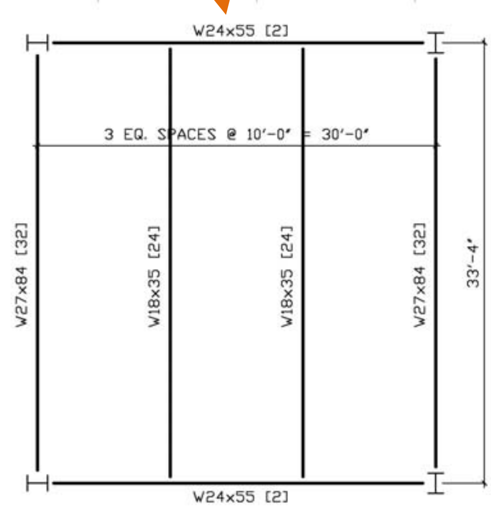
EXISTING SYSTEM

The four-story building of 329 Innovation Boulevard is supported by a steel superstructure. The floor framing system consists of a composite slab and metal deck on wide flange beams and girders. The concrete used is 3½" lightweight concrete with one layer of 6x6xW1.4xW1.4 WWF. The metal decking used is 3" galvanized wide rib type composite deck. The decking is to be continuous over a minimum of three spans. The total thickness of the flooring system comes to 6½" and therefore, the top of steel (beams and girders) is located at -6½" from the finished floor. The typical size of the beams is W18x35 and they span 33'-3" and the girders range from W18x35 to W21x44 and typically span 30'0". There are minimal interferences on each floor, making each of the three floor systems practically identical.

Typical Framing Plan Figure 8.1



Enlarged Typical Bay Framing Plan Figure 8.2

Lateral resistance is provided by several full moment connections of beams, girders, and columns. These connections can be found in the middle bay of the building on each end of the building. There are two columns on each end where the two beams and two girders are all connected by full moment connections. Majority of the moment connections occur in the interior of the building, and there are total of twelve moment connections on the exterior frame. The mechanical penthouse located on the roof utilizes flat strap bracing in plane with the stud wall. The following 3D model shows the location of the moment frames (blue members):

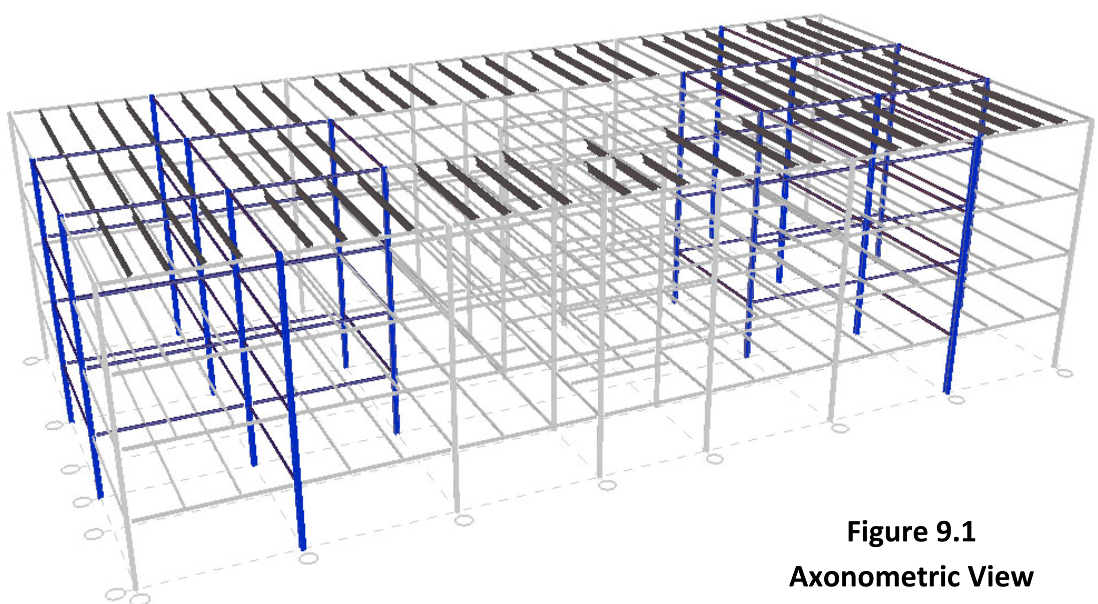


Figure 9.1
Axonometric View

TWO-STORY EXPANSION

A theoretical two-story vertical expansion was proposed for 329 Innovation Boulevard. The two floors will affect the following:

- **Gravity Members**
- **Resistive System Members** (Due to changes in the wind and seismic loads)

This structural depth will go through the process of re-analyzing and re-sizing the gravity members. It will also explore an alternative resisting system, and size the members involved.

DESIGN LOADS

Live Loads	
Corridors	100 PSF
Stairs	100 PSF
Public Areas	100 PSF
Mechanical/Electrical Rooms	175 PSF
Open Plan Office (80 PSF + 20 PSF Partitions)	100 PSF
Slabs-On-Grade (U.N.O.)	100 PSF
Slabs-On-Grade (Dock/Receiving)	200 PSF
Roof Live Loads	
Minimum Roof Live Load	20 PSF
Dead Loads	
Partition Allowance	20 PSF
Lightweight Concrete Slab	115 PCF
MEP	5 PSF
Metal Decking	2-3 PSF (Deck Catalog)
Beam Weight	Specific To Each Member
Snow Loads	
Terrain Category	C
Ground Snow Load (P_g)	40 PSF
Snow Exposure Factor (C_e)	0.9
Thermal Factor (C_t)	1.0
Snow Importance Factor (I_s)	1.0
Wind Loads	
Minimum Wind Load	10 PSF
Uplift On Roof	20 PSF
Basic Wind Velocity	90 MPH
Wind Importance Factor	1.0
Wind Exposure Category	C
Internal Pressure Coefficient	±0.18
Components And Cladding	By Supplier

DESIGN LOADS CONT'D

Seismic Loads

Seismic Importance Factor (I_E)	1.0
Seismic Response Acceleration (S_s)	16.8%
Spectral Response Acceleration (S_1)	5.9%
Spectral Response Coefficient (S_{DS})	13.4%
Spectral Response Coefficient (S_{D1})	6.7%
Seismic Design Category	A
Site Class	C
Long-Period Transition Period (T_L)	6 Sec.
Seismic Force Resisting System	Undetailed
Response Modification Factor (R)	3.0
Seismic Response Coefficient (C_S)	0.045
Deflection Amplification Factor (C_d)	3.0
Design Base Shear	60 Kips
Analysis Procedure	Eq. Lat. F.

WIND ANALYSIS

Due to the change in height of the building, the previous wind analysis done had to be revised. The new height will affect the wind pressures applied to the building, and thus increasing the overturning moment of the initial analysis. The members and foundation will have to be designed to withstand these new loads. The general information remained the same, and is given in the table below:

Wind Loading According to ASCE7-05	
Basic Wind Speed	90 MPH
Exposure Category	C
Enclosure Classification	Enclosed
Building Category	II
Importance Factor	1.0
Internal Pressure Coefficient	0.18

The following page contains tables that include the new pressures used to find the loads applied to each story level. ASCE7-05 was utilized to obtain the values.

North/South Wind Pressure Values						
z (ft)	Kz	q_z	$P_{windwardl}$(PSF)	$P_{leeward}$(PSF)	$P_{sidewall}$(PSF)	P_{total}(PSF)
0-15	0.85	14.98	12.84	-8.43	-14.83	21.27
20	0.90	15.86	13.59	-8.43	-14.83	22.02
25	0.95	16.74	14.35	-8.43	-14.83	22.78
30	0.98	17.27	14.80	-8.43	-14.83	23.23
40	1.04	18.33	15.71	-8.43	-14.83	24.14
50	1.09	19.21	16.46	-8.43	-14.83	24.89
60	1.14	20.09	17.22	-8.43	-14.83	25.65
70	1.17	20.62	17.67	-8.43	-14.83	26.10
80	1.21	21.33	18.28	-8.43	-14.83	26.71
90	1.24	21.86	18.73	-8.43	-14.83	27.16

East/West Wind Pressure Values						
z (ft)	Kz	q_z	$P_{windwardl}$(PSF)	$P_{leeward}$(PSF)	$P_{sidewall}$(PSF)	P_{total}(PSF)
0-15	0.85	14.98	11.34	-4.31	-14.83	15.65
20	0.90	15.86	12.01	-4.31	-14.83	16.32
25	0.95	16.74	12.67	-4.31	-14.83	16.98
30	0.98	17.27	13.07	-4.31	-14.83	17.38
40	1.04	18.33	13.87	-4.31	-14.83	18.18
50	1.09	19.21	14.54	-4.31	-14.83	18.85
60	1.14	20.09	15.21	-4.31	-14.83	19.52
70	1.17	20.62	15.61	-4.31	-14.83	19.92
80	1.21	21.33	16.14	-4.31	-14.83	20.45
90	1.24	21.86	16.54	-4.31	-14.83	20.85

The new story forces in the long direction (North/South) are as follows:

T/ Met. Panel (86')	**88.6 Kips**
Level 6 (60')	**74.9 Kips**
Level 5 (56')	**72.7 Kips**
Level 4 (42')	**60.0 Kips**
Level 3 (28')	**65.0 Kips**
Level 2 (14')	**61.6 Kips**

These values produce an overturning moment of **21,400 $'^{K}$.** This value will be compared to the new overturning moment obtained through seismic analysis to establish the controlling load combination. The overturning moment in the East/West direction is **8,500 $'^{K}$**.

SEISMIC ANALYSIS

Like the wind analysis, the previous seismic analysis needed to be revised. New values needed to be obtained due to the change in height and the change in the building frame system. The framing system is changing from moment frames to braced, which changes the response modification coefficient. The coefficient was taken from ASCE7-05 Table 12.2.1 B-4, ordinary steel concentrically braced frames.

Seismic Loading According to ASCE7-05		
Seismic Design Category		A
Seismic Use Group		II
Importance Factor (I_E)		1.0
S_S		0.168
S_1		0.059
S_{DS}		0.134
$S_{D!}$		0.067
Site Class		C
Response Coefficient		
	N-S	0.041
	E-W	0.041
Response Mod. Factor		
	N-S	3.00
	E-W	3.00
Period		0.555
V (kips)		85
K		1.03

The following table was used to obtain the story forces (F_x), the design base shear, and the overturning moment:

Floor	Weight	Height (ft)	K	h^K	$W*h^K$	Cvx	V (K)	Fx
2	330	14	1.03	15.254	5033.74	0.04	85	3.8
3	330	28	1.03	31.203	10296.86	0.09	85	7.8
4	330	42	1.03	47.425	15650.16	0.14	85	11.8
5	330	56	1.03	63.827	21062.90	0.19	85	15.9
6	330	70	1.03	80.364	26520.25	0.24	85	20.0
Roof	343.3	86	1.03	99.396	34122.71	0.30	85	25.7
Totals	**1993.3**				**112686.62**	**1.00**		**85.0**
Base Shear:				85.0 Kips				
Overturning Moment:				5270 Ft.-Kips				

WIND VS. SEISMIC COMPARISON

The overturning moment caused by wind ($21{,}400'^{K}$) is much greater than the moment produced by seismic loads ($5{,}270'^{K}$). Multiple load combinations where considered, they included:

1.4D

1.2D + 1.6L

1.2D + 0.5L +1.6W (Controlled)

1.2D + 1.6W

0.9D + 1.6W

1.28D + 0.5L + 1.202E

1.28D + 1.202E

0.82D + 1.202E

This load combination controlled the previous design of 329 Innovation Boulevard. State College is located in a region of low seismic activity, so this combination is sensible. The two-story vertical expansion did affect the overturning moment greatly, however. The moment produced by wind ($21{,}400'^{K}$) is over twice the moment produced by the original design of the building ($10{,}035'^{K}$). This new moment inspired the idea of creating a new lateral resisting system. The existing lateral system of moment frames would have to be modified, anyway, to transfer the moments through the beams to the columns, and back down to the foundation. Braced frames will be able to transfer the moments through the structure.

BRACED FRAME SYSTEM

Ordinary moment frames (OMF) are usually used in low seismic region (State College is one) or used as gravity frames in high seismic regions. OMFs are expected to withstand limited inelastic deformations in their members and connections when subjected to forces resulting from the motions of the design. With the increased loads, the connections will become more elaborate and more difficult to design.

Concentrically Braced Frames (CBF) are directed along work-lines that intersect at points and are initially developed to resist wind-induced actions in the linearly elastic range. They are characterized by their high elastic stiffness. The braces are designed to carry all of the lateral force shears. Concentrically braced frames have been selected to carry the new loads produced by the wind pressures. Of course there are advantages and disadvantages to both systems, including the large open areas produced by moment frames and the obstructions caused by braced frames. Braced frames also mean more costs in steel; however, the additional two floors of tenant space will more than likely compensate for these additional costs.

LOCATION OF BRACED FRAMES

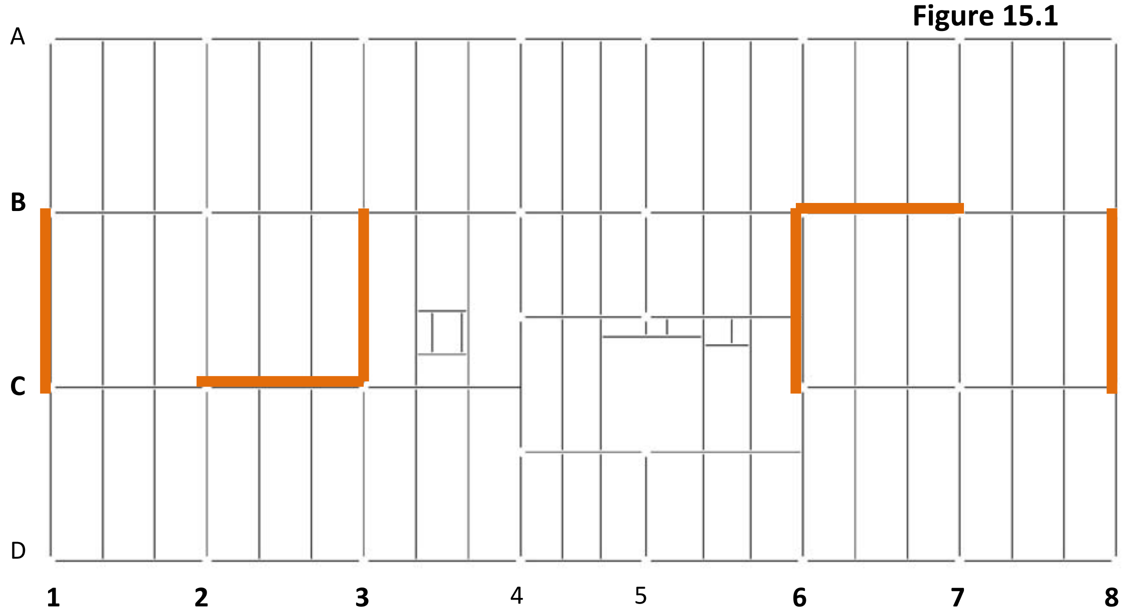
As mentioned previously, a disadvantage of braced frames are obstructions that the form in spaces. This is where good coordination between architect and engineer becomes a priority. These obstructions must be able to coincide with the architectural layout of the space, including window and door locations. Fortunately for 329 Innovation Boulevard, the architectural plans are created after the tenant leases the space. The open floor plan allows for easy placement of the frames.

The engineer is now able to dictate the architectural plans of the space with the position of the frames. There were multiple factors that I had taken into consideration when deciding where the optimal location of the frames would be, they included:

- **Center of Rigidity/Center of Mass**
- **Previous Architectural Aspects**
- **Possible Architectural Schemes w/ Braced Frames**

The center of rigidity and the center of mass was a priority, because by creating the same location for the two centers, I am able to eliminate any torsional effects on the building. For this reason and knowing that the center of mass will be located near the geometric center, I kept the frames symmetrical around the center of building, and located them along the central bays of the building. The following plan shows the preliminary location of the braced frames:

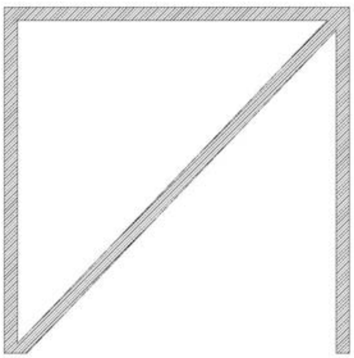
Figure 15.1



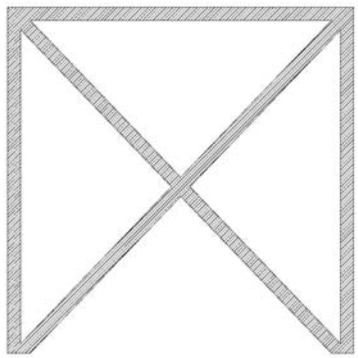
The idea of locating the braces through the central bays stemmed from not wanting to interfere with the façade and its fenestrations. The two braces on the ends are located along the stairways of the building. Entrances/exits to these stairways cannot be obstructed, so that helped with the selection of what type of braced frame to use.

SELECTION OF BRACED FRAMES

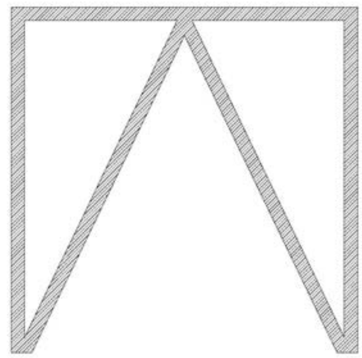
The following images are the three types of braced frames considered:



Diagonal Bracing



X-Bracing

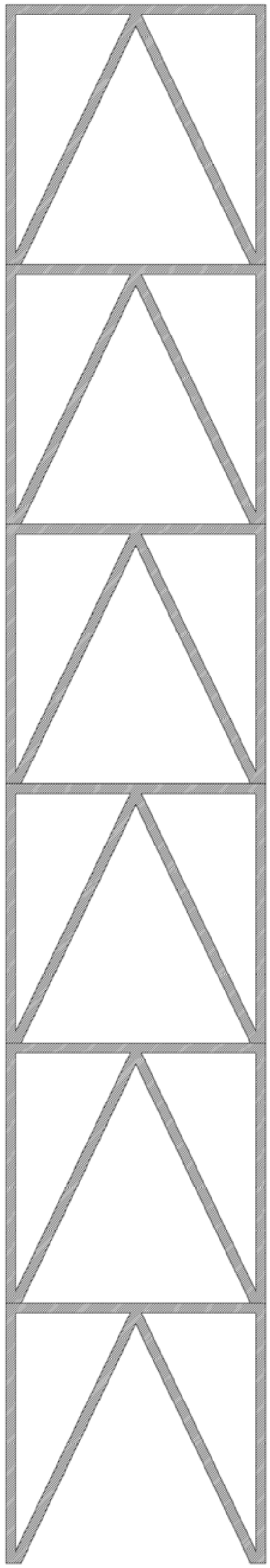


Chevron Bracing

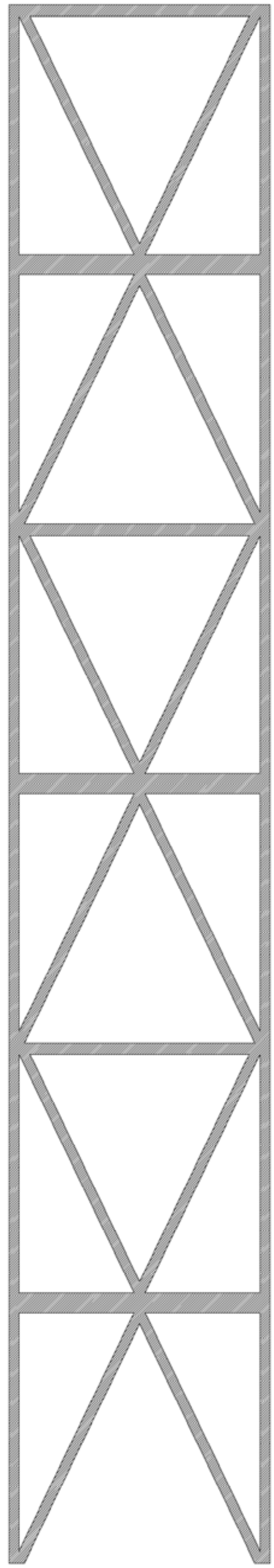
X-bracing clearly makes the most obstructions, and was no longer considered, but aspects of the design were considered. Alternating diagonal bracing, or K-bracing, was initially used for the design. The members for the bracing needed to be large for strength purposes. The large members made the system extremely rigid, and the deflections produced were minimal. Minimal deflections aren't bad, but it was clear that the system did not have to be that rigid. The deflections were much less than the industry standard of H/400, so in order to get a less rigid frame, and thus smaller members, chevron bracing was used. Chevron bracing provides adequate space for doorways, and other possible fenestrations. The inverted V chevron bracing was used on the two frames located on the ends of the buildings. This allowed the location of the planned doorways to the stairway to remain the same. Alternating V and inverted V chevron bracing was used for the four interior frames. This created a two-story "x-bracing" and was used to help create some flexibility in the possible floor plans.

The following page includes the initial elevations of the frames:

CHEVRON BRACING SCHEMATICS



Two Exterior Frames



Four Interior Frames

STRUCTURAL PLAN

Knowing that the cost of a shear stud includes about $10 in steel plus installation costs, I decided to maintain the composite decking; however, I opted not to maintain the composite beams. The new structural system will be composite deck on non-composite beams. I anticipate deeper beams, but I am assuming that it will ultimately create savings in the system.

The concrete used is 3½" lightweight concrete with one layer of 6x6xW1.4xW1.4 WWF. The metal decking used is 3" galvanized wide rib type composite deck. The decking is to be continuous over a minimum of three spans. The total thickness of the flooring system comes to 6½" and therefore, the top of steel (beams and girders) is located at -6½" from the finished floor. The typical size of the beams is W18x35 and they span 33'-3" and the girders range from W18x35 to W21x44 and typically span 30'0".

RAM 3D MODEL

RAM Structural System was utilized to model the building, size the appropriate members, and find the reactions of the members. The following is the 3D RAM model; it shows the location of the braces (red and purple members), and framing of the two-story addition:

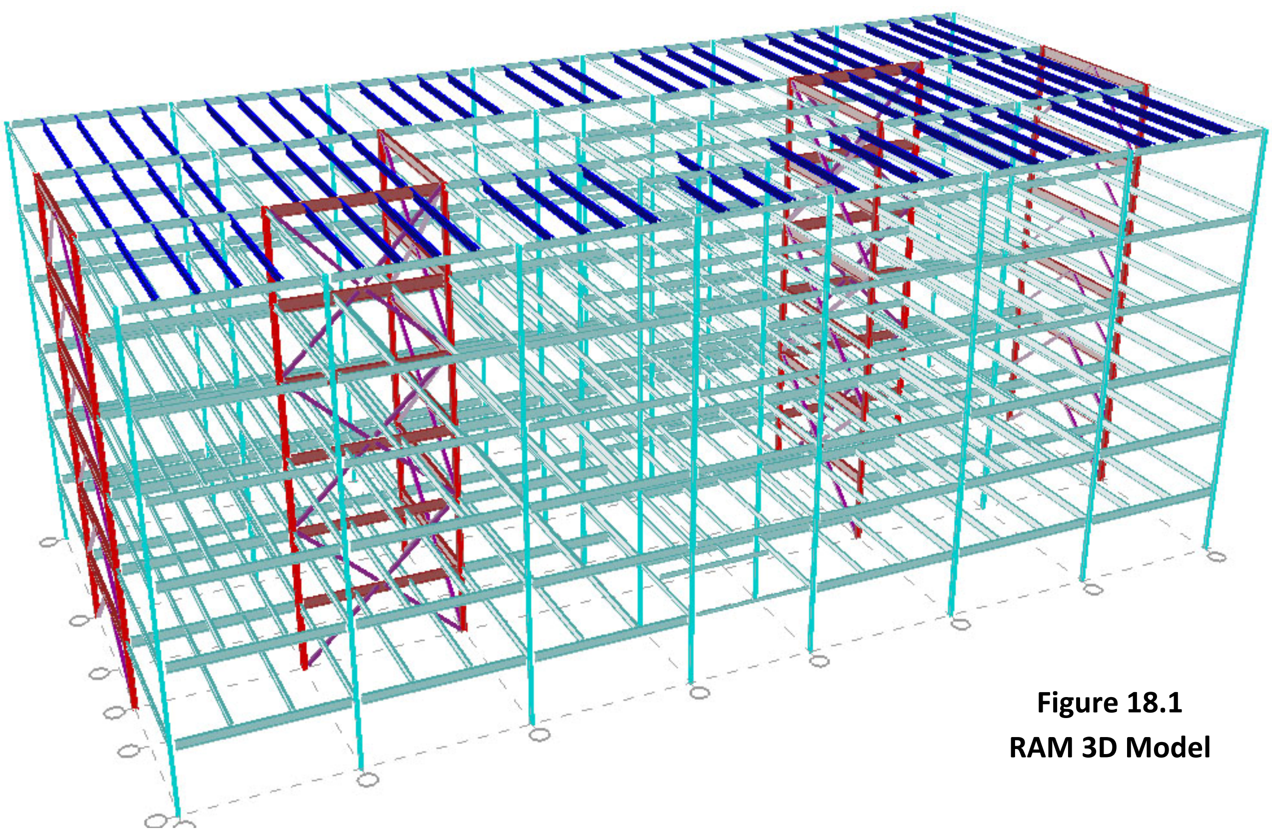


Figure 18.1
RAM 3D Model

INITIAL SIZING OF MEMBERS

The beams utilized in the braced frames were taken from the existing plan. I did this because I found the beams used less than 50% of their capacity in previous technical assignments. I also knew that these beams would have to be large to resist the wind loads. So four W27x84s were used in the long direction and two W24x68s were used in the short direction. In an attempt to reduce the size of the columns, I tried to utilize W10s of different weights.

The bracing member sizes would depend on what shape the braces would be. I considered only two shapes – wide flange and rectangular or square HSS. There are advantages and disadvantages to both, and it comes down to a preference between the two. Due to advances in HSS connections, a new chapter (Chapter K) was added to the Steel Manual. It is titled "Design of HSS and Box Member Connections". *Modern Steel Construction* published an article about this addition of design techniques, and stated that "it ushered in a new era in the use of hollow structural connections." I decided to use these new techniques of design, and opted to use square HSS members for the bracing.

A quick hand check was done, and the initial size of the braces came out to be an HSS8x8x3/8. The overall thickness of these braces (8") is smaller than the width of the flange and web of the columns (12.2" and 9.125", respectively), which would allow the wall thickness to be lesser. These initial members were implemented into RAM Structural System, and check against the various codes and strength checks.

The roof was not redesigned, so the original members (wide flange beams and steel web joists) were used in the model. Their sizes would remain the same, due to the fact that no new loads were applied to them.

STRENGTH CODE CHECK

A strength check was performed using the RAM Structural System model. The results were obtained by loading the model and analyzing it using numerous load combinations. The load combinations were generated by RAM through the load combinations drop-menu. RAM used IBC 2003 LRFD to obtain the combinations. Knowing that wind controls the resisting system - dead, live, and wind loads only were applied to the model. As previously mentioned, the controlling load combination was:

1.2D + 0.5L +1.6W

The strength check dictated the size of the bracing members. The initial size of HSS8x8x3/8 was not large enough for the first two floors for the interior frames, and was too large for top four floors for the exterior frames. The braces on the interior frames were increased from HSS8x8x3/8 to HSS9x9x3/8 for the first two floors. The braces on the exterior frames were decreased from HSS8x8x3/8 to HSS6x6x3/8.

The abovementioned load combination produced the greatest values compared to the other combinations. RAM used the combination to check the resisting members according to strength. It uses a scale so that anything less than 1.0 is an acceptable value. The diagram below shows the color-coded results of RAM's analysis. Note that all members use less than 94% of the maximum strength, with majority less than 70%, meaning that the frames are adequate in strength. The strength code check performed by RAM ultimately dictated the size of the columns. It will be seen later that the system is rigid and produces minimal drift affects, but the members were needed to be larger due to strength. The columns were still reduced in size from the existing plan and are discussed in the next section.

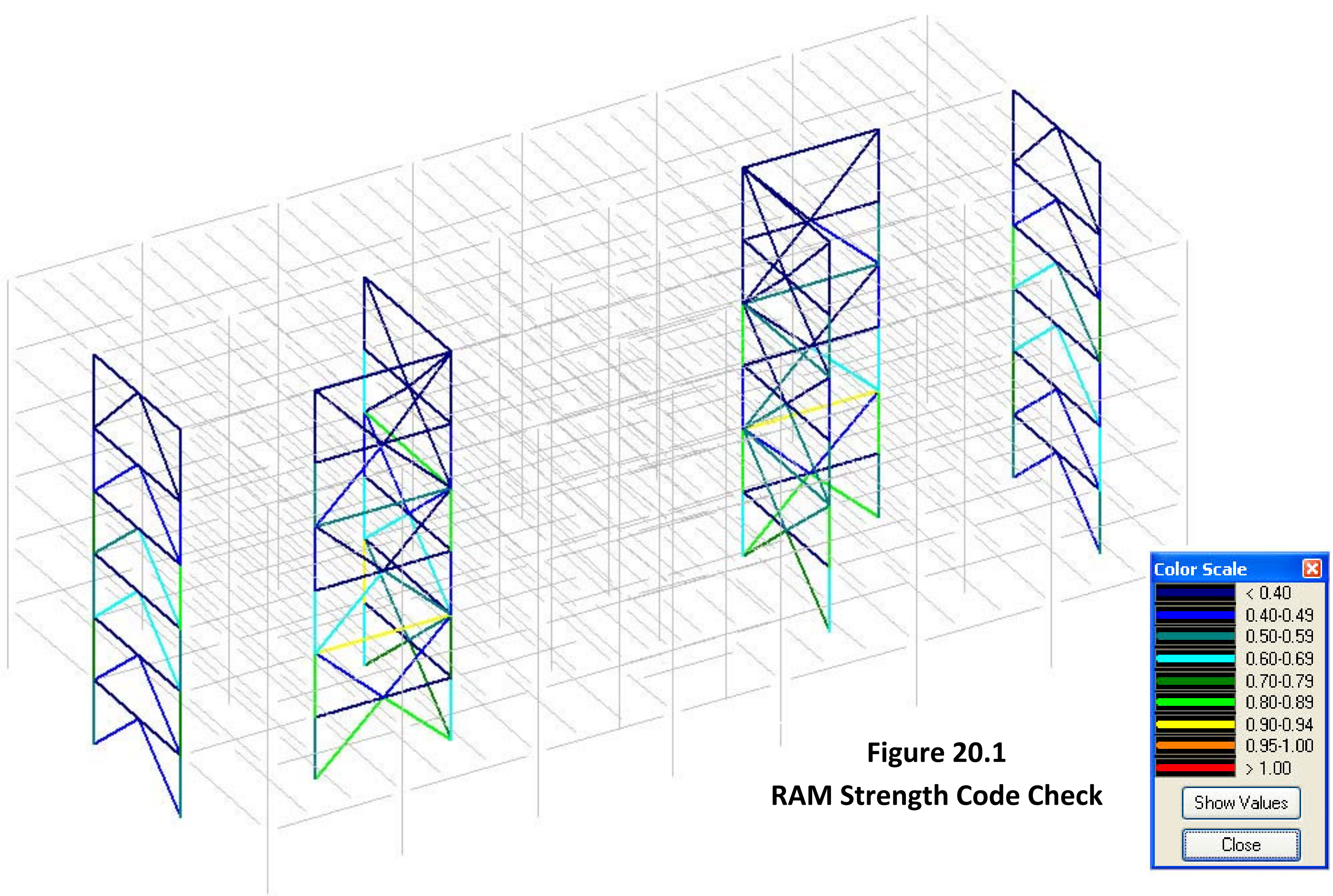


Figure 20.1
RAM Strength Code Check

COLUMN CODE CHECK

RAM Structural System was used to size the gravity member columns. The output showed that the column sizes ranged from W10x33 to W10x49. These column sizes are smaller than the W12x53 and W12x65 columns used in the previous design of the structure. This may be because of the mechanical penthouse loads located on the roof. The columns may also be oversized for the possibility of additional equipment to roof. The mechanical breadth of this report explores the mechanical system, and verifies the assumption of additional equipment needed.

The columns were able to be reduced in size. The new lateral resisting columns consisted of a W10x77 spanning from the ground floor to the third floor and a W10x39 spanning the remaining floors on the east and west ends. The "L" shaped frames consisted of W10x100s spanning the first four floors and a W10x45 spanning the remaining two on the ends. Where the frames meet, the columns consist of a W12x79 that spans the first three floors and a W10x49 spanning the remaining. Clearly, these column sizes are smaller and are shown in a column schedule later in the report. The following is code check performed by RAM, and every column is designed below its max. capacity :

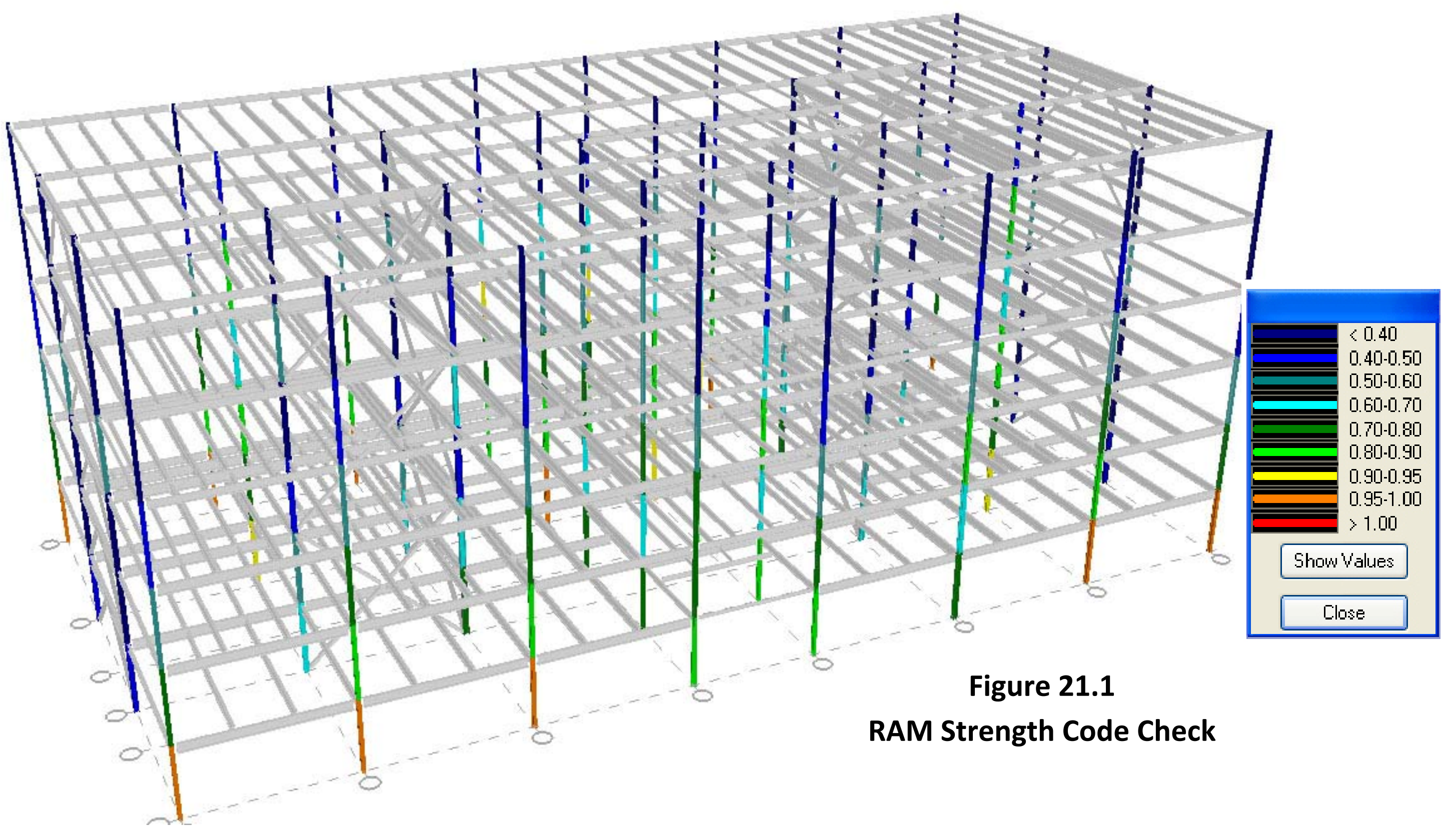


Figure 21.1
RAM Strength Code Check

TORSION ANALYSIS

Along with resisting lateral loads, the braced frames must be able to withstand any torsional forces that may occur. Story shear is assumed to act through the center of mass of each level, and when the center of mass does not coincide with the center of rigidity a moment or torsion force is induced. RAM Frame was used to obtain the centers of rigidity and the centers of mass. The following table contains those values:

Torsion Values				
Floor	**Centers of Rigidity**		**Centers of Mass**	
	X (Ft.)	**Y (Ft.)**	**X (Ft.)**	**Y (Ft.)**
6th Floor	102.35	49.78	101.96	49.88
5th Floor	102.41	49.81	101.68	50.24
4th Floor	102.50	49.84	101.68	50.25
3rd Floor	102.30	49.88	101.68	50.26
2nd Floor	101.92	49.92	101.68	50.26
1st Floor	101.92	49.91	101.68	50.93

A straight comparison of the center of rigidity and center of mass shows that they do coincide almost exactly. The dimensions of 329 Innovation Boulevard are approximately 203'x100', which means the location of the center of mass/rigidity is almost at the geometric center of building. However, according to code, "where diaphragms are not flexible, the mass at each level shall be assumed to be displaced from the calculated center of mass in each direction a distance equal to 5% of the building dimension at that level perpendicular to the direction of the force under consideration. The effect of this displacement on the story shear distribution shall be considered." RAM Frame has accounted for the 5% eccentricity, and the values remain practically identical. The symmetry of 329 Innovation Boulevard in both layout and member sizes aspects have adequately resisted any possible torsional moment created by the lateral loads. No torsional forces have been prepared due to the fact that they will be very minimal.

DRIFT ANALYSIS

The maximum displacement and story drift were calculated using RAM Frame. The maximum values were found under the wind loading, due to the fact that it was the only lateral force applied to the frame. These values were compared to H/400, which yields the acceptable total displacement and story drift. The 329 Innovation Boulevard Expansion is 86' tall, and therefore the acceptable amount of drift is 2.58". Below is a table containing the comparison of the RAM values and the acceptable drift values: Following the comparison table is the deflected shape produced by RAM frame. The values in the comparison table correspond to the red deflected shape of the frames.

Critical Displacements

Floor	Height (ft.)	FF Height (ft.)	H/400 (in.)	RAM Disp. Values (in.)	RAM Drift Values (in.)	H/400 (in.)
Roof	86	16	2.58	0.62	0.11	0.48
6th Floor	70	14	2.58	0.52	0.11	0.42
5th Floor	56	14	2.58	0.41	0.11	0.42
4th Floor	42	14	2.58	0.30	0.11	0.42
3rd Floor	28	14	2.58	0.18	0.10	0.42
2nd Floor	14	14	2.58	0.08	0.08	0.42
1st Floor	0	N/A	N/A	N/A	N/A	N/A

- The drift values do not apply to the 1st floor due to the fact that is considered the ground floor, and the ground prevents any displacement.

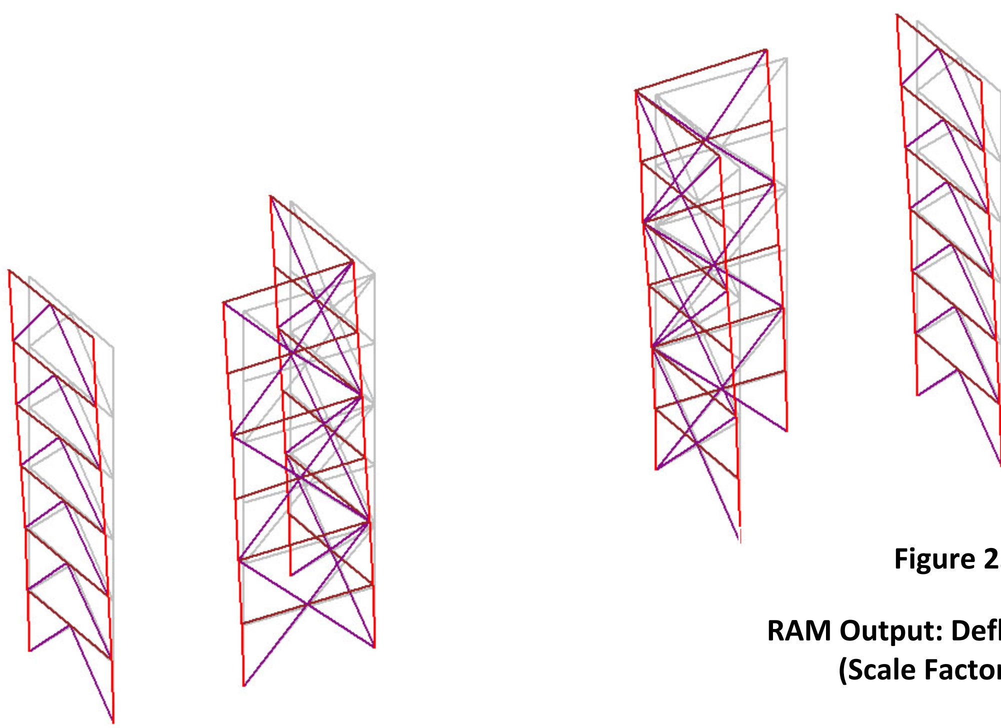


Figure 23.1

RAM Output: Deflected Shape (Scale Factor = 100)

OVERTURNING ANALYSIS

The overall overturning moment was found to be 21,400'^{K} due to the wind load acting in the North/South direction (Refer to Wind Loading under the Wind Analysis Section). Each braced frame will experience an overturning moment as well. This moment will be transferred to the foundation, and it is up to the foundation to resist these moments. Due to the symmetry of 329 Innovation Boulevard, the overturning moments of the frames located on the left side of the plan will be the same as those located on the right side. Refer back to Figure 15.1 for the location of the braced frames. The following table compares the overturning moments of each frame to the resisting moments. If the overturning moment exceeds that of the resisting moment, then additive tension reinforcing is required at the foundation.

Moment Comparison

Frame	Grid Line Location		Overturning Moment	Resisting Moment	Tension
	Left Side	Right Side	(Ft.-Kips)	(Ft.-Kips)	Req'd
B-C	Along #1	Along #8	21,400	205,000	No
B-C	Along #3	Along #6	21,400	205,000	No
B	N/A	6-7	8,500	50,450	No
C	2-3	N/A	8,500	50,450	No

The comparison shows that no tension steel is required to resist the overturning moments. The dead loads alone are adequate. Foundations are discussed in the next section, and it is noted that micropiles are used as anchorage. Although they are not required because of the overturning moment, they may be used for other uplift forces not explored.

FOUNDATIONS

The existing foundation system consists of grade beams and pile caps. The first floor is a slab-on-grade, which consists of 4" normal weight concrete reinforced with fibrous reinforcement. The pile caps are anchored by micropiles, which consist of 7" O.D. steel casing specified by the contractor. These micropiles span a certain length past the competent limestone, which is determined by the specialty contractor. The moments due to the lateral and gravity loads are transferred from the columns into the footings. The foundation should be adequate for the system, but if any redesign was required it would occur at the footings under the braced frames. The foundations would have to be redesigned if moment frames were used, which can ultimately be very expensive.

CONNECTIONS

The connections between the HSS member and the wide flange beams and columns were designed to consist of gusset plates and welds. The gusset plates will be attached to the columns or beams prior to placement and the brace members can then be field welded to the plates. Fillet weld sizes are usually limited to less than 5/16", because that is the maximum size obtained with a single-pass weld. The braces saw a maximum 80 kips, which yielded a weld of 1/4" (< 5/16") with length of 8"on both sides of the HSS member. There actually four welds involved, two on each side of the gusset plate. The plate size is 1/2". The braces that saw lesser forces maintained the 1/4" weld and 1/2" plate size, but only a 6" length of weld was required. Obviously these welds could be smaller (due to the fact that the connection was designed for two welds, rather than four), but I left these lengths for safety purposes. The gusset plates were then sized by making sure that these connections would be possible geometrically. The typical connections included in this report were designed using the worst case loading, so the weld lengths may be even smaller with the braces that saw little force.

COST ANALYSIS

Full-penetration welds for moment connections can cost up to \$1,000 per connection and upwards to \$2,000 if both flanges are engaged. The four-story framing system of 329 did not involve full-penetration welds, but are still very costly. Here's a breakdown:

Bolts:	\$10/bolt
Fillet Welds:	\$35/lb of weld material
	(x 10% for plates)

These values will be used to find the price of a typical moment connection and judged against the pricing of the braced frame system. The braced frame system consists of the connections and the additional HSS members involved. Here's the values used for that system:

HSS:	\$700/ton
Fillet Welds:	\$35/lb of weld material
Plates (1/2" thk.):	\$24.50/S.F.

The following page includes tables of rough estimates for the cost of moment frames vs. the cost of braced frames in the two-story expansion.

Moment Connection Costs					
Material	**Cost/Unit**	**Unit/Connection**	**# of Connections Per Floor**	**# of Floors**	**Total Cost ($)**
Bolts	$10/bolt	18	36	6	38880.00
Welds	$35/lb	4	36	6	30240.00
Plates				(+ 10%)	6912.00
Total					**76032.00**

Braced Connection Costs					
Material	**Cost/Unit**	**Size**	**Tons/Member**	**Quantity**	**Total Cost ($)**
HSS	$700/ton	HSS9x9x3/8	0.439	16	4916.8
		HSS8x8x3/8	0.386	40	10808
		HSS6x6x3/8	0.281	16	3147.2
		Connection Type	**SF of Plare/Connection**		
Plates	$24.50/SF	A	2.80	4	274.40
		B	2.80	20	1372.00
		C	4.70	12	1381.80
		D	3.00	8	588.00
		E	11.10	12	3263.40
		F	6.10	24	3586.80
		Connection Type	**Pounds/Connection**		
Welds	$35/lb	A	0.334	4	50.77
		B	0.334	20	253.84
		C	0.668	12	304.61
		D	0.444	8	134.98
		E	1.777	12	810.31
		F	0.889	24	810.77
Total					**31703.67**

These values are rough estimates, but they do show that it would be beneficial to switch to a braced frame system if the building was designed as six stories. Moment connections are extremely involved, more so than the braced frame connections, and would have more costs of design.

The next cost analysis contains general figures for the composite beam vs. non-composite beam system. A conservative value for total number of shear studs per floor is 1900. Each beam generally has 24 shear studs on it. Let's say that each shear stud is roughly $10 of steel alone (excluding cost of installation)

Cost of Shear Studs = $10(1900 Studs) **= $19,000**

The non-composite system yielded beams larger than the composite system. The typical plan of the composite system included a W18x35 @ 10' O.C.; whereas, the non-composite system used W21x44 @ 10' O.C. RSMeans prices W18x35s at $31/L.F. and W21x44s at $35.50/L.F., which is a difference of $4.50/L.F. There is about 60 W21x44s and W18x35s per floor and each span about 33.33'.

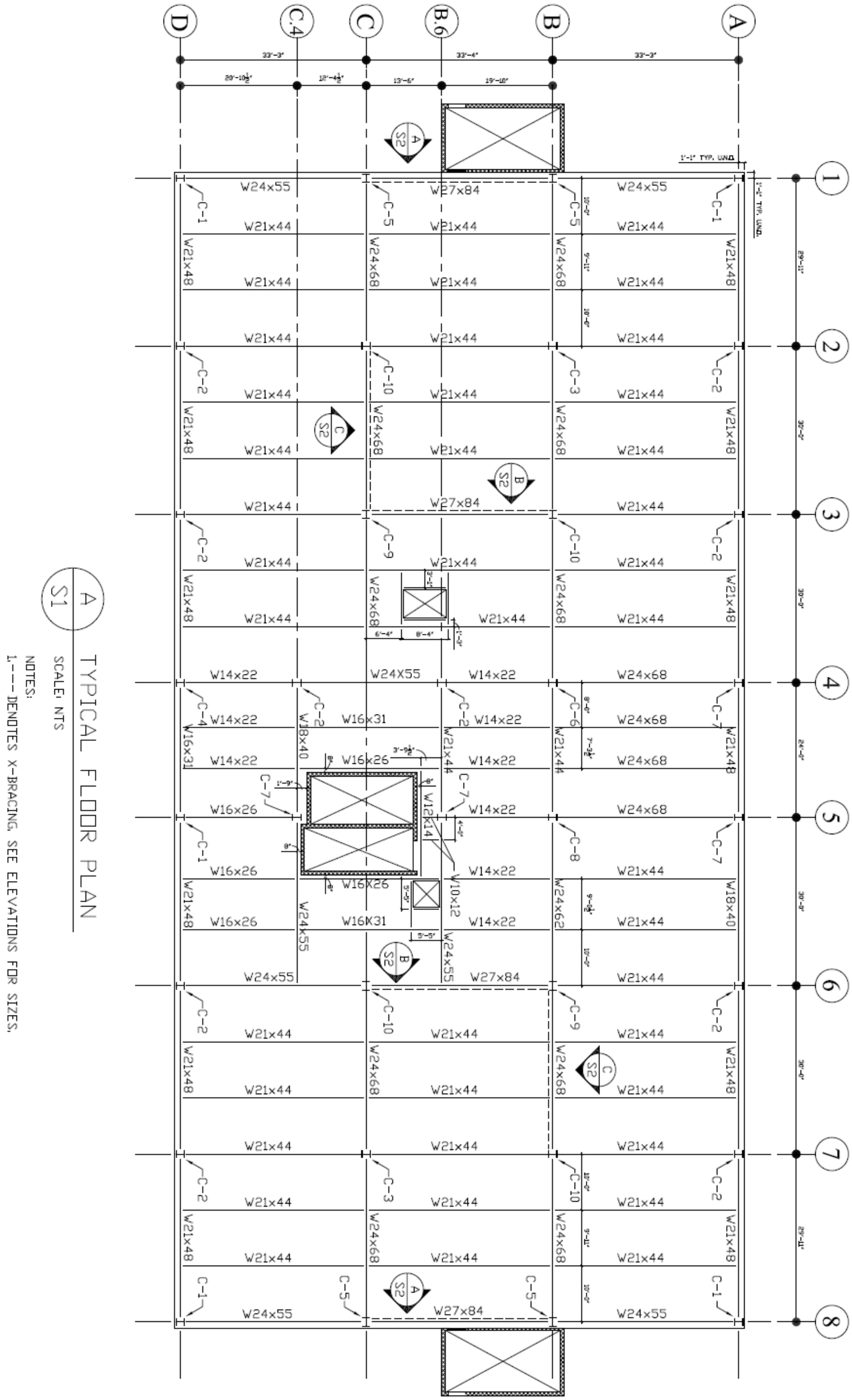
Additive Cost of Beams = $4.50/L.F.(60)(33.33') **= $9,000**

Once again, these are very rough numbers, but they yield about $10,000 in savings per floor. This means a possible total savings of $60,000. This not necessarily a jaw-dropper, but it does show that the non-composite system designed is slightly cheaper.

STRUCTURAL DRAWINGS DESCRIPTION

The drawings included on the following pages are of the culmination of the design process. They include a typical floor plan, the braced frame elevations, and the typical connections used.

STRUCTURAL DRAWINGS



COLUMN SCHEDULE

COLUMN SCHEDULE

COLUMN MARK	ROOF	SIXTH FLOOR	FIFTH FLOOR	FOURTH FLOOR	THIRD FLOOR	SECOND FLOOR	FIRST FLOOR	BOTT. OF BASE PLATE ELEVATION
C-1		W10x33	W10x33	W10x33	W10x33	W10x33	W10x33	
C-2		W10x33	W10x33	W10x39	W10x39	W10x45	W10x45	
C-3		W10x33	W10x33	W10x49	W10x49	W10x68	W10x68	
C-4		W10x33	W10x33	W10x33	W10x33	W10x39	W10x39	
C-5		W10x39	W10x39	W10x39	W10x77	W10x77	W10x77	
C-6		W10x33	W10x33	W10x49	W10x49	W10x54	W10x54	
C-7		W10x33	W10x33	W10x33	W10x33	W10x45	W10x45	
C-8		W10x33	W10x33	W10x45	W10x45	W10x49	W10x49	
C-9		W10x49	W10x49	W10x49	W12x79	W12x79	W12x79	
C-10		W10x45	W10x45	W10x100	W10x100	W10x100	W10x100	

SPLICE TYP.

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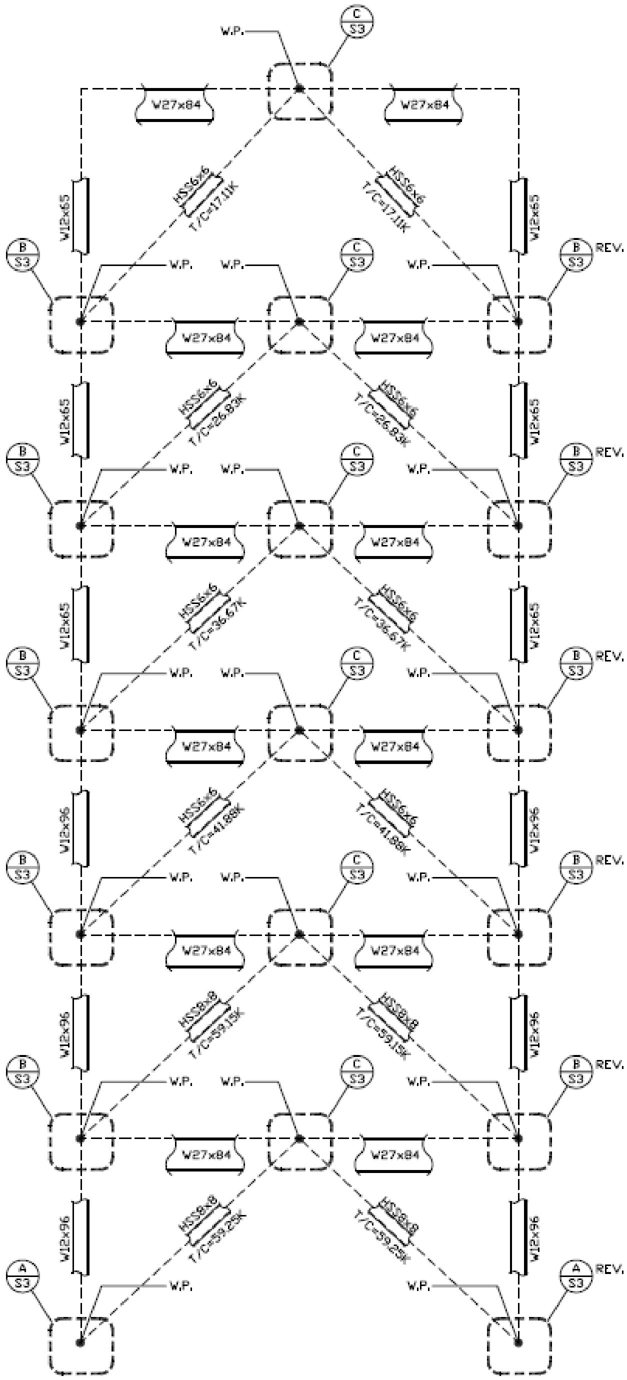


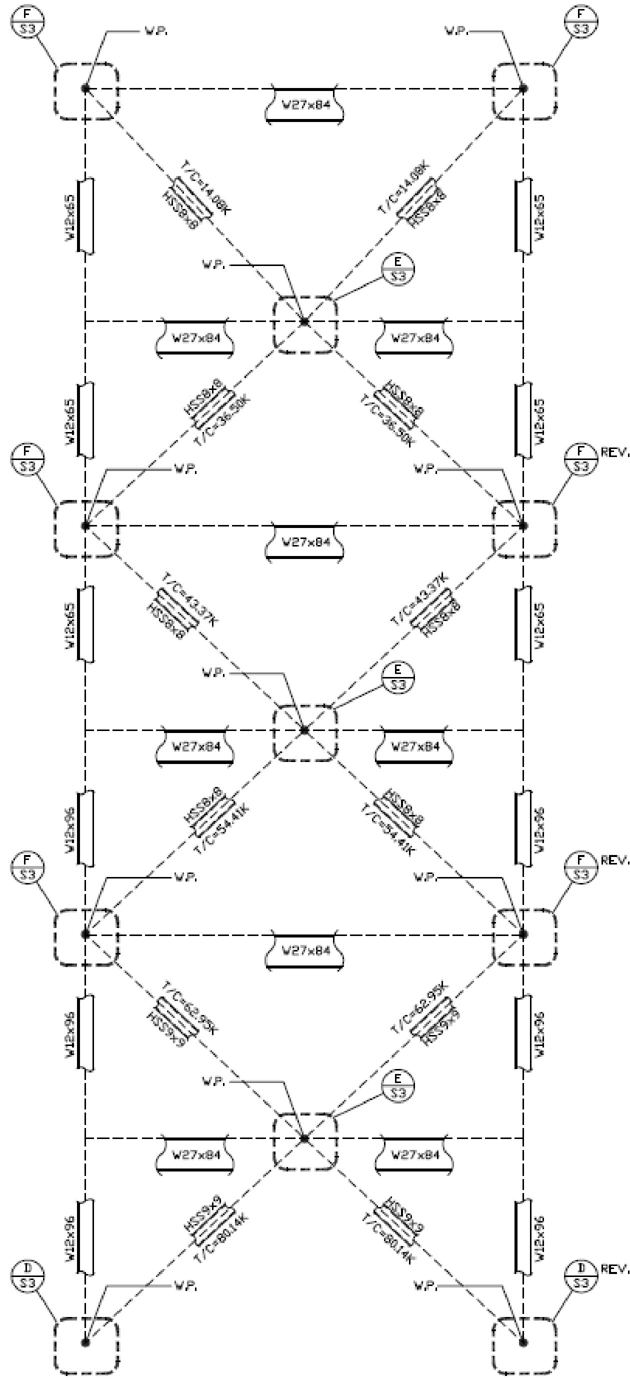
A
S2

ELEVATION

SCALE: NTS

NOTES:

1. REV. DENOTES THE MIRROR IMAGE OF CONNECTION.
2. ALL BRACES ARE 3/8" THICK



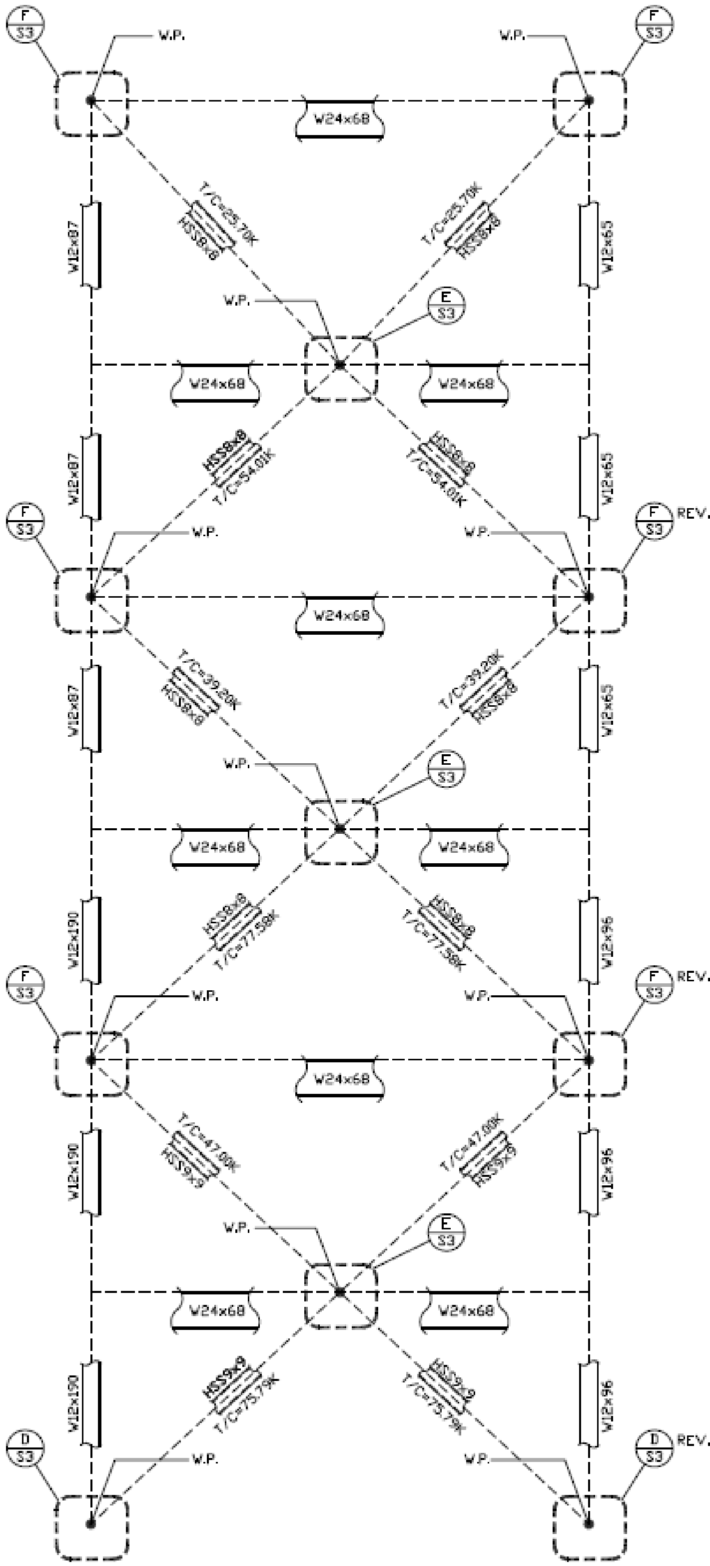
B
S2

ELEVATION

SCALE: NTS

NOTES:

1. REV. DENOTES THE MIRROR IMAGE OF CONNECTION.
2. ALL BRACES ARE 3/8" THICK



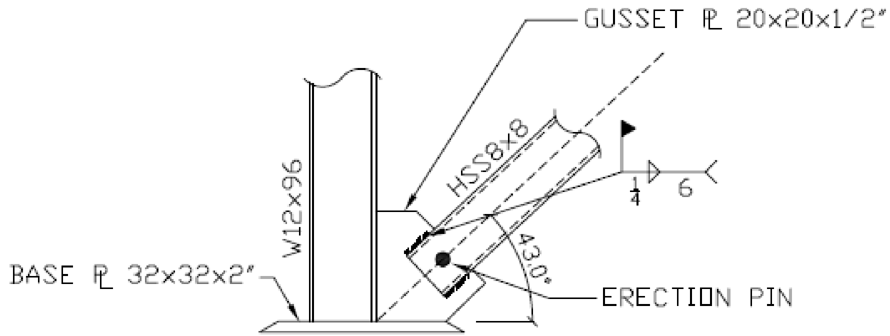
C
S2

ELEVATION

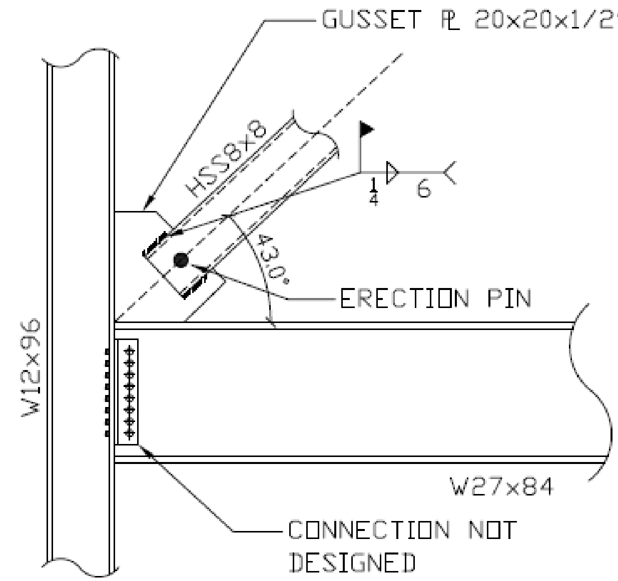
SCALE: NTS

NOTES:

1. REV. DENOTES THE MIRROR IMAGE OF CONNECTION.
2. ALL BRACES ARE 3/8" THICK



A
S3
CONNECTION
SCALE: NTS



B
S3
CONNECTION
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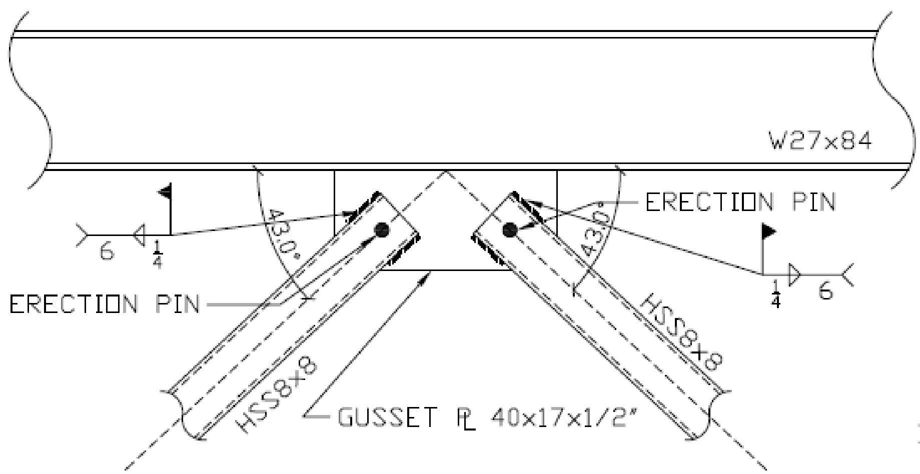
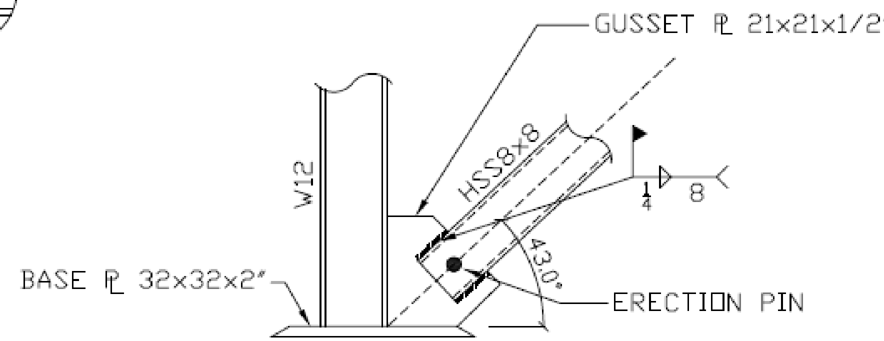
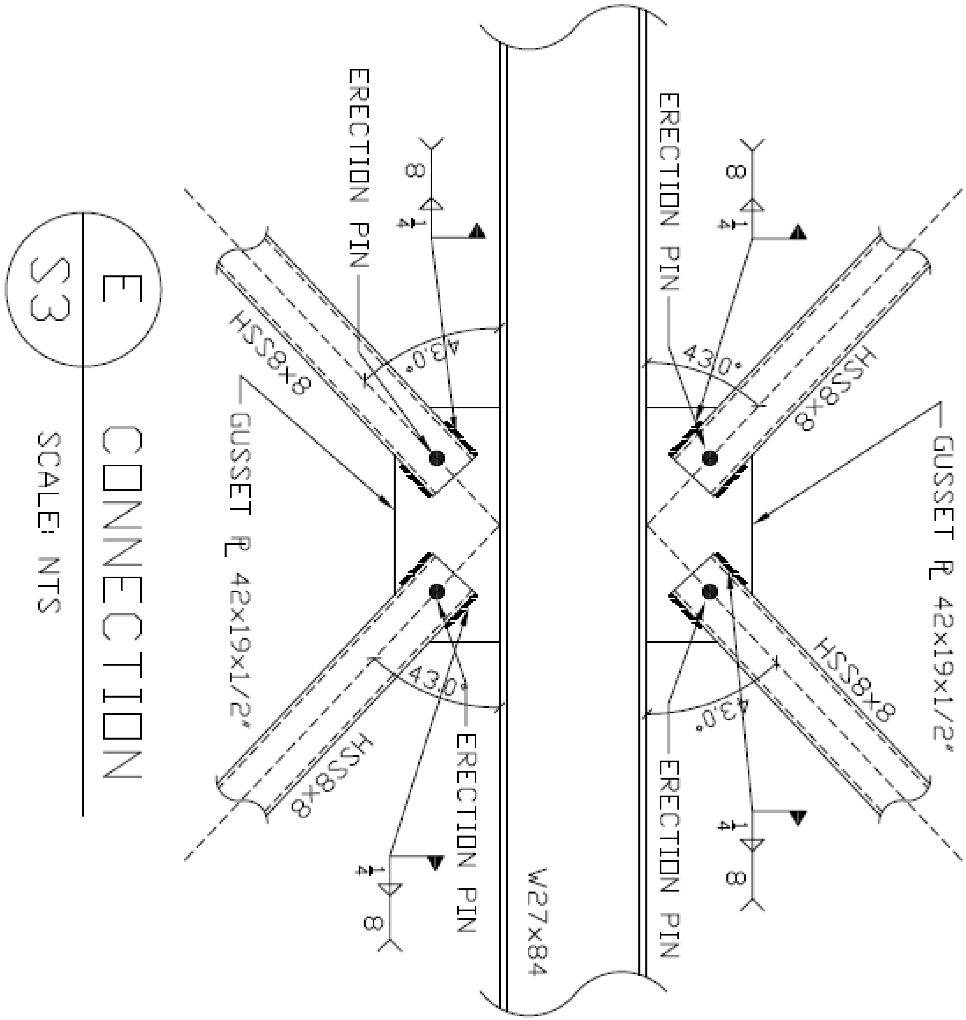
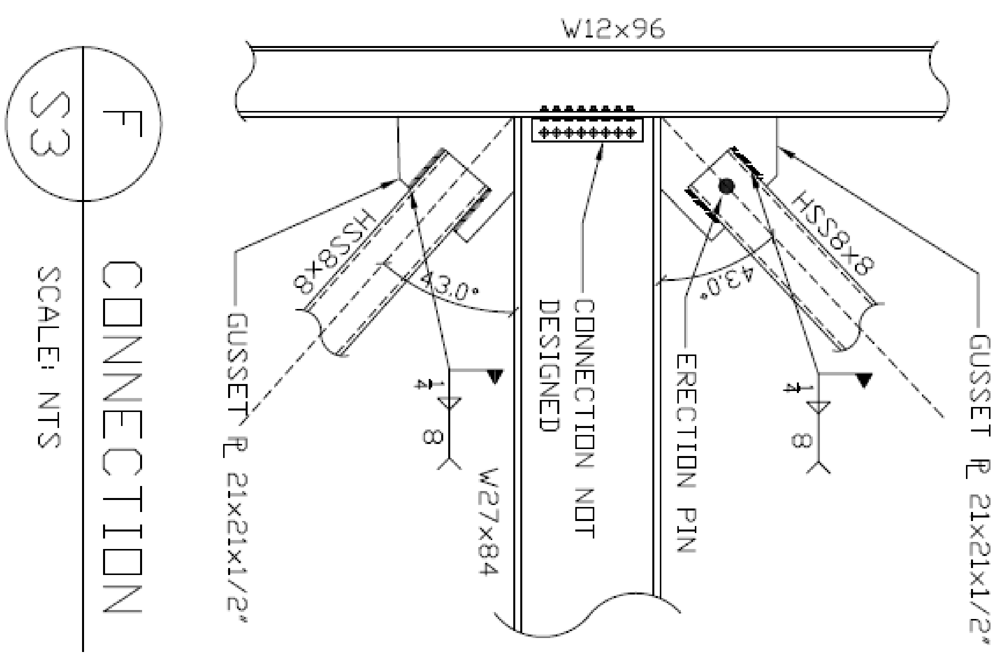


C
S3
CONNECTION
SCALE: NTS



D
S3
CONNECTION
SCALE: NTS


ERECTION PIN
HSS8x8
43.0°
GUSSET PL 42x19x1/2"
W27x84
E
S3
CONNECTION
SCALE: NTS


W12x96
CONNECTION NOT DESIGNED
ERECTION PIN
HSS8x8
43.0°
GUSSET PL 21x21x1/2"
W27x84
F
S3
CONNECTION
SCALE: NTS

CONCLUSIONS

The proposed two-story expansion of 329 Innovation Boulevard required a redesign of the structural members. The height increase proved to greatly affect the wind loads applied to the building. This brought on the alteration of the lateral resisting system. The previous system of moment frames would have to be redesigned to withstand these new loads, and would in most cases involve more elaborate moment connections. Since moment connections are costly and time consuming to design, an alternate resisting system was explored.

Chevron braced frames involving HSS shapes was implemented into the expansion of 329. Six frames were designed along the central bay of the building. The member sizes range from HSS6x6x3/8 to HSS9x9x3/8. Architectural aspects were taken into consideration, and the position of the frames was primarily dictated by not wanting to obstruct the façade. The lateral system yielded extremely small deflections, but the members were unable to be reduced in size due to the fact that the size was controlled by strength. This causes the building to be extremely rigid, which is not a bad thing, but it may not be the most efficient system.

Many changes occurred in the gravity system of the building. This was due to the changes in lateral system and floor system. The usage of a non-composite system caused the beams and girders to increase in size, while the columns were able to be reduced in size. The typical beam sizes increased from W18x35 beams and W24x55 girders to W21x44 beams and W24x68 girders. A price analysis was performed and it can be concluded that the additional cost due to an increase in member sizes does not surpass the cost of shear studs. The deeper beams and girders do mean that the finished floor to finished ceiling may be affected. However, I feel that since top of steel to top of steel is 14', there is plenty of room for any possible mechanical equipment involved.

The columns decreased in size. They were typically W12x96s for the first two floors and spliced to W12x65s for the remaining two. The columns also got as large as W12x190s. This was due to the fact that they were utilized to resist large moments in the moment frame system. The new system of braced frame allowed for a reduction of size due to the interaction between brace and column. The gravity columns were all able to be W10s of numerous sizes ranging from W10x33 to W10x68. The columns in the braced frames were required to be larger than the gravity members, due to the additive moments. The largest columns were located at the corners of the "L" frames.

The consisted of a W12x79 spanning the first three floors, and a W10x49 spans the remaining.

Overall, if an expansion was proposed, this redesign is time and cost saving. It involved the redesign of the lateral resisting system and the gravity members. The time it would take to redesign the moment connections and the cost of them would be much greater than the time and money involved in this redesign. Perhaps the lateral system could be less rigid to make it even more efficient, but this redesign allows for a six-story office building to be designed without starting from square one, which would occur if moment connections and frames were continued.