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Technical Report 1

Executive Summary

This report provides a detailed description and analysis of the existing structural systems at the Heart Hospital. Swedish American Hospital recently completed phase 2 of a 3 phase construction project on their grounds. Part of phase 2 was the construction of the new Heart and Vascular Center, also known as the Heart Hospital.

First, an in depth explanation of each of the main structural systems present in the building is provided. Typical sizes of framing and layouts are listed to help readers visualize a general image of the structure. A summary of the building codes used in the original design is listed, followed by a list of more recent building codes that will be used to analyze the existing structure.

Typical framing plans and elevations are provided to explain the buildings structural components in three dimensions. Brief summaries are included to help convey any elements that could not be shown on the diagrams. Building Dead Loads, Live Loads and Lateral Loads are tabulated and compiled in a set of tables, with corresponding code references as necessary. Spot checks of building elements, such as a beam, girder, and column, are conducted using the calculated gravity and lateral values. LRFD load factors are used in all calculations since the vast majority of the building's structural elements are steel. A simplified check of a lateral framing element is computed to check the validity of the design load assumptions and lateral load calculations. Further assumptions for the gravity element checks and frame analysis can be found in the detailed hand calculations located in the appendix of this report. Additional calculations are available upon request.

Table of Contents

Executive Summary 1

Structural System Overview 3

Codes 5

Material Strengths 5

Typical Framing Plans 6

Design Theory 10

Building Loads and Loading Diagrams 11
- Live Loads
- Snow Loads
- Dead Loads
- Seismic Loads
- Wind Design Loads

Framing Check 14
- Beam Check
- Girder Check
- Column Check
- Lateral Frame Check

Appendix 16

Structural System Overview

Floor System:

The typical building floor framing system is made up of beams and girders acting compositely with a concrete floor slab. Floor sections show 3”-20 gauge LOK Floor galvanized metal deck with 3¼” of lightweight concrete (110 pcf) resting on the steel framing below. Composite action is achieved through 5” long ¾” diameter shear studs welded to the steel framing. Concrete is reinforced with 6x6-W5xW5 welded wire fabric. The span of the metal deck varies depending on the bay location. However, the direction is limited to east-west or northeast-southwest. This assembly has a 2 hour fire rating without the use of spray on fireproofing.

There is no “typical” bay in the structural framing system. However, columns located on the wings are spaced approximately 22’-7 ½” on center. Columns in the interior core area are spaced approximately 32’-0” on center with additional columns located around the core perimeter framing into the wings. The most common and longest span is 32’-0”. Typical beam sizes range from W12x14’s to W27x146 with the larger beams acting as part of the moment framing system.

Roof System:

The roof framing system is very similar to the building floor framing system. Composite design is still used with 3 ¼” of lightweight concrete and 3”-20gauge LOK Floor metal deck on top of steel framing. Deeper steel beams and girders are used to help carry the heavier loads of the mechanical equipment on the roof.

The lobby roof is different from the typical roof framing. It does not use composite action and instead has a 1 ½” deep 20 gauge metal deck spanning north-south. Lower portions of the roof use a 3” deep 20 gauge metal deck to accommodate heavier snow loads and drifts.

Lateral System:

The lateral load resisting system consists of steel moment frames. The majority of the moment frames extend around the perimeter of the building with a few added moment frames on the interior to help stiffen the structure. Larger girders are framed into columns with bolted flange plate moment connections. The prefabricated steel pieces were bolted in place rather than welded to eliminate the need of preheating for welds. Shear walls were not part of the original design analysis; therefore, masonry cores such as the elevator and stairwell cores were not designed for lateral support.

Foundation

The basement footprint is approximately one half of the square footage of the first floor plan. Hence, there are two slabs on grade: one for the basement and one for part of the first floor. Each slab on grade is 5” thick normal weight concrete (145pcf) with 4x4-W5xW5 welded wire fabric reinforcement.

Interior steel columns rest on spread footings with an allowable soil bearing capacity of 4ksf. Exterior columns and basement walls rest on continuous strip footings. Reinforced concrete pilasters are located where exterior columns rest on the basement wall. Footings below columns in the interior core area extend approximately 18’ deep whereas the perimeter strip footings and footings located beneath the wings extend approximately 8’ deep. All footings are required to extend a minimum of 4’ deep for frost protection.

Columns:

Columns are laid out on two different intersecting grids: one running east-west and the other running northwest-southeast. All columns are ASTM A992 Grade 50 wide flange steel shapes. Columns are spliced between the 3rd and 4th floor. Columns acting as part of a moment frame are spliced 5’-6” above the 3rd floor elevation. Columns acting only as gravity columns are spliced 4’-6” above the 3rd floor elevation. All interior columns that extend to the basement level are also spliced 5’-6” above the 1st floor elevation. Future columns for the 6th and 7th floors are designed to be spliced with existing columns at the 5th floor elevation (current mechanical floor and roof).

Codes

Original Design Codes:

- International Building Code (IBC) 2003
  - with City of Rockford, IL amendment
- American Society of Civil Engineers (ASCE)
  - ASCE 7-02 - Minimum Design Loads for Buildings and Other Structures
- American Concrete Institute (ACI)
  - ACI 318-02 - Building Code Requirements for Structural Concrete
  - ACI 530-02 – Building Code Requirements for Masonry Structures
- American Institute of Steel Construction (ASIC)
  - LRFD 1999 - Load and Resistance Factor Design Specification for Structural Steel Buildings
  - AISC 341-02 – Seismic Provisions for Structural Steel Buildings

Thesis Design Codes:

- International Building Code (IBC) 2006
- American Society of Civil Engineers (ASCE)
  - ASCE 7-05 - Minimum Design Loads for Buildings and Other Structures
- American Concrete Institute (ACI)
  - ACI 318-05 – Building Code Requirements for Structure Concrete

Material Strengths

Concrete

Normal Weight Concrete (columns, walls, foundations, slabs on grade)……..…4000psi
Light Weight Concrete (floor slabs on metal deck)………………………………4000psi
Reinforcement ………………………………………………………………60ksi

Structural Steel

Wide Flanges and Channels ……………………………………………………...50ksi
Angles, Bars and Plates…………………………………………………………...36ksi
Hollow Structural Sections (HSS)……………………………………………….. 46ksi
Bolts (A325X or A490X)…………………………………………………………3/4”dia
Shear Studs (5”long)……………………………………………………………… 3/4”dia

Masonry

Design Strength (F'_m)……………………………………………………………2000psi
Block………………………………………………………………………………...4000psi

Typical Framing Plans

Foundation Plans

The building is supported on both spread and strip footings. Spread footings usually support a single interior column. Spread footings located at the corners of the building can support two or three columns. Strip footings extend around the perimeter of the building and below the basement walls (the basement is approximately half the area of the building footprint).

Two slabs on grade are poured for the Heart and Vascular Center. The first slab is at the basement elevation and located at the center core of the building. The area of this slab is approximately 12,500 sq. ft., which is about half of the overall building footprint (25,000 sq.ft.).

The basement walls are assumed to be single span that is laterally supported by the slab on grade at the base and the first floor framing at the top. They are designed to resist load from the lateral earth pressure.

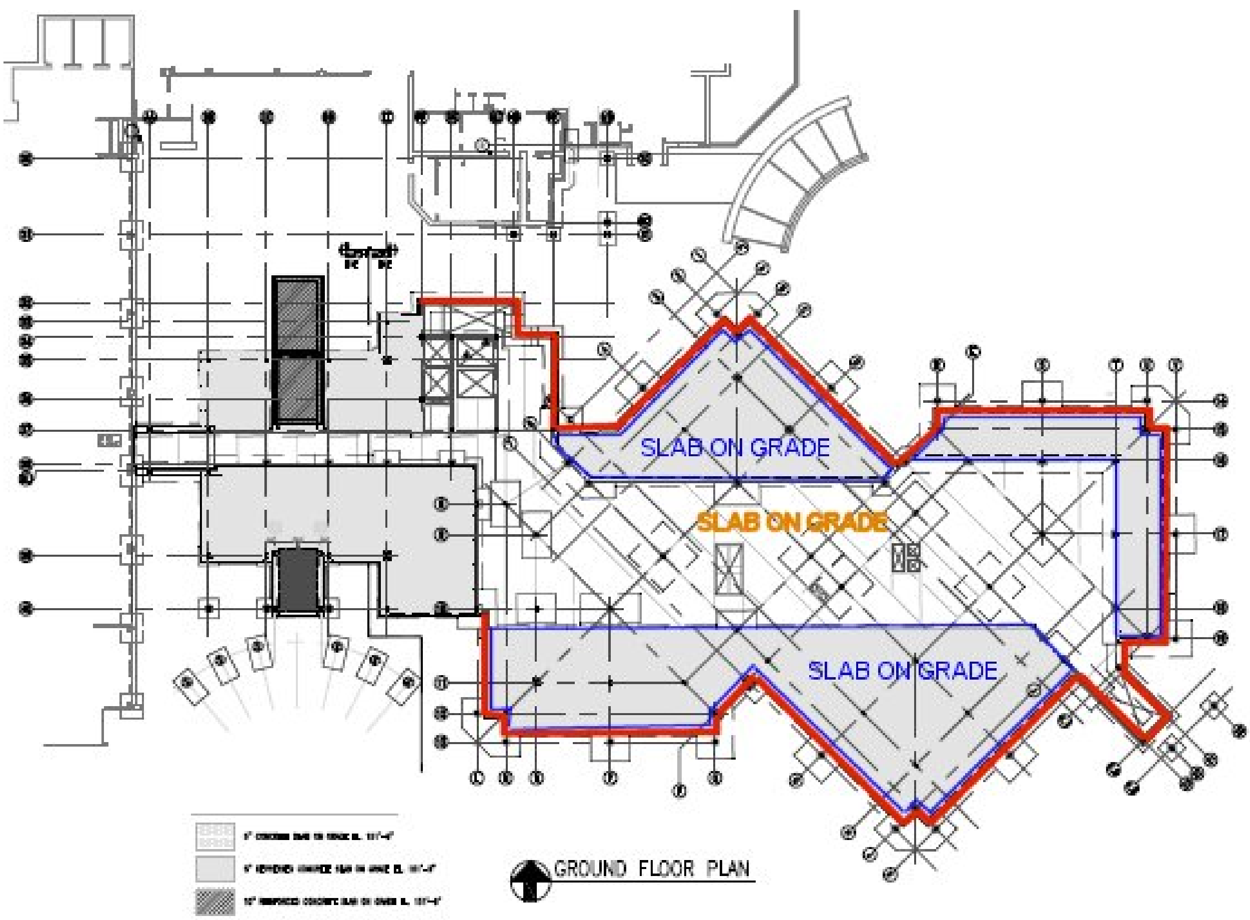


Figure 1: Ground Floor Plan highlighting foundation systems

Strip footings running around the perimeter of the building are highlighted in RED. The First Floor slab on grade (not including the Lobby) is outlined and labeled in BLUE. The area of the basement slab on grade is labeled in ORANGE.

Typical Floor Plans

Open floor plans are achieved through the use of stiff moment frames. This allows for more flexible interior spaces and maximizes the use of natural light. Composite action used to maximize the strength of the structural steel and concrete floor slab. Floor systems utilize 3" galvanized LOK Floor decks with 3¼" of lightweight concrete resting on a grid of beams and girders. The concrete and metal deck system is designed to be unshored for a two span condition. Single spans should be shored unless otherwise specified. The maximum distance for a single span is approximately 11'-6".

Floor framing is oriented north-south and east-west for the wings of the Heart and Vascular Center and the entrance lobby. The framing at the central core is oriented diagonally northwest-southeast and northeast-southwest. Metal deck installed on the hospital wings spans east-west, and spans northeast-southwest at the central core. All concrete and metal deck floor systems are assumed to be rigid systems.

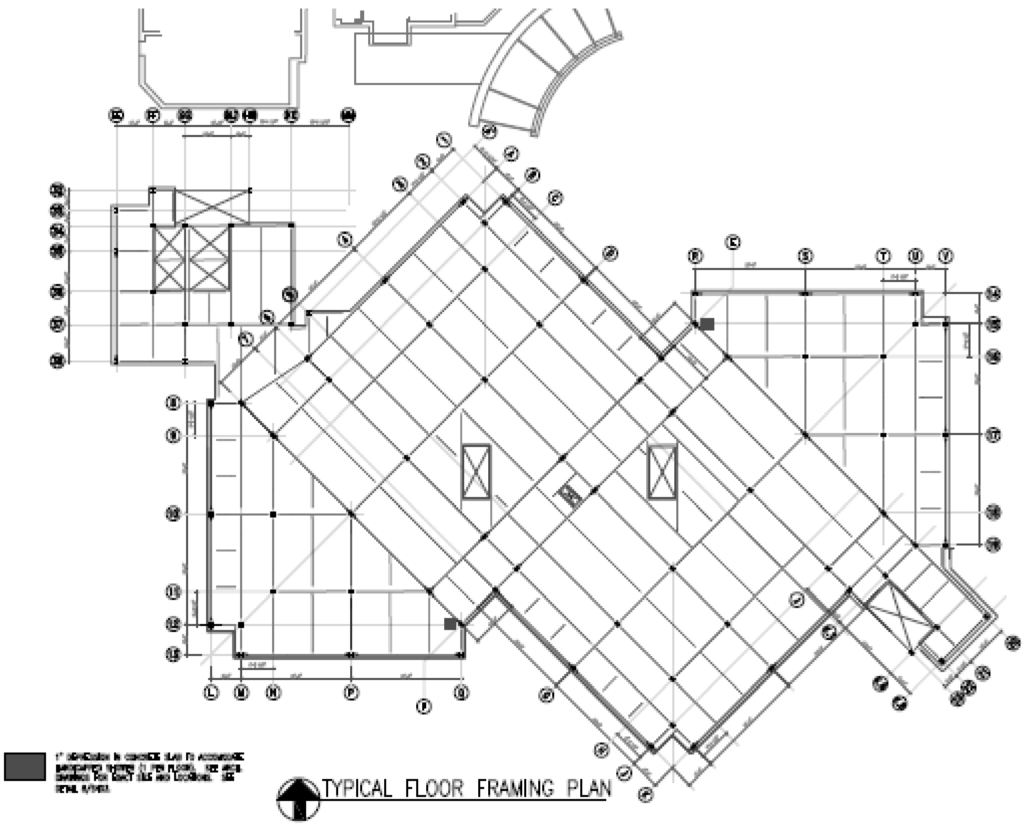


Figure 2: Typical Floor system showing layout of beams and girders

Lateral Framing System

Swedish American Hospital's new Heart and Vascular Center is laterally supported with steel moment frames. The Frames are designed to resist wind and seismic loads. Pieces of these frames are prefabricated then bolted together onsite. Flange plate bolted connections are used in place of welded connections (see Appendix A for details). Bolted connections eliminate the need for preheating steel for welded connections. Since steel erection began in mid February, eliminating the need for preheating helped speed up the erection process and keep the project on schedule.

The majority of the moment frames lie around the perimeter of the building, with some interior moment frames added to help stiffen the structure and reduce drift. Less interior moment frames help reduce the required depth of steel in interior spaces to minimize conflicts with HVAC systems. Moment frames allow for a more open architectural floor plan. Swedish American uses their open floor plan to help increase the amount of natural light that reaches their interior spaces. Braced frames and shear walls could create potential problems with door and window openings. All frames are assumed to be pin-supported on spread footings and concrete piers at the basement level.

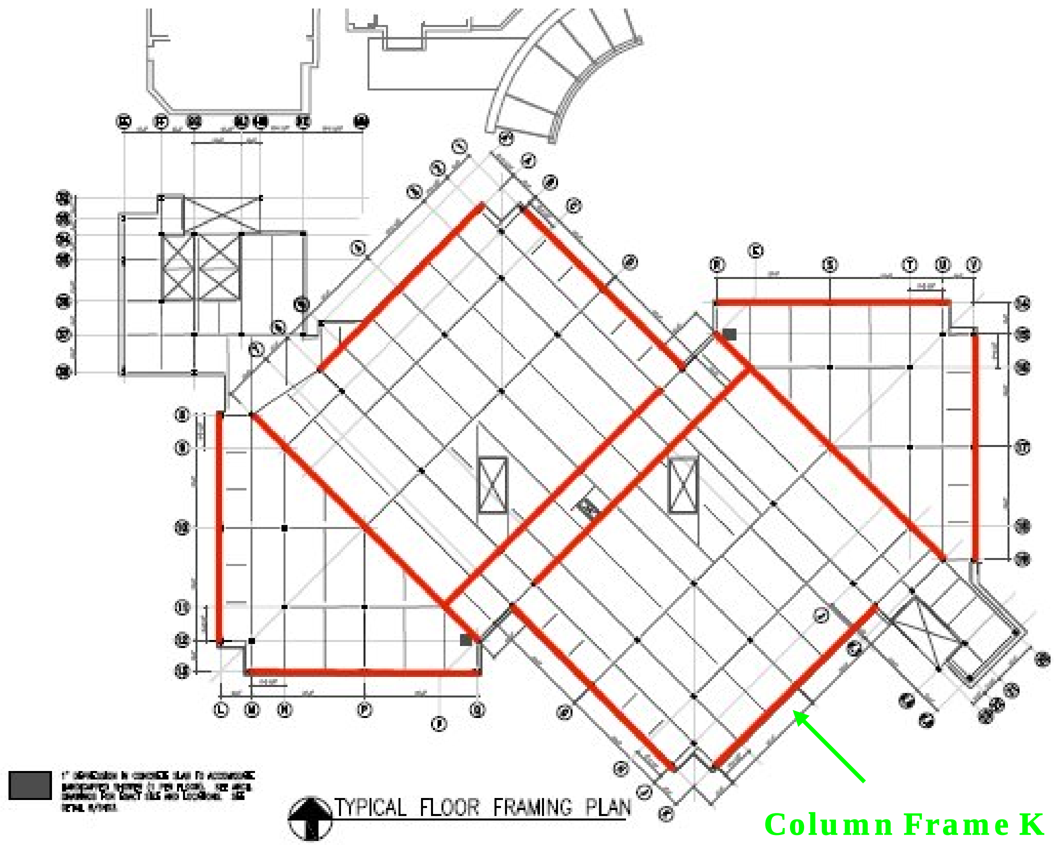


Figure 3: Framing Plan with Highlighted Moment Frames

*Typical moment frames are outlined in RED.

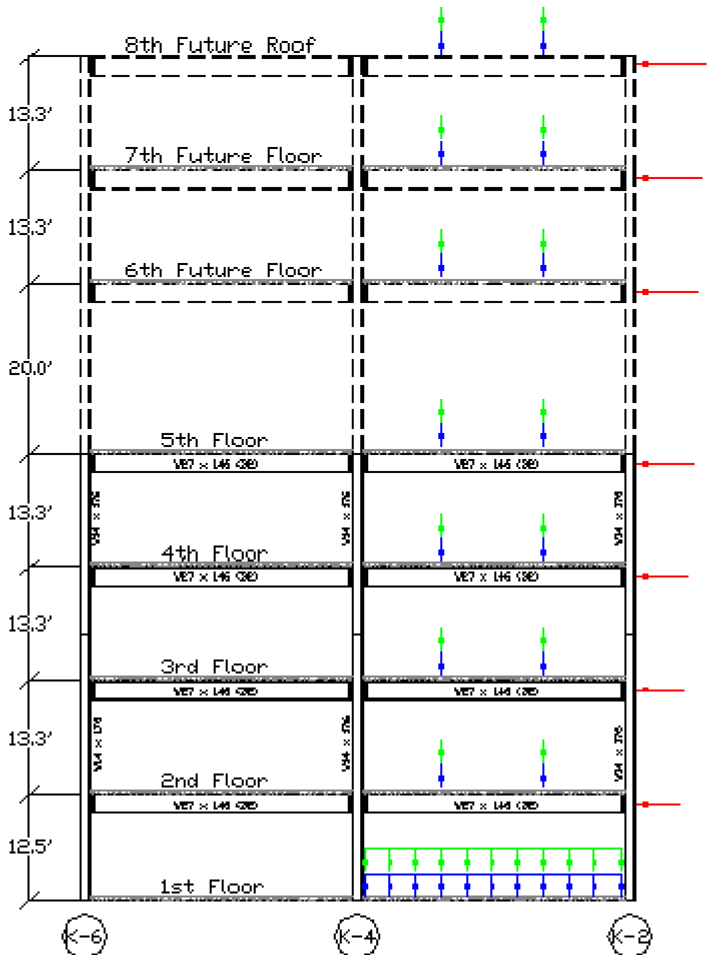


Figure 4: Column Line – Frame K

- GREEN arrows represent concentrated live load from beam framing.
- BLUE arrows represent concentrated dead load from beam framing.
- RED arrows represent lateral load from wind or seismic forces acting at the floor level.

Floor Loading:

The steel deck near column line K runs parallel to the frame. Floor loads are transferred from the slab and deck to beams (typically spanning 18' to 22', also spanning 32' in some locations). These beams run perpendicular to the girders along column line K and frame in near the third points.

When analyzing beams or girders running perpendicular to the steel deck, the load is considered a distributed load along the members' length. When the framing is parallel to the steel deck (as it is for the majority of the girders), the load is considered a concentrated point load where the beam frames into the girder.

Lateral Loading:

For seismic loading, the total base shear is calculated using ASCE 7-05 Sections 11 and 12 (see Seismic Load Table on page 12 and seismic load calculations in Appendix B). The Heart and Vascular Center has a base shear of approximately 875k. This base shear is divided over the entire story height based on the height and weight of each story over the entire height and weight of the structure. This effective story shear is assumed to be taken at the floor level of each story. The shear at the lowest level is small but increases with height for my building.

For wind pressures, the windward pressure acting along the height of the structure is in the form of a parabolic curve. A conservative assumption is to break the curve into a rectangular grid and find the effective pressure acting on an individual story. Windward pressures are calculated using equation 6.19 in ASCE 7-05 Section 6 (see the Wind Design Load Tables on page 13 and Appendix C for wind pressures, diagrams, and Gust Factors). Leeward pressure is assumed to be a constant along the back of the building and calculated using the total building height. Wind pressures are calculated in two main directions (usually acting perpendicular to the building face). Base shears resulting from wind for the Heart Hospital were 555k (N-S direction) and 369k (E-W direction). Therefore, seismic controls the lateral design.

Design Theory

Moment Frame vs Braced Frame or Shear Walls

Moment frames were designed in place of braced frames or shear walls to create an open flexible floor plan. Swedish American Hospital wanted to option to "re-program" various floors and re-layout different floor plans. Braced frames and shear walls are a hindrance when trying to re-layout a floor plan and will minimize the number of available floor plan options.

Swedish American Hospital and Perkins and Will Architects also liked the uniform appearance along the façade faces created by the moment frames. Braced frames could interrupt the uniformity of a façade and could obstruct patient views out their windows. The hospital was willing to accept the extra expenses in exchange for a more aesthetically appealing façade and future flexibility in floor arrangements and layouts.

Possible shear walls exist at the northwest and southeast corners of the building where the stairwells and elevator core lie. However, shear walls were not incorporated into the structural design of the Heart and Vascular Center. Simpson Gumpertz and Heger Inc., the Structural Engineers, prefer to only classify and design one type of lateral system instead of a dual system. For this project, they chose to only analyze and design the steel moment frames. These vertical cores could provide a small contribution to the building's overall stiffness, but were not taken into account during design.

4 Floors vs 7 Floors

The Swedish American Hospital's new Heart and Vascular Center is part of phase 2 in a 3 phase construction project on the hospital grounds. The original 7 floors where designed based on a "Certificate of Need". A Certificate of Need establishes the number of patient rooms required for a hospital based on the local increasing population. Therefore, the new Heart and Vascular center is required to handle that patient increase and must be designed for that load. However, other local hospitals and medical facilities share some of that patient load and is the reason the existing Heart and Vascular Center is only 4 stories.

Phase 3 of the Swedish American construction project is the addition of the 3 floors on top of the new Heart and Vascular Center. However, this causes a problem. To begin construction above an existing medical facility, the facility must first be shut down and all patients evacuated and transferred to another facility. Shutting down the Heart Hospital could cause potential problems with relocating critical patients and getting adequate care to other patients. Closing the hospital, even temporarily, would redirect numerous patients to other local hospitals possibly flooding their medical facilities. Not to mention, many Heart and Vascular Center employees may be without a job (or have a decrease in hours) until construction is nearing completion.

Building Loads and Loading Diagrams

Floor Live Loads

Loaded Area	Building Design Load	ASCE 7-05 Section 4
Basement Floor	100 psf	Table 4-1
First Floor	100 psf	Table 4-1
Typical Floors (2nd, 3rd, 4th, 6th, 7th)	80 psf	Table 4-1
Mechanical/Roof (5th Floor)	150 psf	Set by SAH*, Engineers
Stairwells	100 psf	Table 4-1
Roof (8th Future Roof)	25 psf	ASCE 7-05 Section 7 (Snow)

* SAH – Swedish American Hospital

Roof Snow Loads (Live Load)

Item	Design Load	Code References
Roof Live Load	25 psf	ASCE 7-05 Section 7
Ground Snow Load	30 psf	ASCE 7-05 Figure 7-1
Additional Drift Load	50.4 psf	ASCE 7-05 Section 7-7
Exposure Factor (C_e)	1.0	ASCE 7-05 Table 7-2
Importance Factor (I)	1.2	ASCE 7-05 Table 7-4
Thermal Factor (C_f)	1.0	ASCE 7-05 Table 7-3

Dead Loads

Typical Floors 1 though 4 and Future Floors 6 and 7

Item	Design Load
Partitions	10 psf
Steel Deck with LWC Slab	48 psf
Ponding due to Deflection	5 psf
Steel Self Weight	15 psf
MEP, Misc.	12 psf
Total	90 psf

5th Floor (Roof/Mechanical)

Item	Design Load
Partitions	0 psf
Permanent Equipment	50 psf
Steel Deck with LWC Slab	48 psf
Ponding due to Deflection	5 psf
Steel Self Weight	15 psf
MEP, Misc.	12 psf
Total	130 psf

8th Floor (Future Roof) *

Item	Design Load
Permanent Equipment	0 psf
Steel Deck with LWC Slab	48 psf
Ponding due to Deflection	5 psf
Steel Self Weight	15 psf
MEP, Misc.	12 psf
Total	80 psf

Other Areas (lobby roof, Stair tower roof) *

Item	Design Load
Metal Deck, Insulation, Roofing	25 psf
Steel Self Weight	15 psf
MEP, Misc.	5 psf
Total	45 psf

Wall Dead Loads

Item	Design Load
Exterior Wall Precast Panel	85 psf
Exterior Wall Brick	50 psf
Exterior Aluminum Curtain Wall	15 psf
Shaft Walls around openings	20 psf
Stair Walls around openings	80 psf

* Snow mass is not included in Dead Loads, but a 5 psf snow load is included for seismic massing (ASCE 7-05 Section 12.7.2).

Seismic Loads

Item	Design Value	Code Reference
Occupancy Category	IV	ASCE 7-05 Table 1-1
Site Class	D	* From Geotechnical Report
Spectral Acceleration for Short Periods (S_s)	0.17g	* From Geotechnical Report
Spectral Acceleration for One Sec. Periods (S_1)	0.06g	* From Geotechnical Report
Damped Design for Short Periods (S_{ds})	0.1813g	ASCE 7-05 Section 11.4.4
Damped Design for One Sec. Periods (S_{d1})	0.096g	ASCE 7-05 Section 11.4.4
Seismic Design Category	C	ASCE 7-05 Section 11.6.1.1
Seismic Force Resisting System	Ordinary Steel Moment Frames	ASCE 7-05 Table 12.2-1
Response Modification Factor (R)	3.5	ASCE 7-05 Table 12.2-1
System Overstrength Factor (Ω)	3.0	ASCE 7-05 Table 12.2-1
Deflection Amplification Factor (C_d)	3.0	ASCE 7-05 Table 12.2-1
Importance Factor	1.5	ASCE 7-05 Table 11.5-1
Approximate Period (Ta)	1.106	ASCE 7-05 Section 12.8.2.1
Seismic Response Coefficient (Cs)	0.037	ASCE 7-05 Section 12.8.1.1
Building Mass	23,650k	* From Massing Calculations
Design Base Shear	875k	

Vertical Distribution of Seismic Loads
ASCE 7-05 Section 12.8.3

Level	h (in ft)	W in kips	$w_x h_x^k$	C_{vx}	F_x
8th floor future Roof	99.17	2568	1025315	0.24	208.79
7th floor future	85.83	2977	984759	0.23	200.53
6th floor (future)	72.50	3376	896231	0.21	182.50
5th floor mechanical	52.50	4047	705500	0.16	143.67
4th floor	39.17	3091	367848	0.09	74.91
3rd floor	25.83	3342	231248	0.05	47.09
2nd floor	12.50	3200	85985	0.02	17.51
1st floor	0.00	1049	0	0.00	0.00
		23650k	4296886		875k

Design Base Shear (V) = 875 k
k (by interpolation) = 1.303

Wind Design loads

Level	Total Height	K_z	q	Wind Pressures (psf)					
				N-S Windwar d	N-S Leeward	N-S Side Wall	E-W Windwar d	E-W Leeward	E-W Side Wall
Roof	99.17	1.26	25.54	17.26	-10.79	-15.10	17.43	-8.71	-15.25
7	85.83	1.225	24.83	16.78	-10.79	-15.10	16.94	-8.71	-15.25
6	72.50	1.18	23.92	16.17	-10.79	-15.10	16.32	-8.71	-15.25
5	52.5	1.1	22.30	15.07	-10.79	-15.10	15.21	-8.71	-15.25
4	39.17	1.04	21.08	14.25	-10.79	-15.10	14.38	-8.71	-15.25
3	25.83	0.94	19.05	12.88	-10.79	-15.10	13.00	-8.71	-15.25
2	12.5	0.85	17.23	11.64	-10.79	-15.10	11.76	-8.71	-15.25

Level	Eff. Height	Wind Design (NS - EW)					
		Load (kips)		Shear (kips)		Moment (ft-k)	
		N-S	E-W	N-S	E-W	N-S	E-W
		230'	165'	230'	165'	230'	165'
Roof	6.67	43	29	0	0	4264	2851
7	13.33	85	57	43	29	7318	4889
6	16.67	104	70	128	86	7576	5053
5	16.67	101	67	233	155	5314	3534
4	13.33	78	52	334	223	3056	2026
3	13.33	75	49	412	274	1928	1273
2	12.92	68	45	487	324	856	562
1	6.25	32	21	0	0	0	0
Total	99.15	555	369	555	369	30311	20188

Framing Check

Typical Floor Beam

I checked a typical floor beam on the 5th floor (current roof) spanning from Column G4 to Column H4. This area has a distributed dead load (DL) of 130 psf and a distributed live load (LL) of 150 psf. TL = 1.2(DL) + 1.6(LL) = 396 psf. Assuming a simply supported span, M_u = 287 ft-k

The existing floor beam is a W16x26 with 14 shear studs along its length. Therefore, 7 stubs on each side transfer the shear for the max moment. Using Table 3-19 in the AISC Steel Construction Manual 13th Ed., I found that 7 studs per side do not satisfy the required calculated moment (ϕM_p = 248 ft-k < 287 ft-k). Increasing the beam size to a W16x31 with the same number of stubs will yield a moment ϕM_p = 300 ft-k. Adding additional shear studs to the original beam can also satisfy the moment capacity.

(See Appendix D for detailed calculations and assumptions)

Typical Floor Girder

The typical floor girder I checked is also part of the 5th floor framing spanning from Column H4 to Column H5. It too has a distributed dead load (DL) of 130 psf and a distributed live load (LL) of 150 psf. TL = 1.2(DL) + 1.6(LL) = 396 psf. Assuming a simply supported span, M_u = 516 ft-k

The existing floor girder is a W18x35 with 26 shear studs along its length. With only 13 stubs per side acting to transfer the shear, Table 3-19 gives a moment of ϕM_p = 445 ft-k < 516 ft-k. Increasing the beam size to a W18x46 with an equal number of studs will yield a moment of ϕM_p = 542 ft-k. However, adding the additional shear studs to the smaller beam would be a more efficient and cost effective way to solve the problem.

(See Appendix E for detailed calculations and assumptions)

Typical Column Load

I checked column H4 to determine if it satisfied the strength requirements. A spreadsheet with the live load reduction factors and corresponding live loads can be found in Appendix F. Not all floor live loads can be reduced. The tributary area from each floor is approximately 556.5 sq. ft. acting on the column. I found a maximum of load P = 582 k acting on the column between the 3rd and 4th floor levels. The maximum load on the column is found at the first floor level where P = 759 k.

The column is spliced 4.5' above the 3rd floor. A W14x74 extends from the 1st floor to the 3rd floor splice. From there, a W14x61 continues to the 5th floor level. In the future, a 3rd and potentially 4th column will extend to the top of the 8th floor (future roof framing). The minimum unbraced length for the W14x74 is 12.5' occurring at the first floor. The minimum unbraced length for the W14x61 is 8.5' acting on the 3rd floor above the column splice. The maximum unbraced length for each of the columns is 13.33' at the other floors.

From Table 4-1 in the AISC Steel Manual, a W14x74 has a ϕP_n = 750 k < 759 k. This number is close and probably considered acceptable because it is within 5% of the required value. If more strength is encouraged, a W14x82 has a ϕP_n = 828 k.

For further column calculations and assumptions see Appendix G.

Typical Lateral Frame

The lateral moment frame I checked is part of Column line K, including columns K-2, K-4, and K-6. This frame is part of the exterior wall at the southeast corner of the building (see Figure 3 above). This frame is composed of W14x176 columns spliced 4.5 feet above the 3rd floor level and bear on spread footings below the 1st floor (assumed to be pinned at base support). Floor girders are W27x146 (32) at all upper floors and span 32' from column face to face. Transverse beams frame into the girders at approximately the third points along their span.

I analyzed the frame by constructing a computer model of the frame in SAP. I assigned the existing beams and girders their corresponding structural properties. For the future floors, I assumed the new columns to be continuous from the 5th floor to the 8th floor. This assumption is acceptable because the analysis program is only computing the moments in the frame. Transverse beams are modeled as point loads on the girders at third points along their span. Transverse beams also frame into the columns, but are not considered in the model.

Seismic base shear (V=875k) is greater than the wind force and will control the lateral design. I assumed the central core area to handle the entire seismic load. Since there is 4 framing elements acting in the same direction as Frame K, I assumed Frame K took ¼ of the seismic base shear and distributed it to each of the upper floors based on their ratio of weight and height in comparison to the entire structure.

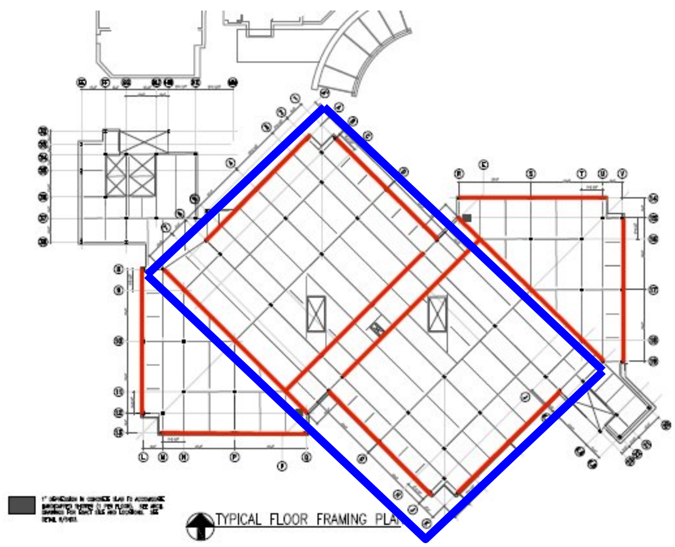


Figure 5: Core Area

(See Appendices B, C and H for detailed calculations and assumptions of lateral and gravity loads on Frame K.)

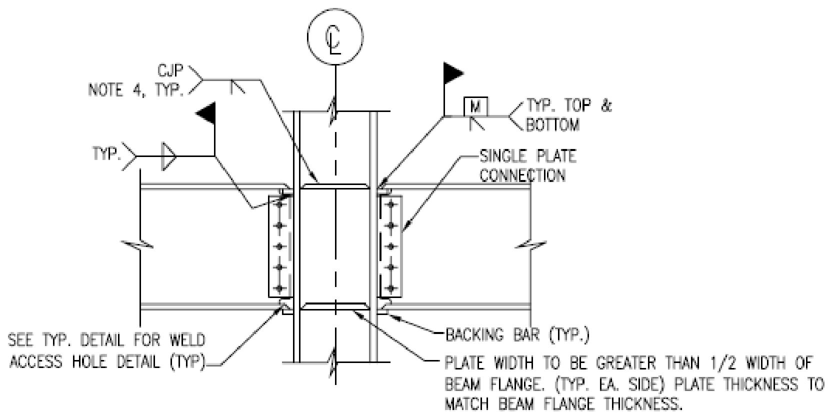
After running the SAP analysis program, I printed the girder and column moments and analyzed the data. The largest moment experienced by a column was M_u = 1106 ft-k at the center column (K-4) on the underside of the 2nd floor. The largest moment experienced by a girder was M_u = 1224 ft-k on the 2nd floor girder between column K-6 and K-4 (see Appendix H – page 30 for figure with moment diagram).

Using Table 3-2 in the AISC Steel Manual, a W14x176 has a ϕM_p = 1200 ft-k which is greater than M_u = 1106 ft-k. Since all columns are W14x176, they all meet the required moment capacity. However, the column design moments decrease as they go up the building. Therefore, smaller columns could be used in place of the larger columns as long as they satisfy the required gravity and lateral loads.

For the girders, Table 3-19 in the AISC Steel Manual does not list W27x146 for composite action. Hand calculations for a W14x146 with shear studs in composite action can be found in Appendix I. Final calculations show the existing girders have a moment capacity ϕM_n = 2152 ft-k > M_u = 1224 ft-k. Like the columns, the design moments slowly decrease as you move your way up the building. Therefore, smaller girders could possibly be used in the design of the upper floors (or future floors).

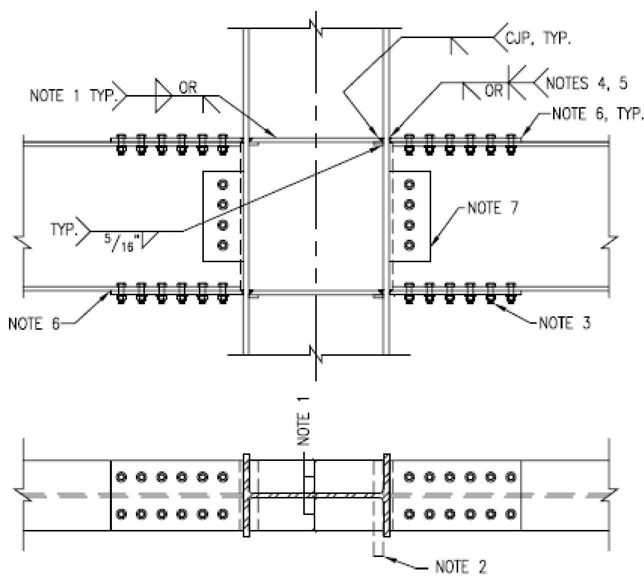
Appendix A

Typical and Alternate Beam to Column Moment Connections



NOTES:
1. MOMENT CONNECTION TO DEVELOP THE FULL CAPACITY OF THE BEAM.
2. SHEAR CONNECTION TO DEVELOP THE LOWER OF THE FACTORED REACTION LISTED IN THE SCHEDULE + 2.4(M_P/SPAN), OR 80% OF THE UNREDUCED SHEAR STRENGTH OF THE COPED WEB. BOLTS ARE SLIP CRITICAL.
3. INSTALL ALL BOLTS SNUG TIGHT PRIOR TO FIRST TORQUING. TENSION BOLTS FULLY PRIOR TO WELDING.
4. EDGE PREPARATION OF CONTINUITY PLATES AT FABRICATORS DISCRETION. BACK GOUGE ROOT PASS OF DOUBLE BEVEL GROOVE WELDS.
5. REMOVE BOTTOM FLANGE BACKING BAR, BACK GOUGE, AND INSTALL REINFORCING 5/16" FILLET ON TOP & BOTTOM OF BOTTOM FLANGE WELD.
6. WELD TOP FLANGE BACKING BAR CONTINUOUSLY TO COLUMN FLANGE OR CONTINUITY PLATE.
7. ADD A 5/16" REINFORCING FILLET TO TOP OF TOP FLANGE WELD.

TYPICAL BEAM–TO–COLUMN MOMENT CONNECTION DETAIL (STRONG AXIS)



BOLTED MOMENT CONNECTION NOTES:
1. MINIMUM WIDTH TO MATCH BEAM FLANGE, THICKNESS TO MATCH OR EXCEED FLANGE OF THICKER BEAM FLANGE AT CONNECTION.
2. REMOVE WELD TABS TO 1/4" MAXIMUM FROM EDGE OF CONTINUITY PLATE. GRIND END OF WELD SMOOTH, NOT FLUSH; DO NOT GOUGE COLUMN FLANGE.
3. ALL BOLTS PRETENSIONED; DESIGNED FOR BEARING. BOLT HOLES IN FLANGE PLATE ARE OVERSIZED, BOLT HOLES IN BEAM FLANGE ARE STANDARD.
4. SHOP WELD: WHEN USING SINGLE BEVEL PREPARATION, REMOVE BACKING AFTER WELDING, BACKGAUGE, AND REINFORCE WITH 5/16" MIN. FILLET WELD.
5. WHEN USING DOUBLE BEVEL PREPARATION, BACKGOUGE FIRST WELD BEFORE WELDING OTHER SIDE.
6. SHIMS BETWEEN BEAM FLANGE AND FLANGE PLATES ARE ALLOWED. USE FULL COVERAGE SHIM PLATES OR FULL DEPTH FINGER SHIMS.
7. HOLES IN SHEAR TABS ARE SHORT SLOTTED HORIZONTAL HOLES
HOLES IN BEAM WEB ARE STANDARD.
8. MOMENT CONNECTION SHALL DEVELOP THE FULL CAPACITY OF THE BEAM.

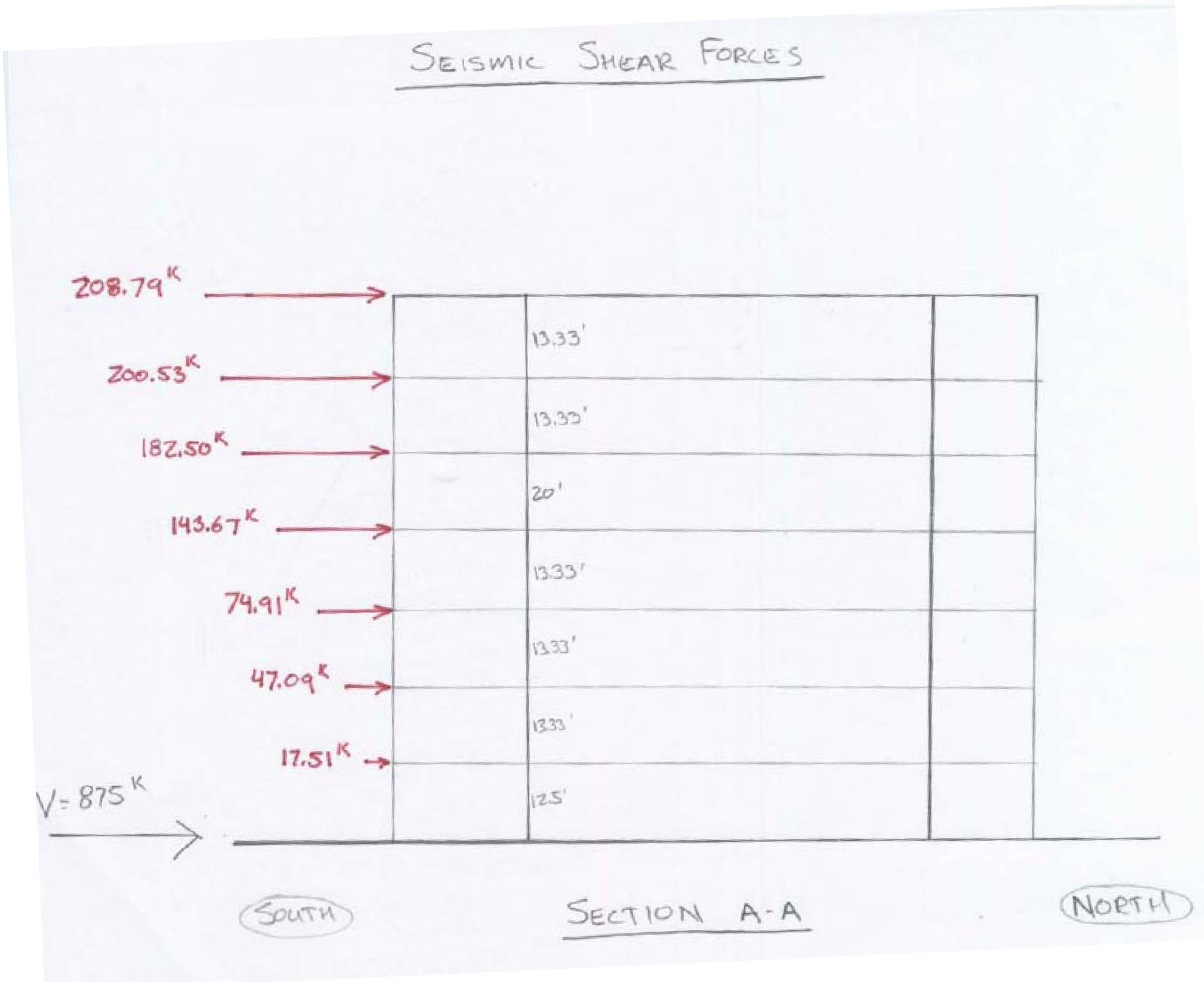
TS–1D ALTERNATE BOLTED STRONG AXIS BEAM–TO–COLUMN MOMENT CONNECTION DETAIL 3/4"=1'–0"

Appendix B

Seismic Massing (Dead Loads)					
Level	Item	Quantity	Units	Unit Weight	Weight
				(ksf or klf)	(kip)
8th Floor (Future Roof)					
h = (ft)	6 ¼" LW conc. 3"-20g LOK-Floor deck	24900	sf	0.048	1195
99.17	Ponding	24900	sf	0.005	125
	Steel self weight	24900	sf	0.015	374
story height	Mechanical, electrical, floor/clg. Misc.	24900	sf	0.012	299
13.33	Snow	24900	sf	0.005	125
	Exterior wall - Precast	575	lf	0.566	325
	Exterior wall - Brick below	100	lf	0.375	38
	Exterior wall - brick above	100	lf	0.508	51
	Exterior wall - Aluminum curtain wall	175	lf	0.100	18
	Interior wall - Elevator core	100	lf	0.203	20
				Total (kips)	2568
7th Floor (Future)	6 ¼" LW conc. 3"-20g LOK-Floor deck	24993	sf	0.048	1200
h = (ft)	Ponding	24993	sf	0.005	125
85.83	Partition	24993	sf	0.015	375
story height	Steel self weight	24993	sf	0.012	300
13.33	Mechanical, electrical, floor/clg. Misc.	24993	sf	0.007	180
	Exterior wall - Precast	575	lf	1.133	652
	Exterior wall - Brick	100	lf	0.667	67
	Exterior wall - Aluminum curtain wall	175	lf	0.200	35
	Interior wall - Shaft	92	lf	0.267	25
	Interior wall - Elevator core	100	lf	0.200	20
				Total (kips)	2977
6th Floor (Future)	6 ¼" LW conc. 3"-20g LOK-Floor deck	24993	sf	0.048	1200
h = (ft)	Ponding	24993	sf	0.005	125
72.50	Partition	24993	sf	0.015	375
story height	Steel self weight	24993	sf	0.012	300
20.00	Mechanical, electrical, floor/clg. Misc.	24993	sf	0.007	180
	Exterior wall - Precast	575	lf	1.700	978
	Exterior wall - Brick	100	lf	1.000	100
	Exterior wall - Aluminum curtain wall	175	lf	0.300	53
	Interior wall - Shaft	92	lf	0.400	37
	Interior wall - Elevator core	100	lf	0.300	30
				Total (kips)	3376
5th Floor Mech	6 ¼" LW conc. 3"-20g LOK-Floor deck	24993	sf	0.048	1200
h = (ft)	Ponding	24993	sf	0.005	125
52.50	Steel self weight	24993	sf	0.015	375
story height	Mechanical, electrical, floor/clg. Misc.	24993	sf	0.012	300
13.33	Permanent Equipment	24993	sf	0.050	1250
	Exterior wall - Precast	575	lf	1.133	651
	Exterior wall - Brick	100	lf	0.667	67
	Exterior wall - Aluminum curtain wall	175	lf	0.200	35
	Interior wall - Shaft	92	lf	0.267	25
	Interior wall - Elevator core	100	lf	0.200	20
				Total (kips)	4047
4th Floor	**Main Floor**				
h = (ft)	6 ¼" LW conc. 3"-20g LOK-Floor deck	24993	sf	0.048	1200
39.17	Ponding	24993	sf	0.005	125
	Partition	24993	sf	0.010	250
story height	Steel self weight	24993	sf	0.015	375

13.33	Mechanical, electrical, floor/clg. Misc	24993	sf	0.012	300
	Architectural Roof				
	6 ¼" LW conc. 3"-20g LOK-Floor deck	600	sf	0.048	29
	Ponding	600	sf	0.005	3
	Steel self weight	600	sf	0.015	9
	Snow	600	sf	0.005	3
	Exterior wall - Precast	575	lf	1.133	652
	Exterior wall - Brick	100	lf	0.667	67
	Exterior wall - Aluminum curtain wall	175	lf	0.200	35
	Interior wall - Shaft	92	lf	0.267	25
	Interior wall - Elevator core	100	lf	0.200	20
				Total (kips)	3091
3rd Floor	**Main Floor**				
h = (ft)	6 ¼" LW conc. 3"-20g LOK-Floor deck	24993	sf	0.048	1200
25.83	Ponding	24993	sf	0.005	125
	Partition	24993	sf	0.010	250
story height	Steel self weight	24993	sf	0.015	375
13.33	Mechanical, electrical, floor/clg. Misc.	24993	sf	0.012	300
	Lobby Roof				
	non composite roof	5300	sf	0.025	133
	Mechanical, electrical, floor/clg. etc	5300	sf	0.005	27
	steel self weight	5300	sf	0.015	80
	Snow	5300	sf	0.005	27
	Exterior wall - Precast	575	lf	1.133	652
	Exterior wall - Brick	100	lf	0.667	67
	Exterior wall - Aluminum curtain wall	325	lf	0.200	65
	Interior wall - Shaft	92	lf	0.267	25
	Interior wall - Elevator core	100	lf	0.200	20
				Total (kips)	3342
2nd Floor	**Main Floor**				
	6 ¼" LW conc. 3"-20g LOK-Floor deck	26283	sf	0.048	1262
h = (ft)	Ponding due to deck/purlin deflection	26283	sf	0.005	131
12.5	partition	26283	sf	0.010	263
	steel self weight	26283	sf	0.015	394
story height	Mechanical, electrical, floor/clg. etc	26283	sf	0.012	315
12.5	**Lobby Roof**				
	non composite roof	1170	sf	0.025	29
	steel self weight	1170	sf	0.015	18
	Mechanical, electrical, floor/clg. etc	1170	sf	0.005	6
	Snow (incl. drift)	1170	sf	0.005	6
	Exterior wall - Precast	575	lf	1.063	611
	Exterior wall - Brick	100	lf	0.625	63
	Exterior wall - Aluminum curtain wall	325	lf	0.188	61
	Interior wall - Shaft	92	lf	0.250	23
	Interior wall - Elevator core	100	lf	0.188	19
				Total (kips)	3200
1st Floor	6 ¼" LW conc. 3"-20g LOK-Floor deck	11650	sf	0.048	559
ground	Ponding due to deck/purlin deflection	11650	sf	0.005	58
h = (ft)	partition	11650	sf	0.010	117
0	steel self weight	11650	sf	0.015	175
	Mechanical, electrical, floor/clg. etc	11650	sf	0.012	140
				Total (kips)	1049
		Total Seismic Massing (kips) =			23650

Seismic Story Shear Forces



Appendix C

MAIN WIND-FORCE RESISTING SYSTEM (ASCE 7-05)

Swedish American Hospital - Rockford, IL

Basic Wind Speed (V) mph	90
Exposure Category	C
Importance Factor (I)	1.2
Wind Directionality Factor (K_d)	0.85
Topographic Factor (K_{zt})	1.0
Building Period (T)	1.106

Number of Floors	7
Building Height (feet)	99.17
N-S Building Length (feet)	165
E-W Building Length (feet)	230
L/B in N-S Direction	0.7
L/B in E-W Direction	1.4

Gust Factor		
	Wind Direction	
Variable	N-S	E-W
Structure	Rigid	Rigid
B	230	165
L	165	230
h	99.17	99.17
z	59.502	59.502
ℓ	500	500
ε	0.2	0.2
α	0.15	0.15
b	0.65	0.65
β	0.2	0.2
L_z	562.6	562.6
c	0.2	0.2
I_z	0.18	0.18
g_q	3.4	3.4
g_v	3.4	3.4
Q	0.83	0.85
n_1	0.90	0.90
g_R	4.17	4.17
V_z	93.95	93.95
N_1	5.41	5.41
R_n	0.05	0.05
η_h	4.39	4.39
η_B	295293.52	211841.01
η_L	24.46	34.09
R_h	0.20	0.20
R_B	0.00	0.00
R_L	0.04	0.03
R	0.00	0.00
G_f	0.84	0.85

Wind Direction	$C_{p, windward}$	$C_{p, leeward}$	$C_{p, side wall}$	Gust Factor	Gcpi (+)	Gcpi (-)
N-S Direction	0.8	-0.5	-0.7	0.84	0.18	-0.18
E-W Direction	0.8	-0.4	-0.7	0.85	0.18	-0.18

Wind Pressure Story Shear Forces

N-S WIND (CONTROLLING)

SECTION A-A

STORY SHEAR

8 (ROOF)	43^k
7	85^k
6	104^k
5	101^k
4	78^k
3	75^k
2	68^k
1	32^k

TOTAL BASE SHEAR

555^k (N-S WIND)

369^k (E-W WIND)

APPENDIX D

Spot Check calculations for a typical roof beam on the 5th floor (current roof/mechanical floor)

SPOT CHECK CALCS

EXISTING ROOF BEAM (FROM COLUMN G4 to H4, NW-SE) [5th FLOOR]

SPAN = 22' 7½" (22.625')

TRIB WIDTH = $\frac{22.625'}{2}$ = 11.3125'

DL = 130 psf

LL = 150 psf

$TL = 1.2(DL) + 1.6(LL)$
$= 1.2(130) + 1.6(150)$
$= 396$ psf

$\omega = 396 \text{ psf} \times (11.3125') = 4.48$ klf.

$M_u = \frac{\omega L^2}{8} = \frac{4.48(22.625^2)}{8}$ • simply supported Beam

$= 287^{1K}$

=> W16 x 26 (14) w/ 3" LOK DECK AND 3¼" LW CONCRETE [EXISTING]

• (14) ¾" φ SHEAR STUDS ⊥ DECK

$\frac{w_r}{h_r} = \frac{6''}{3''} = 2 \geq 1.5$

• 14 STUDS OVER 22' = 1 WEAK STUD/RIB * Assume weak DIRECTION

-> TABLE 3-21

$Q_n = 17.2^K$ /STUD

$Q_T = 17.2(\frac{14}{2}) = 120.4^K$

• $Y_2 = 6¼'' - \frac{1}{2}'' = 5¾''$ * Assume effective depth of conc (a) = 1"

TABLE 3-19

• W16 x 26

• $Q = 96^K \leq 120^K$

• $Y_2 = 5.5'' \leq 5.75''$

=> $\phi M_n = 248 \leq 287^{K'}$ W16 x 26 NO GOOD TO SMALL

CHECK:

$b_{eff} = \min \begin{cases} \text{spacing} = \frac{22.625}{2} = 11.3125' \\ \frac{\text{SPAN}}{4} = \frac{22.625}{4} = 5.66' \end{cases}$

$\frac{Q_T}{.85 f'_c b_{eff}} = a = \frac{120.4}{.85(4)(67.8'')} = 0.52'' \leq 1.00''$

$Y_2 = 5.5'' \leq 6''$ ✓OK STILL CONSERVATIVE

ROOF BEAM CONTINUED

TABLE 3-19:

- TRY W16 x 31
- $Q = 114^k \leq 120^k$
- $Y_2 = 5.5'' \leq 6''$

$\Rightarrow \phi M_n = 300'^k \geq 287'^k$ ✓OK

check:

$b_{eff} = 5.60'$

$\frac{Q_t}{.85 f'_c b_{eff}} = a = \frac{120.4}{.85(4)(67.8'')} = 0.52''$

$Y_2 = 6\frac{1}{4}'' - \frac{0.52}{2} = 6.00'' \geq 5.5''$

$Y_2 = 5.5''$ is conservative

W16 x 31 (14) ✓OK

→ OR Add MORE SHEAR STUDS TO EACH SIDE OF THE W16x26 BEAM

APPENDIX E

Spot Check calculations for a typical roof girder on the 5th floor (current roof/mechanical floor)

EXISTING ROOF GIRDER (FROM COLUMN H4 - H5, NE - SW) [5th FLOOR]

SPAN = 22' 7½" (22.625')

TRIB WIDTH = $\frac{22'7½"}{2} + \frac{18'1½"}{2}$

= 11.3125 + 9.0625 = 20.375'

TRIB AREA = $\frac{22.625'}{2} \times 20.375'$

= 230.5 ft²

DL = 130 psf
LL = 150 psf
TL = 1.2(130) + 1.6(150)
= 396 psf

$P = \frac{(396\ psf)(230.5\ ft^2)}{1000} = 91.3^{K}$

$M_u = \frac{PL}{4} = \frac{91.3^K (22.625')}{4} = 516.4^{1K}$

• BEAM FRAMING IN AT MIDSPAN (POINT LOAD)

=> W18×35 (26) w/ 3" LOK DECK AND 3¼" LW CONCRETE

• (26) ¾" φ SHEAR STUDS // TO DECK

$\frac{w_r}{h_r} = \frac{6}{3} = 2 \geq 1.5$

$Q_r = 21.2^K/\text{stud}$ (TABLE 3-21)

$Q_t = (21.2)(\frac{26}{2}) = 276^K$

• $Y_2 = 6¼" - \frac{1"}{2} = 5¾"$

* Assume effective conc depth a = 1"

TABLE 3-19

- W18×35
- $Q_t = 260^K \leq 276^K$
- $Y_2 = 5.5" \leq 5.75"$

=> $\phi M_n = 445^{1K} \leq 516^{1K}$ NO GOOD / TO SMALL

CHECK:

$b_{eff} = \min \begin{cases} \text{SPACING} = 20.375' \\ \frac{\text{SPAN}}{4} = \frac{22.625}{4} = 5.66' \end{cases}$

$\frac{Q_t}{.85 f'_c b_{eff}} = a = \frac{276}{.85(4)(67.8)} = 1.2"$

$Y_2 = 6¼" - \frac{1.2}{2} = 5.65$ => $Y_2 = 5.5$ is CONSERVATIVE

ROOF GIRDER CONTINUED

TRY W18x40

$Q_T = 272^k \le 276^k$

$Y_2 = 5.5" \le 5.65"$

$\Rightarrow \phi M_n = 501^{'k} \le 516^{'k}$ NO GOOD

TRY W18x46

$Q_t = 240^k \le 276^k$

$Y_2 = 5.5" \le 5.65"$

$\phi M_n = 542^{'k} \le 516^{'k}$ ✓OK

$\Rightarrow$ USE W18x46 w/ 26 STUDS

OR USE SMALLER BEAM
AND INCREASE # OF STUDS

Appendix F

Live load reduction factors and corresponding column loads for a typical gravity column.

Column H-4

Live Load Reduction

Level	AT	AI	Reducible	L.L. Red. (%)
8	556.5	2226	NO	1.00
7	556.5	2226	YES	0.57
6	1113	4452	YES	0.47
5	556.5	2226	NO	1.00
4	1669.5	6678	YES	0.43
3	2226	8904	YES	0.41
2	2782.5	11130	YES	0.40
1	556.5	2226	NO	1.00

Column H4 - Load Takedown

Level	L.L. (psf)	DL	Total Load(k)	Col. Load (k)
8	25	80	75.7	
7	80	90	100.6	75.7
6	80	90	93.9	176.2
5	150	130	220.4	270.2
4	80	90	91.0	490.5
3	80	90	89.2	581.5
2	80	90	88.6	670.8
SOG			Total	759.4

APPENDIX G

Spot Check calculations for a typical gravity column along its length

SPOT CHECK COLUMN H-4

* SEE SPREADSHEET FOR LIVE LOAD REDUCTION CALCS AND TOTAL LOAD CALCS ON COLUMN H-4.

SAMPLE:

8th Floor (Future Roof)

A_T	A_I	L_o	
556.5 ft²	2226	25 psf (SNOW)	UNREDUCIBLE

TYP FLOOR (2nd, 3rd, 4th, 6th, 7th)

	A_T	A_I	L_o	L.L %	L.L
7th	556.5 ft²	2226	80 psf	0.568	45.4 psf

$$L = L_o\left(.25 + \frac{15}{\sqrt{A_I}}\right)$$
$$= 80\left(.25 + \frac{15}{\sqrt{2226}}\right)$$
$$= 80(.568)$$
$$= 45.4 \text{ psf}$$

	A_T	A_I	L_o	L.L %	L.L.
2nd	5(556.5)	11130 SF	80 psf	0.40	32 psf

$$L = 80\left(.25 + \frac{15}{\sqrt{11130}}\right)$$
$$= 80(.39) \quad * \; 0.39 \leq 0.40 \rightarrow \text{use } 0.40$$
$$= 80(0.40)$$
$$= 32 \text{ psf}$$

LOAD TAKEDOWN

LEVEL	DL	LL	LL%	TL (K)
8	80	25	1.00	75.7
7	90	80	0.57	100.6
6	90	80	0.47	93.9
5	130	150	1.00	220.4
4	90	80	0.43	91.0
3	90	80	0.41	89.2
2	90	80	0.40	88.6

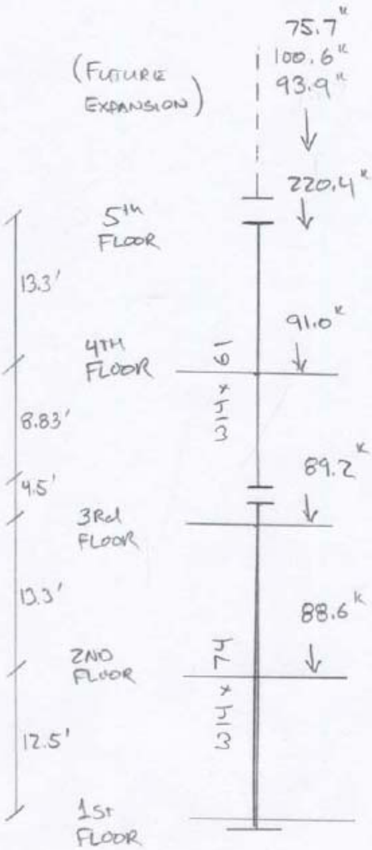


TABLE 4-1

$P = 490.6^k$, $l_b = 13.3'$ ⟶ W14×61, $\phi P_n = 572^k > 490^k$ ✓OK

$P = 581.6^k$, $l_b = 8.83'$ ⟶ W14×61, $\phi P_n = 700^k > 581^k$ ✓OK

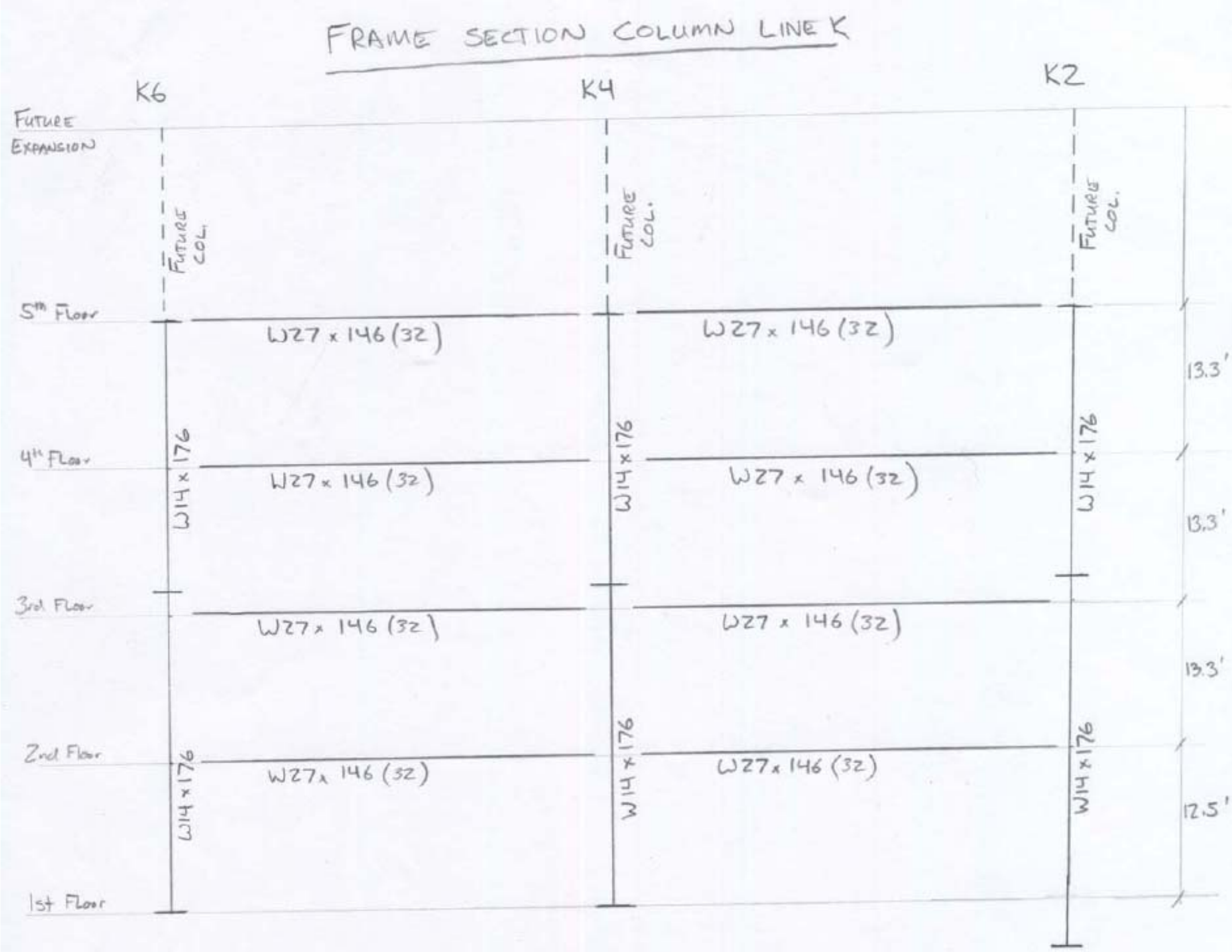
$P = 670.8^k$, $l_b = 13.3'$ ⟶ W14×74, $\phi P_n = 701^k > 670.8^k$ ✓OK

$P = 759.4^k$, $l_b = 12.5'$ ⟶ W14×74, $\phi P_n = 750^k \leq 759^k$ ✗NO GOOD

$\boxed{\text{W14×82},\ \phi P_n = 828^k > 759^k\ \text{✓OK}}$

Appendix H

Simplified check of a lateral framing element: Moment Frame along Column Line K



COLUMNS [TABLE 3-2]

W14 x 176 $\phi M_p = 1200^{'K} \geq 1106^{'K}$

W14 x 145 $\phi M_p = 975^{'K}$

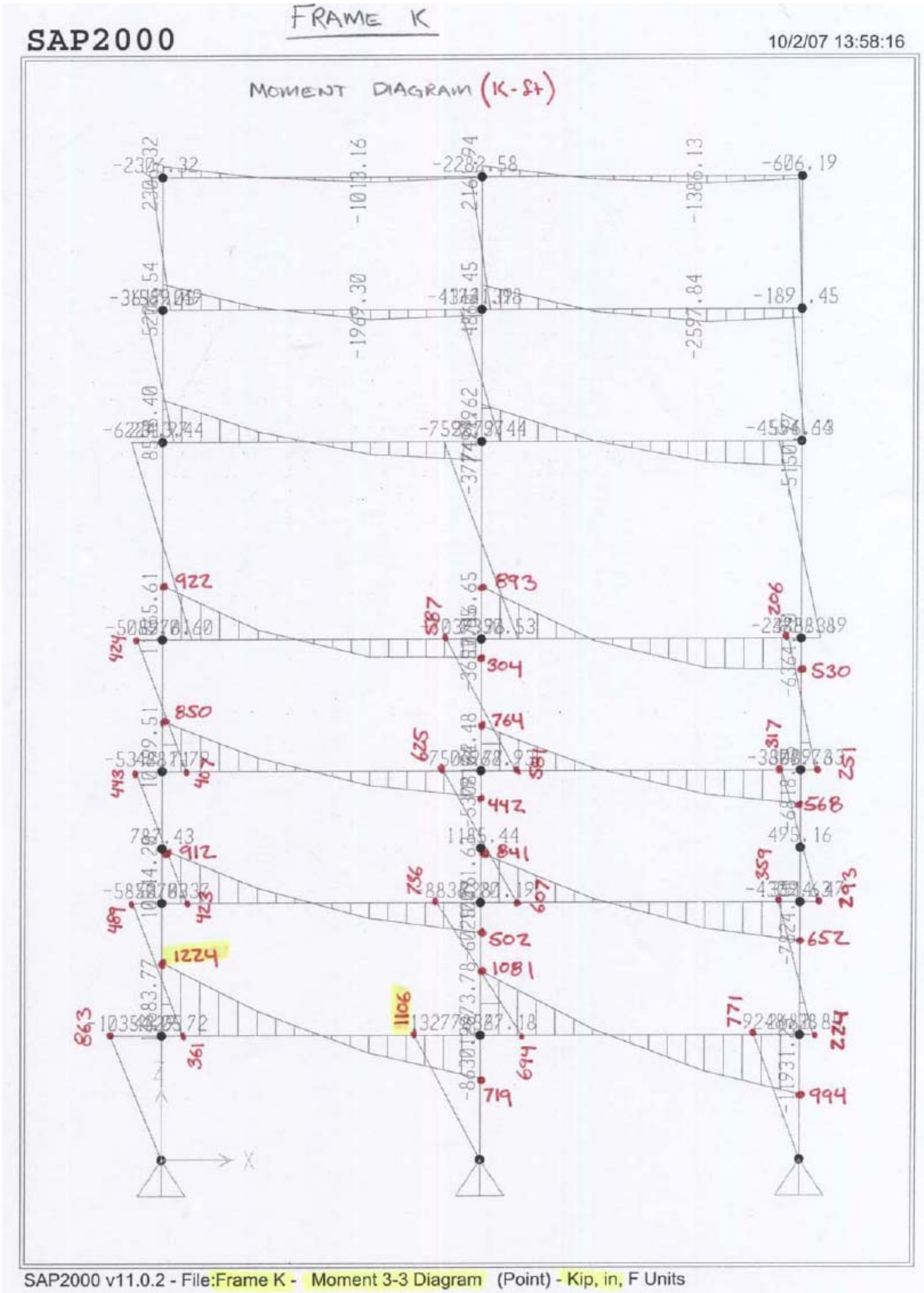
W14 x 109 $\phi M_p = 720^{'K}$

W14 x 82 $\phi M_p = 521^{'K}$

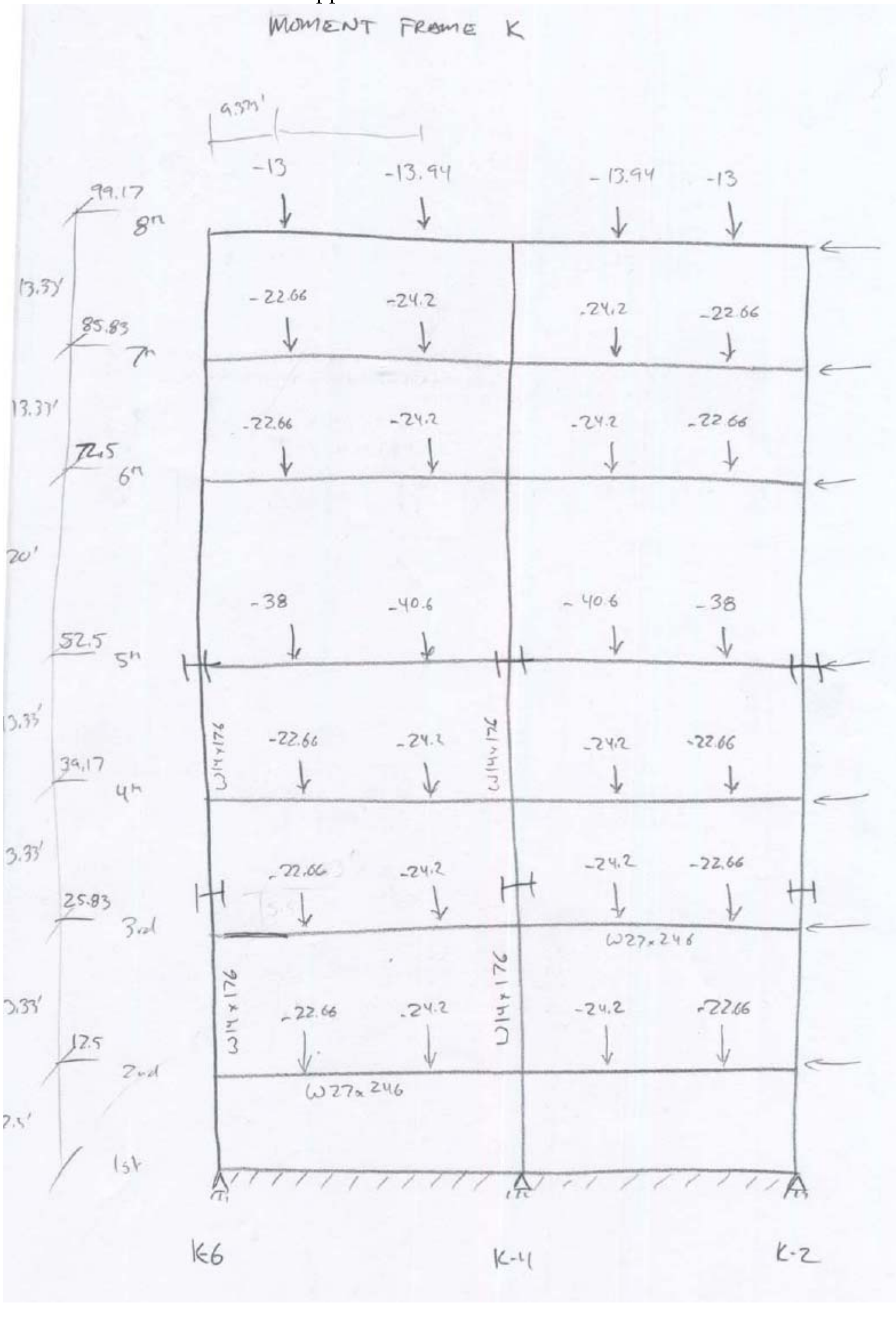
BEAM

HAND CALCS ON NEXT PAGE

SAP Computer Model output listing maximum moments along beam spans and column lengths



Sketch of Frame K with imposed point loads from transverse beams and lateral loads from seismic forces calculated in Appendix B.



Approximate calculation of concentrated framing loads from transverse beams framing into the Floor Girders at all levels along Frame K

8th FLOOR: DL = 80 LL = 25 TL = 1.2(80) + 1.6(25) = 136 psf

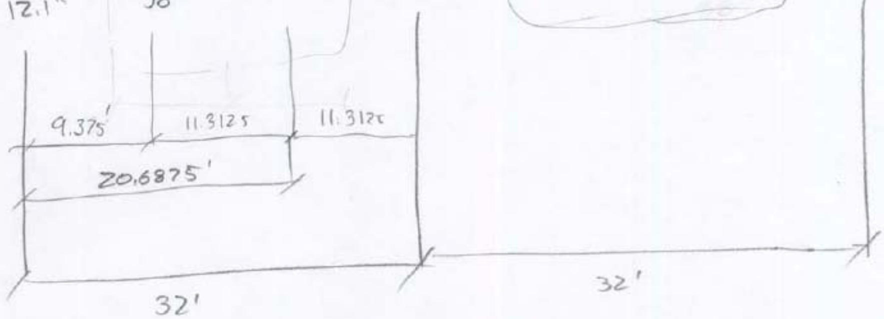
	A	B	C	D	E	F	G	TOTAL
AREA:	30.5 ft²	96 ft²	102.5 ft²	102.5 ft²	102.5 ft²	96 ft²	100 ft²	
P =	4.15 k	13 k	13.94 k	13.94 k	13.94 k	13 k	13.6 k	53.58

TYP FLOOR: DL = 90 LL = 80 TL = 1.2(90) + 1.6(80) = 236 psf
2nd, 3rd, 4th
6th, 7th

	A	B	C	D	E	F	G	TOTAL
P =	7.2 k	22.66 k	24.2 k	24.2 k	24.2 k	22.66 k	23.6 k	468.6

5th FLOOR: DL = 130 psf LL = 150 TL = 1.2(130) + 1.6(150) = 396 psf
MECH

	A	B	C	D	E	F	G	TOTAL
P =	12.1 k	38 k	40.6 k	40.6 k	40.6 k	38 k	39.6 k	157.2
								679.68



LATERAL LOAD DISTRIBUTION

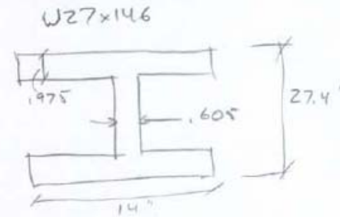
* Assume Central Core MOMENT FRAMES RESIST 100% LATERAL LOAD IN NE-SW DIRECTION
* 4 MOMENT FRAME RESIST LOAD IN THAT DIRECTION
  → FRAME K will RESIST 1/4 of LOAD

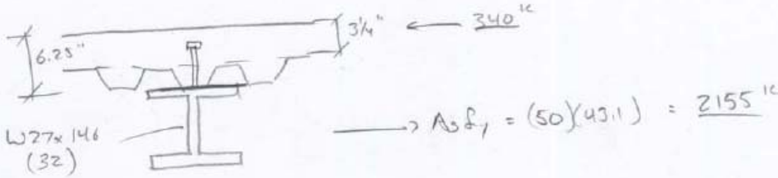
FLOOR	LOAD	FRAME K
8	208.8	52.2 k
7	200.5	50.1
6	182.5	45.6
5	143.7	36.0
4	74.9	18.8
3	47.1	11.8
2	17.5	4.4
		218

Appendix I

Composite Moment capacity of floor girders along Frame K

FRAME K : GIRDER CAPACITY

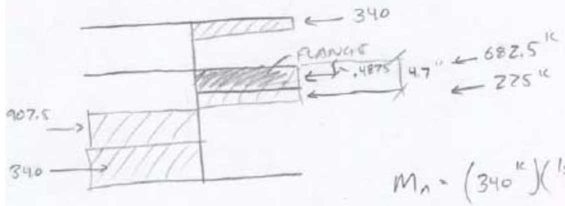




$Q = (16)(21.2^k)$
$= 340^k$

* Assume $b_{eff} = 8'$ (96")

$a = \frac{340}{.85(4)(96)} = 1.04"$



$M_n = (340^k)\left(\frac{1.04"}{2}\right) + (682.5^k)(6.25" + .4875")(2)$
$+ (225^k)(6.25" + 4.7")(2) - (2155^k)\left(\frac{27.4"}{2} + 6.25\right)$

$= 28691\ k''$
$= 2391\ k'$

$\phi M_n = (.9)(2391\ ft\text{-}k)$
$= 2152\ ft\text{-}k > 1224\ ft\text{-}k$

✓OK