The First Albany Building

677 Broadway
Albany, NY

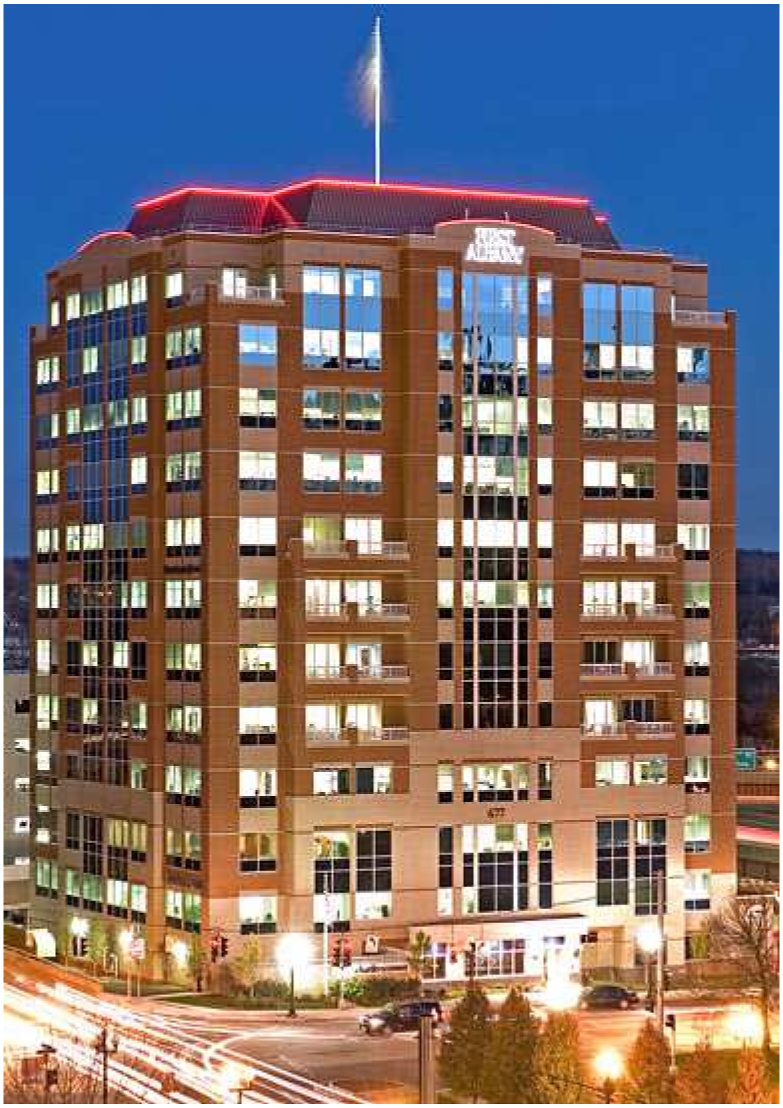

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TABLE OF CONTENTS

Section 1 - EXECUTIVE SUMMARY

In this technical report of The First Albany Building alternative floor systems are investigated. Portions of the structure were analyzed and redesigned and then compared to one another. Comparisons included self-weight, system depth, construction, fire ratings and estimated costs.

The existing system utilizes partial composite beam action and is quite adequate to handle the design parameters. Fewer shear stud connectors are required at the cost of having the use larger structural steel sections, which could be a factor due the variations in the steel market prices. If steel prices are forecast to be lower, larger shapes and less stud connectors would be a better option due to the labor required in installing shear stud connectors. Overall, this system is a good solution given that there are no height restrictions affecting the building and that there is a desire for a short construction period. It is a balanced solution when considering materials and labor.

The three other floor systems explored by this report are:

-Full Composite Beam Action,
-Open Web Steel Joists supported by Wide Flange Girders
-Two Way Reinforced Concrete Flat Plate

A structural steel floor system utilizing full composite action reduces the weight and mass from the existing system and saves a few inches on the total depth. From the portions of the structure analyzed, the use of full composite action reduces the tonnage of structural steel by 33%. However full composite action dramatically increases the number of shear stud connectors requires (up by ~130%). In the right market conditions, this could lead to significant savings on structural steel. The reduced weight of this system would also create savings in other parts of the building in the form of reduced column sizes and perhaps a lighter foundation. If steel prices are low, it becomes a more attractive solution. This system will be studied further beyond this report to increase the benefits and reduce the disadvantages.

Open web joists are light-weight and inexpensive. In the portions of the structure where wide flange shape steel beams were replaced with open web joists; minimal gains (if any at all) were attained. In floors 3-8, where live loads total only 70 pounds per square foot, significant savings were realized. Joist depths could be limited to 18 inches with a 48 inch spacing. In floors 2 & 9-12, live loads are significantly higher; 125 pounds per square foot. Limiting joists to a depth of 18 inches created a system heavier than the existing. When depth restrictions are lessened to 24 inches the system becomes much more lightweight. Further investigation will determine the actual viability of this system when compared to other building systems.

A two-way reinforced concrete flat plate floor system works very well for the portions of the structure analyzed in this report. The total structural depth is only 11”; slightly less than half of the existing composite steel floor system. This could either decrease the overall height of the structure, allow for an added story at the same height, or for higher ceiling heights for more attractive rental space. Labor costs are high compared to the other systems analyzed in this report and the pace of construction may be slowed as well (when compared to structural steel). Considering the added thermal mass in a colder climate, low seismic requirements, and availability of material (3 concrete plants located in the area); a two way flat plate is a very good alternative. Further investigation will refine this design further.

Section 2 - INTRODUCTION

Building Description

The First Albany Building is a 12 story, 180,000 square feet structure designed to house mixed-use office space and condominiums. Dimensions of the building are roughly 115' x 135' and the overall height is about 172' to the mechanical penthouse roof. The first floor is at grade and the building has no basement. The exterior of the building is mostly brick veneer.

The foundation system consists of a mixture of H piles, pile caps, and grade beams to support the structure. The first floor is supported by a 6" concrete slab on grade with the remaining 11 stories (and roof) comprised of a semi-regular grid of simply supported beams and girders. H-piles had to be driven to practical refusal to fully support the building. Six test piles were driven and their capacities tested to verify calculated load capacities of all the piles. Design capacity of each pile was 120 tons.

Gravity loads are resisted by a 4.5" reinforced concrete slab utilizing composite deck design. The floor slab is supported by a semi-regular grid of simply supported beams and girders. Composite beam and composite deck design (partial composite action) was incorporated in to the floor system design and bays are typically about 25'x25' with some variations. Sizes of floor members range between W12 and W18 shapes with varying numbers of shear stud connectors on each member. Column lines transfer loads directly to the ground through pile caps and to the piles themselves. The piles are laid out symmetrically under each cap because there are no eccentricities associated with column loads.

Lateral forces are resisted by sets of concentrically braced steel frames around the core of the building. Bracing patterns include "K", inverted "K", and standard diagonal. The braced frames each act like a vertical, cantilevered truss. There are 2 wide frames in the east-west direction and 3 narrower frames in the north-south direction.

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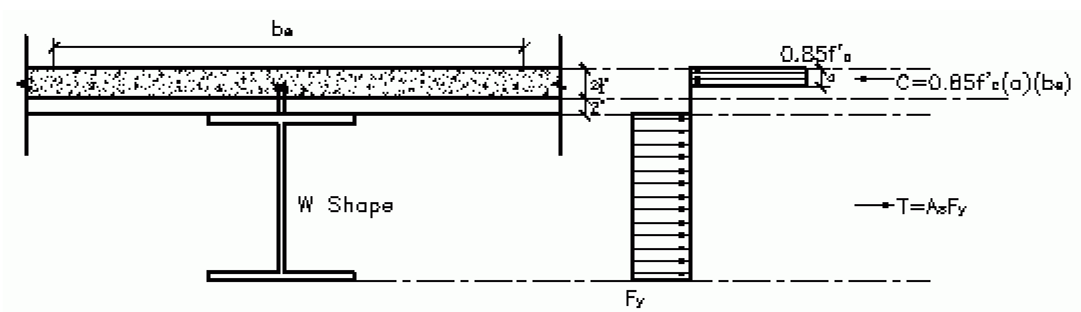
In This Report

Three different structural floor systems will be compared to the existing system.

Full Composite Beam Action:

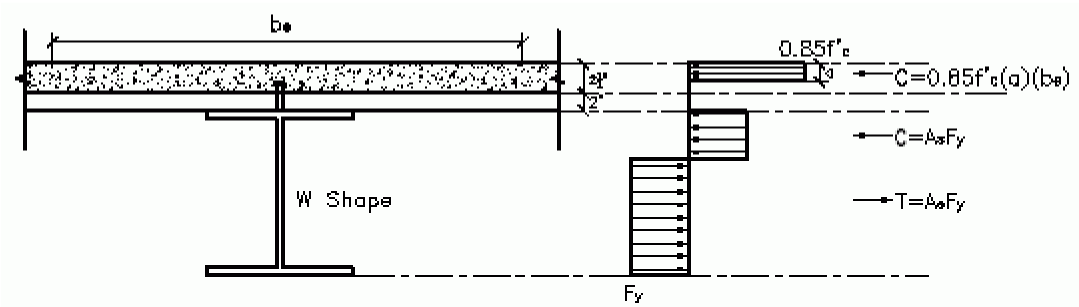
This system utilizes 'full composite action' rather than 'partial composite action' as in the existing system. This allows the concrete floor slab to play a more significant role in the Compression = Tension equation for beam design. All of the compressive forces are taken by the concrete slab while all the tensile forces are carried by the structural steel shape.



Configuration Stress Force

Figure 2.1 – Full Composite Beam Action

Partial composite action happens when the shear stud connectors only transfer a portion of the compressive forces from the structural shape to the concrete slab. A quick spot check easily determines that full composite action was not used in the existing design, $A_s*F_y > \Sigma Q_n$ (appendix C).

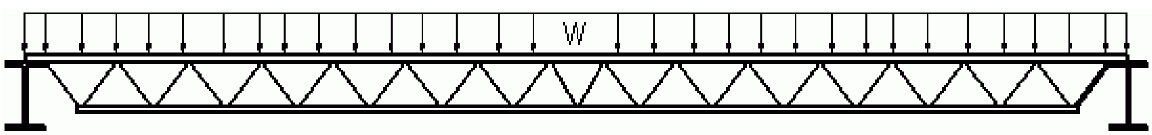


Configuration Stress Force

Figure 2.2 – Partial Composite Beam Action

Open Web Joist System:

This system uses Open Web Joists rather than structural wide flange shapes to carry the gravity loads. The same column sizes and locations are used. Wide flange shape girders are used to support the joists.



W-Shape Girder Joist W-Shape Girder

Figure 2.3 – Typical Joist Elevation

Flat Plate Concrete Floor System:

The entire floor system is converted from structural steel to concrete. Column locations are left unchanged.

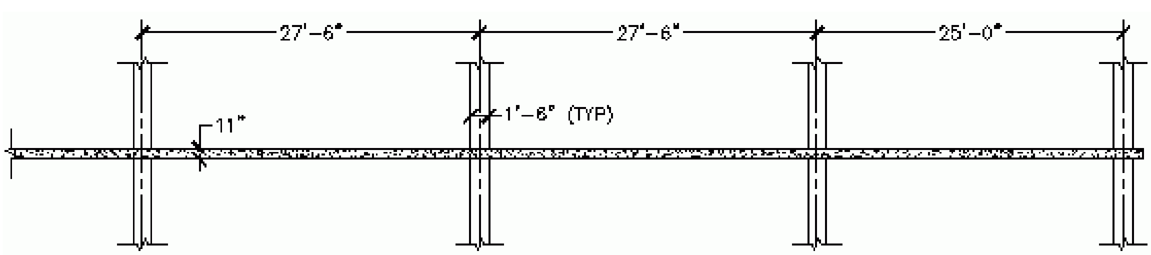


Figure 2.4 – Typical Section

Section 3 - APPLICABLE BUILDING CODES

New York State Building Code 2002
New York State Energy Conservation Code
"Manual of Steel Construction" AISC ASD 9th Ed.
"Building Code Requirements for Structural Concrete" ACI 318-02

Gravity Live Loads

	Loading Used by Engineer	Current Required Loading	
Office Space (2-8)	50 psf +20 psf Partition Allowance	50 psf +15	(ASCE 7-05, Table 4.1) Partition Allowance
Office Space (9-12) +Computer Use	100 psf +15 psf Access Flooring	100 psf	(ASCE 7-05 Table 4.1)
Office Space File Storage	125 psf	125 psf	(ASCE 7-05 Table 4.1)
Stairways	100 psf	100 psf	(ASCE 7-05 Table 4.1)
Roof Snow Load	65 psf	65 psf	(NYS Bldg Code)
Balconies	100 psf	100 psf	(ASCE 7-05 Table 4.1)
Roof	20 psf	20 psf	(ASCE 7-05 Table 4.1)
Restaurants	100 psf	100 psf	(ASCE 7-05 Table 4.1)

Dead Loads

Loading Breakdown	
MEP	15 psf
Structural Steel (Columns Only)	4 psf
Structural Steel (All Other)	10 psf
Lightweight Concrete Slab	34 psf
Deck	2 psf
Finishes	5 psf
Misc	10 psf
Total	80 psf

Live Load Reductions

Reduction Factor (RF) $=0.25+15/\sqrt{(K_{LL}*A_T)}$
For structural members supporting 1 floor; RF ≥ 0.5
For structural members supporting 2 or more floors; RF ≥ 0.4

Section 4.1 - EXISTING SYSTEM

The existing system utilizes partial composite beam action to resist gravity loads. Member sizes and required shear stud connectors are shown on a typical floor plan.

Figure 4.1.1 – Typical Floor Plan

Even though there is a significant change in live loads between floors 1, 2, 9-12 and 3-8; the same member sizes are used in a plan location on every floor. I believe that the reason is for this is that repetitive steel pieces do save money on fabrication. Steel prices and forecasts at the time of design could have also influenced them to select heavier sections and save on the cost of shear stud installation (mostly labor).

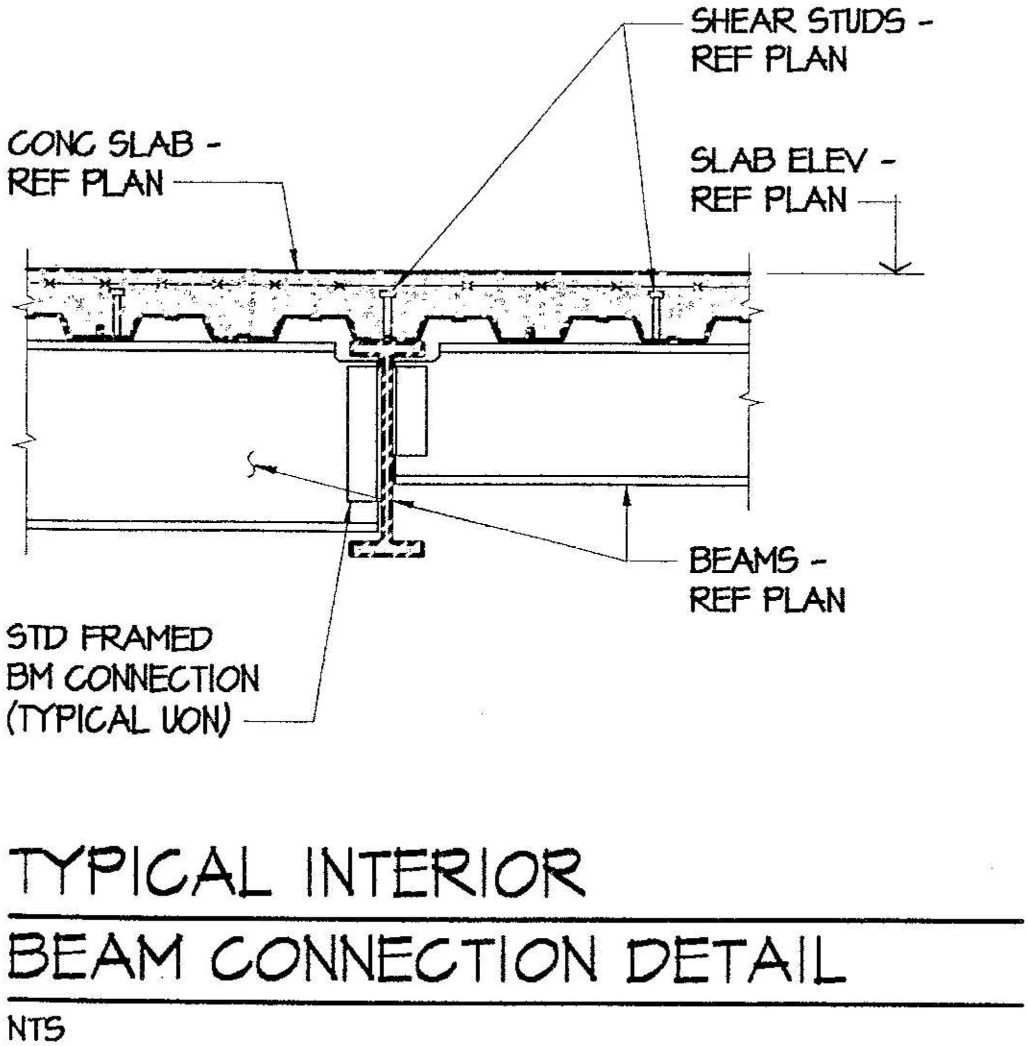
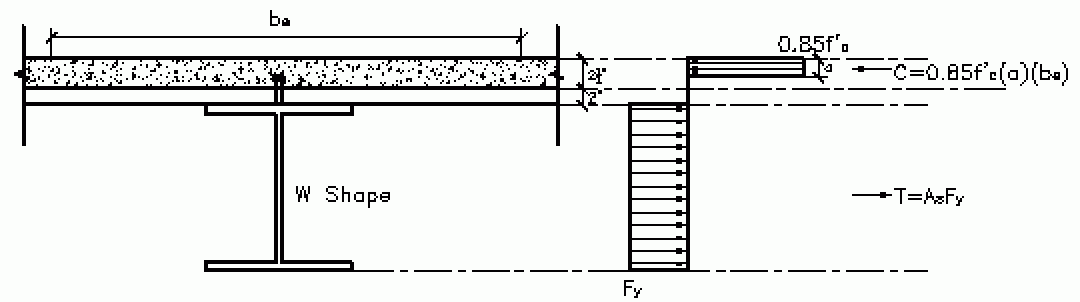


Figure 4.1.2 – Typical Section

Section 4.2 - FULL COMPOSITE ACTION

In this system, the number of shear studs on each beam and girder will be increased so full composite action can be attained.

Total Stud Resistance Force = Tensile Force = Compressive Force
ΣQ_n = (# of studs)(force per stud) $T = A_s F_y$ $C = 0.85 f'_c (a)(b_e)$



Configuration Stress Force

Figure 4.2.1

Checking typical beams and girders yields the following data. The first line (or two) is the original structural member; the last line of each chart section is the selected replacement. The full supporting data and calculation sheet can be found in Appendix D.

Shape	(AsFy) ΣQn	ΦMn Φ=0.9 in-K	ΦMn Φ=0.9 FT-K	ΦVn Φ=1.0 K	AISC Tab3-21 3/4"dia Qn (K)	Stud # req'd	Mu wl^2 / 8	Vu wl / 2	ΦVn>Vu & ΦMn>Mu ?
Column Line C to D (Beams)									
16x 26	384	4032	336.0	117.8	17.2	45	151.91	22.1	OK
12x 19	279	2532	211.0	86.0	17.2	32	151.11	22.0	OK
Column Line A to C & F to H (Beams)									
12x 14	208	1858	154.9	71.4	17.2	24	75.70	15.5	OK
10x 12	177	1432	119.3	59.2	17.2	21	75.58	15.5	OK
Column Line E.1 to F (Beams)									
12x 19	279	2505	208.7	80.5	17.2	32	101.16	18.0	OK
12x 14	208	1871	156.0	71.4	17.2	24	100.78	17.9	OK
Column Line D to D.6 (Beams)									
14x 22	325	3129	260.7	94.5	17.2	38	136.36	21.8	OK
12x 19	279	2520	210.0	86.0	17.2	32	136.08	21.8	OK
12x 16	236	2128	177.3	79.2	17.2	27	135.80	21.7	OK
Column Line C (Short Girders)									
18x 46	690	7574	631.2	195.5	21.2	65	327.37	52.4	OK
16x 31	457	4747	395.6	131.2	21.2	43	325.96	52.2	OK
Column Line C (Long Girders)									
18x 60	880	9529	794.1	226.6	21.2	83	405.08	58.9	OK
16x 45	665	6802	566.8	166.6	21.2	63	403.38	58.7	OK

Table 4.2.1 – Strength Checks

Shape	L ft	Ixx Steel	Y1 (in)	Y2 (in)	Low Bound ILB	DL Δ	LL Δ	L/360 inch	Deflection limits
Column Line C to D (Beams)									
16x 26	27.5	301	0	3.816	822	0.40	0.46	0.92	OK
12x 19	27.5	130	0	4.004	583	0.91	0.65	0.92	OK
Column Line A to C & F to H (Beams)									
12x 14	19.5	88.6	0	3.977	298	0.33	0.32	0.65	OK
10x 12	19.5	53.8	0	4.055	200	0.54	0.48	0.65	OK
Column Line E.1 to F (Beams)									
12x 19	22.5	130	0	3.893	414	0.41	0.41	0.75	OK
12x 14	22.5	88.6	0	4.047	300	0.59	0.57	0.75	OK
Column Line D to D.6 (Beams)									
14x 22	25.0	199	0	3.864	573	0.44	0.50	0.83	OK
12x 19	25.0	130	0	3.954	410	0.67	0.69	0.83	OK
12x 16	25.0	103	0	4.038	341	0.84	0.83	0.83	OK
Column Line C (Short Girders)									
18x 46	25.0	712	0	3.147	1730	0.36	0.49	0.83	OK
16x 31	25.0	375	0	3.605	984	0.68	0.87	0.83	OK
Column Line C (Long Girders)									
18x 60	27.5	984	0	2.931	2335	0.41	0.56	0.92	OK
16x 45	27.5	586	0	3.315	1444	0.67	0.90	0.92	OK

Table 4.2.2 – Deflection Checks

Utilizing full composite action results in the following savings and increases. Gross structural steel weight of members analyzed and replaced decreases by 33%. However the number of shear stud connectors increases by 132%. I believe that the members selected are a good representative sample for the entire structure (except for lateral load resisting members).

Shape	L ft	# of pieces	LF	Weight existing	Studs per existing	Total Studs existing	Weight new	Studs per new	Total Studs new	% saving by weight	% increase stud #
Column Line C to D (Beams)											
16x 26	27.5	14	385	10010	10	140					
12x 19	27.5						7315	32	448		
Column Line A to C & F to H (Beams)											
12x 14	19.5	14	273	3822	10	140					
10x 12	19.5						3276	21	294		
Column Line E.1 to F (Beams)											
12x 19	22.5	14	315	5985	10	140					
12x 14	22.5						4410	24	336		
Column Line D to D.6 (Beams)											
14x 22	25.0	3	75	1650	10	30					
12x 19	25.0	3	75	1425	15	45					
12x 16	25.0						2400	27	162		
Column Line C (Girders)											
18x 46	25.0	2	50	2300	25	50					
16x 31	25.0						1550	43	86		
18x 60	27.5	2	55	3300	40	80					
16x 45	27.5						2475	63	126		
			Totals	28492		625	18951		1452	33%	132%

Table 4.2.3 – Savings & Increases

Section 4.3 - OPEN-WEB JOIST SYSTEM

In this system dead loads are re-calculated into linear loads (excluding structural steel loads) and joists are selected from Nicholas J. Bouras, Inc Steel Joist Catalog. Linear loads are compared to allowable loads (per joist) and joists are selected based on strength and deflection.

	Floors 2, 9-12				Floors 3-8					
	Live Load = 125 PSF				Live Load = 70 PSF					
	Dead Load = 66 PSF				Dead Load = 66 PSF					
	Total Load = 191 PSF				Total Load = 136 PSF					
	Spacing				Spacing					
	2.5'		2'		4'		3'		2.5'	
Span (ft)	Total = 478 plf Live = 313 plf	Self Weight (plf/psf)	382 plf 250 plf	Self Weight (plf/psf)	Total = 544 plf Live = 280 plf	Self Weight (plf/psf)	408 plf 210 plf	Self Weight (plf/psf)	340 plf 175 plf	Self Weight (plf/psf)
15.0	14K1	5.2 / 2.1	12K1	5.0 / 2.5	14K3	6.0 / 1.5	12K1	5.0 / 1.7	10K1	5.0 / 2.0
	12K3	5.7 / 2.3								
17.5	16K3	6.3 / 2.5	16K2	5.5 / 2.8	18K3	6.6 / 1.7	16K2	5.5 / 1.8	14K1	5.2 / 2.1
(18)	14K4	6.7 / 2.7	14K3	6.0 / 3.0	16K4	7.0 / 1.8	14K3	6.0 / 2.0	12K3	5.7 / 2.3
	12K5	7.1 / 2.8	12K5	7.1 / 3.6	14K6	7.7 / 1.9	12K5	7.1 / 2.4		
19.5	20K3	6.7 / 2.7	16K3	6.3 / 3.2	18K4	7.2 / 1.8	18K3	6.6 / 2.2	16K2	5.5 / 2.2
(20)	16K4	7.0 / 2.8	14K4	6.7 / 3.4	16K5	7.5 / 1.9	14K4	6.7 / 2.2	14K3	6.0 / 2.4
	12K5	7.1 / 2.8	12K5	7.1 / 3.6					12K5	7.1 / 2.8
22.5	22K4	8.0 / 3.2	18K4	7.2 / 3.6	22K5	8.8 / 2.2	18K4	7.2 / 2.4	20K3	6.7 / 2.7
(23)	20K5	8.2 / 3.3	16K5	7.5 / 3.8	20K6	8.9 / 2.2	16K6	8.1 / 2.7	16K4	7.0 / 2.8
	18K6	8.5 / 3.4	14K6	7.7 / 3.9	18K7	9.0 / 2.3			14K6	7.7 / 3.1
	16K7	8.6 / 3.4								
25.0	18K7	9.0 / 3.6	20K4	7.6 / 3.8	22K7	9.7 / 2.4	22K4	8.0 / 2.7	18K4	7.2 / 2.9
			18K5	7.7 / 3.9	18K9	10.2/2.6	20K5	8.2 / 2.7	16K5	7.5 / 3.0
			16K7	8.6 / 4.3			18K6	8.5 / 2.8		
							16K7	8.6 / 2.9		
27.5	24K7	10.1/4.0	22K5	8.8 / 4.4	20K9	10.8/2.7	22K6	9.2 / 3.1	20K5	8.2 / 3.3
(28)	20K9	10.8/4.3	20K7	9.3 / 4.7	18K10	11.7/2.9	20K7	9.3 / 3.1	18K7	9.0 / 3.6
	18K10	11.7/4.7	16K9	10.0/5.0			18K10	11.7/3.9	16K9	10.0/4.0

Table 4.3.1 – Open Web Joist Selection

Several options are available for most bays (joist spacing and type). Balancing depth verses weight of a member will help determine spacing and what joist type to choose.

Joists are selected based on a maximum depth of 18" and spacing to maximize economy. For example for a 15' bay, a 10K1 @ 2.5' O.C. equals 2 pounds per square foot supported and a 14K3 @ 4' O.C. equals 1.5 pounds per square foot supported. In a case like that, a deeper joist at a larger spacing is selected.

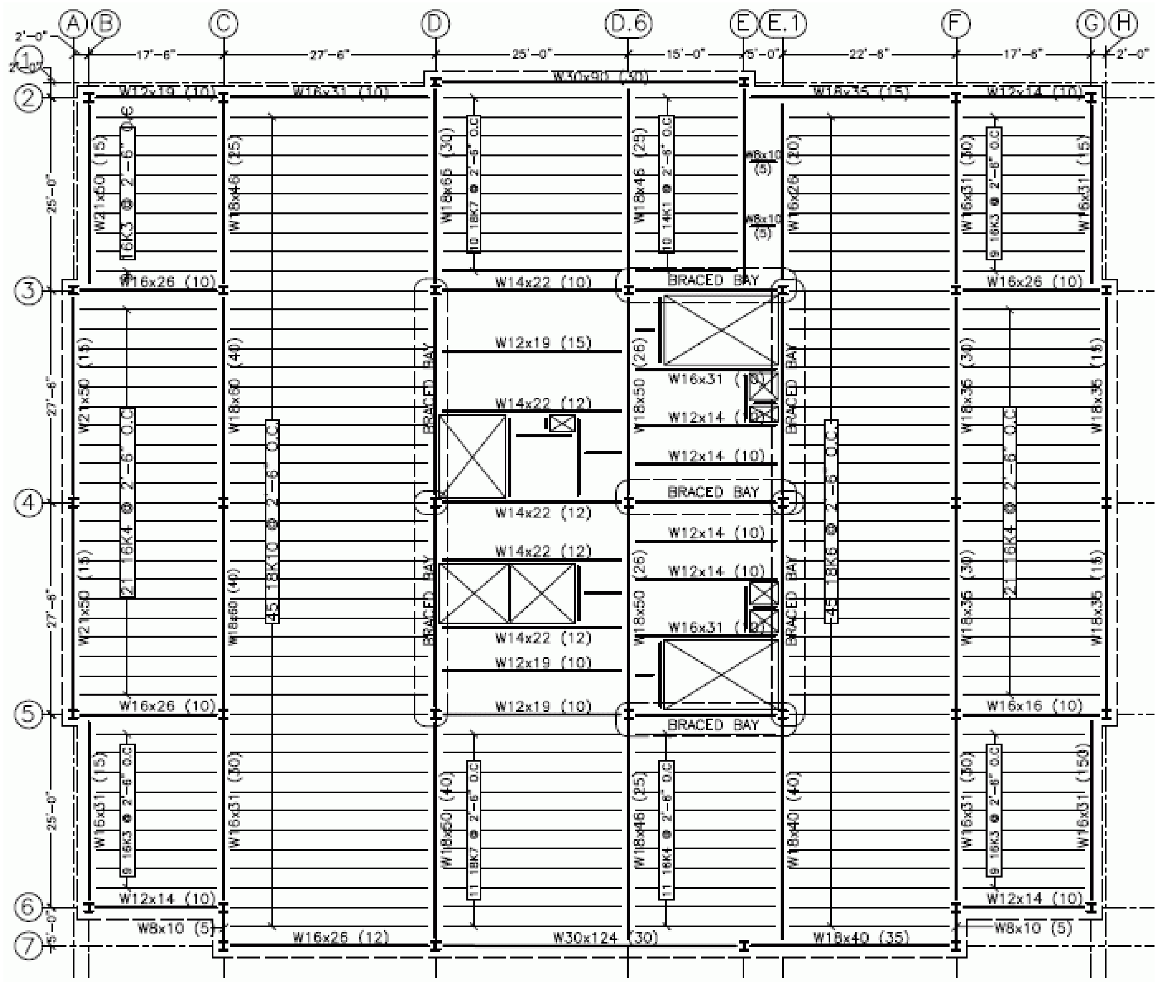


Figure 4.3.1 - Typical Joist Layout – Floors 2, 9 - 12

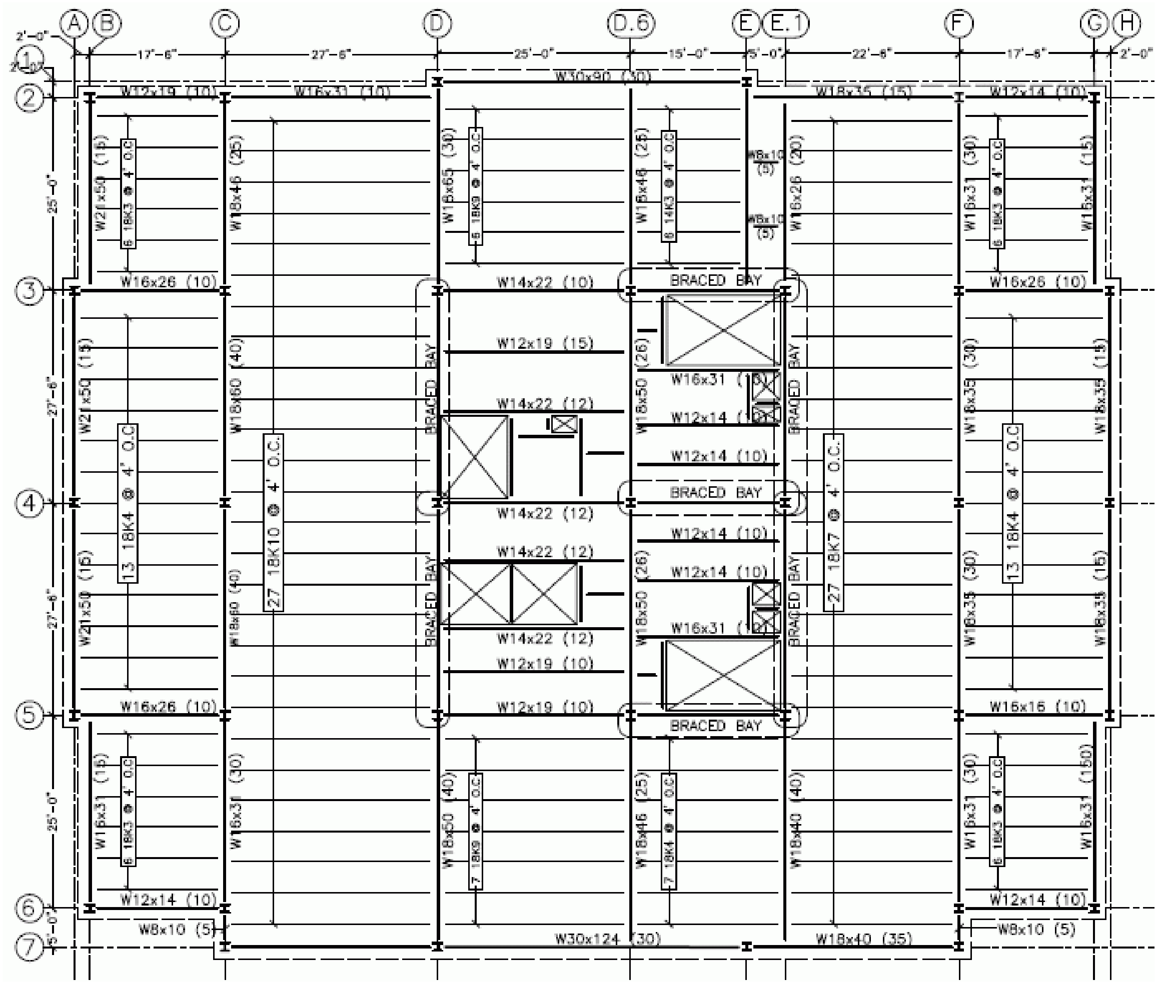


Figure 4.3.2 - Typical Joist Layout – Floors 3 - 8

Joist Weight						
	Joist	Span (ft)	Weight (plf)	Pieces	Total (LF)	Total (lbs)
Floor 2,9-12	14K1	15.0	5.2	10	150.0	780.0
	16K3	17.5	6.3	36	630.0	3969.0
	16K4	19.5	7.0	42	819.0	5733.0
	16K4	20.0	7.0	11	220.0	1540.0
	18K6	22.5	8.5	45	1012.5	8606.3
	18K7	25.0	9.0	21	525.0	4725.0
	18K10	27.5	11.7	45	1237.5	14478.8
					Total joist weight per floor	39832.0
Floor 3-8	14K3	15.0	6.0	6	90.0	540.0
	18K3	17.5	6.6	24	420.0	2772.0
	18K4	19.5	7.2	26	507.0	3650.4
	18K4	20.0	7.2	7	140.0	1008.0
	18K7	22.5	9.0	27	607.5	5467.5
	18K9	25.0	10.2	13	325.0	3315.0
	18K10	27.5	11.7	27	742.5	8687.3
					Total joist weight per floor	25440.2

Table 4.3.2 – Floor Joist Weight

Replaced Beam Weight						
	Beam W	Span (ft)	Weight (plf)	Pieces	Total (LF)	Total (lbs)
Floor 2-12	8x10	6.75	10	3	20.25	202.5
	8x10	7.50	10	9	67.50	675.0
	8x10	8.33	10	11	91.63	916.3
	8x10	15.00	10	3	45.00	450.0
	12x14	17.50	14	8	140.00	1960.0
	12x14	19.50	14	14	273.00	3822.0
	12x14	20.00	14	3	60.00	840.0
	12x19	22.50	19	11	247.50	4702.5
	16x26	22.50	26	3	67.50	1755.0
	12x19	25.00	19	3	75.00	1425.0
	16x26	27.50	26	14	385.00	10010.0
	14x22	25.00	22	3	75.00	1650.0
					Total replaced beam weight per floor	28408.3

Table 4.3.3 – Existing Floor Beam Weight

From the previous tables you can see the potential weight savings. An open web joist system appears to be a good alternative for the mid-level floors only.

Section 4.4 - TWO WAY FLAT PLATE CONCRETE FLOOR SYSTEM

In this system a flat reinforced concrete slab is used to carry gravity loads. In many cases with type of system, punching shear and deflection controls the slab thickness. ACI 9.5.3 outlines minimum slab thicknesses to eliminate the need to check deflections. Drop panels and edge beams have been avoided for this system so minimum thickness is determined by t > Ln / 30. The largest value for Ln is 27.5 feet. From this an initial thickness of 11 inches is chosen. Minimum compressive strength of concrete (f'c) is assumed to be 5000 pounds per square inch and yield strength of reinforcing bars to be 60 ksi. Checking punching shear shows that for the majority of the columns, no punching shear reinforcement is required. Where it is, a worst case scenario shows that ACI code limitations on punching shear strength are sufficiently large so that reinforcing can be used to bridge the gap. Initial column sizes are 18 inches square.

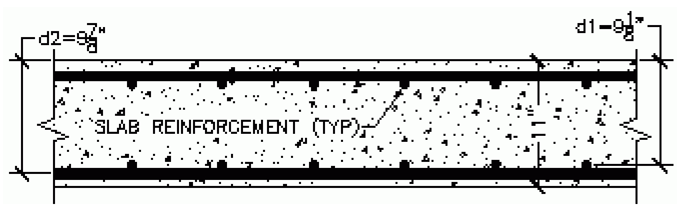


Figure 4.4.1 - Typical Slab Section

Column	Fact.Load 1.2D+1.6L psf	Vu K	(1) ΦVc K	(2) ΦVc K	(3) ΦVc K	ΦVc>Vu ?	(4) ΦVn Limit K	(5) ΦVc K	(6) Req'd ΦVs K
Corner	364.9	44.7	122.2	183.3	149.4	OK			
	348.9	42.8	122.2	183.3	149.4	OK			
	276.9	33.9	122.2	183.3	149.4	OK			
Edge	364.9	97.3	130.6	195.9	197.8	OK			
	348.9	93.0	130.6	195.9	197.8	OK			
	276.9	73.8	130.6	195.9	197.8	OK			
Interior	364.9	223.2	210.0	315.0	281.6	NG	315.0	105.0	118.2
	348.9	213.4	210.0	315.0	281.6	NG	315.0	105.0	108.4
	276.9	169.4	210.0	315.0	281.6	OK			
Interior	364.9	274.1	210.0	315.0	281.6	NG	315.0	105.0	169.1
(worst	348.9	262.1	210.0	315.0	281.6	NG	315.0	105.0	157.1
case)	276.9	208.0	210.0	315.0	281.6	OK			

Table 4.4.1 – Punching Shear

(The full supporting data and punching shear calculations can be found in appendix E)

Adding #3 double stirrups spaced at 4" placed as shown in figure 4.4.2 provides a shear reinforcement strength of 180.7 K.

$\Phi V_s = \Phi(A_v)(f_y)(d) / s = 0.75(1.76)(60)(9.125) / (4) = 180.7\ K$

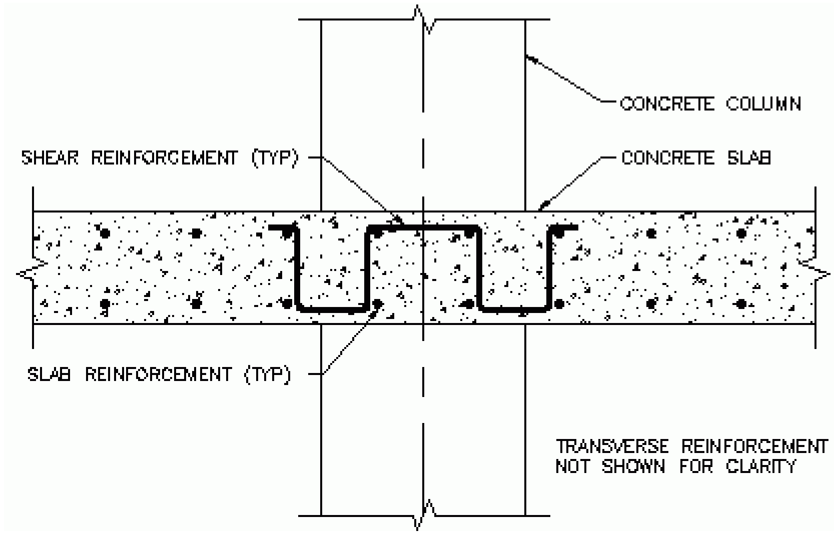


Figure 4.4.2 - Typical Shear Reinforcement Detail

To determine 'd', #6 reinforcing bars are assumed to be used as the flexural reinforcement in the slab. Working backward from a ductility check, a maximum steel ratio of 0.0208 (As = 2.33 in² per foot width) is determined. This provides a maximum moment capacity (ΦMn) of 81.3 ft-k per foot width.

$$0.005 = \frac{0.003(d-c)}{c} = \frac{0.003(9.125-c)}{c} \quad c = 3.42\text{" max}$$

$a = \beta_1(c) = 2.74$ (β_1=0.8 for f'c=5 ksi)

AsFy = 0.85(f'c)(a)(b)   As(60) = 0.85(5)(2.74)(12)   As=2.33 in² / ft max

ΦMn = ΦAsFy(d-a/2) = 0.9(2.33)(60)(9.125-2.74/2) = 975 in-k = 81.3 ft-k per ft width

Taking the Direct Design approach as outlined in ACI 318-08, the total static moment (Mo) for the largest bay is 950 ft-k (w_u*L²/8). Distributed as per ACI 13.6.3.2, the largest factor multiplied to Mo is 0.7 (flat plate, no edge beams). If the column strip for a 27.5' square bay is 13.75' and a minimum of 8.75' due to aspect ratios, the maximum design moment becomes 76 ft-k, which is less than maximum capacity governed by ductile failure (Es>0.005). Five #6 bars per foot equals a steel area of 2.21 in² (per foot) and a ΦMn of 77.8 ft-k per ft width.

AsFy = 0.85(f'c)(a)(b)   2.21(60) = 0.85(5)(a)(12)   a=2.6 in

ΦMn = ΦAsFy(d-a/2) = 0.9(2.21)(60)(9.125-2.6/2) = 934 in-k = 77.8 ft-k per ft width

From these calculations, a flat plate system with a slab thickness of 11 inches and 18 inch square columns can be fully designed for the building. The difference between the punching shear limit and factored shear means that column sizes could be reduced slightly.

Section 5 - CONCLUSIONS

Pro-Con Analysis: Existing Steel Floor System

The existing system utilizes partial composite beam action and is adequate to resist the design needs. Less shear stud connectors are required at the cost of having the use larger structural steel sections, which could be a factor due the variations in the steel market prices. If steel prices are forecasted to be low (relatively speaking), larger shapes and less stud connectors would be the better option due to the labor required in installing shear stud connectors. Steel erection is quicker than forming, placing, and curing concrete and the metal decking used acts as stay in place formwork for the floor slab. Even though partial composite action provides a medium weight structure, the depth of the system reaches 23 inches in places, making for much wasted space in the ceiling cavity that needs to be heated and cooled. A structural steel system also requires the addition of fire-protection which adds cost. Overall, this system is a good solution given that there aren't any height restrictions affecting the building and there is a desire for a short construction period. Even if steel prices are not low, it is a balanced solution when considering materials and labor.

Pro-Con Analysis: Full Composite Beam Action Steel Floor System

A structural steel floor system utilizing full composite action reduces the weight and mass from the existing system and saves a few inches on the depth. From the portions of the structure analyzed, full composite action reduces the tonnage of structural steel by 33%. However it increases the number of shear stud connectors requires (up by ~130%). In the right market conditions, this could lead to significant savings on structural steel. The reduced weight and mass of this system would also create savings in other areas of the building in the form of reduced column sizes needed and perhaps a lighter foundation. Piles could be driven to a shallower depth saving money on materials and installation since contractors pay per foot for the pile and per foot for piling driving/installation. Even though full composite action provides a relatively light weight structure, the depth of the system still reaches 20 inches in places, making for much wasted space in the ceiling cavity that needs to be heated and cooled. This structural steel system also requires the addition of fire-protection which adds cost. Overall, this system is a very good solution given that there aren't any height restrictions affecting the building and there is a desire for speedy construction. If steel prices are low, it becomes an even better solution.

Pro - Con Analysis: Open Web Joist System

Open web joists are traditionally light-weight and inexpensive. In the portions of the structure where wide flange shape steel beams were replaced with open web joists minimal gains (if any at all) were attained. In floors 3-8, live load totals only 70 pounds per square foot, significant savings were had. Joist depths were able to be limited to 18 inches even with a spacing of 48 inches. In floors 2 & 9-12, live loads are significantly higher; 125 pounds per square foot. Limiting joists to a depth of 18 inches created a system heavier than the existing. If depth restrictions were lessened to 24 inches (allowable) the system becomes much more attractive overall. Increasing the maximum depth to 24 inches could cause problems in maintaining the same floor to ceiling height; however other systems could be run *through* the joists, rather than under them, eliminating the problems and even potentially increasing the floor to ceiling height. Construction of open web joist systems is fast and inexpensive – raise, set, connect, repeat. Portions of the floor system can be assembled on the ground and raised as an entire unit, reducing crane time. Connections are simple and require minimal labor. Fire-protection can present an issue as it's hard to use spray applied protection on thin web members, however intumescent paint could be applied at the end of the fabrication stage before the members arrive on site.

Pro - Con Analysis: Two Way Flat Plate

A two-way flat plate floor system works very well for the portions of the structure analyzed in this report. The total structural depth is only 11"; slightly less than half of the existing composite steel floor system. This could either decrease the overall height of the structure, allow for an added story at the same height, or for higher ceiling heights for more attractive rental space. This system is an efficient design for the First Albany Building; however a concrete floor system would need a different lateral force resistance system than the existing steel braced frames. The additional weight of the concrete system also adds significant mass to the building. This is a benefit considering the thermal mass is dramatically increased, perhaps increasing energy efficiency due to slower temperature swings and the ability of concrete to hold onto heat during the winter season (~4 months of the year). The added mass does increase the seismic loads but the seismic requirements for the area are relatively low. Since concrete provides its own fire-protection, a 2 hour fire rating is attained by providing a minimum clear cover of ¾", and no additional fire-protection is required. Labor costs are high compared to the other systems analyzed in this report due to the extensive use of formwork and placing large quantities of concrete. The pace of construction would be slowed as well (when compared to structural steel). Considering the added thermal mass in a colder climate, low seismic requirements, and availability of material (3 concrete plants located in the area); a two way flat plate is a very good alternative only if allowed a longer construction period.

	Existing	Full Composite Action	Open Web Joists	Two Way Flat Plate
Self Weight (psf)	48	44	40 - 50	138
Depth (in)	23	20	18 - 24	11
Construction Difficulty	Moderate	Moderate	Easy	Difficult
Lateral System Impact	-	No	No	Yes
Vibration	Average	Average	Average	Very Good
Fire Rating (hr)	1 (applied)	1 (applied)	1 (applied)	2 (natural)
Thermal Mass Effect	Moderate- Low	Moderate- Low	Low	High
Possible Alternative	-	Yes	Yes	Yes
Additional Investigation	-	Some	Some	Yes

APPENDIX A – PROJECT TEAM MEMBERS

Owner & Developer

Columbia Development Companies
302 Washington Ave. Ext., Albany, NY 12203
http://www.columbiadev.com/

Architect

HCP Architects
302 Washington Ave. Ext., Albany, NY 12203
http://www.hcpdesign.com/

Construction Manager & General Contractor

BBL Construction Services
302 Washington Ave. Ext., Albany, NY 12203
http://www.bblconstructionservices.com/

Structural Engineers

Stroud, Pence, & Associates LTD
204-A Grayson Road, Virginia Beach, VA 23462
http://www.stroudpence.com/

Site Engineers & Surveyor

Hershberg & Hershberg
18 Locust Street, Albany, NY 12203
http://www.hhershberg.com/

Geotechnical Engineers

Dente Engineering, P.C.
594 Broadway, Watervliet, NY 12189
http://www.dente-engineering.com/

Interior Designer / Architect

Woodward, Connor, Gillies, & Seleman
20 Corporate Woods Blvd, Albany, NY 12211
http://www.wcgsarchitects.com/

APPENDIX B – MATERIAL SPECIFICATIONS

Structural Steel –

Miscellaneous shapes, plates, bars	–	ASTM A36, Fy = 36 ksi
Structural Shapes, W8 and larger	–	ASTM A572, Grade 50, Fy = 50 ksi
Hollow Structural Shapes (HSS)	–	A500, Grade B, Fy = 46 ksi (square and rect.)
	–	ASTM A53, Type E or S, Fy = 35 ksi (round shapes)
Anchor Bolts	–	ASTM A307
	–	ASTM A449 (at braced bays)

Cast-in-place Concrete –

Slab on Grade	–	3500 psi (28 day compressive strength)
Supported Floor Slabs	–	4000 psi, lightweight (115 pcf)
Grade Beams, Pile Caps, Walls	–	4000 psi
Foundation Piers	–	6000 psi
Reinforcing bars	–	ASTM A615, Grade 60, deformed
Welded Reinforcing bars	–	ASTM A706, Grade 60
Welded Wire Fabric	–	ASTM A185 (Sheet type only)

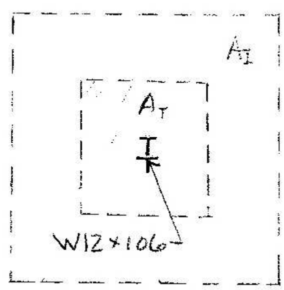
Steel Deck –

Roof Deck	–	1 ½” x 22 Gage Type B Rib Deck
Floor Deck	–	2” x 22 Gage Composite Floor Deck

APPENDIX C – SPOT CHECK CALCULATIONS

COLUMN CHECK @ F-4 LEVEL 1

LL = 70 PSF (2 - 8)
LL = 115 PSF (9 - 12)
SL = 65 PSF (ROOF)
DL = 90 PSF



TRIBUTARY AREA PER FLOOR $A_T = (27.5/2 + 27.5/2)(22.5/2 + 17.5/2)$

$A_T = 550$ SF

INFLUENCE AREA PER FLOOR $A_I = (27.5 + 27.5)(22.5 + 17.5)$

$A_I = 2200$ SF

DL PER FLOOR = 49.5 K
DL TOTAL = 618.8 K

LL TOTAL $= (70 \cdot 550)7 + (115 \cdot 550)4$
= 522.5 K

SL TOTAL $= 65 \cdot 550$
= 35.8 K

LOAD COMBINATIONS:
1.4 DL
1.2 DL + 1.6 LL + 0.5 SL
1.2 DL + 1.6 SL + LL

$1.4(618.8\text{ k}) = 866.3\text{ K}$

$1.2(618.8) + 1.6(0.4)(522.5) + 0.5(35.8) = 1094.9\text{ K} \Leftarrow P_u$

$1.2(618.8) + 1.6(35.8) + 522.5(0.4) = 1008.8\text{ K}$

LIVE LOAD REDUCTION

$RF = 0.25 + 15/\sqrt{A_I} = 0.25 + 15/\sqrt{(2200 \cdot 12)} = 0.34$

LOWER LIMIT = 0.4

AISC MANUAL OF STEEL CONST.

$KL = 14'$ W12×106 $\phi P_n = 1130\text{ K}$ $P_u \geq \phi P_n$

$P_u = 1094.9\text{ K}$

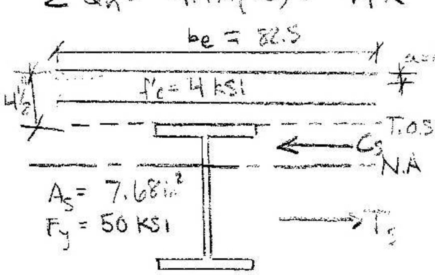
BEAM CHECK @ C-4 : D4 (LEVELS 9-12)

W16x26 (10 STUDS) — 27'-6"

TRIBUTARY WIDTH = 6.875'
LL = 115 PSF
DL = 90 PSF

STUD STRENGTH SPECIFIED – 9.9 K (3/4" ϕ 3½ LONG)

$\Sigma Q_n = 9.9k(10) = 99K$



$C_c = 99k$

$C_c = 0.85 f'_c (b_e) a$
$99 = 0.85(4)(82.5)a$
$a = 0.35"$

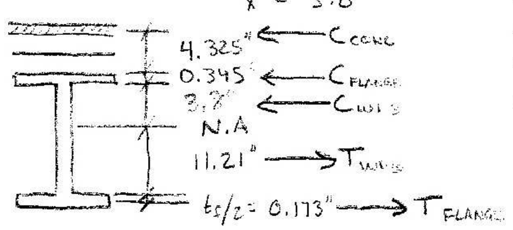
$b_e = S = 82.5"$
$= L/4 = 82.5"$

$C = C_s + C_c = 241.5$ k
$T_s = 241.5$ k

$A_s F_y = 384$ k $C_s = 142.5$ K

$b_f = 5.5"$ $A_f = 1.9"$ $A_f F_y = 95$ K $142.5 - 95 = 47.5$ K
$t_f = 0.345"$

$t_w = 0.25"$ $(F_y) t_w (x) = 47.5$
$x = 3.8"$



$M_n = 99k(38 + 0.345 + 4.325)$	838.6
$+ (50)(5.5 \times 0.34)(3.8 + 0.345/2)$	376.9
$+ (50)(0.25 \times 3.8)(3.8/2)$	90.3
$+ (50)(0.25 \times 11.21)(11.21/2)$	785.4
$+ (50)(5.5 \times 0.345)(11.21 + 0.173)$	1080.0
	= 3171.1 FT·K

$\phi M_n = 0.9 M_n$

$\phi M_n = 2854$ ft·k

LOAD COMBO. LL = 790.6 PLF
DL = 618.8 PLF

1.2DL + 1.6LL

LL REDUCTION:
$A_T = 27.5 \times 13.75 = 378.13 \text{ ft}^2$

$0.25 + 15/\sqrt{378.13} = 1.02 \leftarrow$ N.A.

$W_u = 1.2(618.8) + 1.6(790.6) = 2$ K/ft

$M_u = \frac{w_u l^2}{8} = \frac{2(27.5)^2}{8} = 189.1$ ft·k

*BEAM DESIGN CONTROLLED BY DEFLECTION

APPENDIX D – FULL COMPOSITE BEAM ACTION CALCULATIONS

Shape	As (in²)	be (in)	Trib Width/ Space (in)	d (in)	tw (in)	fy (ksi)	f'c (ksi)	deck t (in)	slab t (in)	(AsFy) ΣQn	a (in)	ΦMn Φ=0.9 in-K	ΦMn Φ=0.9 FT-K	ΦVn Φ=1.0 K
Column Line C to D (Beams)														
16x 26	7.68	82.50	82.5	15.70	0.250	50	4	2	4.5	384	1.369	4032	336.0	117.8
14x 22	6.49	82.50	82.5	13.70	0.230	50	4	2	4.5	325	1.157	3146	262.2	94.5
12x 19	5.57	82.50	82.5	12.20	0.235	50	4	2	4.5	279	0.993	2532	211.0	86.0
Column Line A to C & F to H (Beams)														
12x 14	4.16	58.50	82.5	11.90	0.200	50	4	2	4.5	208	1.046	1858	154.9	71.4
10x 12	3.54	58.50	82.5	9.87	0.200	50	4	2	4.5	177	0.890	1432	119.3	59.2
Column Line E.1 to F (Beams)														
12x 19	5.57	67.50	82.5	12.20	0.220	50	4	2	4.5	279	1.214	2505	208.7	80.5
12x 14	4.16	67.50	82.5	11.90	0.200	50	4	2	4.5	208	0.906	1871	156.0	71.4
10x 12	3.54	67.50	82.5	9.87	0.200	50	4	2	4.5	177	0.771	1442	120.1	59.2
Column Line D to D.6 (Beams)														
14x 22	6.49	75.00	90.0	13.70	0.230	50	4	2	4.5	325	1.273	3129	260.7	94.5
12x 19	5.57	75.00	90.0	12.20	0.235	50	4	2	4.5	279	1.092	2520	210.0	86.0
12x 16	4.71	75.00	90.0	12.00	0.220	50	4	2	4.5	236	0.924	2128	177.3	79.2
12x 14	4.16	75.00	90.0	11.90	0.200	50	4	2	4.5	208	0.816	1880	156.7	71.4
10x 12	3.54	75.00	90.0	9.87	0.200	50	4	2	4.5	177	0.694	1448	120.6	59.2
Column Line C (Girders)														
18x 46	13.80	75.00	270.0	18.10	0.360	50	4	2	4.5	690	2.706	7574	631.2	195.5
16x 40	11.80	75.00	270.0	16.00	0.305	50	4	2	4.5	590	2.314	6023	501.9	146.4
18x 35	10.30	75.00	270.0	17.70	0.300	50	4	2	4.5	515	2.020	5720	476.6	159.3
16x 31	9.13	75.00	270.0	15.90	0.275	50	4	2	4.5	457	1.790	4747	395.6	131.2
18x 60	17.60	82.50	282.0	18.20	0.415	50	4	2	4.5	880	3.137	9529	794.1	226.6
18x 46	13.50	82.50	282.0	18.10	0.360	50	4	2	4.5	675	2.406	7501	625.1	195.5
16x 45	13.30	82.50	282.0	16.10	0.345	50	4	2	4.5	665	2.371	6802	566.8	166.6

$T = C$

$T = (As)(fy)$

$C = 0.85f'c(a)(be)$

$\Phi Mn = \Phi[(AsFy)(d/2) + 0.85f'c(a)(b)(\text{slab } t-a/2)]$

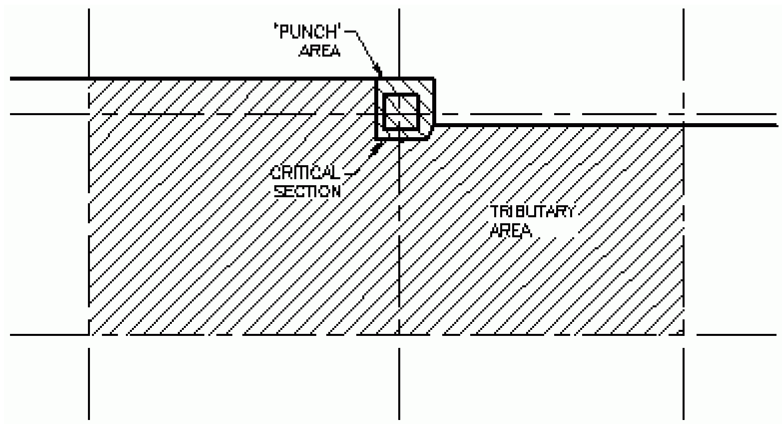
$\Phi Vn = \Phi 0.6(Aw)(fy)$

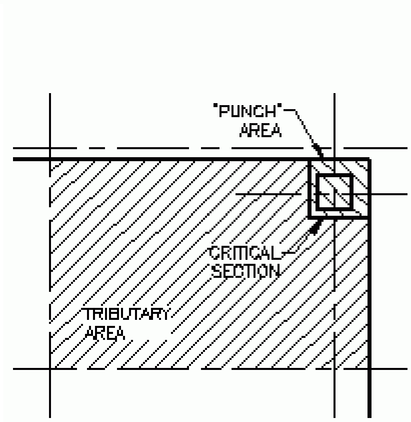
LL reduction $= 0.25+15 / \sqrt{(Ai)} > 0.5$

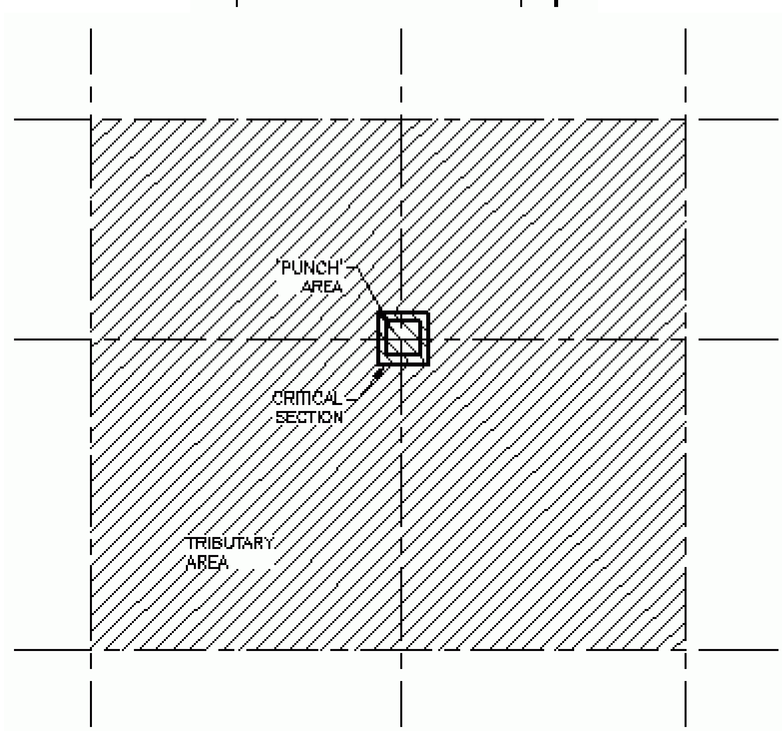
	AISC Tab3-21 3/4"dia Qn (K)	Stud # req'd	1.2(DL+ SELF) plf	Influ. Area AI ft^2	Reduct. Factor (>0.50)	LL psf	1.6LL w/ LL reduct. plf	TL plf	L ft	Mu wl^2/8	Vu wl / 2	ΦVn>Vu & ΦMn>Mu ?
16x26	17.2	45	575.7	378	1.00	125	1031.3	1607.0	27.50	151.91	22.1	OK
14x22	17.2	38	570.9	378	1.00	125	1031.3	1602.2	27.50	151.45	22.0	OK
12x19	17.2	32	567.3	378	1.00	125	1031.3	1598.6	27.50	151.11	22.0	OK
12x14	17.2	24	561.3	268	1.00	125	1031.3	1592.6	19.50	75.70	15.5	OK
10x12	17.2	21	558.9	268	1.00	125	1031.3	1590.2	19.50	75.58	15.5	OK
12x19	17.2	32	567.3	309	1.00	125	1031.3	1598.6	22.50	101.16	18.0	OK
12x14	17.2	24	561.3	309	1.00	125	1031.3	1592.6	22.50	100.78	17.9	OK
10x12	17.2	21	558.9	309	1.00	125	1031.3	1590.2	22.50	100.63	17.9	OK
14x22	17.2	38	620.4	375	1.00	125	1125.0	1745.4	25.00	136.36	21.8	OK
12x19	17.2	32	616.8	375	1.00	125	1125.0	1741.8	25.00	136.08	21.8	OK
12x16	17.2	27	613.2	375	1.00	125	1125.0	1738.2	25.00	135.80	21.7	OK
12x14	17.2	24	610.8	375	1.00	125	1125.0	1735.8	25.00	135.61	21.7	OK
10x12	17.2	21	608.4	375	1.00	125	1125.0	1733.4	25.00	135.42	21.7	NG
18x46	21.2	65	1837.2	1125	0.70	125	2353.1	4190.3	25.00	327.37	52.4	OK
16x40	21.2	56	1830.0	1125	0.70	125	2353.1	4183.1	25.00	326.80	52.3	OK
18x35	21.2	49	1824.0	1125	0.70	125	2353.1	4177.1	25.00	326.34	52.2	OK
16x31	21.2	43	1819.2	1125	0.70	125	2353.1	4172.3	25.00	325.96	52.2	OK
18x60	21.2	83	1933.2	1293	0.67	125	2352.0	4285.2	27.50	405.08	58.9	OK
18x46	21.2	64	1916.4	1293	0.67	125	2352.0	4268.4	27.50	403.50	58.7	OK
16x45	21.2	63	1915.2	1293	0.67	125	2352.0	4267.2	27.50	403.38	58.7	OK

Shape	L ft	be (in)	Space (in)	Const. DL plf	LL plf	Ixx Steel	a (in)	Y2 (in)	Low Bnd ILB	Const. DL Δ	LL Δ	L/360 inch	
Column Line C to D (Beams)													
16x26	27.5	82.5	82.5	273.5	859.4	301	1.369	3.816	822	0.40	0.46	0.92	
14x22	27.5	82.5	82.5	269.5	859.4	199	1.157	3.922	580	0.60	0.66	0.92	
12x19	27.5	82.5	82.5	266.5	859.4	130	0.993	4.004	583	0.91	0.65	0.92	
Column Line A to C & F to H (Beams)													
12x14	19.5	58.5	82.5	261.5	859.4	88.6	1.046	3.977	298	0.33	0.32	0.65	
10x12	19.5	58.5	82.5	259.5	859.4	53.8	0.890	4.055	200	0.54	0.48	0.65	
Column Line E.1 to F (Beams)													
12x19	22.5	67.5	82.5	266.5	859.4	130	1.214	3.893	414	0.41	0.41	0.75	
12x14	22.5	67.5	82.5	261.5	859.4	88.6	0.906	4.047	300	0.59	0.57	0.75	
10x12	22.5	67.5	82.5	259.5	859.4	53.8	0.771	4.114	203	0.96	0.84	0.75	NG
Column Line D to D.6 (Beams)													
14x22	25.0	75.0	90.0	292.0	937.5	199	1.273	3.864	573	0.44	0.50	0.83	
12x19	25.0	75.0	90.0	289.0	937.5	130	1.092	3.954	410	0.67	0.69	0.83	
12x16	25.0	75.0	90.0	286.0	937.5	103	0.924	4.038	341	0.84	0.83	0.83	
12x14	25.0	75.0	90.0	284.0	937.5	88.6	0.816	4.092	299	0.97	0.95	0.83	NG
10x12	25.0	75.0	90.0	282.0	937.5	53.8	0.694	4.153	203	1.59	1.40	0.83	NG
Column Line C (Girders)													
18x46	25.0	75.0	270.0	856.0	2812.5	712	2.706	3.147	1730	0.36	0.49	0.83	
16x40	25.0	75.0	270.0	850.0	2812.5	518	2.314	3.343	1278	0.50	0.67	0.83	
18x35	25.0	75.0	270.0	845.0	2812.5	510	2.020	3.490	1300	0.50	0.66	0.83	
16x31	25.0	75.0	270.0	841.0	2812.5	375	1.790	3.605	984	0.68	0.87	0.83	
18x60	27.5	82.5	282.0	906.0	2937.5	984	3.137	2.931	2335	0.41	0.56	0.92	
18x46	27.5	82.5	282.0	892.0	2937.5	712	2.406	3.297	1818	0.56	0.72	0.92	
16x45	27.5	82.5	282.0	891.0	2937.5	586	2.371	3.315	1444	0.67	0.90	0.92	

APPENDIX E – TWO WAY FLAT PLATE PUNCHING SHEAR CALCULATIONS







Column	α s	x (in)	y (in)	β c	f'c psi	d (in)	t (in)	bo (in)	Conc Weight (pcf)	Trib Area (ft^2)	Punch Area d/2	Net Area (ft^2)
Corner	20	18.00	18.00	1.00	5000	9.125	11.0	63.13	115	129.50	6.92	122.58
	20	18.00	18.00	1.00	5000	9.125	11.0	63.13	115	129.50	6.92	122.58
	20	18.00	18.00	1.00	5000	9.125	11.0	63.13	115	129.50	6.92	122.58
Edge	30	18.00	18.00	1.00	5000	9.125	11.0	67.46	115	273.31	6.8	266.51
	30	18.00	18.00	1.00	5000	9.125	11.0	67.46	115	273.31	6.8	266.51
	30	18.00	18.00	1.00	5000	9.125	11.0	67.46	115	273.31	6.8	266.51
Interior	40	18.00	18.00	1.00	5000	9.125	11.0	108.50	115	616.88	5.11	611.77
	40	18.00	18.00	1.00	5000	9.125	11.0	108.50	115	616.88	5.11	611.77
	40	18.00	18.00	1.00	5000	9.125	11.0	108.50	115	616.88	5.11	611.77
Interior	40	18.00	18.00	1.00	5000	9.125	11.0	108.50	115	756.25	5.11	751.14
(worst	40	18.00	18.00	1.00	5000	9.125	11.0	108.50	115	756.25	5.11	751.14
case)	40	18.00	18.00	1.00	5000	9.125	11.0	108.50	115	756.25	5.11	751.14

Column	Self psf	DL psf	LL psf	Fact.Load 1.2D+ 1.6L psf	Vu K	Φ	(1) ΦVc K	(2) ΦVc K	(3) ΦVc K	ΦVc>Vu ?	(4) ΦVn max K	(5) ΦVc K	(6) Req'd ΦVs K
Corner	105.4	32.0	125.0	364.9	44.7	0.75	122.2	183.3	149.4	OK			
	105.4	32.0	115.0	348.9	42.8	0.75	122.2	183.3	149.4	OK			
	105.4	32.0	70.0	276.9	33.9	0.75	122.2	183.3	149.4	OK			
Edge	105.4	32.0	125.0	364.9	97.3	0.75	130.6	195.9	197.8	OK			
	105.4	32.0	115.0	348.9	93.0	0.75	130.6	195.9	197.8	OK			
	105.4	32.0	70.0	276.9	73.8	0.75	130.6	195.9	197.8	OK			
Interior	105.4	32.0	125.0	364.9	223.2	0.75	210.0	315.0	281.6	NG	315.0	105.0	118.2
	105.4	32.0	115.0	348.9	213.4	0.75	210.0	315.0	281.6	NG	315.0	105.0	108.4
	105.4	32.0	70.0	276.9	169.4	0.75	210.0	315.0	281.6	OK			
Interior	105.4	32.0	125.0	364.9	274.1	0.75	210.0	315.0	281.6	NG	315.0	105.0	169.1
(worst	105.4	32.0	115.0	348.9	262.1	0.75	210.0	315.0	281.6	NG	315.0	105.0	157.1
case)	105.4	32.0	70.0	276.9	208.0	0.75	210.0	315.0	281.6	OK			

(1) $\Phi V_c = \Phi 4\sqrt{f'_c}(b_o)(d)$
(2) $\Phi V_c = \Phi(2+4/\beta c)\sqrt{f'_c}(b_o)(d)$
(3) $\Phi V_c = \Phi(\alpha_s / (b_o/d))\sqrt{f'_c}(b_o)(d)$

(4) $\Phi V_c = \Phi 6\sqrt{f'_c}(b_o)(d)$ (maximum limit)

(5) $\Phi V_c = \Phi 2\sqrt{f'_c}(b_o)(d)$ (if shear reinforcement provided)

(6) $V_u > \Phi V_n = \Phi V_c + \Phi V_s$

bo = critical section perimeter

α_s = 20 for corner column
α_s = 30 for edge column
α_s = 40 for interior column

β_c = 1.0 for square column

APPENDIX F

WIND LOADS as per ASCE 7-05

Wind loads were analyzed using section 6 of ASCE 7-05. Appendix A contains a detailed analysis of wind loads using the equations and factors set forth in ASCE. These factors are dependent on building location and characteristics as well as experimental data.

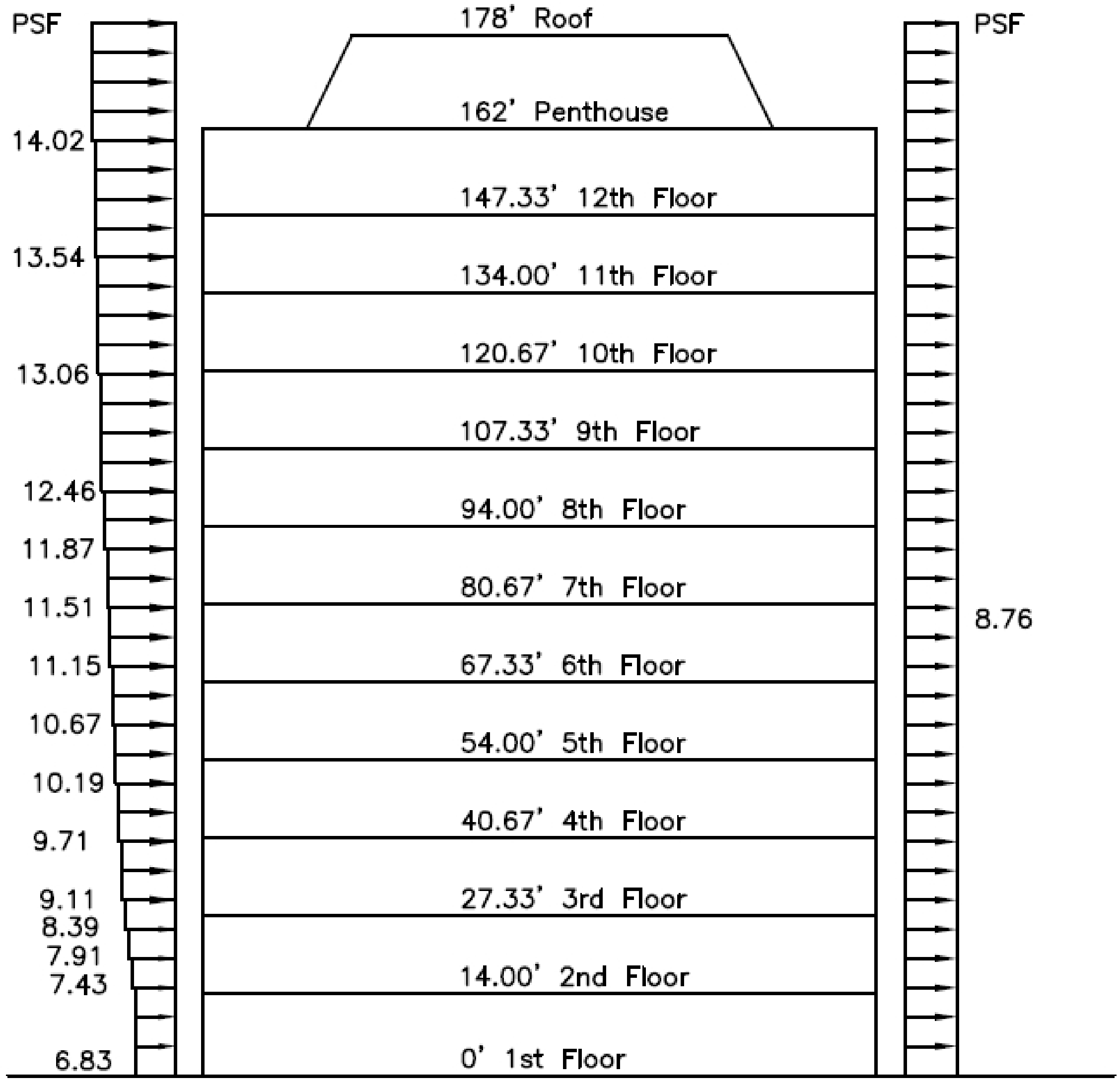
Design Criteria

Height	h			178’
Dimensions				98’x115’
Wind directionality factor	Kd	6.5.4		0.85
Importance Factor	I	6.5.5		1.0
Wind Exposure Category		6.5.6		B
Basic Wind Speed	V			90 MPH
Topographic Factor	Kzt	6.5.7		1.0
Gust Factor	Gf	6.5.8		0.85
External Pressure Coeff.	Cpf	6.5.11.2	Windward	0.8
			Leeward	-0.5
			Sides	-0.7

$q_z = 0.00256(K_z*K_{zt}*K_d*V^2*I)$

h	Kz	Kzt	Kd	V	I	qz	Gf	Cp	Pressure (psf)
									Windward
0-15	0.57	1.00	0.85	90.00	1.00	10.05	0.85	0.80	6.83
20	0.62	1.00	0.85	90.00	1.00	10.93	0.85	0.80	7.43
25	0.66	1.00	0.85	90.00	1.00	11.63	0.85	0.80	7.91
30	0.70	1.00	0.85	90.00	1.00	12.34	0.85	0.80	8.39
40	0.76	1.00	0.85	90.00	1.00	13.40	0.85	0.80	9.11
50	0.81	1.00	0.85	90.00	1.00	14.28	0.85	0.80	9.71
60	0.85	1.00	0.85	90.00	1.00	14.98	0.85	0.80	10.19
70	0.89	1.00	0.85	90.00	1.00	15.69	0.85	0.80	10.67
80	0.93	1.00	0.85	90.00	1.00	16.39	0.85	0.80	11.15
90	0.96	1.00	0.85	90.00	1.00	16.92	0.85	0.80	11.51
100	0.99	1.00	0.85	90.00	1.00	17.45	0.85	0.80	11.87
120	1.04	1.00	0.85	90.00	1.00	18.33	0.85	0.80	12.46
140	1.09	1.00	0.85	90.00	1.00	19.21	0.85	0.80	13.06
160	1.13	1.00	0.85	90.00	1.00	19.92	0.85	0.80	13.54
180	1.17	1.00	0.85	90.00	1.00	20.62	0.85	0.80	14.02
									Leeward
180	1.17	1.00	0.85	90.00	1.00	20.62	0.85	-0.50	-8.76
									Sides
180	1.17	1.00	0.85	90.00	1.00	20.62	0.85	-0.70	-12.27

Through a generalized analysis of the buildings fundamental period set forth in ASCE 7-05 the building was found to behave as a flexible structure. (*See the seismic loads section for the building period calculation*)



WIND PRESSURES

APPENDIX G

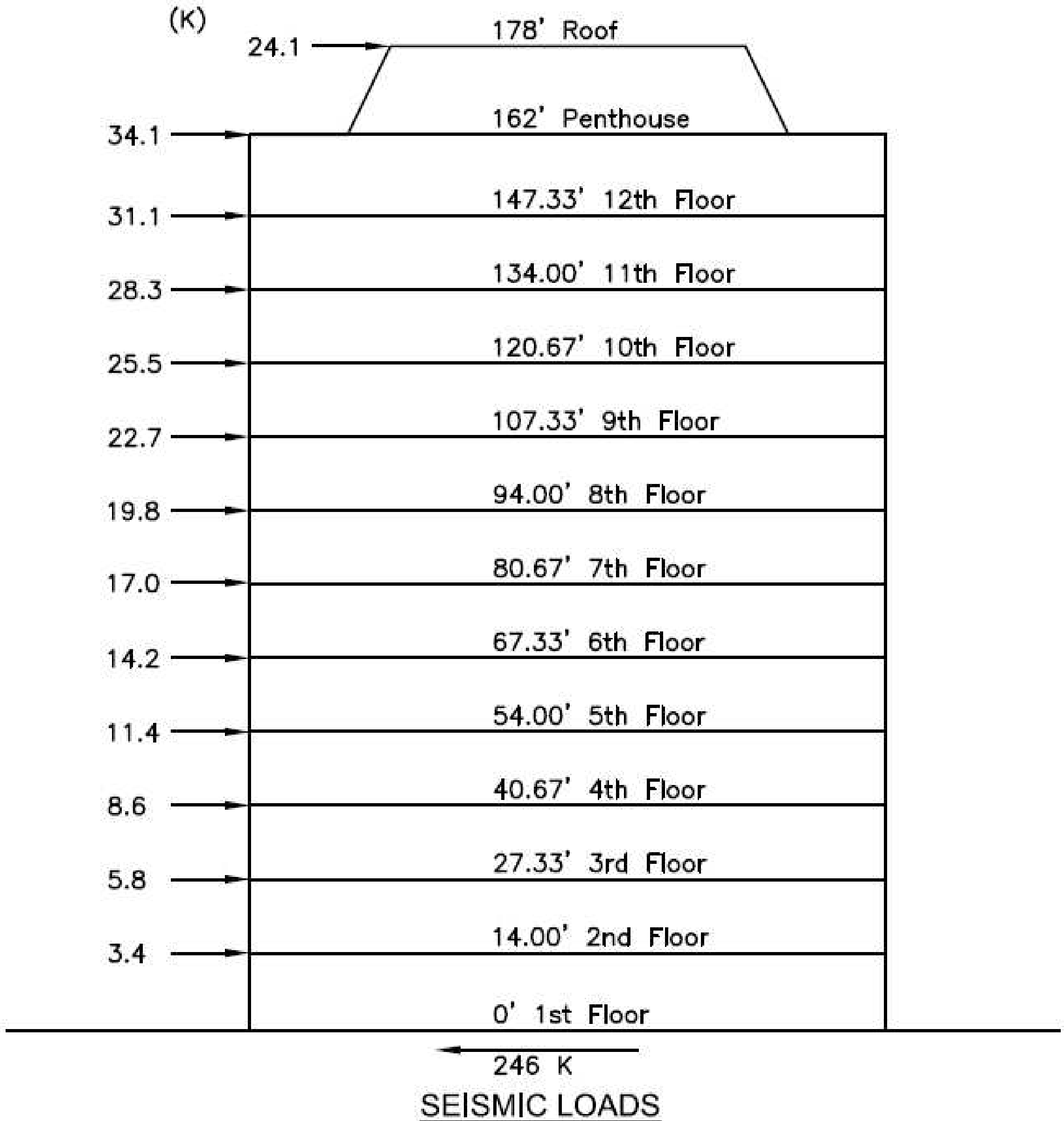
SEISMIC LOADS as per ASCE 7-05

Seismic loads were found using the applicable sections of ASCE 7-05; Equivalent Lateral Force procedure (12.8). All factors and accelerations were found using the tables and equations contained in ASCE. All dead loads used are based on ASCE 7-05 and are listed in the gravity loads section of this report.

Site Class	D	
Occupancy Category	II	
Importance Factor	1.0	
Seismic Design Category	B	
Response Modification Factor (R)	5	
Period (Ta)	1.46	
Ss	0.229	*
S1	0.069	*
SDS	0.28	
SD1	0.12	
T_L	6	Figure 22-15
Cs	0.016	
Base Shear (V)	246 (K)	

*From USGS website - earthquake.usgs.gov/research/hazmaps/design

Level	w_x (k)	h_f (ft)	h_x (ft)	$w_x(h_x)^k$	F_x (K)	V_x (K)	M_x (FT-K)
Pent	750	16.00	178.00	133500.0	24.1	0.0	4286.5
12	1170	14.67	162.00	189540.0	34.2	24.1	5538.8
11	1170	13.33	147.33	172380.0	31.1	58.3	4581.3
10	1170	13.33	134.00	156780.0	28.3	89.4	3789.6
9	1170	13.33	120.67	141180.0	25.5	117.6	3073.0
8	1170	13.33	107.33	125580.0	22.7	143.1	2431.4
7	1170	13.33	94.00	109980.0	19.8	165.8	1864.9
6	1170	13.33	80.67	94380.0	17.0	185.6	1373.3
5	1170	13.33	67.33	78780.0	14.2	202.6	956.9
4	1170	13.33	54.00	63180.0	11.4	216.8	615.4
3	1170	13.33	40.67	47580.0	8.6	228.2	349.0
2	1170	13.33	27.33	31980.0	5.8	236.8	157.7
1	1350	14.00	14.00	18900.0	3.4	242.6	47.7
	14970			1363740.0	246.0	246.0	29065.7


(K)
24.1
178' Roof
34.1
162' Penthouse
31.1
147.33' 12th Floor
28.3
134.00' 11th Floor
25.5
120.67' 10th Floor
22.7
107.33' 9th Floor
19.8
94.00' 8th Floor
17.0
80.67' 7th Floor
14.2
67.33' 6th Floor
11.4
54.00' 5th Floor
8.6
40.67' 4th Floor
5.8
27.33' 3rd Floor
3.4
14.00' 2nd Floor
0' 1st Floor
246 K
SEISMIC LOADS

SEISMIC CALCS

SITE CLASS "D" (FIRM SOILS)

$v_s = 600 - 1200$ ft/s $\bar{N} = 15 - 50$ $\bar{S}_u = 1000 - 2000$ PSF

SDS = 0.28 $S_1 = 0.069$ FROM USGS WEBSITE
SD1 = 0.12 $S_S = 0.229$

OCCUPANCY CATEGORY – II
IMPORTANCE CATEGORY – 1.0
SEISMIC DESIGN CAT. – B
RESPONSE MOD. FACTOR – R = 5

$T_L = 6$ (FIG. 22-15)

$T_a = C_t h_n^x = 0.03(178)^{0.75} = 1.46$
$C_t = 0.03$
$x = 0.75$
$h = 178'$

$C_S = \frac{SD1}{T(R/I)} \leq \frac{SDS}{R/I}$

$= \frac{0.12}{1.46(5/1)} \leq \frac{0.28}{5/1}$

$= \underline{0.016} \leq 0.056$

W: $A_{TOTAL} = 140{,}000$ SF

DL = 90 PSF	= 12,600 k
PARTITIONS = 10 PSF	= 1400 k
20% SNOW LOAD = 13 PSF	= 150 k
ROOF MECHANICAL	= 500 k
	14650 K

BASE SHEAR:

$V = C_S W = 0.016(14970) = 246$ K

$k = 1$ (12.8.3)

$F_x = \frac{w_x h_x^k}{\sum w_i h_i^k} V$

APPENDIX H – PICTURES


677