OFFICE BUILDING-G

Eastern United States

Final Report

Carl Hubben

Structural Option

Advisor: Dr. Ali Memari

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Abstract


OFFICE BUILDING
EASTERN UNITED STATES
Architect: HO+K
General Contractor: Turner Construction
Structural Engineers: SK&A Engineers
MEP: GHT Limited
Civil Engineer: Loiederman Soltesz Associates
General Information:
Size: 649,461 SF
Stories: 14 levels above grade
Garage: 4 levels below grade
Delivery Method: Design-Bid-Build
Construction: Summer 2010 – July 2011
Project Cost: $75,000,000
Architecture:
Curved glass curtain wall on the southern façade
Open floor office space ready for tenant fit outs
Upscale entrance with elegant stone flooring
Street level retail space
LEED Silver Certification
Electrical:
Power provided by Pepco
Stepped down by interior transformer to 277/480 3 phase, 3 wire system
Mechanical:
3 VSD chillers provide chilled water to AHUs
VAV and CAV fans provide airflow to spaces
Fully integrated building automation system (DDC)
Structure:
Post-tensioned girders with 7" thick one-way slabs for floors 4-13
Lateral resistance provided by internal shear walls spanning in both the major and minor axis
Columns ranging from 24" x24" 10,0000 psi in garage to 30" Ø 6000psi in upper floors
4 levels of concrete substructure used for parking
Spread footings supporting loads between 64k to 1025k and range in size between 4' x 4' to 15' x 15'
Construction:
Logistical Issues:
Neighboring existing metro station
On the corner of major roadways
Construction Concerns:
4 story sub-grade parking garage
Geotechnical complications
Vibration effects from trains
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STRUCTURAL OPTION
http://www.engr.psu.edu/ae/thesis/portfolios/2011/cjh5105/index.html

Acknowledgements

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	Robert Holland
	Kevin Parfitt

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Executive Summary

The purpose of this report is to analyze the structural impacts of creating an open floor space for Office Building-G. Office Building-G is a 14-story office building located in the Eastern United States. The redesign of the structure was performed following the provisions set forth by the IBC, local building codes, ASCE and AISC Manuals. Changes to the structural system affected the mechanical and architectural designs and these implications were investigated as part of this report.

Office Building-G's current gravity system is a series of cast-in-place concrete columns located on the interior and perimeter of the floor plan. A concrete shear wall core is also responsible for carrying some of the gravity loads but the main purpose of the core is to resist all of the lateral loads which Office Building-G may experience. The floor system is a 7" one way slab which spans 20' between post-tensioned concrete girders.

The goal of the proposed change is to create a column free space in which the structural system would not impact the design of the tenant fit out spaces. The redesign was performed as if the only core elements were those used for vertical transportation. These criterions were addressed in the redesign by creating an external structural system capable of resisting both gravity and lateral loads. Internal columns were positioned around the elevator shafts and stairwells and were designed to take a portion of the gravity loads. The design process was based on strength and serviceability requirements.

ETABS, RAM Structural System and SAP 2000 models were created to aid in the design of the new structure for Office Building-G. Components of the analysis given by these programs were checked through hand calculations to confirm correct modeling techniques were used. AISC design guides were referenced for the design of castellated beams and braced frames.

The final redesign of Office Building-G makes use of structural steel members. Composite castellated beams distribute gravity loads to the W-Flange interior and exterior columns. Lateral loads are distributed to the braced frames through the floor diaphragm which consists of lightweight concrete on composite metal decking. Two story chevron frames, alternating between normal and inverted frames, resist the lateral forces and distribute them to the foundation.

This new structural system is capable of resisting the design loads and provides a column free space available for tenant fit outs. Through adjusting the way in which the cavity space between floors was used, the floor to floor height and overall building height was not affected. Due to the external braced frame, the existing façade architecture was affected and these changes were considered.

Existing Design

Due to owner restrictions, the building name, location and tenant of Office Building-G cannot be disclosed. Neighboring an existing metro station, this 14 story building will become one the tallest of the modest skyline. Beneath the superstructure is a below grade, 4-story parking garage with space for 662 cars. On the first two floors of the building, a larger floor plan is used to accommodate the rentable space for retail, a restaurant, a bank and a loading dock. Typical floors have a square footage of 25,376 sf with a floor to floor height of 12’-3”. The roof of the mechanical penthouse is 195 ft above grade and the gross square footage of the superstructure and garage combined is 649,461 sf.

The majority of the building façade is precast concrete panels but the southern façade is a curved glass curtain wall. On the first and second floor there is a restaurant which has a glass façade with concrete pilasters between the panes of glass. Figures 1 is a view of the South-West corner of the building. The orange lines outline the restaurant while the blue show the extents of the parking garage.

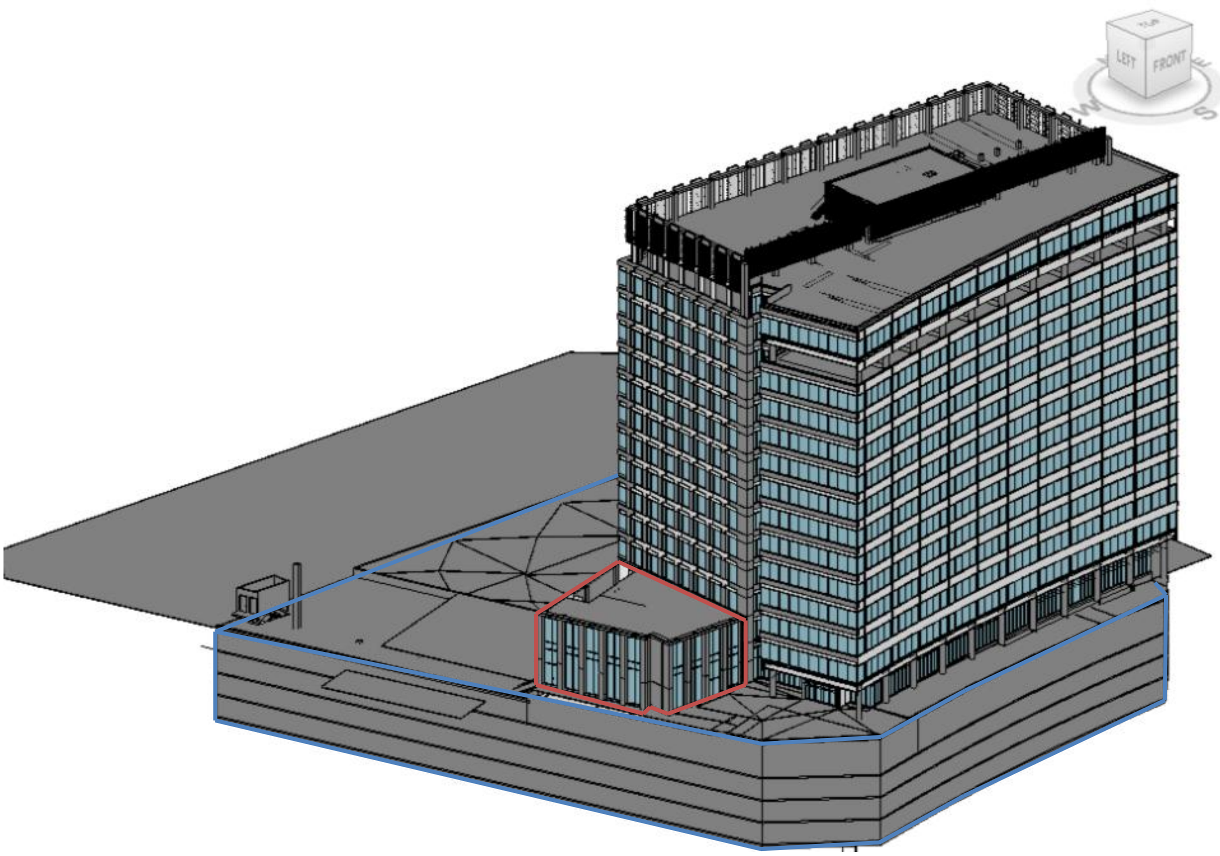


Figure 1

Directly to the right of the restaurant in Figure 1 is the main entrance of the building. The upscale lobby, along with the entire first floor, has a 17 ft floor to floor height, compared to the typical height of 12 ft 3 in. Figure 1 also shows how the perimeter columns supporting the glass façade are on the exterior of the building on the first floor due to a setback on the first floor.

Figure 2 is a view of the North-East corner of Office Building-G. Again, the extents of the below grade parking garage are outlined in blue. Other building aspects displayed in this figure are the bank in green, the loading dock in red, architectural screen wall in purple and the mechanical penthouse in orange.

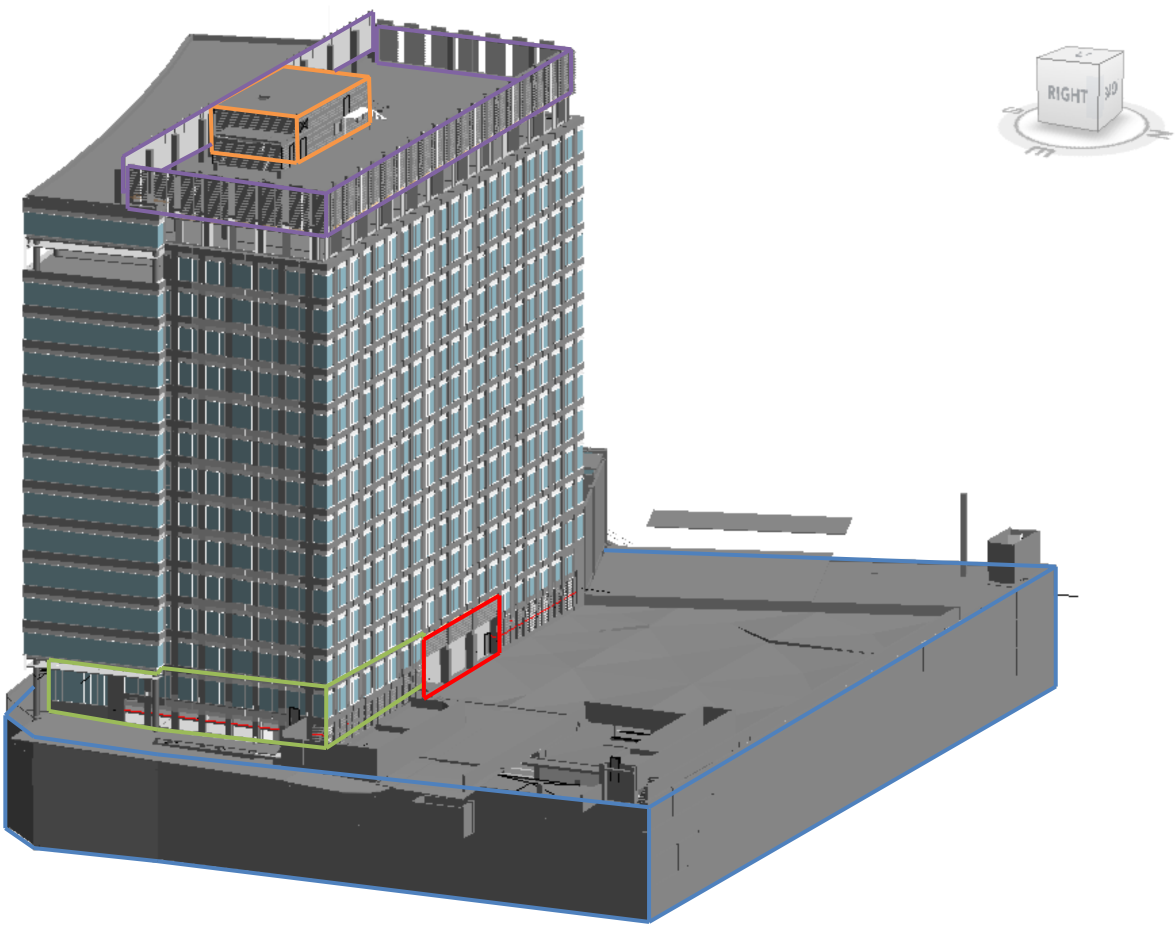


Figure 2

Figure 3 is a typical floor plan which is followed for the majority of the structure. On the 12th and 13th floors of the building the exterior columns slope in, creating a slightly smaller square footage for the 13th and 14th floors. Other building features are described throughout the report as needed.

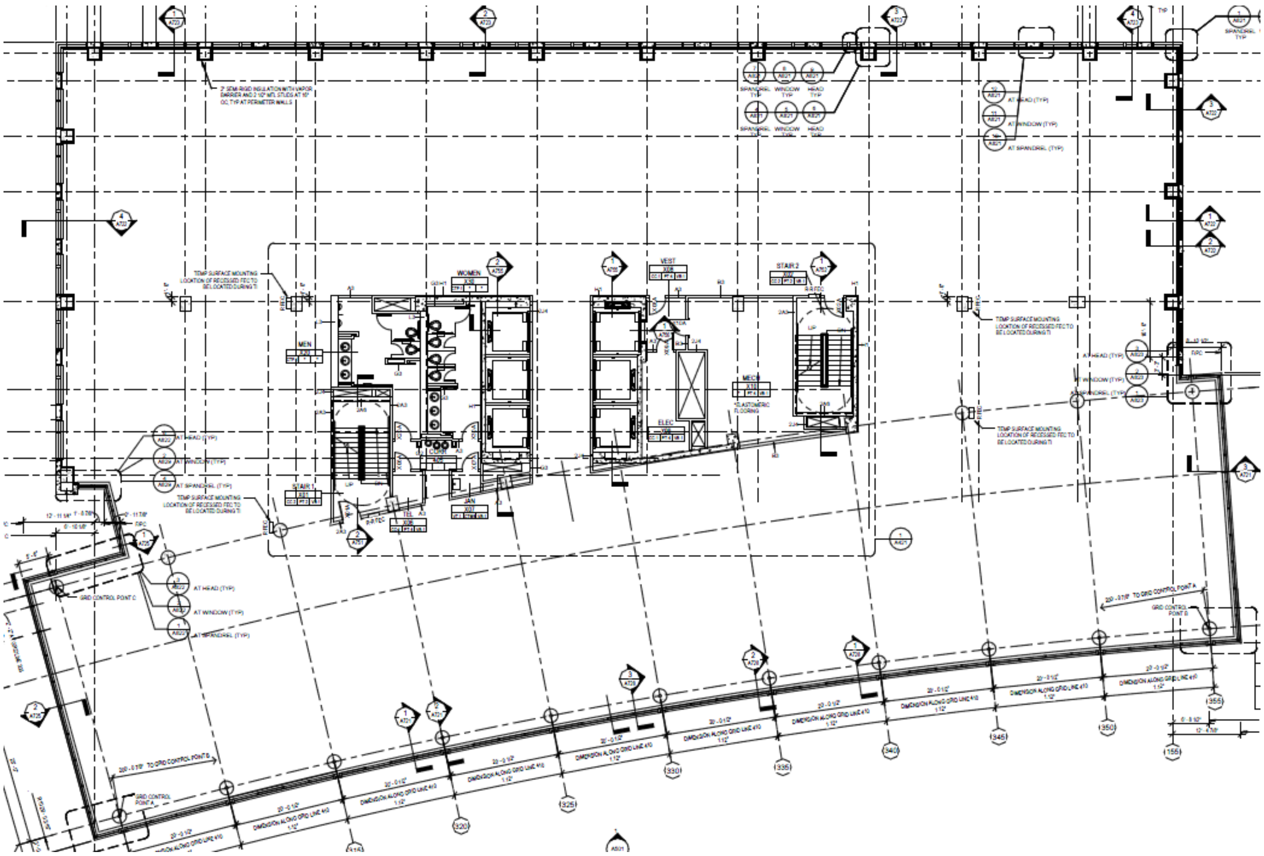


Figure 3

Existing Structural System

Office Building-G is uses cast-in-place concrete for the main structural members. The superstructure relies on a shear wall core and gravity columns to transfer the gravity loads to the foundation. All of the lateral forces are resisted and transferred to the foundation through the shear wall core. The concrete strength ranges between 5,000 psi and 10,000 psi depending on the story level.

The typical floor system is a 7", 5000 psi concrete one-way slab spanning between 18" deep, 5000 psi post-tensioned girders with ½" diameter strand with strength of 270 ksi. The typical bay size is 20' X 45'. The system was chosen to efficiently span the 45' length while minimizing the structural depth.

Gravity System

Gravity loads are carried down the building through a combination of interior and perimeter concrete columns and a shear wall core. The typical floor system is a cast-in-place concrete one-way slab. Thickness changes based on loading conditions but the typical floor is a 7", 5000 psi normal weight concrete slab. On the first floor, there is a 12" concrete slab designed for fire separation between the parking garage and superstructure. The slab system carries the loads to post-tensioned concrete beams with spans between 41'-5" and 45'-1 1/4".

The post tensioned girders range in width from 18" to 48" and have a maximum depth of 24". Forces in the girders are between 162 kips to 675 kips. These beams collect the floor loads from the slab and distribute their reactions to the columns supporting them. Figure 4 and 5 below highlights the post-tensioned beams in yellow, the reinforced beams in purple, and the columns in red.

Figure 4

Rectangular and round concrete columns then transfer the loads down the strictly followed grid. Typical floors have columns sizes of 24" x 24", 24" x 30", and 30" diameter. Smaller columns are used in the mechanical penthouse due to the much lower loads they are carrying. On above grade floors, higher strength concrete is placed below columns and shear walls in the slab to accommodate for any possibility of punching shear. In the parking garage, 8" drop panels are used instead of the different concrete strengths. The typical floor plan shown in Figure 5 highlights the post-tensioned beams in yellow, the reinforced beams in purple, shear walls in green and blue, and the columns in red.

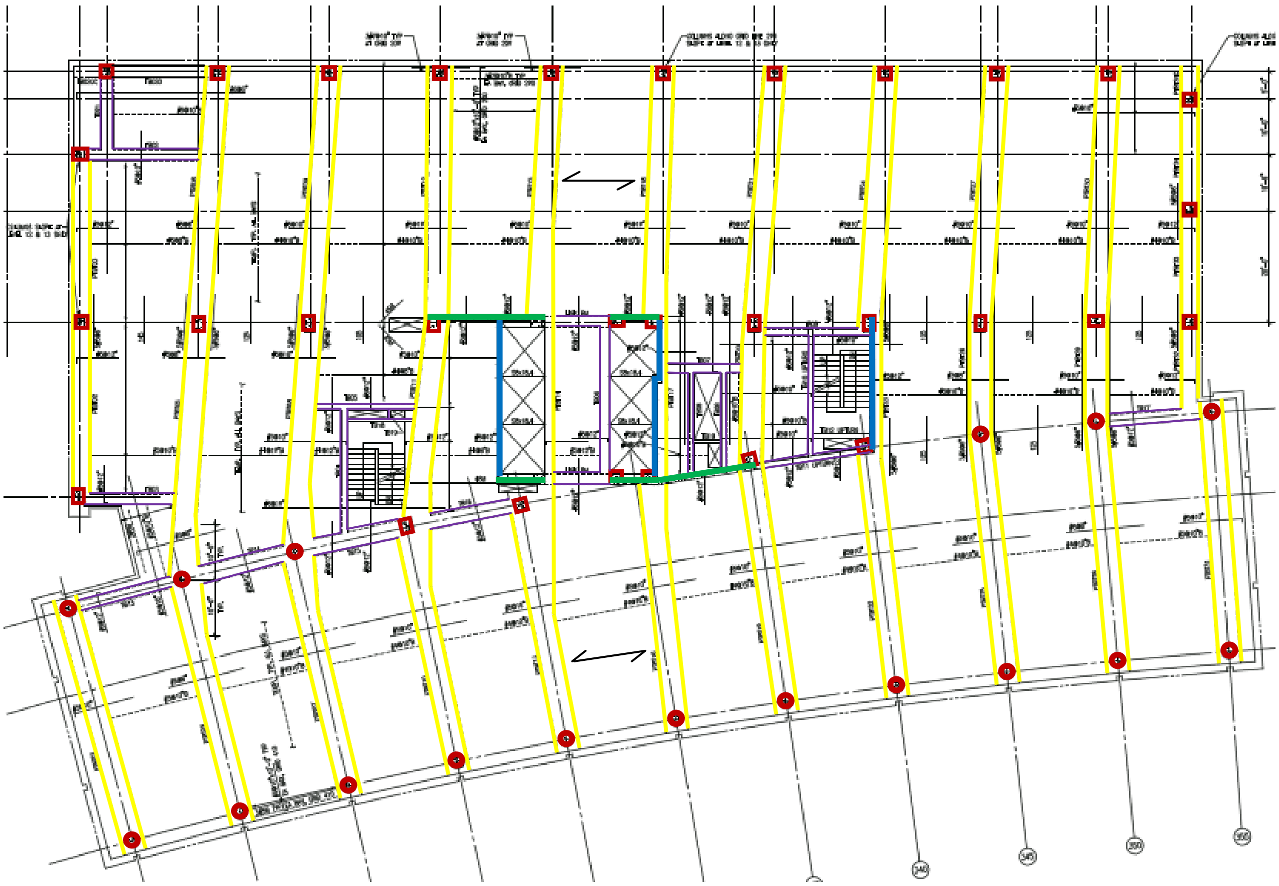


Figure 5

Lateral System

Wind and seismic forces are resisted by an internal shear wall core. The core is made of reinforced concrete walls which have a consistent floor plan from the bottom floor of the parking garage up to the slab of the roof. Basement shear walls were designed with f'c = 10,000 psi, levels 1-4 use f'c = 8,000 psi, and levels 5-14 use f'c = 5,000 psi. Precast concrete beams attached to concrete columns using precast lateral connections provide the required resistance for the mechanical penthouse and elevator machine room. Figure 6 below displays the plan of the shear wall core which is typical for all of the floors of Office Building-G.

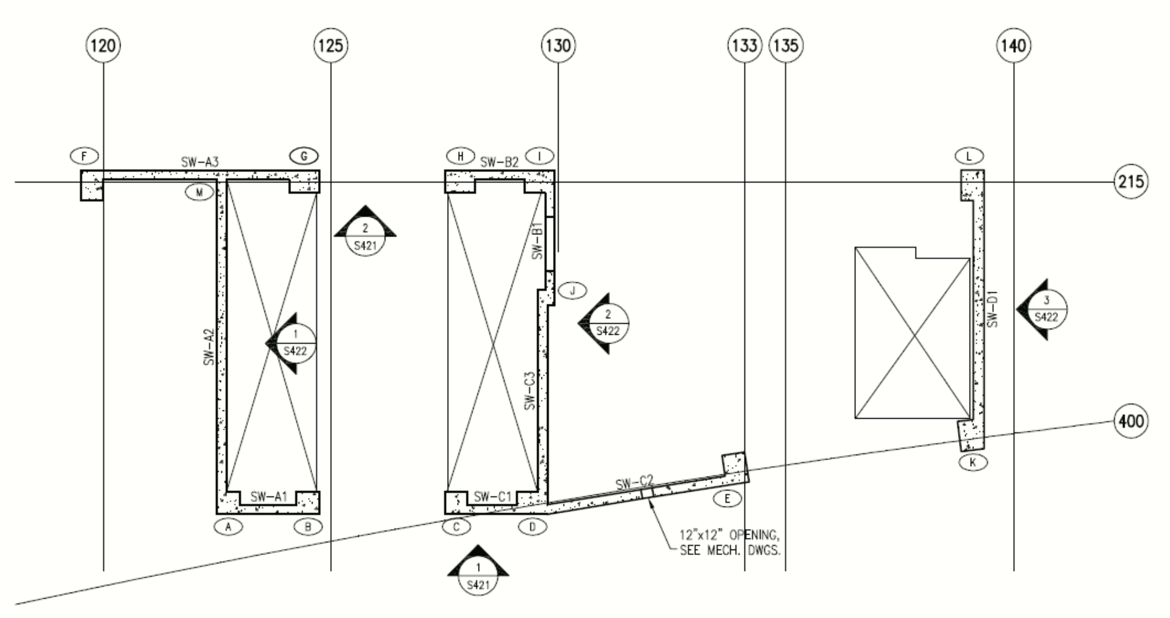


Figure 6

Lateral forces are engaged by the shear walls through the use of floor diaphragms. The building façade collects wind forces that are then transferred to the respective floor diaphragm. Forces then travel through the diaphragm until the shear walls are engaged, at which point the forces are distributed based on the relative stiffness of the walls. Figure 7 is an elevation view of shear wall core.

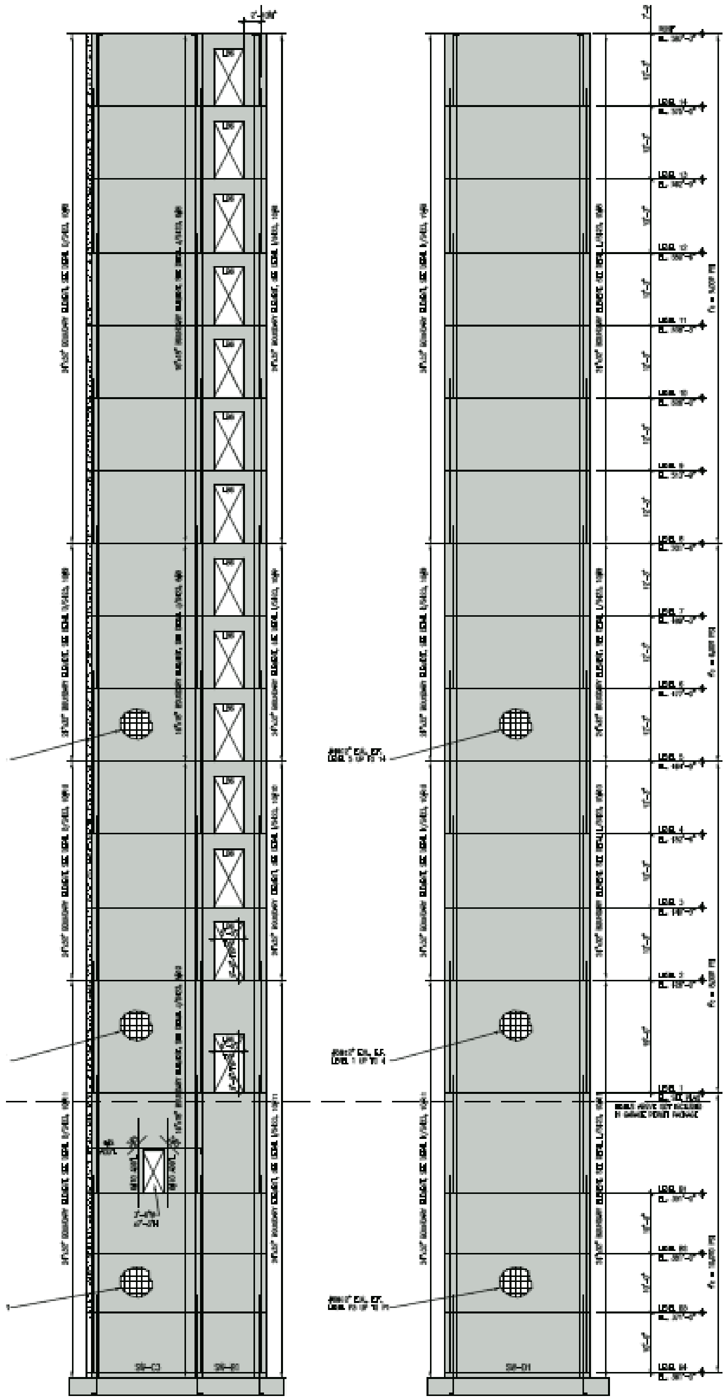


Figure 7

Foundation System:

Schnabel engineering performed a geotechnical study for the location of Office Building-G which determined the possible foundation systems as spread footings, caissons or geopiers. The structural engineers of SK&A decided to use a system of spread footings under the columns, shear walls and along the perimeter concrete bearing wall. Square footage and depth of the footings are based on the load carrying capability of the soil and the vertical load on the column.

Service loads on the columns ranged greatly depending on whether or not the column extended up into the superstructure of the building. Based on the structure above the foundation, the load capacity of soil was determined to support a range of 3,000 psf to 10,000 psf. Loads on the footings varied between 60 kips to 3075 kips, once again depending on which part of Office Building-G they are supporting. Figure 8 is a plan view of the foundation system. The elements outlined in blue are the foundation for the superstructure while those outlined in green only support the below grade parking structure.

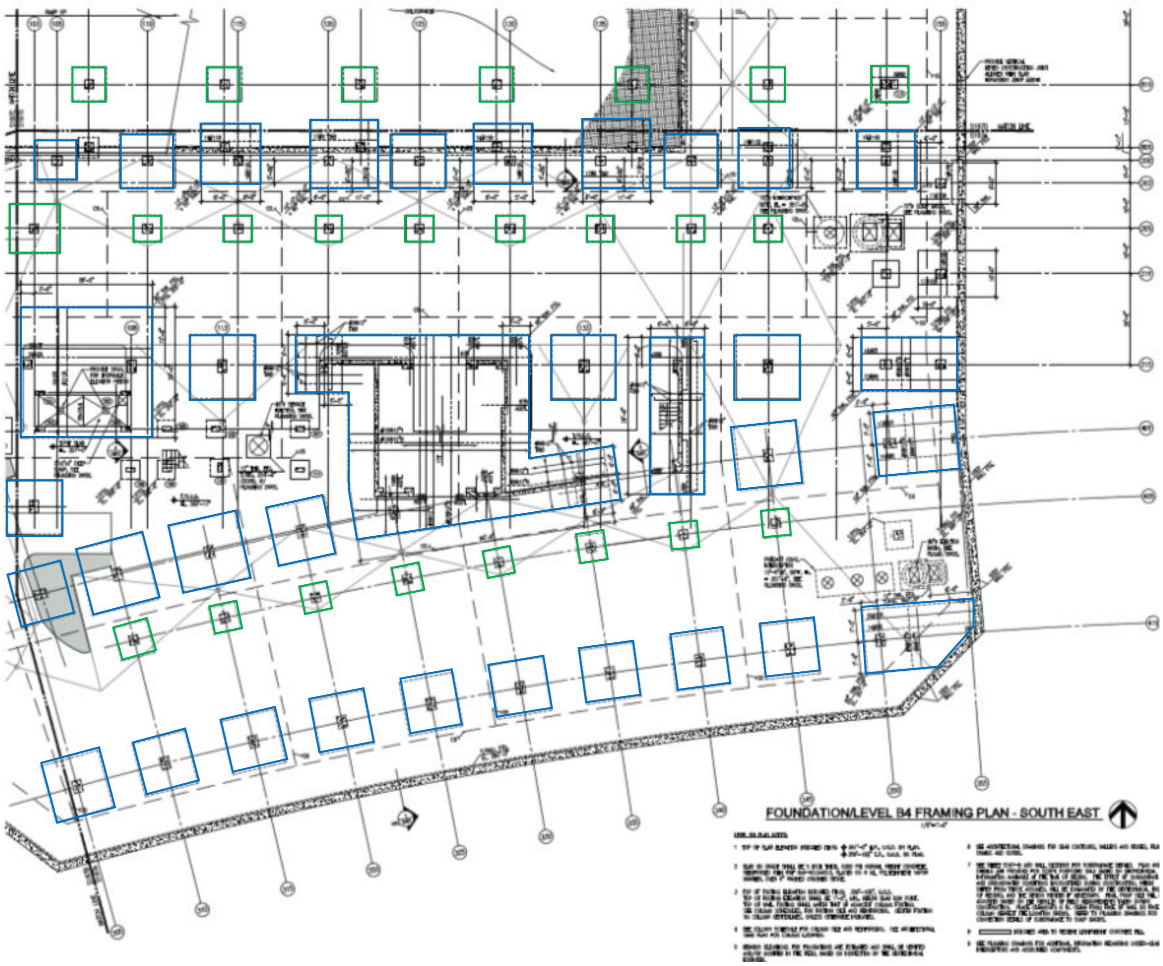


Figure 8

Structural Materials

Structural Materials			
Material	**Element**	**Level**	**Strength**
Cast-in-Place Concrete	Spread Footings	Foundation	f'_c = 10,000 psi
			f'_c = 6,000 psi
			f'_c = 3,000 psi
	Foundation Walls	B4	f'_c = 5,000 psi
		B3-B1	f'_c = 4,000 psi
	Shear Walls	B4-B1	f'_c = 10,000 psi
		L1-L4	f'_c = 8,000 psi
		L5-L7	f'_c = 6,000 psi
		L8-L14	f'_c = 5,000 psi
	Columns	B4-B1	f'_c = 10,000 psi
			f'_c = 6,000 psi
		L1-L4	f'_c = 10,000 psi
			f'_c = 8,000 psi
			f'_c = 6,000 psi
		L5-L7	f'_c = 6,000 psi
		L8-Roof	f'_c = 5,000 psi
	Reinforced Beams	ALL	f'_c = 5,000 psi
	Post-Tensioned Beams	ALL	f'_c = 5,000 psi
Tendons	Post-Tensioned Beams	ALL	F_u= 270 ksi
Reinforcing Steel	Concrete	ALL	F_y = 60 ksi
Structural Steel	Elevator Framing - A36	ALL	F_y = 36 ksi
	Bolts - A325	ALL	F_u= 120ksi

Code and Design Requirements

Design Codes:

National Model Code:

Local building code based on the 2006 International Building Code

Sections: 1603.1.1-1603.1.7, 1603.2, 1607.11, 1608.1, 1608.7, 1608.8, 1609.1

Design Codes:

American Concrete Institute (ACI) 318-08, Building Code Requirements for Structural Concrete and Commentary

ACI 301, Specifications for Structural Concrete for Buildings

ACI 347, Standard Recommended Practice for Concrete Formwork

American Institute for Steel Construction (AISC), Specification for the design, fabrication and erection of structural steel for buildings

Thesis Codes:

National Model Code:

International Building Code, 2006

Design Codes:

ACI 318-08, Building Code Requirements for Structural Concrete and Commentary

American Institute for Steel Construction (AISC), Specification for the design, fabrication and erection of structural steel for buildings

Structural Standards:

American Standards of Civil Engineers (ASCE), ASCE 7-10 Minimum Design Loads for Buildings and Other Structures

Design Guides:

Design of Castellated and Cellular Beams, Dinehart, Coulson, Fares

Proposal

When the construction of Office Building-G is complete the majority of the space will be used as office space. The floor plan was designed with anticipation of tenant fit out spaces. With exception to the building core, which includes the vertical transportation, bathrooms, mechanical and janitorial spaces, the floor plan is open. However, the existing structure of Office Building-G interferes with the space available for tenant fit outs.

All of the concrete columns are exposed, limiting the possibilities of the future tenant fit outs. An external structure designed as part of the building façade would create an entirely open floor plan. However, simply moving the columns to the façade of a building does not create an open floor space. In the case of Office Building-G, the interior location of restrooms and mechanical spaces allowed the structural engineer an opportunity to use an interior shear wall core. Had the restroom location varied with the story levels, the shear wall score would have interfered with the open floor plan. As part of the proposed redesign of Office Building-G, the only elements which will be considered as constant for every level is the location of the elevator shafts and stairwells.

Removal of the shear wall core forces the eternal structure to be capable of resisting the lateral forces which Office Building-G will experience. Braced frames will be used as the lateral force resisting system due to their efficiency and potential for an aesthetically pleasing design.

The structural depth study of this thesis is defined by creating an open floor plan allowing for the maximum freedom of design of the tenant fit outs with the assumption that the only elements impeding the open floor space is the elevator shafts and stairwells. In addition to opening the floor space of Office Building-G, the existing building height and floor to ceiling height will remain as originally designed.

Maintaining the original building height and floor to ceiling height of Office Building-G with the redesigned structure will cause interference between the mechanical duct work and structural members. This issue will be addressed through the work performed as part of a Mechanical Breadth Study.

Integrating the lateral system of a building with the façade has a drastic impact on the façade architecture of the structure. As part of an Architectural Breadth, the aesthetics of the exposed structure will be considered and will have weight in the decision to the type of lateral system designed.

Based on the proposed changes of Office Building-G, it was assumed that the typical floor plan was consistent for every story of the buildings.

MAE Work

To satisfy the requirements of the MAE program the coursework learned in *AE 534 Steel Connections* and *AE 597A Computer Modeling of Building Structures* will be applied. The knowledge learned in AE 534 will be used to design two typical connections used in the braced frame design. A series of computer models will be created using RAM Structural System, SAP 2000 and ETABS and used integrally in the redesign of Office Building-G.

Structural Depth

An entirely new structural system must be designed in order to determine the feasibility of the proposed external structure. Changing from a concrete based design to steel will have large impacts on the gravity and lateral loads, the size of members, and the way in which the building responds to these loads. The following sections of the report lay out the way in which Office Building-G was redesigned.

New Loads

The occupancy spaces of Office Building-G are not changing so the live loads are consistent with the original design values. However, the self-weight of the structure, wind loads, and seismic forces are influenced by the change in the building structure.

Gravity

The live loads for Office Building-G are listed below. A uniform live load of 100 psf was used throughout Office Building-G to allow for flexibility in floor plan design. For members permitting Live Load Reduction the reduced value was used in design.

Floor Live Loads			
Load Description	**Load Location**	**ASCE 7-10 Load (psf)**	**Design Load (psf)**
Office	Levels 1-14	80	80 20 - Partitions

Live Load Reduction was limited to the restrictions of ASCE 7-10 section 4.8 and only used for column and beam design. The reduction of the live load was limited to $0.5L_o$ for members supporting one floor and $0.4L_o$ for members supporting two or more floors. The equation used was:

$L = L_o(0.25 + 15/(K_{LL}A_T))$
$K_{LL} = 2$ for beams and girders
$K_{LL} = 4$ for columns

The dead loads associated with the structure self-weight of Office Building-G are affected due to switching from a concrete building to a steel one. New self-weights of structural members and curtain wall were estimated and later checked to ensure a conservative value was used. The calculation of the new floor slab can be found in the Floor Slab section. The superimposed dead loads have been taken from the assumed values used in the original design and include the weights associated with MEP equipment and any floor and ceiling finishes.

Dead Loads		
Load Description	**Load Location**	**Design Load (psf)**
Superimposed (MEP)	Levels 1-14	15
Composite Deck	Levels 1-14	37
Curtain Wall	Perimeter	20

Wind

ASCE 7-10 was used for the determination of the wind loads for the Main Wind-Force Resisting System (MWRFS) of Office Building-G. Loads were calculated in the North-South and in the East-West direction due to the roughly rectangular shape of the building. The forces were determined using the Chapter 27 guidelines for Enclosed and Partially Enclosed Rigid Buildings.

The first step in calculating wind loads is determining if the building is flexible or rigid. This classification is based on the natural frequency of the structure. ASCE 7-10 allows for an estimation of a buildings frequency based on relationships between the building height and characteristics of the lateral force resisting system. Through this estimation it was determined that the natural frequency of Office Building-G is less than 1, defining the building as flexible. The gust factored cannot be assumed to be 0.85 for flexible buildings so it was calculated based on the resonant response factor, the fundamental natural frequency, damping ratio and the mean hourly wind speed. The coefficients and values used can be found in Appendix A.

The building is fairly square on three sides but the curved southern façade creates a scenario where the West wall has a greater length than the East wall. If the curvature had been so severe that the West wall was wider than the North wall is deep, an additional wind load would have needed to be calculated. Since this is not the case and L/B < 1 a single wind load calculation can be used for both the East and West loads. For the same reason, the North-South wind loads were calculated using the worst case for the different geometries of the building. The building receives the largest wind force in the North-South directions, as these are the longer façades of the building normal to the wind loading.

Figure 9 shows the geometry of Office Building-G in plan and Figure 10 is a list of the values obtained for the wind forces acting in either direction. Figure 10 shows the wind pressures but a complete list of story shears and base shears based on ASCE 7-10 load cases can be found in Appendix A.

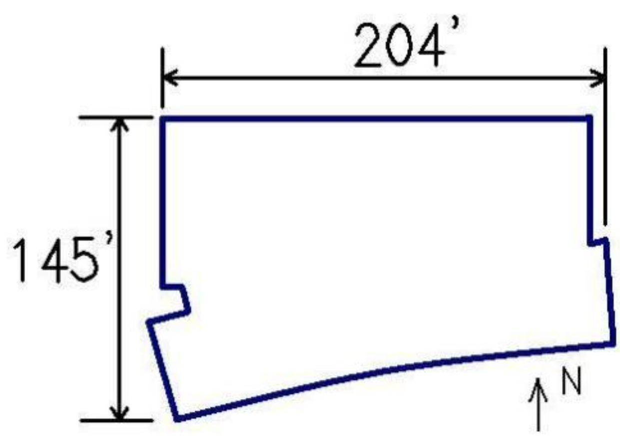


Figure 9

North-South			
Story Level	Story Height (ft)	Windward p_z (psf)	Leeward p_h (psf)
1	0	12.26	-11.86
2	19	12.26	-11.86
3	31.25	14.22	-11.86
4	43.5	15.62	-11.86
5	55.75	16.74	-11.86
6	68	17.72	-11.86
7	80.25	18.69	-11.86
8	92.5	19.44	-11.86
9	104.75	20.09	-11.86
10	117	20.70	-11.86
11	129.25	21.30	-11.86
12	141.5	21.96	-11.86
13	153.75	22.45	-11.86
14	166	22.95	-11.86
ROOF	178.25	23.44	-11.86
EL, MR	186	23.69	-11.86
SCREEN WALL	195	23.96	-11.86

East-West			
Story Level	Story Height (ft)	Windward p_z (psf)	Leeward p_h (psf)
1	0	11.66	-14.25
2	19	11.66	-14.25
3	31.25	13.52	-14.25
4	43.5	14.86	-14.25
5	55.75	15.92	-14.25
6	68	16.86	-14.25
7	80.25	17.78	-14.25
8	92.5	18.49	-14.25
9	104.75	19.11	-14.25
10	117	19.69	-14.25
11	129.25	20.26	-14.25
12	141.5	20.89	-14.25
13	153.75	21.36	-14.25
14	166	21.83	-14.25
ROOF	178.25	22.30	-14.25
EL, MR	186	22.54	-14.25
SCREEN WALL	195	22.79	-14.25

Figure 10

Seismic:

The Equivalent Lateral Force Procedure of ASCE 7-10 was referenced during the calculation of the seismic loads for Office Building-G. General design parameters of the building are a site classification of type D, a seismic design category of B, and a seismic importance factor of 1.0.

The first step taken in determining the seismic forces of the building was to determine the seismic response coefficient; C_S. C_s is based on a variety of factors that take into account the lateral system of the building as well as its geographical. The new lateral system of the building is classified as ordinary concentrically steel braced frames, corresponding to a response modification factor of R=3.25. When determined, C_s can then be multiplied by the total dead load weight of the building to yield the seismic base shear.

The next step was to consider all of the possible areas that could contribute to the dead weight of the building. A typical floor plan was used to determine the dead load of the structure. The building elements considered were: slabs, beams, columns, exterior walls, partitions, and imposed MEP loads. These loads were either a pound per square foot or a total per floor, depending on the nature of the element. It should be noted that partitions are included in a 100 psf live load for office space but since they are secured to the floor of the structure it was assumed that they will not move freely in the instance of an earthquake, becoming a dead load.

Shear forces for each floor were then calculated. Since the lateral system is the same for both loading directions, the seismic forces are the same for the North-South and East-West load cases. When the lateral forces are entered into the load cases, the more severe loads acting on Office Building-G will be used in design. The unfactored seismic forces can be found in Figure 11 below and a complete list of coefficients used can be found in Appendix A.

Seismic Forces					
Story Level	Height (ft)	w_x (k)	w_x*h^k	f_i (k)	V_i (k)
1	0	3700	0	0.0	1444.1
2	19	2038	295304	6.8	1444.1
3	31.25	1882	632324	14.5	1437.3
4	43.5	1645	966252	22.2	1422.7
5	55.75	1645	1469594	33.8	1400.5
6	68	1645	2055813	47.3	1366.7
7	80.25	1645	2719917	62.6	1319.4
8	92.5	1645	3457985	79.5	1256.9
9	104.75	1645	4266816	98.1	1177.4
10	117	1645	5143724	118.3	1079.2
11	129.25	1645	6086410	140.0	960.9
12	141.5	1645	7092869	163.1	820.9
13	153.75	1645	8161333	187.7	657.8

14	166	1645	9290225	213.7	470.1
ROOF	178.25	1549	9869137	227.0	256.4
EL, MR	186	187	1280514	29.5	29.5

Figure 11

Load Cases

ASCE 7-10 section 2.3, Combining Factored Loads Using Strength Design, was used in determining which load cases would be applied to Office Building-G. The load combinations considered are listed below.

1) 1.4(D+F)
2) 1.2(D+F+T) + 1.6(L+H) + 0.5(Lr or S or R)
3) 1.2D + 1.6(Lr or S or R) + (L or 0.5W)
4) 1.2D + 1.0W + L + 0.5(Lr or S or R)
5) 1.2D + 1.0E + L + 0.2S
6) 0.9D + 1.0W
7) 0.9D + 1.0E

Typically, when only gravity loads are being considered, load case 2 will control. However, when lateral forces are being analyzed, cases 4-7 may control based on the magnitude of the forces and whether overturning moment is considered.

Figure 27.4-8 of ASCE 7-10 describes the different loading conditions for wind on a building. All four of the cases for the Main Wind Force Resisting System must be considered in the analysis of the lateral system. These cases account for the effects that wind has on a structure when wind blows from two different directions and are applied slightly off access. As shown in Figure 12, Cases 2 and 4 consider the torsional loads that can be induced by wind loading.

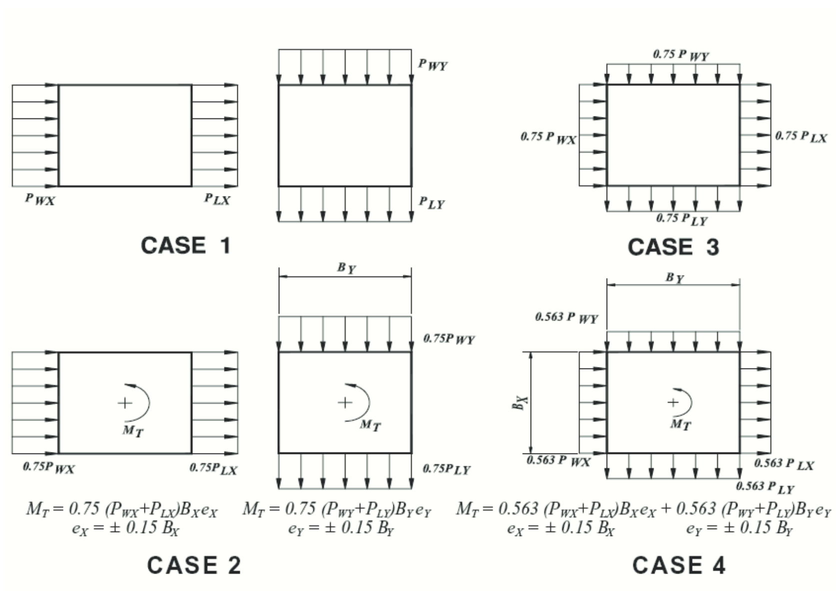


Figure 12

In total, there are 12 different wind loads that Office Building-G can expect to experience. To account for these possibilities, 12 iterations of each of the above load combinations which included a wind component were input into ETABS. The multiple loads cases for wind and earthquake forces changed the number of load combinations from 7 to 43. A complete list of the load combinations, forces used and the confirming wind calculations for Office Building-G can be found in Appendix A.

Floor System

Lightweight concrete on composite metal deck was chosen for the redesigned floor system of Office Building-G. The existing 7” one way slab design would have been sufficient for the proposed system but metal deck was determined to be more economical based on weight, materials, and construction costs. The deck chosen was Vulcraft 2VLI19 and the strength values are shown in Figure 13. The loads accounted for in the design of the floor system were an unreduced live load of 100 psf and a superimposed dead load of 15 psf. The 2VLI19 system can support up to 151 psf and can span 11’-7” unshored during construction, eliminating costs associated with shored construction.

(N=14.15) LIGHTWEIGHT CONCRETE (110 PCF)

TOTAL SLAB DEPTH	DECK TYPE	SDI Max. Unshored Clear Span			Superimposed Live Load, PSF Clear Span (ft.-in.)														
		1 SPAN	2 SPAN	3 SPAN	6'-0	6'-6	7'-0	7'-6	8'-0	8'-6	9'-0	9'-6	10'-0	10'-6	11'-0	11'-6	12'-0	12'-6	13'-0
	2VLI22	8'-1	10'-3	10'-7	238	209	186	167	152	120	108	98	90	82	75	69	64	59	55
4.00	2VLI20	9'-6	11'-8	12'-1	268	235	209	187	169	153	140	129	101	92	84	78	72	66	61
(t=2.00)	2VLI19	10'-10	13'-0	13'-2	297	260	230	206	185	168	153	141	130	121	93	86	79	73	68
30 PSF	2VLI18	11'-7	13'-7	13'-7	324	285	253	227	205	187	171	158	146	136	127	119	92	86	80
	2VLI16	12'-3	14'-3	14'-4	377	330	292	261	235	214	195	179	165	153	143	133	118	98	91
	2VLI22	7'-8	9'-10	10'-2	276	243	216	194	155	139	126	114	104	96	88	81	75	69	64
4.50	2VLI20	9'-0	11'-3	11'-7	312	273	243	217	196	178	163	128	117	107	98	90	83	77	72
(t=2.50)	2VLI19	10'-3	12'-5	12'-9	346	302	268	239	215	195	178	164	151	118	108	100	92	85	79
35 PSF	2VLI18	11'-2	13'-1	13'-1	376	331	294	264	238	217	199	183	170	158	147	116	107	100	93
	2VLI16	11'-7	13'-8	13'-10	400	384	340	303	273	248	227	208	192	178	166	155	123	114	106
	2VLI22	7'-4	9'-5	9'-9	315	277	247	197	176	159	143	130	119	109	100	92	85	79	73
5.00	2VLI20	8'-7	10'-9	11'-2	355	312	276	248	224	203	161	146	133	122	112	103	95	88	82
(t=3.00)	2VLI19	9'-9	11'-11	12'-4	394	345	305	272	245	223	203	187	147	135	124	114	105	97	90

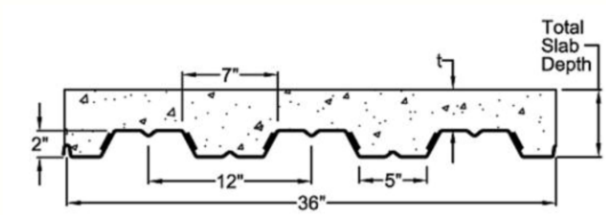
COMPOSITE

Figure 13

The self-weight of the lightweight concrete is 35 psf. This weight is added to the 2.30 psf weight of the metal deck, resulting in total system load of 37.3 psf.

2 VLI

Maximum Sheet Length 42'-0
Extra Charge for Lengths Under 6'-0
ICBO Approved (No. 3415)



Interlocking side lap is not drawn to show actual detail.

STEEL SECTION PROPERTIES

Deck Type	Design Thickness in	Deck Weight psf	Section Properties I_p in^4/ft	S_p in^3/ft	I_n in^4/ft	S_n in^3/ft	V_a lbs/ft	F_y ksi
2VLI22	0.0295	1.62	0.324	0.263	0.321	0.266	1832	50
2VLI20	0.0358	1.97	0.409	0.341	0.406	0.346	2698	50
2VLI19	0.0418	2.30	0.492	0.420	0.489	0.426	3190	50
2VLI18	0.0474	2.61	0.559	0.495	0.558	0.504	3608	50
2VLI16	0.0598	3.29	0.704	0.653	0.704	0.653	3618	40

Figure 14

Framing Plan

The composite metal deck distributes the floor loads between the framing elements of Office Building-G. As shown in Figure 15 the metal deck spans East-West across the framing members spaced at 10' OC.

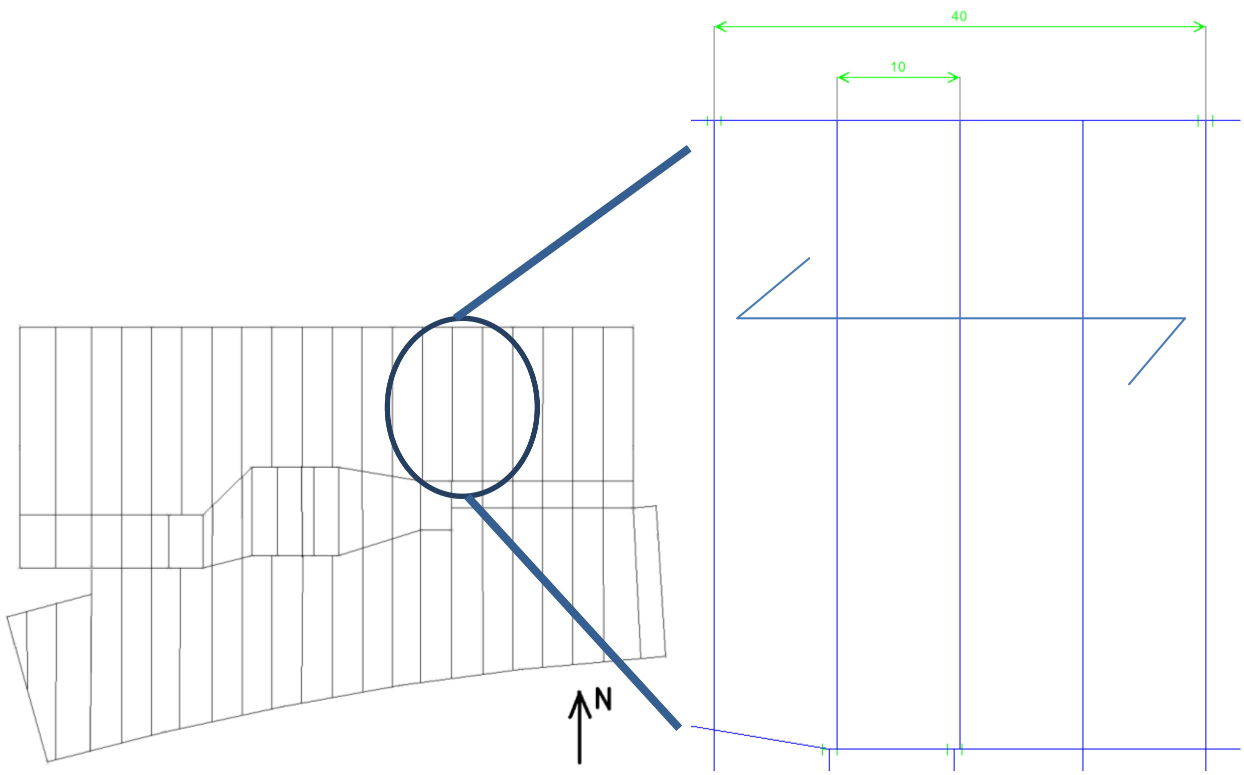


Figure 15

Each story of Office Building-G follows a typical plan which allows a single framing plan to be used for every level. The majority of the framing members are composite castellated beams, span in the North-South direction of Office Building-G and are supported by composite edge beams and core framing. Transfer girders were used on the East and West ends of the building to reduce the span of the main framing members. The castellated beams are spaced at 10 ft on-center, limiting their tributary width in an effort to create manageable bending forces. Figure 16 is a typical floor plan. Green lines represent the core framing elements; blue lines are the transfer girders and red x's show penetrations through the floor for vertical transportation.

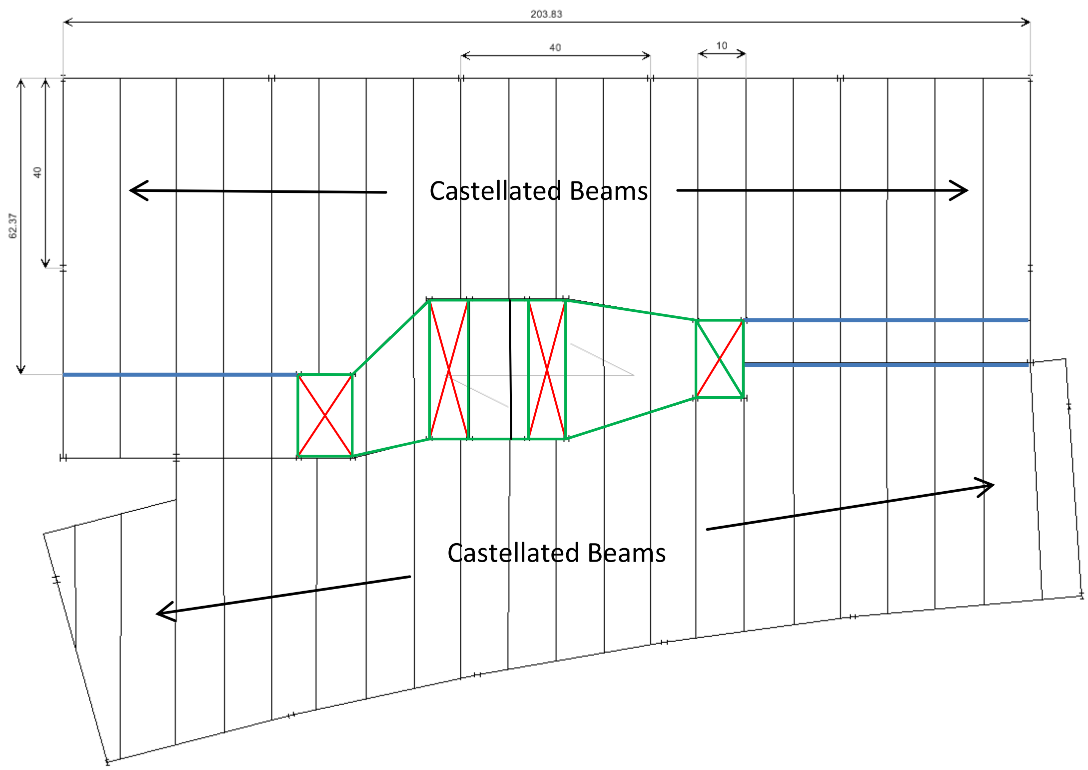


Figure 16

Castellated Beams

The main framing elements of Office Building-G are composite castellated beams. This system was chosen because it is able to fit within the existing ceiling cavity of Office Building-G as well as accommodate the long spans of the structure. The spans and high floor loads were resulting in very deep and heavy members when conventional composite design was used. The castellated beams are roughly the same depth as the typical composite designs but they are able to achieve the necessary bending strength with a lower weight per foot. The voids of castellated beams allow for MEP equipment to pass through the web of the beam, creating an efficient use of the ceiling cavity. Due to serviceability requirements of the duct, the beam designs were based on the opening size rather than required strength, creating a less structurally economical design. Figure 17 is a typical section of a composite castellated beam design showing the mechanical duct work pass through the beam opening.

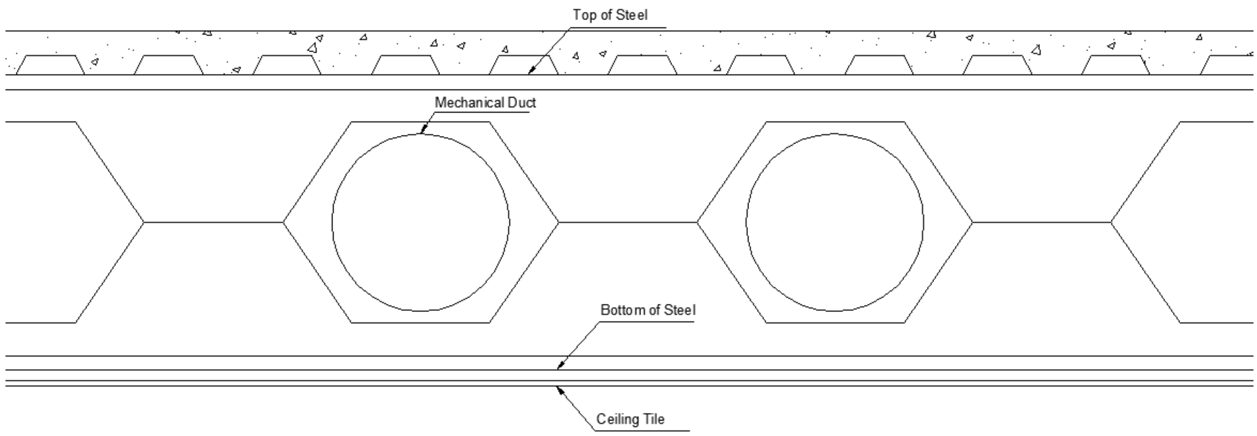


Figure 17

Figure 18 displays the process in which castellated beams are created. Drawing a) shows the original beam with the line in which it is going to be cut. The beam is then separated and realigned to create a deeper member as shown in Drawing b) and c). As depicted in the figure, the shaded region at the end of the beam is discarded as waste. Finally, the two beam halves are then welded together to create the final shape as shown in Drawing d).

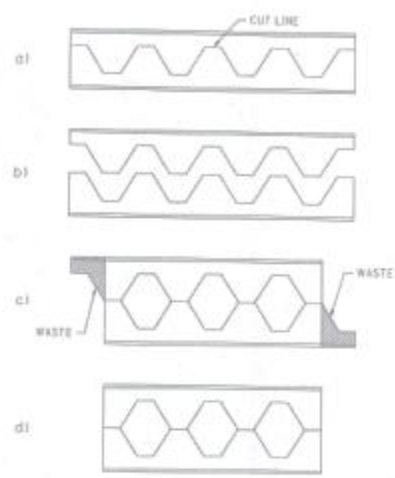


Figure 18

Castellated beams designs are controlled by the effects of Vierendeel Bending. Vierendeel bending occurs when the global bending moment causes a localized compressive and tensile force in the top and bottom cords of the member, known as a primary force. At the location of the opening, secondary forces are created by the shear on the beam. The secondary forces increase the stress experienced by the top and bottom of the beam sections. The castellated beam must be able to withstand the combination of both primary and secondary forces. These forces change at every opening so each must be analyzed to determine the interaction between shear and moment on the section. Figure 19 shows the primary forces acting on a generic composite castellated beam.

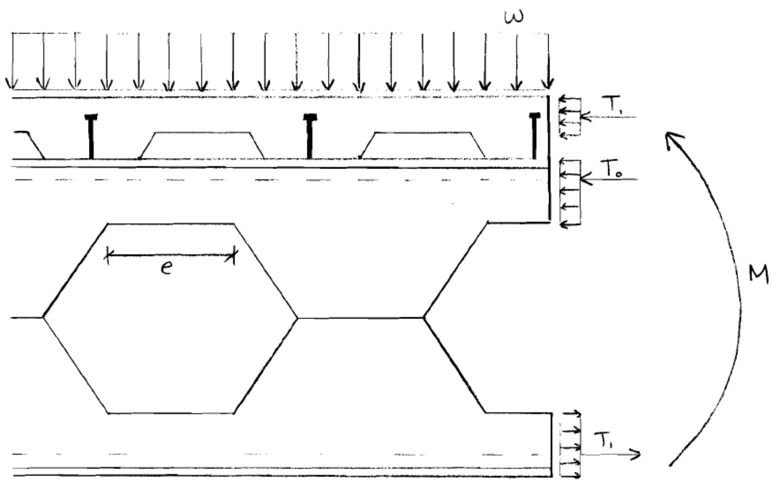


Figure 19

Global moments, M, are calculated based on the net shear at the opening multiplied by the distance from the closest support to the centerline of the opening. The global moments are summed at every opening, resulting in the maximum moment at the midspan of the beam. Local axial forces in the top and bottom tee sections are then calculated to determine the effective depth of the concrete section.

Figure 20 is a larger detail of the top section of a generic castellated beam, displaying the way in which secondary forces act. These forces are calculated over the centerline of the openings. The top and bottom sections resist half of the net shear acting on the beam. Net shear is calculated by subtracting the shear resisted by the concrete from the total. The reduced section is effectively acting as a beam spanning between the solid sections. Thus, the vertical force acting on the reduced section creates a local moment which maximized at the midspan of the opening. The local moment is calculated by the equation shown on Figure 13.

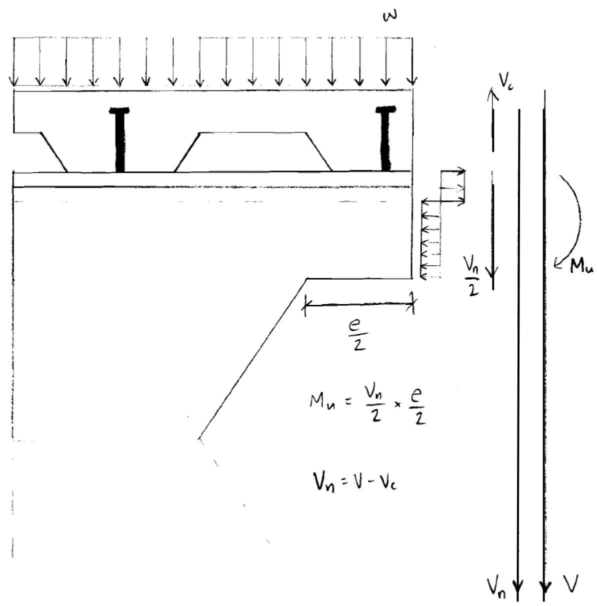


Figure 20

Primary and secondary forces must be calculated at every opening of a castellated beam due of the relationship between total shear and global moment. At the end of a simply supported beam with a uniformly distributed load there is a large shear force but a negligible global moment. A large shear force on a castellated beam creates large secondary forces while a small moment creates minimal primary forces. At midspan, the beam must resist a large global moment but a very small shear. Figure 21 is an example calculation of how global forces and local forces change for the openings of a castellated beam. Since a single beam section was used for the creation of the castellated beams, the local forces experienced by the top and bottom beam sections are identical. If the design used different beam sections, the forces would be distributed based on the relative stiffness of the elements.

Primary and Secondary Forces			
Hole #	Net Shear V_u	Global Moment M (kip-ft)	Local Moment M_u-top (kip-in)
End	55.5	0.0	138.7
1	52.2	98.0	130.6
2	47.4	234.8	118.4
3	42.5	359.5	106.2
4	37.6	472.0	94.1
5	32.8	572.3	81.9
6	27.9	660.4	69.7
7	23.0	736.3	57.5
8	18.1	800.1	45.3
9	13.3	851.7	33.2
10	8.4	891.1	21.0
11	3.5	918.3	8.8

12	0.0	933.3	0.0
CL	0.0	936.1	0.0

Figure 21

The above diagrams and forces represent the way in which the beam section is affected by Vierendeel bending. A composite castellated beam must then be checked for: tension, moment, lateral torsional buckling, flange local buckling, web post buckling, shear, and deflection. A complete set of these calculations can be found in Appendix B.

Due to the relationship between shear and moment in castellated beams described above, large point loads acting on a beam often create too great of secondary forces and result in an uneconomical design. This is why castellated design was not used for the transfer girders, edge beams or interior framing members.

Multiple castellated beams were designed based on the required span of the beam. Slight differences in length did not change the original W shape so beams were designed in 5 foot increments. Figure 22 is a list of the castellated designs with their original size, adjusted depth, size of opening and length.

Castellated Beam Designs						
Design #	**Max Length (ft)**	**# Beams/Floor**	**Size**	**Final Depth (in)**	**Opening Height (in)**	**Max Deflection (in)**
1	65	7	W18X158	29.9	20.4	0.29
2	60	3	W18X143	30	21	0.23
3	55	11	W18X130	30.1	21.6	0.18
4	50	19	W18X119	29.5	21	0.14

Figure 22

Composite Beams

At the ends of the castellated beam spans, there are large shear reactions. As mentioned above, point loads create uneconomical castellated designs. For this reason composite design was used for the transfer girders, edge beam or interior framing members.

The initial design of these members was performed by RAM Structural System with no depth limitation applied. The design was then checked by hand with the aid of Tables 3-19 and 3-20 of the AISC Steel construction manual. These hand calculations verified the process taken by RAM Structural system. Due to the depth limitation of the ceiling cavity, the most economical beam size could not be utilized so RAM was used to check the designs of shallower, heavier members. Sample calculations can be found in Appendix B.

RAM Structural System was used to design all of the regular composite framing members. Due to modeling limitations of RAM the beam layout is not identical to that of the final design but the load path is accurate and this limitation did not have an effect on the design of these members. The RAM model shown in Figure 23 only shows the core of Office Building-G because the edge beams are typical and the longer span members were designed by hand using the castellated design methods described above. The hand calculations and values given by RAM Structural System can be found in Appendix B.

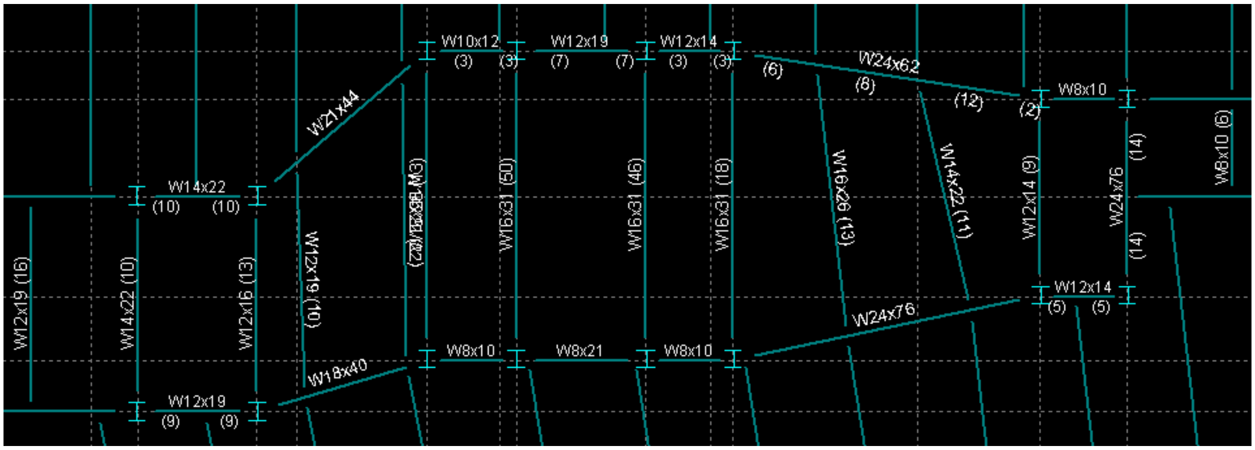


Figure 23

Figure 24 below is a typical floor plan which depicts which beams were designed as castellated in green and those which are regular composite in red. Also shown are the designs of typical edge beams and the transfer girders controlled by the depth limitation.

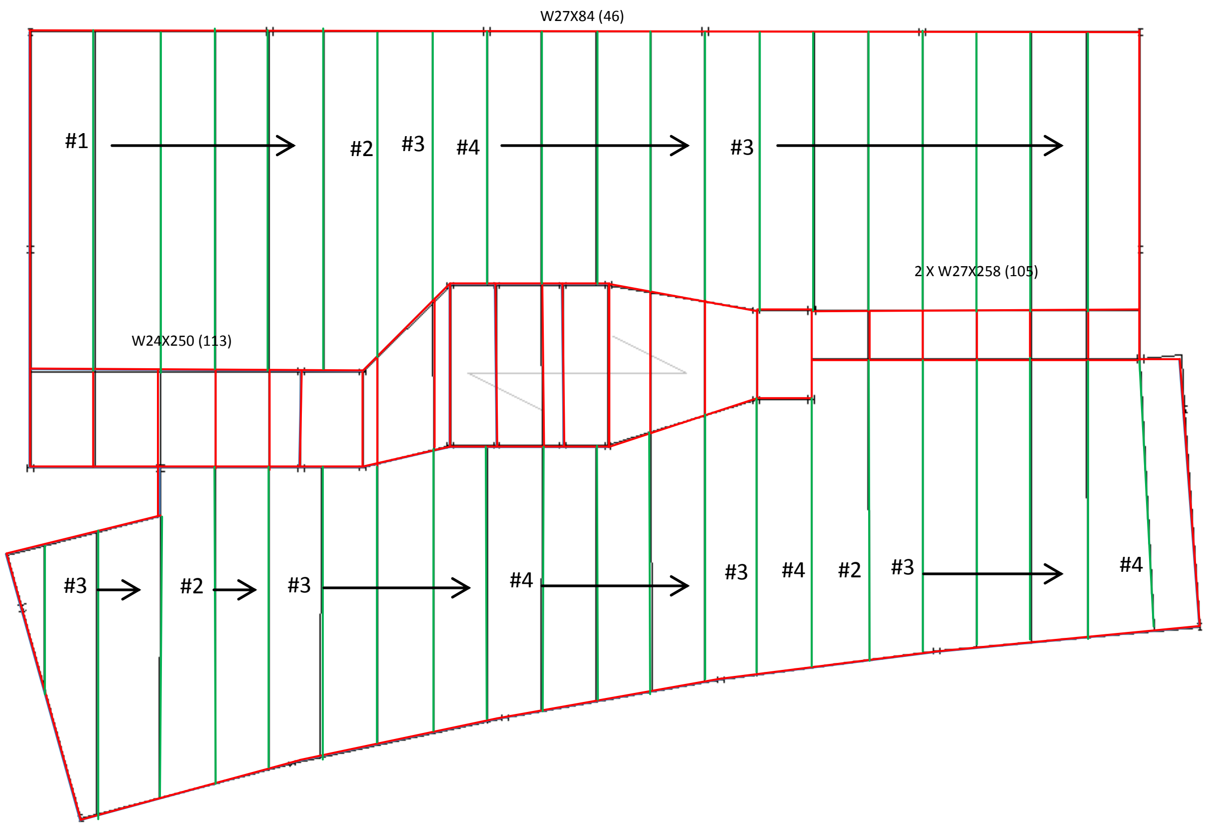


Figure 24

All of the castellated beams were designed to allow for an 18" diameter duct with 1" insulation to pass through them. This limited the factors controlling the design and allowed the beams to be laid out based solely on the span between supports.

Columns

In order to create a column free space for tenant fit outs, all of the exposed interior columns in the original design were removed. Additionally, roughly half of the columns which were on the perimeter were removed and the remaining columns were relocated to the façade of Office Building-G. Figure 25 is the original floor plan which shows the columns which were removed circled in red.

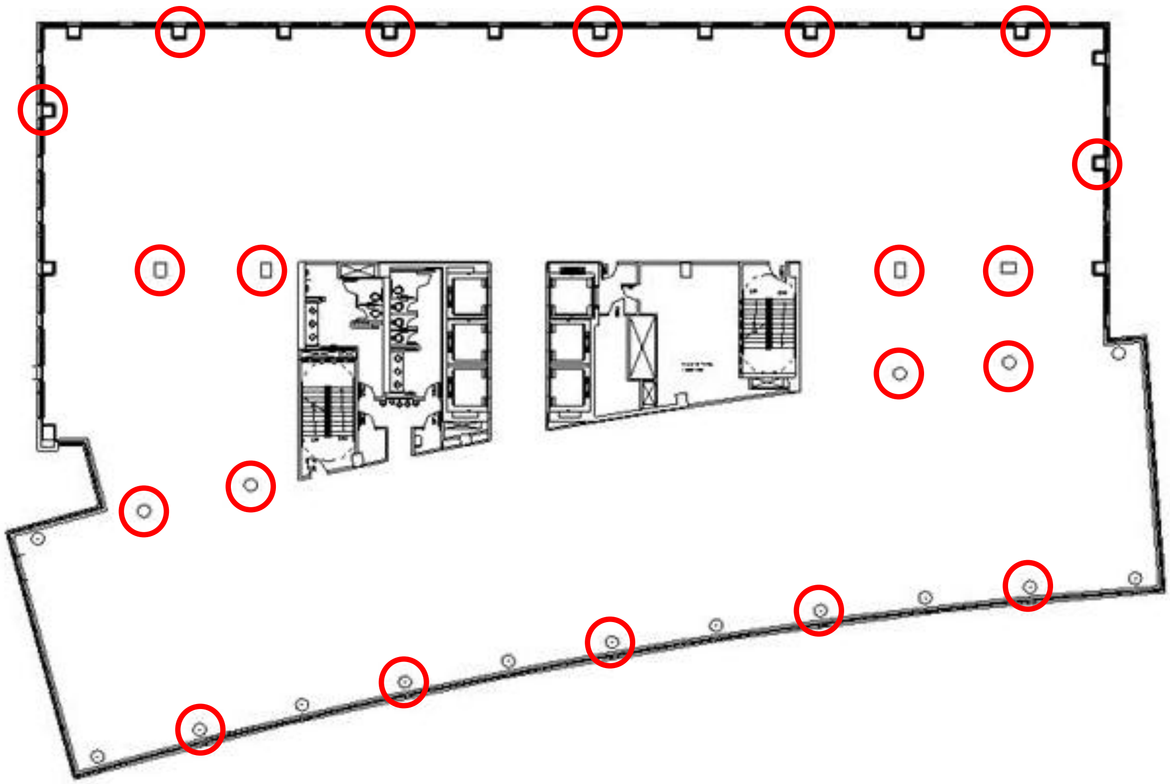


Figure 25

The original design had 29 perimeter columns, 8 interior columns and a shear wall core resisting the gravitational forces. The final design has 19 perimeter columns and 16 interior columns with no structural walls. Figure 26 is a typical floor plan of Office Building-G and displays the new column layout. Columns circled in blue represent columns which are part of the lateral system so they were designed based on gravity and lateral load cases. The remaining columns are strictly gravity members. The orange perimeter represents the tributary area used to check base column loads.

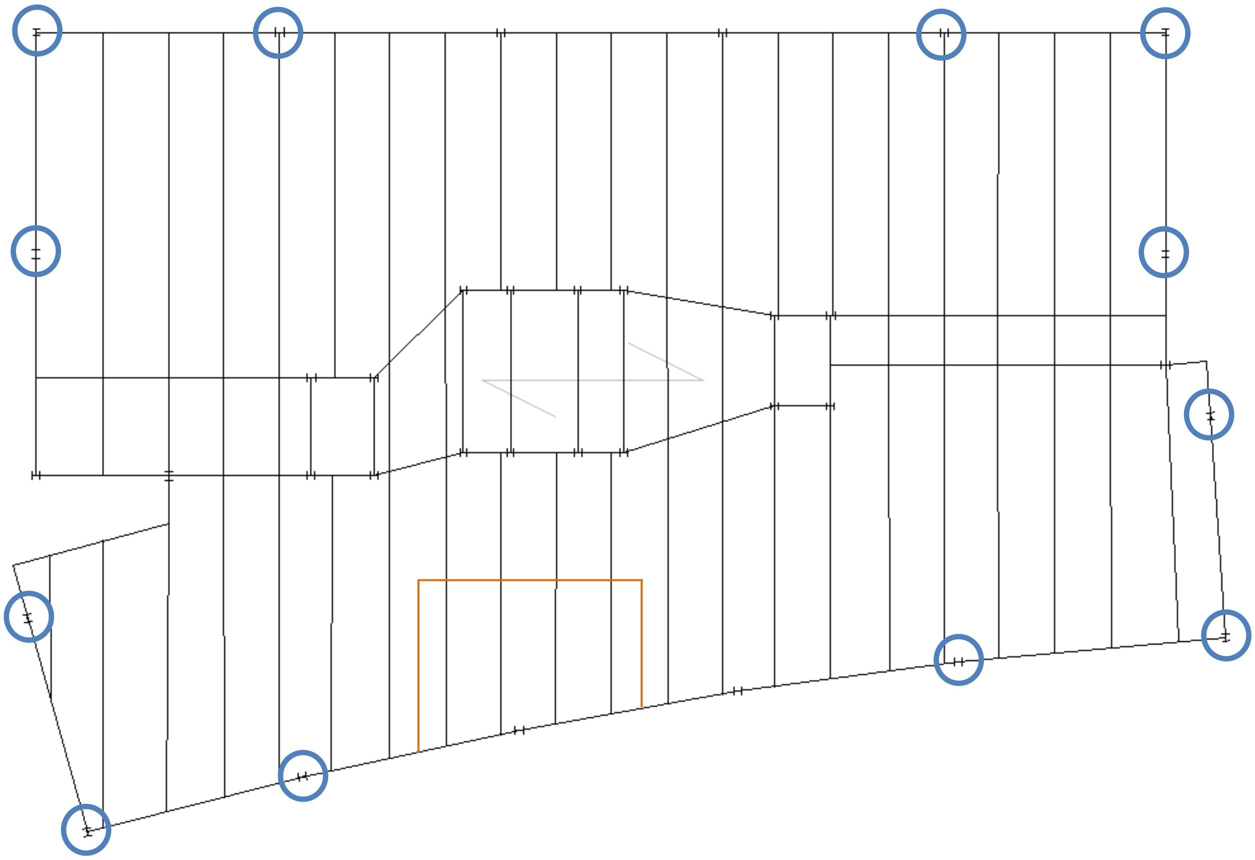


Figure 26

Gravity columns were initially sized through the use of RAM Structural System based on purely axial loads. The summation of loads calculated by RAM was checked through hand calculations to confirm the load path, reduction of live load, and column sizes. These can be found in Appendix C. The initial sizes were then put into ETABS to analyze the influence of p-delta effects on the gravity columns. It was determined through the ETABS analysis and hand calculations that the small story displacements limit the secondary moment applied by the columns and have little effect on the design of the members. The controlling load case on gravity columns was 1.2D + 1.6L.

Columns which are part of lateral force resisting fame were also put into ETABS and resized based on the axial forces they could experience. The controlling load cases for lateral members depended on the direction of the frame but each direction was controlled by 1.2D + 1.0L + E. These members are discussed in greater detail in the Lateral System Section.

Based on the length of steel allowable for transportation speed of construction, columns were designed for a splice every 4 stories. The 14 story design of Office Building-G leaves a 2 column design for the 13th and 14thfloor. In an effort to avoid interference of pouring the concrete and column connections the splicing between columns should be done at 48 inches above the story level. Additionally, columns were designed to be W14 sections to create simple splice details. Figure 27 is a list of the columns used in the redesign of Office Building-G.

Columns	
Section	**NumPieces**
W14X53	40
W14X61	48
W14X68	22
W14X74	14
W14X82	50
W14X90	40
W14X99	32
W14X109	42
W14X120	16
W14X132	38
W14X145	38
W14X159	28
W14X176	14
W14X193	8
W14X211	4
W14X233	24
W14X257	12
W14X283	4
W14X311	4
W14X342	4
W14X370	8

Figure 27

As seen in the framing plan in Figure 26, there are sixteen (16) centrally located columns. These columns are essential for the redesign of Office Building-G's structure to be integrated with the existing architecture. Without interior column the span of the castellated beams would be too large to design a member capable maintaining the existing depth of the cavity space. The interior columns were placed at the corners of the existing vertical transportation shafts in order to be integrated with the existing floor plan. The columns reduce the span of the main framing elements while eliminating the mass of the shear walls.

Lateral System

Geometry

In an effort to create an open floor plan for Office Building-G, it was proposed to implement a system of external steel braced frames. In the initial design stages, a variety of different braced frame geometries were considered. These initial designs made use of different column spacing and frame type. Due to the large architectural impact the exterior frame was going to have on the building, the final design was chosen based on aesthetics. Figures 28 through 31 are images of the different brace designs considered for the North Façade of Office Building-G. Figure 32 is the final design geometry.

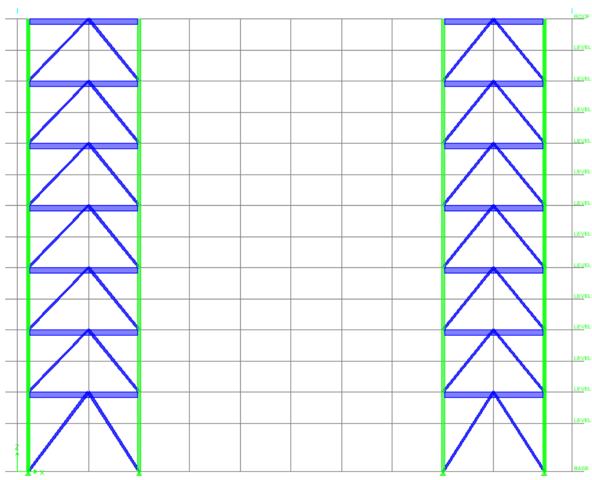


Figure 28

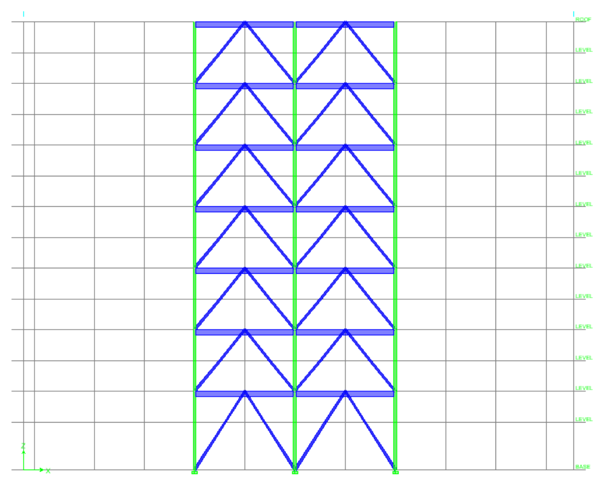


Figure 29

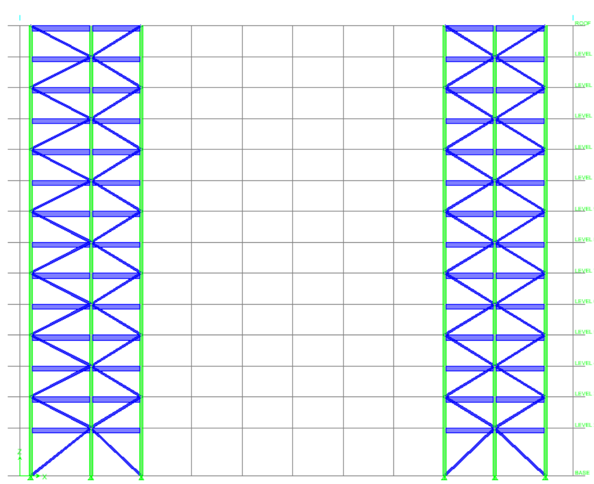


Figure 30

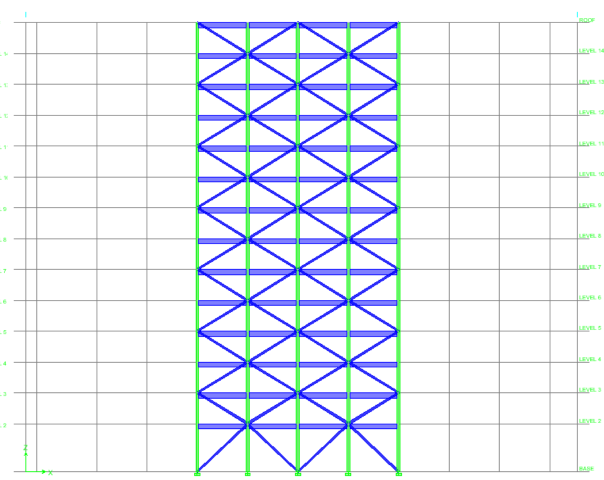


Figure 31

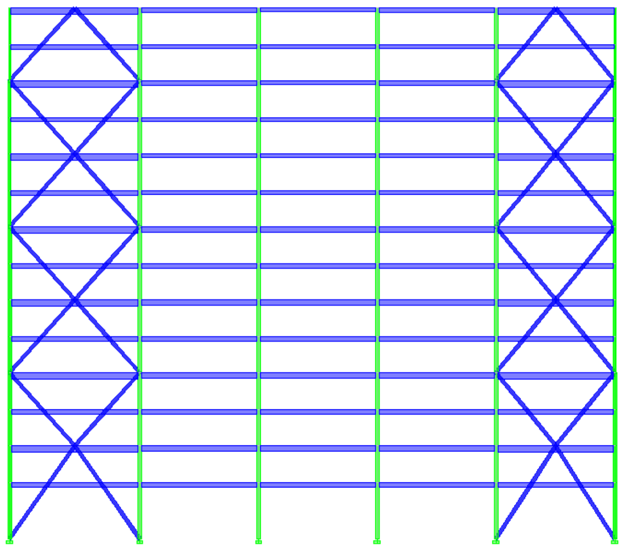


Figure 32

The lateral redesign of Office Building-G uses a system of two-story chevron braces which span 40 feet between columns and alternate between normal and inverted frames, creating an X every four stories. The frames are located at the corners of the building, creating a total of 8 frames, shown in 3D in Figure 33 and highlighted in plan in Figure 34.

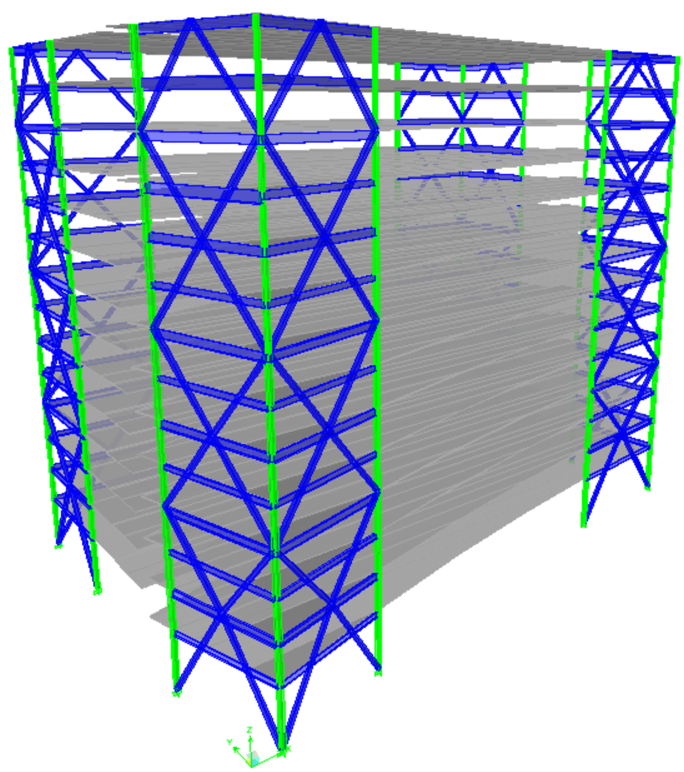


Figure 33

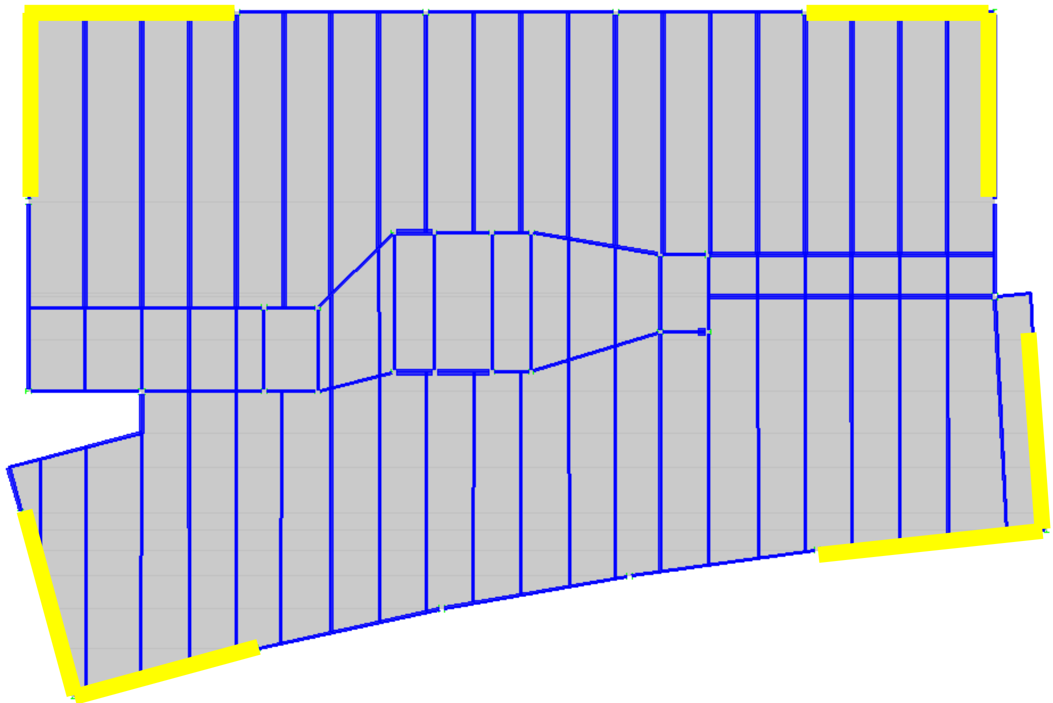
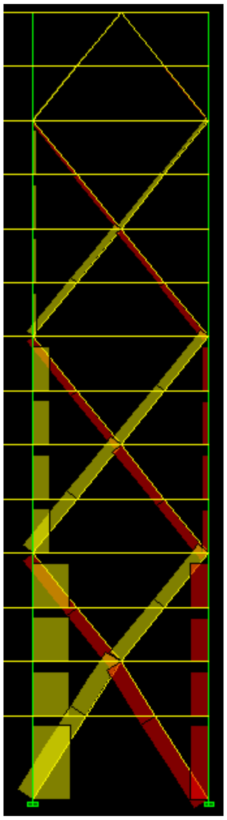


Figure 34

Design

When a chevron frame is loaded with a horizontal force, the load enters the beam of the frame through the shear studs which were engaged by the floor diaphragm. The load is then distributed to the braces and based on the direction of the loading one brace is put in tension while the other is in compression. The tension and compression axial forces of the braces are then transferred to the columns as a gravity load. Figure 35 displays the axial loads on the braces and columns of the highlighted frame due to wind loads on the North Elevation. The yellow force is tension and red is compression.



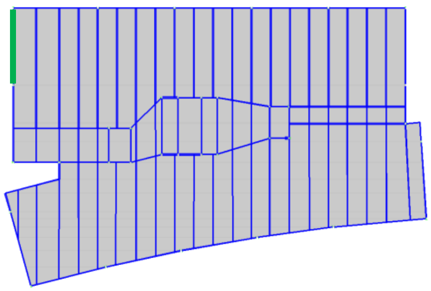


Figure 35

To determine initial member sizes of frame members, a bottom story frame was designed by hand following the provisions described in the AISC Seismic Manual. Columns and beams were designed using W shape members and braces were designed as square HSS. Tributary area estimates were used to determine the gravity loads resisted by the frame and the 1/4th of the seismic base shear was applied as the lateral load. The hand calculations checked the applicable members for compression, tension, axial, local buckling, slenderness and moment capacity. These calculations can be found in the Appendix D.

These initial member sizes were then put into an ETABS model for every frame. A design check within ETABS was performed with the horizontal seismic overstrength factor (Ω_o) and redundancy factor (ρ) included. For the design conditions of Office Building-G these are 2.0 and 1.0 respectively. This check resulted in many of the members being oversized. This was expected because the shear at the bottom floor is much higher than that of the floors above. The oversized members were then resized to create and efficient design. Brace sizes were checked with the aid of ETABS which ran interaction checks for combination loading on each member. Seismic forces control the design forces of the braces in both directions. The controlling load case in either direction is 1.2D + 0.5L + 0.5Lr + 1.0E. A sample of the results given by ETABS for one of the brace designs is shown below in Figure 36.

```
AISC-LRFD93 STEEL SECTION CHECK    Units: Kip-ft  (Summary for Combo and Station)
Level: LEVEL 4  Element: D211  Station Loc: 15.813  Section ID: HSS14X14X.500
Element Type:  Moment Resisting Frame   Classification:  Non-Compact

L=15.813
A=0.171  i22=0.036  i33=0.036  z22=0.072  z33=0.072
s22=0.061  s33=0.061  r22=0.458  r33=0.458
E=4176000.000  fy=7200.000
RLLF=1.000

P-M33-M22 Demand/Capacity Ratio is   0.738 = 0.714 + 0.021 + 0.002

STRESS CHECK FORCES & MOMENTS
                              P           M33         M22          V2          V3
    Combo DSTLS72      -684.261       -11.009       1.151       1.238      -0.073

AXIAL FORCE & BIAXIAL MOMENT DESIGN   (H1-1a)
                             Pu      phi*Pnc      phi*Pnt
                           Load     Strength     Strength
    Axial               684.261      958.221     1107.000

                             Mu       phi*Mn       Cm      B1      B2       K       L      Cb
                         Moment     Capacity   Factor  Factor  Factor  Factor  Factor  Factor
    Major Bending        11.009      459.873    1.000   1.131   1.000   1.000   1.000   1.981
    Minor Bending         1.151      459.873    0.600   1.000   1.000   1.000   1.000

SHEAR DESIGN
                             Vu       Phi*Vn      Stress
                          Force     Strength       Ratio
    Major Shear           1.238      351.540       0.004
    Minor Shear           0.073      351.540       0.000
```

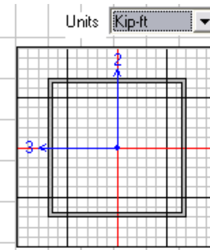


Figure 36

It is important to note a modeling technique used in the analysis of the two story frame in ETABS. In braced frames, the lateral loads enter the frame through the beam. However, the chevron frames of Office Building-G span two stories so the interstory forces have no way of entering the frame. Because of this, a moment connection between the HSS braces and the interstory edge beam was modeled. Figure 37 is a picture of a typical chevron brace with the end moment releases displayed. Note the connection between the interstory edge beam and brace members have no releases.

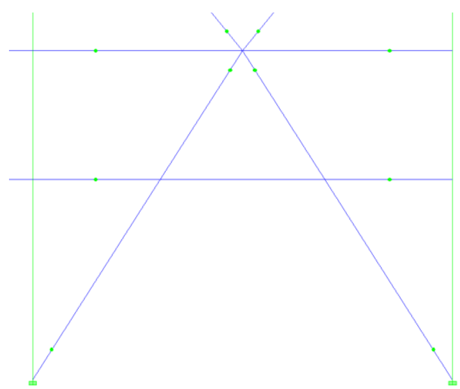


Figure 37

Due to the large amount of symmetry in the design of Office Building-G, there are typical brace and beam sizes for frames which resist loads in the same direction. Column sizes used as part of the frames change throughout the building based on the load path of the framing system. Figures 38 and 39 show a typical frame design for resisting loads in the East-West direction and North-South direction respectively.

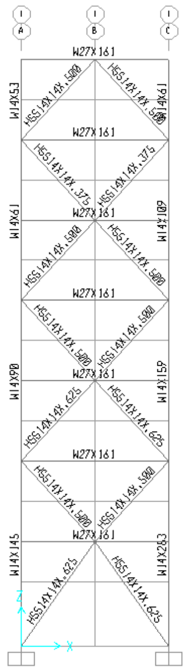


Figure 38

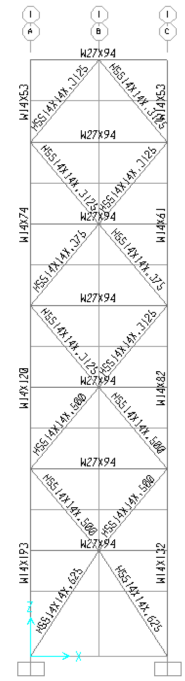
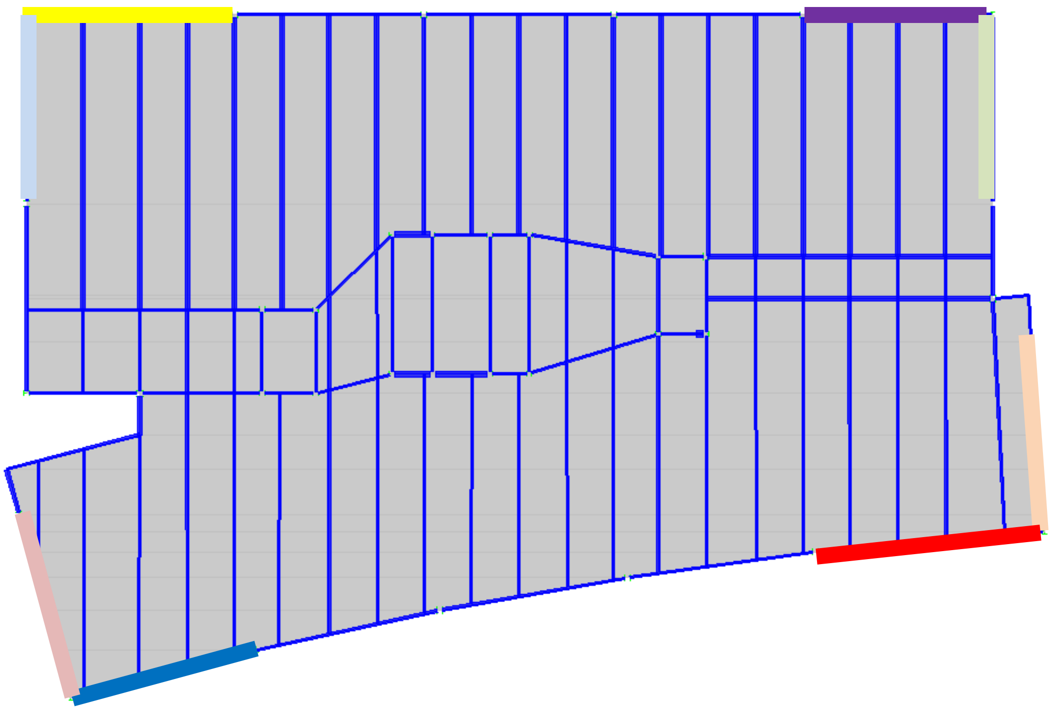


Figure 39

The greatest difference in member sizes of the frame designs are the beam members. Due to the orientation of the castellated beams, the frames on the North and South façade take a much larger gravity load than those of the East and West facade.

Direct Loads

When horizontal forces are applied to a structure, the lateral resisting elements are loaded with direct shear. The distribution of these forces is based on the relative stiffness of the elements. The frames in Office Building-G are designed with minimal differences in them so it is expected that they have similar stiffness's. The stiffness of each frame was calculated with the use of the analysis program SAP 2000. The as designed frames were put into SAP and a 1000 kip force was applied to the top of the frame, causing the frame to deflect. The stiffness was then calculated by dividing the 1000 kip load by the total deflection. Figure 40 is color coded to show the stiffness as well as relative stiffness of the frames in Office Building-G.



Stiffness and Relative Stiffness						
	Brace		Load (k)	Deflection (in)	K	K Rel.
	Elevation	Location				
Resisting E-W Forces	North	NW	1000	20.7	48.30918	0.267468
		NE	1000	26.95	37.10575	0.205439
	South	SW	1000	24.2	41.32231	0.228785
		SE	1000	18.56	53.87931	0.298308
				Total	180.6166	1
Resisting N-S Forces	West	NW	1000	27.23	36.7242	0.269429
		SW	1000	27.98	35.73981	0.262207
	East	NE	1000	29.81	33.54579	0.246111
		SE	1000	33.01	30.29385	0.222253
				Total	136.3037	1

Figure 40

As mentioned above, the frames are made of similar members and have roughly the same dimensions so it is expect that they have similar stiffness's. Those frames with larger k values, such as the frame highlighted in red, have a larger span between columns. The brace members are still connected at mid span of the beam and thus have become more horizontal, allowing the maximum horizontal component of the axial force to increase.

When a lateral load is applied to Office Building-G, the majority of the resultant force in the frames is due to direct shear. The actual distribution of forces can be estimated by the relative stiffness of the frame members. Figure41 is a chart which takes the direct wind loads of Office Building-G and compares the expected forces to the actual.

Wind Loading: E-W Direction, 1st Story					
Brace		**Relative Stiffness**	**Predicted Load (k)**	**Actual Load (k)**	**% Difference**
Elevation	**Location**				
North	NW	0.27	210	189	10.80
	NE	0.21	161	171	5.44
South	SW	0.23	180	174	3.29
	SE	0.30	234	207	12.59

Total Applied Shear (k)	786.00
Total Resisted Shear (k)	740.00

Wind Loading: N-S Direction, 1st Story					
Brace		**Relative Stiffness**	**Predicted Load (k)**	**Actual Load (k)**	**% Difference**
Elevation	**Location**				
West	NW	0.27	286	264	7.86
	SW	0.26	278	257	7.83
East	NE	0.25	261	254	2.67
	SE	0.22	236	255	7.91

Total Applied Shear (k)	1060.00
Total Resisted Shear (k)	1030.00

Figure 41

As expected, the distribution of the force was proportional to the relative stiffness of the frame members. However, there are differences in the expected value versus actual as well as the total applied shear versus the total resisted shear. These differences in values can be explained through the effects of inherenet torsional loads.

Torsional Loads

Inherent torsion occurs when the center of mass and center of rigidity do not directly line up, creating an eccentricity. The center of rigidity is based on the location and relative stiffness of the lateral resisting members. Figure 42 compares the location of center of mass and center of rigidity and calculated the eccentricity in each direction. The hand calculations of center of rigidity can be found in Appendix D.

Center of Mass vs. Center of Rigidity							
CM		CR				e_x (ft)	e_y(ft)
X	Y	Hand		ETABS			
		X	Y	X	Y		
100.4	83.3	102	70	106.6	80.7	6.2	2.6

Figure 42

Loads applied in the East-West (ie the X) direction are applied at the center of mass and multiplied by the e_y length which is acting as a moment arm extended from the center of rigidity. A torsional moment acts on the structure which induces a shear on every frame in Office Building-G. These induced shears can act in the same direction as the direct shear, creating an additive force or in an opposite direction which reduces the shear seen by the brace. Figure 43 shows an example of direct shear outlined in blue with the reactions as solid blue arrows. The inherent torsion is the outlined orange arrow with the induced shears shown as solid orange arrows.

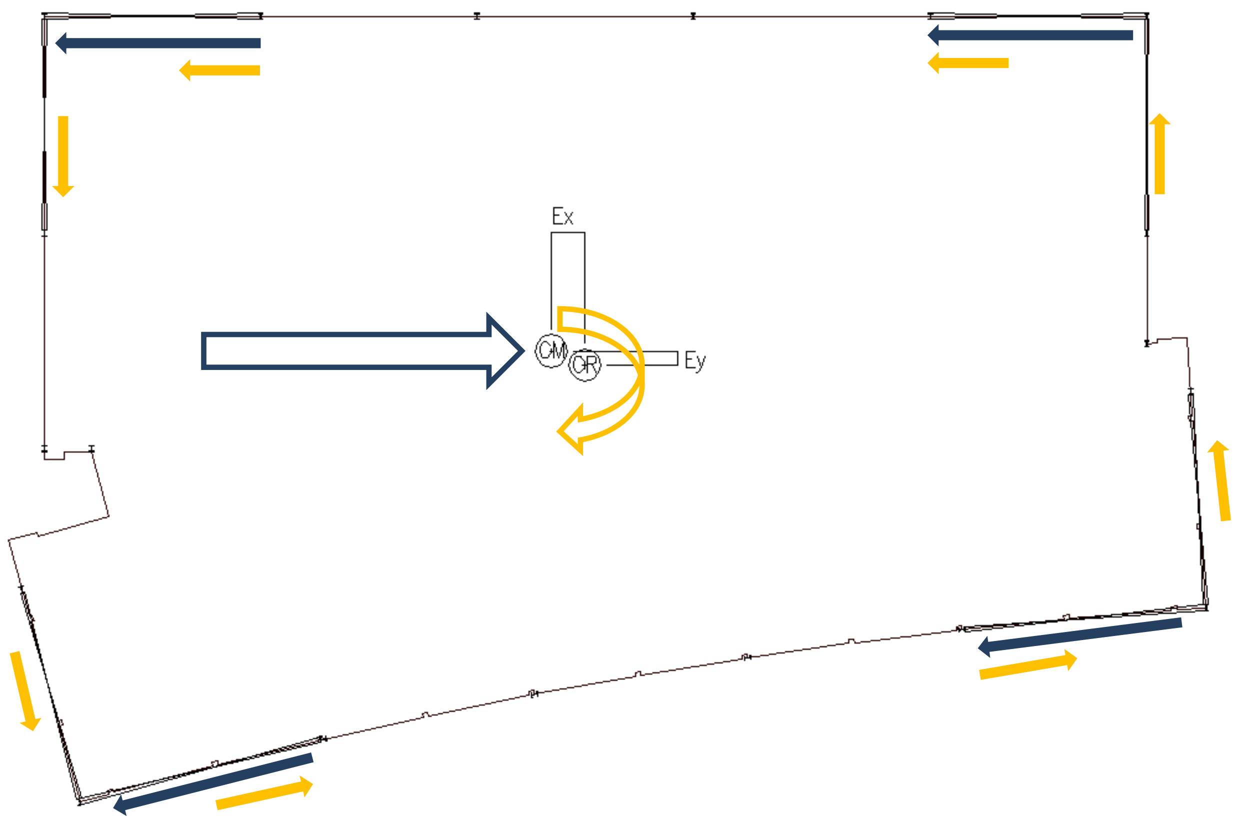


Figure 43

As mentioned above, torsional loading had little effect on the redesign of Office Building-G. Due to the symmetry of the floor plan and the lateral system, the floor center of mass and the structure center of rigidity are very close to each other limiting the effect of eccentric loading.

Displacement and Drift

Total displacement of building is considered a serviceability requirement due to the undesirable sensation of a building swaying back and forth. Wind drift limitations are not directly addressed in building requirements but have been limited to H/400, based on standard engineering practice. Figure 44 is a summary of the allowable story drift compared to the maximum drift values of the load cases acting on Office Building-G. The actual deflections were calculated by ETABS.

Story Deflections (in)										
Story	Height (in)	H/400	Deflections: East-West Loading				Deflections: North-South Loading			
			Seismic Max		Wind Max		Seismic Max		Wind Max	
			UX	UY	UX	UY	UX	UY	UX	UY
ROOF	2139	5.35	3.75	-0.39	1.34	-0.22	-0.30	5.18	-0.16	2.44
LEVEL 14	1992	4.98	3.54	-0.34	1.27	-0.19	-0.27	4.83	-0.15	2.30
LEVEL 13	1845	4.61	3.16	-0.30	1.15	-0.17	-0.24	4.33	-0.13	2.10
LEVEL 12	1698	4.25	2.84	-0.25	1.06	-0.14	-0.21	3.89	-0.12	1.93
LEVEL 11	1551	3.88	2.47	-0.21	0.94	-0.12	-0.18	3.40	-0.10	1.72
LEVEL 10	1404	3.51	2.18	-0.18	0.85	-0.10	-0.15	3.00	-0.09	1.55
LEVEL 9	1257	3.14	1.85	-0.15	0.74	-0.08	-0.13	2.51	-0.07	1.33
LEVEL 8	1110	2.78	1.52	-0.11	0.63	-0.06	-0.09	2.02	-0.05	1.11
LEVEL 7	963	2.41	1.20	-0.08	0.52	-0.04	-0.07	1.55	-0.04	0.88
LEVEL 6	816	2.04	0.98	-0.06	0.44	-0.03	-0.05	1.23	-0.03	0.72
LEVEL 5	669	1.67	0.76	-0.04	0.35	-0.02	-0.03	0.93	-0.02	0.56
LEVEL 4	522	1.31	0.52	-0.02	0.25	-0.01	-0.02	0.61	-0.01	0.39
LEVEL 3	375	0.94	0.29	0.00	0.15	0.00	0.00	0.32	0.00	0.22
LEVEL 2	228	0.57	0.12	0.00	0.07	0.00	0.00	0.13	0.00	0.11

Figure 44

ASCE 7-10 directly addresses the interstory displacements due to the seismic loads of a building as a function of the story height. The allowable deflection by code is calculated as $\Delta_a = 0.02h_{sx}$. This value is compared to the amplified displacement of each story which is calculated as $\delta_x = \frac{C_d \delta_{xe}}{I_e}$. For Office Building-G, C_d (the deflection amplification factor) is equal to 3.25 and I_e (the importance factor) is equal to 1.0. δ_{xe} was calculated as per ASCE section 12.8.6 which states: *story drift shall be computed as the difference of the deflections at the centers of mass at the top and bottom of the story under consideration.* Figure 45 shows the calculated values for each of the building stories and demonstrates how the amplified deflection of δ is well below the allowable value Δ_a.

Story Drift									
		East-West Direction (in)				North-South Direction (in)			
Story	H (in)	δ_e	δ	Δ	Δ_a	δ_e	δ	Δ	Δ_a
ROOF	2139	3.75	12.19	0.70	42.78	5.18	16.85	1.14	42.78

LEVEL 14	1992	3.54	11.50	1.22	39.84	4.83	15.71	1.63	39.84
LEVEL 13	1845	3.16	10.28	1.05	36.90	4.33	14.08	1.44	36.90
LEVEL 12	1698	2.84	9.23	1.19	33.96	3.89	12.64	1.60	33.96
LEVEL 11	1551	2.47	8.04	0.95	31.02	3.40	11.04	1.31	31.02
LEVEL 10	1404	2.18	7.09	1.07	28.08	3.00	9.73	1.57	28.08
LEVEL 9	1257	1.85	6.02	1.08	25.14	2.51	8.17	1.61	25.14
LEVEL 8	1110	1.52	4.94	1.03	22.20	2.02	6.56	1.53	22.20
LEVEL 7	963	1.20	3.91	0.73	19.26	1.55	5.02	1.04	19.26
LEVEL 6	816	0.98	3.18	0.70	16.32	1.23	3.98	0.96	16.32
LEVEL 5	669	0.76	2.48	0.80	13.38	0.93	3.02	1.06	13.38
LEVEL 4	522	0.52	1.68	0.75	10.44	0.61	1.97	0.94	10.44
LEVEL 3	375	0.29	0.93	0.54	7.50	0.32	1.02	0.62	7.50
LEVEL 2	228	0.12	0.39	0.39	4.56	0.13	0.41	0.41	4.56

Figure 45

Connections

As part of the MAE requirements, two typical frame connections were designed. The connections designed were the brace-to-beam connection referred to as the X-Connection and a connection between a column, two beams and four braces, referred to as the Corner Connection. All of the connections were designed as pinned. The specific connections designed are circled in Figure 46 below.

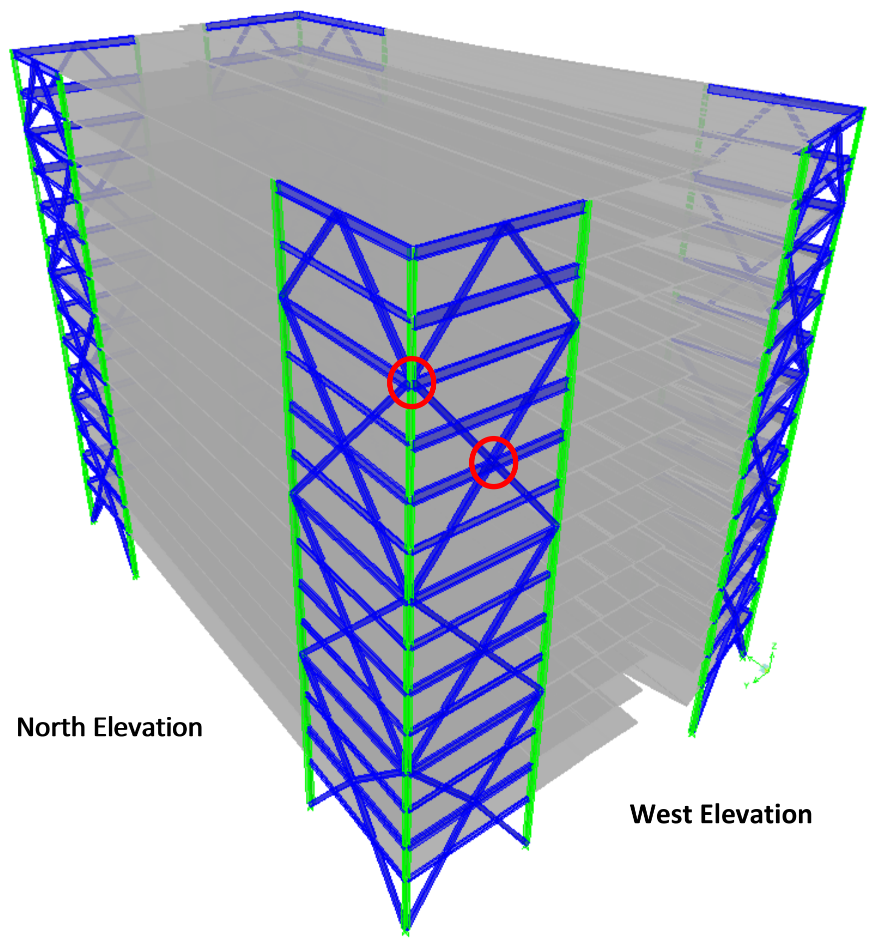


Figure 46

When designing the X-Connection it was broken into four sections, each with a brace framing into it. This was done to ensure the strength of the materials in the connection would not be double counted in resisting the applied load. Figure 47 shows the division of the connection shown by dashed lines in the detail of the designed X-Connection.

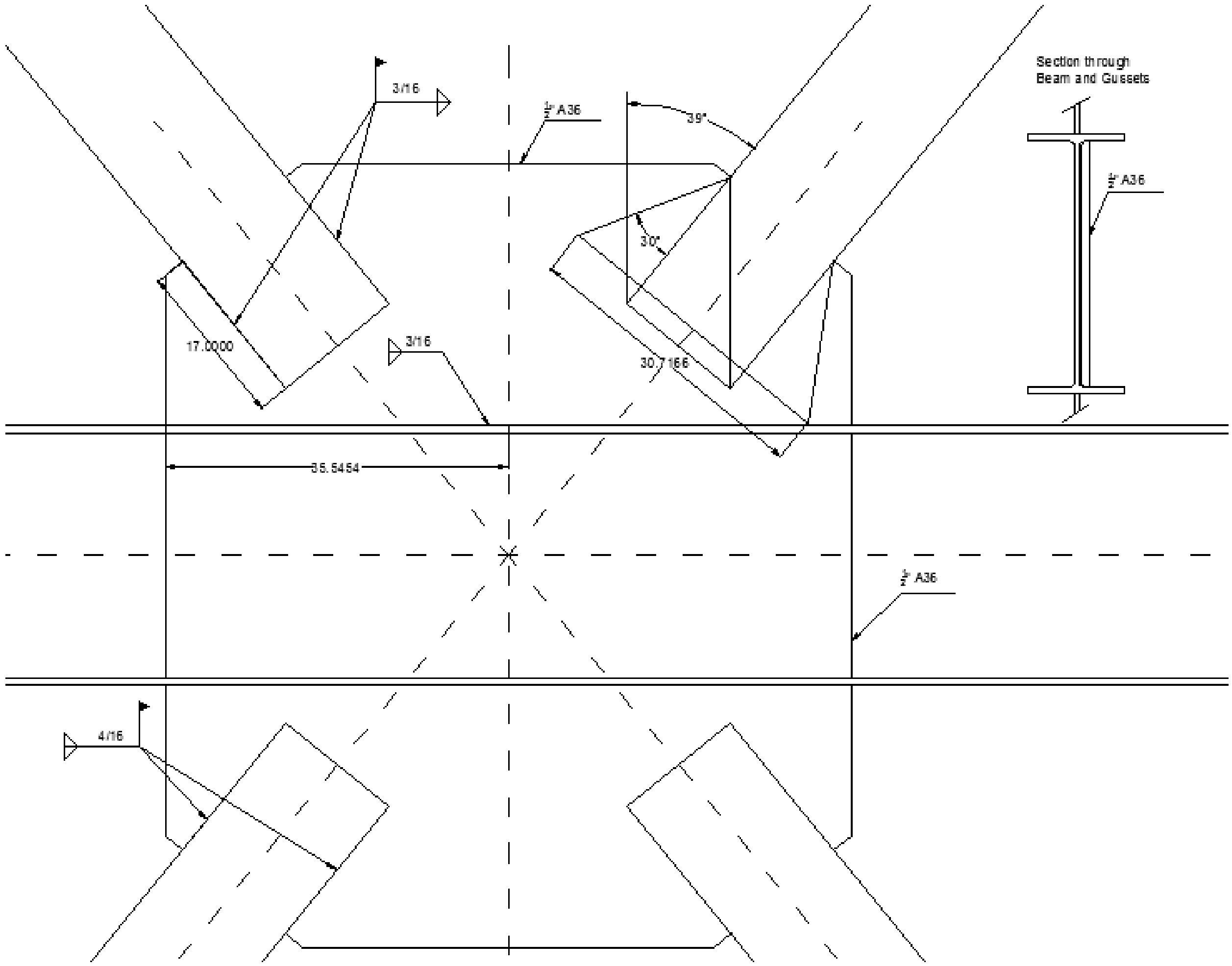


Figure 47

Braces in chevron frames can be in tension or compression based on the direction of the lateral load. This reversal of load direction and magnitude created a symmetrical connection about the vertical axis. Due to slightly higher tension experienced by the bottom braces, the weld size was increased in order to keep the dimensions of the plates used consistent.

The limit states checked for the X-Connection are listed below:

- Brace Limit States:
  - Tension Yielding
  - Tension Rupture
- Brace/Gusset Limit States:
  - Weld Rupture
  - Base Metal Strength
    - Brace
    - Gusset
- Gusset Limit States

    - Tension Yielding
    - Tension Rupture
    - Local Buckling
- Gusset/Beam Limit States
    - Weld Rupture
    - Base Metal Strength
        - Gusset
        - Beam
- Beam Limit States
    - Web Tension Yielding
    - Web Crippling
    - Web Buckling

The Uniform Force Method was used to prevent any moments from being created by the connection. Hand calculations for this connection can be found in Appendix E.

As in the design of the X-Connection, the corner connection was also broken up into four sections. Figure 48 is a detail of the East Elevation elements which frame into the Corner Connection. Due to the similarity of the loads and member size, the top and bottom members were designed with the maximum forces for either member. This simplified the design of the connection as well as created a symmetrical design on either side of the beam.

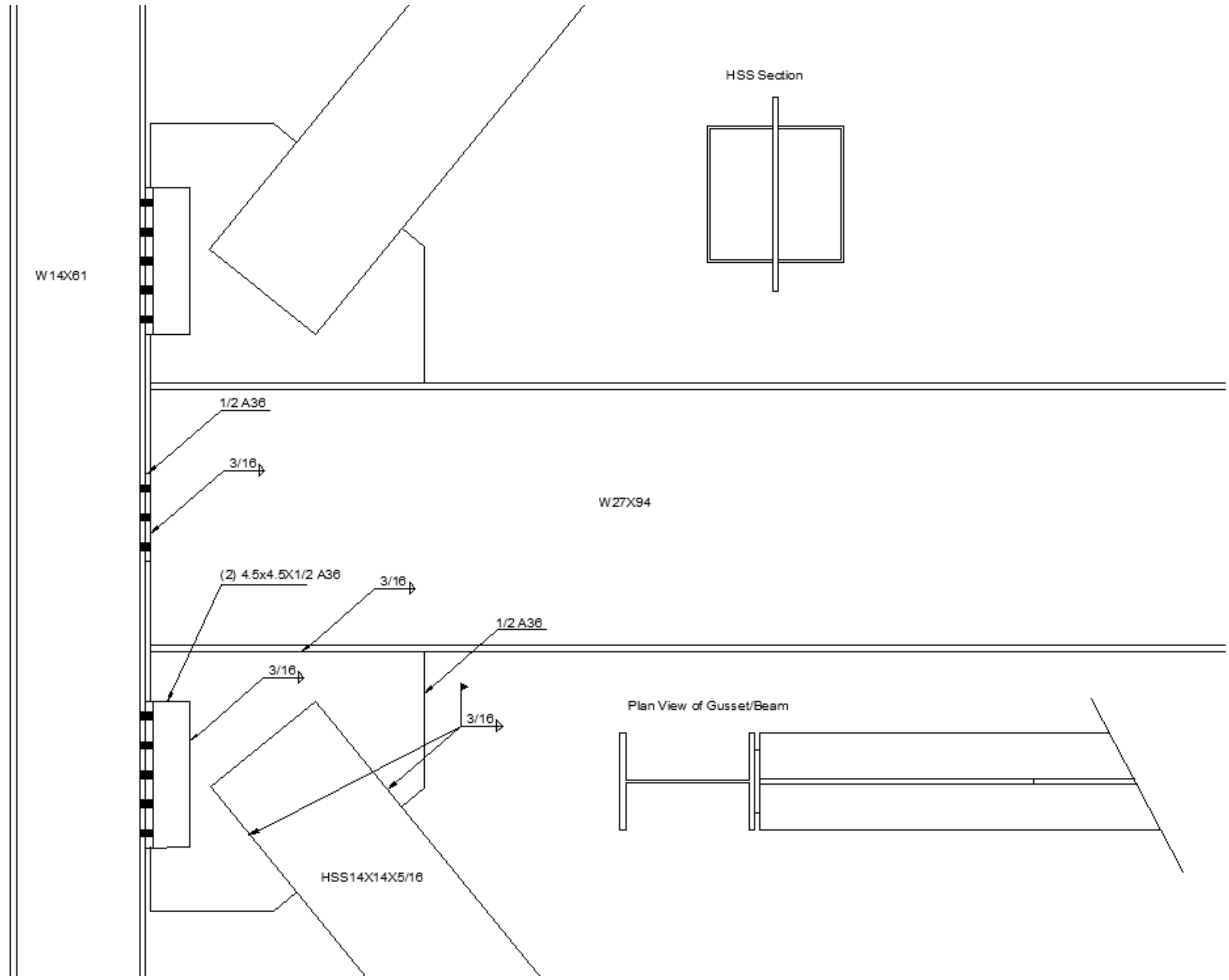


Figure 48

The connection members on the North Elevation were designed similarly to those of the East Elevation. The geometry of the W14X61 column and W27X161 beam resulted in a large end plate being welded to the flanges of the column. This is unique to the specific connection designed because of the column size in the connection. Figure 49 is a detail of the North Elevation elements which frame in to Corner Connection.

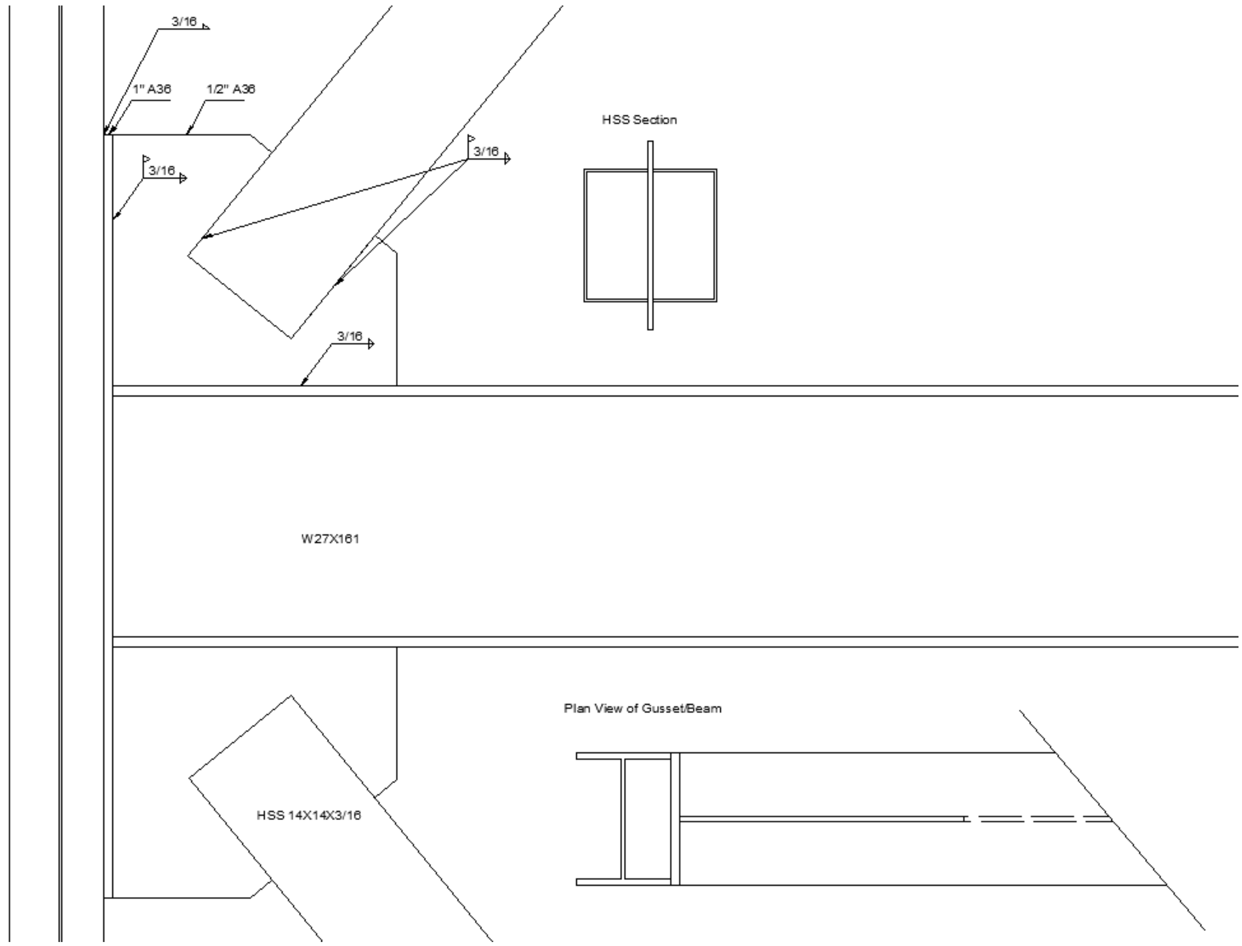


Figure 49

Once both sides of the corner connection were designed they were combined and the column limit states were considered. Again, the Uniform Force Method was used to prevent any moments being imposed on the connection design. Hand calculations and drawing with dimension of the Corner Connection can be found in Appendix E.

The limit states checked for the entire connection are listed below:

- Brace Limit States:
  - Tension Yielding
  - Tension Rupture
- Brace/Gusset Limit States:
  - Weld Rupture
  - Base Metal Strength
    - Brace
    - Gusset
- Gusset Limit States
  - Tension Yielding
  - Tension Rupture

  - Local Buckling
- Gusset/Beam Limit States
  - Weld Rupture
  - Base Metal Strength
    - Gusset
    - Beam
- Beam
  - Web Tension Yielding
  - Web Crippling
  - Web Buckling
  - Shear Yielding
- Beam/End Plate
  - Weld Rupture
  - Base Metal Strength
    - Beam
    - End Plate
- End Plate Limit States
  - Gross Shear
  - Net Shear
  - Block Shear
- Angle Limit States
  - Prying Effects
  - Shear Yielding
  - Shear Rupture
  - Block Shear
- Bolt
  - Shear
  - Tension
  - Bearing and Tear Out
- Gusset/Plate
  - Weld Rupture
  - Base Metal Strength
    - Gusset
    - Plate
- Plate
  - Plate bending
- Plate/Column
  - Weld Rupture
  - Base Metal Strength
    - Plate
    - Column

- Column
  - Local Flange Yielding
  - Local Flange Bending
  - Local Flange Crippling

The material strengths of both connections were: A992 Steel for columns and beams, A500 Grade B HSS sections, A36 Steel plates and angles, A325 N bolts and E 70xx welds.

Foundation Impact

The structural redesign of Office Building-G was focused on the superstructure but impacts on the existing below grade parking garage and foundation were considered. The column layout of the redesign was based on the existing locations so the columns in the parking garage and the spread footings would not need to be altered. Despite an overall reduction of weight for Office Building-G, the columns are spaced further apart, creating larger axial loads. The foundation design would have to be adjusted for these larger forces.

The reduction in weight of Office Building-G is due to a smaller dead load associated with the steel frame redesign. A reduced dead load on the structure creates a greater likelihood of overturning forces affecting the design.

Overturning Moment

With large horizontal forces and a small width to height ratio, buildings have a risk of overturning. This can occur when an upward reaction at the base of a building is greater than a reduced dead load over that column line. Figure 50 is a plan view of the base reaction locations of Office Building-G as well as a list of the reactions which experience an uplift force. This uplift force is compared to a reduced dead load case of (0.9-.2S_{DS})D. At the locations in which the dead load is smaller than the uplift force, overturning effects must be considered. Point 21 and 1873 are circled in red.

Overturning Moment			
Point	**Reaction**	**Dead Load**	**Overturning**
21	-692.53	631.16	Yes
1873	-655.58	550.31	Yes
45	-341.51	645.33	No
2075	-309.53	716.07	No
2077	-184.58	817.16	No
41	-120.81	629.69	No
46	-59.73	570.66	No
1645	-57.68	480.39	No

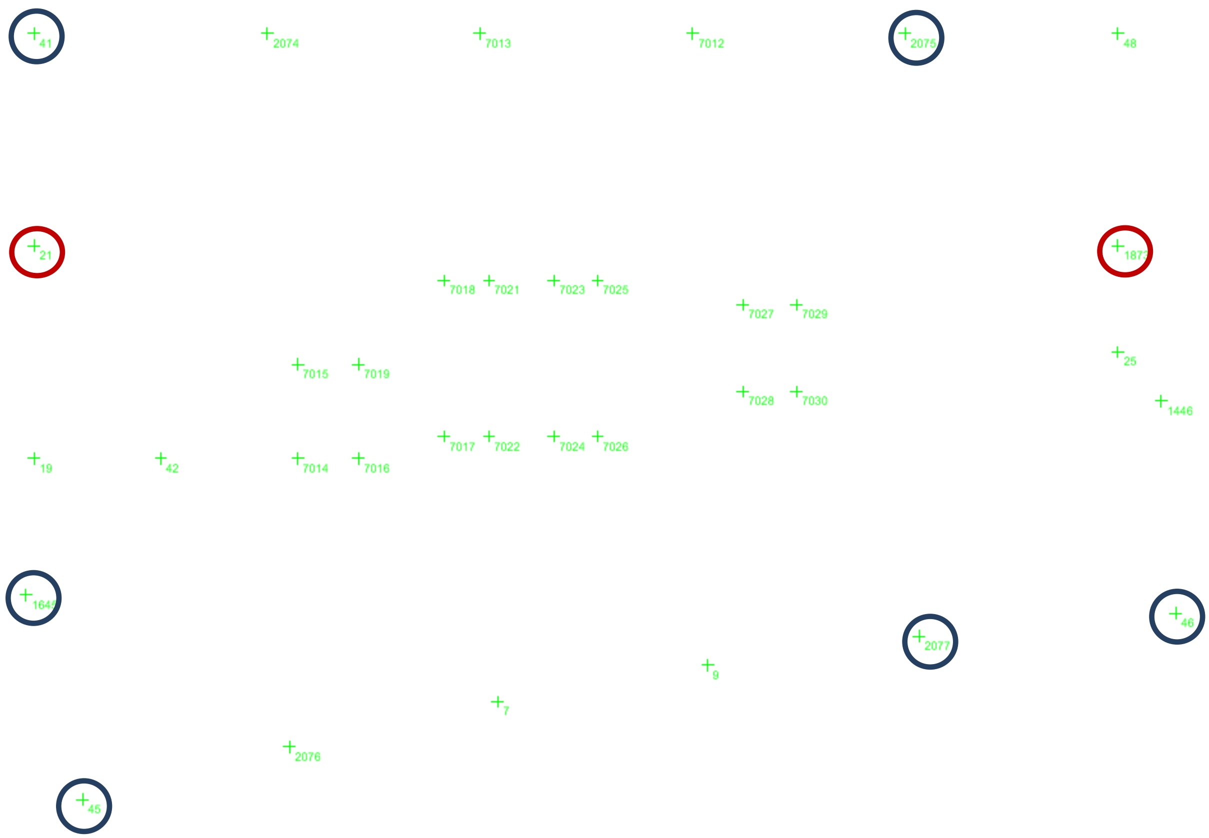


Figure 50

Although the uplift forces are larger than the dead load acting on two of the base reactions, overturning is not a concern for Office Building-G. Below the superstructure there is a four story reinforced concrete parking garage and spread footings. The weight of these concrete elements is more than enough to overcome the slight difference in dead load and uplift force.

Architectural Breadth

Maximizing the open floor space for tenant fit outs was the central focus of the redesign of Office Building-G. Figure 51 is a plan view of the existing design which only shows the façade, elevator shafts, stairwells and structure. Figure 52 is a plan view of the redesign with the same building elements shown.

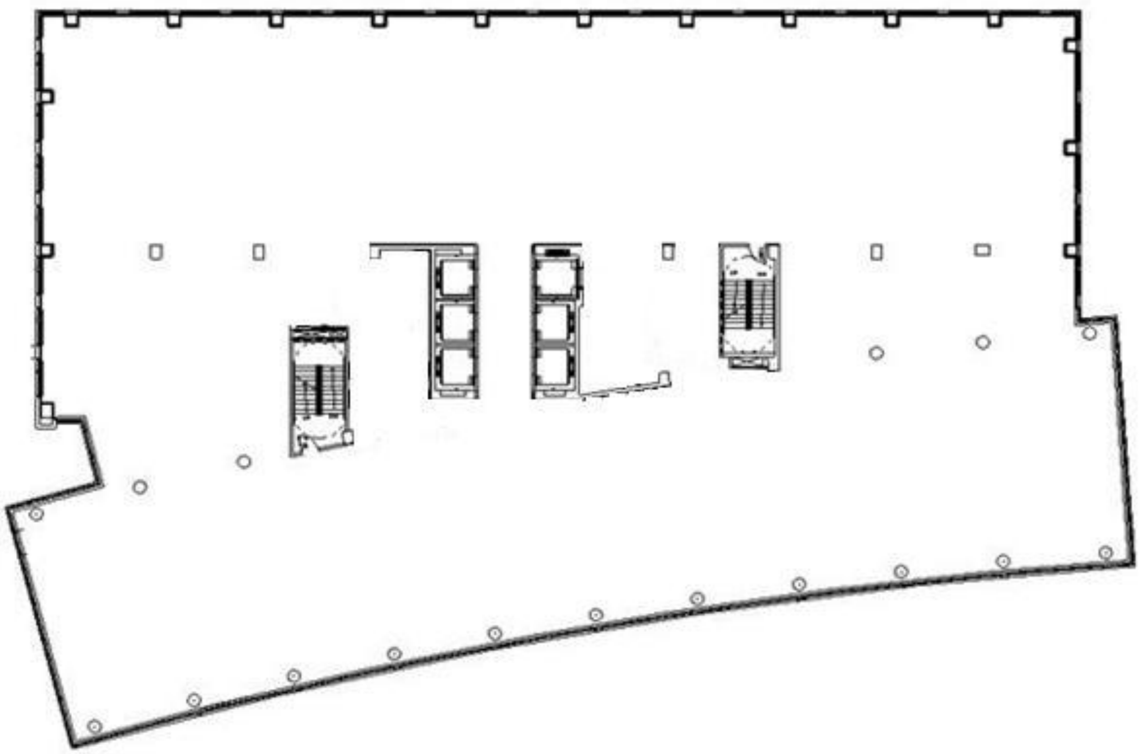


Figure 51

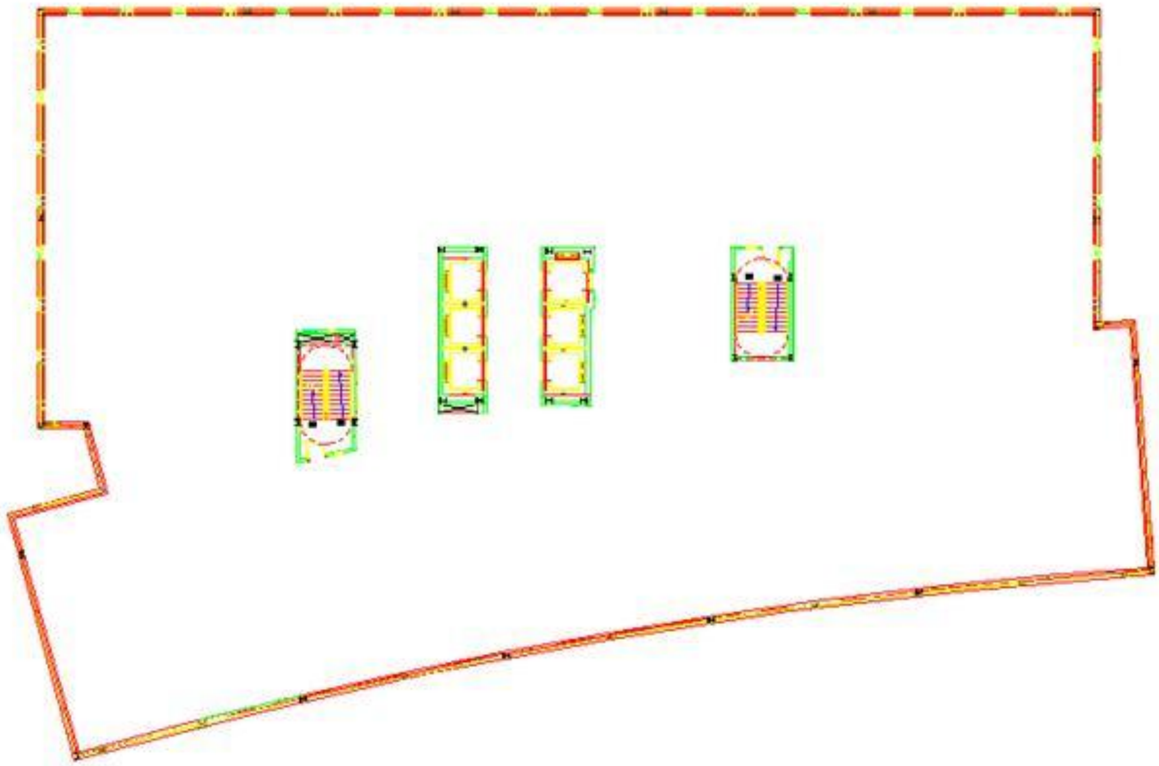


Figure 52

It is clear that the redesigned structure fits well within the permanent building elements while creating an open floor space. The column free space will allow for a greater flexibility for the tenant fit out designs of the office space.

In addition to creating an open floor plan for Office Building-G, the new floor structure was designed to fit within the existing ceiling plenum. This allowed for the original floor-to-ceiling height of building to be maintained, eliminating the need to increase the building height. Coordination between the structure and mechanical duct work was necessary to accomplish this and is discussed in more detail in the Mechanical Breadth section.

The façade of Office Building-G was greatly affected by creating an external structure. Compared to the original design in which none of the structure was exposed, the bracing and perimeter columns are now a major architectural feature. Based on the designed structure, preliminary architectural renderings were made to show the geometry of the building and the change in the exterior view of the building. Figure 54 is an image of the existing structure and Figures 55 and 56 are images of the proposed building.

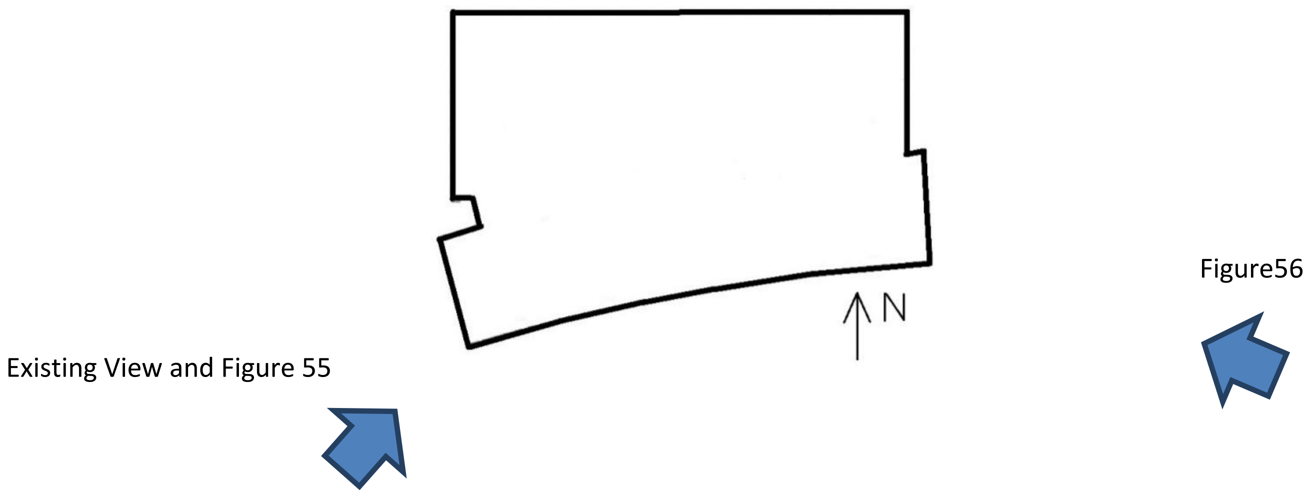


Figure 53

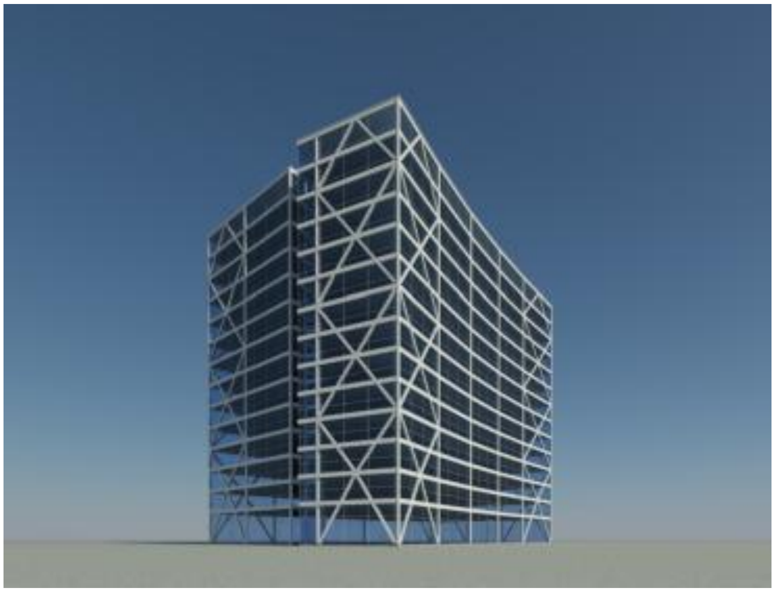


Figure 54

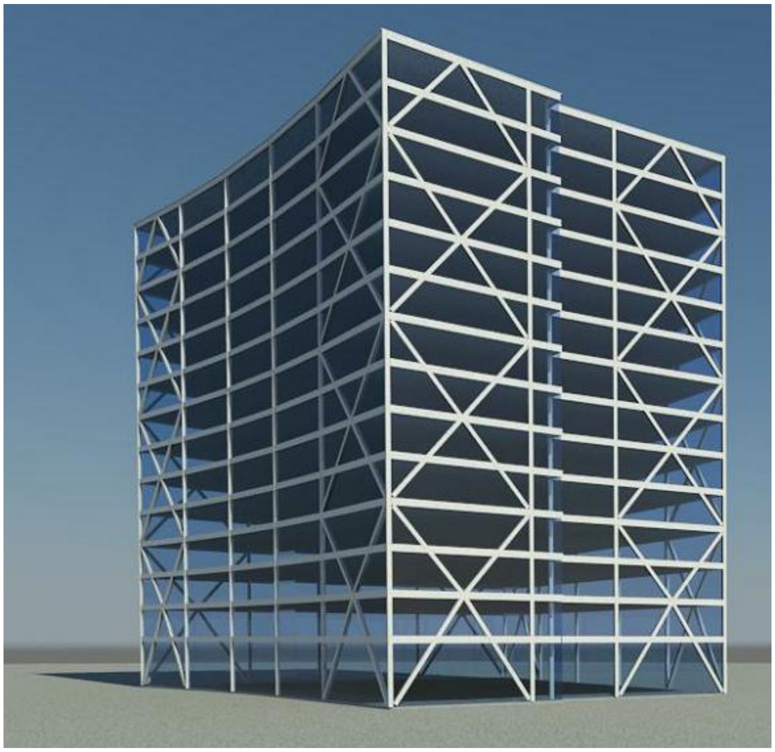


Figure 55

Mechanical Breadth

In the existing design of Office Building-G, the mechanical ducts have a maximum size of 40” wide by 13” deep. The shallow structural depth of 18” allows for this system to fit well within the 36” plenum space. The structural redesign has much deeper framing members which cut into the allowable space for the ducts. As mentioned in the framing plan, the existing ducts were resized to fit through the openings with the castellated beams. This was done by adjusting the original rectangular ducts to circular ducts capable of supplying the same volume of air. When the redesign was finished, pressure drop was checked to ensure that the same air handling units could be used on the floors.

The resizing of ducts was performed with the aid of a Duct Calculator. Duct Calculators relate the needed CFM and duct dimensions to a friction loss. Using the existing CFM values and duct sizes of Office Building-G the as designed pressure drops were solved for. Using this same pressure drop and the needed CFM, round member sizes were calculated. However, creating a single round duct to replace the existing rectangular design was resulting in too great of diameters to fit within the castellated beams. To resolve this issue the space on each floor of Office Building G was broken into interior and perimeter spaces and different duct lines access these spaces. This essentially broke a single duct line into two. Figure 57 is an image of the existing duct layout and Figure58 is an image of the redesign.

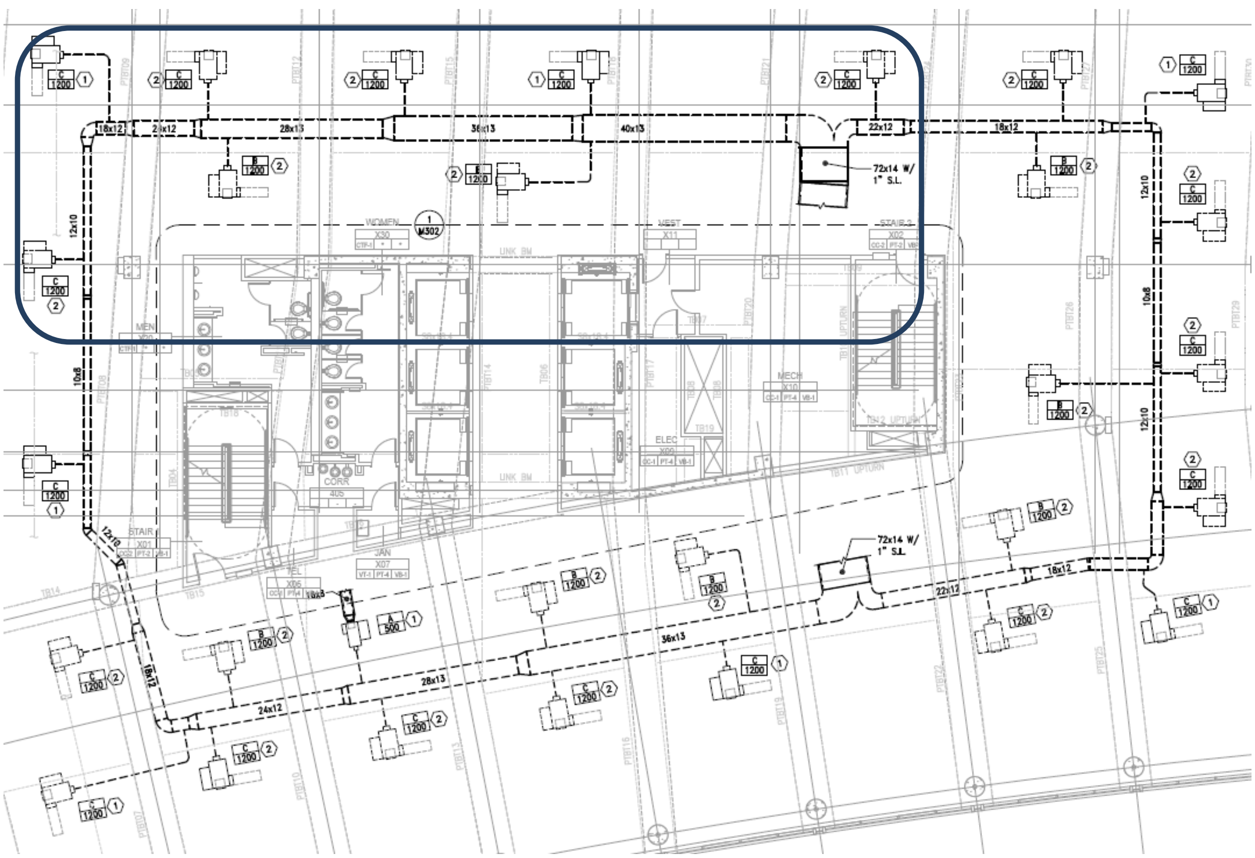


Figure 56

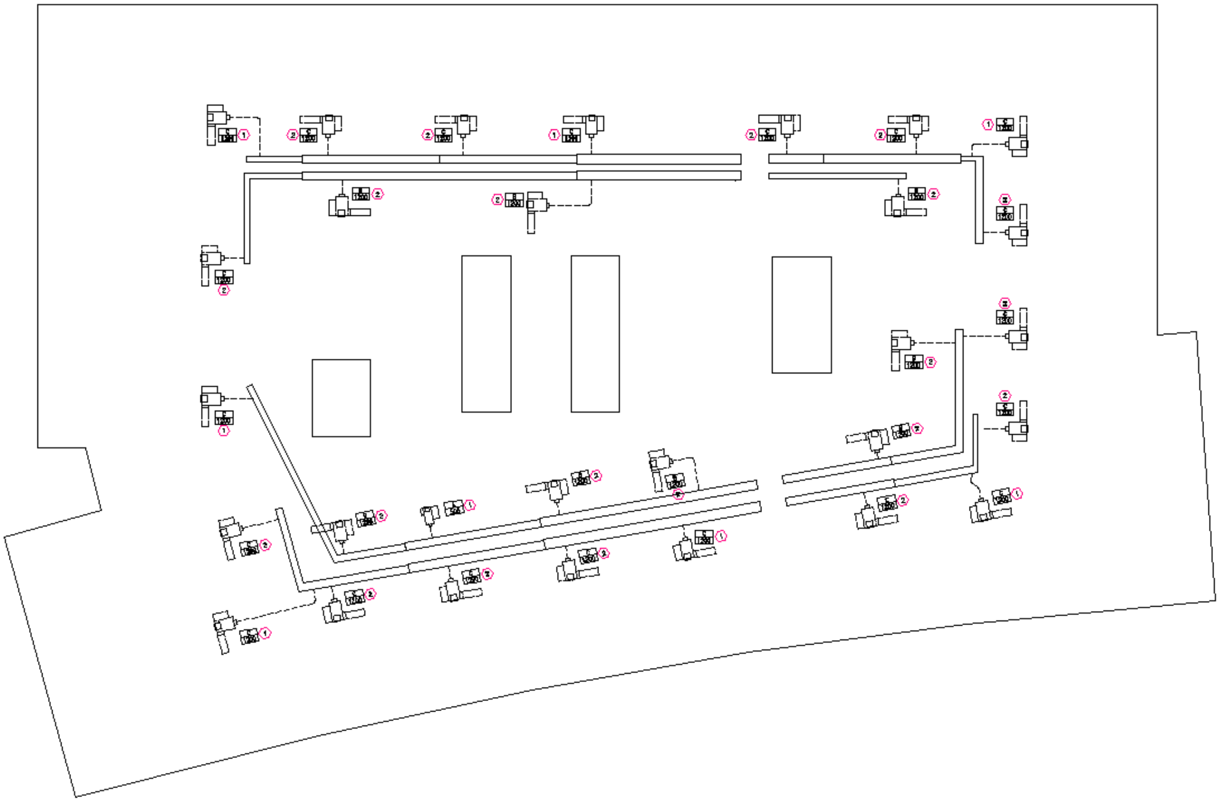


Figure 57

Figure 59 is the section of the mechanical design highlighted in the Figure 57 above. The four VAV boxes circled in green are supplied by one of the new ducts and the remaining VAV boxes (circled in orange) are supplied by a separate duct. The new design is shown in Figure60.

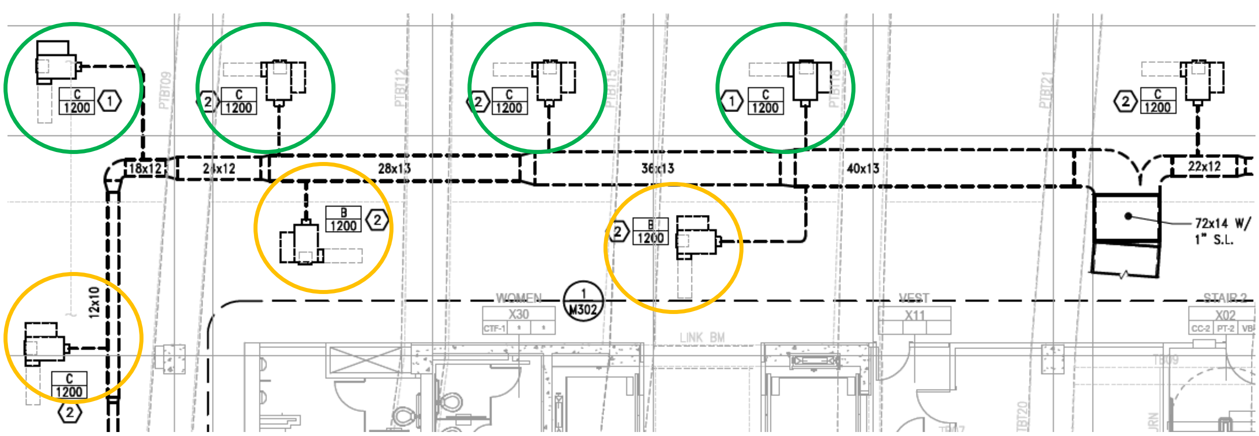


Figure 58

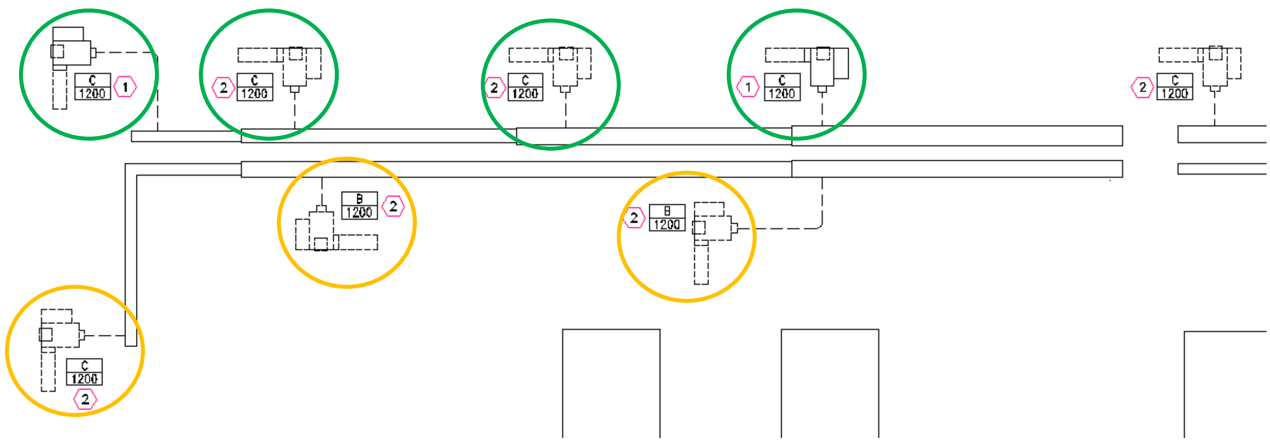


Figure 59

Calculations were performed to estimate the pressure drop due to friction of the straight runs of the ducts. The values for the redesigned system of Figure 60 are shown below in Figure 61. Calculations for the other spaces can be found in Appendix F.

	Perimeter					Interior				
	Duct Dia (in)	CFM	Run	Friction Loss/100'	Friction Loss	Duct Dia (in)	CFM	Run	Friction Loss/100'	Friction Loss
NW	18	4800	30	0.55	0.17	18.5	3600	30	0.3	0.09
	18	3600	55	0.3	0.17	17	2400	80	0.14	0.11
	15.5	2400	80	0.3	0.24	13	1200	208	0.18	0.37
	11.5	1200	120	0.35	0.42					

Figure 60

Limiting the diameter of the ducts allowed for 1" of insulation, creating total diameter of 18". The openings within the castellated beams were designed to have an opening with a minimum diameter of 20". The larger beam opening was required in the design due to construction implications. The larger beam opening also prevent the beam and duct work from colliding with each other when the building is reacting to lateral forces. A section detail is provided below in Figure 62 to show how the cavity space between floors is used.

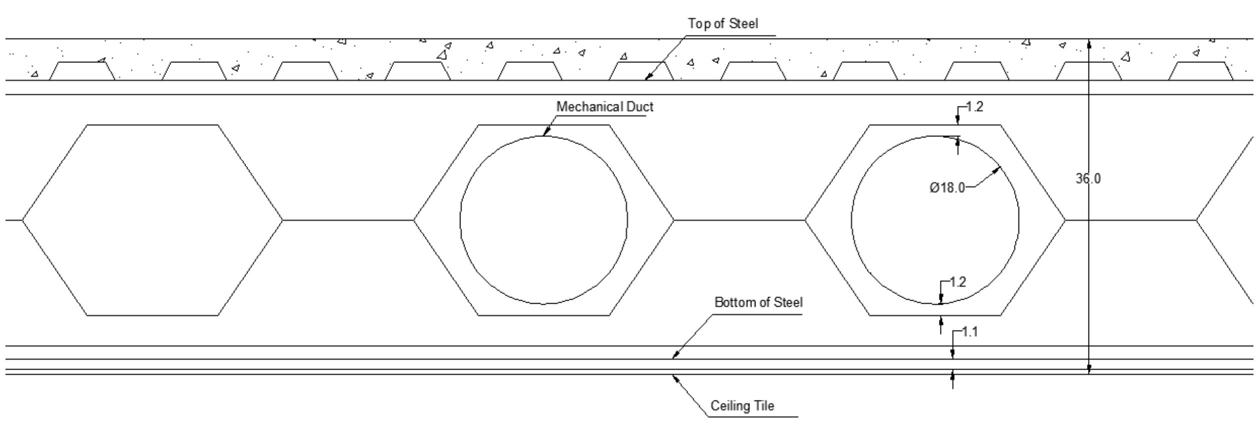


Figure 61

Conclusion

The proposed structural redesign of Office Building-G was focused on creating an open floor plan which allows architectural freedom for the future tenant fit outs. This created the need for a complete redesign of the existing building superstructure. During the redesign process, elements of the architecture and mechanical design were affected. The effects on these components of the building design were addressed through breadth studies. Through the use of an external structure, interior columns and castellated beams, the proposed change to Office Building-G was accomplished.

In every building design, structurally efficiency should be addressed by the structural engineer. Based on the limitations and goals of the proposed change, the redesign is an efficient design and if the proposed changes were a design criteria set forth by the owner the final design would be a viable option for Office Building-G.

Appendix A:

Seismic Values:

Building Dead Loads

Floor Weight					
story	Area (ft^2)	Metal Deck	steel weight (psf)	floor weight (psf)	total (k)
1	27187	37	10	47.0	1278
2	29487	37	10	47.0	1386
3	29628	37	10	47.0	1393
4	25774	37	10	47.0	1211
5	25774	37	10	47.0	1211
6	25774	37	10	47.0	1211
7	25774	37	10	47.0	1211
8	25774	37	10	47.0	1211
9	25774	37	10	47.0	1211
10	25774	37	10	47.0	1211
11	25774	37	10	47.0	1211
12	25774	37	10	47.0	1211
13	25774	37	10	47.0	1211
14	25774	37	10	47.0	1211
ROOF	25774	37	10	47.0	1211
EL, MR	2020	37	10	47.0	95
SCREEN WALL					
				Total =	18688

Façade weight				
story	effective height	perimeter	wall weight (psf)	story weight (k)
1	9.50	755	20	143
2	15.63	803	20	251
3	12.25	804	20	197
4	12.25	708	20	173
5	12.25	708	20	173
6	12.25	708	20	173
7	12.25	708	20	173
8	12.25	708	20	173
9	12.25	708	20	173
10	12.25	708	20	173

11	12.25	708	20	173
12	12.25	708	20	173
13	12.25	708	20	173
14	12.25	708	20	173
ROOF	10.00	708	20	142
EL, MR	8.38	198	20	33
	4.50			
			Total =	2674

Superimposed				
story		Area (ft^2)	S.I. (psf)	Total (k)
1		27187	15	407.805
2		29487	15	442.305
3		29628	15	444.42
4		25774	15	386.61
5		25774	15	386.61
6		25774	15	386.61
7		25774	15	386.61
8		25774	15	386.61
9		25774	15	386.61
10		25774	15	386.61
11		25774	15	386.61
12		25774	15	386.61
13		25774	15	386.61
14		25774	15	386.61
ROOF		25774	15	386.61
EL, MR		2020	15	30.3
SCREEN WALL				
			Total =	5964.15

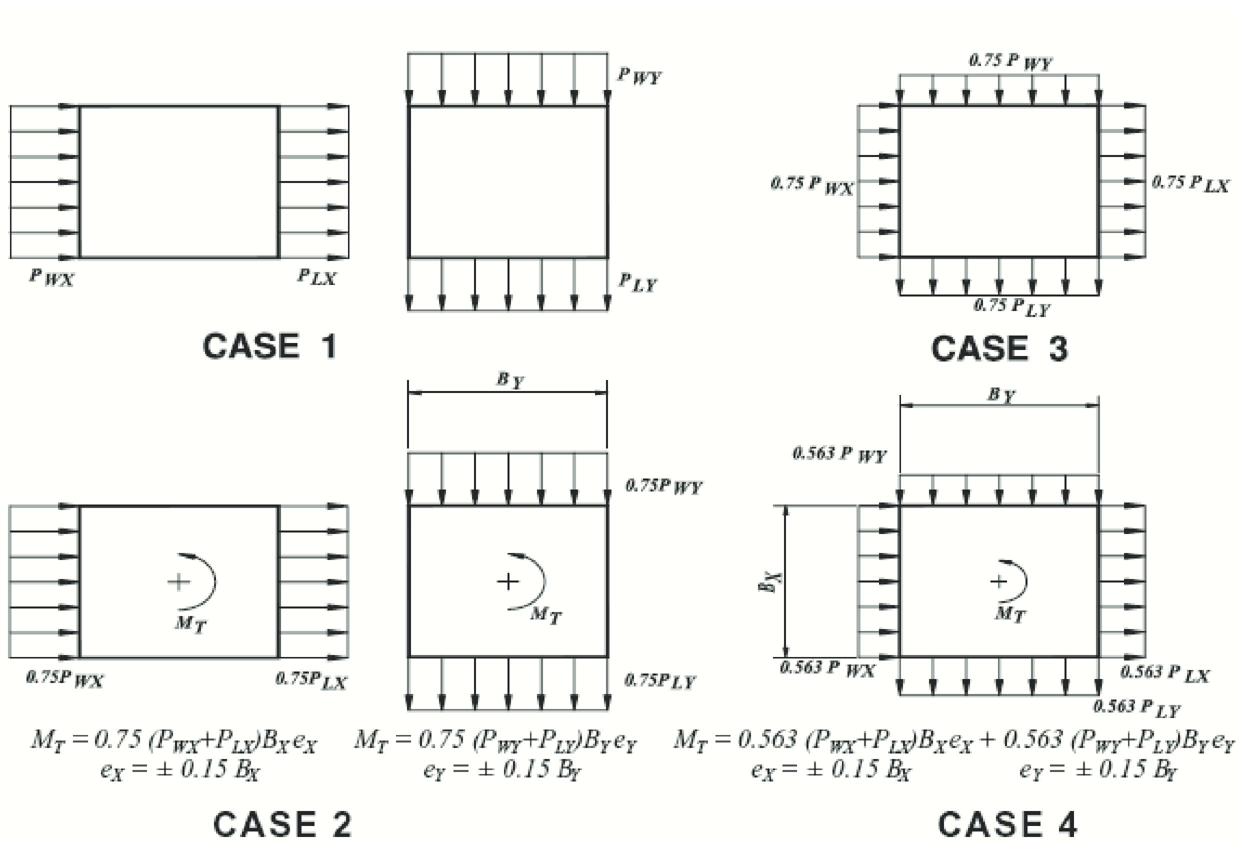
Coefficients and References		
Factor	Coefficient	Reference
Site Class	D	Geo. Report
Design Category	B	T 11.6-1
Importance	1	T 1.5-2
S_s	16	USGS Website
S_1	5.1	USGS Website

F_a	1.6	T 11.4-1
F_v	2.4	T 11.4-2
S_{ms}	0.256	11.4-1
S_{m1}	0.1224	11.4-2
S_{ds}	0.171	11.4-3
S_{d1}	0.0816	11.4-4
C_t	0.028	T 12.8-2
x	0.8	T 12.8-2
h_n	186	Bldg Drawings
T_a	1.83	12.8-7
T_L	8	F 22-12
R	3.25	T 12.2-1
Cs	0.0526	E 12.8-2
W	27435	12.7.2
V_b	1444	12.8-1

Story	**Height, h (ft)**	h_i	N-S Length	E-W Length	w_x	w_x*h^k	C_V	f_i (k)	V_i (k)
1	0.00	9.50	324.5	303.5	1836	0	0.0000	0.0	1444
2	19.00	15.63	267	145	2086	302339.1	0.0044	6.4	1444
3	31.25	12.25	267	145	2041	685775.5	0.0101	14.5	1437
4	43.50	12.25	220.5	145	1779	1045098	0.0154	22.2	1423
5	55.75	12.25	220.5	145	1779	1589513	0.0234	33.7	1400
6	68.00	12.25	220.5	145	1779	2223568	0.0327	47.2	1367
7	80.25	12.25	220.5	145	1779	2941863	0.0432	62.4	1320
8	92.50	12.25	220.5	145	1779	3740157	0.0550	79.3	1257
9	104.75	12.25	220.5	145	1779	4614989	0.0678	97.9	1178
10	117.00	12.25	220.5	145	1779	5563453	0.0818	118.0	1080
11	129.25	12.25	220.5	145	1779	6583063	0.0967	139.6	962
12	141.50	12.25	220.5	145	1779	7671649	0.1127	162.7	822
13	153.75	12.25	220.5	145	1779	8827301	0.1297	187.2	659
14	166.00	12.25	220.5	145	1779	10048310	0.1477	213.1	472
ROOF	178.25	10.00	220.5	145	1747	11130145	0.1636	236.1	259
EL, MR	186.00	8.38	204	81.75	158	1084540	0.0159	23.0	23
SCREEN WALL	195.00	4.50	204	81.75	0	0	0.0000	0.0	0
				Total =	27435	68051766	1.0000	1444	

Wind Values:

Load Cases



Coefficients:

Effective Length Considerations								
Story	Height (ft)	Length (ft)		$\Sigma h_i L_i$		Σh_i	L_{eff} (ft)	
		N-S	E-W	N-S	E-W		N-S	E-W
1	0	324.5	303.5	0	0	0	0	0
2	19	267	145	5073	2755	19	267.0	145.0
3	31.25	267	145	13416.75	7286.25	50.25	267.0	145.0
4	43.5	220.5	145	23008.5	13593.75	93.75	245.4	145.0
5	55.75	220.5	145	35301.375	21677.5	149.5	236.1	145.0
6	68	220.5	145	50295.375	31537.5	217.5	231.2	145.0
7	80.25	220.5	145	67990.5	43173.75	297.75	228.3	145.0
8	92.5	220.5	145	88386.75	56586.25	390.25	226.5	145.0
9	104.75	220.5	145	111484.125	71775	495	225.2	145.0
10	117	220.5	145	137282.625	88740	612	224.3	145.0
11	129.25	220.5	145	165782.25	107481.25	741.25	223.7	145.0
12	141.5	220.5	145	196983	127998.75	882.75	223.1	145.0
13	153.75	220.5	145	230884.875	150292.5	1036.5	222.8	145.0
14	166	220.5	145	267487.875	174362.5	1202.5	222.4	145.0
ROOF	178.25	220.5	145	306792	200208.75	1380.75	222.2	145.0
EL, MR	186	204	81.75	344736	215414.25	1566.75	220.0	137.5
SCREEN WALL	195	204	81.75	384516	231355.5	1761.75	**218.3**	**131.3**

Gust Factor		
	N-S	E-W
n1 = na	0.326833	0.326833
g_Q	3.4	3.4
g_V	3.4	3.4
g_R	3.913861	3.913861
R_n	0.098815	0.098815
l	320	320
ε	0.333333	0.333333
z	117	117
L_z	487.9485	487.9485
Q	0.798367	0.816018
b	0.45	0.45
α	0.25	0.25
V_z	95.09358	95.09358
N_1	1.677058	1.677058
η_h	3.08295	3.08295
η_B	2.29245	3.4782
η_L	11.64441	7.674724
R_h	0.298117	0.298117
R_B	0.389129	0.26686
R_L	0.084034	0.126053
β	0.01	0.01
R	0.807975	0.680605
c	0.3	0.3
I_z	0.242946	0.242946
G_f	1.046962	0.995905

Exposure	
Height	k_z
0	0
19	0.61
31.25	0.7075
43.5	0.7775
55.75	0.833
68	0.882
80.25	0.93
92.5	0.9675
104.75	1
117	1.03
129.25	1.06
141.5	1.093
153.75	1.1175
166	1.142
178.25	1.1665
186	1.179
195	1.1925

Natural Frequency	
h	195
n_a	0.326833
n_a	0.384615

$$n_a = 22.2/h^{0.8}$$

Wall Coefficients									
N-S					E-W				
			Cp					Cp	
B	L	L/B	Windward	Leeward	B	L	L/B	Windward	Leeward
145	220.5	1.52069	**0.8**	**-0.396**	220.5	145	0.657596	**0.8**	**-0.5**

Roof Coefficients									
N-S					E-W				
			Cp					Cp	
h	L	h/L	0 - h	h - 2h	h	L	h/L	0 - h/2	> h/2
195	220.5	0.884354	**-0.75**	**-0.65**	195	145	1.344828	**-1.3**	**-0.7**

Values:

Case 1: EW NO ECC							
Floor		psf		Story Height	plf	Bldg. Width	Shear
		Windward	Leeward				
1	1	11.7	-14.2	15.6	404.8	145.0	58.7
2	1	11.7	-14.2	12.3	317.3	145.0	46.0
3	1	13.5	-14.2	12.3	340.2	145.0	49.3
4	1	14.9	-14.2	12.3	356.6	145.0	51.7
5	1	15.9	-14.2	12.3	369.6	145.0	53.6
6	1	16.9	-14.2	12.3	381.0	145.0	55.2
7	1	17.8	-14.2	12.3	392.3	145.0	56.9
8	1	18.5	-14.2	12.3	401.0	145.0	58.2
9	1	19.1	-14.2	12.3	408.7	145.0	59.3
10	1	19.7	-14.2	12.3	415.7	145.0	60.3
11	1	20.3	-14.2	12.3	422.7	145.0	61.3
12	1	20.9	-14.2	12.3	430.4	145.0	62.4
13	1	21.4	-14.2	12.3	436.2	145.0	63.2
14	1	21.8	-14.2	12.3	441.9	145.0	64.1
ROOF	1	22.3	-14.2	6.1	223.8	145.0	32.5

Case 2: NS NO ECC						
Floor	psf		Story Heig	plf	Bldg. Width	Shear
	Windward	Leeward				
1	12.3	-11.9	15.6	376.8	204.0	76.9
2	12.3	-11.9	12.3	295.4	204.0	60.3
3	14.2	-11.9	12.3	319.4	204.0	65.2
4	15.6	-11.9	12.3	336.7	204.0	68.7
5	16.7	-11.9	12.3	350.3	204.0	71.5
6	17.7	-11.9	12.3	362.4	204.0	73.9
7	18.7	-11.9	12.3	374.2	204.0	76.3
8	19.4	-11.9	12.3	383.4	204.0	78.2
9	20.1	-11.9	12.3	391.4	204.0	79.9
10	20.7	-11.9	12.3	398.8	204.0	81.4
11	21.3	-11.9	12.3	406.2	204.0	82.9
12	22.0	-11.9	12.3	414.3	204.0	84.5
13	22.5	-11.9	12.3	420.4	204.0	85.8
14	22.9	-11.9	12.3	426.4	204.0	87.0
ROOF	23.4	-11.9	6.1	216.2	204.0	44.1

Case 3 & 4: 0.15Bx EW ECC												
Floor		psf		Story Height	plf	Bldg. Width	Shear	CM	CR	0.15Bx	Mz (3)	Mz (4)
		Windward	Leeward									
1	0.75	8.7445292	-10.6843	15.625	303.575	145	**44.0**	72.8	80	21.75	**-640**	**1274**
2	0.75	8.7445292	-10.6843	12.25	238.0028	145	**34.5**	72.8	80	21.75	**-502**	**999**
3	0.75	10.14222	-10.6843	12.25	255.1246	145	**37.0**	72.8	80	21.75	**-538**	**1071**
4	0.75	11.145691	-10.6843	12.25	267.4171	145	**38.8**	72.8	80	21.75	**-564**	**1123**
5	0.75	11.9413	-10.6843	12.25	277.1633	145	**40.2**	72.8	80	21.75	**-585**	**1163**
6	0.75	12.643729	-10.6843	12.25	285.768	145	**41.4**	72.8	80	21.75	**-603**	**1200**
7	0.75	13.331823	-10.6843	12.25	294.1972	145	**42.7**	72.8	80	21.75	**-621**	**1235**
8	0.75	13.869397	-10.6843	12.25	300.7825	145	**43.6**	72.8	80	21.75	**-635**	**1263**
9	0.75	14.335294	-10.6843	12.25	306.4897	145	**44.4**	72.8	80	21.75	**-647**	**1287**
10	0.75	14.765353	-10.6843	12.25	311.7579	145	**45.2**	72.8	80	21.75	**-658**	**1309**
11	0.75	15.195411	-10.6843	12.25	317.0261	145	**46.0**	72.8	80	21.75	**-669**	**1331**
12	0.75	15.668476	-10.6843	12.25	322.8212	145	**46.8**	72.8	80	21.75	**-681**	**1355**
13	0.75	16.019691	-10.6843	12.25	327.1236	145	**47.4**	72.8	80	21.75	**-690**	**1373**
14	0.75	16.370905	-10.6843	12.25	331.4259	145	**48.1**	72.8	80	21.75	**-699**	**1391**
ROOF	0.75	16.72212	-10.6843	6.125	167.8642	145	**24.3**	72.8	80	21.75	**-354**	**705**

Case 5 & 6: NS 0.15By ECC												
Floor		psf		Story Height	plf	Bldg. Width	Shear	CM	CR	0.15By	Mz (5)	Mz (6)
	Multiplier	Windward	Leeward									
1	0.75	9.2	-8.9	15.6	282.6	204.0	**57.7**	105.3	103	30.6	**-1896.9**	**1631.7**
2	0.75	9.2	-8.9	12.3	221.6	204.0	**45.2**	105.3	103	30.6	**-1487.2**	**1279.3**
3	0.75	10.7	-8.9	12.3	239.6	204.0	**48.9**	105.3	103	30.6	**-1608.0**	**1383.2**
4	0.75	11.7	-8.9	12.3	252.5	204.0	**51.5**	105.3	103	30.6	**-1694.7**	**1457.8**
5	0.75	12.6	-8.9	12.3	262.8	204.0	**53.6**	105.3	103	30.6	**-1763.5**	**1516.9**
6	0.75	13.3	-8.9	12.3	271.8	204.0	**55.4**	105.3	103	30.6	**-1824.2**	**1569.2**
7	0.75	14.0	-8.9	12.3	280.7	204.0	**57.3**	105.3	103	30.6	**-1883.7**	**1620.3**
8	0.75	14.6	-8.9	12.3	287.6	204.0	**58.7**	105.3	103	30.6	**-1930.1**	**1660.3**
9	0.75	15.1	-8.9	12.3	293.6	204.0	**59.9**	105.3	103	30.6	**-1970.4**	**1694.9**
10	0.75	15.5	-8.9	12.3	299.1	204.0	**61.0**	105.3	103	30.6	**-2007.6**	**1726.9**
11	0.75	16.0	-8.9	12.3	304.7	204.0	**62.2**	105.3	103	30.6	**-2044.8**	**1758.9**
12	0.75	16.5	-8.9	12.3	310.8	204.0	**63.4**	105.3	103	30.6	**-2085.6**	**1794.0**
13	0.75	16.8	-8.9	12.3	315.3	204.0	**64.3**	105.3	103	30.6	**-2116.0**	**1820.1**
14	0.75	17.2	-8.9	12.3	319.8	204.0	**65.2**	105.3	103	30.6	**-2146.4**	**1846.3**
ROOF	0.75	17.6	-8.9	6.1	162.2	204.0	**33.1**	105.3	103	30.6	**-1088.4**	**936.2**

Case 7: NS & EW NO ECC												
Floor		EW psf		NS psf		Story Height	plf		Bldg. Width		Shear	
	Multiplier	Windward	Leeward	Windward	Leeward		EW	NS	EW	NS	EW	NS
1	0.75	8.7	-10.7	9.2	-8.9	15.6	303.6	282.6	145.0	204.0	**44.0**	**57.7**
2	0.75	8.7	-10.7	9.2	-8.9	12.3	238.0	221.6	145.0	204.0	**34.5**	**45.2**
3	0.75	10.1	-10.7	10.7	-8.9	12.3	255.1	239.6	145.0	204.0	**37.0**	**48.9**
4	0.75	11.1	-10.7	11.7	-8.9	12.3	267.4	252.5	145.0	204.0	**38.8**	**51.5**
5	0.75	11.9	-10.7	12.6	-8.9	12.3	277.2	262.8	145.0	204.0	**40.2**	**53.6**
6	0.75	12.6	-10.7	13.3	-8.9	12.3	285.8	271.8	145.0	204.0	**41.4**	**55.4**
7	0.75	13.3	-10.7	14.0	-8.9	12.3	294.2	280.7	145.0	204.0	**42.7**	**57.3**
8	0.75	13.9	-10.7	14.6	-8.9	12.3	300.8	287.6	145.0	204.0	**43.6**	**58.7**
9	0.75	14.3	-10.7	15.1	-8.9	12.3	306.5	293.6	145.0	204.0	**44.4**	**59.9**
10	0.75	14.8	-10.7	15.5	-8.9	12.3	311.8	299.1	145.0	204.0	**45.2**	**61.0**
11	0.75	15.2	-10.7	16.0	-8.9	12.3	317.0	304.7	145.0	204.0	**46.0**	**62.2**
12	0.75	15.7	-10.7	16.5	-8.9	12.3	322.8	310.8	145.0	204.0	**46.8**	**63.4**
13	0.75	16.0	-10.7	16.8	-8.9	12.3	327.1	315.3	145.0	204.0	**47.4**	**64.3**
14	0.75	16.4	-10.7	17.2	-8.9	12.3	331.4	319.8	145.0	204.0	**48.1**	**65.2**
ROOF	0.75	16.7	-10.7	17.6	-8.9	6.1	167.9	162.2	145.0	204.0	**24.3**	**33.1**

Case 8: NS (+.15) EW(+.15)																			
Floor		EW psf		NS psf		Story Height	plf		Bldg. Width		Shear		CM		CR		0.15B		Mz
	Multiplier	Windward	Leeward	Windward	Leeward		EW	NS	EW	NS	EW	NS	EW	NS	EW	NS	EW	NS	
1	0.563	6.6	-8.0	6.9	-6.7	15.6	227.9	212.2	145.0	204.0	**33.0**	**43.3**	72.8	105.3	80	103	21.75	30.6	**-1904.7**
2	0.563	6.6	-8.0	6.9	-6.7	12.3	178.7	166.3	145.0	204.0	**25.9**	**33.9**	72.8	105.3	80	103	21.75	30.6	**-1493.3**
3	0.563	7.6	-8.0	8.0	-6.7	12.3	191.5	179.8	145.0	204.0	**27.8**	**36.7**	72.8	105.3	80	103	21.75	30.6	**-1611.1**
4	0.563	8.4	-8.0	8.8	-6.7	12.3	200.7	189.5	145.0	204.0	**29.1**	**38.7**	72.8	105.3	80	103	21.75	30.6	**-1695.7**
5	0.563	9.0	-8.0	9.4	-6.7	12.3	208.1	197.2	145.0	204.0	**30.2**	**40.2**	72.8	105.3	80	103	21.75	30.6	**-1762.7**
6	0.563	9.5	-8.0	10.0	-6.7	12.3	214.5	204.0	145.0	204.0	**31.1**	**41.6**	72.8	105.3	80	103	21.75	30.6	**-1821.9**
7	0.563	10.0	-8.0	10.5	-6.7	12.3	220.8	210.7	145.0	204.0	**32.0**	**43.0**	72.8	105.3	80	103	21.75	30.6	**-1879.9**
8	0.563	10.4	-8.0	10.9	-6.7	12.3	225.8	215.9	145.0	204.0	**32.7**	**44.0**	72.8	105.3	80	103	21.75	30.6	**-1925.3**
9	0.563	10.8	-8.0	11.3	-6.7	12.3	230.1	220.4	145.0	204.0	**33.4**	**45.0**	72.8	105.3	80	103	21.75	30.6	**-1964.5**
10	0.563	11.1	-8.0	11.7	-6.7	12.3	234.0	224.5	145.0	204.0	**33.9**	**45.8**	72.8	105.3	80	103	21.75	30.6	**-2000.8**
11	0.563	11.4	-8.0	12.0	-6.7	12.3	238.0	228.7	145.0	204.0	**34.5**	**46.7**	72.8	105.3	80	103	21.75	30.6	**-2037.0**
12	0.563	11.8	-8.0	12.4	-6.7	12.3	242.3	233.3	145.0	204.0	**35.1**	**47.6**	72.8	105.3	80	103	21.75	30.6	**-2076.9**
13	0.563	12.0	-8.0	12.6	-6.7	12.3	245.6	236.7	145.0	204.0	**35.6**	**48.3**	72.8	105.3	80	103	21.75	30.6	**-2106.5**
14	0.563	12.3	-8.0	12.9	-6.7	12.3	248.8	240.1	145.0	204.0	**36.1**	**49.0**	72.8	105.3	80	103	21.75	30.6	**-2136.1**
ROOF	0.563	12.6	-8.0	13.2	-6.7	6.1	126.0	121.7	145.0	204.0	**18.3**	**24.8**	72.8	105.3	80	103	21.75	30.6	**-1082.8**

Case 9: NS (-.15) EW(+.15)																			
Floor		EW psf		NS psf		Story Height	plf		Bldg. Width		Shear		CM		CR		0.15B		Mz
	Multiplier	Windward	Leeward	Windward	Leeward		EW	NS	EW	NS	EW	NS	EW	NS	EW	NS	EW	NS	
1	0.563	6.6	-8.0	6.9	-6.7	15.6	227.9	212.2	145.0	204.0	**33.0**	**43.3**	72.8	105.3	80	103	21.75	30.6	**-467.4**
2	0.563	6.6	-8.0	6.9	-6.7	12.3	178.7	166.3	145.0	204.0	**25.9**	**33.9**	72.8	105.3	80	103	21.75	30.6	**-366.4**
3	0.563	7.6	-8.0	8.0	-6.7	12.3	191.5	179.8	145.0	204.0	**27.8**	**36.7**	72.8	105.3	80	103	21.75	30.6	**-403.1**
4	0.563	8.4	-8.0	8.8	-6.7	12.3	200.7	189.5	145.0	204.0	**29.1**	**38.7**	72.8	105.3	80	103	21.75	30.6	**-429.5**
5	0.563	9.0	-8.0	9.4	-6.7	12.3	208.1	197.2	145.0	204.0	**30.2**	**40.2**	72.8	105.3	80	103	21.75	30.6	**-450.4**
6	0.563	9.5	-8.0	10.0	-6.7	12.3	214.5	204.0	145.0	204.0	**31.1**	**41.6**	72.8	105.3	80	103	21.75	30.6	**-468.9**
7	0.563	10.0	-8.0	10.5	-6.7	12.3	220.8	210.7	145.0	204.0	**32.0**	**43.0**	72.8	105.3	80	103	21.75	30.6	**-487.0**
8	0.563	10.4	-8.0	10.9	-6.7	12.3	225.8	215.9	145.0	204.0	**32.7**	**44.0**	72.8	105.3	80	103	21.75	30.6	**-501.1**
9	0.563	10.8	-8.0	11.3	-6.7	12.3	230.1	220.4	145.0	204.0	**33.4**	**45.0**	72.8	105.3	80	103	21.75	30.6	**-513.3**
10	0.563	11.1	-8.0	11.7	-6.7	12.3	234.0	224.5	145.0	204.0	**33.9**	**45.8**	72.8	105.3	80	103	21.75	30.6	**-524.6**
11	0.563	11.4	-8.0	12.0	-6.7	12.3	238.0	228.7	145.0	204.0	**34.5**	**46.7**	72.8	105.3	80	103	21.75	30.6	**-535.9**
12	0.563	11.8	-8.0	12.4	-6.7	12.3	242.3	233.3	145.0	204.0	**35.1**	**47.6**	72.8	105.3	80	103	21.75	30.6	**-548.4**
13	0.563	12.0	-8.0	12.6	-6.7	12.3	245.6	236.7	145.0	204.0	**35.6**	**48.3**	72.8	105.3	80	103	21.75	30.6	**-557.6**
14	0.563	12.3	-8.0	12.9	-6.7	12.3	248.8	240.1	145.0	204.0	**36.1**	**49.0**	72.8	105.3	80	103	21.75	30.6	**-566.8**
ROOF	0.563	12.6	-8.0	13.2	-6.7	6.1	126.0	121.7	145.0	204.0	**18.3**	**24.8**	72.8	105.3	80	103	21.75	30.6	**-288.0**

Case 10: NS (+.15) EW(-.15)																			
Floor		EW psf		NS psf		Story Height	plf		Bldg. Width		Shear		CM		CR		0.15B		Mz
	Multiplier	Windward	Leeward	Windward	Leeward		EW	NS	EW	NS	EW	NS	EW	NS	EW	NS	EW	NS	
1	0.563	6.6	-8.0	6.9	-6.7	15.6	227.9	212.2	145.0	204.0	**33.0**	**43.3**	72.8	105.3	80	103	21.75	30.6	**744.1**
2	0.563	6.6	-8.0	6.9	-6.7	12.3	178.7	166.3	145.0	204.0	**25.9**	**33.9**	72.8	105.3	80	103	21.75	30.6	**583.4**
3	0.563	7.6	-8.0	8.0	-6.7	12.3	191.5	179.8	145.0	204.0	**27.8**	**36.7**	72.8	105.3	80	103	21.75	30.6	**634.3**
4	0.563	8.4	-8.0	8.8	-6.7	12.3	200.7	189.5	145.0	204.0	**29.1**	**38.7**	72.8	105.3	80	103	21.75	30.6	**670.8**
5	0.563	9.0	-8.0	9.4	-6.7	12.3	208.1	197.2	145.0	204.0	**30.2**	**40.2**	72.8	105.3	80	103	21.75	30.6	**699.8**
6	0.563	9.5	-8.0	10.0	-6.7	12.3	214.5	204.0	145.0	204.0	**31.1**	**41.6**	72.8	105.3	80	103	21.75	30.6	**725.3**
7	0.563	10.0	-8.0	10.5	-6.7	12.3	220.8	210.7	145.0	204.0	**32.0**	**43.0**	72.8	105.3	80	103	21.75	30.6	**750.4**
8	0.563	10.4	-8.0	10.9	-6.7	12.3	225.8	215.9	145.0	204.0	**32.7**	**44.0**	72.8	105.3	80	103	21.75	30.6	**770.0**
9	0.563	10.8	-8.0	11.3	-6.7	12.3	230.1	220.4	145.0	204.0	**33.4**	**45.0**	72.8	105.3	80	103	21.75	30.6	**786.9**
10	0.563	11.1	-8.0	11.7	-6.7	12.3	234.0	224.5	145.0	204.0	**33.9**	**45.8**	72.8	105.3	80	103	21.75	30.6	**802.6**
11	0.563	11.4	-8.0	12.0	-6.7	12.3	238.0	228.7	145.0	204.0	**34.5**	**46.7**	72.8	105.3	80	103	21.75	30.6	**818.2**
12	0.563	11.8	-8.0	12.4	-6.7	12.3	242.3	233.3	145.0	204.0	**35.1**	**47.6**	72.8	105.3	80	103	21.75	30.6	**835.5**
13	0.563	12.0	-8.0	12.6	-6.7	12.3	245.6	236.7	145.0	204.0	**35.6**	**48.3**	72.8	105.3	80	103	21.75	30.6	**848.3**
14	0.563	12.3	-8.0	12.9	-6.7	12.3	248.8	240.1	145.0	204.0	**36.1**	**49.0**	72.8	105.3	80	103	21.75	30.6	**861.0**
ROOF	0.563	12.6	-8.0	13.2	-6.7	6.1	126.0	121.7	145.0	204.0	**18.3**	**24.8**	72.8	105.3	80	103	21.75	30.6	**436.9**

Case 11: NS (-.15) EW(-.15)																			
Floor		EW psf		NS psf		Story Height	plf		Bldg. Width		Shear		CM		CR		0.15B		Mz
	Multiplier	Windward	Leeward	Windward	Leeward		EW	NS	EW	NS	EW	NS	EW	NS	EW	NS	EW	NS	
1	0.563	6.6	-8.0	6.9	-6.7	15.6	227.9	212.2	145.0	204.0	**33.0**	**43.3**	72.8	105.3	80	103	21.75	30.6	**2181.5**
2	0.563	6.6	-8.0	6.9	-6.7	12.3	178.7	166.3	145.0	204.0	**25.9**	**33.9**	72.8	105.3	80	103	21.75	30.6	**1710.3**
3	0.563	7.6	-8.0	8.0	-6.7	12.3	191.5	179.8	145.0	204.0	**27.8**	**36.7**	72.8	105.3	80	103	21.75	30.6	**1842.2**
4	0.563	8.4	-8.0	8.8	-6.7	12.3	200.7	189.5	145.0	204.0	**29.1**	**38.7**	72.8	105.3	80	103	21.75	30.6	**1937.0**
5	0.563	9.0	-8.0	9.4	-6.7	12.3	208.1	197.2	145.0	204.0	**30.2**	**40.2**	72.8	105.3	80	103	21.75	30.6	**2012.1**
6	0.563	9.5	-8.0	10.0	-6.7	12.3	214.5	204.0	145.0	204.0	**31.1**	**41.6**	72.8	105.3	80	103	21.75	30.6	**2078.4**
7	0.563	10.0	-8.0	10.5	-6.7	12.3	220.8	210.7	145.0	204.0	**32.0**	**43.0**	72.8	105.3	80	103	21.75	30.6	**2143.4**
8	0.563	10.4	-8.0	10.9	-6.7	12.3	225.8	215.9	145.0	204.0	**32.7**	**44.0**	72.8	105.3	80	103	21.75	30.6	**2194.1**
9	0.563	10.8	-8.0	11.3	-6.7	12.3	230.1	220.4	145.0	204.0	**33.4**	**45.0**	72.8	105.3	80	103	21.75	30.6	**2238.1**
10	0.563	11.1	-8.0	11.7	-6.7	12.3	234.0	224.5	145.0	204.0	**33.9**	**45.8**	72.8	105.3	80	103	21.75	30.6	**2278.7**
11	0.563	11.4	-8.0	12.0	-6.7	12.3	238.0	228.7	145.0	204.0	**34.5**	**46.7**	72.8	105.3	80	103	21.75	30.6	**2319.3**
12	0.563	11.8	-8.0	12.4	-6.7	12.3	242.3	233.3	145.0	204.0	**35.1**	**47.6**	72.8	105.3	80	103	21.75	30.6	**2364.0**
13	0.563	12.0	-8.0	12.6	-6.7	12.3	245.6	236.7	145.0	204.0	**35.6**	**48.3**	72.8	105.3	80	103	21.75	30.6	**2397.1**
14	0.563	12.3	-8.0	12.9	-6.7	12.3	248.8	240.1	145.0	204.0	**36.1**	**49.0**	72.8	105.3	80	103	21.75	30.6	**2430.3**
ROOF	0.563	12.6	-8.0	13.2	-6.7	6.1	126.0	121.7	145.0	204.0	**18.3**	**24.8**	72.8	105.3	80	103	21.75	30.6	**1231.7**

Load Cases:

Load Combinations	
COMB1	1.4D
COMB2	1.2D+1.6L
COMB3	1.2D+1.6L+0.5Lr
COMB4	1.2D+1.6Lr+0.5W1
COMB5	1.2D+1.6Lr+0.5W2
COMB6	1.2D+1.6Lr+0.5W3
COMB7	1.2D+1.6Lr+0.5W4
COMB8	1.2D+1.6Lr+0.5W5
COMB9	1.2D+1.6Lr+0.5W6
COMB10	1.2D+1.6Lr+0.5W7
COMB11	1.2D+1.6Lr+0.5W8
COMB12	1.2D+1.6Lr+0.5W9
COMB13	1.2D+1.6Lr+0.5W10
COMB14	1.2D+1.6Lr+0.5W11
COMB15	1.2D+1.6Lr+0.5W12
COMB16	1.2D+1.0W1+1.0L+0.5Lr
COMB17	1.2D+1.0W2+1.0L+0.5Lr
COMB18	1.2D+1.0W3+1.0L+0.5Lr
COMB19	1.2D+1.0W4+1.0L+0.5Lr
COMB20	1.2D+1.0W5+1.0L+0.5Lr
COMB21	1.2D+1.0W6+1.0L+0.5Lr
COMB22	1.2D+1.0W7+1.0L+0.5Lr
COMB23	1.2D+1.0W8+1.0L+0.5Lr
COMB24	1.2D+1.0W9+1.0L+0.5Lr
COMB25	1.2D+1.0W10+1.0L+0.5Lr
COMB26	1.2D+1.0W11+1.0L+0.5Lr
COMB27	1.2D+1.0W12+1.0L+0.5Lr
COMB28	1.2D+1.0Ex+1.0L
COMB29	1.2D+1.0Ey+1.0L
COMB30	0.9D+1.0W1
COMB31	0.9D+1.0W2
COMB32	0.9D+1.0W3
COMB33	0.9D+1.0W4
COMB34	0.9D+1.0W5
COMB35	0.9D+1.0W6
COMB36	0.9D+1.0W7
COMB37	0.9D+1.0W8
COMB38	0.9D+1.0W9
COMB39	0.9D+1.0W10
COMB40	0.9D+1.0W11
COMB41	0.9D+1.0W12
COMB42	0.9D+1.0Ex
COMB43	0.9D+1.0Ey

Appendix B:

Example Castellated Beam Calculation

Unfactored Linear Loads		
wDL	723	plf
wLL	676	plf
WTL	1399	plf
WCL	973	plf

Composite Slab Properties		
Deck Height	1.5	in
Conc. Topping	2.5	in
f'c	3	ksi
b_{eff}	67.5	in

Beam Properties	
Original Beam	W16X100
d	17
tw	0.585
bf	10.4
tf	0.985
Ix	1490
Sx	175
Zx	198
A	29.5
Es	29000
f'c	3000
Ec	3155.924
span	62

Resultant Shape	
e	10.0
b	5.0
dt	5.0
h	7.0
ho	14.0
dg	24.0
Φ	54.5
S	30.0
At/2	6.3
Yd	0.6

Net Section	
Anet	25.2
ybs	12.0
yts	12.0
deff	22.1
Ixnet	3102.2
Sx-net-top	258.5
Sx-net-bot	258.5

Composite Section	
n	9.189067
beff	93
Aconc	232.5
Actr	25.30181
Kc	0.501151
ec	2.75
ycc	6.775671
yc	7.391982
Ix-comp	6006.337
Sx-comp-conc	697.7607
Sx-comp-steel	309.733
deff-comp	26.41621

Calculate shear capacity of concrete deck	
ΦvVn	4.9
Vallow	3.3

Global shear and moment at each opening							
		Global Shear			Global Moment		
Hole #	X (ft)	DL (kips)	LL (kips)	Vu	DL (kip-ft)	LL (kip-ft)	Mu (kip-ft)
End	0.0	22.4	21.0	55.5	0.0	0.0	0.0
1	1.7	21.2	19.8	52.2	36.4	34.0	98.0
2	4.2	19.4	18.1	47.4	87.1	81.4	234.8
3	6.7	17.6	16.4	42.5	133.4	124.7	359.5
4	9.2	15.8	14.8	37.6	175.1	163.7	472.0
5	11.7	14.0	13.1	32.8	212.3	198.5	572.3
6	14.2	12.2	11.4	27.9	245.0	229.0	660.4
7	16.7	10.4	9.7	23.0	273.1	255.4	736.3
8	19.2	8.6	8.0	18.1	296.8	277.5	800.1
9	21.7	6.7	6.3	13.3	315.9	295.4	851.7
10	24.2	4.9	4.6	8.4	330.5	309.0	891.1
11	26.7	3.1	2.9	3.5	340.6	318.5	918.3
12	29.2	1.3	1.2	0.0	346.2	323.7	933.3
CL	31.7	-0.5	-0.5	0.0	347.2	324.7	936.1

Effective Depth of concrete														
Hole #	X (ft)	Ma	T1(i)	xc(i+1)	deff-comp	T1(i+1)	T1(i)/T1(i+1)	xc(i+2)	deff-comp	T1(i+2)	T1(i)/T1(i+2)	xc(i+3)	deff-comp	T1(i+3)
End	0.00	0.00	0.00	0.00	27.04	0.00	N/A	0.00	27.04	0.00	N/A	0.00	27.04	0.00
1	1.67	98.00	44.52	0.19	26.95	43.64	1.02	0.18	26.95	43.64	1.02	1.25	26.42	44.52
2	4.17	234.85	106.68	0.45	26.82	105.09	1.02	0.44	26.82	105.08	1.02	1.25	26.42	106.68
3	6.67	359.51	163.31	0.69	26.70	161.60	1.01	0.68	26.70	161.57	1.01	1.25	26.42	163.31
4	9.17	471.99	214.41	0.90	26.59	213.02	1.01	0.90	26.59	212.99	1.01	1.25	26.42	214.41
5	11.67	572.29	259.97	1.10	26.49	259.22	1.00	1.09	26.49	259.20	1.00	1.25	26.42	259.97
6	14.17	660.41	300.00	1.27	26.41	300.09	1.00	1.27	26.41	300.09	1.00	1.25	26.42	300.00
7	16.67	736.35	334.50	1.41	26.34	335.52	1.00	1.41	26.33	335.54	1.00	1.25	26.42	334.50
8	19.17	800.10	363.46	1.53	26.27	365.41	0.99	1.54	26.27	365.47	0.99	1.25	26.42	363.46
9	21.67	851.67	386.89	1.63	26.23	389.70	0.99	1.64	26.22	389.79	0.99	1.25	26.42	386.89
10	24.17	891.06	404.78	1.71	26.19	408.31	0.99	1.72	26.18	408.43	0.99	1.25	26.42	404.78
11	26.67	918.27	417.14	0.82	26.63	413.80	1.01	1.74	26.17	421.08	0.99	1.25	26.42	417.14
12	29.17	933.29	423.96	1.82	26.13	428.62	0.99	1.81	26.14	428.48	0.99	1.25	26.42	423.96
CL	31.67	936.14	425.26	2.82	25.63	438.31	0.97	1.85	26.12	430.13	0.99	1.25	26.42	425.26

Number of studs for full composite action		
V'	592.875	kips
V'	1259.278	kips
V'	592.875	
Qn	21	kips/stud
N	28.23214	studs
N	29	studs
Ntotal	58	
spacing	1.068966	

Local Axial Force at each Opening						
Hole #	X (ft)	T1 = T1(i+2)	NQn (kips)	Status	To (kips)	T1-new (kips)
End	0	0	0	N/A	N/A	N/A
1	1.7	44.5	32.7	Partial	14.1	46.8
2	4.2	106.7	81.9	Partial	29.7	111.6
3	6.7	163.3	131.0	Partial	38.7	169.7
4	9.2	214.4	180.1	Partial	41.1	221.1
5	11.7	260.0	229.2	Partial	36.8	266.0
6	14.2	300.0	278.3	Partial	26.0	304.3
7	16.7	334.5	327.4	Partial	8.5	335.9
8	19.2	363.5	376.5	Full	0.0	363.5
9	21.7	386.9	425.6	Full	0.0	386.9
10	24.2	404.8	474.8	Full	0.0	404.8
11	26.7	417.1	523.9	Full	0.0	417.1
12	29.2	424.0	573.0	Full	0.0	424.0
CL	31.7	425.3	622.1	Full	0.0	425.3

Local Moment at each Opening				
Hole #	X (ft)	Vu (kips)	Mu-top (kip-in)	Mu-bot (kip-in)
End	0.00	55.49	138.74	138.74
1	1.67	52.25	130.61	130.61
2	4.17	47.37	118.43	118.43
3	6.67	42.50	106.25	106.25
4	9.17	37.63	94.07	94.07
5	11.67	32.75	81.89	81.89
6	14.17	27.88	69.70	69.70
7	16.67	23.01	57.52	57.52
8	19.17	18.14	45.34	45.34
9	21.67	13.26	33.16	33.16
10	24.17	8.39	20.97	20.97
11	26.67	3.52	8.79	8.79
12	29.17	0.00	0.00	0.00
CL	31.67	0.00	0.00	0.00

Lateral Torsional Buckling	
J	3.577351
B	-5.84456
	5.844563
Mcr	4233.587

Tension capacity of bottom tee		
Pn	629.6388	kips

Design Tensile strength of bottom tee		
ΦcPn	566.6749	kips

Moment Capacity of tee		
Mp	197.0362	kip-in
	16.41968	kip-ft

Design Flexural Strength		
ΦbMn	177.3325	kip-in

Interaction Values at Each Opening										
		Top Tee			Bottom Tee					
Hole #	X (ft)	Pu (kip)	Mu (kip-in)	Mu/ΦMn	Pu (kip)	Mu (kip-ft)	Pr/Pc	H1-1a	H1-1b	Interaction
End	0	N/A	N/A	N/A	N/A	0	N/A	N/A	N/A	N/A
1	1.67	0.00	130.61	0.74	46.83	130.61	0.08	0.74	0.78	0.78
2	4.17	0.00	118.43	0.67	111.56	118.43	0.20	0.79	0.77	0.77
3	6.67	0.00	106.25	0.60	169.66	106.25	0.30	0.83	0.75	0.83
4	9.17	0.00	94.07	0.53	221.15	94.07	0.39	0.86	0.73	0.86
5	11.67	0.00	81.89	0.46	266.01	81.89	0.47	0.88	0.70	0.88
6	14.17	0.00	69.70	0.39	304.26	69.70	0.54	0.89	0.66	0.89
7	16.67	0.00	57.52	0.32	335.89	57.52	0.59	0.88	0.62	0.88
8	19.17	0.00	45.34	0.26	363.46	45.34	0.64	0.87	0.58	0.87
9	21.67	0.00	33.16	0.19	386.89	33.16	0.68	0.85	0.53	0.85
10	24.17	0.00	20.97	0.12	404.78	20.97	0.71	0.82	0.48	0.82
11	26.67	0.00	8.79	0.05	417.14	8.79	0.74	0.78	0.42	0.78
12	29.17	0.00	0.00	0.00	423.96	0.00	0.75	0.75	0.37	0.75
CL	31.67	0.00	0.00	0.00	425.26	0.00	0.75	0.75	0.38	0.75

Horizontal shear force				
Post #	X (ft)	Vu (i-1)	Vu (i+1)	Vu(i)
1	2.9	52.2	47.4	49.8
2	5.4	47.4	42.5	44.9
3	7.9	42.5	37.6	40.1
4	10.4	37.6	32.8	35.2
5	12.9	32.8	27.9	30.3
6	15.4	27.9	23.0	25.4
7	17.9	23.0	18.1	20.6
8	20.4	18.1	13.3	15.7
9	22.9	13.3	8.4	10.8
10	25.4	8.4	3.5	6.0
11	27.9	3.5	0.0	1.8
12	30.4	0.0	0.0	0.0
CL	32.9	0.0	0.0	0.0

Calculate Web Post Buckling Moment		
Vu	47.73657	kip
Mu-top	601.1359	in-kip
Mu-bot	601.1359	in-kip

Moment Capacity of Web Post	
Mp-top	2925
2h/e	1.4
e/tw	17.09402
Mocr/Mp	0.98301

Design Flexural Strength	
Φb(Mocr/Mp)*Mp	2587.77405

Web Post Buckling	
Imax-top	0.232298

Check Horizontal & Vertical Shear				Cv
e/tw	17.09402	<	29.01999311	1
29.01999311	<	17.09401709	<	36.14308233
17.09401709	>	36.14308233		

Cv	1	
Vn-Horiz	175.5	kips
ΦvVn	157.95	kips

Vertical Shear				
Vu	26.12284			Cv
h/tw	6.350427	<	29.01999311	1
Vn-top	175.5	kips		
ΦvVn	157.95	kips		

Shear at full sections				Cv
h/tw	36.63248	<	59.23681288	1
		<	53.94634371	1
Vn-full	421.2			
ΦvVn	421.2	kips		

RAM Composite Beam Design:

East Edge Beam

Floor Type: Typical **Beam Number = 118**

SPAN INFORMATION (ft): I-End (203.45,84.00) J-End (203.45,104.00)

Beam Size (Optimum) = W24X76 Fy = 50.0 ksi

Total Beam Length (ft) = 20.00

COMPOSITE PROPERTIES (Not Shored):

	Left	Right
Concrete thickness (in)	2.50	2.50
Unit weight concrete (pcf)	115.00	115.00
f'c (ksi)	3.00	3.00
Decking Orientation	perpendicular	perpendicular
Decking type	VULCRAFT 2.0VL	VULCRAFT 2.0VL

beff (in)	=	31.00	Y bar(in)	=	15.14
Mnf (kip-ft)	=	1050.03	Mn (kip-ft)	=	1013.82
C (kips)	=	155.07	PNA (in)	=	15.47
Ieff (in4)	=	3065.78	Itr (in4)	=	3190.28
Stud length (in)	=	4.00	Stud diam (in)	=	0.75

Stud Capacity (kips) Qn = 17.2 Rg = 1.00 Rp = 0.60

# of studs per stud segment: Full = 13,13

Partial = 9,9

Actual = 9,9

Number of Stud Rows = 1 Percent of Full Composite Action = 78.47

POINT LOADS (kips):

Dist	DL	CDL	RedLL	Red%	NonRLL	StorLL	Red%	RoofLL	Red%	PartL	CLL
9.250	50.42	50.42	74.67	38.6	0.00	0.00	0.0	0.00	Snow	14.93	14.93

LINE LOADS (k/ft):

Load	Dist	DL	CDL	LL	Red%	Type	PartL	CLL
1	0.000	0.306	0.306	0.000	0.0%	Red	0.000	0.000
	20.000	0.306	0.306	0.000			0.000	0.000
2	0.000	0.178	0.178	0.000	---	NonR	0.000	0.000
	9.250	0.178	0.178	0.000			0.000	0.000
3	0.000	0.075	0.075	0.500	38.6%	Red	0.100	0.100
	9.250	0.075	0.075	0.500			0.100	0.100
4	9.250	0.176	0.176	0.000	---	NonR	0.000	0.000
	20.000	0.176	0.176	0.000			0.000	0.000
5	9.250	0.074	0.074	0.495	38.6%	Red	0.099	0.099
	20.000	0.074	0.074	0.495			0.099	0.099
6	0.037	0.003	0.003	0.000	---	NonR	0.000	0.000
	20.000	0.003	0.003	0.000			0.000	0.000
7	0.037	0.001	0.001	0.008	38.6%	Red	0.002	0.002
	20.000	0.001	0.001	0.008			0.002	0.002
8	0.000	0.000	0.000	0.000	---	NonR	0.000	0.000
	0.036	0.003	0.003	0.000			0.000	0.000
9	0.000	0.000	0.000	0.000	38.6%	Red	0.000	0.000
	0.036	0.001	0.001	0.008			0.002	0.002

Load	Dist	DL	CDL	LL	Red%	Type	PartL	CLL
10	0.000	0.076	0.076	0.000	---	NonR	0.000	0.000
	20.000	0.076	0.076	0.000			0.000	0.000

SHEAR: Max Va (DL+LL) = 70.29 kips Vn/1.50 = 210.32 kips

MOMENTS:

Span	Cond	LoadCombo	Ma kip-ft	@ ft	Lb ft	Cb	Ω	Mn / Ω kip-ft
Center	PreCmp+	DL+LL	361.7	9.3	0.0	1.00	1.67	499.00
	Init DL	DL	282.4	9.3	---	---		
	Max +	DL+LL	605.2	9.3	---	---	1.67	607.08
Controlling		DL+LL	605.2	9.3	---	---	1.67	607.08

REACTIONS (kips):

	Left	Right
Initial reaction	42.52	37.61
DL reaction	33.48	29.69
Max +LL reaction	36.81	32.23
Max +total reaction (factored)	70.29	61.92

DEFLECTIONS:

Initial load (in)	at	9.80 ft	=	-0.274	L/D	=	875
Live load (in)	at	9.80 ft	=	-0.212	L/D	=	1131
Post Comp load (in)	at	9.80 ft	=	-0.212	L/D	=	1131
Net Total load (in)	at	9.80 ft	=	-0.486	L/D	=	493

North Edge Beam:

Floor Type: Typical **Beam Number = 114**

SPAN INFORMATION (ft): I-End (45.25,144.00) J-End (84.15,144.00)

Beam Size (User Selected) = W27X102 Fy = 50.0 ksi
Total Beam Length (ft) = 38.90

COMPOSITE PROPERTIES (Not Shored):

	Left	Right
Concrete thickness (in)	2.50	2.50
Unit weight concrete (pcf)	115.00	115.00
f'c (ksi)	3.00	3.00
Decking Orientation	parallel	parallel
Decking type	VULCRAFT 2.0VL	VULCRAFT 2.0VL

beff (in) =	59.35	Y bar(in) =	18.18
Mnf (kip-ft) =	1685.91	Mn (kip-ft) =	1419.41
C (kips) =	106.10	PNA (in) =	15.61
Ieff (in4) =	4858.16	Itr (in4) =	5958.19
Stud length (in) =	4.00	Stud diam (in) =	0.75

Stud Capacity (kips) Qn = 17.7 Rg = 1.00 Rp = 0.75

# of studs per stud segment: Full = 11,11,12,11
Partial = 3,3,3,3
Actual = 3,3,3,3

Number of Stud Rows = 1 Percent of Full Composite Action = 28.04

POINT LOADS (kips):

Dist	DL	CDL	RedLL	Red%	NonRLL	StorLL	Red%	RoofLL	Red%	PartL	CLL
10.000	17.48	17.48	29.24	37.6	0.00	0.00	0.0	0.00	Snow	5.85	5.85
19.500	16.33	16.33	27.58	37.6	0.00	0.00	0.0	0.00	Snow	5.52	5.52
29.500	13.46	13.46	23.37	37.6	0.00	0.00	0.0	0.00	Snow	4.67	4.67

LINE LOADS (k/ft):

Load	Dist	DL	CDL	LL	Red%	Type	PartL	CLL
1	0.000	0.306	0.306	0.000	0.0%	Red	0.000	0.000
	38.900	0.306	0.306	0.000			0.000	0.000
2	0.000	0.003	0.003	0.000	---	NonR	0.000	0.000
	38.900	0.003	0.003	0.000			0.000	0.000
3	0.000	0.001	0.001	0.008	37.6%	Red	0.002	0.002
	38.900	0.001	0.001	0.008			0.002	0.002
4	0.000	0.102	0.102	0.000	---	NonR	0.000	0.000
	38.900	0.102	0.102	0.000			0.000	0.000

SHEAR: Max Va (DL+LL) = 66.41 kips Vn/1.50 = 279.13 kips

MOMENTS:

Span	Cond	LoadCombo	Ma	@	Lb	Cb	Ω	Mn / Ω
			kip-ft	ft	ft			kip-ft
Center	PreCmp+	DL+LL	492.6	19.5	10.0	1.13	1.67	760.98
	Init DL	DL	387.4	19.5	---	---		
	Max +	DL+LL	820.5	19.5	---	---	1.67	849.94

Span	Cond	LoadCombo	Ma	@	Lb	Cb	Ω	Mn / Ω
Controlling		DL+LL	820.5	19.5	---	---	1.67	849.94

REACTIONS (kips):

	Left	Right
Initial reaction	40.66	38.75
DL reaction	32.40	30.91
Max +LL reaction	34.01	32.31
Max +total reaction (factored)	66.41	63.22

DEFLECTIONS:

Initial load (in)	at	19.26 ft	=	-0.962	L/D	=	485
Live load (in)	at	19.26 ft	=	-0.793	L/D	=	589
Post Comp load (in)	at	19.26 ft	=	-0.793	L/D	=	589
Net Total load (in)	at	19.26 ft	=	-1.754	L/D	=	266

South Edge Beam:

Floor Type: Typical **Beam Number = 127**

SPAN INFORMATION (ft): I-End (84.15,21.67) J-End (123.90,28.00)

Beam Size (Optimum) = W30X90 Fy = 50.0 ksi
Total Beam Length (ft) = 40.25

COMPOSITE PROPERTIES (Not Shored):

	Left	Right
Concrete thickness (in)	2.50	2.50
Unit weight concrete (pcf)	115.00	115.00
f'c (ksi)	3.00	3.00
Decking Orientation	parallel	parallel
Decking type	VULCRAFT 2.0VL	VULCRAFT 2.0VL

beff (in)	= 61.38	Y bar(in)	=	20.31
Mnf (kip-ft)	= 1631.70	Mn (kip-ft)	=	1337.73
C (kips)	= 106.10	PNA (in)	=	17.01
Ieff (in4)	= 4988.59	Itr (in4)	=	6257.46
Stud length (in)	= 4.00	Stud diam (in)	=	0.75

Stud Capacity (kips) Qn = 17.7 Rg = 1.00 Rp = 0.75
# of studs per stud segment: Full = 12,12,12,11
Partial = 3,3,3,3
Actual = 3,3,3,3

Number of Stud Rows = 1 Percent of Full Composite Action = 27.12

POINT LOADS (kips):

Dist	DL	CDL	RedLL	Red%	NonRLL	StorLL	Red%	RoofLL	Red%	PartL	CLL
10.000	14.71	14.71	23.15	34.4	0.00	0.00	0.0	0.00	Snow	4.63	4.63
20.000	13.68	13.68	21.94	34.4	0.00	0.00	0.0	0.00	Snow	4.39	4.39
30.251	13.52	13.52	21.25	34.4	0.00	0.00	0.0	0.00	Snow	4.25	4.25

LINE LOADS (k/ft):

Load	Dist	DL	CDL	LL	Red%	Type	PartL	CLL
1	0.000	0.306	0.306	0.000	0.0%	Red	0.000	0.000
	40.251	0.306	0.306	0.000			0.000	0.000
2	0.000	0.283	0.283	0.000	0.0%	Red	0.000	0.000
	10.000	0.283	0.283	0.000			0.000	0.000
3	0.000	0.000	0.000	0.000	---	NonR	0.000	0.000
	10.000	0.028	0.028	0.000			0.000	0.000
4	0.000	0.000	0.000	0.000	34.4%	Red	0.000	0.000
	10.000	0.012	0.012	0.080			0.016	0.016
5	10.000	0.000	0.000	0.000	---	NonR	0.000	0.000
	20.000	0.028	0.028	0.000			0.000	0.000
6	10.408	0.001	0.001	0.000	0.0%	Red	0.000	0.000
	20.000	0.271	0.271	0.000			0.000	0.000
7	10.408	0.000	0.000	0.003	34.4%	Red	0.001	0.001
	20.000	0.012	0.012	0.080			0.016	0.016
8	20.000	0.000	0.000	0.000	---	NonR	0.000	0.000
	30.251	0.029	0.029	0.000			0.000	0.000

Load	Dist	DL	CDL	LL	Red%	Type	PartL	CLL
9	20.000	0.000	0.000	0.000	34.4%	Red	0.000	0.000
	30.251	0.012	0.012	0.082			0.016	0.016
10	20.817	0.001	0.001	0.000	0.0%	Red	0.000	0.000
	21.185	0.088	0.088	0.000			0.000	0.000
11	21.185	0.096	0.096	0.000	0.0%	Red	0.000	0.000
	30.251	0.306	0.306	0.000			0.000	0.000
12	30.252	0.000	0.000	0.000	---	NonR	0.000	0.000
	40.251	0.028	0.028	0.000			0.000	0.000
13	30.669	0.001	0.001	0.000	0.0%	Red	0.000	0.000
	40.251	0.319	0.319	0.000			0.000	0.000
14	30.669	0.000	0.000	0.003	34.4%	Red	0.001	0.001
	40.251	0.012	0.012	0.080			0.016	0.016
15	0.002	0.003	0.003	0.000	---	NonR	0.000	0.000
	40.249	0.003	0.003	0.000			0.000	0.000
16	0.002	0.001	0.001	0.008	34.4%	Red	0.002	0.002
	40.249	0.001	0.001	0.008			0.002	0.002
17	0.000	0.090	0.090	0.000	---	NonR	0.000	0.000
	40.251	0.090	0.090	0.000			0.000	0.000

SHEAR: Max Va (DL+LL) = 63.34 kips Vn/1.67 = 249.07 kips

MOMENTS:

Span	Cond	LoadCombo	Ma kip-ft	@ ft	Lb ft	Cb	Ω	Mn / Ω kip-ft
Center	PreCmp+	DL+LL	487.6	20.0	10.3	1.10	1.67	706.09
	Init DL	DL	397.1	20.0	---	---		
	Max +	DL+LL	784.5	20.0	---	---	1.67	801.04
Controlling		DL+LL	784.5	20.0	---	---	1.67	801.04

REACTIONS (kips):

	Left	Right
Initial reaction	40.63	39.35
DL reaction	33.70	32.62
Max +LL reaction	29.63	28.81
Max +total reaction (factored)	63.34	61.43

DEFLECTIONS:

Initial load (in)	at	20.13 ft	=	-1.070	L/D	=	451
Live load (in)	at	20.13 ft	=	-0.743	L/D	=	650
Post Comp load (in)	at	20.13 ft	=	-0.743	L/D	=	650
Net Total load (in)	at	20.13 ft	=	-1.814	L/D	=	266

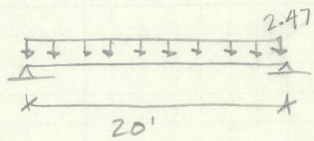
Composite Hand Calculations:

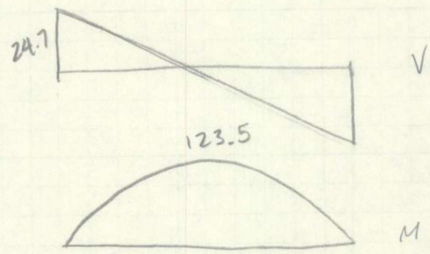
BEAMS SPANNING BTWN D & E @ 1

$L = 20'$

LOADS: $L = 100$ psf TRIB WIDTH = 10'
$D_{SI} = 15$
$D_{SW} = 57.3$

$w = 1.2(15+57.3)(10) + 1.6(10)(100) = 2.47$ klf





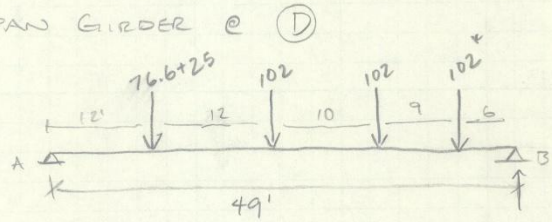
TRY W12x26
$\phi M_x = 140$ ft-k
$I_x = 204$

$\Delta = L/360 = 0.66''$

$\Delta_{LL} = \frac{5wL^4}{384EI} = \frac{5(1.0)(20^4)(1728)}{384(29000)(204)} = 0.61$

∴ W12x26 WORKS

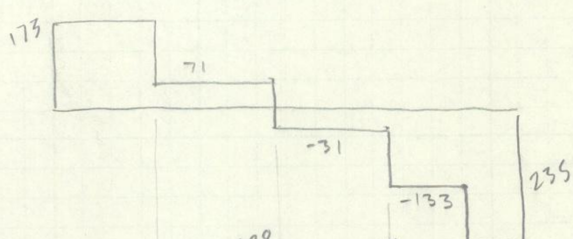
SPAN GIRDER @ D



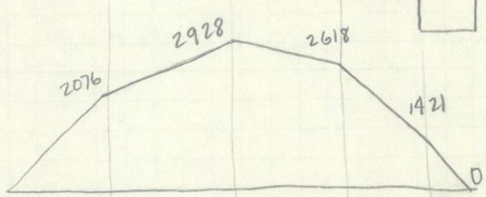
$\Sigma M_A = 0$

$49(R_B) = 102(12+24+34+43)$

$R_B = 235$
$R_A = 173$

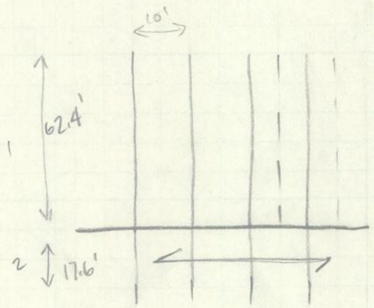


W/ OUT COMPOSITE,
NEED W40x199
∴ TOO DEEP



COMPOSITE BEAM DESIGN 1

G-1



$D_L = 55$ psf

$L_L = 100$ REDUCIBLE

$$L_R = 100\left[0.25 + \frac{15}{\sqrt{2(10)(62)}}\right] = 68 \text{ psf}$$

$P_{D_1} = 55(62.4)(10)/2/1000 = 17.2^K$

$P_{D2} = 55(17.6)(10)/2/1000 = 4.84$

$P_{L1} = 68(62.4)(10)/2/1000 = 21.2^K$

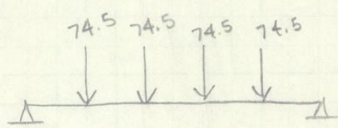
$P_{L2} = 100(17.6)(10)/2/1000 = 8.8^K$

L.C. $1.2D + 1.6L$

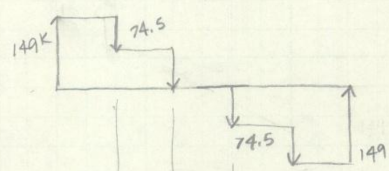
$1.2(17.2 + 4.84) + 1.6(21.2 + 8.8)$

$P_{TOT} = 74.5^K$

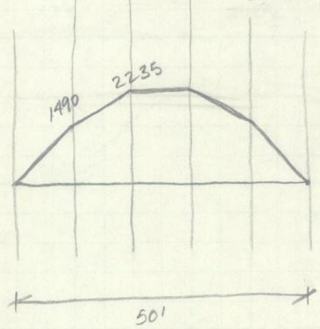
LOADING:



SHEAR:

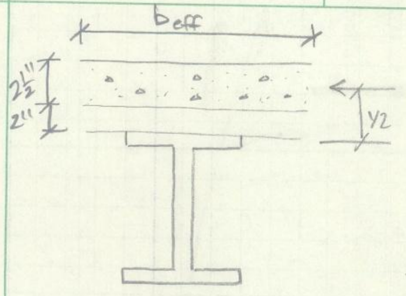


MOMENT:



$M_u = 2235$ ft-K

2



$a \approx 1.0$

$4.5 - 1.0/2 = 4.0 \therefore$ TRY Y2 = 4

POSSIBILITIES : W30x108 $\Sigma Q_n = 1590$
$\phi M_n = 2250$

$\frac{1590}{17.1} = 93 \rightarrow \frac{186 \text{ STUDS}}{50'} = 3.72$ STUDS/FT

$a = \frac{A_s F_y}{0.85 f'_c b_{eff}}$ $b_{eff} = \frac{50(12)}{8}$
$= 75 \times 2 = 150$

$a = \frac{31.7(50)}{.85(3)(150)} = 4.14$

$\therefore$ No Good

$\therefore$ TRY W36x135 w/ Y1 = 9.49
$\Sigma Q_n = 497$, $\phi M_n = 2570$

$a = \frac{497}{.85(3)(150)} = 1.3''$ $\frac{497}{17.1} = 30 \rightarrow 60$ STUDS
1.2 STUDS/FT

$\therefore$ ADJUST Y2

$x = \frac{A_s F_y - 0.85 f'_c b_{eff} t}{2 F_y b_f} = \frac{39.7(50) - .85(3)(150)(2.5)}{2(50)(12)} = 0.857$

$\therefore$ GO TO W36x150

$\Sigma Q_n = 1090 \rightarrow 128$ STUDS $\rightarrow 2.6$ STUDS/FT
$\phi M_n = 3180$

DEFLECTION : ASSUME UNIFORM LOAD

$4(30) = 120^k/50' = 2.4\ ^k/ft$

$\Delta_{LL} = \frac{50(12)}{360} = 1.67$

$\Delta = \frac{5 w_L l^4 (1728)}{384 E I} = \frac{5(2.4)(50^4)(1728)}{384(29000)(15400)} = 0.76 \therefore$ GOOD

Appendix C:

Ram Column Design:

Column Line 1-L

Level	P	Mx	My	LC	Interaction Eq.	Angle	Fy	Size
Story 14	45.5	15.3	13.7	1	0.50 Eq (H1-1b)	90.0	50	W14X43
Story 13	83.5	7.1	6.3	1	0.51 Eq (H1-1a)	90.0	50	W14X43
Story 12	120.3	6.8	6.0	1	0.65 Eq (H1-1a)	90.0	50	W14X43
Story 11	156.4	6.6	5.8	1	0.80 Eq (H1-1a)	90.0	50	W14X43
Story 10	192.1	6.5	5.7	1	0.94 Eq (H1-1a)	90.0	50	W14X43
Story 9	227.6	6.5	5.7	1	0.96 Eq (H1-1a)	90.0	50	W14X48
Story 8	263.0	6.5	5.6	1	0.98 Eq (H1-1a)	90.0	50	W14X53
Story 7	298.2	6.4	6.4	1	0.82 Eq (H1-1a)	90.0	50	W14X61
Story 6	333.3	6.4	6.3	1	0.90 Eq (H1-1a)	90.0	50	W14X61
Story 5	368.3	6.3	6.3	1	0.99 Eq (H1-1a)	90.0	50	W14X61
Story 4	403.3	6.3	6.3	1	0.95 Eq (H1-1a)	90.0	50	W14X68
Story 3	438.3	6.4	6.3	1	0.94 Eq (H1-1a)	90.0	50	W14X74
Story 2	473.3	7.8	7.6	1	0.93 Eq (H1-1a)	90.0	50	W14X82
Base	508.9	4.9	6.4	1	0.89 Eq (H1-1a)	90.0	50	W14X90

Column Line 1-O

Level	P	Mx	My	LC	Interaction Eq.	Angle	Fy	Size
Story 14	168.5	78.4	43.7	1	0.94 Eq (H1-1a)	0.0	50	W14X74
Story 13	315.5	36.3	20.3	1	0.99 Eq (H1-1a)	0.0	50	W14X68
Story 12	471.1	36.1	26.3	1	0.88 Eq (H1-1a)	0.0	50	W14X90
Story 11	628.4	36.5	26.5	1	1.00 Eq (H1-1a)	0.0	50	W14X99
Story 10	786.0	37.1	26.6	1	0.99 Eq (H1-1a)	0.0	50	W14X120
Story 9	943.9	37.7	27.7	1	0.94 Eq (H1-1a)	0.0	50	W14X145
Story 8	1101.9	38.0	27.8	1	0.98 Eq (H1-1a)	0.0	50	W14X159
Story 7	1260.2	38.4	27.9	1	1.00 Eq (H1-1a)	0.0	50	W14X176
Story 6	1418.9	39.4	28.1	1	0.93 Eq (H1-1a)	0.0	50	W14X211
Story 5	1577.8	40.0	28.2	1	0.92 Eq (H1-1a)	0.0	50	W14X233
Story 4	1737.1	40.7	28.3	1	0.91 Eq (H1-1a)	0.0	50	W14X257
Story 3	1896.3	40.7	28.3	1	0.99 Eq (H1-1a)	0.0	50	W14X257
Story 2	2055.9	50.2	34.6	1	0.98 Eq (H1-1a)	0.0	50	W14X283
Base	2218.5	33.6	22.6	1	0.95 Eq (H1-1a)	0.0	50	W14X342

Column Line 1-U

Level	P	Mx	My	LC	Interaction Eq.	Angle	Fy	Size
Story 14	93.0	0.0	32.8	6	0.94 Eq (H1-1a)	0.0	50	W14X48
Story 13	198.3	0.0	15.2	2	0.90 Eq (H1-1a)	0.0	50	W14X53
Story 12	290.0	0.0	16.9	2	0.89 Eq (H1-1a)	0.0	50	W14X61
Story 11	382.3	0.0	16.6	2	0.99 Eq (H1-1a)	0.0	50	W14X68
Story 10	478.1	0.0	16.7	2	0.99 Eq (H1-1a)	0.0	50	W14X82
Story 9	574.1	0.0	21.6	2	0.92 Eq (H1-1a)	0.0	50	W14X90
Story 8	670.1	0.0	21.7	2	0.96 Eq (H1-1a)	0.0	50	W14X99
Story 7	766.3	0.0	21.7	2	0.98 Eq (H1-1a)	0.0	50	W14X109
Story 6	862.7	0.0	21.8	2	0.99 Eq (H1-1a)	0.0	50	W14X120

Story 5	959.1	0.0	21.8	2	0.99 Eq (H1-1a)	0.0	50	W14X132
Story 4	1055.8	0.0	22.7	2	0.97 Eq (H1-1a)	0.0	50	W14X145
Story 3	1152.6	0.0	22.8	2	0.97 Eq (H1-1a)	0.0	50	W14X159
Story 2	1249.6	0.0	27.9	2	0.95 Eq (H1-1a)	0.0	50	W14X176
Base	1348.5	0.0	14.8	1	0.94 Eq (H1-1a)	0.0	50	W14X211

Column Line 1-W

Level	P	Mx	My	LC	Interaction Eq.	Angle	Fy	Size
Story 14	98.7	54.6	15.2	1	1.00 Eq (H1-1a)	0.0	50	W14X43
Story 13	186.7	25.9	7.2	1	0.93 Eq (H1-1a)	0.0	50	W14X48
Story 12	273.0	25.3	8.1	1	0.84 Eq (H1-1a)	0.0	50	W14X61
Story 11	358.5	25.1	8.0	1	0.93 Eq (H1-1a)	0.0	50	W14X68
Story 10	447.7	25.4	8.1	1	0.93 Eq (H1-1a)	0.0	50	W14X82
Story 9	537.6	25.0	10.4	1	0.87 Eq (H1-1a)	0.0	50	W14X90
Story 8	627.5	25.0	10.4	1	1.00 Eq (H1-1a)	0.0	50	W14X90
Story 7	717.6	25.4	10.5	1	0.93 Eq (H1-1a)	0.0	50	W14X109
Story 6	807.9	25.7	10.5	1	0.94 Eq (H1-1a)	0.0	50	W14X120
Story 5	898.3	25.9	10.5	1	0.94 Eq (H1-1a)	0.0	50	W14X132
Story 4	988.9	26.1	10.9	1	0.92 Eq (H1-1a)	0.0	50	W14X145
Story 3	1079.5	26.1	10.9	1	1.00 Eq (H1-1a)	0.0	50	W14X145
Story 2	1170.2	32.0	13.4	1	1.00 Eq (H1-1a)	0.0	50	W14X159
Base	1262.7	21.2	8.7	1	0.97 Eq (H1-1a)	0.0	50	W14X193

Column Line 2-A

Level	P	Mx	My	LC	Interaction Eq.	Angle	Fy	Size
Story 14	79.6	48.3	9.3	1	0.77 Eq (H1-1a)	0.0	50	W14X43
Story 13	150.2	22.7	4.4	1	0.83 Eq (H1-1a)	0.0	50	W14X43
Story 12	219.3	22.2	4.3	1	0.98 Eq (H1-1a)	0.0	50	W14X48
Story 11	287.9	22.0	4.9	1	0.83 Eq (H1-1a)	0.0	50	W14X61
Story 10	355.9	21.7	4.8	1	0.99 Eq (H1-1a)	0.0	50	W14X61
Story 9	424.0	21.9	4.8	1	0.94 Eq (H1-1a)	0.0	50	W14X74
Story 8	495.0	22.0	4.8	1	0.98 Eq (H1-1a)	0.0	50	W14X82
Story 7	566.0	21.6	6.2	1	0.88 Eq (H1-1a)	0.0	50	W14X90
Story 6	637.1	21.6	6.2	1	0.98 Eq (H1-1a)	0.0	50	W14X90
Story 5	708.3	21.9	6.3	1	0.98 Eq (H1-1a)	0.0	50	W14X99
Story 4	779.7	22.0	6.3	1	0.98 Eq (H1-1a)	0.0	50	W14X109
Story 3	851.1	22.2	6.3	1	0.96 Eq (H1-1a)	0.0	50	W14X120
Story 2	922.7	27.3	7.7	1	0.95 Eq (H1-1a)	0.0	50	W14X132
Base	995.7	17.8	5.2	1	0.94 Eq (H1-1a)	0.0	50	W14X159

Column Line 4-C

Level	P	Mx	My	LC	Interaction Eq.	Angle	Fy	Size
Story 14	157.5	11.0	18.4	3	0.96 Eq (H1-1a)	0.0	50	W14X48
Story 13	298.6	9.8	9.9	3	0.87 Eq (H1-1a)	0.0	50	W14X61
Story 12	442.9	9.7	9.8	3	0.99 Eq (H1-1a)	0.0	50	W14X74

Story 11	590.9	9.6	12.7	3	0.92	Eq (H1-1a)	0.0	50	W14X90
Story 10	739.1	9.7	12.8	3	0.93	Eq (H1-1a)	0.0	50	W14X109
Story 9	887.6	10.0	12.8	3	0.91	Eq (H1-1a)	0.0	50	W14X132
Story 8	1036.3	10.0	13.3	3	0.94	Eq (H1-1a)	0.0	50	W14X145
Story 7	1185.1	10.1	13.4	3	0.98	Eq (H1-1a)	0.0	50	W14X159
Story 6	1334.2	10.2	13.5	3	0.99	Eq (H1-1a)	0.0	50	W14X176
Story 5	1483.5	10.4	13.5	3	1.00	Eq (H1-1a)	0.0	50	W14X193
Story 4	1633.2	10.6	13.6	3	0.91	Eq (H1-1a)	0.0	50	W14X233
Story 3	1782.9	10.6	13.6	3	0.99	Eq (H1-1a)	0.0	50	W14X233
Story 2	1933.0	13.1	16.6	4	0.97	Eq (H1-1a)	0.0	50	W14X257
Base	2085.8	0.9	10.8	1	0.96	Eq (H1-1a)	0.0	50	W14X311

Column Line 4-W

Level	P	Mx	My	LC	Interaction Eq.	Angle	Fy	Size
Story 14	179.9	18.7	23.7	3	0.76 Eq (H1-1a)	0.0	50	W14X61
Story 13	335.0	16.6	10.9	3	0.99 Eq (H1-1a)	0.0	50	W14X61
Story 12	502.7	16.7	14.2	3	0.82 Eq (H1-1a)	0.0	50	W14X90
Story 11	670.7	16.8	14.3	3	0.96 Eq (H1-1a)	0.0	50	W14X99
Story 10	838.9	17.1	14.4	3	0.97 Eq (H1-1a)	0.0	50	W14X120
Story 9	1007.4	17.4	15.0	3	0.93 Eq (H1-1a)	0.0	50	W14X145
Story 8	1176.0	17.5	15.0	3	0.99 Eq (H1-1a)	0.0	50	W14X159
Story 7	1345.1	18.0	15.1	3	0.92 Eq (H1-1a)	0.0	50	W14X193
Story 6	1514.4	18.2	15.2	3	0.94 Eq (H1-1a)	0.0	50	W14X211
Story 5	1684.0	18.4	15.3	3	0.94 Eq (H1-1a)	0.0	50	W14X233
Story 4	1853.9	18.8	15.3	3	0.93 Eq (H1-1a)	0.0	50	W14X257
Story 3	2024.0	19.0	15.4	3	0.92 Eq (H1-1a)	0.0	50	W14X283
Story 2	2194.6	23.6	18.8	3	0.91 Eq (H1-1a)	0.0	50	W14X311
Base	2367.8	6.4	12.2	1	0.98 Eq (H1-1a)	0.0	50	W14X342

Column Line KK-O

Level	P	Mx	My	LC	Interaction Eq.	Angle	Fy	Size
Story 14	129.2	10.7	64.6	10	0.94 Eq (H1-1a)	90.0	50	W14X68
Story 13	255.0	5.0	30.1	4	0.96 Eq (H1-1a)	90.0	50	W14X61
Story 12	377.9	5.0	29.9	4	0.93 Eq (H1-1a)	90.0	50	W14X82
Story 11	504.1	4.9	38.7	4	0.91 Eq (H1-1a)	90.0	50	W14X90
Story 10	630.4	5.0	38.9	4	0.99 Eq (H1-1a)	90.0	50	W14X99
Story 9	756.9	5.0	39.1	4	0.95 Eq (H1-1a)	90.0	50	W14X120
Story 8	883.7	5.1	39.1	4	0.98 Eq (H1-1a)	90.0	50	W14X132
Story 7	1010.5	5.1	40.6	4	0.99 Eq (H1-1a)	90.0	50	W14X145
Story 6	1137.8	5.2	41.0	4	0.90 Eq (H1-1a)	90.0	50	W14X176
Story 5	1265.0	5.2	41.0	4	0.99 Eq (H1-1a)	90.0	50	W14X176
Story 4	1392.5	5.3	41.0	4	0.99 Eq (H1-1a)	90.0	50	W14X193
Story 3	1520.2	5.4	41.2	4	0.98 Eq (H1-1a)	90.0	50	W14X211
Story 2	1648.1	6.6	50.4	5	0.97 Eq (H1-1a)	90.0	50	W14X233
Base	1778.6	4.4	30.7	1	0.93 Eq (H1-1a)	90.0	50	W14X283

Column Line KK-R

Level	P	Mx	My	LC	Interaction Eq.	Angle	Fy	Size
Story 14	149.6	10.8	73.9	10	0.98 Eq (H1-1a)	90.0	50	W14X74
Story 13	295.6	4.9	33.8	4	0.98 Eq (H1-1a)	90.0	50	W14X68
Story 12	442.9	4.9	43.8	4	0.85 Eq (H1-1a)	90.0	50	W14X90
Story 11	590.9	5.0	44.0	4	0.96 Eq (H1-1a)	90.0	50	W14X99
Story 10	739.0	5.0	44.3	4	0.95 Eq (H1-1a)	90.0	50	W14X120
Story 9	887.5	5.1	46.1	4	0.90 Eq (H1-1a)	90.0	50	W14X145
Story 8	1036.2	5.2	46.3	4	0.94 Eq (H1-1a)	90.0	50	W14X159
Story 7	1185.0	5.2	46.5	4	0.95 Eq (H1-1a)	90.0	50	W14X176
Story 6	1334.1	5.3	46.5	4	0.96 Eq (H1-1a)	90.0	50	W14X193
Story 5	1483.4	5.4	46.7	4	0.97 Eq (H1-1a)	90.0	50	W14X211
Story 4	1632.9	5.4	47.0	4	0.95 Eq (H1-1a)	90.0	50	W14X233
Story 3	1782.8	5.5	47.2	4	0.94 Eq (H1-1a)	90.0	50	W14X257
Story 2	1933.0	6.8	57.6	4	0.93 Eq (H1-1a)	90.0	50	W14X283
Base	2085.6	4.5	34.3	1	0.99 Eq (H1-1a)	90.0	50	W14X311

Column Line LL-O

Level	P	Mx	My	LC	Interaction Eq.	Angle	Fy	Size
Story 14	53.9	8.2	15.2	6	0.58 Eq (H1-1a)	90.0	50	W14X43
Story 13	118.8	3.8	6.9	2	0.65 Eq (H1-1a)	90.0	50	W14X43
Story 12	171.9	3.6	6.6	2	0.86 Eq (H1-1a)	90.0	50	W14X43
Story 11	224.2	3.6	6.4	2	0.95 Eq (H1-1a)	90.0	50	W14X48
Story 10	276.2	3.5	7.2	2	0.76 Eq (H1-1a)	90.0	50	W14X61
Story 9	330.2	3.5	7.2	2	0.89 Eq (H1-1a)	90.0	50	W14X61
Story 8	385.4	3.5	7.2	2	0.92 Eq (H1-1a)	90.0	50	W14X68
Story 7	440.7	3.6	7.2	2	0.95 Eq (H1-1a)	90.0	50	W14X74
Story 6	496.2	3.6	7.2	2	0.96 Eq (H1-1a)	90.0	50	W14X82
Story 5	551.7	3.5	9.3	2	0.83 Eq (H1-1a)	90.0	50	W14X90
Story 4	607.2	3.5	9.3	2	0.91 Eq (H1-1a)	90.0	50	W14X90
Story 3	662.7	3.5	9.3	2	0.99 Eq (H1-1a)	90.0	50	W14X90
Story 2	718.4	4.4	11.4	2	0.98 Eq (H1-1a)	90.0	50	W14X99
Base	775.1	2.9	5.4	1	0.99 Eq (H1-1a)	90.0	50	W14X120

Column Line LL-R

Level	P	Mx	My	LC	Interaction Eq.	Angle	Fy	Size
Story 14	64.6	7.8	8.7	6	0.48 Eq (H1-1a)	90.0	50	W14X43
Story 13	140.9	5.7	3.9	2	0.69 Eq (H1-1a)	90.0	50	W14X43
Story 12	195.4	5.5	6.2	7	0.96 Eq (H1-1a)	90.0	50	W14X43
Story 11	258.5	5.4	6.3	7	0.97 Eq (H1-1a)	90.0	50	W14X53
Story 10	331.7	5.3	4.0	2	0.87 Eq (H1-1a)	90.0	50	W14X61
Story 9	398.3	5.4	4.0	2	0.92 Eq (H1-1a)	90.0	50	W14X68
Story 8	465.0	5.4	4.1	2	0.97 Eq (H1-1a)	90.0	50	W14X74
Story 7	531.9	5.4	5.2	2	0.79 Eq (H1-1a)	90.0	50	W14X90
Story 6	598.7	5.4	5.2	2	0.89 Eq (H1-1a)	90.0	50	W14X90

Story 5	665.6	5.4	5.2	2	0.98 Eq (H1-1a)	90.0	50	W14X90
Story 4	732.6	5.4	5.3	2	0.98 Eq (H1-1a)	90.0	50	W14X99
Story 3	799.6	5.5	5.3	2	0.96 Eq (H1-1a)	90.0	50	W14X109
Story 2	866.9	6.7	6.4	2	0.95 Eq (H1-1a)	90.0	50	W14X120
Base	935.4	2.8	1.4	1	0.94 Eq (H1-1a)	90.0	50	W14X145

Column Line 6-P

Level	P	Mx	My	LC	Interaction Eq.	Angle	Fy	Size
Story 14	62.8	8.3	28.0	1	0.88 Eq (H1-1a)	90.0	50	W14X43
Story 13	116.3	3.8	12.9	1	0.76 Eq (H1-1a)	90.0	50	W14X43
Story 12	168.2	3.7	12.5	1	0.97 Eq (H1-1a)	90.0	50	W14X43
Story 11	219.5	3.6	12.2	1	0.93 Eq (H1-1a)	90.0	50	W14X53
Story 10	270.4	3.6	13.8	1	0.82 Eq (H1-1a)	90.0	50	W14X61
Story 9	322.6	3.6	13.8	1	0.95 Eq (H1-1a)	90.0	50	W14X61
Story 8	376.5	3.6	13.8	1	0.96 Eq (H1-1a)	90.0	50	W14X68
Story 7	430.6	3.6	13.8	1	0.99 Eq (H1-1a)	90.0	50	W14X74
Story 6	484.7	3.6	13.8	1	0.99 Eq (H1-1a)	90.0	50	W14X82
Story 5	539.0	3.6	17.9	1	0.86 Eq (H1-1a)	90.0	50	W14X90
Story 4	593.2	3.6	17.9	1	0.94 Eq (H1-1a)	90.0	50	W14X90
Story 3	647.6	3.6	18.0	1	0.92 Eq (H1-1a)	90.0	50	W14X99
Story 2	702.1	4.4	21.9	1	0.91 Eq (H1-1a)	90.0	50	W14X109
Base	757.5	2.9	15.3	1	1.00 Eq (H1-1a)	90.0	50	W14X120

Column Line 6-T

Level	P	Mx	My	LC	Interaction Eq.	Angle	Fy	Size
Story 14	62.1	25.3	13.2	10	0.66 Eq (H1-1a)	90.0	50	W14X43
Story 13	120.9	11.7	6.1	4	0.68 Eq (H1-1a)	90.0	50	W14X43
Story 12	175.0	11.3	5.9	4	0.90 Eq (H1-1a)	90.0	50	W14X43
Story 11	228.4	11.1	5.7	4	0.99 Eq (H1-1a)	90.0	50	W14X48
Story 10	281.4	11.0	6.5	4	0.79 Eq (H1-1a)	90.0	50	W14X61
Story 9	336.6	11.0	6.5	4	0.93 Eq (H1-1a)	90.0	50	W14X61
Story 8	392.9	11.0	6.5	4	0.95 Eq (H1-1a)	90.0	50	W14X68
Story 7	449.3	11.1	6.5	4	0.98 Eq (H1-1a)	90.0	50	W14X74
Story 6	505.7	11.2	6.5	4	0.99 Eq (H1-1a)	90.0	50	W14X82
Story 5	562.3	11.0	8.4	4	0.86 Eq (H1-1a)	90.0	50	W14X90
Story 4	618.9	11.0	8.4	4	0.94 Eq (H1-1a)	90.0	50	W14X90
Story 3	675.6	11.1	8.4	4	0.93 Eq (H1-1a)	90.0	50	W14X99
Story 2	732.4	13.6	10.3	4	0.92 Eq (H1-1a)	90.0	50	W14X109
Base	790.4	9.0	6.1	1	0.92 Eq (H1-1a)	90.0	50	W14X132

Column Line 7-E

Level	P	Mx	My	LC	Interaction Eq.	Angle	Fy	Size
Story 14	147.2	9.7	16.4	3	0.99 Eq (H1-1a)	0.0	50	W14X43
Story 13	280.3	8.7	8.9	3	0.81 Eq (H1-1a)	0.0	50	W14X61
Story 12	414.2	8.5	8.8	3	0.92 Eq (H1-1a)	0.0	50	W14X74

Level	P	Mx	My	LC	Interaction Eq.	Angle	Fy	Size
Story 11	552.6	8.4	11.5	3	0.86 Eq (H1-1a)	0.0	50	W14X90
Story 10	691.1	8.5	11.4	3	0.96 Eq (H1-1a)	0.0	50	W14X99
Story 9	829.9	8.6	11.4	3	0.93 Eq (H1-1a)	0.0	50	W14X120
Story 8	968.9	8.7	11.4	3	0.98 Eq (H1-1a)	0.0	50	W14X132
Story 7	1108.2	8.9	12.0	3	0.91 Eq (H1-1a)	0.0	50	W14X159
Story 6	1247.6	9.0	12.0	3	0.92 Eq (H1-1a)	0.0	50	W14X176
Story 5	1387.3	9.1	12.0	3	0.93 Eq (H1-1a)	0.0	50	W14X193
Story 4	1527.2	9.2	12.1	3	0.93 Eq (H1-1a)	0.0	50	W14X211
Story 3	1667.4	9.3	12.1	3	0.92 Eq (H1-1a)	0.0	50	W14X233
Story 2	1807.9	11.5	14.8	4	0.91 Eq (H1-1a)	0.0	50	W14X257
Base	1950.6	0.2	9.6	1	0.99 Eq (H1-1a)	0.0	50	W14X283

Column Line 7-W

Level	P	Mx	My	LC	Interaction Eq.	Angle	Fy	Size
Story 14	139.0	12.0	15.4	3	0.95 Eq (H1-1a)	0.0	50	W14X43
Story 13	263.0	11.0	8.3	3	0.77 Eq (H1-1a)	0.0	50	W14X61
Story 12	388.7	10.7	8.2	3	0.95 Eq (H1-1a)	0.0	50	W14X68
Story 11	518.6	10.7	10.6	3	0.81 Eq (H1-1a)	0.0	50	W14X90
Story 10	648.6	10.7	10.6	3	0.99 Eq (H1-1a)	0.0	50	W14X90
Story 9	778.8	10.9	10.7	3	0.97 Eq (H1-1a)	0.0	50	W14X109
Story 8	909.3	11.1	10.7	3	0.93 Eq (H1-1a)	0.0	50	W14X132
Story 7	1039.9	11.1	11.2	3	0.94 Eq (H1-1a)	0.0	50	W14X145
Story 6	1170.7	11.2	11.2	3	0.97 Eq (H1-1a)	0.0	50	W14X159
Story 5	1301.7	11.4	11.3	3	0.96 Eq (H1-1a)	0.0	50	W14X176
Story 4	1432.9	11.5	11.3	3	0.96 Eq (H1-1a)	0.0	50	W14X193
Story 3	1564.4	11.6	11.3	3	0.96 Eq (H1-1a)	0.0	50	W14X211
Story 2	1696.1	14.4	13.9	3	0.94 Eq (H1-1a)	0.0	50	W14X233
Base	1830.3	2.2	9.0	1	0.93 Eq (H1-1a)	0.0	50	W14X283

Column Line 8-P

Level	P	Mx	My	LC	Interaction Eq.	Angle	Fy	Size
Story 14	54.6	16.3	17.9	1	0.68 Eq (H1-1a)	90.0	50	W14X43
Story 13	100.6	7.5	8.2	1	0.62 Eq (H1-1a)	90.0	50	W14X43
Story 12	145.1	7.2	7.9	1	0.80 Eq (H1-1a)	90.0	50	W14X43
Story 11	188.8	7.0	7.7	1	0.97 Eq (H1-1a)	90.0	50	W14X43
Story 10	232.3	7.0	7.7	1	0.91 Eq (H1-1a)	90.0	50	W14X53
Story 9	275.4	6.9	8.7	1	0.79 Eq (H1-1a)	90.0	50	W14X61
Story 8	320.1	6.8	8.6	1	0.90 Eq (H1-1a)	90.0	50	W14X61
Story 7	366.1	6.9	8.6	1	0.90 Eq (H1-1a)	90.0	50	W14X68
Story 6	412.1	6.9	8.6	1	1.00 Eq (H1-1a)	90.0	50	W14X68
Story 5	458.2	7.0	8.7	1	0.91 Eq (H1-1a)	90.0	50	W14X82
Story 4	504.3	7.0	8.7	1	0.99 Eq (H1-1a)	90.0	50	W14X82
Story 3	550.5	6.9	11.2	1	0.85 Eq (H1-1a)	90.0	50	W14X90
Story 2	596.8	8.4	13.7	1	0.93 Eq (H1-1a)	90.0	50	W14X90
Base	644.0	5.5	9.1	1	0.93 Eq (H1-1a)	90.0	50	W14X109

Column Line 8-T

Level	P	Mx	My	LC	Interaction Eq.	Angle	Fy	Size
Story 14	51.2	16.2	11.6	10	0.53 Eq (H1-1a)	90.0	50	W14X43
Story 13	105.3	7.4	5.3	4	0.58 Eq (H1-1a)	90.0	50	W14X43
Story 12	152.0	7.2	5.0	4	0.77 Eq (H1-1a)	90.0	50	W14X43
Story 11	198.0	7.0	4.9	4	0.95 Eq (H1-1a)	90.0	50	W14X43
Story 10	243.6	6.9	4.8	4	0.91 Eq (H1-1a)	90.0	50	W14X53
Story 9	288.9	6.9	5.4	4	0.79 Eq (H1-1a)	90.0	50	W14X61
Story 8	336.9	6.8	5.4	4	0.90 Eq (H1-1a)	90.0	50	W14X61
Story 7	385.2	6.9	5.4	4	0.91 Eq (H1-1a)	90.0	50	W14X68
Story 6	433.7	7.0	5.5	4	0.93 Eq (H1-1a)	90.0	50	W14X74
Story 5	482.2	7.0	5.5	4	0.93 Eq (H1-1a)	90.0	50	W14X82
Story 4	530.8	6.9	7.1	4	0.80 Eq (H1-1a)	90.0	50	W14X90
Story 3	579.4	6.9	7.1	4	0.87 Eq (H1-1a)	90.0	50	W14X90
Story 2	628.1	8.4	8.6	4	0.95 Eq (H1-1a)	90.0	50	W14X90
Base	677.6	5.5	4.5	1	0.96 Eq (H1-1a)	90.0	50	W14X109

Column Line 9-P

Level	P	Mx	My	LC	Interaction Eq.	Angle	Fy	Size
Story 14	53.3	16.2	17.3	1	0.66 Eq (H1-1a)	90.0	50	W14X43
Story 13	98.0	7.4	7.9	1	0.60 Eq (H1-1a)	90.0	50	W14X43
Story 12	141.3	7.1	7.6	1	0.78 Eq (H1-1a)	90.0	50	W14X43
Story 11	183.8	6.9	7.5	1	0.95 Eq (H1-1a)	90.0	50	W14X43
Story 10	226.0	6.9	7.3	1	0.99 Eq (H1-1a)	90.0	50	W14X48
Story 9	268.0	6.8	8.3	1	0.77 Eq (H1-1a)	90.0	50	W14X61
Story 8	310.9	6.8	8.3	1	0.87 Eq (H1-1a)	90.0	50	W14X61
Story 7	355.5	6.8	8.3	1	0.98 Eq (H1-1a)	90.0	50	W14X61
Story 6	400.1	6.8	8.3	1	0.97 Eq (H1-1a)	90.0	50	W14X68
Story 5	444.9	6.9	8.4	1	0.97 Eq (H1-1a)	90.0	50	W14X74
Story 4	489.7	6.9	8.4	1	0.96 Eq (H1-1a)	90.0	50	W14X82
Story 3	534.6	6.8	10.8	1	0.83 Eq (H1-1a)	90.0	50	W14X90
Story 2	579.5	8.3	13.1	1	0.90 Eq (H1-1a)	90.0	50	W14X90
Base	625.2	5.4	9.3	1	1.00 Eq (H1-1a)	90.0	50	W14X99

Column Line 9-T

Level	P	Mx	My	LC	Interaction Eq.	Angle	Fy	Size
Story 14	52.0	16.2	7.9	1	0.46 Eq (H1-1a)	90.0	50	W14X43
Story 13	95.5	7.4	4.8	4	0.53 Eq (H1-1a)	90.0	50	W14X43
Story 12	137.6	7.1	4.5	4	0.70 Eq (H1-1a)	90.0	50	W14X43
Story 11	179.1	7.0	4.4	4	0.86 Eq (H1-1a)	90.0	50	W14X43
Story 10	220.1	6.9	4.3	4	0.91 Eq (H1-1a)	90.0	50	W14X48
Story 9	260.9	6.8	4.3	4	0.95 Eq (H1-1a)	90.0	50	W14X53
Story 8	302.0	6.8	4.8	4	0.81 Eq (H1-1a)	90.0	50	W14X61
Story 7	345.4	6.8	4.8	4	0.92 Eq (H1-1a)	90.0	50	W14X61
Story 6	388.8	6.8	4.8	4	0.91 Eq (H1-1a)	90.0	50	W14X68

Story 5	432.2	6.9	4.9	4	0.92 Eq (H1-1a)	90.0	50	W14X74
Story 4	475.8	6.9	4.9	4	0.91 Eq (H1-1a)	90.0	50	W14X82
Story 3	519.4	6.9	4.9	4	0.99 Eq (H1-1a)	90.0	50	W14X82
Story 2	563.0	8.3	7.7	4	0.85 Eq (H1-1a)	90.0	50	W14X90
Base	607.5	5.4	4.2	1	0.95 Eq (H1-1a)	90.0	50	W14X99

Column Line 11-P

Level	P	Mx	My	LC	Interaction Eq.	Angle	Fy	Size
Story 14	86.0	13.0	37.4	1	0.94 Eq (H1-1a)	90.0	50	W14X53
Story 13	160.8	6.0	17.5	1	0.93 Eq (H1-1a)	90.0	50	W14X48
Story 12	233.8	5.9	19.5	1	0.80 Eq (H1-1a)	90.0	50	W14X61
Story 11	306.7	5.8	19.1	1	0.97 Eq (H1-1a)	90.0	50	W14X61
Story 10	383.6	5.9	19.3	1	0.95 Eq (H1-1a)	90.0	50	W14X74
Story 9	460.6	5.9	19.3	1	1.00 Eq (H1-1a)	90.0	50	W14X82
Story 8	537.7	5.8	24.9	1	0.90 Eq (H1-1a)	90.0	50	W14X90
Story 7	614.9	5.9	25.0	1	0.91 Eq (H1-1a)	90.0	50	W14X99
Story 6	692.2	5.9	25.0	1	0.92 Eq (H1-1a)	90.0	50	W14X109
Story 5	769.7	5.9	25.1	1	0.91 Eq (H1-1a)	90.0	50	W14X120
Story 4	847.1	5.9	25.1	1	0.99 Eq (H1-1a)	90.0	50	W14X120
Story 3	924.7	6.0	25.1	1	0.98 Eq (H1-1a)	90.0	50	W14X132
Story 2	1002.5	7.3	31.8	1	0.96 Eq (H1-1a)	90.0	50	W14X145
Base	1081.9	4.8	20.7	1	0.93 Eq (H1-1a)	90.0	50	W14X176

Column Line 11-T

Level	P	Mx	My	LC	Interaction Eq.	Angle	Fy	Size
Story 14	80.8	12.3	24.1	6	0.89 Eq (H1-1a)	90.0	50	W14X43
Story 13	175.4	5.7	11.1	2	0.98 Eq (H1-1a)	90.0	50	W14X43
Story 12	255.6	5.6	12.3	2	0.77 Eq (H1-1a)	90.0	50	W14X61
Story 11	337.5	5.6	12.2	2	0.97 Eq (H1-1a)	90.0	50	W14X61
Story 10	422.1	5.7	12.3	2	0.96 Eq (H1-1a)	90.0	50	W14X74
Story 9	506.9	5.6	15.8	2	0.81 Eq (H1-1a)	90.0	50	W14X90
Story 8	591.7	5.6	15.8	2	0.93 Eq (H1-1a)	90.0	50	W14X90
Story 7	676.7	5.7	15.9	2	0.95 Eq (H1-1a)	90.0	50	W14X99
Story 6	761.8	5.7	15.9	2	0.96 Eq (H1-1a)	90.0	50	W14X109
Story 5	846.9	5.7	16.0	2	0.96 Eq (H1-1a)	90.0	50	W14X120
Story 4	932.3	5.8	16.0	2	0.96 Eq (H1-1a)	90.0	50	W14X132
Story 3	1017.8	5.8	16.7	2	0.93 Eq (H1-1a)	90.0	50	W14X145
Story 2	1103.5	7.2	20.4	3	0.93 Eq (H1-1a)	90.0	50	W14X159
Base	1190.6	4.7	9.8	1	1.00 Eq (H1-1a)	90.0	50	W14X176

Column Line 12-G

Level	P	Mx	My	LC	Interaction Eq.	Angle	Fy	Size
Story 14	142.9	10.1	15.0	2	0.95 Eq (H1-1a)	0.0	50	W14X43
Story 13	272.5	9.2	8.1	2	0.78 Eq (H1-1a)	0.0	50	W14X61
Story 12	401.9	8.9	8.0	2	0.98 Eq (H1-1a)	0.0	50	W14X68

Story 11	536.2	8.9	10.5	2	0.83 Eq (H1-1a)	0.0	50	W14X90
Story 10	670.7	8.9	10.4	2	0.93 Eq (H1-1a)	0.0	50	W14X99
Story 9	805.3	9.0	10.4	2	1.00 Eq (H1-1a)	0.0	50	W14X109
Story 8	940.2	9.2	10.4	2	0.95 Eq (H1-1a)	0.0	50	W14X132
Story 7	1075.2	9.2	10.9	2	0.97 Eq (H1-1a)	0.0	50	W14X145
Story 6	1210.4	9.3	10.9	2	0.99 Eq (H1-1a)	0.0	50	W14X159
Story 5	1345.8	9.4	11.0	2	0.99 Eq (H1-1a)	0.0	50	W14X176
Story 4	1481.4	9.6	11.0	2	0.99 Eq (H1-1a)	0.0	50	W14X193
Story 3	1617.3	9.6	11.0	2	0.99 Eq (H1-1a)	0.0	50	W14X211
Story 2	1753.4	11.9	13.5	5	0.97 Eq (H1-1a)	0.0	50	W14X233
Base	1892.0	0.8	8.8	1	0.96 Eq (H1-1a)	0.0	50	W14X283

Column Line 12-W

Level	P	Mx	My	LC	Interaction Eq.	Angle	Fy	Size
Story 14	140.2	11.1	16.6	2	0.98 Eq (H1-1a)	0.0	50	W14X43
Story 13	265.1	10.0	9.0	2	0.78 Eq (H1-1a)	0.0	50	W14X61
Story 12	392.0	9.7	8.8	2	0.97 Eq (H1-1a)	0.0	50	W14X68
Story 11	523.1	9.7	11.5	2	0.82 Eq (H1-1a)	0.0	50	W14X90
Story 10	654.3	9.9	11.5	2	0.91 Eq (H1-1a)	0.0	50	W14X99
Story 9	785.6	9.9	11.5	2	0.98 Eq (H1-1a)	0.0	50	W14X109
Story 8	917.1	10.1	11.6	2	0.93 Eq (H1-1a)	0.0	50	W14X132
Story 7	1048.9	10.2	12.1	2	0.95 Eq (H1-1a)	0.0	50	W14X145
Story 6	1180.8	10.3	12.1	2	0.97 Eq (H1-1a)	0.0	50	W14X159
Story 5	1312.9	10.4	12.2	2	0.97 Eq (H1-1a)	0.0	50	W14X176
Story 4	1445.2	10.5	12.2	2	0.97 Eq (H1-1a)	0.0	50	W14X193
Story 3	1577.8	10.6	12.2	2	0.97 Eq (H1-1a)	0.0	50	W14X211
Story 2	1710.6	13.1	15.0	2	0.95 Eq (H1-1a)	0.0	50	W14X233
Base	1846.0	1.2	9.7	1	0.94 Eq (H1-1a)	0.0	50	W14X283

Column Line 13-Q

Level	P	Mx	My	LC	Interaction Eq.	Angle	Fy	Size
Story 14	72.9	7.7	25.7	10	0.87 Eq (H1-1a)	90.0	50	W14X43
Story 13	154.8	3.6	11.9	4	0.90 Eq (H1-1a)	90.0	50	W14X43
Story 12	225.0	3.5	11.5	4	0.94 Eq (H1-1a)	90.0	50	W14X53
Story 11	294.4	3.4	12.9	4	0.87 Eq (H1-1a)	90.0	50	W14X61
Story 10	368.2	3.5	12.9	4	0.93 Eq (H1-1a)	90.0	50	W14X68
Story 9	442.0	3.5	13.0	4	1.00 Eq (H1-1a)	90.0	50	W14X74
Story 8	516.1	3.5	16.7	4	0.82 Eq (H1-1a)	90.0	50	W14X90
Story 7	590.2	3.5	16.7	4	0.93 Eq (H1-1a)	90.0	50	W14X90
Story 6	664.4	3.5	16.8	4	0.93 Eq (H1-1a)	90.0	50	W14X99
Story 5	738.7	3.5	16.8	4	0.93 Eq (H1-1a)	90.0	50	W14X109
Story 4	813.2	3.6	16.9	4	0.93 Eq (H1-1a)	90.0	50	W14X120
Story 3	887.8	3.6	16.9	4	0.91 Eq (H1-1a)	90.0	50	W14X132
Story 2	962.3	4.4	20.6	5	1.00 Eq (H1-1a)	90.0	50	W14X132
Base	1038.3	2.9	11.2	1	0.97 Eq (H1-1a)	90.0	50	W14X159

Column Line 13-S

Level	P	Mx	My	LC	Interaction Eq.	Angle	Fy	Size
Story 14	84.6	8.4	39.8	1	0.96 Eq (H1-1a)	90.0	50	W14X53
Story 13	158.1	3.9	18.6	1	0.93 Eq (H1-1a)	90.0	50	W14X48
Story 12	229.9	3.8	20.7	1	0.80 Eq (H1-1a)	90.0	50	W14X61
Story 11	301.0	3.7	20.3	1	0.97 Eq (H1-1a)	90.0	50	W14X61
Story 10	376.5	3.8	20.5	1	0.94 Eq (H1-1a)	90.0	50	W14X74
Story 9	452.1	3.8	20.5	1	0.99 Eq (H1-1a)	90.0	50	W14X82
Story 8	527.7	3.8	26.4	1	0.88 Eq (H1-1a)	90.0	50	W14X90
Story 7	603.4	3.8	26.4	1	0.99 Eq (H1-1a)	90.0	50	W14X90
Story 6	679.2	3.8	26.5	1	1.00 Eq (H1-1a)	90.0	50	W14X99
Story 5	755.1	3.8	26.5	1	0.99 Eq (H1-1a)	90.0	50	W14X109
Story 4	831.1	3.8	26.7	1	0.98 Eq (H1-1a)	90.0	50	W14X120
Story 3	907.3	3.9	26.7	1	0.96 Eq (H1-1a)	90.0	50	W14X132
Story 2	983.7	4.8	33.8	1	0.95 Eq (H1-1a)	90.0	50	W14X145
Base	1061.6	3.1	22.0	1	0.92 Eq (H1-1a)	90.0	50	W14X176

Column Line 14-Q

Level	P	Mx	My	LC	Interaction Eq.	Angle	Fy	Size
Story 14	82.5	54.8	6.3	1	0.75 Eq (H1-1a)	90.0	50	W14X43
Story 13	154.3	25.6	2.9	1	0.83 Eq (H1-1a)	90.0	50	W14X43
Story 12	224.5	25.0	2.8	1	0.99 Eq (H1-1a)	90.0	50	W14X48
Story 11	293.9	24.7	3.2	1	0.84 Eq (H1-1a)	90.0	50	W14X61
Story 10	366.0	24.7	3.2	1	0.90 Eq (H1-1a)	90.0	50	W14X68
Story 9	439.5	25.0	3.2	1	0.97 Eq (H1-1a)	90.0	50	W14X74
Story 8	513.2	24.7	4.1	1	0.80 Eq (H1-1a)	90.0	50	W14X90
Story 7	586.8	24.7	4.1	1	0.91 Eq (H1-1a)	90.0	50	W14X90
Story 6	660.6	25.0	4.2	1	0.92 Eq (H1-1a)	90.0	50	W14X99
Story 5	734.5	25.1	4.2	1	0.92 Eq (H1-1a)	90.0	50	W14X109
Story 4	808.6	25.3	4.2	1	0.91 Eq (H1-1a)	90.0	50	W14X120
Story 3	882.6	25.3	4.2	1	0.99 Eq (H1-1a)	90.0	50	W14X120
Story 2	956.8	31.1	5.1	1	0.98 Eq (H1-1a)	90.0	50	W14X132
Base	1032.3	20.4	3.4	1	0.97 Eq (H1-1a)	90.0	50	W14X159

Column Line 14-S

Level	P	Mx	My	LC	Interaction Eq.	Angle	Fy	Size
Story 14	188.6	44.8	64.5	12	0.97 Eq (H1-1a)	90.0	50	W14X82
Story 13	382.2	20.3	29.9	3	0.98 Eq (H1-1a)	90.0	50	W14X82
Story 12	573.5	20.2	38.8	3	0.95 Eq (H1-1a)	90.0	50	W14X99
Story 11	765.0	20.5	39.0	3	0.98 Eq (H1-1a)	90.0	50	W14X120
Story 10	956.9	20.9	40.6	3	0.96 Eq (H1-1a)	90.0	50	W14X145
Story 9	1149.1	21.3	41.0	3	0.93 Eq (H1-1a)	90.0	50	W14X176
Story 8	1341.6	21.6	41.0	3	0.97 Eq (H1-1a)	90.0	50	W14X193
Story 7	1534.5	22.1	41.4	3	0.91 Eq (H1-1a)	90.0	50	W14X233
Story 6	1727.8	22.5	41.6	3	0.91 Eq (H1-1a)	90.0	50	W14X257

Story 5	1921.3	22.9	41.8	3	0.91 Eq (H1-1a)	90.0	50	W14X283
Story 4	2114.9	22.9	41.8	3	1.00 Eq (H1-1a)	90.0	50	W14X283
Story 3	2308.8	23.3	42.0	3	0.99 Eq (H1-1a)	90.0	50	W14X311
Story 2	2503.1	28.8	51.5	3	0.97 Eq (H1-1a)	90.0	50	W14X342
Base	2700.8	13.5	33.5	1	0.98 Eq (H1-1a)	90.0	50	W14X398

Column Line 15-I

Level	P	Mx	My	LC	Interaction Eq.	Angle	Fy	Size
Story 14	181.8	19.3	19.4	2	0.99 Eq (H1-1a)	0.0	50	W14X53
Story 13	341.9	17.5	10.3	2	0.90 Eq (H1-1a)	0.0	50	W14X68
Story 12	511.2	17.4	13.4	2	0.83 Eq (H1-1a)	0.0	50	W14X90
Story 11	681.9	17.5	13.4	2	0.97 Eq (H1-1a)	0.0	50	W14X99
Story 10	852.9	17.8	13.5	2	0.98 Eq (H1-1a)	0.0	50	W14X120
Story 9	1024.2	18.1	14.1	2	0.95 Eq (H1-1a)	0.0	50	W14X145
Story 8	1195.9	18.5	14.2	2	0.90 Eq (H1-1a)	0.0	50	W14X176
Story 7	1367.8	18.7	14.2	2	0.93 Eq (H1-1a)	0.0	50	W14X193
Story 6	1539.9	18.9	14.3	2	0.96 Eq (H1-1a)	0.0	50	W14X211
Story 5	1712.3	19.2	14.3	2	0.96 Eq (H1-1a)	0.0	50	W14X233
Story 4	1885.0	19.6	14.4	2	0.95 Eq (H1-1a)	0.0	50	W14X257
Story 3	2058.0	19.8	14.5	2	0.94 Eq (H1-1a)	0.0	50	W14X283
Story 2	2231.3	24.6	17.7	5	0.93 Eq (H1-1a)	0.0	50	W14X311
Base	2407.9	8.1	11.6	1	0.92 Eq (H1-1a)	0.0	50	W14X370

Column Line 15-W

Level	P	Mx	My	LC	Interaction Eq.	Angle	Fy	Size
Story 14	146.1	10.4	17.6	3	0.90 Eq (H1-1a)	0.0	50	W14X48
Story 13	275.5	9.2	9.5	3	0.80 Eq (H1-1a)	0.0	50	W14X61
Story 12	408.4	9.1	9.4	3	0.92 Eq (H1-1a)	0.0	50	W14X74
Story 11	544.8	9.0	12.1	3	0.85 Eq (H1-1a)	0.0	50	W14X90
Story 10	681.4	9.1	12.2	3	0.95 Eq (H1-1a)	0.0	50	W14X99
Story 9	818.3	9.3	12.2	3	0.92 Eq (H1-1a)	0.0	50	W14X120
Story 8	955.3	9.4	12.2	3	0.97 Eq (H1-1a)	0.0	50	W14X132
Story 7	1092.4	9.4	12.7	3	0.99 Eq (H1-1a)	0.0	50	W14X145
Story 6	1230.0	9.6	12.9	3	0.91 Eq (H1-1a)	0.0	50	W14X176
Story 5	1367.7	9.7	12.9	3	0.92 Eq (H1-1a)	0.0	50	W14X193
Story 4	1505.7	9.8	12.9	3	0.92 Eq (H1-1a)	0.0	50	W14X211
Story 3	1643.9	10.0	13.0	3	0.91 Eq (H1-1a)	0.0	50	W14X233
Story 2	1782.1	12.1	15.8	3	0.99 Eq (H1-1a)	0.0	50	W14X233
Base	1922.9	0.0	10.3	1	0.97 Eq (H1-1a)	0.0	50	W14X283

Column Line 17-R

Level	P	Mx	My	LC	Interaction Eq.	Angle	Fy	Size
Story 14	193.6	98.9	40.1	16	0.92 Eq (H1-1a)	0.0	50	W14X82
Story 13	361.5	45.7	18.6	4	0.92 Eq (H1-1a)	0.0	50	W14X82
Story 12	542.4	45.0	24.0	4	0.99 Eq (H1-1a)	0.0	50	W14X90

Story 11	723.6	46.2	24.5	4	0.93 Eq (H1-1a)	0.0	50	W14X120
Story 10	905.1	46.9	25.3	4	0.91 Eq (H1-1a)	0.0	50	W14X145
Story 9	1086.8	47.4	25.4	4	0.98 Eq (H1-1a)	0.0	50	W14X159
Story 8	1269.0	48.6	25.5	4	0.92 Eq (H1-1a)	0.0	50	W14X193
Story 7	1451.3	49.1	25.7	4	0.95 Eq (H1-1a)	0.0	50	W14X211
Story 6	1633.9	49.8	25.8	4	0.96 Eq (H1-1a)	0.0	50	W14X233
Story 5	1816.8	50.7	25.9	4	0.96 Eq (H1-1a)	0.0	50	W14X257
Story 4	2000.1	51.4	26.0	4	0.95 Eq (H1-1a)	0.0	50	W14X283
Story 3	2183.6	52.4	26.1	4	0.93 Eq (H1-1a)	0.0	50	W14X311
Story 2	2367.6	64.9	32.1	4	0.92 Eq (H1-1a)	0.0	50	W14X342
Base	2554.9	43.1	20.7	1	0.93 Eq (H1-1a)	0.0	50	W14X398

Column Line 17-U

Level	P	Mx	My	LC	Interaction Eq.	Angle	Fy	Size
Story 14	73.6	0.0	26.3	6	0.85 Eq (H1-1a)	0.0	50	W14X43
Story 13	153.2	0.0	12.1	2	0.88 Eq (H1-1a)	0.0	50	W14X43
Story 12	223.4	0.0	11.7	2	0.92 Eq (H1-1a)	0.0	50	W14X53
Story 11	292.8	0.0	13.1	2	0.85 Eq (H1-1a)	0.0	50	W14X61
Story 10	361.8	0.0	12.9	2	0.91 Eq (H1-1a)	0.0	50	W14X68
Story 9	434.3	0.0	13.0	2	0.97 Eq (H1-1a)	0.0	50	W14X74
Story 8	507.1	0.0	16.7	2	0.80 Eq (H1-1a)	0.0	50	W14X90
Story 7	579.9	0.0	16.7	2	0.90 Eq (H1-1a)	0.0	50	W14X90
Story 6	652.8	0.0	16.8	2	0.91 Eq (H1-1a)	0.0	50	W14X99
Story 5	725.8	0.0	16.8	2	0.91 Eq (H1-1a)	0.0	50	W14X109
Story 4	798.8	0.0	16.8	2	1.00 Eq (H1-1a)	0.0	50	W14X109
Story 3	872.0	0.0	16.9	2	0.98 Eq (H1-1a)	0.0	50	W14X120
Story 2	945.3	0.0	20.6	2	0.97 Eq (H1-1a)	0.0	50	W14X132
Base	1020.0	0.0	11.8	1	0.95 Eq (H1-1a)	0.0	50	W14X159

Column Line 17-W

Level	P	Mx	My	LC	Interaction Eq.	Angle	Fy	Size
Story 14	80.9	46.9	10.9	1	0.80 Eq (H1-1a)	0.0	50	W14X43
Story 13	152.2	22.0	5.2	1	0.85 Eq (H1-1a)	0.0	50	W14X43
Story 12	222.0	21.5	5.1	1	0.90 Eq (H1-1a)	0.0	50	W14X53
Story 11	291.1	21.1	5.8	1	0.84 Eq (H1-1a)	0.0	50	W14X61
Story 10	359.7	21.0	5.7	1	0.90 Eq (H1-1a)	0.0	50	W14X68
Story 9	431.1	21.2	5.7	1	0.96 Eq (H1-1a)	0.0	50	W14X74
Story 8	503.3	20.9	7.4	1	0.80 Eq (H1-1a)	0.0	50	W14X90
Story 7	575.6	20.9	7.4	1	0.90 Eq (H1-1a)	0.0	50	W14X90
Story 6	647.9	21.2	7.5	1	0.91 Eq (H1-1a)	0.0	50	W14X99
Story 5	720.4	21.3	7.5	1	0.91 Eq (H1-1a)	0.0	50	W14X109
Story 4	792.9	21.3	7.5	1	0.99 Eq (H1-1a)	0.0	50	W14X109
Story 3	865.5	21.5	7.5	1	0.98 Eq (H1-1a)	0.0	50	W14X120
Story 2	938.3	26.4	9.1	1	0.97 Eq (H1-1a)	0.0	50	W14X132
Base	1012.5	17.3	6.1	1	0.95 Eq (H1-1a)	0.0	50	W14X159

Column Line 18-K

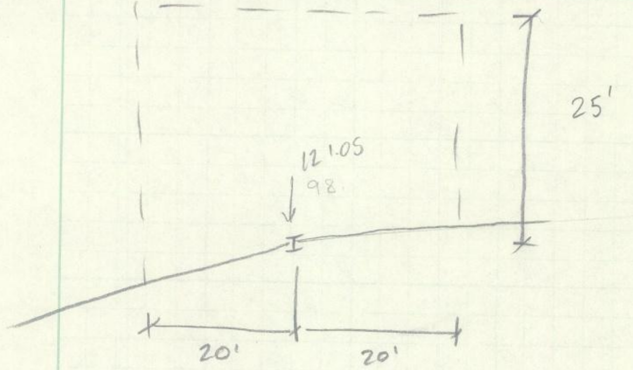
Level	P	Mx	My	LC	Interaction Eq.	Angle	Fy	Size
Story 14	108.1	16.1	53.4	1	0.90 Eq (H1-1a)	90.0	50	W14X61
Story 13	203.7	7.7	25.3	1	0.80 Eq (H1-1a)	90.0	50	W14X61
Story 12	297.5	7.5	24.6	1	0.91 Eq (H1-1a)	90.0	50	W14X68
Story 11	390.6	7.6	24.4	1	0.92 Eq (H1-1a)	90.0	50	W14X82
Story 10	488.0	7.4	31.5	1	0.86 Eq (H1-1a)	90.0	50	W14X90
Story 9	585.6	7.5	31.7	1	0.91 Eq (H1-1a)	90.0	50	W14X99
Story 8	683.4	7.6	31.7	1	0.93 Eq (H1-1a)	90.0	50	W14X109
Story 7	781.3	7.6	31.9	1	0.95 Eq (H1-1a)	90.0	50	W14X120
Story 6	879.3	7.7	31.9	1	0.96 Eq (H1-1a)	90.0	50	W14X132
Story 5	977.5	7.8	33.1	1	0.94 Eq (H1-1a)	90.0	50	W14X145
Story 4	1075.8	7.8	33.3	1	0.94 Eq (H1-1a)	90.0	50	W14X159
Story 3	1174.4	7.9	33.5	1	0.92 Eq (H1-1a)	90.0	50	W14X176
Story 2	1273.2	9.8	40.7	1	0.91 Eq (H1-1a)	90.0	50	W14X193
Base	1373.6	6.4	26.4	1	0.98 Eq (H1-1a)	90.0	50	W14X211

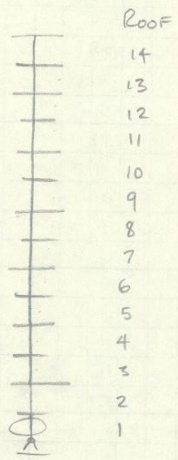
Column Line 18-Q

Level	P	Mx	My	LC	Interaction Eq.	Angle	Fy	Size
Story 14	30.4	18.4	1.6	12	0.21 Eq (H1-1b)	90.0	50	W14X43
Story 13	57.9	8.4	0.8	3	0.30 Eq (H1-1a)	90.0	50	W14X43
Story 12	83.4	8.0	0.7	3	0.40 Eq (H1-1a)	90.0	50	W14X43
Story 11	108.5	7.8	0.7	3	0.50 Eq (H1-1a)	90.0	50	W14X43
Story 10	133.3	7.6	0.7	3	0.60 Eq (H1-1a)	90.0	50	W14X43
Story 9	157.9	7.5	0.7	3	0.70 Eq (H1-1a)	90.0	50	W14X43
Story 8	182.4	7.4	0.7	3	0.80 Eq (H1-1a)	90.0	50	W14X43
Story 7	206.7	7.4	0.7	3	0.90 Eq (H1-1a)	90.0	50	W14X43
Story 6	231.0	7.3	0.7	3	0.89 Eq (H1-1a)	90.0	50	W14X48
Story 5	255.2	7.3	0.7	3	0.98 Eq (H1-1a)	90.0	50	W14X48
Story 4	279.5	7.3	0.7	3	0.96 Eq (H1-1a)	90.0	50	W14X53
Story 3	303.7	7.2	0.8	3	0.77 Eq (H1-1a)	90.0	50	W14X61
Story 2	327.9	8.8	0.9	4	0.84 Eq (H1-1a)	90.0	50	W14X61
Base	352.9	5.5	1.2	1	0.93 Eq (H1-1a)	90.0	50	W14X82

Hand Calculations:

AXIAL COLUMN LOAD CHECK





DEAD LOAD = 45 psf
CURTAIN WALL = 15 psf

LIVE LOAD = 100 psf (REDUCIBLE)

$$L_R = 0.25 + \frac{15}{\sqrt{4(40)(25)(13)}} = 0.315 < 0.4 \therefore \text{USE } 0.4$$

$$L_{LR} = 0.4(100) = 40 \text{ psf}$$

AXIAL LOAD ON COLUMN: (PER FLOOR)

$$(40')(25')(40 \text{ psf}) = 40^k$$

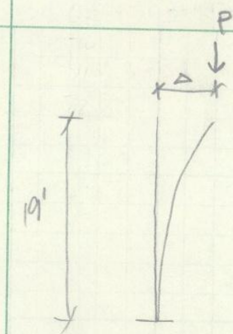
$$(40')(25')(50 \text{ psf}) = 50^k$$

$$(40')(178')(15 \text{ psf}) = 106.8^k \text{ (TOTAL)}$$

$$P_D = 1.2(50^k/\text{floor})(13 \text{ floors}) + 1.2(106.8) = 908^k$$

$$P_L = 1.6(40^k/\text{floor})(13 \text{ floors}) = 832^k$$

$$\text{TOTAL} = 1740^k$$



$\Delta_{max} = 0.12$ in — Story Deflection

$M = P\Delta = 2025^k(.12/12) = 20.25$ ft-k

STRONG AXIS BENDING

COLUMN SIZE = 14 x 342

$P_r = 2025$

$P_c = 3680$

$\frac{P_r}{P_c} = \frac{2025}{3680} = .55 > 0.2 \therefore$ H1-1a

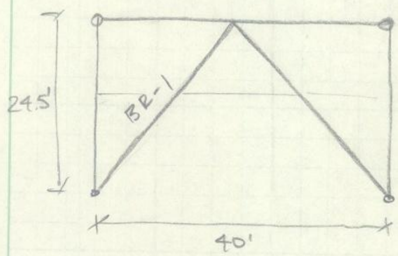
$\frac{P_r}{P_c} + \frac{8}{9}\left(\frac{M_{rx}}{M_{cx}}\right) \leq 1.0$ $\qquad M_{cx} = 1060$

$\frac{2025}{3680} + \frac{8}{9}\left(\frac{20}{1060}\right) = 0.55 + 0.0168 = 0.567 \therefore$ GOOD

P-Δ HAS MINIMAL EFFECT ON BEAM DESIGN

Appendix D:

BRACE DESIGN 1



HSS $F_y = 46$ ksi
$F_u = 58$ ksi

$P_D = 75^k$
$P_L = 75^k$
$E = 361^k$

SEISMIC DESIGN CATEGORY B

$\Omega_o = 2.0$ $\rho = 1.0$ $S_{ds} = .171$

$0.2S_{DS} = 0.2(.171) = 0.0342$

HORIZONTAL FORCE ON BRACE:

$\frac{20}{31.6}(361) = 228^k$ $\frac{228}{361} = 0.63$ ∴ GOOD, $0.3 < 0.63 < 0.7$

MAX COMPRESSIVE FORCE:

$P_u = (1.2 + 0.2S_{DS})P_D + \rho P_{QE} + 0.5P_L$

$= (1.2 + .0342)75 + 1.0(361) + 0.5(75) = 491$

MAX TENSION:

$T_u = (0.9 - 0.2S_{DS})P_D + \rho P_{QE} =$

$= (0.9 - 0.0342)75 + 1.0(-361) = -300$

$L = \sqrt{24.5^2 + 20^2} = 31.6(2/3)$

TRY HSS 14×14×0.5

$A_g = 24.6$ $r = 5.49$
$t = 0.465$ $H = 12.6$

LOCAL BUCKLING

$\lambda = 27.1$

$\lambda_{ps} = 0.044\frac{E}{F_y} = 0.044\left(\frac{29000}{46}\right) = 27.73$

$27.73 > 27.1$ SEISMICALLY COMPACT

2

SLENDERNESS:

$$\frac{KL}{r} \leq 4.0\sqrt{\frac{E}{F_y}} = 115 \qquad K = 1.0$$

$$\frac{KL}{r} = \frac{1.0(31.6)(12)}{5.49} = 70$$

$70 < 115 \therefore$ GOOD

COMPRESSIVE STRENGTH:

$$4.71\sqrt{\frac{E}{F_y}} = 136$$

$$F_{CR} = \left[0.658^{F_y/F_e}\right] F_y = 0.658^{46/58.4}(46) = 33.1$$

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{\pi^2(29000)}{(70^2)} = 58.4$$

$$\phi P_n = 0.9(33.1)(24.6) = 732$$

$732 > 491 \therefore$ OK

TENSION STRENGTH OF BRACE:

$$\phi P_n = 0.9 F_y A_g$$
$$= 0.9(46)(24.6) = 1018$$

$1018 > 300 \therefore$ OK

COLUMN DESIGN 1

$P_D = 1000^k$
$P_L = 1000^k$
$P_{QE} = 361$

MAX COMPRESSIVE FORCE:

$P_u = (1.2 + 0.2S_{DS})P_D + \rho P_{QE} + 0.5P_L$
$= 1.2342(1000) + 361 + 0.5(1000) = 2095^k$

MAX TENSION:

$T_u = (0.9 - 0.2S_{DS})P_D + \rho P_{QE}$
$= 505$ (COMP)

TRY W14x233

$A_g = 68.5$ $d = 16.0$ $b_f = 15.9$
$r_x = 6.79$ $t_w = 1.07$ $t_f = 1.72$
$r_y = 4.17$

CHECK SLENDERNESS:

$$\lambda_f = \frac{b_f}{2t_f} = \frac{15.9}{2(1.72)} = 4.62$$

$$\lambda_{ps} = 0.3\sqrt{\frac{E}{F_y}} = 0.3\sqrt{\frac{29000}{50}} = 7.22$$

$7.22 > 4.62 \therefore$ SEISMICALLY COMPACT

$$\lambda_w = \frac{h}{t_w} = \frac{d-2t_f}{t_w} = \frac{16-2(1.72)}{1.07} = 11.74$$

$$C_a = \frac{P_u}{\phi_b P_y} = \frac{P_u}{0.9F_yA_g} = \frac{2095}{0.9(50)(68.5)} = 0.679 \geq 0.125$$

$$\lambda = 1.12\sqrt{\frac{E}{F_y}}(2.33 - C_a) \geq 1.49\sqrt{\frac{E}{F_y}} = 35.88$$

$$= 1.2\sqrt{\frac{29000}{50}}(2.33 - .679)$$

$47.7 > 35.88$

$\lambda_w < \lambda_{ps} \therefore$ WEB SEISMICALLY COMPACT

2

COMPRESSION STRENGTH

$$\frac{KL}{r} = \frac{1.0(24.5)(12)}{4.17} = 70.5$$

$$4.71\sqrt{\frac{29000}{50}} = 113$$

$113 > 57.6$

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{\pi^2(29000)}{(70.5^2)} = 57.6$$

$$F_{cr} = \left[0.658^{(50/57.6)}\right](50) = 34.8$$

$$\phi P_n = 0.9 F_{cr} A_g = 0.9(34.8)(68.5) = 2145.42 > 2095 \therefore OK$$

TENSILE STRENGTH: OK. NEVER OUT OF COMPRESSION

1

AXIAL LOAD CHECK : EW

BRACE TYPE : HSS14×14×.625

$P_t = 1.4(46)(30.3) = 1951^k$

MAX AXIAL IN BEAM

$P_{tx} = \frac{20}{31.6}(1951) = 1234^k$

$P_{tx} = \frac{1234}{2} = 617$

$\phi F_{cr} = 40$ ksi → TABLE 4-22

$A_g = 30.0$ → W27×102

$\phi P_n = 40(30) = 1200$

$\frac{P_r}{P_c} = \frac{617}{1200} = .514$ ∴ H1-1a

$M_{rx} = 1179.5$

$M_{cx} = 1259$

$\frac{617}{1200} + \frac{8}{9}\left(\frac{1180}{1260}\right) = 0.514 + 0.83 = 1.34$

∴ No Good

TRY W27×129

$A_g = 37.8$

$\phi P_n = (40)(37.8) = 1512$

$\frac{P_r}{P_c} = \frac{617}{1512} = 0.41$

$\frac{617}{1512} + \frac{8}{9}\left(\frac{1180}{1541}\right)$

W27×161

$A_g = 47.6$ → $\phi P_n = 1904$

$\frac{P_r}{P_c} = \frac{617}{1904} = .32$

$.32 + \frac{8}{9}\left(\frac{1180}{1970}\right) = .92$ ∴ Good

CHECK AXIAL LOADS : NS 2

HSS 14×14 × 0.5

$P_t = 1.4(46)(24.6) = 1584$

$P_{tx} = \frac{20}{31.6}(1584) = 1002^k \qquad 1002/2 = 501$

W 27×94

$A_g =$

$\phi P_n = 40(26.9) = 1076$

$\frac{P_r}{P_c} = \frac{501}{1076} = 0.47$

$\frac{P_r}{P_c} + \frac{8}{9}\left(\frac{M_r}{M_c}\right) \leq 1.0$

$.47 + \frac{8}{9}\left(\frac{316}{833}\right) = 0.81$

BEAM DESIGN

ASSUMED TENSION IN BRACE

$P_t = R_y F_y A_g$

$= 1.4(46)(24.6) = 1584.2$

ASSUMED COMPRESSION:

$P_c = 0.3 P_n = 0.3 F_{cr} A_g$

$= 0.3(33.1)(24.6) = 244^k$

UNBALANCED LOAD:

$P_{ty} = \frac{24.5}{31.6}(1584) = 1227$

$P_{cy} = \frac{24.5}{31.6}(244) = 189$

$Q_b = P_{ty} - P_{cy}$

$= 1227 - 189 = 1038$

AXIAL FORCE IN BEAM:

$P_{tx} = \frac{20}{31.6}(1584) = 1002^k$

$P_{cx} = \frac{20}{31.6}(244) = 154.4^k$

$P_u = \frac{P_{tx} + P_{cx}}{2} = \frac{1002 + 154.4}{2} = 578^k$

MOMENTS IN BEAM

ASSUME S.W. BEAM = 100 lb/ft
CW WEIGHT = 12.5 × 15 = 187.5 lb/ft
LIVE LOAD = 110 lb/ft

$M_D = \frac{(.186 + .100)(40^2)}{8} + \frac{16(25)}{4} = 122.3 \text{ ft-k}$

$M_L = \frac{.110(25^2)}{8} + \frac{14(25)}{4} = 96.1 \text{ ft-k}$

$M_{Qb} = \frac{361(25)}{4} = 2256 \text{ ft-k}$

$M_u = 1.2 M_D + 0.5 M_L + 1.0 M_Q$

$= 1.2(122.3) + .5(96.1) + 1.0(2256) = 2451$

2

TRY W27 x 307 : $A_g = 90.4$, $d = 30.0$, $t_w = 1.26$, $b_f = 14.6$, $t_f = 2.28$, $r_x = 12.0$, $r_y = 3.41$, $I_x = 13100$, $Z_x = 1030$, $S_x = 887$

ELEMENT SLENDERNESS

$$\lambda_f = \frac{b_f}{2t_f} = \frac{14.6}{2(2.28)} = 0.35$$

$$\lambda_p = 0.38\sqrt{\frac{E}{F_y}} = 9.15$$ $\lambda_f < \lambda_p$ ∴ Flange Compact

$$\lambda_w = \frac{h}{t_w} = \frac{30-2(2.28)}{1.26} = 20.2$$

$$\lambda_p = 3.76\sqrt{\frac{E}{F_y}} = 90.6$$ $90.6 > 20.2$ ∴ Compact Web

UNBRACED LENGTH

$L_p = 12.0$

COMPOSITE BEAM → $L_b = 0$ ∴ OK

FLEXURAL STRENGTH :

$M_n = M_p = F_y Z_x$

$M_p = 50(1030)(1/12) = 4292$ k-ft

$\phi M_n = 0.9(4292) = 3862$

COMPRESSIVE STRENGTH :

$$\frac{K_xL_x}{r_x} = \frac{1.0(40)(12)}{12.0} = 40 \qquad \frac{KL_y}{r_y} = 0$$

$\phi F_{cr} = 40$ ksi

$\phi P_n = 40(90.4) = 3620^k$

3

SECOND ORDER EFFECTS

$$B_1 = \frac{C_m}{1-\left[\alpha P_r / P_{e1}\right]} \geq 1$$

$$P_{e1} = \frac{\pi^2 EI}{(KL)^2} = \frac{\pi^2 (29000)(13100)}{[40(12)]^2} = 16{,}300^k$$

$$B_1 = \frac{1.0}{1-\left[578/16300\right]} = 1.04$$

$B_2 = 1.0$

$$P_r = P_{nt} + B_2 P_{lt}$$
$$= 0 + 1.0(578) = 578$$

$$M_{rx} = B_1 M_{nt} + B_2 M_{lt}$$
$$= 1.04(2451) = 2549$$

COMBINED LOADING CHECK:

$$\frac{P_r}{P_c} = \frac{578}{3620} = 0.16 < 2.0 \quad \therefore H1\text{-}1b$$

$$\frac{P_r}{2P_c} + \frac{M_{rx}}{M_{cx}} < 1.0 \Rightarrow 0.080 + .66 = 0.74 \quad \therefore \text{GOOD}$$

DETERMINE SHEAR IN BEAM

$$V_D = \frac{(.186 + .307)(40)}{2} + \frac{16}{2} = 17.86$$

$$V_L = \frac{.11(40)}{2} + \frac{15}{2} = 9.7$$

$$V_{PQ} = \frac{1038}{2} = 519$$

$$V_u = 1.2V_D + 0.5V_L + 1.0V_{Q_e} = 1.2(17.86) + 0.5(9.7) + 1.0(519)$$

$$V_u = 545$$

4

SHEAR STRENGTH OF BEAM

$$2.24\sqrt{\frac{E}{F_y}} = 2.24\sqrt{\frac{29000}{50}} = 53.9$$

$$\frac{h}{t_w} = 20.2 < 53.9$$

$$\therefore V_n = 0.6 F_y A_w C_v$$

$$= 1.0(0.6)(50)(25.44 \times 1.26)(1.0) = 962$$

$962 > 545 \therefore OK$

TORSIONAL FORCES

CENTER OF RIGIDITY:

$$\bar{x} = \frac{\Sigma K_{iy} X_i}{\Sigma K_{iy}} \qquad \bar{y} = \frac{\Sigma K_{ix} y_i}{\Sigma K_{ix}}$$

$\bar{x}$:

$K_1 X_1 = 36.7(0) = 0$

$K_2 X_2 = 35.7(6.22') = 222$

$K_3 X_3 = 33.5(203.8) = 7235$

$K_4 X_4 = 30.3(212.4) = 6436$

$\Sigma K_{iy} = 136.2 \qquad \Sigma K_{iy} X_i = 13893$

$$\bar{x} = \frac{13893}{136.2} = 102.0 \text{ ft}$$

$\bar{y}$:

$K_1 y_1 = 48.3(138.7) = 6699$

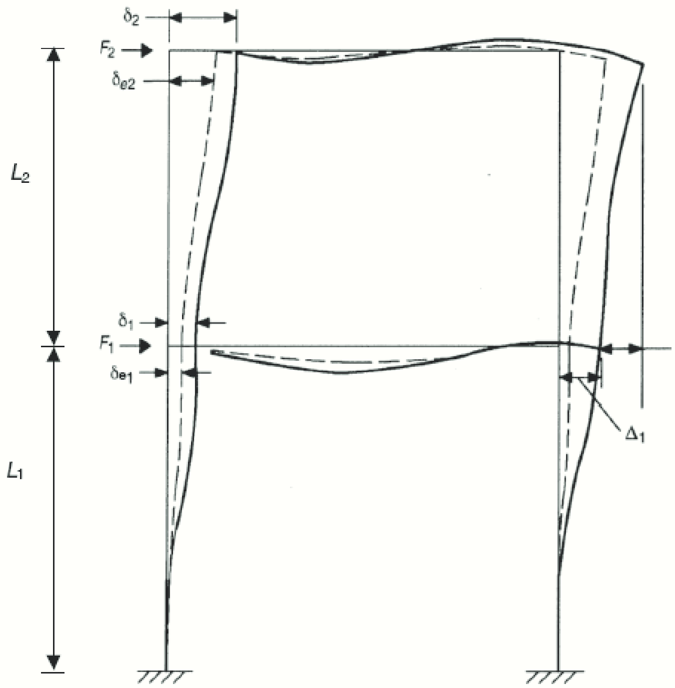
$K_2 y_2 = 37.1(138.7) = 5146$

$K_3 y_3 = 41.3(0) = 0$

$K_4 y_4 = 53.9(14.7) = 792.3$

$\Sigma K_{ix} = 180.6 \qquad \Sigma K_{ix} y_i = 12637$

$$\bar{y} = \frac{12637}{180.6} = 70.0'$$



Story Level 2

F_2 = strength-level design earthquake force
δ_{e2} = elastic displacement computed under strength-level design earthquake forces
δ_2 = $C_d\, \delta_{e2}/I_E$ = amplified displacement
Δ_2 = $(\delta_{e2} - \delta_{e1})\, C_d / I_E \leq \Delta_a$ (Table 12.12-1)

Story Level 1

F_1 = strength-level design earthquake force
δ_{e1} = elastic displacement computed under strength-level design earthquake forces
δ_1 = $C_d\, \delta_{e1}/I_E$ = amplified displacement
Δ_1 = $\delta_1 \leq \Delta_a$ (Table 12.12-1)

Δ_i = Story Drift
Δ_i/L_i = Story Drift Ratio
δ_2 = Total Displacement

FIGURE 12.8-2 Story Drift Determination

Table 12.12-1 Allowable Story Drift, Δ_a [a,b]

Structure	Risk Category		
	I or II	III	IV
Structures, other than masonry shear wall structures, 4 stories or less above the base as defined in Section 11.2, with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts.	$0.025h_{sx}$ [c]	$0.020h_{sx}$	$0.015h_{sx}$
Masonry cantilever shear wall structures [d]	$0.010h_{sx}$	$0.010h_{sx}$	$0.010h_{sx}$
Other masonry shear wall structures	$0.007h_{sx}$	$0.007h_{sx}$	$0.007h_{sx}$
All other structures	$0.020h_{sx}$	$0.015h_{sx}$	$0.010h_{sx}$

Appendix E:

CORNER CONNECTION 11

PLATE BLOCK SHEAR :

TABLE 9.3a = 46.2
9.3b = 121
9.3c = 139 } 121

$\phi V_n = 2(121 + 46.2)(.5) = 167.2$

BOLT LIMIT STATES

SHEAR = 15.9

PLATE BEARING $= 0.75(2.4)(58)(0.75)(.5) = 39.2$

PLATE EDGE $= 0.75(1.2)(58)(1.5 - \frac{.75 + 3/16}{2})(.5) = 27.7$

$\phi V_n = 3(15.9)(2) = 95.4 > 22^k \therefore$ OK

X – CONNECTION 1

DESIGN OF TOP CONNECTION

MAX TENSION = 278^k L.C. 71

MAX COMPRESSION = 210^k L.C. 80

BRACE TYPE: HSS 14x14x.3125, A500 GRADE B

BRACE LIMIT STATES

TENSION YIELDING: $\phi R_n = \phi F_y A_g$

$\phi R_n = 0.9(46)(15.7) = \boxed{650^k}$ > 278 ∴ OK

TENSION RUPTURE: $\phi R_n = \phi F_u A_e$

$A_e = U A_n$

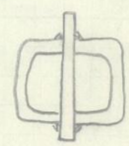
$U = 1 - \frac{\bar{x}}{L} \Rightarrow \bar{x} = \frac{B^2 + 2BH}{4(B+H)}$ TABLE D3.1 $(L \geq H)$

$U = 1 - \frac{5.25}{14}$ $\bar{x} = \frac{14^2 + 2(14)(14)}{4(14+14)} = 5.25$

$U = 0.625$

$A_e = 0.625(15.7) = 9.81\ in^2$

$\phi R_n = 0.75(58)(9.81) = \boxed{427^k}$ > 278 ∴ OK



BRACE/GUSSET LIMIT STATES:

WELD RUPTURE: $\phi R_n = 1.392(D)(L_w)(\#)$

$t_{HSS} = 0.291 \rightarrow$ Min Weld = 3/16 MAX WELD = 3/16

$\phi R_n = 1.392(3)(14)(4) = 233.8$ ↑ WELD LENGTH

$L_w = \frac{278}{1.392(3)(4)} = 16.6 \rightarrow 17''$

$\phi R_n = 1.392(3)(17)(4) = \boxed{284^k}$ > 278 ∴ OK

X - CONNECTION 2

BRACE/GUSSET LIMIT STATE:

BASE METAL: BRACE: $\phi R_n = \phi(0.6) F_u A_{nw}$ #

$A_{nw} = t_{br} \times L_w = 0.291(17) = 4.95 \text{ in}^2$

$\phi R_n = 0.75(0.6)(58)(4.95)(4) = \boxed{517^k}$ > 278 ∴ OK

BASE METAL: GUSSET, ½" A36

$A_{nw} = 0.5(17) = 8.5 \text{ in}^2$

$\phi R_n = 0.75(0.6)(58)(8.5)(2) = \boxed{444^k}$ > 278 ∴ OK

GUSSET LIMIT STATES

TENSION YIELDING: $\phi R_n = \phi F_y A_g$

$\phi R_n = 0.9(36)(.5)(18) = \boxed{292^k}$ > 278 ∴ OK

TENSION RUPTURE: $\phi R_n = \phi F_u A_e$ ∴ WHITORE SECTION NOT NEEDED

$A_e = A_g$

$\phi R_n = 0.75(58)(9) = \boxed{392^k}$ > 278 ∴ OK

LOCAL BUCKLING: $\phi P_n = \phi A_g F_{cr}$

$$\lambda_c = \frac{Kl}{r\pi}\sqrt{\frac{F_y}{E}} = \frac{0.5(22)}{\left(\frac{.5}{\sqrt{12}}\right)\pi}\sqrt{\frac{36}{29000}} = 0.85 \leq 1.5$$

$$F_{cr} = 0.658^{\lambda_c^2} F_y = 0.658^{(.85^2)}(36) = 26.6 \text{ ksi}$$ 210

$\phi P_n = 0.9(0.5)(30)(26.6) = 359.1 > 210$ ∴ OK

(30 ← WHITMORE)

GUSSET/BEAM LIMIT STATES

WELD RUPTURE: $\phi R_n = 1.392(D)(L_w)(\#)$

Min weld = 3/16, $\theta = 39°$

$(1 + .5\sin(\theta)^{1.5}) = 1 + .5(\sin 39)^{1.5} = 1.25$

$\phi R_n = 1.392(1.25)(3)(35.5)(2) = \boxed{370.6^k}$ > 278 ∴ OK

X - CONNECTION 3

GUSSET / BEAM LIMIT STATES:

BASE METAL : GUSSET : $\phi R_n = \phi(0.6) F_u A_{nw}$

$A_{nw} = t_G \times L_w = 0.5(35.5) = 17.75 \text{ in}^2$

$\phi R_n = 0.75(0.6)(58)(17.75) = \boxed{462} > 278 \therefore OK$

BASE METAL : BEAM : $\phi R_n = \phi(0.6) F_u A_{nw}$ #

$A_{nw} = t_f \times L_w = 0.745(35.5) = 26.4$

$\phi R_n = 0.75(.6)(65)(26.4)(2) = \boxed{1544.4} > 278 \therefore OK$

X - CONNECTION 4

DESIGN OF BOTTOM CONNECTION

MAX TENSION = 311^K

MAX COMPRESSION = 196^K

HSS: 14×14×.375

TENSION YIELDING: 650 > 311 ∴ OK

TENSION RUPTURE: 427 > 311 ∴ OK

BRACE/GUSSET LIMIT STATES:

WELD RUPTURE: $\phi R_n = 1.392(D)(L_w)(\#)$

$min_{weld} = 3/16$ $max_{weld} = .349 - 1/16 = 0.287$

$t_w = 4/16 = 0.25$

$\phi R_n = 1.392(4)(17)(4) = 379^K > 311$ ∴ OK

BASE METAL: BRACE 517 > 311 ∴ OK

: GUSSET 887 > 311 ∴ OK

GUSSET LIMIT STATES:

TENSION YIELDING: $\phi R_n = \phi F_y A_g$

- CHECK w/ WHITMORE SECTION ⇒ L = 30.7" (SEE DETAIL)

$\phi R_n = 0.9(36)(0.5)(30.7) = 497.3 > 311$ ∴ OK

TENSION RUPTURE: $\phi R_n = \phi F_u A_e$

$\phi R_n = 0.75(58)(15.35) = 668^K > 311$ ∴ OK

LOCAL BUCKLING: 359 > 196 ∴ OK

GUSSET/BEAM LIMIT STATES:

WELD RUPTURE: 370.6 > 311 ∴ OK

BASE METAL: 927 > 311 ∴ OK

: 1544 > 311 ∴ OK

X - CONNECTION 5

BEAM LIMIT STATES:

BEAM WEB TENSION YIELDING: $\phi R_n = \phi F_y A_g$

$\phi R_n = 0.9(50)(.49)(35.5) = 782 > 311 \therefore OK$

WEB CRIPPLING: $R_n = 0.8 t_{wb}^2 \left[1 + 3\left(\frac{N}{d}\right)\left(\frac{t_{wb}}{t_{fb}}\right)^{1.5}\right]\sqrt{\frac{E F_{yb} t_{fb}}{t_{wb}}}$

$N = t_p = 0.5$

$R_n = 0.8(.49)^2\left[1 + 3\left(\frac{.5}{26.9}\right)\left(\frac{.49}{.745}\right)^{1.5}\right]\sqrt{\frac{29000(50)(.745)}{.49}}$

$R_n = 292^K$

$\phi R_n = 0.75(292) = 219^K > 210^K \therefore OK$

WEB BUCKLING: $R_n = \frac{24 t_{wb}^3 \sqrt{E F_y}}{h}$

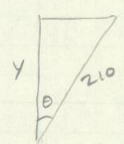
$h = \frac{h}{t_w}(t_w) = 49.5(.49) = 24.26$

$R_n = \frac{24(.49^3)\sqrt{29000(50)}}{24.26} = 140.1$ $\phi R_n = 0.9(140.1)$

$\phi R_n = 126.1$

VERTICAL COMPONENT OF 210^K

$\theta = 39°$



$y = 210 \cos 39 = 163.2^K$

$\therefore$ DOUBLER PLATE REQUIRED

X - CONNECTION 6

DOUBLER PLATE DESIGN

$$t_p^3 = \frac{C_u \cdot h}{\sqrt{EF_y}} \times \frac{1}{24} \qquad C_u = 170^k$$

$$= \frac{170(24.26)}{\sqrt{29000(50)}} \times \frac{1}{24} = 0.1427 = t_p^3$$

$$t_p = \sqrt[3]{.1427} = 0.52 \quad \therefore \text{USE } t_p = 0.75''$$

$$R_n = \frac{24(.75)^3\sqrt{29000(50)}}{24.26} = 502.5^k > 163 \therefore \text{OK}$$

CORNER CONNECTION 1

EAST ELEVATION PORTION

MAX TENSION = 139^K

MAX COMPRESSION = 119^K

BRACE TYPE : HSS 14x14x.3125 , A500 GRADE B

BRACE LIMIT STATES : $\phi R_n = \phi F_y A_g$

$\phi R_n = 0.9(46)(15.7) = 650^K$ $650^K > 139^K \therefore OK$

TENSION RUPTURE : $\phi R_n = \phi F_u A_e$

$U = 1 - \frac{5.25}{14} = 0.625$

$A_e = 0.625(15.7) = 9.8125$

$\phi R_n = 0.75(58)(9.8125) = 426.8^K > 139^K \therefore OK$

BRACE / GUSSET

WELD RUPTURE : $\phi R_n = 1.392(D)(L_w)(\#)$

$\phi R_n = 1.392(3)(14)(4) = 234^K > 139^K \therefore OK$

BASE METAL STRENGTH : $\phi R_n = \phi(0.6) F_u A_{nw} \#$

BRACE : $A_{nw} = 0.291(14) = 4.074$

$\phi R_n = 0.75(0.6)(58)(4.074)(4) = 425^K > 139 \therefore OK$

GUSSET : $\frac{1}{4}$" A36 $\Rightarrow A_{nw} = 0.25(14) = 3.5$

$\phi R_n = 0.75(0.6)(58)(3.5)(2) = 182^K > 139 \therefore OK$

GUSSET :

TENSION YIELDING : $\phi R_n = \phi F_y A_g$

$\phi R_n = 0.9(36)(0.25)(30) = 243^K > 139^K \therefore OK$

WHITMORE
(SEE DETAIL)

TENSION RUPTURE : $\phi R_n = \phi F_u A_e$

$\phi R_n = 0.75(58)(0.5)(30) = 652.5^K > 139 \therefore OK$

CORNER CONNECTION 2

GUSSET :

BUCKLING : $\phi P_n = \phi A_g F_{CR}$

$$\lambda_c = \frac{Kl}{r\pi}\sqrt{\frac{F_Y}{E}} = \frac{0.5(18)}{\left(\frac{.25}{\sqrt{12}}\right)\pi}\sqrt{\frac{36}{29000}} = 1.40 < 1.5$$

$$F_{CR} = 0.658^{\lambda_c^2} F_Y = 0.658^{1.4^2}(36) = 15.85 \text{ psi}$$

$$\phi P_n = 0.9(15.85)(0.25)(30) = 106.9 < 119$$

$\therefore$ PLATE BUCKLES, INCREASE t_P

$$\lambda_c = \frac{0.5(18)}{\left(\frac{.5}{\sqrt{12}}\right)\pi}\sqrt{\frac{36}{29000}} = 0.7 < 1.5$$

$$F_{CR} = 0.658^{.7^2}(36) = 29.3$$

$$\phi P_n = 0.9(29.3)(.5)(30) = 396^K > 119^K \quad \therefore \text{OK}$$

GUSSET / BEAM

WELD RUPTURE : $\phi R_n = 1.392(D)(L_w)(\#)$

$$(1 + 0.5\sin(\theta)^{1.5}) = 1 + .5(\sin(39))^{1.5} = 1.25$$

$$\phi R_n = 1.392(1.25)(3)(28.6)(2) = 298.6^K > 139^K \quad \therefore \text{OK}$$

BASE METAL : $\phi R_n = \phi(0.6) F_u A_{nw} \#$

GUSSET : $0.75(.6)(58)(.5)(28.6) = 373.2^K > 139^K$

BEAM : $0.75(0.6)(65)(.745)(28.6)(2) = 1246 > 139^K$ $\therefore$ OK

CORNER CONNECTION 3

BEAM LIMIT STATES

WEB YIELDING : $\phi R_n = \phi F_y A_g$

$\phi R_n = 0.9(50)(.49)(28.6) = 630.6^k > 139$

WEB CRIPPLING : $R_n = 0.8 t_{wb}^2 \left[1 + 3\left(\frac{N}{d}\right)\left(\frac{t_{wb}}{t_{fb}}\right)^{1.5} \right] \sqrt{\frac{E F_y t_f}{t_w}}$; $N = t_p$

$R_n = 0.8(.49)^2 \left[1 + 3\left(\frac{.5}{26.9}\right)\left(\frac{.49}{.745}\right)^{1.5} \right] \sqrt{\frac{29000(50)(.745)}{(.49)}} = 292$

$\phi R_n = 0.75(292) = 219^k > 108 \therefore OK$

HORIZONTAL COMPONENT OF AXIAL FORCE :

$139 \cos 39 = 108^k$

WEB BUCKLING : $R_n = \frac{24 t_{wb}^3 \sqrt{E F_y}}{h}$

$R_n = \frac{24(.49^3)\sqrt{29000(50)}}{24.26} = 140.1^K > 108^K \therefore OK$

CORNER CONNECTION 4

ANGLE/COLUMN CONNECTION:

NUMBER OF BOLTS NEEDED: $\frac{3}{4}$" ø A325-N, $\phi r_n = 15.9^k$

$\phi r_{nt} = 29.8^k$

$T_u = 139 \sin 39 = 87.5^k$

80 − 11.9

72 − 21.8

$V_u = 108 + 21.8 = 129.8$

$F_{nv} = 48$

$F_{nt} = 90$

∴ TRY 2 ROWS OF 5 BOLTS

SHEAR STRESS IN BOLTS:

$$f_v = \frac{129.8}{10(.442)} = 29.4^{ksi}$$

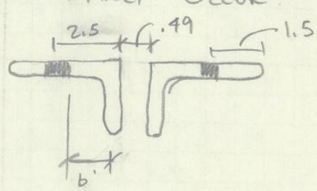
AVAILABLE TENSILE STRENGTH:

$$F_t' = 1.3F_{nt} - \frac{F_{nt}}{\phi F_{nv}} f_v$$

$$= 1.3(90) - \frac{90}{36}(29.4) = 43.5^{ksi} < 90^{ksi}$$

$$\phi r_{nt} = 0.75(43.5)(.442) = 14.42^k$$

WILL PRYING OCCUR



t_a

$$b = 2.5 - \frac{.75}{2} = 2.125$$

d_b

$$b' = 2.125 - \frac{.75}{2} = 1.75$$

$$Z_{p_1} = \frac{pt_f^2}{4} = \frac{3.6(.75^2)}{4} = .506$$

$$a = \begin{cases} a_{actual} = 1.5 \\ 1.25b = 1.25(2.125) = 2.656 \end{cases}$$

$$\phi M_{n_1} = 0.9F_u Z_{p_1}$$

$$= 0.9(50)(.506)$$

$$= 22.77 \text{ kip-in}$$

$$a' = 1.5 + \frac{.75}{2} = 1.875$$

$$p = \begin{cases} \text{gage} = 5.49 \\ \frac{18}{5} = 3.6 \leftarrow \end{cases}$$

$\phi r_{nt} b' = 14.42(1.75) = 25.2$ k-in

∴ PRYING OCCURS

CORNER CONNECTION 5

PRYING EFFECTS:

$\phi M_{n2} = 0.9 F_u Z_{P2}$ $\quad Z_{P2} = \frac{(p - d_h) t_f^2}{4}$

$= 0.9(50)(.4)$

$= 18$ $\quad = (3.6 - .75)(.75^2)/4 = 0.40$

$\phi M_{n1} + \phi M_{n2}\left(1 + \frac{1.75}{1.875}\right) = 22.77 + 18(1.93) = 57.57$

∴ BOLT RUPTURE w/ PRYING ACTION

ANGLE LIMIT STATES:

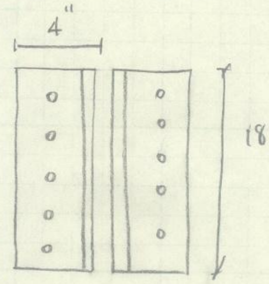
SHEAR YIELDING: $\phi V_n = \phi(0.6) F_y 2 L_p t_p$

$\phi V_n = 1.0(0.6)(36)(2)(18)(.75)$

$= 583.2$

$V_c = \frac{B}{r} P_u$

$= \frac{9.39}{30}(139) = 43.5$ $\quad 583.2 > 43.5$ ∴ OK



SHEAR RUPTURE: $\phi V_n = 0.75(0.6 F_u)(A_n)$

$A_n = 18(.75) - 5(3/4 + 2/16)(.75) = 10.22 \text{ in}^2$

$\phi V_n = 0.75(.6)(58)10.22 = 266.7 > 43.5$ ∴ OK

BLOCK SHEAR:

TABLE 9.3a = 46.2
9.3b = 219
9.3c = 250 } 219

$\phi V_n = 2(219 + 46.2)(.75) = 382.8 > 43.5$ ∴ OK

CORNER CONNECTION 6

BOLT LIMIT STATES :

SHEAR & TENSION GOOD DUE TO PRYING CHECK

BOLT BEARING & TEAR OUT :

ANGLE : $\phi r_n = 0.75(2.4) d_b F_u t$

$= 0.75(2.4)(.75)(58)(.75) = 58.7$ } BEARING

COLUMN : $\phi r_n = 0.75(2.4)(.75)(65)(.645) = 56.6$

ANGLE : $\phi r_n = 0.75(1.2)(L_c)(F_u)t$

EDGE $L_c = 1.5 - \frac{.75 + 1/16}{2} = 1.09$

$\phi r_n = 0.75(1.2)(1.09)(58)(.75) = 42.6$

OTHER $L_c = 3.0 - (.75 + 1/16) = 2.19$ } TEAR OUT

$\phi r_n = 0.75(1.2)(2.19)(58)(.75) = 85.7$

COLUMN : $\phi r_n = 0.75(1.2)(2.19)(65)(.645) = 82.6$

∴ CONTROLLED BY SHEAR, OK

GUSSET/ANGLE LIMIT STATES

WELD RUPTURE : $\phi R_n = 1.392(D)(L_w)\#$

$\phi R_n = 1.392(\overset{\text{min}}{3})(18)(2) = 150^k > 139$ ∴ OK

BASE METAL STRENGTH : $\phi R_n = \phi(0.6) F_u A_{nw} \#$

ANGLE $= 0.75(0.6)(58)(18)(.75)(2) = 704.7 > 139$ ∴ OK

GUSSET $= 0.75(0.6)(58)(18)(.5) = 234 > 139$ ∴ OK

GUSSET :

TENSION YIELDING : $\phi R_n = \phi F_y A_g$

$\phi R_n = 0.9(36)(18)(.5) = 291 > 139$ ∴ OK

TENSION RUPTURE : $\phi R_n = \phi F_u A_e$

$\phi R_n = 0.75(58)(9) = 391 > 139$ ∴ OK

CORNER CONNECTION 7

NORTH ELEVATION PORTION

MAX TENSION = 278^K

MAX COMPRESSION = 226^K

BRACE TYPE : HSS 14×14×0.5

BRACE LIMIT STATES: TENSION YIELD = 650 > 278 ∴ OK
RUPTURE = 427 > 278 ∴ OK

BRACE/GUSSET LIMIT STATES

WELD RUPTURE = $1.392(4)(14)(4) = 312 > 278$ ∴ OK

BASE METAL STRENGTH: BRACE = 425 > 278 ∴ OK

GUSSET ($\frac{1}{2}$") $= 0.75(0.6)(58)(14)(.5)(2) = 365 > 278$ ∴ OK

GUSSET LIMIT STATES

TENSION YIELDING : $\phi R_n = 0.9(36)(.5)(30) = 486^K > 278$ ∴ OK

TENSION RUPTURE = 652 > 278 ∴ OK

BUCKLING = 396 > 226 ∴ OK

GUSSET/BEAM LIMIT STATE > 278 ∴ OK (W27×94 DIMENSIONS ARE CONSERVATIVE)

BEAM LIMIT STATES:

WEB YIELDING : > 278 ∴ CONSERVATIVE, OK

WEB CRIPPLING:

$$\phi R_n = 0.75(0.8(.66^2)\left[1 + 3\left(\frac{.5}{27.6}\right)\left(\frac{.66}{1.08}\right)^{1.5}\right]\sqrt{\frac{29000(50)(1.08)}{.66}}$$

$\phi R_n = 413^K > 226$ ∴ OK

BUCKLING

$$\phi R_n = \frac{0.75(24)(.66^3)\sqrt{29000(50)}}{23.8}$$

$h = 36.1(.66) = 23.8$

$\phi R_n = 261.8 > 226$ ∴ OK

CORNER CONNECTION

8

GUSSET/PLATE CONNECTION:

$H_c = \frac{e_c}{r} P_u$

$= \frac{4.57}{30} 278 = 42$

WELD RUPTURE: $\phi R_n = 1.392(D)(L)(\#)$

$\phi R_n = 1.392(1.25)(26)(2)(3) = 271 > 42 \therefore OK$

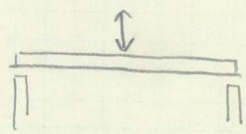
BASE METAL STRENGTH: $\phi R_n = \phi(0.6) F_u A_{nw}$

GUSSET $= 0.75(0.6)(58)(26)(.5) = 340 > 42 \therefore OK$

PLATE (1" A36 $\Rightarrow \therefore$ OK)

PLATE LIMIT STATES

PLATE BENDING:



$\phi R_n = \phi 6.25 t_p^2 F_y$

$= 0.9(6.25)(1.0^2)(36) = 202.5$

FROM BEAM

MAX HORIZONTAL FORCE $= 278 \sin 39 = 175$

$202.5 > 175^k \therefore OK$

PLATE/COLUMN LIMIT STATES

WELD RUPTURE: $1.392(3)(26)(2) = 217 > 175 \therefore OK$

BASE METAL STRENGTH: $\phi R_n = \phi(0.6) F_u A_{nw}$

PLATE $= 0.75(0.6)(58)(26)(1.0)(2) = 1357 > 175 \therefore OK$

COLUMN $= 0.75(0.6)(58)(26)(.645)(2) = 875 > 175 \therefore OK$

CORNER CONNECTION 9

COLUMN LIMIT STATES

LOCAL FLANGE YIELDING : $\phi R_n = \phi F_y A_n$

$\phi R_n = 1.0(50)(.645)(26)(2) = 1677 > 278 \therefore OK$

ECCENTRIC LOADING FROM NORTH ELEVATION :

$e = 5.6''$

74 - 71

$V_u = \frac{e}{r} P_u + V_u = 74 + \frac{9.4}{30}(278) = 161$

$M_{u2} = 902 \text{ in-k} \rightarrow 75 \text{ ft-k}$

$M_{u1} = 6.95(43+22) = 452 \text{ in-k} \rightarrow 38 \text{ ft-k}$

COMBINED LOADING $\frac{P_u}{P_r} = \frac{376}{626} = 0.6 > 0.2 \therefore H1-1a$

$$\frac{P_u}{P_r} + \frac{8}{9}\left(\frac{M_1}{M_{n1}} + \frac{M_2}{M_{n2}}\right) = \frac{376}{626} + \frac{8}{9}\left(\frac{38}{382.5} + \frac{75}{242}\right) = 0.96$$

$\therefore$ OK , COULD INCREASE COLUMN SIZE

CORNER CONNECTION 10

BEAM COLUMN CONNECTION

EAST ELEVATION

DIAPHRAGM LOAD $= \frac{187.7^k}{4} = 47^k$

$H_b = \frac{\alpha}{r} P_u = \frac{12.02}{30}(119) = 48^k$

95^k AXIAL FORCE IN BEAM

SHEAR END PLATE DESIGN

$V_u = 22^k$

BEAM LIMIT STATES

SHEAR YIELDING : $\phi R_n = (1.0)(0.6) F_y h t_w$

$\phi R_n = 0.6(50)(24.26) = 727.6^k > 22 \therefore OK$

BEAM/END PLATE :

WELD RUPTURE : $\phi V_n = 1.392(D)(L_p)(\#)$

$\phi V_n = 1.392(3)(9 - 2(3/16))(2) = 72.0^k > 22 \therefore OK$

BASE METAL STRENGTH : $\phi V_n = \phi(0.6) F_u A_{nw}$

$\phi V_n = 0.75(0.6)(65)(9 - 6/16)(.49) = 124 > 22 \therefore OK$

$\phi V_n = 0.75(0.6)(58)(9 - 6/16)(.5) = 113 > 22 \therefore OK$

END PLATE LIMIT STATES $\frac{1}{2}''$, A36

GROSS SHEAR : $\phi V_n = \phi(0.6) F_y(2)(L_p t_p)$

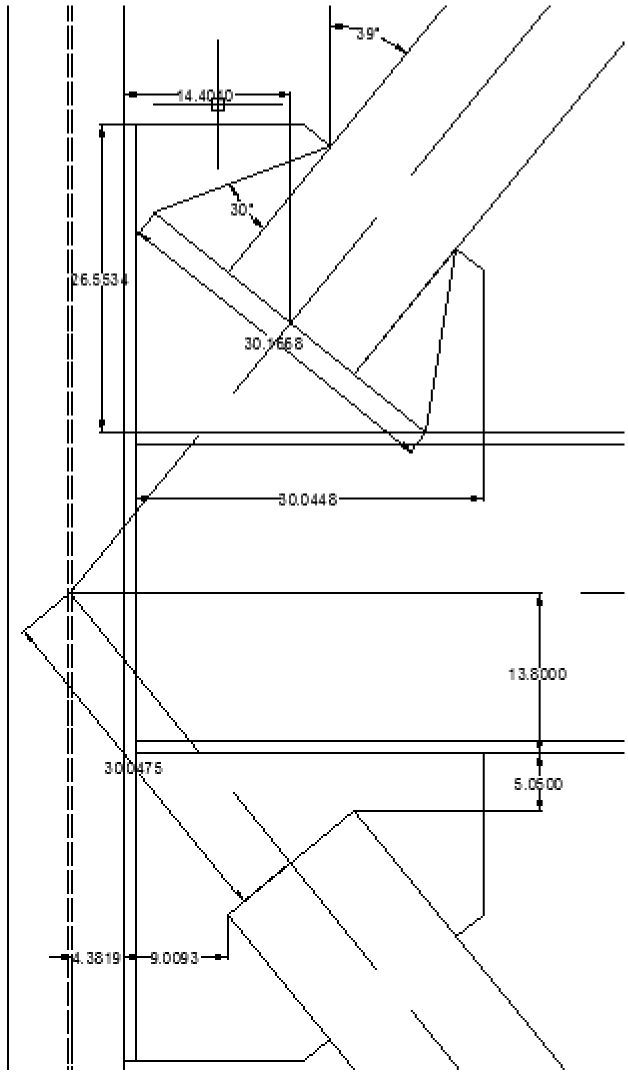
$\phi V_n = 0.6(36)(2)(9)(.5) = 194 > 22 \therefore OK$

NET SHEAR : $\phi V_n = \phi(0.6) F_u A_n$

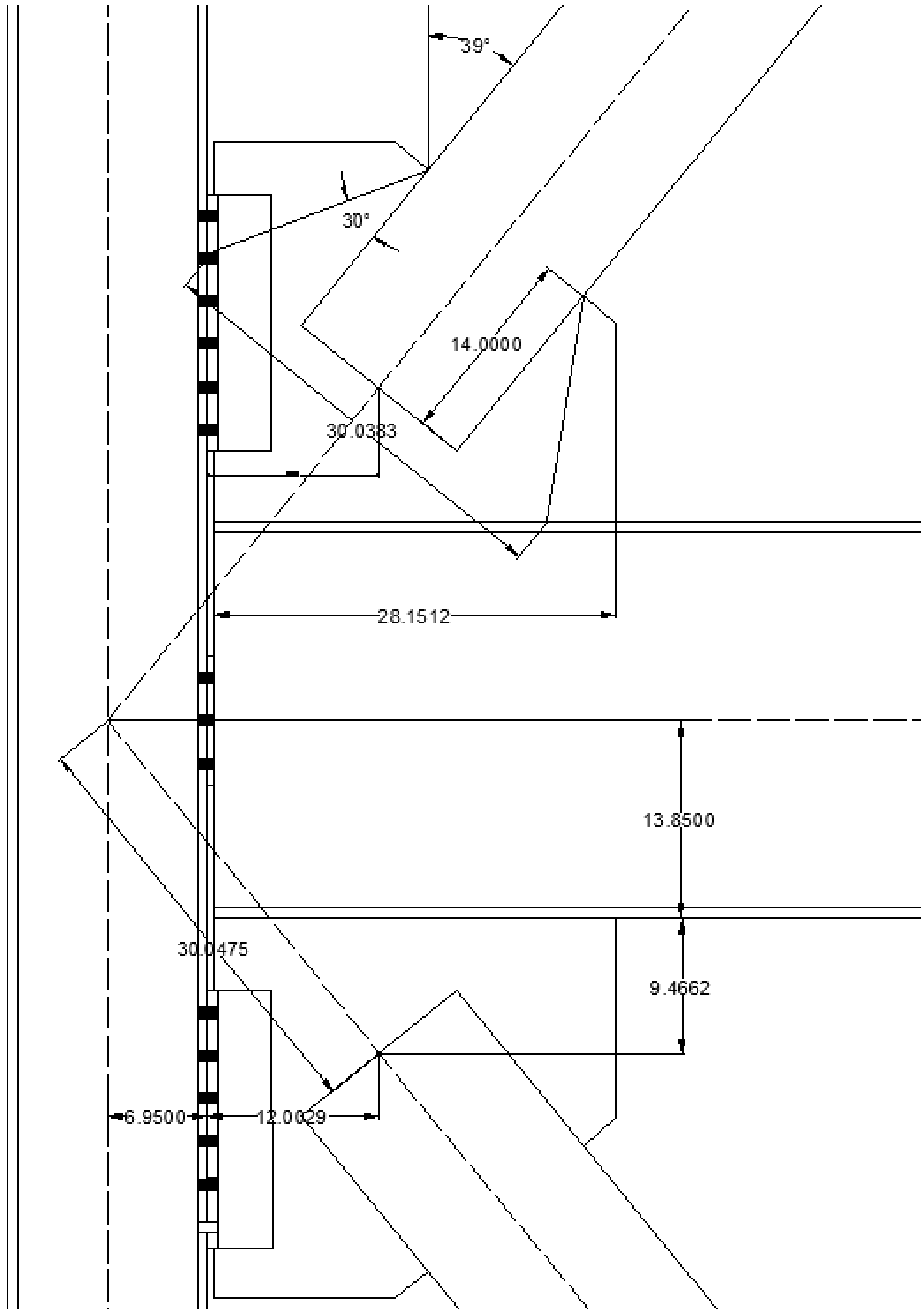
$A_n = 9(.5) - 3(7/8 + 2/16)(.5) = 3.19\ in^2$

$\phi V_n = 0.75(0.6)(58)(3.19) = 83.3 > 22 \therefore OK$

North Elevation:



West Elevation:



Appendix F:

Duct Size and Pressure Drop Calculations:

	Perimeter					Interior				
	Duct Dia (in)	CFM	Run	Friction Loss/100'	Friction Loss	Duct Dia (in)	CFM	Run	Friction Loss/100'	Friction Loss
NW	18	4800	30	0.55	0.17	18.5	3600	30	0.3	0.09
	18	3600	55	0.3	0.17	17	2400	80	0.14	0.11
	15.5	2400	80	0.3	0.24	13	1200	208	0.18	0.37
	11.5	1200	120	0.35	0.42					
NE	18	4800	10	0.5	0.05	11.5	1200	25	0.35	0.09
	18	3600	35	0.3	0.11					
	16	2400	65	0.26	0.17					

	Perimeter					Interior				
	Duct Dia (in)	CFM	Run	Friction Loss/100'	Friction Loss	Duct Dia (in)	CFM	Run	Friction Loss/100'	Friction Loss
SW	18	7200	15	1.2	0.18	18.00	5300.00	15.00	0.70	0.11
	18	6000	40	0.8	0.32	18.00	4100.00	40.00	0.40	0.16
	18	4800	65	0.45	0.29	16.50	2900.00	65.00	0.33	0.21
	17	3600	85	0.4	0.34	15.00	2400.00	85.00	0.35	0.30
	16	2400	100	0.26	0.26	13.50	1200.00	130.00	0.16	0.21
SE	17	3600	20	0.4	0.08	17.00	3600.00	20.00	0.40	0.08
	15	2400	40	0.35	0.14	17.00	2400.00	60.00	0.19	0.11
	11.5	1200	60	0.35	0.21					