
Nicholas Szakelyhidi — Structural — Office Building — Washington, DC
M. Kevin Parfitt, Advisor — Spring 2006

# Office Building*

## Washington, DC

Nick Szakelyhidi — Structural — 2005

**Primary Project Team**

Owner – Louis Dreyfus Property Group
CM – Centex Construction
Architect – Kevin Roche John Dinkeloo and Associates
Electrical – Truland Systems Corporation
MEP/Fire Protection – Tolk, Inc
Structural – Tadjer-Cohen-Edelson Associates, Inc

**General**

- Urban Class A Office Building
- Multi-Use Design
- 382,091 Square feet above grade
- 12 Stories with 3 underground parking levels
- To be completed Mid 2007
- Design-Bid-Build Contract
- 50 Million Dollar Estimated Project Cost

**Structure**

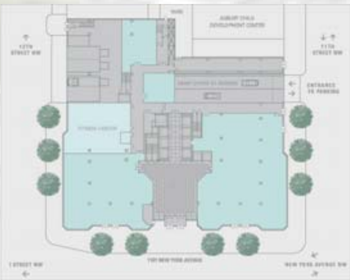
- Interior concrete piers with reinforced concrete columns
- 12" Two-Way PT Concrete slabs on main floors
- Perimeter curb strip footing
- Mixed office occupancy load
- 20" cantilever above 1st floor
- Column free facade

**Architecture**

- Colorless glass façade
- No columns visible from exterior
- 11 foot uninterrupted windows
- High quality finishes
- Dark granite base
- LEED rated design

**MEP**

- 8 watts/square foot power availability
- High speed fiber optic connections
- Variable air volume (VAV) system
- Independent tenant climate controls
- Prevailing natural light supplemented with fluorescent and recessed incandescent
- LEED rated enclosure
- 7 passenger and 1 freight elevators



*Buildng specifics withheld at owners request

ARCHITECTURAL ENGINEERING

http://www.arche.psu.edu/thesis/eportfolio/current/portfolios/nss131/

Nick Szakelyhidi
Office Building, Washington DC
Structural
M. K. Parfitt

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Nick Szakelyhidi
Structural
M.K. Parfitt
Office Building, Washington, DC
Final Report
4-3-06

All renderings courtesy of Dreyfus Property Group and KRJDA

# Executive Summary

The building being evaluated is a mixed use office building located in downtown Washington, DC. A continuous glass façade covers the exterior of the building. On 3 sides, there is a 20' cantilevered bay which allows the façade to be uninterrupted by vertical supports. The structure of this unique building is a cast in place two-way post-tensioned flat slab with drop panels at columns. Lateral forces are resisted by monolithic concrete moment frames in the north-south and east-west directions.

As an area of investigation, the structural system was designed using a composite steel equivalent. The thesis research assessed the design of this equivalent and the affect on gravity and lateral systems. The result was a decrease in building weight by over 300%. Seismic effects on the building were reduced significantly. Lateral stiffness was decreased by approximately half but the braced frames and moment frames were still able to bear the lateral forces due to the seismic force reduction. A consequence of the building composite steel framing system was an increase in floor depth. The original structure maintained a maximum depth of 24 inches, whereas the provisional steel structure saw a maximum depth of 26 ½ inches.

The cost comparison yielded a price estimate for the composite steel structure that closely rivaled the cost of the concrete structure. Composite steel construction proved to be a marginally quicker than the existing concrete frame. LEED analysis make certain that steel attain an equal or higher rating that the concrete equivalent.

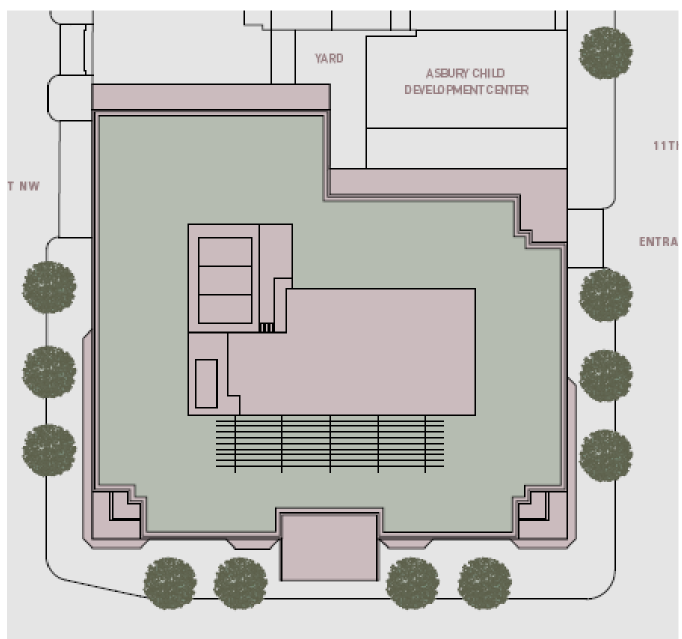
After all aspects of the project were considered, It was decided that there were not enough clear benefits using the composite steel system to warrant its use over the existing post-tensioned cast in place concrete structural system.


## GENERAL BUILDING STATISTICS

The northwest quadrant of Washington, DC is a vibrant and inspiring neighborhood. As part of a downtown revitalization, many new construction projects have been undertaken in the recent years. This building is a forerunner to upcoming projects, and must become exemplary of the quality construction to follow. The 12 story structure exudes elegance with its granite base and uninterrupted colorless glass façade. No expense was spared for the mixed use office interior either. Stone, wood and metal finishes accent the interior of this Class A rated building. These fine materials are also selected for their environmental friendliness. The glazing on the exterior is insulated and layered with a low-E coating to minimize solar gain in the summer. The building rests atop 3 levels of below grade parking and an occupied lower level. Typical floor areas are 31,115 square feet for a total leasable floor area of 393,000 square feet.

Being located in downtown Washington DC, strict height requirements are required per the zoning. A maximum building height of 12 stories is dictated by the DC Downtown Development (DD) and C-3-C High Bulk Major Business and Employment zones. Additional height restraints are assigned by the Downtown Development zoning overlay.

The property is owned by Louis Dreyfus Property Group. For the architectural design, the owner enlisted the distinguished Kevin Roche of Kevin Roche John Dinkeloo and Associates LLC. The structural design of this unconventional building was tasked to Tadjer Cohen Edelson Associates. Providing MEP services is Tolk Engineering, with Truland Systems performing the electrical work. Centex Construction is assuring everything comes together correctly and on time. Greenshape LLC will make sure that the designs incorporate necessary LEED items and the construction is carried out in a sustainable and ecologically friendly manner. The uniqueness of this building is exhibited in every aspect of the design and construction. An important feature is the column free façade on the primary sides of the site. This was accomplished by incorporating a 20 foot cantilever bay along three of the building faces. This will prove to be a crucial factor in the structural evaluation and alternate design of the structure. The cantilever is in effect on the east, west, and south sides of the property, as can be seen in the site plan.

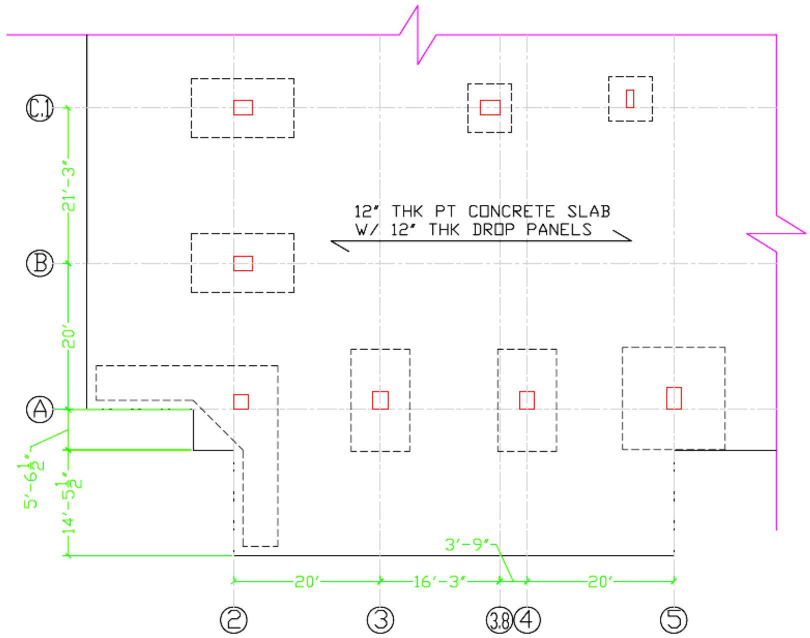


## FLOOR SYSTEM

The existing floor system for this office building has been designed as a post-tensioned flat slab with drop panels around columns. The bays are spaced at roughly 20' intervals. The primary building core houses the stair and elevator transportation systems. Stepped out 40' is an outer ring of columns. The building gravity loading and design is governed by IBC 2000 with a 2003 Washington DC code supplement. Concrete design complies with ACI code specifications. Design of the prestressed floor slabs was according to PCI design regulations. The prestressed slab is typically 12" thick with additional 12" drop panels around primary columns. Columns vary in size and strength as they move further toward the base, and toward the exterior of the structure. Compressive strength of the concrete varies from 4000 psi, up to 10,000 psi in base columns.

Below the occupied office floors is a lobby, lower concourse level, and below grade parking. The structure in this area is similar to the above floors in layout. The difference is that the system is a traditional reinforced concrete slab without post-tensioning. This can be used because there is less emphasis on limiting the thickness of the structural floor depth in the area, and it does not affect the overall height of the building.

On the Upper floors the structure is kept as thin as possible so that as many floors as possible can be fit within the height restriction. This maximizes rentable floors space and allows for premium finished ceiling heights in the tenant spaces. Post tensioning is an ideal system to meet such criteria. Post tensioning eliminates deflection problems that would plague the longer spans in the building. The floor depth can be kept to a minimum and materials familiar to the area, such as concrete, can be used extensively in the construction. The post tension slab does largely impact the possibility of future renovations of the space. The plan shown below is typical of the post-tensioned flat slab structural system used in the majority of the building.



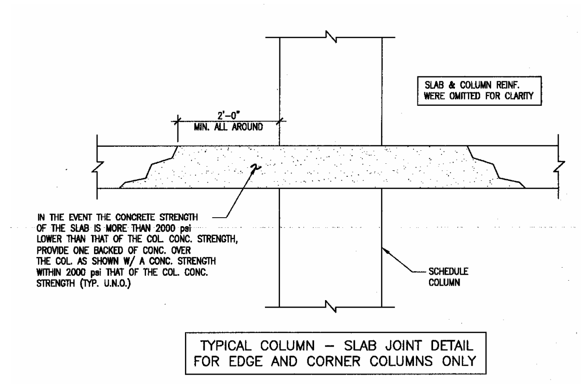

SW CORNER FLOORS 6-12 (TYP.)
NTS

## FOUNDATION

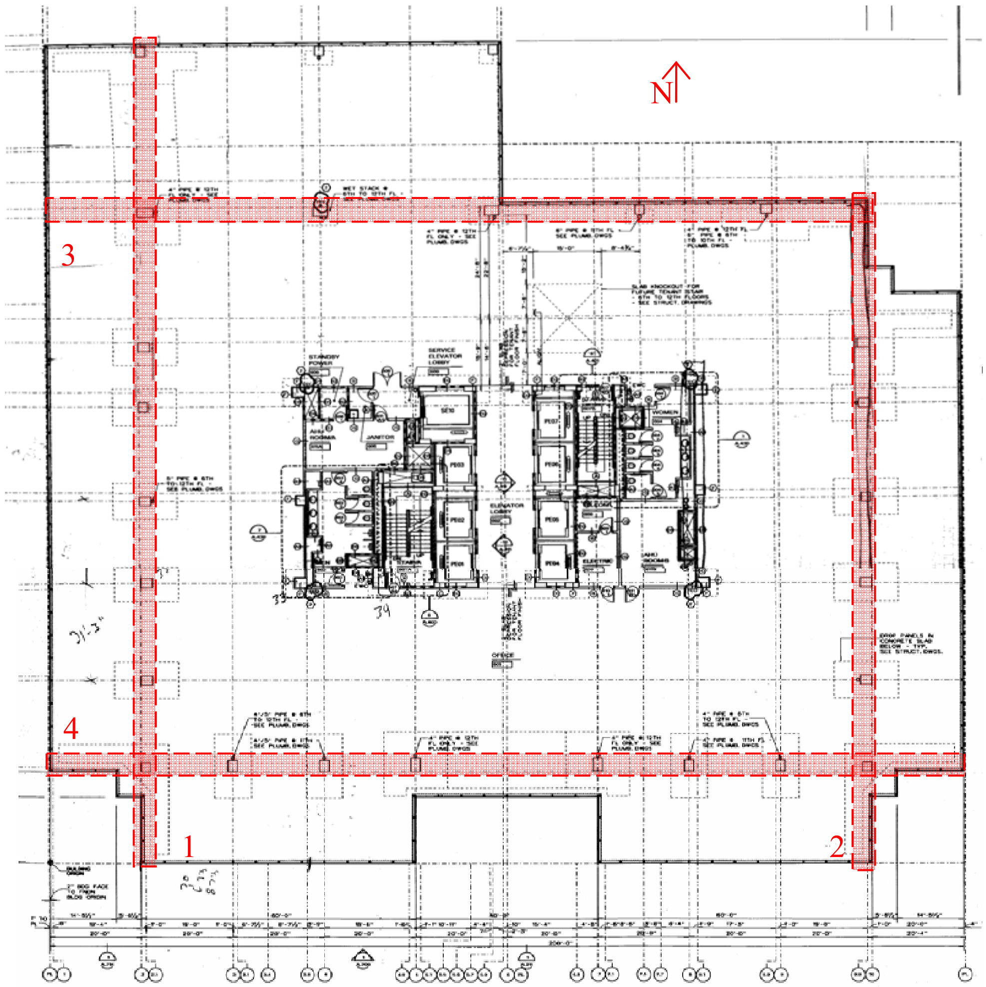
After passing through the below grade parking structure, the gravity columns terminate into conventional spread footings. The lowest level of parking rests on a traditional slab on grade floor. The SOG is 4" under parking areas. Where non-bearing walls exist above, the slab is thickened to 10". At the perimeter, the slab forms a strip footing with cropped toe which can be up to 24" deep. The footings for the outermost columns vary in thickness from 59" to a full 75" deep. Footing depths of 40" to 61" are needed for the interior columns. Minimum soil bearing requirements for the spread footings was designated to be 12,000 psf. The retaining wall in the underground structure is made of 16 inches of reinforced concrete. The huge structure weight of the building dictates such a significant foundation.

## LATERAL SYSTEM

The frame of this building is made completely of cast in place concrete. When properly reinforced, CIP concrete can be adequate to transfer shear and moment from beam slabs to columns. When the columns are aligned and of substantial size, it can be recognized that these elements act as a moment frame. To achieve this, the concrete slab-column connection must be cast monolithically. Typical details from the original building design are shown to illustrate this important characteristic. Reinforcing must also tie together all the elements of the connection. These can be difficult to construct, but are necessary to ensure that the elements act as a rigid moment frame. As the building is designed, the curtain wall receives lateral loads and then concentrates them to the slab edges. The slabs then act as a rigid diaphragm and distribute the load to the columns that make up the moment frames in the direction of the applied lateral loads. Loads are carried primarily by the rigid moment frames, according to the stiffness of each frame. Two major moment frames exist in the north-south direction, and two more work in the east-west direction. The location of the moment frames are shown on the plan below.




DETAIL 4-S.202




There are no marked shear walls on the plan. Typically elevator and stair cores are designed as shear walls. Also prominent CIP concrete or masonry walls are specifically intended to be shear walls. There are no significant walls of any type in this building. The stairs and elevators are enclosed by 8" masonry walls. These walls are not considered as shear walls, but instead are used to meet fire ratings for egress. Furthermore, the walls do not fully enclose the cores, and are not connected to each other. There are columns located at the corners of each stair and elevator core. Because they do not from a closed section, these regions cannot provide torsional resistance either. All lateral load (direct and torsional) resistance comes from the rigid moment frames in the building.

The lateral loads on the building are controlled by both wind and seismic. The seismic loading on the building, despite the low-moderate risk category, are quite large. This is due mainly to the substantial structure weight. Regardless of this, the structure meets drift requirements very nicely. This is thanks to the robust lateral resistance built into the CIP concrete frame.


## DESIGN CHALLENGE

The most difficult part of this study was finding a feasible alternative to the existing system. Several restrictions and design aspects prescribe the use of system that was originally designed. The zoning restrictions on height are very stringent. That creates a need for the thinnest floor structure possible. It is also well known that concrete is the construction material of choice in downtown Washington, DC. Additionally, there are a few critical design situations in the structure that lend themselves to post-tensioned concrete systems. The exterior cantilever and interior 40' span are largely governed by deflection issues, which are effectively mitigated by the PT concrete structure. The

challenge of the revised design is to choose a solution that addresses these concerns as effectively as the solution provided by the project engineers. The proposed solution will consider the inherent limitations of site and usage, and then compare the resulting design to the existing.

## THESIS PROPOSAL

Many solutions were initially considered. These included plans to alter column layout and spacing in order to accommodate different systems. Among those considered were a one-way skip-joist system, two-way flat slab with drop panels (not post-tensioned), precast sections, two-way waffle slab and composite steel with composite metal deck. It was decided that altering the column layout was an unnecessary impact on the architectural intention on the building. The openness of the office plan is a vital selling point of the building and should not be disturbed. Local zoning codes were not to be violated, and the end result needed to be as close to the initial design as possible. The proposed system for redesign would be composite steel frame with a composite metal deck. This is a reasonable structural design decision, often used in similar situations. As long as structure weight can be minimized, floor loads will be dispersed into the columns with the smallest beam depth possible. The composite steel deck system will require a complete redesign of the structural system. Verco™ deck catalogs will be used to design the composite steel deck. The composite members will be designed initially with AISC 3$^{rd}$ Edition LRFD specifications. The composite floor system will then be checked and further designed in RAM Structural System. The cantilever area will be of special concern as the members and connections must resist excessive deflection. The lateral resisting system will try to utilize braced frames in the building core, and moment frames where additional horizontal stiffness is needed. The braced frames are not a possibility in the exterior bays due to the open office plan. The lateral resisting frames will be modeled individually in SAP2000 to determine relative stiffness and drift.

# Structural Depth

## GENERAL ISSUES

Altering the structural system from post-tensioned concrete to composite steel brings up many design considerations. The weight of the structure will change considerably. The increase in structure weight will impact the seismic response of the structure, and therefore the equivalent lateral forces to be designed for. This issue will be addressed more thoroughly in the discussion of the lateral system. Other issues include the overall structural depth of the system, which will be determined in gravity analysis. An important point of comparison for any alternative system is the cost increase or decrease. The change of material will also necessitate a change in project delivery. These issues will be explored in the construction management breadth. Assuring that steel is as environmentally conscious as concrete is the focus of the LEED breadth.

## FLOOR SYSTEM

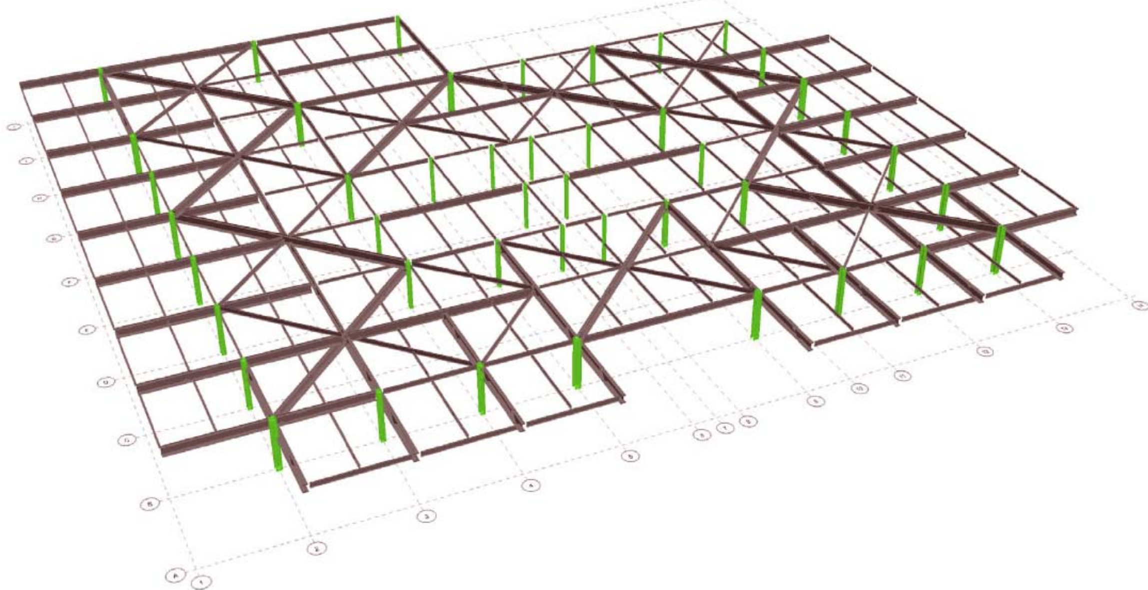
Loading on the floor system was governed by IBC 2003 building loads. Mixed-use office buildings are specified at 60 psf, with corridors taking 80 psf of live load. Being that the office is designed as an open plan, no specific corridor areas are fixed. The office live load was chosen to be 80 psf to allow for corridors to be effectively placed anywhere. Purposely, freedom for tenant configuration is a goal of the

architecture. ASCE 7-02 has superimposed dead load provisions. These were taken to include MEP, finished ceiling, floor covering, and light gauge framing members. The total design SDL was used as 25 psf. In the absence of actual dead loads used, the line load due to the curtain wall on the exterior slab was conservatively taken to be 500 plf. This will make the member size conservative. It will also allow the deflection at the curtain wall to be less than the L/360 design criteria applied in the model. Deflection at the cantilevered end where the curtain wall is attached is important to prevent differential movement of curtain wall supports, which could cause significant problems in the building envelope.

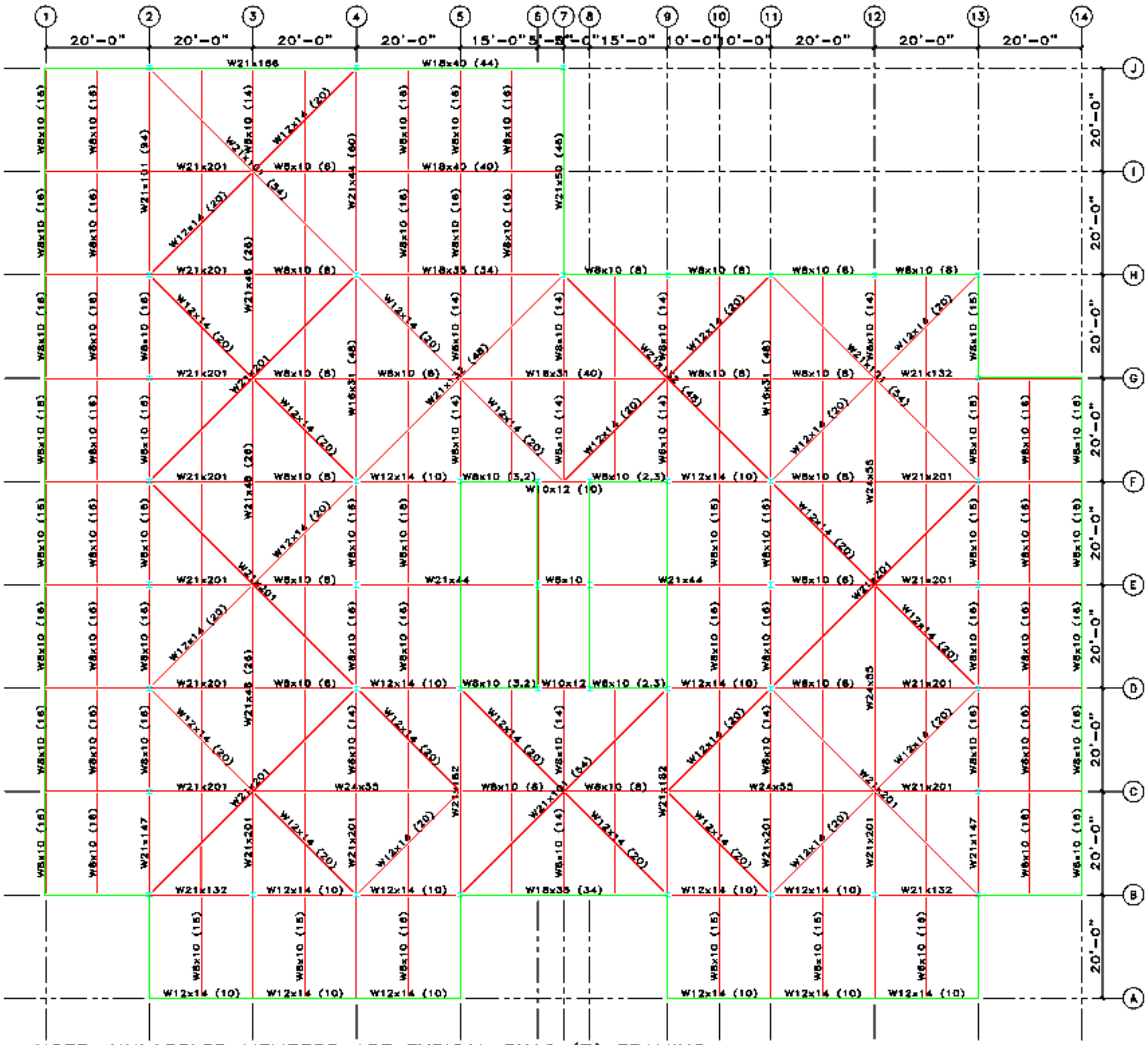
When redesigning the floor system, the first step in the composite steel system was design of the steel deck. Composite steel deck was chosen because of the increased span capabilities that it provides. Issues of fire protection are intrinsic with deck and floor slab design. At the same time, structure depth is an ongoing concern. Because of these, the floor slabs were specified to be a 4 ½ inch deep CIP lightweight concrete slabs. Assuming a 2" deck, this allowed for 2 ½ inches of lightweight concrete, which provides a 1 hour fire rating by UL standards. This is before any other fireproofing is applied. Using normal weight concrete, a 3 ½ inch slab would be required to achieve the same rating. The savings of an inch may not seem like a lot, but every increase in depth adds up. Also the additional dead load will lead to increased member sizes. The increased cost of lightweight concrete can be justified by the reduction in floor depth. Lightweight concrete also makes a better thermal and acoustical insulator.

The decking chosen was a 2" composite W2 FormLok® Deck. A span distance of 10' was chosen. The basis for this is that it allowed 20' bays to be divided into two 10' spans, which will work well for infill beam framing. The alternative would have be 6'-8" spans which is less than decking tables are even tabulated for. This case would also increase the amount of infill beams and contribute considerably more self-weight to girders. The 10' span can be bridged with 2 inch deep 20 gage composite deck, and does not require shoring during construction. In a 3 span condition, the unshored deck can withstand 215 psf of superimposed service load, much more than the 105 psf applied.

The load on intermediate framing members is equal to the applied live loads, superimposed dead load, plus the dead load due to concrete and decking. Lightweight concrete in a 4 ½ slab accounts for 32.1 psf of additional load. 20 gage decking adds another 2 psf of dead load to beams. The dead load is factored by 1.2 and live loads are factored by 1.6. The loads over the beams tributary area result in the design load for that member. A typical floor member sees a design load of 1982.2 plf, or 1.99 plf. Beams were designed utilizing full composite capabilities of the concrete slab. RAM Structural System, using 3rd edition LRFD criteria, was used to design the floor system. The member sizes were initially limited to W18 shapes. This proved impossible to design with the preliminary framing plan that had been developed. Member sizes were then limited to W21 shapes or smaller. Again the members were insufficient to meet the strength and/or deflection criteria imposed. A revised framing plan was developed that shortened the spans of key members that had been failing in the pervious layout. The basic layout of this framing plan is shown on the typical floor below.



Designing the floor system for 1.2(dead) + 1.6(live) gravity loads yields the following member sizes.

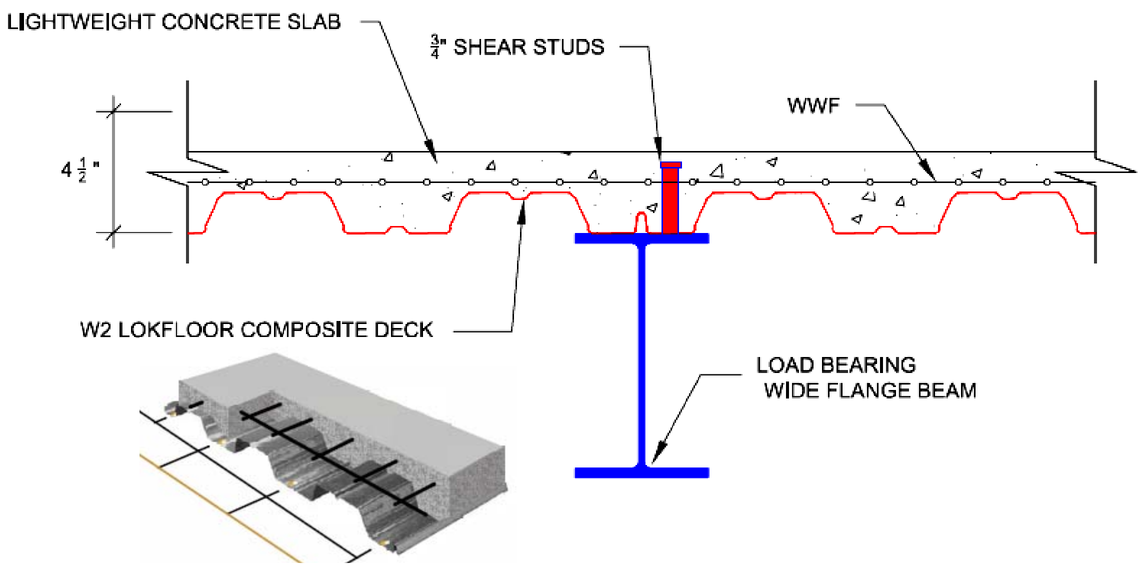



NOTE: UNLABELED MEMBERS ARE TYPICAL 8X10 (7) FRAMING

TYPICAL FLOOR

The beams requiring shear studs are labeled with the number of shear studs following in parenthesis. The shear studs are assumed to be evenly distributed along the length of the member. Concrete compressive strength for the slab was increased from the initial 3000 psi to 4000 psi. This value is still readily achievable by conventional batching techniques and should not increase the structure cost significantly. 4000 psi concrete is approximately $5.00 more per cubic yard, resulting in a $2000 dollar cost increase. This

value is easily offset by the reduction in the size of many steel members. A typical cross section of the composite floor system is shown below.




TYPICAL COMPOSITE FLOOR WITH COMPOSITE BEAM

## LATERAL SYSTEM

While the reduction in structure weight was not instrumental in the redesign of the buildings foundation system, it makes a sizeable impact on the lateral forces. Wind loads remain similar to those found for the existing structure. A more detailed analysis was done using a more accurate building geometry to determine the storey forces and shears due to wind. The basic wind data used for calculation of the wind loads is shown below:

| | | |
|---|---|---|
| **Building Data** | Location | Washington, DC |
| | N-S | 200 ft |
| | E-W | 180 ft |
| | Height | 140.28 ft |
| | Floor Ht | 11.69 ft |

| | | |
|---|---|---|
| **Velocity Pressure** | Kzt | 1.00 |
| | Kd | 0.85 |
| | V | 90 mph |
| | Use Group | II |
| | Importance | 1.00 |
| | Exposure | B |

| | | |
|---|---|---|
| **Gust Factor N-S** | B | 180.00 |
| | L | 200 |
| | h | 140.28 |
| | Ct | 0.02 |
| | x | 0.75 |
| | G | 0.821 |

| | | |
|---|---|---|
| **Gust Factor E-W** | B | 180.00 |
| | L | 200 |
| | h | 140.28 |
| | Ct | 0.028 |
| | x | 0.08 |
| | G | 0.824 |

The ASCE 7 design method 2 was used for this building, as it does not meet requirements for the simplified design method. The factored were used in the equations of the method and geographical and topographical effects were considered. Storey forces and shear forces were determined as follows.

| Wind | Shear | | Storey Force | |
|---|---|---|---|---|
| | E-W | N-S | E-W | N-S |
| **Roof** | | | 21.51 | 21.73 |
| **12** | 21.51 | 21.73 | 42.26 | 42.78 |
| **11** | 63.77 | 64.51 | 41.45 | 42.05 |
| **10** | 105.22 | 106.56 | 40.63 | 41.32 |
| **9** | 145.85 | 147.87 | 39.76 | 40.54 |
| **8** | 185.62 | 188.41 | 38.79 | 39.66 |
| **7** | 224.40 | 228.07 | 37.59 | 38.59 |
| **6** | 261.99 | 266.66 | 36.29 | 37.42 |
| **5** | 298.28 | 304.08 | 34.93 | 36.20 |
| **4** | 333.21 | 340.28 | 33.19 | 34.64 |
| **3** | 366.41 | 374.92 | 30.91 | 32.60 |
| **2** | 397.32 | 407.52 | 28.69 | 30.60 |
| **Base** | 426.01 | 438.12 | | |

Whereas the wind loading did not see significant change from the existing structure, the equivalent seismic load decreased significantly. Equivalent seismic loading is based on the structures response to earthquakes. The resulting storey forces are based on many factors, including structure weight. The lighter steel structure has less inertia and responds better to the lateral movements in an earthquake. The factors and conditions used for design are as follows:

| Building Data | Location | Washington, DC |
|---|---|---|
| | N-S | 200 ft |
| | E-W | 180 ft |
| | Height | 140.28 ft |
| | Floor Ht | 11.69 ft |
| | Design Cat | B |

| Velocity Pressure | Ss | 0.143 |
|---|---|---|
| | S1 | 0.0713 |
| | Fa | 1.2 |
| | Fv | 1.7 |
| | Use Group | I |
| | Site Class | C |
| | Importance | 1.0 |

| Response N-S | R | 5 |
|---|---|---|
| | Cs | 0.023 |
| | Ct | 0.02 |
| | x | 0.75 |
| | T | 0.82 |

| Response N-S | R | 3.5 |
|---|---|---|
| | Cs | 0.033 |
| | Ct | 0.028 |
| | x | 0.8 |
| | T | 1.46 |

ASCE 7-02 also gives guidelines for the equivalent lateral force method of designing for seismic loads. The frames used for seismic resisting are braced frames in the N-S direction, and steel moment frames in the E-W direction. The design resulted in decreased storey forces and shears as was expected. The forces are summarized in the following table.

| Seismic | Shear | | Storey Force | |
|---|---|---|---|---|
| | E-W | N-S | E-W | N-S |
| **Roof** | | | 34.00 | 37.27 |
| **12** | 34.00 | 37.27 | 47.44 | 53.49 |
| **11** | 81.44 | 90.77 | 41.20 | 47.91 |
| **10** | 122.64 | 138.67 | 35.25 | 42.41 |
| **9** | 157.88 | 181.08 | 29.61 | 37.00 |
| **8** | 187.49 | 218.08 | 24.29 | 31.70 |
| **7** | 211.78 | 249.78 | 19.34 | 26.52 |
| **6** | 231.12 | 276.30 | 14.76 | 21.47 |
| **5** | 245.88 | 297.77 | 10.61 | 16.59 |
| **4** | 256.49 | 314.36 | 6.93 | 11.89 |
| **3** | 263.42 | 326.25 | 3.80 | 7.43 |
| **2** | 267.22 | 333.68 | 1.36 | 3.33 |
| **Base** | 268.58 | 337.02 | | |

The decrease in lateral force is evident when the redesign values are compared to values found for the original design. It is also of note that when comparing the tables that seismic loading no longer controls the majority of the forces at floor level. The next table

highlights which force controls at each floor, in each direction. It is beneficially that the forces have decreased, because resisting lateral forces with steel structures can become difficult and expensive because of the complex connections required.

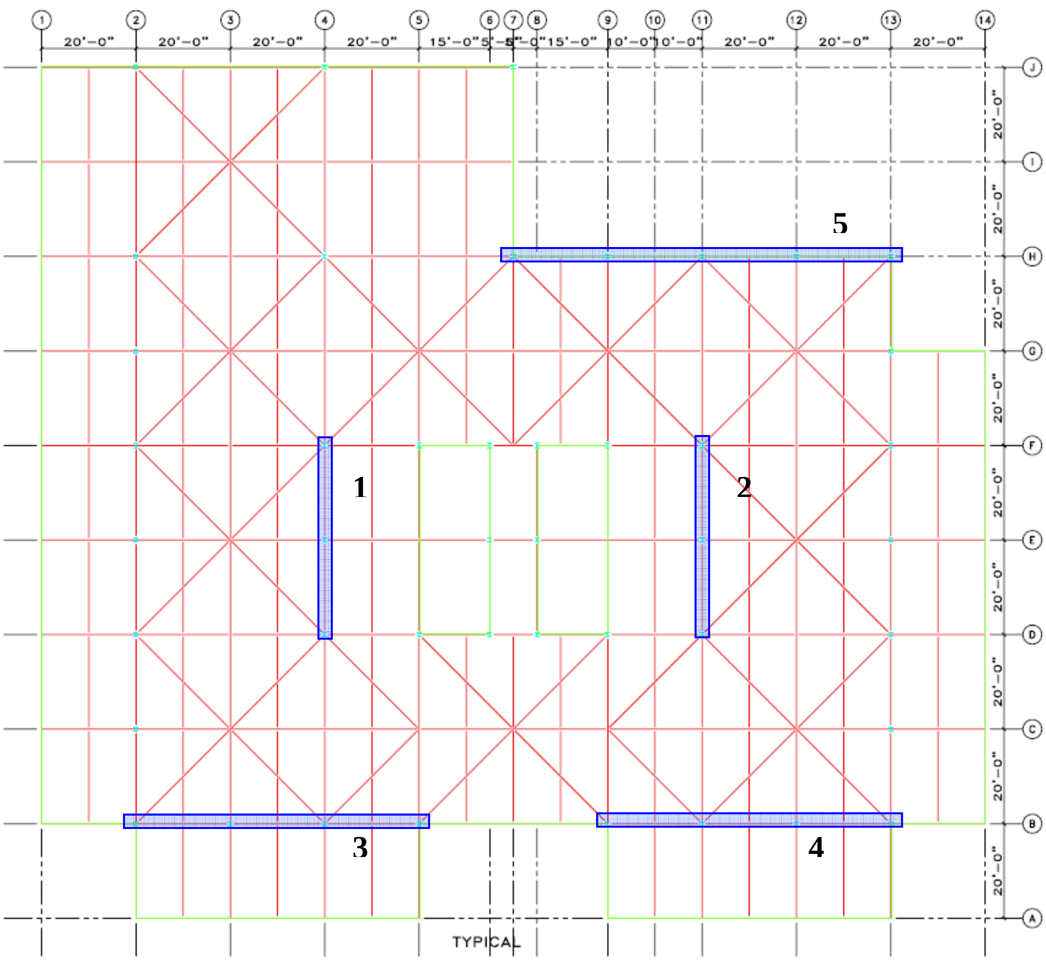
| | Shear | | | | Storey Force | | | |
|---|---|---|---|---|---|---|---|---|
| | E-W | | N-S | | E-W | | N-S | |
| Storey | Wind | Seismic | Wind | Seismic | Wind | Seismic | Wind | Seismic |
| Roof | | | | | 21.51 | 34.00 | 21.73 | 37.27 |
| 12 | 21.51 | 34.00 | 21.73 | 37.27 | 42.26 | 47.44 | 42.78 | 53.49 |
| 11 | 63.77 | 81.44 | 64.51 | 90.77 | 41.45 | 41.20 | 42.05 | 47.91 |
| 10 | 105.22 | 122.64 | 106.56 | 138.67 | 40.63 | 35.25 | 41.32 | 42.41 |
| 9 | 145.85 | 157.88 | 147.87 | 181.08 | 39.76 | 29.61 | 40.54 | 37.00 |
| 8 | 185.62 | 187.49 | 188.41 | 218.08 | 38.79 | 24.29 | 39.66 | 31.70 |
| 7 | 224.40 | 211.78 | 228.07 | 249.78 | 37.59 | 19.34 | 38.59 | 26.52 |
| 6 | 261.99 | 231.12 | 266.66 | 276.30 | 36.29 | 14.76 | 37.42 | 21.47 |
| 5 | 298.28 | 245.88 | 304.08 | 297.77 | 34.93 | 10.61 | 36.20 | 16.59 |
| 4 | 333.21 | 256.49 | 340.28 | 314.36 | 33.19 | 6.93 | 34.64 | 11.89 |
| 3 | 366.41 | 263.42 | 374.92 | 326.25 | 30.91 | 3.80 | 32.60 | 7.43 |
| 2 | 397.32 | 267.22 | 407.52 | 333.68 | 28.69 | 1.36 | 30.60 | 3.33 |
| Base | 426.01 | 268.58 | 438.12 | 337.02 | | | | |

| Base Moment | | | |
|---|---|---|---|
| E-W | | N-S | |
| Wind | Seismic | Wind | Seismic |
| 33955.15 | 27213.48 | 34664.82 | 32744.16 |

*all values in k and Ft-k

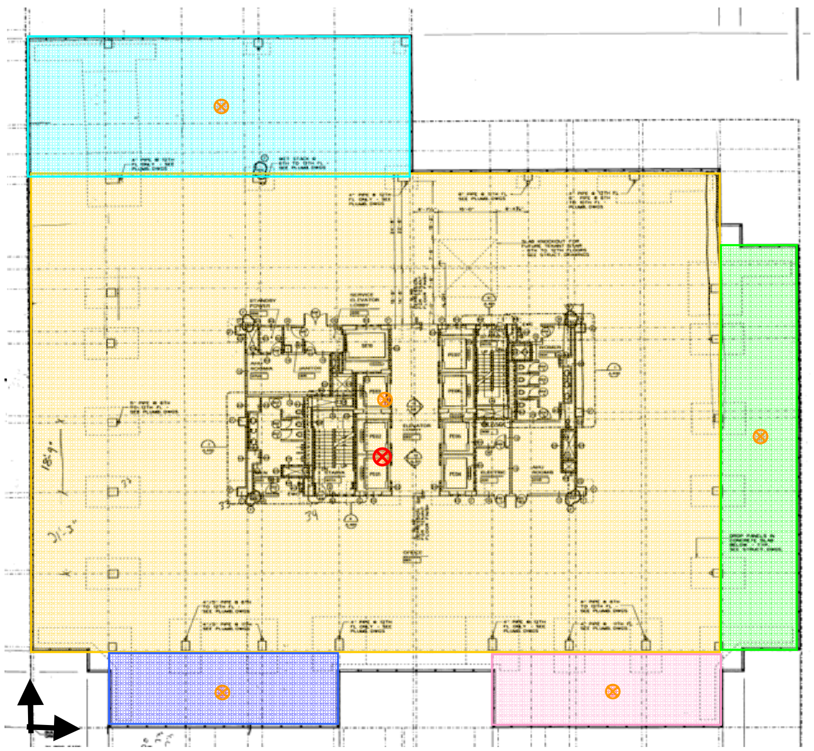
In the north-south direction, the easternmost and westernmost core walls are available to be used as shear walls or braced frames. They have no openings in plan and are not part of a fire-rated stairwell or elevator core. For this reason braced frames will be utilized to resist forces in the N-S direction. Perpendicularly, in the E-W direction, there are no walls where braces can be hidden. All the core walls in that direction require openings for restrooms and mechanical spaces. Due to limitations set forth by the building architecture, the lateral forces must be resisted by frames with moment carrying connections. Along the southern edge of the building at the first column line, there is potential for a moment frame. Instead of a single frame, two frames will be designated. This is because of the long 40’ span that effectively separates the frame and would be

difficult to transfer forces through. Two bays of each three bay frame will be used as moment frames. This is for reasons of relative stiffness which will be elaborated upon shortly. At the northern side of the building, a four bay frame will serve as a moment frame. The location and label of the lateral resisting frames is shown in the following plan:




Frames 1 and 2 act in the N-S direction, and frames 3, 4, and 5 act in the E-W direction. The location of the frames is generally equidistant from the center of mass. The center of mass for the structure was calculated previously based on a uniform mass distribution. The floor is considered to be drastically heavier than the column system,

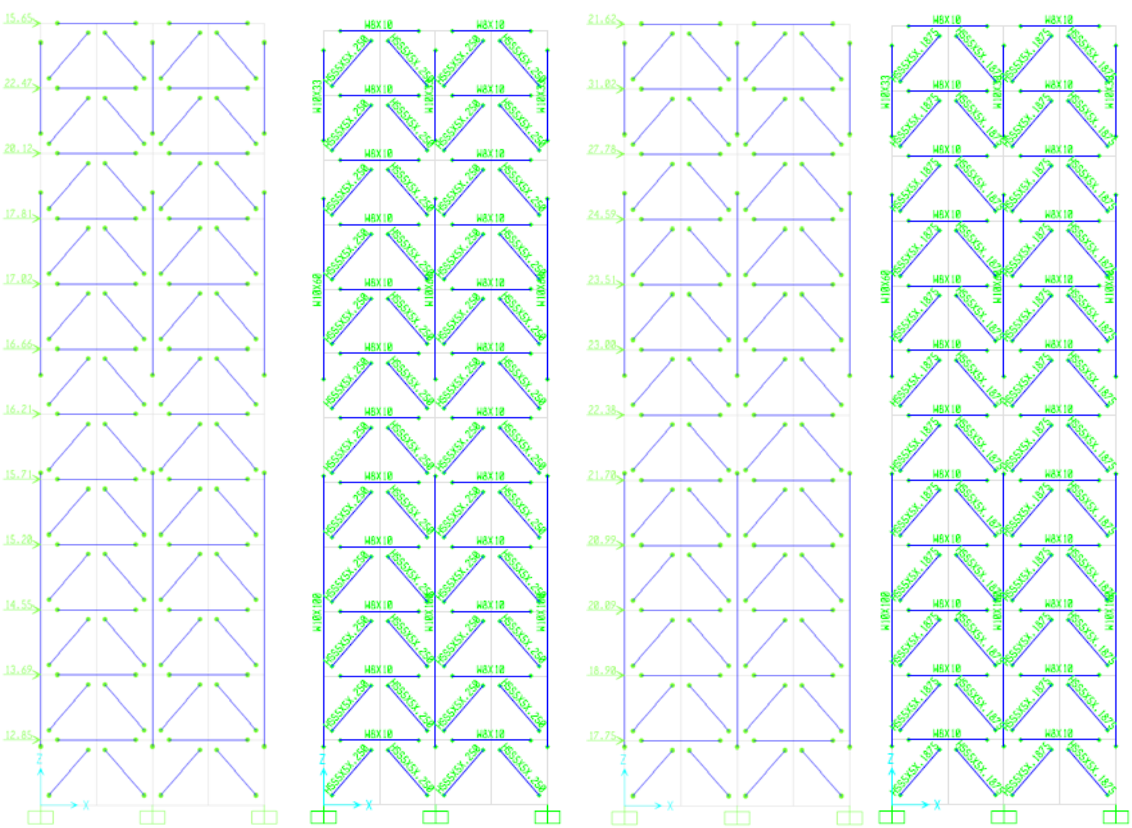
therefore allowing such an assumption to be made. The mass center location of each regular section was calculated, and then used to find the center of mass for the entire structure. The graphical representation is shown below.



For the braced frames in the N-S direction, a vertical K, or Chevron configuration was chosen. This was chosen due to the lateral stiffness, and ease of construction. Frames 1 and 2 will be constructed with HSS braces. Frames 3, 4, and 5 will be able to resist moment by the connections between beam and column members.

To minimize torsional effects before they become a problem, attention was paid to the location of the center of rigidity. The frames located closer to the center of mass received an increase in rigidity. The factor of increase is related to the perpendicular distance to the center of mass, divided by the total perpendicular distance to the resisting frame on the opposite side. This required frame 1 to be 36.6 percent stiffer than frame 2. The same method was used in the E-W direction, but with the total stiffness of frames 3

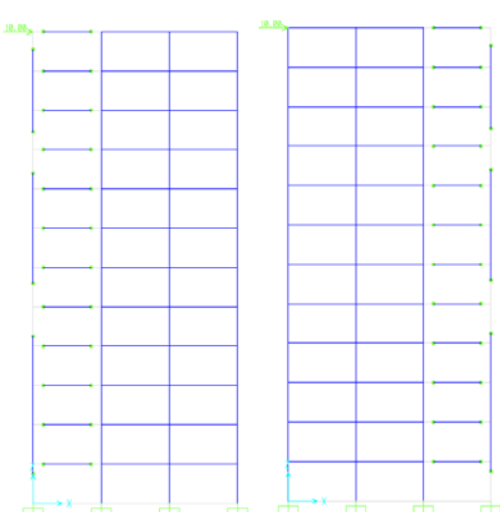
and 4 being compared to frame 5. Frames 3 and 4 collectively need to be 42 percent stiffer than frame 5 alone. Frames were then modeled using SAP2000. The frame stiffness was determined using the equation ki=P/Δ. P was a 10k force applied at the top joint of the frame. Delta is the lateral deflection that resulted from that force. Member sizes were adjusted until relative stiffness near those needed were obtained. The resulting frame member sizes for frames 1 and 2 are shown below. The outer dimension of the bracing members was kept constant to aid in detailing and fabrication. HSS 5x5x0.25 members brace frame 1 while the less stiff frame 2 is braced by HSS 5x5x0.1875 members. The frames as modeled in SAP2000 are shown below.



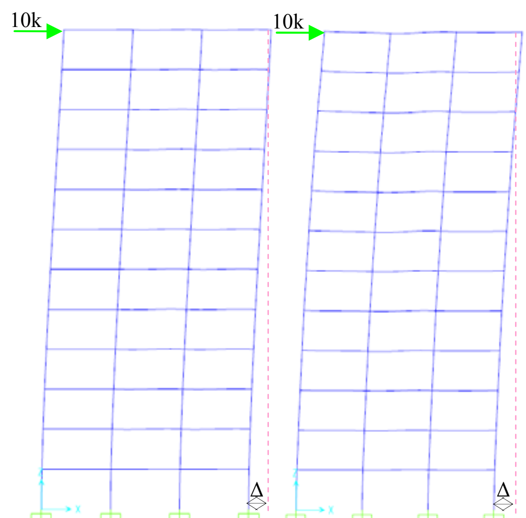
*Frame 1 (load)* *Frame 1 (sizes)* *Frame 2 (load)* *Frame 2 (sizes)*

The same procedure was carried out for the moment frames in the E-W direction. A decision was made to utilize only 2 bays of each 3 bay frame in frames 3 and 4. This was

done to reduce the stiffness of these frames relative to frame 5. The bay adjacent to the building corner was not used because it was unlike the other 2 bays. Also the end column is that bay is a primary gravity column, and it is not desirable to induce extra moments from lateral loading. Also the column is much larger than those in the other two bays, which would require additional detailing of larger moment connections, which would undoubtedly be costly. The moment frames in the E-W direction under their 10k unit loads are shown below. Here you can see the deflection in the frame that is used to compute the stiffness. For frame 3, the deflection Δ = 0.4817". The stiffness associated with that frame becomes 10 / 0.4817 = 20.75.



*Frame 3 & 4 (unit loads)*

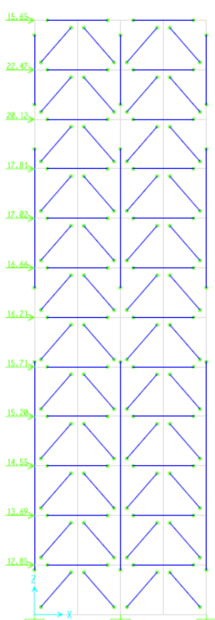


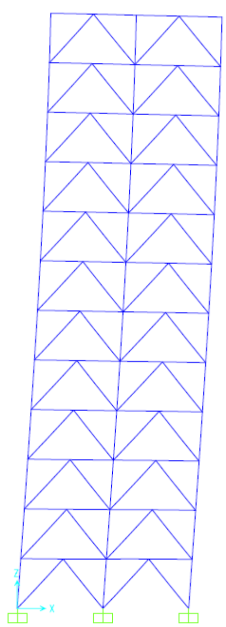

*Frames 3 & 4 (unit deflections)*

Each frame carries a load that is relative to its stiffness. The stiffer frames carry more lateral load as a direct proportion of their relative stiffness. The following table summarizes the load distribution to each frame in the primary directions.

| Force Distribution | N-S | | E-W | |
|---|---|---|---|---|
| | Frame 1 | Frame 2 | Frames 3,4 | Frame 5 |
| **Proportion** | 0.58 | 0.42 | 0.28 | 0.43 |
| **Roof** | 21.62 | 15.66 | 9.52 | 14.62 |
| **12** | 31.03 | 22.47 | 13.28 | 20.40 |
| **11** | 27.79 | 20.12 | 11.61 | 17.82 |
| **10** | 24.60 | 17.81 | 11.38 | 17.47 |
| **9** | 23.51 | 17.03 | 11.13 | 17.10 |
| **8** | 23.00 | 16.66 | 10.86 | 16.68 |
| **7** | 22.38 | 16.21 | 10.53 | 16.16 |
| **6** | 21.70 | 15.72 | 10.16 | 15.60 |
| **5** | 21.00 | 15.20 | 9.78 | 15.02 |
| **4** | 20.09 | 14.55 | 9.29 | 14.27 |
| **3** | 18.91 | 13.69 | 8.66 | 13.29 |
| **2** | 17.75 | 12.85 | 8.03 | 12.34 |

When the appropriate loads are applied to each frame, lateral effects and drifts can be analyzed. A total drift limit of L/400 is a good target to shoot for. The drifts and deflections would be best minimized due to the glass façade, and a suspended plaster veneer ceiling system. With a 140’ building height, L/400 drift comes to 0.3507’ or 4.2”. Under lateral loads, all resisting frames meet this drift limitation. A sample frame and drift evaluation is shown below. Consult appendices for exhaustive lateral frame modeling.

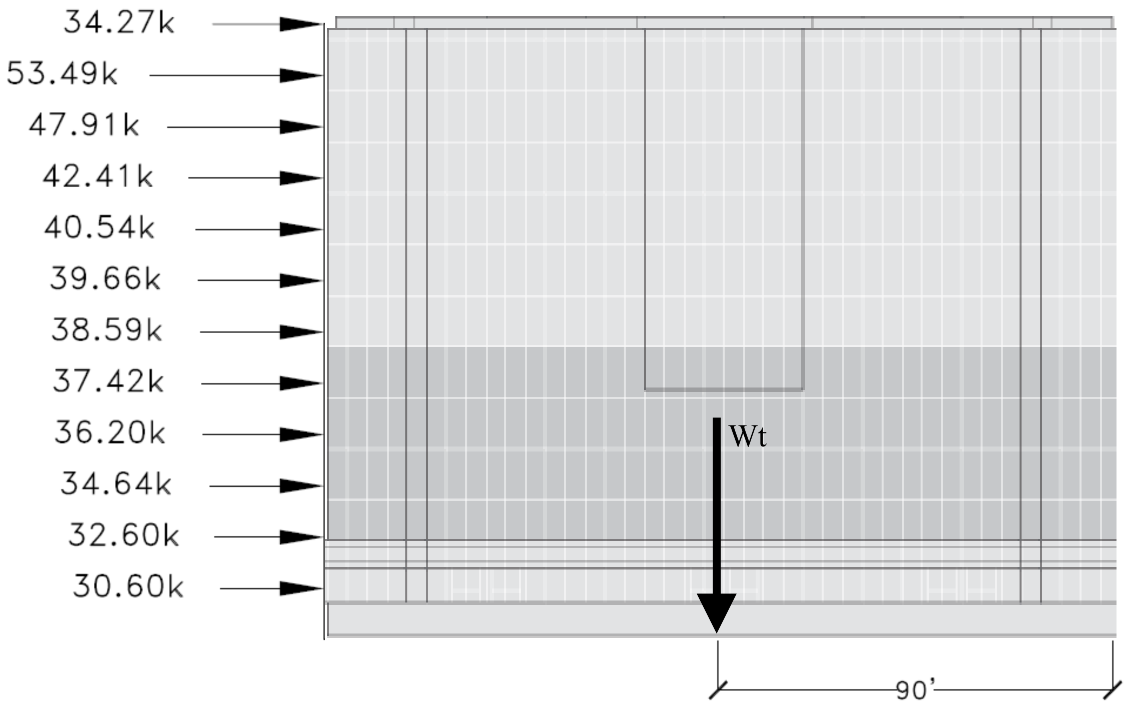




| Frame 1 Drift | Storey Ht | Drift | L/x Ratio |
|---|---|---|---|
| **Roof** | 11.69 | 0.251 | 559 |
| **12** | 11.69 | 0.280 | 501 |
| **11** | 11.69 | 0.310 | 453 |
| **10** | 11.69 | 0.334 | 420 |
| **9** | 11.69 | 0.351 | 400 |
| **8** | 11.69 | 0.361 | 389 |
| **7** | 11.69 | 0.374 | 375 |
| **6** | 11.69 | 0.380 | 369 |
| **5** | 11.69 | 0.380 | 369 |
| **4** | 11.69 | 0.372 | 377 |
| **3** | 11.69 | 0.352 | 399 |
| **2** | 11.69 | 0.322 | 435 |
| **Total** | 140.28 | 4.067 | 414 |

The lateral frames in the revised steel design are less stiff than the original concrete moment frames. In the E-W direction, the moment frames had a stiffness of 169 compared to just 71 for the steel frame. The stiffness is less than half of the original design. Before, high equivalent seismic loads controlled the storey forces. Now the storey forces have been dramatically reduces, which explains why a more ductile system is still able to resist applied lateral forces.

Overturning can be an issue for lighter buildings with a low width to height ratio. An overturning investigation was conducted based on maximum lateral loads in the more critical N-S direction where the building is narrower.




Overturning Moment = 37707.38 ft-k
Resisting Weight = 337703.38 ft-k / 90 ft = 418.97 k
Building Wt >> 418.97 k


## COST ESTIMATE

A vital point of comparison between two alternate systems is initial cost. A takeoff was preformed for the steel system. The gravity beams were totaled in linear foot per floor for each shape. Composite decking was estimated per square foot of floor area. A floor area of 28800 square feet per floor was used for occupied tenant floors. The sub-grade structure was not considered for the cost estimate, as it did not change from the existing system. When system costs were compared, only the area redesigned for the thesis research was compared. Bare costs were determined using RS Means CostWorks 2005 data. Because construction of this building started when these costs would be relevant, inflation should not have a significant impact on the comparable unit costs. Cost was broken down into bare material, bare labor, and bare equipment. Cost data for the existing structure was acquired from the actual contractor's estimates.

The part of the existing structure equivalent to the part redesigned had an approximate cost of 7,990,336 dollars. Approximately 15 percent of this cost was due to the post tensioning in the slab. The value of the redesigned structure was found to be 8,037,148 dollars. This is a cost increase of one half of a percent. These figures include error from several sources. Error in the original cost stems from separating the cost of the equivalent part of the original structure to the estimate for the entire structural system. The entire structural system was not redesigned, only a portion of it. Error was

introduced into the figure from the alternative steel system in several places. Moment connections were not designed, so an accurate estimate of the cost per connection could not be determined. A rough figure based on steel tonnage in the moment frames was used. Because of these discrepancies and the closeness of the estimates, it is very difficult to say which system would actually cost more upfront. This came as quite a surprise as it was expected that the steel system would cost considerably more than the existing concrete frame. RS Means data does not take into account price fluctuations due to geographic location. Steel producers, fabricators, and erectors are less common in the area than concrete contractors. Actual quotes would likely differ from those found in Means data, and a project cost history would be a better method for calculating local costs. That being said, the actual costs should not differ much more than those estimated. A full detailed estimate can be found in the appendix. The chart below summarizes costs for existing and alternate systems.

| Estimate | Existing Concrete Structure | | Alternate Steel Structure | |
|---|---|---|---|---|
| | Item | Cost | Item | Cost |
| | Crane | $351,120.00 | Crane | $351,120.00 |
| | Formwork | $2,811,062.25 | Steel | $4,968,106.38 |
| | Concrete | $2,010,426.00 | Decking | $660,096.00 |
| | Reinforcing | $1,438,959.00 | Shear Studs | $54,180.00 |
| | Post-tensioning | $1,378,769.00 | WWF | $126,835.20 |
| | | | Concrete | $571,369.20 |
| | | | Fireproofing | $425,088.00 |
| | | | Connections | $880,354.00 |
| | | | | |
| **Total** | | **$7,990,336.25** | | **$8,037,148.78** |

## PROJECT SCHEDULE

There is much variation between concrete and steel construction methods. Steel must be designed and fabricated before it can be shipped to site for erection. Concrete requires little lead time but must be placed and cured which can cause a slower actual construction period. The post-tensioning tendons cannot be jacked until after the concrete has reached a certain percentage of its full 28 day strength. This is usually about half of the strength, so an additional 14 day wait for post tensioning should be assumed.

The schedule submitted by the general contractor calls for a 188 day duration on the concrete structure. That schedule is increased to 265 days if the end date is counted as when the reshoring is removed. This is when the interior trades would be able to start their work. Post-tensioning was not included in the timeframe given. It should be assumed that post-tensioning can be applied while the reshoring is in place, and therefore would not impact the 265 day estimate. In the repetitive floors at the middle of the building, the floor to floor turnaround is 14 days.

The alternative system requires a 6-8 week procurement time for steel. The total schedule length comes out to be 224 days with an 8 week lead on the steel. If the lead time happens to fall on the low end, the total schedule can be decreased to 218 days. The durations with various assumptions are summarized below.

| **Schedule** | Existing Concrete Structure | | Alternate Steel Structure | |
|---|---|---|---|---|
| | Assumption | Duration | Assumption | Duration |
| | Without curing, PT or reshoring | 188 days | 2 deck/concrete crews, 8 wk lead | 270 days |
| | With removal of reshoring | 265 days | 3 deck/concrete crews, 8 wk lead | 224 days |
| | | | 3 deck/concrete crews, 6 wk lead | 218 days |

Overall it appears that a steel project could be delivered more quickly. How much more quickly the steel is depends on the fabricator. The concrete would be expected to take longer because it must be formed and reinforced before it can be placed. The amount of material involved in the project allows the steel system to make up for lead time with rapid erection.


## EXISTING CERTIFICATION

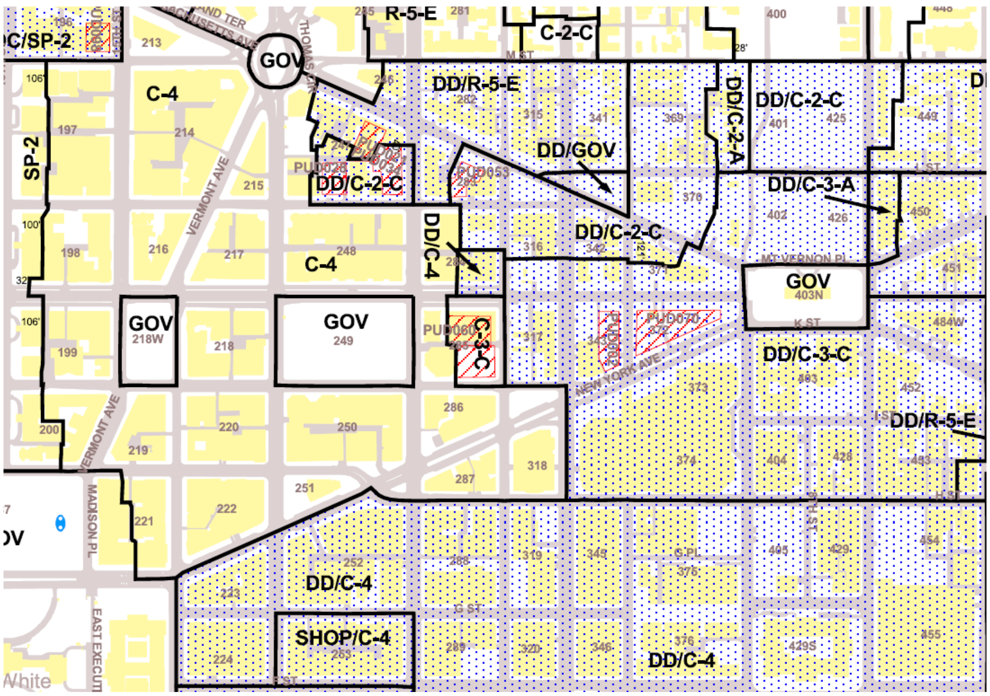
The existing design is to be LEED rated silver. This means that the project has met minimum LEED requirements and also amassed 30 to 35 additional points. The rating system used for the building is based on a past version of the LEED-NC standard. As of February 2006, the existing project was on track for 29 points, with 9 points pending. Silver rating is easily attainable from the current standing. The current version of LEED-NC is now 2.2.

## REVISED LEED CERTIFICATION

Updating the current project to LEED-NC 2.2 is the first step in evaluating the LEED performance of the alternate structural design. Requirements will be checked for design and materials in areas that have been affected by the design revision. Then the entire project will be checked to make sure that a LEED Silver rating is achieved. After a silver rating is guaranteed, recommendations will be made to possibly improve the project to the Gold rating level.

Converting the existing project to the new standard 2.2 yields 31 points, as some credits have changed from the version used for the original design. Due to these changes in the LEED requirements, only 7 points are now pending. In version 2.2 a silver rating requires a minimum of 32 points, making it also very likely to be achieved. A copy of the LEED Scorecard for the existing project can be found in the appendix. Some credits

have been altered and require additional points to be met. SS credit 2 now additionally requires the new construction to be with ½ mile of both a residential zone and 10 basic services. The zoning map below shows the proximity to R-5-E high density residential housing. The remaining facilities exist within the ½ mile radius but are not labeled below.




Also a new credit, EQ 3.2 for adjusting indoor air quality before occupancy, has been added. It requires that 14,000 cubic feet per square foot of floor area of outside air be flushed through the space. With all air handling units operating at maximum capacity, this can be accomplished in less than a week's time. This will not impact tenant move-in as it can be started prior to completely finishing work on the building. Alternatively occupancy can start when 3,500 cubic feet per square foot has been delivered. This can be accomplished in a few days. Then ventilation may continue at a reduced rate outlined in the LEED guidelines. These are the only credits that have been altered and required attention from previous version to the current version.

The change of material brings attention to several credits in the materials and resources (MR) credit area. Credit MR 1 will not change as means for recycling of

common recyclables will not be altered. Credits 4.1, 4.2, 5.1 and 5.2 all deal with specifying recycled materials and materials produced in the region. This can be achieved for the revised structural system by using steel produced within 500 miles of the project site. Also the steel must be more than 10% recycled content to meet both 4.1 and 4.2. According to the Green Building Council, the location of the steel fabricator can be counted toward local production. Many fabricators lie within the 500 mile radius of the job site, so those points can be awarded. Steel is the most recycled material in the USA and the world. Steel regularly contains up to 90 percent recycled material. Specifying 10% recycled steel is very feasible. Steel is more easily recycled than concrete so that is a benefit not rated by the LEED system that will provide ecological advantages in the future.

There are several credits that were skipped likely in the interest of architectural concerns. SS 4.2 allows a point for providing bicycle racks and changing facilities. The building already provides a gym with shower and changing facilities that can be used mutually for accommodating bicyclists. Secure bicycle parking can be integrated into the rear entrance without a considerable impact on architecture. Credit SS 8 limits the light pollution created by the building. To acquire this point, a control system can be placed on the tenant lighting that will automatically turn it off after business hours. In addition all exterior lighting will be designed according to zone LZ3 requirements. Exterior lighting may cause no more than 0.20 footcandles at the site boundary. WE credit 2 is intended to reduce creation of wastewater and reduce potable water demand. A way to implement this into the building system is collecting rainwater for use in the automated irrigation system for the building. Additionally captured rainwater can be used for toilet and urinal flushing. Where reclaimed water use is not practical, water conserving fixtures will be installed. The total reduction in potable water use for sewage conveyance needs to be reduced by 50%.

Utilizing these LEED criteria, the project point total can be realistically increased to 40 points. This value puts the building into the LEED Gold rating category. The revised LEED Scorecard for the supplementary credits is shown below.

**LEED™ Scorecard**

| 40 | 3 | 26 | Total Project Score | | Possible Points 69 |
|---|---|---|---|---|---|

Certified 26 to 32 points Silver 33 to 38 points Gold 39 to 51 points Platinum 52 or more points

| Y | ? | N | | Sustainable Sites | Possible Points |
|---|---|---|---|---|---|
| 12 | 0 | 2 | | | 14 |
| Y | | | Prereq 1 | Erosion & Sedimentation Control | 0 |
| 1 | | | Credit 1 | Site Selection | 1 |
| 1 | | | Credit 2 | Development Density & Community Connectivity | 1 |
| 1 | | | Credit 3 | Brownfield Redevelopment | 1 |
| 1 | | | Credit 4.1 | Alternative Transportation, Public Transportation Access | 1 |
| 1 | | | Credit 4.2 | Alternative Transportation, Bicycle Storage & Changing Rooms | 1 |
| 1 | | | Credit 4.3 | Alternative Transportation, Alternative Fuel Refueling Stations | 1 |
| 1 | | | Credit 4.4 | Alternative Transportation, Parking Capacity | 1 |
| | | 1 | Credit 5.1 | Reduced Site Disturbance, Protect or Restore Open Space | 1 |
| | | 1 | Credit 5.2 | Reduced Site Disturbance, Development Footprint | 1 |
| 1 | | | Credit 6.1 | Stormwater Management, Rate and Quantity | 1 |
| 1 | | | Credit 6.2 | Stormwater Management, Treatment | 1 |
| 1 | | | Credit 7.1 | Landscape & Exterior Design to Reduce Heat Islands, Non-Roof | 1 |
| 1 | | | Credit 7.2 | Landscape & Exterior Design to Reduce Heat Islands, Roof | 1 |
| 1 | | | Credit 8 | Light Pollution Reduction | 1 |

| Y | ? | N | | Materials & Resources | Possible Points |
|---|---|---|---|---|---|
| 5 | 0 | 8 | | | 13 |
| Y | | | Prereq 1 | Storage & Collection of Recyclables | 0 |
| | | 1 | Credit 1.1 | Building Reuse, Maintain 75% of Existing Shell | 1 |
| | | 1 | Credit 1.2 | Building Reuse, Maintain 100% of Existing Shell | 1 |
| | | 1 | Credit 1.3 | Building Reuse, Maintain 100% Shell & 50% Non-Shell | 1 |
| 1 | | | Credit 2.1 | Construction Waste Management, Divert 50% | 1 |
| | | 1 | Credit 2.2 | Construction Waste Management, Divert 75% | 1 |
| | | 1 | Credit 3.1 | Resource Reuse, Specify 5% | 1 |
| | | 1 | Credit 3.2 | Resource Reuse, Specify 10% | 1 |
| 1 | | | Credit 4.1 | Recycled Content, Specify 25% | 1 |
| 1 | | | Credit 4.2 | Recycled Content, Specify 50% | 1 |
| 1 | | | Credit 5.1 | Local/Regional Materials, 20% Manufactured Locally | 1 |
| 1 | | | Credit 5.2 | Local/Regional Materials, of 20% Above, 50% Harvested Locally | 1 |
| | | 1 | Credit 6 | Rapidly Renewable Materials | 1 |
| | | 1 | Credit 7 | Certified Wood | 1 |

| Y | ? | N | | Water Efficiency | Possible Points |
|---|---|---|---|---|---|
| 5 | 0 | 0 | | | 5 |
| 1 | | | Credit 1.1 | Water Efficient Landscaping, Reduce by 50% | 1 |
| 1 | | | Credit 1.2 | Water Efficient Landscaping, No Potable Use or No Irrigation | 1 |
| 1 | | | Credit 2 | Innovative Wastewater Technologies | 1 |
| 1 | | | Credit 3.1 | Water Use Reduction, 20% Reduction | 1 |
| 1 | | | Credit 3.2 | Water Use Reduction, 30% Reduction | 1 |

| Y | ? | N | | Energy & Atmosphere | Possible Points |
|---|---|---|---|---|---|
| 6 | 1 | 10 | | | 17 |
| Y | | | Prereq 1 | Fundamental Building Systems Commissioning | 0 |
| Y | | | Prereq 2 | Minimum Energy Performance | 0 |
| Y | | | Prereq 3 | CFC Reduction in HVAC&R Equipment | 0 |
| 2 | | | Credit 1.1 | Optimize Energy Performance, 20% New / 10% Existing | 2 |
| 2 | | | Credit 1.2 | Optimize Energy Performance, 30% New / 20% Existing | 2 |
| | | 2 | Credit 1.3 | Optimize Energy Performance, 40% New / 30% Existing | 2 |
| | | 2 | Credit 1.4 | Optimize Energy Performance, 50% New / 40% Existing | 2 |
| | | 2 | Credit 1.5 | Optimize Energy Performance, 60% New / 50% Existing | 2 |
| | | 1 | Credit 2.1 | Renewable Energy, 5% | 1 |
| | | 1 | Credit 2.2 | Renewable Energy, 10% | 1 |
| | | 1 | Credit 2.3 | Renewable Energy, 20% | 1 |
| 1 | | | Credit 3 | Additional Commissioning | 1 |
| | | 1 | Credit 4 | Ozone Depletion | 1 |
| 1 | | | Credit 5 | Measurement & Verification | 1 |
| | 1 | | Credit 6 | Green Power | 1 |

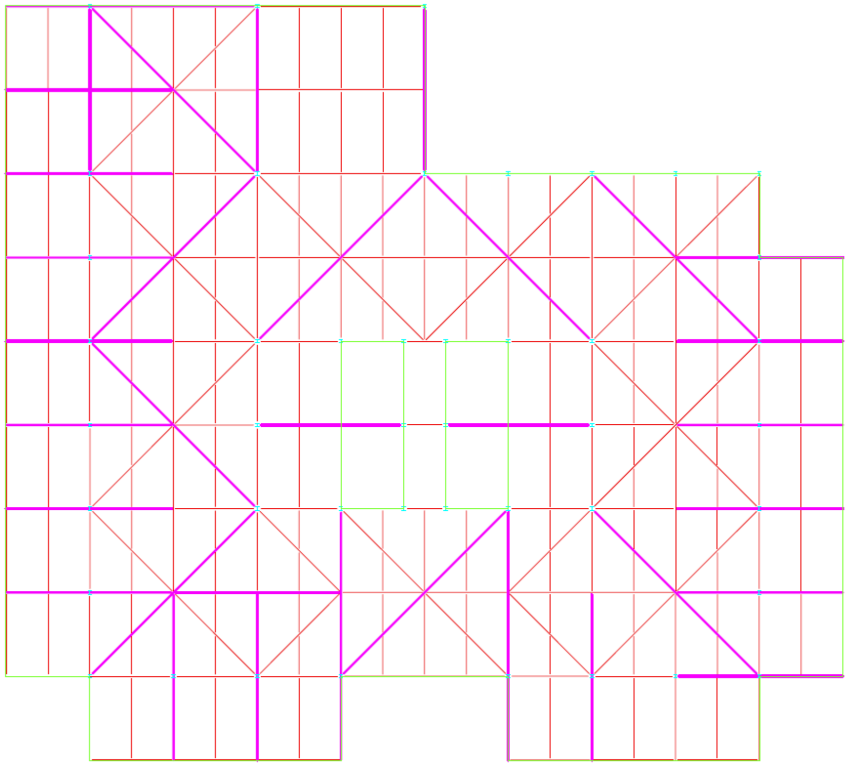
| Y | ? | N | | Indoor Environmental Quality | Possible Points |
|---|---|---|---|---|---|
| 9 | 0 | 6 | | | 15 |
| Y | | | Prereq 1 | Minimum IAQ Performance | 0 |
| Y | | | Prereq 2 | Environmental Tobacco Smoke (ETS) Control | 0 |
| 1 | | | Credit 1 | Carbon Dioxide ($CO_2$) Monitoring | 1 |
| | | 1 | Credit 2 | Increase Ventilation Effectiveness | 1 |
| 1 | | | Credit 3.1 | Construction IAQ Management Plan, During Construction | 1 |
| 1 | | | Credit 3.2 | Construction IAQ Management Plan, Before Occupancy | 1 |
| 1 | | | Credit 4.1 | Low-Emitting Materials, Adhesives & Sealants | 1 |
| 1 | | | Credit 4.2 | Low-Emitting Materials, Paints | 1 |
| 1 | | | Credit 4.3 | Low-Emitting Materials, Carpet | 1 |
| 1 | | | Credit 4.4 | Low-Emitting Materials, Composite Wood | 1 |
| 1 | | | Credit 5 | Indoor Chemical & Pollutant Source Control | 1 |
| | | 1 | Credit 6.1 | Controllability of Systems, Perimeter | 1 |
| | | 1 | Credit 6.2 | Controllability of Systems, Non-Perimeter | 1 |
| 1 | | | Credit 7.1 | Thermal Comfort, Comply with ASHRAE 55-1992 | 1 |
| | | 1 | Credit 7.2 | Thermal Comfort, Permanent Monitoring System | 1 |
| | | 1 | Credit 8.1 | Daylight & Views, Daylight 75% of Spaces | 1 |
| | | 1 | Credit 8.2 | Daylight & Views, Views for 90% of Spaces | 1 |

| Y | ? | N | | Innovation & Design Process | Possible Points |
|---|---|---|---|---|---|
| 3 | 2 | 0 | | | 5 |
| | 1 | | Credit 1.1 | Innovation in Design: 41% water savings | 1 |
| 1 | | | Credit 1.2 | Innovation in Design: Green houskeeping | 1 |
| | 1 | | Credit 1.3 | Innovation in Design: Exceed MR 5.1 | 1 |
| 1 | | | Credit 1.4 | Innovation in Design: SS 7.1 100% underground parking | 1 |
| 1 | | | Credit 2 | LEED™ Accredited Professional | 1 |

# Conclusions

## RESULT COMPARISON

During the structural breadth study, an alternate framing system of composite concrete was investigated. The goal was to keep the trial floor system as thin as possible. Creating a gravity system with W shapes of 18" or less in depth was not possible. Using W21 shapes the floor system would likely increase in depth. The resulting plan includes many W21 shapes, but primarily along the perimeter. It was speculated that if the deeper beams were not covering too much floor area that the HVAC ductwork could be run below the smaller beams and complete air distribution could be achieved without a duct passing under a W21 member. The resulting floor plan with W21 members highlighted is shown below.



It is apparent that the air distribution ducts must pass below a W21 beam at some point, so therefore it will be best to consider this case and note the consequential loss of ceiling height. The story height is 11.69' = 140.28 inches. The total depth of the floor system is attributed to 4 ½ inches of slab and deck, 21 inches of steel beam, 1 inch of fireproofing, and 10 inch ducts with 1 inch of insulation on either side. The floor depth before the ceiling is applied comes to 101.78 inches. Assuming ¾ inch plaster ceiling leaves 101 inches, or 8'-5" of ceiling height. The architectural plans for the original structure call for 9' finished ceilings. This is a loss of 7 inches.

Concerning the lateral resisting system, the steel frame reduces overall building weight by over 300%. This results in a decreased equivalent load on the structure. Steel is inherently more ductile than concrete and generally behaves better in seismic events anyway. The steel cannot meet the most stringent of drift limitations that the concrete system was able to, but passes baseline restrictions. The concrete moment frame requires attention to construction practices, and precise rebar layout. Similarly, great care must be

paid to the moment connections in the steel moment frame. Field welding will be required for many connections. This can cause a considerable cost and schedule delay.

The first cost for the existing concrete system ended up being very comparable to the alternate steel frame design. Fluctuations in actual cost could have either system prove more cost effective. When schedule in considered, steel tends to be somewhat quicker when the time for reshoring and post-tensioning of concrete is considered. The lead time for the steel holds back the steel schedule, but still results in a comparably shorter project with 224 days versus 265 days for concrete work and curing.

The LEED rating of the building will not be hurt by the choice to use steel construction. Steel can achieve high levels of recycled content. Either system with proper planning and design can reach a respected LEED rating.

## RECOMMENDATION

Based on all the factors considered in this design, and attributes of the existing structural system for the office building in question, I would encourage use of the original buildings structural system. This is largely due to the fact that the composite steel system creates an increase in structural floor depth. Cost did not largely contribute to the decision, as the systems seemed similar. There is a certain level of uncertainty with the material costs used for the cost estimate, but any variation should not have a considerable impact. Steel erection is a specialized skill that requires a competent contractor. While many contractors exist in every city, it is more likely that quality concrete construction can be achieved in Washington, DC. However, there seem to be advantages in the steel schedule duration. If lead time can be minimized, structural steel does not require the wait time associated with concrete shoring and curing. LEED considerations do not vary greatly between projects. A LEED gold rating seems attainable with some extra effort. The comparison of this added effort and cost to the benefit should be investigated further.

The existing post-tension concrete design appears to actively address all of the design concerns associated with the project. The true surprise is that any alternative system was able to create such a reasonable substitute to the original.

# Acknowledgements

This research has not been a solo effort. I have received help from many individuals, corporations, organizations, and resources. I would like to thank the following.

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www.truland.com

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www.krjda.com

Centex Construction
www.centex-construction.com

Tadjer Cohen Edelson Associates
www.tadjerco.com

My thesis advisor M. Kevin Parfitt, and the entire Architectural Engineering faculty at The Pennsylvania State University

## RESOURCES

American Concrete Institute
ACI 318-05 - Building Code Requirements for Structural Concrete.

American Institute of Steel Construction
Manual of Steel Construction – Load and Resistance Factor Deign, 3rd Edition.
"LEED Credits Given For Locally Fabricated Steel" Feb 24, 2004
"Steel takes LEED with recycled content" Jan 2005

American Society of Civil Engineers
ASCE Standard 7-02 – Minimum Design Loads for Buildings

International Code Council
International Building Code, IBC 2003

Verco Manufacturing
http://www.vercodeck.com/

DC zoning
http://dcoz.dc.gov/info/map.shtm

Underwriters Laboratory Certifications Directory
http://database.ul.com/cgi-bin/XYV/template/LISEXT/1FRAME/showpage.html?name=BXUV.D764&ccnshorttitle=Fire+Resistance+Ratings+-+ANSI/UL+263&objid=1074327207&cfgid=1073741824&version=versionless&parent_id=1073984818&sequence=1

United States Green Building Council
LEED for New Construction Version 2.2

RS Means
Building Construction Cost Data 2005