Structural Breadth

Landmark Façade

As mentioned before, the building has two distinct pieces: the historic landmark façade and the new office tower. The two portions are brought together by a skylighting system and clearstory that wraps the base of the new tower. The 200 square foot base building will house lobby space and 80 foot atrium space.

Preserving the 1928 landmark façade posed several challenges for the structural design team and construction manager. Demolition of the existing Hearts Headquarters called for the six story building to be completely gutted leaving three remaining facades on 56th and 57th street and Eighth Avenue. Aside from simply constructing the new tower, the construction team also introduced a new framing plan to support the exterior walls and upgraded the existing façade to meet the seismic requirements of the New York City Building Code.

Foundation

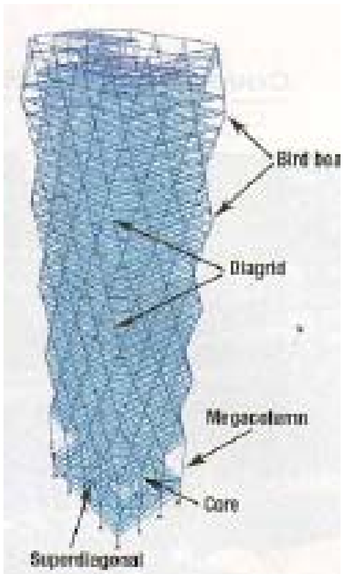
Tests done at the site revealed that there is a sharp drop in the elevation of the rock below grade. The rock elevation varied from 30 feet down to only a few feet in some spots. Because of this variation in elevation, half of the tower is supported by spread footings on rock, while the other half is supported by caissons embedded into the rock below.

Lateral System

Three sides of the building are open to the street as mentioned above, however the fourth face is against an existing high-rise to the west. From an architectural program standpoint, the most logical placement for the service corridor was toward the west side of the tower, since having a window view in these areas is unnecessary. Locating the corridor in this offset manner however eliminated the benefit of using the core as the "main spine of the tower" (Rahimian 2006). To address the issue of instability, the structural design team developed a plan to use the perimeter structure to create the needed stability. The system design resulted in a triangulated system of horizontal A-frames.

Secondary Lateral System

Although the primary lateral system will produces the needed amount of strength and stiffness, the diagonal elements need additional bracing between the 4o foot nodal modules. The goal of the secondary lateral system is to provide stability for gravitational loads and construction tolerances at each level. The secondary lateral system in this case is a braced frame at the service core.



Diagrid

The most striking feature of the Hearst Tower is the diagrid design that forms a triangular curtain wall façade. Starting at the tenth floor, each triangular glass pane is 4 stories high and the intersections of these panes form diagrid nodes. The nodes, set on 40 foot modules, then act to redirect member forces. Essentially, all four faces come together at chamfered "Bird's Mouth" corners and create a very efficient tube structure.

The office tower begins at the tenth floor and utilizes a composite steel and concrete flooring system. Each floor has 40 foot column free spans to allow for maximum open office planning. The office floors are typically 16,000 sf in area with 9.5' ceilings.

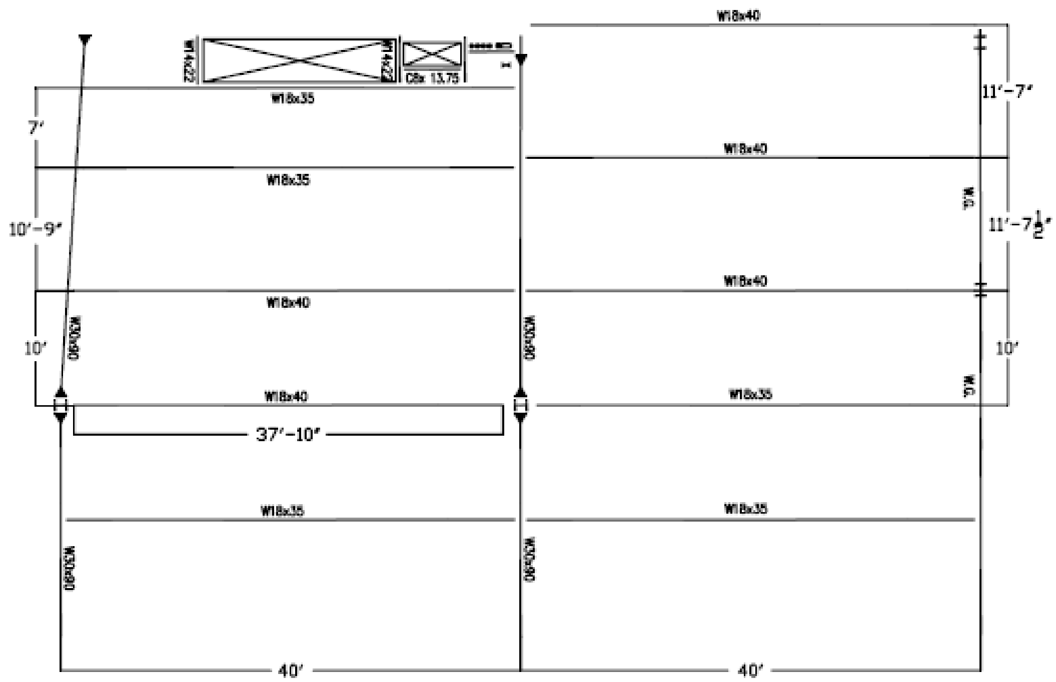
Having no vertical members on the façade allowed for a savings of 2,000 tons of steel. In all, the diagrid uses approximately 10,000 tons of wide flange rolled sections.

The left hand side of the **Figure 1s** below shows an interior bay of the Hearst Tower. Since the proposed mechanical design calls for radiant flooring, checking the fire rating for the new slab and subsequent loading on the members is necessary.

Columns, Mega Columns, and Super Diagonals

The diagrid façade is used to transfer loads at the tenth floor to 12 perimeter megacolumns. These megacolumns have an unbraced length of 85 feet and continue all the way to the foundation. The megacolumn system is designed to respond to the large unbraced height. Eight-ninety foot super diagonals slope inward from the megacolumn nodes on the third floor to the columns lines at floor ten. Additional superdiagonals are provided below floor ten to assist the composite core system with stability. The megacolumns are made out of built-up steel tube sections filled with concrete.

Figure 1s Typical Interior Bay



Procedure for Checking

Using the Vulcraft Steel Deck and Floor Deck Catalog, under the Section titled "Floor-Ceiling Assemblies with Composite Deck," 3VLI deck was chosen to satisfy the 2 hour fire rating required by the 2003 International Fire Code (IFC 2003). Since the current slab is 6" deep, an additional 2" of concrete is needed to protect the tubes.

Once this deck was selected, a check was performed to make sure the 19 gauge deck used currently in the Hearst Tower was sufficient to hold the additional 2" of slab. With the additional slab weight, the total load is 95 PSF. According to the deck catalog, and looking at the 3VLI19 deck with 6" slab (to be conservative, since top 2" is not structural), the total superimposed live load that can be supported for 12' spans is 110 PSF.

After checking the deck, four interior beams were checked. Using the Load Resistance Factor Design Method (LRFD Method), in conjunction with ASCE7-2005, it was determined that all of the beam sizes need to increase to withstand the additional loading. Since this is a structural breadth study, the beams were checked as if they are noncomposite with the slab and deck. A table of the existing beams and the required beams for the new design are shown below in **Table 1s**. Appendix A contains hand calculations for each beam check performed.

Table 1s Beam Sizes

Existing Structure	New Structure
W18X35	W21X48
W18X40	W21X55
W18X40	W21X55
W18X35	W18X40

After the beams were checked and resized as necessary, the girder on the right hand side of Figure # was also checked. Hand calculations of this procedure are also included in the appendix of this report.

Cost Considerations

The purpose of this analysis was not necessarily to determine cost savings or increases, but rather to show the impact the changes in one building system (Mechanical in this case) may have on other systems of the building. For the case of the Hearst Tower, using an estimate of $1/lbm, the increased beam sizes only result in an increase of $48/ typical bay. Since this number is small in comparison to the $38 million dollar steel contract, structural cost will not serve as an argument for or against the proposed mechanical system redesign.

Structural Breadth

Appendix A

Hand Calculations for Beam Checks
Hand Calculations for Girder Checks

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Building ______ Project No. ______
Subject 18x40 TO.4 Date ______
Sheet No. ____ of ____ Computed by ______ Checked by ______

LOADS

DEAD
57 PSF (3VLI19 6" SLAB)
2 PSF (CARPET)
2 PSF (FINISH CEILING)
10 PSF (MEP)
24 PSF (XTRA 2" CONCRETE)

LIVE
80 PSF (OPEN OFFICE W/ CORRIDOR THROUGHOUT)

$$A_T = (40')\left(\frac{10'+10'}{2}\right) = 400\ FT^2$$

$$T_W = \frac{10'+10'}{2} = 10'$$

$K_{LL} = 2$ (T.4-2 ASCE7-2005 INTERIOR BEAM)

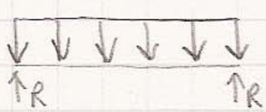
LL REDUCTION (B/C $K_{LL} \cdot A_T > 400$ SF)

$$L = L_o\left(.25 + \frac{15}{\sqrt{2(400)}}\right) = 0.78 L_o > 0.4 \therefore \text{USE } \underline{\underline{0.78 L_o}}.$$

$$L = 0.78(80\ PSF) = 62.4\ PSF$$

$$(w_u)_{L+D} = 1.2(95) + 1.6(62.4) = 213.84\ PSF$$

$$W_u = T_W w_u = (10')(213.84\ P/SF) = 2.14\ KLF$$



$$R = \frac{(2.14\ KLF)(40\ FT)}{2} = 42.8^K$$

$$M_{MAX} = \frac{WL^2}{8} = \frac{(2.14\ KLF)(40\ FT)^2}{8} = 428\ FT\text{-}K$$

CURRENT = 18x40 $\phi M_N = 294$ FT-K

$\boxed{\text{TRY W21x55}} \Rightarrow \phi M_N = 473\ FTK > 428\ FTK \Rightarrow \underline{\underline{OK}}$

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Building ______ Project No. ______
Subject W18x40 (2ND one) Date ______
Sheet No. ____ of ____ Computed by ______ Checked by ______

LOADS

DEAD	LIVE
51 PSF (3VLI19 6" SLAB)	80 PSF (OPEN OFFICE W/ CORRIDOR) THROUGHOUT
2 PSF (CARPET)	
2 PSF (FINISH CEILING)	
10 PSF (NEP)	
24 PSF (XTRA 2" CONCRETE)	

$$A_T = (39.33')\left(\frac{10' + 10.75'}{2}\right) = 408.05\ FT^2$$

$$T_W = \frac{10' + 10.75'}{2} = 10.375\ FT$$

$K_{LL} = 2$ (T4-2 ASCE-7-2005 INTERIOR BEAM)

LL REDUCTION (B/C $K_{LL} \cdot A_T > 400$ SF)

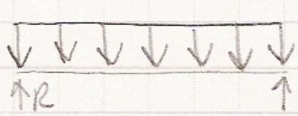
$$L = L_0\left(0.25 + \frac{15}{\sqrt{K_{LL} A_T}}\right) = 0.775 L_0 > 0.4 \therefore \text{USE } 0.775 L_0$$

$$L = 0.775(80\ PSF) = 62\ PSF$$

$$(W_u)_{L+D} = 1.2(95\ PSF) + 1.6(62\ PSF)$$

$$W_u = 213.2\ PSF$$

$$W_u = T_W W_u = (10.375\ FT)(213.2\ PSF) = 2.21\ KLF$$



$$R = \frac{2.21\ KLF(39.33')}{2} = 43.46\ K$$

$$M_{MAX} = \frac{WL^2}{8} = \frac{(2.21\ KLF)(39.33)^2}{8} = 427.3\ FT\text{-}K$$

CURRENT W18x40 $\phi M_N = 294$ FT-K $\qquad \phi M_N = 0.9 \cdot F_y \cdot Z$

$\boxed{\text{NEED W21x55}}$ $\phi M_N = 473$ FT-K

Building ______ Project No. ______
Subject 18x35 (1ST) ______ Date ______
Sheet No. ____ of ____ Computed by ______ Checked by ______

LOADS

DEAD
51 PSF (3VLI19 6" SLAB)
2 PSF (CARPET)
2 PSF (FINISH CLG.)
10 PSF (MEP)
24 PSF (XTRA 2" SLAB)

LIVE
80 PSF

$$A_T = (38.64')\left(\frac{10.75' + 7.0'}{2}\right) = 343\ FT^2$$

$$T_W = \left(\frac{10.75' + 7.0'}{2}\right) = 8.875\ FT$$

$K_{LL} = 2$ (T.4-2 ASCE7-2005 Interior BEAM)

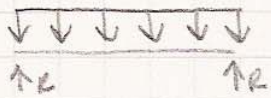
LL Reduction (BC $K_{LL} \cdot A_T > 400\ FT^2$)

$$L = L_o\left(0.25 + \frac{15}{\sqrt{2 \cdot (343)}}\right) = 0.823 L_o > 0.4 \therefore \text{use } 0.823 L_o$$

$$L = 0.823(80\ PSF) = 65.8\ PSF$$

$$\omega_{u_{L+D}} = 1.2(95) + 1.6(65.8) = 219.3\ PSF.$$

$$W_u = T_W \omega_u = (8.875')(219.3\ PSF) = 1.95\ KLF$$



$$R = \frac{(1.95\ KLF)(38.64\ FT)}{2} = 37.67\ K$$

$$M_{MAX} = \frac{W_u L^2}{8} = \frac{(1.95\ KLF)(38.64\ FT)^2}{8} = 364\ FT\text{-}K$$

CURRENT: 18x35 $\phi M_N = 249$ FT-K

$\boxed{\text{TRY: W21x48}}$ $\phi M_N = 398\ FT\text{-}K > 364\ FT\text{-}K \Rightarrow$ OK!!

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Building ______________ Project No. ______

Subject 18x35 (2nd) Date ______

Sheet No. ____ of ____ Computed by ______________ Checked by ______________

LOADS
57 PSF (3VLI19 6" SLAB)
2 PSF (CARPET)
2 PSF (FINISH CEILING)
10 PSF (MEP)
24 PSF (xtra 2" CONCRETE)

LIVE
80 PSF (OPEN OFFICE/ corridor)

$A_T = (38.2')\left(\frac{7.0' + 6.0'}{2}\right) = 248.3 \text{ FT}^2$

$T_W = \left(\frac{7.0' + 6.0'}{2}\right) = 6.5'$

$K_{LL} = 2$ (T.4-2 ASCE7-2005 INTERIOR BEAM)

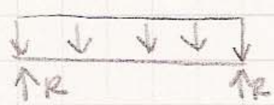
LL REDUCTION (B/C $K_{LL} \cdot A_T > 400 \text{ FT}^2$)

$L = L_o\left(0.25 + \frac{15}{\sqrt{2(248.3)}}\right) = 0.92 L_o > 0.4 \therefore$ USE $0.92 L_o$.

$L = 0.92(80 \text{ PSF}) = 73.8 \text{ PSF}$

$(\omega_u)_{L+D} = 1.2(95) + 1.6(73.8) = 232.1 \text{ PSF}$

$W_u = T_W \omega_u = (6.5')(232.1 \text{ PSF}) = 1.51 \text{ KLF}$



$R = \frac{(1.51 \text{ KLF})(38.2')}{2} = 28.84 \text{ K}$

$M_{max} = \frac{WL^2}{8} = \frac{1.51 \text{ KLF} (38.2 \text{ ft})^2}{8} = 275.4 \text{ K-FT}$

CURRENT: W18x35 $\phi M_N = 249$ K-FT

TRY: $\boxed{\text{W18x40}}$ => $\phi M_N = 294$ FT-K > 275 FT-K => OK!!

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