second technical report

a pro-con analysis of alternate structures for the new york times building

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IPD/BIM Structural Thesis

October 30, 2009
New York, NY

TABLE OF CONTENTS

EXECUTIVE SUMMARY

The purpose of the second technical report is to investigate alternate structural floor systems as a replacement for the existing composite steel beam and slab system of the New York Times Building. Three alternate systems were studied to assess their applicability as replacements:

1. Non-composite Steel
2. Precast Hollow Core Planks
3. Two-Way Reinforced Concrete Slab

For the analyses, two 30'-0" by 40'-0" bays were selected from the eighth floor, as shown in Figure 1 on the next page, to represent the typical structural framing of the office floors in the tower. The sixth floor was specifically selected to maintain consistency between options in the BIM Thesis group, as it is representative of a typical tower floor. Although this floor appears to be an exterior bay, there is a 5'-0" cantilevered framing section at column line A, visible to the left in Figure 1, that supports the exterior façade. For simplification, it was assumed that the façade loads on the cantilevered section would not impact the loads of the bays considered in the analyses.

Through hand calculations and software analyses, typical alternate framing systems were developed for the New York Times Building. The design of these systems was performed using the 13th Edition AISC Manual, ACI 318-08, and Nitterhouse Concrete Products' specifications. A pro-con analysis was then completed for each system in order to display the merits and drawbacks of the designs. These analyses were compared according to nine criteria, including constructability, operability with other systems, and structural weight impact on foundations, to determine the best alternative; results are tabulated in Table 9 on page 24. Through this comparison, the two-way reinforced concrete slab was found to be the most a feasible alternative considered for a high-rise structure. However, the changes it would impose on the structure, including a 15" slab depth, more than 2.7 times more weight, and a change in the architectural aesthetic, are very significant. Since the building will be undergoing many changes in the future through the BIM proposal, these effects may not have as great of an impact on the final redesign as they do on the original structure. Elements can be completely updated according to the combined proposal. Additionally, it would be worth looking into post-tensioning the two-way slab to reduce the overall structural depth and weight.

The non-composite steel system added 6" to the structural depth and an overall 5% increase in structural weight, not including columns. It was less efficient than the composite system, yielding W24x68 members as opposed to W18x35 members. Because of this, it is not considered to be a viable alternative. In addition, it was found that the hollow core system is not an appropriate option for a building of this height. Although the average structural depth was approximately equal to that of the existing composite system, the weight of the system was approximately 30% higher. This increase would impact foundations, but is potentially manageable with a change in the foundation system.

However, construction of the hollow core system requires a good deal of maneuverability between steel framing during construction. A crane must also be able to place the planks on all levels, which would be difficult on a small site in New York City.

INTRODUCTION

The 52-story New York Times Headquarters Building is located on Eighth Avenue between 41st and 42nd Streets. Home to the New York Times newsroom, 26 floors of Times administrative offices, and several law firms, it was intended to be a flagship building promoting sustainability, lightness, and transparency. The architectural façade reflects the ever-changing environment surrounding the building, an appropriate acknowledgement of the heart of New York City. Thornton Tomasetti worked closely with architect Renzo Piano to create a building that displayed not only transparency in

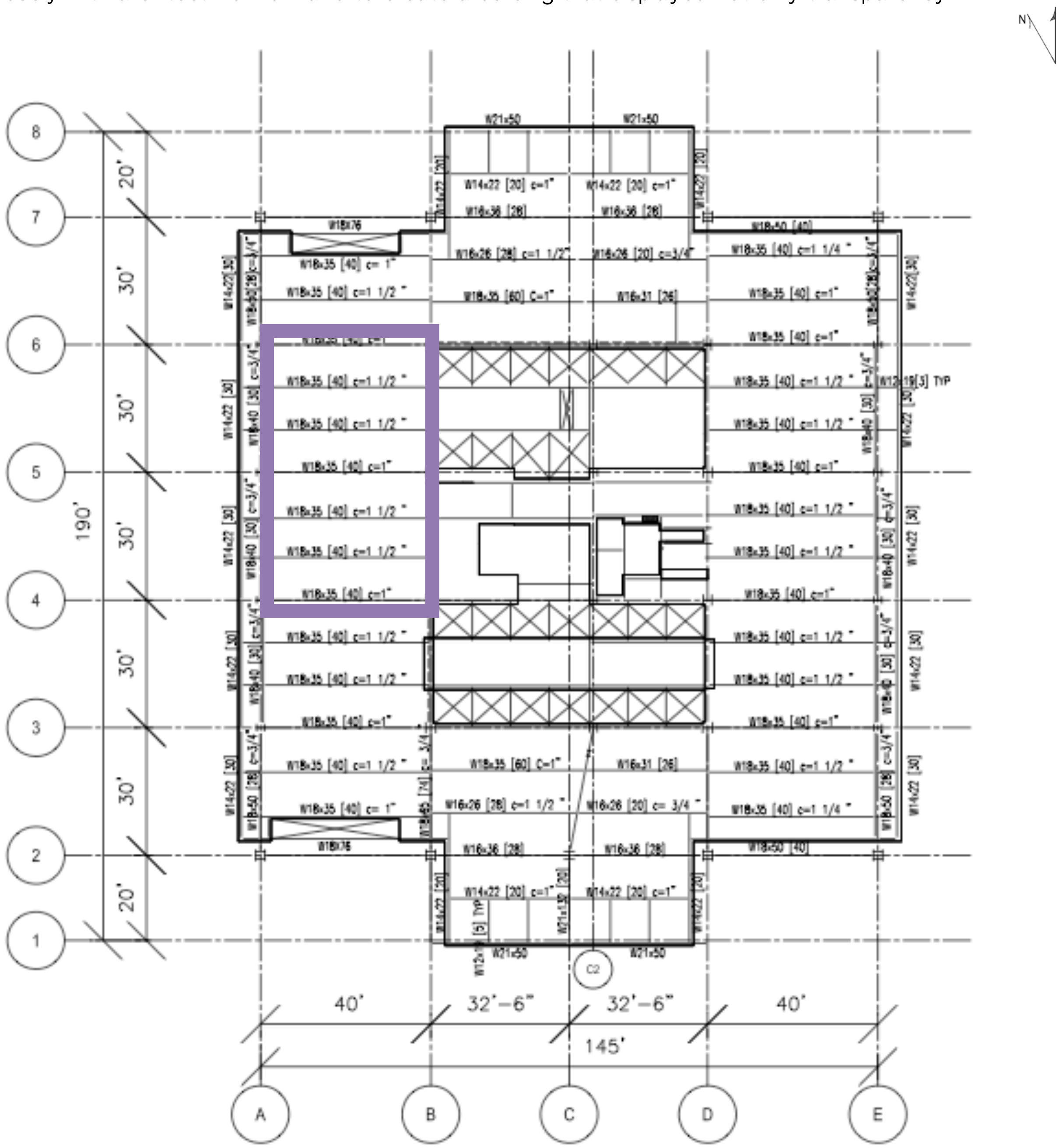

Figure 1: Typical tower framing plan

the media, but also structural transparency. For this reason, exterior columns, X-bracing, and beams were shifted outside of the façade, and the visual appearance of these elements and connections was given special attention. This architectural aesthetic was considered when selecting feasible alternate structures.

The office floors are intended to be open plans, with minimal disturbance from columns and other structural elements. For this reason, two-story outriggers were used at mechanical levels (floors 28 and 51) to engage exterior columns in the lateral system and increase stiffness. Story heights average approximately 13'-9", and floor-to-ceiling heights are approximately 10'-9" due to the 16" allowance for an under-floor air distribution system. The existence of this mechanical system was also taken into consideration when designing alternate systems.

CODES AND REFERENCES

Original Design Codes:

National Model Code:
- 1968 Building Code of the City of New York

Structural Standards:
- ASCE 7-98, Minimum Design Loads for Buildings and other Structures

Structural Design Codes:
- AISC – LRFD, Steel Construction Manual 2nd edition, American Institute of Steel Construction, 1998
- National Building Code of Canada, 1995
- Uniform Building Code, 1997

Design Deflection Criteria:

Lateral Deflections:
- Total building sway deflection for 10-year wind loading is limited to H/450

Thermal Deflections:
- The shortening and elongating effects due to thermal fluctuations are designed to L/300.
- At this point in time additional gravity and lateral deflections were not disclosed.

Thesis Design Codes:

National Model Code:
- 2006 International Building Code

Structural Standards:
- ASCE 7-05, Minimum Design Loads for Buildings and other Structures

Structural Design Codes:
- AISC, Steel Construction Manual 13th Edition
- ACI 318-08, Building Code Requirements for Structural Concrete
- PCI Design Handbook, Precast and Prestressed Concrete, 6th Edition via Nitterhouse Concrete Products

MATERIAL STRENGTHS

Structural Steel:

Wide Flanges Shapes……………………………………………ASTM A572 or A992, Grade 50
Built-Up Sections……………………………………………ASTM A572, Grade 50 & Grade 42
HSS Shapes…………………………………………………………………ASTM A500 Grade B
Diagonal & X-Braced Rod…………………………………………………ASTM A572, Grade 65
Connection Plates………………………………………………………………………ASTM A36

Concrete:

Caissons…………………………………………………………………………………f'_c = 6000 psi
Spread Footings………………………………………………………………………f'_c = 6000 psi
Slabs on Deck (normal weight concrete) U.N.O……………………………………f'_c = 4000 psi

Metal Decking:

3" Composite Deck…………………………………………………………………F_y = 40 ksi

At this point in time, the designer did not disclose shear stud, weld, bolt, and reinforcement strengths.

STRUCTURAL SYSTEM

Foundation

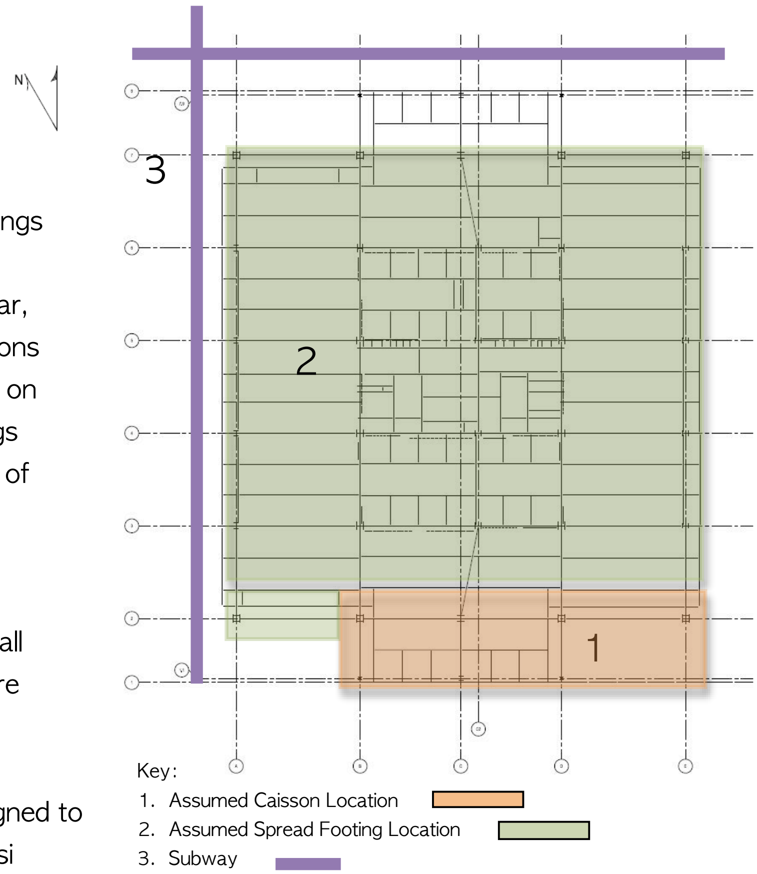

Figure 2: Foundation locations

The foundation of the New York Times Headquarters combines typical spread footings with caissons to achieve its maximum axial capacity. Below the building's 16-foot cellar, the tower and podium mostly bear on 20 tons per square foot rock; in this area, indicated on Figure 2 in green, 6,000-psi spread footings were used under each column (dimensions of footings not disclosed by the design team). However, at the southeast corner of the tower, the rock only has 8 tons per square foot capacity. At the seven columns that fall within this area, indicated in orange on Figure 2, 24-inch diameter concrete-filled steel caissons were used to replace the original foundation designs. Each caisson was designed to support a load of 2,400 kips with 6,000 psi concrete. The structural engineers did not disclose the depth of the caissons; it is only known that they extend until they reach rock with a bearing capacity of 20 tons per square foot or greater.

The New York City Subway passes below Eighth Avenue to the west and 41st Street to the north of the New York Times Building. However, this is not a major site restriction since the transit system is not directly beneath the structure.

Floor System

The floor system is a steel composite system with a typical bay size of 30'-0"x 40'-0", with 2½" normal weight concrete on 3" metal deck. Typical beam sizes are W18x35 with a 10'-0" typical spacing, bearing on W18x40 girders. The girders frame into the various built-up columns, box columns along the exterior and built-up non-box columns in the core. Framing of the core consists of W12 and HSS shapes framing into W14 and W16 shapes, which bear on W33 girders.

In the New York Times spaces, the structural steel is 16 inches below the finished floor to accommodate the under-floor air distribution plenum. Because the façade is transparent and office spaces are visible from the exterior, the architect wanted members passing through to the outside to line up with the perceived floors. To align the girder with the office floor level and not the level of the structure, engineers created a "dog leg" at the end of the girders on these floors. Figure 3 depicts the

Figure 3: 'Dog-leg' beam connection

dog leg during construction; shaded glass was used to mask the location of the girder, as shown in Figure 4. The top of steel of the girder is at the bottom of the shaded glass in the figure, and the shaded glass covers up the plenum.

Figure 4: 'Dog-leg' beam connection

Columns

The 30" by 30" box columns (Figure 5), exposed at the exterior corners of the tower, as seen in Figure 4, consist of two 30-inch wide flange plates and two web plates inset three inches from the exterior of the column on either side. Each web plate decreases in thickness from 7 inches at the bottom of the building to adjust to the loads at each level. The flange plates decrease thickness from 4 inches to conform to the "lightness" of the architecture with an increase in elevation. Although the yield strength of the plates also varies with tower height, the strength was assumed to be a uniform 50 ksi for calculations. Interior columns are a combination of built-up sections and rolled shapes. Column locations stay consistent throughout the height of the building, spaced with the grid at 30 feet in one direction and 40 feet in the other. Every column is engaged in the lateral system via connections to bracing and outriggers; this system is described in more detail in the lateral system section.

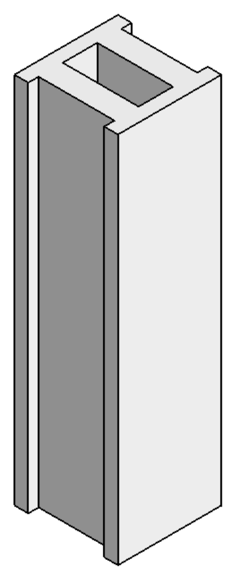

Figure 5: Box column as modeled in Revit

Lateral System

The main lateral load resisting system for the tower of the New York Times Building consists of a centralized steel braced frame core with outriggers on the two mechanical floors (Levels 28 and 51) to engage the exterior columns. The structural core consists of single diagonal bracing and concentric and eccentric chevron bracing, which surround elevator shafts, MEP shafts, and stairwells. At this time, the member sizes of these braces have yet to be disclosed. The core configuration remains consistent from the ground level to the 27th floor as shown in Figure 6 on this page. But above the 28th floor, some elevators were no longer required due to capacity. In order to optimize the rentable space on the upper levels of the tower, the number of bracing lines in the North-South direction was reduced from two to one (Figure 7). Please refer to Figures 8 and 9 on the next page to view the typical core bracing elevations.

The outriggers on the mechanical floors consist of eccentric and single diagonal braces (seen in Figures 30-32 in Appendix E). The outrigger system was designed to increase the efficiency and redundancy of the tower by engaging the perimeter columns into the lateral system.

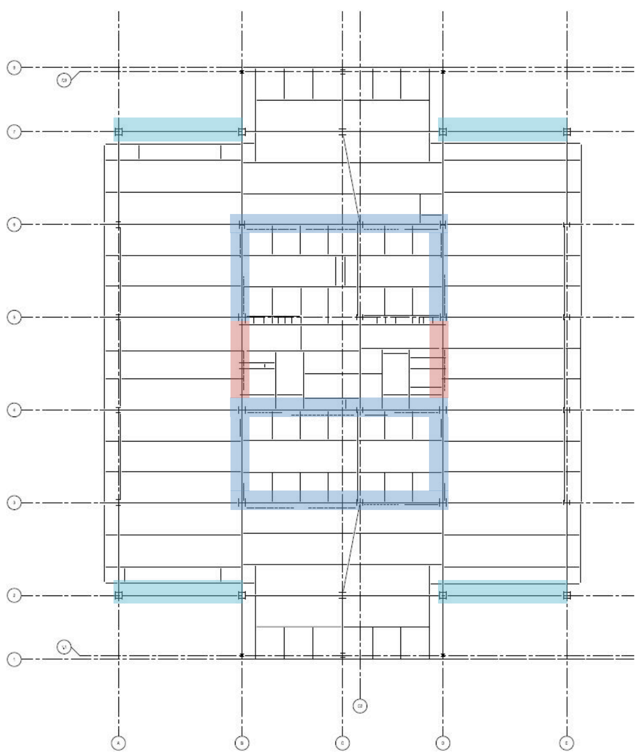

Figure 6: Typical Lateral System (Floors 1-27)

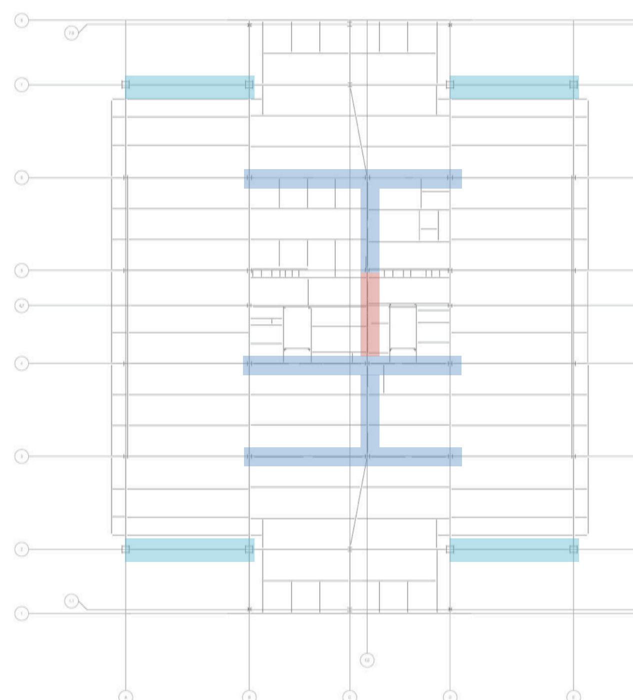

Figure 7: Typical Lateral System (Floors 29-50)

Key:
1. Single Diagonal Bracing
2. Pre-Tensioned Steel Rod X-Bracing
3. Chevron Bracing

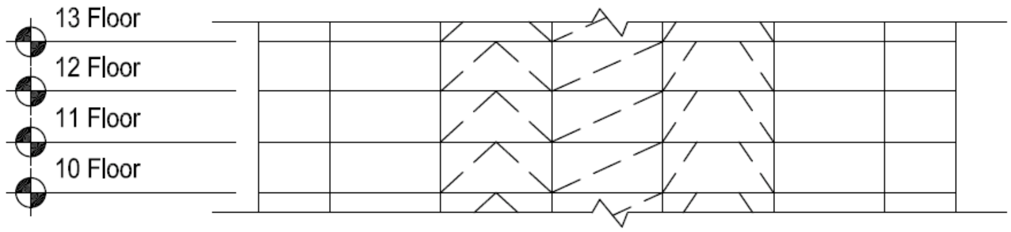

Figure 8: Typical Core N/S Core Bracing Elevation

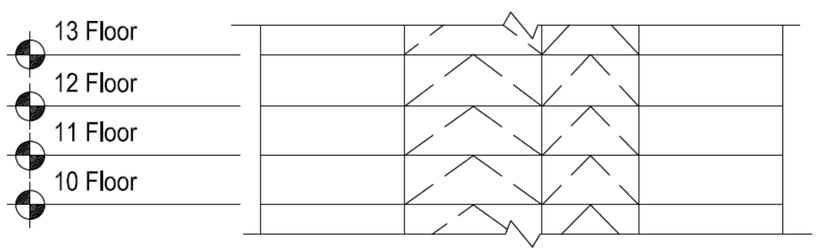

Figure 9: Typical Core E/W Core Bracing Elevation

During the design of the tower, the engineers at Thornton Tomasetti sized the members of the main lateral force resisting system merely for strength. In order to increase stiffness and meet drift and deflection criteria, the structural engineers utilized the double story steel rod X-braces (original to Renzo Piano's exterior design) instead of increasing the member sizes of the main lateral force resisting system. These X-brace locations can be seen in Figure 30 of Appendix E and in Figures 6 and 7 above, as well as in the photo on the cover page. The high strength steel rods transition from 2.5" to 4" in diameter and were prestressed to 210 kips. This induced tensile load prevents the need for large compression members that would not conform to the architectural vision of the exterior.

Although the X-braces reduced the need for an overall member size increase, the lateral system still did not completely conform to the deflection criterion. Therefore, some of the 30" by 30" base columns were designed as built-up solid sections that reduced the building drift caused by the building overturning moment. After combining these solid base columns and the X-braces with the main lateral force resisting system, the calculated deflection of the tower due to wind was L/450 with a 10 year return period and a building acceleration of less than 0.025g for non-hurricane winds.

According to information obtained from the structural engineer, the podium of the New York Times Building was designed with a separate lateral system. Though the owner did not disclose information about the podium, it is known that the lateral system is comprised of concrete shear walls.

EXISTING SYSTEM: COMPOSITE STEEL

Design Process

Composite beams and girders were analyzed by hand using the 13th Edition AISC Manual; calculations for the typical bays are available in Appendix A. This analysis was supplemented with a RAM Structural System model of the typical bays, shown in Figure 10 on page 15, to check for accuracy. Flexural strength controlled the design of the typical beam and girder, although they were cambered 2 inches and ¾ inch, respectively, to control deflections. This camber was necessary to meet the deflection criterion of L/360 for loads during construction that occur before composite action is achieved. The girder size obtained through analysis was a W21x44, larger than the W18x40 specified by Thornton Tomasetti. However, this difference, as well as the lower obtained number of shear studs, can be attributed to differences in live load reductions and levels of composite action.

Design Considerations

Several elements were taken into consideration for comparison of the systems. For the composite steel system, the weight on the foundations was lower than all other systems analyzed. The cost of the system is also approximately the same as the cost of a concrete or non-composite system, depending on availability and different resources consulted. Deflections are a potential problem in this system, due to the deflection value on the girder that was just barely under the L/360 construction load criterion. Vibrations should also be taken into consideration due to the 40-foot span length. However, the system is likely heavy enough with the slab to counteract these effects. The fireproofing is assumed to be a 2-hour rated system, compliant with specifications. In terms of constructability, composite steel systems are typical for buildings in New York City, so there are enough skilled trades available to easily construct the system. One issue is with the lateral system, though; it is made up of several types of bracing in order to achieve the necessary stiffness, and it is also supplemented with exterior bracing. This is a relatively complicated design, but was executed in this case. Other merits of this system include its compliance with the original architectural vision, the flexibility of the framing plans, and the recycled content of steel. The design criteria as well as a comparison of the most weighted factors are shown on the next page, in Tables 1 and 2.

PROS	CONS
* Architectural aesthetic remains intact	* Vulnerable to vibrations and deflections
* Weight of system is relatively low	* Lateral system is complicated to achieve stiffness
* Easy to construct- skilled trades in the area	
* Steel is recycled; allows for open plans & daylighting	
* Flexible layout; can change with others' proposals	
* Cost is comparable to that of other systems	

Table 1: Composite steel system pro-con survey

DESIGN CRITERIA
Structural depth = 24"
f'c = 4000 psi
fy = 60 ksi
Self weight = 71.4 psf
Superimposed DL = 37 psf
Construction DL = 69 psf
LL = 70 psf

Table 2: Composite steel system design criteria

Floor Map

RAM Steel v12.1
DataBase: RAMstlcomp_10_28_09
Building Code: IBC

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Steel Code: AISC360-05 LRFD

Floor Type: sixth floor

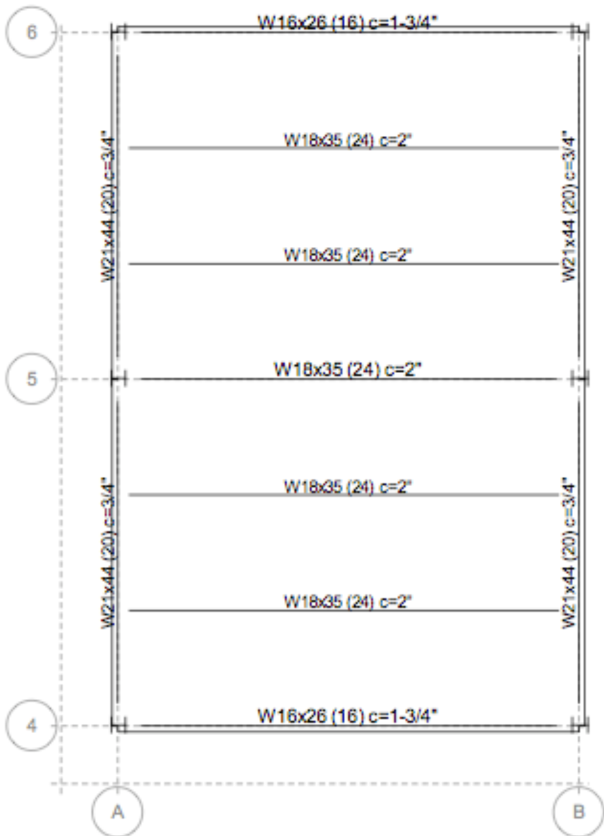

Figure 10: Composite steel system typical bays

FIRST ALTERNATE SYSTEM: NON-COMPOSITE STEEL

Design Process

Non-composite beams and girders were analyzed by hand using the 13th Edition AISC Manual; calculations for the typical bays are available in Appendix B. This analysis was supplemented with a RAM Structural System model of the typical bays, shown in Figure 12 on the page 18, to check for accuracy. The deck was first checked to ensure it was adequate to brace the beams before proceeding with calculations. Total load deflections at L/240 controlled the design of both the typical beam and girder. The beam was originally analyzed for flexure before it was determined that deflection would control; a larger section was then selected, assuming it was acceptable for flexure. The girder was then chosen based on its moment of inertia, and subsequently checked for flexure and deflections. Finally, shear was checked on both members, although it was likely that it would not control due to the 30-foot and 40-foot typical span dimensions.

Design Considerations

The weight of the non-composite steel system was higher than that of the composite steel system by 3.8 psf; beam sizes changed from W18x35 to W24x68 and girder sizes increased from W21x44 to W21x62. Although this does not seem like a large difference, it translates to an approximately 5% increase in the total load on the footings and caissons. The cost of the system is about the same as the cost of a concrete or composite system, depending on availability and different resources consulted; costs will be saved because there are no shear studs, but that cost will likely be regained after factoring in the extra weight of the structural steel. Vibrations are also more likely to have an impact than with the concrete systems, and should be taken into consideration due to the 40-foot span length. However, vibrations are typically only a problem for lighter floor systems; the large members have a much stiffer section than other systems such as open web joist framing. The fireproofing easily achieves a 2-hour rating using the same spray-on application as the existing system. The non-composite system is similar in construction to a composite system, and likewise there are plenty of skilled trades available in the area to erect the steel and pour the slabs. The same problem is also encountered with the complicated lateral system, although in both cases this system could be simplified by using only one or two types of bracing. Architectural aesthetics do not have to be compromised for the design aside from a potential increase in the size of the exposed box columns and framing plans are flexible to potential changes in floor layout. The design criteria for the non-composite system as well as a comparison of the most weighted factors are shown on the next page, in Tables 3 and 4.

DESIGN CRITERIA
Structural depth = 30"
f'c = 4000 psi
fy = 60 ksi
Self weight = 74.8 psf
Superimposed DL = 37 psf
LL = 70 psf

Table 3: Non-composite steel system design criteria

PROS	CONS
* Architectural aesthetic remains intact	* Vulnerable to vibrations and deflections
* Weight of system is relatively low	* Lateral system is complicated to achieve stiffness
* Easy to construct- skilled trades in the area	* Floor-to-ceiling height reduced
* Steel is recycled	* Added weight on foundations
* Flexible layout; can change with others' proposals	
* Cost is comparable to that of other systems	

Table 4: Non-composite steel framing pro-con study

Floor Map

RAM Steel v12.1
DataBase: RAMstlnon_10_28_09
Building Code: IBC

10/29/09 18:40:26
Steel Code: AISC360-05 LRFD

Floor Type: sixth floor

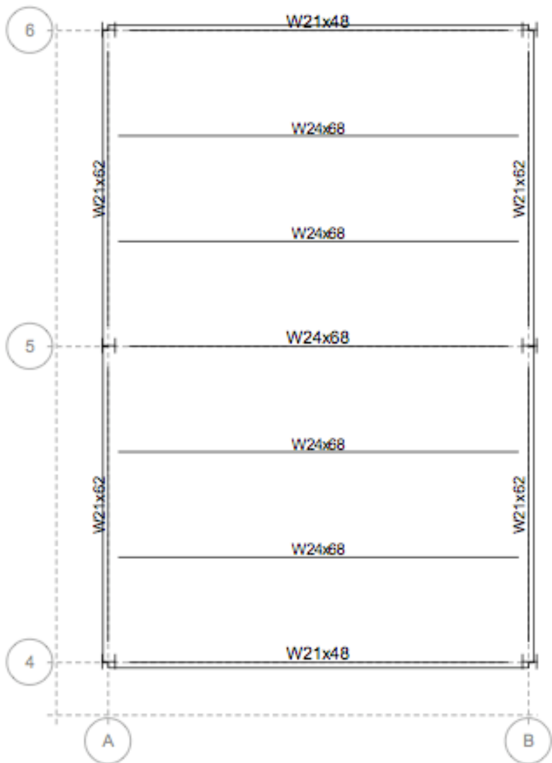

Figure 11: Non-composite steel framing layout

SECOND ALTERNATE SYSTEM: HOLLOW CORE PLANKS

Design Process

Precast Hollow Core Planks were designed using the PCI Design Handbook and loading values from the Nitterhouse Concrete Products specifications. A lower dead load was selected for use with this system; 12 pounds per square foot was removed that originally accounted for a ceiling treatment. In this case, the bottom of the planks can be finished to replace the drop acoustical tile ceiling. Calculations and design specifications for the typical bays are available in Appendix C; the framing view shows that two planks will have to be cut in order to fit in the 30-foot bays. Alternatively, planks could possibly span over the beam, but this could create some unwanted moments and forces in the plank. The design was checked for live load deflection, which was much lower than the L/360 required. The total factored load controlled the design of the thickness, which is 16 inches plus a 2-inch topping slab for fire protection. The girder supporting the planks was also designed using AISC to be a W21x62 steel member.

Design Considerations

This system had a larger impact on the framing of the building as compared with the non-composite steel system. Total weight of the system increased 30% from the weight of the composite system, which would greatly affect the weight transferred to the foundations. At 16 inches, depth stays fairly consistent between this system and the composite system, but there is much less space to fit MEP as well as the under floor air distribution system. It is assumed that some of the larger structural depth of the composite system is occupied by MEP as well. The cost of the system is also higher than the cost of a concrete or non-composite system, since labor is required to place and install the planks. Deflections did not prove to be an issue in this system, as expected of a stiffer concrete floor, and vibrations will probably not have a large effect due to the depth of the planks. The fireproofing is also a 2-hour rated system, consistent with the steel framing systems. However, a major issue that would need to be dealt with for a hollow core framing system is constructability. The site in New York City is not large, and cranes were not allowed to extend outside of the block on which the New York Times Building was being constructed. Therefore, it would be difficult to get all of the precast planks to their respective floors given these constraints. In addition, planks must be maneuvered between steel framing in order to be placed, which would further limit movements. This framing system would not necessarily affect the lateral system; since primary girders and columns are steel, the same connections and bracing layouts can be achieved. Fly ash can be added into the concrete as a practice of sustainability. Unfortunately, though, the flexibility of the framing plans and open office spaces is compromised by this system as it ideally needs equal, rectangular spans to function most efficiently and cost effectively. Planks can be cut to fit irregular shapes, but this has a large added cost. The design criteria as well as a comparison of the most weighted factors are shown on this page, in Tables 5 and 6.

DESIGN CRITERIA
Structural depth = 18″
f′c = 4000 psi
fy = 60 ksi
Self weight = 91.8 psf
Superimposed DL = 25 psf
LL = 70 psf

Table 5: Hollow core plank design criteria

PROS	CONS
* Fly ash can be added to concrete mix for sustainability	* Construction tight with crane to place planks
* Weight of system is relatively low	* Complicated lateral system to achieve stiffness
* Architectural aesthetic not drastically altered	* Floor-to-ceiling height reduced due to rerouted MEP
	* Added weight on foundations
	* May be difficult to coordinate with UFAD system
	* Cost higher than average
	* Bays must be rectangular and uniform for ease of construction

Table 6: Hollow core plank pro-con study

THIRD ALTERNATE SYSTEM: TWO-WAY FLAT PLATE

Design Process

The two-way flat plate system was analyzed by hand using ACI 318-08; calculations for the typical bays are available in Appendix D. A superimposed dead load of 25 psf was used, as shown in Figure 10 on the following page, as opposed to the 37 psf dead load used for the steel analyses. Again, this lower value was obtained after subtracting the weight of the ceiling from the total dead load. The design moments for the two-way slab were determined using the Direct Design Method, and were then distributed longitudinally and transversely using a combination of hand calculations and excel spreadsheets, as shown in Appendix D. Frame "A" and Frame "B" were analyzed as typical interior bays, although the thickness for an exterior bay was used as the controlling thickness. This is because the typical bay used in this report is somewhat of an exterior bay, since it receives minimal loading from one side of its framing. Calculations were performed to check wide beam and punching shears, and in both cases it was determined that the thickness was sufficient to prevent shear failure. Deflections were not calculated separately since the minimum thickness value from ACI Table 9.5(c) is meant to control deflections.

Design Considerations

Ten main elements were taken into consideration for analysis of this system. First, the system weight was calculated to be a massive 2.7 times greater than the system weight of the composite steel framing. Although the cost of the system is also approximately the same as the cost of a composite or non-composite system, the large effect that the building weight would have on foundations almost ensures that the overall cost would increase. Deflections and vibrations are likely not much of a problem in this system, due to the deep slab thickness. This slab thickness, at 15.5 inches, is comparable to the 24-inch composite framing depth once MEP is taken into consideration. However, the necessary change in lateral system would provide a large difference in the project schedule as well as the way in which the structure is built; finding skilled cast-in-place professionals would not be hard in New York City, though, where many concrete structures are built. Since there is no longer a steel skeleton to connect bracing elements, the lateral system would be changed to a concrete shear wall system. This has the benefit of providing added stiffness to the structure, but the concrete in general imposes other problems. The architecture would have to drastically change, since much of it is based on transparency of the façade and exposed structural elements. However, it is possible that the architecture will change during the BIM proposal process, so this is not taken as a harsh drawback. Fireproofing is also assumed to be a 2-hour rated system, sufficient for the original design. Finally, the concrete can be made into a more sustainable material through the addition of fly ash and other admixtures. The design criteria as well as a comparison of the most weighted factors are shown on this page, in Tables 7 and 8.

DESIGN CRITERIA
Structural depth = 15.5″
f′c = 5000 psi
fy = 60 ksi
Self weight = 194 psf
Superimposed DL = 25 psf
LL = 70 psf

Table 7: Two-way flat plate design criteria

PROS	CONS
* Fly ash can be added to concrete mix for sustainability	* Some MEP will need rerouted due to slab mass
* Easier to achieve stiffness with concrete core	* Large load increase on foundations
* Cost is comparable to that of other systems	* Drastic architectural aesthetic difference
* Constructability on par with steel	
* Less problems with deflection and vibrations	

Table 8: Two-way flat plate pro-con study

COMPARISONS & CONCLUSIONS

Three alternate structural systems (steel non-composite, hollow core plank, and two-way flat plate) were analyzed according to factors deemed important in the design, construction, and ownership of the building. Overall, the structures were judged based on nine criteria to determine which was the most feasible alternative, as seen in Table 9 on the next page. After in-depth analyses and calculations for each of these systems, it was determined that a two-way flat plate system would be the most appropriate substitute for a composite steel structure. This is not to say that it is a better option than the original; the two-way slab is not without its problems. For instance, the structural depth of the system decreased from 24 inches to 15.5 inches overall, but there is no usable space in the slab as there is between framing members in the composite system. In addition, the weight of the structure increased almost 300%, which will have a huge impact on the feasibility of the existing foundations. More than likely, the entire foundation system will have to be redesigned. The lateral system would also need completely redesigned for the concrete system, as steel braced frames are not the best pairing for a concrete floor system when it comes to load transfer, stiffness, and connections. However, the system does prove promising for future study with the Building Information Modeling Thesis Team; combined with post-tensioning, it could be an extremely viable alternative to the existing structure. Post-tensioning would decrease the thickness of the slab, as well as lighten up the system. Concrete can be used to incorporate a more inherent sustainability, as well. This is an option to be explored in further detail for the proposal.

The other two systems, steel non-composite and hollow core plank, did not necessarily fail all of the criteria. Each has its merits: the non-composite system ensures there is no change in the lateral system, and its framing layout is flexible for changes that could be imposed by the BIM Team at a later date; the hollow core plank system lightens up the weight of the concrete slab, and also improves serviceability concerns such as deflections and vibrations. However, non-composite seems much more inefficient due to the upsize from a W18x35 to a W24x68. It is also not taking advantage of the shear capacity of the slab. In addition, the hollow core planks are still bulky members, and also would be difficult to maneuver around the small New York Times Building site.

All in all, there needs to be further investigation into many systems that were not discussed in detail in this report, including the lateral and foundation systems. For the future proposal, more coordination between disciplines is necessary to fully reap the benefits of the project team and the BIM process. Modeling will be done to gain more insight into the structure and how it performs, including the drift under wind and seismic loading conditions. The steel composite structure seems like the best choice out of these four options, but there are more possibilities that will be explored in more detail.

Criteria/System	Steel Composite	Steel Non-Composite	Hollow Core Plank	Two-Way Flat Plate
System Weight		-	-	-
Cost		-	-	
Structural Depth		-	+	+
Serviceability			+	+
Constructability			-	
Architecture	*	-	-	-
Lateral System				+
BIM Proposal				+
Sustainability				

-	underperforms existing system
	on par with existing system
+	outperforms existing system
*	best system for criterion

Table 9: Pro-con comparison of alternate systems

APPENDIX A:

EXISTING COMPOSITE STEEL FRAMING

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NY TIMES

* EXISTING COMPOSITE STEEL FRAMING:

$f'_c = 4000$ psi
$f_y = 60$ ksi

3" deck + 2 1/2" NWC
UNSHORED CONSTR.

DL = 94 psf : SLAB = 55 psf
FMG = 4 psf
MEP = 25 psf
RAISED FLR = 12 psf

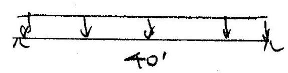
LL = 70 psf ; ASCE 7-05 OFFICE
+20 psf PARTITIONS

$DL_{constr} = 65$ psf

> DESIGN TYPICAL BEAM:

$L = L_o\left(0.25 + \frac{15}{\sqrt{A_I}}\right)$ $A_I = 30 \cdot 40 = 1200$ sf

$= L_o(0.658)$



$= 47.8$ psf

$W_u = 1.2(94 \cdot 10) + 1.6(47.8 \cdot 10) = 1.893$ KLF

$M_u = wl^2/8 = 1.893(40^2)/8 = 378.6'^k$

* assume $a = 1.0''$:

$Y_2 = 5.5 - 1.0/2 = 5.0''$

T3-19:
- W16x31 3 $M_p = 396^{'k}$ $\phi M_p = 203^{'k}$ $\Sigma Q_n = 335^k$
- W16x36 BFL $M_p = 395^{'k}$ $\phi M_p = 240^{'k}$ $\Sigma Q_n = 228^k$
- W18x35 6 $M_p = 404^{'k}$ $\phi M_p = 249^{'k}$ $\Sigma Q_n = 194^k$

→ select W18x35:

$b_{eff} = \min \begin{cases} l/4 = 10' \leftarrow \text{governs} \\ s = 10' \end{cases}$

$a = \frac{404}{0.85(4)(120'')} = 0.990'' < 1.0''$ OK

so 1.0 was a bit conservative

Figure 12: First page of composite steel hand calculations

NY TIMES

CHECK CONSTR. STRENGTH

$M_u = 1.2\,(69 \cdot 10/1000)\,(40^2)/8 = 165.6^{'k} < 249^{'k}$ OK

CHECK DEFLECTION

$\Delta_{LL} \le 1.33'' \therefore \Delta_{LL} = \dfrac{5\,(0.70)(40^4)}{384(29000)(1050)} = 1.32'' < 1.33''$ OK

$\Delta_{CL} \le 1.33'': \quad \Delta_{CL} = \dfrac{5\,(0.69)(40^4)}{384(29000)(510)} = 2.69''$

$-2''$ CAMBER $= 0.69'' < 1.33''$ OK

SHEAR STUDS: 3/4", $Q_n = 17.2^k$, + DECK

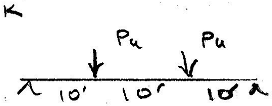
$\# = \dfrac{194^k}{17.2^k} = 11.28 \Rightarrow 12$ STUDS / SIDE

$= \boxed{24 \text{ TOTAL}}$

> DESIGN TYPICAL GIRDER:

$P_u = 1.893\,(20) = 37.9K$

$M_u = 37.9\,(10) = 379^{'k}$



T3-19:
- W16×31 $\quad M = 443^{'k} \quad \Sigma Q_n = 456^k$ TFL
- W16×36 $\quad M = 455^{'k} \quad \Sigma Q_n = 378^k$ 3
- W18×35 $\quad M = 435^{'k} \quad \Sigma Q_n = 260^k$ BFL
- W21×44 $\quad M = 516^{'k} \quad \Sigma Q_n = 162^k$ 7

→ try W18×35:

$b_{eff} = \min \begin{cases} 30/4 = 90'' \leftarrow \text{controls} \\ 40' \end{cases}$

$a = \dfrac{435}{(0.85)(4)(90)} = 1.42'' > 1.0$

$\therefore Y_2 = 5.5 - \dfrac{1.42}{2} = 4.79 \rightarrow 4.5$ in T3-19

W18×35 $\quad M = 448^{'k} \quad \Sigma Q_n = 260^k$ BFL

$a = \dfrac{448}{0.85(4)(90)} = 1.46'' < 1.5''$ OK

$\therefore$ assumption conservative

Figure 13: Second page of composite steel hand calculations

NYTIMES

CHECK CONSTR. STRENGTH

$M_u = 1.2(69 \cdot 10 \cdot \frac{40}{2})\frac{(10')}{1000} = 165.6^{'k}$

$< \phi M_p = 249^{'k}$ OK

CHECK DEFLECTIONS

$\Delta_{LL} = \frac{Pa}{24EI}(3l^2 - 4a^2) \leq \Delta_{LL} = L/360 = 1''$

$= \frac{(70 \cdot 10 \cdot 20/1000)(10)}{24(29000)(1130)}(3(30^2) - 4(10^2))$

$= 0.796'' < 1.0''$ OK

$\Delta_{CL} = \frac{(69 \cdot 10 \cdot 20/1000)(10)}{24(29000)(510)}(3(30^2) - 4(10^2))$

$= 1.739'' > 1.0'' \rightarrow$ camber $3/4''$

$\Delta = 0.99''$ OK

$\Delta_{TL} = \frac{(94+70)(10)(20/1000)(10)}{24(29000)(1130)}(3(30^2) - 4(10^2))$

$= 1.66'' > L/240 = 1.5''$ NG

try W21x44; $M = 498^{'k}$ $\Sigma Q_n = 162^k$ BFL

$a = \frac{516}{0.85 \cdot 4(90)}$

$= 1.686'' > 1.0$

$\therefore Y_2 = 3.5''$

$a = 1.63'' > 2''$ OK

$\Delta_{TL} = \frac{(\frac{94+70}{1000})(10)(20)(10)}{24(29000)(1340)}(3(30^2) - 4(10^2))$

$= 1.40'' < 1.5''$ OK

STUDS $= \frac{162^k}{17.2^k} = 9.4 \rightarrow \boxed{20 \text{ TOTAL}}$

$\therefore$ SEE NEXT PAGE FOR TYPICAL FRAMING LAYOUT.

Figure 14: Third page of composite steel hand calculations

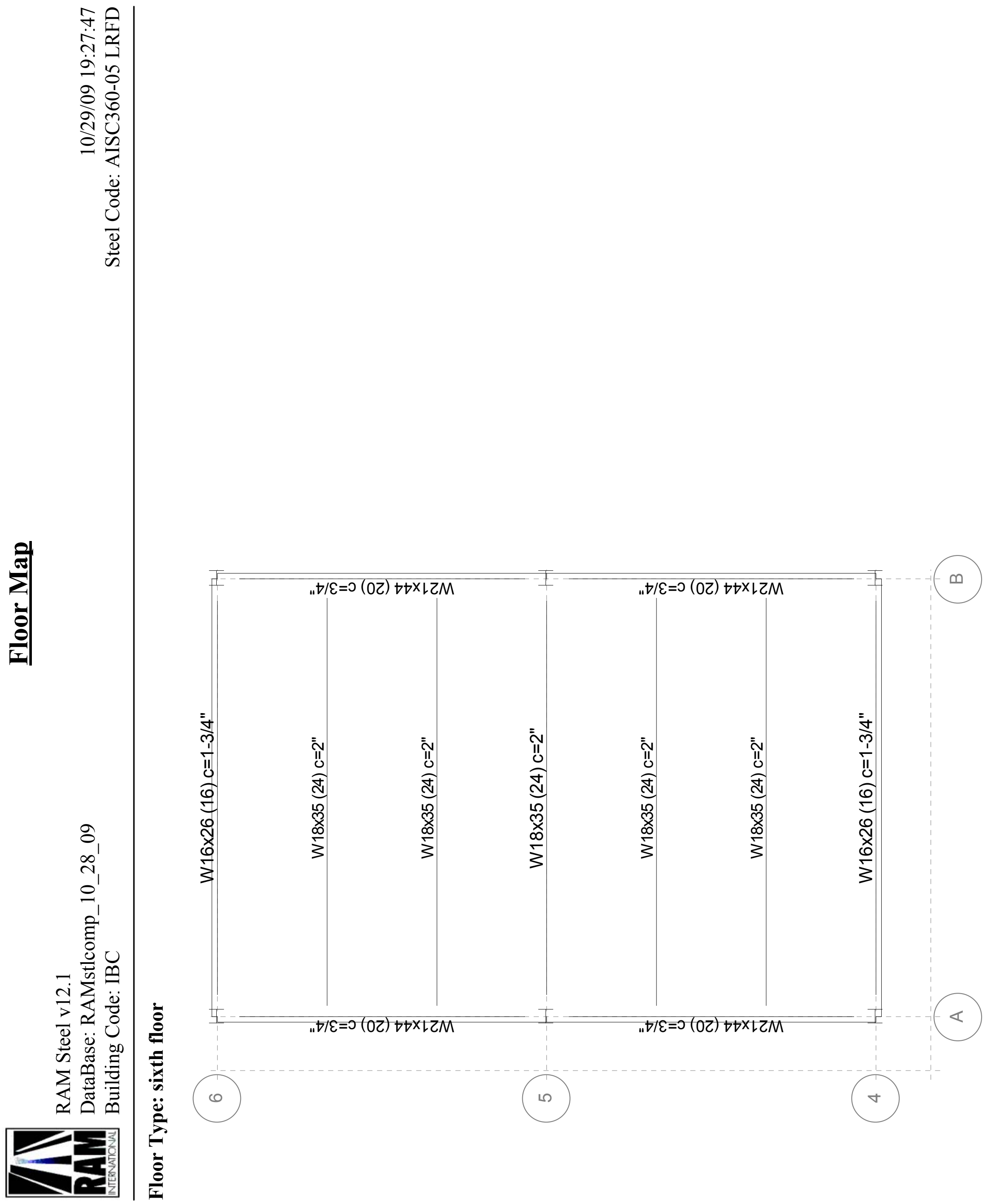


Figure 15: RAM Structural System composite framing plan

APPENDIX B:

NON-COMPOSITE STEEL FRAMING

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NY TIMES

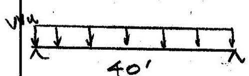
* NON-COMPOSITE STEEL FRAMING:

DL = 94 psf
LL = 70 psf
> DESIGN TYPICAL BEAM:

$w_u = 1.2(94 \cdot 10) + 1.6(47.8 \cdot 10) = 1.893 \text{ KLF}$

$L = L_o(0.658) = 47.8 \text{ psf}$

$L = L_o\left(0.25 + \frac{15}{\sqrt{A_I}}\right)$ $A_I = 30' \cdot 40' = 1200 \text{ sf}$



$= L_o(0.25 + 15/\sqrt{1200})$

$= L_o(0.658)$

$M_u = w l^2/8 = (1.893)(40)^2/8 = 378.6^{\text{'K}}$

* CAN YOU ASSUME BM. IS FULLY BRACED BY DECK?

FROM APP. 6.3: BRACE STRENGTH:

$P_{br} = 0.004 \, M_u C_t \cancel{C_d} / h_o$ $\; C_d \to 1.0$

$C_t = 2.0$: top flange loading

$h_o \, (21 \times 44) = 20.2$

$P_{br} = 0.004(378.6)(2)/20.2$

$= 0.150 \text{ K}$

3" DECK CAN CARRY 3.3K > 0.15K, OK
/FROM

SO, $L_b = 0"$ → FROM Z_x TABLES:

W21×48 $\phi M_p = 398^{\text{'K}}$

CHECK DEFLECTION:

$\Delta_{LL} \le L/360 = 1.33"$

$$\Delta_{LL} = \frac{5(70 \cdot 10)(40)^4}{384(29000)(959)} = 0.839" < 1.33 \text{ OK}$$

Figure 16: First page of non-composite steel hand calculations

NY TIMES

$\Delta_{TL} \le L/240 = 2.0"$

$\Delta_{TL} = \dfrac{5(0.70+0.940)(40)^4 \cdot 1728}{384 \cdot 29000 \cdot 959}$

$= 3.39" > 2.0"$ NG

$I_{req} = \dfrac{5(0.7+0.94)(40^4)}{384 \cdot 29000 \cdot 2} \cdot 1728$

$= 1629 \text{ in}^4$

$\rightarrow$ try W 24×68 $I = 1830 \text{ in}^4$ OK

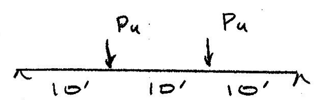
CHECK SHEAR

$V_u = \dfrac{wl}{2} = \dfrac{1.893(40)}{2} = 37.9K$

$\phi V_{n\,(W24\times68)} = 295K > 37.9K$ OK

> DESIGN TYPICAL GIRDER:

DL = 94 psf
LL = 70 psf



$P_u = (1.893)(20) = 37.86K$

$M_u = 37.9^K(10') = 379^{1K}$

$L_b = 10'$ → from T3-10, try W 21×62

$\phi M_p = 540^{1K}$ BF = 17.4 $L_p = 6.25' < 10'$ OK

$C_b = 1.14$

$\phi M_n = \phi M_p - BF(L_b - L_p)$

$= 540 - 17.4(10-6.25) = 474.8^{1K}$

$474.8(1.14) = 541.2^{1K} > 540^{1K}$

$540^K > 379^{1K}$ OK

Figure 17: Second page of non-composite steel hand calculations

NYTIMES

CHECK DEFLECTION

$\Delta_{LL} \leq L/360 = 30'/360 = 1''$

$\Delta_{LL} = \frac{Pa}{24EI}(3l^2 - 4a^2)$

$= \frac{(0.7 \cdot 20)(10)}{24(29000)(1330)}(3(30^2) - 4(10)^2)$

$= 0.601'' < 1.0''$ OK

$\Delta_{TL} \leq L/240 = 1.5''$

$\Delta_{TL} = \frac{(0.7 + 0.94)(20)(10)}{24(29000)(1330)}(3(30^2) - 4(10^2))$

$= 1.41'' < 1.5''$ OK

CHECK SHEAR

$V_u = 39.5^k < \phi V_n = 252^k$ OK

∴ TYPICAL FRAMING AS SHOWN ON NEXT PAGE.

Figure 18: Third page of non-composite steel hand calculations

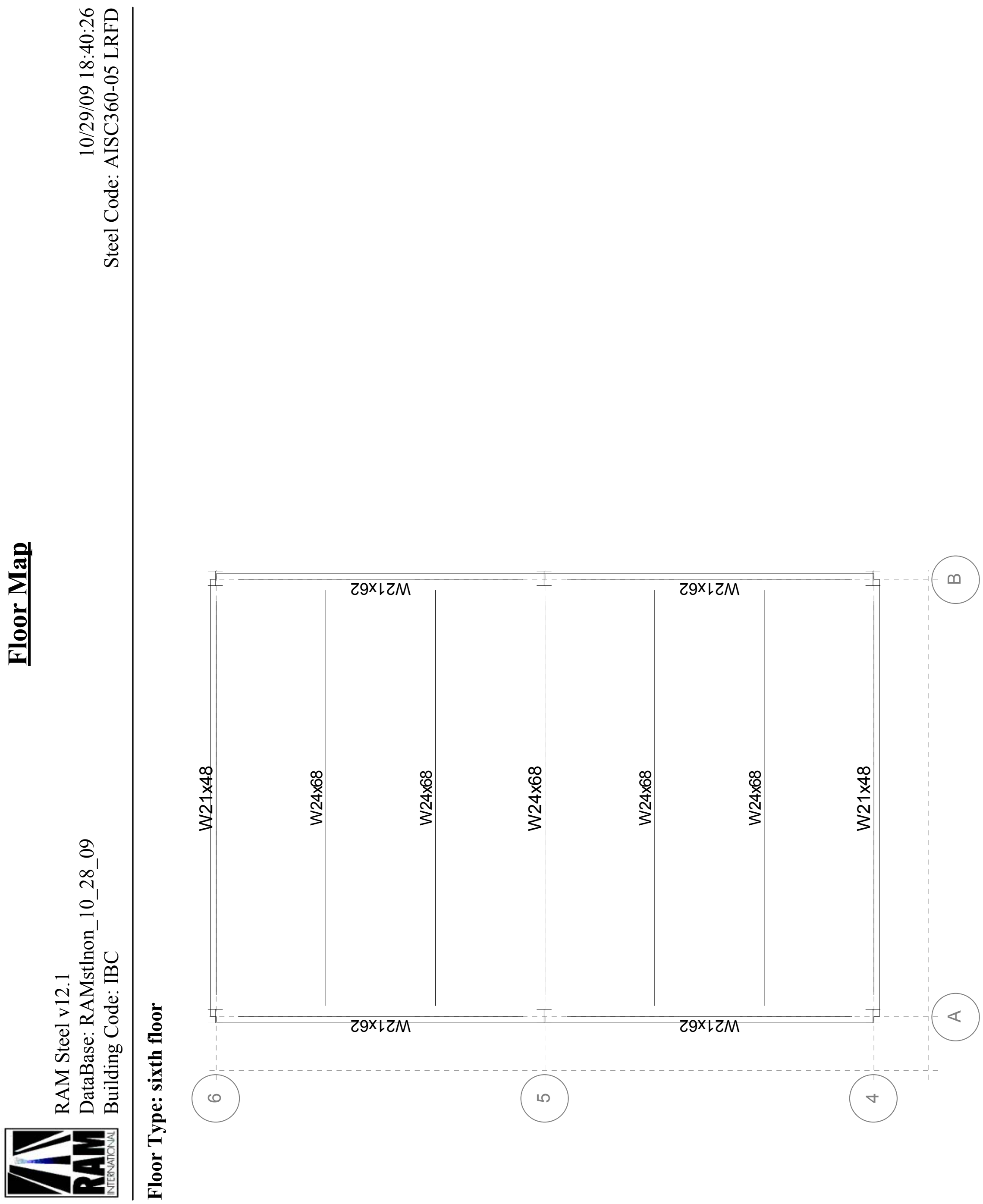


Figure 19: RAM Structural System non-composite steel framing plan

APPENDIX C:

HOLLOW CORE PLANK SYSTEM

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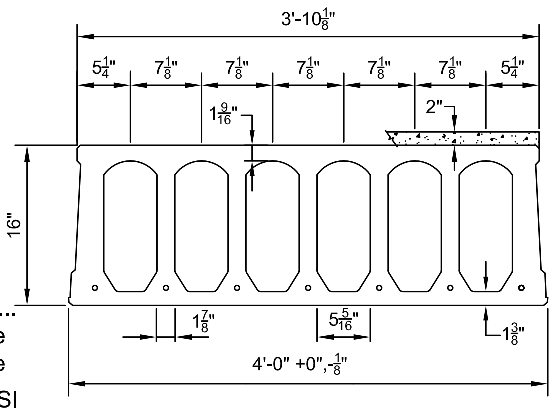
Prestressed Concrete
16"x4'-0" Hollow Core Plank

2 Hour Fire Resistance Rating With 2" Topping

PHYSICAL PROPERTIES
Composite Section

A_c = 418 in.^{2} Precast b_w = 14.25 in.
I_c = 15498 in.^{4} Precast S_{bcp} = 1653 in.^{3}
Y_{bcp} = 9.38 in. Topping S_{tct} = 2542 in.^{3}
Y_{tcp} = 6.62 in. Precast S_{tcp} = 2340 in.^{3}
Y_{tct} = 8.82 in. Precast Wt. = 367 PLF
Precast Wt. = 91.75 PSF



DESIGN DATA

1. Precast Strength @ 28 days = 6000 PSI
2. Precast Strength @ release = 3500 PSI
3. Precast Density = 150 PCF
4. Strand = 1/2"Ø and 0.6"Ø 270K Lo-Relaxation.
5. Strand Height = 1.75 in.
6. Ultimate moment capacity (when fully developed)...
   7-1/2"Ø, 270K = 323.1 k-ft at 60% jacking force
   7-0.6"Ø, 270K = 441.9 k-ft at 60% jacking force
7. Maximum bottom tensile stress is $10\sqrt{f'c}$ = 775 PSI
8. All superimposed load is treated as live load in the strength analysis of flexure and shear.
9. Flexural strength capacity is based on stress/strain strand relationships.
10. Deflection limits were not considered when determining allowable loads in this table.
11. Topping Strength @ 28 days = 3000 PSI. Topping Weight = 25 PSF.
12. These tables are based upon the topping having a uniform 2" thickness over the entire span. A lesser thickness might occur if camber is not taken into account during design, thus reducing the load capacity.
13. All load values are controlled by ultimate flexural strength or fire endurance limits.
14. Load values may be different for IBC 2000 & ACI 318-99. Load tables are available upon request.
15. Camber is inherent in all prestressed hollow core slabs and is a function of the amount of eccentric prestressing force needed to carry the superimposed design loads along with a number of other variables. Because prediction of camber is based on empirical formulas it is at best an estimate, with the actual camber usually higher than calculated values.

SAFE SUPERIMPOSED SERVICE LOADS		IBC 2006 & ACI 318-05 (1.2 D + 1.6 L)																		
Strand Pattern		SPAN (FEET)																		
		39	40	41	42	43	44	45	46	47	48	49	50	51	52	53	54	55	56	57
7 - 1/2"ø	LOAD (PSF)	131	118	107	96	87	77	69	61	53	46	41	34							
7 - 0.6"ø	LOAD (PSF)								129	119	109	100	91	83	76	68	62	55	49	50

NITTERHOUSE
CONCRETE PRODUCTS
2655 Molly Pitcher Hwy. South, Box N
Chambersburg, PA 17202-9203
717-267-4505 Fax 717-267-4518

This table is for simple spans and uniform loads. Design data for any of these span-load conditions is available on request. Individual designs may be furnished to satisfy unusual conditions of heavy loads, concentrated loads, cantilevers, flange or stem openings and narrow widths. The allowable loads shown in this table reflect a 2 Hour & 0 Minute fire resistance rating.

11/03/08

16F2.0T

Figure 20: Nitterhouse Concrete Products 16"x4' Hollow Core Plank Specification

NY TIMES

* HOLLOW CORE PLANK SYSTEM :

FROM NITTERHOUSE CONCRETE PRODUCTS SPEC. :

16" x 4'-0" Hollow Core Plank w/ 2" Topping
→ 2HR FIRE RATING

for 40' spans - safe superimposed load
= 118 psf

1.2D + 1.6L ≤ 118 psf   DL = 25 psf MEP/ceiling
LL = 70 psf

$1.2(25) + 1.6(47.8) = 106.5\text{psf} < 118 \text{ psf}$ OK

• $L = L_o\left(.25 + \frac{15}{\sqrt{A_I}}\right)$ → $A_I = 30 \cdot 40 = 1200\text{sf}$

$= 0.683L_o = 47.8\text{ psf}$

CHECK DEFLECTION

$$\Delta = \frac{5(70 \cdot 4/1000)(40^4)}{384(15498)(4415)} = 0.236'' < l/360 = 1.33''$$ OK

• $f'_c = 6000\text{psi}$

• $E = 57\sqrt{f'_c} = 4415 \text{ ksi}$

• $I = 15498 \text{ in}^4$

> DESIGN SUPPORT GIRDER :

$W_{plank} = 91.75 \text{ psf}$

$W_u = 1.2(25 + 91.75)(20) + 1.6(47.8)(20)$

$= 4.33 \text{ KLF}$

$M_u = \frac{4.33(30^2)}{8} = 487.3^{'K}$ $L_b = 0'$

→ from T 3-2 :

W21x62 $\phi M_p = 540'K$

Figure 21: First page of hollow core plank hand calculations

NY TIMES

CHECK DEFLECTION

$\Delta_{LL} \leq L/360 = 1.0''$

$\Delta_{LL} = \frac{5w\ell^4}{384EI} = \frac{5(0.0478 \cdot 20)(30^4)}{384(29000)(1550)} = 0.388''$

$< 1.0''$ OK

$\Delta_{TL} \leq L/240 = 1.5''$

$\Delta_{TL} = \frac{5(25+91.75+47.8)(20/1000)(30)^4}{384(29000)(1550)} = 1.33''$

$< 1.5''$ OK

CHECK SHEAR

$V_u = \frac{w\ell}{2} = \frac{4.33(30)}{2} = 65.0^k$

$< \phi V_n = 252^k$ OK

* FOR HOLLOWCORE FRAMING LAYOUT, SEE NEXT PAGE.

Figure 22: Second page of hollow core plank hand calculations

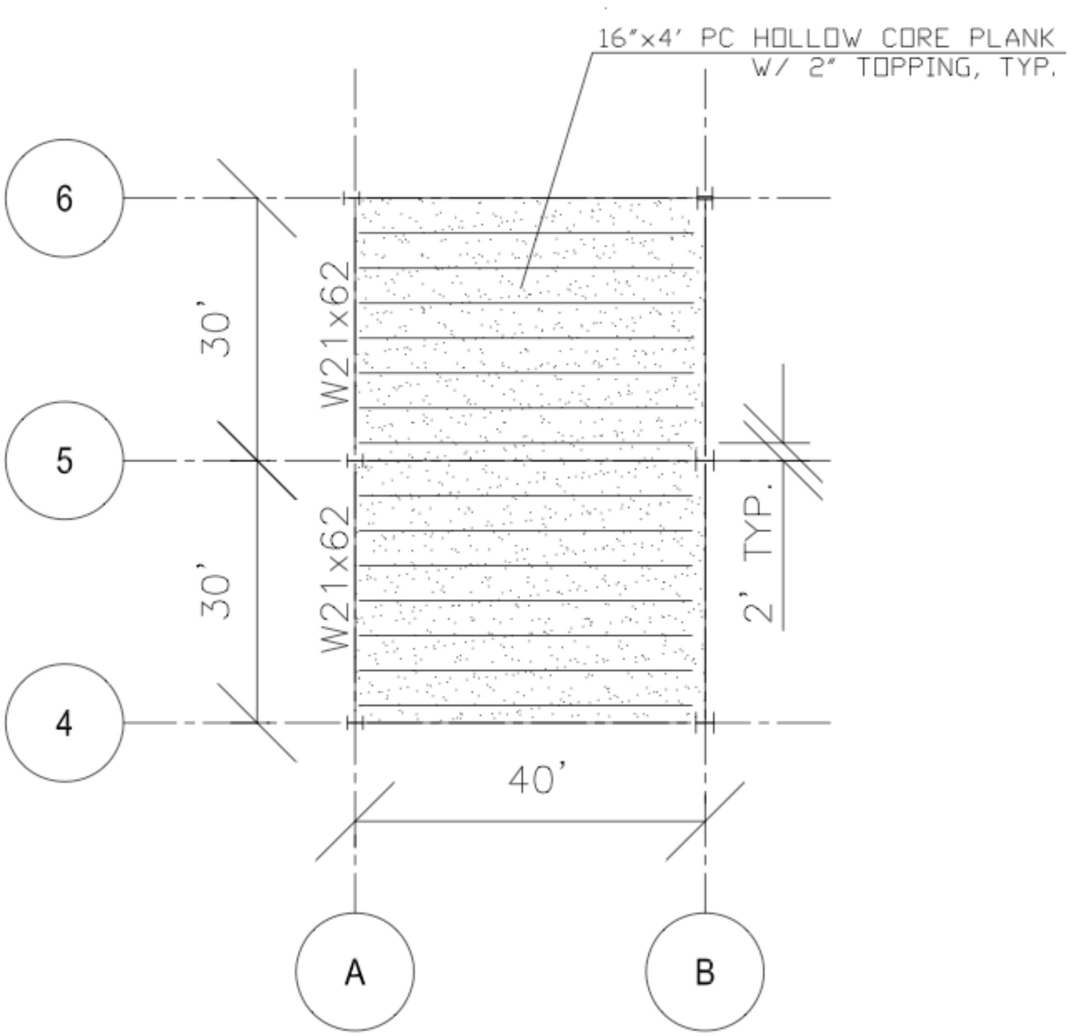


Figure 23: Framing plan for hollow core plank system

APPENDIX D:

TWO-WAY FLAT PLATE SYSTEM

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NY TIMES

* TWO-WAY FLAT PLATE FLOOR SYSTEM:

$f'_c = 5000$ psi

DL = 25 psf MEP

LL = 70 psf OFFICE + PARTITIONS

24×24" COLUMNS ASSUMED FOR SLAB CALCULATIONS

+ CHECK FOR USE OF DIRECT DESIGN METHOD:

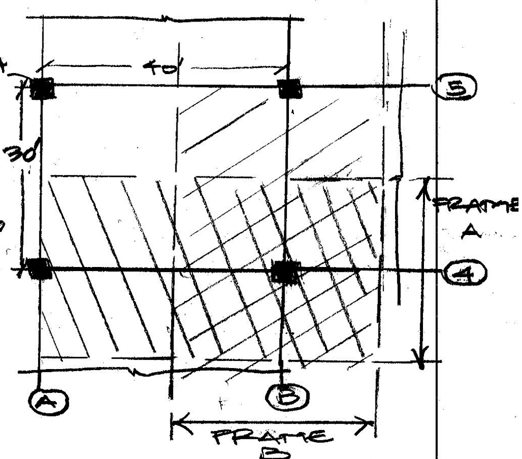
✓ 3 CONTINUOUS SPANS IN EACH DIRECTION

✓ $\ell_2/\ell_1 < 2.0$: $40/30 = 1.33$

✓ $\ell_2 - 32'\text{-}6" < \ell_2/3 = 13.3'$
$7.5 < 13.3$

✓ LOADS UNIFORMLY DISTRIBUTED

✓ LL < 2 DL
(SLAB WEIGHT > LL)



* MINIMUM THICKNESS PER ACI T.9.5(c)

ext.: $h_{min} = \ell_n/30 = (40 - \frac{24}{12})(12)/30 = 15.2"$

int.: $h_{min} = \ell_n/33 = (40-2)(12)/33$ DNC.

$w_{D,S} = (\frac{15.5}{12})(150) = 193.8$ psf

$w_u = 1.2(193.8 + 25) + 1.6(70)$
$= 374.5$ psf

FRAME A: $M_o = \dfrac{w_u \ell_2 \ell_n^2}{8} = \dfrac{0.3745(40)(28)^2}{8}$

$= 1468'^k$

Figure 24: First page of two-way flat plate hand calculations

NY TIMES

› DISTRIBUTE M_o:

CS width = min $\begin{cases} \ell_2/2 = 20' \\ \ell_1/2 = \underline{15'} \leftarrow \text{governs} \end{cases}$

$\beta_t = 0 \quad \alpha_1 = 0$

END SPAN		INT SPAN	
exterior negative	$0.26 = -381.7^{1k}$	negative	$0.65 = -954.2^{1k}$
positive	$0.5 = 734^{1k}$	positive	$0.35 = 513.8^{1k}$
interior negative	$0.7 = -1027.6^{1k}$		

	COL. STRIP	MID. STRIP
-ext	$1.0 = -381.7^{1k}$	0
+ext	$0.6 = 440.4^{1k}$	293.6^{1k}
-int	$0.75 = -770.7^{1k}$	-256.9^{1k}
-int	$0.75 = -715.7^{1k}$	-238.6^{1k}
+int	$0.6 = 308.3^{1k}$	205.5^{1k}

LONGITUDINAL REINFORCEMENT DESIGN:

in EXCEL: $d_{short} = 15.5 - 0.75 - \frac{1}{2}(0.75)$
$= 14.3125''$

$\rho_{required}$ calculated from the quadratic:

$$R = \rho f_y \left(1 - 0.59 \rho \frac{f_y}{f'_c}\right)$$

CHECK STRENGTH REQUIREMENTS:

• FLEXURE: $d \geq \sqrt{\dfrac{M_u}{\phi b f_y \rho_{max}\left(1 - 0.59 \rho_{max} \frac{f_y}{f'_c}\right)}}$

$\geq \sqrt{\dfrac{31.38 \cdot 12000}{0.9(12)(60)(0.0243)\left[1 - 0.59\left(0.0243 \cdot \frac{60}{5}\right)\right]}}$

$15.5'' \geq 6.88''$ $\underline{\underline{OK}}$

• SHEAR:

WIDE-BM.: $V_u = 0.3745\,(1')\left(\frac{40}{2} - \frac{24}{12 \cdot 2} - \frac{14.3125}{12}\right)$
$= 1.76^k$

Figure 25: Second page of two-way flat plate hand calculations

NY TIMES

$V_c = 2\sqrt{f'_c}\,bd = 2\sqrt{\frac{5000}{1000}}(12)(14.3125)$ * controlling equation

$= 24.3^k$

$\phi V_c = 0.75(24.3) = 18.2^k > 1.76^k$ OK

PUNCHING: $V_u = 0.3745\left(30 \cdot 40 - \left(\frac{24+14.3125}{12}\right)^2\right) = 435.2^k$

$V_c = 4\sqrt{f'_c}\,bd = 4\sqrt{\frac{5000}{1000}}(4(24+14.3125))(14.3125)$

$= 620.4^k$

$\phi V_c = 0.75(620.4) = 465.3^k > 435.2^k$ OK

FRAME B: $M_o = \frac{w_u l_2 l_n^2}{8} = \frac{0.3745(30)(38)^2}{8}$

$= 2028^{'k}$

* DISTRIBUTE M_o:

END SPAN		INT. SPAN	
− ext	0.26 = −527.3'k	− int	0.65 = −1318'k
+ ext	0.5 = 1014'k	+ int	0.35 = 709.8'k
− int	0.7 = −1420'k		

	COL. STRIP	MID. STRIP
− ext	1.0 = −527.3'k	0
+ ext	0.6 = 608.4'k	405.6'k
− int	0.75 = −1065'k	−355'k
− int	0.75 = −988.5'k	−329.5'k
+ int	0.6 = 425.9'k	283.9'k

LONGITUDINAL REINFORCEMENT DESIGN:

In EXCEL: $d_{long} = 15.5 - 0.75 - 0.75$

$= 13.94''$

CHECK STRENGTH REQUIREMENTS:

• FLEXURE: $d \geq \sqrt{\frac{71.00(12000)}{0.9(12)(60)(0.0243)\left[1-0.59(0.0243)\frac{60}{5}\right]}}$

$15.5'' \geq 8.08''$ OK

Figure 26: Third page of two-way flat plate hand calculations

Figure 25: Second page of two-way flat plate hand calculations

NY TIMES

• SHEAR:

WIDE-BM.: $V_u = 0.3745(1)\left(\frac{30}{2} - \frac{24}{12 \cdot 2} - \frac{13.94''}{12}\right)$

$= 4.81^K$

$\phi V_c = 0.75(2)\sqrt{5000}\,/\,1000\,(12)(13.94'')$

$= 17.74^k > 4.81^k$ OK

PUNCHING: $V_u = 0.3745\left[30.40 - \left(\frac{24+13.94}{12}\right)(12)\right]$

$= 435.2^K$

$\phi V_c = 0.75(4)\sqrt{5000}\,/\,1000\,(4(24+13.94))(13.94)$

$= 448.8^K > 435.2^K$ OK

* TYPICAL REINFORCEMENT DETAILS ON NEXT PAGES.

Figure 27: Fourth page of two-way flat plate hand calculations

		FRAME A									
		column strip = 15'					middle strip = 25'				
		exterior			interior		exterior			interior	
item	description	M_{ext}^-	M^+	M_{int}^-	M^-	M^+	M_{ext}^-	M^+	M_{int}^-	M^-	M^+
1	M_u	-381.7	440.4	-770.7	-715.7	308.3	0	293.6	-256.9	-238.6	205.5
2	b	180	180	180	180	180	300	300	300	300	300
3	d	14.31	14.31	14.31	14.31	14.31	14.31	14.31	14.31	14.31	14.31
4	M_u*12/b	-25.4	29.4	-51.4	-47.7	20.6	0	11.7	-10.3	-9.54	8.22
5	$M_n=M_u/\Phi$	-424.1	489.3	-856.3	-795.2	342.6	0	326.2	-285.4	-265.1	228.3
6	$R=M_u/(bd^2)$	-124.2	143.3	-250.8	-232.9	100.3	0	57.3	-50.2	-46.6	40.1
7	$\rho_{req'd}$	0.00210	0.00244	0.00432	0.00400	0.00170	0	0.000962	0.000841	0.000781	0.000672
8	$A_{s,req'd}$	5.41	6.29	11.1	10.3	4.38	0	4.13	3.61	3.35	2.89
9	$A_{s,min}$	5.02	5.02	5.02	5.02	5.02	8.37	8.37	8.37	8.37	8.37
10	$N=A_s/0.6$	9.02	10.5	18.5	17.2	8.37	14.0	14.0	14.0	14.0	14.0
11	N_{min}	5.81	5.81	5.81	5.81	5.81	9.68	9.68	9.68	9.68	9.68
12	$N_{prov,\ \#7}$	10	11	19	18	9	14	14	14	14	14

Table 10: Frame A moment & reinforcement distribution

		FRAME B									
		column strip = 15'					middle strip = 15'				
		exterior			interior		exterior			interior	
item	description	M_{ext}^-	M^+	M_{int}^-	M^-	M^+	M_{ext}^-	M^+	M_{int}^-	M^-	M^+
1	M_u	-527.3	608.4	-1065	-988.5	425.9	0	405.6	-355	-329.5	283.9
2	b	180	180	180	180	180	180	180	180	180	180
3	d	13.94	13.94	13.94	13.94	13.94	13.94	13.94	13.94	13.94	13.94
4	M_u*12/b	-35.15	40.56	-71.00	-65.90	28.39	0	27.04	-23.67	-21.97	18.93
5	$M_n=M_u/\Phi$	-585.9	676.0	-1,183	-1,098	473.2	0	450.7	-394.4	-366.1	315.4
6	$R=M_u/(bd^2)$	-181.0	208.8	-365.5	-339.2	146.2	0	139.2	-121.8	-113.1	97.43
7	$\rho_{req'd}$	0.002954	0.00340	0.00638	0.00590	0.00248	0	0.00236	0.00206	0.00191	0.00164
8	$A_{s,req'd}$	7.41	8.53	16.0	14.8	6.22	0.00	5.92	5.17	4.79	4.11
9	$A_{s,min}$	5.02	5.02	5.02	5.02	5.02	5.02	5.02	5.02	5.02	5.02
10	$N=A_s/0.6$	12.4	14.2	26.7	24.7	10.4	8.37	9.87	8.61	8.37	8.37
11	N_{min}	5.81	5.81	5.81	5.81	5.81	5.81	5.81	5.81	5.81	5.81
12	$N_{prov,\ \#7}$	13	15	27	25	11	9	10	9	9	9

Table 11: Frame B moment & reinforcement distribution

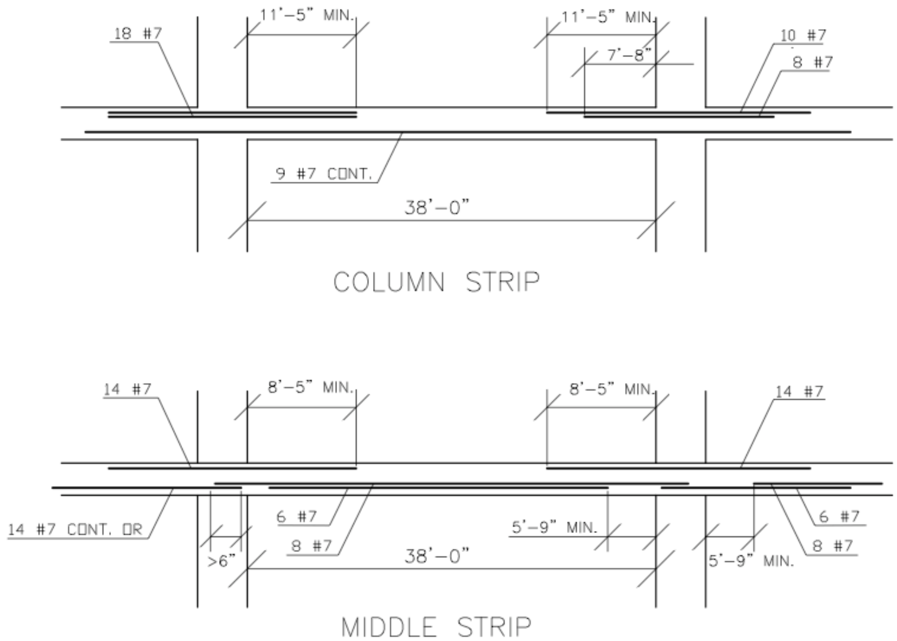


Figure 28: Frame A reinforcement details

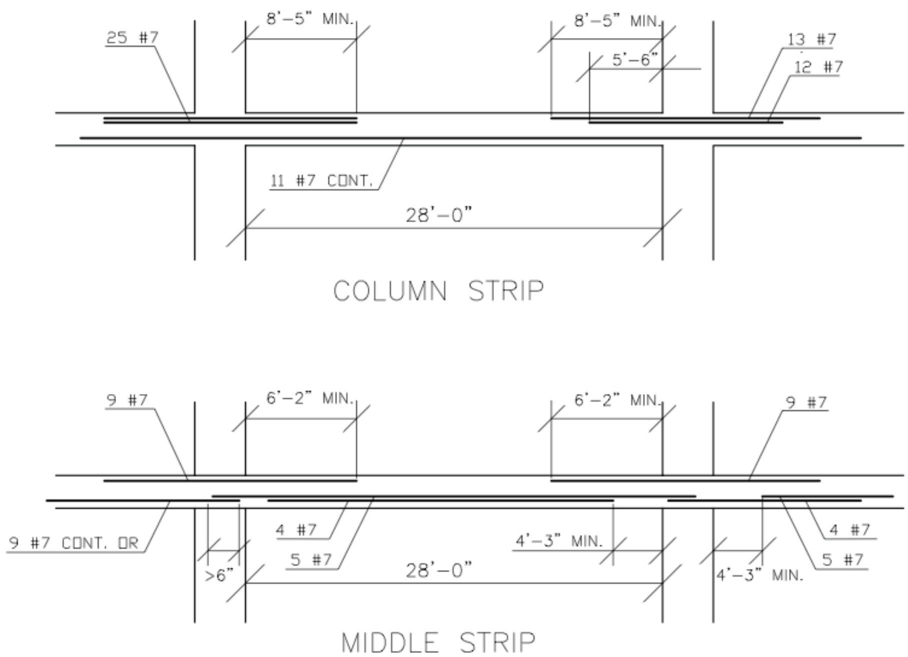


Figure 29: Frame B reinforcement details

APPENDIX E:
MISCELLANEOUS FIGURES & TABLES

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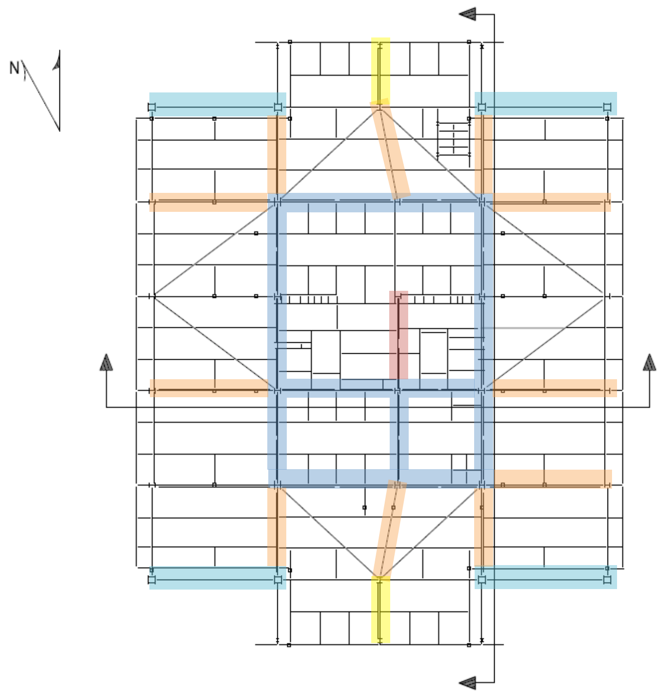

Key:
Single Diagonal Bracing
Pre-Tensioned Steel Rod X-Bracing
Chevron Bracing
Outrigger Bracing
Single Diagonal Brace at Cantilever

Figure 30: Mechanical Floor Framing Plan (Floors 28 & 51)

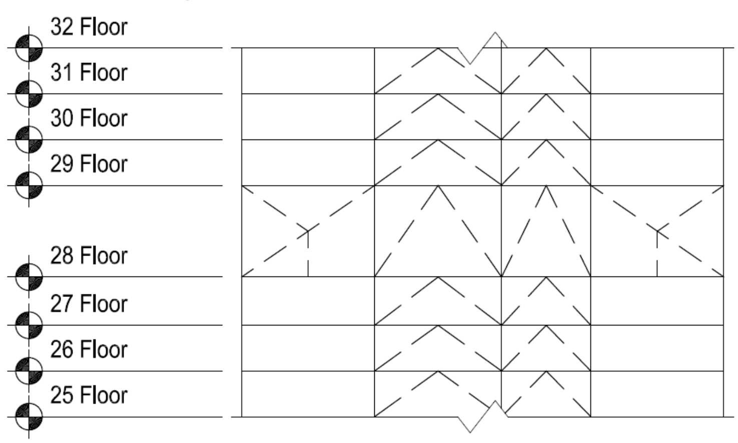

Figure 31: Typical E/W Outrigger Section (28th Floor)

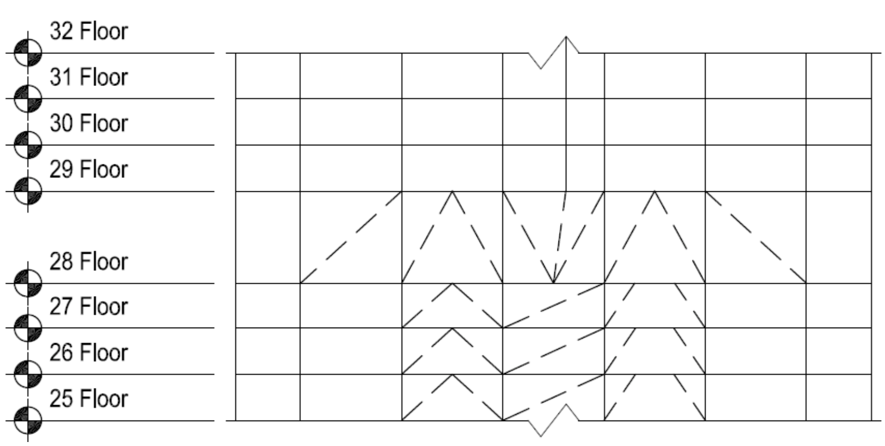

Figure 32: Typical N/S Outrigger Section (28th Floor)