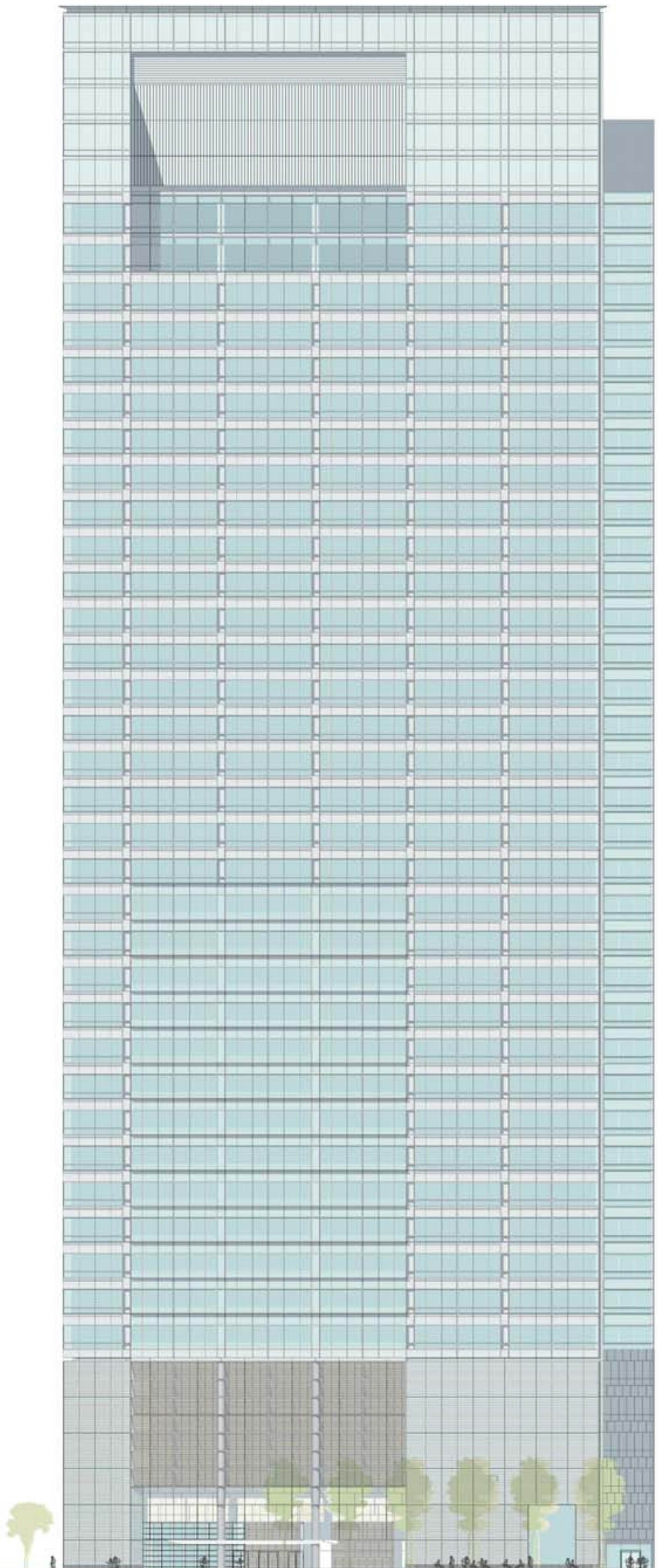


ONE SOUTH DEARBORN

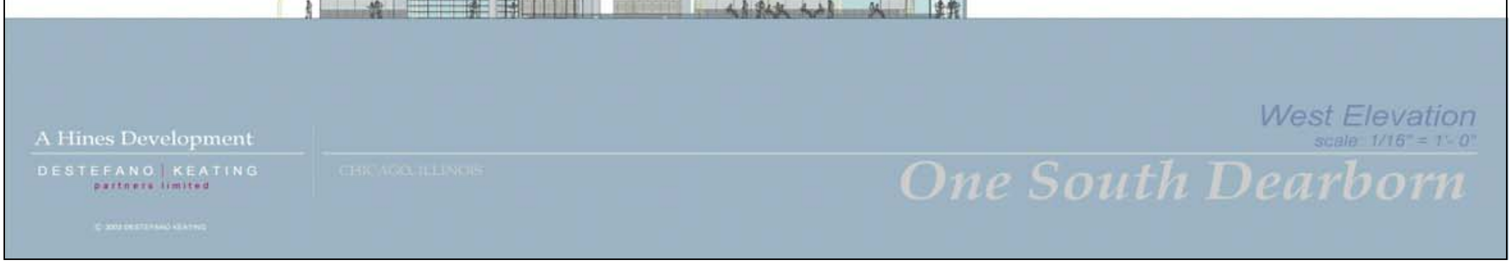
CHICAGO
ILLINOIS

JOSEPH R. BLASKO

ARCHITECTURAL ENGINEERING

STRUCTURAL EMPHASIS

SENIOR THESIS
2005



One South Dearborn, Chicago IL

JOSEPH R. BLASKO

STRUCTURAL OPTION

Project Overview

- **Owner**: Hines Interests Limited Partnership
- **General Contractor**: Turner Construction Company
- **Architect**: DeStefano Keating Partners Limited
- **Structural Engineer**: Halvorson Kaye
- **MEP Engineer**: Alvine & Associates, INC.
- **Total Project Cost**: $99.82 Million - 97.55 $/SF
- **Size**: 40 Stories — 1,023,294 GSF
- **Construction Started**: November 01, 2003
- **Substantially Complete**: August 31, 2005

Structural System

- **Superstructure**: Steel floor framing and steel perimeter columns using 50 ksi steel
- **Lateral System**: Reinforced concrete core using 5,000 to 8,000 psi concrete and is approximately 50' by 60'
- **Envelope**: Custom all glass and metal panel unitized interior set curtain wall supported from slab edge
- **Foundation**: Grade beam system supported by a combination of belled and rock caissons
- **Feature**: Most of the ground floor is slab on grade, but elevator pits and a meter room will extend below grade as a partial basement.

Mechanical System

- **Equipment** : Two 1,500 ton water cooled centrifugal chillers located on the 7th floor, four 900 ton cooling towers located on the roof, four 162,000 cfm custom air handlers (two located on the 7th floor serving floors 9-22 & two located on the 39th floor serving floors 23-38), each floor has a medium pressure duct loop and four fan powered terminal units (FPTU) with electric reheat coils. A 25,000 cfm AHU in the lobby and five other AHU ranging from 920 — 7,150 cfm through a zoned VAV duct system.
- **Fire Protection** : Fully sprinklered building with standpipe risers and two fire pumps (high zone and low zone)

Electrical/Lighting System

- **Primary**: 480 HVAC Panel, 3 Phase, 4 Wire
- **Secondary**: 277V HVAC Panel, 3 Phase, 4 Wire
- **Primary**: 208 Remaining Load , 3 Phase, 4 Wire
- **Secondary**: 120V Remaining Load, 3 Phase, 4 Wire
- **Emergency Back-up**: 1250 KW Diesel Generator
- **General Lighting**: Primarily Indirect/Direct Fluorescent
- **Architectural Lighting**: Primarily Strip Fluorescent & Metal Halide Fixtures — LED strip at edges of each cast glass panel

PENNSTATE

1855

HTTP://WWW.ARCHE.PSU.EDU/THESIS/2005/JRB914/

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EXECUTIVE SUMMARY

This report contains the results on the structural analysis and redesign of One South Dearborn in Chicago, Illinois. It evaluates the structural system of the building, code and specification requirements and calculations from snow, dead, wind and seismic loads.

One South Dearborn is a forty story, one million square foot commercial office tower. This high-rise is a composite design; the core of the building is constructed of reinforced concrete shear walls with perimeter steel floor framing and columns. This type of design is limited by the strength and torsional stiffness of the wall for large lateral loads. Also in a composite design, there is differential shortening where the steel members are erected out-of-level with the concrete core. These are some of the possible problems that can be encountered with a composite structure.

This report analyzes alternative structural systems to determine if the use of a composite structure was appropriate. One possible solution examines an alterative to the existing lateral system, which makes the building entirely of steel. The proposed design incorporates a steel braced core with outriggers in the mechanical room of the penthouse. Another possible solution examines an alterative to the existing floor framing system, which makes the building entirely of concrete. The proposed floor system uses post-tensioned concrete.

By redesigning the structural systems of the building, adjustments of cost, construction schedule and construction methods of One South Dearborn were examined. Also as a result of altering the existing structural system, mechanical system issues took place. This report takes a look at thermal energy storage which can lower building operating costs and the capital cost of cooling and heating equipment as smaller devices can be installed.

After designing One South Dearborn as an all-steel and all-concrete structure, I have come to the conclusion that the existing composite structure is a better solution. Even though the all-steel structure had a lower expenditure and shorter construction phase, it didn't function as well as the composite structure.

This redesign is a case study and should not be treated as a professional design. It is intended to study the use of different structural systems, and observe the system's practicality. Any questions pertaining to this report may be directed to jrb914@psu.edu.

INTRODUCTION & BACKGROUND

On November 01, 2003, Hines broke ground on One South Dearborn, a new office tower in downtown Chicago, designed by DeStefano Keating Partners Ltd. One South Dearborn shares the site with the Inland Steel building on the Southeast corner of Madison and Dearborn streets in downtown Chicago.

Location and Site

Street : 1 South Dearborn Street
City: Chicago
State: Illinois
Zipcode: 60603
District: Downtown
Neighborhood: Loop

History of the Site

This site was once home to the Tribune Building which was built in 1901. In 1958, an addition was added to the original Tribune Building and the named was changed to 19 South Dearborn. Five years ago, the building was demolished for the proposed 7 South Dearborn building, which would have been one of the tallest buildings in the world. However, that deal fell through and the site sat vacant leaving the site with existing caissons. These existing conditions of the site may require modifications to the foundation design.

Project Team

Owner / Developer:	Hines Interests Limited Partnership
Financing:	Bank One Corp. [JP Morgan Chase & Co.]
Architect:	Rick Keating w/ De Stefano + Partners
Landscape Architect:	Daniel Weinbach and Partners, Ltd.
General Contractor / CM:	Turner Construction Company
Engineers:	Halvorson Kaye Structural Engineers
	Alvine & Associates (MEP)
	McClier (Civil)
	STS Consultants, Ltd. (Geotechnical)
Consultants:	Cerami & Associates (Acoustics)
	Persohn / Hahn Associates (Elevator)
	Curtain Wall Design and Consulting, Inc.
Suppliers:	The Prairie Group (Concrete)
	Cives Steel Company
	ThyssenKrupp Elevator U.S.A.
Excavation:	Brandenburg Industrial Service Co.
Foundation:	Case Foundation Company

Construction Details

One South Dearborn was developed through a Design-Bid-Build project delivery method. The overall project cost of the one million square foot office tower is $100 Million. The project is being constructed under contract by a Hines custom agreement. It has actual and consequential damages capped at the dollar amount. The construction started on November 1, 2003 and through the course of communication with the project team it was projected that the building will be substantially complete on August 31, 2005. In November 2005, One South Dearborn will be fully turned over to Hines Interests Limited Partnership.

Building Function

One South Dearborn is a forty-story modern high-rise commercial office tower. The one million square foot building is broken into two main areas. The office space of the building occupies 820,000 square feet while the parking garage and mechanical rooms occupy the remaining 180,000 square feet. The primary tenant is the Chicago-based law firm of Sidley Austin Brown & Wood LLP and this law firm will occupy 500,000 square feet of office space. Other amenities include an on-site fitness center, conference facilities, 8,000 square-feet of retail space and four floors of above-grade covered parking that will accommodate 160 vehicles.

Major National Codes

The building is being built around the City of Chicago Building Code.

Zoning

All zoning requirements are typical of commercial zoning in the Chicago downtown area. However, Chicago's City Council has approved a new zoning code and the ordinance went into effect on November 1, 2004. To preview the revised zoning code, you can visit the following web address: http://www.cityofchicago.org/Mayor/Zoning/.

Building Envelope

The project consists of a back lit stone clad precast for the first six floors, an etched sandblast pattern at the base of the building, and the remaining skin of the building is a custom curtain wall. The components of the system include composite panels, painted aluminum spandrel glass with custom frit, and clear glass with an energy efficient, low-e coating.

Fire Protection

One South Dearborn is a fully sprinklered building. There are two fire pumps located in the lower level. One pump serves the "high zone" and the other the "low zone". There are also two standpipes that run vertically up the stairwells. The fire alarm system includes smoke detectors, heat detectors, visual and audio devices, stairwell door release devices, elevator override, firefighter communication system and flow and tamper devices on the sprinkler systems.

Telecommunications

There are two underground incoming services from the street to the building. The incoming services go to a “net pop / telecommunication” room on the 2nd floor, room 211. From there a raceway system of conduits feed two telecommunication / data closets on each typical floor, which the tenants will tie into. The security system includes card readers on some doors, cameras in elevators and alleys.

Vertical Transportation

Vertical transportation consists of elevators and stairs. There are nineteen elevators in the building. Two elevators serve the parking garage (floors ground through 6th floor). There are seven "low rise" passenger cars that serve the ground and 7 through 23rd floors, eight "high rise" passenger cars that serve the ground and 23 through 38th floors, and two service cars that serve ground through roof. All elevators in the building are traction elevators.

Elevator	Capacity	Speed	H.P.	Heat Release
Low rise	4000 lbs	800 fpm	57	30,000
High Rise	4000 lbs	1200 fpm	85	34,200
Service	4500 lbs	500 fpm	44	25,600
Garage	3000 lbs	350 fpm	30	15,400

EXISTING CONDITIONS

Existing Architectural

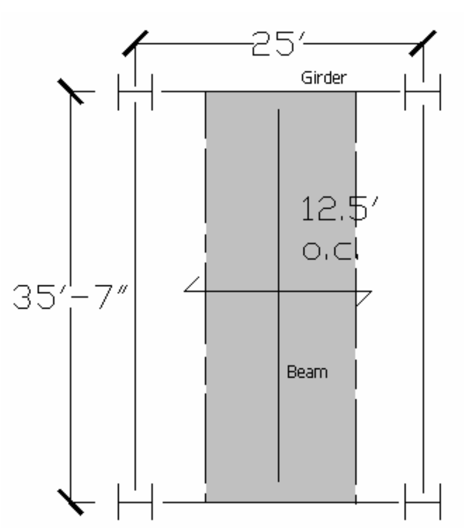
This forty-story, one million square-foot office tower, features a structural steel frame, reinforced concrete core and glass curtain wall cladding. One South Dearborn shares the site with the Inland Steel building on the Southeast corner of Madison and Dearborn streets. It is located to the rear of the site to provide a new 18,000 square foot granite-paved civic plaza and to create a relationship to this space for both buildings. The plaza is graced by parallel rows of trees that will be installed in natural soil at 40 feet in height.

The highest point of the building is 571 feet above grade. At the top of the building is a crown that is made of an extension of the curtain wall and has a rectangular indentation over the outside terrace. Each of these elements will manipulate the light both in the day and at night to create a visual conclusion to this tower. The base of One South Dearborn will include a three-story, 5,000 square-foot lobby finished in granite, marble, textured glass and stainless steel, with floor-to-ceiling windows for clear views of Madison and Dearborn streets. Floors three through six will contain covered parking and the remaining levels will hold offices. An interesting feature about One South Dearborn is that it has a very limited underground basement. Only the core descends significantly below grade, because the elevator pits have to extend that far.

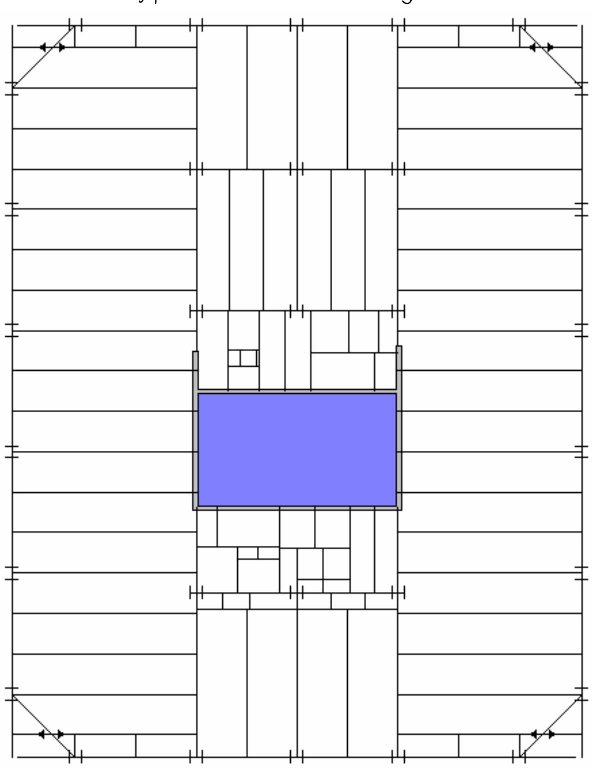
Existing Structural

Floor System

The floors are column-free 42-foot spans, with extra-wide stairways to enhance life safety, nine-foot high windows, 30-foot structural spans and dual risers for redundant wiring. The floor framing is rolled structural steel shapes that act compositely with the floor slab. The floor slab itself is a composite steel deck with lightweight concrete topping at the typical office levels. On parking floors, steel framing will be made composite with normal weight formed concrete slab with epoxy reinforcing. Typical bay size is 25' x 35'-7" with intermediate beams spaced at 12.5' o.c. The composite beam has a total slab depth of 6.25" and deck depth is 3".



Typical Floor Framing Plan



- Grey – Shear walls
- Blue – Elevators
- Beam moment connections
- Each corner of the floor is cantilevered

Columns

Columns are rolled structural shapes. Where increased capacities are required, rolled shapes will be reinforced with structural steel plates at flange tips, parallel to their web.

Roof

The roof is a composite steel deck slab consisting of two placements of normal weight concrete with slope topping. The concrete depth on the roof is six inches with a three inch deck. Also the roof is composed of an American hydrotech mop down roofing membrane with pre-cast pavers.

Foundation

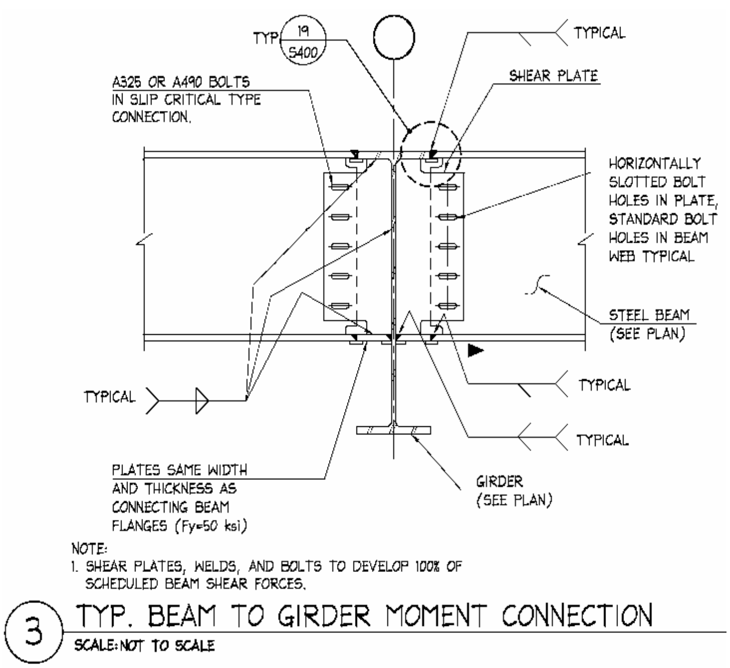
The tower is supported on belled and rock caissons, linked by grade beams as required. Most of the ground floor is on an 8" slab on grade, but the elevator pits and a meter room will extend below grade as a partial basement. The new caissons have been placed to avoid existing caissons on the site. However, alternates have been provided if other existing caissons are encountered during construction.

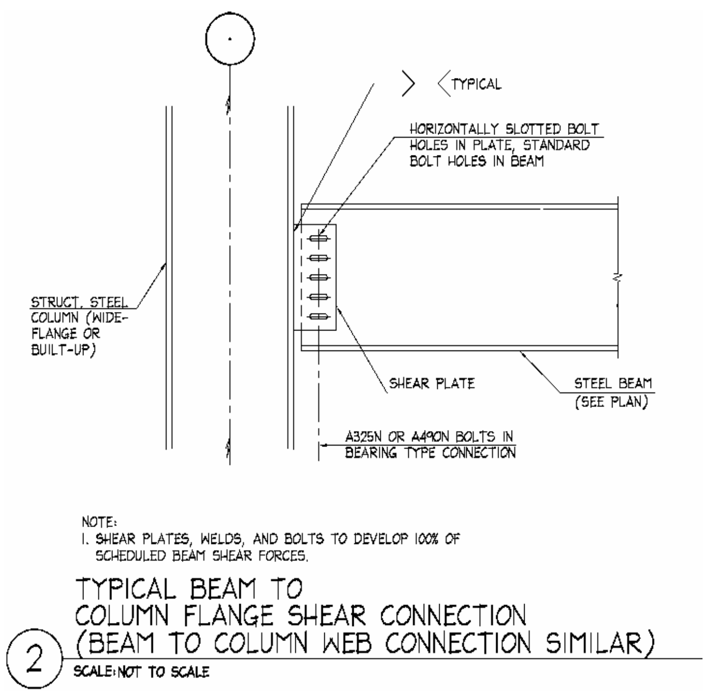
Enclosure

The structure's primary cladding, glass curtain wall, will be supported from slab edge. Pre-cast cladding at the lower levels will be supported on perimeter beams immediately adjacent to the column. The allowable deflection will be held to 0.625" for superimposed loads. The components of the system include composite panels, painted aluminum spandrel glass with custom frit, and clear glass with an energy efficient, low-e coating.

Connections

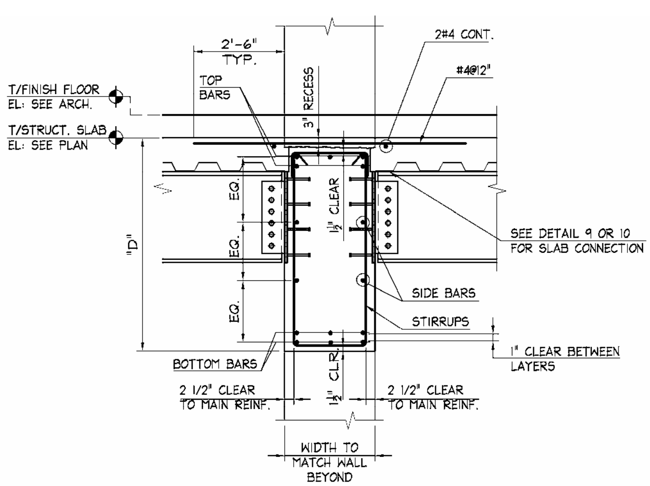
Most connections are simple shear connections utilizing short slotted holes and high-strength bolts in bearing type connections with threads included in the shear plane connections requiring friction type bolts. Some beam-to-column and beam-to-beam connections are designed with moment connections, mainly on the catwalk framing above the roof.



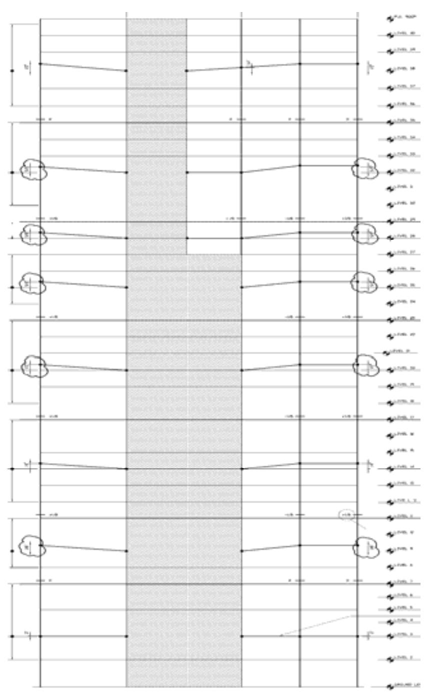


Lateral System

The tower is a composite design – a reinforced concrete core with steel floor framing and steel perimeter columns. The core is approximately 50' by 60' and provides all the lateral strength and stiffness for the tower, in addition to resisting gravity loads. All lateral loads on the tower are resisted by the central reinforced concrete shear walls that extend from the foundations to the roof level. The interior core walls are a constant 12" thickness, while the exterior core walls vary in thickness above grade from 22" to 18". Segments of the wall are linked together with "link beams," which can be seen in the details below. The beams are the same thickness as the wall and are reinforced as conventional beams. The structural steel floor framing and slabs between the shear walls and steel columns are erected out-of-level. As differential column/wall shortening occurs over time, the floor will tend to become more level. Below is a schematic section for differential shortening compensations.



2 TYPICAL LINK BEAM SECTION
SCALE: NOT TO SCALE



Structural Design Codes

Chicago Building Code (New Draft Code)

Structural Design Specifications and Standards

Structural Concrete Design
American Concrete Institute, Building Code Requirements for Structural Concrete
ACI 318-83 [Also refer to 1999 edition]

Structural Steel Design
American Institute of Steel Construction, Load and Resistance Factor Design
Specififcation for Structural Steel Buildings, Second Edition, 1994
[Also refer to 2001 edition]

Welding
American Welding Society, Structural Welding Code – Steel
(AWS D1. 1-92)

Steel Deck Slabs
Specification for the Design of Cold Formed Steel Structural Members
(AISI 1968 edition, as modified by Addendum No. 1 - dated Nov. 19, 1970)

Reinforced Masonry Design
American Concrete Institute, Building Code Requirements for Masonry Structures
(ACI 530-92 / ASCE 5-92 / TMS 402-92) [Also refer to 1999 edition]

Project Material Strengths

Concrete

Cast-In Place – Normal weight concrete** with a 28-day strength of:

Belled Caissons	8,000psi
Gradebeams	6,000psi
Caisson Caps	6,000psi
Shear walls	5,000 - 8,000psi
Link Beams	5,000 - 8,000psi
Parking slab floor	5,000psi
Misc. Foundation Walls	5,000psi
Slabs on Grade	4000psi

Steel Deck Slabs – Lightweight concrete** with a 28-day strength of:

Typical slab floors	4,000psi

Note: Air entrain to all exposed concrete
**Normal weight concrete – 145 PCF
**Lightweight concrete – 115 PCF

<u>Concrete Reinforcement</u>

ASTM A615, Grade 60, reinforcing bars.
Epoxy coat reinforcing bars to ASTM A775 at exterior exposed and garage areas.
ASTM A185 welded wire fabric.
Epoxy coat welded wire fabric to ASTM A884 at exterior exposed and garage areas.

<u>Metal Deck</u>

Mill coated steel sheet conforming to ASTM A653-94, Structural Quality
Galvanized to G-90 for decks at all ground floor and roof areas
Galvanized to G-60 for floor decks
Minimum Fy = 33 KSI

<u>Structural Steel</u>

Wide Flanges, WT's	ASTM A992 (Fy = 50 KSI)
Channels	ASTM A36 (Fy = 36 KSI)
Other Rolled Shapes	ASTM A36 (Fy = 36 KSI)
Continuity Plates	ASTM A992
Misc. Plates	ASTM A36, unless noted otherwise
Column Base Plates	ASTM A36, typical (ASTM A992, where noted)
Connection Materials*	ASTM A36 or ASTM A992, Grade 50
Rectangular Tubes (HSS)	ASTM A500, Grade B (Fy = 46 KSI)
Round Tubes (HSS)	ASTM A500, Grade C (Fy = 46 KSI)
Round Pipes	ASTM A53, Type S, Grade B (Fy = 35 KSI)
Anchor Bolts	ASTM A36, typical; (ASTM A992, where noted)

*Sized by the Structural Steel Fabricator

Shear Studs

0.75" diameter headed shear studs per ASTM A108
(Lengths vary with slab thickness)

Welding

AWS E70XX electrodes for shop welding
AWS E7018 electrodes for field welding

Bolts

ASTM A325 typical, ASTM A490 where noted
¾" diameter minimum

Structural Steel Fireproofing

Spray-on cementitious fireproofing on all steel in the field

Project Design Gravity Loads

SDL = Superimposed Dead Load (Dead Loads in excess of structural self weight)

LL = Live Load, reduced per Chicago Code for floor framing, walls, columns & foundations

Parking

MEP/Ceilings/etc.	5
SDL =	5
LL =	50
Total Superimposed Load	55 PSF

Typical Office

MEP/Ceilings/etc.	5
SDL =	5
LL =	100 (includes 20 psf for partitions)
Total Superimposed Load	105 PSF

Level 1 Lobby

Floor Finishes	35
MEP/Ceilings/etc.	10
SDL =	45
LL =	100
Total Superimposed Load	145 PSF

Roof

Roofing	15	
Slope to drain	35	
MEP/Ceilings/etc.	20	
DL =	70	
LL =	25	+ Drifts
Total =	95 PSF	+ Drifts

Typical Core – Electrical / Telecom Rooms

MEP/Ceilings/etc.	5
DL =	5
LL =	150
Total =	155 PSF

Typical Core – Active Elevator Lobbies

MEP/Ceilings/etc.	5
DL =	5
LL =	100
Total =	105 PSF

"Light" Mechanical Plant

CMU Partitions	linear loads, as required
Housekeeping pads	40, as required
MEP/Ceilings/etc.	5
SDL =	85
LL =	150
Total Superimposed Load	235 PSF

“Heavy” Mechanical Plant

CMU Partitions	linear loads, as required
Housekeeping pads	40, as required
MEP/Ceilings/etc.	5
SDL =	85
LL =	250
Total Superimposed Load	335 PSF

Other Design Criteria

Exterior Cladding

Gravity loads:

Glass walls	20 PSF
Solid walls	90 PSF

Spandrel beam deflections:

Less than 5/8 inch for (SDL + LL)

Snow Loads

Snow loads were determined by using ASCE7-02.

Ground Snow Load – $p_g = 30$ PSF (Figure 7-1)

Flat Roof Snow Load – $p_f = 0.7C_eC_tIp_g = 21$ PSF

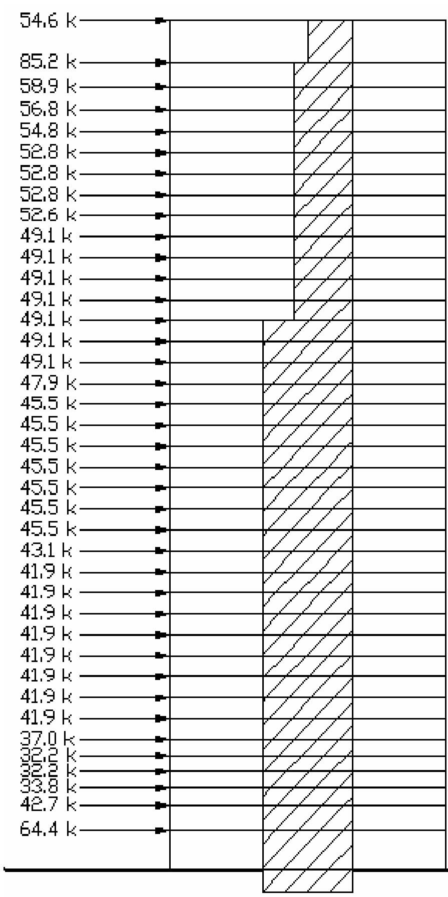
C_e	–	Exposure Factor	–	1.0
C_t	–	Thermal Factor	–	1.0
I	–	Importance Factor	–	1.0

Note: For Total Floor Dead Loads – See Appendix

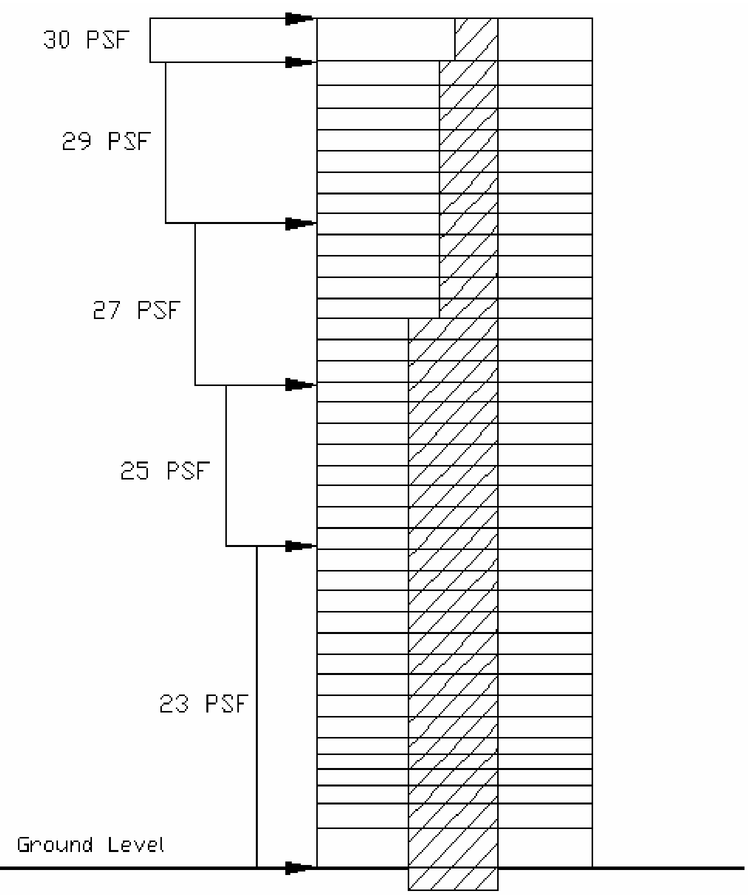
Wind Anaylsis

The project design wind loads were specified by the Chicago Building Code (proposed revised Draft September 20, 2002). The design wind loads are also specified by Wind Tunnel Testing.

Height – Feet	Wind Loads for Frame Design
200 or less	23 PSF
200 to 300	25 PSF
300 to 400	27 PSF
400 to 500	29 PSF
500 to 600	30 PSF

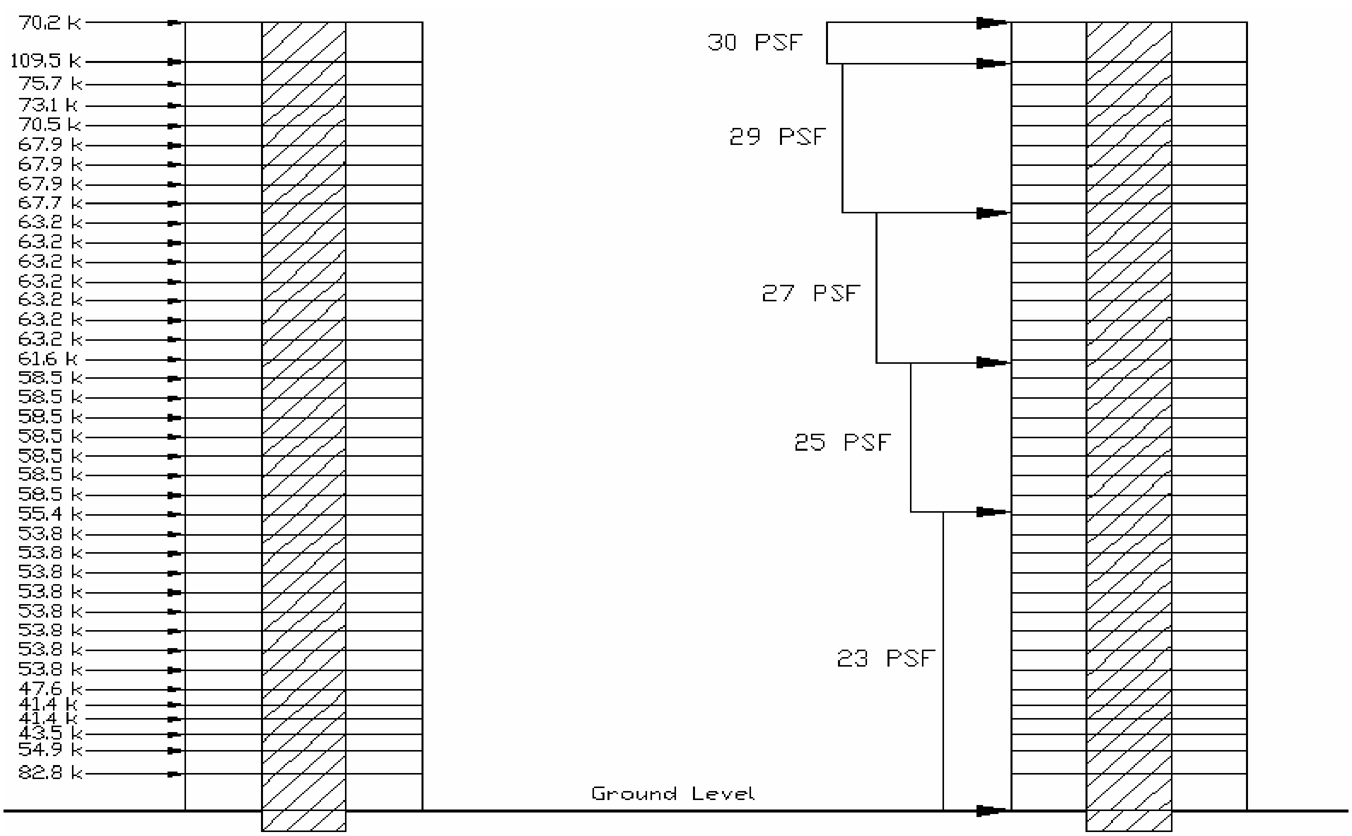


West Elevation
(Showing Shear Forces From Wind Loads on North Elevation)



West Elevation
(Showing Pressures From Wind Loads on North Elevation)

The diagram above displays the shear forces acting on each floor in the North-South direction. The diagram below shows the shear forces acting on each floor in the West-East direction. Also both diagrams show the pressure distribution over the height of the building. Refer to the wind load spreadsheets in the appendix for the calculation of shear forces due to wind loads. The Wind Tunnel Test and the Chicago Code cover both leeward and windward pressures in their design. Also in the appendix is a comparison of design load values between the Wind Tunnel Test, The Chicago Code(2001) and The Chicago Code(2002). That is follwed by graphical charts displaying the differences amongst the three.



South Elevation
(Showing Shear Forces From Wind Loads on West Elevation)

South Elevation
(Showing Pressures From Wind Loads on West Elevation)

Seismic Analysis

A Seismic Analysis was not performed in the design of One South Dearborn since it is not required by the City of Chicago Building Code. However, I have used ASCE7-02 to analyze the seismic shear on the structure.

Try Equivalent Lateral Force Analysis – Section 9.5.5.

- Category II – Table 1-1
- Seismic Use Group I – Table 9.1.3.
- Site Classification D – Table 9.4.1.2.
- $S_s = 0.17$ – Figure 9.4.1.1a.
- $S_1 = 0.06$ – Figure 9.4.1.1b.

Adjust For Site Class

- $Fa = 1.6$ – Table 9.4.1.2.4a. (Site Class D & $S_s \leq 0.25$)
- $Fv = 2.4$ – Table 9.4.1.2.4b. (Site Class D & $S_1 < 0.1$)
- $S_{ms} = F_aS_s = 1.6(0.17) = 0.272$
- $S_{m1} = F_vS_1 = 2.4(0.06) = 0.144$

Design Spectral Response Acceleration Parameters

- $S_{Ds} = 2/3S_{ms} = 2/3(0.272) = 0.181$
- $S_{D1} = 2/3S_{m1} = 2/3(0.144) = 0.096$

Seismic Design Category

- Seismic Design Catergory B – Table 9.4.2.1a. ($0.167 \leq S_{Ds} < 0.330$)
- Seismic Design Catergory B – Table 9.4.2.1b. ($0.067 \leq S_{D1} < 0.133$)

**Therefore Equivalent Lateral Load Method can be used.

Seismic Base Shear

- $W = 75{,}116^k$ – Appendix 4.7.
- R = 3 – Table 9.5.2.2. (Not Specifically Designed for Seismic Resistance)
- I = 1.0 – Table 9.1.4. (Seismic Use Group I)
- $T = C_th_n^x = 0.02(527\text{ ft})^{0.75} = 2.20$ s (N-S & E-W Direction)
- $C_s = S_{Ds}/(R/I) = 0.181/(3/1.0) = 0.0603$
- $C_{sMAX} = S_{D1}/[T(R/I)] = 0.096/[2.20(3/1.0)] = 0.0145$ ←Governs (N-S & E-W)
- $C_{sMIN} = 0.044IS_{Ds} = 0.044(1.0)(0.181) = 0.00796$
- $V = C_sW = 0.0145(75{,}116^k) = 1089.2^k$ (N-S & E-W Direction)

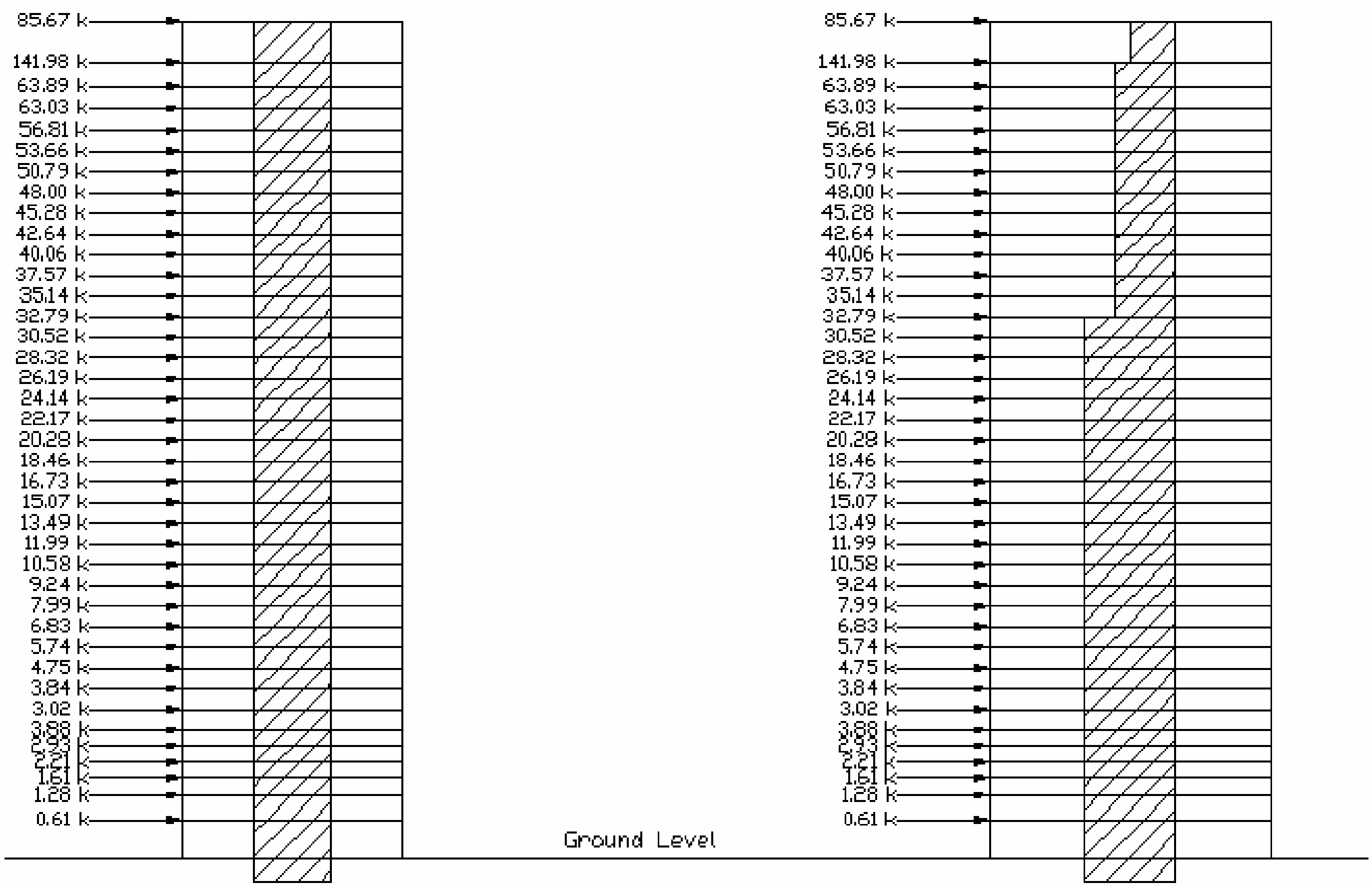
Vertical Distribution of Seismic Forces

$$C_{vx} = \frac{W_x h_x^{k}}{\sum_{i=x}^{n} W_i h_i^{k}} \quad \text{– Vertical Distribution Factor}$$

$$M_x = \sum_{i=x}^{n} f_i (h_i - h_x) \quad \text{– Overturning Moment}$$

$$K = 1 + \frac{2.20 - 0.5}{2} = 1.85$$

Refer to spreadsheet in appendix for the calculation of seismic loads. The diagram shows the distribution of seismic shear on each level in the N-S & W-E direction. To find the design shear from seismic loads at any given level, all point loads above and including that level must be summed. Therefore, the greatest shear occurs at the base. The base shear in this building is 1089.20^{k}.



South Elevation
(Showing Shear Forces From Seismic Loads on West Elevation)

West Elevation
(Showing Shear Forces From Seismic Loads on North Elevation)

Existing Mechanical

Two 1,500 ton water cooled centrifugal chillers located on the 7th floor, four 900 ton cooling towers located on the roof, four 162,000 cfm custom air handlers (two located on the 7th floor serving floors 9-22 & two located on the 39th floor serving floors 23-38), each floor has a medium pressure duct loop and four fan powered terminal units (FPTU) with electric reheat coils. A 25,000 cfm AHU in the lobby and five other AHU ranging from 920 - 7,150 cfm through a zoned VAV duct system.

Existing Electrical

Power for the building is taken from a 13 kV incoming service. There are two 2000 amp switchgears located on the 2nd floor, two 2000 amp switchgears located on the 8th floor and two 2000 amp switchgears located on the 39th floor. Switchgear feed transformers and panels on every floor for distribution and step-down. The voltage of the HVAC panel is 480/277V, 3 phase, 4 wire. The remaining load is then transformed down to 208/120V, 3 phase, 4 wire panelboards, where it is fed to branch circuits. There is also a 1250 kW emergency back up diesel generator located on the ground floor. The general lighting will primarily be indirect/direct fluorescent. The architectural lighting will be comprised of primary of strip fluorescent and metal halide fixtures. LED strips will be placed at the edges of each cast glass panel.

PROBLEM STATEMENT

Local history and material tendencies are most likely to be the influential factors in designing a building's structural system. As an example, in Chicago, almost every office building built in the past 20 years has been a steel frame with a concrete core. In other areas of the country not as used to designing and constructing steel around a concrete core, they tend to stick with an all-steel or an all-concrete structure based upon regional preferences. For example, an all-steel system tends to be more likely in the Northeast, while an all-concrete system tends to be more likely in the Southeast.

One South Dearborn is a composite design; it is a reinforced concrete core with steel floor framing and steel perimeter columns. The concrete shear walls are placed in the core of the building surrounding the elevator shaft and stairwells and jump-formed first. A simple steel frame is then attached to the concrete walls to complete the structure. Combining steel and concrete so that they act together structurally in composite elements can lead to very efficient frame solutions. The combination makes the best use of the materials and their respective benefits. Steel is used where long spans are desirable to make initial layouts more open and flexible for future modifications. Concrete is best used for its lateral drift control. Also a stiffer lateral system is attained because of the high level of damping due to composite behavior.

The design of this type of construction is limited by the strength and lateral and torsional stiffness of the wall for taller heights and large lateral loads. From a construction standpoint, the plumbness of the wall and the resulting tolerances for attachment of the steel framing members are the major challenges. If out-of-plumbness of the walls exceeds required tolerances, adjustments to steel members or chipping of the concrete add cost and time to the project. Also phasing of the concrete and steel erection and efficient utilization of the equipment affect the economics of the project. When the core can be started and substantially completed during the lead time required for structural steel, maximum benefits are achieved. If the site is congested and access to pickup points is limited, logistics become a problem. In a composite design you have to deal with differential shortening, so you have to erect the members out-of-level. This is another disadvantage of the composite design, because it increases the duration of the schedule due to its complexity. With such small tolerances, errors in construction may occur. The composite design also has costly connections between the steel and the concrete such as big embeds, field applied connection plates or angles, and slotted connections.

This structural system was chosen due to owner's preferences and location. But was this really the most economical choice when compared to other conventional systems? By comparing the existing system to alternative systems I will be able to determine if the use of a composite design was appropriate.

PROPOSAL

Proposed Solutions

Lateral Steel Braced Core and Outriggers

One possible solution will examine an alterative to the existing lateral system, which will make the building entirely of steel. The proposed design will incorporate a steel braced core with outriggers in the mechanical room of the penthouse. This will be compared to the existing cast-in-place concrete core and shear walls in order to examine the possibility of an alternate design solution. The flexibility of the proposed design of lateral system will have to be investigated. Also we will have to examine the placement of the bracing, so that it works for the elevator openings. Although, the connections of the steel system are still labor extensive and expensive. The steel system has a faster erection time, decreasing the duration of the schedule.

Post-Tension Concrete System

Another possible solution will examine an alterative to the existing floor framing system, which shall make the building entirely of concrete. The proposed floor system will use post-tensioned concrete. Post-tensioned concrete was chosen because of its rapid construction, economy, minimum story heights and optimum clear spans. This concrete system is also an efficient choice when issues

regarding reduced sound and vibration, as well as flexibility arise. For most multi-story buildings this is a suitable concrete framing system. For spans greater than 20 ft, post-tensioned slabs start to become cost-effective, and can be used alone or combined with reinforced concrete to provide a complementary range of casted-in place concrete floor options. The three main forms of construction are beam and slab, flat slab and ribbed slab. Next semester these different flooring selections will be analyzed and design decisions will be made to choose the appropriate type of post-tension slab. For office construction, flexibility is mostly concerned with changes in the internal space planning. In many cases adding openings do not substantially affect the structure. Core areas, primary services distribution and other major items usually remain fixed, although some additional holes for minor services may be required afterward. Regardless of construction type, forming large holes in any existing structure is not simple. In post-tensioned design, careful consideration is necessary before cutting out any openings.

Solution Method

Lateral Steel Braced Core and Outriggers

The design of all structural steel elements shall conform to the specifications of LRFD, 3rd Edition. A portion of the braced frame analysis may be more efficiently examined by manual methods or by the implementation of a computer program such as STAAD, RISA or RAM. Torsional effects on the overall building can be considered when using these types of programs. However, if the proposed lateral system is permitted to be situated in a symmetric layout, then torsional effects become negligible. Since the torsional effects become negligible, code states that the structure needs to account for five percent. For this method of analysis it is necessary to determine the stiffness for the uniform structure, bending and racking shear stiffness of the braced frame and outriggers in addition to a stiffness parameter representing the axial lengthening and shortening of the exterior columns. The analysis allows a simple procedure for obtaining the optimum location of the outrigger in the structure and a rapid assessment of the impact of the outrigger on the behavior of the high-rise structure. It is concluded that all the stiffnesses should be included in the preliminary analysis of a proposed high-rise building structure as the reductions in horizontal deflections and bending moments of the braced frame are influenced by all stiffness parameters.

Post-Tension Concrete System

Checks at both serviceability and ultimate limit state are carried when designing the overall structural performance. At the serviceability condition the concrete section is checked at all positions to ensure that both the compressive and tensile stresses lie within acceptable limits. Stresses are checked in the concrete section at the initial condition when the prestress is applied, and at serviceability conditions when calculations are made to determine the deflections for various load combinations. At the ultimate limit state the pre-compression in the section is ignored and checks are made to ensure that the section has sufficient moment capacity. Shear stresses are also checked at the ultimate limit state in a similar manner to that for reinforced concrete design, although the benefit of the prestress across the shear plane may be taken into account. Research will still have to be done early in the semester next spring to get caught up to speed with the post-tension concrete system. However, to carry out the checks above, ADAPT-PT, a computer program will be used or you could design the system using the ACI Handbook.

Breadth Proposal

The depth work of this thesis proposal is dealing with issues that will lead to the selection of an all concrete or all steel structure. The work in the areas of mechanical design and construction management aids to that selection process.

Mechanical

As a result of altering the structural systems, mechanical system issues will arise. Changes in the floor system will affect the plenum space. A modified mechanical system could be used to adjust to the new floor height. The proposed steel braced core and outrigger system may affect the mechanical penthouse on the roof and the placement of the chillers and air handlers in this location. In addition, the ductwork and air distribution may need to be rerouted throughout the building. Calculations will be performed to determine whether the current mechanical system is adequate to service the new structural systems. Also will look into adding thermal energy storage which will lower building operating costs and the capital cost of cooling and heating equipment as smaller devices can be installed. Adjustments to the mechanical system will be made as necessary.

Construction Management

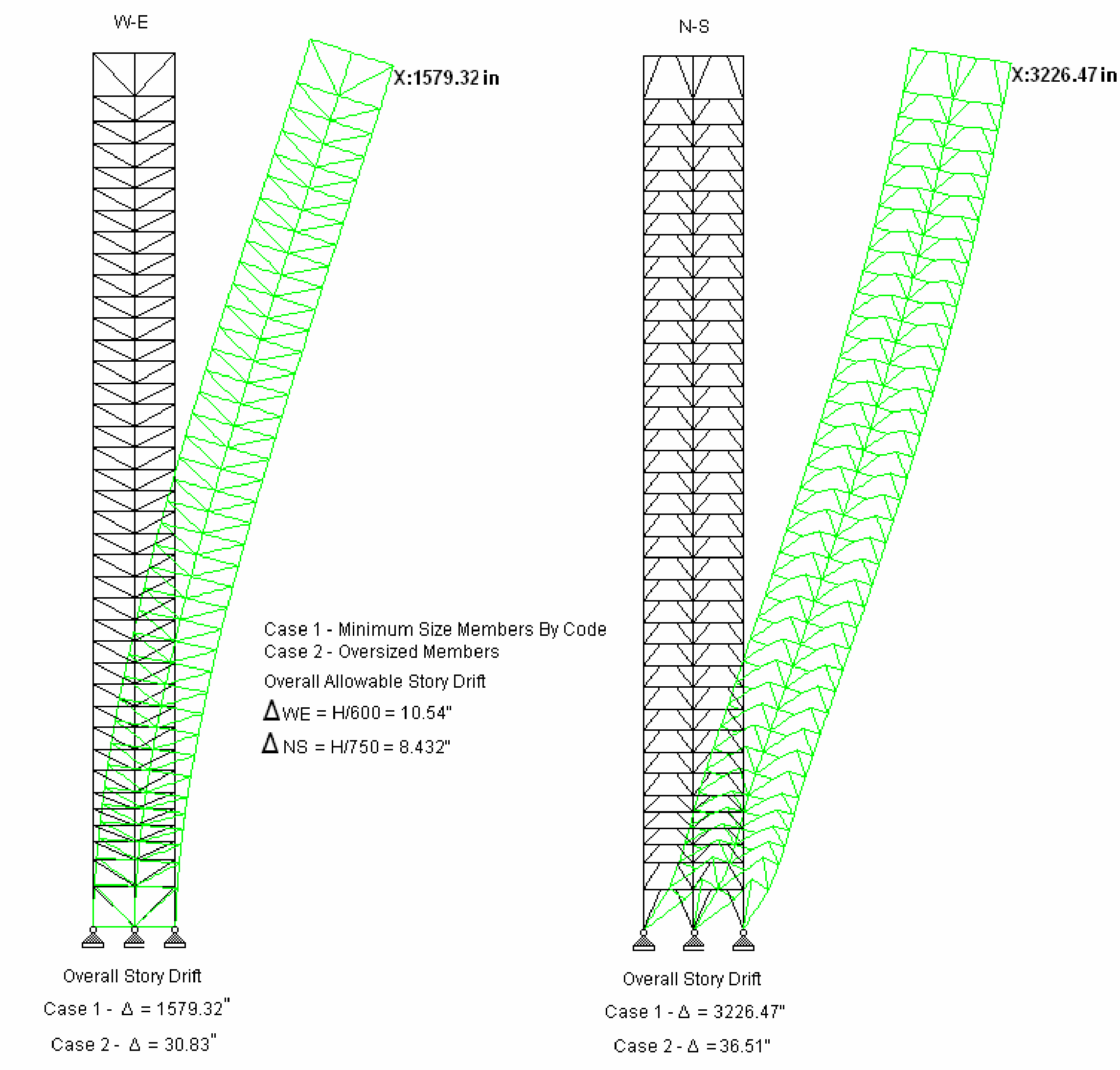
By redesigning the structural systems of the building, adjustments of cost, construction schedule and construction methods of One South Dearborn will have to be examined. So a study of each change in the depth work will be a crucial part in determining which solution is most economical and feasible. One analysis will compare the cost of using post-tension flooring system against the existing steel floor framing. Another comparison will be done between the reinforced concrete core and a proposed lateral system of a steel braced core and outriggers. In addition to looking at the costs of a post-tension concrete system and a steel braced framing system, schedules will be created for the two alternatives. These schedules will then be compared to the schedule used in constructing the existing structural system. This will include all structural and mechanical changes. With all the changes made to the structure, there will be additional construction issues such as cost, constructability and labor forces.

STRUCTURAL REDESIGN

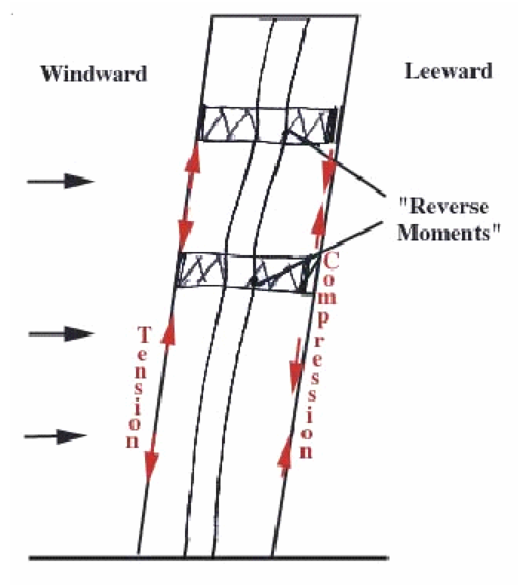
Steel Braced Core & Outriggers – Lateral Redesign

The lateral system redesign of One South Dearborn is a steel braced core and outriggers system. A building can be stiffened effectively by adding a single level of outriggers at the top of the structure, in which case it is sometimes referred to as "hat trusses" or a "top-hat" structure. Each additional level of outriggers increases the lateral stiffness, but by a smaller amount than the previous additional level. While the outrigger system is very effective in increasing the structure's flexural stiffness, it does not increase its resistance to shear, which has to be carried mainly by the core.

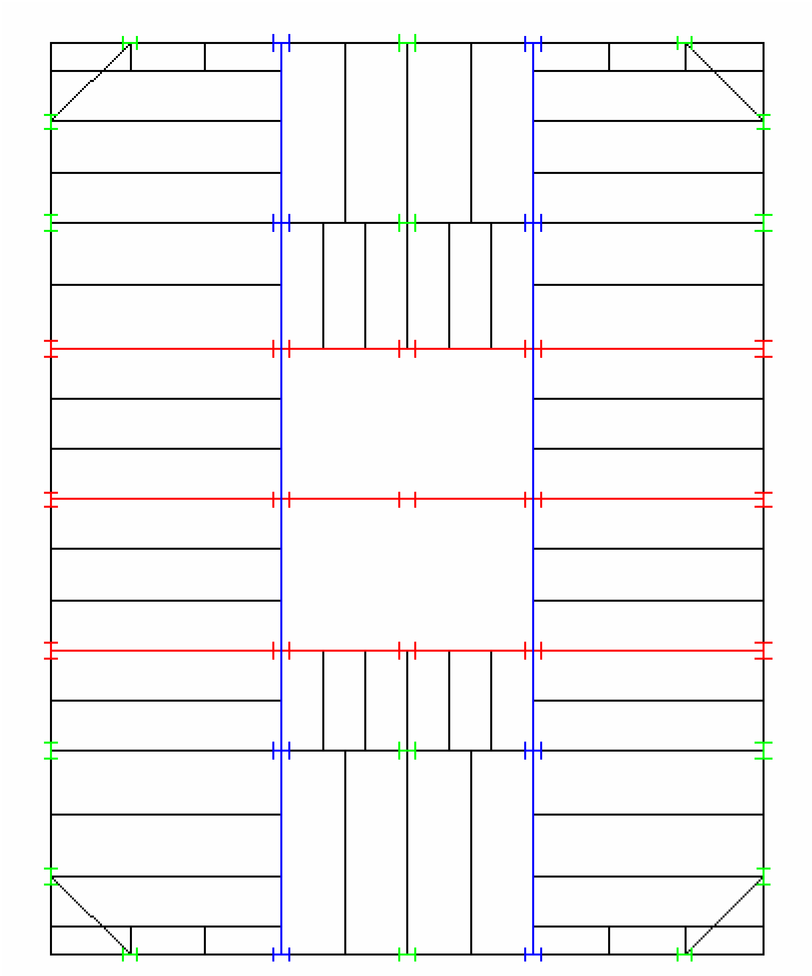
Several iterations of different configurations were taken to solve the concentrically and eccentrically braced frames with and without outriggers. The worst case loading from wind and seismic loads were first applied assuming a concentric load evenly distributed to all frames in that direction. When the lateral loading acted on the building, the column restrained outriggers resisted the rotation of the core, causing lateral deflections and moments in the core to be smaller than if the freestanding core alone resisted the loading. The figure below illustrates the overall drift of the freestanding core when the members are minimum size by code. The frames in the West-East direction are concentrically braced, while the frames in the North-South direction are eccentrically braced.



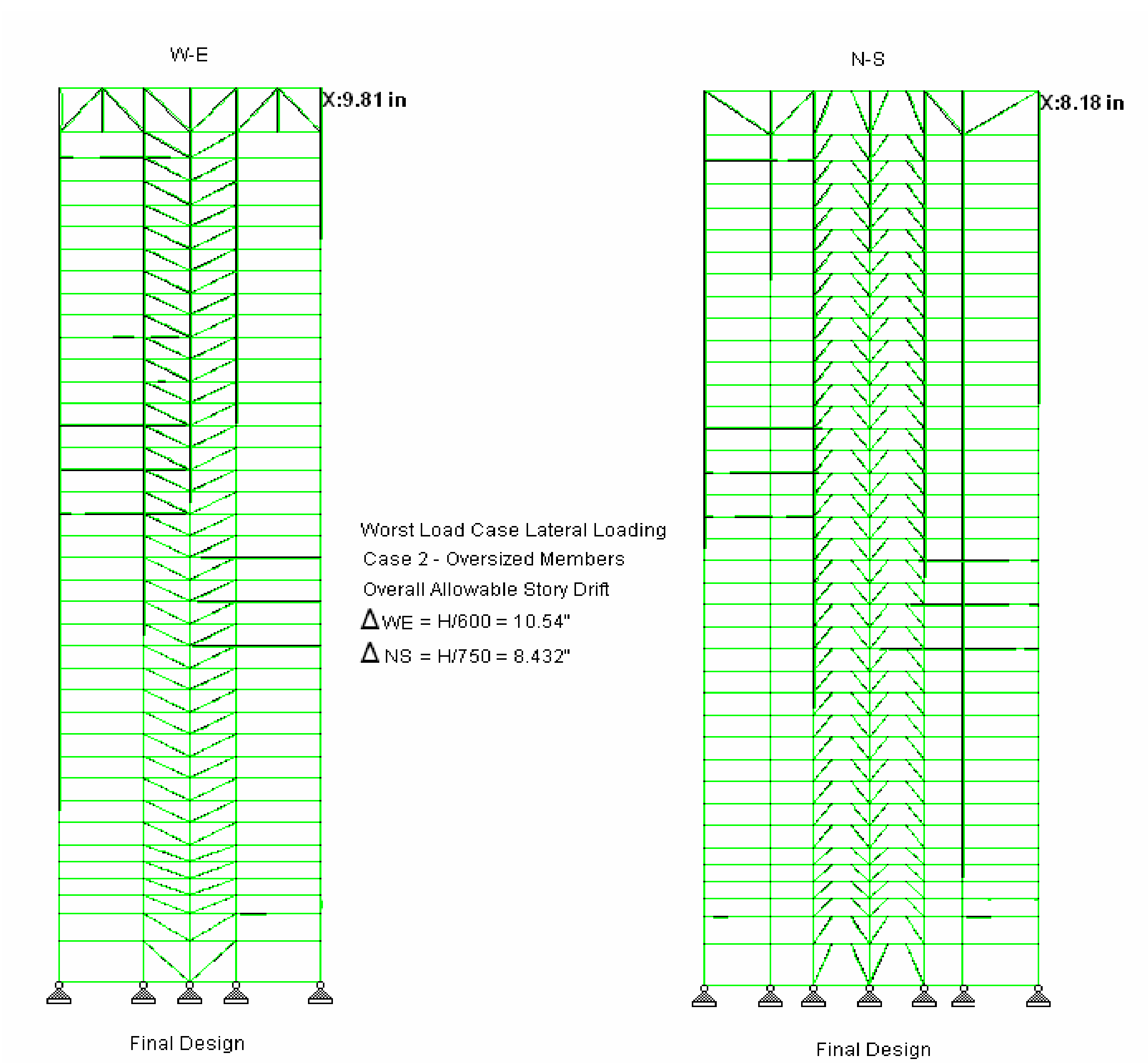
The outrigger configuration in this analysis consisted of linking the core of the building to the exterior columns by way of a truss. This increased the effective depth of the structure when it flexed as a vertical cantilever, by inducing tension in the windward columns and compression in the leeward columns. The outriggers served to reduce the overturning moment and transfer the reduced moment to columns in the core by the way of a tension-compression couple, which takes advantage of the increased moment arm between the columns.



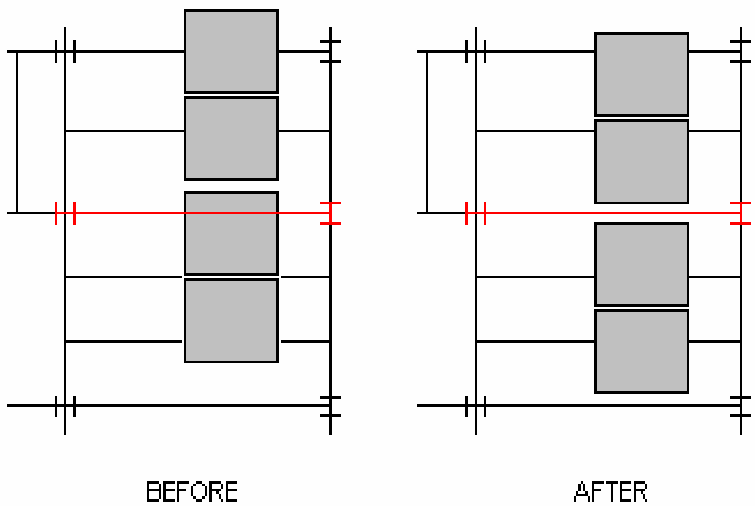
One South Dearborn is made up of a central core with column free floor space between the core and the exterior support columns. While this results in greater functional efficiency, it also effectively disconnects the two major structural elements available to resist the overturning moment present in a high-rise building. This uncoupling of the interior core and the perimeter frame reduces the overall resistance of the structure. The incorporation of outriggers in this same system joins these two components and enhances the system's ability to resist overturning forces dramatically. For rectangular buildings, outriggers can engage the middle columns on the long faces of the building under the application of lateral loads in the more critical direction. In a freestanding core system, these columns that carry significant gravity load are either not incorporated or underutilized. In the redesign of One South Dearborn, the outrigger system efficiently incorporated almost every gravity column into the lateral load resisting system, which can be seen in the figure below. The green columns are not integrated into the outrigger system.



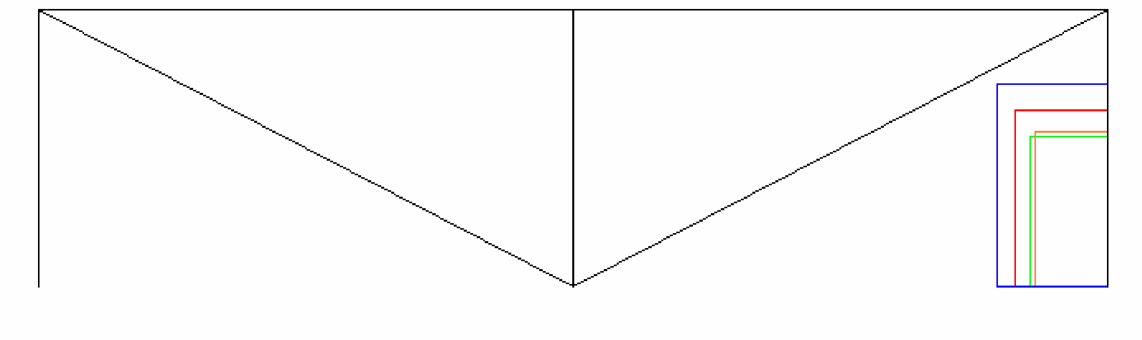
After solving for member sizes under this worst case loading, the relative stiffness of each frame were found. The lateral loads were than applied according to the relative stiffness and considering a torsional eccentricity of five percent of the length of the building. The frames were then redesigned under this new loading condition. The analysis of the braced frames and outriggers within the lateral system were done with STAAD. The Chicago Building Code set that drift must be restricted to Height/750 in the North-South direction and Height/600 in the West-East direction. To maintain the allowable drift the members of the steel braced core were oversized dramatically. As seen in the picture below, the drift of the W-E frame was 9.81" and the N-S frame was 8.18" which were less than the allowable drift of 10.54" and 8.432" respectively.

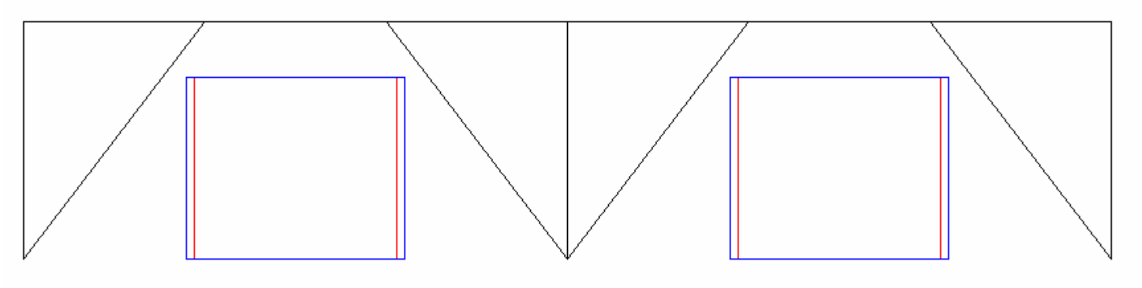


The most significant drawback with the use of outrigger systems is their potential interference with unoccupied or mechanical space. The redesign of One South Dearborn has outriggers interfering with the mechanical room on the 39th and 40th floor. A set of two 900 ton cooling towers had to be moved ten feet to clear an outrigger. Shown below is a before and after placement of the cooling towers.



Also the diagonal bracing of the core is inherently obstructive to the architectural plan and posed problems in the organization of internal space and traffic as well as locating door openings. In many locations the type of bracing has to be selected primarily on the basis of allowing the necessary openings through a bay, often at the expense of efficiency in resisting the lateral forces. The figures below show how the doors were positioned due to the concentric and eccentric bracing of the steel core.





Another potential drawback from a construction point of view is the impact the outrigger installation can have on the erection process. The construction of an outrigger if not approached properly can have a negative impact on the assembly procedure. Also avoid adding additional outrigger levels for borderline force or deflection control. This will have a significant positive impact on the overall construction costs.

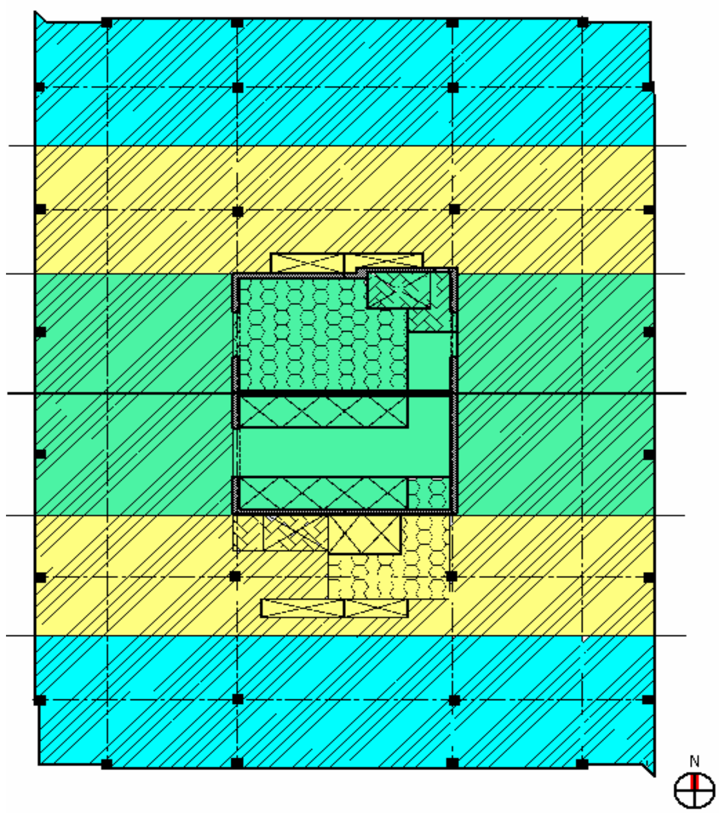
The floor framing plan was changed to accommodate the newly redesigned core with outriggers. The size of the bays in the W-E direction is 25 feet, while the bays in the N-S direction are 30 feet. Unlike the existing core, the redesigned core remained constant throughout the height of the building. Also with the changed flooring plan the building was able to utilize interior columns for

lateral resistance. The floors were redesigned in RAM Steel using the gravity loads set forth from the existing conditions section. The floor layout and member sizes of different loading conditions can be found in the appendix.

The outriggers provided many structural and functional benefits to the building's overall design. There was significant reduction of uplift and net tension forces throughout the columns and the foundation system. Though an analysis and redesign of the foundation system was not performed, it should definitely be an area of significant interest. The redesign structure creates a noticeable decrease in loads transferred from the superstructure to the foundation. This would greatly affect the foundation design. A caisson system still would be used in the redesign; however the diameter and reinforcement of the individual caissons would decrease.

Post-Tensioned Concrete – Floor Framing Redesign

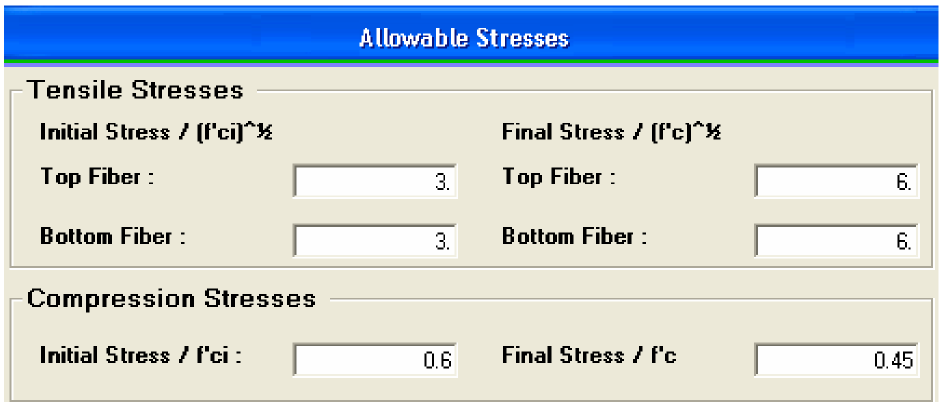
The floor framing redesign of One South Dearborn is made up of wide, shallow, post-tensioned concrete beams. This system was chosen because it best fit the design criteria. This system allows long-span floors to achieve the desirable column-free space and the spans extend between the façade and the core. The new three span system is oriented in the West-East direction. The advantage of this system is that the band and slab system can maintain tributary width equal to the column spacing. The picture below shows the post-tensioned design strips. The green is a core strips, the yellow is a column strips and the blue is an end strips.



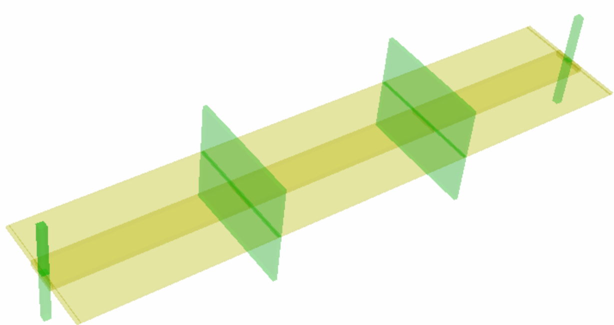
Concrete is best utilized for compression and cannot carry a large amount of tensile load. Post-tensioned concrete is concrete that is pre-stressed in the field using jacking devices. Post-tensioned floor systems use tendons, high-strength steel strands. These tendons have resulted in two systems: bonded or unbonded. In the bonded system the prestressing tendons run through small continuous flattened ducts which are grouted after the tendons are stressed. With an unbonded system the tendon is not grouted and remains free to move independently of the concrete. The tendons are cast into the concrete slabs with small anchorages fixed to each end. When the concrete has obtained a specified compressive strength, the tendon is stressed using a small hand-held jack, completing the post-tensioning procedure. The unbonded system was utilized in the redesign of One South Dearborn. In an unbonded system, tendons can be located close to the surface of the concrete to maximize the eccentricity. Also in this system, the tendons are flexible and can be easily fixed to different profiles. They can be displaced locally around holes and to accommodate changes in slab shape. Furthermore unbonded post-tensioning usually requires fewer strands due to lower friction and greater available drape.

The form of construction selected for the redesign of One South Dearborn was a band beam and slab system. Slab bands or wide shallow beams are basically a thickening of the slab along the column lines to allow additional drape. Although there is no absolute maximum value for the band depth, it is recommended that the band width is at least three times the band depth. The picture below shows how the unbonded tendons are positioned in a band beam and slab system.

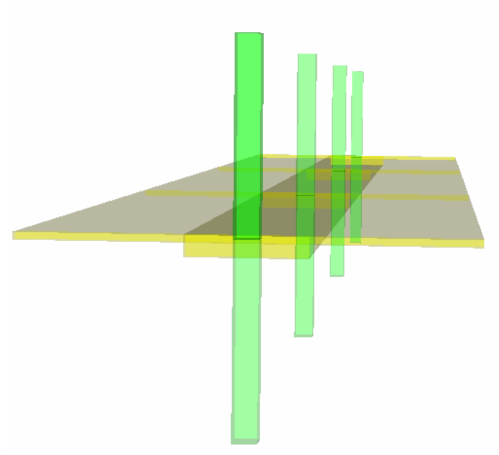
The ADAPT-PT design program supplied by the ADAPT Corporation was used for the design of the post-tensioned band beams and slab for the three-span condition. By adjusting the strand drapes and jacking force the program provides the compressive and tensile stress checks for the top and bottom of the members. The screen shot from the ADAPT-PT program below shows the ACI Codes for allowable stresses that will be inputed. Two strips were designed for each different gravity loading condition. One design strip evaluated the shear walls in the design, while the other design strip evaluated the strip just with columns. These loadings consisted of typical, mechanical, parking and level 2 loading. See the post-tensioned section of the appendix for the design criteria.



For a typical loading case, the three-span band beam was 96" wide by 11" deep with a 6" slab, spanning 45.58 feet, then 50 feet, then 45.58 feet. Other dimensions for different loadings can be found in the appendix. The wide shallow beams were helpful in making the strip an economical design by reducing the force per area to less than 300 psi. A partial parabola drape was used in the design of the post-tensioned concrete system. This is where the tendons are positioned straight over the supports.



The picture shown above is the design strip for the three-span band beam and slab system with shear walls and below is the design strip for the three-span system with columns. The jacking forces and stresses for the different loading conditions can be found in the appendix.



The ½" diameter strand has an area of 0.153in², which has a final effective force of 27 kips which corresponds to an effective stress after allowance for all prestress losses. The number of tendons equal the required tendon force divided by the final effective force. The chart below shows the number of tendons per design strip. ACI 318-02 Section 18.12.4 states that the

Design Strip Cases	Number of Tendons
Typical with Columns	35
Typical with Core	35
Typical End Span	35
Level 2 with Columns	53
Level 2 with Core	51
Mechanical with Columns	51
Mechanical with Core	49
Parking with Columns	31
Parking with Core	33

spacing of the uniform tendons cannot exceed eight times the slab thickness or five feet. It also requires that a minimum of two tendons be provided in each direction through the critical section over the columns.

When stresses exceed the cracking limits of concrete, a cracked section deflection estimate was performed using PTI'S bilinear elastic modulus approach. A strength analysis and design was conducted to determine if any mild reinforcement was necessary to meet the factored ultimate conditions. The design strip summaries in the appendix show the mild reinforcement and its placement in the band beam and slab. According to the ACI code the deflection of the flooring system is limited to L/360 or for the max span 1.67". All the deflections are well below the limited deflection; this can be seen in the appendix.

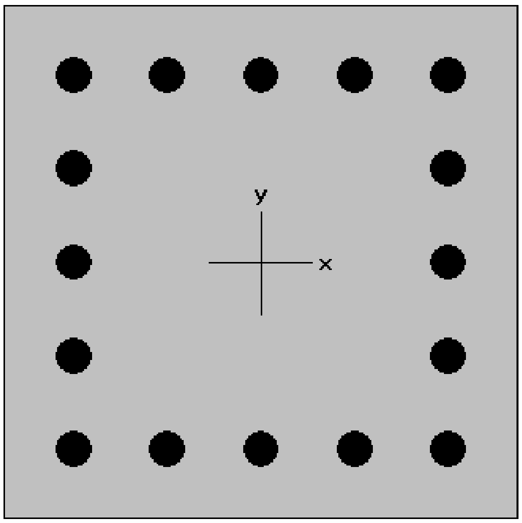
Because post-tensioning causes axial shortening of the prestressed member, it is necessary to consider the effects of axial restraint. Such restraint can overstress the columns or walls in flexure and shear. Since the core of One South Dearborn is fairly centralized the axial shortening of the floor is in a direction toward the core. The perimeter columns move inward, but because they moved by the same amount story to story, no permanent bending stresses occured except in the first story above the non-prestressed floor. Since One South Dearborn's first floor is higher than a typical story, the flexibility of the columns is greater and induced bending moments were easily accommodated.

Several advantages were gained from using a post-tensioning system such as a reduction in slab and band depth due to the upward force provided by the tendons. The smaller floor thickness either could maximize the ceiling zone available for horizontal services or keep down the overall height of the building. Alternatively, it minimizes the exterior surface area to be enclosed, as well as the vertical runs of mechanical and electrical system. The reduced building volume will save on cladding costs and may reduce running costs of HVAC equipment.

For office construction, flexibility is mostly concerned with likely future modifications in the internal space. In many cases these do not substantially affect the structure. Core areas, primary services distribution and other major items remain fixed, although some additional holes for minor services may be required subsequently. The positions of the tendons can be marked on the slab's soffit to aid identification for future openings.

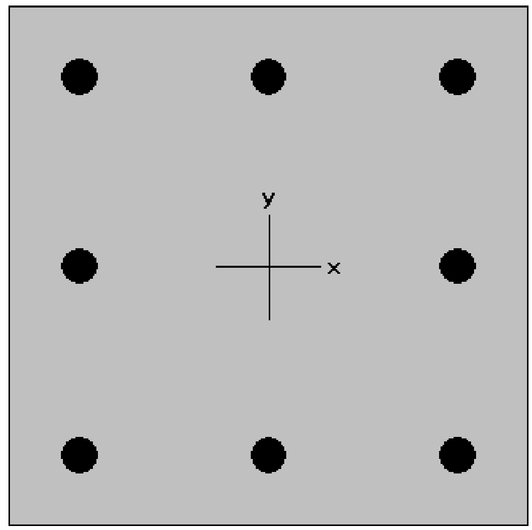
ADAPT-PT was used to perform a frame and loading analysis which gave the moments and axial loadings of the columns. A set of the interior columns and exterior columns were designed for the worst case moments and axial loading as well as the moments and axial loading for a typical loading case. This design information was then entered into PCA Column to size the column and reinforcement. See the appendix for calculations of moments and axial loads. All of the columns were designed as twenty inch square columns for constructibility reasons. The reinforcing was equally distributed along the sides of the columns to take the bending forces that the building experiences. Below are sections of the worst case and typical loading case exterior and interior columns. These sections show the column size and reinforcement that were designed in PCA Column.

WORST CASE EXTERIOR COLUMN



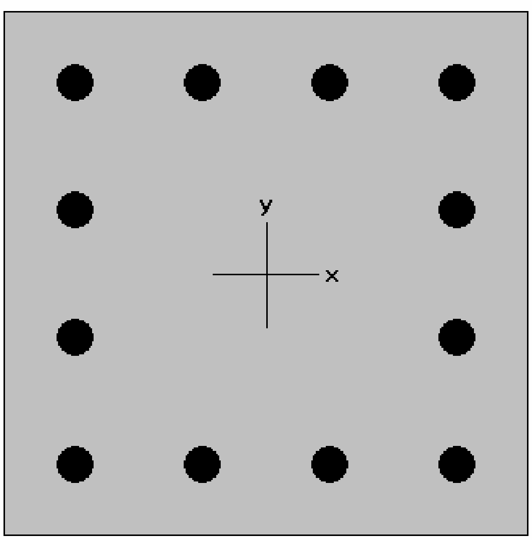
20.0 x 20.0 inch
6.24% reinf.
16 - #11

WORST CASE INTERIOR COLUMN



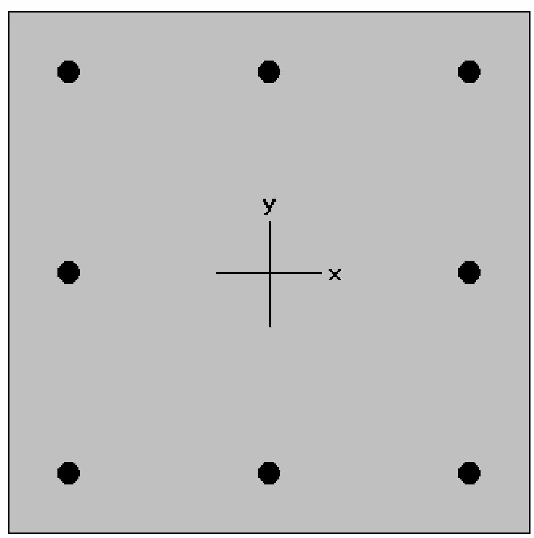
20.0 x 20.0 inch
3.12% reinf.
8 - #11

TYPICAL EXTERIOR COLUMN



20.0 x 20.0 inch
4.68% reinf.
12 - #11

TYPICAL INTERIOR COLUMN



20.0 x 20.0 inch
1.20% reinf.
8 - #7

The shears walls from the existing structure were used when it came to desiging the lateral system. The lateral loads were determined based on the wind and seismic calculations. Since there are two identical shear walls in the North-South direction the load was divided into half to determine the design loads for the shear walls. For the West-East direction, the top fifteen floors have 2 shear walls and below that the remaining levels have three identical shear walls, which took half and third the load, respectively. The shear walls were designed to resist more than just these lateral forces, based on the shear calculations included in the appendix. Additional shear wall area may have been needed to resist overturning moments from uplift. Reinforcement was placed throughout the caissons, so it is possible that uplift was a factor. The shear walls act as vertical cantilever beams which transfer lateral forces from the superstructure to the foundation. Even though the building has coupled shear walls which increases the stiffness, it was analyzed as independent cantilevers ignoring the coupling effect. This resulted in a conservative wall design. This means the link beams were ignored in the distribution of forces and the calculation of the stiffness. The portion of the total lateral force which each wall resists depends on the bending and shear resistance of the wall and the characteristics of the foundation. It was assumed that the floors acted as rigid elements for loads in the plane of the floor and the deformations of the soil were neglected. Thus, the shear walls alone are assumed to resist all lateral forces. The center of rigidity is in the center of the building in both the North-South and West-East direction, since the shear walls are placed symmetrically about the center of the building. However, a minimum five percent torsional moment has been incorporated in the design of the shear walls. This information is referenced in the Existing Conditions section of the appendix. The load on each shear wall was determined by combining the effects produced by rigid body translation and rotation. Torsional effects were based on the difference between the center of rigidity of the shear walls and the eccentricity of the lateral load. It was found that the torsional effects were greater on the upper tier of the nonproportionate shear walls.

A drift analysis was performed using ETABS, and the results can be seen Appendix A. Drift limitations were specified by the Chicago Building Code. This analysis produced a maximum drift at the top of the building of ten inches in the North-South direction and of eight inches in the West-East direction.

$$\Delta x = 10'' \quad H/600 = 527'/600 = 10.54'' \quad \text{OK}$$

$$\Delta y = 8'' \quad H/750 = 527'/750 = 8.432'' \quad \text{OK}$$

The shear reinforcement was checked in all of the shear walls. The checks are based on the fraction of the factored load going to each wall. Shear strength calculations are shown in spreadsheets in Appendix A. The design basis for shear walls was according to ACI Code 11.10.

$$V_u \leq \Phi V_n \quad \text{where } V_n = V_c + V_s$$

Though an analysis and redesign of the foundation system was not performed, it should definitely be an area of significant interest. The redesign structure creates a noticeable increase in loads transferred from the superstructure to the foundation. This would greatly affect the foundation design. A caisson system still would be used in the redesign; however the diameter and reinforcement of the individual caissons would increase.

BREADTH ANALYSIS

Mechanical Breadth Study

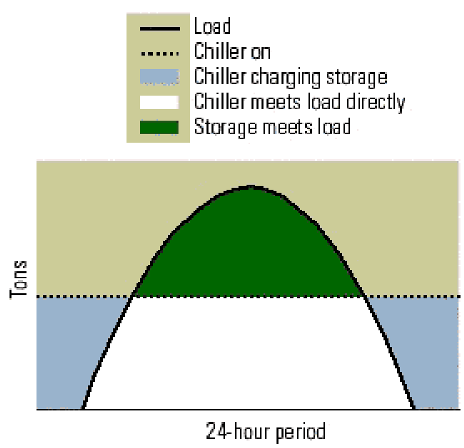
The mechanical systems of One South Dearborn was investigated to lower building operating costs and the capital cost of cooling and heating equipment so that smaller devices can be installed. This was achieved by using thermal energy storage. Thermal energy storage systems allow a shift in part of the actual load required to off-demand hours to take advantage of cheaper time based utility rates. For the past 50 years, energy management design approaches have been on the forefront of mechanical design for buildings. Not only has the owner profited from savings on electrical costs, but the energy provider and the environment also benefited from thermal energy storage. Thermal energy storage for large buildings, such as the one being analyzed in this report, is generally more effective than those for smaller buildings. During the design phase the owner decides whether his building will use an energy efficient system with high initial costs and lower operating costs or a standard system with low initial costs and higher operating cost. One South Dearborn has chosen to use a standard system due to the low initial costs.

In spite of how resourceful a building's mechanical system is, it still requires energy from an external source, most commonly electricity. Electricity is required to run chillers, air handling units, pumps, cooling towers, etc. In large cities such as Chicago, electricity costs fluctuate with demand. On-peak hours of operation, the time period that a building is occupied, generally have a higher cost of electricity than the off-peak hours, the time period where the city demand for electricity is lower. Due to this fact, thermal energy storage can play a big role by reducing a buildings on-peak electrical demand.

The air-conditioning and distribution system will generally have means to heat, cool, humidify, dehumidify, clean and distribute air to various conditioned spaces. A cooling fluid must be supplied to the cooling coil in the air handler. The liquid is cooled with chillers and then pumps are required to circulate the liquid through the piping. The operation of these chillers accounts for a significant portion of the total electrical demand of the mechanical system. If the chillers were run at night at off-demand hours then it is possible to reduce operating costs. In order to run the chillers during off-peak hours and provide cooling during peak hours, ice storage can be utilized. The ice produced during the night, which is created and stored in modular tanks can be used the next day to meet the building's air-conditioning load requirement. These storage systems not only dramatically reduce the use of peak period, high-cost energy; they also can potentially reduce total building energy usage by 15%. Off-peak cooling systems may reduce the size of a building's air conditioning equipment, including chillers, pumps, fan coils and cooling towers. The existing mechanical system of One South Dearborn has two 1,500 ton water cooled centrifugal chillers located on the 7th floor.

This mechanical breadth studied the capabilities of ice thermal storage and its affect on the chiller configuration. Several different operating strategies are available for charging and discharging storage to meet cooling demand during peak hours. A full-storage approach can be used, which shifts the entire on-peak cooling load to off-peak hours. A partial-storage strategy can also be designed where the chiller runs to meet part of the peak period cooling load, and the remainder is met by drawing from storage. The redesign of One South Dearborn concentrated on the partial-storage operating system because it is very first cost-effective. Partial-storage systems may be run as load-leveling or demand-limiting; both are evaluated in this study.

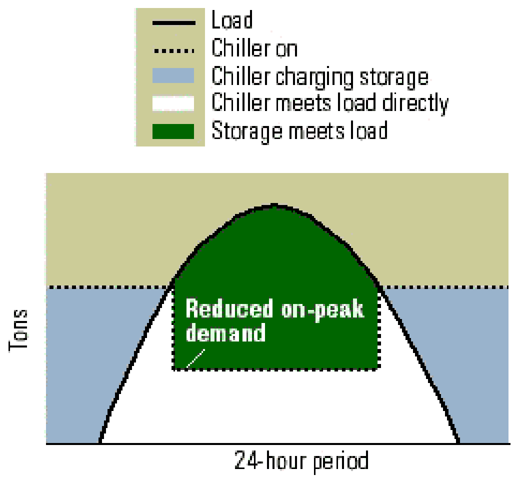
In a load-leveling system, the chiller runs at its full capacity for 24 hours on the design day. When the load is less than the chiller output, the excess cooling is stored. When the load exceeds the chiller capacity, the additional requirement is discharged from storage.



Load-leveling

Source: ASHRAE Design Guide for Cool Thermal Storage

In a demand-limiting system, the chiller runs at reduced capacity during on-peak hours and is often controlled to limit the facility's peak demand charge. Usually demand savings and equipment costs are higher than the load-leveling system. However both approaches minimize the required chiller and storage capacities for a given load.



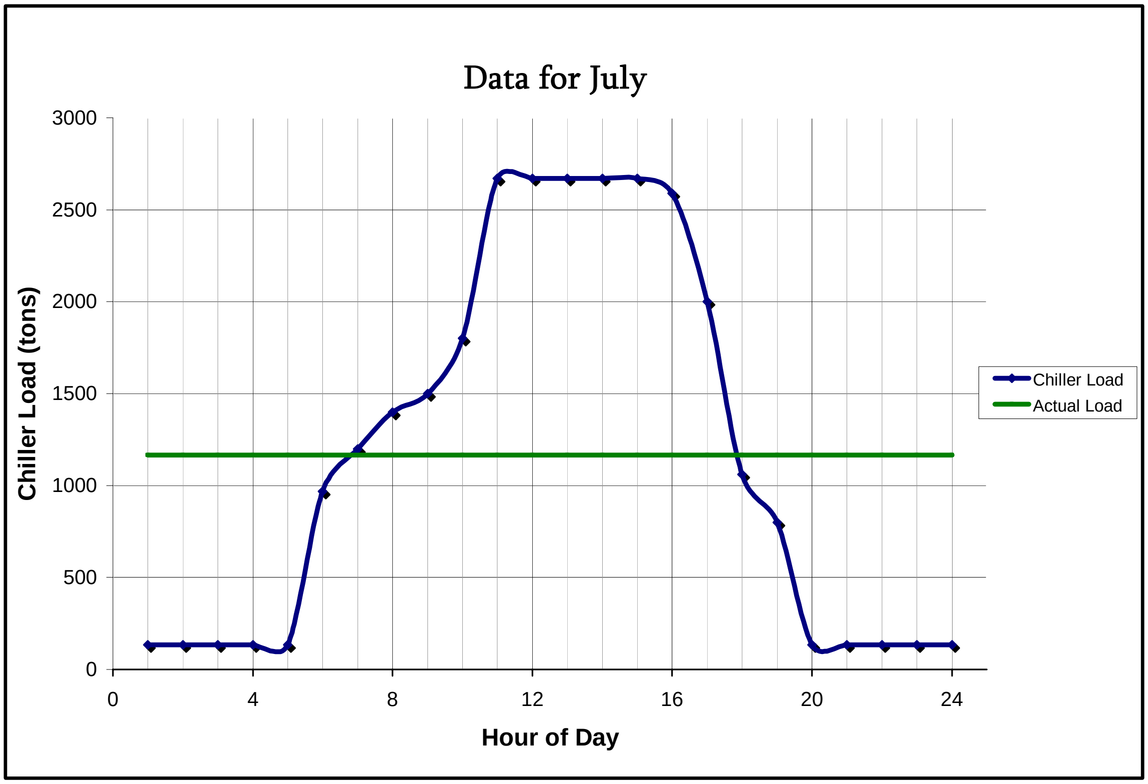
Demand-limiting

Source: ASHRAE Design Guide for Cool Thermal Storage

The worst-month hourly chiller load profile was obtained to perform the mechanical redesign. This data provides a breakdown of hours to tons of cooling. The worst design month for One South Dearborn was in July. The total building load was found to be 28008 ton-hrs with a peak load of 2670 tons. Other information needed for the redesign was accumulated. The electrical rates for on-demand and off-demand hours were obtained for Chicago, Illinois. The on-demand time period runs from 6 A.M. to 6 P.M. and costs $0.0575 per kwh. While the off-demand time period runs from 6 P.M. to 6 A.M. and costs $0.0249 per kwh. Liquid running through the pipes for both load-leveling and demand-limiting is 25% Ethylene Glycol.

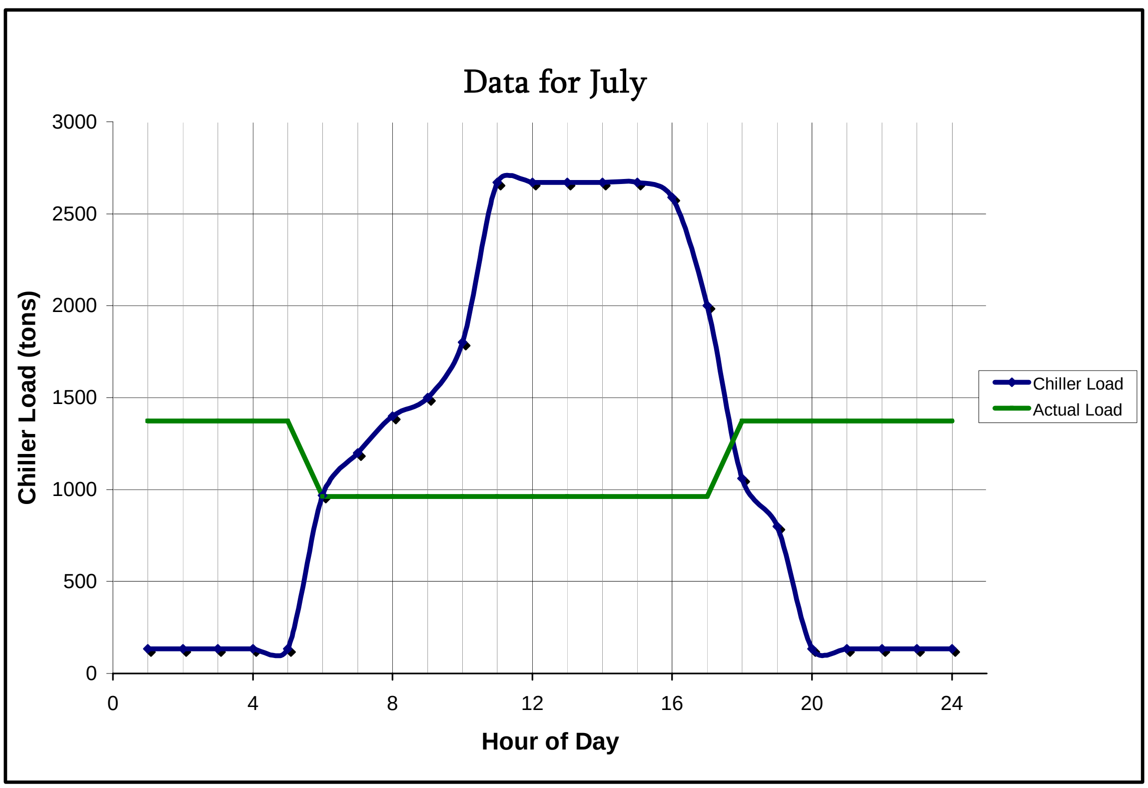
For the load-leveling redesign, the system consists of two 584 ton Carrier 19XR Evergreen chillers. In addition, there needs to be an extra pump added for the ice making cycle. The pump needs to supply a maximum of 4247 GPM to the loop. The pump selected was a Bell & Gossett Series HSC³ that flows up to 6500 GPM. CALMAC Icebank Model 1500C was selected for the ice storage tanks. The total storage capacity of the tank is 570 ton-hrs. The total number of tanks needed for this configuration is 19 units. With load-leveling, the cooling cost would have an average monthly savings of $1,238.72.

Load-Leveling Hourly Chiller Load Profile



For the demand-limiting redesign, the system consists of a 962 ton and a 412 ton Carrier 19XR Evergreen chiller. The 412 ton chiller is designated to making ice during the off-demand hours. However there needs to be an additional pump added to this ice making cycle as well. The pump needs to supply a maximum of 1694 GPM. The pump selected was a Bell & Gossett Series 1531 that flows up to 2300 GPM. CALMAC Icebank Model 1500C was selected for the ice storage tanks. The total storage capacity of the tank is 570 ton-hrs. The total number of tanks needed for this configuration is 24 units. With demand-limiting, the cooling cost would have an average monthly savings of $1,521.91. Calculations and cut-sheets for both systems can be found in the appendix.

Demand-Limiting Hourly Chiller Load Profile



A preliminary payback analysis was calculated to determine which operating system is better suited for One South Dearborn. The load-leveling system had a 23% savings using ice storage with a monthly savings of $1,238.72. The demand-limiting system had a 29% savings using ice storage with a monthly savings of $1,521.91. The demand-limiting had a $238.19 monthly savings over load-leveling. However, the equipment prices for the demand-limiting costs more than the load-leveling equipment. The demand-limiting needed 24 units which came to a price of $1,327,600 and load-leveling needed 19 units that cost $1,080,400. It cost $917,378 for the existing two 1500 ton chillers. The load-leveling had a chiller savings of $560,215 and the demand-limiting had a savings of $497,222. The initial cost of the load-leveling is $520,185 with a payback of 420 months. The initial cost of the demand-limiting is $830,378 with a payback of 546 months. This preliminary payback analysis does not account for the savings of the smaller pipes, fan coils, cooling towers and pumps that would attribute to the final cost. From the calculations and cost analysis performed I would suggest using partial-storage load-leveling over demand-limiting due to the lower initial costs. If the load-leveling system was considered in the design phase rather than renovating the existing mechanical system with thermal energy storage. Then I would also recommend the partial-storage load-leveling system over the existing mechanical system.

Structural Impact

The structure of One South Dearborn was impacted due to the changes in the mechanical system. Although the chillers and cooling towers decreased in size, the addition of storage tanks affected the gravity loads in the mechanical room on the 7^{th} floor. The storage tanks produced a floor loading of 391 psf. The floor framing on the 7^{th} floor of the all-concrete and all-steel structure needed to be redesigned.

The redesign of the post-tensioned floor system due to the mechanical changes affected the depth of the bands, the jacking force in the tendons and provided additional cost to the structure. There was four more inches of concrete added to the depth of each band. That came to a total of an extra 27 cubic yards of concrete costing $7,956. The largest jacking force in the spans had a 280.6^k increase from the system without thermal storage.

The redesign of the structural steel floor system due to the mechanical changes affected the size of the flooring members, the number of shear studs and provided additional cost to the structure. The redesigned members weighed 26 tons extra with the addition of the storage tanks. There was a smaller number of shear studs, 218 less. This system costs an additional $423,330 with the storage tanks. The price of the steel system was greatly increased compared to that of the concrete system. A redesigned 7^{th} floor steel framing plan and takeoff can be found in the appendix.

Construction Management Breadth Study

In order to successfully evaluate the different structural systems proposed for One South Dearborn, two main criteria had to be considered; the structural economics and the duration of structural construction. Since One South Dearborn was built in 2004, all costs and durations were determined for that year.

I will start with a breakdown of One South Dearborn's existing structural cost and schedule. The costs and schedule information was obtained directly from the Turner Construction Company, the construction management firm whom supervised the project. The total cost of the composite structure was $26,797,392. This cost can be broken down as follows:

Concrete	$4,796,542
Concrete Core Walls	$4,814,091
Miscellaneous Walls	$488,979
Structural Steel	$14,848,258
Miscellaneous / Ornamental Metals	$1,849,522

For the analysis breakdown of existing structural costs, see the Appendix. The total cost of the composite structural system per square foot, was $26.19/ft^2. Also the steel needs fireproofing and that costs an additional $1,137,149.

The schedule for the erection of the composite structural system is very complex. The core has to be started and substantially completed during the lead time required for structural steel. The phasing of the concrete and steel erection and utilization of the equipment affects the economics of the project. The starter

concrete wall of the core was started on March 15, 2004 and the final steel tier erection was finished on January 10, 2005. Since the steel needs spray-on fireproofing, the complete structure was finished one month later on February 10, 2005. The entire structural system of One South Dearborn was completed in 44 weeks. The whole project took 96 weeks from the contract construction start to the final completion of the building. For further durations of the existing structure, see the Appendix.

The cost estimates and activity durations for the redesign of One South Dearborn were determined using *R.S. Means 2004 Heavy Construction Cost Data* and Turner Construction's cost estimate data. The cost estimates for the redesigns were performed using the unit price material, labor and equipment costs found in *R.S. Means.* The location factor for material in Chicago, Illinois during 2004 is 0.996. The unit costs included a limited number of activities involved in the structural construction of the redesigned One South Dearborn. Just the material and labor costs were analyzed since the figures from Turner only deal with those costs and it was necessary for an equivalent comparison. The activities involved in the construction, as well as the cost breakdown associated with each activity, can be found in the spreadsheet in the appendix for each redesign. Notes about the Turner information as follows: The steel member pricing includes connections and welds in the cost and the core walls include formwork and rebar in the cost. A re-analysis of the existing structural system came to a total of $21,878,358. This re-analysis was performed to develop a better comparison to weigh against the two different redesign costs. The total cost of the steel braced core and outrigger system came to be $18,555,217; while the total cost of the post-tensioned concrete system came to be $29,749,563. All these values were calculated using Turner cost information data, because when using R.S. Means to estimate the cost for the post-tensioned concrete system the final total came to be $11,462,203. It was realized that the estimation was like comparing apples to oranges and there would not be a good evaluation of the systems, so Turner was contacted to receive their post-tensioning cost data. An economical comparison is presented in the final conclusions section of the report.

From the activities determined, durations for each activity could be calculated based on crew type and/or Turner's scheduling data. The daily output for each crew type is given by R.S. Means and the post-tensioning's duration was calculated using this data. The durations are in units of crew weeks, which were established to be the equivalent of five-day work weeks. Thus given the durations, a structural schedule for each system was developed. The existing structural system was completed in 44 weeks. The braced steel core and outrigger system was completed in 40 weeks. The post-tensioned concrete system was completed in 54 weeks. A schedule comparison is presented in the final conclusions section of the report.

FINAL CONCLUSIONS

Structural Comparisons

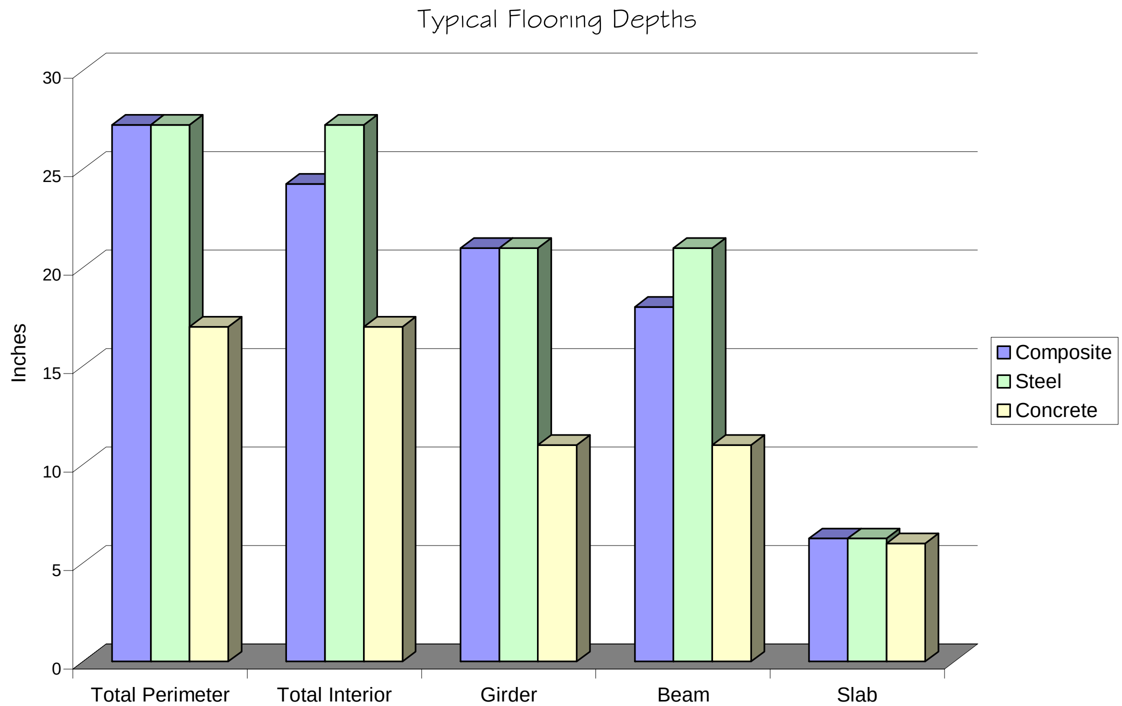
The redesigned systems were examined to make sure the required design criteria were met, which were established by the owner's preferences. Not only did the alternate designs need to meet the owner's preferences, the designs had to maintain similar square footage, floor heights, exterior aesthetics and architectural layout of the building. The perceptibility of lateral motion, uninterrupted floor space and the ability for future modifications were all concerns of the owner.

In a high-rise, there is a need for damping to reduce the occupant perceptibility of lateral motion. The all-steel structure needed to be made comparatively stiff so that it had a damping in the range of 1 percent. The composite or all-concrete structure provided more damping, upwards of 1.5 percent to 2 percent. This is due to the inelastic behavior of the concrete. With the all-concrete and composite structure, it is much easier with that damping to make the motion hardly noticeable. The Chicago Building Code established design limitations for allowable lateral drift in the city. It was set that drift must be restricted to Height/750 in the North-South direction and Height/600 in the West-East direction. As seen in the chart below, the overall drifts for the different structures are all adequate according to code.

Direction	Code	Allowable	Composite	All-Steel	All-Concrete
North-South	H/750	8.432"	8"	8.18"	8"
West-East	H/600	10.54"	10"	9.81"	10"

In a typical office setting, there is a need for long spans to have uninterrupted floor space for aesthetic and functional preferences. With these open initial layouts, there may be the need for future modifications. One South Dearborn has long-span floors to achieve the desirable column-free space and the spans reach extend across the façade and the core. A deep floor system in steel or reinforced concrete is utilized to achieve longs spans while still maintaining acceptable deflections. Post-tensioning provided the advantage of allowing a reduction in depth of the floor system. This could change the floor-to-floor height or give more room to mechanical system in the plenum space. Below in the chart and graph is a comparison of floor depths amongst the three different structures. The post-tensioning floor system had considerable reduction in floor depth while the all steel structure increased its floor depth in the interior spans.

Typical Sizes	Slab Depth					
	Typical	Mechanical	Parking			
Composite Structure	6 1/4"	9"	7 1/2"			
All-Steel Structure	6 1/4"	9"	7 1/2"			
All-Concrete Structure	6"	9"	6"			
Typical Sizes	Girder Depth			Beam Depth		
	Typical	Mechanical	Parking	Typical	Mechanical	Parking
Composite Structure	21"	27"	21"	18"	27"	18"
All-Steel Structure	21"	27"	18"	21"	27"	18"
All-Concrete Structure	11"	15"	8"	11"	15"	8"
Max Depths	Perimeter Total Depth			Interior Total Depth		
	Typical	Mechanical	Parking	Typical	Mechanical	Parking
Composite Structure	27 1/4"	36"	28 1/2"	24 1/4"	36"	25 1/2"
All-Steel Structure	27 1/4"	36"	25 1/2"	27 1/4"	36"	25 1/2"
All-Concrete Structure	17"	24"	14"	17"	24"	14"



Furthermore the post-tensioned flooring system is limited to minor structural modifications such as core drilling and services holes, while there is flexibility for future modifications with steel floor framing.

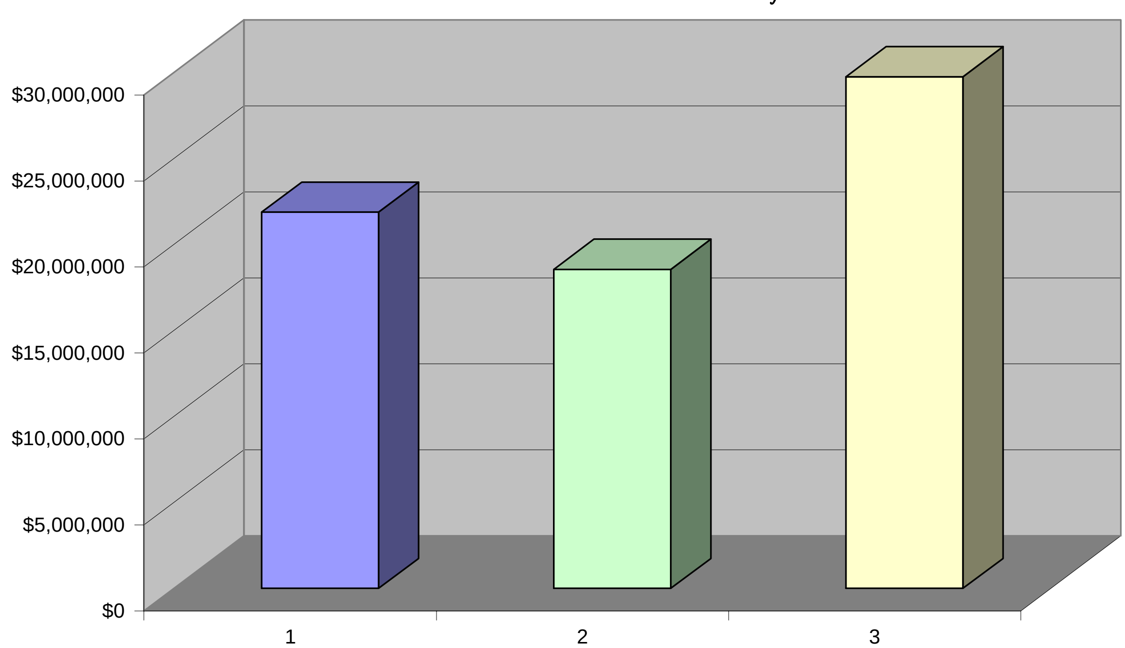
All the required design criteria were met by both of the redesigned systems, with some advantages and limitations compared to the existing system. It can be concluded from the information above, in the design of One South Dearborn, that the existing composite structural system is better suited structurally and functionally than that of the all-steel and all-concrete structural system. It was found that the composite structure made the best use of the materials and their respective benefits. The steel was used where long spans were desirable and future modifications likely and reinforced concrete where it was best used to control lateral drift.

Cost Comparisons

Given the differences between the structural characteristics of the existing and alternate structures, the cost and schedule duration of each system was evaluated to determine if it would play a role in selecting a definitive design. The cost is usually an important factor in deciding the building's structure. For this study, the total cost of the structural systems represented only the material and labor costs. The table and graph below illustrates a comparison between the total costs of each structural system.

	Composite Structure	All-Steel Structure	All-Concrete Structure
Graph Correspondence	1	2	3
Total Costs	$21,878,358	$18,555,217	$29,749,563
Cost Per Square Foot	$21.38	$18.13	$29.07

Total Cost of Structural Systems



The existing composite structural system came to a total cost of $21,878,358. The redesigned system with the steel braced core and outriggers had projected cost of $18,555,217. While the post-tensioned concrete flooring system had an estimated cost of $29,749,563. The all-concrete structure was $7,871,205 more expensive than the existing structure, this was about a 36% cost increase. However, the all-steel structure was $3,323,141 less expensive than the existing structure, this was about an 18% cost decrease. The total cost was broken down into square foot per system. One South Dearborn is 1,023,294 square foot in size. The existing structural system cost $21.38/SF, the all-steel structure cost $18.13/SF and the all-concrete structure cost $29.07/SF. The costs for the redesigns was $3.25/SF less for the all-steel system and $7.69/SF more for the all-concrete system.

It can be concluded from the information above, in the design of One South Dearborn, that the all-steel structural system is significantly less expensive than that of the existing and all-concrete structural system. Note that these costs are both time and location sensitive and were calculated for the Chicago, Illinois area in 2004.

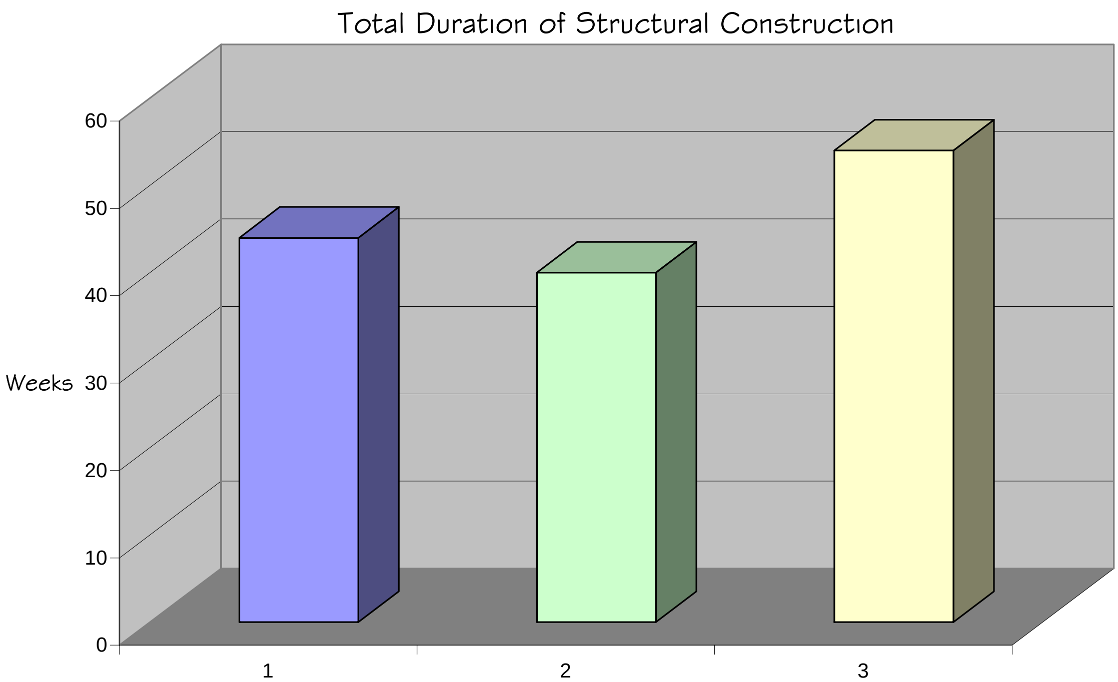
Schedule Duration Comparisons

Another significant factor in comparing the structural systems is the duration of structural construction. As mention previously, the schedule length of the structural phase for One South Dearborn was completed in 44 weeks, staring on March 15, 2004 and finishing on February 10, 2005. The structural schedule for the alternate designs was based on the assumption that all other aspects of the constructing and finishing of One South Dearborn would experience negligible deviations from the existing schedule. The activity durations were determined by using the crew day's information from R.S. Means. The construction start date for both of the redesigns began on March 15, 2004. Since an early construction deadline from Turner Construction was not stress there is no immediate deadline to complete the building.

The redesigned system with the steel braced core and outriggers had projected schedule length of 40 weeks. While the post-tensioned concrete flooring system had an estimated schedule length of 54 weeks. The all-concrete structure took 10 more weeks to be erected than the existing structure; this was a 35% increase in duration. The all-concrete structure was delayed because it had to wait for the formwork to be placed, the rebar and tendons to be positioned, the concrete to be poured and once the concrete dried to strip the formwork. However, the all-steel structure was erected 4 weeks faster than the existing structure; this was about a 10% decrease in duration. The shorter

duration for the all-steel structure occurred because it didn't have to be concerned about the phasing of the concrete and steel erection. The table and graph below illustrates a comparison between the total durations of each system.

	Composite Structure	All-Steel Structure	All-Concrete Structure
Graph Correspondence	1	2	3
Structural Duration	44 Weeks	40 Weeks	54 Weeks



Based on the schedule length information above, it can be concluded that the all-steel structure has a more efficient assembly than the existing and all-concrete structural system. Because proper installation of these systems requires a highly skilled labor force in its use and execution these durations are location sensitive and were calculated for the Chicago, Illinois area.

RECOMMENDATIONS

After designing One South Dearborn as an all-steel and all-concrete structure, I have come to the conclusion that the existing composite structure is a better solution. Even though the all-steel structure had a lower expenditure and shorter construction phase, it didn't function as well as the composite structure. Although cost and schedule are important factors, they were not the most vital for this design. High-end construction will often use a slower or more costly system if there are long-term benefits and advantages it can provide. The composite structure made the best use of the materials and their respective benefits. The concrete core shear walls has lateral drift control against the direct shear caused by wind and seismic loads while the perimeter framing provides long spans and resists torsion generated by the offset loading criteria.

Local history and material tendencies were also influential factors in advocating this design. If One South Dearborn wasn't built in Chicago, where almost every office building erected in the past 20 years has been a steel frame with a concrete core. Then this structural design would not be recommended. The city's local market needs to understand the differing tolerances between steel and concrete and know how to design and construct buildings to accommodate these tolerances for the two materials. For example, the Lincoln Tower in Rochester, New York had sloping floor problems and the wheels of all the chairs on the top floor had to be taken off due to this issue.

ACKNOWLEDGEMENTS

During my past five years at Penn State, many people have provided me with advice, assistance, support, and encouragement. Now, I would like to take some time to thank some of the many who have made this thesis report, my senior year, and my entire Penn State career such a worthwhile experience. First and foremost, I would like to thank Mom, Dad, Tim, Bubba and Jason for supporting and encouraging my actions and ideas. With their insight and guidance, I achieved academic success and have a bright future ahead.

I would like to thank the faculty and staff in the Penn State Architectural Engineering department for all of the opportunities to develop as an engineer. Special thanks to Dr. Boothby, Dr. Geschwindner, Professor Parfitt and Jonathan Dougherty for their one-on-one consultations during the thesis process.

I would like to thank the many professional contacts that I have had the pleasure of meeting and working with through the thesis writing process. Thanks to Carrie Warner, Dan Cronin, Gerard McGuire, Ray Clark and Hines for the chance to work on one of their projects, for their engineering and construction insight, and for all of their assistance in obtaining project information.

I am also thankful for my many great colleagues in the AE Class of 2005. I owe a lot to many of you and am thankful for the example that you have been to me and the impact that you have had on my life.

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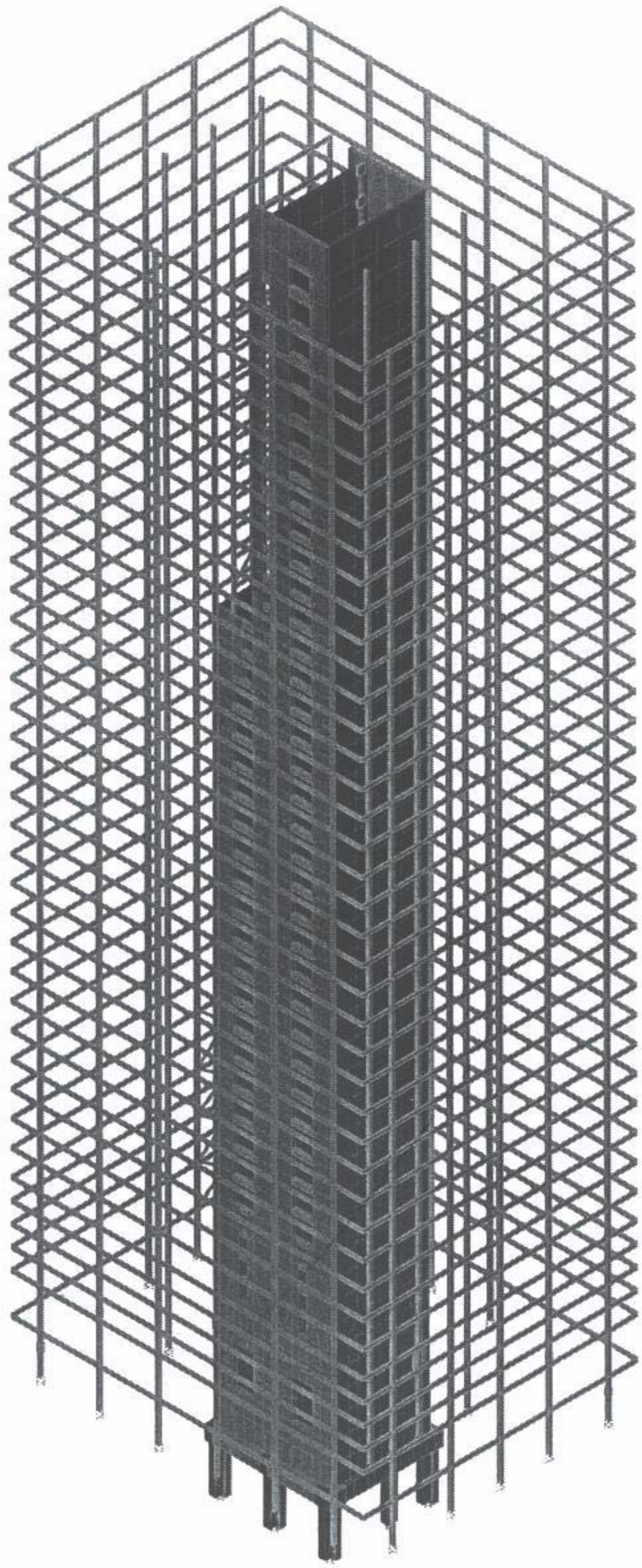
APPENDICES

APPENDIX A

EXISTING CONDITIONS INVESTIGATION

<u>ETABS 3D Model</u>



W-E Wind Loads

Level	Story Height (ft)	Elevation (ft)	Tributary Height (ft)	Tributary Width (ft)	Tributary Area (ft²)	Wind Load (PSF)	Wind Load (kip)	Shear (kip)	Moment (ft-kip)
41 - Roof	0	527	13	180	2340	30	70.2	70.2	0
39	26	501	6.5 + 14	180	1170 + 2520	29 / 30	109.5	179.7	1825.2
38	15	486	14.5	180	2610	29	75.7	255.4	4520.7
37	14	472	14	180	2520	29	73.1	328.5	8096.16
36	14	458	13.5	180	2430	29	70.5	398.9	12694.74
35	13	445	13	180	2340	29	67.9	466.8	17880.96
34	13	432	13	180	2340	29	67.9	534.7	23949.36
33	13	419	13	180	2340	29	67.9	602.5	30899.94
32	13	406	0.5 + 12.5	180	90 + 2250	27 / 29	67.7	670.2	38732.7
31	13	393	13	180	2340	27	63.2	733.4	47445.56
30	13	380	13	180	2340	27	63.2	796.6	56979.76
29	13	367	13	180	2340	27	63.2	859.8	67335.3
28	13	354	13	180	2340	27	63.2	922.9	78512.18
27	13	341	13	180	2340	27	63.2	986.1	90510.4
26	13	328	13	180	2340	27	63.2	1049.3	103330
25	13	315	13	180	2340	27	63.2	1112.5	116970.9
24	13	302	4.5 + 8.5	180	810 + 1530	25 / 27	61.6	1174.1	131433.1
23	13	289	13	180	2340	25	58.5	1232.6	146696.1
22	13	276	13	180	2340	25	58.5	1291.1	162719.7
21	13	263	13	180	2340	25	58.5	1349.6	179503.7
20	13	250	13	180	2340	25	58.5	1408.1	197048.3
19	13	237	13	180	2340	25	58.5	1466.6	215353.3
18	13	224	13	180	2340	25	58.5	1525.1	234418.8
17	13	211	13	180	2340	25	58.5	1583.6	254244.9
16	13	198	8.5 + 4.5	180	1530 + 810	23 / 25	55.4	1639.0	274831.4
15	13	185	13	180	2340	23	53.8	1692.8	296138.2
14	13	172	13	180	2340	23	53.8	1746.6	318144.6
13	13	159	13	180	2340	23	53.8	1800.4	340850.6
12	13	146	13	180	2340	23	53.8	1854.3	364256.3
11	13	133	13	180	2340	23	53.8	1908.1	388361.7
10	13	120	13	180	2340	23	53.8	1961.9	413166.8
9	13	107	13	180	2340	23	53.8	2015.7	438671.5
8	13	94	13	180	2340	23	53.8	2069.5	464875.8
7	13	81	11.5	180	2070	23	47.6	2117.2	491779.8
6	10	71	10	180	1800	23	41.4	2158.6	512951.3
5	10	61	10	180	1800	23	41.4	2200.0	534536.8
4	10	51	10.5	180	1890	23	43.5	2243.4	556536.3
3	11	40	13.25	180	2385	23	54.9	2298.3	581214
2	15.5	24.5	20	180	3600	23	82.8	2381.1	616837.2

N-S Wind Loads

Level	Story Height (ft)	Elevation (ft)	Tributary Height (ft)	Tributary Width (ft)	Tributary Area (ft^2)	Wind Load (PSF)	Wind Load (kip)	Shear (kip)	Moment (ft-kip)
41 - Roof	0	527	13	140	1820	30	54.6	54.6	0
39	26	501	6.5 + 14	140	910 + 1960	29 / 30	85.2	139.8	1419.6
38	15	486	14.5	140	2030	29	58.9	198.7	3516.6
37	14	472	14	140	1960	29	56.8	255.5	6297.98
36	14	458	13.5	140	1890	29	54.8	310.3	9875.12
35	13	445	13	140	1820	29	52.8	363.1	13909.28
34	13	432	13	140	1820	29	52.8	415.9	18629.58
33	13	419	13	140	1820	29	52.8	468.7	24036.02
32	13	406	0.5 + 12.5	140	70 + 1750	27 / 29	52.6	521.3	30128.6
31	13	393	13	140	1820	27	49.1	570.4	36904.98
30	13	380	13	140	1820	27	49.1	619.5	44320.18
29	13	367	13	140	1820	27	49.1	668.7	52374.2
28	13	354	13	140	1820	27	49.1	717.8	61067.04
27	13	341	13	140	1820	27	49.1	767.0	70398.7
26	13	328	13	140	1820	27	49.1	816.1	80369.18
25	13	315	13	140	1820	27	49.1	865.2	90978.48
24	13	302	4.5 + 8.5	140	630 + 1190	25 / 27	47.9	913.1	102226.6
23	13	289	13	140	1820	25	45.5	958.6	114097.4
22	13	276	13	140	1820	25	45.5	1004.1	126559.7
21	13	263	13	140	1820	25	45.5	1049.6	139613.6
20	13	250	13	140	1820	25	45.5	1095.1	153258.9
19	13	237	13	140	1820	25	45.5	1140.6	167495.7
18	13	224	13	140	1820	25	45.5	1186.1	182324
17	13	211	13	140	1820	25	45.5	1231.6	197743.8
16	13	198	8.5 + 4.5	140	1190 + 630	23 / 25	43.1	1274.7	213755.2
15	13	185	13	140	1820	23	41.9	1316.6	230326.8
14	13	172	13	140	1820	23	41.9	1358.5	247442.6
13	13	159	13	140	1820	23	41.9	1400.3	265102.6
12	13	146	13	140	1820	23	41.9	1442.2	283306.7
11	13	133	13	140	1820	23	41.9	1484.0	302055.1
10	13	120	13	140	1820	23	41.9	1525.9	321347.6
9	13	107	13	140	1820	23	41.9	1567.8	341184.3
8	13	94	13	140	1820	23	41.9	1609.6	361565.2
7	13	81	11.5	140	1610	23	37.0	1646.7	382490.2
6	10	71	10	140	1400	23	32.2	1678.9	398956.7
5	10	61	10	140	1400	23	32.2	1711.1	415745.2
4	10	51	10.5	140	1470	23	33.8	1744.9	432855.7
3	11	40	13.25	140	1855	23	42.7	1787.5	452049.2
2	15.5	24.5	20	140	2800	23	64.4	1851.9	479755.8

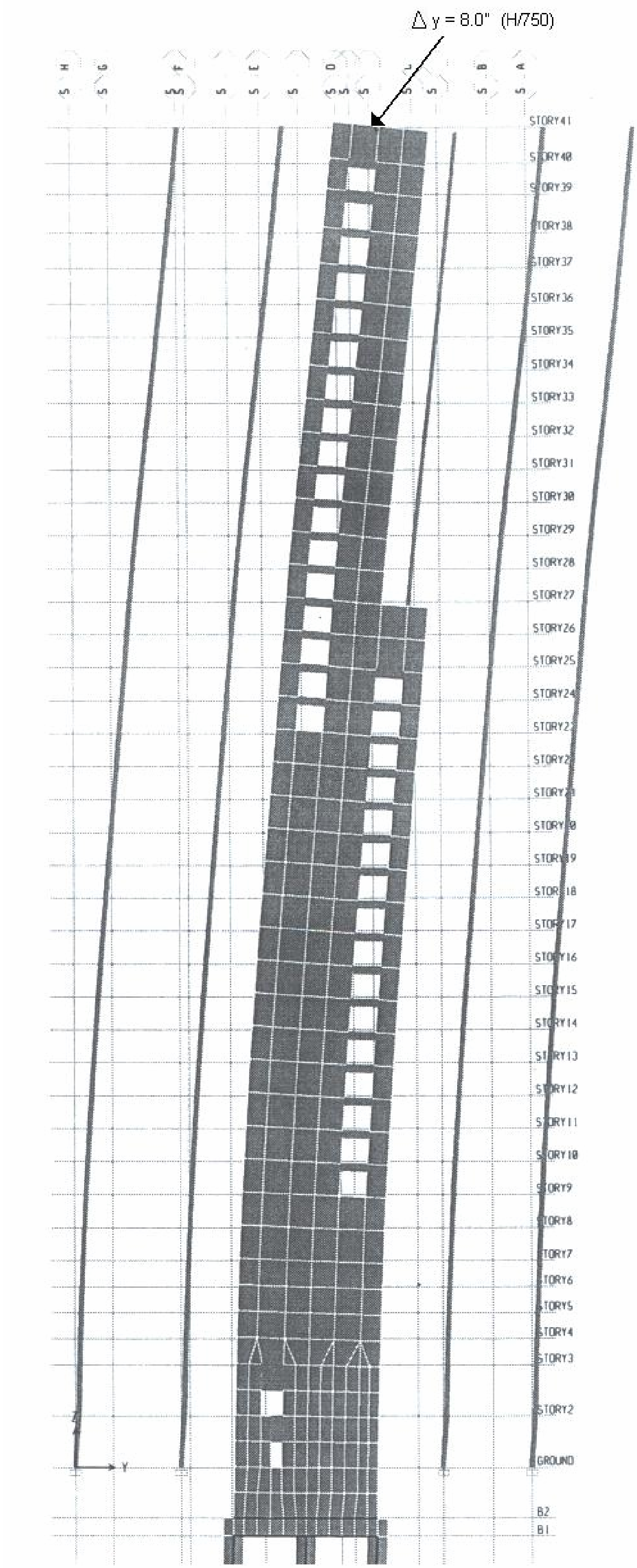
Wind Load Comparison

Story	Wind Tunnel			Chicago Code (Proposed)			Chicago Code (2001)		
	WX (kip)	WY(kip)	WT(k-ft)	WX (kip)	WY(kip)	WT(k-ft)	WX (kip)	WY(kip)	WT(k-ft)
STORY41	219.5	107.9	4809	320	250	3600	324	252	3641
STORY39	194.6	139.3	5765	108	84	1215	102	79	1145
STORY38	81.4	58.1	2342	70.5	54.8	788	68	53	765
STORY37	78.2	55.9	2295	73.1	56.8	788	71	55	794
STORY36	74.5	52.9	2207	70.5	54.8	788	68	53	765
STORY35	70.6	50.1	2121	67.9	52.8	788	66	51	737
STORY34	68.2	48.2	2044	67.9	52.8	788	66	51	737
STORY33	65.7	46.1	1967	67.9	52.8	788	66	51	737
STORY32	63.2	44.2	1873	67.9	52.8	788	66	51	737
STORY31	60.8	42.1	1796	63.2	49.1	711	59	46	658
STORY30	58.3	40.1	1685	63.2	49.1	711	59	46	658
STORY29	55.8	38	1592	63.2	49.1	711	59	46	658
STORY28	53.4	36.1	1498	63.2	49.1	711	59	46	658
STORY27	51.1	34.4	1404	63.2	49.1	711	59	46	658
STORY26	49.2	33	1316	63.2	49.1	711	59	46	658
STORY25	47.9	32.1	1279	63.2	49.1	711	59	46	658
STORY24	45.5	30.4	1216	63.2	49.1	711	59	46	658
STORY23	42.9	28.6	1171	58.5	45.5	658	49	38	553
STORY22	41	27.1	1111	58.5	45.5	658	49	38	553
STORY21	38.7	25.3	1066	58.5	45.5	658	49	38	553
STORY20	36.4	23.7	1005	58.5	45.5	658	49	38	553
STORY19	34.4	21.9	943	58.5	45.5	658	49	38	553
STORY18	32.1	20.2	882	58.5	45.5	658	49	38	553
STORY17	30	18.9	823	58.5	45.5	658	49	38	553
STORY16	28	18.9	781	53.8	41.9	605	47	36	527
STORY15	26	18.9	741	53.8	41.9	605	47	36	527
STORY14	24.7	18.9	710	53.8	41.9	605	47	36	527
STORY13	24.7	18.9	697	53.8	41.9	605	47	36	527
STORY12	24.7	18.9	684	53.8	41.9	605	47	36	527
STORY11	24.7	18.9	666	53.8	41.9	605	47	36	527
STORY10	24.7	18.9	642	53.8	41.9	605	47	36	527
STORY9	24.7	18.9	658	53.8	41.9	605	47	36	527
STORY8	24.7	18.9	581	53.8	41.9	605	47	36	527
STORY7	22.1	16.9	555	49.7	38.6	506	43	34	486
STORY6	19.5	14.9	502	43.5	33.8	506	38	29	425
STORY5	19.5	14.9	488	41.4	32.2	506	36	28	405
STORY4	19.5	14.9	464	41.4	32.2	506	36	28	405
STORY3	28.7	21.9	694	60	46.7	788	52	41	587
STORY2	38	29	822	82.8	64.4	788	72	56	810
STORY1	19	14.5	355						
	1987	**1352**	**54250**	**2632**	**2048**	**29670**	**2405**	**1870**	**27051**

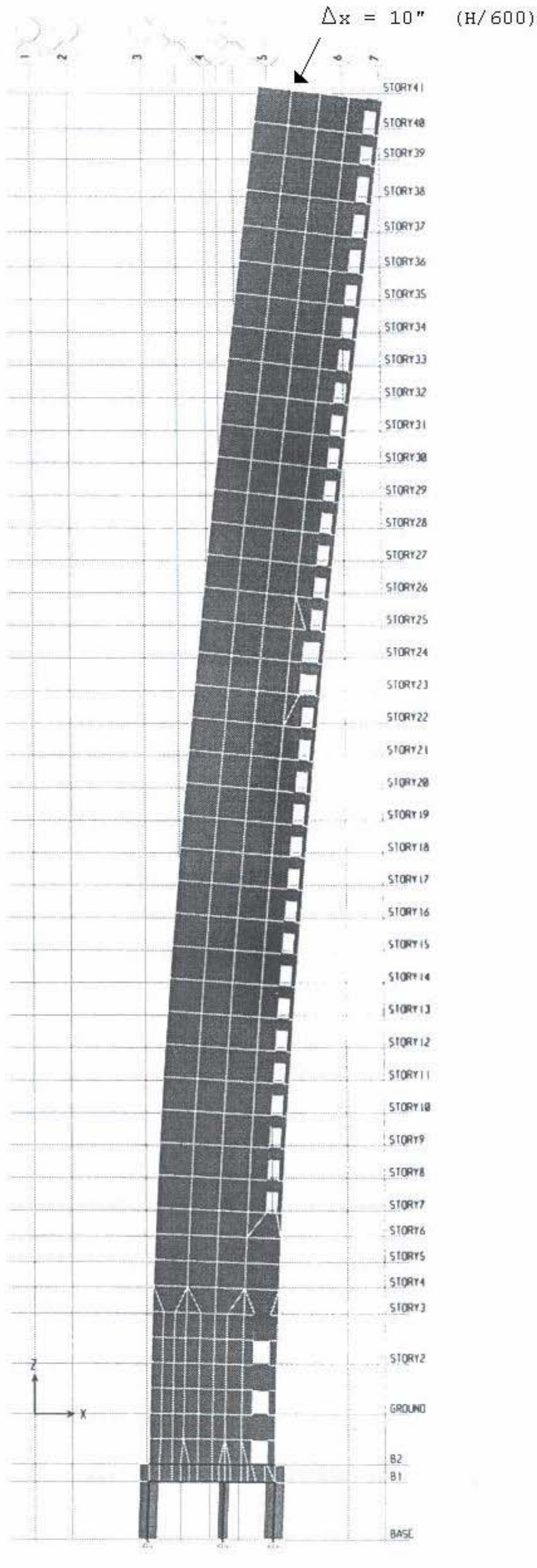
Seismic Loads

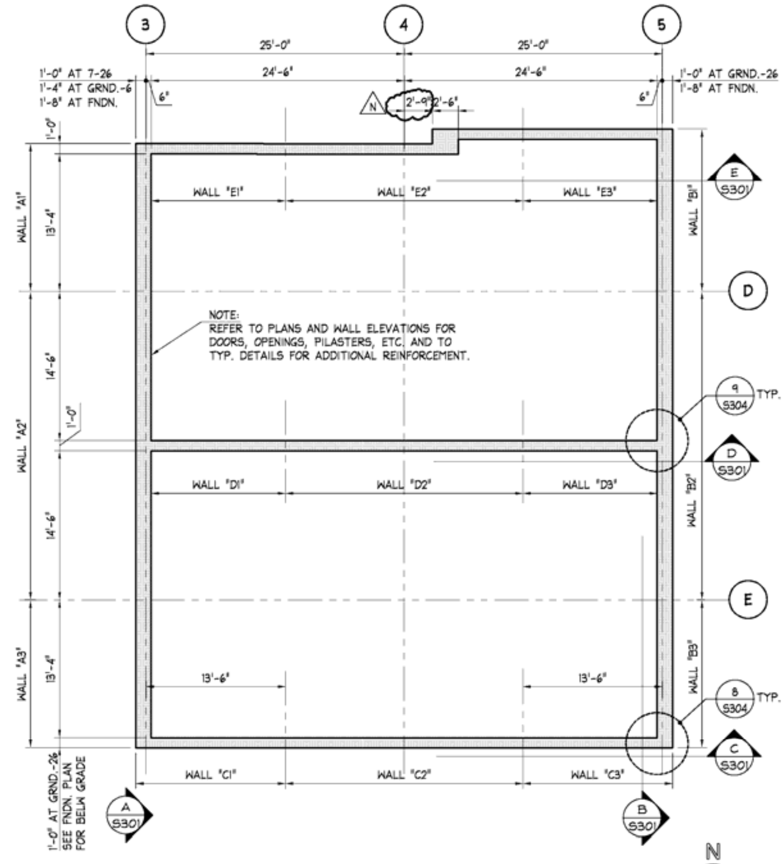
Level	Story Height (ft)	Elevation h_x (ft)	Tributary Height (ft)	Perimeter (ft)	Floor Area (ft²)	DL_{total} (psf)	Cladding Load (psf)	DL_{total} w_x (kip)	$DL_{total(cum)}$ (kip)	$w_x h_x^{1.85}$ (ft-k)	C_{vx}	Seismic Force F_x (kip)	Seismic Shear (kip)	Overturning Moment (ft-kip)
41 - Roof	0	527	13	640	25200	70	20	1930.4	1930.4	209409773	0.0787	85.67	85.67	0.00
39	26	501	20.5	640	25200	129	20	3513.2	5443.6	347058485	0.1304	141.98	227.65	5918.89
38	15	486	14.5	640	25200	59	20	1672.4	7116.0	156177065	0.0587	63.89	291.54	4373.12
37	14	472	14	640	25200	62	20	1741.6	8857.6	154078171	0.0579	63.03	354.57	4964.04
36	14	458	13.5	640	25200	59	20	1659.6	10517.2	138868761	0.0522	56.81	411.38	5759.39
35	13	445	13	640	25200	59	20	1653.2	12170.4	131156983	0.0493	53.66	465.04	6045.53
34	13	432	13	640	25200	59	20	1653.2	13823.6	124156748	0.0466	50.79	515.83	6705.83
33	13	419	13	640	25200	59	20	1653.2	15476.8	117333315	0.0441	48.00	563.83	7329.84
32	13	406	13	640	25200	59	20	1653.2	17130.0	110687495	0.0416	45.28	609.12	7918.50
31	13	393	13	640	25200	59	20	1653.2	18783.2	104220131	0.0391	42.64	651.75	8472.78
30	13	380	13	640	25200	59	20	1653.2	20436.4	97932096	0.0368	40.06	691.82	8993.60
29	13	367	13	640	25200	59	20	1653.2	22089.6	91824298	0.0345	37.57	729.38	9481.95
28	13	354	13	640	25200	59	20	1653.2	23742.8	85897679	0.0323	35.14	764.52	9938.78
27	13	341	13	640	25200	59	20	1653.2	25396.0	80153224	0.0301	32.79	797.31	10365.05
26	13	328	13	640	25200	59	20	1653.2	27049.2	74591958	0.0280	30.52	827.83	10761.75
25	13	315	13	640	25200	59	20	1653.2	28702.4	69214952	0.0260	28.32	856.14	11129.85
24	13	302	13	640	25200	59	20	1653.2	30355.6	64023327	0.0240	26.19	882.33	11470.35
23	13	289	13	640	25200	59	20	1653.2	32008.8	59018261	0.0222	24.14	906.48	11784.22
22	13	276	13	640	25200	59	20	1653.2	33662.0	54200987	0.0204	22.17	928.65	12072.48
21	13	263	13	640	25200	59	20	1653.2	35315.2	49572809	0.0186	20.28	948.93	12336.12
20	13	250	13	640	25200	59	20	1653.2	36968.4	45135100	0.0170	18.46	967.40	12576.16
19	13	237	13	640	25200	59	20	1653.2	38621.6	40889315	0.0154	16.73	984.12	12793.62
18	13	224	13	640	25200	59	20	1653.2	40274.8	36836998	0.0138	15.07	999.19	12989.53
17	13	211	13	640	25200	59	20	1653.2	41928.0	32979794	0.0124	13.49	1012.69	13164.92
16	13	198	13	640	25200	59	20	1653.2	43581.2	29319462	0.0110	11.99	1024.68	13320.85
15	13	185	13	640	25200	59	20	1653.2	45234.4	25857891	0.0097	10.58	1035.26	13458.37
14	13	172	13	640	25200	59	20	1653.2	46887.6	22597116	0.0085	9.24	1044.50	13578.55
13	13	159	13	640	25200	59	20	1653.2	48540.8	19539348	0.0073	7.99	1052.50	13682.46
12	13	146	13	640	25200	59	20	1653.2	50194.0	16686997	0.0063	6.83	1059.32	13771.21
11	13	133	13	640	25200	59	20	1653.2	51847.2	14042710	0.0053	5.74	1065.07	13845.89
10	13	120	13	640	25200	59	20	1653.2	53500.4	11609425	0.0044	4.75	1069.82	13907.63
9	13	107	13	640	25200	59	20	1653.2	55153.6	9390428	0.0035	3.84	1073.66	13957.58
8	13	94	13	640	25200	59	20	1653.2	56806.8	7389447	0.0028	3.02	1076.68	13996.87
7	13	81	11.5	640	25200	105	20	2793.2	59600.0	9479803	0.0036	3.88	1080.56	14047.29
6	10	71	10	640	25200	84	90	2692.8	62292.8	7161963	0.0027	2.93	1083.49	10834.91
5	10	61	10	640	25200	84	90	2692.8	64985.6	5408344	0.0020	2.21	1085.70	10857.03
4	10	51	10.5	640	25200	84	90	2721.6	67707.2	3924905	0.0015	1.61	1087.31	10873.09
3	11	40	13.25	640	25200	105	90	3409.2	71116.4	3136631	0.0012	1.28	1088.59	11974.51
2	15.5	24.5	20	640	25200	113	90	3999.6	75116.0	1485846	0.0006	0.61	1089.20	16882.60

ETABS W-E Drift



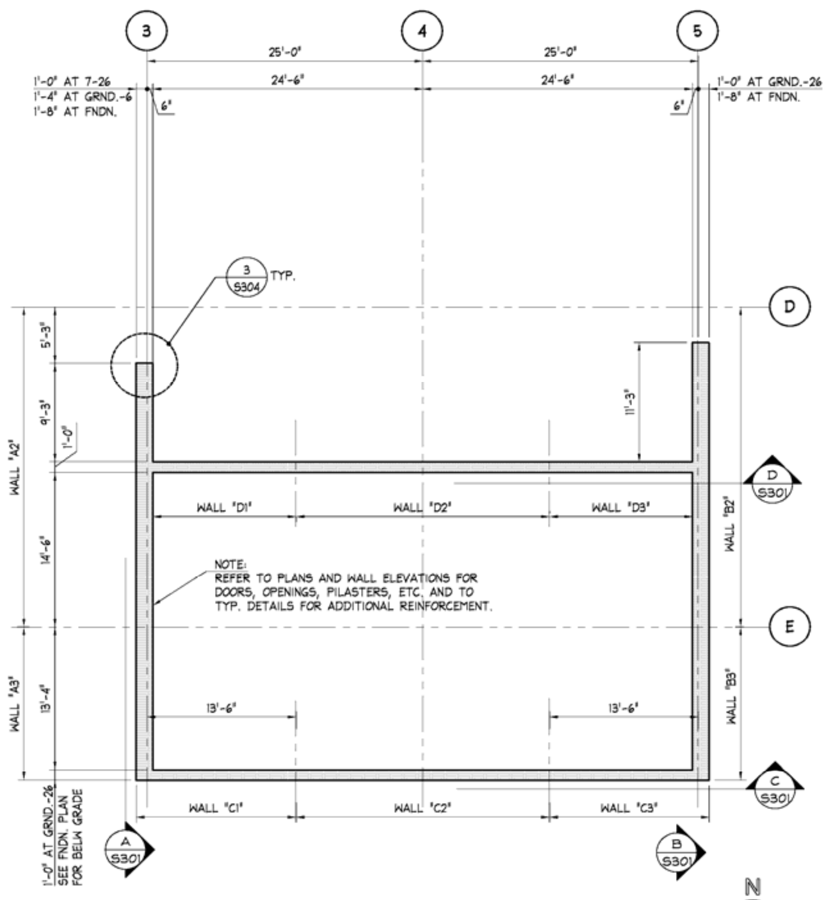
ETABS N-S Drift





1 SHEAR WALL KEY PLAN - FOUNDATION THRU LEVEL 26

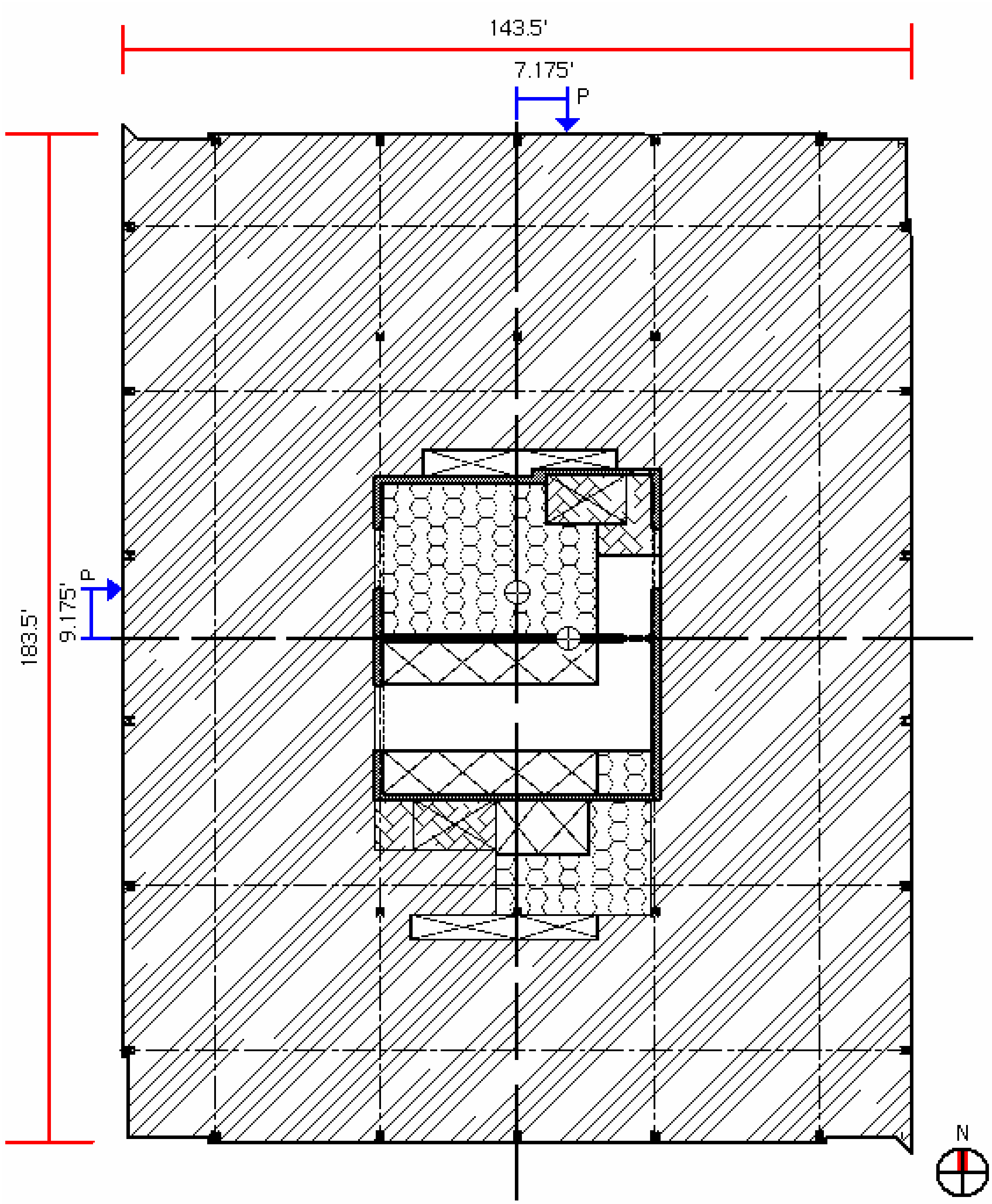
SCALE: 1/8"=1'-0"



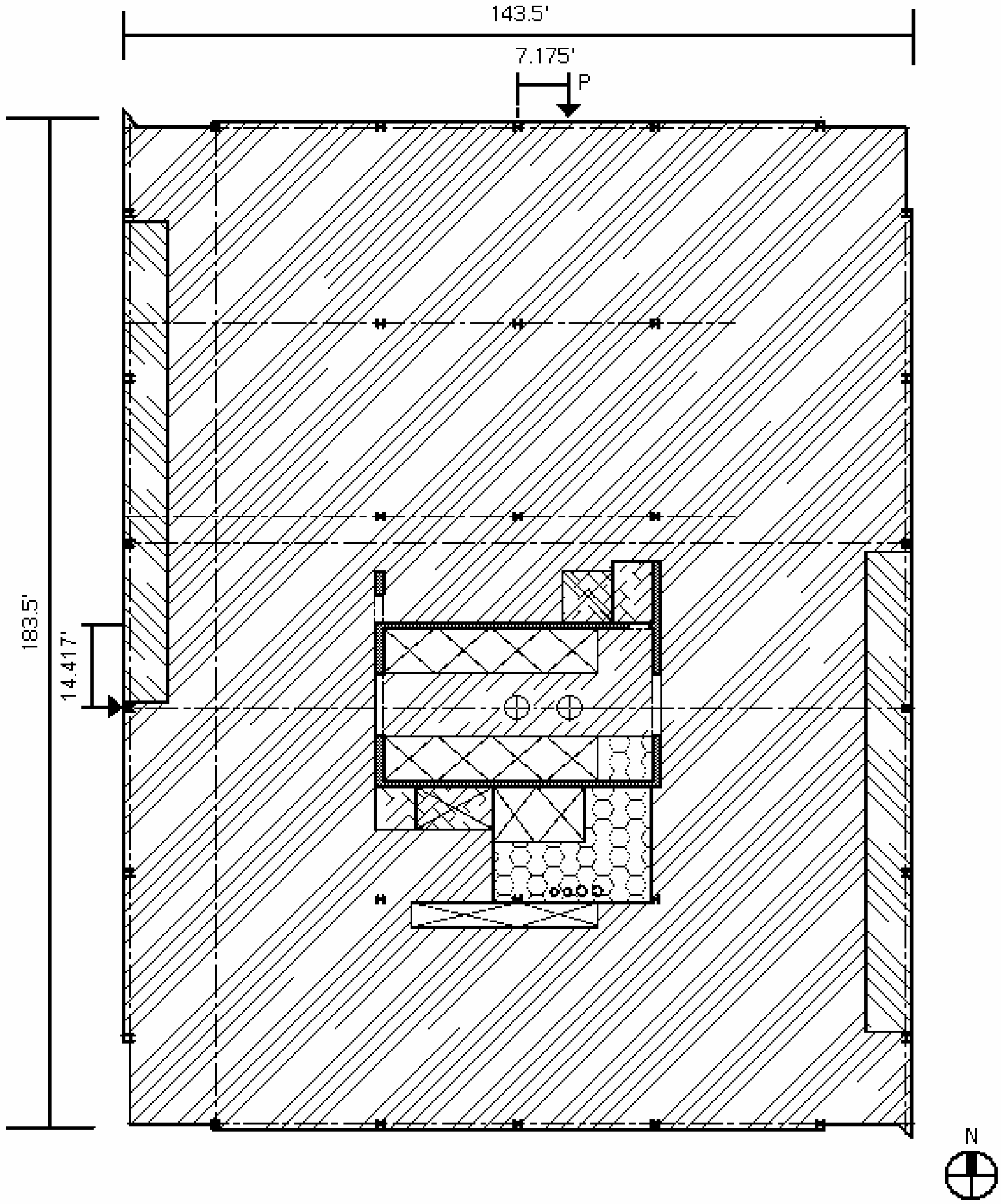
2 SHEAR WALL KEY PLAN - LEVELS 27 THRU P.H. ROOF

SCALE: 1/8"=1'-0"

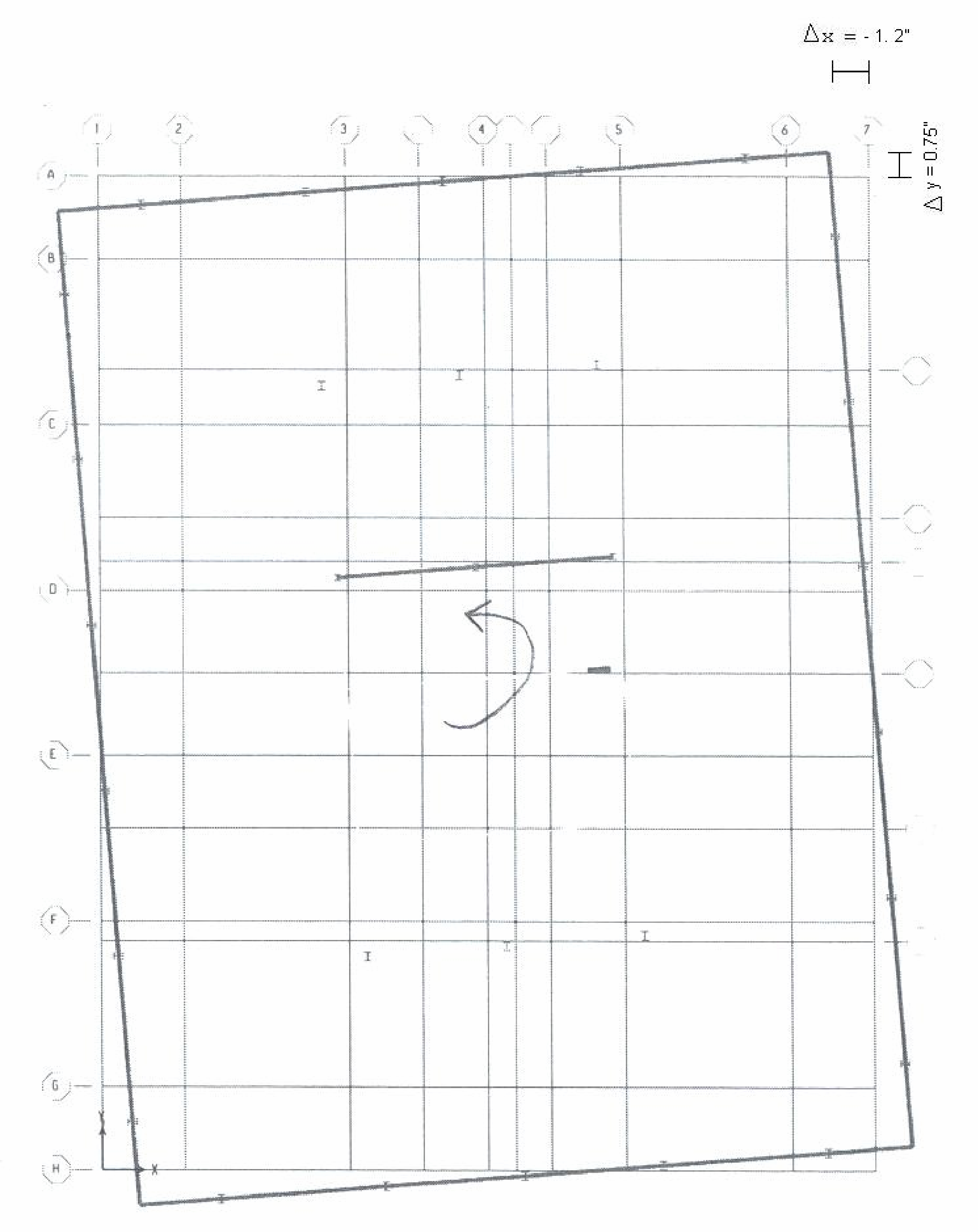
Center of Rigidity for Levels 1 - 26



Center of Rigidity for Levels 27-41



ETABS Torsional Moment



Torsional Moments for Wind Loads

Story	5% Offset (ft)	W-E Wind Load (kip)	5% Offset (ft)	N-S Wind Load (kip)	W-E Torsional Moment (ft-kip)	N-S Torsional Moment (ft-kip)
41	14.417	70.20	7.175	54.60	-1012.07	-391.76
39	14.417	109.50	7.175	85.20	-1578.66	-611.31
38	14.417	75.69	7.175	58.87	-1091.22	-422.39
37	14.417	73.08	7.175	56.84	-1053.59	-407.83
36	14.417	70.47	7.175	54.81	-1015.97	-393.26
35	14.417	67.86	7.175	52.78	-978.34	-378.70
34	14.417	67.86	7.175	52.78	-978.34	-378.70
33	14.417	67.86	7.175	52.78	-978.34	-378.70
32	14.417	67.70	7.175	52.60	-976.03	-377.41
31	14.417	63.18	7.175	49.14	-910.87	-352.58
30	14.417	63.18	7.175	49.14	-910.87	-352.58
29	14.417	63.18	7.175	49.14	-910.87	-352.58
28	14.417	63.18	7.175	49.14	-910.87	-352.58
27	14.417	63.18	7.175	49.14	-910.87	-352.58
26	9.175	63.18	7.175	49.14	-579.68	-352.58
25	9.175	63.18	7.175	49.14	-579.68	-352.58
24	9.175	61.60	7.175	47.90	-565.18	-343.68
23	9.175	58.50	7.175	45.50	-536.74	-326.46
22	9.175	58.50	7.175	45.50	-536.74	-326.46
21	9.175	58.50	7.175	45.50	-536.74	-326.46
20	9.175	58.50	7.175	45.50	-536.74	-326.46
19	9.175	58.50	7.175	45.50	-536.74	-326.46
18	9.175	58.50	7.175	45.50	-536.74	-326.46
17	9.175	58.50	7.175	45.50	-536.74	-326.46
16	9.175	55.40	7.175	43.10	-508.30	-309.24
15	9.175	53.82	7.175	41.86	-493.80	-300.35
14	9.175	53.82	7.175	41.86	-493.80	-300.35
13	9.175	53.82	7.175	41.86	-493.80	-300.35
12	9.175	53.82	7.175	41.86	-493.80	-300.35
11	9.175	53.82	7.175	41.86	-493.80	-300.35
10	9.175	53.82	7.175	41.86	-493.80	-300.35
9	9.175	53.82	7.175	41.86	-493.80	-300.35
8	9.175	53.82	7.175	41.86	-493.80	-300.35
7	9.175	47.61	7.175	37.03	-436.82	-265.69
6	9.175	41.40	7.175	32.20	-379.85	-231.04
5	9.175	41.40	7.175	32.20	-379.85	-231.04
4	9.175	43.47	7.175	33.81	-398.84	-242.59
3	9.175	54.85	7.175	42.67	-503.25	-306.16
2	9.175	82.80	7.175	64.40	-759.69	-462.07

Torsional Moments for Seismic Loads

Story	5% Offset (ft)	W-E Seismic Load (kip)	5% Offset (ft)	N-S Seismic Load (kip)	W-E Torsional Moment (ft-kip)	N-S Torsional Moment (ft-kip)
41	14.417	85.67	7.175	85.67	-1235.09	-614.67
39	14.417	141.98	7.175	141.98	-2046.94	-1018.71
38	14.417	63.89	7.175	63.89	-921.13	-458.42
37	14.417	63.03	7.175	63.03	-908.75	-452.26
36	14.417	56.81	7.175	56.81	-819.04	-407.62
35	14.417	53.66	7.175	53.66	-773.56	-384.98
34	14.417	50.79	7.175	50.79	-732.27	-364.43
33	14.417	48.00	7.175	48.00	-692.03	-344.41
32	14.417	45.28	7.175	45.28	-652.83	-324.90
31	14.417	42.64	7.175	42.64	-614.69	-305.91
30	14.417	40.06	7.175	40.06	-577.60	-287.46
29	14.417	37.57	7.175	37.57	-541.58	-269.53
28	14.417	35.14	7.175	35.14	-506.62	-252.13
27	14.417	32.79	7.175	32.79	-472.74	-235.27
26	9.175	30.52	7.175	30.52	-279.98	-218.95
25	9.175	28.32	7.175	28.32	-259.80	-203.16
24	9.175	26.19	7.175	26.19	-240.31	-187.93
23	9.175	24.14	7.175	24.14	-221.52	-173.23
22	9.175	22.17	7.175	22.17	-203.44	-159.09
21	9.175	20.28	7.175	20.28	-186.07	-145.51
20	9.175	18.46	7.175	18.46	-169.41	-132.48
19	9.175	16.73	7.175	16.73	-153.48	-120.02
18	9.175	15.07	7.175	15.07	-138.27	-108.13
17	9.175	13.49	7.175	13.49	-123.79	-96.80
16	9.175	11.99	7.175	11.99	-110.05	-86.06
15	9.175	10.58	7.175	10.58	-97.06	-75.90
14	9.175	9.24	7.175	9.24	-84.82	-66.33
13	9.175	7.99	7.175	7.99	-73.34	-57.35
12	9.175	6.83	7.175	6.83	-62.63	-48.98
11	9.175	5.74	7.175	5.74	-52.71	-41.22
10	9.175	4.75	7.175	4.75	-43.58	-34.08
9	9.175	3.84	7.175	3.84	-35.25	-27.56
8	9.175	3.02	7.175	3.02	-27.74	-21.69
7	9.175	3.88	7.175	3.88	-35.58	-27.83
6	9.175	2.93	7.175	2.93	-26.88	-21.02
5	9.175	2.21	7.175	2.21	-20.30	-15.87
4	9.175	1.61	7.175	1.61	-14.73	-11.52
3	9.175	1.28	7.175	1.28	-11.77	-9.21
2	9.175	0.61	7.175	0.61	-5.58	-4.36

Shear Strength of W-E Shear Walls

Story	Vert E. F.	Av (in²)	fc (psi)	h (in)	d (in)	S (in)	fy (psi)	Vc (kip)	Vs (kip)	Vn (kip)	ΦVn (kip)
41	No. 4	0.20	5000	12	618	12	60000	1048.78	618	1666.78	1416.76
39	No. 4	0.20	5000	12	618	12	60000	1048.78	618	1666.78	1416.76
38	No. 4	0.20	5000	12	618	12	60000	1048.78	618	1666.78	1416.76
37	No. 4	0.20	5000	12	618	12	60000	1048.78	618	1666.78	1416.76
36	No. 4	0.20	5000	12	618	12	60000	1048.78	618	1666.78	1416.76
35	No. 4	0.20	5000	12	618	12	60000	1048.78	618	1666.78	1416.76
34	No. 4	0.20	5000	12	618	12	60000	1048.78	618	1666.78	1416.76
33	No. 4	0.20	5000	12	618	12	60000	1048.78	618	1666.78	1416.76
32	No. 4	0.20	5000	12	618	12	60000	1048.78	618	1666.78	1416.76
31	No. 4	0.20	5000	12	618	12	60000	1048.78	618	1666.78	1416.76
30	No. 4	0.20	6000	12	618	12	60000	1148.88	618	1766.88	1501.85
29	No. 4	0.20	6000	12	618	12	60000	1148.88	618	1766.88	1501.85
28	No. 4	0.20	6000	12	618	12	60000	1148.88	618	1766.88	1501.85
27	No. 4	0.20	6000	12	618	12	60000	1148.88	618	1766.88	1501.85
26	No. 4	0.20	6000	12	618	12	60000	1148.88	618	1766.88	1501.85
25	No. 4	0.20	6000	12	618	12	60000	1148.88	618	1766.88	1501.85
24	No. 4	0.20	6000	12	618	12	60000	1148.88	618	1766.88	1501.85
23	No. 4	0.20	6000	12	618	12	60000	1148.88	618	1766.88	1501.85
22	No. 4	0.20	6000	12	618	12	60000	1148.88	618	1766.88	1501.85
21	No. 4	0.20	6000	12	618	12	60000	1148.88	618	1766.88	1501.85
20	No. 4	0.20	6000	12	618	12	60000	1148.88	618	1766.88	1501.85
19	No. 4	0.20	6000	12	618	12	60000	1148.88	618	1766.88	1501.85
18	No. 4	0.20	6000	12	618	12	60000	1148.88	618	1766.88	1501.85
17	No. 4	0.20	6000	12	618	12	60000	1148.88	618	1766.88	1501.85
16	No. 5	0.31	6000	12	618	12	60000	1148.88	958	2106.78	1790.76
15	No. 5	0.31	8000	12	618	12	60000	1326.61	958	2284.51	1941.84
14	No. 6	0.44	8000	12	618	12	60000	1326.61	1360	2686.21	2283.28
13	No. 6	0.44	8000	12	618	12	60000	1326.61	1360	2686.21	2283.28
12	No. 6	0.44	8000	12	618	12	60000	1326.61	1360	2686.21	2283.28
11	No. 6	0.44	8000	12	618	12	60000	1326.61	1360	2686.21	2283.28
10	No. 7	0.60	8000	12	618	12	60000	1326.61	1854	3180.61	2703.52
9	No. 7	0.60	8000	12	618	12	60000	1326.61	1854	3180.61	2703.52
8	No. 7	0.60	8000	12	618	12	60000	1326.61	1854	3180.61	2703.52
7	No. 7	0.60	8000	12	618	12	60000	1326.61	1854	3180.61	2703.52
6	No. 7	0.60	8000	12	618	12	60000	1326.61	1854	3180.61	2703.52
5	No. 7	0.60	8000	12	618	12	60000	1326.61	1854	3180.61	2703.52
4	No. 7	0.60	8000	12	618	12	60000	1326.61	1854	3180.61	2703.52
3	No. 7	0.60	8000	12	618	12	60000	1326.61	1854	3180.61	2703.52
2	No. 7	0.60	8000	12	618	12	60000	1326.61	1854	3180.61	2703.52

Shear Strength of N-S Shear Wall 1

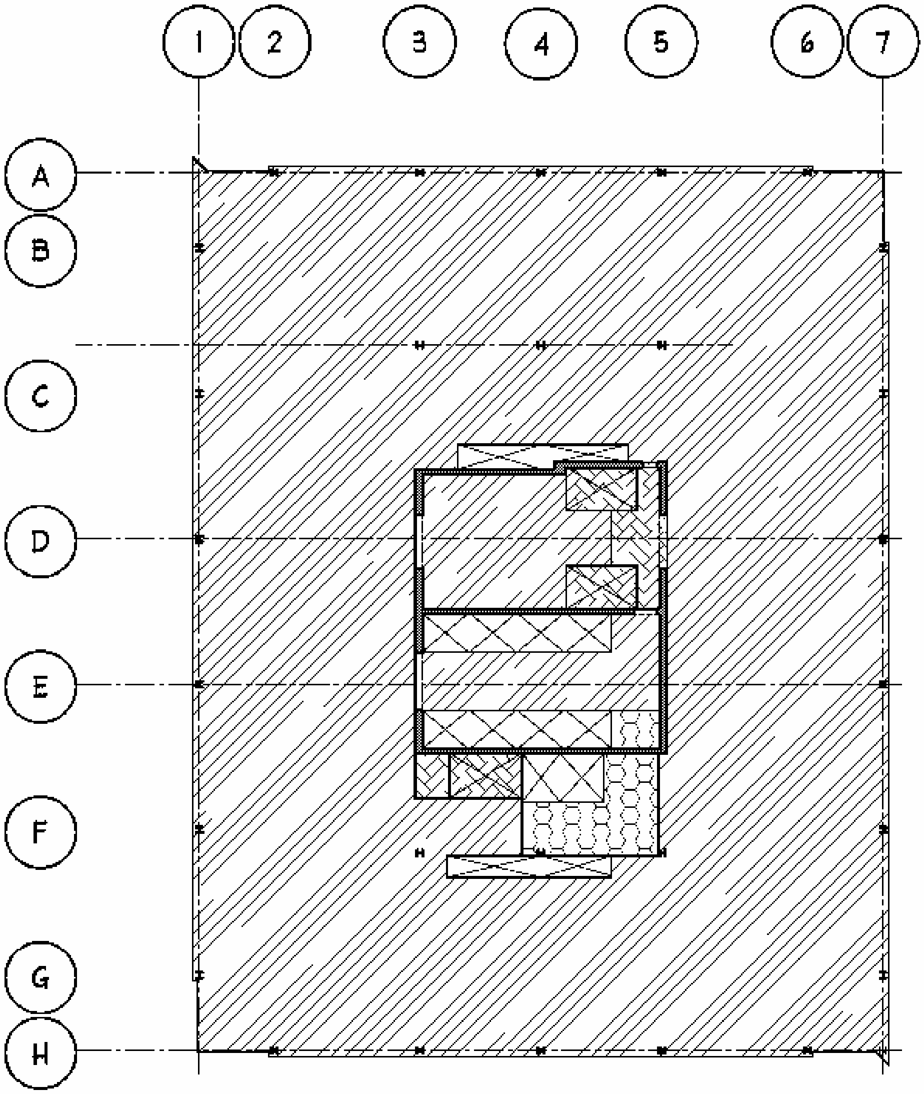
Story	Vert E. F.	Av (in²)	fc (psi)	h (in)	d (in)	S (in)	fy (psi)	Vc (kip)	Vs (kip)	Vn (kip)	ΦVn (kip)
41	No. 5	0.31	5000	18	352	12	60000	896.05	545.60	1441.65	1225.40
39	No. 5	0.31	5000	18	487	12	60000	1239.70	754.85	1994.55	1695.37
38	No. 5	0.31	5000	18	487	12	60000	1239.70	754.85	1994.55	1695.37
37	No. 5	0.31	5000	18	487	12	60000	1239.70	754.85	1994.55	1695.37
36	No. 5	0.31	5000	18	487	12	60000	1239.70	754.85	1994.55	1695.37
35	No. 5	0.31	5000	18	487	12	60000	1239.70	754.85	1994.55	1695.37
34	No. 5	0.31	5000	18	487	12	60000	1239.70	754.85	1994.55	1695.37
33	No. 5	0.31	5000	18	487	12	60000	1239.70	754.85	1994.55	1695.37
32	No. 5	0.31	5000	18	487	12	60000	1239.70	754.85	1994.55	1695.37
31	No. 5	0.31	5000	18	487	12	60000	1239.70	754.85	1994.55	1695.37
30	No. 5	0.31	6000	18	487	12	60000	1358.02	754.85	2112.87	1795.94
29	No. 6	0.44	6000	18	487	12	60000	1358.02	1071.40	2429.42	2065.01
28	No. 6	0.44	6000	18	487	12	60000	1358.02	1071.40	2429.42	2065.01
27	No. 6	0.44	6000	18	487	12	60000	1358.02	1071.40	2429.42	2065.01
26	No. 7	0.60	6000	18	698	12	60000	1946.41	2094.00	4040.41	3434.35
25	No. 6	0.44	6000	18	698	12	60000	1946.41	1535.60	3482.01	2959.71
24	No. 6	0.44	6000	18	698	12	60000	1946.41	1535.60	3482.01	2959.71
23	No. 6	0.44	6000	18	698	12	60000	1946.41	1535.60	3482.01	2959.71
22	No. 6	0.44	6000	18	698	12	60000	1946.41	1535.60	3482.01	2959.71
21	No. 6	0.44	6000	18	698	12	60000	1946.41	1535.60	3482.01	2959.71
20	No. 6	0.44	6000	18	698	12	60000	1946.41	1535.60	3482.01	2959.71
19	No. 7	0.60	6000	18	698	12	60000	1946.41	2094.00	4040.41	3434.35
18	No. 7	0.60	6000	18	698	12	60000	1946.41	2094.00	4040.41	3434.35
17	No. 7	0.60	6000	18	698	12	60000	1946.41	2094.00	4040.41	3434.35
16	No. 7	0.60	6000	18	698	12	60000	1946.41	2094.00	4040.41	3434.35
15	No. 8	0.78	8000	18	698	12	60000	2247.52	2722.20	4969.72	4224.26
14	No. 8	0.78	8000	18	698	12	60000	2247.52	2722.20	4969.72	4224.26
13	No. 9	1.00	8000	18	698	12	60000	2247.52	3490.00	5737.52	4876.89
12	No. 9	1.00	8000	18	698	12	60000	2247.52	3490.00	5737.52	4876.89
11	No. 9	1.00	8000	18	698	12	60000	2247.52	3490.00	5737.52	4876.89
10	No. 10	1.27	8000	18	698	12	60000	2247.52	4432.30	6679.82	5677.84
9	No. 10	1.27	8000	18	698	12	60000	2247.52	4432.30	6679.82	5677.84
8	No. 10	1.27	8000	18	698	12	60000	2247.52	4432.30	6679.82	5677.84
7	No. 10	1.27	8000	18	698	12	60000	2247.52	4432.30	6679.82	5677.84
6	No. 10	1.27	8000	22	698	12	60000	2746.96	4432.30	7179.26	6102.38
5	No. 10	1.27	8000	22	698	12	60000	2746.96	4432.30	7179.26	6102.38
4	No. 10	1.27	8000	22	698	12	60000	2746.96	4432.30	7179.26	6102.38
3	No. 11	1.56	8000	22	698	12	60000	2746.96	5444.40	8191.36	6962.66
2	No. 11	1.56	8000	22	698	12	60000	2746.96	5444.40	8191.36	6962.66

Shear Strength of N-S Shear Wall II

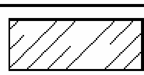
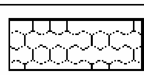
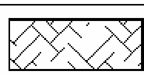
Story	Vert E. F.	Av (in²)	f'c (psi)	h (in)	d (in)	S (in)	fy (psi)	Vc (kip)	Vs (kip)	Vn (kip)	ΦVn (kip)
41	No. 5	0.31	5000	18	352	12	60000	896.05	545.60	1441.65	1225.40
39	No. 5	0.31	5000	18	487	12	60000	1239.70	754.85	1994.55	1695.37
38	No. 5	0.31	5000	18	487	12	60000	1239.70	754.85	1994.55	1695.37
37	No. 5	0.31	5000	18	487	12	60000	1239.70	754.85	1994.55	1695.37
36	No. 5	0.31	5000	18	487	12	60000	1239.70	754.85	1994.55	1695.37
35	No. 5	0.31	5000	18	487	12	60000	1239.70	754.85	1994.55	1695.37
34	No. 5	0.31	5000	18	487	12	60000	1239.70	754.85	1994.55	1695.37
33	No. 5	0.31	5000	18	487	12	60000	1239.70	754.85	1994.55	1695.37
32	No. 5	0.31	5000	18	487	12	60000	1239.70	754.85	1994.55	1695.37
31	No. 5	0.31	5000	18	487	12	60000	1239.70	754.85	1994.55	1695.37
30	No. 5	0.31	6000	18	487	12	60000	1358.02	754.85	2112.87	1795.94
29	No. 5	0.31	6000	18	487	12	60000	1358.02	754.85	2112.87	1795.94
28	No. 5	0.31	6000	18	487	12	60000	1358.02	754.85	2112.87	1795.94
27	No. 5	0.31	6000	18	487	12	60000	1358.02	754.85	2112.87	1795.94
26	No. 5	0.31	6000	18	698	12	60000	1946.41	1081.90	3028.31	2574.06
25	No. 5	0.31	6000	18	698	12	60000	1946.41	1081.90	3028.31	2574.06
24	No. 5	0.31	6000	18	698	12	60000	1946.41	1081.90	3028.31	2574.06
23	No. 5	0.31	6000	18	698	12	60000	1946.41	1081.90	3028.31	2574.06
22	No. 5	0.31	6000	18	698	12	60000	1946.41	1081.90	3028.31	2574.06
21	No. 5	0.31	6000	18	698	12	60000	1946.41	1081.90	3028.31	2574.06
20	No. 5	0.31	6000	18	698	12	60000	1946.41	1081.90	3028.31	2574.06
19	No. 5	0.31	6000	18	698	12	60000	1946.41	1081.90	3028.31	2574.06
18	No. 5	0.31	6000	18	698	12	60000	1946.41	1081.90	3028.31	2574.06
17	No. 6	0.44	6000	18	698	12	60000	1946.41	1535.60	3482.01	2959.71
16	No. 6	0.44	6000	18	698	12	60000	1946.41	1535.60	3482.01	2959.71
15	No. 6	0.44	8000	18	698	12	60000	2247.52	1535.60	3783.12	3215.65
14	No. 7	0.60	8000	18	698	12	60000	2247.52	2094.00	4341.52	3690.29
13	No. 7	0.60	8000	18	698	12	60000	2247.52	2094.00	4341.52	3690.29
12	No. 8	0.78	8000	18	698	12	60000	2247.52	2722.20	4969.72	4224.26
11	No. 8	0.78	8000	18	698	12	60000	2247.52	2722.20	4969.72	4224.26
10	No. 9	1.00	8000	18	698	12	60000	2247.52	3490.00	5737.52	4876.89
9	No. 9	1.00	8000	18	698	12	60000	2247.52	3490.00	5737.52	4876.89
8	No. 8	0.78	8000	18	698	12	60000	2247.52	2722.20	4969.72	4224.26
7	No. 8	0.78	8000	18	698	12	60000	2247.52	2722.20	4969.72	4224.26
6	No. 8	0.78	8000	18	698	12	60000	2247.52	2722.20	4969.72	4224.26
5	No. 8	0.78	8000	18	698	12	60000	2247.52	2722.20	4969.72	4224.26
4	No. 8	0.78	8000	18	698	12	60000	2247.52	2722.20	4969.72	4224.26
3	No. 10	1.27	8000	18	698	12	60000	2247.52	4432.30	6679.82	5677.84
2	No. 10	1.27	8000	18	698	12	60000	2247.52	4432.30	6679.82	5677.84

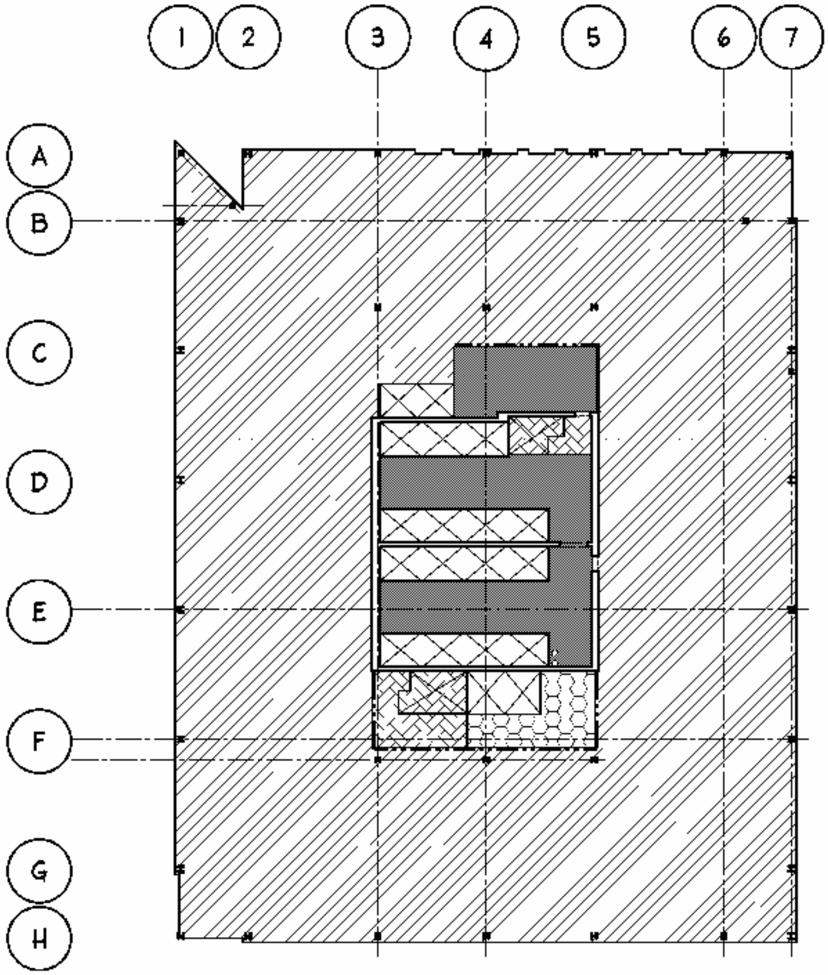
Design Gravity Dead Loads

Level	Total Slab Depth (in)	Steel Deck Depth (in)	Concrete Weight (pcf)	Steel Average Size (plf)	Concrete Load (psf)	Steel Deck (psf)	Steel (psf)	MEP, Ceiling, etc. (psf)	DL_{total} (psf)
41 - Roof	9	3	145	(See pg 19 - for gravity design loads)				70	70
39	11	3	145	46	117	2	5	5	129
38	6.25	3	115	46	47	2	5	5	59
37	6.25	3	115	76	47	2	8	5	62
36	6.25	3	115	46	47	2	5	5	59
35	6.25	3	115	46	47	2	5	5	59
34	6.25	3	115	46	47	2	5	5	59
33	6.25	3	115	46	47	2	5	5	59
32	6.25	3	115	46	47	2	5	5	59
31	6.25	3	115	46	47	2	5	5	59
30	6.25	3	115	46	47	2	5	5	59
29	6.25	3	115	46	47	2	5	5	59
28	6.25	3	115	46	47	2	5	5	59
27	6.25	3	115	46	47	2	5	5	59
26	6.25	3	115	46	47	2	5	5	59
25	6.25	3	115	46	47	2	5	5	59
24	6.25	3	115	46	47	2	5	5	59
23	6.25	3	115	46	47	2	5	5	59
22	6.25	3	115	46	47	2	5	5	59
21	6.25	3	115	46	47	2	5	5	59
20	6.25	3	115	46	47	2	5	5	59
19	6.25	3	115	46	47	2	5	5	59
18	6.25	3	115	46	47	2	5	5	59
17	6.25	3	115	46	47	2	5	5	59
16	6.25	3	115	46	47	2	5	5	59
15	6.25	3	115	46	47	2	5	5	59
14	6.25	3	115	46	47	2	5	5	59
13	6.25	3	115	46	47	2	5	5	59
12	6.25	3	115	46	47	2	5	5	59
11	6.25	3	115	46	47	2	5	5	59
10	6.25	3	115	46	47	2	5	5	59
9	6.25	3	115	46	47	2	5	5	59
8	6.25	3	115	46	47	2	5	5	59
7	9	3	145	50	93	2	5	5	105
6	7.25	3	145	46	72	2	5	5	84
5	7.25	3	145	46	72	2	5	5	84
4	7.25	3	145	46	72	2	5	5	84
3	9	3	145	46	93	2	5	5	105
2	9	3	145	76	93	2	8	10	113



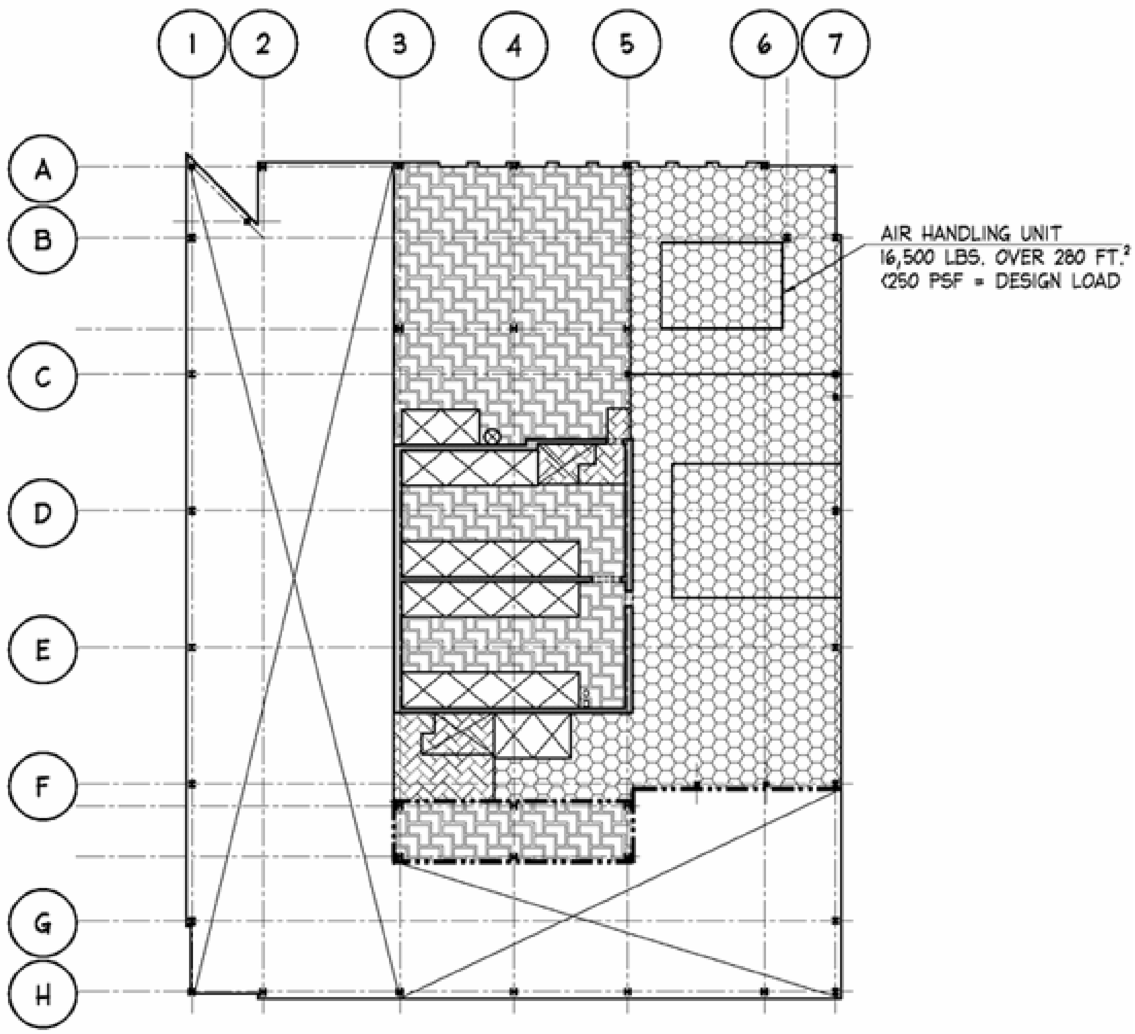
Typical Office Loading Diagram

SUPERIMPOSED LOAD TABLE			
MARK	AREAS	SDL (PSF)	LL (PSF)
	TYPICAL OFFICE (LOBBY, RESTROOMS AND FITNESS)	5	100
	HEAVY MECHANICAL	5	250
	STAIR	---	100



Parking Loading Diagram

SUPERIMPOSED LOAD TABLE			
MARK	AREAS	SDL (PSF)	LL (PSF)
	PARKING	5	50
	HEAVY MECHANICAL	5	250
	LIGHT STORAGE	5	125
	EXIT STAIR	---	100
	PARTITION	65 PSF x H	



LEVEL 2 LOADING DIAGRAM

SUPERIMPOSED LOAD TABLE			
MARK	AREAS	SDL (PSF)	LL (PSF)
	HEAVY MECHANICAL	5	250
	LIGHT STORAGE	5	150
	EXIT STAIR	---	100
	PARTITION	65 PSF x H	

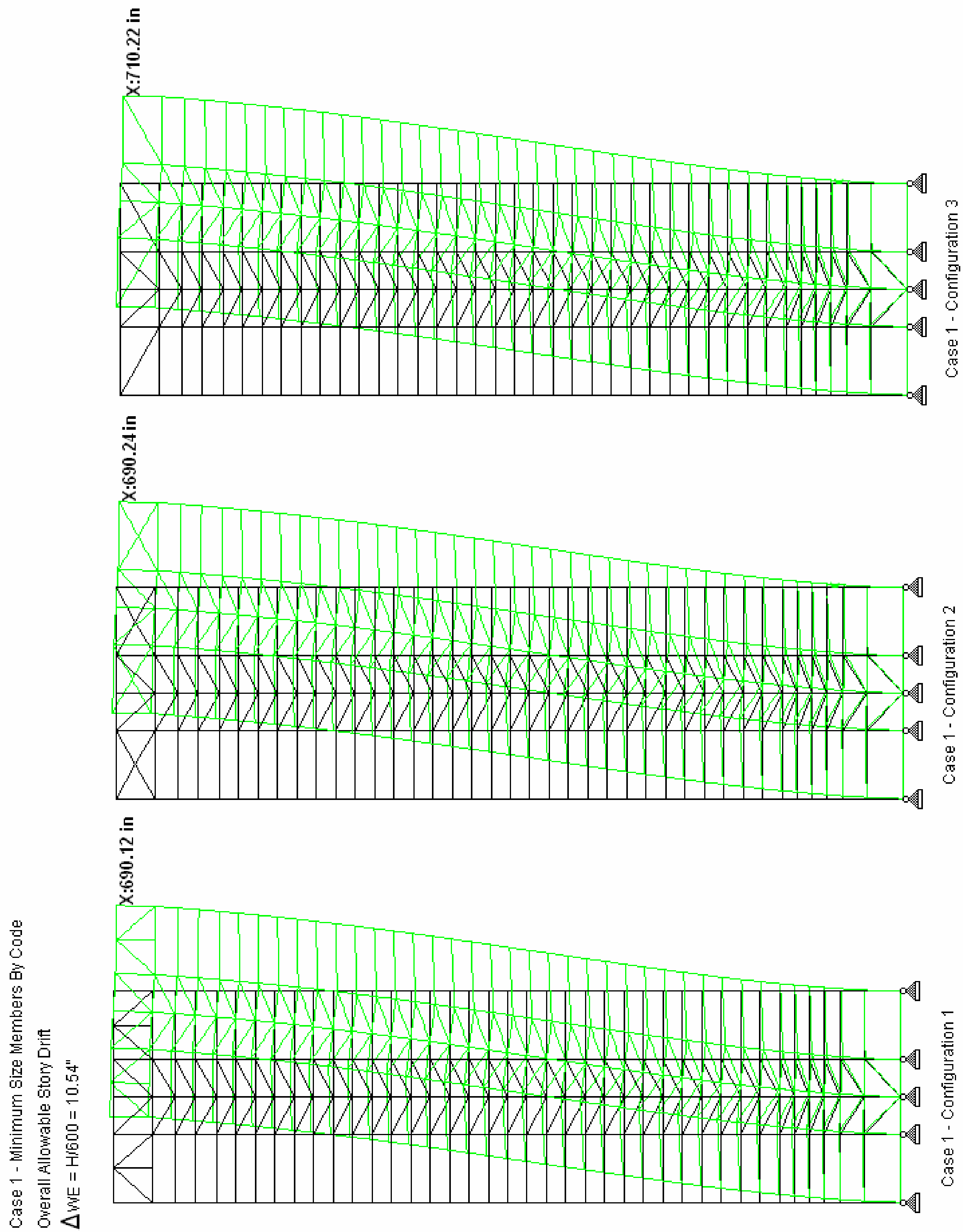
APPENDIX B

BRACED STEEL CORE & OUTRIGGERS ANALYSIS

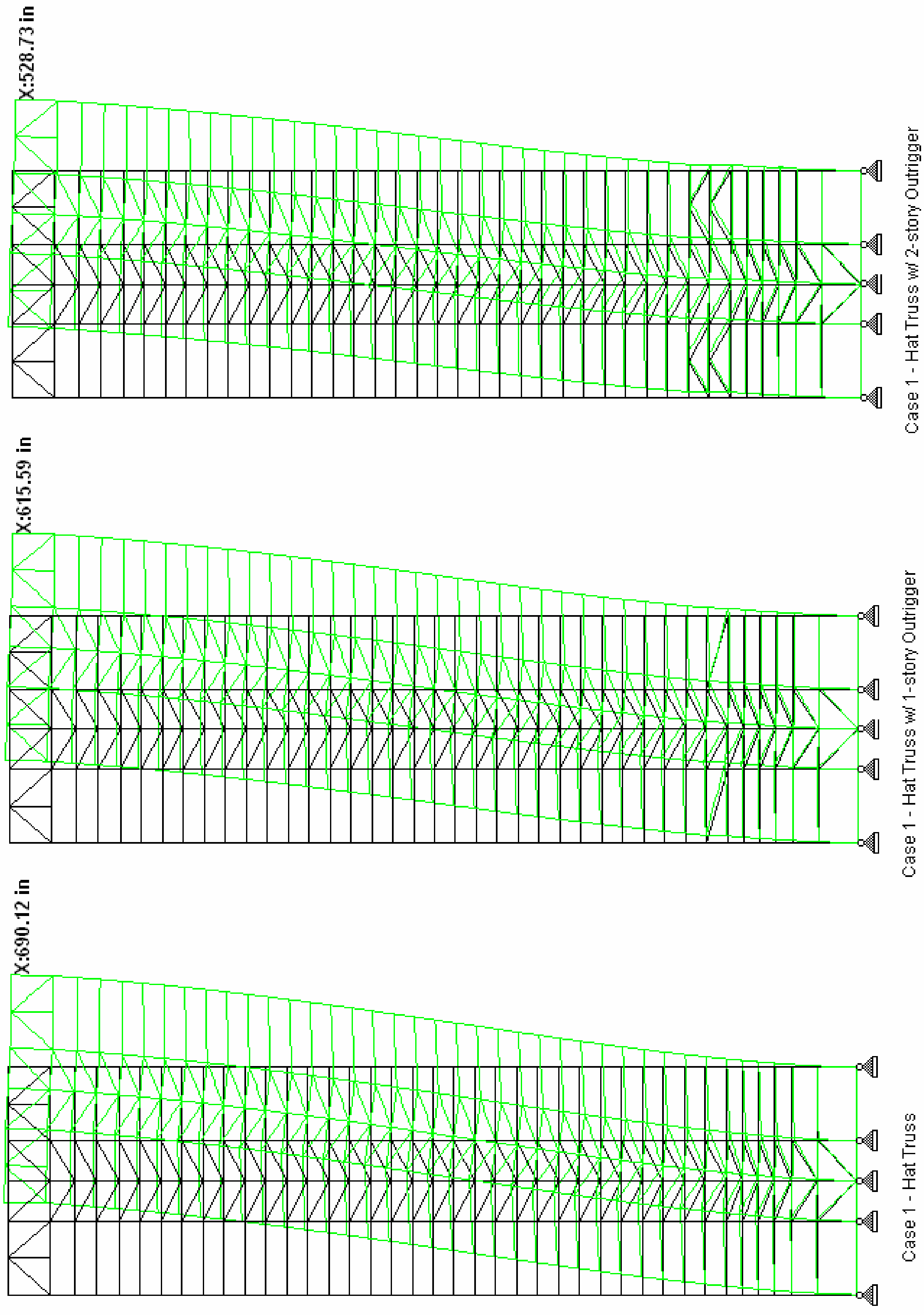
Worst Case Loads in Steel Braced Frames

W-E Wind Load (kip)	Seismic Force (kip)	N-S Wind Load (kip)	Seismic Force (kip)	W-E Worst Case Load (kip)	W-E Shear Wall Load (kip)	W-E Shear Wall Load Cum (kip)	N-S Worst Case Load (kip)	N-S Shear Wall Load (kip)	N-S Shear Wall Load Cum (kip)
70.2	85.67	54.6	85.67	85.67	28.56	28.56	85.67	42.83	42.83
109.5	141.98	85.2	141.98	141.98	47.33	75.88	141.98	70.99	113.82
75.69	63.89	58.87	63.89	75.69	25.23	101.11	63.89	31.95	145.77
73.08	63.03	56.84	63.03	73.08	24.36	125.47	63.03	31.52	177.29
70.47	56.81	54.81	56.81	70.47	23.49	148.96	56.81	28.41	205.69
67.86	53.66	52.78	53.66	67.86	22.62	171.58	53.66	26.83	232.52
67.86	50.79	52.78	50.79	67.86	22.62	194.20	52.78	26.39	258.91
67.86	48.00	52.78	48.00	67.86	22.62	216.82	52.78	26.39	285.30
67.7	45.28	52.6	45.28	67.7	22.57	239.39	52.6	26.30	311.60
63.18	42.64	49.14	42.64	63.18	21.06	260.45	49.14	24.57	336.17
63.18	40.06	49.14	40.06	63.18	21.06	281.51	49.14	24.57	360.74
63.18	37.57	49.14	37.57	63.18	21.06	302.57	49.14	24.57	385.31
63.18	35.14	49.14	35.14	63.18	21.06	323.63	49.14	24.57	409.88
63.18	32.79	49.14	32.79	63.18	21.06	344.69	49.14	24.57	434.45
63.18	30.52	49.14	30.52	63.18	21.06	365.75	49.14	24.57	459.02
63.18	28.32	49.14	28.32	63.18	21.06	386.81	49.14	24.57	483.59
61.6	26.19	47.9	26.19	61.6	20.53	407.34	47.9	23.95	507.54
58.5	24.14	45.5	24.14	58.5	19.50	426.84	45.5	22.75	530.29
58.5	22.17	45.5	22.17	58.5	19.50	446.34	45.5	22.75	553.04
58.5	20.28	45.5	20.28	58.5	19.50	465.84	45.5	22.75	575.79
58.5	18.46	45.5	18.46	58.5	19.50	485.34	45.5	22.75	598.54
58.5	16.73	45.5	16.73	58.5	19.50	504.84	45.5	22.75	621.29
58.5	15.07	45.5	15.07	58.5	19.50	524.34	45.5	22.75	644.04
58.5	13.49	45.5	13.49	58.5	19.50	543.84	45.5	22.75	666.79
55.4	11.99	43.1	11.99	55.4	18.47	562.31	43.1	21.55	688.34
53.82	10.58	41.86	10.58	53.82	17.94	580.25	41.86	20.93	709.27
53.82	9.24	41.86	9.24	53.82	17.94	598.19	41.86	20.93	730.20
53.82	7.99	41.86	7.99	53.82	17.94	616.13	41.86	20.93	751.13
53.82	6.83	41.86	6.83	53.82	17.94	634.07	41.86	20.93	772.06
53.82	5.74	41.86	5.74	53.82	17.94	652.01	41.86	20.93	792.99
53.82	4.75	41.86	4.75	53.82	17.94	669.95	41.86	20.93	813.92
53.82	3.84	41.86	3.84	53.82	17.94	687.89	41.86	20.93	834.85
53.82	3.02	41.86	3.02	53.82	17.94	705.83	41.86	20.93	855.78
47.61	3.88	37.03	3.88	47.61	15.87	721.70	37.03	18.52	874.30
41.4	2.93	32.2	2.93	41.4	13.80	735.50	32.2	16.10	890.40
41.4	2.21	32.2	2.21	41.4	13.80	749.30	32.2	16.10	906.50
43.47	1.61	33.81	1.61	43.47	14.49	763.79	33.81	16.91	923.40
54.85	1.28	42.67	1.28	54.85	18.28	782.07	42.67	21.34	944.74
82.8	0.61	64.4	0.61	82.8	27.60	809.67	64.4	32.20	976.94

W-E Braced Frame & Outrigger Configurations I

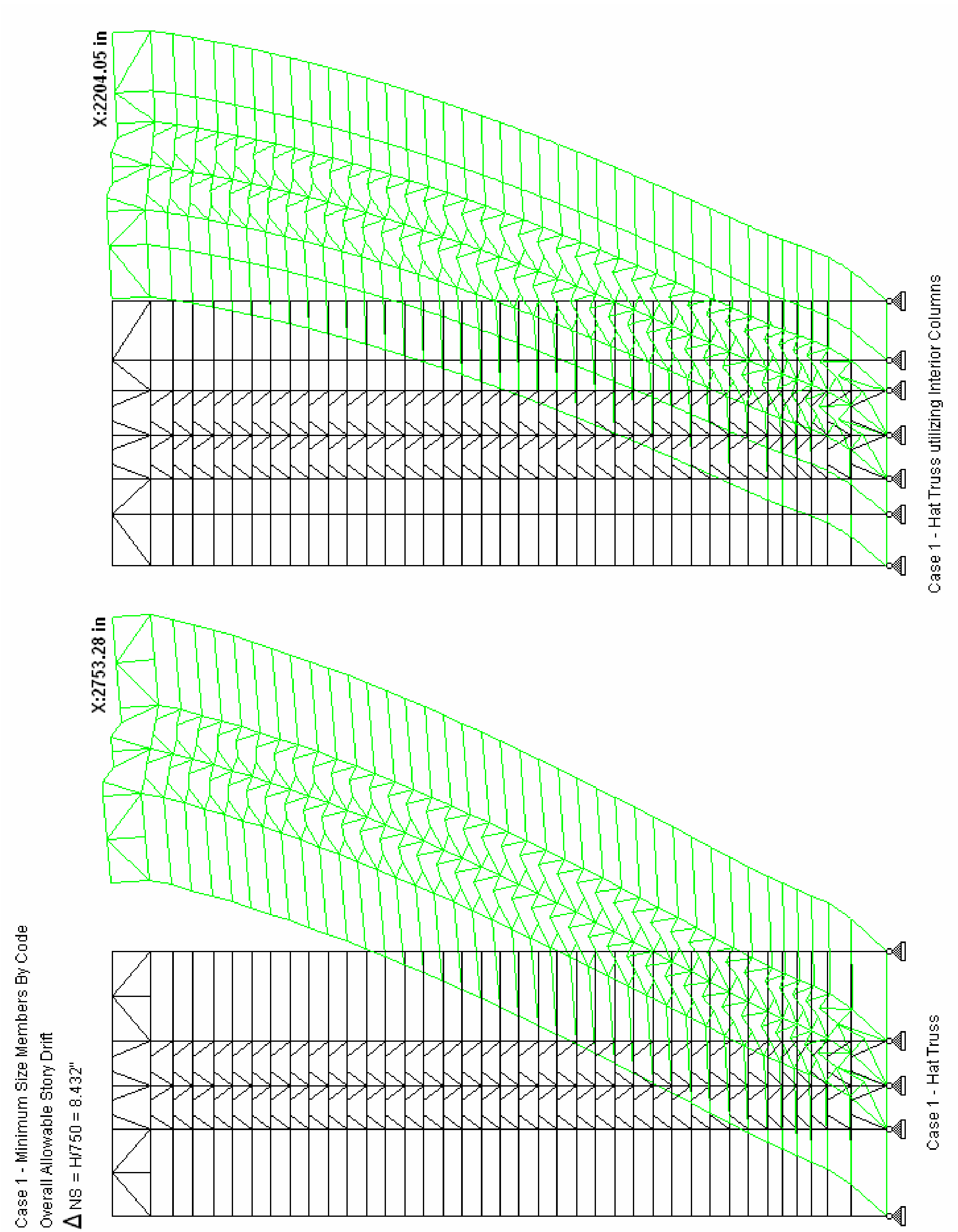


W-E Braced Frame & Outrigger Configurations II

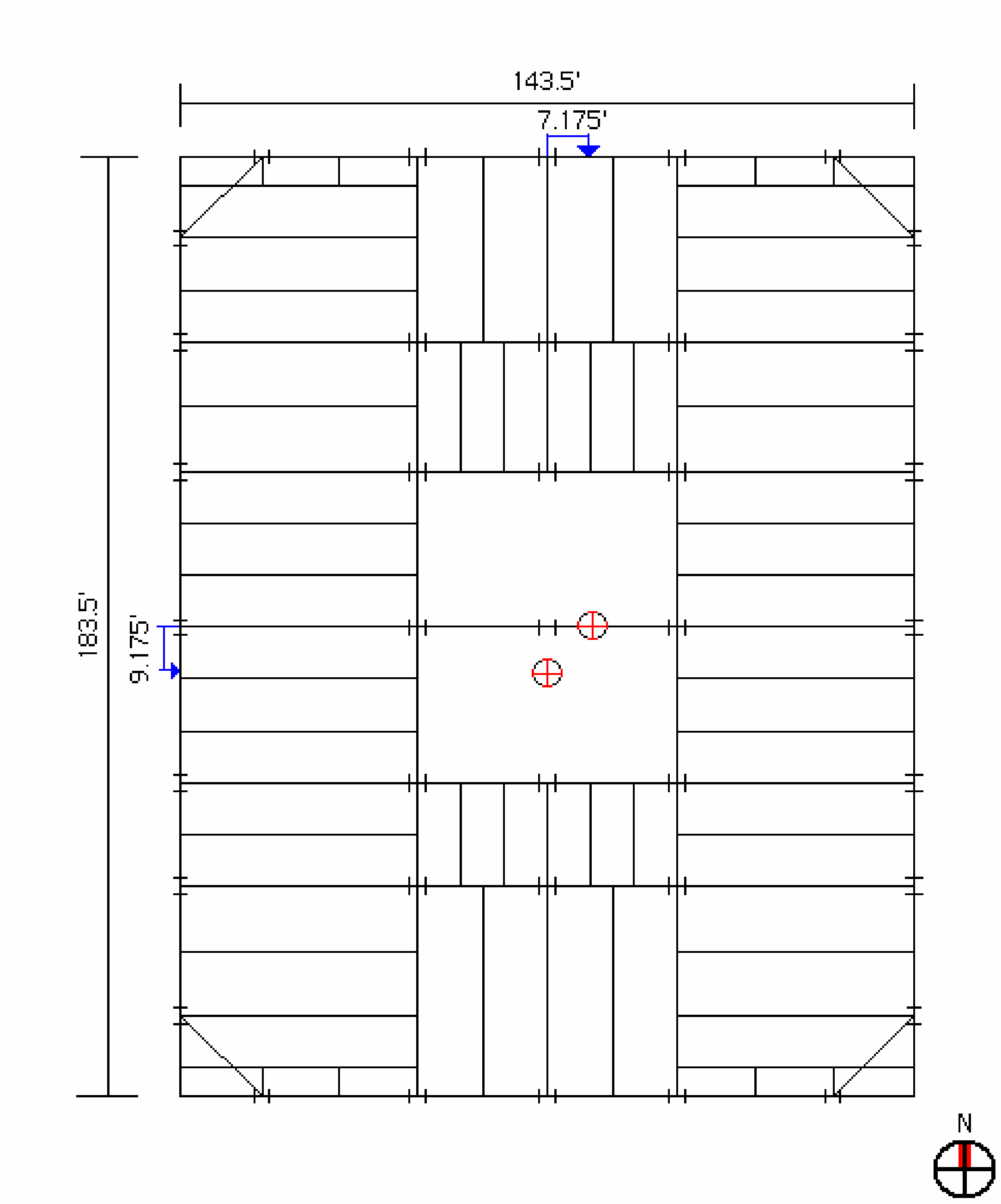


Case 1 - Minimum Size Members By Code
Overall Allowable Story Drift
Δ_{WE} = H/600 = 10.54"

N-S Braced Frame & Outrigger Configurations



Center of Rigidity



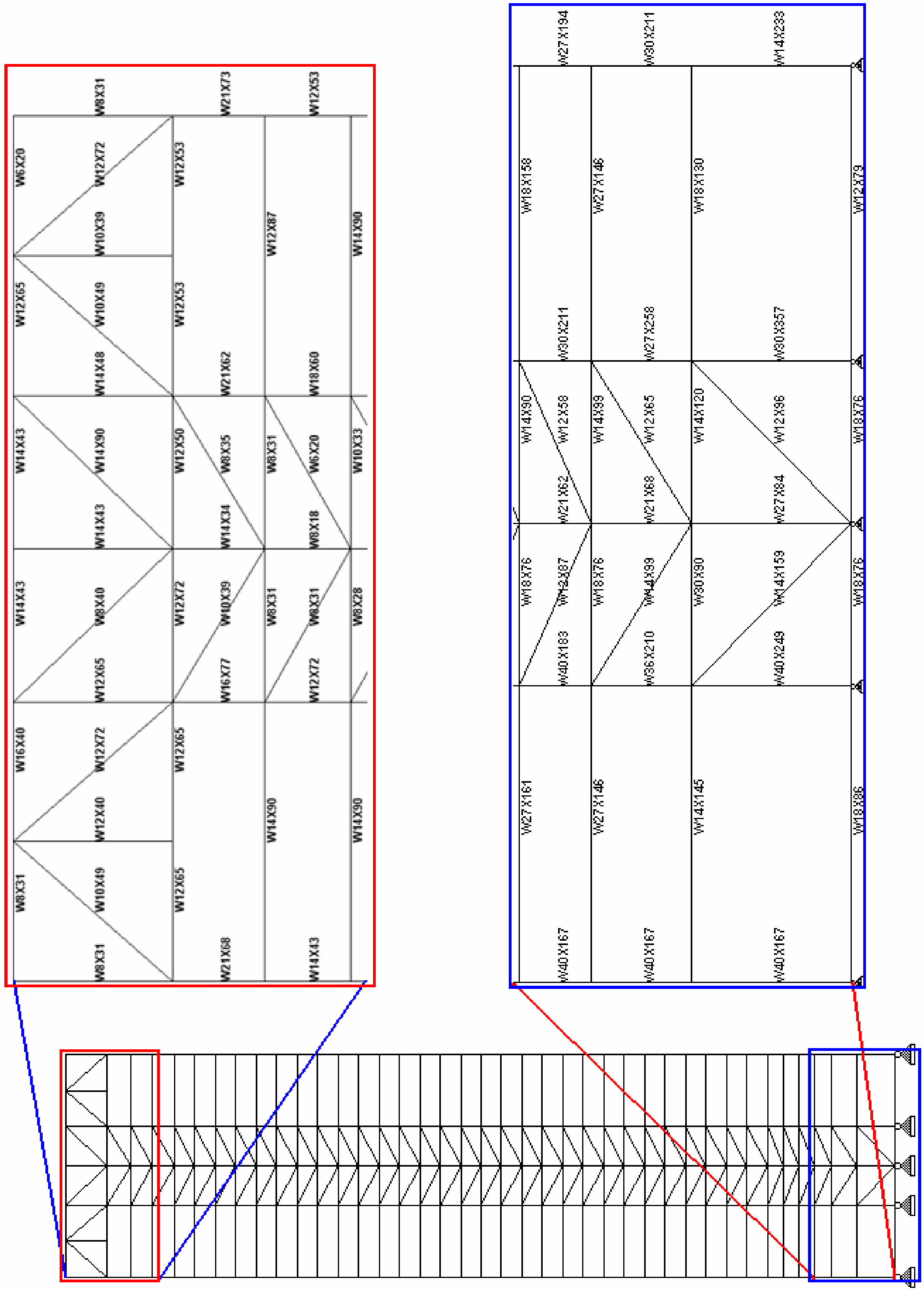
Torsional Moments for Worst Case Loads

Story	5% Offset (ft)	W-E Worst Case Load (kip)	5% Offset (ft)	N-S Worst Case Load (kip)	W-E Torsional Moment (ft-kip)	N-S Torsional Moment (ft-kip)
41	9.175	85.67	7.175	85.67	-786.01	-614.67
39	9.175	141.98	7.175	141.98	-1302.67	-1018.71
38	9.175	75.69	7.175	63.89	-694.46	-458.42
37	9.175	73.08	7.175	63.03	-670.51	-452.26
36	9.175	70.47	7.175	56.81	-646.56	-407.62
35	9.175	67.86	7.175	53.66	-622.62	-384.98
34	9.175	67.86	7.175	52.78	-622.62	-378.70
33	9.175	67.86	7.175	52.78	-622.62	-378.70
32	9.175	67.7	7.175	52.6	-621.15	-377.41
31	9.175	63.18	7.175	49.14	-579.68	-352.58
30	9.175	63.18	7.175	49.14	-579.68	-352.58
29	9.175	63.18	7.175	49.14	-579.68	-352.58
28	9.175	63.18	7.175	49.14	-579.68	-352.58
27	9.175	63.18	7.175	49.14	-579.68	-352.58
26	9.175	63.18	7.175	49.14	-579.68	-352.58
25	9.175	63.18	7.175	49.14	-579.68	-352.58
24	9.175	61.6	7.175	47.9	-565.18	-343.68
23	9.175	58.5	7.175	45.5	-536.74	-326.46
22	9.175	58.5	7.175	45.5	-536.74	-326.46
21	9.175	58.5	7.175	45.5	-536.74	-326.46
20	9.175	58.5	7.175	45.5	-536.74	-326.46
19	9.175	58.5	7.175	45.5	-536.74	-326.46
18	9.175	58.5	7.175	45.5	-536.74	-326.46
17	9.175	58.5	7.175	45.5	-536.74	-326.46
16	9.175	55.4	7.175	43.1	-508.30	-309.24
15	9.175	53.82	7.175	41.86	-493.80	-300.35
14	9.175	53.82	7.175	41.86	-493.80	-300.35
13	9.175	53.82	7.175	41.86	-493.80	-300.35
12	9.175	53.82	7.175	41.86	-493.80	-300.35
11	9.175	53.82	7.175	41.86	-493.80	-300.35
10	9.175	53.82	7.175	41.86	-493.80	-300.35
9	9.175	53.82	7.175	41.86	-493.80	-300.35
8	9.175	53.82	7.175	41.86	-493.80	-300.35
7	9.175	47.61	7.175	37.03	-436.82	-265.69
6	9.175	41.4	7.175	32.2	-379.85	-231.04
5	9.175	41.4	7.175	32.2	-379.85	-231.04
4	9.175	43.47	7.175	33.81	-398.84	-242.59
3	9.175	54.85	7.175	42.67	-503.25	-306.16
2	9.175	82.8	7.175	64.4	-759.69	-462.07

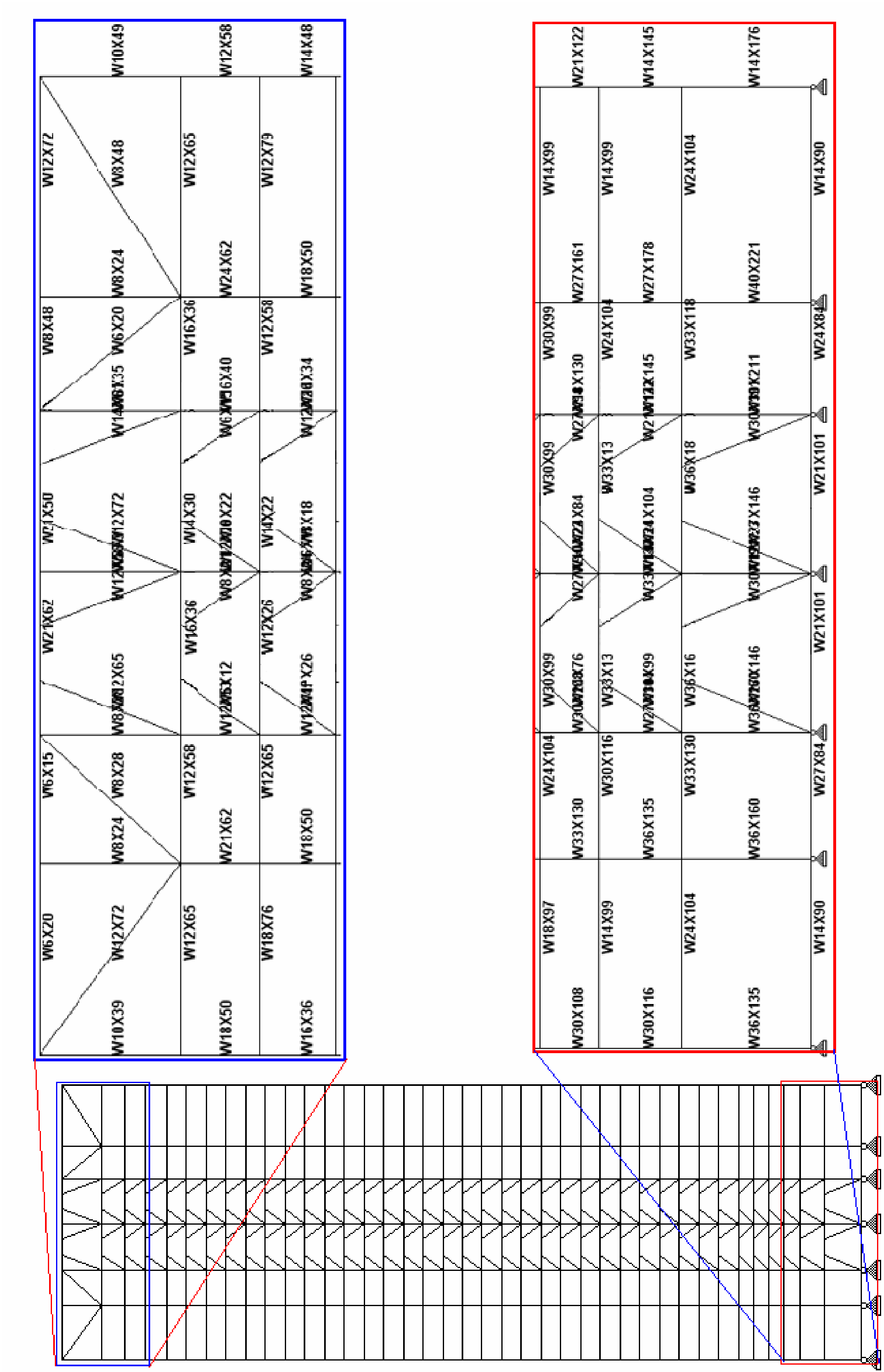
Worst Case Loads with Torsion in Steel Braced Frames

W-E Shear Wall Load (kip)	%5 W-E Torsion Worst Case	N-S Shear Wall Load (kip)	%5 N-S Torsion Worst Case	Worst Case W-E Shear Wall Load (kip)	Worst Case N-S Shear Wall Load (kip)	Worst Case W-E Shear Wall Load Cum (kip)	Worst Case W-E Shear Wall Load Cum (kip)
28.56	6.53	42.83	9.00	35.09	51.83	35.09	51.83
47.33	10.83	70.99	14.91	58.15	85.90	93.24	137.73
25.23	5.77	31.95	6.71	31.00	38.65	124.24	176.38
24.36	5.57	31.52	6.62	29.93	38.14	154.18	214.52
23.49	5.37	28.41	5.97	28.86	34.37	183.04	248.89
22.62	5.17	26.83	5.63	27.79	32.46	210.84	281.35
22.62	5.17	26.39	5.54	27.79	31.93	238.63	313.28
22.62	5.17	26.39	5.54	27.79	31.93	266.43	345.22
22.57	5.16	26.30	5.52	27.73	31.82	294.15	377.04
21.06	4.82	24.57	5.16	25.88	29.73	320.03	406.77
21.06	4.82	24.57	5.16	25.88	29.73	345.91	436.50
21.06	4.82	24.57	5.16	25.88	29.73	371.79	466.23
21.06	4.82	24.57	5.16	25.88	29.73	397.67	495.96
21.06	4.82	24.57	5.16	25.88	29.73	423.54	525.69
21.06	4.82	24.57	5.16	25.88	29.73	449.42	555.42
21.06	4.82	24.57	5.16	25.88	29.73	475.30	585.15
20.53	4.70	23.95	5.03	25.23	28.98	500.53	614.13
19.50	4.46	22.75	4.78	23.96	27.53	524.49	641.66
19.50	4.46	22.75	4.78	23.96	27.53	548.45	669.18
19.50	4.46	22.75	4.78	23.96	27.53	572.41	696.71
19.50	4.46	22.75	4.78	23.96	27.53	596.37	724.24
19.50	4.46	22.75	4.78	23.96	27.53	620.34	751.77
19.50	4.46	22.75	4.78	23.96	27.53	644.30	779.29
19.50	4.46	22.75	4.78	23.96	27.53	668.26	806.82
18.47	4.22	21.55	4.53	22.69	26.08	690.95	832.90
17.94	4.10	20.93	4.40	22.04	25.33	712.99	858.22
17.94	4.10	20.93	4.40	22.04	25.33	735.04	883.55
17.94	4.10	20.93	4.40	22.04	25.33	757.08	908.87
17.94	4.10	20.93	4.40	22.04	25.33	779.13	934.20
17.94	4.10	20.93	4.40	22.04	25.33	801.17	959.53
17.94	4.10	20.93	4.40	22.04	25.33	823.21	984.85
17.94	4.10	20.93	4.40	22.04	25.33	845.26	1010.18
17.94	4.10	20.93	4.40	22.04	25.33	867.30	1035.50
15.87	3.63	18.52	3.89	19.50	22.40	886.80	1057.91
13.80	3.16	16.10	3.38	16.96	19.48	903.76	1077.39
13.80	3.16	16.10	3.38	16.96	19.48	920.72	1096.87
14.49	3.31	16.91	3.55	17.80	20.46	938.52	1117.32
18.28	4.18	21.34	4.48	22.47	25.82	960.99	1143.14
27.60	6.31	32.20	6.76	33.91	38.96	994.90	1182.10

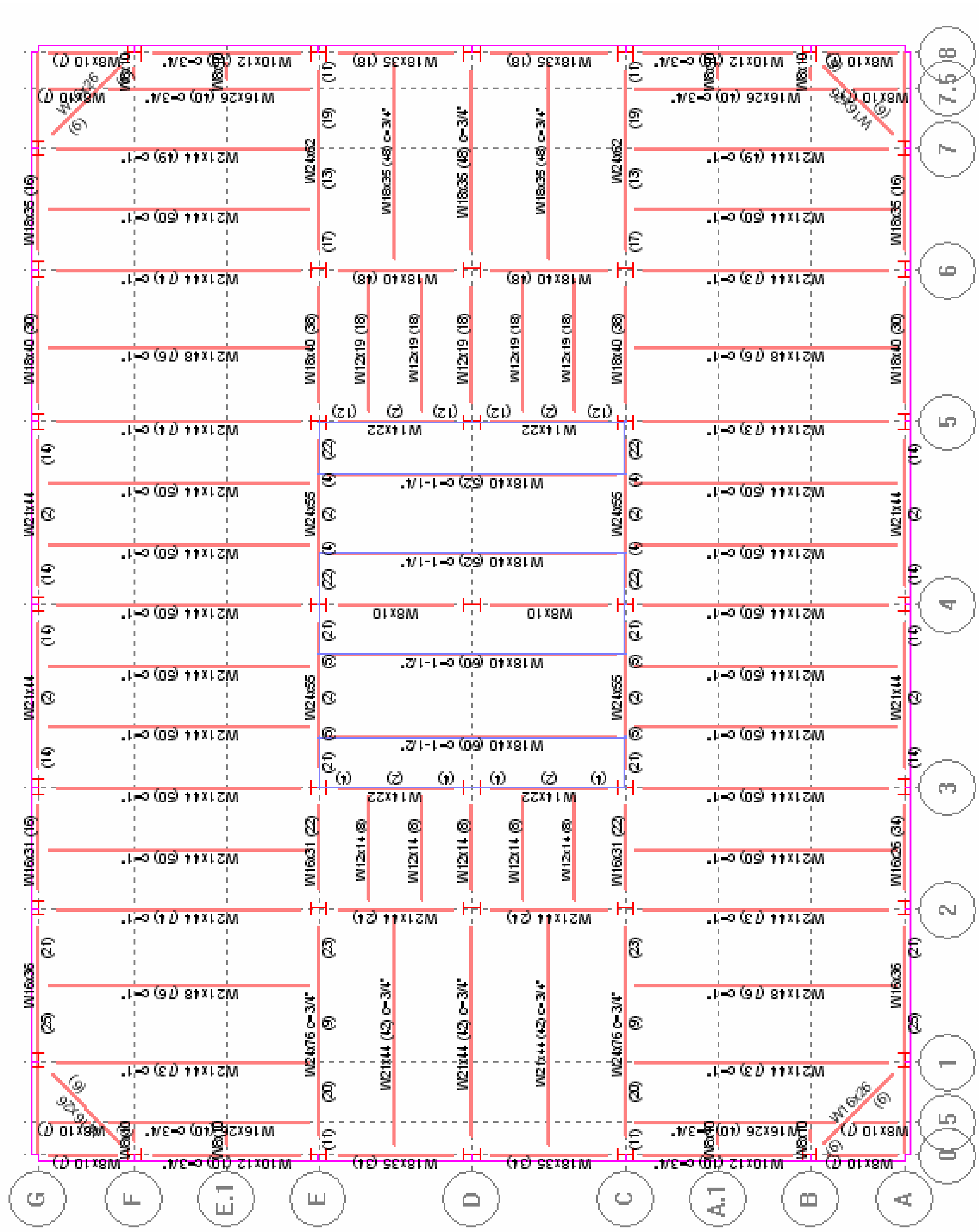
Member Sizes for W-E Worst Case Loading



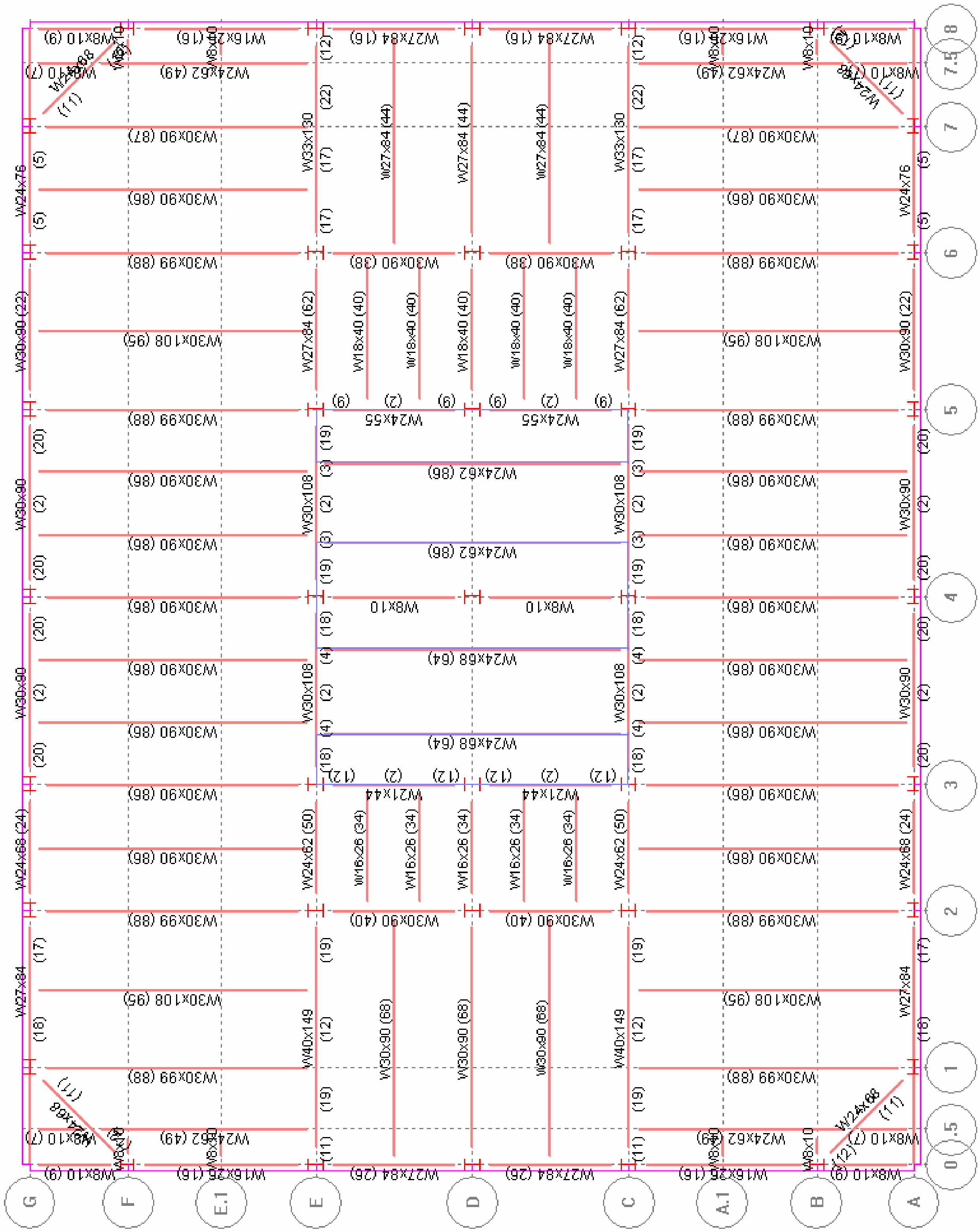
Member Sizes for N-E Worst Case Loading



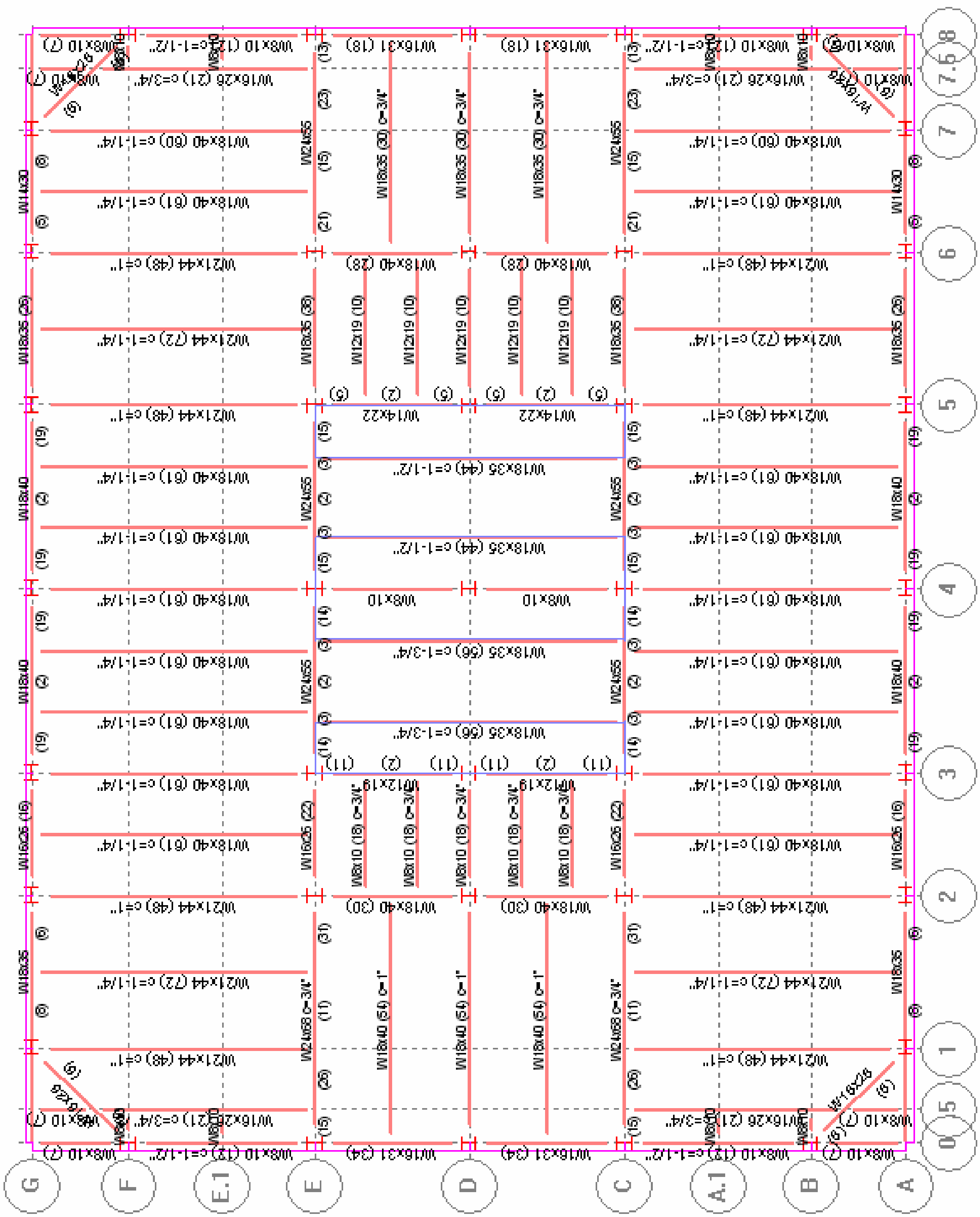
Typical Steel Floor Framing Redesign



Mechanical Steel Floor Framing Redesign



Parking Steel Floor Framing Redesign



APPENDIX C

POST-TENSIONING FLOOR SYSTEM ANALYSIS

General Design Parameters

CONCRETE:

STRENGTH at 28 days, for BEAMS/SLABS/COLUMNS	4000.00 psi
MODULUS OF ELASTICITY for BEAMS/SLABS/COLUMNS	3605.00 ksi
SELF WEIGHT - NORMAL WEIGHT CONCRETE	145.00 pcf
CREEP factor for deflections for BEAMS/SLABS	2.00

TENSION STRESS limits (multiple of $f'c^{1/2}$)

At Top	6.000
At Bottom	6.000

COMPRESSION STRESS limits (multiple of f'c)

At all locations	0.450

REINFORCEMENT:

Yield Strength	60.00 ksi
Minimum Cover at TOP	1.00 in
Minimum Cover at BOTTOM	1.00 in

POST-TENSIONING:

Post tensioning system	UNBONDED
Ultimate strength of strand	270.00 ksi
Average effective stress in strand (final)	175.00 ksi
Strand area	0.153 in^2
Strand diameter	0.50 in
Min CGS of tendon from TOP	1.00 in
Min CGS of tendon from BOTTOM for INTERIOR spans	1.00 in
Min CGS of tendon from BOTTOM for EXTERIOR spans	1.75 in
Min average precompression	125.00 psi
Max spacing between strands (factor of slab depth)	8.00
Tendon profile type	PARTIAL PARABOLA

DESIGN OPTIONS USED:

Structural system	ONE-WAY
Moment of Inertia over support is	NOT INCREASED
Moments REDUCED to face of support	YES
Limited plastification allowed (moments redistributed)	NO

Slab & Band Concrete Strip Dimensions

One-Way Slab						
Typical Loading Condition						
			Beam		Strip	
Span	Section	Length (ft)	Width (in)	Height(in)	Width (in)	Thick. (in)
1	T	45.58	96	17	360	6
2	T	50	96	17	360	6
3	T	45.58	96	17	360	6

One-Way Slab						
Parking Loading Condition						
			Beam		Strip	
Span	Section	Length (ft)	Width (in)	Height(in)	Width (in)	Thick. (in)
1	T	45.58	96	14	360	6
2	T	50	96	14	360	6
3	T	45.58	96	14	360	6

One-Way Slab						
Mechanical Loading Condition						
			Beam		Strip	
Span	Section	Length (ft)	Width (in)	Height(in)	Width (in)	Thick. (in)
1	T	45.58	120	24	360	9
2	T	50	120	24	360	9
3	T	45.58	120	24	360	9

One-Way Slab						
Level 2 Loading Condition						
			Beam		Strip	
Span	Section	Length (ft)	Width (in)	Height(in)	Width (in)	Thick. (in)
1	T	50	120	24	360	9
2	T	45.58	120	24	360	9

One-Way Slab						
Typical Loading Condition						
			Beam		Strip	
Span	Section	Length (ft)	Width (in)	Height(in)	Width (in)	Thick. (in)
1	T	45.58	96	17	367	6
2	T	50	96	17	367	6
3	T	45.58	96	17	367	6

Loading Conditions

One-Way Slab				
Typical Loading Condition				
Span	Class	Type	Load (k/ft²)	Total on Tributary (k/ft)
1	Live	Uniform	0.1	3
1	Dead	Uniform	0.005	0.15
1	Self W.	Uniform		3.238
2	Live	Uniform	0.1	3
2	Dead	Uniform	0.005	0.15
2	Self W.	Uniform		3.238
3	Live	Uniform	0.1	3
3	Dead	Uniform	0.005	0.15
3	Self W.	Uniform		3.238

One-Way Slab				
Parking Loading Condition				
Span	Class	Type	Load (k/ft²)	Total on Tributary (k/ft)
1	Live	Uniform	0.05	1.5
1	Dead	Uniform	0.005	0.15
1	Self W.	Uniform		2.948
2	Live	Uniform	0.05	1.5
2	Dead	Uniform	0.005	0.15
2	Self W.	Uniform		2.948
3	Live	Uniform	0.05	1.5
3	Dead	Uniform	0.005	0.15
3	Self W.	Uniform		2.948

One-Way Slab				
Mechanical Loading Condition				
Span	Class	Type	Load (k/ft²)	Total on Tributary (k/ft)
1	Live	Uniform	0.15	3
1	Dead	Uniform	0.005	0.15
1	Self W.	Uniform		5.075
2	Live	Uniform	0.25	7.5
2	Dead	Uniform	0.005	0.15
2	Self W.	Uniform		5.075
3	Live	Uniform	0.25	7.5
3	Dead	Uniform	0.005	0.15
3	Self W.	Uniform		5.075

One-Way Slab				
Typical End Loading Condition				
Span	Class	Type	Load (k/ft²)	Total on Tributary (k/ft)
1	Live	Uniform	0.1	3.058
1	Dead	Uniform	0.005	0.153
1	Self W.	Uniform		3.281
2	Live	Uniform	0.1	3.058
2	Dead	Uniform	0.005	0.153
2	Self W.	Uniform		3.281
3	Live	Uniform	0.1	3.058
3	Dead	Uniform	0.005	0.153
3	Self W.	Uniform		3.281

One-Way Slab				
Level 2 Loading Condition				
Span	Class	Type	Load (k/ft²)	Total on Tributary (k/ft)
1	Live	Uniform	0.15	4.5
1	Dead	Uniform	0.005	0.15
1	Self W.	Uniform		5.075
2	Live	Uniform	0.25	7.5
2	Dead	Uniform	0.005	0.15
2	Self W.	Uniform		5.075

Typical Loading Strip - Columns

	Tension Stresses / (fc')^1/2			Compression Stresses / (fc')			Final Allowable Values		
	Left	Center	Right	Left	Center	Right	Tens (top)	Tens (bot)	Comp
1	3.362	4.634	5.675	0.227	0.181	0.384	6.000	6.000	0.450
2	5.986	2.557	5.986	0.393	0.217	0.393	6.000	6.000	0.450
3	5.676	4.634	3.362	0.384	0.181	0.227	6.000	6.000	0.450

		<—— Tendon Height ——>					Governing Forces		
	Force	Left	Center	Right	P/A (mid)	Wbal %DL	Left	Center	Right
1	919.8	-5.79	-15.25	-1.00	286	129	433.9	849.5	899.4
2	919.8	-1.00	-16.00	-1.00	286	136	919.7	772.6	919.8
3	919.8	-1.00	-15.25	-5.79	286	129	899.5	849.5	433.9

Typical Loading Strip - Core

	Tension Stresses / (fc')^1/2			Compression Stresses / (fc')			Final Allowable Values		
	Left	Center	Right	Left	Center	Right	Tens (top)	Tens (bot)	Comp
1	2.772	3.629	5.986	0.207	0.161	0.398	6.000	6.000	0.450
2	5.979	0.235	5.979	0.398	0.192	0.398	6.000	6.000	0.450
3	5.985	3.629	2.772	0.398	0.161	0.207	6.000	6.000	0.450

		<—— Tendon Height ——>					Governing Forces		
	Force	Left	Center	Right	P/A (mid)	Wbal %DL	Left	Center	Right
1	941.2	-5.79	-15.25	-1.00	293	132	429.3	806.6	941.1
2	941.2	-1.00	-16.00	-1.00	293	139	940.7	693.1	940.7
3	941.2	-1.00	-15.25	-5.79	293	132	941.1	806.6	429.4

Typical Loading Strip – End Span

	Tension Stresses / (fc')^1/2			Compression Stresses / (fc')			Final Allowable Values		
	Left	Center	Right	Left	Center	Right	Tens (top)	Tens (bot)	Comp
1	3.494	4.700	5.668	0.229	0.184	0.388	6.000	6.000	0.450
2	5.986	2.619	5.986	0.397	0.220	0.397	6.000	6.000	0.450
3	5.668	4.700	3.494	0.388	0.184	0.229	6.000	6.000	0.450

		<—— Tendon Height ——>					Governing Forces		
	Force	Left	Center	Right	P/A (mid)	Wbal %DL	Left	Center	Right
1	936.4	-5.76	-15.25	-1.00	287	130	460.2	869.3	915.4
2	936.4	-1.00	-16.00	-1.00	287	136	936.4	791.1	936.4
3	936.4	-1.00	-15.25	-5.76	287	130	915.4	869.3	460.2

Mechanical Loading Strip – Columns

	Tension Stresses / (fc')^1/2			Compression Stresses / (fc')			Final Allowable Values		
	Left	Center	Right	Left	Center	Right	Tens (top)	Tens (bot)	Comp
1	4.120	5.326	4.946	0.096	0.107	0.237	6.000	6.000	0.450
2	5.007	3.599	3.129	0.239	0.162	0.272	6.000	6.000	0.450
3	3.419	5.560	5.023	0.280	0.217	0.153	6.000	6.000	0.450

		←——— Tendon Height ———→					Governing Forces		
	Force	Left	Center	Right	P/A (mid)	Wbal %DL	Left	Center	Right
1	750.0	-8.79	-22.25	-1.00	149	100	630	697	649
2	650.0	-1.00	-23.00	-1.00	129	91	652	630	1059
3	1372.7	-1.00	-22.25	-8.79	272	183	1108	1332	1216

Mechanical Loading Strip – Core

	Tension Stresses / (fc')^1/2			Compression Stresses / (fc')			Final Allowable Values		
	Left	Center	Right	Left	Center	Right	Tens (top)	Tens (bot)	Comp
1	4.295	5.382	5.752	0.094	0.104	0.252	6.000	6.000	0.450
2	5.338	1.910	3.169	0.241	0.124	0.263	6.000	6.000	0.450
3	5.502	5.793	5.295	0.327	0.183	0.150	6.000	6.000	0.450

		←——— Tendon Height ———→					Governing Forces		
	Force	Left	Center	Right	P/A (mid)	Wbal %DL	Left	Center	Right
1	700.0	-8.79	-22.25	-1.00	139	93	630	648	677
2	700.0	-1.00	-23.00	-1.00	139	98	638	630	985
3	1300.9	-1.00	-22.25	-8.79	258	173	1252	1282	1187

Parking Loading Strip – Columns

	Tension Stresses / (fc')^1/2			Compression Stresses / (fc')			Final Allowable Values		
	Left	Center	Right	Left	Center	Right	Tens (top)	Tens (bot)	Comp
1	2.332	2.738	5.512	0.276	0.132	0.371	6.000	6.000	0.450
2	5.985	0.902	5.985	0.386	0.141	0.386	6.000	6.000	0.450
3	5.511	2.737	2.333	0.371	0.132	0.276	6.000	6.000	0.450

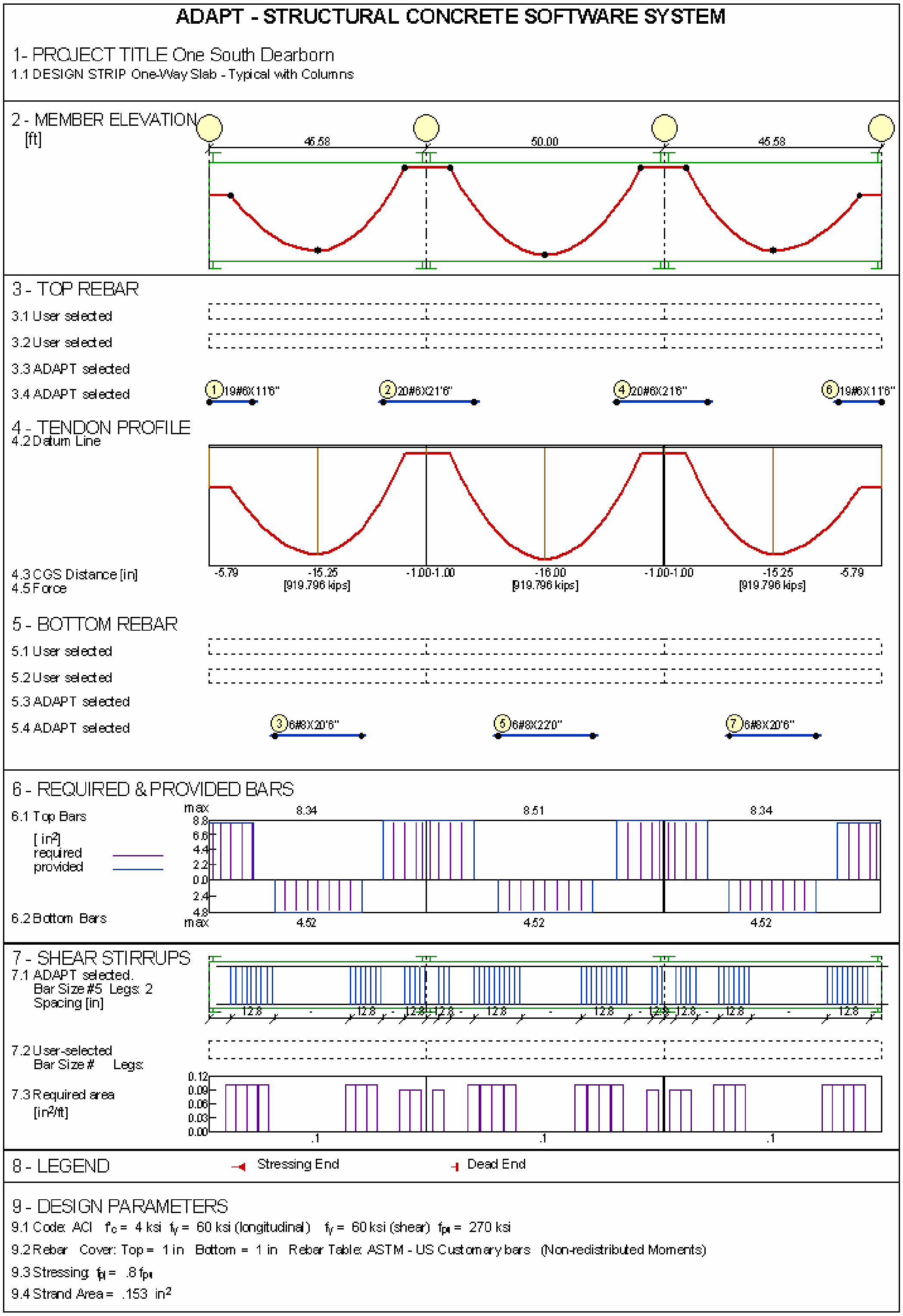
		←——— Tendon Height ———→					Governing Forces		
	Force	Left	Center	Right	P/A (mid)	Wbal %DL	Left	Center	Right
1	833.9	-4.84	-12.25	-1.00	285	101	548	675	806
2	833.9	-1.00	-13.00	-1.00	285	108	834	645	834
3	833.9	-1.00	-12.25	-4.84	285	101	806	675	548

Parking Loading Strip – Core

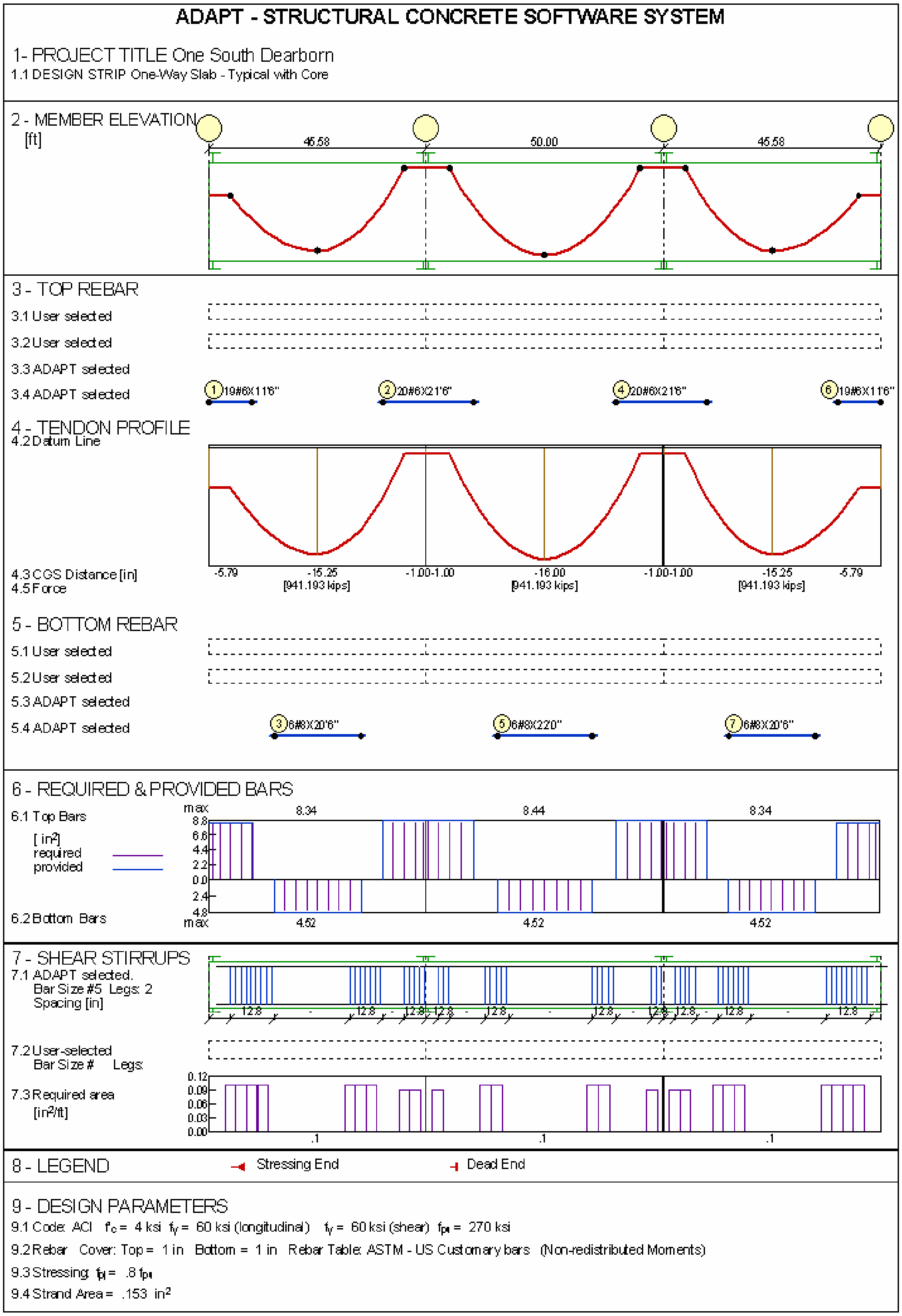
	Tension Stresses / (fc')^1/2			Compression Stresses / (fc')			Final Allowable Values		
	Left	Center	Right	Left	Center	Right	Tens (top)	Tens (bot)	Comp
1	1.516	1.868	5.714	0.260	0.129	0.386	6.000	6.000	0.450
2	5.984	0.000	5.984	0.394	0.141	0.394	6.000	6.000	0.450
3	5.713	1.868	1.517	0.386	0.129	0.260	6.000	6.000	0.450

		←——— Tendon Height ———→					Governing Forces		
	Force	Left	Center	Right	P/A (mid)	Wbal %DL	Left	Center	Right
1	867.8	-4.84	-12.25	-1.00	296	105	520.4	660.4	851.4
2	867.8	-1.00	-13.00	-1.00	296	112	867.8	610.2	867.8
3	867.8	-1.00	-12.25	-4.84	296	105	851.3	660.4	520.5

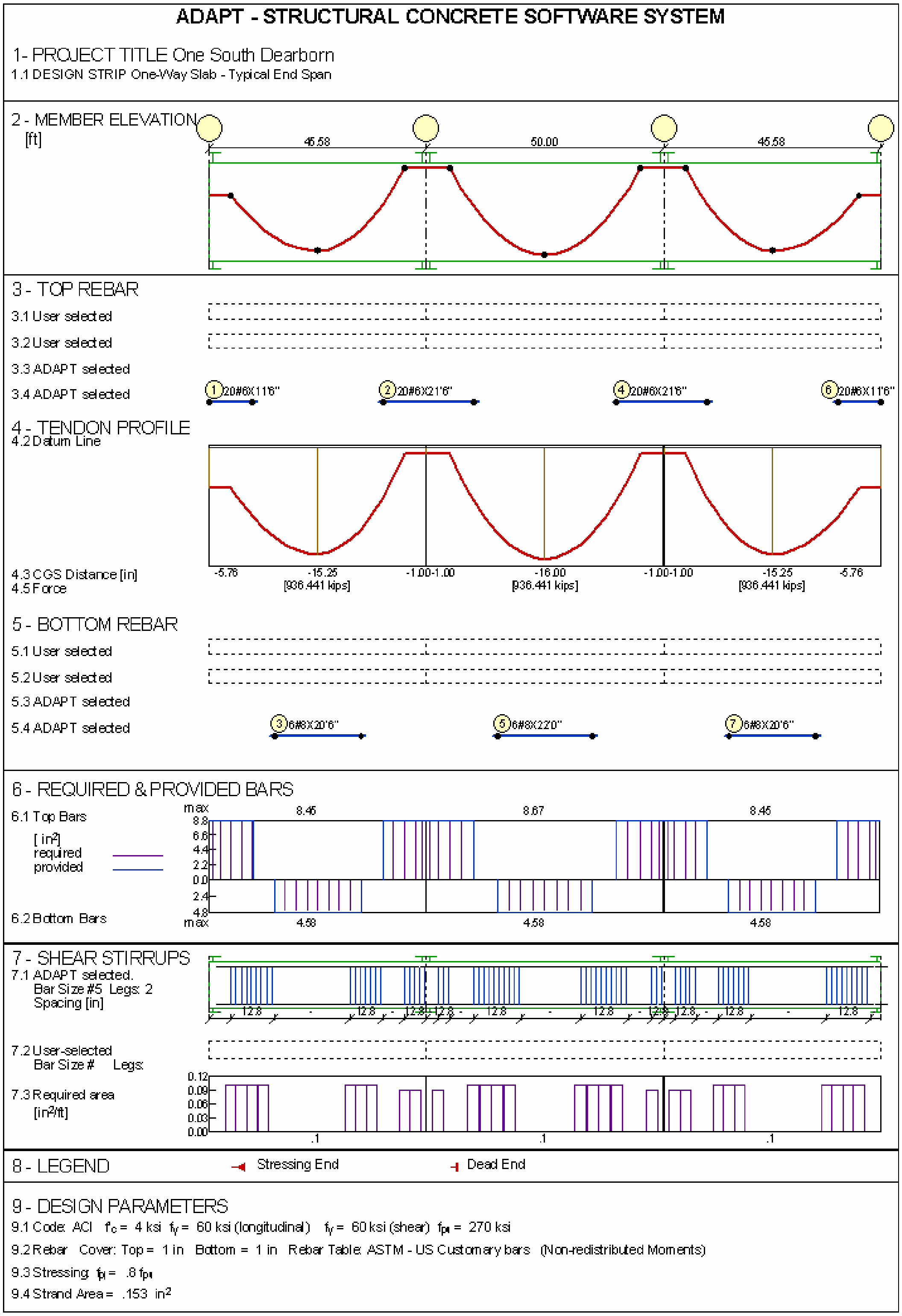
Post-Tensioning Strip Summary – Typical – Columns



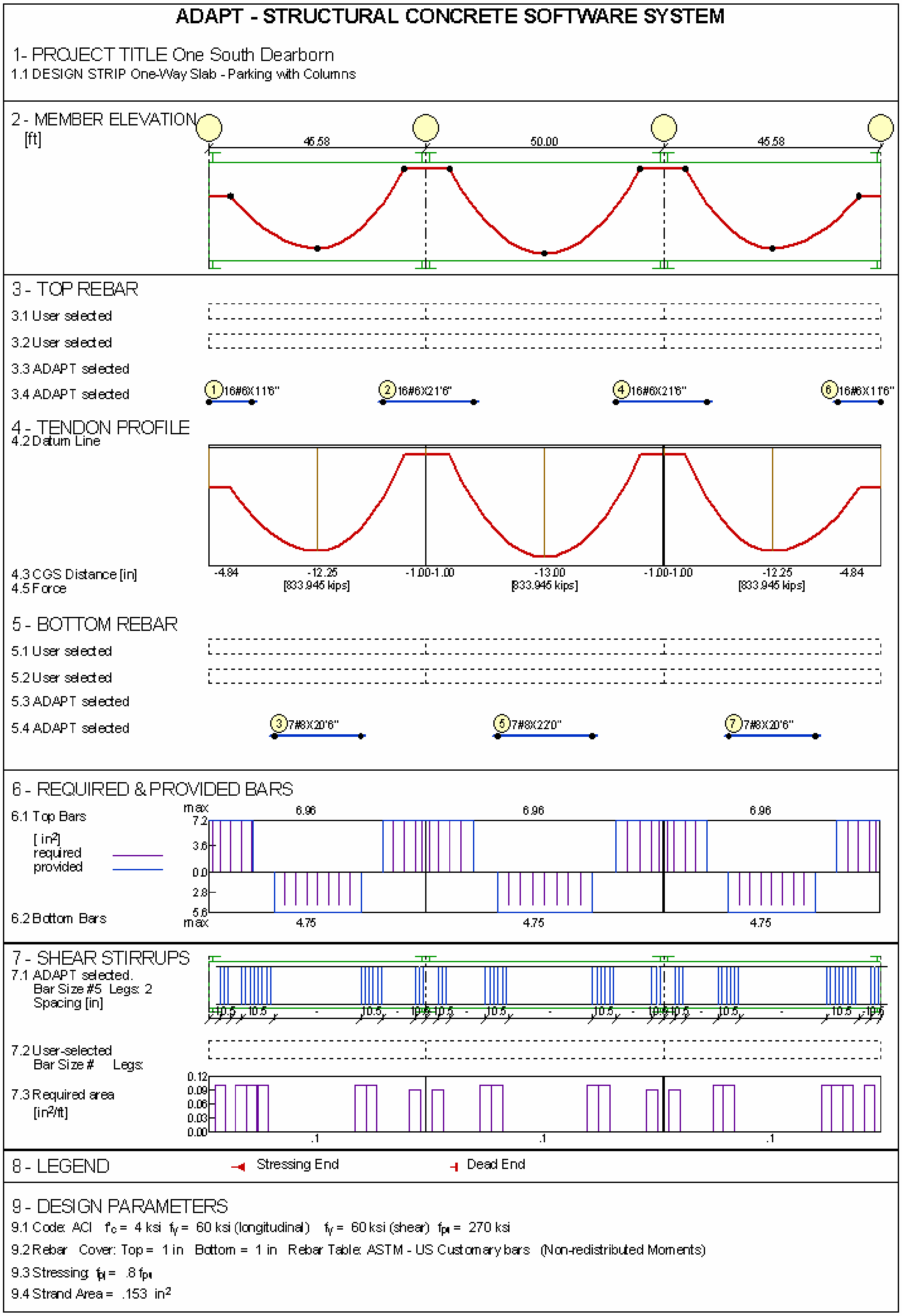
Post-Tensioning Strip Summary – Typical – Core



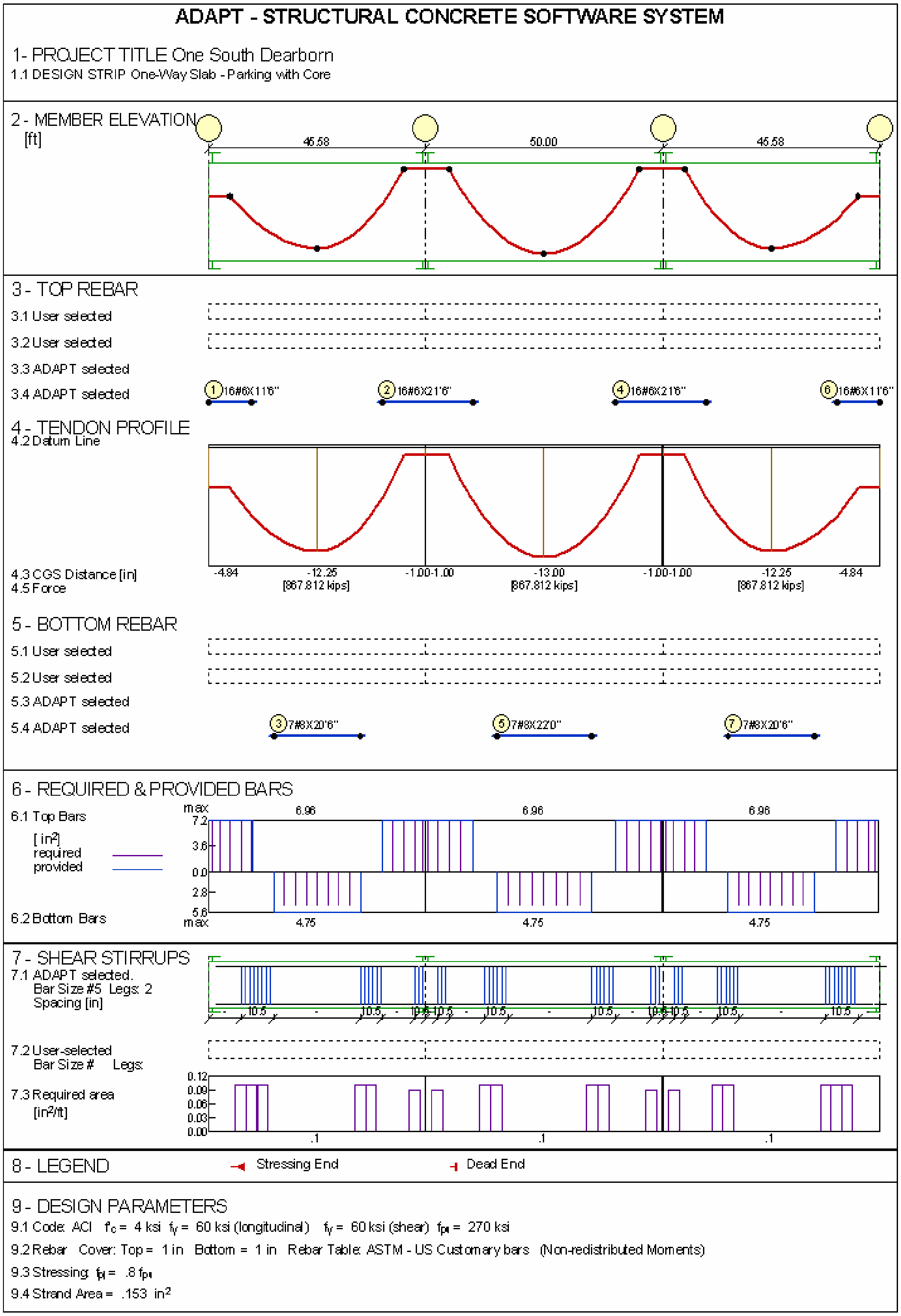
Post-Tensioning Strip Summary – Typical End Span



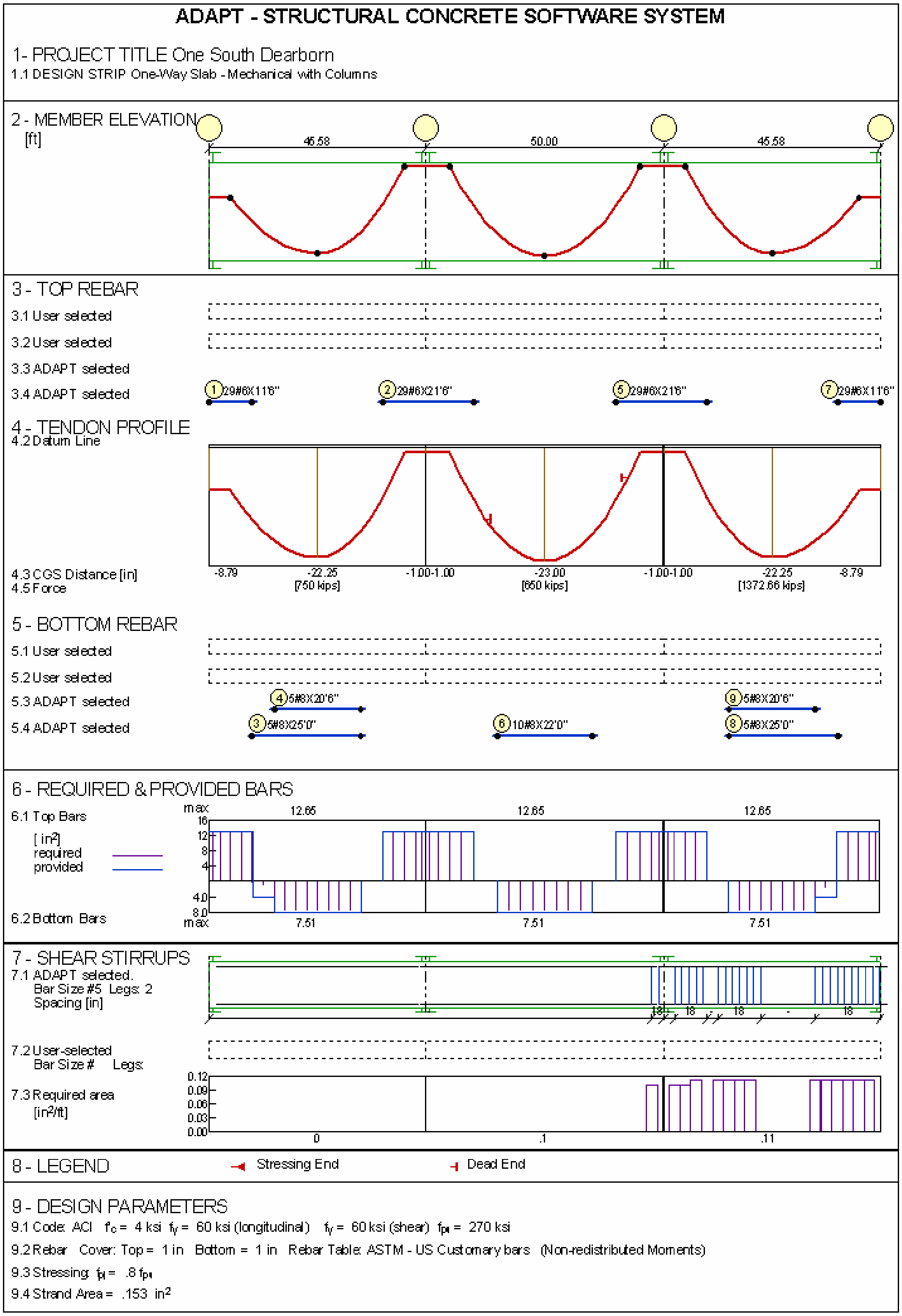
Post-Tensioning Strip Summary – Parking - Columns



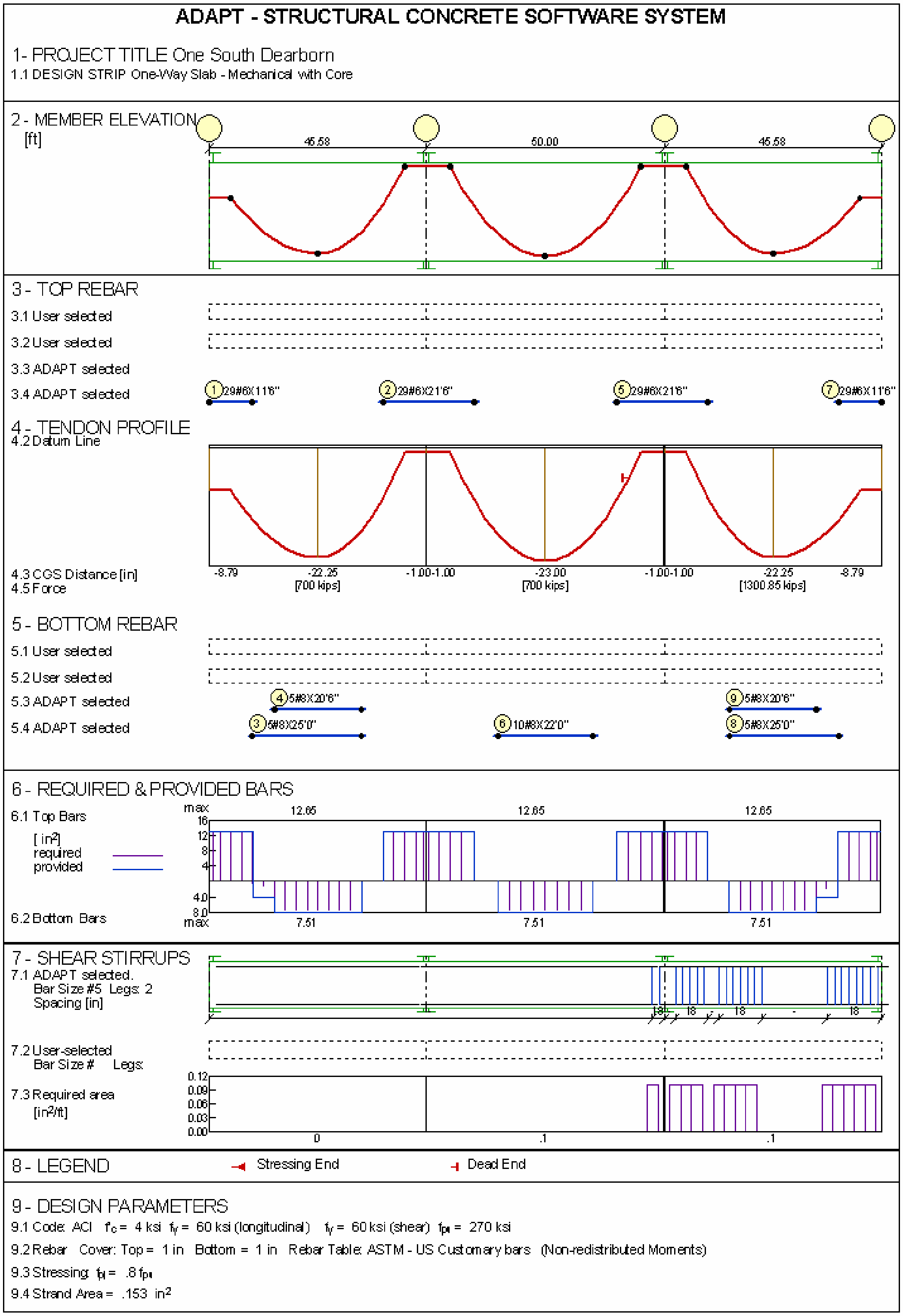
Post-Tensioning Strip Summary – Parking - Core



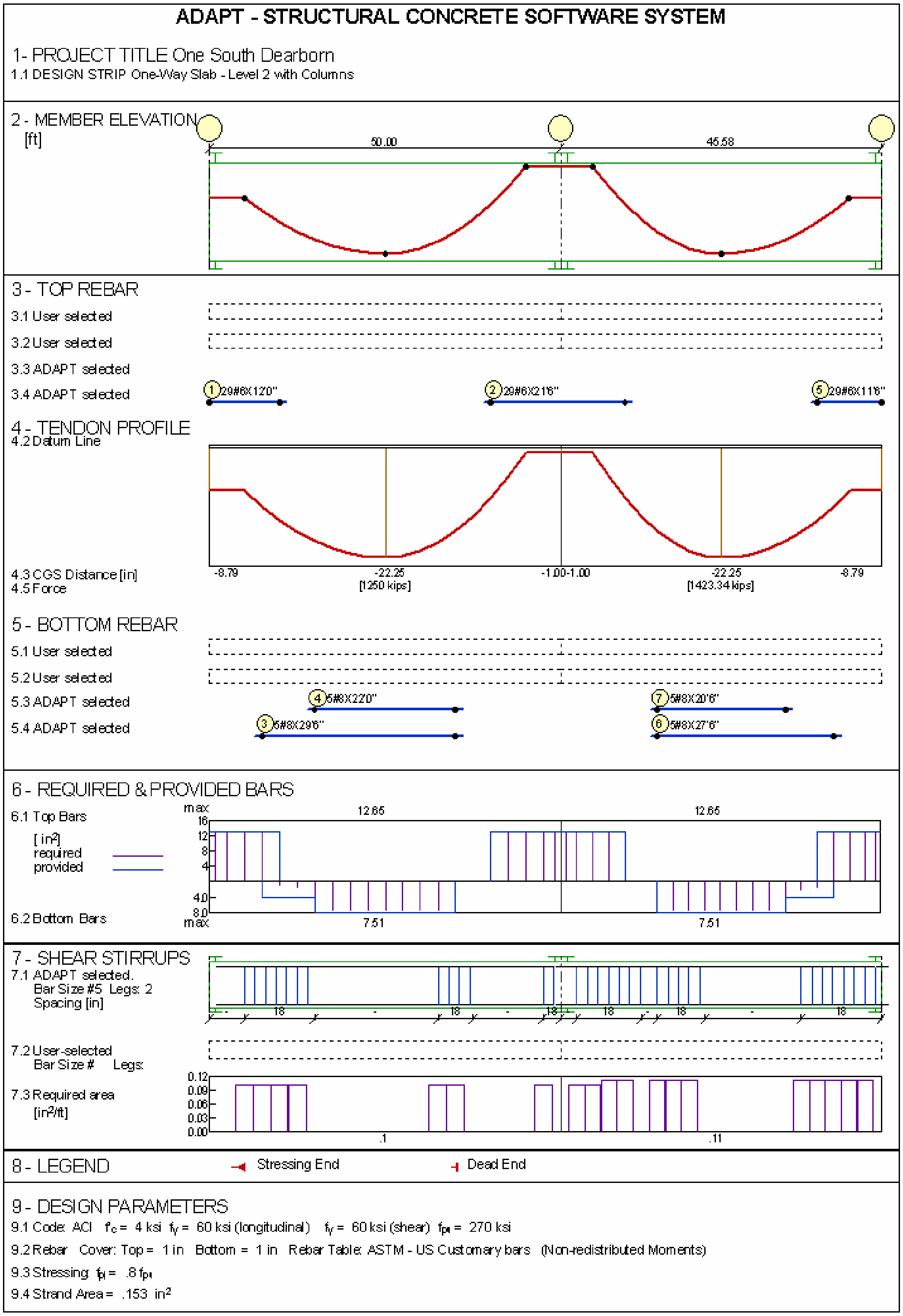
Post-Tensioning Strip Summary – Mechanical - Columns



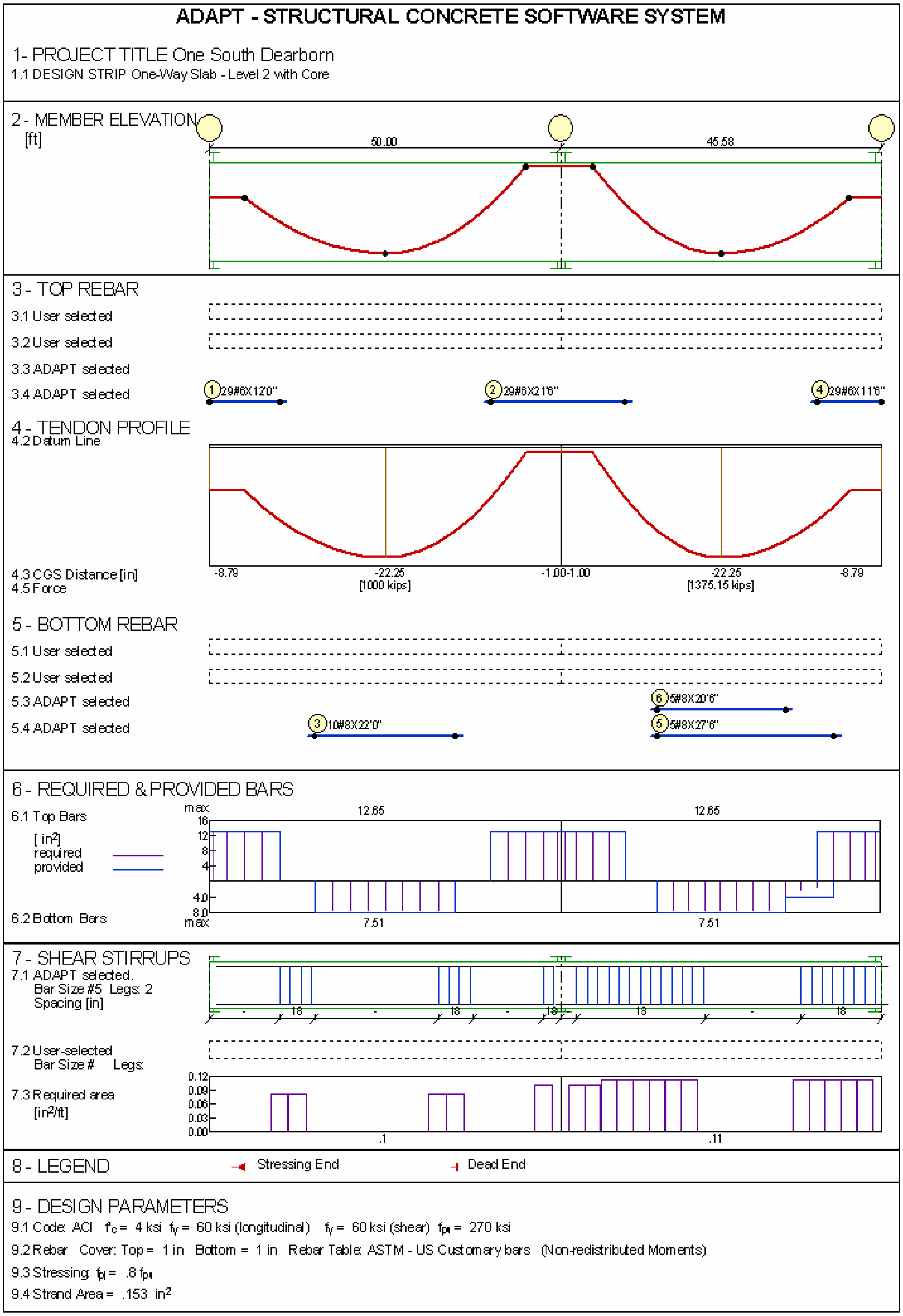
Post-Tensioning Strip Summary – Mechanical - Core



Post-Tensioning Strip Summary – Level 2 - Columns



Post-Tensioning Strip Summary – Level 2 - Core



Column Properties

Column Design

Material Properties		Units
f'c	4	ksi
Ec	3834.25	ksi
fc	3.4	ksi
fy	60	ksi
Es	29000	ksi
erup	0	in/in
eu	0.003	in/in
Beta 1	0.85	

Geometry		Units
Width	20	in
Depth	20	in
Ag	400	in^2
Ix	13333.3	in^4
Iy	13333.3	in^4

Typical Exterior Column			Worst Case Exterior Column		
Reinforcement		Units	Reinforcement		Units
phi c	0.7		phi c	0.7	
phi b	0.9		phi b	0.9	
a	0.8		a	0.8	
#4 ties			#4 ties		
12 - #11			16 - #11		
As	18.72	in^2	As	24.96	in^2
cover	1.5	in	cover	1.5	in

Typical Interior Column			Worst Case Interior Column		
Reinforcement		Units	Reinforcement		Units
phi c	0.7		phi c	0.7	
phi b	0.9		phi b	0.9	
a	0.8		a	0.8	
#3 ties			#4 ties		
8 - #7			8 - #11		
As	4.8	in^2	As	12.48	in^2
cover	1.5	in	cover	1.5	in

Column Design Criteria

Service Loads

Typical Exterior Column							
		Moments about X-axis		Applied Loads		Computed Strength	
Load Case	Axial Load (kips)	at Top (ft-k)	at Bottom (ft-k)	Pu (kips)	Mu (ft-k)	Pn (kips)	Mn (ft-k)
Dead	69.12	169.56	169.56	190	497	195	503
Live	67.16	183.31	183.31				

Typical Interior Column							
		Moments about X-axis		Applied Loads		Computed Strength	
Load Case	Axial Load (kips)	at Top (ft-k)	at Bottom (ft-k)	Pu (kips)	Mu (ft-k)	Pn (kips)	Mn (ft-k)
Dead	170.03	1.16	1.16	454	254	497	280
Live	156.04	158.17	158.17				

Worst Case Exterior Column							
		Moments about X-axis		Applied Loads		Computed Strength	
Load Case	Axial Load (kips)	at Top (ft-k)	at Bottom (ft-k)	Pu (kips)	Mu (ft-k)	Pn (kips)	Mn (ft-k)
Dead	99.82	139.7	139.7	377	584	398	620
Live	160.57	260.51	260.51				

Worst Case Interior Column							
		Moments about X-axis		Applied Loads		Computed Strength	
Load Case	Axial Load (kips)	at Top (ft-k)	at Bottom (ft-k)	Pu (kips)	Mu (ft-k)	Pn (kips)	Mn (ft-k)
Dead	268.96	16.62	16.62	777	331	767	334
Live	284.02	194.28	194.28				

Typical Strip with Columns – Design Criteria

One-Way Slab				
Typical Strip with Columns				
Section Properties				
Span	Area (in²)	I (in⁴)	Yb (in)	Yt (in)
1	3216	68370	11.21	5.79
2	3216	68370	11.21	5.79
3	3216	68370	11.21	5.79

Dead Load Moments, Shears & Reactions					
Span	Span Moments (k-ft)			Span Shears (k)	
	Left	Mid	Right	Left	Right
1	-339.12	356.09	-708.54	-69.12	85.32
2	-706.23	352.63	-706.22	-84.71	84.71
3	-708.55	356.1	-339.1	-85.33	69.11

Joint	Column Moments (k-ft) Upper & Lower Columns	Reactions (k)
1	-169.56	69.12
2	1.15	170.03
3	-1.15	170.03
4	169.55	69.11

Live Load Moments (k-ft), Shears (k) & Reactions (k)								
Span	Left Moments		Midspan Moments		Right Moments		Shear Forces	
	max	min	max	min	max	min	left	right
1	-366.63	66.38	384.87	-69.59	-653.32	-179.58	-67.16	76.3
2	-691.01	-105.43	417.65	-105.44	-691	-105.44	-79.74	79.74
3	-653.33	-179.57	384.87	-69.59	-366.61	66.38	-76.3	67.16

Joint	Reactions		Upper & Lower Column Moments	
	max	min	max	min
1	67.16	-5.97	33.19	-183.32
2	156.04	69.58	158.17	-157.15
3	156.04	69.58	157.14	-158.17
4	67.16	-5.97	183.31	-33.19

Reduced Dead Load Moments (k-ft)			
Span	Left	Midspan	Right
1	-282.67	356.08	-638.58
2	-636.83	352.67	-636.83
3	-638.67	356.08	-282.67

Reduced Live Load Moments (k-ft)						
Span	Left		Midspan		Right	
	max	min	max	min	max	min
1	-311.67	61.41	384.83	-69.59	-590.75	-175.25
2	-625.58	-105.42	417.67	-105.42	-625.58	-105.42
3	-590.75	-175.25	384.83	-69.59	-311.67	61.41

Sum of Dead and Live Moments (k-ft)						
Span	Left		Midspan		Right	
	max	min	max	min	max	min
1	-594.33	-221.26	740.92	286.49	-1229.33	-813.83
2	-1262.42	-742.25	770.33	247.25	-1262.42	-742.25
3	-1229.42	-813.92	740.92	286.49	-594.33	-221.26

Post-Tensioning Balanced Moments, Shears & Reactions					
Span	Span Moments (k-ft)			Span Shears (k)	
	Left	Mid	Right	Left	Right
1	277.75	-472.75	594	1.16	1.16
2	607.67	-542.08	607.67	0	0
3	594	-472.75	277.75	-1.16	-1.16

Joint	Column Moments (k-ft) Upper & Lower Columns	Reactions (k)
1	139.333	-1.162
2	7.351	1.162
3	-7.341	1.161
4	-139.333	-1.161

Factored Design Moments (k-ft)						
1.2D +1.6L + 1.0 Secondary Moment Effects						
Span	Left		Midspan		Right	
	max	min	max	min	max	min
1	-565.61	34.26	1295.28	568.14	-1490.25	-822.49
2	-1530.9	-695.29	1331.82	494.87	-1530.84	-695.27
3	-1490.23	-822.45	1295.32	568.19	-565.56	34.28

Secondary Moments (k-ft)			
Span	Left	Midspan	Right
1	277.67	252.17	226.67
2	240.42	240.42	240.42
3	226.75	252.17	277.67

Joint	Factored Reactions (k)		Factored Column Moments (k-ft) Upper & Lower Columns	
	max	min	max	min
1	189.24	72.24	-11.06	-357.5
2	454.76	316.49	261.8	-242.73
3	454.76	316.49	242.73	-261.8
4	189.23	72.23	357.5	11.07

Forces & Stresses of Typical Strip - Columns

SELECT POST-TENSIONING FORCES AND TENDON DRAPE

Span	Force (k/-)	<Distance of CGS (in)> Left	Center	Right	P/A (psi)	Wbal (k/-)	Wbal (%DL)
--1--	2	3	4	5	6	7	8--
1	919.796	-5.79	-15.25	-1.00	286.01	4.374	129
2	919.796	-1.00	-16.00	-1.00	286.01	4.599	136
3	919.796	-1.00	-15.25	-5.79	286.01	4.374	129

Approximate weight of strand 2623.5 LB

REQUIRED MINIMUM POST-TENSIONING FORCES (kips)

SPAN	<Based on Stress Conditions> LEFT	CENTER	RIGHT	<Based on Minimum P/A> LEFT	CENTER	RIGHT
- -1--	2	3	4	5	6	7----
1	363.47	815.51	899.40	402.00	402.00	402.00
2	919.74	772.57	919.75	402.00	402.00	402.00
3	899.48	815.52	363.48	402.00	402.00	402.00

SERVICE STRESSES(psi)

	LEFT				RIGHT			
	TOP		BOTTOM		TOP		BOTTOM	
	max-T	max-C	max-T	max-C	max-T	max-C	max-T	max-C
-1-	2	3	4	5	6	7	8	9------
1	35.20	-343.97	-----	-908.55	358.90	-63.39	-----	-1535.13
2	378.56	-150.11	-----	-1573.18	378.57	-150.10	-----	-1573.19
3	358.98	-63.32	-----	-1535.27	35.21	-343.96	-----	-908.58

	CENTER			
	TOP		BOTTOM	
	max-T	max-C	max-T	max-C
-1-	2	3	4	5----------------------
1	-----	-558.12	240.71	-653.36
2	14.19	-517.45	161.99	-867.09
3	-----	-558.13	240.73	-653.33

Reinforcement & Deflections of Typical Strip - Columns

MILD STEEL

SPECIFIC CRITERIA for ONE-WAY SYSTEM
- Minimum steel - 0.004A
- Moment capacity > factored design moment

Support cut-off length for minimum steel (length/span)	0.17
Span cut-off length for minimum steel (length/span)	0 .33
Top bar extension beyond where required	12.00 in
Bottom bar extension beyond where required	12.00 in

TOTAL WEIGHT OF REBAR = 3566.1 lb AVERAGE = 0.8 psf
TOTAL AREA COVERED = 4234.8 ft²

MAXIMUM SPAN DEFLECTIONS (in) (downward positive)

Concrete`s modulus of elasticity	Ec = 3605.00 ksi
Creep factor	K = 2.00
Ieffective/Igross (due to cracking)	K = 1.00

Values in parentheses are (span/max deflection) ratios

Span	DL	DL+PT	DL+PT+CREEP	LL	DL+PT+LL+CREEP
1	2	3	4	5	6
1	.38	-.02	-.05(11258)	.34(1611)	.29(1881)
2	.38	-.09	-.28(2162)	.34(1773)	.06(9865)
3	.38	-.02	-.05(11212)	.34(1611)	.29(1882)

Typical Strip with Core – Design Criteria

One-Way Slab				
Typical Strip with Core				
Section Properties				
Span	Area (in²)	I (in⁴)	Yb (in)	Yt (in)
1	3216	68370	11.21	5.79
2	3216	68370	11.21	5.79
3	3216	68370	11.21	5.79

Dead Load Moments, Shears & Reactions					
Span	Span Moments (k-ft)			Span Shears (k)	
	Left	Mid	Right	Left	Right
1	-338.83	355.79	-709.43	-69.09	85.35
2	-705.78	353.08	-705.76	-84.71	84.71
3	-709.46	355.79	-338.81	-85.35	69.09
Joint	Column Moments (k-ft) Upper & Lower Columns			Reactions (k)	
1	-169.42			69.09	
2	1.82			170.06	
3	-1.85			170.06	
4	169.41			69.09	

Live Load Moments (k-ft), Shears (k) & Reactions (k)								
Span	Left Moments		Midspan Moments		Right Moments		Shear Forces	
	max	min	max	min	max	min	left	right
1	-327.06	27.06	343.38	-28.37	-633.2	-78.72	-63.6	75.72
2	-660.12	-42.98	355.6	-42.98	-660.1	-42.98	-77.27	77.27
3	-633.22	-78.72	343.38	-28.37	-327.04	27.06	-75.72	63.6
Joint	Reactions		Upper & Lower Column Moments					
	max	min	max		min			
1	63.6	-2.43	13.53		-163.53			
2	152.99	73.14	250.67		-249.05			
3	152.99	73.14	249.04		-250.68			
4	63.6	-2.43	163.52		-13.53			

Reduced Dead Load Moments (k-ft)			
Span	Left	Midspan	Right
1	-282.42	355.83	-667.17
2	-663.83	353.08	-663.83
3	-667.17	355.75	-282.42

Reduced Live Load Moments (k-ft)						
Span	Left		Midspan		Right	
	max	min	max	min	max	min
1	-275.08	25.03	343.42	-28.37	-595.75	-77.58
2	-621.83	-42.98	355.58	-42.98	-621.83	-42.98
3	-595.75	-77.58	343.42	-28.37	-275.08	25.03

Sum of Dead and Live Moments (k-ft)						
Span	Left		Midspan		Right	
	max	min	max	min	max	min
1	-557.5	-257.38	699.25	327.47	-1262.92	-744.7
2	-1285.67	-706.81	708.67	310.1	-1285.67	-706.82
3	-1262.92	-744.74	699.17	327.38	-557.5	-257.38

Post-Tensioning Balanced Moments, Shears & Reactions					
Span	Span Moments (k-ft)			Span Shears (k)	
	Left	Mid	Right	Left	Right
1	286	-485.75	601.67	1.36	1.36
2	624.83	-551.67	624.83	0	0
3	601.67	-485.75	285.92	-1.36	-1.36
Joint	Column Moments (k-ft) Upper & Lower Columns			Reactions (k)	
1	143.583			-1.358	
2	11.925			1.358	
3	-11.9			1.358	
4	-143.583			-1.357	

Factored Design Moments (k-ft)						
1.2D +1.6L + 1.0 Secondary Moment Effects						
Span	Left		Midspan		Right	
	max	min	max	min	max	min
1	-498.38	-15.28	1232.58	637.78	-1531.76	-700.6
2	-1546.91	-618.3	1241.78	604.05	-1546.86	-618.29
3	-1531.78	-700.59	1232.6	637.8	-498.33	-15.26

Secondary Moments (k-ft)			
Span	Left	Midspan	Right
1	286	256.25	225.92
2	249.08	249.08	249.08
3	226	256.25	286

Joint	Factored Reactions (k)		Factored Column Moments (k-ft) Upper & Lower Columns	
	max	min	max	min
1	183.31	77.66	-38.08	-321.32
2	450.28	322.5	415.18	-384.42
3	450.28	322.5	384.42	-415.18
4	183.31	77.66	321.32	38.08

Forces & Stresses of Typical Strip - Core

SELECTED POST-TENSIONING FORCES AND TENDON DRAPE

Span	Force (k/-)	<Distance of CGS (in)> Left	Center	Right	P/A (psi)	Wbal (k/-)	Wbal (%DL)
-1-	-2-	-3-	-4-	-5-	-6-	-7-	-8-
1	941.193	-5.79	-15.25	-1.00	292.66	4.475	132
2	941.193	-1.00	-16.00	-1.00	292.66	4.706	139
3	941.193	-1.00	-15.25	-5.79	292.66	4.475	132

Approximate weight of strand .... 2698.5 LB

REQUIRED MINIMUM POST-TENSIONING FORCES (kips)

Span	<Based on Stress Conditions> Left	Center	Right	<Based on Minimum P/A> Left	Center	Right
-1-	-2-	-3-	-4-	-5-	-6-	-7-
1	301.97	750.98	941.14	402.00	402.00	402.00
2	940.72	693.07	940.74	402.00	402.00	402.00
3	941.12	750.98	301.98	402.00	402.00	402.00

SERVICE STRESSES (psi)

	LEFT				RIGHT			
	TOP		BOTTOM		TOP		BOTTOM	
	max-T	max-C	max-T	max-C	max-T	max-C	max-T	max-C
-1-	-2-	-3-	-4-	-5-	-6-	-7-	-8-	-9-
1	--------	-322.26	-----	-826.59	378.57	-148.07	-----	-1592.74
2	378.12	-210.20	-----	-1591.88	378.14	-210.17	-----	-1591.92
3	378.55	-148.10	-----	-1592.70	------	-322.25	-----	-826.63

	CENTER			
	TOP		BOTTOM	
	max-T	max-C	max-T	max-C
-1-	-2-	-3-	-4-	-5-
1	-----	-509.10	126.30	-605.04
2	-----	-451.68	15.16	-768.97
3	-----	-509.10	126.30	-605.04

Reinforcement & Deflections of Typical Strip - Core

MILD STEEL

SPECIFIC CRITERIA for ONE-WAY SYSTEM
- Minimum steel - 0.004A
- Moment capacity > factored design moment

Support cut-off length for minimum steel (length/span)	0.17
Span cut-off length for minimum steel (length/span)	0.33
Top bar extension beyond where required	12.00 in
Bottom bar extension beyond where required	12.00 in

TOTAL WEIGHT OF REBAR = 3566.10 lb AVERAGE = 0.8 psf
TOTAL AREA COVERED = 4234.79 ft^2

MAXIMUM SPAN DEFLECTIONS(in) (downward positive)

Concrete`s modulus of elasticity	Ec = 3604.00 ksi
Creep factor	K = 2.00
Ieffective/Igross (due to cracking)	K = 1.00

Values in parentheses are (span/max deflection) ratios

Span	DL	DL+PT	DL+PT+CREEP	LL	DL+PT+LL+CREEP
--1--	--2--	--3--	--4--	--5--	--6--
1	.38	-.03	-.09(6052)	.34(1613)	.25(2199)
2	.38	-.10	-.29(2045)	.34(1768)	.05(13080)
3	.38	-.03	-.09(6052)	.34(1613)	.25(2199)

Worst Case Loads in Shear Walls

W-E Wind Load (kip)	Seismic Force (kip)	N-S Wind Load (kip)	Seismic Force (kip)	W-E Worst Case Load (kip)	W-E Shear Wall Load (kip)	W-E Shear Wall Load Cum (kip)	N-S Worst Case Load (kip)	N-S Shear Wall Load (kip)	N-S Shear Wall Load Cum (kip)
70.2	85.67	54.6	85.67	85.67	42.83	42.83	85.67	42.83	42.83
109.5	141.98	85.2	141.98	141.98	70.99	113.82	141.98	70.99	113.82
75.69	63.89	58.87	63.89	75.69	37.85	151.67	63.89	31.95	145.77
73.08	63.03	56.84	63.03	73.08	36.54	188.21	63.03	31.52	177.29
70.47	56.81	54.81	56.81	70.47	35.24	223.44	56.81	28.41	205.69
67.86	53.66	52.78	53.66	67.86	33.93	257.37	53.66	26.83	232.52
67.86	50.79	52.78	50.79	67.86	33.93	291.30	52.78	26.39	258.91
67.86	48.00	52.78	48.00	67.86	33.93	325.23	52.78	26.39	285.30
67.7	45.28	52.6	45.28	67.7	33.85	359.08	52.6	26.30	311.60
63.18	42.64	49.14	42.64	63.18	31.59	390.67	49.14	24.57	336.17
63.18	40.06	49.14	40.06	63.18	31.59	422.26	49.14	24.57	360.74
63.18	37.57	49.14	37.57	63.18	31.59	453.85	49.14	24.57	385.31
63.18	35.14	49.14	35.14	63.18	31.59	485.44	49.14	24.57	409.88
63.18	32.79	49.14	32.79	63.18	31.59	517.03	49.14	24.57	434.45
63.18	30.52	49.14	30.52	63.18	21.06	538.09	49.14	24.57	459.02
63.18	28.32	49.14	28.32	63.18	21.06	559.15	49.14	24.57	483.59
61.6	26.19	47.9	26.19	61.6	20.53	579.69	47.9	23.95	507.54
58.5	24.14	45.5	24.14	58.5	19.50	599.19	45.5	22.75	530.29
58.5	22.17	45.5	22.17	58.5	19.50	618.69	45.5	22.75	553.04
58.5	20.28	45.5	20.28	58.5	19.50	638.19	45.5	22.75	575.79
58.5	18.46	45.5	18.46	58.5	19.50	657.69	45.5	22.75	598.54
58.5	16.73	45.5	16.73	58.5	19.50	677.19	45.5	22.75	621.29
58.5	15.07	45.5	15.07	58.5	19.50	696.69	45.5	22.75	644.04
58.5	13.49	45.5	13.49	58.5	19.50	716.19	45.5	22.75	666.79
55.4	11.99	43.1	11.99	55.4	18.47	734.65	43.1	21.55	688.34
53.82	10.58	41.86	10.58	53.82	17.94	752.59	41.86	20.93	709.27
53.82	9.24	41.86	9.24	53.82	17.94	770.53	41.86	20.93	730.20
53.82	7.99	41.86	7.99	53.82	17.94	788.47	41.86	20.93	751.13
53.82	6.83	41.86	6.83	53.82	17.94	806.41	41.86	20.93	772.06
53.82	5.74	41.86	5.74	53.82	17.94	824.35	41.86	20.93	792.99
53.82	4.75	41.86	4.75	53.82	17.94	842.29	41.86	20.93	813.92
53.82	3.84	41.86	3.84	53.82	17.94	860.23	41.86	20.93	834.85
53.82	3.02	41.86	3.02	53.82	17.94	878.17	41.86	20.93	855.78
47.61	3.88	37.03	3.88	47.61	15.87	894.04	37.03	18.52	874.30
41.4	2.93	32.2	2.93	41.4	13.80	907.84	32.2	16.10	890.40
41.4	2.21	32.2	2.21	41.4	13.80	921.64	32.2	16.10	906.50
43.47	1.61	33.81	1.61	43.47	14.49	936.13	33.81	16.91	923.40
54.85	1.28	42.67	1.28	54.85	18.28	954.42	42.67	21.34	944.74
82.8	0.61	64.4	0.61	82.8	27.60	982.02	64.4	32.20	976.94

Torsional Moments for Worst Case Loads

Story	5% Offset (ft)	W-E Worst Case Load (kip)	5% Offset (ft)	N-S Worst Case Load (kip)	W-E Torsional Moment (ft-kip)	N-S Torsional Moment (ft-kip)
41	14.417	85.67	7.175	85.67	-1235.09	-614.67
39	14.417	141.98	7.175	141.98	-2046.94	-1018.71
38	14.417	75.69	7.175	63.89	-1091.22	-458.42
37	14.417	73.08	7.175	63.03	-1053.59	-452.26
36	14.417	70.47	7.175	56.81	-1015.97	-407.62
35	14.417	67.86	7.175	53.66	-978.34	-384.98
34	14.417	67.86	7.175	52.78	-978.34	-378.70
33	14.417	67.86	7.175	52.78	-978.34	-378.70
32	14.417	67.7	7.175	52.6	-976.03	-377.41
31	14.417	63.18	7.175	49.14	-910.87	-352.58
30	14.417	63.18	7.175	49.14	-910.87	-352.58
29	14.417	63.18	7.175	49.14	-910.87	-352.58
28	14.417	63.18	7.175	49.14	-910.87	-352.58
27	14.417	63.18	7.175	49.14	-910.87	-352.58
26	9.175	63.18	7.175	49.14	-579.68	-352.58
25	9.175	63.18	7.175	49.14	-579.68	-352.58
24	9.175	61.6	7.175	47.9	-565.18	-343.68
23	9.175	58.5	7.175	45.5	-536.74	-326.46
22	9.175	58.5	7.175	45.5	-536.74	-326.46
21	9.175	58.5	7.175	45.5	-536.74	-326.46
20	9.175	58.5	7.175	45.5	-536.74	-326.46
19	9.175	58.5	7.175	45.5	-536.74	-326.46
18	9.175	58.5	7.175	45.5	-536.74	-326.46
17	9.175	58.5	7.175	45.5	-536.74	-326.46
16	9.175	55.4	7.175	43.1	-508.30	-309.24
15	9.175	53.82	7.175	41.86	-493.80	-300.35
14	9.175	53.82	7.175	41.86	-493.80	-300.35
13	9.175	53.82	7.175	41.86	-493.80	-300.35
12	9.175	53.82	7.175	41.86	-493.80	-300.35
11	9.175	53.82	7.175	41.86	-493.80	-300.35
10	9.175	53.82	7.175	41.86	-493.80	-300.35
9	9.175	53.82	7.175	41.86	-493.80	-300.35
8	9.175	53.82	7.175	41.86	-493.80	-300.35
7	9.175	47.61	7.175	37.03	-436.82	-265.69
6	9.175	41.4	7.175	32.2	-379.85	-231.04
5	9.175	41.4	7.175	32.2	-379.85	-231.04
4	9.175	43.47	7.175	33.81	-398.84	-242.59
3	9.175	54.85	7.175	42.67	-503.25	-306.16
2	9.175	82.8	7.175	64.4	-759.69	-462.07

Worst Case Loads with Torsion in Shear Walls

W-E Shear Wall Load (kip)	%5 W-E Torsion Worst Case	N-S Shear Wall Load (kip)	%5 N-S Torsion Worst Case	Worst Case W-E Shear Wall Load (kip)	Worst Case N-S Shear Wall Load (kip)	Worst Case W-E Shear Wall Load Cum (kip)	Worst Case W-E Shear Wall Load Cum (kip)
42.83	14.16	42.83	10.04	56.99	52.87	56.99	52.87
70.99	23.46	70.99	16.64	94.45	87.63	151.45	140.50
37.85	12.51	31.95	7.49	50.35	39.43	201.80	179.93
36.54	12.08	31.52	7.39	48.62	38.90	250.42	218.84
35.24	11.65	28.41	6.66	46.88	35.06	297.30	253.90
33.93	11.21	26.83	6.29	45.14	33.12	342.44	287.01
33.93	11.21	26.39	6.18	45.14	32.57	387.59	319.59
33.93	11.21	26.39	6.18	45.14	32.57	432.73	352.16
33.85	11.19	26.30	6.16	45.04	32.46	477.77	384.63
31.59	10.44	24.57	5.76	42.03	30.33	519.80	414.96
31.59	10.44	24.57	5.76	42.03	30.33	561.83	445.28
31.59	10.44	24.57	5.76	42.03	30.33	603.86	475.61
31.59	10.44	24.57	5.76	42.03	30.33	645.90	505.94
31.59	10.44	24.57	5.76	42.03	30.33	687.93	536.27
21.06	5.70	24.57	4.69	26.76	29.26	714.68	565.53
21.06	5.70	24.57	4.69	26.76	29.26	741.44	594.79
20.53	5.56	23.95	4.57	26.09	28.52	767.53	623.32
19.50	5.28	22.75	4.34	24.78	27.09	792.31	650.41
19.50	5.28	22.75	4.34	24.78	27.09	817.08	677.50
19.50	5.28	22.75	4.34	24.78	27.09	841.86	704.60
19.50	5.28	22.75	4.34	24.78	27.09	866.64	731.69
19.50	5.28	22.75	4.34	24.78	27.09	891.41	758.79
19.50	5.28	22.75	4.34	24.78	27.09	916.19	785.88
19.50	5.28	22.75	4.34	24.78	27.09	940.96	812.97
18.47	5.00	21.55	4.12	23.46	25.67	964.43	838.64
17.94	4.85	20.93	4.00	22.79	24.93	987.22	863.57
17.94	4.85	20.93	3.76	22.79	24.69	1010.02	888.26
17.94	4.85	20.93	3.74	22.79	24.67	1032.81	912.93
17.94	5.24	20.93	3.72	23.18	24.65	1055.99	937.57
17.94	5.27	20.93	3.69	23.21	24.62	1079.20	962.19
17.94	5.31	20.93	3.67	23.25	24.60	1102.45	986.79
17.94	5.35	20.93	3.64	23.29	24.57	1125.74	1011.36
17.94	5.38	20.93	3.61	23.32	24.54	1149.06	1035.90
15.87	4.80	18.52	3.17	20.67	21.68	1169.73	1057.58
13.80	3.65	16.10	3.13	17.45	19.23	1187.19	1076.82
13.80	3.68	16.10	3.12	17.48	19.22	1204.66	1096.03
14.49	3.88	16.91	3.26	18.37	20.16	1223.03	1116.19
18.28	4.92	21.34	4.09	23.21	25.43	1246.24	1141.62
27.60	7.48	32.20	6.14	35.08	38.34	1281.32	1179.96

APPENDIX D

MECHANICAL BREADTH STUDY

Monthly Profiles

LOAD-LEVELING

HOUR	JULY TOTAL COOLING (Tons) Non-Storage	JULY TOTAL COOLING (Tons) Ice-Storage
0	134	1167
100	134	1167
200	134	1167
300	134	1167
400	134	1167
500	134	1167
600	968	1167
700	1200	1167
800	1400	1167
900	1500	1167
1000	1800	1167
1100	2670	1167
1200	2670	1167
1300	2670	1167
1400	2670	1167
1500	2670	1167
1600	2590	1167
1700	2000	1167
1800	1060	1167
1900	800	1167
2000	134	1167
2100	134	1167
2200	134	1167
2300	134	1167
Total ton-hrs	28008.0	28008.0

Base Chiller size:	584	tons
Ice Chiller size:	584	tons
Ice distribution:	10804	ton-hrs

DEMAND-LIMITING

HOUR	JULY TOTAL COOLING (Tons) Non-Storage	JULY TOTAL COOLING (Tons) Ice-Storage
0	134	1373
100	134	1373
200	134	1373
300	134	1373
400	134	1373
500	134	1373
600	968	961
700	1200	961
800	1400	961
900	1500	961
1000	1800	961
1100	2670	961
1200	2670	961
1300	2670	961
1400	2670	961
1500	2670	961
1600	2590	961
1700	2000	961
1800	1060	1373
1900	800	1373
2000	134	1373
2100	134	1373
2200	134	1373
2300	134	1373
Total ton-hrs	28008.0	28008.0

Base Chiller size:	962	tons
Ice Chiller size:	412	tons
Ice distribution:	13276	ton-hrs

# of on-peak hours	12
# of off-peak hours	12

On-peak Demand Charge:	5.75 ¢/kWh
Off-peak Demand Charge:	2.49 ¢/kWh

Design Day Load Profiles

Load-Leveling Design Day Load Profile

	Load			Load			Load	
Hour	Tons	kW	Hour	Tons	kW	Hour	Tons	kW
1	134	471.28	9	1500	5275.50	17	2000	7034.00
2	134	471.28	10	1800	6330.60	18	1060	3728.02
3	134	471.28	11	2670	9390.39	19	800	2813.60
4	134	471.28	12	2670	9390.39	20	134	471.28
5	134	471.28	13	2670	9390.39	21	134	471.28
6	968	3404.46	14	2670	9390.39	22	134	471.28
7	1200	4220.40	15	2670	9390.39	23	134	471.28
8	1400	4923.80	16	2590	9109.03	24	134	471.28

# of on-peak hours	12
# of off-peak hours	12

	Ton-hr	kWh
On-peak	24,808.00	87,249.74
Off-Peak	3,200.00	11,254.40
Total Load	28,008.00	98,504.14

Demand-Limiting Design Day Load Profile

	Load			Load			Load	
Hour	Tons	kW	Hour	Tons	kW	Hour	Tons	kW
1	134	471.28	9	1500	5275.50	17	2000	7034.00
2	134	471.28	10	1800	6330.60	18	1060	3728.02
3	134	471.28	11	2670	9390.39	19	800	2813.60
4	134	471.28	12	2670	9390.39	20	134	471.28
5	134	471.28	13	2670	9390.39	21	134	471.28
6	968	3404.46	14	2670	9390.39	22	134	471.28
7	1200	4220.40	15	2670	9390.39	23	134	471.28
8	1400	4923.80	16	2589.9	9108.68	24	134	471.28

# of on-peak hours	12
# of off-peak hours	12

	Ton-hr	kWh
On-peak	24,807.90	87,249.38
Off-Peak	3,200.00	11,254.40
Total Load	28,007.90	98,503.78

Carrier
Turn to the Experts.

19XR EVERGREEN®

High-Efficiency Hermetic R-134a Centrifugal Chiller

19XR - Base unit:
200 to 1,500 Nominal Tons (703 to 5275 kW)

19XRV - w/unit-mounted variable frequency drive:
200 to 800 Nominal Tons (703 to 2813 kW)

Nominal Capacity (Tons) †	Heat Exchanger Size	Length with Nozzle-in-Head Waterbox				Width		Height **	
		2-Pass*		1 or 3-Pass					
		ft-in.	mm	ft-in.	mm	ft-in.	mm	ft-in.	mm
200	10 TO 12	11-4	3454	11-11	3632	5-2 7/8	1597	6-1 1/4	1861
225	15 TO 17	13-7 1/2	4153	14-2 1/2	4331	5-2 7/8	1597	6-1 1/4	1861
250	20 TO 22	11-4 3/4	3473	11-11 3/4	3651	5-6 7/8	1688	6-3 1/4	1911
300	30 TO 32	13-8 1/4	4172	14-3 1/4	4350	5-6 7/16	1688	6-3 1/4	1911
350	35 TO 37	15-4 3/4	4693	15-11 3/4	4870	5-6 7/16	1688	6-3 1/4	1911
450	40 TO 42	14-3 1/8	4347	14-9	4496	6-3 1/8	1908	7-0 3/4	2153
500	45 TO 47	15-8 5/8	4867	16-5 1/2	5017	6-3 1/8	1908	7-0 3/4	2153
550	50 TO 52	14-4 1/2	4382	14-10	4521	6-6 1/2	1994	7-1	2159
600	55 TO 57	16-1	4902	16-6 1/2	5042	6-6 1/2	1994	7-1	2159
700	60 TO 62	14-5 1/4	4401	14-11	4547	6-11 5/8	2124	7-4 7/8	2257
800	65 TO 67	16-1 3/4	4921	16-7 1/2	5067	6-11 5/8	2124	7-4 7/8	2257
1000	70 TO 72	16-6 1/8	5032	17-0 1/2	5194	8-1 1/4	2470	9-9 1/2	2985
1200	75 TO 77	18-6 1/8	5642	19-0 1/2	5804	8-1 1/4	2470	9-9 1/2	2985
1300	80 TO 82	16-9 1/8	5109	17-3 1/2	5271	8-10 3/4	2711	9-11 1/4	3029
1500	85 TO 87	18-9 1/8	5718	19-3 1/2	5880	8-10 3/4	2711	9-11 1/4	3029

*Assumes all customer connections are on the same end of the chiller.
**Height is for chiller without unit-mounted starter.
†Heat exchangers are rated for typical applications at ARI Conditions (44F leaving chilled water and 2.4 gpm/ton in the cooler, and 85F entering condenser water with 3.0 gpm/ton flow in the condenser).

Pump Sizing Calculations

LOAD-LEVELING

Charge Loop Design		
25% Ethelene Glycol	Tons Required =	1033

ρ (Glycol)	ρ (Water)	Fraction of Glycol	Fraction of Water	ρ (Total)	ρ (Total)
Kg/m^3	Kg/m^3			Kg/m^3	Lb/ft^3
1096.78	1000	0.25	0.75	1024.195	63.94

Charge	Specific Heat	Density	Ice Bank Temp Drop	Conversion	Flow Rate
Btu/Hr	Btu/(lb ºF)	Lb/ft^3	ºF	gpm/cfh	Gpm
12,396,000.00	0.91	63.94	6.00	0.12	4247

DEMAND-LIMITING

Charge Loop Design		
25% Ethelene Glycol	Tons Required =	412

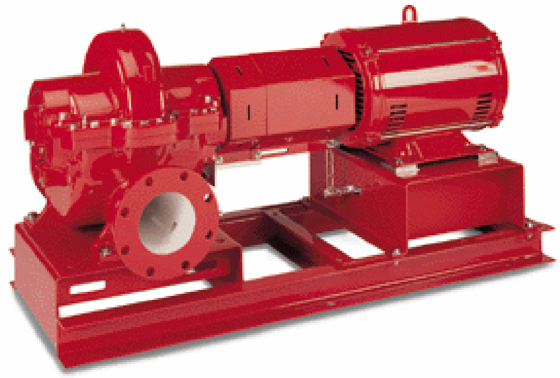
ρ (Glycol)	ρ (Water)	Fraction of Glycol	Fraction of Water	ρ (Total)	ρ (Total)
Kg/m^3	Kg/m^3			Kg/m^3	Lb/ft^3
1096.78	1000	0.25	0.75	1024.195	63.94

Charge	Specific Heat	Density	Ice Bank Temp Drop	Conversion	Flow Rate
Btu/Hr	Btu/(lb ºF)	Lb/ft^3	ºF	gpm/cfh	Gpm
4,944,000.00	0.91	63.94	6.00	0.12	1694

LOAD-LEVELING PUMP

Series HSC³

Double-suction, base-mounted Series HSC³ pumps are available in 2" through 10" sizes. Motor sizes through 300 HP. Flows to 6500 GPM and heads to 400 feet. Bronze-fitted construction with a maximum working pressure of 175 psig.



The HSC³ features internally self-flushing mechanical seals for maximum lubrication, debris removal and heat dissipation. The internally flushed seals allow for a shorter shaft for reduced deflection for longer seallife. Closed-end steel baseplate with full seam welds for strength and an open top for easy grouting. The pump bearings and mechanical seals can be serviced without disturbing the upper casing half, piping connections or electrical motor connections. An ANSI-OSHA-compliant coupling guard shields the flexible coupler.

APPLICATIONS:

HYDRONIC HEATING AND COOLING SYSTEMS, GENERAL SERVICE AND INDUSTRIAL.

DEMAND-LIMITING PUMP

Series 1531

End suction, close-coupled, foot-mounted pump available in 1-1/4" through 6" sizes. Motor size 3/4 to 50 HP at 1750 RPM, 2 to 60 HP at 3500 RPM. Available in bronze-fitted, all-iron and all-bronze construction with flows to 2300 GPM, heads to 400 feet. Close-coupled, space-saving economy plus rugged construction make the Series 1531 an ideal selection for a variety of horizontal and vertical mount applications. Available with the B&G standard self-flushing Carbon/Ceramic mechanical seal in 175# working pressure design.

Features

- Back pullout design allows ease of maintenance.
- Self-flushing mechanical seals.
- Aluminum bronze shaft sleeve.
- Enclosed, balanced impeller for quiet, vibration free performance.
- Heavy duty cast iron volute construction for 175 PSI working pressure.
- Jacking bolts provide ease of volute disassembly.
- Gauge tappings.
- Hydrostatic testing of each pump is standard.

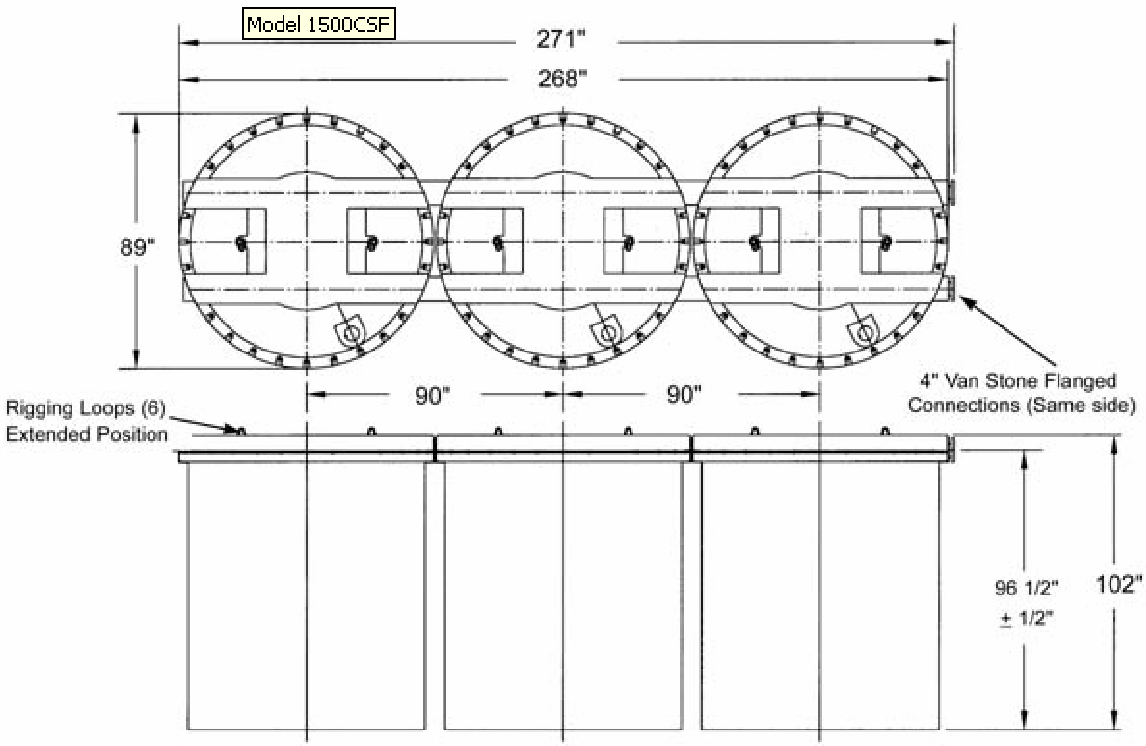
APPLICATIONS:

Hydronic heating and cooling systems, general service, cooling towers and industrial uses.

Bell & Gossett Domestic Pump Hoffman McDonnell & Miller

ICE BANK

Thermal Energy Storage



Off-Peak Cooling - ICEBANK Model C

Specifications

TANK MODELS	1500C
Total Storage Capacity, Ton-Hr	570
With Mix Air	550
Latent Capacity Ton-Hr	486
With Mix Air	469
Sensible Capacity Ton-Hr	84
With Mix Air	81
Max. Operating Temp.,°F	100

TANK MODELS	1500C
Factory Tested Pres., PSI	250
Max. Operating Pres., PSI	90
Dimensions (WxLxH), in.	89x273x102(RF) 89x271x102(SF)
Shipping Weight,lb	6,000
Weight, Filled, lb	50,600
Floor Loading, lb/Sq Ft	391

Monthly Mechanical Costs

LOAD-LEVELING

SYSTEM	JANUARY
Non-Storage	
On-Peak Demand:	5016.86
Off-Peak Demand:	280.23
Totals:	$5,297.09
Ice-Storage	
On-Peak Demand:	2831.99
Off-Peak Demand:	1226.38
Totals:	$4,058.37
Difference:	1,238.72

Average Monthly Savings: $1,238.72

23% savings

1 ton-hr = 3.5168 kWh

Mechanical Equipment Additional Costs

Storage Tanks:	**$1,080,400.00**	@ $100 per ton-hr of storage
Additional Pump:		per Manufacturer's Specifications
Chiller Savings:	**$560,215.00**	
Total:	**$520,185.00**	

Payback Analysis

First Cost	**$520,185.00**
Monthly Savings:	**$1,238.72**
Pay Back:	**420 months**

DEMAND-LIMITING

SYSTEM	JANUARY
Non-Storage	
On-Peak Demand:	5016.86
Off-Peak Demand:	280.23
Totals:	$5,297.09
Ice-Storage	
On-Peak Demand:	2332.33
Off-Peak Demand:	1442.86
Totals:	$3,775.19
Difference:	1,521.91

Average Monthly Savings: $1,521.91

29% savings

1 ton-hr = 3.5168 kWh

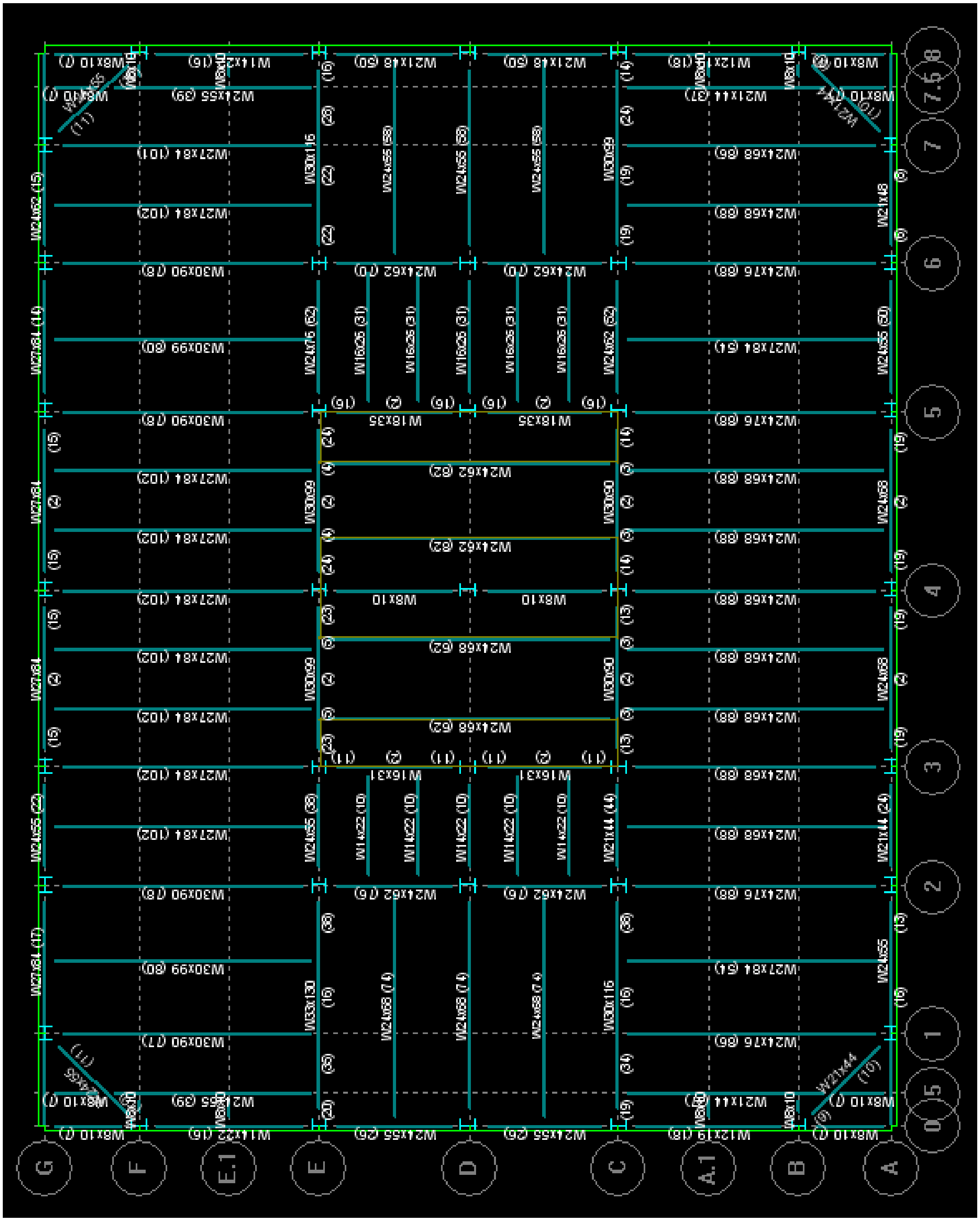
Mechanical Equipment Additional Costs

Storage Tanks:	**$1,327,600.00**	@ $100 per ton-hr of storage
Additional Pump:		per Manufacturer's Specifications
Chiller Savings:	**$497,222.00**	
Total:	**$830,378.00**	

Payback Analysis

First Cost:	**$830,378.00**
Monthly Savings:	**$1,521.91**
Pay Back:	**546 months**

Mechanical – 7th Floor Structural Redesign



Mechanical – 7th Floor Structural Redesign

Gravity Beam Design Takeoff

RAM Steel v8.1
Building Code: IBC

Steel Code: AISC LRFD

STEEL BEAM DESIGN TAKEOFF:

Floor Type: Typical Floor
Story Level 1
Steel Grade: 50

SIZE	#	LENGTH (ft)	WEIGHT (lbs)
W8X10	18	197.00	1984
W16X26	9	220.00	5749
W18X40	5	125.00	5019
W21X44	2	50.00	2212
W24X55	2	50.00	2773
W24X62	8	282.33	17581
W24X68	8	228.15	15605
W24X76	2	71.17	5425
W27X84	11	337.92	28517
W30X90	31	1212.25	108901
W30X99	8	364.67	36110
W30X108	8	302.33	32612
W33X130	2	71.17	9275
W40X149	2	81.17	12097
	-----		-------------
	116		**283860**

Total Number of Studs = **5404**

APPENDIX E

CONSTRUCTION MANAGEMENT BREADTH STUDY

Post-Tension Redesign

Duration & Cost Estimate I

							Duration	Total Costs
Item No.		Cost	Unit	Crew	Daily-Output	Quantity	Crew Weeks	$
03110	**Structural C.I.P. Forms**							
	Forms in place, Columns							
7700	Steel Framed Plywood, 4 use/mo., rent, 20"x 20"	1.53	SFCA	C-1	420	24594	12	37629
	Forms in place, Elevated slabs							
1150	Flat plate, job-built plywood, to 15' high, 4 use	0.81	SF	(6) C-2	560	865540	52	701087
03210	**Reinforcing Steel**							
	Reinforcing in place, A615 Grade 60, incl. reinf. access							
0250	Columns, #8 to #18	555	Ton	4 Rodm	2.3	288	25	159840
0400	Elevated slabs, #4 to #7	605	Ton	4 Rodm	2.9	407	28	246235
03230	**Stressing Tendons**							
	Prestressing steel, post-tensioned in field							
1250	Ungrouted strand, 50' span, 300 kip	2.10	Lb	(2) C-4	1475	603858	41	1268102
03310	**Structural Concrete**							
	Concrete, Ready Mix, Normal weight							
0300	4000 psi	76.50	CY	-	-	53692	49	4107438
Turner	**Shear Walls**							
03300	Core walls - floors 1-25	590.88	CY	-	-	5649	27	3337856
03300	Core walls - floors 26-40	722.58	CY	-	-	2043	10	1476235
							Total =	$11,334,422

Existing Design
Duration & Cost Estimate

Item No.		Cost	Unit	Quantity	Duration Crew Weeks	Total Costs $
Turner	**Shear Walls**					
03300	Core walls - floors 1-25	590.88	CY	5649	27	3337856
03300	Core walls - floors 26-40	722.58	CY	2043	10	1476235
Turner	**Structural Steel**					
05120	Floor Framing / Columns	1639.79	Ton	6510	32	10675000
Turner	**Steel Deck**					
05120	Metal Decking	1.85	SF	865540	33	1604426
Turner	**Metal Fastenings**					
05120	Shear Studs	2.38	Each	130200	-	309420
Turner	**Concrete**					
03300	Concrete fill on metal deck	3.85	SF	865540	31	3338272
Turner	**Fireproofing**					
07810	Spray-on Fireproofing	1.11	SF	1022430	28	1137149
					Total =	$21,878,358

Outrigger Redesign
Duration & Cost Estimate

Item No.		Cost	Unit	Quantity	Duration Crew Days	Total Costs $
Turner	**Structural Steel**					
05120	Floor Framing / Columns	1639.79	Ton	6785	36	11125975
05120	Braced Core / Outrigger	1639.79	Ton	2634	-	4319207
Turner	**Steel Deck**					
05120	Metal Decking	1.85	SF	865540	33	1604426
Turner	**Metal Fastenings**					
05120	Shear Studs	2.38	Each	154815	-	368460
Turner	**Concrete**					
03300	Concrete fill on metal deck	3.85	SF	865540	31	3338272
Turner	**Fireproofing**					
07810	Spray-on Fireproofing	1.11	SF	1022430	28	1137149
					Total =	$18,555,217

PT Redesign
Duration & Cost Estimate II

Item No.		Cost	Unit	Quantity	Duration Crew Days	Total Costs $
Turner	**Post-Tensioned Concrete**					
03300	Concrete incl. formwork, rebar, tendons	25.69	SF	970629.5	-	24935472
Turner	**Shear Walls**					
03300	Core walls - floors 1-25	590.88	CY	5649	27	3337856
03300	Core walls - floors 26-40	722.58	CY	2043	10	1476235
					Total =	$29,749,563

Steel Braced Frame – W-E

PROFILE	LENGTH(ft)	WEIGHT(k)
W18X76	165.00	12.496
W30X90	103.00	9.234
W14X120	116.17	13.926
W14X99	145.58	14.387
W14X90	297.82	26.802
W16X67	126.00	8.430
W14X61	89.00	5.410
W18X86	95.58	8.212
W12X58	157.42	9.088
W12X79	180.79	14.244
W12X53	216.61	11.476
W12X72	241.43	17.300
W14X48	64.00	3.065
W12X65	574.93	37.292
W12X45	106.36	4.768
W14X43	237.53	10.164
W12X40	107.36	4.302
W10X39	440.11	17.188
W14X38	53.18	2.023
W8X35	135.99	4.757
W10X33	281.36	9.278
W10X49	438.22	21.430
W8X31	337.98	10.479
W8X48	81.36	3.896
W10X45	25.00	1.129
W12X30	51.00	1.522
W8X28	184.53	5.170
W12X50	25.00	1.248
W21X62	39.00	2.424
W14X34	41.00	1.392
W16X77	41.00	3.147
W8X18	53.00	0.947
W18X60	27.00	1.614
W14X74	27.00	1.999
W6X15	14.00	0.211
W24X62	37.00	2.287
W10X17	13.00	0.220
W21X68	82.50	5.603
W21X83	104.00	8.582
W8X21	39.00	0.816
W24X68	195.00	13.311
W10X22	39.00	0.860
W24X76	65.00	4.945
W10X26	52.00	1.344
W12X26	39.00	1.013
W24X84	52.00	4.362
W27X84	63.50	5.348
W30X99	65.00	6.424
W16X36	13.00	0.468
W21X111	39.00	4.331
W16X40	48.79	1.955
W33X118	39.00	4.596
W30X108	39.00	4.198
W33X130	65.00	8.454
W30X116	39.00	4.530

PROFILE	LENGTH(ft)	WEIGHT(k)
W33X141	39.00	5.510
W30X124	13.00	1.611
W24X146	26.00	3.797
W21X50	13.00	0.649
W27X161	553.41	89.083
W18X50	27.00	1.348
W30X173	1334.91	230.294
W40X149	39.00	5.801
W18X55	26.00	1.430
W27X178	39.00	6.927
W36X160	59.00	9.417
W24X55	20.00	1.100
W24X192	56.00	10.707
W40X167	81.00	13.506
W27X194	44.00	8.517
W36X182	10.00	1.820
W24X207	10.00	2.061
W40X183	11.00	2.006
W30X211	26.50	5.580
W36X210	15.50	3.253
W27X258	15.50	3.985
W40X249	24.50	6.099
W30X357	24.50	8.653
W8X40	36.07	1.433
W6X20	51.44	1.026
W8X24	85.01	2.044
W10X54	28.18	1.512
W10X60	28.18	1.684
W12X87	72.90	6.337
W14X159	35.00	5.551
W12X96	35.00	3.352
W14X109	91.17	9.907
W24X117	45.58	5.325
W24X131	149.75	19.579
W14X145	182.33	26.440
W21X132	45.58	6.006
W27X146	299.50	43.633
W27X94	13.00	1.223
W18X158	91.17	14.334
W24X104	13.00	1.351
W21X122	13.00	1.585
W30X191	273.50	52.106
W36X135	26.00	3.505
W24X176	26.00	4.565
W36X150	13.00	1.951
W18X130	45.58	5.913
W14X233	24.50	5.699
W21X73	15.00	1.095

TOTAL = 1034.406^k

Steel Braced Frame – N-S

PROFILE	LENGTH(ft)	WEIGHT(k)
W36X160	102.00	16.280
W36X182	20.00	3.641
W33X118	175.33	20.661
W33X130	288.28	37.495
W40X149	10.00	1.487
W12X53	148.51	7.867
W30X211	24.50	5.159
W27X146	65.92	9.604
W30X191	52.92	10.083
W30X90	367.87	32.981
W30X108	119.00	12.811
W30X99	295.11	29.164
W18X76	404.59	30.640
W24X84	463.30	38.863
W27X114	107.33	12.211
W14X34	159.30	5.410
W14X145	31.00	4.495
W24X104	678.95	70.555
W21X122	406.70	49.583
W10X22	86.23	1.901
W18X130	11.00	1.427
W24X76	275.47	20.955
W27X94	121.97	11.473
W21X62	220.60	13.710
W27X84	327.56	27.587
W24X62	232.80	14.389
W24X68	299.89	20.470
W6X9	10.00	0.091
W21X68	252.09	17.122
W27X102	44.00	4.483
W12X14	10.00	0.141
W18X119	10.00	1.192
W16X67	306.18	20.484
W12X22	30.00	0.660
W21X111	352.58	39.154
W21X83	62.00	5.116
W8X18	40.20	0.718
W18X97	197.74	19.138
W10X17	36.00	0.610
W18X86	248.70	21.368
W6X15	159.86	2.405
W21X73	40.23	2.938
W18X60	210.01	12.552
W24X55	119.00	6.547
W18X55	271.60	14.942
W21X50	208.00	10.384
W10X15	182.00	2.726
W8X 13	26.00	0.339
W14X61	156.86	9.535
W14X74	45.80	3.391
W6X12	58.03	0.700
W18X50	245.60	12.261
W12X65	190.23	12.339
W21X44	133.00	5.872
W14X48	246.81	11.818

PROFILE	LENGTH(ft)	WEIGHT(k)
W16X40	298.60	11.966
W16X36	166.60	5.997
W16X45	29.40	1.328
W14X43	131.01	5.606
W12X58	162.89	9.404
W18X35	40.00	1.399
W14X38	65.80	2.503
W14X30	66.40	1.996
W12X30	116.41	3.475
W10X33	60.01	1.979
W12X26	125.81	3.268
W10X26	53.61	1.385
W14X22	30.00	0.661
W8X24	89.20	2.145
W6X20	79.01	1.575
W12X16	10.00	0.160
W8X21	28.03	0.586
W12X19	10.00	0.189
W16X31	10.00	0.310
W12X45	73.80	3.308
W8X31	42.40	1.315
W8X28	61.27	1.717
W8X35	26.00	0.909
W12X72	112.68	8.074
W14X176	24.50	4.310
W21X101	368.00	37.242
W24X131	26.00	3.399
W14X109	94.50	10.270
W16X77	13.00	0.998
W10X49	26.00	1.271
W36X135	40.00	5.393
W30X116	317.83	36.914
W10X39	26.00	1.015
W30X124	43.00	5.330
W14X90	269.75	24.276
W14X99	274.92	27.168
W12X79	117.08	9.225
W24X117	611.00	71.379
W40X221	24.50	5.391
W27X178	15.50	2.753
W27X161	24.00	3.863
W24X146	13.00	1.898
W33X141	13.00	1.837
W8X48	69.34	3.320
W12X87	40.75	3.543

TOTAL = 1081.979^k

Typical Steel Flooring System Redesign

Gravity Beam Design Takeoff

RAM Steel v8.1
Building Code: IBC

Steel Code: AISC LRFD

STEEL BEAM DESIGN TAKEOFF:

Steel Grade: 50

SIZE	#	LENGTH (ft)	WEIGHT (lbs)
W8X10	18	197.00	1984
W10X12	4	120.00	1446
W12X14	5	100.00	1416
W12X19	5	125.00	2369
W14X22	4	100.00	2208
W16X26	9	250.49	6546
W16X31	3	60.00	1864
W18X35	9	277.92	9741
W16X36	2	81.17	2928
W18X40	10	350.00	14053
W21X44	35	1476.92	65333
W21X48	4	182.33	8748
W24X55	4	120.00	6656
W24X62	2	71.17	4432
W24X76	2	81.17	6187
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	116		**135910**

Total Number of Studs = **3973**

Parking Steel Flooring System Redesign

Gravity Beam Design Takeoff

RAM Steel v8.1
Building Code: IBC

Steel Code: AISC LRFD

STEEL BEAM DESIGN TAKEOFF:

Steel Grade: 50

SIZE	#	LENGTH (ft)	WEIGHT (lbs)
W8X10	27	417.00	4200
W12X19	7	175.00	3317
W14X22	2	50.00	1104
W16X26	12	310.49	8114
W14X30	2	71.17	2143
W16X31	4	100.00	3107
W18X35	13	487.92	17101
W18X40	29	1162.25	46668
W21X44	12	547.00	24197
W24X55	6	191.17	10603
W24X68	2	81.17	5551
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	116		**126106**

Total Number of Studs = **3764**

Gravity Beam Design Takeoff

RAM Steel v8.1
Building Code: IBC

Steel Code: AISC LRFD

STEEL BEAM DESIGN TAKEOFF:

Steel Grade: 50

SIZE	#	LENGTH (ft)	WEIGHT (lbs)
W8X10	18	197.00	1984
W12X19	2	60.00	1137
W14X22	7	160.00	3533
W16X26	5	125.00	3267
W16X31	2	50.00	1553
W18X35	2	50.00	1752
W21X44	6	155.24	6867
W21X48	3	85.58	4106
W24X55	13	377.58	20943
W24X62	8	260.58	16227
W24X68	16	692.00	47330
W24X76	5	207.33	15803
W27X84	15	627.00	52912
W30X90	6	242.34	21770
W30X99	5	186.75	18492
W30X116	2	76.17	8864
W33X130	1	40.58	5289
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	116		**231831**

Total Number of Studs = **5532**