	-7.12		
Leeward Roof Pressure			
P (psf) (Pos. GCpi)	-7.33	<<governs	
P (psf) (Neg. GCpi)	-3.51		
16 PSF MINIMUM!!			
Net Pressure	15.57		

Elevator Wind Calculations

APPENDIX E

SEISMIC CALCULATIONS

Seismic Design

Building: Tower 1 and 3

S_s	1.5
S_1	0.6
Site Class	D
F_a	1
F_v	1.5
S_{Ms}	1.5
S_{M1}	0.9
S_{Ds}	1
S_{D1}	0.6
Risk Category	II
Importance Factor	1
Building Specific: R	8
Building Specific: Ω	3
Cd	5.5
Weight (lb)	311960
C_s	0.125
C_t	0.028
x	0.8
h_n (ft)	50
$T=C_t h_n^x$	0.64

Cs			
0.125	Cs not exceed	0.12	T<TL
	Cs Shall be greater than	0.01	

Where S1 is greater than or equal to 0.6	
Cs	
0.125	Cs shall not be less than	0.0375

Base Shear

C_s	0.12
W	311960
V (lb)	36544.85
V(kips)	36.54

Level	Level Weight wx (kip	hx (ft)	K	$w_x h_x^k$	C_{vx}	Fx (kips)
Roof	146.125	50	2	365312.5	0.742974159	27.15
2	132.03	30	2	118827	0.241670872	8.83
1	33.555	15	2	7549.875	0.015354969	0.56

Towers 1 and 3 Seismic Calculations

Building:	**Tower 2**

S_s	1.5
S_1	0.6
Site Class	D
F_a	1
F_v	1.5
S_{Ms}	1.5
S_{M1}	0.9
S_{Ds}	1
S_{D1}	0.6
Risk Category	II
Importance Factor	1
Building Specific — R	8
Ω	3
Cd	5.5
Weight (lb)	214605
C_s	0.125
C_t	0.028
x	0.8
h_n (ft)	46
$T=C_t h_n^x$	0.60

Cs

0.125	Cs not exceed	0.13	$T<T_L$
	Cs Shall be greater than	0.01	

Where S1 is greater than or equal to 0.6		
Cs		
0.125	Cs shall not be less than	0.0375

Base Shear

C_s	0.13
W	214605
V (lb)	26874.28
V(kips)	26.87

Level	Level Weight wx (k	hx (ft)	K	$w_x h_x^k$	C_{vx}	Fx (kips)	
Roof	34.875	46	2	73795.5	0.482656878	12.97	
2	115	26	2	77740	0.508455742	13.66	48.66
1	11.23	11	2	1358.83	0.00888738	0.24	

Note: 35kips added to level 2 due to bridge bearing connection bringing total force at level 2 (48.66kips)

Tower 2 Seismic Calculations

Building: Commercial A

	S_s	1.5
	S_1	0.6
	Site Class	D
	F_a	1
	F_v	1.5
	S_{Ms}	1.5
	S_{M1}	0.9
	S_{Ds}	1
	S_{D1}	0.6
	Risk Category	II
	Importance Factor	1
Building Specific	R	8
	Ω	3
	Cd	5.5
	Weight (lb)	2896140
	C_s	0.125
	C_t	0.028
	x	0.8
	h_n (ft)	34
	$T=C_t h_n^x$	0.47

Cs

0.125	Cs not exceed		0.16	T<TL
	Cs Shall be greater than		0.01	

Where S1 is greater than or equal to 0.6		
Cs		
0.125	Cs shall not be less than	0.0375

Base Shear

C_s	0.16
W	2896140
V (lb)	461891.16
V(kips)	461.89

Level	vel Weight wx (kip	hx (ft)	K	$w_x h_x^k$	C_{vx}	Fx (kips)	Per Frame	Total	Per Node
							Visual Analysis		
Roof	510.36	34	2	589976.16	0.49	224.45	56.11		18.70
1	2159.60	17	2	624124.40	0.51	237.44	47.49	103.60	34.53

Commerical Building A Seismic Calculations

Building: Commercial B

S_s	1.5
S_1	0.6
Site Class	D
F_a	1
F_v	1.5
S_{Ms}	1.5
S_{M1}	0.9
S_{Ds}	1
S_{D1}	0.6
Risk Category	II
Importance Factor	1

Building Specific	R	8
	Ω	3
	Cd	5.5

Weight (lb)	3208200
C_s	0.125
C_t	0.028
x	0.8
h_n (ft)	46
$T=C_t h_n^x$	0.60

Cs

0.125	Cs not exceed		0.13
	Cs Shall be greater than	0.01	

Where S1 is greater than or equal to 0.6		
Cs		
0.125	Cs shall not be less than	0.0375

Base Shear

C_s	0.13
W	3208200
V (lb)	401752.33
V(kips)	401.75

							Visual Analysis		
Level	vel Weight wx (kip	hx (ft)	K	$w_x h_x^k$	C_{vx}	Fx (kips)	Per frame	Total	Per node
Roof	561	34	2	648516.00	0.48	193.68	38.74		7.75
1	2410.8	17	2	696721.20	0.52	208.07	41.61	80.35	16.07

Commercial Building B Seismic Calculations

Building:	**Commercial C**

	S_s	1.5
	S_1	0.6
	Site Class	D
	F_a	1
	F_v	1.5
	S_{Ms}	1.5
	S_{M1}	0.9
	S_{Ds}	1
	S_{D1}	0.6
	Risk Category	II
	Importance Factor	1
Building Specific	R	8
	Ω	3
	Cd	5.5
	Weight (lb)	416000
	C_s	0.125
	C_t	0.028
	x	0.8
	h_n (ft)	46
	$T=C_t h_n^x$	0.60

Cs			
0.125	Cs not exceed		0.13
	Cs Shall be greater than		0.01

Where S1 is greater than or equal to 0.6		
Cs		
0.125	Cs shall not be less than	0.0375

Base Shear

C_s	0.13
W	416000
V (lb)	52094.31
V(kips)	52.09

Level	vel Weight wx (kip	hx (ft)	K	$w_x h_x^k$	C_{vx}	Fx (kips)
Roof	390.5	17	2	112854.5	1	52.09

Commercial Building C Seismic Calculations

Elevators @ commercial buildings

	S_s	1.5
	S_1	0.6
	Site Class	D
	F_a	1
	F_v	1.5
	S_{Ms}	1.5
	S_{M1}	0.9
	S_{Ds}	1
	S_{D1}	0.6
	Risk Category	II
	Importance Factor	1
Building Specific	R	8
	Ω	3
	Cd	5.5
	Weight (lb)	11640
	C_s	0.125
	C_t	0.028
	x	0.8
	h_n (ft)	46
	$T=C_t h_n^x$	0.60

Cs

0.125	Cs not exceed		0.13
	Cs Shall be greater than		0.01

Where S1 is greater than or equal to 0.6		
Cs		
0.125	Cs shall not be less than	0.0375

Base Shear

C_s	0.13
W	11640
V (lb)	1457.64
V(kips)	1.46

Level	vel Weight wx (kip	hx (ft)	K	$w_x h_x^k$	C_{vx}	Fx (kips)	Per fram	Shear
							Visual Analysis	
Roof	8.36	27	2	6094.44	0.865397395	1.26	0.630718	0.819933984
1	3.28	17	2	947.92	0.134602605	0.20	0.82692	1.074996573

Commercial Building Elevator Seismic Calculations

APPENDIX F

BANNAN ENGINEERING BORING LOGS

Project: UNIVERSITY OF SANTA CLARA ENGINEERING BUILDING

Log of Boring No. 3

Date Drilled: Oct. 14, 1983

Type of Boring: 6" Auger

Hammer Weight: 140 Lbs.

Remarks: See Figure 4A for Sampler Type

Surface Elevation:

Depth, Ft.	Samples	Blows/Ft.	MATERIAL DESCRIPTION	LABORATORY TESTS: Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf
			CLAYEY SILT FILL (ML) Poorly to moderately compacted, moist, brown, with some gravel			
	1	6	Becoming light brown Gravelly Clay (CL)	—	—	—
	2	14	Becoming black Silty Clay, with traces of sand and gravel	24	94	—
5	3	12	(Fill)	22	102	2680
			CLAYEY SILT (ML) Stiff, moist, light gray			
10	4	22	**SAND (SP–SM)** Medium dense, moist, brown, fine to very fine	—	—	—
			▽ Water at Time of Drilling			
			SILTY CLAY (CL–CH) Medium stiff, brown-gray			
15	5	6	Changing to Clayey Silt	40	80	1070
			SILTY CLAY (CL–CH) Stiff, greenish-gray			
			SAND (SP–SM) Loose, brown, very fine to fine, traces of gravel			
20	6	9		—	—	—

Proj. No. 16086V

Woodward·Clyde Consultants

Project: UNIVERSITY OF SANTA CLARA ENGINEERING BUILDING

Log of Boring No. 3

(Continued)

Depth, Ft.	Samples	Blows/Ft.	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf
25			SILTY CLAY (CL) Medium stiff, mottled greenish-gray and brown			
	7	10		29	95	1170
30						
	8	12		23	102	1660
			Bottom of Boring at 31.5 feet			
35						
40						
45						

LEGEND FOR SAMPLER TYPE

2-Inch Modified California Sampler

Proj. No. 16086V | **Woodward-Clyde Consultants** | Figure 4A

APPENDIX G

VULCRAFT TRUSS SPECIFICATIONS

LRFD

STANDARD LOAD TABLE FOR OPEN WEB STEEL JOISTS, K-SERIES
Based On A 50 ksi Maximum Yield Strength - Loads Shown In Pounds Per Linear Foot (plf)

Joist Designation	10K1	12K1	12K3	12K5	14K1	14K3	14K4	14K6	16K2	16K3	16K4	16K5	16K6	16K7	16K9
Depth (in.)	10	12	12	12	14	14	14	14	16	16	16	16	16	16	16
Approx. Wt (lbs./ft.)	5.0	5.0	5.7	7.1	5.2	6.0	6.7	7.7	5.5	6.3	7.0	7.5	8.1	8.6	10.0
Span (ft.) ↓															
10	825 550														
11	825 542														
12	825 455	825 550	825 550	825 550											
13	718 363	825 510	825 510	825 510											
14	618 289	750 425	825 463	825 463	825 550	825 550	825 550	825 550							
15	537 234	651 344	814 428	825 434	766 475	825 507	825 507	825 507							
16	469 192	570 282	714 351	825 396	672 390	825 467	825 467	825 467	825 550	825 550	825 550	825 550	825 550	825 550	825 550
17	415 159	504 234	630 291	825 366	592 324	742 404	825 443	825 443	768 488	825 526	825 526	825 526	825 526	825 526	825 526
18	369 134	448 197	561 245	760 317	528 272	661 339	795 397	825 408	684 409	762 456	825 490	825 490	825 490	825 490	825 490
19	331 113	402 167	502 207	681 269	472 230	592 287	712 336	825 383	612 347	682 386	820 452	825 455	825 455	825 455	825 455
20	298 97	361 142	453 177	613 230	426 197	534 246	642 287	787 347	552 297	615 330	739 386	825 426	825 426	825 426	825 426
21		327 123	409 153	555 198	385 170	483 212	582 248	712 299	499 255	556 285	670 333	754 373	822 405	825 406	825 406
22		298 106	373 132	505 172	351 147	439 184	529 215	648 259	454 222	505 247	609 289	687 323	747 351	825 385	825 385
23		271 93	340 116	462 150	321 128	402 160	483 188	592 226	415 194	462 216	556 252	627 282	682 307	760 339	825 363
24		249 81	312 101	423 132	294 113	367 141	442 165	543 199	381 170	424 189	510 221	576 248	627 269	697 298	825 346
25					270 100	339 124	408 145	501 175	351 150	390 167	469 195	529 219	576 238	642 263	771 311
26					249 88	313 110	376 129	462 156	324 133	360 148	433 173	489 194	532 211	592 233	711 276
27					231 79	289 98	349 115	427 139	300 119	334 132	402 155	453 173	493 188	549 208	658 246
28					214 70	270 88	324 103	397 124	279 106	310 118	373 138	421 155	459 168	510 186	612 220
29									259 95	289 106	348 124	391 139	427 151	475 167	570 198
30									241 86	270 96	324 112	366 126	399 137	444 151	532 178
31									226 78	252 87	304 101	342 114	373 124	415 137	498 161
32									213 71	237 79	285 92	321 103	349 112	388 124	466 147

LRFD

STANDARD LOAD TABLE FOR OPEN WEB STEEL JOISTS, K-SERIES
Based On A 50 ksi Maximum Yield Strength - Loads Shown In Pounds Per Linear Foot (plf)

Joist Designation	18K3	18K4	18K5	18K6	18K7	18K9	18K10	20K3	20K4	20K5	20K6	20K7	20K9	20K10	22K4	22K5	22K6	22K7	22K9	22K10	22K11
Depth (In.)	18	18	18	18	18	18	18	20	20	20	20	20	20	20	22	22	22	22	22	22	22
Approx. Wt. (lbs./ft.)	6.4	7.2	7.7	8.4	8.9	10.1	11.6	6.5	7.2	7.7	8.4	8.9	10.1	11.6	7.3	7.7	8.5	9.0	10.2	11.7	11.9
Span (ft.) ↓																					
18	825 550	825 550	825 550	825 550	825 550	825 550	825 550														
19	771 494	825 523	825 523	825 523	825 523	825 523	825 523	825 550	825 550	825 550	825 550	825 550	825 550	825 550							
20	694 423	825 490	825 490	825 490	825 490	825 490	825 490	775 517	825 550	825 550	825 550	825 550	825 550	825 550							
21	630 364	759 426	825 460	825 460	825 460	825 460	825 460	702 453	825 520	825 520	825 520	825 520	825 520	825 520	825 550	825 550	825 550	825 550	825 550	825 550	825 550
22	573 316	690 370	777 414	825 438	825 438	825 438	825 438	639 393	771 461	825 490	825 490	825 490	825 490	825 490	825 548	825 548	825 548	825 548	825 548	825 548	825 548
23	523 276	630 323	709 362	774 393	825 418	825 418	825 418	583 344	703 402	793 451	825 468	825 468	825 468	825 468	777 491	825 518	825 518	825 518	825 518	825 518	825 518
24	480 242	577 284	651 318	709 345	789 382	825 396	825 396	535 302	645 353	727 396	792 430	825 448	825 448	825 448	712 431	804 483	825 495	825 495	825 495	825 495	825 495
25	441 214	532 250	600 281	652 305	727 337	825 377	825 377	493 266	594 312	669 350	729 380	811 421	825 426	825 426	657 381	739 427	805 464	825 474	825 474	825 474	825 474
26	408 190	492 222	553 249	603 271	672 299	807 354	825 361	456 236	549 277	618 310	673 337	750 373	825 405	825 405	606 338	682 379	744 411	825 454	825 454	825 454	825 454
27	378 169	454 198	513 222	558 241	622 267	747 315	825 347	421 211	508 247	573 277	624 301	694 333	825 389	825 389	561 301	633 337	688 367	768 406	825 432	825 432	825 432
28	351 151	423 177	477 199	519 216	577 239	694 282	822 331	391 189	472 221	532 248	579 269	645 298	775 353	825 375	522 270	588 302	640 328	712 364	825 413	825 413	825 413
29	327 136	394 159	444 179	483 194	538 215	646 254	766 298	364 170	439 199	495 223	540 242	601 268	723 317	825 359	486 242	547 272	597 295	664 327	798 387	825 399	825 399
30	304 123	367 144	414 161	451 175	502 194	603 229	715 269	340 153	411 179	462 201	504 218	561 242	675 286	799 336	453 219	511 245	556 266	619 295	745 349	825 385	825 385
31	285 111	343 130	387 146	421 158	469 175	564 207	669 243	318 138	384 162	433 182	471 198	525 219	631 259	748 304	424 198	478 222	520 241	580 267	697 316	825 369	825 369
32	267 101	322 118	363 132	396 144	441 159	529 188	627 221	298 126	360 147	406 165	442 179	492 199	592 235	702 276	397 180	448 201	489 219	544 242	654 287	775 337	823 355
33	252 92	303 108	342 121	372 131	414 145	498 171	589 201	280 114	339 134	381 150	415 163	463 181	556 214	660 251	373 164	421 183	459 199	511 221	615 261	729 307	798 334
34	237 84	285 98	321 110	349 120	390 132	468 156	555 184	264 105	318 122	358 137	391 149	435 165	523 195	621 229	352 149	397 167	432 182	481 202	579 239	687 280	774 314
35	223 77	268 90	303 101	330 110	367 121	441 143	523 168	249 96	300 112	339 126	369 137	411 151	493 179	585 210	331 137	373 153	408 167	454 185	546 219	648 257	741 292
36	211 70	253 82	286 92	312 101	348 111	417 132	495 154	235 88	283 103	319 115	348 125	388 139	466 164	553 193	313 126	354 141	385 153	429 169	516 201	612 236	700 269
37								222 81	268 95	303 106	330 115	367 128	441 151	523 178	297 116	334 130	364 141	406 156	487 185	579 217	663 247
38								211 74	255 87	286 98	312 106	348 118	418 139	496 164	280 107	316 119	345 130	384 144	462 170	549 200	628 228
39								199 69	241 81	271 90	297 98	330 109	397 129	471 151	267 98	300 110	327 120	364 133	438 157	520 185	595 211
40								190 64	229 75	258 84	282 91	313 101	376 119	447 140	253 91	285 102	310 111	346 123	417 146	495 171	565 195
41															241 85	271 95	295 103	330 114	396 135	471 159	538 181
42															229 79	259 88	282 96	313 106	378 126	448 148	513 168
43															219 73	247 82	268 89	300 99	360 117	427 138	489 157
44															208 68	235 76	256 83	286 92	343 109	408 128	466 146

APPENDIX H

COST TABLES

Slab On Metal Deck			
Building	**Area**	**CY**	**Price**
1	1650.00	23.81	$4,761.57
2	675.00	9.74	$1,947.92
3	1650.00	23.81	$4,761.57
A	22960.00	331.29	$66,258.02
B	26660.00	384.68	$76,935.49
C	4000.00	57.72	$11,543.21
		Total Price	$166,207.79

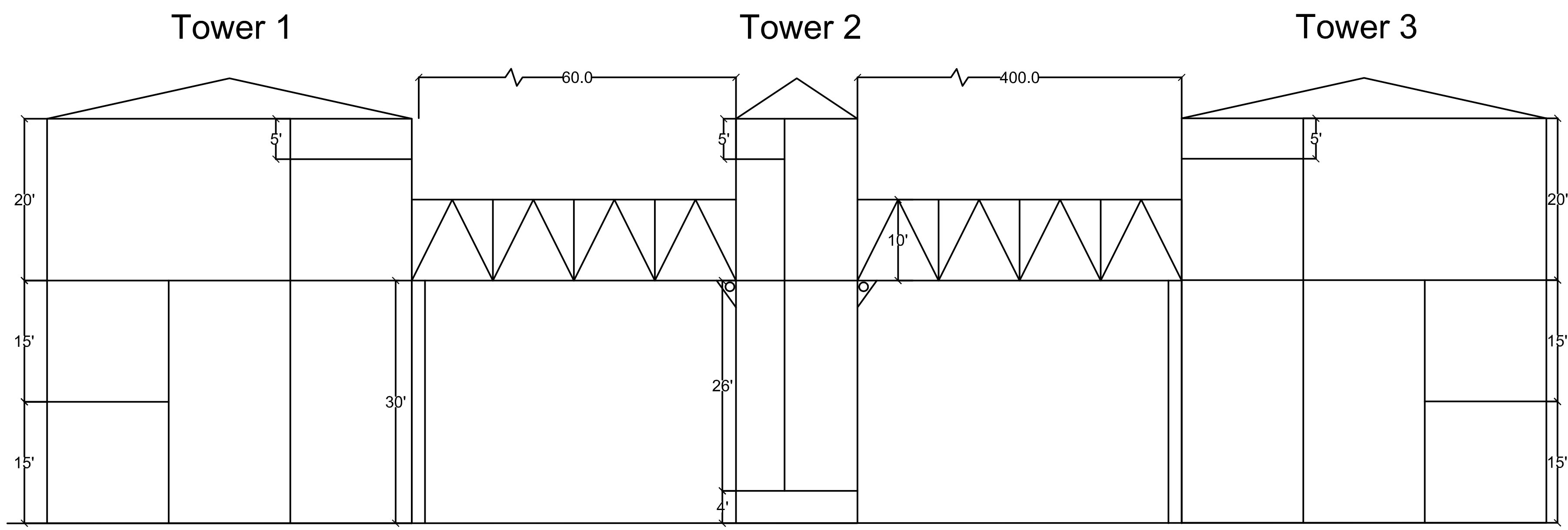
Slab On Grade			
Building	**Area**	**CY**	**Price**
1	2025.00	37.50	$5,625.00
2	675.00	12.50	$1,875.00
3	2025.00	37.50	$5,625.00
A	22960.00	425.19	$63,777.78
B	26660.00	493.70	$74,055.56
C	4000.00	74.07	$11,111.11
		Total Price	$162,069.44

Steel Pricing		
Building	**Weight of steel(Kips)**	**Price**
1	129.34	$226,336.25
2	70.74	$123,786.25
3	129.34	$226,336.25
A	639.34	$1,118,845.00
B	575.40	$1,006,950.00
C	96.00	$168,000.00
	Total Price	$2,870,253.75

APPENDIX I

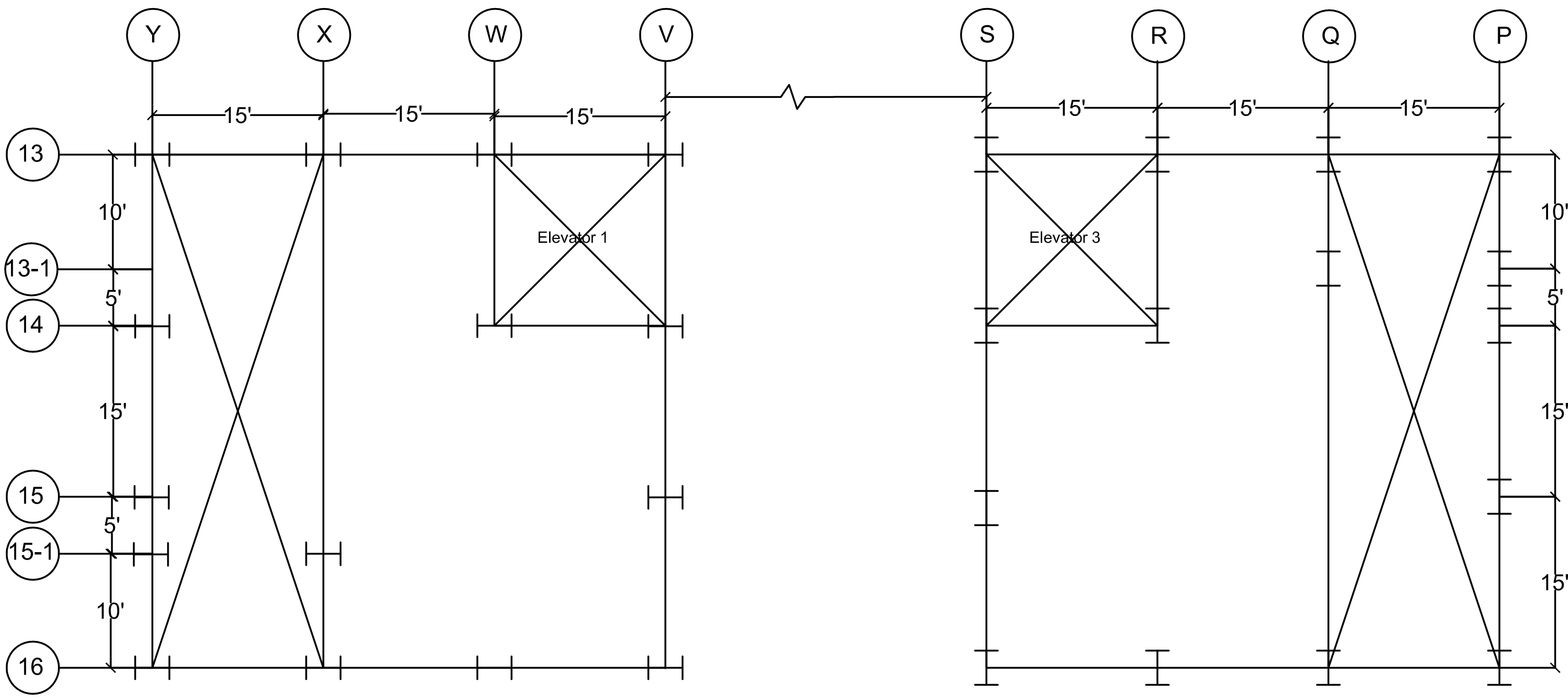
DETAILED DESIGN DRAWINGS

Santa Clara University	
DEPARTMENT OF CIVIL ENGINEERING	
PROJECT TITLE:	CALTRAIN BRIDGING STRUCTURE AND COMMERCIAL BUILDINGS
DESIGNED BY:	MR. GUADALUPE GONZALEZ
	MR. ANTHONY NAVARRETE
ADVISOR:	REYNAUD SERRETTE
DATE: 06/07/2013	SCALE: NTS
SHEET NAME: S100	SHEET NO. 1 OF: 15



Tower Elevations 1

Santa Clara University	
DEPARTMENT OF CIVIL ENGINEERING	
PROJECT TITLE: CALTRAIN BRIDGING STRUCTURE AND COMMERCIAL BUILDINGS	
DESIGNED BY:	MR. GUADALUPE GONZALEZ
	MR. ANTHONY NAVARRETE
ADVISOR: REYNAUD SERRETTE	
DATE: 06/07/2013	SCALE: NTS
SHEET NAME: S101	SHEET NO.: 2 OF: 15

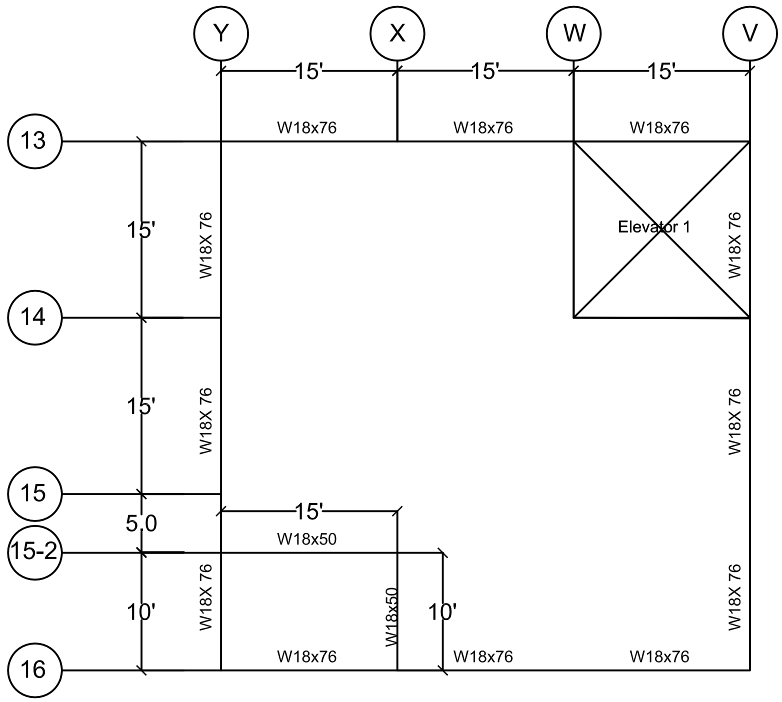


Tower 1 Column Layout 1

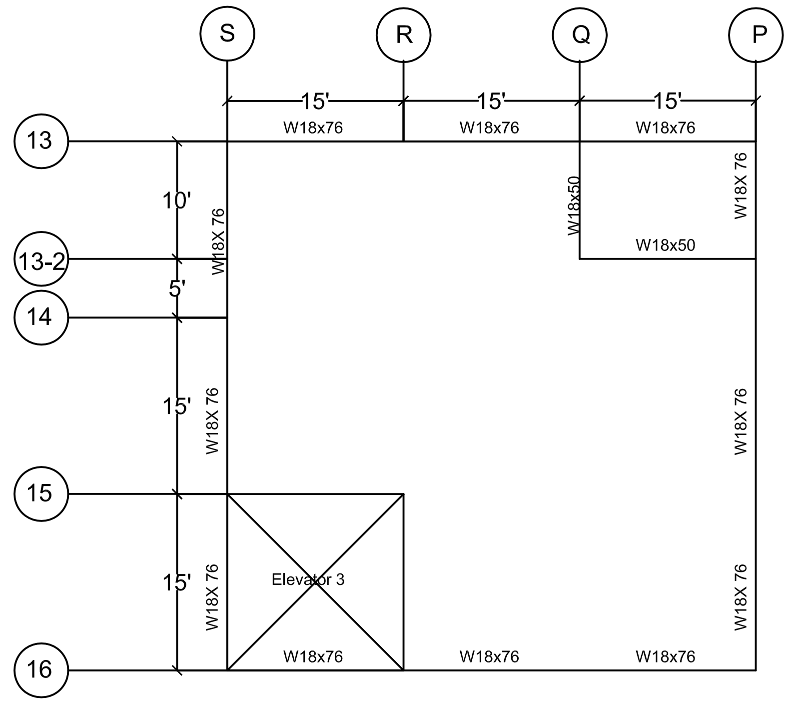
Tower 3 Column Layout 2

Note: All columns are W21X182

Tower 1 and 3 Floor Plans	**S101**



Tower 1 Beam Layout 1



Tower 3 Beam Layout 2

Santa Clara University	
DEPARTMENT OF CIVIL ENGINEERING	
PROJECT TITLE:	CALTRAIN BRIDGING STRUCTURE AND COMMERCIAL BUILDINGS
DESIGNED BY:	MR. GUADALUPE GONZALEZ MR. ANTHONY NAVARRETE
ADVISOR: REYNAUD SERRETTE	
DATE: 06/07/2013	SCALE: NTS
SHEET NAME: S102	SHEET NO.: 3 OF: 15

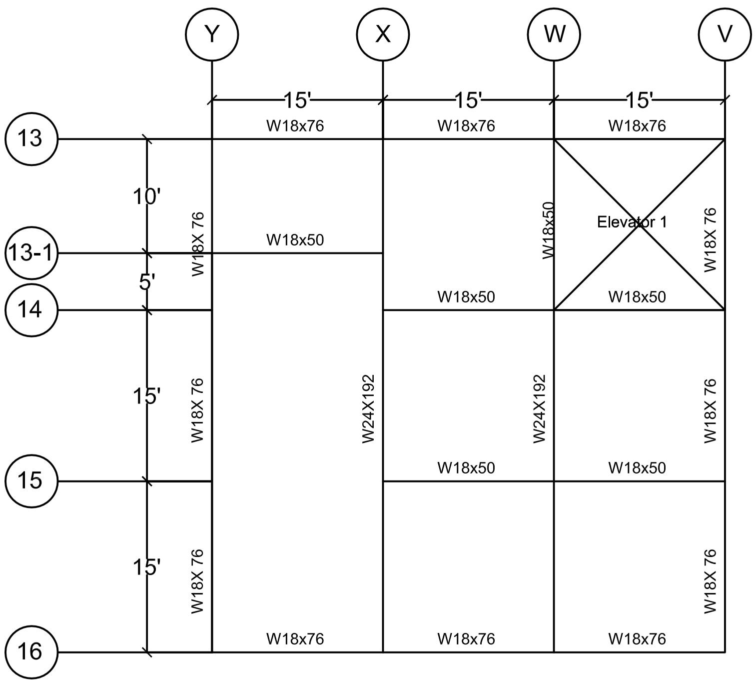
Tower 1 and 3 First Floor

S102

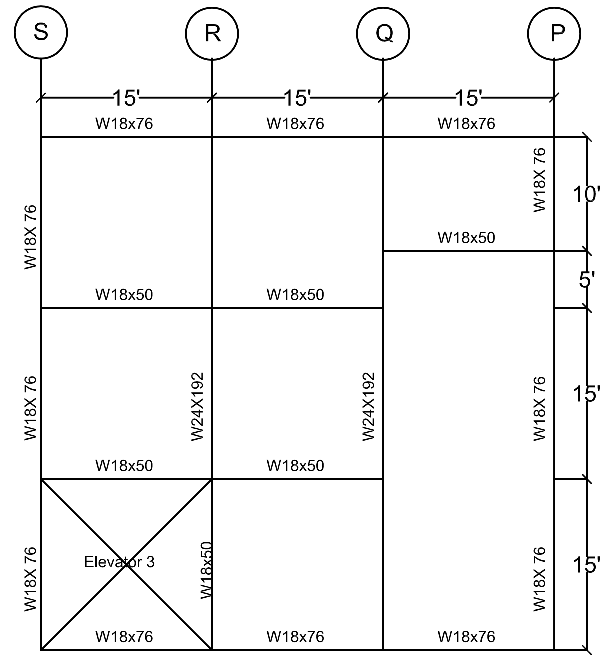
Santa Clara University

DEPARTMENT OF CIVIL ENGINEERING

PROJECT TITLE:	CALTRAIN BRIDGING STRUCTURE AND COMMERCIAL BUILDINGS
DESIGNED BY:	MR. GUADALUPE GONZALEZ
	MR. ANTHONY NAVARRETE
ADVISOR:	REYNAUD SERRETTE
DATE: 06/07/2013	SCALE: NTS
SHEET NAME: S103	SHEET NO.: 4 OF: 15



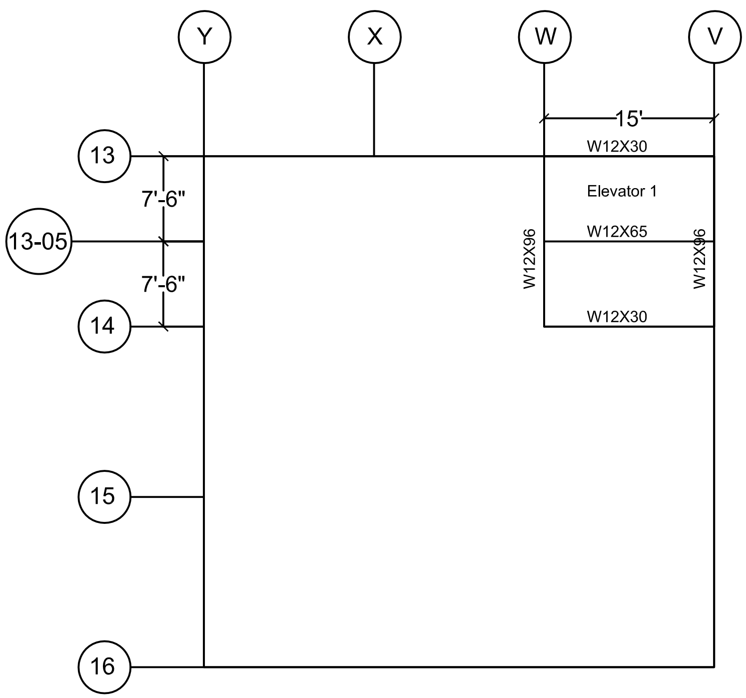
Tower 1 Beam Layout (1)



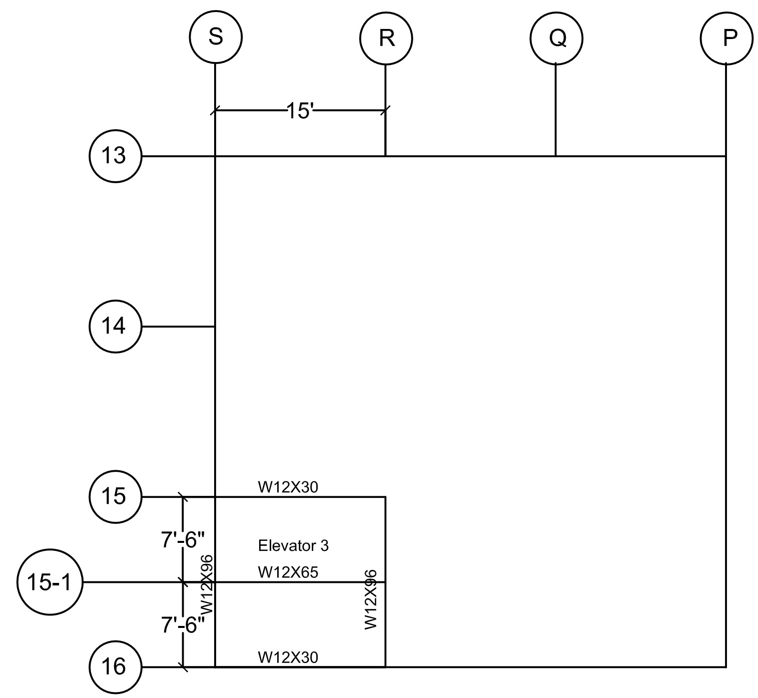
Tower 3 Beam Layout (2)

Tower 1 and 3 Second Floor

S103



Tower 1 Beam Layout 1



Tower 3 Beam Layout 2

Santa Clara University	
DEPARTMENT OF CIVIL ENGINEERING	
PROJECT TITLE:	CALTRAIN BRIDGING STRUCTURE AND COMMERCIAL BUILDINGS
DESIGNED BY:	MR. GUADALUPE GONZALEZ
	MR. ANTHONY NAVARRETE
ADVISOR: REYNAUD SERRETTE	
DATE: 06/07/2013	SCALE: NTS
SHEET NAME: S104	SHEET NO.: 5 OF: 15

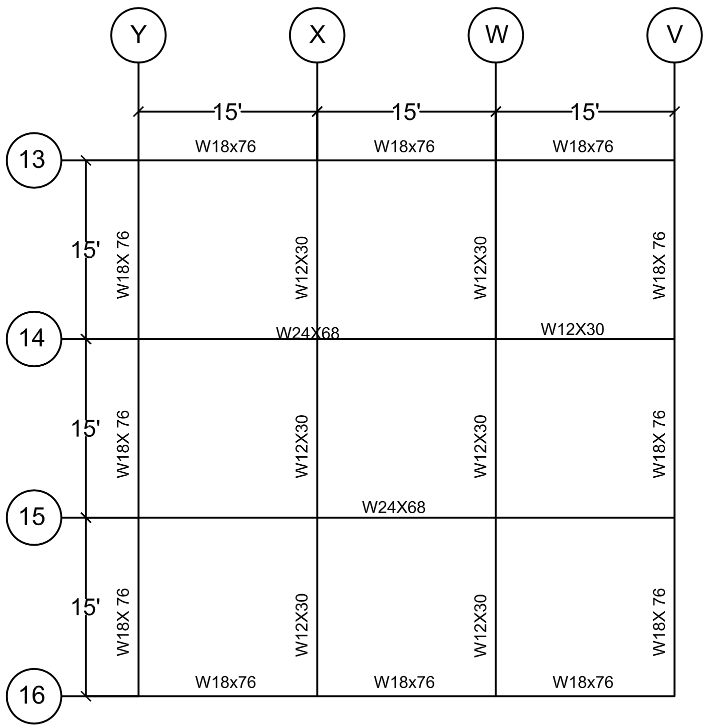
Tower 1 and 3 Mechanical Level

S104

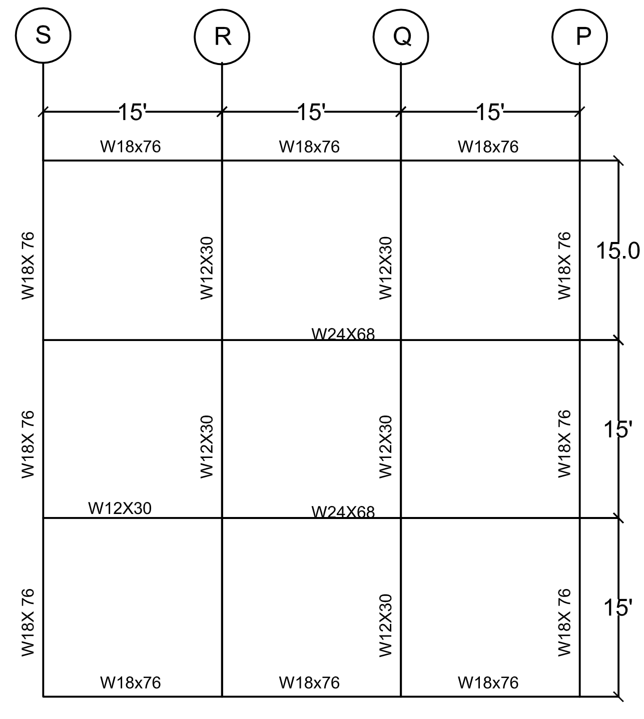
Santa Clara University

DEPARTMENT OF CIVIL ENGINEERING

PROJECT TITLE:	CALTRAIN BRIDGING STRUCTURE AND COMMERCIAL BUILDINGS		
DESIGNED BY:	MR. GUADALUPE GONZALEZ		
	MR. ANTHONY NAVARRETE		
ADVISOR: REYNAUD SERRETTE			
DATE: 06/07/2013		SCALE: NTS	
SHEET NAME: S105		SHEET NO.: 6	OF: 15



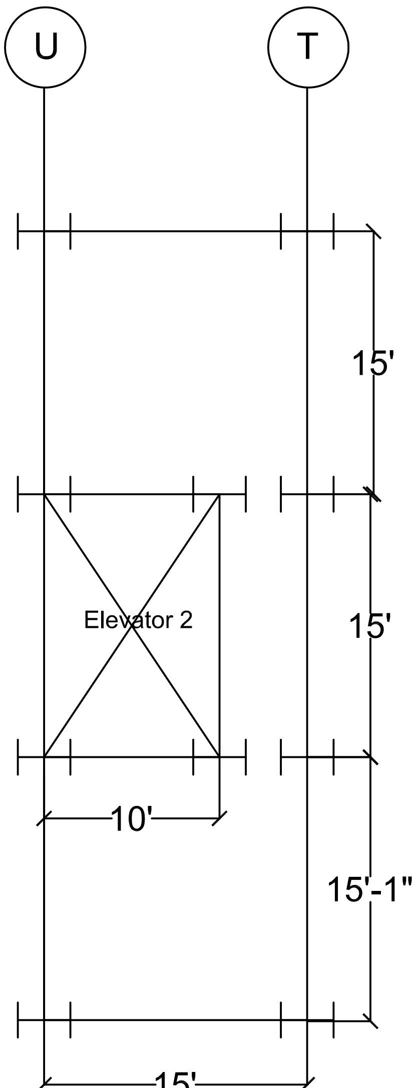
Tower 1 Beam Layout 1



Tower 3 Beam Layout 2

Tower 1 and 3 Roof

S105



Tower 2 Column Layout 1

Santa Clara University	
DEPARTMENT OF CIVIL ENGINEERING	
PROJECT TITLE:	CALTRAIN BRIDGING STRUCTURE AND COMMERCIAL BUILDINGS
DESIGNED BY:	MR. GUADALUPE GONZALEZ
	MR. ANTHONY NAVARRETE
ADVISOR: REYNAUD SERRETTE	
DATE: 06/07/2013	SCALE: NTS
SHEET NAME: S106	SHEET NO.: 7 OF: 15

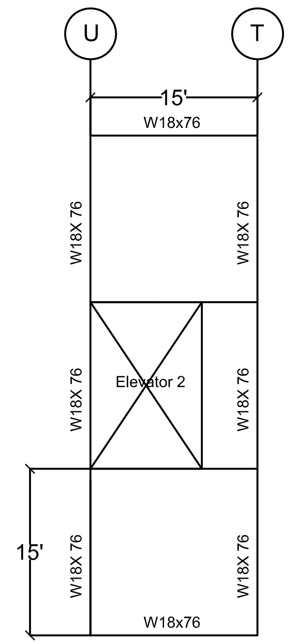
Tower 2 Floor Plans

S106

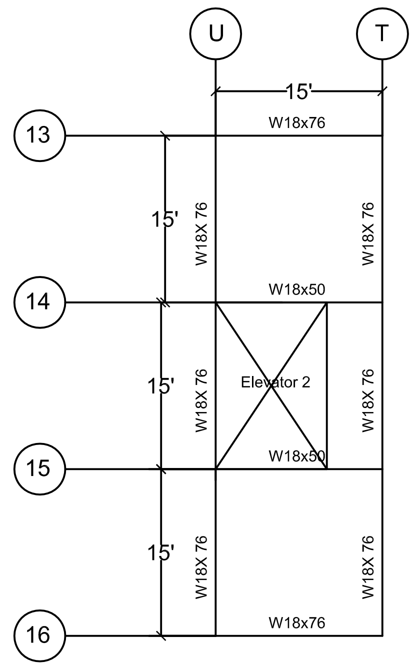
Santa Clara University

DEPARTMENT OF CIVIL ENGINEERING

PROJECT TITLE:	CALTRAIN BRIDGING STRUCTURE AND COMMERCIAL BUILDINGS
DESIGNED BY:	MR. GUADALUPE GONZALEZ
	MR. ANTHONY NAVARRETE
ADVISOR: REYNAUD SERRETTE	
DATE: 06/07/2013	SCALE: NTS
SHEET NAME: S107	SHEET NO.: 8 OF: 15



Tower 2 First Floor 1

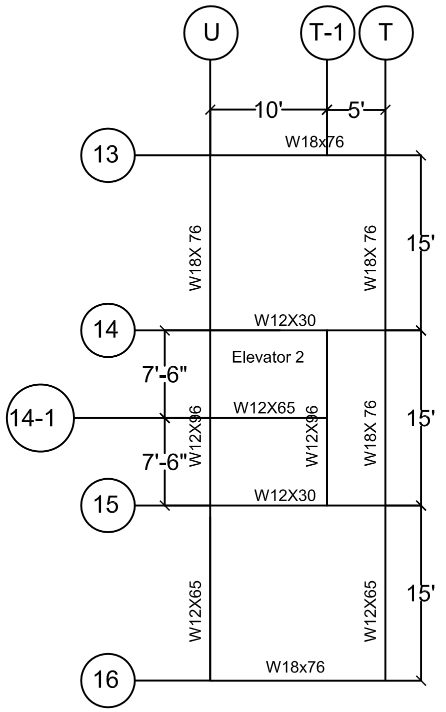


Tower 2 Second Floor 2

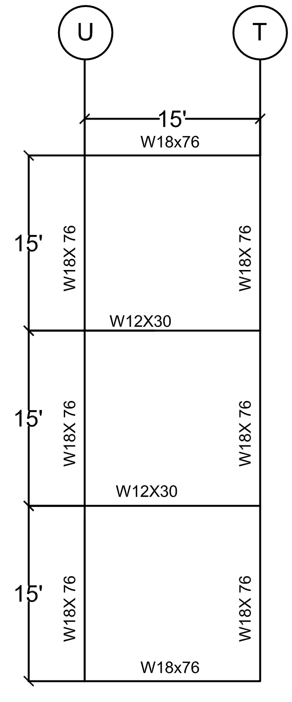
Tower 2 First and Second Floor

S107

Santa Clara University	
DEPARTMENT OF CIVIL ENGINEERING	
PROJECT TITLE:	CALTRAIN BRIDGING STRUCTURE AND COMMERCIAL BUILDINGS
DESIGNED BY:	MR. GUADALUPE GONZALEZ
	MR. ANTHONY NAVARRETE
ADVISOR: REYNAUD SERRETTE	
DATE: 06/07/2013	SCALE: NTS
SHEET NAME: S108	SHEET NO.: 9 OF: 15



Tower 2 Mechnaical Level 1



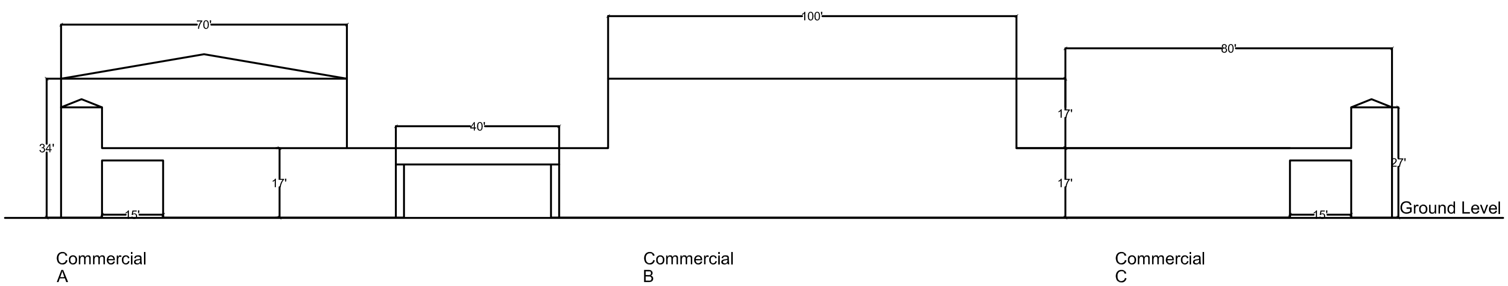
Tower 2 Roof 2

Tower 2 Mechanical and Roof

S108

Santa Clara University
DEPARTMENT OF CIVIL ENGINEERING

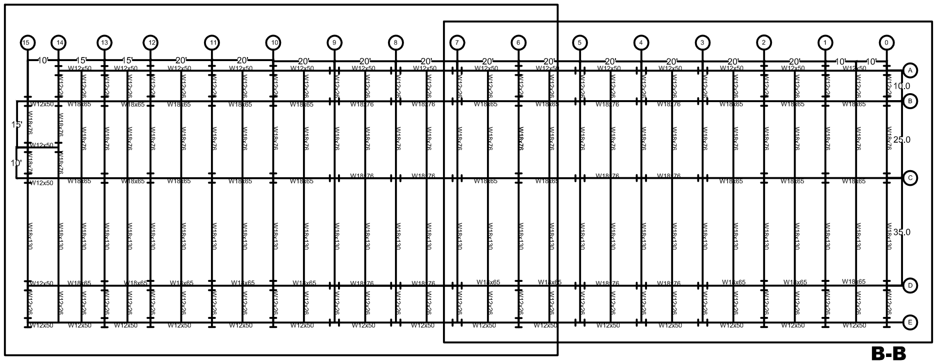
PROJECT TITLE: CALTRAIN BRIDGING STRUCTURE AND COMMERCIAL BUILDINGS			
DESIGNED BY:	MR. GUADALUPE GONZALEZ		
	MR. ANTHONY NAVARRETE		
ADVISOR: REYNAUD SERRETTE			
DATE: 06/07/2013	SCALE: NTS		
SHEET NAME: S109	SHEET NO.: 10	OF: 15	



Commercial Building Elevations 1

Santa Clara University
DEPARTMENT OF CIVIL ENGINEERING

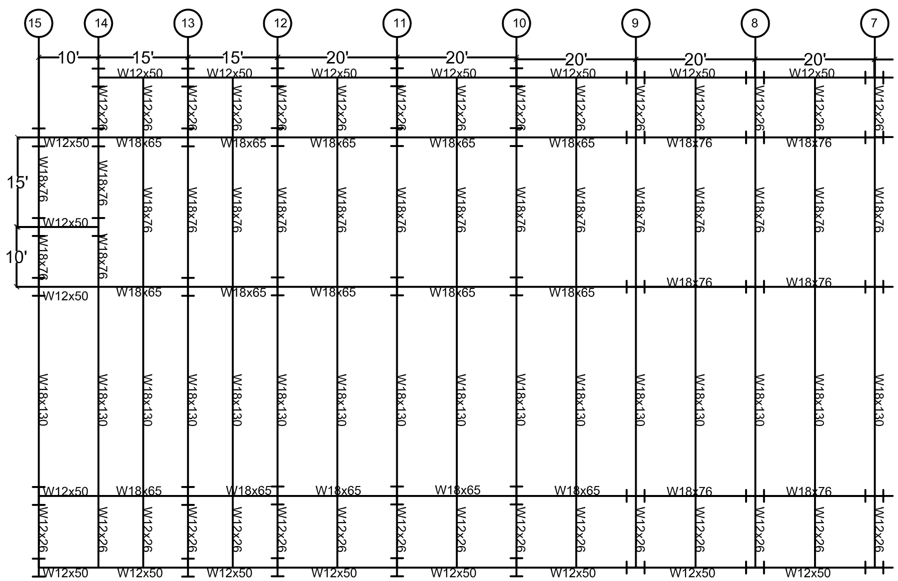
PROJECT TITLE:	CALTRAIN BRIDGING STRUCTURE AND COMMERCIAL BUILDINGS
DESIGNED BY:	MR. GUADALUPE GONZALEZ
	MR. ANTHONY NAVARRETE
ADVISOR:	REYNAUD SERRETTE
DATE: 06/07/2013	SCALE: NTS
SHEET NAME: S110	SHEET NO.: 11 OF: 15



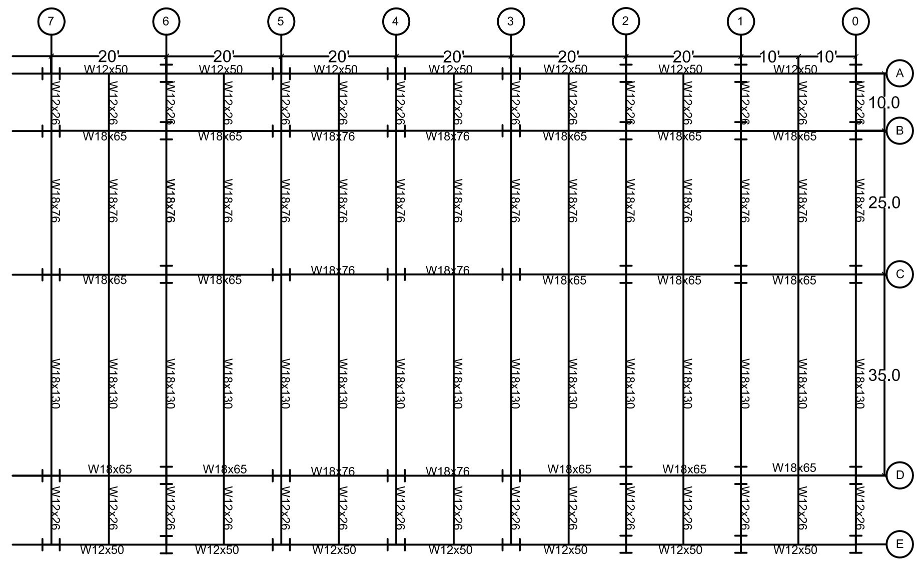
A-A

B-B

Commercial Building A Beam Layout (1)



Commercial Building A Beam Layout (A-A)



Commercial Building A Beam Layout (B-B)

Note: All columns are W24X104

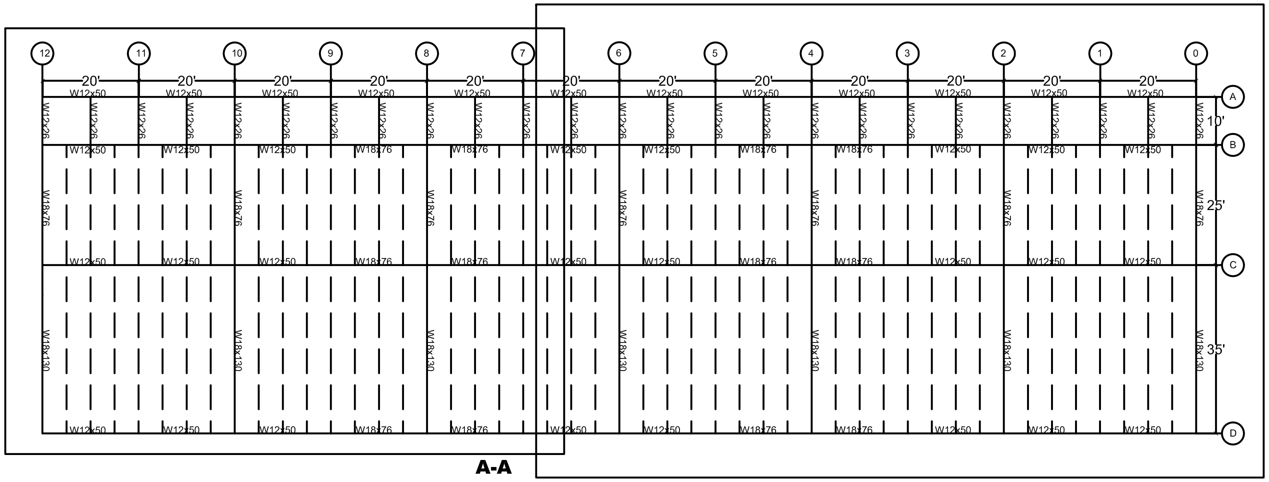
Commercial Building A First Floor

S110

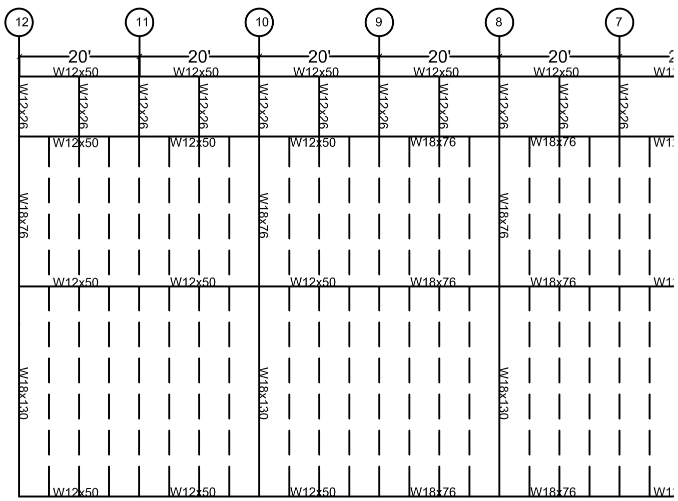
Santa Clara University

DEPARTMENT OF CIVIL ENGINEERING

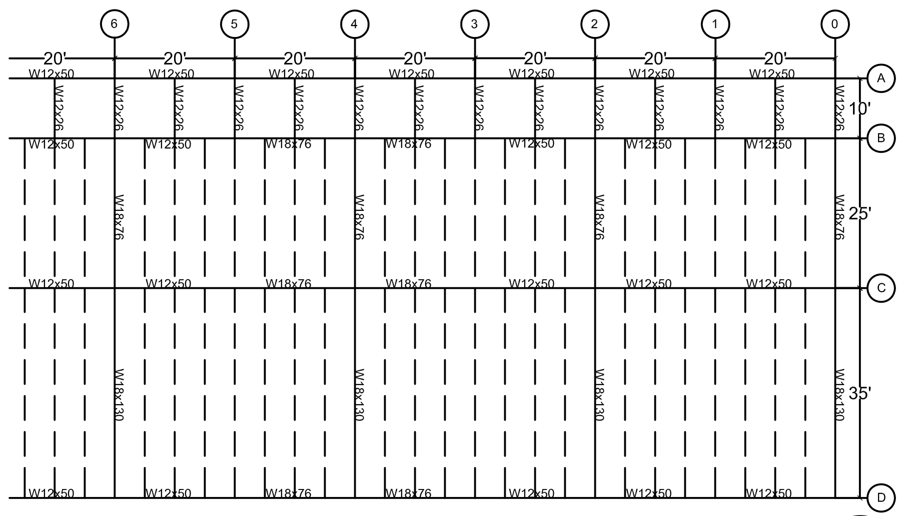
PROJECT TITLE:	CALTRAIN BRIDGING STRUCTURE AND COMMERCIAL BUILDINGS
DESIGNED BY:	MR. GUADALUPE GONZALEZ
	MR. ANTHONY NAVARRETE
ADVISOR: REYNAUD SERRETTE	
DATE: 06/07/2013	SCALE: NTS
SHEET NAME: S111	SHEET NO.: 12 OF: 15



Commercial Building A (1)



Commercial Building A Beam Layout (A-A)



Commercial Building A Beam Layout (B-B)

Commercial Building A Roof Plan

S111

Santa Clara University

DEPARTMENT OF CIVIL ENGINEERING

PROJECT TITLE: CALTRAIN BRIDGING STRUCTURE AND COMMERCIAL BUILDINGS

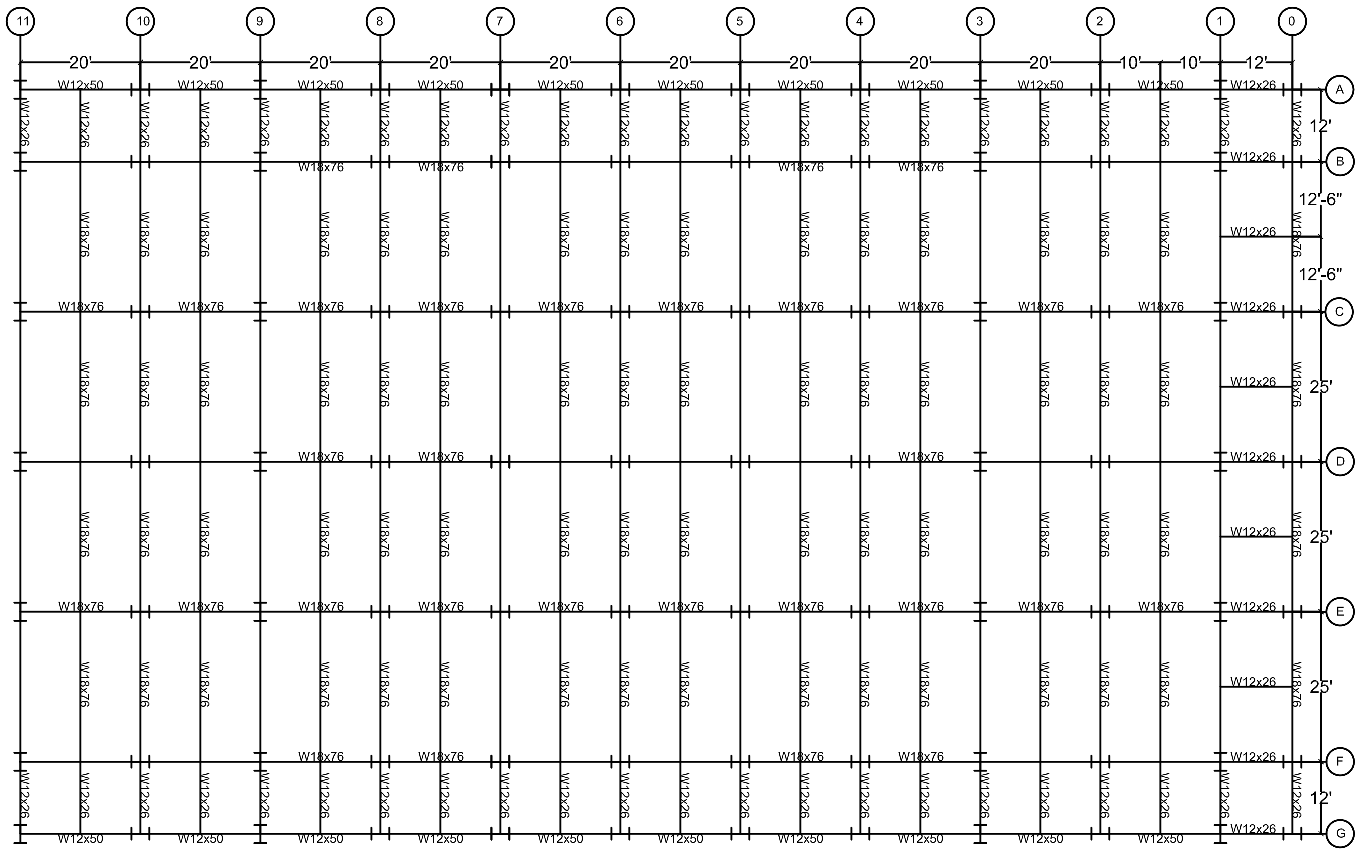
DESIGNED BY: MR. GUADALUPE GONZALEZ

MR. ANTHONY NAVARRETE

ADVISOR: REYNAUD SERRETTE

DATE: 06/07/2013 | SCALE: NTS

SHEET NAME: S112 | SHEET NO.: 13 | OF: 15



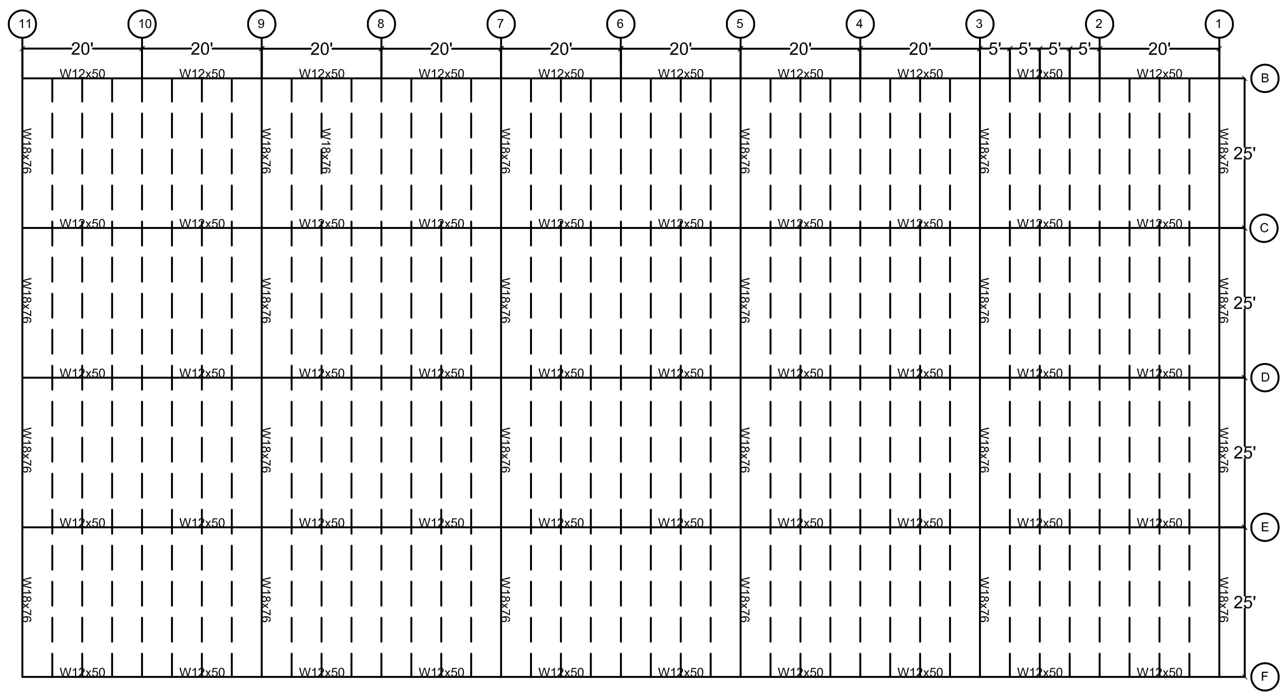
Note: All columns in N-S direction are W14X132
All columns in E-W direction are W24X94

Commercial Building B Beam Layout

1

Commercial Building A Floor Plan

S112



Commercial Building B Roof Beam Layout

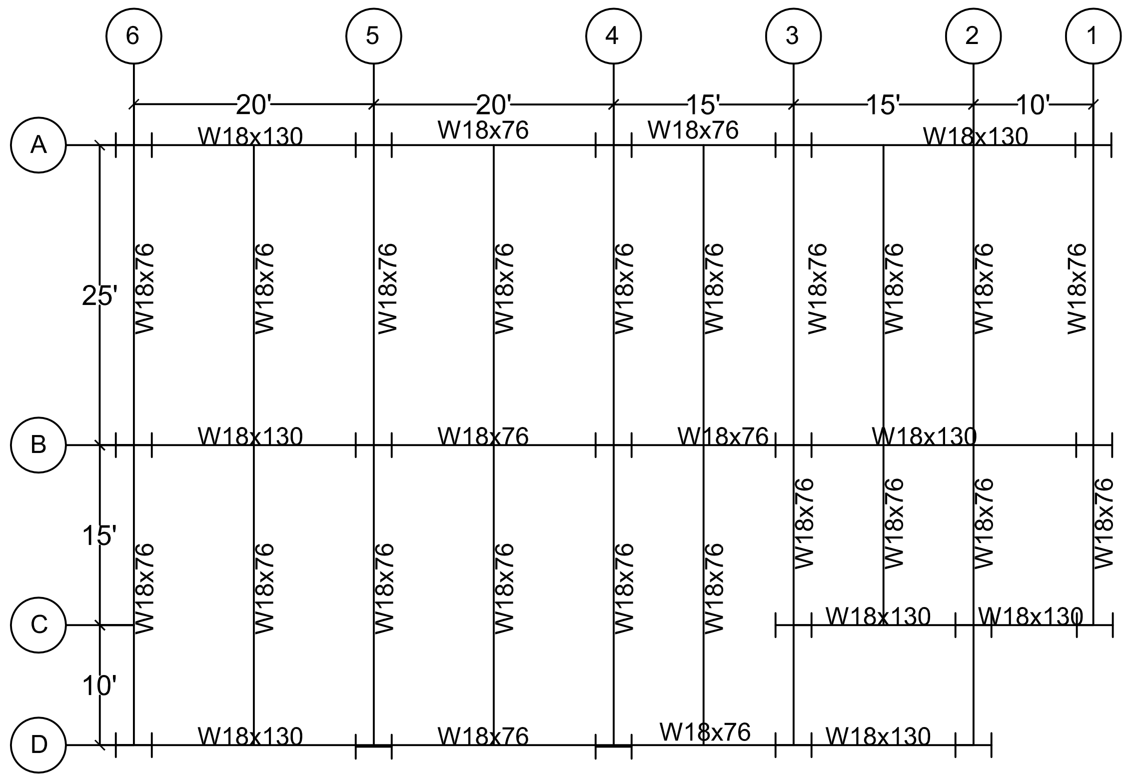
1

Santa Clara University	
DEPARTMENT OF CIVIL ENGINEERING	
PROJECT TITLE: CALTRAIN BRIDGING STRUCTURE AND COMMERCIAL BUILDINGS	
DESIGNED BY: MR. GUADALUPE GONZALEZ	
MR. ANTHONY NAVARRETE	
ADVISOR: REYNAUD SERRETTE	
DATE: 08/07/2013	SCALE: NTS
SHEET NAME: S113	SHEET NO.: 14 OF: 15

Commercial Building B Roof Plan

S113

Santa Clara University	
DEPARTMENT OF CIVIL ENGINEERING	
PROJECT TITLE:	CALTRAIN BRIDGING STRUCTURE AND COMMERCIAL BUILDINGS
DESIGNED BY:	MR. GUADALUPE GONZALEZ
	MR. ANTHONY NAVARRETE
ADVISOR: REYNAUD SERRETTE	
DATE: 06/07/2013	SCALE: NTS
SHEET NAME: S114	SHEET NO.: 15 OF: 15



Commercial Building C Layout

1

Note: All columns are W12X87

Commercial Building C

S114