Structural Redesign
Gravity System

Gravity Loads

Gravity loads used in design can be seen in Table 2 and Table 3. Loads used can be referenced to the Engineer of Record, and are in accordance with ASCE7-05.

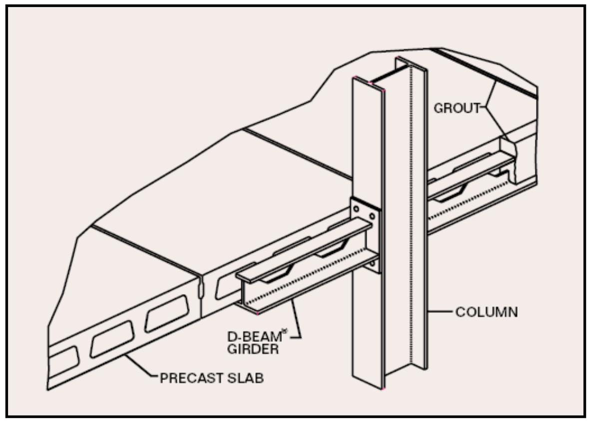
Table 2: Superimposed Dead Loads

Area	PSF
Roofs	30
Penthouse Roof	40
Penthouse Floor	20
Guestroom Floors	10
Second Floor	10
First Floor	10
Pool Deck	40

Table 3: Live Loads

Area	PSF		Area	PSF
Roof Live Load	30		Garage Level	150
Penthouse Floor	150		Pool Deck	100
Guestroom Floors	40		First Floor	100
Second Floor	100		First and second Floor Storage Kitchen and Laundry	125
Second Floor Mechanical Rooms	150		Meeting Rooms	100
Meeting Rooms	100		Stairs	100
Stairs	100		Garage Level	150

Girder-Slab System



Girder-Slab system was developed by Girder-Slab Technologies LLC. It is the first of its kind to utilize steel and pre-cast plank as a composite monolithic structural floor assembly. A modified steel girder supports pre-cast concrete plank on either side with its bottom flange. The modified steel girder is called a dissymmetric beam or D-beam. There are two basic D-beam sections available for use with 8" pre-cast slabs, DB-8 and DB-9. Each beam is cut from a parent wide flange section which produces two D-beams. Beams are corrugated cut in half, and then a piece of steel is welded to the web to produce a small top flange. The corrugated web of the girder allows for grout to flow through the beam and the hollow core plank openings. Upon curing this transformed grouted section acts compositely with the pre-cast plank. The transformed section has over twice the moment capacity of sole D-beam. Girder-slab system and D-beam girders are only distributed and assembled by steel contractors authorized by Girder-slab technologies LLC of New Jersey. Construction of girder slab system is fairly quick and saves on labor costs compared to cast-in-place concrete (Girder-Slab Design Guide).

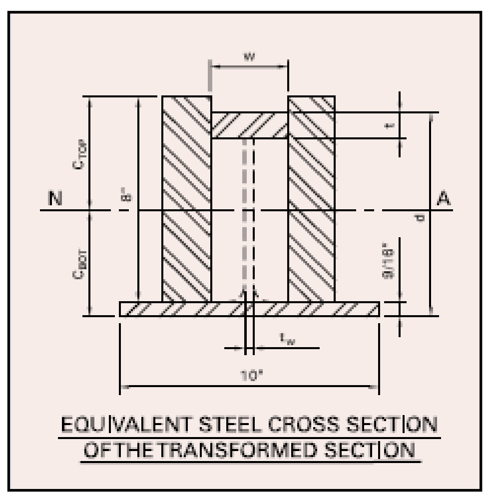


Figure 4: Left: Composite D-beam, Right: Composite D-beam with equivalent cross-section

Girder-Slab system was implemented for typical floors 4 -11 during the structural redesign. It was chosen on these floors to match the similar floor thickness (8”) of the existing post tension floor system (7-1/2”). For typical floors bay sizes are 27’-0”x 20’-0”. Eight inch pre-cast planks will span the length of 27’-0” while a DB-8x42 will span 16’-0” with a 2’-0” D-beam tree connection on either side. J952 8”x 4’ Span Deck planks with 6- ½” Ø strands will be used in the Girder-Slab system. A ¾” topping will be used to level the floor from differential deck cambers. Typical Girder-slab layout can be seen in Figure 5.

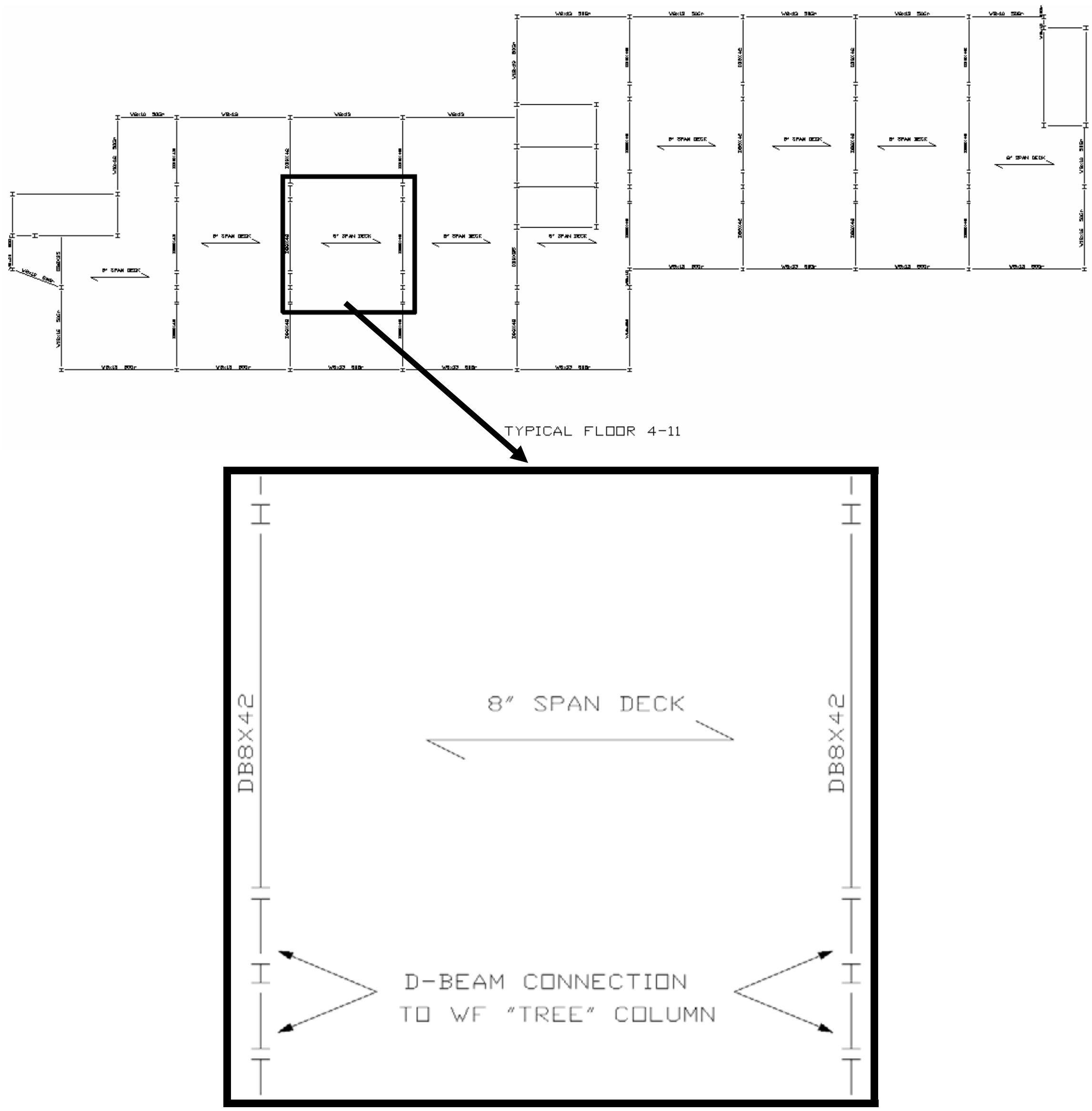


Figure 5: Typical Girder-Slab plan for floors 4-11

Girder-Slab system was designed in accordance with the design specifications and examples outlined in the Girder-Slab Design Guide. Girder-slab utilizes Allowable Stress Design specifications of the American Institute of Steel Construction (AISC).

When calculating allowable loads on the system, the system must be checked twice, for pre-composite action and full composite. Pre-composite action occurs before the grouting and curing during construction. Initial load during construction is the weight of the pre-cast hollow core planks. After curing has occurred, the transformed section is checked against the dead load of the plank, the superimposed dead loads of partitions, etc., and the live load for the occupancy according to ASCE7-05. The required section modulus is calculated and compared to the given transformed sections of the composite D-beam and plank system. Equation 1 shows the calculation to find the required section modulus.

Equation 1: $S_{Reg} = \dfrac{M_{TL}}{0.6F_y}$

Where: M_{TL} is the bending moment due to total loading

F_y is the yield strength of the steel

Deflections of the section are also checked and compared against industry standard of L/360. Compression stress on the concrete is checked against allowable stress. Next the bottom flange of the D-beam should be checked for tensile stresses from the total load. This tensile stress is then compared to the allowable yield stress of the steel section. Equation 2 illustrates this computation where F_y is equal to 50 ksi.

Equation 2: $f_b = \dfrac{M_{DL}}{S_b} + \dfrac{M_{SUP}}{S_{b(Transformed)}} \leq 0.9F_y$

Where: S_b is the section modulus of the D-beam before composite action

$S_{b(Transformed)}$ is the section modulus of the transformed section

The last strength check is allowable shear stress of the D-beam against the total loading.

Equation 3: $f_v = \dfrac{R}{netAreaweb} \leq 0.4F_y$

Where: R is support reaction

For calculation results please see the Appendix.

Tree Column

To use a DB8 beam with the given loading, spans had to be limited to 16'. Given the desirable spans of 19' to 20', a wide flange "tree" column had to be utilized. In this connection WT section is welded to a wide-flange column with a bevel weld and a fillet on both sides. This detail can be seen in Figure 6. The WT section has to be the same depth as the D-beam. In this case a DB8 was used, therefore the tree beam selected had to be a WT8 section. A typical connection was designed producing a WT8x22.5 section. This beam is able to resist a negative moment caused by this fixed connection type of 52.3 ft-kips. The D-beam transfers a shear force of 23.3 K to the tree beam with a single plate with two bolts in each member. A 9"x6-1/2"x 7/16" plate will be used with 1" A325N bolts. Calculations for member and connection may be found in the Appendix.

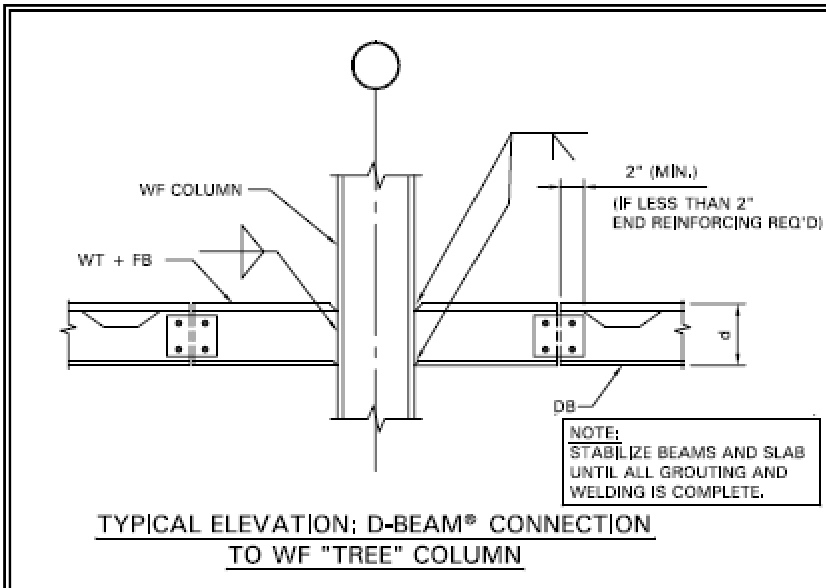


Figure 6: Tree Column detail

Tree column connections are sure to be costly. Another alternative to the tree column connection would be to decrease span lengths of the D-beam. This could be achieved by adding more columns to the framing plan. This however would not be an applicable alternative for the 'BWI Hilton'. The column spacing given allows for a more wide open floor layout desirable for hotels.

Composite Beam

Floors Ground through 3 were designed using a composite steel beam and concrete slab system. Bays sizes were kept the same as the existing concrete system so not to disrupt the architecture. Columns and beams were laid out using RAM structural system. Typical bay sizes, referenced from construction drawings, are 27’-0” x 20’-0”. Composite concrete and deck span perpendicular to beams spanning the 20’-0” distance and spaced 9’-0” o.c. Beams will frame into girders spanning 27’-0”, which in turn will frame into W-shaped columns at the web.

Decking used was a 2” Lok-Floor deck with a 3” concrete slab having a compressive strength of 3000 psi. Deck was capable of being unshored during construction with a unshore span of 9.6ft and a loading capacity of 295 psf. Studs used were Grade 60 with dimensions of 3.5” - ¾” Ø. Composite deck has a fire rating of 2 hours.

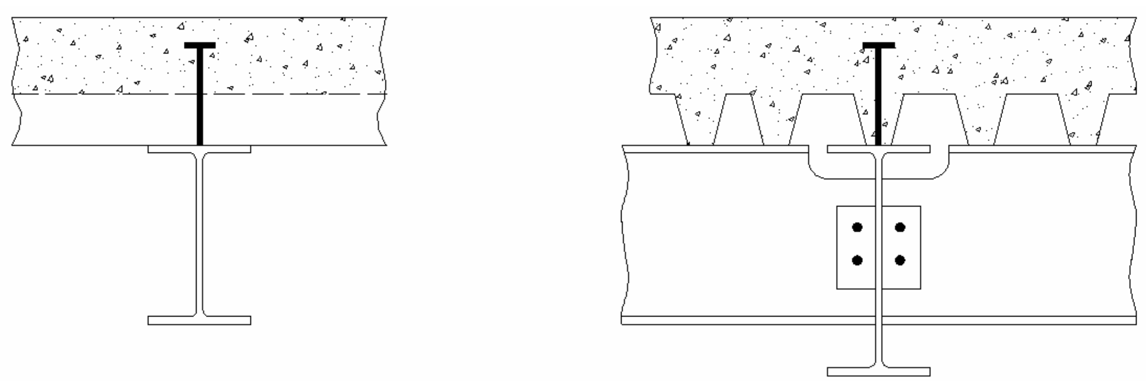


Figure 7: Left: Composite beam with concrete slab, Right: Composite girder and slab

Ground floor layout can be seen in Figure 8 on the following page.

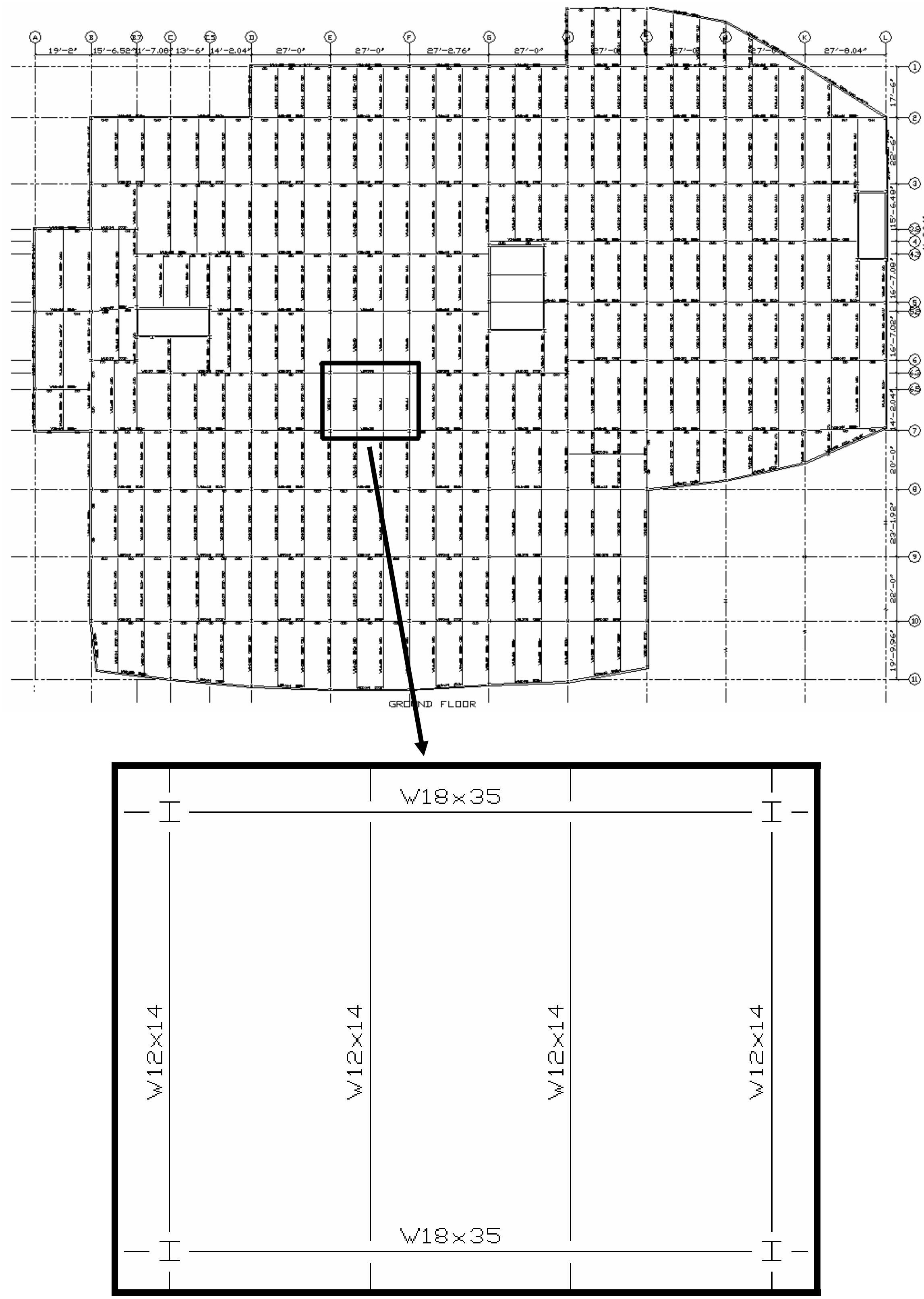


Figure 8: Ground floor plan with typical framing layout

Composite beams and girders were designed in accordance with American Institute of Steel Construction (AISC) Manual 13th Edition Allowable Strength Design (ASD). A load combination of D+L was used for gravity beams and girders. Hand calculations produced a W10x26 beam with 16-3/4"Ø studs in the weak direction. Beams were required to resist a max moment at mid-span of 69 ft.-kips. Sizing of beams were controlled by deflection limitation. A moment of inertia required to limit deflection, for construction loading, was 105 in^4. This I_x value is the I_x value of the beam itself before composite action. Loads to be considered during construction are the weight of the wet concrete, workers, equipment and the beam self weight. Deflection should also be checked against live loads and total loads after concrete cures and system acts compositely. Beam sizes were well within the deflection limit of L/360 = 0.64". Total deflection of composite beam required a lower bound I_x of 171 in^4. This value was computed by setting the deflection equation of a simple supported beam with a distributed load equal to the deflection limit of L/240. By manipulating the equation the value of I_x can be solved, as seen in Equation 4.

Equation 4: $$I_x = \frac{5wL^4}{384E\left(\dfrac{L}{240}\right)}$$

Where: *w* is the distributed load

L is the span of the beam

E is the modulus of elasticity of steel = 29,000 ksi

See Appendix for hand calculations.

RAM results produced typical sizes for beams of W12x14 (14 studs), W12x19 (16 studs), W14x22 (10 studs). These sizes were check against I_x values calculated to limit deflection. Girders spanning the length of 27 ft. were also designed for a typical bay by hand and then checked against RAM results. Typical girder designs by RAM were W18x35 and W16x31, which also worked for deflection. All beams and girders were designed as simply supported by the columns.

Vibration Analysis

When designing typical composite slab and beam for a floor system, vibration of the system should be checked against acceptable human perception levels. Thin slabs and smaller beams in a composite system produce smaller moment of inertia values which in turn allows for larger deflections. The weight of the structure also effects the deflection with a heavier structure deflecting more than a lighter structure. These two variables have to be considered when calculating deflection. Natural Frequency of the system is inversely related to the systems deflection. Depending on the cause of vibration, there is varying criteria for the system's natural frequency.

Vibrations caused by walking can be disturbing to human perception. While designing the 2nd floor system walking vibrations were considered. Floor accelerations for a typical bay (J-K, 3-4) were checked in an area were offices are located in the 'BWI Hilton'. The check was performed in accordance with AISC Design Guide 11 Ch. 4 Design for Walking Excitation. Typical bays have a 5" composite concrete slab and deck spanning perpendicular to W12x14 beams seen in Figure 9.

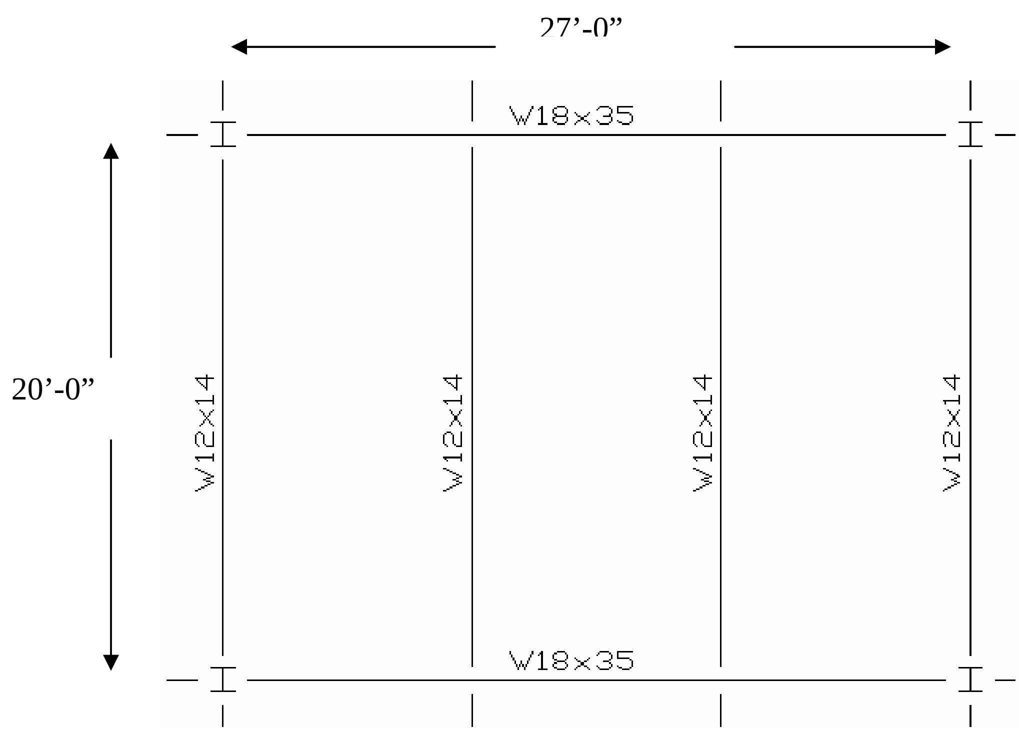


Figure 9: Typical bay framing plan 2nd floor

Trying to determine the critical mode of a floor system in resonance with a harmonic step frequency may be difficult. There are varying factors both structural and non-structural that affect the floor system's natural frequency. The natural frequency of a critical mode can be estimated by first analyzing a beam panel mode and then girder panel mode, and then the combined beam-girder panel mode (AISC DG 11 Pg11).

The lowest of these natural frequencies should be used in determination of the peak acceleration, a_p, as a fraction of the acceleration of gravity, g. The ratio, a_p/g, can be determined using Equation 5.

Equation 5: $$\frac{a_p}{g} = \frac{P_0 \exp(-0.35 f_n)}{\beta W}$$

Where: P_0 is a constant force representing excitation

f_n is fundamental natural frequency of critical panel mode

β is the modal damping ratio

W is the effective weight supported by critical panel

According to design guide criteria, the floor system is satisfactory if the a_p/g ratio does not exceed the appropriate value given in Table 4.1 in the design guide which can be seen as Figure 10. This floor system has an equivalent mode natural frequency of 5.93 Hz and therefore accelerates 0.31%g under a constant force of 65 lbs. Recommended excitation force of 65 lbs comes from Table 4.1 in the design guide.

Table 4.1 Recommended Values of Parameters in Equation (4.1) and a_o/g Limits			
	Constant Force P_o	**Damping Ratio** β	**Acceleration Limit** $a_o/g \times 100\%$
Offices, Residences, Churches	0.29 kN (65 lb)	0.02–0.05*	0.5%
Shopping Malls	0.29 kN (65 lb)	0.02	1.5%
Footbridges—Indoor	0.41 kN (92 lb)	0.01	1.5%
Footbridges—Outdoor	0.41 kN (92 lb)	0.01	5.0%

* 0.02 for floors with few non-structural components (ceilings, ducts, partitions, etc.) as can occur in open work areas and churches,
0.03 for floors with non-structural components and furnishings, but with only small demountable partitions, typical of many modular office areas,
0.05 for full height partitions between floors.

Figure 10: Table 4.1 from AISC Design Guide 11

This acceleration is well below the recommended acceleration limit of 0.5%g for offices given in Table 4.1 of the design guide, therefore the structure is acceptable for human vibration perception. Values given in this table are for natural frequencies between 4 Hz and 8 Hz. A damping ratio of 0.05 was used in calculations because the offices have full height partitions. Calculations may be found in the appendix.

Another area of the building where vibration might be an issue is the ballroom floor located on the ground floor. With people dancing on areas of the floor while others will be dining on the same framed floor, the occurrence of shaking wine glasses might cause some discomfort. The recommended acceleration limit due to rhythmic activities occurring simultaneously with dining is between 1.5 - 2.5 %g. This value was used to determine an adequate natural frequency (f_n) of the system. Equation 6 illustrates the calculation for required natural frequency.

Equation 6: $$f_n(req'd) = f\sqrt{1 + \frac{k}{\frac{a_0}{g}}\frac{\alpha_i w_p}{w_t}}$$

Where: f is the forcing function

k is a constant, 1.3 for dancing

$\frac{a_0}{g}$ is the peak acceleration ratio

α_i is the dynamic coefficient found in Table 2.1 of design guide

w_p is the effective weight per unit area of participants

w_t is effective total weight per unit area

Computations produced a natural frequency of 7.22 Hz. Using this natural frequency, deflection was found which then in turn could be used to find a required moment of inertia to keep the floor acceleration within the recommended limit. A required I_{tr} of the beams was found to be 463.6in^4. In previous calculations to find the effective I_{tr} for walking vibrations, an I_{tr} of 480in^4 was found for a W12x14 beam with a 5" composite slab and deck. The beams supporting the floor of the ballroom are W14x22 and a W12x19, and by inspection would have a larger moment of inertia, therefore would accelerate within the limits for dining.

In previous studies it has been found that industrial washers used in hotels produce a steady-state sinusoidal motion which will transfer to the framed floor on which it is supported upon. This motion can be excited by a load imbalance in the washer, e.g. laundry lumped on one side, while washer is running. Excitation of the steady state sinusoidal wave potentially could have adverse effects on the structure of the building. If the washer extract speeds are equal to the natural frequency of the building, then resonance will occur, causing increasing vibrations over time (Hanagan).

Measures need to be taken to prevent the washers from causing vibrations that may be perceived as uncomfortable or in the worse case perceived as dangerous. Isolating the structure supporting the washing units may be the best solution. Though completely isolating the framing from other members may be difficult, columns may be shared but beams can be designed not to share the same girder. Existing location of the washers in the 'BWI Hilton' are on the second floor adjacent to the elevator shaft, which can be seen in Figure 11. Since framing into the lateral brace frame system would not be ideal, moving the washers two bays over would be a possible solution. Since plumbing could be stacked over the locker rooms below the laundry room, this is possible. Beams in this bay will span parallel to girders and frame into beams that frame into the columns.

In this layout the girders will not be shared by beams of adjacent bays. This is not a fail-safe solution and further analysis for this particular case would have to be completed to determine the best solution.

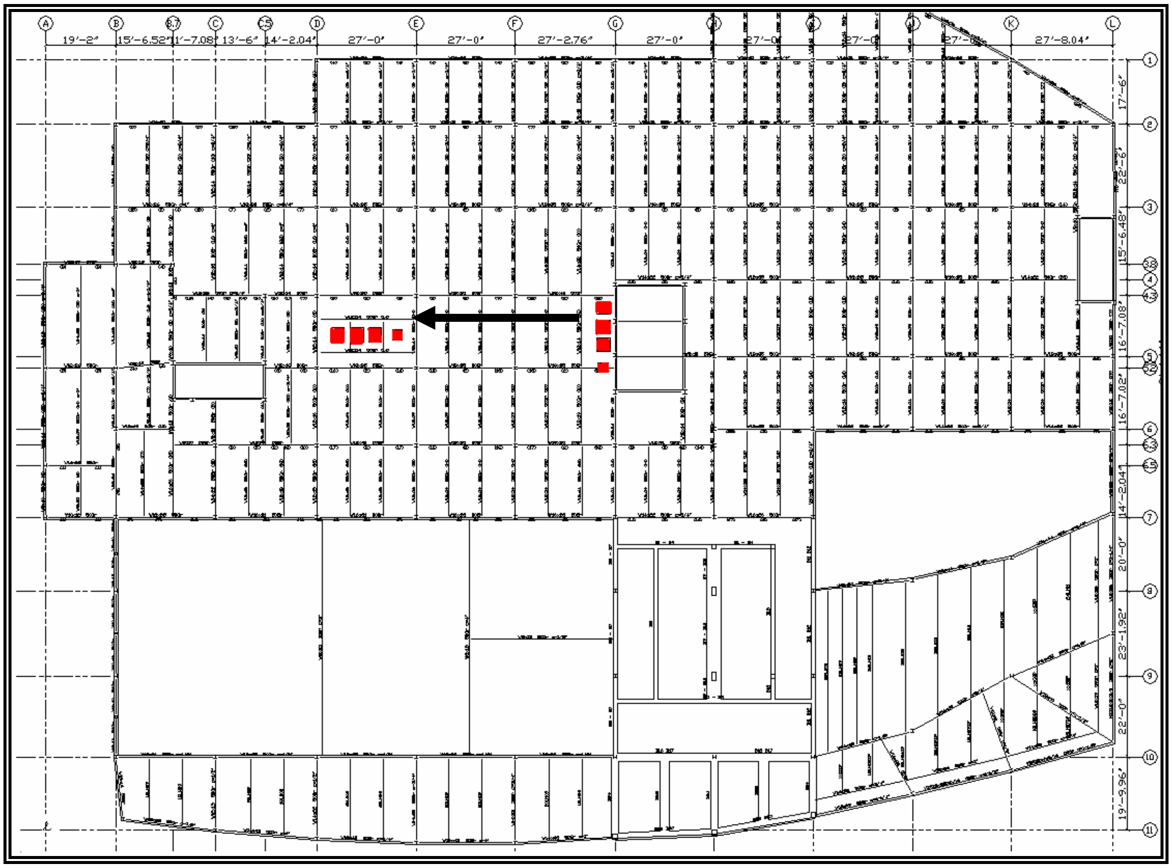


Figure 11: Movement of washers and frame layout

Columns

Columns in the 'BWI Hilton Hotel' were designed for ASD using RAM structural system and manual calculations in accordance with the Steel Construction Manual 13th ed. Columns in RAM Structural system were modeled having no eccentric loads. Therefore columns are subjected to pure axial loading and be can designed without an interaction equation. Columns subjected to this type of loading were designed using Tables 4-1 in the Steel Construction Manual 13th ed. assuming a k =1.0. Columns were modeled to be spliced every 3rd floor.

Girder-Slab floors (4-11) utilize a tree connection to allow for larger spans. A typical detail of this connection type can be seen in Figure 6. This connection type subjects columns to combined loading of axial and bending. Since this connection could not be modeled in RAM, hand calculations were performed to determine the bending moment induced on the column. All spans using this connection are equivalent therefore one span was used to determined the max bending moment. Computations produced a design bending moment of 52.5 ft-kips, calculations may be found in the appendix.

Interaction equation H1-1a governed the designed for all combined loaded columns.

Equation H1-1a: $\frac{\mathrm{Pr}}{Pc} + \frac{8}{9}\left(\frac{Mr}{Mc}\right) \leq 1$

Where P_r is the axial load

M_r is the bending moment

P_c is the axial strength of the column

M_c is the bending strength of the column

Columns designed by RAM were checked manually for the loading condition and value of the interaction equation. If column interaction equation values were not less than 1, then columns were resized accordingly and updated in RAM. Some of RAM's original designs produced shapes that were slender according to AISC. Columns sizes were manually updated accordingly. Figure 12 shows an elevation of column line F-5.2. Interior gravity columns are typical for this elevation.

For column line F-5.2 designs produced the following sizes:

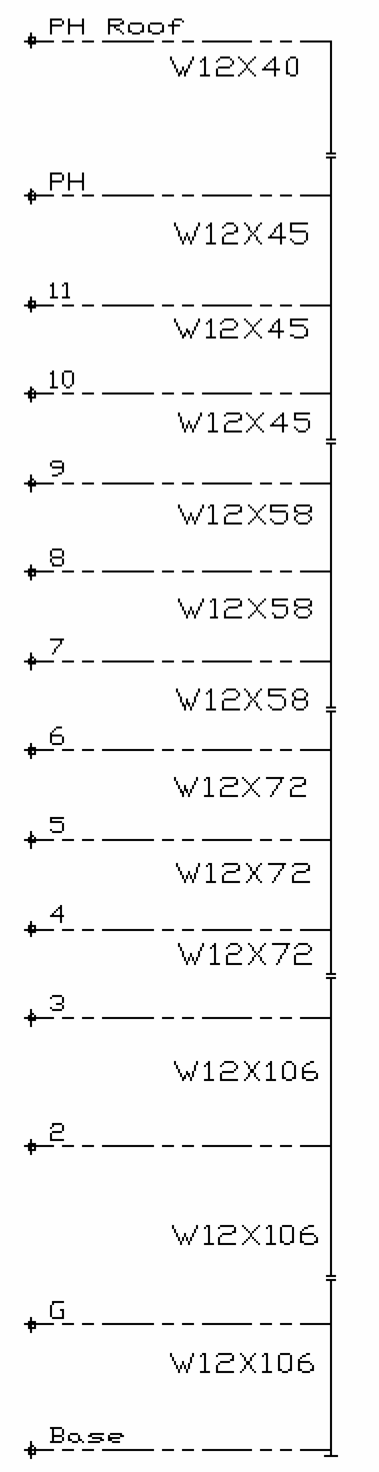


Figure 12: Column line F-5.2

Connections

All beams and girders resisting gravity loads in the composite beam system were modeled as pin-pin, therefore connections would need to be designed as shear connections. In a typical bay there are three connections types that need to be addressed: connection between beam web to girder web (1), connection of girder web to column web (2), and the final connection would be from beam web to column flange (3).

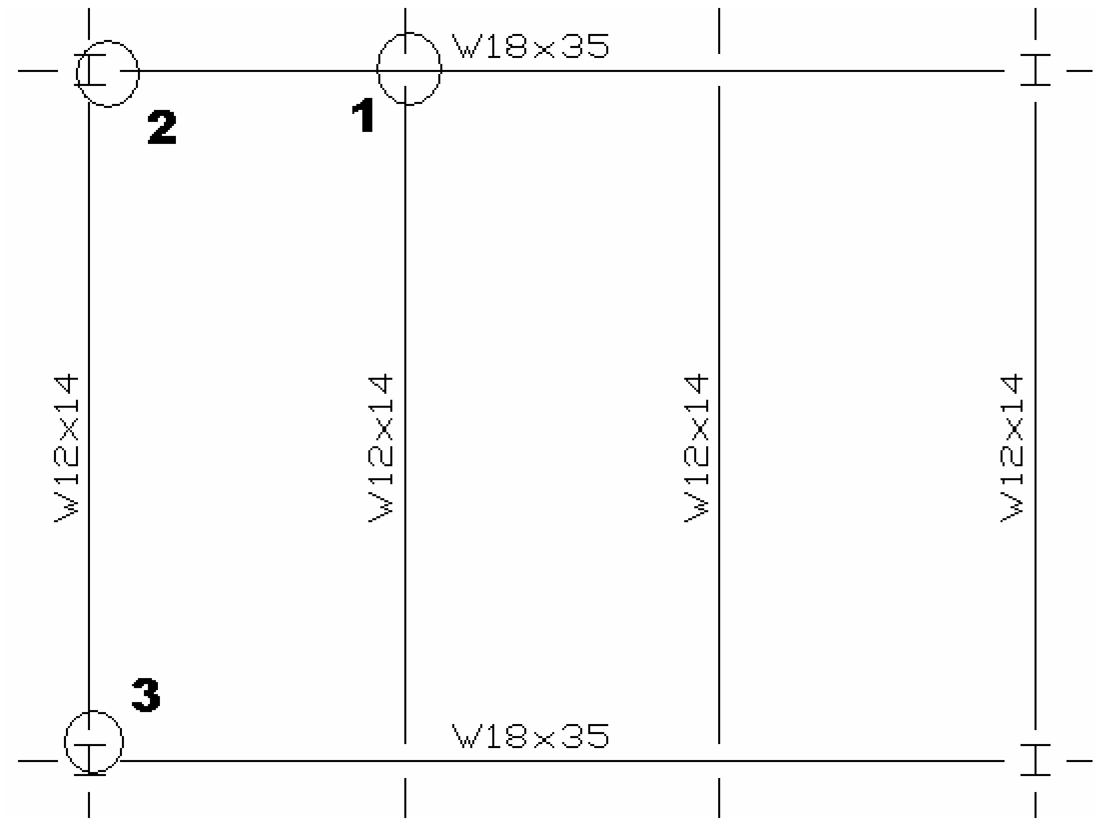
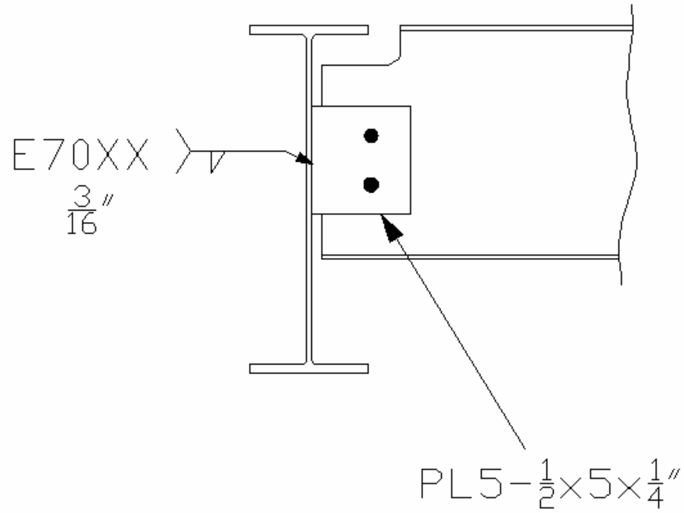
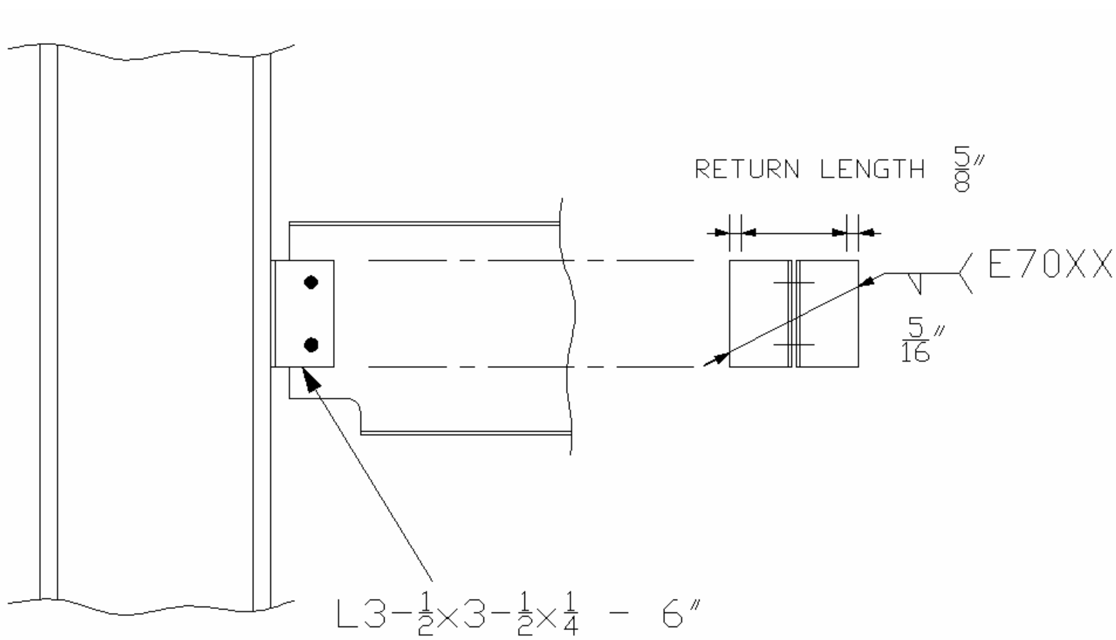


Figure 13: Typical connection plan



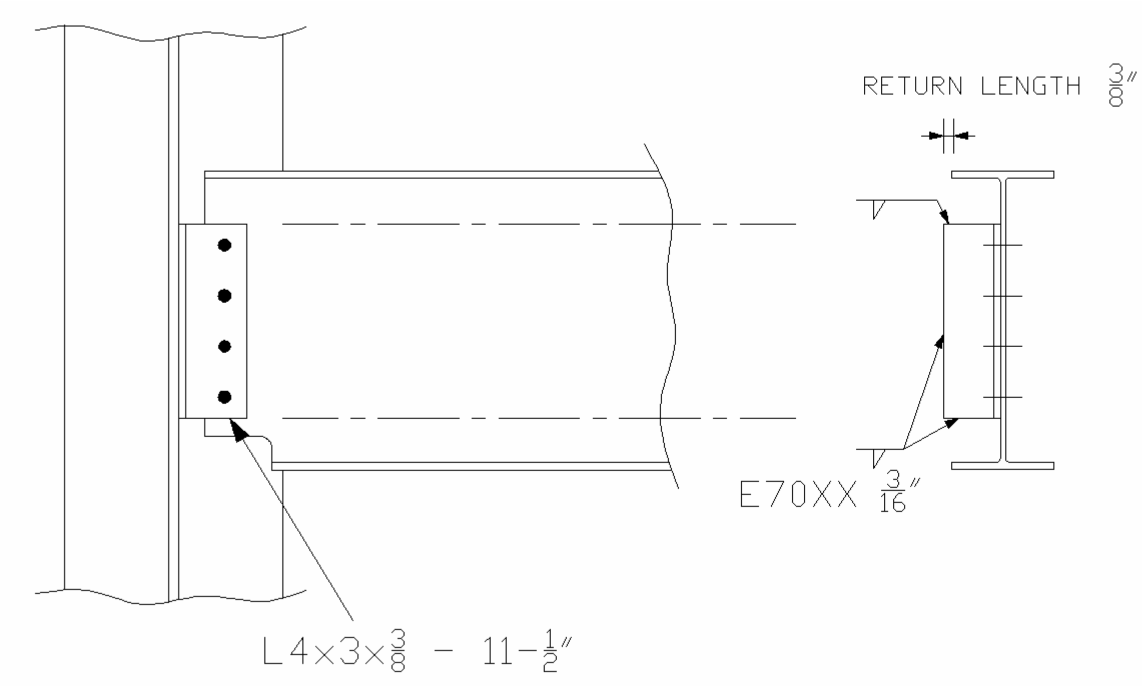
Connection 1: Beam web to girder web

A 5-1/2” x 5”x ¼” shear tab with 2- ¾” A325 bolts will be used for connections between beams and girders. The beam will be coped at the top to allow for connection. A 3/16” E70XX fillet weld will be used to connect the shear tab to the girder web.



Connection 2: Girder web to column web

Connection between girder web and column web will utilize a single angle, L4 x 3 x 3/8, 11-1/2" long with 4- ¾" A-325N bolts, and a 3/16" weld to the column web.



Connection 3: Beam web to column flange

The third type of connection will be a double-angle bolted to the beam and welded to the supporting column flange. A L3-1/2" x 3-1/2" x ¼", 6 " long will be used with 2- ¾" A325 N bolts and a 5/16" weld to the column flange. The beam will be coped at the bottom for constructability. Design aids in chapter 10 and eccentric weld tables in chapter 8 of the ASIC Manual 13th ed. were used in connection design. Calculations may be found in the Appendix.

Foundation Redesign

A footing was redesigned for column line F-5.2. Changing from a much heavier concrete system to a lighter steel system should allow for utilization of smaller footings. At the base of the ground column a force a 675 kips must be transferred to the ground. In accordance with the geotechnical report an allowable bearing capacity of 12,000 psf can be used for foundations placed on undisturbed soil. Designs produced a column size of W12x106. A base plate designed in accordance with AISC Manual 13th edition ASD produced a size of a 26" square plate 2-1/2" thick. Column will be welded to the base plate and the plate would have to be attached to the concrete pier by four anchor bolts. The concrete pier would then transfer the axial force to the footing.

The footing designed was an 8 ft square footing 29” thick reinforced by (12) #8 bar each way. Compared to the existing F-5.2 footing (10 ft square footing 40” thick), this is a decrease in concrete volume by 53%. Two other footings were sized for the steel structure, at column line D-3, and J-6. Both resized footings gave a decrease in concrete volume by 50% and 63% respectively to the existing footings. This trend of decreased volume will be assumed for all footings. Completing a volume take-off of the existing footings, then assuming a 50% reduction for the steel structure, produced an overall volume of concrete savings for the footings. Footings under the ‘adjacent structure’ were not accounted for in the take-off since this part of the structure will remain constant. The existing footing volume of those counted in the take-off is 390 cubic yards. Using only 50% of this total volume for the steel structure, 195 cubic yards of concrete will be used for footings. According to R.S. Means 2007 Cost Data, which prices square concrete footings at $370 per cubic yard, a savings of $72,150.00 will be made.

Structural Redesign
Lateral System

Lateral Loads

Wind

Winds loads were computed in accordance with ACSE7-05 Chapter 6. Basic wind speeds for the Baltimore were taken as 90 mph with a building exposure category B. Parameters were inputted into a RAM frame model and RAM calculated wind forces using ASCE7-05. A comparison of hand computations to those calculated by RAM may be seen in Table 4 below.

Wind Applied Story Forces (k)					
Ht. (ft)	Level	Manual N/S	RAM Output N/S	MANUAL E/W	RAM Output E/W
129.67	ph roof	11.60	11.3	23.95	24.75
114	ph floor	19.88	19.02	52.99	52.78
103	11th floor	15.06	14.51	52.81	51.43
94	10th floor	13.31	12.79	46.84	45.52
85	9th floor	12.91	12.52	45.74	44.76
76	8th flooor	12.62	12.23	44.91	43.93
67	7 floor	12.28	11.91	43.94	43.03
58	6 floor	11.88	11.56	42.84	42.03
49	5th floor	11.49	11.17	41.73	40.92
40	4th floor	11.38	10.83	42.04	39.6
31	3rd floor	20.16	19.34	54.26	47.56
18	2nd floor	43.38	43.86	72.00	66.83
		195.95	191.04	564.05	543.14

Table 4: Applied wind force comparison

Applied forces computed are within 4% of each other which will be acceptable for analysis and design. Allowing RAM to compute the 4 different load cases given in Figure 6-9 of ASCE7-05 Ch.6, the controlling load case was Case 1.

Seismic

Seismic loads applied to the building were computed in accordance with ACSE7-05 chapters 11, 12 and 19. ‘BWI Hilton’ has a seismic design category B, therefore the method of seismic analysis procedure allowed by code is the Equivalent Lateral Force. Again the parameters were input into RAM Frame and RAM calculated the ELF forces on the building. A comparison of these forces may be seen in Table 5 below.

Equivalent Lateral Forces (k)				
Ht. (ft)	Level	W_x (k)	Manual Force (k)	RAM Output Force (k)
129.67	ph roof	319.2	26.02	26.5
114	ph floor	872.3	61.30	62.57
103	11th floor	941.7	58.85	60.2
94	10th floor	938.1	52.73	54.07
85	9th floor	939.0	46.96	48.28
76	8th flooor	939.8	41.26	42.56
67	7 floor	939.9	35.62	36.89
58	6 floor	941.8	30.13	31.39
49	5th floor	943.7	24.75	25.98
40	4th floor	943.7	19.44	20.64
31	3rd floor	1862.1	28.23	30.49
18	2nd floor	3198.5	28.22	28.27
		13779.8	453.5	467.8

Table 5: Equivalent Lateral Force comparison

Base shear was reduced from 695 K to 470 K by changing from a concrete structural system to a steel system. This is a 32% reduction of equivalent applied seismic forces.

Braced Frames

To keep consistent with the change of the gravity system from concrete to steel, existing shear walls were replaced with braced frames. A layout of the braces frames can be seen in Figure 14. Brace frame #11 extends vertically only to the second floor.

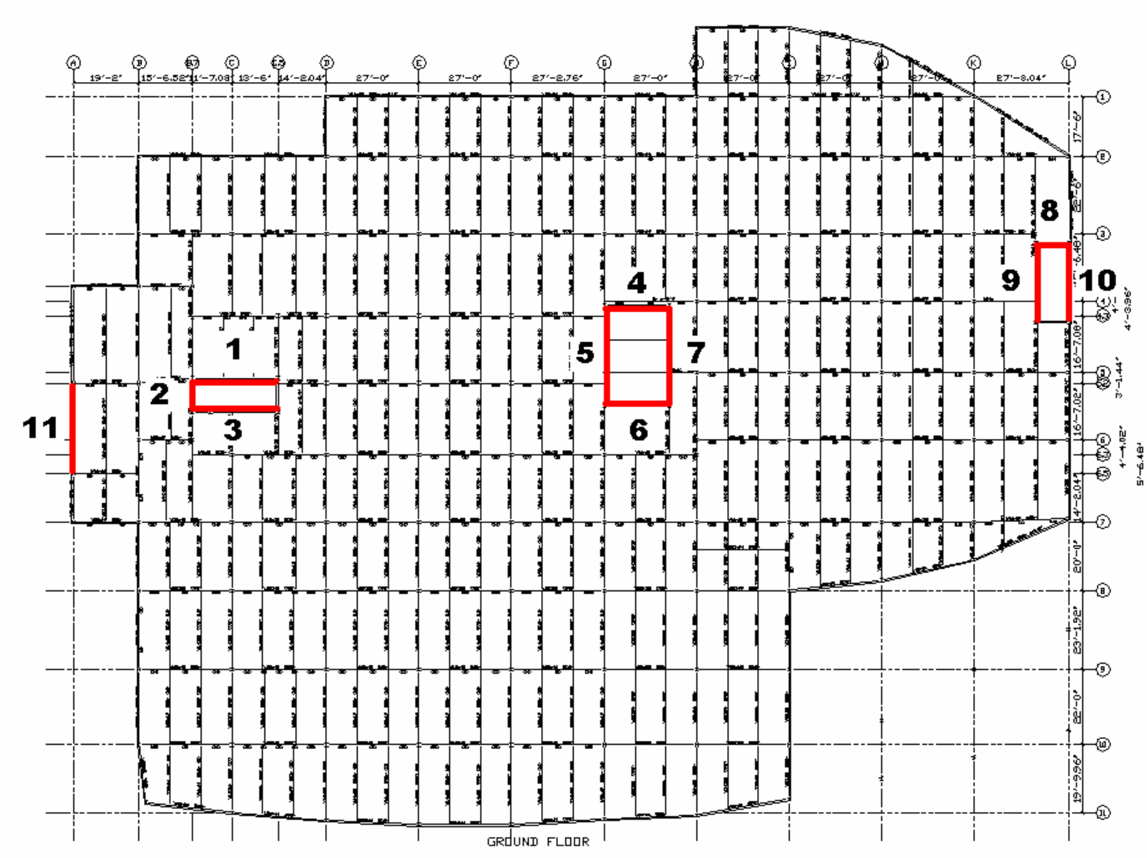


Figure 14: Braced frame layout

Three types of layouts were used for braces were: a chevron brace using double angles or HSS members, a separated chevron brace using HSS members, and cross braces using HSS shapes. Elevations for brace frames in each direction can be seen in Figures 15 and 16. Sizes of members can be found in a table located in the Appendix.

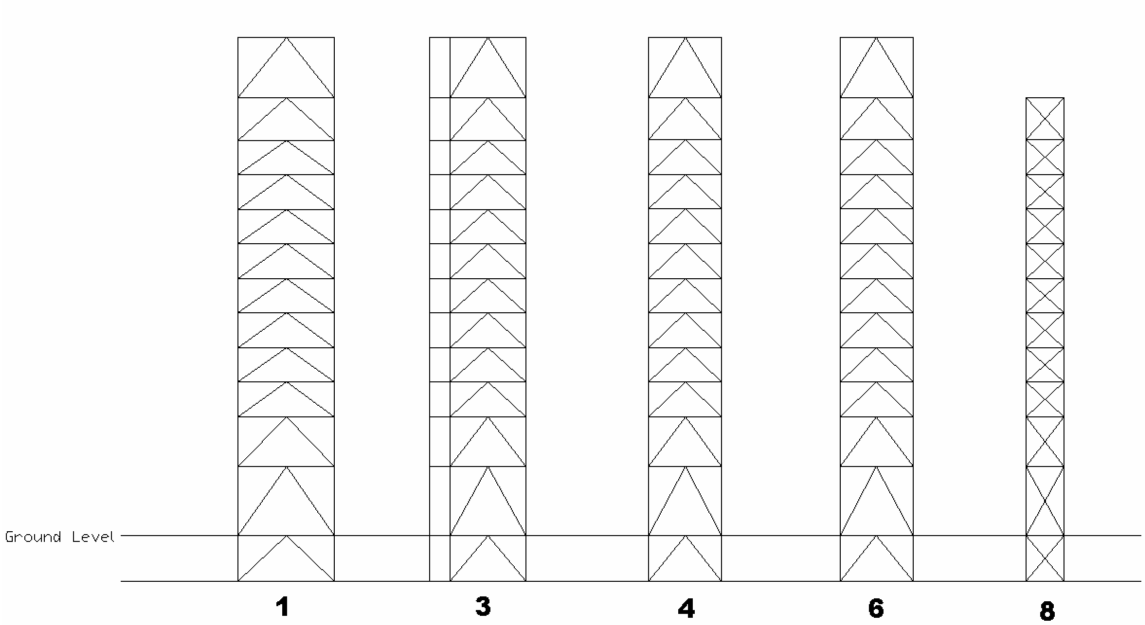


Figure 15: Elevation of Braced Frames in East-West direction

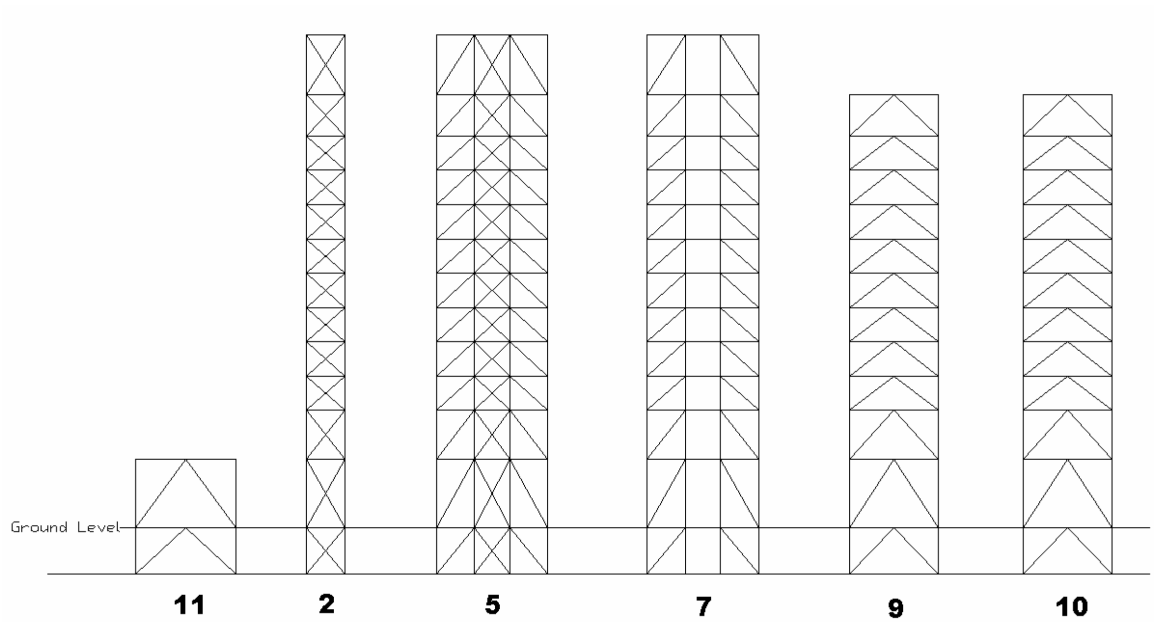
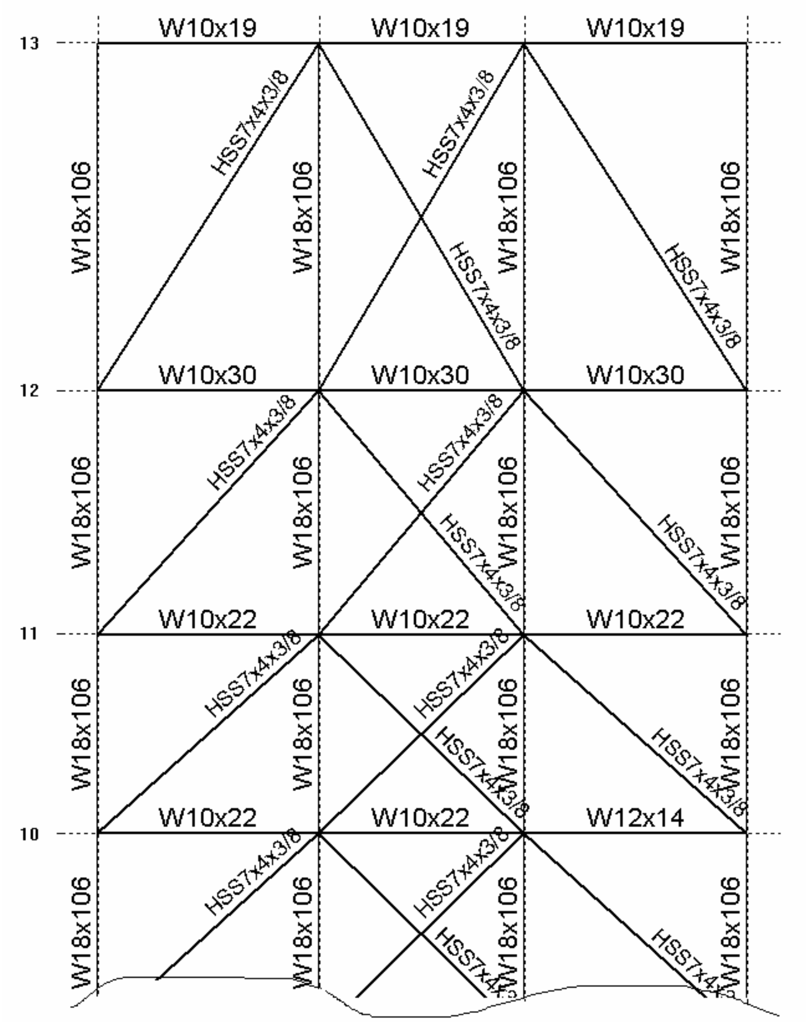


Figure 16: Elevation of Braced Frames in North-South direction

Initial braced frame sizes were found using RAM Advanse. Forces applied to advance model were taken from wind forces manually computed. The distribution factors for each frame were taken from the relative stiffness' of the concrete shear walls computed in previous technical assignments. Frames were designed using ASD load combinations taken from ASCE7-05. Initial sizes were then input into RAM structural system to determine overall building displacement and torsion. Overall building displacement was found to be the controlling design factor. Using an industry standard of L/400 for overall building displacement equated to a displacement limit of 3.9". Cross-braces were added to the interior opening frame #5 increase rigidity of the system. Cross-braces could not be added to the interior opening of frame #7 because egress to the elevators is through this opening. Figure 17 shows members sizes of frame #5 from floors: 10 to the penthouse roof, and then from: foundation to 3rd floor.



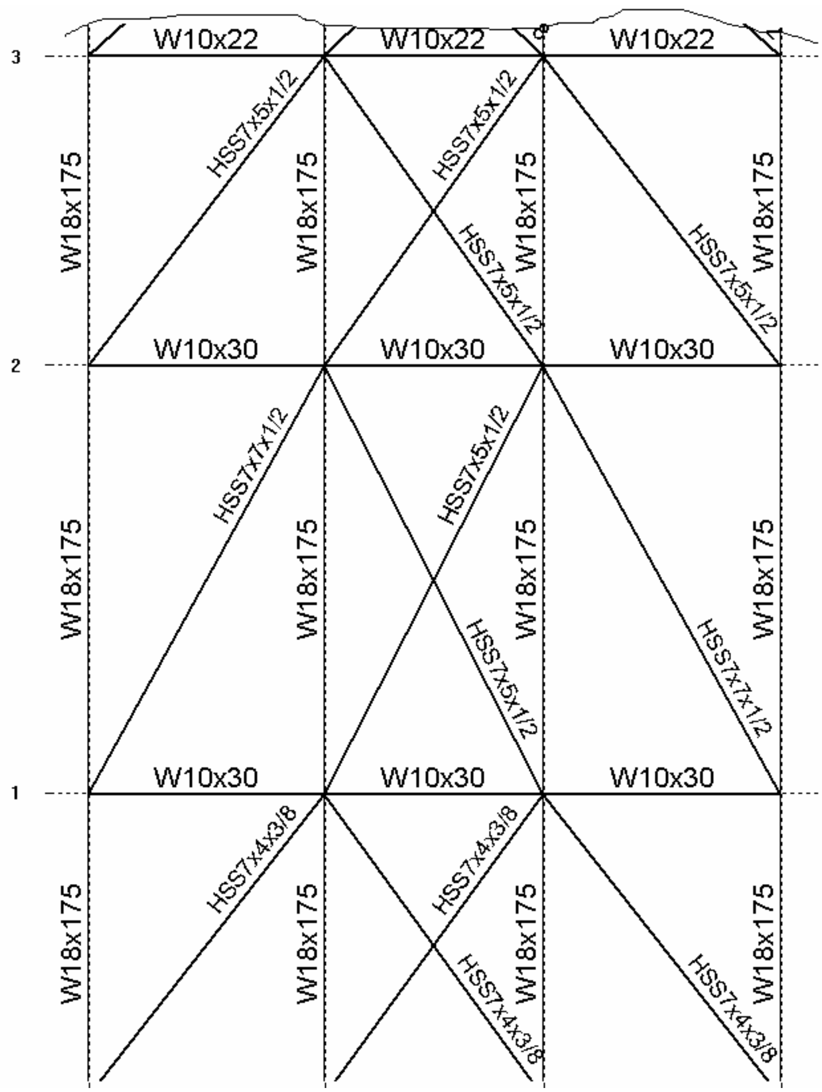


Figure 17: Partial elevation of Frame 5

Columns sizes of the frames were also increased. This became an iterative process with sizing members and checking overall building displacement. When the displacement was within limits, members were then checked using RAM steel check and ASD load combinations from ASCE7-05. Controlling load combinations for members varied throughout the frames. Members were sized accordingly to meet code requirements.

Story drift caused by seismic loading was within acceptable code values. The max story displacement occurred at the penthouse roof with a drift of 0.2691". Multiplying by a Cd value of 3.25 to get the code drift value produced a story drift of

0.875”. The multiplier for story height below story ‘x’ is 0.02 for Occupancy category II and braced frame resisting system. With a h_{sx} value of 132” the max allowed story drift by code is 2.64”, which is significantly greater than 0.875”. Torsional irregularity can be ignored by code since the ‘BWI Hilton’ falls in the seismic design category B.

Overturning moment of the lateral system was checked for punching shear of the frame columns through the mat slab foundation. Calculations require the mat slab at the central elevator core to be 29” thick. The existing mat foundation is 36” thick and therefore can resist the punching shear.

RAM Structural System Model

The ‘BWI’ Hilton structural steel system was modeled in RAM. Typical Girder-Slab floors 4-11 were modeled in RAM by using a one-way deck with the same weight as the specified hollow core concrete plank with parent beam sizes for the girders. Girder-slab members were not designed in RAM, but a somewhat accurate representative of the system needed to be including in the model to determine loads on columns due to the system weight. Floors ground through 3 were designed as composite steel beam and concrete slab system. Loads prescribed by the EOR were used on floors or portions of floors in the model. The “adjacent area’ was modeled the same as the original system. Materials and layouts were not changed. This area was modeled to gain an accurate deflection and torsional moment created by building shape and lateral forces. A 3D image of the model can be seen in Figure 18.

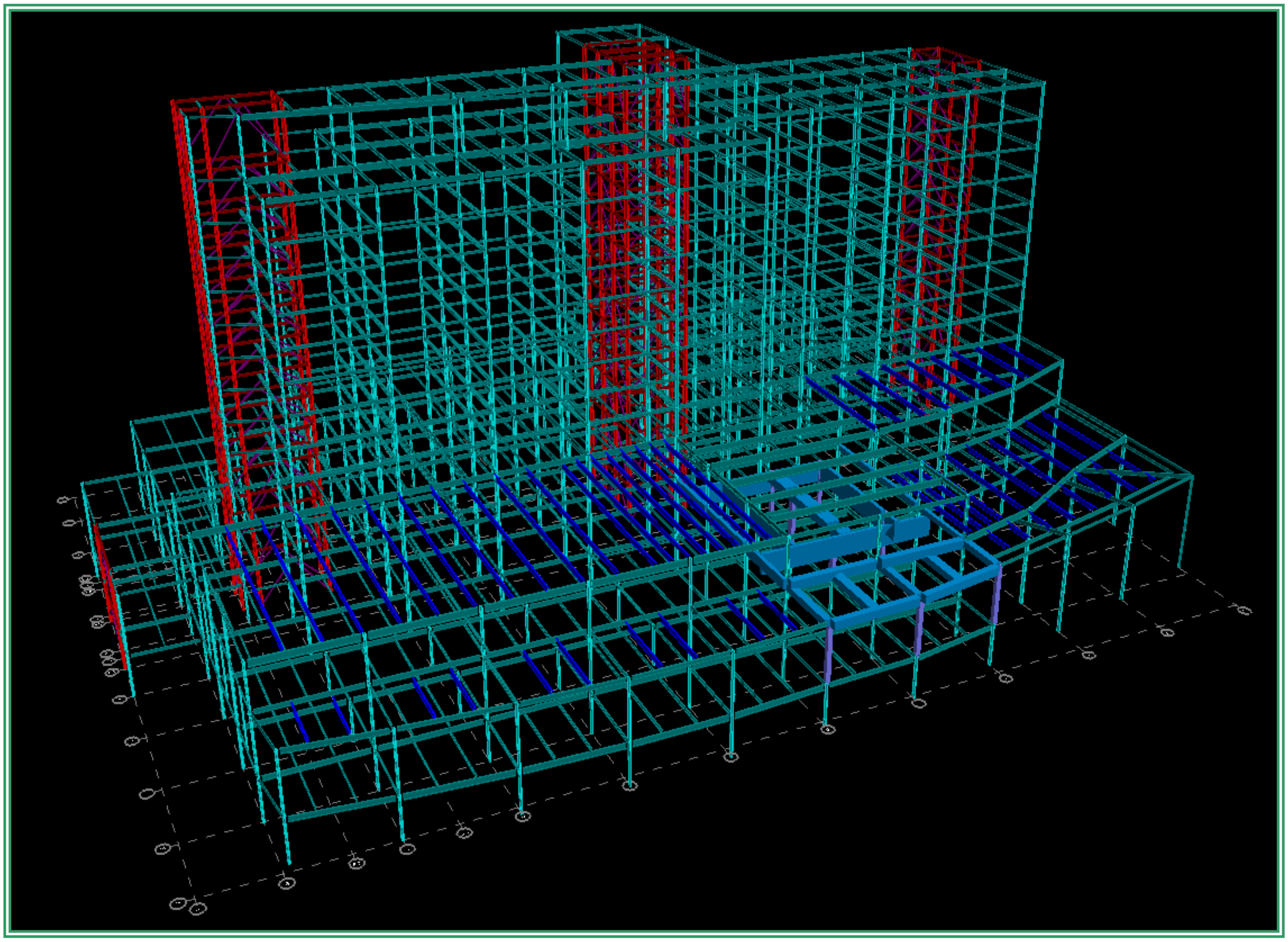


Figure 18: 3D RAM Structural System Model