Butler Memorial Hospital

Butler, PA

James D. Rotunno

Senior Thesis Final Report

Structural Option

Advisor: Dr. Ali Memari

April 11, 2010

BHS
Butler Health System

Project Team:

Owner: Butler Healthcare Providers
Owners Representative: Ritter Construction Management Inc.
Construction Manager: Turner Construction
Architect: Design Group
Design Architect: Hammel, Green, Abrahamson

Building Statistics

Size: 206,000 Square Feet
Height: 134'-3"
Levels: 6 above grade & 2 below grade
Construction Dates: Sept. 2008 - Summer 2010
Function: Primarilly Surgery & Recovery
Cost: 93 Million

The foundation system is made up of drilled piers at varying depths and diameters from 30"-78", reinforced concrete grade beams and reinforced concrete foundation walls

Structural System

Structural steel framing with wide flange members for beams and columns and HSS braces
Floor systems are comprised of wide flange beams supporting 3" composite metal decking and 3-1/2" composite concrete floor slabs.
The lateral force resisting system is composed Cevron type braces made from HSS sections

Mechanical System

HVAC requirements are being provided through the use of AHU's at the roof top levels.
Variable air volume (VAV) boxes closer to the discharge destination readjusts the air quality again before delivering it to individual rooms.
Radient ceiling panels are utilized on perimeter walls, and bare fin tube radient heat is used in the overhanging floor areas.

Electrical & Lighting

The electrical system is a 480/277V, 3 phase, 4 wire system for equipment and florescent lighting and steps down toa 208/120 system for general use, recepticles and incandescent lighting.

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M. Kevin Parfitt, P.E. – Thesis Faculty Director
Faculty & Staff

Family and Friends: A special thanks to my wife who has never complained about my time away from home, even when the snow needs cleared or the furnace breaks down.

Executive Summary:

This is the fourth report in a yearlong senior thesis project for The Pennsylvania State University, Department of Architectural Engineering. The subject of this thesis project is The Butler Health System – New Inpatient Tower Addition and Remodel involving a structural depth topic, two breadth area studies, and a member connection design. The primary structural topic is whether or not the proposed redesign of the gravity system; a girder-slab system, for this type of structure is not only theoretically possible but a practical solution as well based on depth and breadth studies.

Existing structural design features are initially discussed including foundation and gravity with a primary focus on the lateral force resisting system. An analysis of the design codes and standards are included as well as a determination as wind being the controlling lateral force. The lateral load analysis contains force, distributions, methods, deflection criteria, over-turning moment, and member checks. Conclusions drawn at the end of the lateral analysis reveal that the structures lateral system is designed for strength rather than drift criteria.

The gravity force resisting system was redesigned from a composite deck and composite beam system with a total depth of six and one half inch lightweight concrete to a girder-slab floor system which uses precast hollow-core planks with partially grouted cores, a two inch structural concrete topping and a system of modified castellated W-shape steel members. The slabs rest on the bottom flange of the modified members or HSS shapes used as "shims" and are approximately ½" above the top flange adding approximately one foot of unobstructed ceiling cavity without increasing floor-to-floor heights.

Connections were designed to complete the load path from the gravitational and lateral loads to the columns. Several typical connection designs were completed to ensure functionality and constructability of the systems. Breadth topics of construction management and an acoustical study of conflicting use spaces; which includes an architectural redesign were completed.

Conclusions at the end of each section and the report found that on this particular structure the proposed solution is possible but may not be a practical solution due to costs, delivery method and location; however, the same structure located elsewhere requiring lower floor-to-floor heights may benefit from the use of this type of system.

Introduction:

Butler Health System's new addition located in Butler, PA consists of two sub grade levels which have limited facade and entrances at ground level on the plan west end of the structure. There are five other at or above grade levels that comprise the bulk of the hospitals general facilities. One more final level, the penthouse level, encompasses the mechanical equipment on the roof top.

The structure is approximately 206,000 square feet with floor to floor heights of 14'-8"each. It stands at just a little over 100' tall above the highest grade level and is situated on the middle-top of a hillside. With the exception of the slightly arcing plan north facade the floor plan is quite regular with typical bay sizes being 28' x30'.

Drilled caissons were used for the foundation system which range from 30"–78" in diameter and reach depths of up to 79'. Grade beams between the caissons on the below grade level areas transfer wall loads to the foundation system and provide interior perimeter walls for the lower levels as well as provide support for the slab on grade at the second level. The superstructure is composed of steel W-shape members for the gravity load transfer components and steel HSS members in primarily an inverted chevron bracing pattern which provides the lateral force resisting system for the structure. Almost all member connections are shear connections with the exception of a few moment connections at cantilevering beams. These moment connections however do not contribute to the lateral force resisting system.

The main focus and depth study for this report is on the redesign of the gravity load resisting system. The redesigned system is a fairly new concept in structural design and has only been used since early 2000. This type of system is generally referred to as girder-slab construction and has been limited primarily to housing units, dormitories and hotels. Generally current practices, standards, and research limit this type of system to 15' spans and relatively low live loading (60psf or less). As part of this gravity system five W-shape members were selected and modified into a built up castellated sections with a large compression bar for the top flange.

Also included is a the lateral force resisting system, how loads are applied to the system, the load combinations used to determine the system, and how the system reacts to and distributes these lateral forces. A 2D frame computer analysis is performed as well as hand calculations to compare to the computer output results and to verify minimal spot checks. The braced frames at or above level two; the first level that is completely exposed above grade, will primarily be the focus for both the computer and hand calculation analysis and spot checks.

Included as part of the depth study is how the structure will be connected at different member intersections. Several of these connections are shown as typical connections of different element types to illustrate the load path and how the load is transmitted through the connection. All relevant limit states are considered and calculated to determine the controlling state at each connection and all connections are designed as shear connections.

As part of the two breadth studies done for this project the first is a construction management analysis of the gravity systems effectiveness from a time and cost perspective. This is one of the deciding factors as to the systems viability for a structure of this size and loading requirements.

The second breadth option studied the difference in acoustical performance of the redesigned floor system over the existing one particularly in sound transmission between the first and second levels where there are chillers, boilers and compressors on the first level, directly below conference and board rooms on the second level. This was also looked at using an architectural redesign as a solution to any acoustical issues that were determined.

The proposed system is evaluated in the final conclusions section based on all of the above information, research and designs for its technical and practical viability for this type of structure use as well as other building types.

Structural System:

Existing System: Rigid Diaphragm

Existing conditions for the originally designed floor system consists of composite steel decking with lightweight concrete (f'c = 3500psi @28 days). It has 20 gauge steel decking with 3" deep flutes, ¾" diameter 5" long shear studs and an additional 3.5" of concrete. The girders supporting the beams and floor system are typically W21x50, 28' long with 38 shear studs. There are typically four beams per bay including the ones at each column line. The beams are typically W18x40 evenly spaced at ten foot intervals and are 30 feet long with 28 shear studs each.

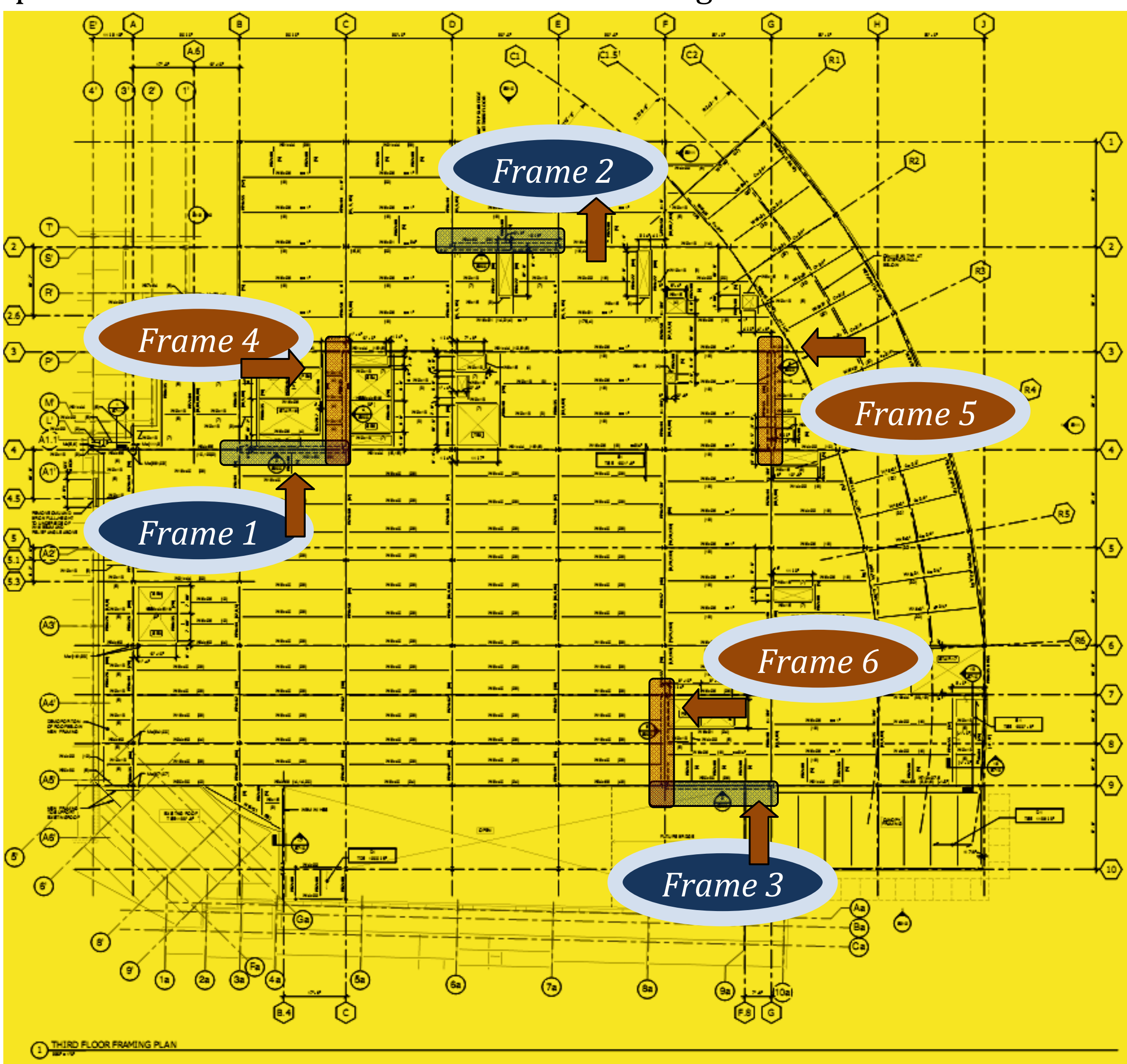


Figure 4.1: Third floor framing plan with braced frame locations shown

The composite deck and composite beam floor system is what comprises the rigid diaphragm to transfer the lateral loads into the lateral load resisting system as shown in the partial system of level 3 in Figure 4.2 below. The highlighted areas indicate the braced frame locations.

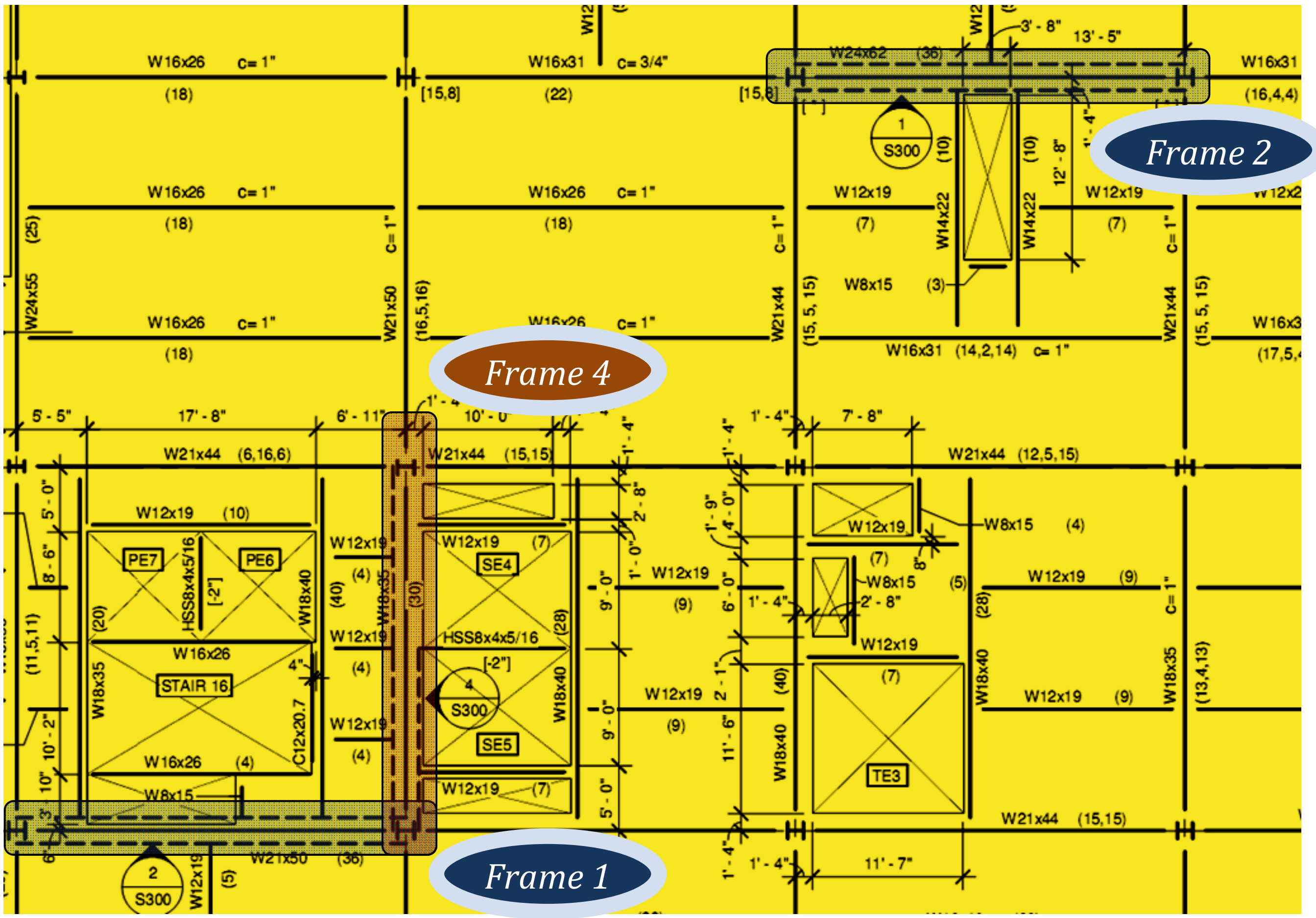


Figure 4.2: Enlarged view from Figure 4.1

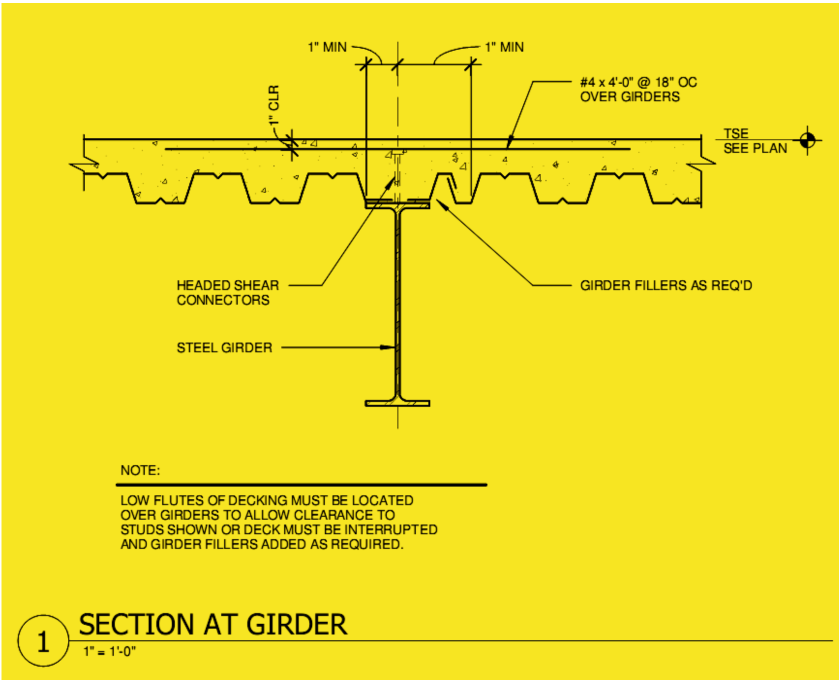


Figure 4.3: Existing slab & beam/girder conditions

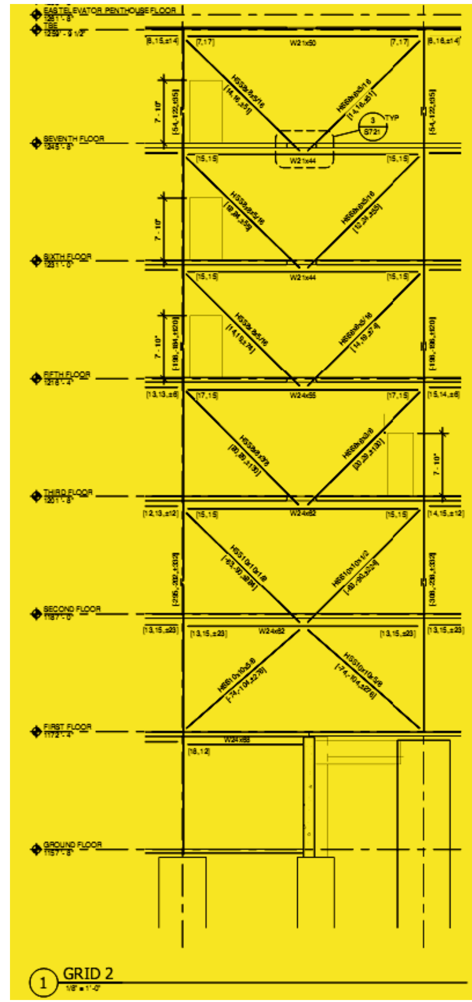


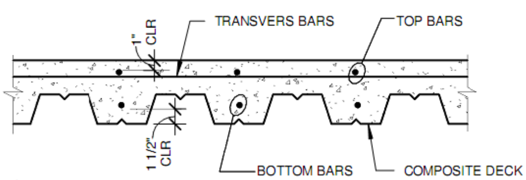
Figure 4.4: Typical lateral bracing elevation

SLAB/DECK SCHEDULE

MARK	TOTAL THICKNESS	TYPE	DECK			CONCRETE		STUD LENGTH	REINFORCING	
			DEPTH	GAGE	FINISH	THICKN	TYPE		REINF	DETAIL
S1	6 1/2"	COMP DECK	3"	20	GALV	3 1/2"	LW	5"	WWF 6x6 W2.1xW2.1	
S2	6 1/2"	COMP DECK	3"	18	GALV	3 1/2"	LW	5"	#5 @ 12"OC T & B #4 @ 12"OC TRANSVERSE	
D1	3"	ROOF DECK	3"	20	GALV	-	-	-	-	

NOTES:

1. ALL COMPOSITE SHEAR CONNECTORS (STUDS) ARE 3/4"Ø UNO.
2. NW=NORMAL WEIGHT CONCRETE; LW=LIGHTWEIGHT CONCRETE.
3. STUD LENGTHS ARE LENGTHS AFTER WELDING.
4. SEE DETAILS 1,2,3/S701 FOR SLAB REINFORCING.
5. SEE 14-16/S700 FOR DECK WELDING.
6. SEE 17/S700 FOR COMPOSITE DECK STUD PLACEMENT.



NOTE:
SEE SLAB/DECK SCHEDULE
FOR BAR SIZES & SPACING

1 SLAB/DECK SCHEDULE
1" = 1'-0"

Figure 4.5: Existing slab/deck schedule

Existing System: Foundation

Drilled caissons were used for the foundation system which range from 30"–78" in diameter and reach depths of up to 79' and are socketed 3' into competent rock. Grade beams between the caissons on the below grade level areas transfer wall loads to the foundation system and provide interior perimeter walls for the lower levels as well as provide support for the slab on grade at the second level. The piers have been designed for both end bearing and skin friction with an allowable end bearing pressure of 20 TSF and an allowable lateral earth pressure that varies with the depth of the soil strata from a minimum of 3TSF through fill and decomposed rock to a maximum of 12 TSF in the limestone/siltstone layer. They are comprised of 4000 psi @ 28 days strength concrete, ASTM A615 Grade 60 deformed bars with 12" minimum Class B tension lap splices where required and conform to ACI 318 design code.

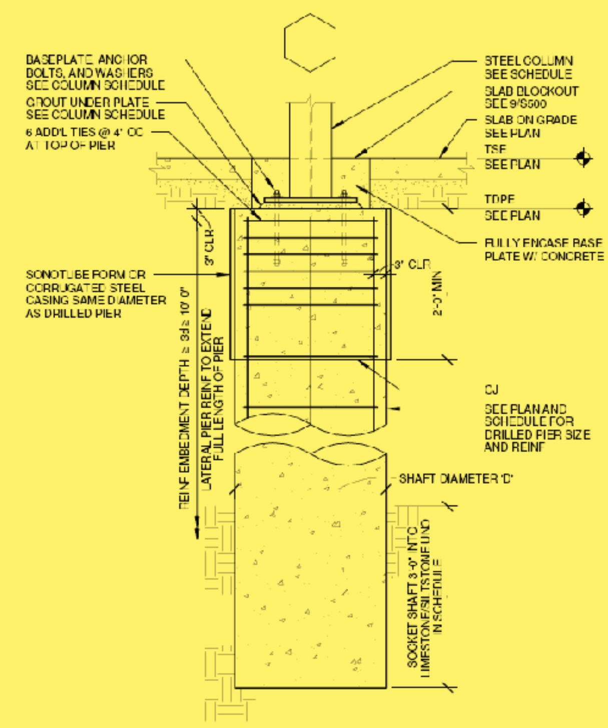
DRILLED PIER SCHEDULE

MARK	SHAFT DIAMETER	VERTICAL REINFORCEMENT	TIES	SOCKET DEPTH INTO LIMESTONE/SILTSTONE LAYER, 36" MIN UNO
DP30	30"	(6)-#7	#4 @ 14"	
DP36	36"	(7)-#8	#4 @ 16"	
DP42	42"	(7)-#9	#4 @ 18"	
DP48	48"	(7)-#10	#4 @ 18"	
DP54	54"	(9)-#10	#4 @ 18"	
DP60	60"	(9)-#11	#4 @ 18"	
DP66	66"	(11)-#11	#4 @ 18"	
DP72	72"	(13)-#11	#4 @ 18"	
LDP54	54"	(25)-#10	#5 @ 12"	
LDP60	60"	(25)-#11	#5 @ 6" TOP 10'-0" #5 @ 12" REM	
LDP66	66"	(30)-#11	#5 @ 6" TOP 10'-0" #5 @ 12" REM	
LDP72	72"	(36)-#11	#5 @ 4" TOP 20'-0" #5 @ 12" REM	
LDP78	78"	(42)-#11	#5 @ 4" TOP 20'-0" #5 @ 12" REM	5'-0" SOCKET AT GRID C-3

NOTES

1. TOP OF DRILLED PIER ELEVATION IS AS FOLLOWS, UNO ON PLAN:
   DPXXA (TDPE = 1156'-8")
   DPXXB (TDPE = 1171'-4")
   DPXXC (TDPE = 1186'-0")
2. LDPXX INDICATES LATERAL PIER. REINFORCING TO EXTEND FULL LENGTH OF PIER.
3. SEE 3/S511 FOR ADDITIONAL TIES AT TOP OF DRILLED PIERS.
4. ALL DRILLED PIERS TO BEAR ON AND BE EMBEDDED 36" MINIMUM INTO THE LIMESTONE/SILTSTONE.
5. REFER TO GEOTECHNICAL REPORT FOR ASSUMED TOP OF LIMESTONE/SILTSTONE LAYER.
6. SEE 1/S511 FOR APPROXIMATE DRILLED PIER LENGTHS.
7. ALL PIER LENGTHS TO BE REPORTED TO STRUCTURAL ENGINEER DURING CONSTRUCTION.

2 DRILLED PIER SCHEDULE
1" = 1'-0"



3 TYPICAL DRILLED PIER
1/2" = 1'-0"

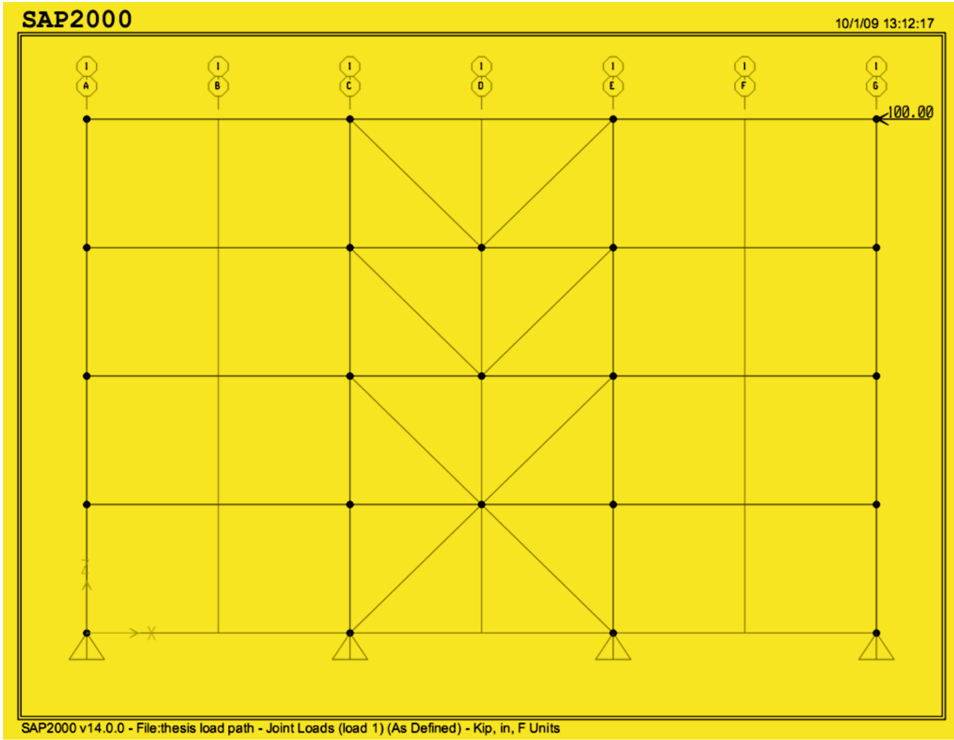
Figure 4.6: Drilled pier schedule

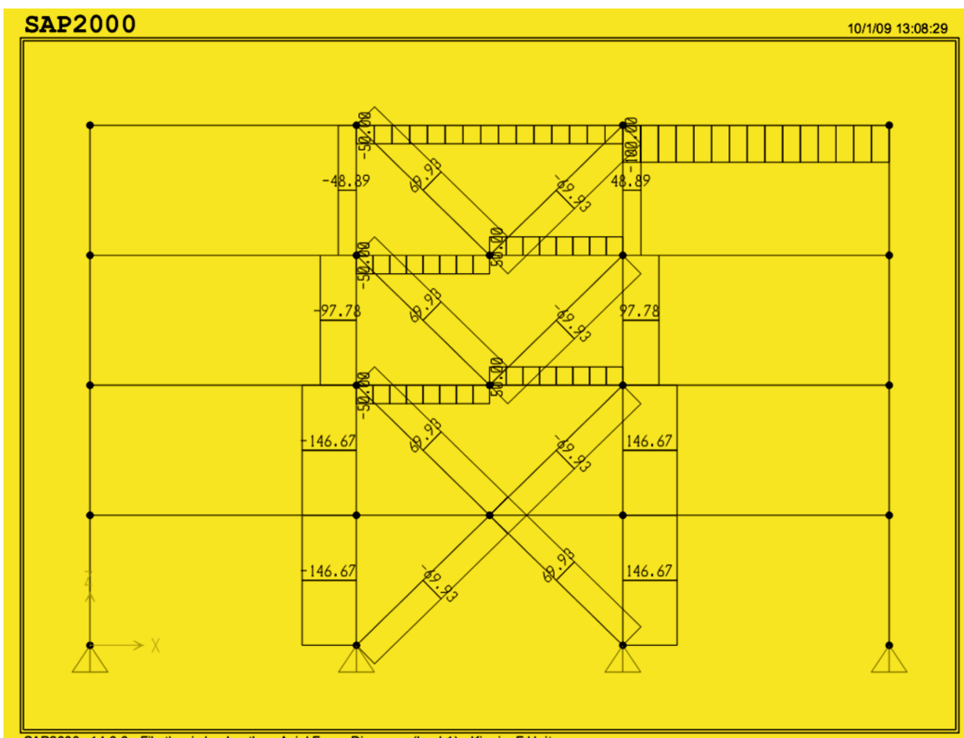
Existing System: Lateral Resistance

Lateral loads caused by wind pressures / earthquake loading are calculated using ASCE 7-05 and are resisted by the structure through the use of several different diagonal inverted Chevron, Chevron, and/or X bracing configurations (see Figure 4.4) located at every floor level in both directions.
The differing wind pressures on the exterior facade are converted to forces per square foot of wall area and are distributed to each floor level by tributary areas through the glazing and brick facade system. From there the floors are assumed to act as rigid diaphragms and distribute each floor load to the braced frames at each level according to their relative stiffness's. This assumption can be made by viewing the composite floor system as being approximately 22-30" thick including the reinforced composite slab and composite beam/girder construction. Where there are openings in the floor, extra beams are located along side/through them to help keep rigidity around/through them. Braced frames #1 & 4 are located in the elevator and stairwell core area to collect and maintain rigidity in that area where there are larger openings.

These loads are then transferred axially through the HSS members and into their corresponding beams and columns. At the beam/girder to HSS connection there is a concentric compressive force from one brace and a concentric tension force from the other brace which cancel each other's vertical components being transferred into the beam/girder; therefore, the force transferred into the member is axial.

See Figures 4.7 & 4.8 on the following page for how the load is distributed from the initial lateral force to the individual bracing and framing elements. Note how the single lateral force at the top of the structure creates the same compressive/tensile force from top to bottom in all bracing members, but the load being transferred axially into the columns increases linearly by the force in the top column until the frame reaches its foundation support. From there the load is transferred to the ground.





Figures 4.7 & 4.8: Simplified example of lateral force distribution to braced frame and lateral load columns

Design Standards & Codes:

2006 IBC
2000 NFPA 101
2006 Guidelines for Design & Construction of Health Care Facilities
1998 Pennsylvania Department of Health Rules and Regulations for Hospitals
ASCE 7-05: for wind, seismic, snow and gravity loads
ACI 318-08: for concrete construction
AISC Thirteenth Edition: for steel members
ASHRAE Handbook: HVAC Applications & Fundamentals
PCI 2003 for vibration
ATC 1999 for vibration (ADAPT technical note TN209 3/21/09 for reference)

Possible load case combinations: From ASCE 7-05 § 1605.2.1

(Only combinations that include Wind, Earthquake and/or Snow)

*Note: The snow load would be added to the total weight of the building for the earthquake loading calculations; therefore, snow by itself would not be considered.

D=Dead, L=Live, W=Wind, E=Earthquake, S=Snow, F=Fluid, T=Temperature, H=Lateral Earth Pressure, L_r=Live roof, R=Rain

1) $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L \text{ or } 0.8W)$
   $1.2D + 1.6L_r + 0.8W$ for gravity and lateral
   $0.8W$ for just lateral
2) $1.2D + 1.6W + L + 0.5(L_r \text{ or } S \text{ or } R)$
   $1.2D + 1.6W + L + 0.5L_r)$ for gravity and lateral
   $1.6W$ for just lateral
3) $1.2D + 1.0E + L + S$
   $1.2D + 1.0E + L + S$ for gravity and lateral
   $1.0E$ for just lateral
4) $0.9D + 1.6W + 1.6H$
   $0.9D + 1.6W$ for gravity and lateral
   $1.6W$ for just lateral
5) $0.9D + 1.0E + 1.6H$
   $0.9D + 1.0E$ for gravity and lateral
   $1.0E$ for just lateral

1.6W or 1.0E will control for just lateral loading on the structure, whichever proves to be higher.

Design Load Summary:

Gravity Loads					
Description/location	DL/ LL	ASCE 7-05/ IBC 1607.9 values	HGA's values	Reduction available/used	Design value
Concrete floors	DL	90-115pcf	115pcf	NO/NO	115pcf
MEP/partitions/finishes	SDL	20-25psf	44psf	NO/NO	35psf
1st floor mechanical	LL		125psf	YES/NO	125psf
2nd floor/ lobby	LL	100psf	100psf	YES/NO	100psf
Hospital floors	LL	40-80psf	80psf	YES/YES	80psf
Stairs & exits	LL	100psf	100psf	NO/NO	100psf
5th floor roof	LL		115psf	NO/NO	115psf
Mech. Penthouse floor	LL		125psf	NO/NO	125psf
Elevator Machine room floor	LL		125psf	YES/NO	
Roof top equipment areas	LL		125psf (or actual equipment wt.)	NO/NO	125psf
Balconies	LL	100psf	100psf	YES/YES	psf
*Snow	LL	24-30psf	24-30psf	NO/NO	24-30psf

See Appendix C for calculations

Table 4.1: For total dead weight of building for seismic loading

Wind Loads are determined using ASCE 7-05 Section 6.5, which is Main Wind Force Resisting System (MWFRS) method 2- analytical procedure. See ASCE 7-05 Section 6.5 Table 1B for design factor values needed in calculations. All values, factors and equations are derived from section 6. To Determine the Gust Effect Factor (G) the structure had to be determined as a rigid structure. To make this assumption 100/h has to be ≤ 1. Making the assumption that h was just under 100 feet based on the fact that the first two levels are minimal compared to the rest of the structure and there is only one wall face exposed on each; therefore the bulk of the structure completely exposed above ground would meet the requirement.
See Appendix A of structure under construction for clarity; the lowest level faces west. The wind and seismic calculations from the previous technical reports were revisited and final values were adjusted based on more accurate factor values.
See Appendix B for wind calculations.
See Appendix D for seismic calculations.

WIND LOAD
BASIC WIND SPEED (3 SECOND GUST) .... 90 MPH
WIND IMPORTANCE FACTOR 1.15
WIND EXPOSURE CATEGORY C
MEAN ROOF HEIGHT .... 122 FT
INTERNAL PRESSURE COEFFICIENT .... ±0.18
TOPOGRAPHIC FACTOR, Kzt .... 1.62 MAX AT BASE
1.09 MIN AT MEAN ROOF HEIGHT

Figure 4.9: Wind load data from construction documents

Wind Load Data for Calculations

East-West direction			ASCE section
Basic wind speed	V	90mph	6.5.4 (Figure 6-1)
Mean roof height	h	122ft	
Wind directionality factor	K_d	0.85	6.5.4 (Table 6-4)
Importance Factor (Occupancy category IV)	I	1.15	6.5.5 (Table 6-1)
Exposure category		C	6.5.6.3
Velocity pressure coefficient	K_z	varies	6.5.6 (Table 6-3)
Topographic factor	K_{zt}	varies	6.5.7.1 (Figure 6-4)
Gust effect factor	G	0.856	6.5.8
Enclosure Classification		Enclosed	6.5.9
Internal pressure coefficient	GC_{pi}	±0.18	6.5.11.1 (Table 6-3)
External pressure coefficients windward side	C_p	0.8	6.5.11.2 (Figure 6-6)
External pressure coefficients leeward side	C_p	-0.5	(Figure 6-6)
Velocity pressure @ height Z	q_z	varies	6.5.10
Velocity pressure @ mean roof height	q_h	30.41lb/ft^2	6.5.10
Design wind load	F	determined	

Table 4.2: Wind load data table for East - West loading

East – West Base & Story Shears with Overturning Moment

Level	**Height** (ft)	**Pressure** (lbs/ft ^{2})	**Force** (F)kips	**Shear** (V) kips	**Moment** (M) Kips*ft
		Windward + leeward			
0- Ground	0	24.59	21.64	545.75	4000.3
1	14'-8"	24.59	52.10	524.11	3841.7
2	29'-4"	26.48	69.29	472.01	3459.8
3	44'-0"	27.30	81.26	402.72	2951.9
5	58'-8"	27.57	84.91	321.46	2356.3
6	73'-4"	27.59	84.64	236.55	1733.9
7	88'-0"	27.38	83.51	151.91	1113.5
8-Roof	102'-8"	26.87	49.5	68.4	501.4
9- P.H. 1	122'-0"	26.30	13.52	18.9	182.7
10- P.H. 2	135'- 0"	25.87	5.38	5.38	34.97
			Base Shear =	**545.75**	
			Overturning Moment =		**20176.52**

Table 4.3: See Appendix B for calculations and drawings

Wind Load Data for Calculations

North-South direction			ASCE section
Basic wind speed	V	90mph	6.5.4 (Figure 6-1)
Mean roof height	h	122ft	
Wind directionality factor	K_d	0.85	6.5.4 (Table 6-4)
Importance Factor	I	1.15	6.5.5 (Table 6-1)
Exposure category		C	6.5.6.3
Velocity pressure coefficient	K_z	varies	6.5.6 (Table 6-3)
Topographic factor	K_{zt}	varies	6.5.7 (Figure 6-4)
Gust effect factor	G	0.857	6.5.8
Enclosure Classification		Enclosed	6.5.9
Internal pressure coefficient	GC_{pi}	±0.18	6.5.11.1 (Table 6-3)
External pressure coefficients windward side	C_p	0.8	6.5.11.2 (Figure 6-6)
External pressure coefficients leeward side	C_p	-0.5	(Figure 6-6)
Velocity pressure @ height Z	q_z	varies	6.5.10
Velocity pressure @ mean roof height	q_h	30.41/ft^2	6.5.10
Design wind load	F	determined	

Table 4.4: Wind load data table for North – South loading

North - South Base & Story Shears with Overturning Moment

Level	**Height** (ft)	**Pressure** (lbs/ft 2)	**Force** (F)kips	**Shear** (V) kips	**Moment** (M) Kips*ft
		Windward + leeward			
0- Ground	0	0	0	557.55	4086.84
1	14’-8”	24.60	15.69	557.55	4086.84
2	29’-4”	26.61	72.10	541.86	3971.83
3	44’-0”	27.33	98.45	469.76	3443.34
5	58’-8”	27.61	100.27	371.31	2721.70
6	73’-4”	27.63	93.73	271.04	1986.72
7	88’-0”	27.43	86.37	177.31	1299.68
8-Roof	102’-8”	26.91	62.53	90.94	666.59
9- P.H. 1	122’-0”	26.34	23.96	28.41	274.58
10- P.H. 2	135’- 0”	25.90	4.45	4.45	28.93
			Base Shear =	**557.55**	
			Overturning Moment =		**22567.05**

Table 4.5: See Appendix B for calculations and drawings

Snow loads are determined using ASCE 7-05 Chapter 7. The design values in sections 7.1-7.3 all agree with HGA’s values (see Appendix C notes on snow loads.) A minimum roof design load of 30psf will be used for calculations.

SNOW LOAD
GROUND SNOW LOAD, Pg .... 25 PSF
FLAT ROOF SNOW LOAD, Pf .... 24 PSF
MINIMUM ROOF DESIGN LOAD .... 30 PSF
SNOW IMPORTANCE FACTOR 1.2
SNOW EXPOSURE FACTOR, Ce 1.0
THERMAL FACTOR, Ct (BUILDING) 1.0
THERMAL FACTOR, Ct (CANOPIES) 1.2

Figure 4.10: Construction document values

As per ASCE 7-05 § 12.7.2; effective seismic weight:

4) where the flat roof snow load exceeds 30psf use 20%; otherwise it is not required. (P_f designed and calculated = 24psf (Therefore not applicable)

Seismic Design:

Criteria are based off of ASCE 7-05 Chapters 11, 12, 14 & 22 for seismic design. Initially in Technical Report #3 (lateral system analysis) a C_s value of 0.046 was calculated to multiply with the total building weight (W_T) to determine the base shear and then distribute this base shear to the individual levels. The effective seismic weight (W_T) is determined using information from ASCE 7-05, §12.7.2., and totaled using an excel spreadsheet found in Appendix D.

SEISMIC DESIGN DATA

SPECTRAL RESPONSE ACCELERATION, Ss	0.0127
SPECTRAL RESPONSE ACCELERATION, S1	0.0055
SITE CLASS	C
SEISMIC IMPORTANCE FACTOR	1.5
SEISMIC DESIGN CATEGORY (SDC)	A

Figure 4.11: Construction document data for seismic

Technical Report #3 Calculations

V= base shear = Cs*W_T

Cs= 0.0456

W_T= 18675.1 kips

V= 851.58 kips

Rechecking and revaluating the seismic data and calculations from the previous report it was determined from Chapter 11, § 4-7 that the structure is located in a an area where the Seismic Design Category (SDC) is A. ASCE 7-05 §11.7.2 for design category A lets the designer use a more simplified and less stringent lateral design force for the structure.

11.7.2 Lateral Forces. Each structure shall be analyzed for the effects of static lateral forces applied independently in each of two orthogonal directions. In each direction, the static lateral forces at all levels shall be applied simultaneously. For purposes of analysis, the force at each level shall be determined using Eq. 11.7-1 as follows:

$$F_x = 0.01 w_x \qquad (11.7\text{-}1)$$

where

F_x = the design lateral force applied at story x, and
w_x = the portion of the total dead load of the structure, D, located or assigned to Level x

Figure 4.12: ASCE 7-05 §11.7.2

This will effectively reduce the previous calculated design loads by approximately 3 times; which will result in drastically lower design values.

			Total Dead Load for Seismic Calculation										
				W_T									
				Load type									
Floor Level	square footage	wall	Plank & Topping	Superimposed	Columns	Beams	equipment	roof	exterior walls		Floor weight	Fx	
		square footage	psf	MEP/Partitions	kips	lb/ft^2	psf	psf	psf/wall		Totals		
			93.0	35.0		10.0	1.0	93.0	28.6		w_t	kips	
Ground	8240												
Level 1	20405	170	1897.67	714.18	70.07	204.05	20.41	0	4.86		2906.4	29.06	
Level 2	45545	458	4235.69	1594.08	60.70	455.45	45.55	0	13.10		6391.5	63.91	
Level 3	42165	458	3921.35	1475.78	82.79	421.65	42.17	0	13.10		5943.7	59.44	
Level 5	31525	458	2931.83	1103.38	50.20	315.25	31.53	0	13.10		4432.2	44.32	
Level 6	27720	678	2577.96	970.20	47.40	277.20	27.72	0	19.39		3900.5	39.00	
Level 7	27760	678	2581.68	971.60	35.83	277.60	27.76	0	19.39		3894.5	38.94	
Level 8 (roof)	45545							4235.69			4235.7	42.36	
TOTALS	**248905**	**2900**	**18146.16**	**6829.2**	**346.99**	**1951.2**	**195.12**	**4235.7**	**82.94**				
						W_T =	31787.3 kips						
					Base Shear =		317.04 kips						

Figure 4.13: EXCEL spreadsheet calculating seismic base and story shear with additional loading of proposed system

<u>Controlling lateral load combination:</u> 1.6W or 1.0E for just lateral loading

<u>1.6W = 1.6(557.550) =892kips, from wind N-S; CONTROLS</u>

1.6W = 1.6(545.75) =873kips, from wind E-W

1.0E = 1.0(317.04) =317kips, from Seismic

A factored load of 1.6 times the wind force at each level will be used in calculations to determine, relative stiffness of braces on each level, distribution of load to braces, and eventually the force in the members.

Lateral System Analysis:

With the design of a different gravity load system the existing lateral resisting system has to be checked for compatibility of the two systems. The system being designed and implemented is commonly referred to as a girder-slab floor system. This particular girder configuration cannot have moment connections at its end supports, due to the fact that if the top flange is in tension (-moment) then the composite member properties/strength would be reduced to just the tensile capacity of the top bar since the concrete in tension theoretically has no tensile capacity that can be relied upon. A concentrically braced frame is the preferred and most economical lateral resistance system for this type of construction. This is also the as designed system type; however, the connections on the drawings were not included and were left up to the contractors design as per my construction document set. The connections to the columns and girders from the lateral elements will be designed as an additional aspect of the lateral load transfer to the gravity components. The lateral elements will again consist of HSS members. The following section of this report goes into detail about the analysis method and force distribution for the lateral force resisting system.

Force Distribution:

For the scope and purpose of this report the braced frame section from level 3 to level 5 along grid line 2 between grid lines D-E will be analyzed; which is what I am calling frame #2 and will be assumed to be resisting N-S applied wind forces. See Figure 4.14 below for frame detail.

See enlarged view on following page Figure 4.15 for more as designed details

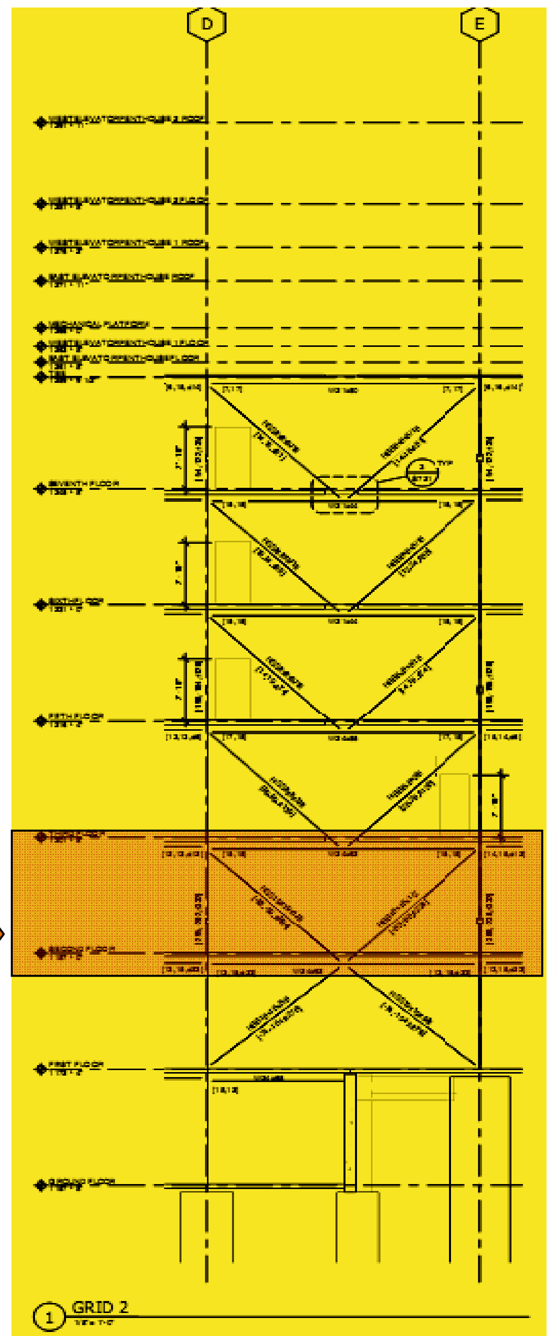


Figure 4.14: Braced frame at grid 2 between D & E

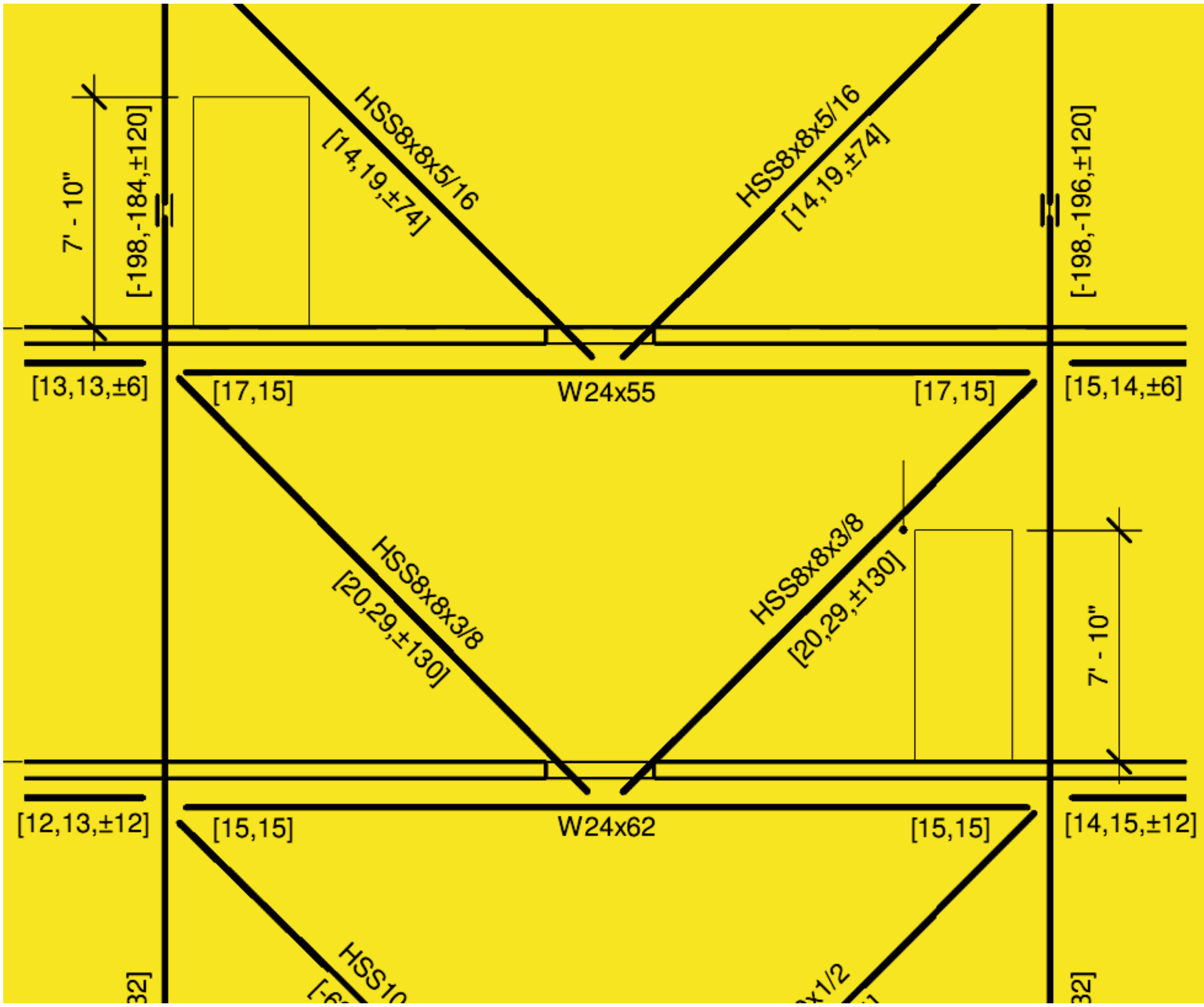


Figure 4.15: Enlarged view of braced frame at grid 2 between D & E

Analysis Method:

As shown earlier in Figures 4.7 & 4.8 a force on an upper level of a Chevron type braced frame will induce a compressive force in one brace and a tension force in the other that will carry itself down through all bracing members below that level. It will also introduce a compressive force in one column and a tensile force in the other that will compound itself linearly in each respective vertical element.

Therefore the forces at each level cannot be analyzed individually; they will have to be combined with the forces acting on the levels above to get a more accurate result. This is also part of the reason why the HSS member sizes increase in section and wall thickness as more floors are added above even as the forces at each level are relatively the same.

The first step in the analysis process is to assume the floor levels are acting as rigid diaphragms and to determine the center of mass for the rigid diaphragm above the level being analyzed, which is the area/mass that is applying the load to the braces. See Appendix E for these calculations.

Next would be to calculate the center of rigidity for each of these levels to determine how much of the force at the respective level will go into each brace at that level based on their relative stiffnesses to each other and torsional effects due to eccentric differences in center of mass versus center of rigidity. This is the axial force being introduced into the bracing elements below the level. Note: Only the diagonal braces in the same direction of the loading will be considered to be resisting the lateral load in that direction; and the columns and beams that make up part of the braced frame are not considered for stiffness criteria.

Once the level forces, center of mass, center of rigidity and relative stiffnesses have been determined then the direct force and eccentric force at that level can be calculated. These two forces can then be added together to determine the force being applied at that level to each individual frame. The value for the eccentric force being added to or subtracted from the direct force will always be considered positive since load reversal can be applied and the eccentric forces would switch signs but he direct forces would remain the same.

These forces can then be applied to the Free Body Diagram for the frame and the element member forces can be determined and checked against the computed design values and subsequent sizes.

See Appendix E for drawings and calculations including FBD of braced frame #2 and SAP verification of FBD and member forces.

Tabulated values of hand calculations

Level	Frame Stiffness (kip/in)						Center of Rigidity		Story Shear (kips)		Eccentricity	
	1	2	3	4	5	6	X (ft)	Y (ft)	N-S	E-W	e_x (ft)	e_y (ft)
3	1198.01	1198.01	2419.07	1573.30	2050.06	2120.89	106.06	121.8	98.45	81.26	19.77	12.3
5	1198.01	1198.01	1520.56	1573.30	2050.06	2120.89	106.06	107.53	100.27	84.91	19.77	1.97
6	1009.10	1198.01	1198.01	1239.57	1573.30	1627.66	105.51	101.37	93.73	84.64	25.89	3.67
7	1009.10	1009.10	1009.10	1044.10	1239.57	1080.18	103.12	100.67	86.37	83.51	7.20	32.13
8	1009.10	1009.10	1009.10	1044.10	1239.57	1080.18	103.12	100.67	62.53	49.50	7.20	32.13
Average	1355.83	1403.06	1788.96	1618.59	2038.14	2007.45	130.9675	133.01			19.9575	20.55
								Total=	441.35	383.82		

Table 4.6: Tabulated values to evaluate member forces

Level	Direct Shear (kips)						Torsional Shear (kips) *5% minimum of Direct					
	1	2	3	4	5	6	1	2	3	4	5	6
3	24.50	24.50	49.46	22.26	29.00	30.00	1.91	8.05	16.668	9.604	3.844	*1.50
5	30.67	30.67	38.93	23.26	30.30	31.35	2.80	11.13	13.93	1.49	*1.52	*1.57
6	32.98	32.98	27.78	23.63	29.99	31.02	2.26	14.35	16.61	2.76	2.07	*1.55
7	28.79	28.79	28.79	25.92	30.77	26.82	*1.44	3.63	4.28	23.78	18.10	5.68
8	20.84	20.84	20.84	15.36	18.24	15.90	*1.04	2.63	3.10	14.10	10.73	3.37
Total	137.78	137.78	165.80	110.43	138.30	135.09	9.45	39.79	37.92	42.13	36.26	13.67

Total Shear (kips)						
Frame	1	2	3	4	5	6
Level 3	26.41	57.51	66.13	31.86	32.84	31.50
Level 5	33.47	41.80	52.86	24.75	31.82	32.92
Level 6	35.24	47.33	44.39	26.39	32.06	32.57
Level 7	30.23	32.42	33.07	49.70	48.87	32.50
Level 8	21.88	23.47	23.94	29.46	28.97	19.27
Total	147.23	202.53	220.39	162.16	174.56	148.76

Table 4.7: Resulting shears due to wind loads

Deflection criteria as per 2006 International Building Code:

Allowable building drift: Δ_{wind} = H/400

Allowable story drift: $\Delta_{seismic}$ = 0. $10h_{sx}$ (Table 12.12-1 ASCE 7-05)

Equation used to calculate story drift Δ_s: $K=P/\Delta_p$ $\Delta_p=P/K$

Wind Drift Comparison of Frame #2									
Level	Story Height (ft)	Story Drift (in)	Allowable Story Drift Δ_{wind} = H/400 (in)			Total Drift (in)	Allowable Total Drift Δ_{wind} = H/400 (in)		
3	14.67	0.0782	<	0.44	Acceptable	0.0782	<	1.32	Acceptable
5	14.67	0.0837	<	0.44	Acceptable	0.162	<	1.76	Acceptable
6	14.67	0.0782	<	0.44	Acceptable	0.240	<	2.2	Acceptable
7	14.67	0.0856	<	0.44	Acceptable	0.326	<	2.64	Acceptable
8/roof	14.67	0.0620	<	0.44	Acceptable	0.388	<	3.08	Acceptable

Table 4.8: Drift Values from hand calculations

SAP 2000 2d Frame Analysis to compare with hand calculations:

The relative stiffness of each frame can be approximated by taking the inverse of the deflection of each frame and relating them to each other by taking its value and dividing by the sum of the other frames in the same participating direction. This could also be done on a level by level basis to get a more accurate assumption. Since the second approach was used for the hand calculations the computer analysis will be done the same way for more consistency.

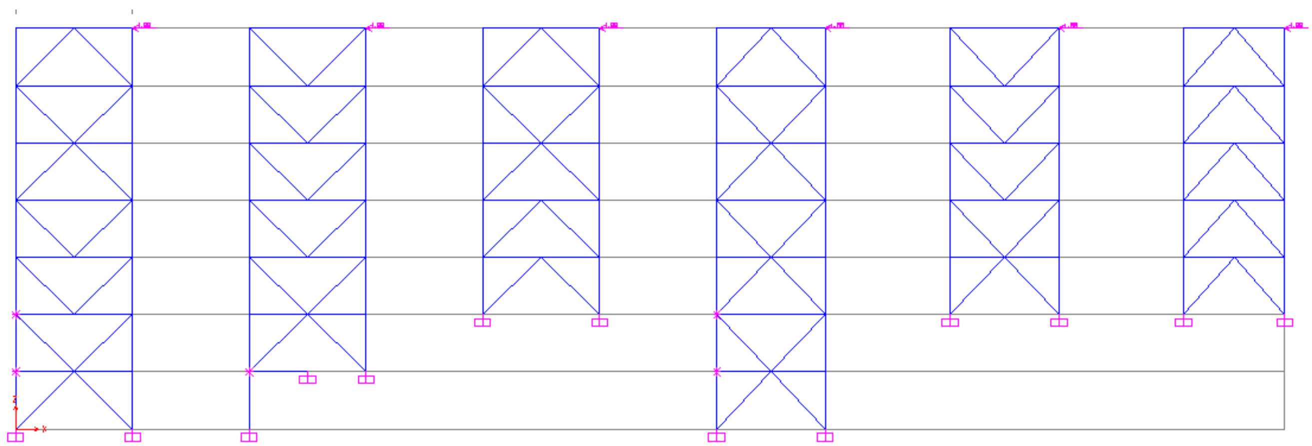
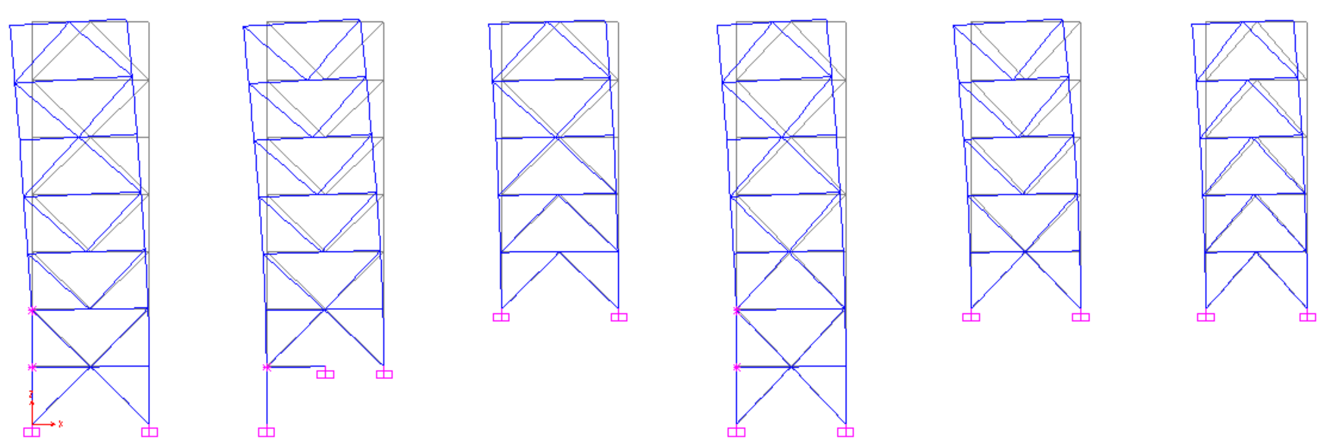


Figure 4.16: Frames 1-6 with 1 kip load applied to determine relative stiffnesses of frames.

Δ_T=-0.01342 Δ_T=-0.01423 Δ_T=-0.00773 Δ_T=-0.01156 Δ_T=-0.01078 Δ_T=-0.00840



#1 #2 #3 #4 #5 #6

Figure 4.17: Deflected shapes with total displacements caused by 1 kip at top

Level	Displacement for Stiffness Calculations (Δ_s)					
	Frames 1-6					
3	0.00251	0.00220	0.00005	0.00157	0.00005	0.00007
5	0.00507	0.00461	0.00168	0.00371	0.00203	0.00182
6	0.00741	0.00761	0.00317	0.00581	0.00445	0.00353
7	0.01042	0.01093	0.00545	0.00860	0.00756	0.00585
8/roof	0.01342	0.01423	0.00773	0.01156	0.01078	0.00840

Table 4.9: Frames 1-6 showing displaced shape due to 1 kip load @ top of frame and relative displacements at each level.

Level	K for each brace ($1/\Delta_s$)					
	Frame 1	Frame 2	Frame 3	Frame 4	Frame 5	Frame6
3	398.406	454.545	20000	636.943	20000	14285.71
5	197.239	216.920	595.238	269.542	492.611	549.451
6	134.953	131.406	315.457	172.117	224.719	283.286
7	95.9691	91.4913	183.486	116.279	132.275	170.940
8/roof	74.5156	70.2741	129.366	86.5052	92.7644	119.048

Table 4.10: Stiffness of each brace at each level (k/in) based off of 1k load @ top of frame

Level	% Stiffness per Brace ($K/\Sigma K$)					
	Frame 1	Frame 2	Frame 3	Frame 4	Frame 5	Frame6
3	1.91	2.18	95.91	1.83	57.31	40.86
5	19.54	21.49	58.97	20.55	37.56	41.89
6	23.20	22.59	54.22	25.31	33.04	41.65
7	25.87	24.66	49.46	27.72	31.53	40.75
8/roof	27.18	25.54	47.19	29.00	31.10	39.91

Table 4.11: Percentage of load to each frame at each level based off of 1k load @ top of frame

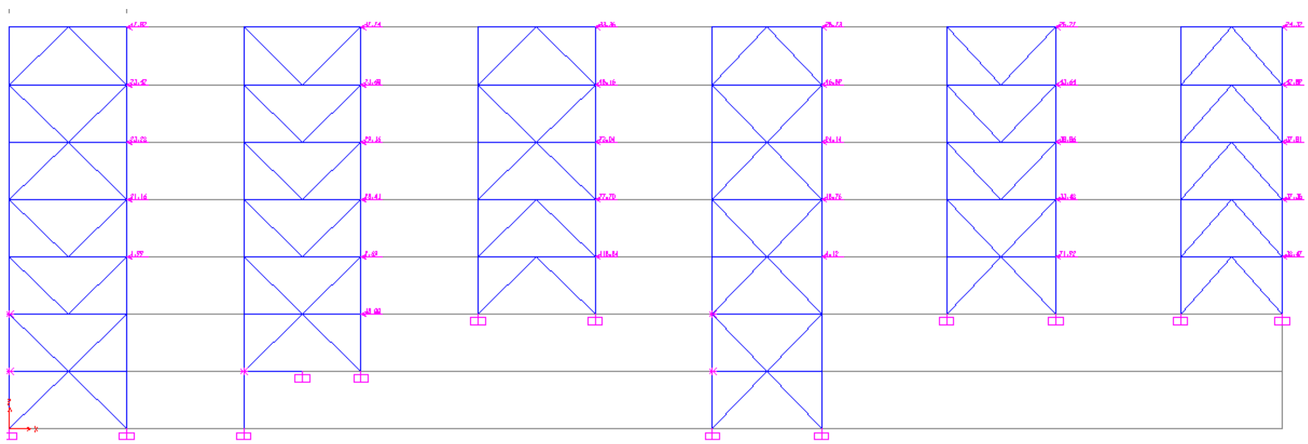
To determine the total force that is transmitted into each brace on each level the values from Table 4.11; as a fraction, are multiplied by the story shear at the corresponding level, which can be found in Tables 4.3 & 4.5. This however does not account for the torsional shear; which can be seen from Table 4.7 in the hand calculations could be close to 30% of the direct shear. To try and reasonably account for these torsional shears the eccentricities calculated by hand are assumed to be accurate here.

SAP Model Calculations												
Level	**Direct Shear** (kips)						**Torsional Shear** (kips) *5% minimum of Direct					
	1	2	3	4	5	6	1	2	3	4	5	6
3	1.88	2.15	94.42	1.49	46.57	33.20	0.11	0.54	24.42	2.63	25.35	5.27
5	19.59	21.55	59.13	17.45	31.89	35.57	1.57	6.86	18.57	1.31	1.59	1.78
6	21.75	21.17	50.82	21.42	27.97	35.25	1.53	7.99	22.22	2.72	2.09	1.76
7	22.34	21.30	42.72	23.15	26.33	34.03	1.08	2.30	5.44	23.74	17.31	8.06
8	17.00	15.97	29.51	14.36	15.39	19.76	0.82	1.77	3.85	14.37	9.88	4.56
Total	82.56	82.14	276.60	77.87	148.15	157.81	5.11	19.46	74.50	44.76	56.22	21.43

Total Shear (kips)						
Frame	1	2	3	4	5	6
Level 3	1.99	2.69	118.84	4.12	71.92	38.47
Level 5	21.16	28.41	77.70	18.76	33.48	37.35
Level 6	23.28	29.16	73.04	24.14	30.06	37.01
Level 7	23.42	23.60	48.16	46.89	43.64	42.09
Level 8	17.82	17.74	33.36	28.73	25.27	24.32
Total	87.67	101.60	351.10	122.63	204.37	179.24

Table 4.12: Resulting shears due to wind loads from SAP 2000

The computed total story shears from Table 4.12 are placed at the nodes of the frames on their corresponding levels in the 2D frame model to evaluate total drift and compare the values with the hand calculations and the 2006 IBC deflection criteria.



#1 #2 #3 #4 #5 #6

Figure 4.18: Frames 1-6 with loads applied to determine deflections of frames to compare with hand calculations.

Comparisons:

Wind Drift Comparison of Frame #2 using SAP 2000 2D									
Level	Story Height (ft)	Story Drift (in)	Allowable Story Drift Δ_{wind} = H/400 (in)			Total Drift (in)	Allowable Total Drift Δ_{wind} = H/400 (in)		
3	14.67	0.1214	<	0.44	Acceptable	0.18813	<	1.32	Acceptable
5	14.67	0.1858	<	0.44	Acceptable	0.37396	<	1.76	Acceptable
6	14.67	0.1912	<	0.44	Acceptable	0.56512	<	2.2	Acceptable
7	14.67	0.1703	<	0.44	Acceptable	0.73544	<	2.64	Acceptable
8/roof	14.67	0.1456	<	0.44	Acceptable	0.88103	<	3.08	Acceptable
Wind Drift Comparison of Frame #2 using hand calculations									
Level	Story Height (ft)	Story Drift (in)	Allowable Story Drift Δ_{wind} = H/400 (in)			Total Drift (in)	Allowable Total Drift Δ_{wind} = H/400 (in)		
3	14.67	0.0782	<	0.44	Acceptable	0.0782	<	1.32	Acceptable
5	14.67	0.0837	<	0.44	Acceptable	0.162	<	1.76	Acceptable
6	14.67	0.0782	<	0.44	Acceptable	0.240	<	2.2	Acceptable
7	14.67	0.0856	<	0.44	Acceptable	0.326	<	2.64	Acceptable
8/roof	14.67	0.0620	<	0.44	Acceptable	0.388	<	3.08	Acceptable

Table 4.13: Drift comparison table

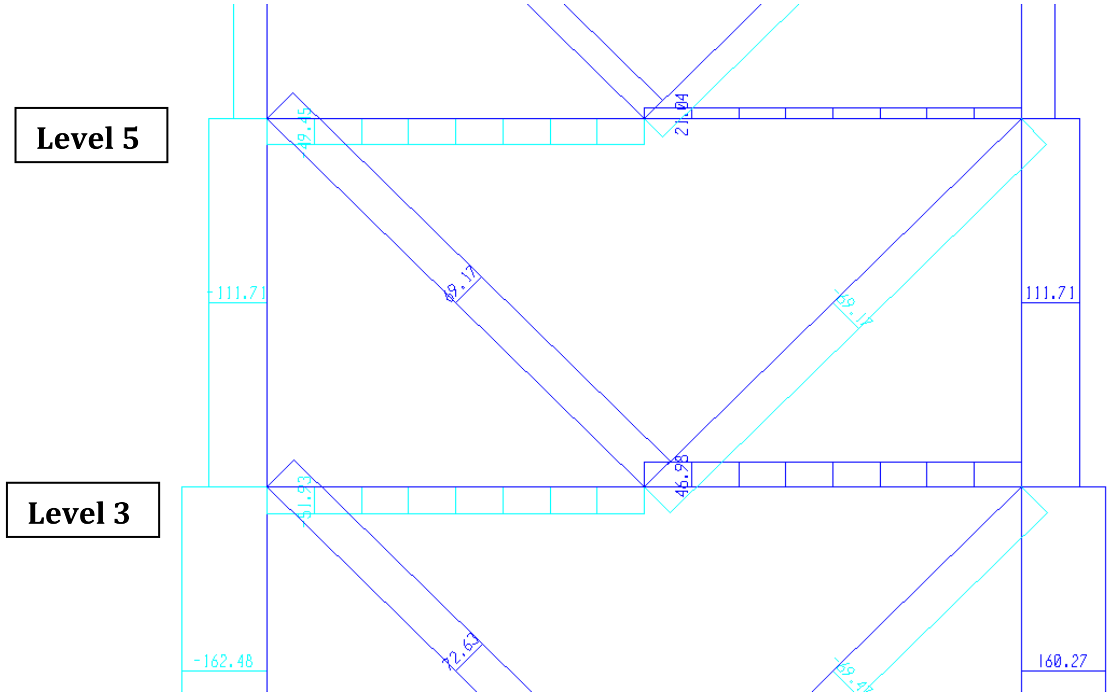


Figure 4.19: SAP 2000 frame #2 axial load output

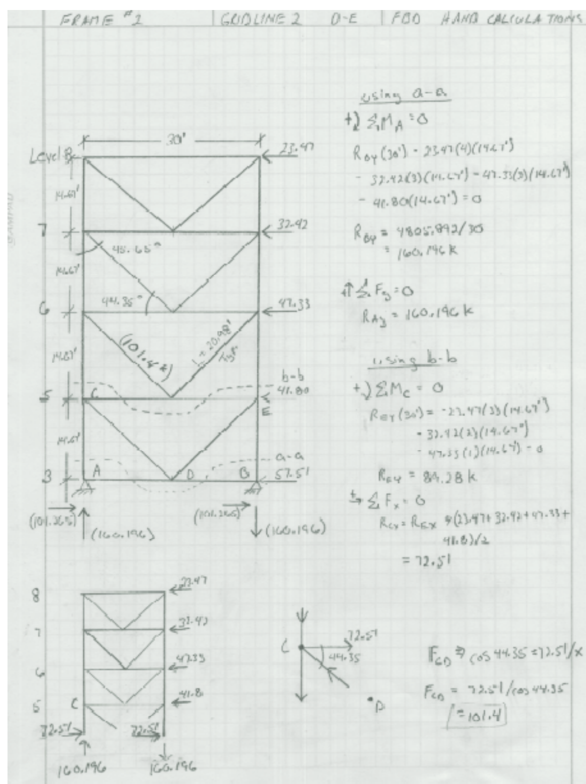
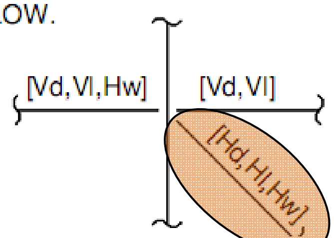


Figure 4.20: FBD of hand calculation for frame #2 to compare values with Figure 4.19. An enlarged view of this figure can be found in the beginning of Appendix F.

BRACED FRAME CONNECTION NOTES

1. SEE PLAN, BRACE ELEVATIONS, AND COLUMN SCHEDULE FOR MEMEBERS SIZES.
2. BRACING AND BEAM MEMBER SERVICE (UNFACTORED) FORCES ARE INDICATED ON ELEVATIONS, AS SHOWN IN FIGURE BELOW.
   Hd=AXIAL DEAD LOAD
   Hl=AXIAL LIVE LOAD
   Hw= AXIAL WIND LOAD
   Vd=SHEAR DEAD LOAD
   Vl=SHEAR LIVE LOAD



   TENSION AXIAL FORCE AND DOWNWARD SHEAR FORCE ARE POSITIVE.
   COMPRESSION AXIAL FORCE AND UPWARDS SHEAR FORCE ARE NEGATIVE.
3. NO REDUCTION IN SERVICE LEVEL FORCES OR INCREASES IN ALLOWABLE STRESSES SHALL BE ALLOWED IN DESIGN OF CONNECTIONS.
4. FABRICATOR TO ENSURE BRACE LENGTHS AND SLOT DIMENSIONS ALLOW PLACEMENT OF BRACE BETWEEN GUSSET [PLATES.
5. SEE GENERAL STRUCTURAL NOTES ON SHEET S001 FOR ADDITIONAL INFORMATION.
6. BEAMS AND COLUMNS ARE NOT DESIGNED FOR BENDING MOMENT DUE TO CONNECTION ECCENTRICITY. PROPORTIN CONNECTION TO TO ELIMINATE ADDITIONAL MOMENTS ON BEAMS AND COLUMNS
7. AXIAL FORCES IN BRACED FRAME BEAMS ARE NOT SHOWN. DETERMINE FORCE REQUIRED TO OBTAIN CONNECTION FORCE EQUILIBRIUM.

1 BRACED FRAME NOTES

1" = 1'-0"

Figure 4.21: Description from print to show value meanings and to compare with SAP and hand calculations.

Axial Force in Brace from Level 3 to Level 5 in Frame #2			
	Print	Hand Calculations	SAP 2000
H_w	130kips	101.4kips	69.17kips

Table 4.14: Brace comparison values

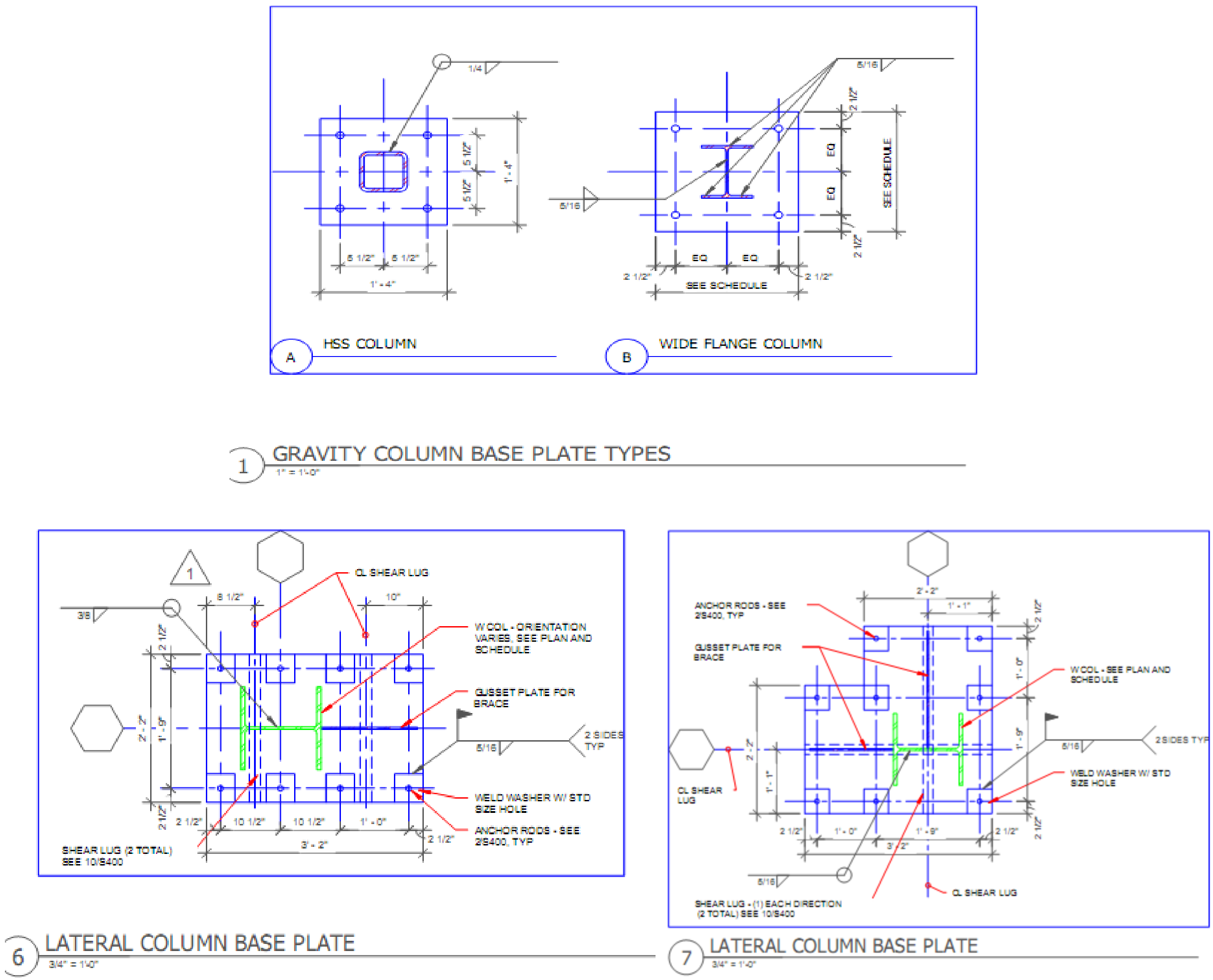


Figure 4.22: Gravity and Lateral base plate to foundation connection detail

Overturning:

The drawings in Figure 4.22 depict the differences in the base plate to caisson connection details for lateral versus gravity columns. The reason for the difference in anchor size, depth, number and layout is because of the overturning moment caused by the lateral loading on the structure. As shown in Figures 4.19 & 4.20 at each braced frame location there will be one side of the frame columns in compression and the other in tension.

Depending on the lateral loading direction there will also be a moment of approximately 20,000-23,000 foot kips applied to the base of the columns, this load (moment) would be distributed among the columns which are participating in the loaded direction similar to the manner in which the lateral load is distributed to the braced frames.
The uplift force seen in the columns that are in tension would be negated by the gravity forces in the columns imposed by dead and live loading of the structure as well as the connected weight of the 30"-78" Ø and up to 79' deep caissons; therefore overturning issues would not be a concern or issue.

Member Checks:

The bracing member compared in Table 4.14 is checked for strength and size using the hand calculation and the value given on the construction documents. One column in the same braced frame between levels 3 and 5 is also checked for compression, lateral stability and size. To compare and evaluate the members in the design documents the gravity loads applied to the columns, beams and HSS members and any moments that are applied to the columns also have to be considered. After determining the gravity loads, the loads will be applied to a simple 2D SAP model to get the member forces to be added to the lateral analysis.

These calculations can be found in Appendix F at the end of this report.

Lateral System Conclusions:

Based on the calculations and comparisons in this report the lateral force resisting system is designed for strength rather than for drift considerations. This conclusion seems completely plausible since two of the levels are relatively small compared to the rest of the structure and are only minimally exposed on one side. There are five other main levels above ground and a smaller penthouse level on the roof. The height of these levels compared to the length and width of the structure is approximately 1:2 making the building relatively short, almost symmetrically square and stocky.

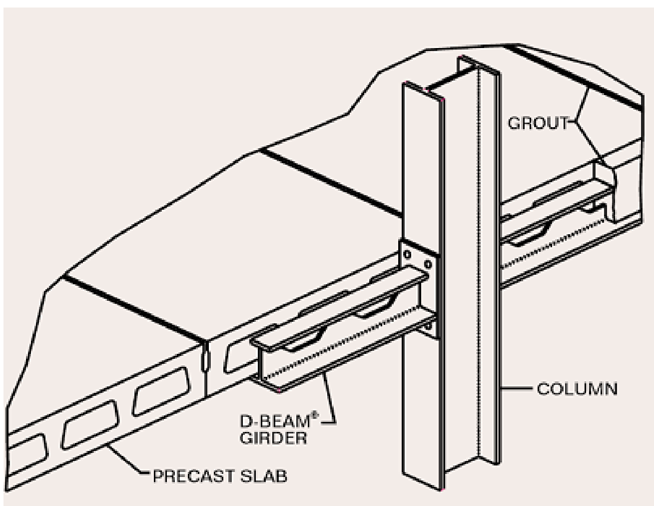
These features would indicate that the structures lateral deformations should be less than code standards as compared to taller and thinner structures and therefore the bracing elements would be designed more for a governing strength limit state.

Hand and SAP calculated drift values compiled in Table 3.13 on page 26 for code vs. calculated values shows that the story and total drifts are approximately 3.5 times less than code standards indicating that a smaller profile could have been used to control building drift.
It was also shown that the construction document data for the lateral system and bracing members was oversized as compared to the hand calculations by a factor of 40-50%. The bracing member that was checked shows a service wind load (unfactored) of 130kips on the construction documents while the hand calculated values are 101kips factored.

This discrepancy in values and subsequent member sizes could be from multiple reasons. First some of the assumptions and simplifications of the wind values may have been different than design values and led to lower than designed for wind loads. Secondly only the wind was considered as contributing to the axial load in the braces. The gravity and live loads will also induce axial loads in these members. The design loads were also done with the original penthouse designs being larger and a full height rooftop screen wall (13′ high), both of which will increase the lateral design loads. The screen wall was omitted and the penthouses reduced in size. The controlling limit states for the connections have also not been considered at this point and may contribute to an increase in member sizes. Vibration concerns in hospital operating rooms and rooms with sensitive equipment may also have an effect on member sizes.

Redesigned Gravity System

In the second of the three previous technical reports, alternate floor systems were briefly introduced and analyzed. As part of this process the girder-slab gravity type floor system appeared to be a possible viable substitute for the existing design; however, its concept is relatively new and current use has been restricted to smaller spans and much smaller loading conditions. To determine if this is in fact a theoretical as well as practical solution for the building structure several aspects will have to be examined more closely. Starting with the list of advantages and disadvantages listed in Technical Report #2, each entry will have to be evaluated and accepted or dispelled for this particular building type, bay sizing and loading configuration.



Figures 4.23 & 4.24: Modified castellated sections

Disadvantages:

- Large lead times with this type of system
- Girders and columns would need fireproofing
- Much more efficient and cost effective at shorter spans
- Column spacing may have to be reduced, increasing footing requirements
- Floor penetrations must be well coordinated with the slab designer/manufacture

Advantages:

- Easy & fast to install
- The lateral system can still be utilized
- No formwork required and concrete slabs are already at usable capacity when they arrive
- No intermediate beams in interior of bays needed
- Can be installed in any type of weather
- Other trades can start work underneath almost immediately
- Additional unobstructed ceiling space for MEP's.
- Meets or exceeds floor fireproofing requirements
- Reduce noise transmission from floor to floor through baffled cavities
- No increase in floor to floor heights
- Reduces overall weight of the structure

To evaluate these two lists an initial girder-slab floor design process will have to be determined and followed. The following is a list of steps in the redesign procedure. Steps in the redesign process:

1. Determine the design loads that the structure will be resisting both gravity and lateral per ASCE 7-05.
2. Design of the hollow core planks to find the total depth required and the weight per square foot of the floor system.
3. Configure the load path to be followed including which type of connections will be used between members.
4. Calculate the shape and size of the castellated girders needed to resist the shears and moments induced on them by the floor loading.
5. Assuming the use of the existing column sizes, calculate the total weight of the building.
6. Compare the base shear values for wind and seismic, rework load combinations as per ASCE 7-05 and find which combination is the controlling combination.
7. Distribution of lateral loads on the structure.
8. Size column, girder and bracing members for total loads.

Analysis Process:

To determine the design loads for the first step of this design process, two separate approaches were used. The first was to determine which areas of the structure would experience the most gravitational loading as per ASCE 7-05 since this type of loading would be the majority or sole loading condition on almost all girders. The lateral force induced into the girders axially will be minimal compared to that of the gravity loading. It was determined that these areas would be in the girders that support the rooftop level where there are large live loads from equipment, and hallways and corridors on the lower levels where live loads are larger and non-reduced.

The second approach was to cross check these areas with the as designed beam and girder sizes to determine the location and sizes of the largest members. The same areas that were determined to carry the largest loads in the first step coincided with the locations of the largest designed members. From these two combined approaches the largest as designed composite member moment was determined and compared with the values of the simply supported girder moment value.

The largest calculated M_u value is approximately 77% of the largest ΦM_n of the as designed W-Shapes; therefore, this gives a starting point to develop a composite modified castellated section to carry the applied loading and an identical non-composite castellated section to carry the construction loading and control deflections until the grout in the composite section reaches its 28 day compressive strength.

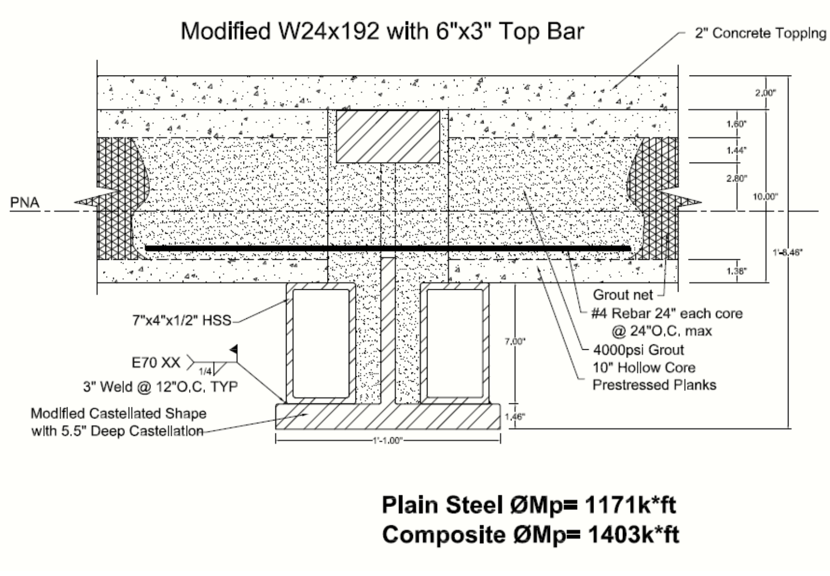


Figure 4.25: 1 of 5 designed sections

Calculated Values:

Span (ft)	M_u @ 80psf LL & Constant DL (k*ft)	M_u @ 125psf LL & Constant DL (k*ft)
14	207	260
27	770	967
28	828	1040
29	888	1115
30	950	1193
31	1014	1274
32	1080	1358

Table 4.15: Calculated values for M_u

Modified Girder Shape Size (Modified)	Shear Capacity@ Least Section (kips)	Total Depth Inc. 2" Concrete Topping (in)	Non-composite Plastic Moment Capacity (ΦM_p) (k*ft)	Composite Plastic Moment Capacity (ΦM_{pc}) (k*ft)
W_m27x217	359.8	22.50	1328	1674
W_m24x192	314.0	20.46	1171	1403
W_m18x211	345.0	18.91	985	1287
W_m14x193	233.6	15.44	652	877
W_m10x68	70.1	12	286	Uncalculated

Table 4.16: Calculated values for Modified Girders

Calculations and data can be found in Appendix G at the end of this report for loading, girder sizing and girder capacities.

The load path determination in the second step of the design process is determined through the design of the connection details which is covered later in this report.

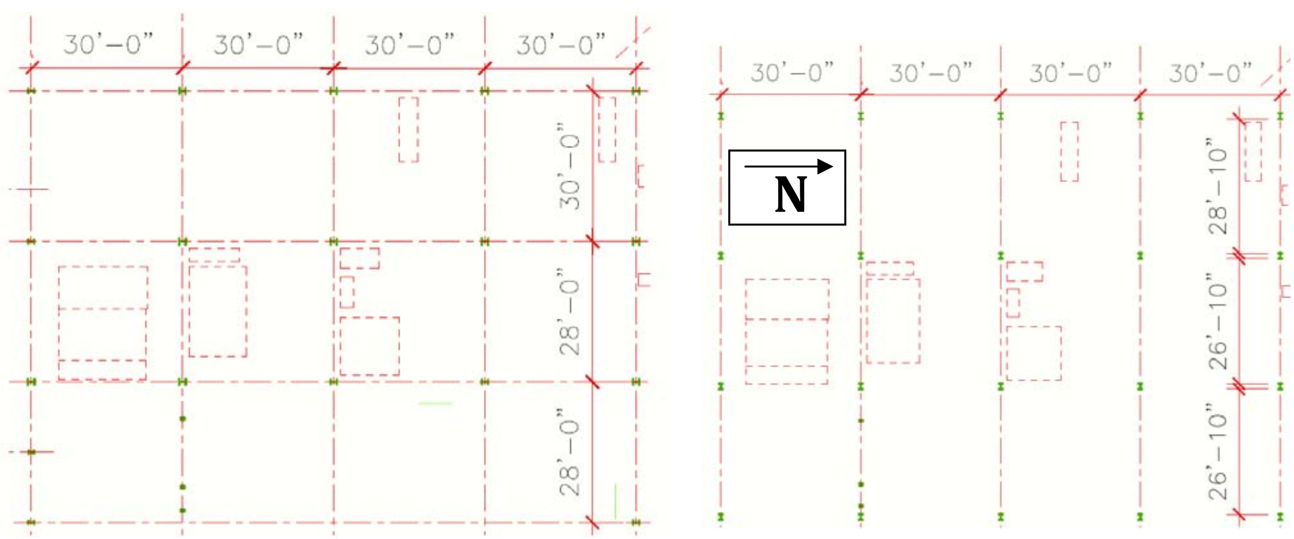
For the design and determination of the prestressed concrete hollow core planks a plank size and type was selected from Nitterhouse Concrete Products design literature. See Appendix G for selected plank and loading capacities.

The total weight of the structure was compiled and tabulated in an EXCEL spreadsheet and shown in Figure 4.13 for seismic calculations.

System Component Viability:

Once these initial steps have been completed a closer inspection of the disadvantages has to be completed to determine if any of the negative drawbacks of the system can be mitigated. The most detrimental aspects of the system would be the longer spans, loads that are 2.5-3 times larger than conventional girder-slab systems, and accommodating multiple and larger openings. These will be the main focus; if these issues cannot be properly addressed then the system is not going to be an option.

An initial step to reduce the moment at midspan of the modified girders was to analyze whether or not the columns could be rotated 90° about their axes to minimize the span length and make connections from girder to columns in the strong axis direction. The majority of the columns strong axes run in the N-S direction which is the same direction in which all of the bay sizes have 30' spans. In the E-W direction the majority of the spans are 28' or less. Having the columns in this orientation could effectively reduce most spans and subsequently their maximum applied moment; however, there would still be some remaining bay spans in this direction that would remain at 30'. These bays however would be in areas where there is reduced loading, somewhat compensating for the increased length.



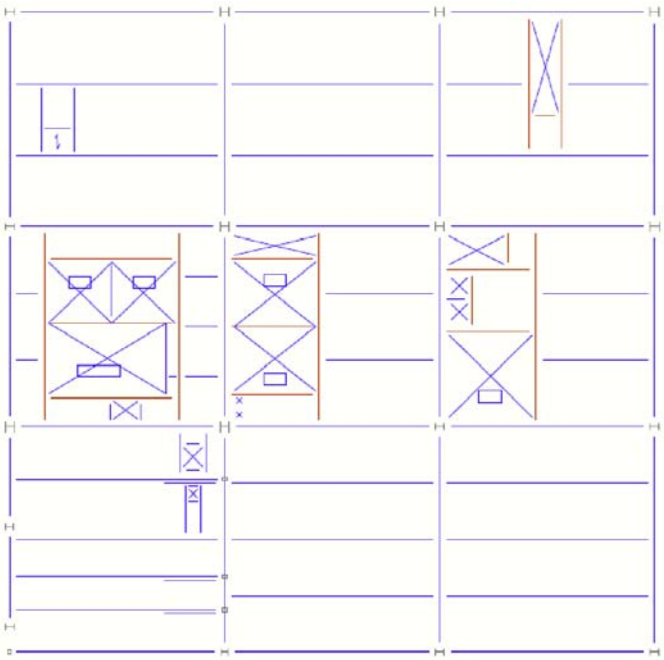
Figures 4.26 & 4.27: Partial 3rd floor as designed column layout with girders; proposed column layout with girder direction and spacing.

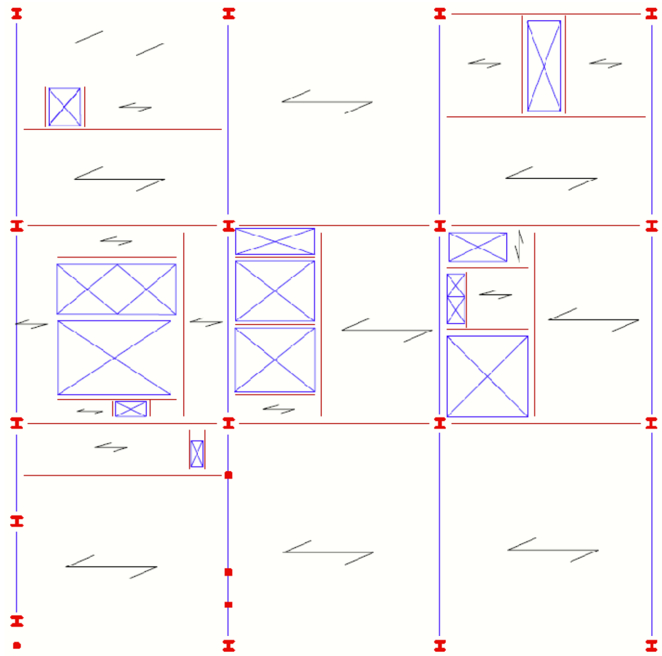
With the original design and the subsequent proposed redesign both having concentric shear connections at the columns, and the columns not participating in the lateral load resistance except to carry the transferred axial loads, the orientation of the columns is left up to how the connections will be made as well as any architectural considerations.

Since the bottom flange of the proposed modified girders would not fit between the flanges of the columns a better and more constructible connection location would be to the column flanges. With the columns rotated 90° from their present axes this would make the connection easier, more constructible and shorten the span approximately one foot. This would eliminate the need for an extended shear tab back into the web of the column on heavily loaded main girder spans.

Columns in the inside curved radius section of the building are already in this orientation and the columns along the exterior curved radius do not have to be adjusted for the girders. The existing girders along the outside perimeter of the structure would remain as normal W-shaped sections supporting the hollow-core slab from underneath. Some of these girders may have to be increased to carry the additional applied load of the slabs. Keeping these members the same would eliminate the need for a structural exterior facade redesign.

The remaining issue of multiple/large openings in the floor slabs would have to be handled in a manner similar to that of the original design. The original design accomplished this by the use of additional beams in and around the openings to support the slab edges. These additional beams/girders would have to be of the modified designs since the only place to attach them would be to the webs of the support girders to maintain non-moment (shear) type connections; and the slabs would be supported by the bottom flange of these additional beams. The direction of the hollow-core slabs would have to be rotated 90° in some of these areas to minimize the use of additional beams; however rotating the slab directions would compromise the composite action of the girder sections since the cores from one slab would not be able to be grouted integrally with the cores on the other side of the modified girder which run perpendicular to them.





Figures 4.28 & 4.29: Original opening support beams and; proposed slab layout and support girders.

As shown in Figures 4.28 & 4.29 above, additional beams can be added to the system. The direction of the hollow-core slabs will remain as consistent as possible to maintain rigidity and stability from slab to slab. Where the girders/beams are alongside an opening the member would not be fully laterally braced along its compression flange on both sides and the calculated ΦM_{pc} value of the member may not be obtainable. Therefore it is suggested that in these instances 3/8"Ø x 1-1/2" long shear studs be attached along the top flange of the girder at 2' O.C. spacing to "fully laterally brace" the top compression flange after the cells have been grouted and the 2" topping has been placed. It is also suggested that steel detailing around the inside of the open areas will need to be completed to let the grout flow through the castellation and be able to provide some composite action with the girder.

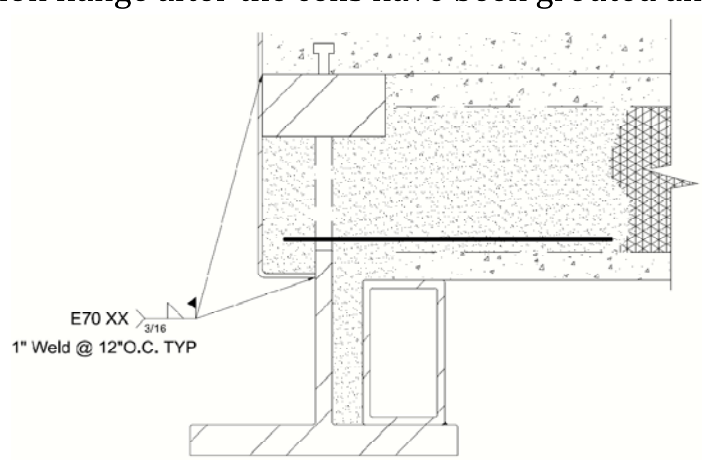


Figure 4.30: Opening Detail

Connections:

Proposed system MAE considerations:

Another aspect of the proposed systems viability as an alternative to the original design is if the connections at bracing and typical maximum load bearing areas can be designed as simple shear connections. To design these connections AISC 13 is used for design specifications. Since all of the connections include modified members and smaller depth areas with higher loads the design manual tables and aids will not be applicable and all connections will have to be designed and checked with all relevant limit states in the steel manual specifications section J and Parts 9 & 10.

Load Determinations:

To determine the design loads for the three typical connections a full factored dead load plus a factored live load of 125psf was used on all braced framed sections and modeled in SAP 2000. The calculated factored lateral loads were additionally added to the 2D frame and all frames were analyzed with just gravity and a lateral – gravity combination. The loads on all these members' intersections were then used to determine the areas where the connections would have to resist the most shear and tensile force limit states since the connections were designed as simple shear connections and contained no moments. After the locations and magnitudes of the forces were determined they were increased by 30% to make sure that the shear connections would be able to be designed with even larger loads in a reduced depth situation and to compensate for possible differences between calculated loads and as designed loads.

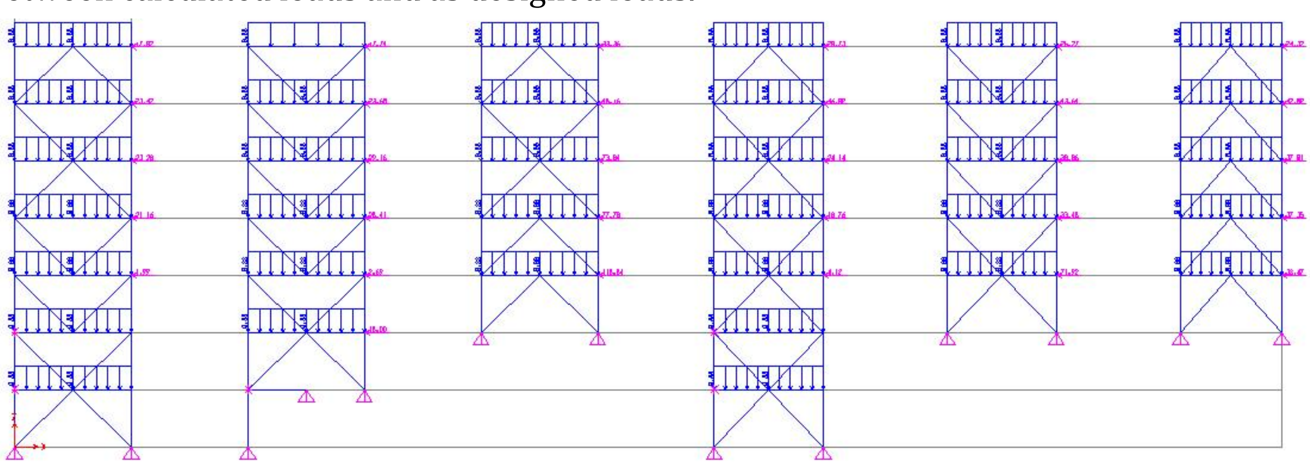


Figure 4.31: Full gravity and calculated lateral loads for connection designs

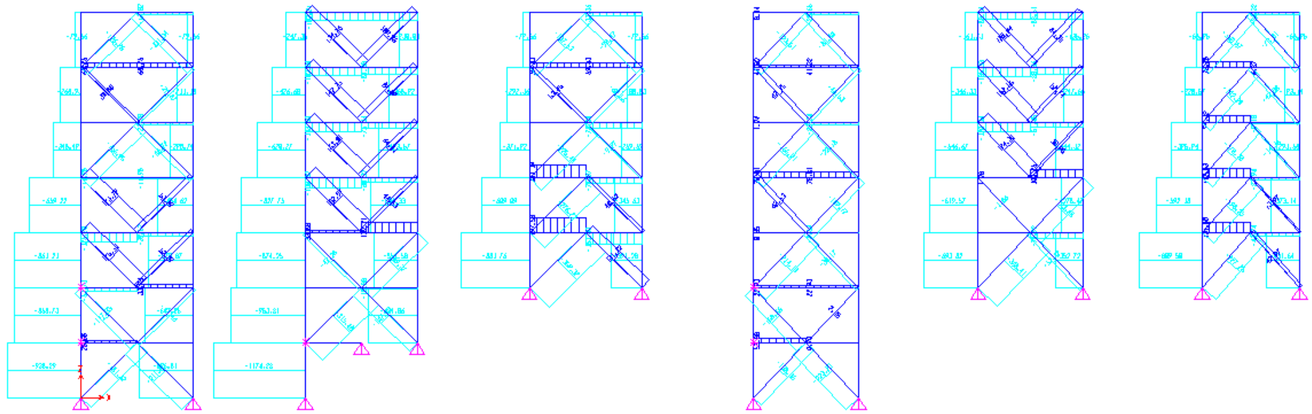


Figure 4.32: Axial loads generated by Figure 4.31

Compressive loads Tensile loads

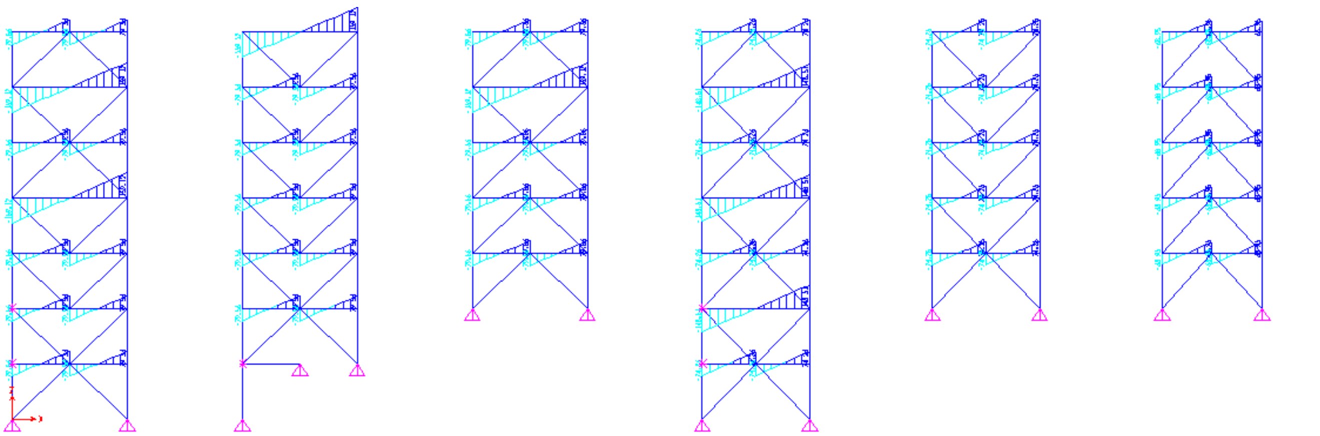


Figure 4.33: Shears generated by Figure 4.31

Positive Negative

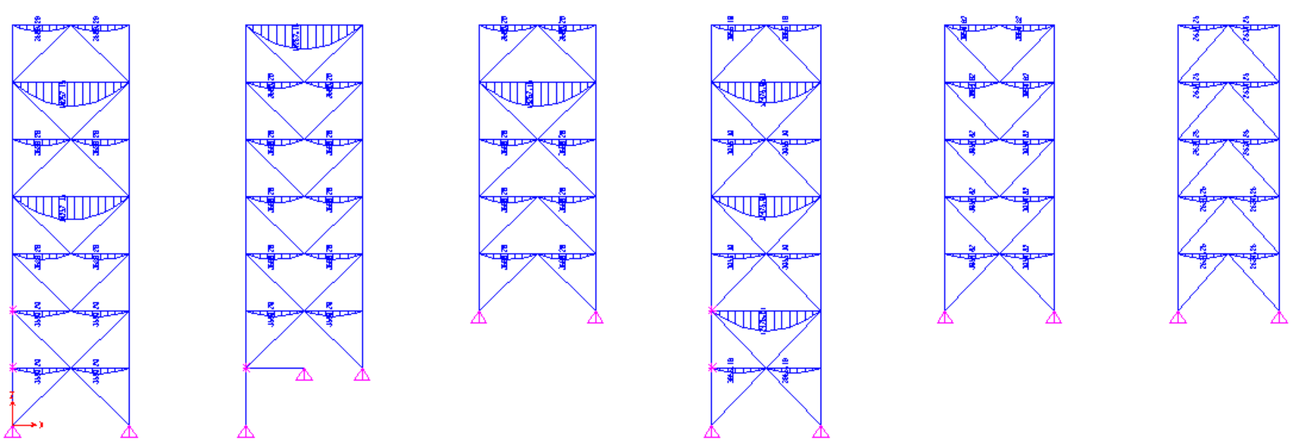
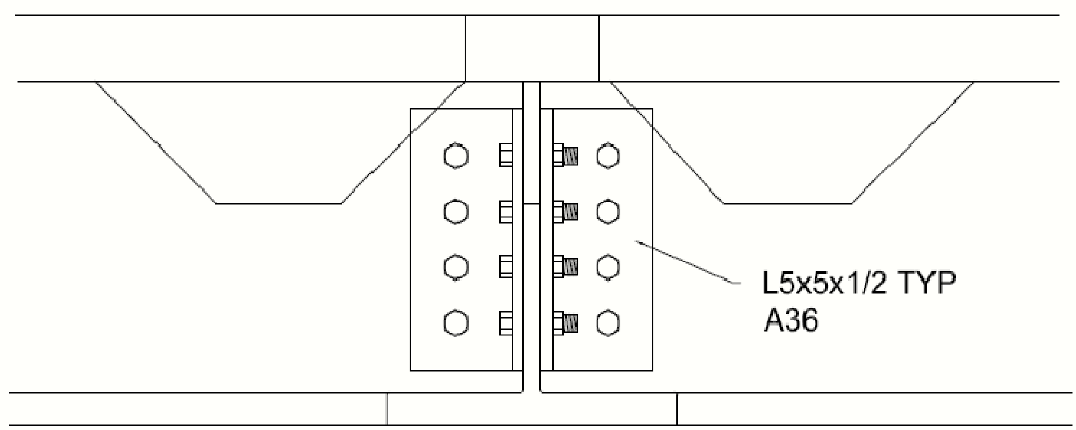


Figure 4.34: Moments induced by loading from Figure 4.31 (ALL ARE POSITIVE)

Designs:

Three connection types were designed to simulate the most frequently used connections with the most loading.

1) Modified Girder to Modified Girder (where openings occur)
2) Column to Girder to HSS Brace combination
3) Girder to Column web using an extended shear tab
4) Girder to Column flange (same as 1 above)



Modified W24x192 Connected to Modified W24x192
189 kip Capacity

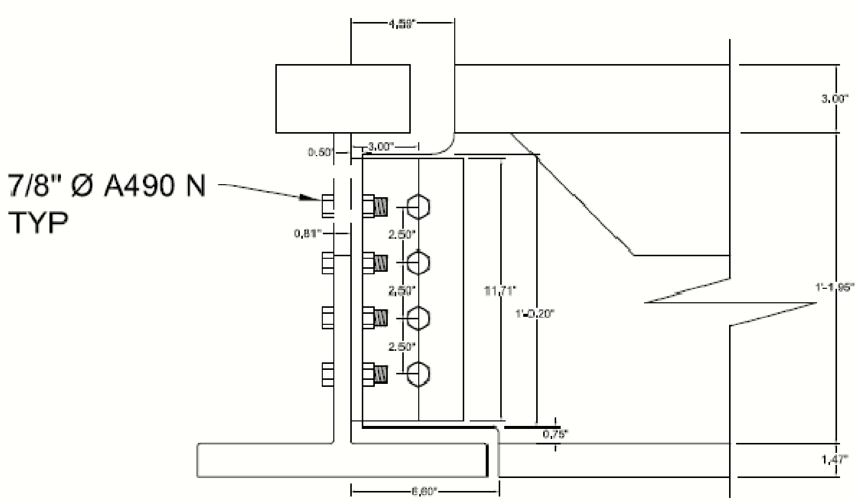


Figure 4.35: Connection 1 also used for column flange to girder web

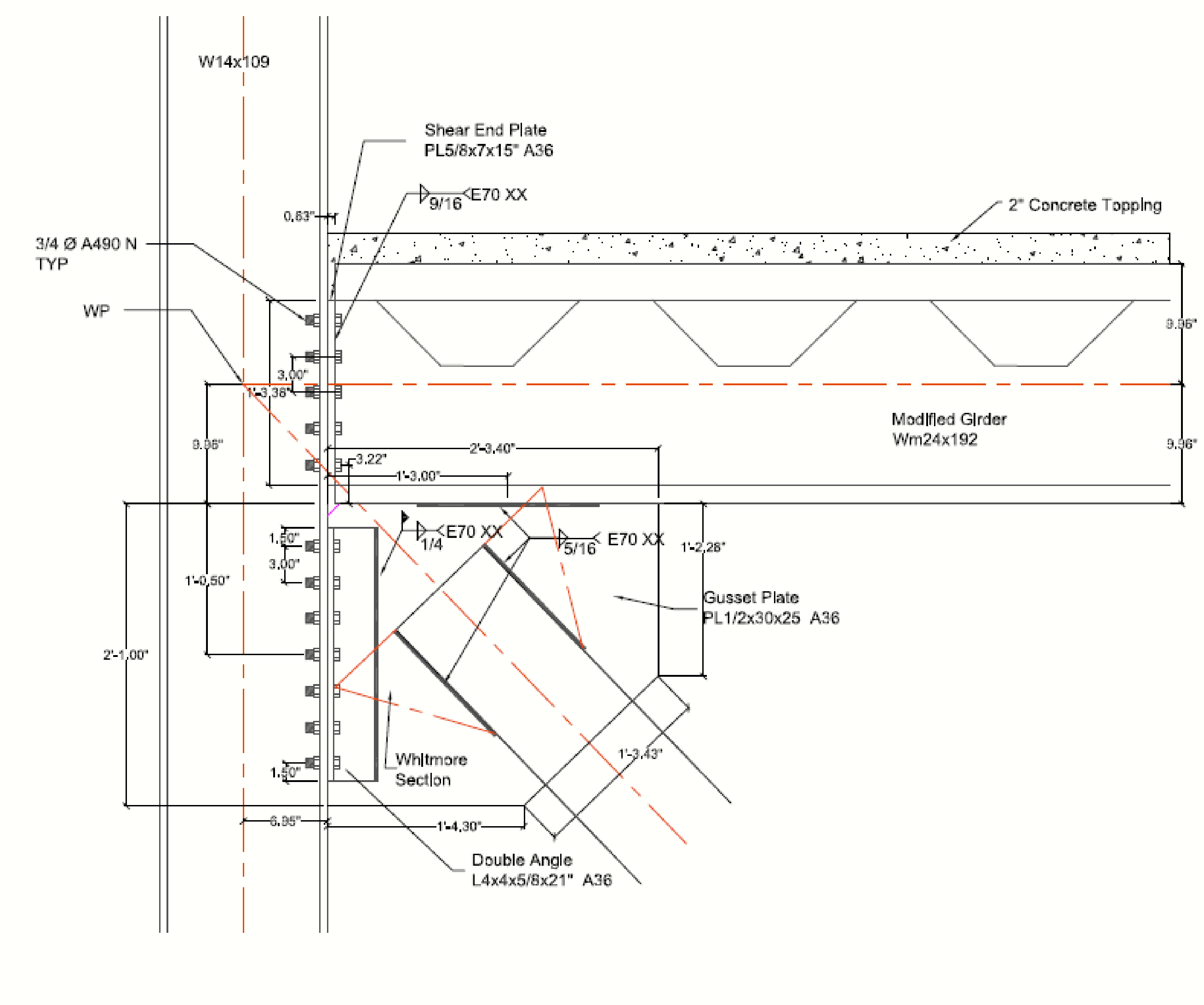


Figure 4.36: Connection 2

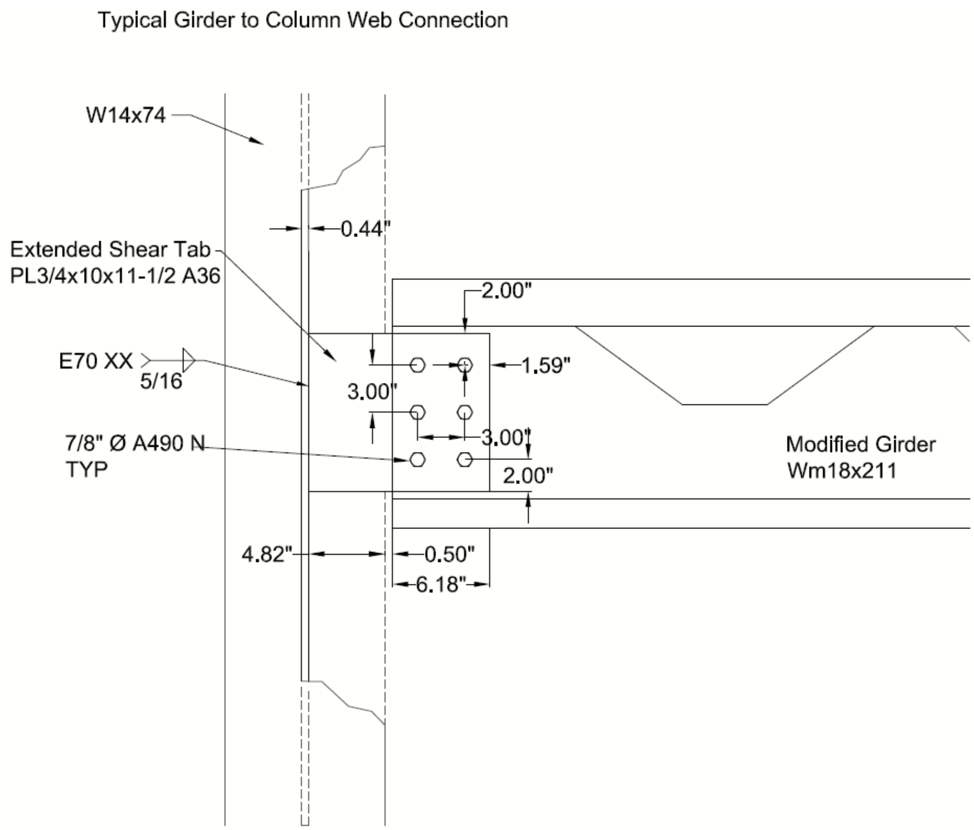


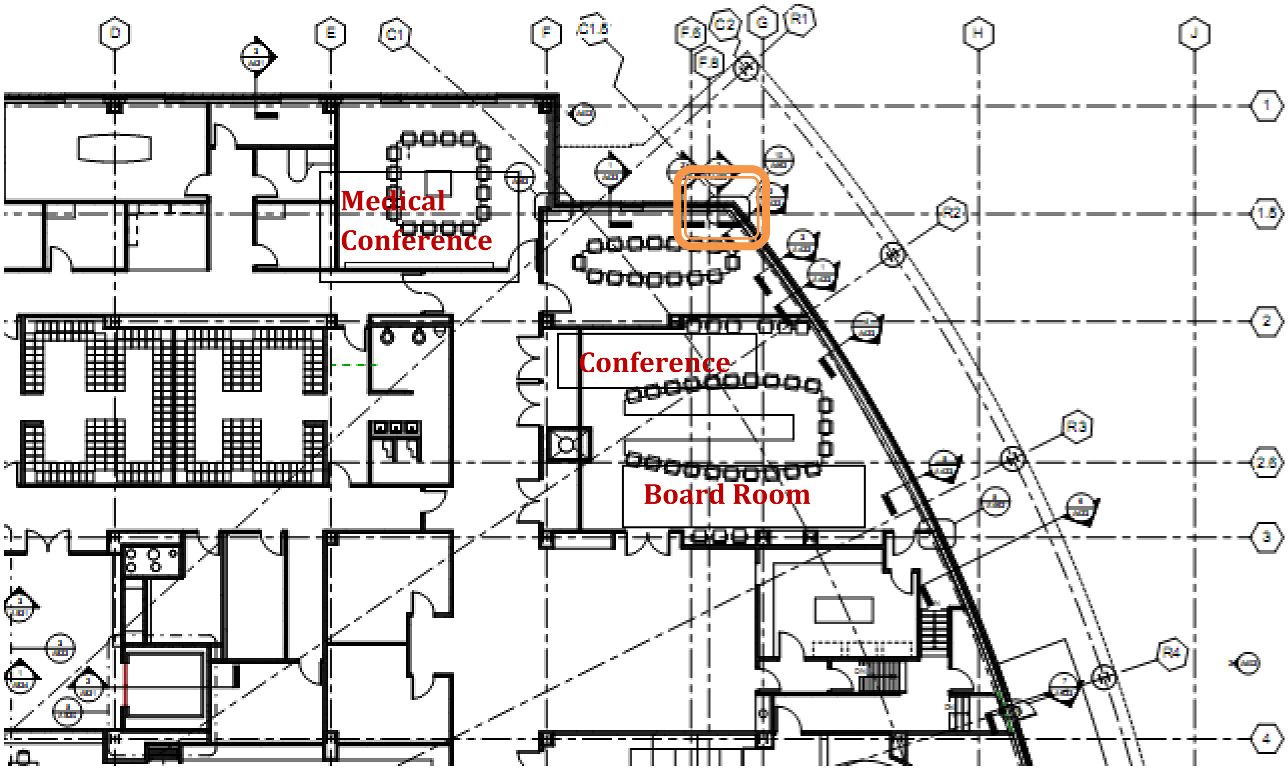
Figure 4.37: Connection 3
See Appendix J for the design calculations.

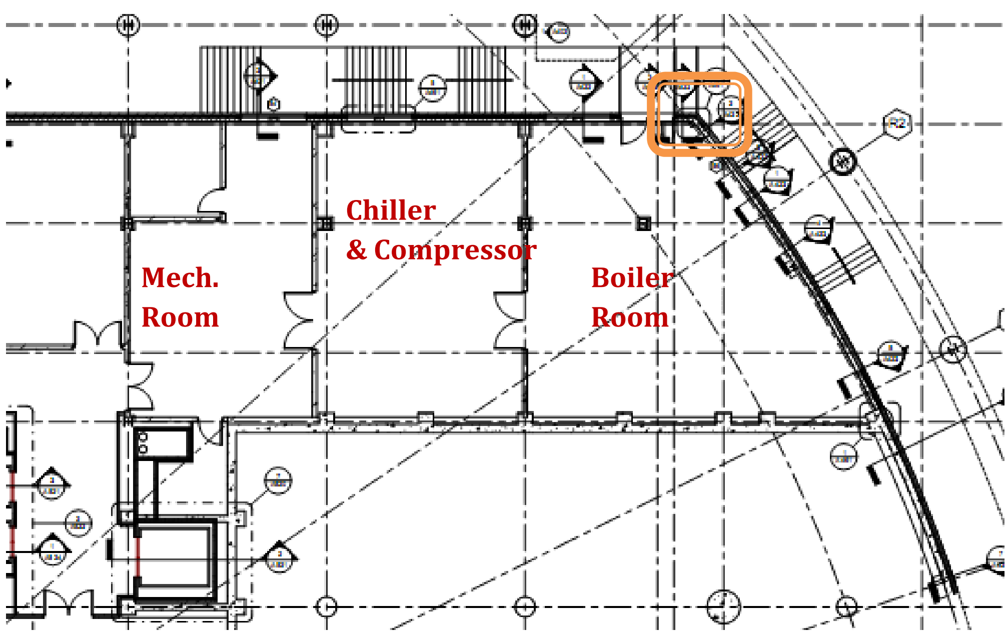
Connection Conclusions:

Reduced depth dimensions and increased loading requirements made the designs more challenging since most values could not be pulled from AISC Tables and simple shear design considerations had to be adhered to such as rotational ductility requirements found in AISC 13; however, based on the three types of connections that were designed to transfer the forces between members the results determined that the calculated factored loads plus an additional 30% can be accomplished in connecting the lateral and gravity systems to the vertical elements ; however, it was determined that in order to achieve some of these connections a larger than necessary modified girder (W_m 24x192) was needed for its depth. This practically makes the design of the structure using only one size girder.

Breadth Options:

Acoustical Considerations:





Figures 4.38 & 4.39: Spaces with acoustical conflicts Match Point

Part of level 1 is slab on grade and within this section are the chiller room and the boiler room. The walls enclosing the chiller room are 8" CMU's on three sides and a 16" thick reinforced concrete wall on the other. The ceiling separating the two spaces consists of 3-1/2" thick concrete on a 3" deep corrugated metal deck with carpeting on the conference room floor. There are two centrifugal chillers located directly below the medical staff conference room which also cantilevers out over the sidewalk.

Design values for the acoustical analysis were taken from various tables and charts in the ASHRAE Handbooks. ASHRAE 2003 Applications Handbook Figure 12 gives typical values for maximum and minimum sound levels for a centrifugal chiller. The maximum values are used and adjusted using Figure 14 to get the built-up estimated sound level. To be able to use this figure an estimated "boxed" size of the chiller is compared to the overall room size to get a ratio for the horizontal axis, and an average sound absorption coefficient is determined based on the room surfaces. For surfaces of concrete and CMU's a coefficient of 0.10 is used. The calculated sound level from both chillers operating at the same time at maximum levels is approximately 105 dBA including sound build-up from the almost all concrete room surfaces and dBA weighting effects.

Calculations, charts and figures for sound levels can be found in Appendix K at the end of this report.

From Table 34 in ASHRAE 2003 Applications Handbook 47.29 a design guideline for HVAC-related background sound level in the medical staff conference room can be estimated at 25-30dBA; therefore the transmission loss from the chiller room to the conference room through the ceiling/floor system needs to be approximately 75 dBA. The Sound Transmission Class (STC); a single number representation of transmission loss (TL) for all octave bands, for the as designed composite deck system with carpeting above is approximately 51 dB and 57 dB for the proposed redesign floor system. If the assumption is made that the TL = STC then the background sound level from the chiller room exceeds the acceptable by 24 and 18 dBA respectively. The dBA levels associated with the two boilers is much lower than the chiller room levels and are not high enough to be a concern.

Receiver Room Sound Correction As Designed								
Hz	63	125	250	500	1000	2000	4000	8000
Max. dB	80	75	92	88	90	87	79	67
Build up	+9	+9	+9	+9	+9	+9	+9	+9
total	89	84	101	97	99	96	88	76
A weighting	-25	-15	-8	-3	+0	+1	+1	+1
A weighted adjusted	64	69	93	94	99	97	89	75
TOTAL (dBA)	64	70	93	95	100	102	102	**102**

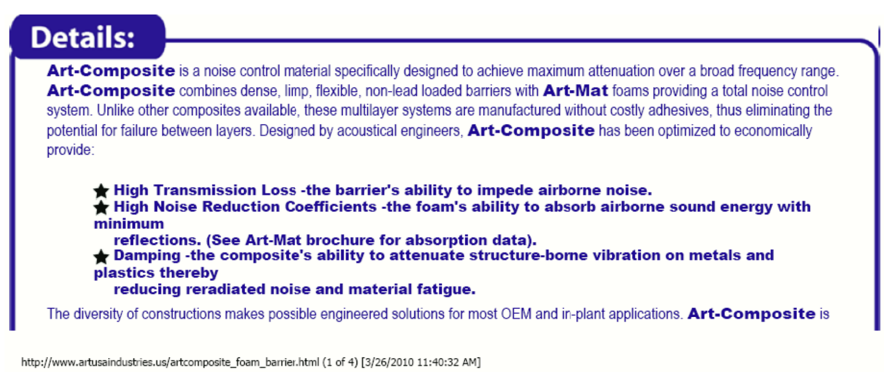
Table 4.17: dBA sound level in conference room from one chiller

To account for both chillers operating at the same time and at the same level the dBA for two chillers would be combined to give a background sound level of **105dBA**; however, the TL values for the individual octave bands for the floor construction were not obtained and subtracted from the above table. The STC values of 51(as designed) and 57 (proposed) for the systems were obtained and subtracted from the 105dBA to obtain a receiver room background noise level from the equipment. A background noise level range of 25-35 from ASHRAE Applications Handbook 1993, Chapter 43.5 Table 2 is used for comparison.

Floor Systems Effectiveness Comparison	
As Designed	Proposed
25≥105-51=54	25≥105-57=48
25<54 NOT ACCEPTABLE	25<48 NOT ACCEPTABLE

Table 4.18

The tables on the following page compare the two systems when using a sound barrier. A product from ArtUSA was used for determinations


Details:

Art-Composite is a noise control material specifically designed to achieve maximum attenuation over a broad frequency range. **Art-Composite** combines dense, limp, flexible, non-lead loaded barriers with **Art-Mat** foams providing a total noise control system. Unlike other composites available, these multilayer systems are manufactured without costly adhesives, thus eliminating the potential for failure between layers. Designed by acoustical engineers, **Art-Composite** has been optimized to economically provide:

- ★ **High Transmission Loss -the barrier's ability to impede airborne noise.**
- ★ **High Noise Reduction Coefficients -the foam's ability to absorb airborne sound energy with minimum reflections. (See Art-Mat brochure for absorption data).**
- ★ **Damping -the composite's ability to attenuate structure-borne vibration on metals and plastics thereby reducing reradiated noise and material fatigue.**

The diversity of constructions makes possible engineered solutions for most OEM and in-plant applications. **Art-Composite** is

http://www.artusaindustries.us/artcomposite_foam_barrier.html (1 of 4) [3/26/2010 11:40:32 AM]

Figure 4.40: Details of sound barrier material

Acoustical Properties:

Sound Transmission Loss, dB, (ASTM E90-75)							
Barrier Weight	Frequency (Hz) (In. / Nom.)						
lb/ft^2	125	250	500	1000	2000	4000	STC
.5	10	12	16	21	26	32	20
.75	12	16	20	25	20	34	23
1	15	17	21	27	32	36	26
1.5	14	19	25	36	33	37	30

Figure 4.41: Acoustical TL Values for sound barrier

Receiver Room Sound Correction As Designed								
Hz	63	125	250	500	1000	2000	4000	8000
Max. dB	80	75	92	88	90	87	79	67
Build up	+6	+6	+6	+6	+6	+6	+6	+6
total	86	81	98	94	96	93	85	73
(+A)	+1	+0	-1	-2	-3	-4	-5	-5
(+B)	-9	-9	-9	-9	-9	-9	-9	-9
total	78	72	88	83	84	80	71	59
Art composite TL	-	-10	-12	-16	-21	-26	-32	-
total	-	62	76	67	63	58	39	-
A weighting	-25	-15	-8	-3	+0	+1	+1	+1
A weighted adjusted	-	47	68	64	63	59	40	-
TOTAL (dBA)	-	47	68	69	70	70	70	70

Table 4.19: dBA sound level in conference room from one chiller using barrier

The dBA levels for two chillers is **73**dBA

Floor Systems Effectiveness Comparison with Sound Barrier	
As Designed	Proposed
25≥73-51=22	25≥73-57=16
25>22 ACCEPTABLE	25>16 ACCEPTABLE

Table 4.20:

Acoustical Conclusions:

Since both designs are above the acceptable limits for background sound levels produced from HVAC systems then corrective measures should be taken to reduce these levels. Only direct sound transmission through the floor system was evaluated therefore sound isolation techniques for vibrational transmission should also be considered for final design measures.

According to ASHRAE 1995 Application Handbook 43.9 there is actually little data available to accurately estimate the sound levels associated with chillers, and it is recommended that these levels should be measured in the rooms in which they are installed. To accurately assess the sound levels and the amount of sound absorptive material to apply to the bottom of the decking; as well as other possible measures, it is recommended that sound level measurements be taken at the peak time of year when the chillers are operating.

The primary sound level reduction technique would be to apply a sound barrier material to the underside of composite metal deck. This will change the absorption coefficient within the room and the sound build up level which will initially reduce the overall sound level. It will also change the STC value for the overall constructed system by changing the density of the materials the sound waves are traveling through and will reduce/dissipate the sound energy more effectively.

The particular sound barrier material used for these calculations and the description, application and specifications can be obtained @ http://www.artusaindustries.us/artcomposite_foam_barrier.html

Other recommended sound isolation techniques related to vibratory transmission would include:

- Spring/duct isolation hangers for any ducts or pipes coming to or going from the equipment for at least 150x pipe diameter
- Thick ribbed neoprene pad at connection to housekeeping pad
- Flexible duct/pipe connectors located close to equipment
- Pack any pipe slab penetrations with fibrous material & seal with non-hardening caulking

Better acoustical performance will be realized from the proposed redesign based on the relative masses of the two systems; however both systems will need extra acoustical measures to be able to meet the medical staff conference room background sound level needs from the chillers.

Architectural Redesign of Partial Ground & First Levels:

Relocating the chiller room was investigated from an architectural viewpoint as an alternative to using acoustical treatments in the chiller room. To do this the chiller room was dropped straight down to the ground level and a storage area on the first level was moved to the location of the original chiller location. An additional area of 44'x44' (1936 sq. ft) needs to be excavated for the new space but an area of 32'x22' (704 sq. ft) for the storage area does not have to be excavated.

Acoustical Treatment VS. Architectural Redesign			
Acoustical Considerations	**Estimated Cost ($)**	**Redesign Considerations**	**Estimated Cost ($)**
Sound Barrier	7,500.00	Excavation of 8400ft^3	1440.00
Adhesive	450.00	Additional 60' of Foundation Walls (Ground)	25,645.00
Labor	15,840.00	Additional 44' of 8" Reinforced CMU Wall	4818.00
		Additional Slab On Grade	9800.00
		Less 5 Columns @15'	-6263.00
		Additional 2 sets of double doors	6000.00
		Additional 30' of interior wall for storage area	1200.00
		Less 54' of Foundation Wall (1st)	-23,528.00
		Mechanical Considerations (pipes, ducts, sprinkler)	3500.00
TOTAL	**23,790.00**	TOTAL	**22,612.00**

Table 4.21: Alternatives cost comparison

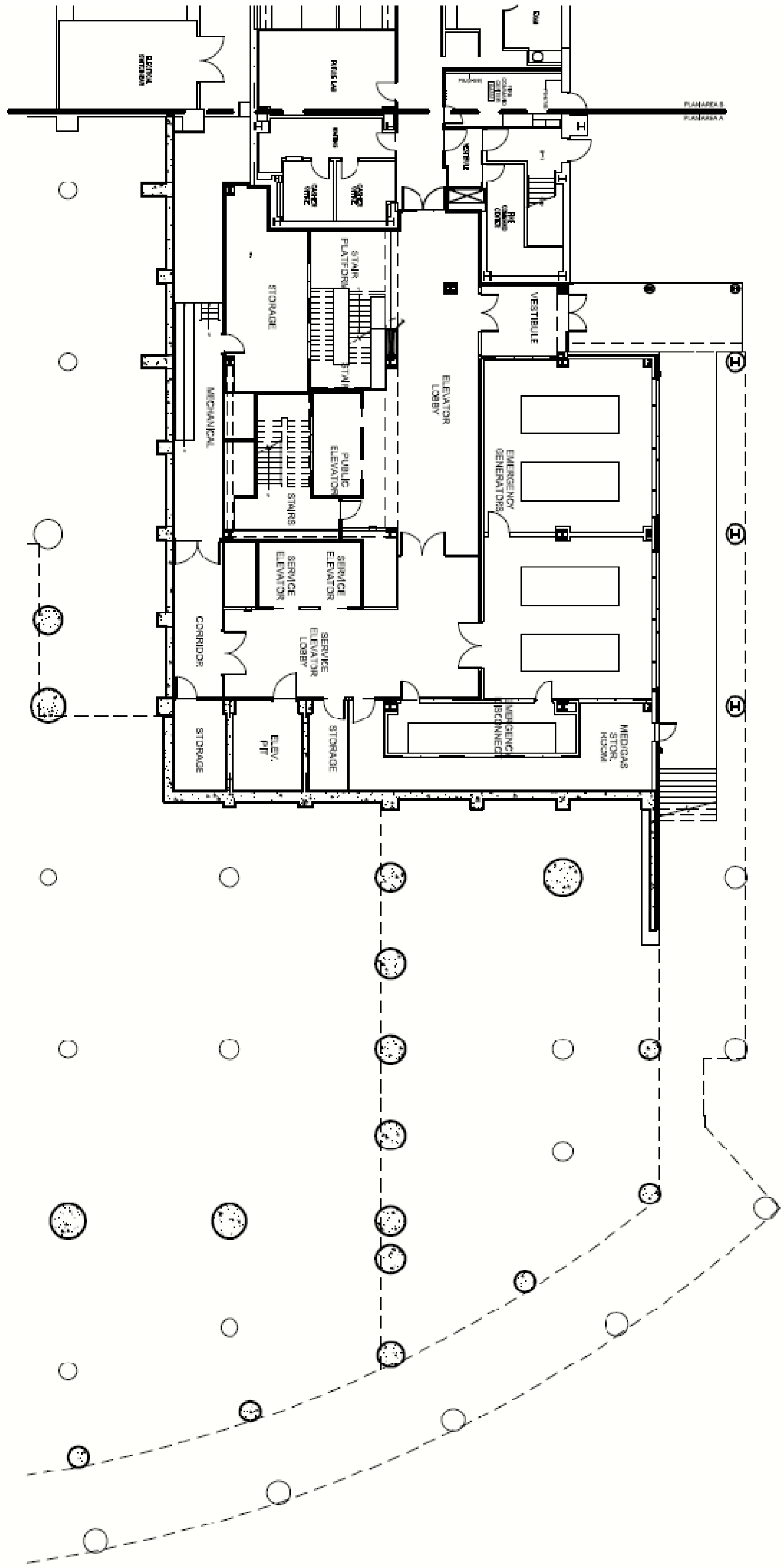


Figure 4.42:
Ground level
as designed

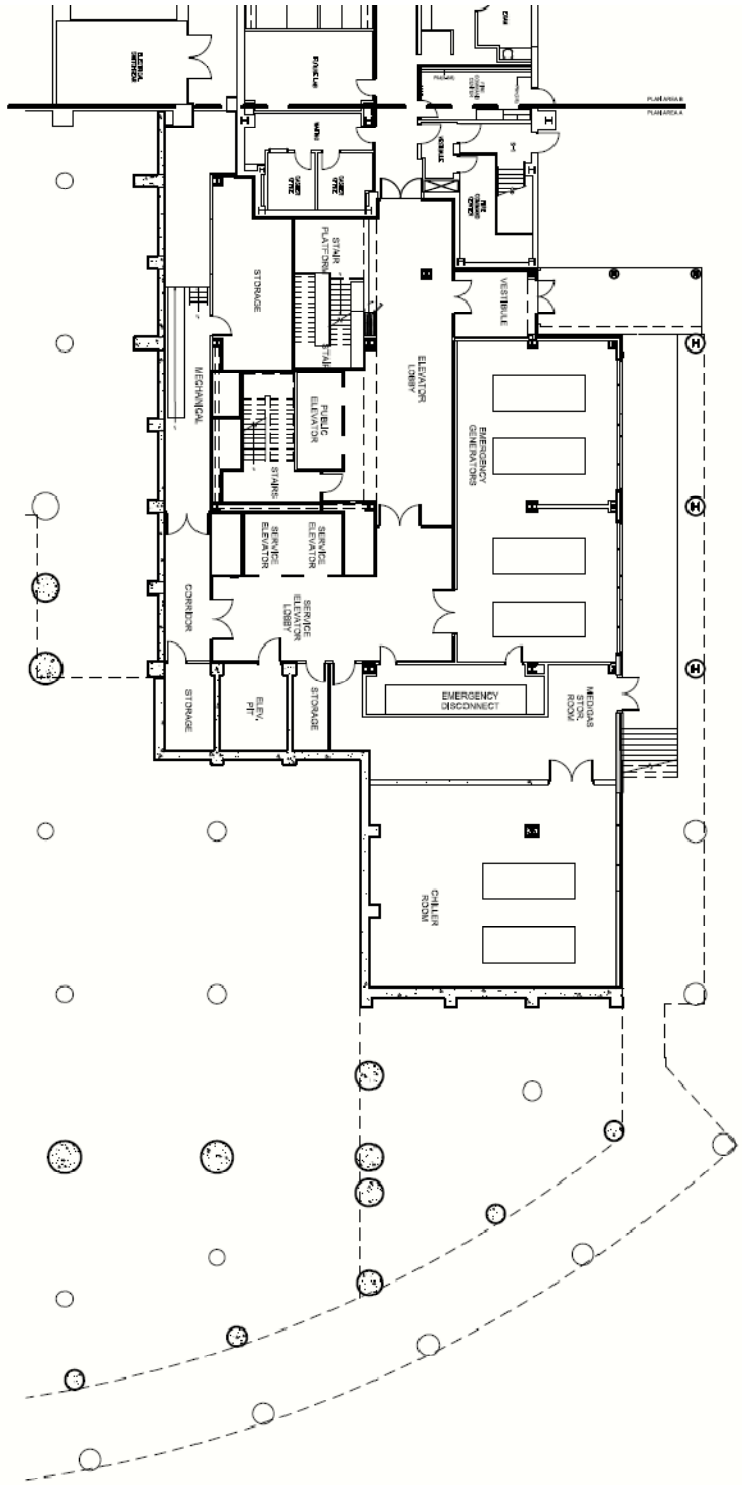


Figure 4.43: Ground level redesigned

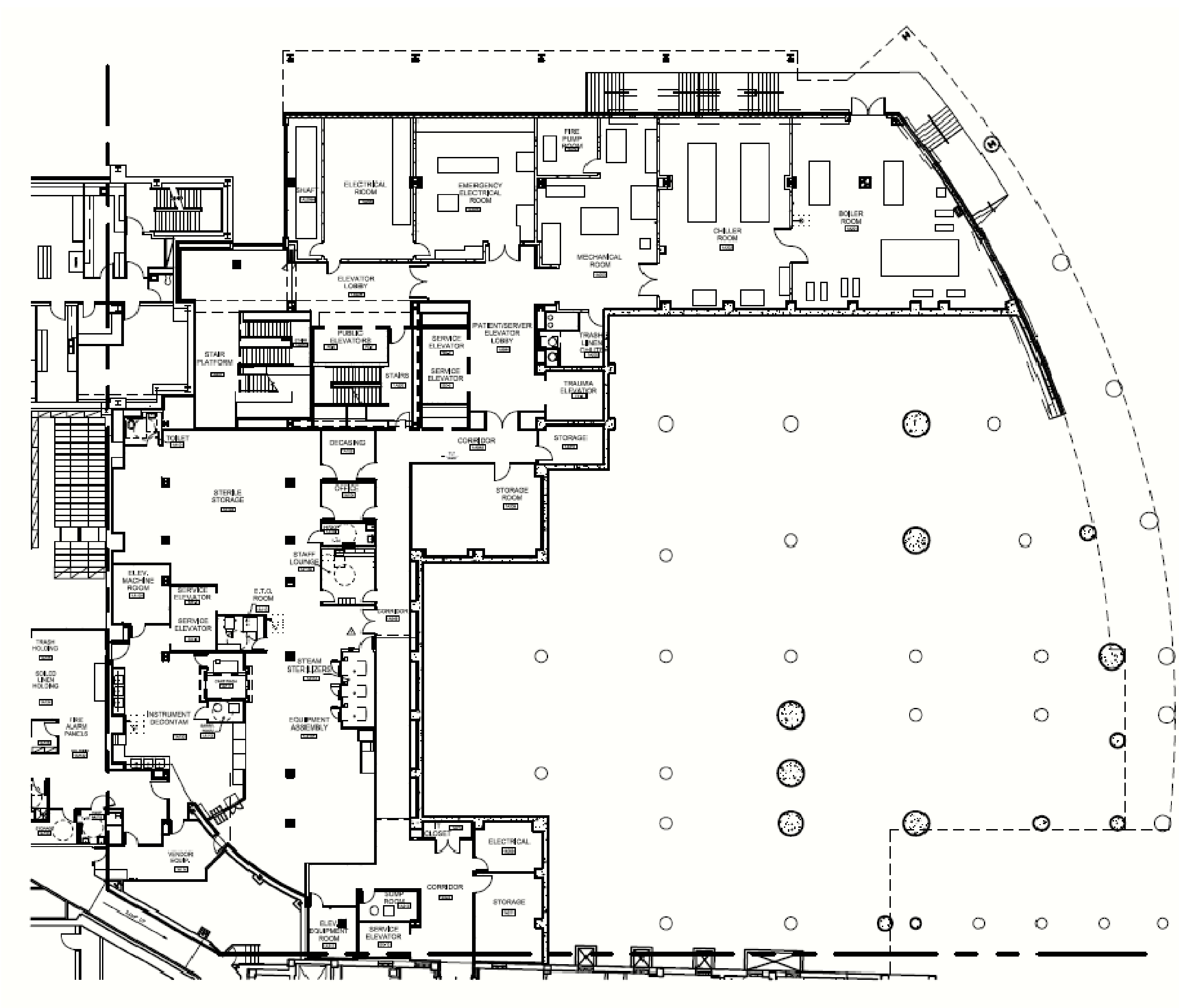


Figure 4.44: Level one as designed

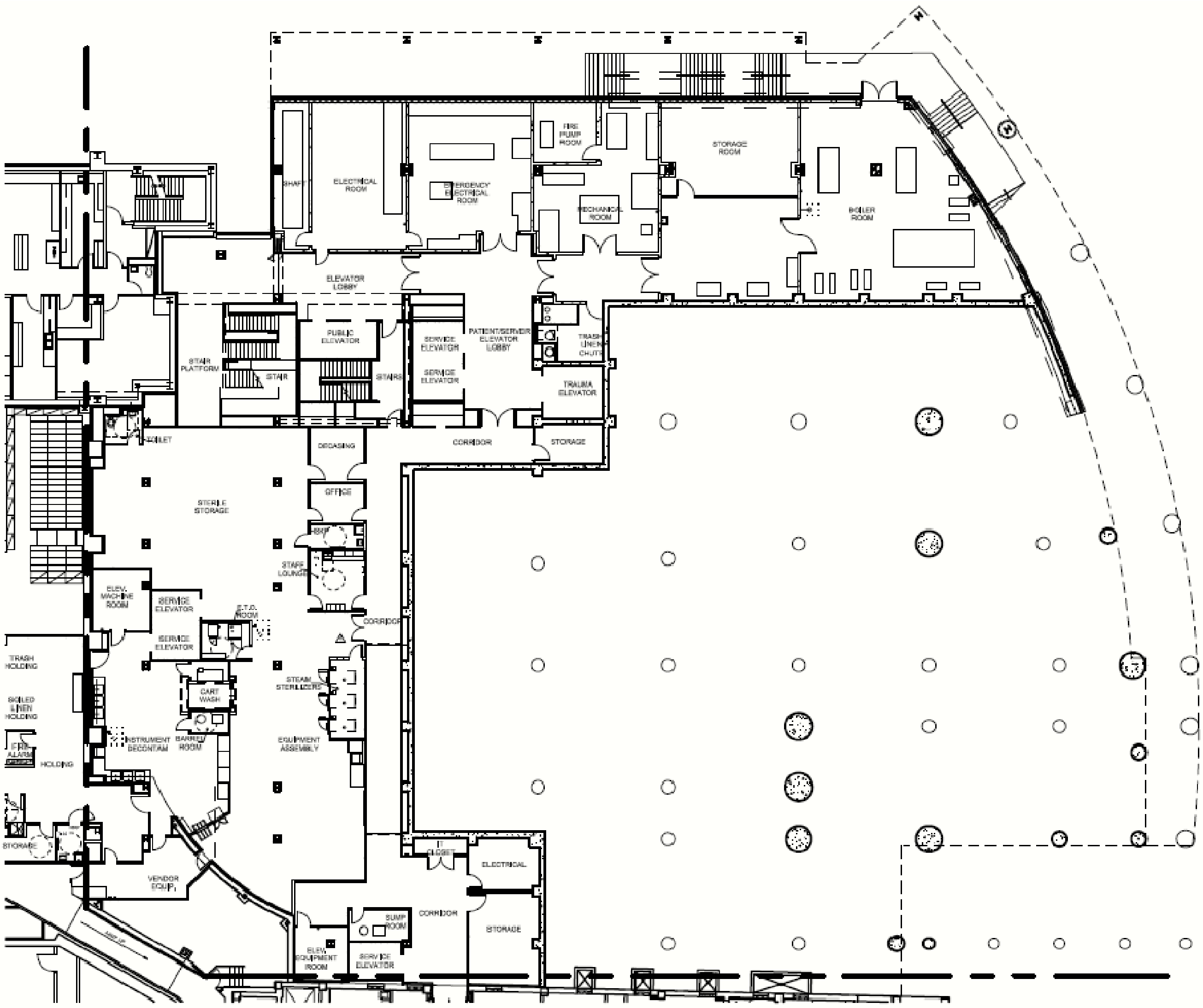


Figure 4.45: Level one redesigned

Proposed System Vibrations Due to Walking:

The proposed floor system was evaluated using PCI Chapter 9.7 (Vibration in Concrete Structures) & an ADAPT Technical Note (TN290_vibrationbs_floor_032109) which refers to ATC, 1999

To determine if the proposed system will be acceptable floor system for hospital operating rooms, the natural frequency as determined in PCI was used to compare against Figure 4 as found in ATC 1999. Figure 4 compares the frequency of the floor system with the peak acceleration as a function the natural frequency in %g.

Equation used: $a/g=P_o\, e^{(-0.35*f_n)}/\beta W$

Where Po=assumed weight of an individual walker*0.53

0.53=dynamic load factor for first harmonic of walking force with an assumed walking frequency of 2 Hz. From Figure 1 in ADAPT TN290
=150*0.53=79.5

B=0.05; damping factor, From Table 1 in ADAPT TN290

W=weight of the floor section; actual attached DL
= 107.5k

f_n=natural frequency of floor
=5.20Hz

$a =[P_o\, e^{(-0.35*f_n)}/\beta W]g \leq 0.25\%g$

$a =[79.5\, e^{(-0.35*5.2)}/0.05*107.5*1000]g = 0.002396g$

0.002396g=0.2396%g<0.25%g Acceptable

Additional calculations can be found in Appendix L

Construction Cost Comparison:

For a complete and accurate cost and scheduling analysis a take-off of each individual level would have to be done for the structural system and the two systems compared by using both total costs and scheduling implications. However for the scope of this report; a typical area will be analyzed based on the following criteria.

- Total cost of steel for both systems
- Fabrication costs for both systems
- Licensing fee for proposed system
- Steel detailing
- Increasing column size for proposed system based on additional weight
- Number of girders in both systems and their total weight
- Number of beams in both systems and their total weight
- Shear studs & decking vs. just shear studs for proposed
- Concrete pours vs. hollow-core slabs
- Opening, installing rebar & grouting hollow-core planks
- 2" concrete topping for the proposed system
- Fireproofing both systems

N

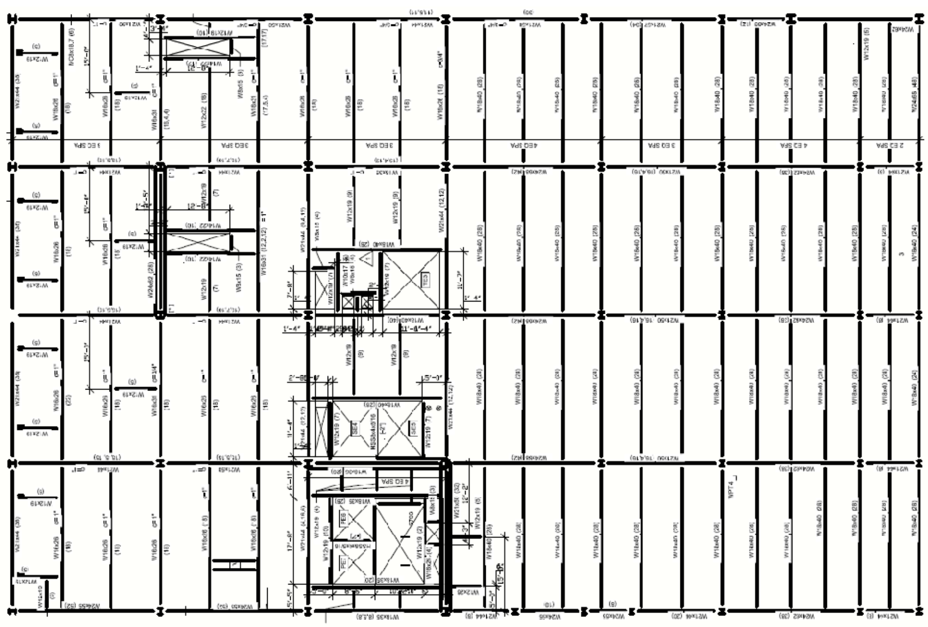


Figure 4.46: Section of Level #3 used for CM analysis

Cost Comparison of Structural Systems			
As Designed	**Estimated Cost ($)**	**Proposed**	**Estimated Cost ($)**
		Licensing Fee	206,000
Columns (42 @ 69kips) Labor to install	3500	Columns (42 @ 82.8kips) Labor to install	3500
Fabrication	411,337	Fabrication	1,319,640
Girders (37 @ 45.7kips) Labor to install	3500	Girders (37 @ 203.3kips) Labor to install	3500
Beams (121 @ 102.7kips) Labor to install	10,500	Beams (40 @ 116.2kips) Labor to install	3500
Connections (336)	252,000	Connections (142)	106,500
Shear studs & decking (2177) & (22,080ft²)	135,667	Shear studs (175)	347
Concrete forming & placement 3pours @ 7360ft² ea.	155,142	Hollow-core slab & install	234,048
		Opening & grouting HCS	44,160
		2" Concrete topping	34,707
Fireproofing (200 full members)(Total feet=4649)	19,850	Fireproofing (119 members) (Total feet=2150)	5850
TOTAL	**991,400**	**TOTAL**	**1,755,800**
		DIFFERENCE	**764,400**

Table 4.22: Construction Management Cost Comparison Based on section of Level 3
Total square feet = 22,080

Conclusions:

Based on the cost analysis in Table 4.22 for approximately 1/2 of level 3 it can be seen that even though there are the same number of girders and 2/3 less beams in the proposed system the overall weight of the girders, beams, and columns is almost double the weight of the original design, which directly translates into much higher building costs since steel is purchased mainly by the ton. Assuming the figures from this area are indicative of the entire structure then the assumption could be made that the proposed structural system will be approximately 75% more expensive based only on the above criteria.

Final System Summary & Conclusions:

It was stated at the beginning of the redesigned gravity system that a closer inspection of the advantages and disadvantages would have to be done to evaluate the systems viability. By analyzing the system it was determined that all of the disadvantages listed are correct. It has been shown that large lead times are required with this type of system to be able to coordinate the size of the girders, span and direction of HCP's, and floor penetrations. These elements alone contribute to the systems inflexibility during construction should changes in design or use of space become necessary. This would also make any project of this size and magnitude a design-bid-build type of project, prolonging the completion and delaying the use of the structure. This is not the preferred method of completing a structure in today's building environment where time and opening delays could have cost effects into the millions of dollars.

On the advantages side 5 out of the 11 advantages listed are actually not exactly true for the bay sizes, loading and use of the structure. Starting with the system will reduce the overall weight of the structure; it was proven the overall weight will increase by approximately 25%. Secondly, no intermediate beams in the interior of bays would be needed. Additional beams are needed to frame around larger openings in the floor system. Next it was stated that the system can be installed in any type of weather and trades can begin work underneath almost immediately. While the system may be able to be installed in any type of weather; the grouting of the cores cannot be done in lower temperatures and adverse conditions without additional and possibly costly measures being taken. Without the grouting and setting requirements of the cores being completed; construction materials and equipment cannot be stockpiled or stored on the system because of possible instability issues. This would negate the last two advantages and slow down construction time and scheduling. The first advantage listed as easy and fast to install would not apply when there are multiple and large openings because this would slow down the beam and slab setting process versus larger straighter sections where more square footage can be covered quicker.

Structural construction cost estimates for a typical section of the structure also shows that the costs of this type of system on this building type would increase somewhere in the range of 50-75%. This would be too large of an increase to justify unless the system would provide additional benefits which other cost effective systems would not be able to provide.

Some of the benefits the system is able to provide over the as designed is better acoustical and vibrational considerations; however these same benefits can be achieved with concrete systems which would be less costly also.

The design size and composite strength capacity of the Girder-slab D-beam shapes are determined by testing methods rather than by analytical engineering calculations. It is estimated that the strength of the D-beams is actually 2-3 times larger than the estimated allowable strength of the shape. With this in mind; the modified proposed shapes may be able to be much smaller than the designed proposed shapes; however, even in the areas where shapes are connected and could have a smaller section, the increased loading required the depth of the section to be deeper to be able to meet the requirements for a shear connection and maintain rotational ductility.

Overall the initial sizes of the modified shapes are dictated by the construction loading (pre-composite action) and the requirements needed for the shear connections; therefore making the depth of the modified designed members non-reducible and all of the above conclusions are still valid.

Although the redesigned proposed systems disadvantages outweigh its advantages for this type of structure, some of the advantages of the system for different building uses could very possibly make it a viable solution. These would include reducing the overall building height without compromising floor space or reducing open unobstructed ceiling cavity areas.

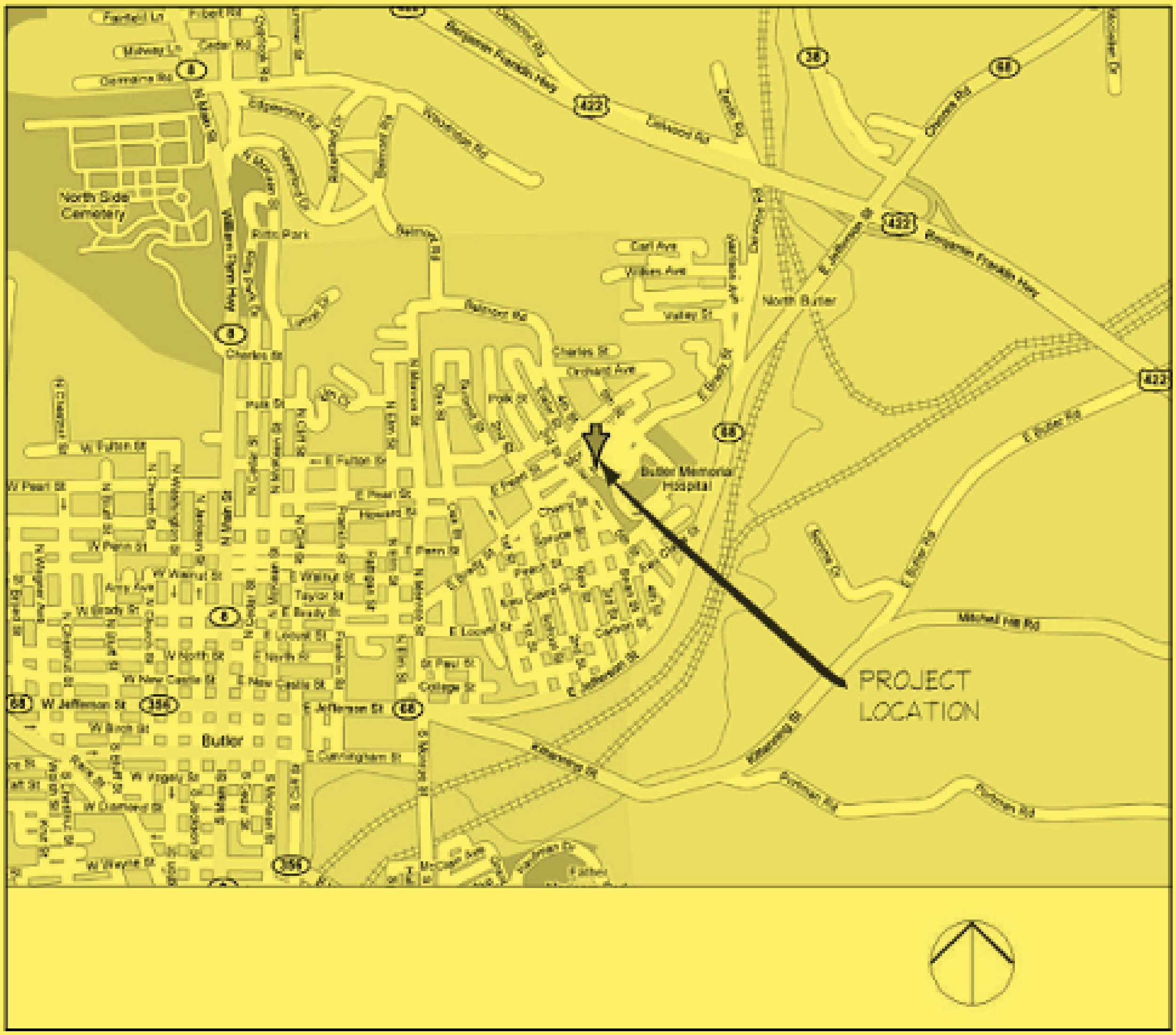
This single advantage would equate to savings in the facade, elevators, MEP runs, column fireproofing, column lengths and sizes, bracing lengths and sizes, foundations, and stair runs. In addition to the material saving provided on the structural system the reduced level heights would also reduce the overall loading on the structure which would possibly reduce member sizes even more. Taking these and possible scheduling advantages into consideration should definitely overcome the cost difference of the structural proposed system making this a good optional alternative to modern conventional practices.

I believe with further research and testing done on this type of expanded system, that some day it will be used in larger span and loading situations, but not in a hospital situation where floor-floor heights are generally large anyway to accommodate all of the additional building system infrastructure needed.

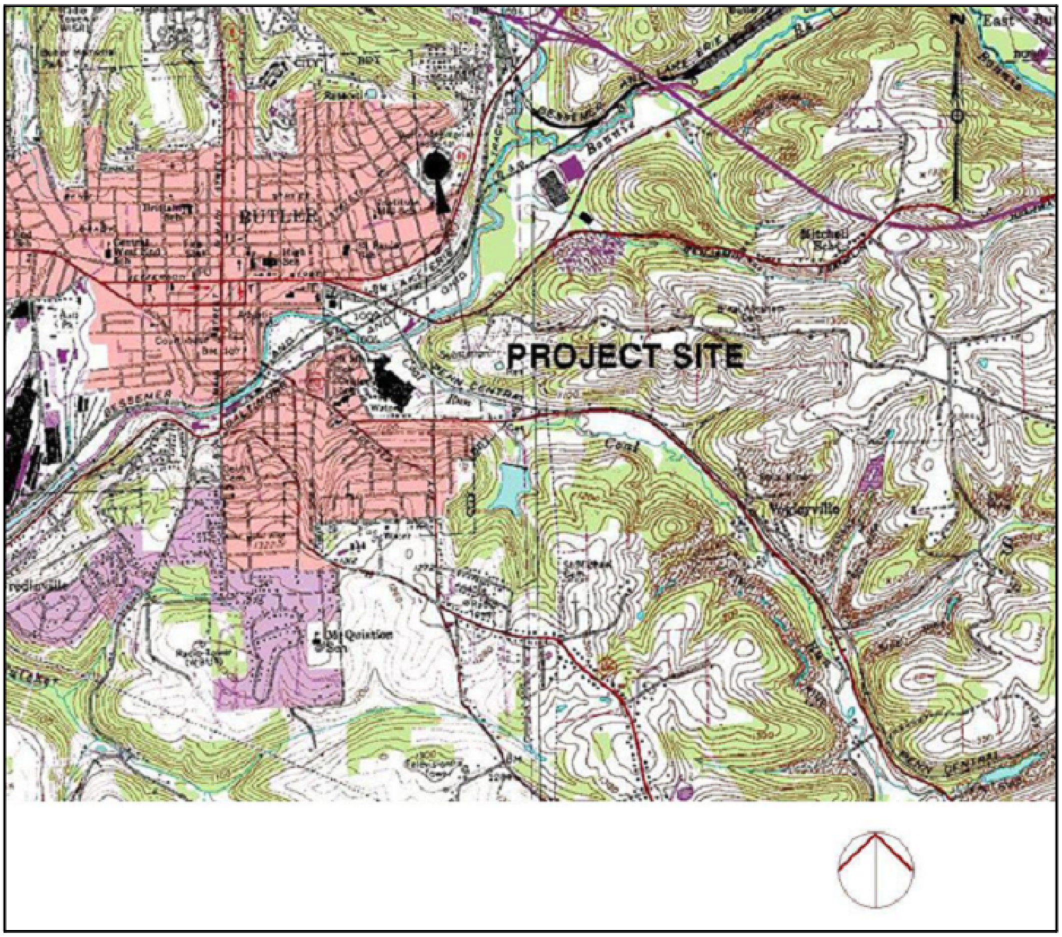
Appendix: A

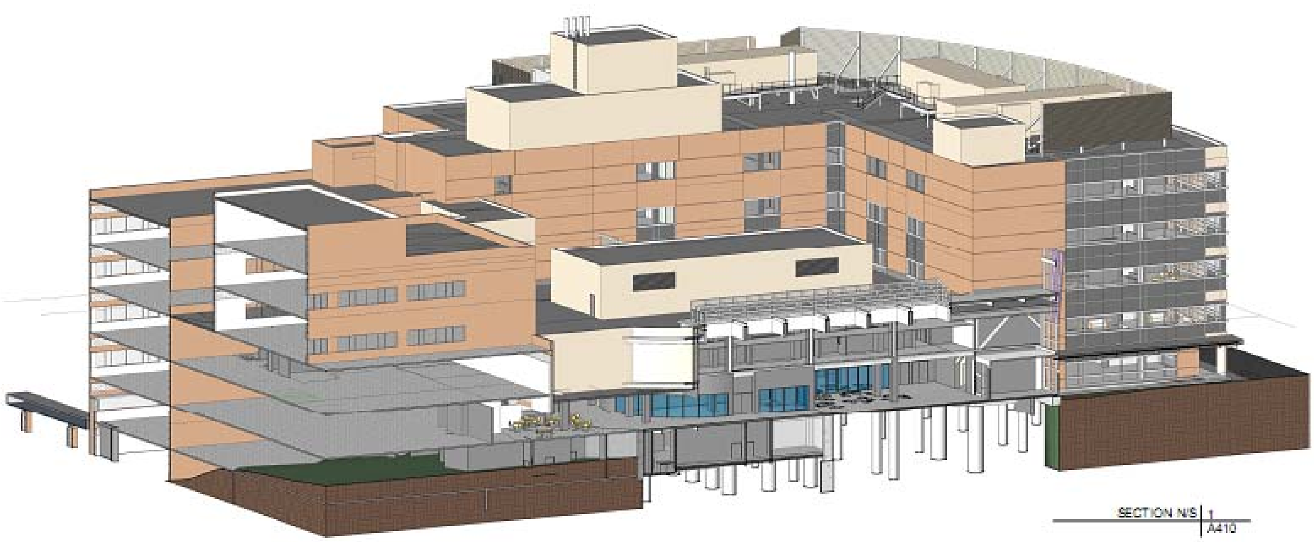
View looking from magnetic north

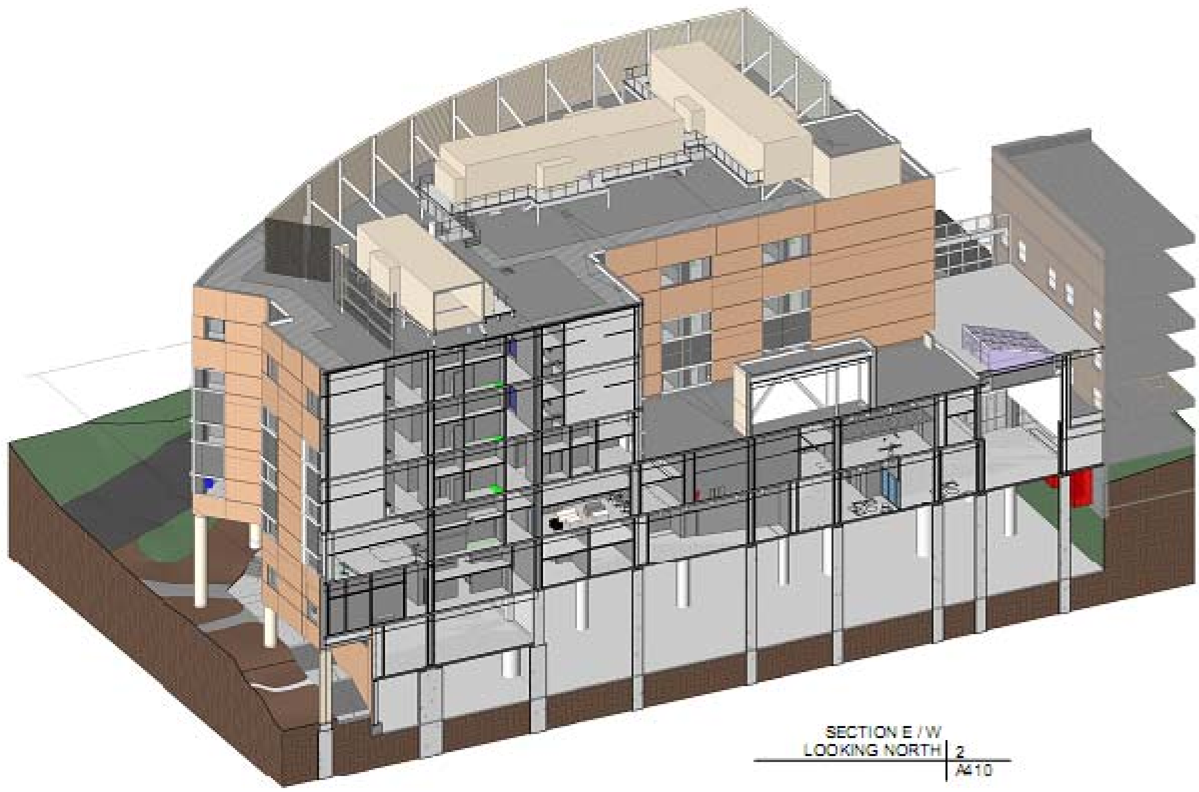
LOCATION MAP



VICINITY MAP






SECTION E / W
LOOKING NORTH
2
A410


MAIN FACADE
NORTHWEST CORNER
3
A400

Appendix B: Wind

Topographic Factor, K_{zt} – Method 2

Figure 6-4 (cont'd)

Equations:

$$K_{zt} = (1 + K_1 K_2 K_3)^2$$

K_1 determined from table below

$$K_2 = (1 - \frac{|x|}{\mu L_h})$$

$$K_3 = e^{-\gamma z / L_h}$$

Parameters for Speed-Up Over Hills and Escarpments						
Hill Shape	$K_1/(H/L_h)$ Exposure			γ	μ	
	B	C	D		Upwind of Crest	Downwind of Crest
2-dimensional ridges (or valleys with negative H in $K_1/(H/L_h)$)	1.30	1.45	1.55	3	1.5	1.5
2-dimensional escarpments	0.75	0.85	0.95	2.5	1.5	4
3-dimensional axisym. hill	0.95	1.05	1.15	4	1.5	1.5

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JIM ROTUNNO TECH III WIND CALCULATIONS

9-26-09

Gust effect Factor: Rigid Structure

$$G = 0.925\left(\frac{1+1.7 g_Q I_z Q}{1+1.7 g_v I_z}\right) \Rightarrow I_z = c\left(\frac{33}{\bar{z}}\right)^{1/6}$$

$\bar{z} = 0.6h = 0.6(122') = 73.2' > 30' = \bar{z}_{min}$

$c = 0.20$ (Table 6-2)

$I_z = 0.2\left(33/73.2\right)^{1/6}$

$0.2(0.876) = 0.1752$

$g_Q = 3.4$, $g_v = 3.5$ > 6.5.8.1

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_z}\right)^{0.63}}}$$

B = horizontal dimension normal to wind direction

E-W 198'

N-S 210'

$h = 122'$

$L_z = l\left(\frac{\bar{z}}{70}\right)^{\bar{\epsilon}}$ $\Rightarrow \bar{z} = 73.2$

$= 500\left(\frac{73.2}{70}\right)^{1/5}$ $\bar{\epsilon} = 1/5.0$ > Table 6-2

$= 744.5'$ $l = 500$

(N-S) wind $Q = \sqrt{\frac{1}{1+0.63\left(\frac{198+122}{744.5}\right)^{0.63}}} = \sqrt{\frac{1}{1.372}}$

$= 0.854$

(E-W) wind $Q = \sqrt{\frac{1}{1+0.63\left(\frac{210+122}{744.5}\right)^{0.63}}} = \sqrt{\frac{1}{1.3788}}$

$= 0.851$

(E-W)

$$G = 0.925\left(\frac{1+1.7(3.4)(0.1752)(0.851)}{1+1.7(3.4)(0.1752)}\right)$$

$= 0.856$

(N-S)

$$G = 0.925\left(\frac{1+1.7(3.4)(0.1752)(0.854)}{1+1.7(3.4)(0.1752)}\right)$$

$= 0.857$

TECH III Wind Calculations

ASCE 7-05 § 6.5.12.2.1 MWFRS 11-13-09

Velocity pressure (q_z) evaluated @ height z → height above ground

$q_z = 0.00256\, k_z\, k_{zt}\, k_d\, V^2\, I$ units (lb/ft²)

k_z varies Table 6-3

k_{zt} varies Figure 6-4

$k_d = 0.85$

$V^2 = 90^2$

$I = 1.15$

Exposure B CASE I

$K_{zt} = (1 + k_1 k_2 k_3)^2$

K_{z1}	@ 14'-8"	= 0.85	K_{zt1}	= 1.62	from documents
k_{z2}	29'-4"	0.975	K_{zt2}	= 1.552	
k_{z3}	44'-0"	1.06	k_{zt3}	= 1.484	
k_{z5}	58'-8"	1.125	k_{zt5}	= 1.415	linear interpolation
k_{z6}	73'-4"	1.183	K_{zt6}	= 1.347	
k_{z7}	88'-0"	1.234	k_{zt7}	= 1.279	
k_{z8}	102'-8"	1.267	K_{zt8}	= 1.217	
k_{z9}	121'-0"	1.315	k_{zt9}	= 1.141	
k_{z10}	135'-0"	1.348	K_{zt10}	= 1.09	from documents

$K_1 = 0.75/(H/L_h)$ $H/L_h \leq 0.5$

$K_2 = \left(1 - \frac{|x|}{\mu L_h}\right)$

$K_3 = e^{-\gamma z/L_h}$

$\gamma = 2.5$ $\mu = 4$

$q_{z1} = 0.00256(0.85)(1.62)(0.85)(90^2)(1.15) = 27.91$

$_2 = 0.00256(0.975)(1.552)(0.85)(90^2)(1.15) = 30.67$

$_3 = 0.00256(1.06)(1.484)(0.85)(90^2)(1.15) = 31.88$

$_5 = 0.00256(1.125)(1.415)(0.85)(90^2)(1.15) = 32.27$

$_6 = 0.00256(1.183)(1.347)(0.85)(90^2)(1.15) = 32.3$

$_7 = 0.00256(1.234)(1.279)(0.85)(90^2)(1.15) = 31.99$

$_8 = 0.00256(1.267)(1.217)(0.85)(90^2)(1.15) = 31.35$

$_9 = 0.00256(1.315)(1.141)(0.85)(90^2)(1.15) = 30.41$

$_{10} = 0.00256(1.348)(1.09)(0.85)(90^2)(1.15) = 29.78$

TECH III

Wind Calculations
11-13-09

$P_1 = q_{z_i} G C_p - q_i (GC_{pi})$ lb/ft²

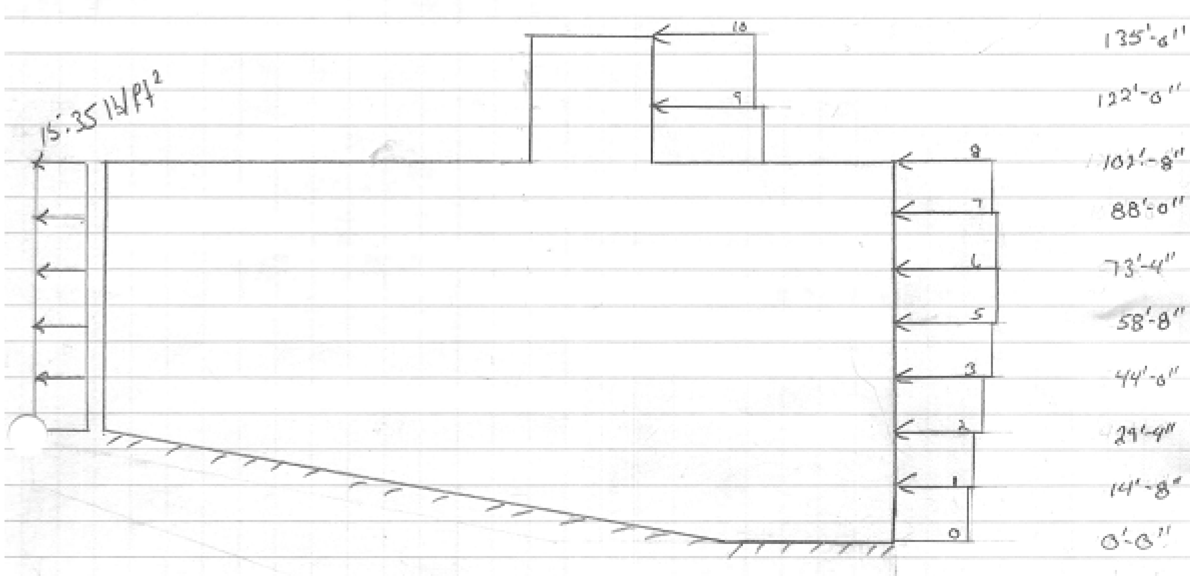
$q_i = 30.41$
$G = 0.856$
$C_p = 0.8$
$(GC_{pi}) = \pm 0.18$

P (East - west) windward side

$P_1 = 27.91(0.856)(0.8) - 30.41(-0.18) = 24.59$ lb/ft²
$_2 = 30.67(0.856)(0.8) - 30.41(-0.18) = 26.48$
$_3 = 31.88(0.856)(0.8) + 5.474 = 27.30$
$_5 = 32.27(0.856)(0.8) + 5.474 = 27.57$
$_6 = 32.3(0.856)(0.8) + 5.474 = 27.59$
$_7 = 31.99(0.856)(0.8) + 5.474 = 27.38$
$_8 = 31.25(0.856)(0.8) + 5.474 = 26.87$
$_9 = 30.41(0.856)(0.8) + 5.474 = 26.30$
$_{10} = 29.78(0.856)(0.8) + 5.474 = 25.87$

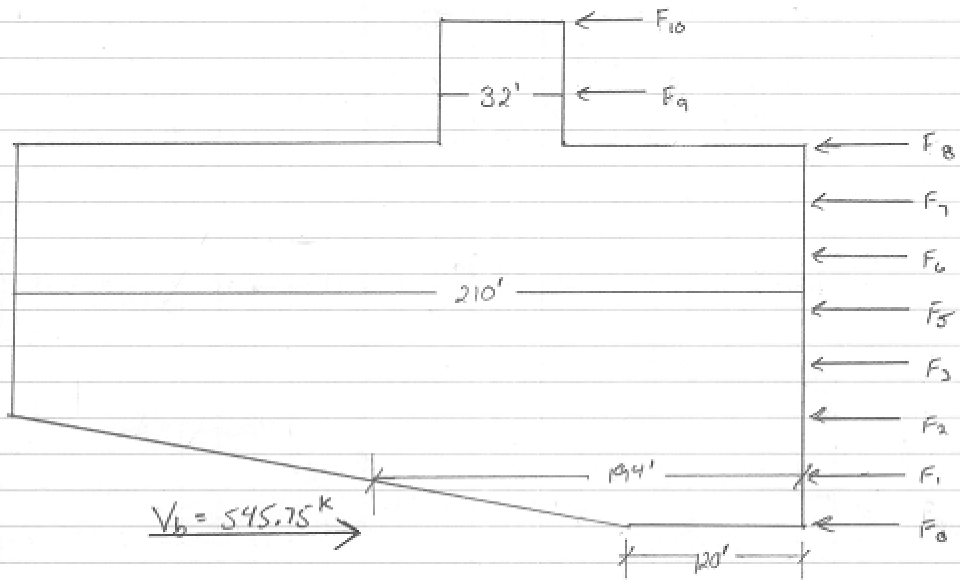
Leeward Side

$P = 30.41(0.856)(0.8) - 30.41(0.18) = 15.35$ lb/ft²



TECH III Wind Calculations

11-13-09



WEST ELEVATION

$F_0 = \frac{1}{2}(14.67')(120')(24.59) = 21.644\ k$

$F_1 = 7.33'(120')(24.59) + 7.33'(157')(26.48) = 52.10\ k$

$F_2 = 7.33'(157')(26.48) + 7.33(194)(27.36) = 69.29\ k$

$F_3 = 7.33(194)(27.36) + 7.33(210)(27.57) = 81.26\ k$

$F_5 = 7.33(210)(27.57) + 7.33(210)(27.59) = 84.91\ k$

$F_6 = 7.33(210)(27.59) + 7.33(210)(27.38) = 84.64\ k$

$F_7 = 7.33(210)(27.38) + 7.33(210)(26.87) = 83.51\ k$

$F_8 = 7.33(210)(26.87) + \frac{19.33}{2}(32')(26.30) = 49.5\ k$

$F_9 = \frac{19.33}{2}(32')(26.3) + \frac{13}{2}(32)(25.87) = 13.52\ k$

$F_{10} = \frac{13}{2}(32)(25.87) = 5.38\ k$

Total = 545.75

TECH III Wind Calculations
11-13-09

North – South

Same K_z, K_{zt}, K_d V^2 + I as East - West

$q_h = 30.41$

$(GC_{pi}) = -0.18$

$G = 0.857$

q_{zi} is the same as E-W

$P_i = q_{zi}(0.857)(0.8) - 30.41(-0.18)$

$P_1 = 27.91(0.6856) + 5.4738 = 24.60$ lb/ft^2

$P_2 = 30.67(0.6856) + 5.4738 = 26.61$

$P_3 = 31.88 = 27.33$

$P_5 = 32.27 = 27.61$

$P_6 = 32.3 = 27.63$

$P_7 = 32.0 = 27.43$

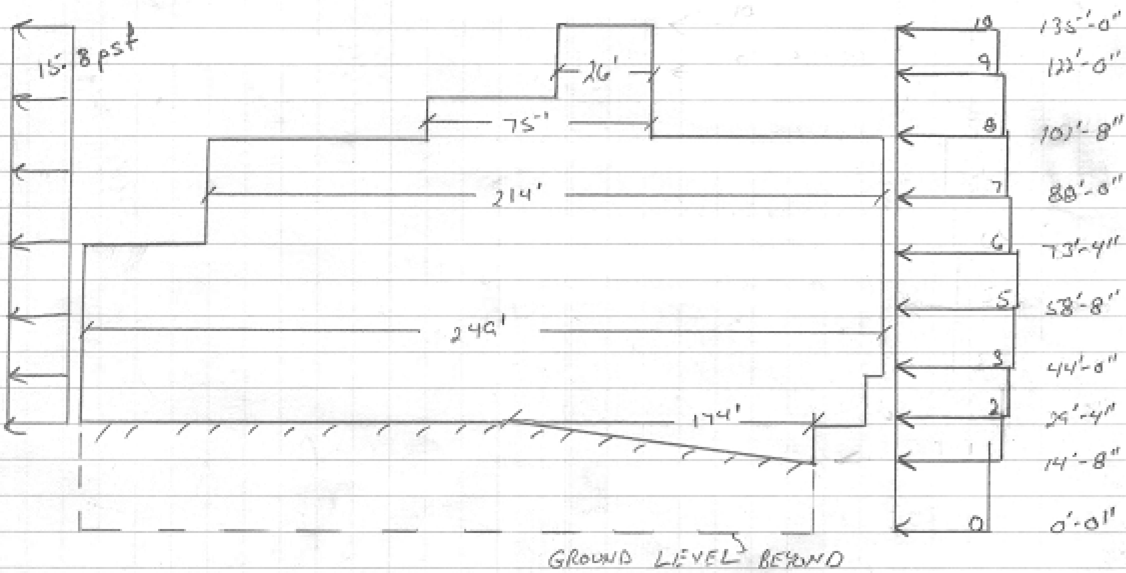
$P_8 = 31.25 = 26.91$

$P_9 = 30.41 = 26.34$

$P_{10} = 29.78 = 25.90$

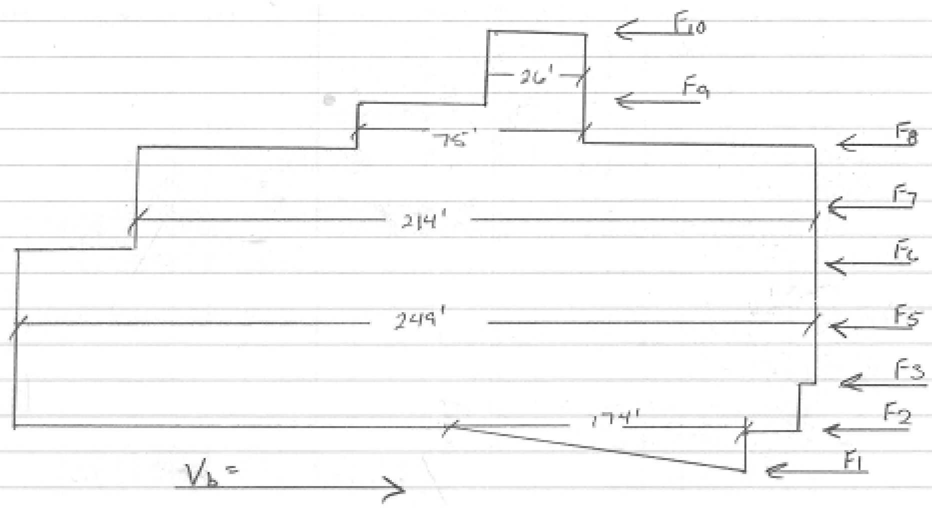
Leeward side

$P = 30.41(0.857)(0.8) - 30.41(+0.18) = 15.8$ lb/ft^2



TECH III Wind Calculations
11-13-09

North - South



$F_0 = 0$

$F_1 = \frac{1}{2}(174')(\frac{1}{2})(14.67')(24.60) = 15.69$ kips

$F_2 = 7.33'(\frac{3}{4})(174')(24.60) + 7.33'(249)(26.61) = 72.10$

$F_3 = 7.33(249)(26.61) + 7.33(249)(27.33) = 98.45$

$F_5 = 7.33(249)(27.33) + 7.33(249)(27.61) = 100.27$

$F_6 = 7.33(249)(27.61) + 7.33(214)(27.63) = 93.73$

$F_7 = 7.33(214)(27.63) + 7.33(214)(27.43) = 86.37$

$F_8 = 7.33(214)(27.43) + \frac{19.33}{2}(75')(26.91) = 62.53$

$F_9 = \frac{19.33}{2}(75')(26.91) + \frac{13}{2}(26')(26.34) = 23.96$

$F_{10} = \frac{13}{2}(26')(26.34) = 4.45$

Appendix C: Snow

JIM ROTUNNO TECH I + III SNOW LOAD CALCULATIONS

USING ASCE 7-05 CHAPTER 7

Flat roof snow load (p_f)

$p_f = 0.7\, C_e\, C_t\, I\, p_g$

$p_g \Rightarrow$ Butler, PA Figure 7-1 = ground snow load
= 25 psf

$C_e \Rightarrow$ Table 7-2 terrain category C roof partially exposed
= 1.0

$C_t \Rightarrow$ Table 7-3
= 1.0

I = Table 7-4 Category IV
= 1.2

$p_f = 0.7(1.0)(1.0)(1.2)(25) = 21$ psf

7.3 where p_g exceeds 20 lb/ft²

$p_f = 20I$

$= \boxed{24 \text{ psf}}$ not 21 psf

Appendix D: Seismic calculations

JIM ROTUNNO TECH I + III SEISMIC ANALYSIS

Occupancy Category IV

Determine the design spretral responce acceleration

$S_{DS} = 2/3\, S_{MS}$

$S_{MS} = F_a S_S$ ⇒ F_a = site coefficient Table 11.4.1

Site Class C

$S_S = .12$ $S_1 = 0.046 \Rightarrow 0.0055$

$F_a = 1.2$

$S_{MS} = 1.2(0.12) = 0.144$

$S_{DS} = 2/3\, S_{MS} = 2/3(0.144) = 0.096$

$S_{D1} = 2/3\, S_{M1} \Rightarrow S_{M1} = F_v S_1$

$= 1.7(0.046) = 0.0782$

$S_{D1} = 2/3(0.0782) = 0.0521$

Importance Factor 1.5

SDC ⇒ Seismic design category = A ⇒ Table 11.6.1 occupancy IV

$S_{DS} < 0.167$

From Table 11.6.2

occupancy IV

$S_{D1} < 0.067$ ∴ A

Calculate the seismic base shear

$V = C_S W$ W = Total dead load for seismic load determination

C_S = Seismic Responce coefficient ASCE 7-05 §12.8.1

$= S_{DS} / (R/I)$

$= 0.096 / (3.25/1.5) = 0.0443$ and $\le \frac{S_{D1}}{T(R/I)}$ for $T \le T_L$

$T: T_a = C_t h_n^x$ eq. 12.8.7

T_L = §11.4.5 Fig 22-15

$= 12$

$T = 0.02(135\,ft)^{0.75} = 0.792\,sec$

$S_{D1}/T(R/I) = 0.0782 / 0.792(3.25/1.5)$

$C_S = 0.0456$

JIM ROTUNNO TECH I & III SEISMIC ANALYSIS

Effective seismic weight W_T as defined 12.7.2
1) 25% LL for for storage areas
2) partitions - minimum of 10 psf
3) total operating weight of permanent equipment use 10psf
4) where the flat roof snow load exceeds 30psf use 20% p_f < 30 ∴ Not required
plus the total Dead load

51psf weight of concrete slab + metal deck concrete 3½" ↕ metal 3" ↕

$$115\text{ pcf}\left(\frac{3.5 + 3/2}{12''}\right) + 3\text{ psf} = 50.92\text{ lb/ft}^2 \quad \text{use } \underline{51\text{ psf}}$$

Superimposed Dead Loads ⇒ MEP, partitions, finishes

25psf use 25psf

289.16 kips columns from column schedule (tabulated using excel)

beams (see next page)

1.0psf operating weight of permanent equipment - 1.0psf of total building area

42psf roof - concrete deck, insulation, EDPM - 42psf

building square footage from construction documents

L_1 = 20 405 sq. ft
L_2 = 45 545 sq. ft
L_3 = 42 165 sq. ft
L_5 = 31 525 sq. ft
L_6 = 27 720 sq. ft
L_7 = 27 760 sq. ft
Roof ⇒ L_8 = 46000

Exterior wall weight ⇒ brick veneer 120 lb/ft³

$$120\frac{\text{lb}}{\text{ft}^3}\left(\frac{3.25''}{12''}\right) = 32.5\text{ lb/ft}^2$$

Story height = 14'-8"

JIM ROTUNNO TECH I + III SEISMIC CALCULATIONS

Level by Level weight analysis (Example)

	concrete	MEP	Equip.	columns	beams	brick facade
$W_1 =$	51(20 405) +	25(20 405) +	1.0(20 405) +	58.39 +	142.84 +	4.86

= 1772.4 k

exterior walls are estimated @ 20% glass + 80% brick

average curtain wall weighs 12.5-15 lb/ft² use 15 psf

0.20(15)(sq. ft. of wall per floor) + 0.80(32.5)(sq. ft. of ext. wall per floor) = lb of exterior wall per level ⇒ divide this by the level's square footage to get a lb/sq. ft per level then add into excel spreadsheet

0.20(15) + 0.80(32) = 28.6 lb/ft² of exterior wall

Beams

weight of beams @ each floor level is tabulated by taking (3) spot checks of average bays and calculating the % steel per floor area and superimposing it to all floors

spot check 1: @ level 3

Typical bay = 30' x 28' = 840 sq. ft

⇒ 4 - 18x40-30' 1 21x50-28' ⇒ only 1 girder per bay because they are shared between bays

4(40)(30) + 1(50)(28) = 6200 lb

6200 lb/840 sq. ft = 7.38 lb/ft²

2: Typical bay level 2 30'x30' = 900 sq. ft

4- 16x26 1-24x55

4(26)(30) + 1(55)(30) = 4770

4770/900 = 5.3 lb/ft²

3: Typical bay level 5 30x28 = 840 sq. ft

5- 18x40-30' 1-24x62-28'

5(40)(30) + 1(62)(28) = 7736 lb

7736 lb / 840 = 9.2 lb/ft²

(7.38 + 5.3 + 9.2)/3 = 7.3 average

USE 7.0 lb/ft²

JIM ROTUNNO TECH I + III SEISMIC CALCULATIONS

Distributing the total base shear to individual levels
The roof square footage needs to be divide up between levels 3, 5, 7, + 8

3 = 45 545 - 31 525 = 14 020 14020/45545 = 31%
5 = 31 525 - 27760 = 3765 3765/45545 = 8%
7 = 27760 - 2060 = 25700 25700/45545 = 56%
8 = 45545 - 14020 - 3765 - 25700 = 2060 2060/45545 = 5%

From excel take the floor weight totals (summed across) and add the % of roof weight to levels 3 5 + 7 to get total floor level weights

1 = 1772.4k
2 = 3876.4k
3 = 3610.9 + 0.31(1913) = 4203.93
5 = 2689.9 + 0.08(1913) = 2842.94
6 = 2368.0k
7 = 2361.7 + 0.56(1913) = 3432.98
8 = 0.5(1913) = 95.65k

Total = 18676.1 kips

USE THESE LEVEL weights to estimate the % of Total Base Shear that acts at each level.

1 ⇒ 1772.4/22616.5 = 7.84% ⇒ 0.0784(851.58) = 66.76 k
2 ⇒ 3876.4/22616.5 = 17.14% ⇒ 0.1714(851.58) = 145.96 k
3 ⇒ 4203.93/22616.5 = 18.59% ⇒ 0.1859(851.58) = 158.31 k
5 ⇒ 2842.94/22616.5 = 12.57% ⇒ 0.1257(851.58) = 107.04 k
6 ⇒ 2368.0/22616.5 = 10.47% ⇒ 0.1047(851.58) = 89.16 k
7 ⇒ 3432.98/22616.5 = 15.18% ⇒ 0.1518(851.58) = 129.27 k
8 ⇒ 95.65/22616.5 = 0.423% ⇒ 0.00423(851.58) = 3.6 k

		Column Load Summary												
W Shapes	**weight**							**Levels**						
14.667	lbs													
			1		**2**		**3**		**5**		**6**		**7**	
8x40	40	1	0.587	0	0.000	0	0.000	0	0.000	0	0.000	0	0.000	
12x40	40	1	0.587	2	1.173	2	1.173	2	1.173	1	0.587	0	0.000	
12x45	45	0	0.000	2	1.320	2	1.320	0	0.000	0	0.000	0	0.000	
12x50	50	1	0.733	2	1.467	2	1.467	1	0.733	1	0.733	0	0.000	
12x53	53	3	2.332	3	2.332	3	2.332	0	0.000	0	0.000	0	0.000	
12x58	58	0	0.000	0	0.000	0	0.000	2	1.701	2	1.701	0	0.000	
12x65	65	2	1.907	1	0.953	1	0.953	0	0.000	0	0.000	0	0.000	
12x72	72	1	1.056	1	1.056	1	1.056	0	0.000	0	0.000	0	0.000	
12x87	87	0	0.000	1	1.276	1	1.276	0	0.000	0	0.000	0	0.000	
12x96	96	2	2.816	1	1.408	1	1.408	0	0.000	0	0.000	0	0.000	
14x43	43	0	0.000	0	0.000	0	0.000	1	0.631	0	0.000	9	5.676	
14x48	48	0	0.000	0	0.000	0	0.000	3	2.112	4	2.816	11	7.744	
14x53	53	0	0.000	0	0.000	0	0.000	3	2.332	4	3.109	5	3.887	
14x61	61	2	1.789	0	0.000	3	2.684	17	15.210	17	15.210	8	7.157	
14x68	68	2	1.995	0	0.000	2	1.995	10	9.974	9	8.976	3	2.992	
14x74	74	4	4.341	0	0.000	4	4.341	1	1.085	1	1.085	0	0.000	
14x82	82	7	8.419	0	0.000	9	10.824	0	0.000	0	0.000	2	2.405	
14x90	90	7	9.240	18	23.761	18	23.761	4	5.280	4	5.280	0	0.000	
14x99	99	0	0.000	1	1.452	1	1.452	0	0.000	0	0.000	0	0.000	
14x109	109	4	6.395	9	14.388	7	11.191	1	1.599	0	0.000	0	0.000	
14x120	120	3	5.280	0	0.000	1	1.760	0	0.000	0	0.000	0	0.000	
14x132	132	2	3.872	0	0.000	0	0.000	0	0.000	0	0.000	0	0.000	
14x145	145	1	2.127	0	0.000	0	0.000	0	0.000	0	0.000	0	0.000	
14x159	159	1	2.332	0	0.000	0	0.000	0	0.000	0	0.000	0	0.000	
14x176	176	1	2.581	0	0.000	0	0.000	0	0.000	0	0.000		0.000	
			58.389	kips	50.586	kips	68.994	kips	41.830	kips	39.498	kips	29.862	kips
		Total column weight=			289.160	kips								

Appendix E: Frame Stiffness and Load Distribution Calculations

Level 1 TECH III JIM ROTONNO

Area and Center of Mass Calculation 11-14-09

$\bar{y}$ or $\bar{x}$ = $\Sigma\, \bar{y}_i(A_i) / \Sigma A_i$

$\bar{y}_{1a}$ = 16' + 18' + 50'/2 = 59'
A_{1a} = (50')(77') = 3850

$\bar{x}_{1a}$ = 77/2 = 38.5
A_{1a} = 77(50) = 3850

$\bar{y}_{1b}$ = 29.75'
A_{1b} = 7.333'(8.5') = 62.334

$\bar{x}_{1b}$ = 33.67 + 11.33 + 8 + 16.67 + 7.33/2 = 73.335'

$\bar{x}_{1c}$ = 33.67 + 11.33 + 8/2 = 49'

$\bar{y}_{1c}$ = 29.75'
A_{1c} = 8'(8.5') = 68

$\bar{x}_{1d}$ = 33.67/2 = 16.835'

$\bar{y}_{1d}$ = 25'
A_{1d} = 18'(33.67') = 606.06

$\bar{x}_{1e}$ = 45 + 32/2 = 61'

$\bar{x}_{1f}$ = 45/2 = 22.5'

$\bar{y}_{1e}$ = 12.75'
A_{1e} = (32)(25.5') = 816

$\bar{x}_2$ = 77 + 12/2 = 83'

$\bar{y}_{1f}$ = 8'
A_{1f} = 16'(45') = 720

$\bar{x}_3$ = 77 + 12 + 12/2 = 95'

$\bar{x}_4$ = 77 + 12 + 9/2 = 93.5'

$\bar{y}_2$ = 9 + 30 = 39'
A_2 = 12'(60) = 720

$\bar{x}_5$ = 77 + 12 + 21/2 = 99.5'

$\bar{y}_3$ = 9 + 10 + 19 + 17/2 = 46.5
A_3 = 12(17) = 204

$\bar{x}_6$ = 86' - 9/2 = 81.5'

$\bar{X}$ = 349957.284 / 7458.394 = 46.92'

$\bar{y}_4$ = 9 + 10 + 19/2 = 28.5
A_4 = 9(19) = 171

$\bar{y}_5$ = 9 + 10/2 = 14
A_5 = 10(16) = 160

CENTER OF MASS
(46.92', 41.21')

$\bar{y}_6$ = 4.5'
A_6 = 9(9) = 81

$\bar{Y}$ = 307386.937 / 7458.394 = 41.21'

CENTER OF MASS CALC. TECH III LEVEL 2

$\bar{y}_1 \approx 11.5' + 12' + 12.5' + \frac{1}{3}(48)' = 52'$

$A_1 \approx \frac{1}{2}(48)(30) = 720$

$\bar{y}_2 = 11.5 + 12 + 12.5 + 48/2 = 60'$

$A_2 = 48(69) = 3312$

$\bar{y}_3 = \bar{y}_2 = 60'$

$A_3 = 75(48) = 3600$

$\bar{y}_4 = 84 - 15 - 14/2 = 62'$

$A_4 = 12(14) = 168$

$\bar{y}_5 = 84 - 29 - 17/2 = 46.5'$

$A_5 = 17(24) = 408$

$\bar{y}_6 = 84 - 48 - 3/2 = 34.5'$

$A_6 = 3(18) = 54$

$\bar{y}_7 = 84 - 48 - 13.67/2 = 29.167'$

$A_7 = 164$

$\bar{y}_8 = (84 - 29 - 30.67)/2 = 12.167'$

$A_8 = (12.167)(2)(28.67) = 349.79$

$\bar{y}_9 = (16 + 3)/2 = 9.5'$

$A_9 = 19(8) = 152$

$\bar{y}_{10} = 8'$

$A_{10} = 13(16) = 208$

$\bar{y}_{11} = 11.5/2 = 5.75'$

$A_{11} = 11.5(18) = 207$

$\bar{y}_{12} = 11.5 + 24.5/2 = 23.75'$

$A_{12} = 8(24.5') = 196$

$\bar{y}_{13} = 5.75'$

$A_{13} = 11.5(8 + 11) = 218.5$

$\bar{y}_{14} = 11.5 + 3/2 = 13'$

$A_{14} = 5(3) = 15$

$\bar{y}_{15} = 36/2 = 18'$

$A_{15} = 17(36) = 612$

$\bar{y}_{16} = 11.5 + 12 + 12.5/2 = 29.75'$

$A_{16} = 12.5(15') = 187.5$

$\bar{y}_{17} = 5.75'$

A_{17} $11.5(15) = 172.5$

$$\bar{y} = \frac{520446.9079}{10744.29} = 48.44''$$

$\bar{x}_1 = -69 - 10 = -79'$

$\bar{x}_2 = -69/2 = -34.5'$

$\bar{x}_3 = 75/2 = 37.5'$

$\bar{x}_4 = 75' + 12/2 = 81'$

$\bar{x}_5 = 75' + 12 = 87'$

$\bar{x}_6 = 15 + 17 + 11 + 8 + 18/2 = 60'$

$\bar{x}_7 = 75 + 24 - 3 - 12/2 = 90'$

$\bar{x}_8 = 75' + 24 - 3 - ((12 + 16.67)/2) = 81.67'$

$\bar{x}_9 = 75 + 24 + 5' - 4 = 100'$

$\bar{x}_{10} = \bar{x}_9 + 4 + 13/2 = 110.5'$

$\bar{x}_{11} = 15 + 17 + 11 + 8 + 18/2 = 60'$

$\bar{x}_{12} = 15' + 17 + 11 + 4 = 47'$

$\bar{x}_{13} = 15' + 17 + 11/2 = 37.5'$

$\bar{x}_{14} = 15' + 17 + 11 + 8 + 5/2 = 53.5'$

$\bar{x}_{15} = 15' + 17/2 = 23.5'$

$\bar{x}_{16} = 15/2 = 7.5'$

$\bar{x}_{17} = 15/2 = 7.5'$

$$\bar{x} = \frac{145421.5993}{10744.29} = 13.53'$$

LEVEL 3 TECH III 11-14-09

CENTER OF MASS CALC.

$\bar{y}_1 = 52'$	$A_1 = 720$	$\bar{x}_1 = -79'$
$\bar{y}_2 = 60'$	$A_2 = 1200$	$\bar{x}_2 = -44' - 12.5' = -56.5'$
$\bar{y}_3 = 60'$	$A_3 = 7440$	$\bar{x}_3 = 120/2 - 44 = 16'$
$\bar{y}_4 = 7'$	$A_4 = 359.324$	$\bar{x}_4 = 60 + 16 + 25.67/2 = 88.83'$
$\bar{y}_5 = 11.5 + 12.25 = 23.75'$	$A_5 = 261.332$	$\bar{x}_5 = 89.33'$
$\bar{y}_6 = 1.5'$	$A_6 = 36$	$\bar{x}_6 = 89.33 + 10.67/2 + 7.5 + 6 = 108.16'$
$\bar{y}_7 = 3'$	$A_7 = 22.5$	$\bar{x}_7 = 108.16 - 6 - 7.5/2 = 98.413'$
$\bar{y}_8 = 34.5'$	$A_8 = 54$	$\bar{x}_8 = 60'$
$\bar{y}_9 = 1.5'$	$A_9 = 13.5$	$\bar{x}_9 = 62.5'$
$\bar{y}_{10} = 27.125'$	$A_{10} = 13.5$	$\bar{x}_{10} = 51'$
$\bar{y}_{11} = 23.75'$	$A_{11} = 207$	$\bar{x}_{11} = 23.75'$
$\bar{y}_{12} = 18'$	$A_{12} = 416.5$	$\bar{x}_{12} = 23.75'$
$\bar{y}_{13} = 34.5'$	$A_{13} = 20$	$\bar{x}_{13} = 3.33'$
$\bar{y}_{14} = 28.25'$	$A_{14} = 112.78$	$\bar{x}_{14} = 5.833'$
$\bar{y}_{15} = 5.75$	$A_{15} = 168.67$	$\bar{x}_{15} = 7.33'$
$\bar{y}_{16} = (33.5/2) - 11.5 = -5.25'$	$A_{16} = 3350$	$\bar{x}_{16} = 64.67'$
$\bar{y}_{17} = -22 - 15.67/2 = -29.83'$	$A_{17} = 172.33$	$\bar{x}_{17} = 114.67 - 11/2 = 109.17'$
$\bar{y}_{18} = -29.83'$	$A_{18} = 757.22$	$\bar{x}_{18} = 61.17'$
$\bar{y}_{19} = -60.08'$	$A_{19} = 3153.28$	$\bar{x}_{19} = 79.5'$
$\bar{y}_{20} = -89.17'$	$A_{20} = 200$	$\bar{x}_{20} = 81.33'$
$\bar{y}_{21} = -92.5'$	$A_{21} = 606.67$	$\bar{x}_{21} = 59.5'$
$\bar{y}_{22} = -110'$	$A_{22} = 225$	$\bar{x}_{22} = 63.83'$
$\bar{y}_{23} = -100'$	$A_{23} = 1038.33$	$\bar{x}_{23} = 29.5'$
	20 597.936	

$$\bar{y} = \frac{146\,892.9641}{20\,547.936} = 7.15'$$

$$\bar{x} = \frac{707090.7863}{20547.936} = 34.41'$$

LEVEL 345 TECH III 11-14-09

CENTER OF MASS CALCULATION (Subtracting voids from Areas)

$\bar{y}_1 = [22080(92') - 71.5(396) - 71.5(220) - 71.5(312) - 36.5(52) - 36(36)]/A_1 = 93.136'$

$A_1 = (158+26)120 - \text{voids} = 22080 - 1016 = 21064$

$\bar{x}_1 = [22080(60') - 75(52) - 117.5(36) - 19.5(396) - 35(220) - 66(312)]/A_1 = 60.90'$

$\bar{y}_2 = 214'$ $A_2 = 1560$ $\bar{x}_2 = 83.67'$

$\bar{y}_3 = 214'$ $A_3 = 288$ $\bar{x}_3 = 136.67'$

$\bar{y}_4 = 183(4500) - 162(144)/A_4$
$800172/4356 = 183.7'$

$A_4 = 50(90) - \text{voids} = 4500 - 144 = 4356$

$\bar{x}_4 = 165(4500) - 128(144)/A_4 = 170.45'$

$\bar{y}_5 = ((158/2)(30 \times 158) - 45'(32')/A_5$
$79.23'$

$A_5 = 30(158) - 2(16) = 4708$

$\bar{x}_5 = (135'(4740) - 122(16) - 148(16)) / 4708 = 135'$

$\bar{y}_6 = [105(\tfrac{1}{2} \times 60 \times 158) - 151(52) - 151(52) - 151(117) - 130(40) - 90(52) - 51.5(15)]/A_6 = 96.05'$

$A_6 = 4740$

$\bar{x}_6 = (170(4740) - 152(52) - 178(52) - 195(117) - 155.5(40) - 156(52) - 161.5(15))/A_6 = 158.06'$

$\bar{y}_7 = 45 + 14/2 = 52'$ $A_7 = 359.324$ $\bar{x}_7 = -12.833'$

$\bar{y}_8 = 45 + 14 + 11 = 70'$ $A_8 = 498.66$ $\bar{x}_8 = -11.333'$

$\bar{y}_9 = 133.25(3349) - 136.25(170.5)/A_9 = 133.09'$

$A_9 = 3349 - 170.5 = 3178.5$

$\bar{x}_9 = -17(3349) - (-22.5)(170.5)/A_9 = -16.71'$

$\bar{y} = \sum \bar{y}_i A_i / \sum A_i = \dfrac{4462395.357}{40752.484} = 109.5'$

$\bar{x} = \sum \bar{x}_i A_i / \sum A_i = \dfrac{3516573.106}{40752.484} = 86.29'$

LEVEL 6 TECH III 11-14-09

CENTER OF MASS CALCULATION (by Subtracting Voids from Areas) similar to level 4

$\bar{y}_1 = (22080(92') - 71.5(396) - 71.5(220) - 71.5(312) - 36.5(52) - 36(36) - 10(120)(6) - 3(142)(6) - 10(149)(6))/A_1$
$= 93.44'$

$A_1 = 22080 - \text{Voids}$
$22080 - 396 - 220 - 312 - 52 - 36 - 23(243) = 20926$

$\bar{x}_1 = [60(22080) - 75(52) - 112.5(36) - 14.5(396) - 35(220) - 66(312) - 2(2')6 - 2(8)6 - 2(22)6 - 2(27)(6) - 32(6) - 37(6) - 42(6) - 52(6) - 2(65)6 - 2(68)6 - 3(83)6 - 94(6)]/A_1$
$= 61.05'$

$\bar{y}_2 = 170(2568) - 174(16 \times 88)/A_2$
$= 168'$

$A_2 = 1184$

$\bar{x}_2 = 75'(2568) - 75(16 \times 88)/A_2$
$= 73.5'$

$\bar{y}_3 = 170'$ $A_3 = 540$ $\bar{x}_3 = 131.25'$

$\bar{y}_4 = 79.23'$ $A_4 = 4708$ $\bar{x}_4 = 135'$

$\bar{y}_5 = (128(3480) - 161(52) - 161(52) - 130(40) - 151(234) - 90(52) - 81.5(15))/A_5 = 112.35'$

$A_5 = 3480 - 52 - 52 - 40 - 234$
$= 3102$

$\bar{x}_5 = (180(3480) - 152(52) - 178(52) - 210.5(234) - 155.5(40))/A_5$
$= 178.37'$

$\bar{y}_6 = 106.67'$ $A_6 = 560$ $\bar{x}_6 = 200'$

$\bar{y}_7 = (88(2240) - 90(52) - 77(15))/A_7$
$= 88.03'$

$A_7 = 2240 - 52 - 15$
$= 2173$

$\bar{x}_7 = (170'(2240) - 155.5(40) - 151.5(15))/A_7$
$= 171.33'$

$\bar{y}_8 = 50'$ $A_8 = 1200$ $\bar{x}_8 = 26.67'$

$\bar{y}_9 = 52'$ $A_9 = 359.324$ $\bar{x}_9 = -12.833$

$\bar{y}_{10} = 70'$ $A_{10} = 498.66$ $\bar{x}_{10} = -11.333$

$\bar{y}_{11} = 133.25'$ $A_{11} = 3349$ $\bar{x}_{11} = -17$

$\Sigma A = 38599.984$

$$\bar{y} = \frac{3771090.868}{38599.984} = 97.7'$$

$$\bar{x} = \frac{3073423.611}{38599.984} = 79.62'$$

LEVEL 7 & 8 TECH III 11-14-09

CENTER OF MASS CALCULATION

$\bar{Y}_1 = (44'(10560) - 71.5(396) - 71.5(220) - 71.5(312) - 36.5(52) - 36(36))/A_1 = 41.397'$

$A_1 = 10560 - 396 - 220 - 312 - 52 - 36 = 9544$

$\bar{X}_1 = (60(10560) - 76(52) - 112.5(36) - 14.5(396) - 35(226) - 66(312))/A_1 = 61.99'$

$\bar{Y}_2 = 79.23'$ $A_2 = 4708$ $\bar{X}_2 = 135'$

$\bar{Y}_3 = 112.36'$ $A_3 = 3102$ $\bar{X}_3 = 178.37'$

$\bar{Y}_4 = 88.03'$ $A_4 = 2173$ $\bar{X}_4 = 171.33'$

$\bar{Y}_5 = 106.67'$ $A_5 = 560$ $\bar{X}_5 = 200'$

$\bar{Y}_6 = 50'$ $A_6 = 1200$ $\bar{X}_6 = 26.67'$

$\bar{Y}_7 = 52'$ $A_7 = 359.324$ $\bar{X}_7 = -12.833'$

$\bar{Y}_8 = 70'$ $A_8 = 498.66$ $\bar{X}_8 = -11.333'$

$\bar{Y}_9 = 164'$ $A_9 = 874$ $\bar{X}_9 = 21'$

$\bar{Y}_{10} = 122'$ $A_{10} = 164$ $\bar{X}_{10} = -39.33'$

23122.984

$$\bar{Y} = \frac{1584816.946}{23122.984} = 68.54'$$

$$\bar{X} = \frac{2556953.551}{23122.984} = 116.32'$$

$\bar{Y}_b = 136.25$ $\bar{X}_b = 48'$

LEVEL 9 TECH III 11-14-09

CENTER of MASS Calculations

$\bar{y}_1 = (44'(10560) - 36.5(52) - 36(36) - 54(2100))/A_1 = 41.573'$

$A_1 = 10560 - 52 - 36 - 2100 = 8372$

$\bar{x}_1 = (60(10560) - 76(52) - 112.5(36) - 37.5(2100))/A_1 = 65.32'$

$\bar{y}_2 = 79.23'$ $A_2 = 4708$ $\bar{x}_2 = 135'$

$\bar{y}_3 = (118(3480) - 151(52) - 151(52) - 90(52) - 136(6) - 145(8))/A_3 = 117.31'$

$A_3 = 3480 - 52 - 52 - 52 - 6 - 8 = 3310$

$\bar{x}_3 = (180(3480) - 130(52) - 151(52) - 151(52) - 136(6) - 168(8))/A_3 = 181.8$

$\bar{y}_4 = 88.03'$ $A_4 = 2173$ $\bar{x}_4 = 171.33'$

$\bar{y}_5 = 106.67'$ $A_5 = 560$ $\bar{x}_5 = 200'$

$\bar{y}_6 = 50'$ $A_6 = 1200$ $\bar{x}_6 = 26.67'$

$\bar{y}_7 = 52'$ $A_7 = 359.324$ $\bar{x}_7 = -12.833'$

$\bar{y}_8 = 74'$ $A_8 = 634.66$ $\bar{x}_8 = -11.333'$

$\underline{}\ 21316.984$

$\bar{y} = \dfrac{1486634.174}{21316.984} = 69.71'$

$\bar{x} = \dfrac{2288697.323}{21316.984} = 107.365'$

LEVEL 10 $\bar{y}_1 = 38.167'$ $\bar{x} = 49.25$

$\bar{y}_2 = (74(2100) - 71(8.75) - 70(88) - 63(25) - 2(72)(6) - 70(6) - 78.5(171))/A_2 = 73.71'$

$A_2 = 2100 - 8.75 - 88 - 25 - 6 - 6 - 171 = 1795.25$

$\bar{x}_2 = (37.5(2100) - 4(8.75) - 16(171) - 36(25) - 36.5(88) - 63(6)(2) - 72(6))/A_2 = 39.465'$

AT LEVEL 3 TECH III 11-19-09

CENTER OF RIGIDITY

$K = AE/L$ assuming only braces are participating

$k_1 \Rightarrow$ HSS 8x8x3/8 $\Rightarrow A = 10.4 in^2$ $L = 251.75''$

$= 10.4(29000)/251.75$

$= 1198.01$ k/in

$k_2 = k_1$

$k_3 \Rightarrow$ HSS 10x10x5/8 $\Rightarrow A = 21.0 in^2$ $L = 251.75''$

$= 21(29000)/251.75$

$= 2419.066$ k/in

$k_4 \Rightarrow$ HSS 10x10x3/8 $A = 13.2 in^2$ $L = 243.31''$

$= 13.2(29000)/243.31$

$= 1573.3$ k/in

$k_5 \Rightarrow$ HSS 10x10x1/2 $A = 17.2 in^2$ $L = 243.31''$

$= 17.2(29000)/243.31$

$= 2050.06$ k/in

$k_6 \Rightarrow$ HSS 10x10x1/2 $A = 17.2 in^2$ $L = 235.184''$

$= 17.2(29000)/235.184$

$= 2120.89$ k/in

$$\bar{X} = \frac{\Sigma k_{iy} X_i}{\Sigma k_{iy}} \qquad \bar{Y} = \frac{\Sigma k_{ix} Y_i}{\Sigma k_{ix}}$$

$$\bar{X} = \frac{k_4(30') + k_5(150') + k_6(120')}{k_4 + k_5 + k_6} \qquad \bar{Y} = \frac{k_1(88') + k_1(30') + k_3(184')}{k_1 + k_2 + k_3}$$

$$= \frac{1573.3(30') + 2050.06(150') + 2120.89(120')}{1573.3 + 2050.06 + 2120.89} \qquad = \frac{1198.01(88) + 1198.01(30) + 2419.066(184)}{1198(2) + 2419.066}$$

$= 106.06$ $\qquad = 121.8$

AT LEVEL 5 TECH III 11-15-09

CENTER OF RIGIDITY

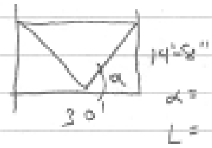
Assuming loading perpendicular to frame direction (strong axis)

$K = AE/L \qquad \delta = PL/AE$

$k_1 \Rightarrow$ HSS $8 \times 8 \times 3/8 \Rightarrow A = 10.4 \quad L = 251.75''$

$K_1 = 10.4(29000)/251.75$

$= 1198.01 \text{ k/in}$



$\alpha = 64.36^\circ$

$L = 20.98' = 251.75''$

$K_2 = K_1$

$k_3 \Rightarrow$ HSS $10 \times 10 \times 3/8 \Rightarrow A = 13.2 \text{ in}^2 \quad L = 251.75''$

$= 13.2(29000)/251.75$

$= 1520.556 \text{ k/in}$

$k_4 \Rightarrow$ HSS $10 \times 10 \times 3/8 \Rightarrow A = 13.2 \text{ in}^2 \quad L = 243.31''$

$= 13.2(29000)/243.31$

$= 1573.3 \text{ k/in}$

$k_5 \Rightarrow$ HSS $10 \times 10 \times 1/2 \Rightarrow A = 17.2 \text{ in}^2 \quad L = 243.31''$

$= 17.2(29000)/243.31$

2050.06 k/in

$k_6 \Rightarrow$ HSS $10 \times 10 \times 1/2 \Rightarrow A = 17.2 \text{ in}^2 \quad L = 235.184''$

$17.2(29000)/235.184$

2120.89 k/in

$$\bar{x} = \frac{\sum k_{iy} x_i}{\sum k_{iy}} \qquad \bar{y} = \frac{\sum k_{ix} y_i}{\sum k_{ix}}$$

$$\bar{x} = \frac{k_4(30') + k_5(150') + k_6(120')}{k_4 + k_5 + k_6} \qquad \bar{y} = \frac{k_1(88') + k_2(30') + k_3(184')}{k_1 + k_2 + k_3}$$

$$= \frac{1573.3(30) + 2050.06(150) + 2120.89(120)}{1573.3 + 2050.06 + 2120.89} \qquad = \frac{1198.01(88) + 1198(30) + 1520.556(184)}{2(1198.01) + 1520.556}$$

$$= 106.86' \qquad = 107.53'$$

AT LEVEL 6 TECH III 11-15-09

CENTER of RIGIDITY

$K_1 \Rightarrow$ HSS $8 \times 8 \times 5/16$ $A = 8.76\ in^2$ $L = 251.75''$

$= 8.76(29000)/(251.75'')$

$= 1009.10\ k/in$

$K_2 \Rightarrow$ HSS $8 \times 8 \times 3/8$ $A = 10.4\ in^2$ $L = 251.75''$

$= 10.4(29000)/251.75$

$= 1198.01\ k/in$

$K_3 = K_2 = 1198.01\ k/in$

$K_4 \Rightarrow$ HSS $8 \times 8 \times 3/8$ $A = 10.4\ in^2$ $L = 243.31''$

$= 10.4(29000)/243.31$

$= 1239.57\ k/in$

$K_5 \Rightarrow$ HSS $10 \times 10 \times 3/8$ $A = 13.2\ in^2$ $L = 243.31''$

$= 13.2(29000)/243.31$

$= 1573.3\ k/in$

$K_6 \Rightarrow$ HSS $10 \times 10 \times 3/8$ $A = 13.2\ in^2$ $L = 235.184''$

$= 13.2(29000)/235.184$

$= 1627.66\ k/in$

$$\bar{x} = \frac{k_4(30') + k_5(150') + k_6(120')}{k_4 + k_5 + k_6} \qquad \bar{y} = \frac{k_1(88') + k_2(30') + k_3(184')}{k_1 + k_2\ k_3}$$

$$= \frac{1239.57(30) + 1573.3(150) + 1627.66(120)}{1239.57 + 1573.3 + 1627.66} \qquad = \frac{1009.1(88) + 1198.01(30) + 1198.01(184)}{1009.1 + 2(1198.01)}$$

$$= 105.51' \qquad = 101.37'$$

AT LEVEL 7 + 8 TECH III 11-15-09

CENTER of RIGIDITY

$K_1 \Rightarrow$ HSS 8×8×5/16 $A = 8.76\,in^2$ $L = 251.75''$

$= 8.76(29000)/251.75$

$= 1009.1$ k/in

$K_2 = K_1 = K_3$

$K_4 \Rightarrow$ HSS 8×8×5/16 $A = 8.76\,in^2$ $L = 243.31''$

$= 8.76(29000)/243.31$

$= 1044.1$ k/in

$K_5 \Rightarrow$ HSS 8×8×3/8 $A = 10.4\,in^2$ $L = 243.31''$

$= 10.4(29000)/243.31$

$= 1239.571$ k/in

$K_6 \Rightarrow$ HSS 8×8×5/16 $A = 8.76\,in^2$ $L = 235.184''$

$= 8.76(29000)/235.184$

$= 1080.176$ k/in

$$\bar{X} = \frac{k_4(30') + k_5(150') + k_6(120')}{k_4 + k_5 + k_6} \qquad \bar{Y} = \frac{K_1(88') + k_2(30') + k_3(184')}{K_1 + k_2 + k_3}$$

$$= \frac{1044.1(30) + 1239.571(150) + 1080.176(120)}{1044.1 + 1239.571 + 1080.176} \qquad = \frac{1009.1(88) + 1009.1(30) + 1009.1(184)}{3(1009.1)}$$

$$= 103.12' \qquad = 100.67'$$

DIRECT FORCE INTO EACH FRAME ON LEVEL 3 TECH III 11-A-09

$P_y = 98.45k$ Table 3.5

$\Sigma k_y = k_1 + k_2 + k_3$

$= 1198.01 + 1198.01 + 2419.066$

$= 4815.086$

$P_x = 81.26k$ Table 3.3

$\Sigma k_x = k_4 + k_5 + k_6$

$= 1573.3 + 2050.06 + 2120.89$

$= 5744.25$

$$F_1 = \frac{k_1}{\Sigma k_y} P_y = \frac{1198.01(98.45)}{4815.086} = 24.495\ k$$

$$F_2 = F_1 = 24.495\ k$$

$$F_3 = \frac{k_3}{\Sigma k_y} P_y = \frac{2419.066(98.45)}{4815.086} = 49.461\ k$$

$$F_4 = \frac{k_4}{\Sigma k_x} P_x = \frac{1573.3(81.26)}{5744.25} = 22.256\ k$$

$$F_5 = \frac{k_5}{\Sigma k_x} P_x = \frac{2050.06(81.26)}{5744.25} = 29.0\ k$$

$$F_6 = \frac{k_6}{\Sigma k_x} P_x = \frac{2120.89(81.26)}{5744.25} = 30.0\ k$$

DIRECT FORCE INTO EACH FRAME ON EACH LEVEL TECH III 11-15-09

LEVEL 5 $P_y = 100.27\text{ k}$ $\sum k_y = k_1 + k_2 + k_3$
$= 1198.1(2) + 1520.556 = 3916.756$

$$F_1 = \frac{k_1}{\sum k_y}(P_y) = \frac{1198.01(100.27)}{3916.756}$$

$= 30.669\text{ k}$ from Table 3.5

$$F_2 = \frac{k_2}{\sum k_y}(P_y) = \frac{1198.01(100.27)}{3916.756} = 30.669\text{ k}$$

$$F_3 = \frac{k_3}{\sum F_y}(P_y) = \frac{1520.556(100.27)}{3916.756} = 38.927\text{ k}$$

$P_x = 84.91\text{ k}$ from Table 3.3 $\sum k_x = k_4 + k_5 + k_6$
$= 5744.25$

$$F_4 = \frac{k_4}{\sum k_x}(P_x) = \frac{1573.3(84.91)}{5744.25}$$
$= 23.256\text{ k}$

$$F_5 = \frac{k_5}{\sum k_x} P_x = \frac{2050.06(84.91)}{5744.25}$$
$= 30.30\text{ k}$

$$F_6 = \frac{k_6}{\sum k_x} P_x = \frac{2120.89(84.91)}{5744.25}$$
$= 31.35\text{ k}$

DIRECT FORCE INTO EACH FRAME ON LEVEL 6   TECH III   11-15-09

$P_x = 84.64\,k$ Table 3.3

$\sum k_y = k_1 + k_2 + k_3$

$= 2(1198.01) + 1009.1$

$= 3405.12$

$P_y = 93.73$ Table 3.5

$\sum k_x = k_4 + k_5 + k_6$

$= 1239.57 + 1573.3 + 1627.66$

$= 4440.53$

$$F_1 = \frac{k_1}{\sum k_y} P_y = \frac{1198.01(93.73)}{3405.12} = 32.98\,k$$

$$F_2 = \frac{k_2}{\sum k_y} P_y = \frac{1198.01(93.73)}{3405.12} = 32.98\,k$$

$$F_3 = \frac{k_3}{\sum k_y} P_y = \frac{1009.1(93.73)}{3405.12} = 27.78$$

$$F_4 = \frac{k_4}{\sum k_x} P_x = \frac{1239.57(84.64)}{4440.53} = 23.627\,k$$

$$F_5 = \frac{k_5}{\sum k_x} P_x = \frac{1573.3(84.64)}{4440.53} = 29.99\,k$$

$$F_6 = \frac{k_6}{\sum k_x} P_x = \frac{1627.66(84.64)}{4440.53} = 31.02\,k$$

DIRECT FORCE INTO EACH FRAME ON LEVEL 7 — TECH III — 11-15-09

$P_y = 86.37$ Table 3.5

$\Sigma k_y = k_1 + k_2 + k_3$

$= 1009.1(3)$

$= 3027.3$

$P_x = 83.51$ Table 3.3

$\Sigma k_x = k_4 + k_5 + k_6$

$= 1044.1 + 1239.571 + 1080.176$

$= 3363.847$

$$F_1 = \frac{k_1}{\Sigma k_y} P_y = \frac{1009.1(86.37)}{3027.3} = 28.79\,k$$

$$F_2 = \frac{k_2}{\Sigma k_y} P_y = F_1 = 28.79\,k$$

$$F_3 = \frac{k_3}{\Sigma k_y} P_y = F_1 = 28.79\,k$$

$$F_4 = \frac{k_4}{\Sigma k_x} P_x = \frac{1044.1(83.51)}{3363.847} = 25.92\,k$$

$$F_5 = \frac{k_5}{\Sigma k_x} P_x = \frac{1239.571(83.51)}{3363.847} = 30.77\,k$$

$$F_6 = \frac{k_6}{\Sigma k_x} P_x = \frac{1080.176(83.51)}{3363.847} = 26.82\,k$$

DIRECT FORCE INTO EACH FRAME ON LEVEL 8 TECH III 11-15-09

$P_y = 62.53k$ Table 3.5

$\Sigma k_y = k_1 + k_2 + k_3$

$= 1009.1(3)$

$= 3027.3$

$P_x = 49.5k$ Table 3.3

$\Sigma k_x = k_4 + k_5 + k_6$

$= 1044.1 + 1239.571 + 1080.176$

$= 3363.847$

$$F_1 = \frac{k_1}{\Sigma k_y} P_y = \frac{1009.3(62.53)}{3027.3} = 20.84\,k$$

$$F_2 = F_1 = F_3$$

$$F_4 = \frac{k_4}{\Sigma k_x} P_x = \frac{1044.1(49.5)}{3363.847} = 15.36\,k$$

$$F_5 = \frac{k_5}{\Sigma k_x} P_x = \frac{1239.571(49.5)}{3363.847} = 18.24\,k$$

$$F_6 = \frac{k_6}{\Sigma k_x} P_x = \frac{1080.176(49.5)}{3363.847} = 15.90\,k$$

FORCE IN EACH FRAME AT EACH LEVEL DUE TO ECCENTRICITY

TECH III

11-19-09

LEVEL 3

$e_x = 106.06' - 86.29 = 19.77'$ $P_y = 98.45\,k$ Table 3.5

$e_y = 121.8' - 109.5' = 12.3'$ $P_x = 81.26\,k$ Table 3.3

d_i = distance from CR to frame i

5% minimum of direct force

$$F_i = \frac{k_{iy}\, d_i\, P_y\, e_x}{\Sigma\, k_{jy}\, d_j^2}$$

y: $F_1 = \dfrac{1198.01(18.06)(98.45)\,19.77}{1198.01(18.06)^2 + 1198.01(76.06)^2 + 2419.066(77.94)^2} = \dfrac{42111487.18}{22016346.97}$

$= \boxed{1.91k} > 5\%$.

$F_2 = \dfrac{1198.01(76.06)(98.45)(19.77)}{22016346.97} = \boxed{8.056k}$

$F_3 = \dfrac{2419.066(77.94)(98.45)(19.77)}{22016346.97} = \boxed{16.668k}$

x: $F_4 = \dfrac{1573.3(91.8)(81.26)(12.3)}{1573.3(91.8^2) + 2050.06(28.2^2) + 2120.89(8.2^2)} = \dfrac{144356436.7}{15031475.05}$

$= \boxed{9.604\,k}$

$F_5 = \dfrac{2050.06(28.2)(81.26)(12.3)}{15031475.05} = \boxed{3.844\,k}$

$F_6 = \dfrac{2120.89(8.2)(81.26)(12.3)}{15031475.05} = 1.156$

5% minimum $= \boxed{1.5\,k}$

FORCE IN EACH FRAME TECH III 11-15-09
AT EACH LEVEL
DUE TO ECCENTRICITY LEVEL 5

$e_x = 106.06 - 86.29 = 19.77'$ $P_y = 100.27\,k$ Table 3.5
$e_y = 109.5 - 107.53 = 1.97'$ $P_x = 84.91\,k$ Table 3.3
d_i = distance from CR to frame i

5% minimum of direct force

$$F_i = \frac{k_{iy}\, d_i\, P_y\, e_x}{\Sigma k_{jy}\, d_j^2}$$

y: $$F_1 = \frac{1198.1(19.53)(100.27)(19.77)}{1198.1(19.53^2) + 1198.1(77.53)^2 + 1520.56(76.47^2)} = \frac{46\,384\,512.41}{16\,550\,360.01}$$
$= \boxed{2.80}\,k$ 5% minimum = 1.53 k

$$F_2 = \frac{1198.1(77.53)(100.27)(19.77)}{16\,550\,360.01} = \boxed{11.13\,k}$$

$$F_3 = \frac{1520.56(76.47)(100.27)(19.77)}{16\,550\,360.01} = \boxed{13.93\,k}$$

x: $$F_4 = \frac{1573.3(76.06)(84.91)(1.97)}{1573.3(76.06^2) + 2050.06(43.94^2) + 2120.89(13.94)^2} = \frac{20\,016\,720.77}{13\,471\,973.16}$$
$= \boxed{1.49\,k}$ 5% minimum = 1.16k

$$F_5 = \frac{2050.06(43.94)(84.91)(1.97)}{13\,471\,973.16} = 1.12\,k \quad \text{5\% minimum} = \boxed{1.52k}$$

$$F_6 = \frac{2120.89(13.94)(84.91)(1.97)}{13\,471\,977.16} = .637k \quad \text{5\% minimum} = \boxed{1.57k}$$

FORCE IN EACH FRAME DUE TO ECCENTRICITY — TECH III LEVEL 6 — 11-15-09

$e_x = 105.51' - 79.62' = 25.89'$ $P_y = 93.73\ k$ Table 3.5

$e_y = 101.37' - 97.7' = 3.67'$ $P_x = 84.64\ k$ Table 3.3

d_i = distance from CR to frame i

5% minimum of direct force

$$F_i = \frac{k_{iy}\, d_i\, P_y\, e_x}{\sum k_{iy}\, d_j^2}$$

Y:

$$F_1 = \frac{1009.1(13.37)(93.73)(25.89)}{1009.1(13.37^2) + 1198.1(71.37^2) + 1198.1(82.63^2)} = \frac{32739819.51}{14463405.5} = \boxed{2.26\ k}$$

5% minimum = 1.65 k

$$F_2 = \frac{1198.1(71.37)(93.73)(25.89)}{14463405.5} = \boxed{14.35\ k}$$

$$F_3 = \frac{1198.1(82.63)(93.73)(25.89)}{14463405.5} = \boxed{16.61\ k}$$

X:

$$F_4 = \frac{1239.57(75.51)(84.64)(3.67)}{1239.57(75.51^2) + 1573.3(44.49^2) + 1627.66(14.49^2)} = \frac{29074834.15}{10523601.67} = \boxed{2.76\ k}$$

$$F_5 = \frac{1573.3(44.49)(84.64)(3.67)}{10523601.67} = \boxed{2.07\ k}$$

$$F_6 = \frac{1627.66(14.49)(84.64)(3.67)}{10523601.67} = 0.696\ k$$

5% minimum = $\boxed{1.55\ k}$

FORCE IN EACH FRAME AT EACH LEVEL DUE TO ECCENTRICITY

TECH III

LEVEL: 7

11-15-09

$e_x = 110.32' - 103.12' = 7.2'$ $\quad P_y = 86.37\,k$ Table 3.5

$e_y = 100.67' - 68.54' = 32.13'$ $\quad P_x = 83.51\,k$ Table 3.3

d_i = distance from CR to frame i

5% minimum of direct force

$$F_i = \frac{k_{iy}\, d_i\, P_y\, e_x}{\sum k_{jy}\, d_j^2}$$

y: $F_1 = \dfrac{1009.1(12.67)(86.37)(7.2')}{1009.1(12.67^2) + 1009.1(70.67^2) + 1009.1(83.33^2)} = \dfrac{7\,950\,715.934}{12\,208\,764.57}$

$= 0.65\,k \quad$ 5% minimum $= \boxed{1.44\,k}$

$F_2 = \dfrac{1009.1(70.67)(86.37)(7.2)}{12\,208\,764.57} = \boxed{3.63\,k}$

$F_3 = \dfrac{1009.1(83.33)(86.37)(7.2)}{12\,208\,764.57} = \boxed{4.28\,k}$

x: $F_4 = \dfrac{1044.1(73.12)(83.51)(32.13)}{1044.1(73.12^2) + 1239.57(46.88^2) + 1080.18(16.88^2)} = \dfrac{204\,845\,999.9}{8\,614\,342.637}$

$= \boxed{23.78\,k}$

$F_5 = \dfrac{1239.57(46.88)(83.51)(32.13)}{8\,614\,342.637} = \boxed{18.1\,k}$

$F_6 = \dfrac{1080.18(16.88)(83.51)(32.13)}{8\,614\,342.637} = \boxed{5.68\,k}$

FORCE IN EACH FRAME AT EACH LEVEL DUE TO ECCENTRICITY

TECH III

LEVEL 8

11-15-09

$e_x = 7.2'$ $P_y = 62.53$ Table 3.5

$e_y = 32.13'$ $P_x = 49.5$ Table 3.3

d_i = distance from CR to frame i

5% minimum of direct force

Y: $F_1 = \dfrac{1009.1(12.67)(62.53)(7.2)}{12\ 208\ 764.57} = 0.47\text{k}$ 5% minimum = $\boxed{1.04\text{ k}}$

$F_2 = \dfrac{1009.1(70.67)(62.53)(7.2)}{12\ 208\ 764.57} = \boxed{2.63\text{ k}}$

$F_3 = \dfrac{1009.1(83.33)(62.53)(7.2)}{12\ 208\ 764.57} = \boxed{3.1\text{ k}}$

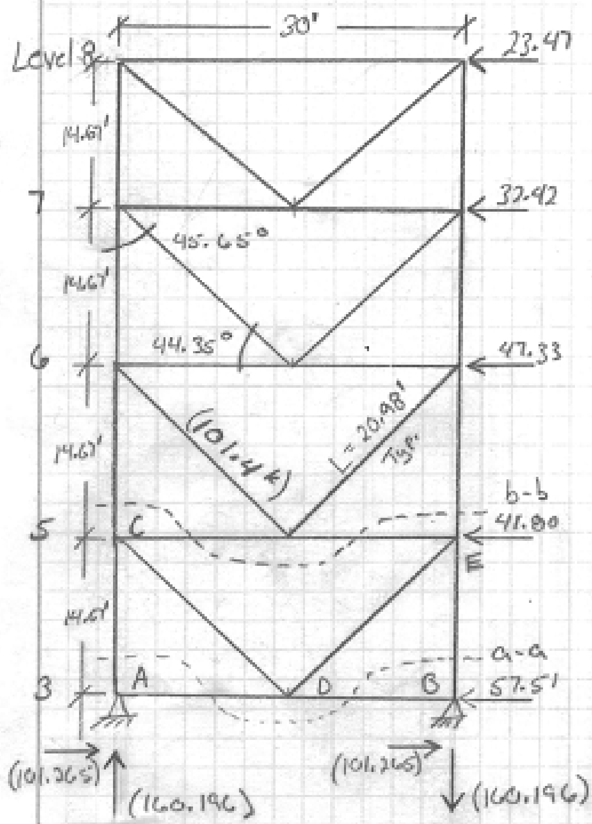
X: $F_4 = \dfrac{1044.1(73.12)(49.5)(32.13)}{8\ 614\ 342.637} = \boxed{14.10\text{ k}}$

$F_5 = \dfrac{1239.57(46.88)(49.5)(32.13)}{8\ 614\ 342.637} = \boxed{10.73\text{ k}}$

$F_6 = \dfrac{1080.18(16.88)(49.5)(32.13)}{8\ 614\ 342.637} = \boxed{3.37\text{ k}}$

Appendix F: Member Spot Checks

FRAME #2 | GRIDLINE 2 D-E | FBD HAND CALCULATIONS



using a-a

$+\circlearrowright \sum M_A = 0$

$R_{By}(30') - 23.47(4)(14.67') - 32.42(3)(14.67') - 47.33(2)(14.67') - 41.80(14.67') = 0$

$R_{By} = 4805.892/30 = 160.196\text{ k}$

$+\uparrow \sum F_y = 0$

$R_{Ay} = 160.196\text{ k}$

using b-b

$+\circlearrowright \sum M_C = 0$

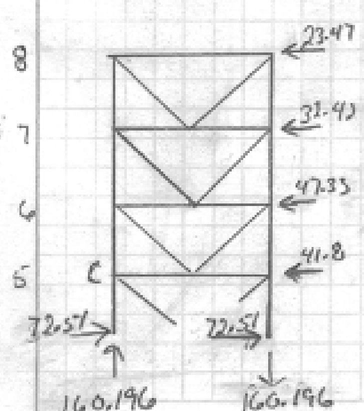
$R_{Ey}(30') = -23.47(3)(14.67') - 32.42(2)(14.67') - 47.33(1)(14.67') = 0$

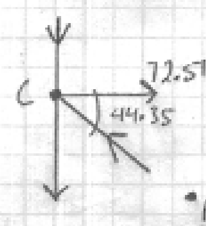
$R_{Ey} = 89.28\text{ k}$

$\xrightarrow{+} \sum F_x = 0$

$R_{Cx} = R_{Ex} = (23.47 + 32.42 + 47.33 + 41.8)/2$

$= 72.51$





$F_{CD} \Rightarrow \cos 44.35 = 72.51/x$

$F_{CD} = 72.51/\cos 44.35$

$\boxed{= 101.4}$

GRAVITY LOADS TO BEAMS TO BRACES | MEMBER SPOT CHECKS | 11-27-09

Roof Area contributing to brace frame

Trib. Area = 7.5' x 30'

DL = 48 psf for decking
+3 psf for metal deck

LL = 115 psf

$W_u = (48+3)(7.5)(1.2) + 115(7.5)(1.6)$
$= 1.839\ k/ft$

Floor Area contributing to brace frame

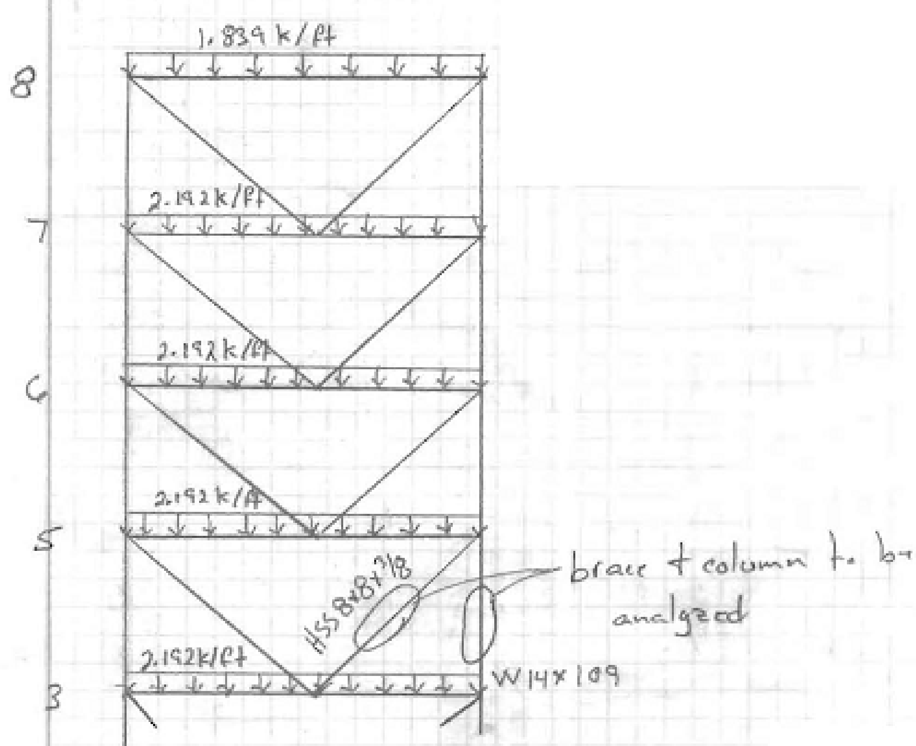
Levels 7, 6 + 5

Tributary Areas = 10' x 30'

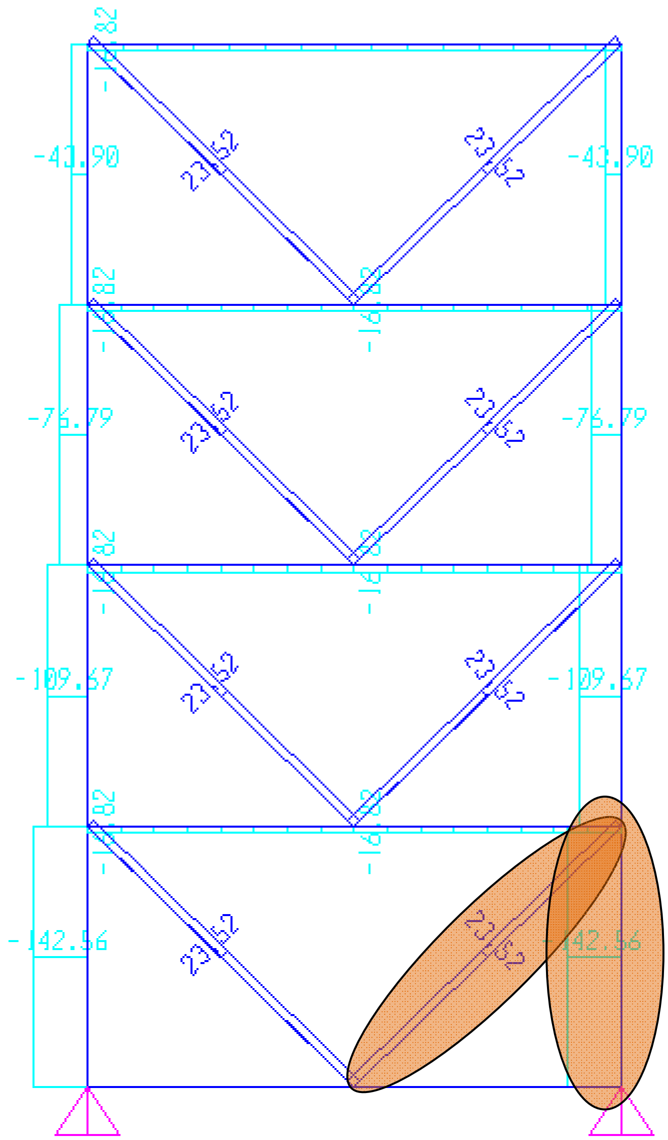
DL = 48 + 3 + 25 psf for MEP

LL = 80 psf no reductions

$W_u = 1.2(76)(10') + 1.6(80)(10)$
$= 2.192\ k/ft$



1



Axial loads in columns and braces determined in SAP 2000 for previous page to be added to hand calculated lateral axial load analysis.

BRACE CHECK | TECH III | 11-27-09

LATERAL AXIAL LOAD IN BRACE (HAND CALCULATION)

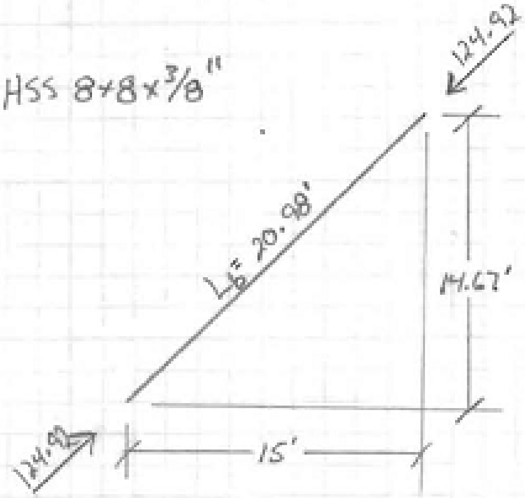
101.4 kips

GRAVITY AXIAL LOAD IN BRACE (FACTORED LIVE + DEAD LOADS)

23.52 kips

Total axial load in brace 124.92 kips

HSS 8x8x3/8"



F_y = 46 ksi

A_g = 10.4 in^2

$r_x = r_y$ = 3.10 in

$KL_x/r = 1.0(20.98')(12")/3.1$ in

$= 81.21 < 113$

$\therefore$ use $F_{cr} = 0.658^{(F_y/F_e)}(F_y)$

Limit State $4.71\sqrt{\frac{29000}{46}} = 118.26$

$81.21 < 118.26 \therefore$ inelastic

$F_e = \pi^2 E/(KL_x/r)^2 = \pi^2(29000)/81.21^2 = 43.40$

$F_{cr} = 0.658^{(46/43.4)}(46) = 29.52$ k

$\phi P_n = \phi F_{cr} A_g = 0.9(29.52)(10.4) = 265.68$ kips

ϕP_n from AISC 13 @ $L_b = 21' \Rightarrow 275$k

$\phi P_n \geq P_u$

$265.68 > 124.92 \therefore$ ok

STRESS RATIO

$\frac{P_u}{\phi P_n} = \frac{124.92}{265.68} = 0.47 < 1.0 \therefore$ OK

COLUMN CHECK	TECH III	11-27-09

W14×109 COMBINED AXIAL LOADS

$P_u \Rightarrow 160.196^k$ From lateral loading

16.34^k From gravity load on columns due to brace tension/compression

$142.56 - [(1.839(30')) + (3\times 2.192\times 30')]/2$

4.67^k From gravity load of columns above

$109(14.67') + 74(2)(14.67') + 61(14.67') = 4.67\,k$

From DL + LL for all floors above

Tributary Area $= 30' \times 30' = 900\,ft^2$

Roof $\Rightarrow 1.2(51psf)(900) + 1.6(115)(900) = 226.680\,k$

Floors $\Rightarrow 3[1.2(76)(900) + 1.6(80)900] = 591.840\,k$

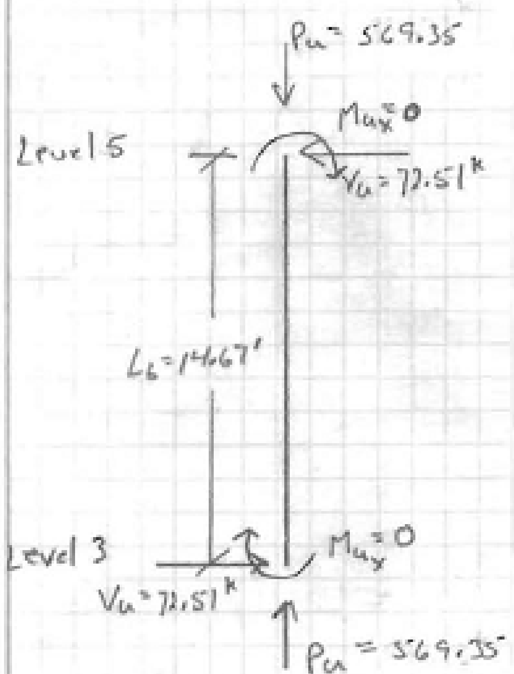
$812.52^k \Rightarrow 226.68 + 591.84 = 812.52\,k$

993.73^k

Load Combinations

$1.2D + 1.6L_r + 0.8W = 374.65$

$1.2D + 1.6W + L + 0.5L_r = 569.35$



$KL = 14.67'(r_x/r_y) = 14.67(1.67)$

$= 24.5'$

ϕP_n From Table 4-1 @ 24.5'

$= 915^k > P_u = 569.35^k \therefore OK$

$\dfrac{P_u}{\phi P_n} = \dfrac{569.35}{915} = 0.622 < 1.0$

$\therefore OK$

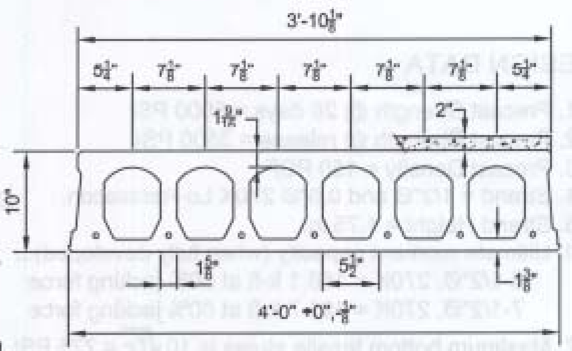
Appendix G: Girder & Slab Sizing

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

2 Hour Fire Resistance Rating With 2" Topping

PHYSICAL PROPERTIES
Composite Section

A_c = 327 in.^{2}	Precast b_w = 13.13 in.
I_c = 5102 in.^{4}	Precast S_{bcp} = 824 in.^{3}
Y_{bcp} = 6.19 in.	Topping S_{tct} = 1242 in.^{3}
Y_{tcp} = 3.81 in.	Precast S_{tcp} = 1340 in.^{3}
Y_{tcp} = 5.81 in.	Precast Wt. = 272 PLF
	Precast Wt. = 68.00 PSF



DESIGN DATA

1. Precast Strength @ 28 days = 6000 PSI
2. Precast Strength @ release = 3500 PSI
3. Precast Density = 150 PCF
4. Strand = 1/2"Ø and 0.6"Ø 270K Lo-Relaxation.
5. Strand Height = 1.75 in.
6. Ultimate moment capacity (when fully developed)...
   6-1/2"Ø, 270K = 168.1 k-ft at 60% jacking force
   7-1/2"Ø, 270K = 191.7 k-ft at 60% jacking force
7. Maximum bottom tensile stress is $10\sqrt{f'c}$ = 775 PSI
8. All superimposed load is treated as live load in the strength analysis of flexure and shear.
9. Flexural strength capacity is based on stress/strain strand relationships.
10. Deflection limits were not considered when determining allowable loads in this table.
11. Topping Strength @ 28 days = 3000 PSI. Topping Weight = 25 PSF.
12. These tables are based upon the topping having a uniform 2" thickness over the entire span. A lesser thickness might occur if camber is not taken into account during design, thus reducing the load capacity.
13. Load values to the left of the solid line are controlled by ultimate shear strength.
14. Load values to the right are controlled by ultimate flexural strength or fire endurance limits.
15. Load values may be different for IBC 2000 & ACI 318-99. Load tables are available upon request.
16. Camber is inherent in all prestressed hollow core slabs and is a function of the amount of eccentric prestressing force needed to carry the superimposed design loads along with a number of other variables. Because prediction of camber is based on empirical formulas it is at best an estimate, with the actual camber usually higher than calculated values.

SAFE SUPERIMPOSED SERVICE LOADS — IBC 2006 & ACI 318-05 (1.2 D + 1.6 L)

Strand Pattern		SPAN (FEET)																		
		26	27	28	29	30	31	32	33	34	35	36	37	38	39	40	41	42	43	44
6 - 1/2"ø	LOAD (PSF)	202	181	161	144	128	114	101	90	79	69	60	52	45	38					
7 - 1/2"ø	LOAD (PSF)	246	222	200	180	162	146	131	118	105	94	84	74	66	58					

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

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

11/03/08

10F2.0T

January 08, 2010

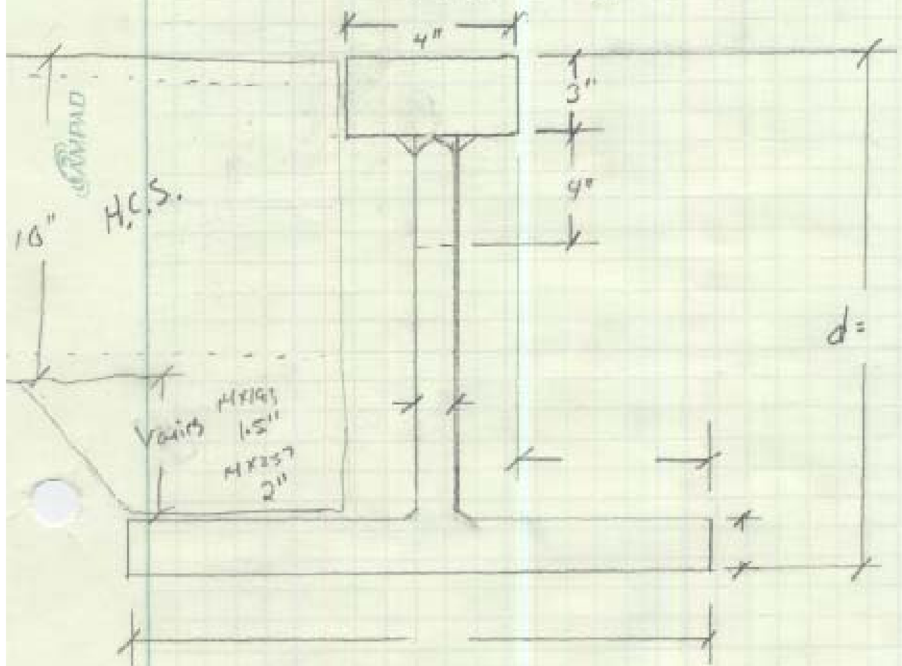
STARTING POINT FOR COMPARISON

Largest girder with highest ϕM_n

24x84 ⇒ ϕM_n = 840 k·ft ⇒ composite ϕM_n = 1630

next 24x68 ⇒ ϕM_n = 664 k·ft ⇒ composite ϕM_n = 1310

24x62 ⇒ ϕM_n = 574 k·ft



W Shapes
Try sizes
- W 18x112
- W14x193
- W14x257
- W18x119
- W18x175

Girder-Slab design guide
PCI standard ⇒ 2" min bearing on bottom flange
U.S.O.

if using a 4" wide top bar then bottom flange needs to be at least 11" wide for 3" bearing on each end and 1/2" for adjustment on each side of beam
5" ⇒ 12" bf

Bar
A572 GR 50
AISC Table 2-4
f_y = 50 ksi

W Shapes
A572 GR 50 or 65
AISC Table 2-3 pg. 2-38
f_y = 50 or 65 ksi

1

January 24, 2010

MAXIMUM GIRDER DESIGN LOAD

The weight of the Hollow core planks is 68.00 psf
(Nitterhouse 10F2.0T) 2HR Fire with 2" topping
Plank span is approximately 29'
SAFE SUPERIMPOSED SERVICE LOAD IS 180 psf
using 7-1/2" Ø Strand Pattern
topping = 25 psf
All superimposed load is treated as LL

Weight of topping = 25
LL no reduction = 80
ASCE 7-05 Hospitals (50-80)
MEP, Partitions = 35
ceiling, med. equip.

= 140 psf < 180
∴ OK

DL = weight of plank + topping + MEP
= 68 + 25 + 35 = 128 psf

LL = 80

$1.2D + 1.6L = 1.2(128) + 1.6(80) = 281.6$ psf

Distributed weight along the girder

$281 \frac{lb}{ft^2} \cdot 30\,ft = 8448\ lb/ft = W_u$

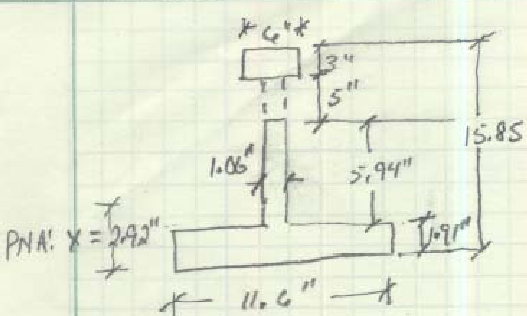
$M_u = \left[\frac{8448}{1000}(30')^2\right]/8 = \boxed{950.4\ k\cdot ft}$

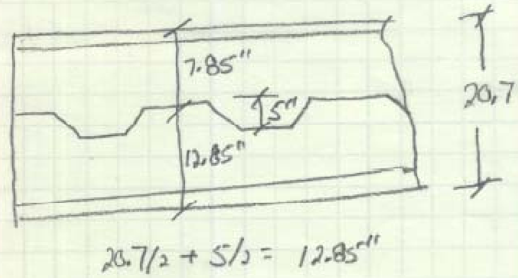
@ 125 psf LL
$\boxed{1107\ k\cdot ft}$

@29' = 888 k·ft ⇒ 1034 k·ft ✓
@28' = 828 k·ft ⇒ 964 k·ft ✓
@27' = 770 k·ft ⇒ 897 k·ft ✓
@31' = 1014 k·ft ⇒ 1182 k·ft
@32' 1080 k·ft ⇒ 1260 k·ft

2

✓ W18x211 Modified   January 24, 2010





$20.7/2 + 5/2 = 12.85''$

$d = 20.7''$
$t_w = 1.06''$
$b_f = 11.6''$
$t_f = 1.91''$

bearing for planks
$(11.6'' - 6'' - 1'')/2 = 2.3'' > 2''$ ∴ OK

PNA for Bare Beam

$3''(6'') + 5.94''(1.06'') + 11.6''(1.91'')$

$18 + 6.2964 + 22.156 = 46.4524$

$46.4524/2 = 23.2262 \text{ in}^2$ ∴ PNA is in the web

$23.2262 - 22.156 = 1.0702$

$1.06'' x = 1.0702$

$x = 1.01''$ up from the top of the bottom flange $1.91 + 1.01 = 2.92''$

$C_f = 3''(6'')(50\text{ ksi}) = 900\text{ k}$

$C_w = 1.06''(5.94 - 1.01)50\text{ ksi} = 261.3\text{ k}$

$T_w = 1.06''(1.01'')50\text{ ksi} = 53.53\text{ k}$

$T_f = 11.6''(1.91'')50\text{ ksi} = 1107.8\text{ k}$

$M_p = 900(3/2 + 5 + (5.94 - 1.01)) + 261.3\left(\frac{5.94 - 1.01}{2}\right) + 53.53\left(\frac{1.01}{2}\right) + 1107.8\left(\frac{1.91}{2} + 1.01\right)$

$= 10287 + 644.1 + 27.03 + 2177$

$= 13135\text{ k}\cdot\text{in}$

$= 1095\text{ k}\cdot\text{ft}$

$\phi M_p = \boxed{985\text{ k}\cdot\text{ft Non composite}}$

$\frac{3(6)}{144}(490)$

bar $= 61.25\text{ lb/ft}$

$W/ft = 61.25 + 211/2 = \boxed{166.75\text{ lb/ft}}$

$I_x = bh^3/12 + Ad^2$ of each shape about ENA

$\left[\frac{6(3^3)}{12} + 6(3)(3/2 + 5 + (5.94 - 1.01))^2\right] + \left[\frac{1.06(5.94^3)}{12} + 1.06(5.94)\left(\frac{5.94}{2} - 1.01\right)^2\right] + \left[\frac{11.6(1.91^3)}{12} + 11.6(1.91)\left(\frac{1.91}{2} + 1.01\right)^2\right]$

$(13.5 + 2352) + (18.5 + 24.19) + (6.74 + 85.55) = \boxed{1933\text{ in}^4}$

(8)

FINAL REPORT	MODIFIED W18 x 211 DEFLECTION	January 24, 2010

DEFLECTION & CAMBER CALCULATION for modified 18x211

$\phi M_p = 985$ k·ft $W_u = 8.448$ k/ft

$M_u = 950$ k·ft

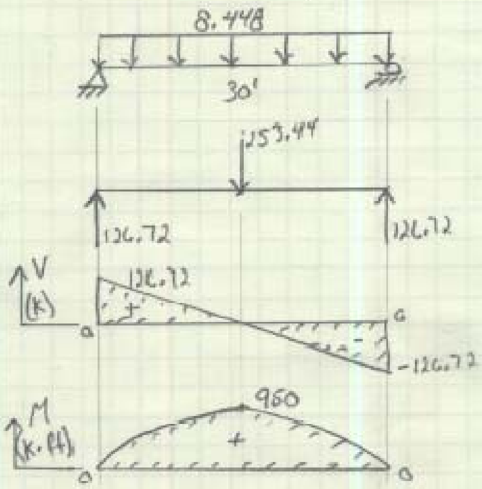
$\Delta_{max} = L/360$ for LL $= 1''$

$L/360$ for construction load $= 1''$

$L/240$ for Total $= 1.5''$

Assuming girders are simply supported (shear connections)

max tributary area = 30'



DOES NOT INCLUDE SELF WEIGHT OF GIRDER

$V_u = 127$ k

$$\Delta_{max} = \frac{5 W_u l^4 (1728)}{384 EI} = \frac{5(8.448)(30^4)(1728)}{384(29000)(1933)} = 2.75''$$

$2.75'' > 1.5''$ ∴ CAMBER IS REQUIRED

CAMBER GIRDERS 1.00" for construction loading

$$\Delta_{CL} = \frac{5 W_u l^4 (1728)}{384 EI} = \frac{5(2.97)30^4(1728)}{384(29000)\ 1933} = 0.966''$$

$W_u = (68+25)(30')/1000 = 2.8$ k/ft + self w $= 2.8 + 0.166 = 2.97$

$$\Delta_{LL} = \frac{5 W_u l^4 (1728)}{384 EI} = \frac{5(3.84)30^4(1728)}{384(29000)\ 1933} = 1.25''$$

$W_u = 80(1.6)(30') = 3.84$

(9)

MODIFIED W18X211 | $\bar{y}$ + ENA | January 30, 2010

$$\bar{y} = \frac{3''(6'')(15.85''-1.5'') + 10.94''(1.06'')\left(\frac{10.94''}{2} + 1.91''\right) + 1.91''(11.6'')\left(\frac{1.91''}{2}\right)}{3''(6'') + 16.94''(1.06'') + 1.91''(11.6'')}$$

$$= \frac{258.3 + 85.58 + 21.159}{18 + 11.596 + 22.156} = \frac{365.039}{51.755}$$

$= 7.053''$ up from the bottom of the beam

$$I_x = \left(\frac{6(3^3)}{12} + 6(3)(5.797+1.5)^2\right) + \left(\frac{1.06(16.94^3)}{12} + 1.06(10.94)(0.327^2)\right)$$

$$+ \left(\frac{11.6(1.91^3)}{12} + 11.6(1.91)(6.955+5.143)^2\right)$$

$$= 13.5 + 958.43 + 115.658 + 1.24 + 20.4 + 823.88$$

$$= 1933\ \text{in}^4$$

Modified
W18X 211

COMPOSITE ACTION (M_p) January 30, 2010

PNA CALCULATED FROM C = T

$T_f = 11.6''(1.91'')(50ksi) = 1107.8k$ (T)

$T_w = 1.06''(5.94'')(50ksi) = 314.82k$ (T)

$C_{bar} = 3''(6'')(50ksi) = 900k$ (C)

$C_{c\,17''} = 17''(0.85)(4)(4.06'') = 234.7k$ (C)

$C_{c\,23''} = 287.92/(23''(0.85)(4)) = 3.68'' < 5'' \therefore ok$

$$\begin{array}{r} 1107.8 \\ +314.82 \\ \hline 1422.62 \\ -900.00 \\ \hline 522.62 \\ -234.7 \\ \hline 287.92 \end{array}$$

$M_p = 1107.8(\frac{1.91''}{2} + 5.94'' + 1.32'') + 314.82(\frac{5.94''}{2} + 1.32'') + 900(\frac{3''}{2} + 3.68'')$

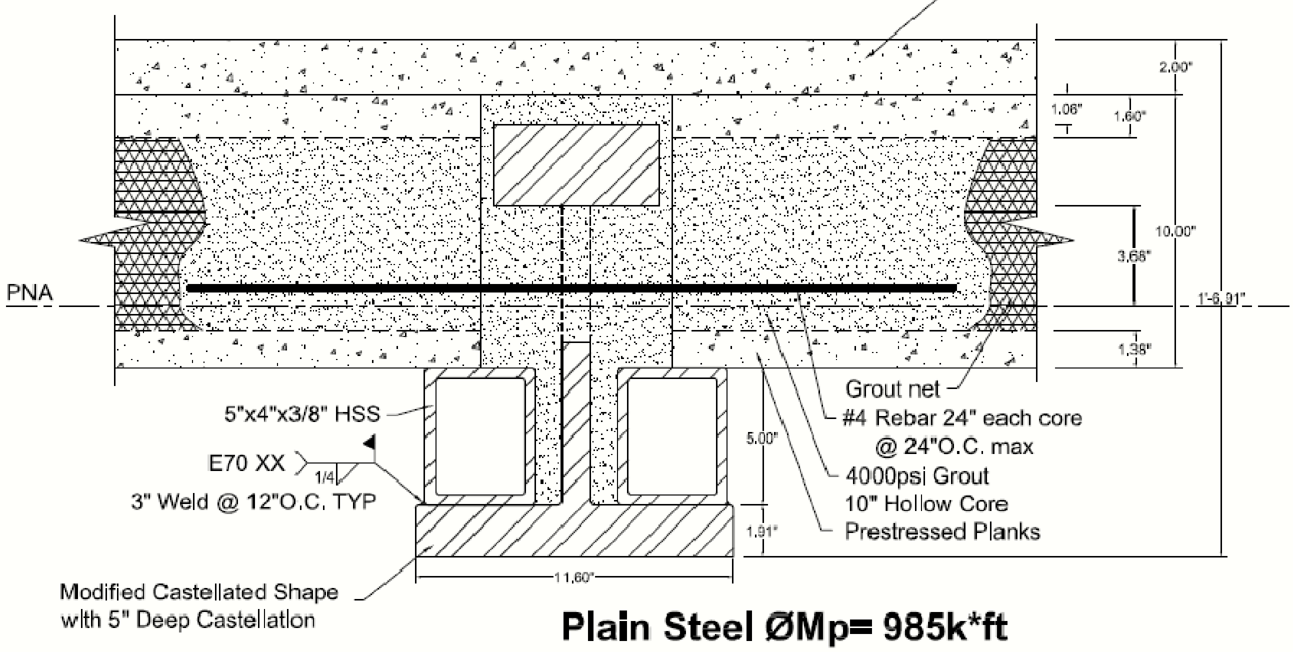
$+ 234.7(\frac{4.06}{2} + 3.68) + 384.62(\frac{3.68}{2})$

$= 9101 + 1351 + 4662 + 1340 + 707$

$= 17161 \; k \cdot in = 1430 \; k \cdot ft$

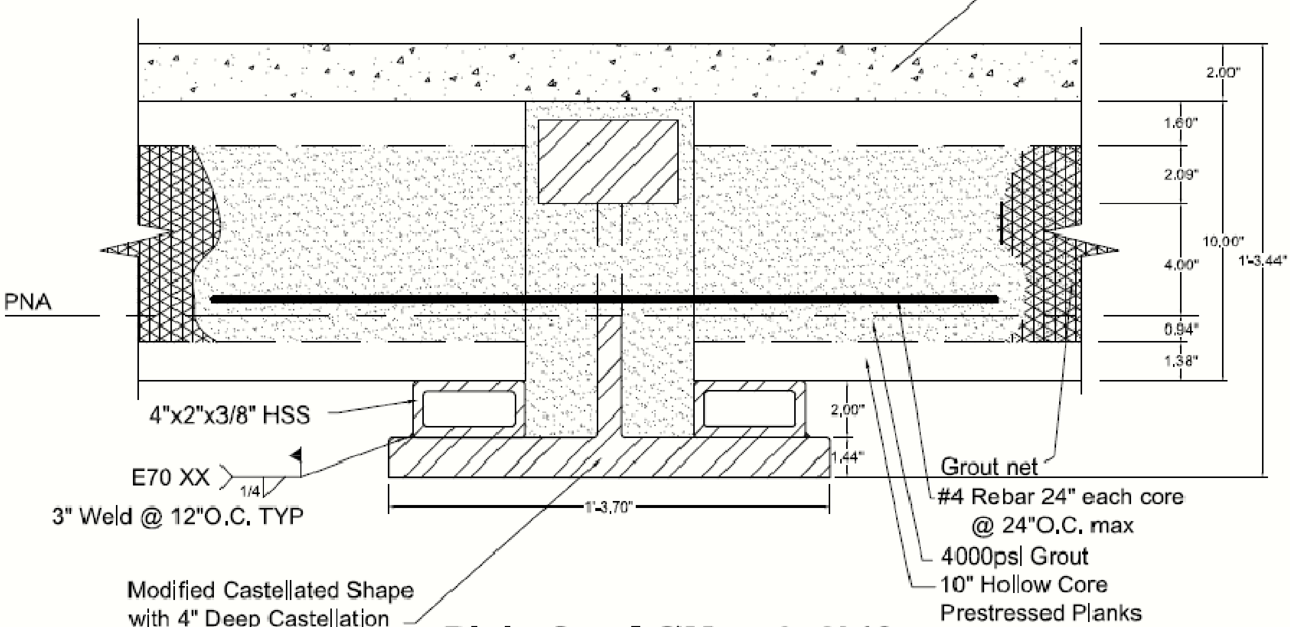
$\boxed{\phi M_{pc} = 1287 \; k \cdot ft}$ COMPOSITE

Modified W18x211 with 6"x3" Top Bar



Plain Steel ØMp= 985k*ft
Composite ØMp= 1287k*ft

Modified W14x193 with 5"x3" Top Bar

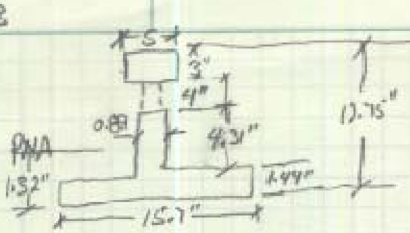


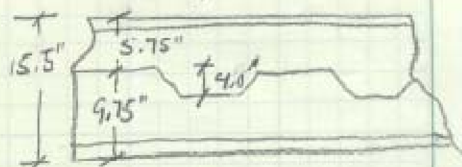
Plain Steel ØMp= 652k*ft
Composite ØMp= 877k*ft

MODIFIED
W 14 x 193

January 10, 2010

$d = 15.5''$
$t_w = 0.89''$
$b_f = 15.7''$
$t_f = 1.44''$





(b_f - bar width - ½" spacing each side)/2 = bearing from end of b_f

$(15.7'' - 5'' - 2(½''))/2 = 4.85''$

assuming $f_y = 50$ ksi for bar and W

PNA

$3''(5'') + 4.31''(0.89'') + 15.7(1.44'')$

$15 + 3.8359 + 22.608 = 41.4439 \text{ in}^2$

$41.4439/2 = 20.722 \text{ in}^2$ ∴ PNA is in the bottom flange

$15.7''(x'') = 20.722 \text{ in}^2$

$x = 1.32''$ up from the bottom of the bottom flange

$C_{tf} = 3''(5'')\,50\text{ksi} = 750\text{k}$

$C_w = 4.31''(0.89'')\,50\text{ksi} = 191.8\text{k}$

$C_{bf} = 15.7''(1.44''-1.32'')\,50\text{ksi} = 94.2\text{k}$

$T_{bf} = 15.7''(1.32'')\,50\text{ksi} = 1036.2\text{k}$

$M_p = 750(3/2 + 4 + 4.31 + 0.12) + 191.8(\frac{4.31}{2} + 0.12) + 94.2(\frac{0.12}{2}) + 1036.2(\frac{1.32}{2} + 0.12)$

$= 7447.5 + 436.345 + 5.652 + 808.236$

$= 8697.7 \text{ k·in}$

$= 724.8 \text{ k·ft}$ Non composite

$\boxed{\phi M_p = 652 \text{ k·ft}}$

weight of bar
= 51 lb/ft

weight of beam =
$\frac{193}{2} + 51 = 149.5$ lb/ft

16

MODIFIED W14 x 193 | I_x, Δ + ENA | January 31, 2010

$$\bar{y} = \frac{3''(5'')(12.75''-15'') + 8.31''(0.89'')\left(\frac{8.31''}{2}+1.44''\right) + 1.44''(15.7'')\left(\frac{1.44''}{2}\right)}{3''(5'') + 8.31''(0.89'') + 1.44''(15.7'')}$$

$$= \frac{168.75 + 41.38 + 16.278}{15 + 7.396 + 22.608} = \frac{226.41}{45.004}$$

$= 5.031''$ up from the bottom of the beam

$$I_x = \left(\frac{5''(3'')^3}{12} + 5''(3'')(1.5''+4.719'')^2\right) + \left(\frac{15.7''(1.44'')^3}{12} + 15.7''(1.44'')(0.72''+3.591'')^2\right)$$

$$+\left(\frac{0.89''(8.31'')^3}{12} + 0.89''(8.31'')(4.155''-3.591'')^2\right)$$

$$= 11.25 + 580.14 + 3.91 + 420.16 + 42.56 + 443.76$$

$$= 1502 \text{ in}^4$$

$$\Delta_{CL} = \frac{5(2.97+0.1465)\,30^4(1728)}{384\,(29000)\,(1502)} = 1.31''$$

CAMBER $\Rightarrow 1.31(3/4) = 0.98 \Rightarrow \boxed{1''}$ @ 30' Long

<u>PNA from C = T</u>

$C_{bar} = 750\ k$

$T_f = 15.7(1.44)50 = 1130.4\ k$

$T_w = 0.89(4.31)50 = 191.8\ k$

$C_{c\,19''} = 0.85(4)(19'')(3.69) = 238.37k$

$C_{c\,23''} = 23''(4.27'')3.4 = 333.83$

$1130.4 + 191.8 - 750 = 572.2\ k$

Needed from joist + plank

$572.2 - 238.37 = 333.826$

$333.83/(23(4)(0.85)) = 4.27''$

$\therefore$ PNA

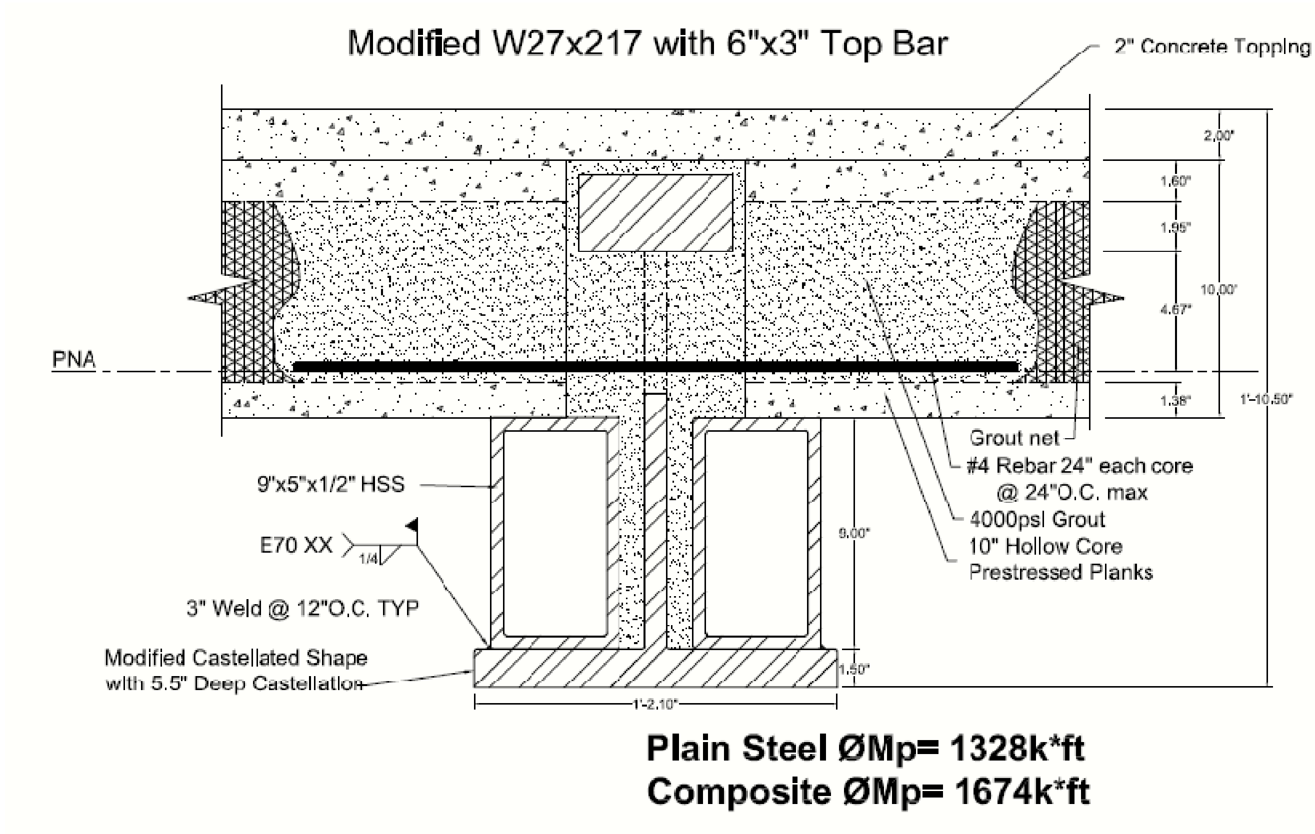
$$M_p = 750\left(\tfrac{3}{2}+4\right) + 1130.4\left(\tfrac{1.44}{2}+4.22\right) + 191.8\left(\tfrac{4.31}{2}\right) + 142.12\left(\tfrac{2.09}{2}+4\right) + 368\left(4/2\right)$$

$$= 4125 + 5697 + 413 + 717 + 736 = 11688\ k\cdot in$$

$$= 974\ k\cdot ft$$

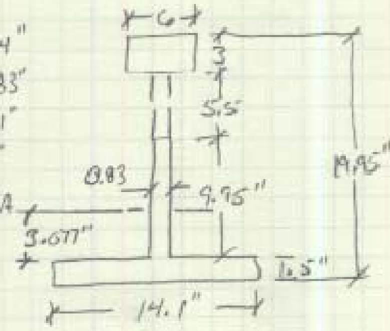
$\boxed{\phi M_p = 876.6\ k\cdot ft}$ COMPOSITE

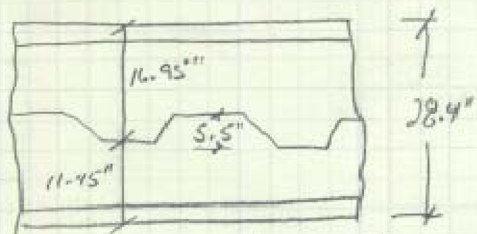
17


Modified W27x217 with 6"x3" Top Bar
2" Concrete Topping
PNA
2.00"
1.60"
1.95"
4.67"
1.38"
10.00"
1'-10.50"
Grout net
#4 Rebar 24" each core
@ 24"O.C. max
4000psi Grout
10" Hollow Core
Prestressed Planks
9"x5"x1/2" HSS
E70 XX
1/4
3" Weld @ 12"O.C. TYP
Modified Castellated Shape
with 5.5" Deep Castellation
9.00"
1.50"
1'-2.10"
Plain Steel ØMp= 1328k*ft
Composite ØMp= 1674k*ft

MODIFIED W27×217 | M_p Non Composite | January 31, 2010

$d = 28.4"$
$t_w = 0.83"$
$b_f = 14.1"$
$t_f = 1.5"$





$28.4/2 + 5.5/2 = 16.95"$

bearing for planks
$(14.1" - 6" - 1")/2 = 3.55" > 2" \therefore$ ok

PNA of Bare Steel Beam

$3"(6") + 0.83"(9.95") + 14.1"(1.5") = 18 + 8.2585 + 21.15 = 47.409\ in^2$

$47.409/2 = 23.704 > 21.15 \therefore$ PNA is in the web

$23.704 - 21.15 = 2.554$

$2.554/0.83 = 3.077"$ up from the top of the bottom flange

$C_{bar} = 3(6)(50) = 900$

$C_w = 0.83(9.95 - 3.077)(50) = 285.23$

$T_w = 0.83(3.077)(50) = 127.7$

$T_f = 14.1(1.5)(50) = 1057.5$

$M_p = 900(3/2 + 5.5 + 6.873) + 285.23(\frac{6.873}{2}) + 127.7(\frac{3.077}{2}) + 1057.5(\frac{1.5}{2} + 3.077)$

$= 12486 + 980 + 196 + 4047 = 17709\ k \cdot in$

$= 1476\ k \cdot ft$

$\boxed{\phi M_p = 1328\ k \cdot ft \text{ NON COMPOSITE}}$

$$ENA \rightarrow \bar{y} = \frac{6"(3")(18.45") + 0.83"(15.45")(15.45/2 + 1.5) + 14.1"(1.5")(1.5"/2)}{18 + 12.824 + 21.15}$$

$$= \frac{332.1 + 118.3 + 15.86}{51.974} = \frac{466.263}{51.974}$$

$= 8.97"$ up from the bottom of the beam

Weight of bar
$= 61.25\ lb/ft$

Weight of beam
$= \frac{217}{2} + 61.25$
$= 169.75\ lb/ft$

(19)

MODIFIED W27 X 217 | I_x, Δ_{CL}, ϕM_{pc} | January 31, 2010

$$I_x = \left(\frac{6(3^3)}{12} + 6(3)(1.5+5.5+2.48)^2\right) + \left(\frac{0.83(15.45^3)}{12} + 0.83(15.45)(0.255^2)\right)$$

$$+\left(\frac{14.1(1.5^3)}{12} + 14.1(1.5)(0.75+7.47)^2\right) = 13.5 + 1618 + 255 + 1 + 4 + 1429$$

$$= 3321\ in^4$$

$$\Delta_{CL} = \frac{5(2.97+0.170)(30^4)(1728)}{384(29000)\,3321} = 0.594''$$

for l = 30' Camber 1/2"
for l = 32' Camber 3/4"

PNA from C = T for composite action

T_f = 14.1"(1.5")(50 ksi) = 1057.5 k
T_w = 0.83"(9.95")(50 ksi) = 412.93 k
C_{bar} = 3"(6") (50 ksi) = 900 k
$C_{c\,17''}$ = 3.55"(17") 4(0.85) = 205.2 k
$C_{c\,23''}$ = 23"(4.67") 4(0.85) = 365.24 k

1057.5 + 412.93 − 900
= 570.43 k
Needed from grout + plank
570.43 − 205.2 = 365.24 k
365.24 / ((23)(4)(0.85)) = 4.67
4.67" < 5.5" ∴ ok
4.67" down from the bottom of the bar

$$M_p = 1057.5\left(\tfrac{1.5}{2} + 9.95 + 6.83\right) + 412.93\left(\tfrac{9.95}{2} + 6.83\right) + 900\left(\tfrac{3}{2} + 4.67\right)$$

$$+ 205.2\left(\tfrac{3.55}{2} + 4.67\right) + 365.24\left(\tfrac{4.67}{2}\right)$$

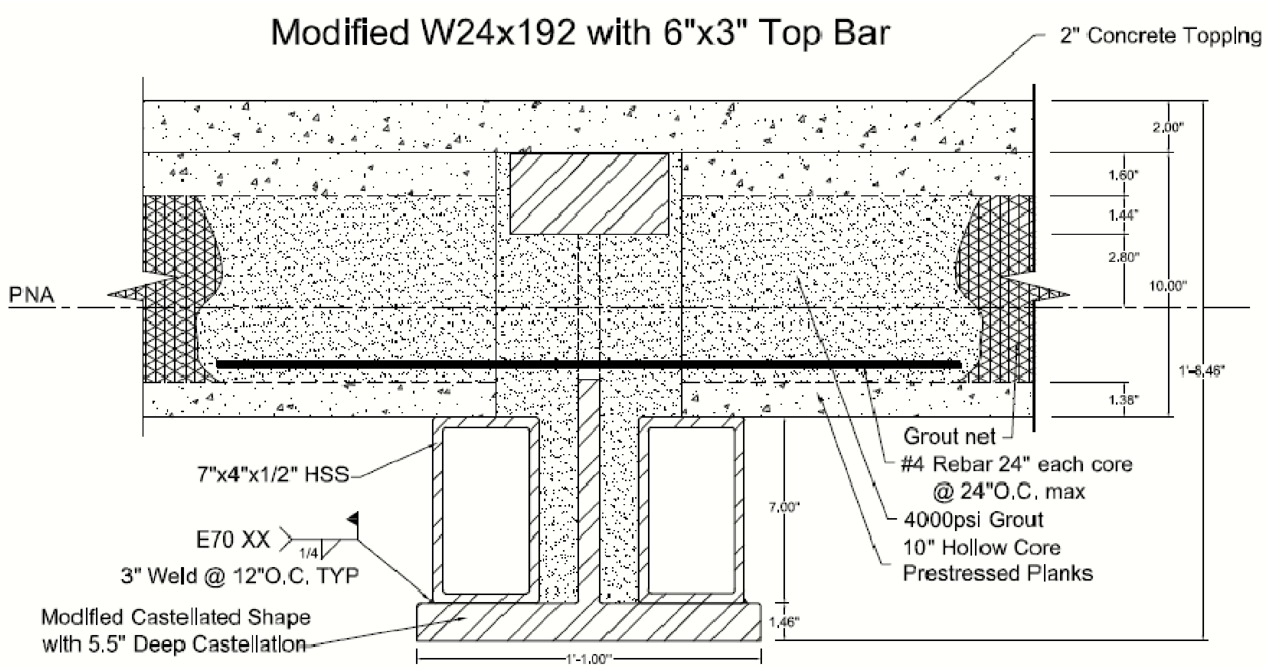
$$= 12193 + 2397 + 5553 + 1322 + 853$$

$$= 22318\ k\cdot in$$

$$= 1860\ k\cdot ft$$

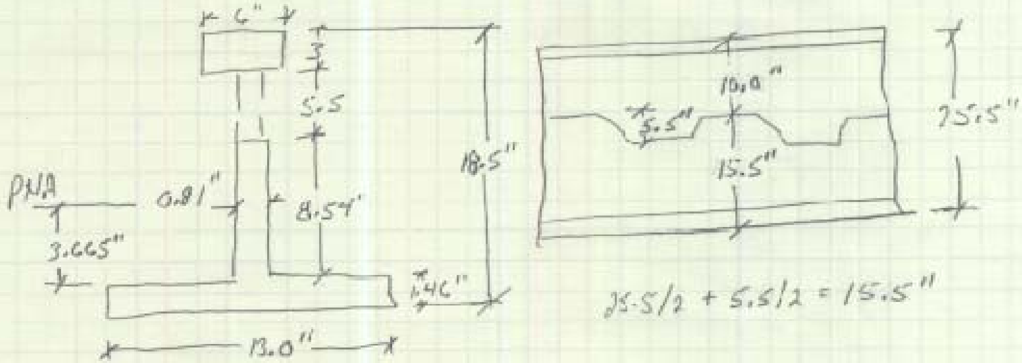
$\boxed{\phi M_{pc} = 1674\ k\cdot ft}$ COMPOSITE

20



Plain Steel ØMp= 1171k*ft
Composite ØMp= 1403k*ft

Modified W24x192 | Non composite ϕM_p | January 31, 2010



$25.5/2 + 5.5/2 = 15.5''$

PNA

$3''(6'') + 0.81''(8.54'') + 13''(1.46'')$

$18 + 6.9174 + 18.98 = 43.897$

$43.897/2 = 21.949 > 18.98$ ∴ PNA is above bottom flange

$21.9487 - 18.98 = 2.9687 \Rightarrow 2.9687/0.81 = 3.665''$ up from flange

$C_{bar} = 3''(6'')(50ksi) = 900k$

$T_f = 13''(1.46'')(50ksi) = 949\ k$

$T_w = 0.81''(3.665'')(50ksi) = 148.43\ k$

$C_w = 0.81''(8.54''-3.665'')(50) = 197.44\ k$

$M_p = 900(3/2 + 5.5 + 4.875) + 949(\frac{1.46}{2} + 3.665) + 148.43(\frac{3.665}{2}) + 197.44(\frac{4.875}{2})$

$= 10687.5 + 4170.8 + 272 + 481.3 = 15611\ k \cdot in$

$= 1301\ k \cdot ft$

$\boxed{\phi M_p = 1171\ k \cdot ft}$ Non composite

Weight of bar $= 61.25\ lb/ft$

Weight of beam $= \frac{192}{2} + 61.25 = 157.25\ lb/ft$

$ENA \Rightarrow \bar{y} = \frac{3(6)(17) + 14.04(0.81)(8.48) + 13(1.46)(0.73)}{3(6) + 14.04(0.81) + 13(1.46)}$

$= \frac{306 + 96.438 + 13.855}{18 + 11.3724 + 18.98} = \frac{416.2934}{48.3524}$

$= 8.61''$ up from the bottom

22

Modified W 24 × 192 | I_x Δ ϕM_{pc} | January 31, 2010

$$I_x = \left(\frac{6(3^3)}{12} + 6(3)(1.5+5.5+1.39)^2\right) + \left(\frac{0.81(14.04^3)}{12} + 0.81(14.04)(0.06)^2\right)$$

$$+\left(\frac{13.0(1.46^3)}{12} + 13(1.46)\left(\frac{1.46}{2}+7.15\right)^2\right)$$

$$= 13.5 + 1267 + 187 + 0 + 3.4 + 1173$$

$$= 2644 \text{ in}^4$$

$$\Delta_{CL} = \frac{5(2.97+0.157)(30^4)(1728)}{384(29000)\,2644} = 0.74''$$

CAMBER ⇒ 5/8"

PNA from C-T for composite action

$C_{bar} = 3''(6'')(50\text{ksi}) = 900k$

$T_f = 13(1.46)(50) = 949k$

$T_w = 0.81(8.54)(50) = 345.87k$

$C_{c\,17''} = 3.04''(17'')(4)(0.85) = 175.7k$

$C_{c\,23''} = (23'')(2.8'')4(0.85) = 219.16$

$949+345.87-900 = 394.87k$

Needed from grout + plank

$394.87 - 175.7 = 219.16\ k$

$219.16/(23(4)(0.85)) = 2.80''$

$$M_{pc} = 900(1.5+2.8) + 949\left(\frac{1.46}{2}+8.54+2.7\right) + 346(4.27+2.7) + 175.7\left(\frac{3.04}{2}+2.8\right)$$

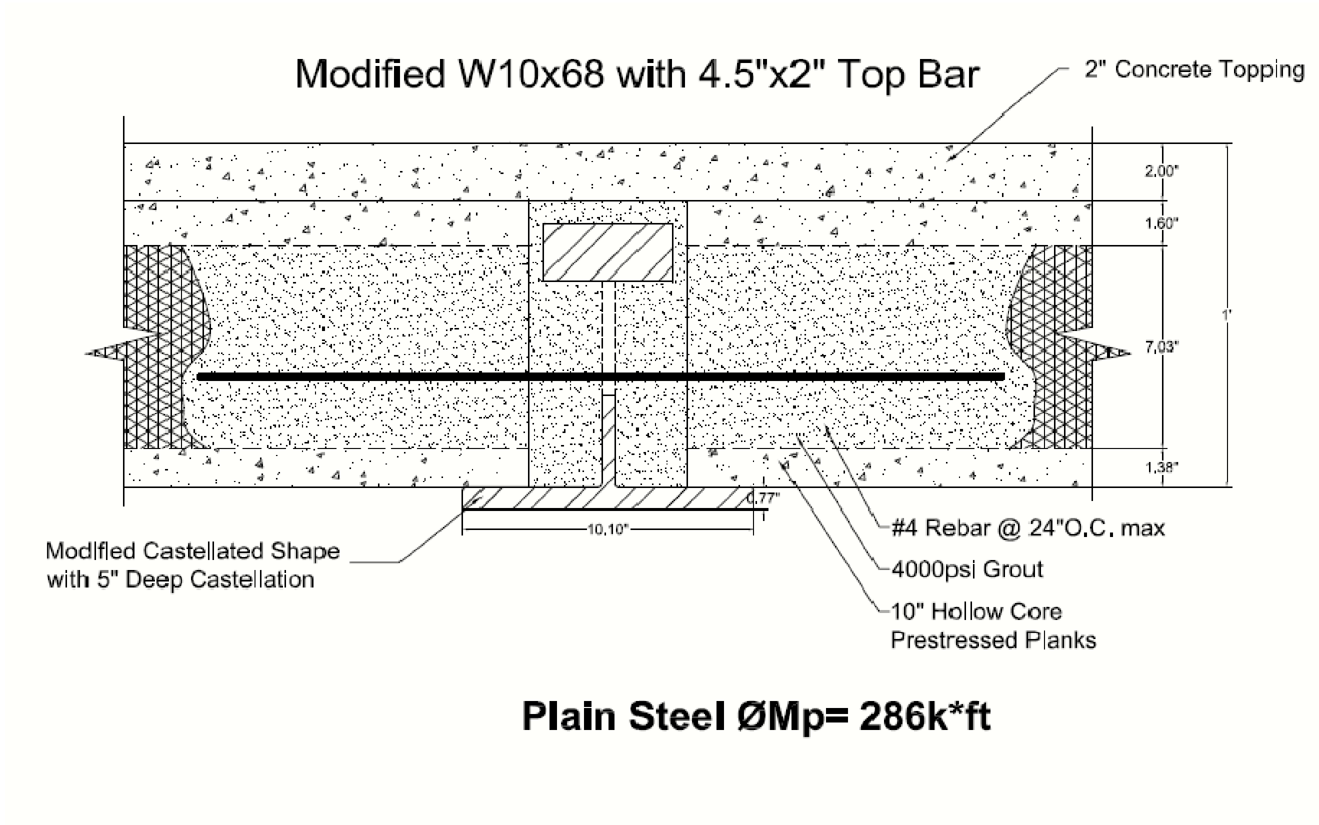
$$+219.16\left(\frac{2.8}{2}\right) = 3870 + 11359 + 2412 + 761 + 307$$

$$= 18709\ k\cdot in$$

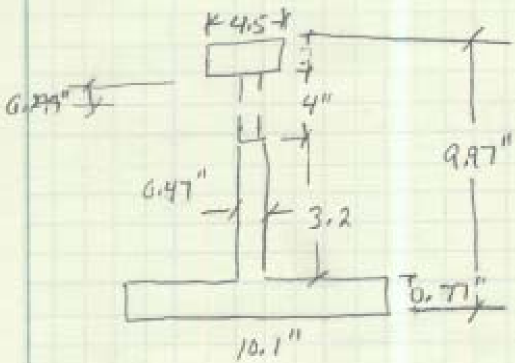
$$= 1559\ k\cdot ft$$

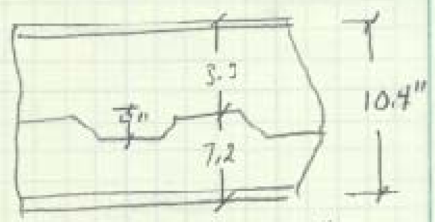
$\boxed{\phi M_{pc} = 1403\ k\cdot ft}$ COMPOSITE

23


Modified W10x68 with 4.5"x2" Top Bar
2" Concrete Topping
2.00"
1.60"
7.03"
1'
1.38"
0.77"
10.10"
#4 Rebar @ 24"O.C. max
4000psi Grout
10" Hollow Core Prestressed Planks
Modified Castellated Shape with 5" Deep Castellation
Plain Steel ØMp= 286k*ft

MODIFIED W10 X68 | SHORT SPANS | February 02, 2010





$10.4/2 + 4/2 = 7.2''$

PNA

$2(4.5) + 0.47(3.2) + 10.1(.77)$

$9 + 1.504 + 7.777 = 18.281$

$18.281/2 = 9.1405$

$9.1405 - 9 = 0.1405 \quad 0.1405/0.47 = 0.299''$ ↓ from bottom of bar

$C_{bar} = 2(4.5)50 = 450$

$C_f = 10.1(0.77)50 = 388.85$

$t_w = 0.47(3.2)50 = 75.2$

$M_p = 450(1.299) + 388.85(7.286) + 75.2(5.301)$

$= 584.55 + 2833 + 398.63 = 3816$ k·in

$= 318$ k·ft

$\phi M_p = 286$ k·ft

ENA ⇒ $\bar{y} = \dfrac{2(4.5)(8.97) + 7.2(0.47)(4.37) + 10.1(0.77)(0.385)}{2(4.5) + 7.2(0.47) + 10.1(0.77)}$

$= \dfrac{80.73 + 14.79 + 2.99}{9 + 3.384 + 7.777} = \dfrac{98.51}{20.161} = 4.886''$ up from bottom

$I_x = \left(\dfrac{4.5(2^3)}{12} + 4.5(2)(5.104^2)\right) + \left(\dfrac{0.47(7.2^3)}{12} + 0.47(7.2)(0.516^2)\right) + \left(\dfrac{10.1(0.77^3)}{12} + 10.1(0.77)(4.50^2)\right)$

$= 3 + 234.46 + 14.62 + 1 + 0.38 + 157.55$

$= 411 \text{ in}^4$

(25)

MODIFIED W 10×68	SHORT SPANS	February 02, 2010

Δ_{CL} for 14' spans or shorter

$$\Delta_{CL} = \frac{5(2.97+0.0646)14^4(1728)}{384(29000)(411)}$$

$= 0.22 \text{ in}^4$

NO CAMBER

$M_u = 209 \text{ k}\cdot\text{ft}$

$\phi M_p > M_u$ No need to calculate composite action

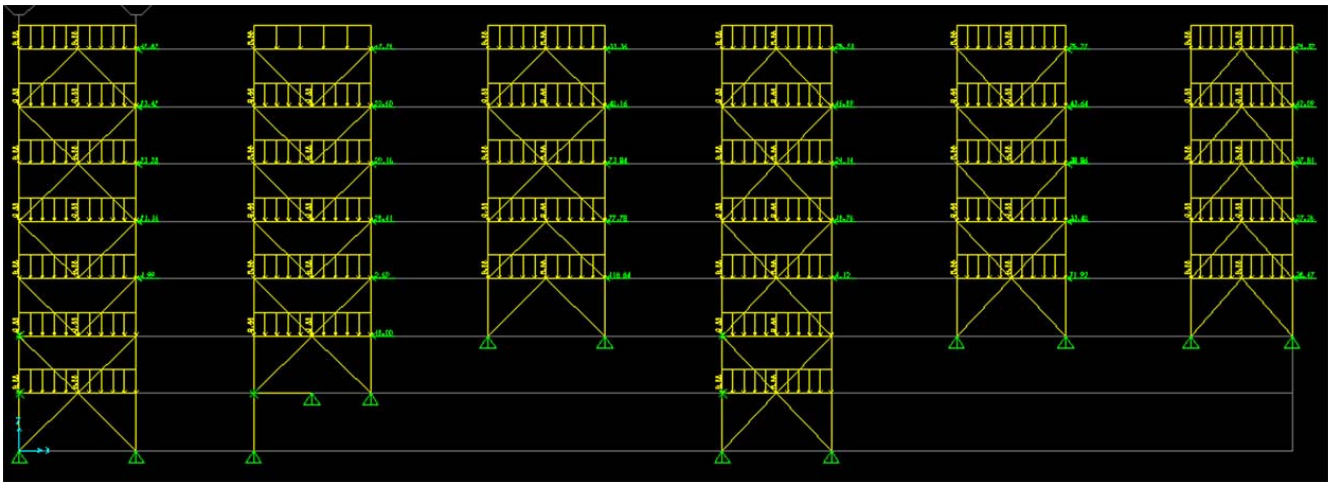
Weight of bar

$\frac{2}{12}\left(\frac{4.5}{12}\right)490 = 30.63 \text{ lb/ft}$

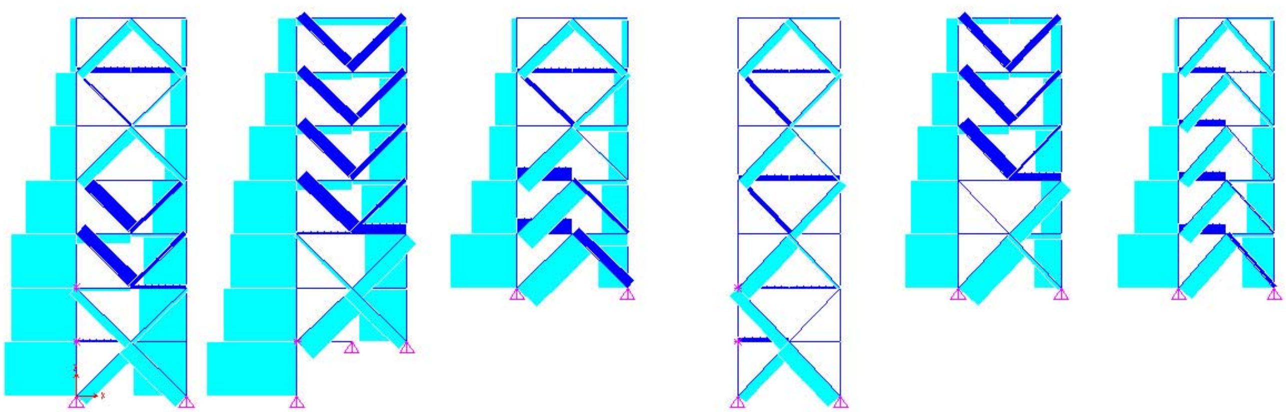
Weight of beam

$68/2 + 30.63 = 64.63 \text{ lb/ft}$

Appendix H: Connection Load Diagrams



Full gravity and lateral loading on members shown



Magnitude of axial loads due to the above loading

Dark Blue = Tension

Light Blue = Compression

Appendix J: Connection Designs and Calculations

GIRDER - GIRDER | SHEAR TAB CONNECTION | February 14, 2010 | 2

Plate : A572 GR 50

$l = 11.71''$

$l_{eh} = 2.0''$

$l_{ev} = 2.427''$

$a = 3.0''$

$F_u = 65$ ksi

$F_y = 50$ ksi

width = 5.0''

$t_p = 9/16''$

Bolts: $n = 4$

A 490

$\phi R_n = 44.2$ k / bolt

1'' ϕ

LIMIT STATE CHECKS:

Dimension limits

$2 \le \#\text{bolts} \le 12$ ∴ ok

$3 = a \le 3.5''$ ∴ ok

STANDARD HOLES ARE ASSUMED ∴ ok

$L_{eh} \geq 2 d_b \Rightarrow 2.0 = 2(1.0)$ ∴ ok

L_{ev} meets Table J3.4 ∴ ok

minimum edge distances

$t_p \le \frac{d_b}{2} + 1/16 = 0.5 + 0.0625 = 0.5625$

$= 9/16''$ ∴ ok

or

Beam: $t_w \le \frac{d_b}{2} + 1/16 = 9/16 = 0.5625$

$0.81 >$

Eccentricity can be ignored

W_m 24X192 - W_m 24X192 GIRDER- GIRDER	SHEAR TAB CONNECTION	February 14, 2010	3

LIMIT STATES: $V_u = 140$ kips

W_m 24X192 @ 28' Long and full DL + 125 psf LL

ϕV_n Rupture = 150 k

ϕV_n Yielding = 339^k ∴ OK

Bolt Shear ⇒ $4(44.2^k) = 176.8^k$ ∴ OK

Bolt Bearing on Plate

$\phi r_n = \phi\, 2.4\, F_u\, d_b\, t_p \geq 35.3$

$= 0.75(2.4)(65)(1)(0.5625)$

$= 65.81$ ∴ OK

Bolt Bearing on Beam

$\phi r_n = \phi\, 2.4\, F_u\, d_b\, t_w \geq 35.3$

$= 0.75(2.4)(65)(1)(0.81)$

$= 94.77$ ∴ OK

Bolt Tear out @ edge

$\phi\, 1.2\, F_u\, L_c\, t_p$

$0.75(1.2)(65)(2.105 - 1.125/2)(0.5625) = 50.75^k$

Bolt Tear out other bolts

$\phi\, 1.2\, F_u\, L_c\, t_p$

$0.75(1.2)(65)(2.50 - 1.125)(0.5625) = 45.25^k$

Bolt T/O, Bearing + Shear combined

Shear is the lowest always

∴ 176.8^k ∴ OK

Wm 24X92 – Wm 24X192 GIRDER TO GIRDER | SHEAR TAB CONNECTION | February 14, 2010 | 4

LIMIT STATES

Block Shear in the Shear Plate will control over the beam

Block Shear

rupture   yielding

Plate $\Rightarrow \phi[0.6 F_u A_{nv} + U_{bs} F_u A_{nt}] \leq \phi[0.6 F_y A_{gv} + U_{bs} F_u A_{nt}]$

$F_u = 65$

$F_y = 50$

$A_{gv} = 11.71''(0.5625) = 6.59 \text{ in}^2$

$A_{nv} = A_{gv} - (\# \text{ bolt holes}(thk)(d_b + 1/8)$

$= 6.59 - (4(0.5625)(1.125))$

$= 4.06 \text{ in}^2$

$A_{nt} = t_p(l_{eh} - \tfrac{1}{2} d_b) = 0.81(2 - 1.125/2) = 1.164$

$U_{bs} = 1.0$

beam + plate have the same F_y + F_u ∴
Shear rupture in the plate will control over block shear

Plate

Shear yielding $\Rightarrow \phi\, 0.6 F_y A_{gv}$

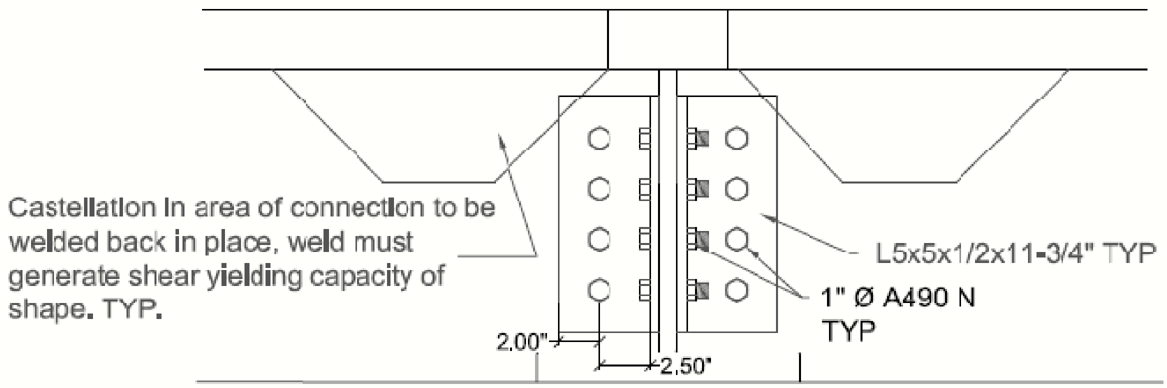
$= 1(0.6)(50)(6.59) = 198^k > 160$ ∴ ok

Shear Rupture $\Rightarrow \phi\, 0.6 F_u A_{nv}$

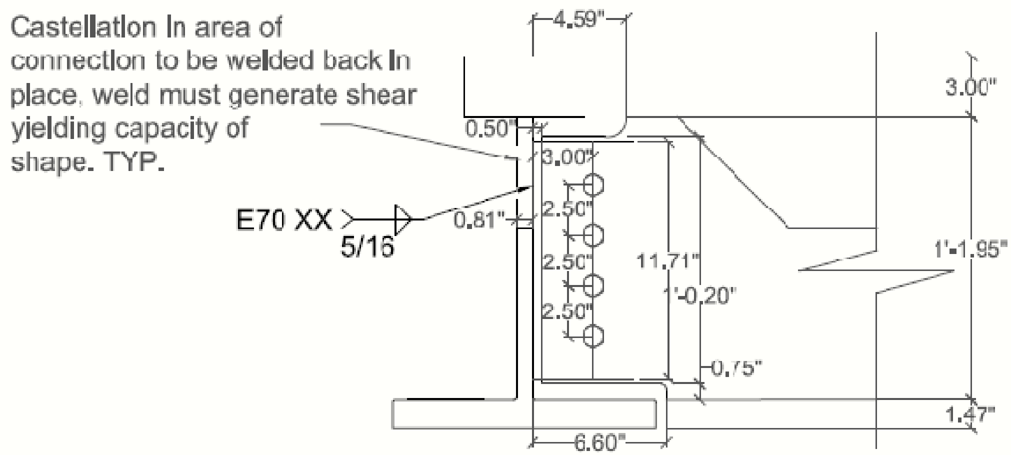
$= 0.75(0.6)(65)(4.06) = 119 < 160$

NO GOOD

WILL HAVE TO TRY A DOUBLE ANGLE



Modified W24x192 Connected to Modified W24x192
189 kip Capacity @ Connection



W_m 24X192 - W_m 24X192 GIRDER - GIRDER	DOUBLE ANGLE CONNECTION	February 15, 2010	5

BOLTED - BOLTED

Angles : A 36
5 x 5 x 1/2
l = 11.71"
A = 4.75 in²
$\bar{y} + \bar{x}$ = 1.42"
F_u = 58 k
F_y = 36 k
l_{eh} = 2"
l_{ev} = 2.11"
a = 3"
$V_{u_{max}}$ = 140 kips

Bolts : 7/8" ϕ
A 490 N
n = 4
ϕr_n = 54.1 k double shear
27.1 k single shear

Bolts in beam:

Bolt Shear in beam ⇒ $4(54.1) = 216.4^k > 140$ ∴ OK

Bolt Bearing on beam ⇒ $\phi\, 2.4\, F_u\, d_b\, t_b = 0.75(2.4)(65)(1")\,0.81 = 94.77^k$

Bolt Tear out on beam edge ⇒ $\phi\, 1.2\, F_u\, L_c\, t_b = 0.75(1.2)(65)(2.11 - 1/2)(0.81) = 76.3^k$

Bolt Tear out beam others ⇒ $\phi\, 1.2\, F_u\, L_c\, t_b = 0.75(1.2)(65)(2.5-1)(0.81) = 71.1^k$

Bolt Shear Controls on all ∴ OK

Beam:

Shear Yielding ⇒ $\phi\, 0.6\, F_y\, A_{gv} = 1.0(0.6)(50)(11.96)(0.81) = 339^k$ OK

Shear Rupture ⇒ $\phi\, 0.6\, F_u\, A_{nv} = 0.75(0.6)(65)(0.81)(11.96 - 4(1")) = 188.6^k$ OK

Block Shear

$\phi[0.6 F_u A_{nv} + U_{bs} F_u A_{nt}] \le \phi[0.6 F_y A_{gv} + U_{bs} F_u A_{nt}]$

$0.6 F_u A_{nv} < 0.6 F_y A_{gv}$ ∴

$\phi[0.6 F_u A_{nv} + U_{bs} F_u A_{nt}]$ ⇒ $\phi = 0.75$
$U_{bs} = 0.5$
$A_{nt} = t_b\,(a - d_b/2)$
$= 0.875\,(3 - 0.5) = 2.188$

$= 0.75(251.5 + 54.7)$
$= 230^k > 140$ ∴ OK

$W_m 24\times192$ - $W_m 24\times192$ GIRDER - GIRDER	DOUBLE ANGLE CONNECTION	February 15, 2010	6

Angles:

Shear Yielding $\Rightarrow \phi\, 0.6\, F_y A_{gv} = 1.0(0.6)(36)(11.71)(0.5)(2)$ # angles

$= 252.9^k > 140 \therefore$ OK

Shear Rupture $\Rightarrow \phi\, 0.6\, F_u A_{nv} \Rightarrow A_{nv} = A_{gv} - \#_{bolt}(d_b)t_a$

$11.71(0.5) - 4(1)(0.5)$

$= 3.855$

$= 0.75(0.6)(58)(3.855)2$

$= 201.2^k > 140 \therefore$ OK

Block Shear

$\phi[0.6 F_u A_{nv} + U_{bs} F_u A_{nt}] \leq \phi[0.6 F_y A_{gv} + U_{bs} F_u A_{nt}]$

$\phi = 0.75 \quad U_{bs} = 0.5 \quad A_{nv} = 3.855$

$\phi[0.6 F_u A_{nv} + U_{bs} F_u A_{nt}] \Rightarrow A_{nt} = t_a(l_{ev} - d_b/2)$

$= 0.5(2 - 0.5) = 0.75$

$= 0.75(0.6(58)(3.855)(2) + 0.5(58)(0.75)(2))$

$= 233.9^k > 140^k \therefore$ ok

Since THE BEAM IS DOUBLE COPED CHECK FOR COPED BEAM FLEXURE

Flexural Yielding $\Rightarrow \phi_b M_n = 0.9 F_y S_{net} \Rightarrow S_{net} = \dfrac{t_w h_o^2}{6}$

$= 0.9(50)(20.13) \qquad = \dfrac{0.81(12.21^2)}{6}$

$= 905.7$ k·in $\qquad = 20.13$

Double cope limitations

$c \leq 2d$

$d_c \leq 0.2d$

$d_{cb} = d_{ct}$ not met

$M_a = V_a \cdot e \Rightarrow e = 4.594$

$= 140(4.594) = 643 < 905.7 \therefore$ ok

Local Web Buckling $\Rightarrow \phi_b M_n = \phi F_{bc} S_{net} > M_a = 643$

$\therefore \phi F_{bc} S_{net} = \phi F_{cr} = \phi F_y Q \Rightarrow Q = 1$ for $\lambda \leq 0.7$

$= (1.34 - 0.486\lambda)$ for $0.7 < \lambda \leq 1.41$

$= (1.30/\lambda^2)$ for $\lambda > 1.41$

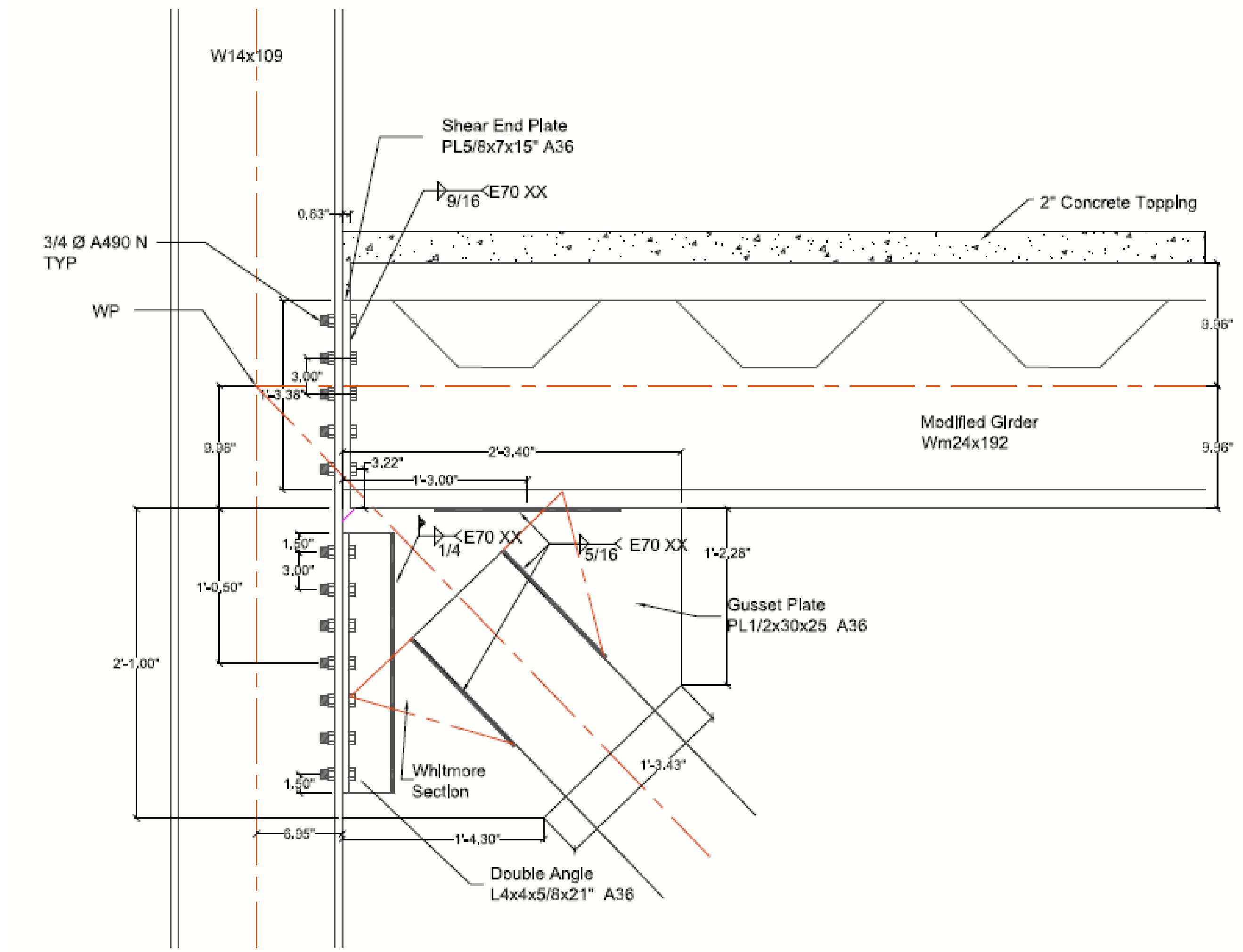
$$\lambda = \frac{h_o\sqrt{F_y}}{10 t_w\sqrt{475 + 280\left(\frac{h_o}{c}\right)^2}} = \frac{12.21\sqrt{50}}{10(0.81)\sqrt{475+280\left(\frac{12.21}{6}\right)^2}}$$

$= 0.264 \therefore Q = 1$

$\phi F_y = 0.9(50) = 45 \leq F_y \therefore$ OK

AISC pg. 9-7

Typical connection @ column and girder bracing location



DESIGN FORCES AND LENGTHS | CONNECTION COLUMN - BEAM - BRACE | February 16, 2010 | 1

ASSUMING NO ECCENTRICITIES

i.e. UNIFORM FORCE METHOD

$e_b = 9.9567''$ $e_c = 6.95''$

$P_u = 350^k$

$\theta = 44°$ $\tan\theta = 0.966$

$\tan\theta = \dfrac{e_c + \alpha}{e_b + \beta}$

$\alpha \Rightarrow$ assumed to be $15''$

$0.966 = \dfrac{6.95 + 15}{9.9567 + \beta}$

$\beta = 12.76''$

$\beta = 12.76''$

$r = \sqrt{(e_c + \alpha)^2 + (e_b + \beta)^2} = 31.59''$

$\dfrac{P_{uc}}{r} = \dfrac{V_{uc}}{\beta} \Rightarrow \dfrac{350}{31.59} = \dfrac{V_{uc}}{12.76}$

$V_{uc} = 141.37^k$

$\dfrac{V_{uc}}{\beta} = \dfrac{V_{ub}}{e_b} \Rightarrow \dfrac{141.37}{12.76} = \dfrac{V_{ub}}{9.9567}$

$V_{ub} = 110.31^k$

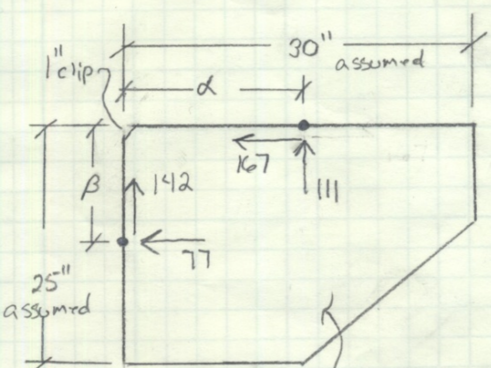
$\dfrac{H_{uc}}{e_c} = \dfrac{V_{uc}}{\beta} \Rightarrow H_{uc} = \dfrac{141.37}{12.76}(6.95)$

$H_{uc} = 77^k$

$\dfrac{H_{ub}}{\alpha} = \dfrac{V_{uc}}{\beta} \Rightarrow H_{ub} = \dfrac{141.37}{12.76}(15)$

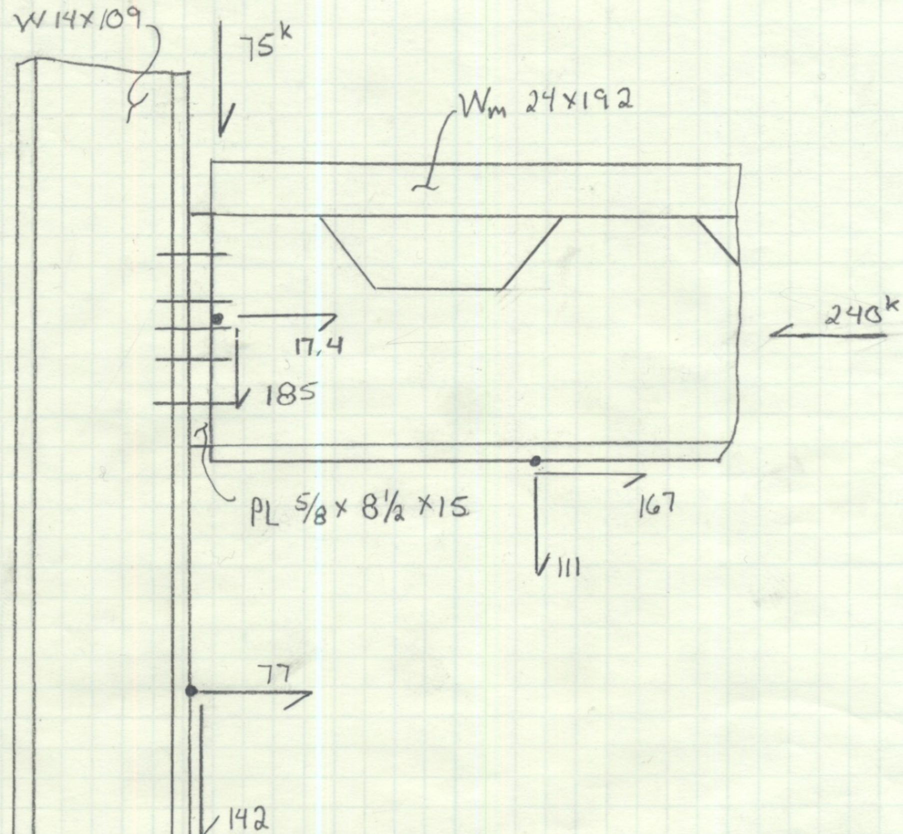
$H_{ub} = 166.19^k$

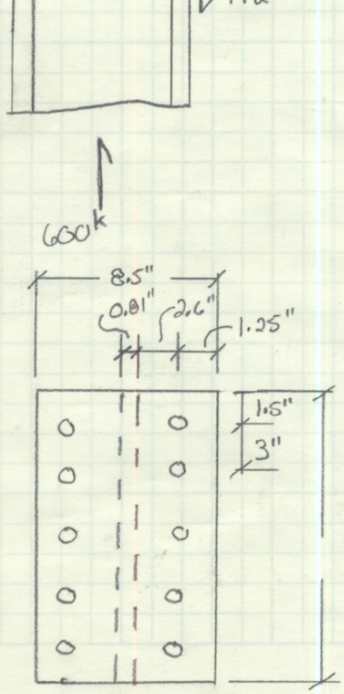
GUSSET REACTIONS



PL ½ × 30 × 25
A 36

DESIGN FORCES — CONNECTION COLUMN-BEAM-BRACE — February 16, 2010 — 2





GIRDER TO COLUMN REACTIONS

V_c = 185k (Shear)

H_c = 17.4k (Tension)

GUSSET TO COLUMN REACTIONS

V_{uc} = 142k (Shear)

H_{uc} = 77k (Tension)

GUSSET TO BEAM REACTIONS

V_{ub} = 111k (Tension)

H_{ub} = 167k (Shear)

LIMIT STATES | CONNECTION COLUMN - BEAM - BRACE | February 16, 2010 | 3

CHECK COLUMN TO GIRDER ; BOLTED END PLATE

$V_u = 185^k$ BOLTS : 3/4 Ø A490 N

GIRDER LIMIT STATES

$\phi V_n (k)$

Shear Yielding $\Rightarrow \Phi\, 0.6\, F_y\, h_o\, t_w$

484 $\phi V_n = 1(0.6)(50)(19.91)(0.81) = 484^k$

368 Shear Rupture $\Rightarrow \Phi\, 0.6\, F_u\, A_n$

$\phi V_n = 0.75(0.6)(65)(19.91 - 5(7/8))0.81 = 368^k$

329 Girder Web Strength @ weld

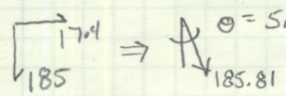
Plate length = 15 , $t_p = 5/8''$

$t_{weld} = 5/8 - 1/16 = 9/16'' = 0.5625''$

$\Phi V_n = \Phi\, 0.6\, F_u\, (l_p - 2 t_{weld})(t_{web})$

$= 0.75(0.6)(65)(15 - 2(0.5625))(0.81) = 329^k$

520^k Weld Rupture



$\theta = 5.35°$ $90 - \theta = 84.65°$

$\phi V_n = 1.392(l_p - 2 t_{weld})(\#_{welds})(t_{weld})(1 + 0.5 \sin^{1.5}(90-\theta))$

$= 1.392(15 - 2(0.5625))(2)(9)(1.497) = 520^k$

Prying on Plate will be checked with bolts

PLATE LIMIT STATES

Shear Yielding $\Rightarrow \phi\, 0.6\, F_y\, l_p\, t_p$ # shear planes

405 $\phi V_n = 1.0(0.6)(36)(15)(5/8)\,2 = 405$

Shear Rupture $\Rightarrow \phi\, 0.6\, F_u\, A_n$

$A_n = 2(l_p - \text{holes})\, t_p$

$= 2(15 - 5(7/8))(0.625) = 13.28\ in^2$

346 $\phi V_n = 0.75(0.6)(58)(13.28) = 346.6$

LIMIT STATES | CONNECTION COLUMN - BEAM - BRACE | February 16, 2010 | 4

COLUMN TO GIRDER CONT.

PLATE LIMIT STATES CONT.

320^k Plate Block Shear

$\phi R_n = \phi[0.6 F_u A_{nv} + U_{bs} F_u A_{nt}] \le \phi[0.6 F_y A_{gv} + U_{bs} F_u A_{nt}]$

$U_{bs} = 1.0$ for uniform tension

$A_{gv} = (l_p - l_{ev}) t_p \,\#_{\text{shear planes}} = 13.5(0.625)2 = 16.875\ in^2$

$A_{nv} = A_{gv} - (\text{holes}(t_p)) = 16.875 - (8(7/8)(0.625)) = 12.5\ in^2$

$A_{nt} = 2 t_p (l_{eh} - d_b/2) = 2(0.625)(1.25 - 3/4/2) = 1.094\ in^2$

$\phi R_n = 0.75[0.6(58)(12.5) + 58(1.094)] \le 0.75[0.6(36)(16.875) + 58(1.094)]$

$= 373.8 \le 320 \therefore 320$

226^k Plate strength @ weld

$\phi V_n = \phi\, 0.6\, F_u (l_p - 2t_{weld})(t_p)$

$= 0.75(0.6)(58)(15 - 2(0.5625))(0.625) = 226^k$

BOLT LIMIT STATES ⇒ Bolts connecting girder to column

$\boxed{199^k}$ Bolts: $3/4\phi$ A490 N

$\phi r_n = 19.9$ k/bolt in shear

37.4 k/bolt in tension

Bearing on plate ⇒ controls over column

$\phi R_n = \phi\, 2.4\, F_u t_p d_b$

$= 0.75(2.4)(58)(0.625)(3/4) = 48.93\ ^k/\text{bolt}$

Tear out on plate edge

$\phi R_n = \phi\, 1.2\, F_u (l_{ev} - d_h/2) t_p$

$= 0.75(1.2)(58)(1.25 - 7/8/2)(0.625) = 26.5\ k$

Tear out off other bolts on plate

$\phi R_n = \phi\, 1.2\, F_u (\text{bolt spacing} - d_h)(t_p)$

$= 0.75(1.2)(58)(3 - 7/8)(0.625) = 69.3$

Shear, bearing, T.O combined $= 10(19.9) = 199^k$

LIMIT STATES | CONNECTION COLUMN - BEAM - BRACE | February 16, 2010 | 5

COLUMN TO GIRDER CONT.

BOLT LIMIT STATES CONT.

CHECK PRYING @ Girder to Column

Calculate the Shear Stress in the bolts

$f_v = V_u / \#\text{bolts}(A_b) = 185/10(0.442) = 41.86 \text{ ksi}$

calculate the available tensile strength / bolt

$F_t' = 1.3F_{nt} - (F_{nt}/\phi F_{nv})f_v \Rightarrow F_{nt} = 113 \text{ ksi}$ AISC

$= 1.3(113) - (113/0.75(60))41.86 \quad F_{nv} = 60 \text{ ksi}$ J3.2

$= 41.78 \text{ k/bolt}$

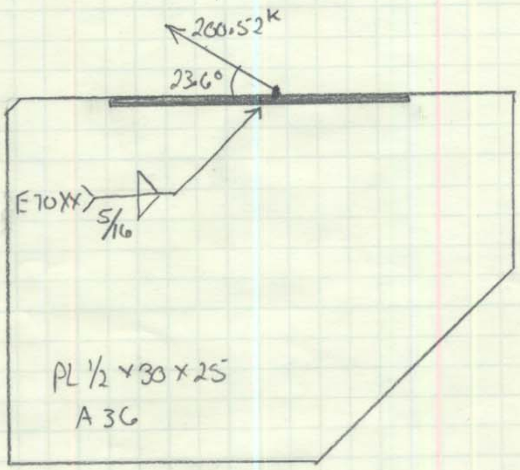
$T_u = 17.4 \text{ k}$

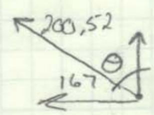
$r_{ut} = T_u / \#\text{bolts} = 17.4/16 = 1.74$

$\phi r_{nt} = \phi F_t' A_b = 0.75(41.78)(0.442) = 13.85 \text{ k/bolt}$

$\phi r_{nt} > r_{ut} \Rightarrow 13.85 > 1.74 \therefore \text{OK}$

CHECK GIRDER TO GOSSET $V_u = 201^k$





$\theta = 56.4°$

limitations

$3/16" < t_{weld} < 7/16$

$L_{weld} \geq 4\, t_{weld}$

LIMIT STATES	CONNECTION COLUMN - BEAM - BRACE	February 16, 2010	6

GIRDER TO GUSSET CONT. $V_u = 201^k$

Weld Rupture ⇒ assume 5/16" weld

$\boxed{288^k}$

$\phi R_n = 1.392(l_{weld})(\#\ welds)(t_{weld})(1.0+0.5\sin^{1.5}\theta) \geq 201$

$= 1.392(l_w)(2)(5)(1.38) = 201$

$l_{weld} \geq 10.46''$

USE 15" weld 5/16" long both sides

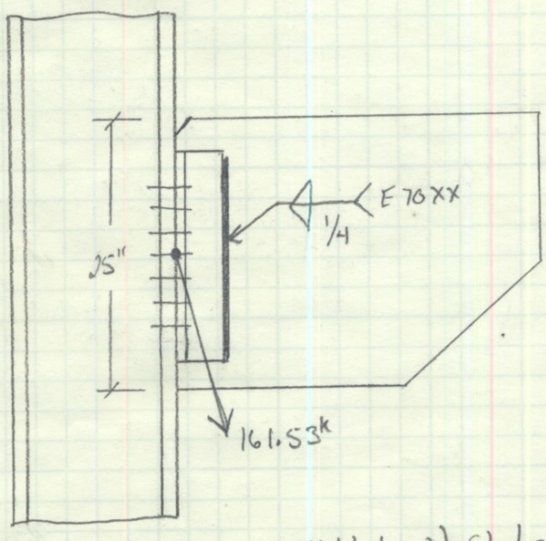
$\phi R_n = 1.392(15)(2)(5)(1.38) = 288^k$

391^k Base Metal Rupture ; 1/2 plate controls

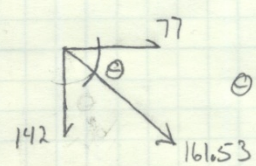
$\phi R_n = \phi\, 0.6\, F_u\, t_p\, l_{weld}\ \#welds$

$= 0.75(0.6)(58)(0.5)(15)(2) = 391.5^k$

CHECK COLUMN TO GUSSET $V_u = 162$



Angles: 4x4x5/8
A36
$A = 4.61\ in^2$
$l = 21''$
Bolts: (7) 3/4 Ø A490 N
Weld: 1/4"



$\theta = 51.5°$

Weld Limit States

Weld Rupture

307^k

$\phi R_n = 1.392(l_{weld})(\#\ welds)(t_{weld})(1+0.5\sin^{1.5}\theta)$

$= 1.392(21'' - 1/2'')(2)(4)(1.35) = 307.3^k > 162$

∴ OK

LIMIT STATES | CONNECTION COLUMN - BEAM - BRACE | February 17, 2010 | 8

COLUMN TO GUSSET CONT.

278^k BOLT LIMIT STATES

Shear 3/4 ϕ A490 N

$\phi r_n = 19.9k$ (Shear)

$37.4k$ (tension)

Bolt Bearing ⇒ angles control over column

$\phi V_n = \phi\, 2.4\, F_u\, t_a\, d_b$

$= 0.75(2.4)(58)(5/8)(3/4) = 48.94k$

Tear out @ angle edge

$\phi V_n = \phi\, 1.2\, F_u\, (l_{ev} - d_h/2)\, t_a$

$= 0.75(1.2)(58)(1.5 - \frac{7/8}{2})(5/8) = 34.66k$

Tear out of other bolt on angle

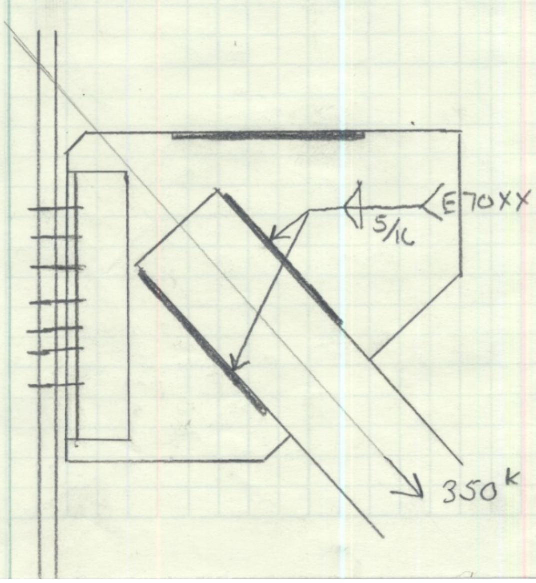
$\phi V_n = \phi\, 1.2\, F_u\, (\text{spacing} - d_h)\, t_a$

$= 0.75(1.2)(58)(3 - 7/8)(5/8) = 36.7$

COMBINATION ⇒ Bolt Shear Controls all bolts

$14(19.9) = 278.6^k$

BRACE TO GOSSET



HSS: 10x10x5/8

$A = 21.0$

$L_b = 20'$

$F_y = 46$ ksi

$F_u = 58$ ksi

$\phi P_n = 666^k$ axial Compression @ $kl = 20'$

LIMIT STATES | CONNECTION COLUMN - BEAM - BRACE | February 17, 2010 | 9

BRACE TO GUSSET $R_u = 350^k$

501^k Weld Rupture ⇒ all longitudinal welds ⇒ 1.5 factor
assume E70XX + $t_{weld} = 5/16''$

$\phi R_n = 1.392\, l_{weld}\, \#_{welds}\, t_{weld}\, 1.5 \geq 350$
$1.392(l_{weld})(4)(5)\,1.5 \geq 350$
$l_{weld} \geq 8.38''$ (all four sides)
assume 12" long
$\phi R_n = 1.392(12)(4)(5)(1.5) = 501.12^k$

624^k Base metal Rupture ⇒ HSS $F_u = 58 = F_u$ of plate
$t_p = 0.5 < t_{HSS} = 0.625$
plate controls
$\phi R_n = \phi\, 0.6\, F_u\, t_p\, l_{weld}\, \#_{welds}$
$= 0.75(0.6)(58)(0.5)(12)(4) = 626.4^k$

869^k HSS Tensile Yielding $= 869^k$ ⇒ AISC 5-6 Table 5-5

$\boxed{365^k}$ HSS Tensile Rupture ⇒ $\phi\, 0.6\, F_u A_e$ ⇒ $A_e = A_n U$
$A_n = A_g - \text{slots} = 21.0 - \tfrac{1}{2}(5/8)2$
$= 20.375\, in^2$

$U = 1 - \bar{x}/l_{weld}$

$\bar{x} = \dfrac{B^2 + 2BH}{4(B+H)} = \dfrac{10^2 + 2(10)(10)}{4(10+10)}$
$= \dfrac{300}{80} = 3.75$

$U = 1 - \dfrac{3.75}{12} = 0.6875$

$\phi R_n = 0.75(0.6)(58)(20.375)(0.6875) = 365.6^k$

587^k Block Shear of plate
$\phi R_n = \phi[0.6\, F_y A_{gv} + U_{bs} F_u A_{nt}]$
$A_{gv} = (12''\text{weld} + 5.85 \text{ edge distance})(t_p)(2)$
$= (17.85)(0.5)(2) = 17.85\, in^2$
$A_{nt} = 10(0.5) = 5\, in^2$
$\phi R_n = 0.75[0.6(46)(17.85) + 1.0(58)(5)] = 587^k$

LIMIT STATES | CONNECTION COLUMN - BEAM - BRACE | February 17, 2010 | 9

BRACE TO GUSSET $\quad R_u = 350^k$

501^k Weld Rupture ⇒ all longitudinal welds ⇒ 1.5 factor

assume E70XX + $t_{weld} = 5/16''$

$\phi R_n = 1.392\ l_{weld}\ \#_{welds}\ t_{weld}\ 1.5 \geq 350$

$1.392(l_{weld})(4)(5)\ 1.5 \geq 350$

$l_{weld} \geq 8.38''$ (all four sides)

assume 12" long

$\phi R_n = 1.392(12)(4)(5)(1.5) = 501.12^k$

624^k Base metal Rupture ⇒ HSS $F_u = 58 = F_u$ of plate

$t_p = 0.5 < t_{HSS} = 0.625$

plate controls

$\phi R_n = \phi\ 0.6\ F_u\ t_p\ l_{weld}\ \#_{welds}$

$= 0.75(0.6)(58)(0.5)(12)(4) = 626.4^k$

869^k HSS Tensile Yielding $= 869^k$ ⇒ AISC 5-C Table 5-5

$\boxed{365^k}$ HSS Tensile Rupture ⇒ $\phi\ 0.6\ F_u A_e$ ⇒ $A_e = A_n U$

$A_n = A_g - \text{slots} = 21.0 - \frac{1}{2}(5/8)2$

$= 20.375\ in^2$

$U = 1 - \bar{x}/l_{weld}$

$$\bar{x} = \frac{B^2 + 2BH}{4(B+H)} = \frac{10^2 + 2(10)(10)}{4(10+10)}$$

$$= \frac{300}{80} = 3.75$$

$$U = 1 - \frac{3.75}{12} = 0.6875$$

$\phi R_n = 0.75(0.6)(58)(20.375)(0.6875) = 365.6^k$

587^k Block Shear of plate

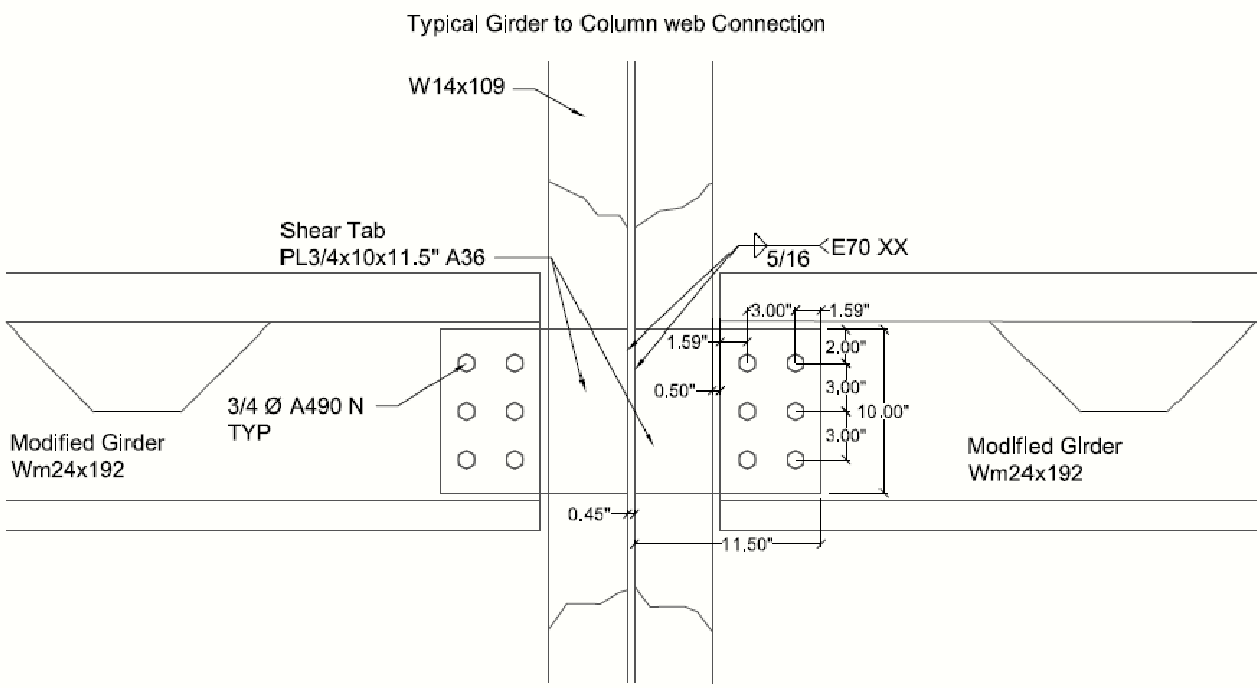
$\phi R_n = \phi[0.6\ F_y A_{gv} + U_{bs} F_u A_{nt}]$

$A_{gv} = (12''\text{ weld} + 5.85\text{ edge distance})(t_p)(2)$

$= (17.85)(0.5)(2) = 17.85\ in^2$

$A_{nt} = 10(0.5) = 5\ in^2$

$\phi R_n = 0.75[0.6(46)(17.85) + 1.0(58)(5)] = 587^k$

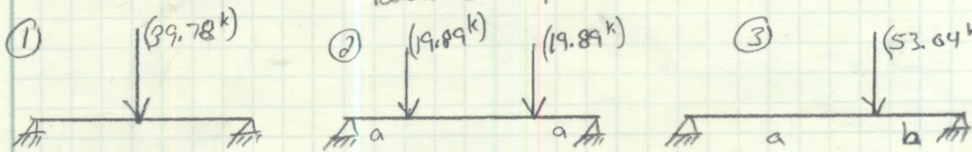

Typical Girder to Column web Connection
W14x109
Shear Tab
PL3/4x10x11.5" A36
5/16
E70 XX
3.00"
1.59"
1.59"
2.00"
3.00"
10.00"
3.00"
0.50"
3/4 Ø A490 N
TYP
Modified Girder
Wm24x192
Modified Girder
Wm24x192
0.45"
11.50"

DESIGN FORCES | CONNECTION GIRDER – COLUMN WEB | February 20, 2010 | 1

CONNECTION OF EXTRA SUPPORT GIRDERS IN THE NORTH-SOUTH DIRECTION TO SUPPORT OTHER BEAMS THAT PROVIDE FOR OPENINGS.

Differing Conditions; maximum loading

loaded @ center — loaded @ 1/4 points — loaded @ 1/3 point



loading:

$128\ DL(1.2) + 125\ LL(1.6) = 0.3536\ ksf$

for half point loading; loading comes from only one side

$0.3536\left(\frac{15'}{2}\right)\left(\frac{30'}{2}\right) = 39.78k$

for 1/4 point loading $\Rightarrow 0.3536\ ksf\left(\frac{7.5'}{2}\right)\left(\frac{30'}{2}\right) = 19.89^k$ @ each point

for 1/3 point loading $\Rightarrow 0.3536\left(\frac{2/3(30)}{2}\right)\left(\frac{30}{2}\right) = 53.04^k$

Maximum: Deflections Moments & Reactions for each load case

assuming $W_m 24\times 192$ I_x of NON-COMPOSITE

① $\Delta = P\ell^3/48EI = 40(30^3)(1728)/48(29000)(2634) = 1/2''$

$M = P\ell/4 = 40(30)/4 = 300\ k\cdot ft$

$R = 40/2 = 20^k$ each end

② $\Delta = \frac{Pa}{24EI}(3\ell^2 - 4a^2) = \frac{20(30/4)}{24(29000)(2634)}\left(3(30^2) - 4(7.5^2)\right)(1728) = 0.3456''$

$M = Pa = 20(30/4) = 150\ k\cdot ft$

$R = 20^k$ @ each end

③ $\Delta = \frac{Pab(a+2b)\sqrt{3a(a+2b)}}{27EI\ell} = \frac{53(20)(10)(20+2(10))\sqrt{3(20)(20+2(10))}(144)}{27(29000)(2634)(30)} = 0.48''$

CONTROLS V & M

$M = Pab/\ell = 53(20)(10)/30 = 354\ k\cdot ft$

$R_R = Pa/\ell = 53(20)/30 = 35.3\ k$

$R_L = Pb/\ell = 53(10)/30 = 17.7\ k$

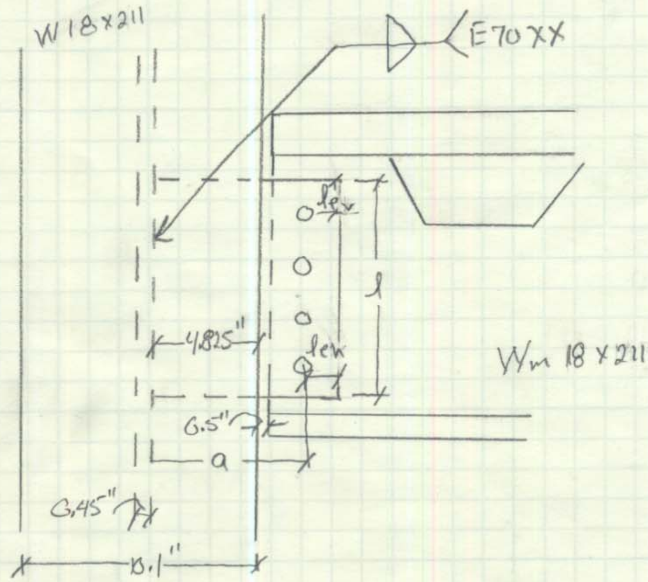
DESIGN FORCES | CONNECTION GIRDER - COLUMN WEB | February 21, 2010 | 2

TRY AN EXTENDED SHEAR TAB (SINGLE-PLATE) CONNECTION

SHEAR (V_u) Due to maximum DEAD + LIVE loading and Load cases 1-3. Load case 3 controlled with a single point load located at the 1/3 point from one end.

$V_{u_{max}} = 35.3^k$ $M_{u_{max}} = 354$ k·ft

The extended shear tab would give the best constructibility options.



W14x74:

$d = 14.2"$

$t_w = 0.45"$

$b_f = 10.1"$

$t_f = 0.785"$

Wm 18 x 211:

$t_w = 1.06"$

$d = 15.85"$

$b_f = 11.60"$

$t_f = 1.91"$

Dimensional limitations:

Bolts are not limited

The distance a is not limited

Holes satisfy AISC J3.2

Edge distances meet AISC Table J3.4 for minimum edge distances

PLATE SIZES & LIMITS | CONNECTION GIRDER – COLUMN WEB | February 21, 2010 | 3

Maximum Plate length (vertical depth) is limited to the T dimension of Wm 18x211 section ≃ 10".

Minimum plate width has to be wider than

$(\frac{1}{2} b_{fc} - \frac{1}{2} t_{wc}) + \frac{1}{2}"\text{gap} + l_{eh} + \text{spacing} + l_{eh}$

$4.825 + 0.5 + 1.5875 + 3 + 1.5875 = 11.5"$

Eccentricity must be considered

$e = 4.825 + 0.5 + 1.5875 + \frac{3}{2} = 8.413"$ for a double row of bolts

loading angle = 0°

Plate thickness limit

$t_{max} \le 6\, M_{max} / F_y d^2$

$\le \frac{6(712.185)}{36(10^2)}$

$\le 1.19"$

$M_{max} = 1.25\, F_v\, A_b\, C'$

$F_v = 60\text{ksi}$ for A490

$A_b = 0.601\ \text{in}^2$

$C' = 15.8$ for $e = 8.413$

$M_{max} = 1.25(60)(0.601)(15.8)$

$= 712.185$

Try a PL $\overset{t}{3/4} \times \overset{l}{10} \times \overset{w}{11\frac{1}{2}}$

Plate: A36

$F_y = 36$

$F_u = 58$

$l_{ev} = 2"$

$l_{eh} = 1.5875"$

Bolts:

7/8 ⌀ A490 N

$\phi R_n = C\,\phi r_n = 1.71(27.1)$

$= 46.34^k > 36 \therefore OK$

$V_u = 36^k$ $M_u = V_u \cdot e = 36(8.413) = 303\,k\cdot ft$

46^k

COMBINED BOLT GROUP LIMIT STATE

bearing $\Rightarrow \phi r_n = \phi\, 2.4 (F_u)\, t_p\, d_b$

$= 0.75(2.4)(58)(0.75)(7/8) = 68.5^k$

tear out on edge controls over others $\Rightarrow \phi r_n = \phi\, 1.2\, F_u\, (l_{ev} - d_h/2)\, t_p$

$= 0.75(1.2)(58)(2 - \frac{1}{2})0.75 = 58.73^k$

BOLT SHEAR CONTROLS $\phi R_n = 46^k > 36^k$

CONNECTIONS

LIMIT STATES | GIRDER – COLUMN WEB | February 20, 2010 | 4

PLATE LIMIT STATES

162^k Shear Yielding $\Rightarrow \phi R_n = \phi\, 0.6\, F_y A_g \Rightarrow A_g = h(t_p)$

$= 1.0(0.6)(36)(10)(0.75) = 162^k$

137^k Shear Rupture $\Rightarrow \phi R_n = \phi\, 0.6\, F_u A_n \Rightarrow A_n = A_g - ((3d_h)(t_p))$

$= 7.5 - (3(1'')(0.75))$

$= 5.25$

$= 0.75(0.6)(58)(5.25)$

$= 137^k$

218^k Block Shear $\Rightarrow \phi R_n = \phi[0.6\, F_u A_{nv} + U_{bs} F_u A_{nt}] \le \phi[0.6\, F_y A_{gv} + U_{bs} F_u A_{nt}]$

$U_{bs} = 1.0$

$A_{gv} = 10(0.75) = 7.5\ in^2$

$A_{nv} = (10 - l_{ev} - 2.5 d_h)(t_p) = (10 - 2.0 - 2(1))(0.75) = 4.5\ in^2$

$A_{nt} = ((l_{eh} + spacing) - 1.5 d_h)\, t_p = ((1.5875 + 3) - 1.5(1))(0.75) = 2.32$

$\phi R_n = 0.75[0.6(58)(4.5) + 1.0(58)(2.32)] \le 0.75[0.6(36)(7.5) + 1.0(58)(2.32)]$

$= 218.37 \le 222.42$

$607\ k{\cdot}in$ Plate flexure using Von-Mises Shear Reduction

$M_u \le \phi M_n \Rightarrow M_u = 303\ k \cdot in$

$\phi M_n = \phi F_{cr} Z \Rightarrow F_{cr} = \sqrt{F_y^2 + 3f_v^2} \Rightarrow f_v = \dfrac{V_u/\phi}{A_p}$

$= \sqrt{36^2 - 3(0.348^2)}$ $\qquad = \dfrac{36/0.9}{10(11.5)}$

$= 36$ $\qquad = 0.348$

$\phi M_n = 0.9(36)(18.75) = 607\ k\cdot in > M_u$

$\therefore$ OK

$Z_p = d_p^2\, t_p/4$

$= 10^2(0.75)/4$

$= 18.75\ in^3$

$861\ k{\cdot}in$ Plate Flexural Rupture $\Rightarrow \phi F_u Z_{net} \ge M_u$

$Z_{net} = Z_p - Z_{holes} \Rightarrow Z_{holes} = t_p(d_h)(2(dist))$

$= 18.75 - 2.25$ $\qquad = 0.75(1'')(2(1.5))$

$= 16.5\ in^3$ $\qquad = 2.25$

$0.9(58)(16.5) = 861.3\ k\cdot in > 303 \therefore$ OK

LIMIT STATES | CONNECTION GIRDER - COLUMN WEB | February 21, 2010 | 5

PLATE LIMIT STATES CONT.

Plate buckling ⇒ top and bottom "copes" are not =
therefore use the equation for $d_c > 0.2d$

$F_{cr} \le F_y$

$F_{cr} = F_y Q \Rightarrow Q = 1$ for $\lambda \le 0.7$

$$\lambda = \frac{h_o \sqrt{F_y}}{10 t_p \sqrt{475 + 280\left(\frac{h_o}{c}\right)^2}} \Rightarrow h_o = 10''$$

$c = 4.825 + 0.5 + 1.6h$
$= 6.9125$

$$= \frac{10\sqrt{36}}{10(0.75)\sqrt{475 + 280\left(\frac{10}{6.9125}\right)^2}} = \frac{60}{7.5(\sqrt{475 + 586})}$$

$= 0.246 \therefore Q = 1$ & $F_{cr} = F_y$ and plate buckling does not control

WELD LIMIT STATES

NO ECCENTRICITIES FOR WELDS

assume $l_{weld} = l_p - 2t_{weld}$ & $t_{weld} = 5/16$ on both sides

130^k Weld Rupture $\Rightarrow \phi R_n = 1.392\, l_w \, \#\text{welds}\, t_w (1 + 0.5 \sin^{1.5}\theta)$ $\theta = 0°$

$= 1.392(10 - 10/16)(2)(5)1$

$= 130.5^k > 36 \therefore OK$

246^k Base Metal Rupture $\Rightarrow \phi R_n = \phi\, 0.6\, F_u\, t_{wc}\, l_{weld}\, \#\text{welds}$

or

$\phi\, 0.6\, F_u\, t_p\, l_{weld}\, \#\text{welds}$

$\phi R_n = 0.75(0.6)(65)(0.45)(9.375)(2) = \boxed{246.8}$
column ⇒

or

plate ⇒ $0.75(0.6)(58)(0.75)(9.375)(2) = 367.63$

Appendix K: Acoustical Calculations

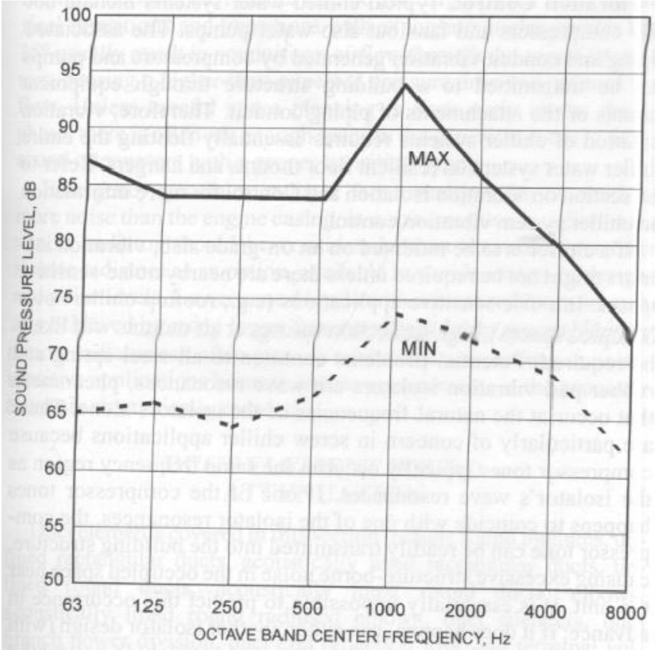


Fig. 12 Typical Minimum and Maximum ARI 575 L_p Values for Centrifugal Chillers (130 to 1300 Tons)

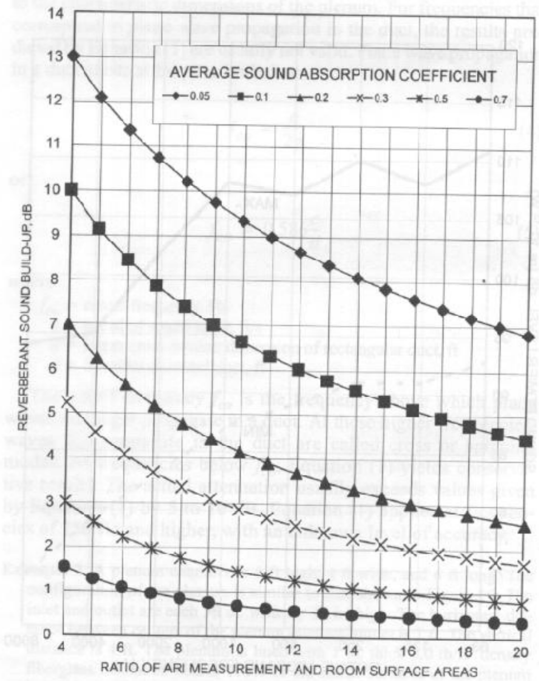


Fig. 14 Estimated dB Buildup in Mechanical Room for ARI 575 Chiller Sound Levels

Chillers and Air-Cooled Condensers

All chillers and their associated systems produce significant amounts of both broadband and tonal noise. The broadband noise is caused by flows of both refrigerant and water, whereas the tonal noise is caused by the rotation of compressors, motors, and fans (in fan-cooled equipment). Chiller noise is usually significant in the octave bands from 250 through 1000 Hz.

Indoor Water-Cooled Chillers. The dominant noise source in most water-cooled chillers is the compressor. Water-cooled chillers can use any compressor type, but most use either centrifugal or screw compressors.

Factory sound data for indoor chillers are obtained via ARI *Standard* 575. The standard requires measuring the A-weighted and octave band sound pressure level (L_p) values at several locations 3.28 ft from the chiller and 4.92 ft above the floor. ARI 575 ratings are generally available at operating points of 25, 50, and 100% of a chiller's nominal full capacity. The ranges of ARI 575 values for typical centrifugal and screw chillers are shown in Figures 12 and 13, respectively.

ARI 575 measurements are usually made in very large rooms with large amounts of sound absorption. Measured levels must be adjusted for each chiller installation to account for the size and surface treatment conditions of the mechanical room. For a given chiller at a given operating point, a small equipment room, or one

ALL are taken from ASHRAE 2003 Applications Handbook 47.10 & 47.11

Acoustical (dbA) | Chiller Calculations | March 13, 2010 ①

For Figure 14 in ASHRAE 2003 Applications Handbook 47.11

<u>For Only ONE Chiller</u>

Add 3.28' to height

Add 6.56' to both length + width on each side

$h = 3.28' + 7'\text{-}0'' = 10.28'$

$l = (2)6.56' + 7'\text{-}0'' = 20.12'$

$w = (2)6.56' + 4'0'' = 17.12'$

$10.28'(20.12')(17.12') = 3541\ ft^3$

room size

$30.5'(45')(14.17') = 19444\ ft^3$

$$\frac{19444}{3541} \cong 5.5$$

From Figure 14 the estimated db buildup is 9

∴ 9 is added to the 8 octave band center frequencies

(Hz)	63	125	250	500	1000	2000	4000	8000
	80	75	92	88	90	87	79	67
Build up	+9	+9	+9	+9	+9	+9	+9	+9
	89	84	101	97	99	96	88	76
A weighting	-25	-16	-8	-3	0	+1	+1	-1
A weighted sound level	64	69	93	94	99	97	89	75
Total (dbA)	64	70	93	95	100	102	102	$\boxed{102}$

Criteria for Acceptable HVAC Sound Levels in Rooms

Sound associated with HVAC systems is usually considered part of the background sound in a building. Therefore, to be judged acceptable, it must neither noticeably mask sounds people want to hear nor be otherwise intrusive or annoying in character. In an office

47.29

Table 34 Design Guidelines for HVAC-Related Background Sound in Rooms

Room Types	RC(N) (QAI ≤ 5 dB[a,b])
Residences, Apartments, Condominiums	25 to 35
Hotels/Motels	
Individual rooms or suites	25 to 35
Meeting/banquet rooms	25 to 35
Corridors, lobbies	35 to 45
Service/support areas	35 to 45
Office Buildings	
Executive and private offices	25 to 35
Conference rooms	25 to 35
Teleconference rooms	25 (max)
Open-plan offices	30 to 40
Corridors and lobbies	40 to 45
Hospitals and Clinics	
Private rooms	25 to 35
Wards	30 to 40
Operating rooms	25 to 35
Corridors and public areas	30 to 40
Performing Arts Spaces	
Drama theaters	25
Concert and recital halls[c]	
Music teaching studios	25
Music practice rooms	30 to 35
Laboratories (with fume hoods)	
Testing/research, minimal	
Speech communication	45 to 55
Research, extensive telephone use, speech communication	40 to 50
Group teaching	35 to 45
Church, Mosque, Synagogue	
General assembly with critical music programs[c]	25 to 35
Schools[d]	
Classrooms	25 to 30
Large lecture rooms	25 to 30
(without speech amplification)	25
Libraries	30 to 40
Courtrooms	
Unamplified speech	25 to 35
Amplified speech	30 to 40
Indoor Stadiums, Gymnasiums	
Gymnasiums and natatoriums[e]	40 to 50
Large seating-capacity spaces with speech amplification[e]	45 to 55

[a]Values and ranges are based on judgment and experience, not on quantitative evalua-

9.2 ACOUSTICAL PROPERTIES OF PRECAST CONCRETE

9.2.1 Definitions

Hertz (Hz). A measure of sound wave frequency, i.e., the number of complete vibration cycles per second.

STC. Sound Transmission Class

IIC. Impact Insulation Class

9.2.2 General

The basic purpose of architectural acoustics is to provide a satisfactory environment in which desired sounds are clearly heard by the intended listeners and unwanted sounds (noise) are isolated or absorbed.

Under most conditions, the architect/engineer can determine the acoustical needs of the space and then design the building to satisfy those needs.

Figure 9.2.4.1 Sound transmission class as a function of weight of floor or wall

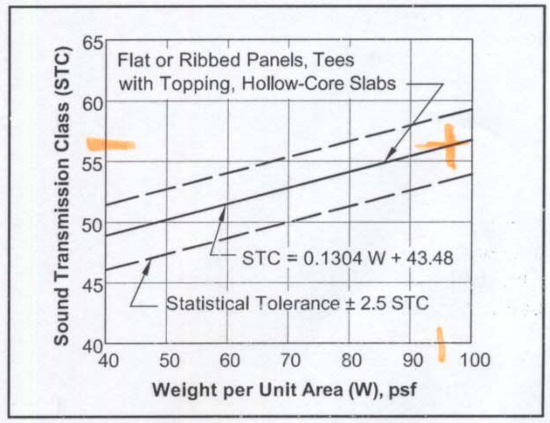


Table 1. Suggested acoustical criteria for some occupancies

	Recommended Minimum Sound Attenuation		Recommended Range for Background Noise, dB(A)	Reverberation Time, seconds
	ASTC	FIIC		
Multi-family homes	55	50	35-40	
Bedrooms in residences	55	50	30-35	
Private offices	45		40-45	
Meeting rooms	50		35-40	0.5
Bedrooms in hotels, motels and hospitals	50	50	35-40	
Classrooms up to 300 m^3	50		35-40	0.6
Cafeterias			40-45	0.8
Large lecture rooms, classrooms over 300 m^3	50		30-35	0.7
Gymnasiums			40-45	1.0
Libraries			40-45	0.7

Taken from the Precast Concrete Institute (PCI)
PCI Design Handbook / Sixth Edition

Acoustical (db A) | Chiller / Conference | March 13, 2010 | 2

Since there are two chillers in the same room the sound level for both is assumed the same and therefore the dbA for the room equals

$$102 + 102$$

$$L_p = \boxed{105 \text{ dbA}} \cong L_I$$

L_2 = Sound level in Conference Room

L_1 = Sound level in Chiller Room

$TL = STC \Rightarrow STC_2 = 57 \text{ dBA}$

$STC_1 = 51 \text{ dBA}$

BNC = Background Noise Criteria for HVAC Systems in Conference Room = 30 dBA

$$L_2 = BNC = L_1 - STC \leq L_2$$

As designed	Proposed
$30 > 105 - 51 = 54$	$30 > 105 - 57 = 48$
NOT ACCEPTABLE	NOT ACCEPTABLE

Chiller Room Calcs.

Acousitical | with absorptive cieling | March 13, 2010 | 3

estimated absorptive coefficient for Figure 14 = 0.2

Hz	63	125	250	500	1000	2000	4000	8000
	80	75	92	88	90	87	79	67
Build up	+ 6	+ 6	+ 6	+ 6	+ 6	+ 6	+ 6	+ 6
	86	81	98	94	96	93	85	73
A weighting	-25	-15	-8	-3	0	+1	+1	-1
A weighted Ajusted	61	66	90	91	96	94	86	72
Total (dBA)	61	67	90	94	98	99	99	$\boxed{99}$

For 2 chillers

$99 + 99 \Rightarrow \boxed{102 \text{ dBA}}$

assuming the absorptive material can provide and additional 10 dB to the STC ratings

As Designed

$30 \geq 102 - (51 + 10)$

$30 < 41$ ∴ NOT ACCEPTABLE

Proposed

$30 \geq 102 - (57 + 10)$

$30 < 35$

Within the 25-35 range

By taking into consideration the reciever room sound correction found in ASHRAE 2003 Applications Handbook 47.25 the proposed sound level should be acceptable

see page 4

Acoustical Analysis | RECIEVER ROOM SOUND CORRECTION | March 13, 2010 | 4

Reciever room volume ⇒ Medical Staff Conference Room

$30.5'(22')(9') = 6039\ ft^3 \cong 6000\ ft^3 < 15000\ ft^3$

for Table 28 A values

The room has carpeting and a sound absorptive ceiling

$B = 9$ for a distance of 8' away for Table 29

$L_p = L_w + A - B$ equation 19

L_w

(Hz)	63	125	250	500	1000	2000	4000	8000
Max dB	80	75	92	88	90	87	79	67
Build up	+6	+6	+6	+6	+6	+6	+6	+6
	86	81	98	94	96	93	85	73
(+ A)	+1	+0	-1	-2	-3	-4	-5	-5
(- B)	-9	-9	-9	-9	-9	-9	-9	-9
	78	72	88	83	84	80	71	59
A weighting	-25	-15	-8	-3	0	+1	+1	-1
A weighted Adjusted	53	57	80	80	84	81	72	58
Total (dBA) L_p	53	58	80	83	87	88	88	$\boxed{88}$

2 Chillers = $\boxed{91\ dBA}$

As Designed

$30 \geq 91 - (51+10) = 30$

$30 = 30$ ∴ OK

PROPOSED

$30 \geq 91 - (57+10) = 24$

$30 > 24$ ∴ OK

Point Sound Sources

Most normally furnished rooms of regular proportions have acoustic characteristics that range from *average* to *medium dead*. These usually include carpeted rooms with sound-absorptive ceilings. If such a room has a volume less than 15,000 ft^3 and the sound source is a single point source, sound pressure levels associated with the sound source can be obtained from

$$L_p = L_w + A - B \quad (19)$$

where

L_p = sound pressure level at specified distance from sound source, dB
L_w = sound power level of sound source, dB

Table 25 TL_{in} Versus Frequency for Rectangular Ducts

Duct Size, in. × in.	Gage	TL_{out}, dB Octave Midband Frequency, Hz							
		63	125	250	500	1000	2000	4000	8000
12 × 12	24	16	16	16	25	30	33	38	42
12 × 24	24	15	15	17	25	28	32	38	42
12 × 48	22	14	14	22	25	28	34	40	42
24 × 24	22	13	13	21	26	29	34	40	42
24 × 48	20	12	15	23	26	28	36	42	42
48 × 48	18	10	19	24	27	32	38	42	42
48 × 96	18	11	19	22	26	32	38	42	42

Note: Data are for duct lengths of 20 ft, but values may be used for cross-section shown regardless of length.

Table 26 Experimentally Measured TL_{in} Versus Frequency for Circular Ducts

Diameter, in.	Length, ft	Gage	TL_{in}, dB Octave Midband Frequency, Hz						
			63	125	250	500	1000	2000	4000
Long Seam Ducts									
8	15	26	>17	(31)	39	42	41	32	31
14	15	24	>27	43	43	31	31	28	22
22	15	22	>28	40	30	30	30	24	22
32	15	22	(35)	36	23	23	21	19	35
Spiral Wound Ducts									
8	10	26	>20	>42	>59	>62	53	43	26
14	10	26	>20	>36	44	28	31	32	22
26	10	24	>27	38	20	23	22	19	33
26	10	16	>30	>41	30	29	29	25	38
32	10	22	>27	32	25	22	23	21	37

Note: In cases where background sound swamped the sound radiated from duct walls, a lower limit on TL_{in} is indicated by >. Parentheses indicate measurements in which background sound produced greater uncertainty than usual.

Table 27 TL_{in} Versus Frequency for Flat Oval Ducts

Duct Size, in. × in.	Gage	TL_{in}, dB Octave Midband Frequency, Hz						
		63	125	250	500	1000	2000	4000
12 × 6	24	18	18	22	31	40	—	—
24 × 6	24	17	17	18	30	33	—	—
24 × 12	24	15	16	25	34	—	—	—
48 × 12	22	14	14	26	29	—	—	—
48 × 24	22	12	21	30	—	—	—	—
96 × 24	20	11	22	25	—	—	—	—
96 × 48	18	19	28	—	—	—	—	—

Note: Data are for duct lengths of 20 ft, but values may be used for cross-section shown regardless of length.

furnished room has a volume greater than 15,000 ft^3 and the sound source is a single point source, sound pressure levels associated with the sound source can be obtained from

$$L_p = L_w - C - 5 \quad (20)$$

Values for C are given in Table 30. Equation (20) can be used for room volumes of up to 150,000 ft^3, with accuracy typically within 2 to 5 dB.

Distributed Array of Ceiling Sound Sources

In many office buildings, air supply outlets are located flush with the ceiling of the conditioned space and constitute an array of distributed ceiling sound sources. The geometric pattern depends on the floor area served by each outlet, the ceiling height, and the thermal load distribution. In the interior zones of a building where thermal load requirements are essentially uniform, air delivery per outlet is usually the same throughout the space; thus, these outlets emit nominally equal sound power levels. One way to calculate sound pressure levels in a room with a distributed array is to use Equation (19) or (20) to calculate the sound pressure levels for each individual air outlet at specified locations in the room and then log-

Table 28 Values for A in Equation (19)

Room Volume, ft^3	Value for A, dB Octave Midband Frequency, Hz						
	63	125	250	500	1000	2000	4000
1,500	4	3	2	1	0	−1	−2
2,500	3	2	1	0	−1	−2	−3
4,000	2	1	0	−1	−2	−3	−4
6,000	1	0	−1	−2	−3	−4	−5
10,000	0	−1	−2	−3	−4	−5	−6
15,000	−1	−2	−3	−4	−5	−6	−7

Table 29 Values for B in Equation (19)

Distance from Sound Source, ft	Value for B, dB
3	5
4	6
5	7
6	8
8	9
10	10
13	11
16	12
20	13

Table 30 Values for C in Equation (20)

Distance from Sound Source, ft	Value for C, dB Octave Midband Frequency, Hz						
	63	125	250	500	1000	2000	4000
3	5	5	6	6	6	7	10
4	6	7	7	7	8	9	12
5	7	8	8	8	9	11	14
6	8	9	9	9	10	12	16
8	9	10	10	11	12	14	18
10	10	11	12	12	13	16	20
13	11	12	13	13	15	18	22
16	12	13	14	15	16	19	24
20	13	15	15	16	17	20	26
25	14	16	16	17	19	22	28
32	15	17	17	18	20	23	30

Chiller Room Final Calculations with Acoustical Treatment

Acoustical Final Calc. TREATMENT Transmission Loss | March 18, 2010 5

USING THE ART USA ART COMPOSITE SOUND BARRIER
½ lb/ft²

Lw Chiller Room

(Hz)	63	125	250	500	1000	2000	4000	8000
Max dBA buildup	80 +6	75 +6	92 +6	88 +6	90 +6	87 +6	79 +6	67 +6
 +A −B	86 +1 −9	81 +0 −9	98 −1 −9	94 −2 −9	96 −3 −9	93 −4 −9	85 −5 −9	73 −5 −9
Art composite TL	78 —	72 −10	88 −12	83 −16	84 −21	80 −26	71 −32	59 —
A weighting	— −25	62 −15	76 −8	67 −3	63 0	58 +1	39 +1	— −1
A weighted Adjusted	—	47	68	64	63	59	40	—
TOTAL dBA		47	68	69	70	70	70	$\boxed{70}$

2 chillers ≈ 73 dBA

As Designed

25 ≥ 73 − 51 = 22

∴ OK

Proposed

25 > 73 − 57 = 16

∴ OK

Boiler Room With No Acoustical Treatments

Acoustical Final Calc. | Boiler Room Calculations | March 18, 2010 | 6

NO Acoustical Treatments

(Hz)	63	125	250	500	1000	2000	4000	8000
dBA	50	62	62	69	69	61	54	46
+ Build up	+7	+7	+7	+7	+7	+7	+7	+7
	57	69	69	76	76	68	61	53
+A	+1	0	−1	−2	−3	−4	−5	−5
−B	−9	−9	−9	−9	−9	−9	−9	−9
A weighting	49	60	59	65	64	55	47	39
	−25	−15	−8	−3	0	+1	+1	−1
A weighted Adjusted	24	45	51	62	64	56	48	38
TOTAL dBA	24	45	52	62	66	66	66	$\boxed{66}$

2 Boiler ≅ 69 dbA

As designed

25 ≥ 69 − 51 = 18 dBA

∴ ok

Proposed

25 ≥ 69 − 57 = 12 dBA

∴ ok

Appendix L: Vibration due to Walking

9.7.2 Human Response to Building Vibrations

This section is a condensation of the material contained in Ref. 1, which is based on information in Refs. 2 to 6.

Limits are stated as a minimum natural frequency of a structural system. These, in turn, depend on the permissible peak accelerations (as a fraction of gravitational acceleration), on the mass engaged during an activity, the degree of continuity of the floor system, the environment in which the vibration occurs, the effectiveness of interaction between connected structural components, and the degree of damping. Much vibration theory derives from experience with steel and wood floors. In general, floor vibrations are much less likely to be a problem with stiffer, more massive, concrete floors.

Some building types common in precast construction are not dealt here, because of a lack of source information. Choice of limits for usage not listed may be selected, with judgment, from other types listed here.

It must be emphasized that the calculations presented are very approximate. The actual natural frequency of a floor can be estimated to a reasonable degree of accuracy, but the calculation of the required frequency is based on damping and on human response, both of which are subject to much variation. When in doubt about the acceptability of a proposed floor system, the best way to decide is to compare it to existing similar systems that are known to be acceptable or unacceptable, using the same method of analysis.

9.7.3 Types of Vibration Analysis

Three types of vibration analysis are described. These analyses differ because the inputs causing the vibration differ.

9.7.3.1 Walking

As a walking person's foot touches the floor, a vibration of the floor system is caused. This vibration may be annoying to other persons sitting or lying in the same area, such as an office, a church, or a residence. Although more than one person may be walking in the same area at the same time, their footsteps are normally not synchronized. Therefore, the analysis is based on the effect of the impact of the steps of individual walking persons.

9.7.3.2 Rhythmic Activities

In some cases, several or many people may engage in a coordinated activity that is at least partially synchronized. Spectators at sporting events, rock concerts, and other entertainment events often move in unison in response to music, a cheer, or other stimuli. The people engaged in the rhythmic activity have a higher level of tolerance for the induced vibrations, while those nearby will have a lower level of tolerance.

9.7.3.3 Mechanical Equipment

Mechanical equipment may produce a constant impulse at a fixed frequency, causing the structure to vibrate.

9.7.3.4 Analysis Methods

Each of the three input types described above requires a somewhat different solution. But, all require knowledge of an important response parameter of the floor system, its natural frequency of vibration.

9.7.3.5 Using Consistent Units

All the equations in this section are dimensionally correct. Provided one is careful to be sure that the units used cancel out to produce the desired units for the answer, a correct result will be obtained using either customary or SI units.

9.7.4 Natural Frequency of Vibration

The natural frequency of a floor system is important in determining how human occupants will perceive vibrations. It has been found that certain frequencies seem to set up resonance with internal organs of the human body, making these frequencies more annoying to people.

The human body is most sensitive to frequencies in the range of 4 to 8 Hertz (cycles per sec). This range of natural frequencies is commonly found for typical floor systems.

9.7.4.1 Computing the Natural Frequency

The natural frequency of a vibrating beam is determined by the ratio of its mass (or weight) to its stiffness. The deflection of a simple span beam is also dependent on its weight and stiffness. A simple relationship exists between deflection and natural frequency of a uniformly loaded simple span beam on rigid supports: [2,3]

$$f_n = 0.18\sqrt{\frac{g}{\Delta_i}} \qquad \text{(Eq. 9.7.4.1)}$$

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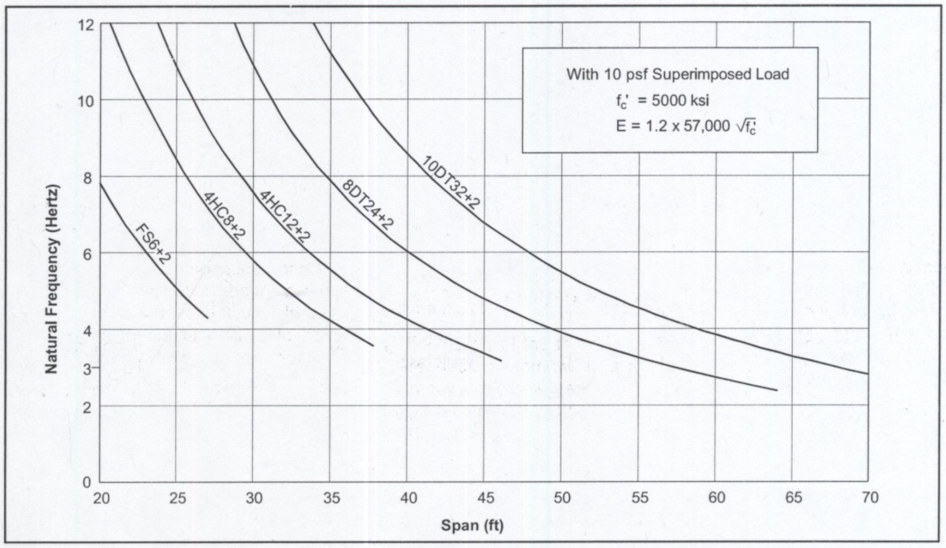
9–67

Figure 9.7.4.1 Natural frequency of selected floor units



9.7.4.2 Computing Deflection

The deflection, Δ_j, for a uniformly loaded simple span floor unit is:

$$\Delta_j = \frac{5w\ell^4}{384EI} \qquad \text{(Eq. 9.7.4.2)}$$

Many vibration problems are more critical when the mass (or weight) is low. When computing Δ_j, use a minimum realistic live load when computing w, not the maximum live load.

For continuous spans of equal length, the natural frequency is the same as for simple spans. During vibration, one span deflects down while the adjacent spans deflect upward. An inflection point exists at the supports, and the deflection and natural frequency are the same as for a simple span.

For unequal continuous spans, and for partial continuity with supports, the natural frequency may be increased by a small amount. Refs. 2 and 3 suggest how this increase may be computed.

9.7.4.3 Effect of Supporting Girders

The deflection of beams or girders supporting the floor system also affect the natural frequency of the floor system. The simple-span deflection, Δ_g, of the floor girder may be calculated in the same manner as Δ_j. The natural frequency of the floor system may then be estimated by the following formula: [2,3]

$$f_n = 0.18\sqrt{\frac{g}{\Delta_j + \Delta_g}} \qquad \text{(Eq. 9.7.4.3)}$$

For concrete floor systems supported on walls, Δ_g may be assumed to be zero. For concrete floor systems supported by concrete girders, Δ_g is normally small, and is often neglected, unless the girders are unusually long or flexible. For concrete floor units supported on steel beams, the beam deflection can have a significant effect, and should usually be included in computing f_n.

9.7.4.4 Minimum Natural Frequency

Floors with natural frequencies lower than 3 Hertz are not recommended, because people may more readily synchronize their actions at lower frequencies. [3]

9.7.4.5 Graphs of Natural Frequency

Eqs. 9.7.4.1 and 9.7.4.2 may be combined to produce the following Eq. 9.7.4.4, for a floor unit on stiff supports:

$$f_n = \left(\frac{1.58}{\ell^2}\right)\sqrt{\frac{EIg}{w}} \qquad \text{(Eq. 9.7.4.4)}$$

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Figure 9.7.4.1 shows the relation between span and expected natural frequency for various topped floor units given in Chapter 2.

9.7.5 Damping

Damping usually is expressed is a fraction or percent of critical damping. Real building structures have damping from 1 percent to a few percent of critical.

9.7.5.1 Types of Damping

Damping is not a well understood phenomenon. In the literature, differing methods are used for calculation. This section and its references are based on modal damping. Do not mix values of damping from other sources with damping values in the equations of this section, as they may be based on a different calculation method.

9.7.5.2 Estimation of Damping

Damping of a floor system is highly dependent on the non-structural items (partitions, ceilings, furniture, etc.) present. The modal damping ratio of a bare structure can be very low, on the order of 0.01. Non-structural elements may increase this, up to 0.05.

The results of a vibration analysis are highly influenced by the choice of the assumed damping, which can vary widely. Yet, this choice is based more on judgment than science.

9.7.6 Vibrations Caused by Walking

Vibrations caused by walking are seldom a problem in concrete floor systems because of their mass and stiffness. When using concrete floor systems of ordinary proportions, it is usually not necessary to check for vibrations caused by walking.

When designing concrete floor systems of long-span or slender proportions, this section may be used to evaluate their serviceability with respect to vibrations.

9.7.6.1 Minimum Natural Frequency

An empirical formula, based on resonant effects of walking, has been developed to determine the minimum natural frequency of a floor system needed to prevent disturbing vibrations caused by walking: [4]

$$f_n \geq 2.86\left[\ln\left(\frac{K}{\beta W}\right)\right] \qquad \text{(Eq. 9.7.6.1)}$$

The constant 2.86 has the units 1/sec.

9.7.6.2 Effective Weight

The effect of an impact such as a footfall is strongly influenced by the mass (or weight) of the structure affected by the impact. This weight, W, is normally taken as the unfactored dead load (per square foot) of the floor units plus some (not full code) live load, multiplied by the span and by a width B. For solid or hollow-core slabs, which are stiff in torsion, it is recommended to take B equal to the span. [2] For double tees, it is recommended to take B varying from 0.8ℓ for 18-in. double tees with 3-in. topping to 0.6ℓ for 32-in. double tees with 3-in. topping. [5] For continuous spans, W may be increased 50 percent. [2,3] At an unstiffened edge of a floor, the width B used for estimating floor system weight should be halved. [2]

9.7.6.3 Recommended Values

The recommended values of K and β for use in Eq. 9.7.6.1 are given in Table 9.7.6.1 below.

Table 9.7.6.1 Values of K and β for use in Eq. 9.7.6.1 (based on Table 3 of Ref. 4)

Occupancies Affected by the Vibrators	K Kips	K kN	β
Offices, Residences, Churches	13	58	0.02[a] 0.03[b] 0.05[c]
Shopping Malls	4.5	20	0.02
Outdoor Footbridges	1.8	8	0.01

a. For floors with few non-structural components and furnishings, open work area, and churches.
b. For floors with non-structural components and furnishings, cubicles.
c. For floors with full-height partitions.

PCI 9.7 Analysis | Floor Vibration From Walking | March 21, 2010 | 1

USING PCI CHAPTER 9.7 for Analysis

Fundamental Equation: $f_n = 0.18\sqrt{\dfrac{g}{\Delta_{slab} + \Delta_{girder}}}$ 9.7.4.3

$g = 386\ in/sec^2$

$\Delta_{slab} = \dfrac{5 w_s l_s^4}{384 E_c I_s}$ $\Rightarrow$ $w_s = 93\,plf + 30\,plf = 120\,plf$

$I_s = 5102\ in^4$

$E_c = 1.35(57)\sqrt{6000} = 5960\ ksi$

$l_s = 30'$

1.35 for Dynamic loading

$= \dfrac{5(0.120)(30^4)(1728)}{384(5960)(5102)} = 0.0732\,in$

$\Delta_{girder} = \dfrac{5 w_g l_g^4}{384 E_g I_g}$ $\Rightarrow$ $w_g = 120\,psf(30') + 221\,plf = 3.821\,plf$

$l_g = 28'$

$E_s = 29000\ ksi$

$I_g \Rightarrow I_c + I_s$ for composite shape

$E_g = E$ composite girder

$I_g = I$ composite girder

$I_s = \Sigma \dfrac{bh^3}{12} + (Ad^2)$

$I_s = \left(\dfrac{6''(3^3)}{12} + \overset{bar}{18(4.3'')^2}\right) + \left(\dfrac{0.81(8.46^3)}{12} + \overset{web}{6.85(6.918)^2}\right)$

$+ \left(\dfrac{13(1.46^3)}{12} + \overset{flange}{18.98(11.95^2)}\right) + 4\left(\dfrac{0.5(7^3)}{12} + \overset{HSS\ sides}{3.5(7.66^2)}\right)$

$+ 2\left(\dfrac{0.5(4^3)}{12} + \overset{Hss\ Tops}{2(4.41^2)}\right) + 2\left(\dfrac{0.5(4^3)}{12} + \overset{Hss\ bottoms}{2(11.035^2)}\right)$

$= 346.32 + 368.7 + 2713.76 + 878.63 + 83.12 + 492.42$

$= 4883\ in^4$

PCI 9.7 Analysis | Floor Vibration From Walking | March 21, 2010 | 2

$$I_c = 2\left(\frac{8.5(5.84^3)}{12} + 49.64\left(\frac{5.84}{2}\right)\right) = 572 \text{ in}^4$$

$$E_c = (1.35)(57)\sqrt{4000} = 4866 \text{ ksi}$$

dynamic loading

$$\Delta_{girder} = \frac{5 w_g l^4}{384(E_s I_s + E_c I_c)} = \frac{5(3.821)\,28^4\,(1728)}{384(29000(4883) + 4866(572))}$$

$$= \frac{2.029 \times 10^{10}}{384(143669060)} = 0.366 \text{ in}$$

$$f_n = 0.18\sqrt{\frac{g}{\Delta_{slab} + \Delta_{girder}}} = 0.18\sqrt{\frac{386}{0.0971 + 0.366}}$$

$$= 5.20 \text{ Hz}$$

minimum f_n

$$f_{n_{min}} \geq 2.86\left[\ln\left(\frac{K}{\beta W}\right)\right] \Rightarrow \text{estimated}$$

$\beta = 0.05$

$K = 25$ kips

$$W = \omega_s(B)l \Rightarrow B = l$$

$$= 0.12(30')(30') \quad l = 30'$$

$$= 108 \text{ k}$$

$$\geq 2.8\left[\ln\left(\frac{25}{0.05(108)}\right)\right] = 4.29 \text{ Hz}$$

$$f_n \geq f_{n_{min}}$$

$$5.20 > 4.29 \therefore \text{OK}$$

Floor Vibration

ADAPT ANALYSIS March 23, 2010 3

f_n as a function of %g

$$\frac{a}{g} = \frac{P_o e^{(-0.35 f_n)}}{\beta W} = a = \frac{P_o e^{(-0.35 f_n)}}{\beta W} g < 0.25\%g$$

From ATC 1999
Figure 4

P_o = 150 assumed weight (0.53) From Figure 1 ADAPT TN290
= 79.5 with walking speed of 2 Hz

f_n = 5.2 Hz ⇒ from calculation on previous page

β = 0.05 ⇒ ADAPT Technical note TN290 3/21/09 Table 1

g = gravitational constant

W = weight of floor section

= 30' × 28' (68 psf + 25 psf topping + 35 psf) = 107.52 k

$$a = \frac{79.5\, e^{(-0.35(5.2))}}{0.05(107.52)(1000)} g = 0.002396 g \Rightarrow 0.2396\% g$$

0.2396 % g < 0.25 % g ∴ OK

ADAPT Technical Note

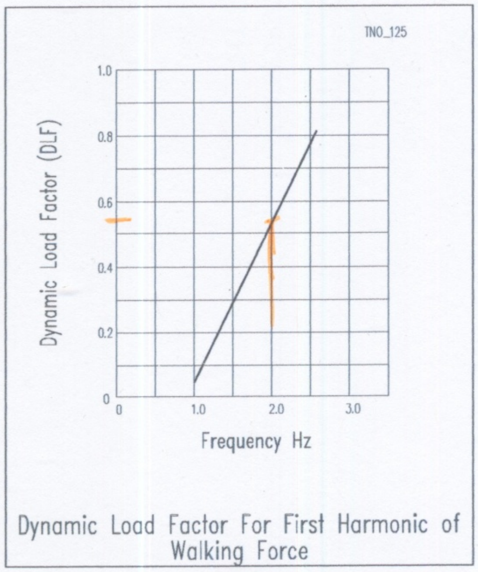

FIGURE 1

Damping

Damping has an inherently high variability that is difficult to determine before a floor system is placed in service. The recommended values from reference [Allen, D.E., and Murray, T. M., 1993] vary from 2-3% for bare concrete floors to 5-8% with full height partitions. Damping factors suggested in the same reference are listed in Table 1.

TABLE 1 RECOMMENDED DAMPING FACTORS FOR VARIOIUS OCCUPANCIES

Occupancy	Damping factor β
Bare concrete floor	0.02
Furnished, low partition	0.03
Furnished, full height partition	0.05
Shopping malls	0.02

Extent of Cracking

Cracking reduces floor stiffness and, consequently, lowers its natural frequency. For conventionally reinforced concrete it is important to allow for cracking. Otherwise, the results are likely to be on the unconservative side. For conventionally reinforced flat slab construction with span to depth ratio of 30 or larger, a 30% reduction in stiffness is reasonable. For post-tensioned floors designed according to IBC [IBC, 2006], allowable tensile stresses are low so reduction in stiffness is not necessary. Designs

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[ATC, 1999] addresses the same issue and recommends the threshold of human sensitivity to vertical vibration as shown in Fig. 4. Other references state somewhat different values. In most cases the perceptibility is related to the response acceleration of the floor system for different natural frequencies of the floor. The common consensus among the investigators is that humans are most sensitive to vibration for frequencies between 4 to 8 Hz. Larger acceleration values can be tolerated at higher or lower frequencies.

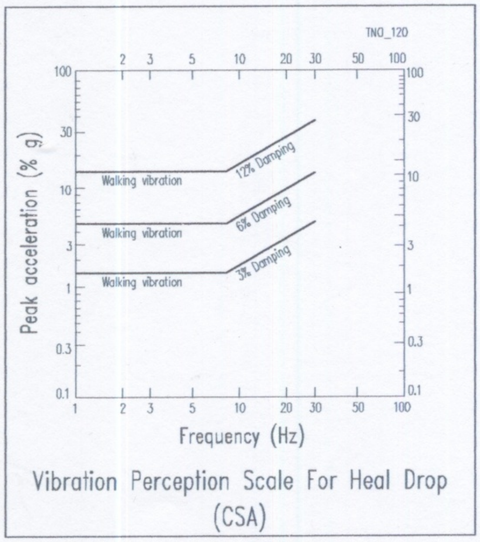

FIGURE 3

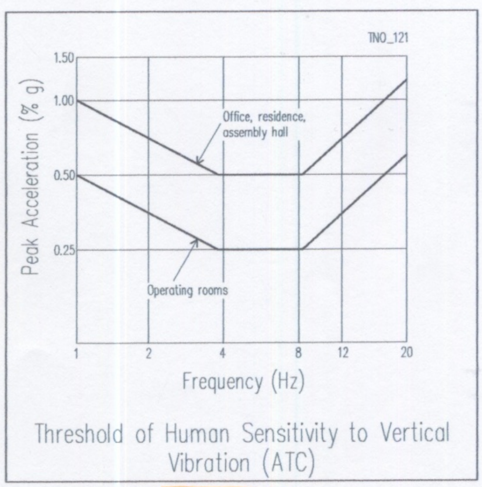

FIGURE 4

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PEAK ACCELERATION AND ACCEPTABILITY OF VIBRATION

To evaluate the vibration of a floor system, designers must determine the floor's peak acceleration response from foot drop, since the acceleration response is one of the two prime parameters in perception of vibration. Peak acceleration is obtained from the first natural frequency of a floor. [ATC, 1999; AISC/CISC 1997] recommends the following relationship:.

$$\frac{a_p}{g} \leq \frac{P_0 e^{-0.35fn}}{\beta W} \qquad (7)$$

where

a_p = peak acceleration;
g = gravitational acceleration [32.2 ft/sec^2; 9.81 m/sec^2];
P_o = constant force representing the walking force;
β = modal damping ratio, recommended in Table 1;
W = effective weight of the panel and the superimposed dead load; and
f_n = first natural frequency.

The calculated response acceleration is compared with the minimum acceptable value given by equation 8 [walking [Allen, D.E., and Murray, T.M., 1993]] and the levels per perceptibility (Fig. 4).

Quoting from [Mast, 2001] people are most sensitive to vibration when engaged in sedentary activity while seated or lying. Much more is tolerated by people who are standing, walking, or active in other ways. The following empirical formula, based on resonant effects of walking, has been developed to determine the minimum natural frequency of a floor system needed to prevent disturbing vibration caused by walking [Allen, D.E., and Murray, T.M., 1993]

$$f_n \geq 2.86 \ln\left(\frac{K}{\beta W}\right) \qquad (8)$$

where

K = a constant, given in [Table 3];
β = modal damping ratio [Table 2];;
W = weight of area of floor panel affected by the point load (heel drop); and
f_n = minimum frequency.

For the first natural frequency and the peak acceleration calculated the acceptability of the floor for vibration perception is compared to and matched against the suggested values of Fig. 4.

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