Fraser Centre

State College, Pennsylvania

Thesis Final Report

Tyler Strange

Structural Option

AE Consultant: Dr. Thomas Boothby

April 7, 2011

Table of Contents

Project Abstract

STATE COLLEGE, PA

Fraser Centre

Tyler Strange
Structural

http://www.engr.psu.edu/ae/thesis/portfolios/2011/trs5062/index.html

PROJECT INFORMATION

Location: Fraser St./Beaver Ave. State College, PA
Occupancy Type: Retail/Office/Hotel/Condos
Size: 230,000 SF
Stories Above Grade: 10 (1 Below Grade)
Construction Dates: Fall 2010 - Fall 2012
Project Cost : Estimated $40M
Project Delivery Method: Design - Bid - Build

PROJECT TEAM

General Contractor: Leonard S. Fiore
Construction Manager: Leonard S. Fiore
Architect: Wallace Roberts & Todd, LLC
Landscape Architect: Wallace Roberts & Todd, LLC
Structural Engineer: David Chou & Associates, Inc.
MPE/Fire Prot. Engineer: AKF Engineers
Civil Engineer: L. Robert Kimball & Associates
Theatre Engineer: JKR Partners, LLC

ARCHITECTURE

The Fraser Centre is a mixed-use building containing a parking garage, retail stores, a theatre, condominiums, and penthouse suites. The base of the Fraser Centre is a glass curtain wall drawing passers-by off of the sidewalk and into the retail stores on the ground level. Above the curtain wall is an appealing mixture of glass and aluminum composite panels creating an attractive addition to down town State College. This building is the only one in State College that has an all glass and aluminum façade.

STRUCTURE

Foundation consists of isolated and continuous footings
Isolated footings range from 4'x4' and 24" thick to 15'x15' and 42" thick
Continuous footings range from 3' wide 16" thick to 6' wide 54" thick
CIP concrete slab and columns resist gravity loads
Lateral loads are resisted by shear walls throughout the building
Slabs are primarily 12" thick reinforced with #5 rebar
Mechanical floor slab is 16" thick and reinforced with # 9 rebar

MEP

The HVAC System of Fraser Centre consists of the following:

- Type of System
  - Constant volume air system (12,000 cfm)
  - Constant flow hydronic system
  - Variable lfow hydronic system
- Unit Heaters (Electric-resistance heating coils)
  - Cabinet unit heaters with centrifugal fans
  - Propeller unit heaters
- Open-circuit, induced draft, cross flow cooling tower
- Condensing Boilers
  - Gas fired
  - Pulse combustion
  - Fire-tube
  - Water-tube
  - Water-jacket
- Hydronic Pumps
  - Closed-coupled/in-line pump
  - Closed-coupled/end-suction pump
  - Automatic condensate pump unit
- Axial HVAC Fans
  - Tubeaxial fans
  - Vaneaxial fans
  - Mixed-flow fans
- Centrifugal HVAC Fans
  - Airfoil fans
  - Forward-curved fans
- Diffusers
  - Rectangular/square ceiling diffusers
  - Perforated diffusers
  - Louver faced diffusers

Executive Summary

Fraser Centre is a mixed-use, high-rise development located in State College, Pa. The 11 story structure has been designed using a two-way concrete slab with concrete shear walls.

In Technical Report 3, lateral loads were found to be resisted by two shear walls on the east end of the building. In an effort to reduce the torsion created by this configuration, shear walls on the theater level were extended throughout the building. The new shear walls were then redesigned for the new load distribution. With the new layout of shear walls an alternate floor system, composite deck, was also studied.

Two non-structural breadth analyses were also undertaken. An analysis and slight redesign of the architectural layout of the residential floors was conducted. This analysis determined that the shear wall layout had a minimal impact on the architectural floor plan. In addition to the architectural redesign a cost and schedule analysis was completed for the existing design and the new design. This analysis helped determine if the proposed changes were economical.

With these analyses it was determined that the proposed changes were not economical or recommended. The floor system did reduce the building weight, but it also increased the building height. The cost and schedule analysis showed that the new floor system reduced the construction time but also significantly increased the cost of the project. The new shear wall layout had very little impact on the architecture of the residential floors but is not a recommended change if the proposed floor system is not going to be used.

Acknowledgements

I would like to extend acknowledgments to the following individuals and firms:

- Susquehanna Real Estate LP
- Leonard S. Fiore, Inc.
- The Pennsylvania State University
    - Professor M. Kevin Parfitt
    - Professor Robert Holland
    - Dr. Thomas Boothby
    - The entire AE faculty and staff

I would also like to give a special thank you to my family, friends, and fellow classmates for all the support and encouragement.

Introduction

Figure 1: Site view of Fraser Centre (blue) bounded by Fraser St., Calder Way, Miller Alley, and Beaver Ave. Photo courtesy of Bing Maps.

The Fraser Centre is a mixed-use, high-rise development located in downtown State College, Pennsylvania (See Fig. 1). The site will encompass an entire block on the corner of Beaver Avenue and Fraser Street, at an approximate elevation of 1100 feet above sea level. The development was designed by Wallace, Roberts, and Todd LLC, to be the only building in State College to have an all glass and aluminum façade. The structure was engineered by David Chou and Associates, Inc.; the MEP was engineered by AKF Engineers; and the theater was engineered by JKR Partners, LLC.

Fraser Centre is an eleven story multi-use building. The first floor is exclusively parking; with 94 parking spaces. Residential parking takes up the majority of the second floor along with the theater lobby and 3 retail spaces. The entire third floor is occupied by the ten-auditorium movie theatre. The mechanical equipment is located on the fourth floor, or mechanical floor. At the fourth floor the building foot print reduces from roughly 270ft x 165ft to 190ft x 76ft. Floors five through eleven are all residential levels; floor five consists of nine units, levels six through ten all have eight units, and three penthouse suites makes up the penthouse or eleventh floor.

The structural system of Fraser Centre is reinforced concrete. The gravity load resisting system consists of concrete columns, shear walls, and two-way slabs. The lateral system is composed of reinforced concrete shear walls located throughout the entire building.

Existing Structural Systems

Gravity System

Columns are designed with 5000 psi concrete for the columns below the sixth level and 4000 psi concrete will be used for columns above the sixth level. Figure 2 in the Appendix shows the column locations and the column size and reinforcement can be found in Figure 3a through 3g. Column sizes vary from 18”x24” and 16”x32” to 24”x72” and 36”x60” and there are also 24” diameter columns.

Beams on level 2 garage vary in width from 10” to 36” with 18” being the most common and a depth between 24” and 111”, 30” is the most common depth. The theater level beams vary from 12” to 72” and 20” to 48” in width and depth respectively. Beams vary in depth from 24” to 40” and 16” to 48” on the mechanical floor. 12”x 78” and 48”x30” is the range of beams on the roof. All beams are made with 4000 psi concrete.

The parking garage has 9” slabs on grade reinforced with 13#5 bars on top and a bottom grid of #4 bars at 12” each way. 4000psi concrete will be used for the slab on grade. 18#5 top bars and a grid of #5 bottom bars at 12” reinforce the 14” concrete slab of the theatre level. In addition to #7 bottom bars at 9” East-West and #5 bottom bars North-South in the 16” slab, the mechanical floor also has a 12’-6”x7’ transfer girder with 40 #11 bottom bars and 20 #11 top bars. The residential levels and penthouse (5 through 11) as well as the roof have 12” slabs reinforced with a grid of #5 bars at 14” east-west and 12” north-south. All of the structural slabs will have 5000 psi concrete and a typical span of 40 feet. Steel beams are used for the projection of the mezzanine floor, and they vary from W8x10 to W12x22.

Lateral System

Concrete shear walls will be used in Fraser Centre to resist lateral loads. Shear walls are composed of 5000 psi concrete and reinforced with #5 horizontal bars and #6 vertical bars. Shear walls are located along column lines 3, 4, 5, 6, and 7 as shown in Figures 2 and 3. The theatre level has 14” shear walls and 16” walls are typical of the parking levels and the residential levels.

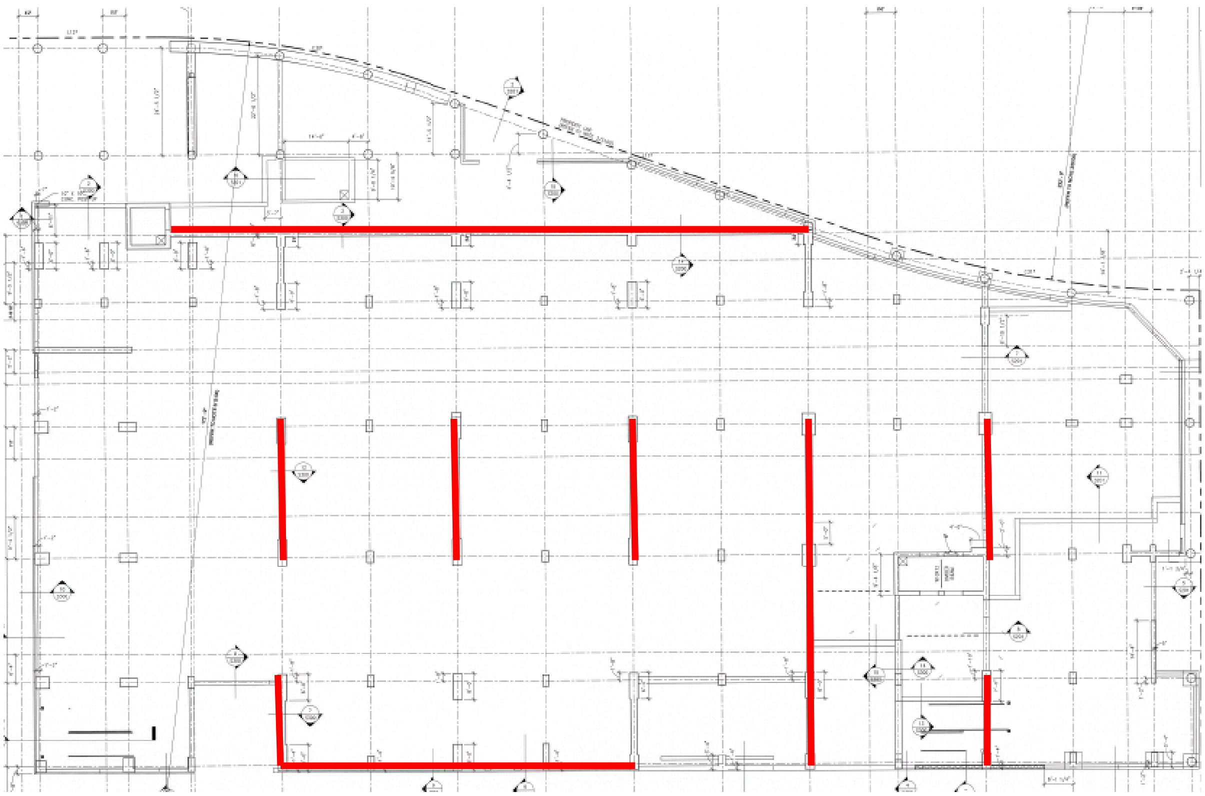


Figure 2: First Floor Shear Wall Plan

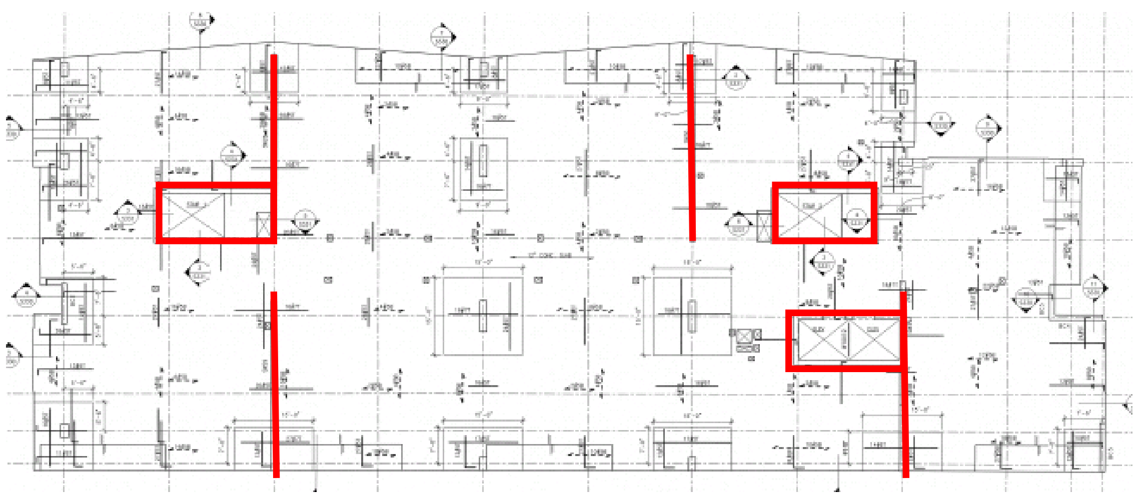


Figure 3: Typical Residential Floor Shear Wall Plan

Current shear walls

Design Criteria

The following data is provided to illustrate the general design criteria for Fraser Centre.

Codes & Design Standards

Applied to Original Design
International Building Code **IBC 2006**
American Concrete Institute Building Code **ACI 318-05**
American Institute of Steel Connection **AISC, 9th Edition**
Steel Deck Institute **SDI Specification**
Building Code Requirements for Masonry Structures **ACI 530-05**
American Society for Civil Engineers **ASCE 7-05**

Substituted for Analysis
International Building Code **IBC 2006**
American Concrete Institute Building Code **ACI 318-08**
American Institute of Steel Connection **AISC, 13th Edition**
American Society for Civil Engineers **ASCE 7-10**

Table 1: Codes and Standards used for Original Design and Analysis.

Material Strength Requirements

Material	Strength Requirement
Cast –In-Place Concrete:	
Footings	4 ksi NWC
Basement and Bearing Walls	4 ksi NWC
Shear Walls and Columns	5 ksi NWC
Grade Beams and Slab on Grade	4 ksi NWC
Structural Slab	5 ksi NWC
Reinforcement	ASTM A615, Grade 60
Structural Steel:	
Steel Shapes	ASTM A992
Structural Tubes	ASTM A500
Plates	ASTM A36

Table 2: Material Strength Requirements per drawing S001

Dead and Live Loads

Area	Design Live Load (psf)
Roof/Ground Snow (from drawing S001)	Min 40
Mechanical	125
Rooms	40
Stairs/Public Rooms/Corridors/ Balconies	100
Theater	60
Retail Sales	100
Light Storage	125

	Design Super-Imposed Dead Load (psf)
Roofing	10
Partitions	20
4” Hollow Non-Bearing Block	30 (/sf of wall)
8” Hollow Non-Bearing Block	55 (/sf of wall)
Brick Veneer	40 (/sf of wall)

Table 3: Design Live and Super-Imposed Dead Loads per drawing S001

Problem Statement

In Technical Report 3, wind loads were found to be the controlling load condition for the structure of the Fraser Centre. There are two components of every lateral load, direct and torsional. One way to reduce the lateral loads experienced by the lateral force resisting system is to reduce the eccentricity between the center of rigidity and the resultant force.

Proposed Solution

Since the controlling lateral load was determined to be seismic, a center of rigidity closer to the center of mass would reduce the torsional component of the wind load. By continuing the shear walls of the theater up to the roof the lateral force resisting system will be more evenly distributed than the current layout. The current layout only has two shear walls that continue from foundation to roof located on the east side of the building.

With the change to the shear wall layout, a composite slab and beam floor system will be used instead of the current two-way concrete floor. The composite nature of the proposed slab also allows for a thinner slab which reduces the weight and there for the seismic loads felt by the building. This will further reduce the load the shear walls will experience.

Changing the floor system to a composite system could also reduce the construction time allowing for earlier occupancy. This would result the not having to place and remove formwork for the floor. A composite floor will allow faster construction by erecting beams, girders, and metal deck instead of formwork.

Gravity System Design

Composite Deck/Slab Design:

Utilizing the Vulcraft design guide and catalog a deck/slab section was chosen with the following properties:

3VLI 18 Deck

f'c=4 ksi (normal weight concrete)

total slab thickness= 4.5"

18 gage

Composite Weight= 75 psf

3 span construction

Maximum Unshored Span= 12'-0"

Clear span= 10'-0"

Maximum Superimposed Load= 246 psf

Superimposed Load= Live + Dead= 135 psf

Full Composite Action:

The floor system employs full composite action. Full composite action allows the concrete floor slab to play a more significant role in the beam design. Shear connectors transfer all the compressive forces to the concrete slab while the tensile forces are resisted by the steel beam.

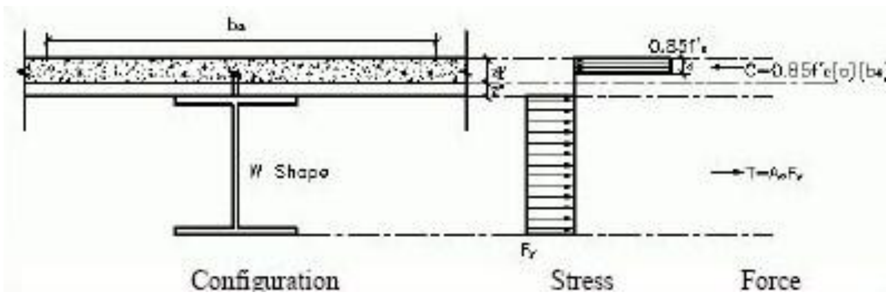


Figure 4: Full Composite Action

Partial Composite Action:

In partial composite action, the shear connectors transfer only a fraction of the compressive forces from the beam to the concrete slab.

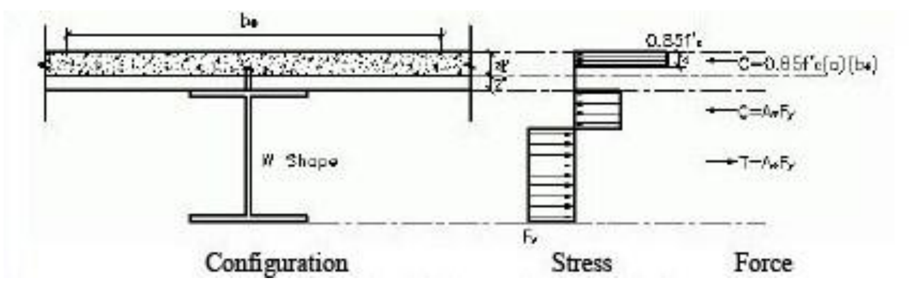


Figure 5: Partial Composite Action

The composite deck design will add an additional 13” to the floor depth. This increase in depth was managed by decreasing the floor to ceiling height 7” and increasing the floor to floor height 6”, this added an additional 3’ the overall building height. Calculations of individual structural members can be found in Appendix A.

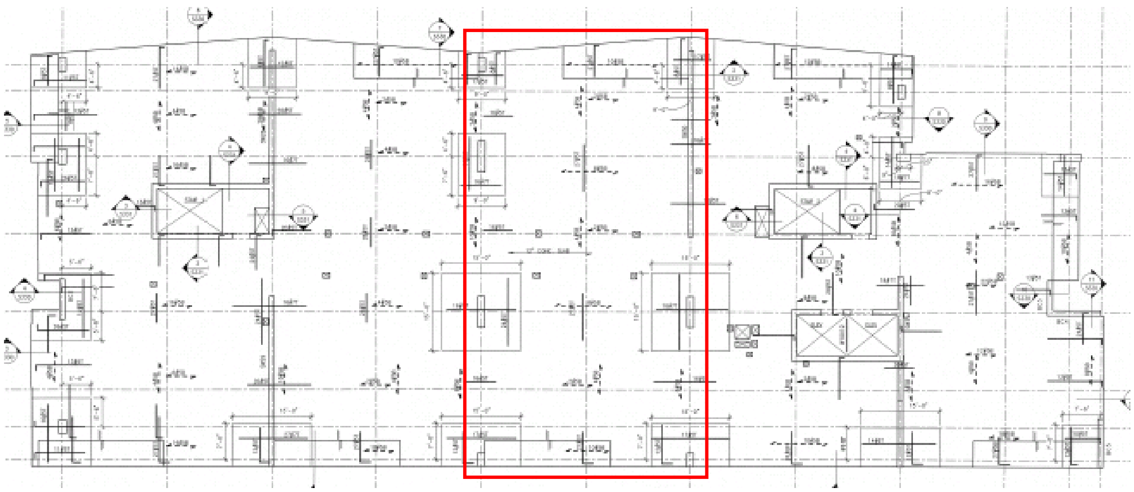
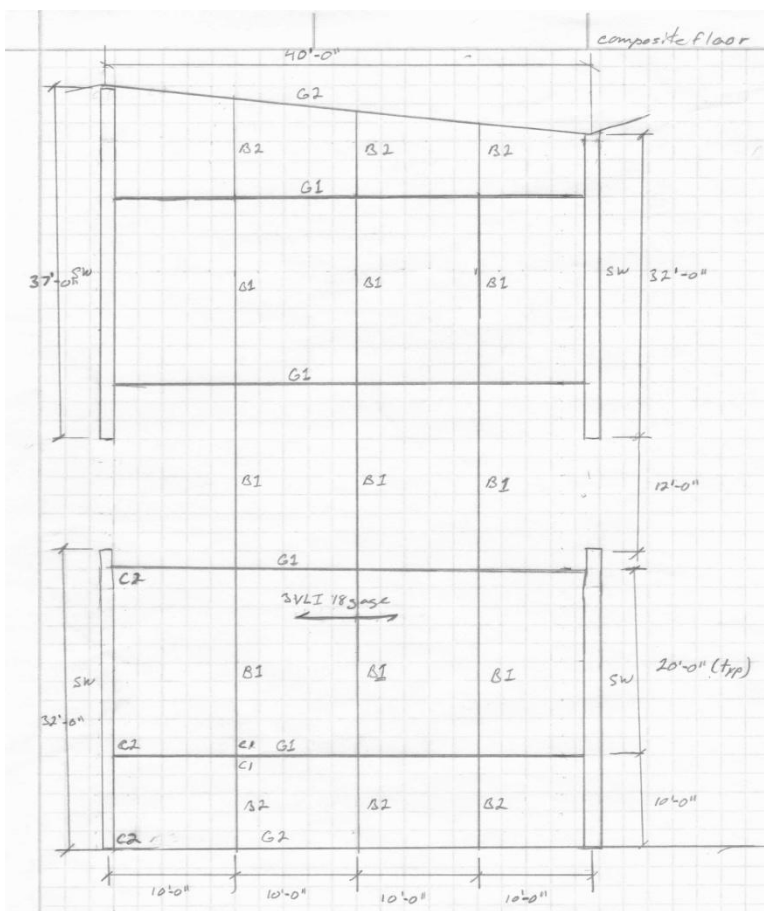


Figure 6: Typical Bay Location



Beams:
B1: W12x16
B2: W10x12
Girders:
G1: W18x76
G2: W16x31

Figure 7: Typical Composite Layout

Lateral System Design

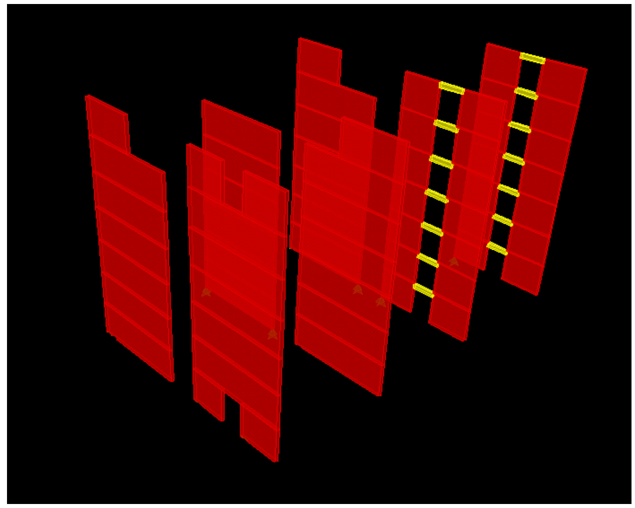


Figure 8: ETABS Model of Residential Shear Walls

The lateral system was modified to utilize more continuous concrete shear walls. The shear walls were designed using an ETABS model and checked with hand calculations. Due to the lack of flexibility of the parking level floor plan, the shear walls were unable to be extended from the theatre to the foundations. As a result of this the shear walls were only extended upwards through the residential levels. The ETABS model of the shear walls of the residential floors was created as shown in Figure 8. Wind and seismic loads were applied to the model and drift was compared for the wind and seismic load cases. It was determined that seismic loads will control the design of the building. The shear walls are 14" thick. The design output from ETABS as well as the hand check can be found in Appendix B.

The deflections caused by the wind load were compared to L/400 for building displacement story drifts were also compared to 0.3 in, the lower limit of L/400 for story drift. At the roof, the maximum deflection above the 5th floor was 0.187" which passed the L/400 requirement of 3.31". Table 4 summarizes the displacement and story drifts due to the wind load.

Level	Displacement	Drift
Roof	0.187	0.036
10	0.151	0.034
9	0.117	0.033
8	0.084	0.030
7	0.054	0.026
6	0.028	0.018
5	0.010	0.010

Table 4: Wind Displacements

Deflections due to the seismic load were compared to 0.025h, the allowable seismic drift. The maximum displacement at the roof was 0.392" which passed the 0.025h requirement of 2.64". Table 5 summarizes the displacement and story drifts due to the seismic load.

Level	Displacement	Drift
Roof	0.392	0.077
10	0.315	0.072
9	0.243	0.070
8	0.173	0.067
7	0.110	0.053
6	0.057	0.039
5	0.018	0.018

Table 5: Seismic Displacements

MAE Topics

As required for this project, important concepts and skill sets pertaining to MAE coursework were integrated into the proposed solution. To redesign the lateral system computer modeling software and techniques learned in AE 597A will be used. Material properties were modified as necessary. Point masses were employed for floor mass as well as the application of seismic loads. A rigid diaphragm was accomplished by imposing translation constraints on points of individual floors. Shear wall meshing was used to obtain more realistic results.

Steel connections were also designed for the composite floor using knowledge from AE 534. Using processes learned in AE 534 steel connections were designed using Table 10-11 from the AISC Steel Manual. Welded/bolted connections were implored for beam/girder connections and bolted/bolted connections were used for girder/shear wall connections. Steel connection calculations can be found at the end of Appendix A.

Construction Management Breadth

The goal of the construction management breadth was to compare the impact on the cost and schedule when changing the original system to a composite system. A cost comparison of the proposed and original systems was undertaken to determine the economic effects of the changes. This comparison included the impact the changes will have on the schedule. The material and construction costs of the composite system were then compared to the costs of the current system. Since a delay in opening would cost the owner in lost income, it is important to not impact the schedule in a negative manner.

From the cost analysis of the original system, the floor for a single residential level would cost approximately $2,310,000 with 95% of the cost coming from placing form work. The composite floor is estimated at $7,050,000 and 77% of the cost coming from the metal deck. The price difference suggests that a composite floor system is not a viable alternative; the total increase in cost for all seven residential levels would be roughly $33,000,000.

Description	Total Cost (O+P)	Cost Estimate
In Place Forms	11.29	$2,212,344
In Place Reinforcing	1625.00	$3,432
4 ksi Ready Mix	92.50	$60,492
Placing Concrete	20.50	$37,658
	Total	**$2,313,926**

Table 6: Current System Cost Analysis

Description	Total Cost (O+P)	Cost Estimate
Structural Steel W16x31	38.50	$260,360
Structural Steel W18x76	88.00	$829,142
Structural Steel W10x12	21.00	$74,388
Structural Steel W12x16	22.63	$333,574
Metal Decking	3.12	$5450,667
Weld Shear Conn	2.11	$66,409
4 ksi Ready Mix	92.50	$20,234
Placing Concrete	20.50	$12,596
	Total	**$7,047,374**

Table 7: Alternative System Cost Analysis

When comparing the impacts on the schedule a single crew was originally assumed with a slight overlap in progress. This resulted in the form work requiring 29 days to place and the original system taking 26 days longer to complete one floor. In an effort to reduce the schedule difference two crews were used to place the form work, all other tasks were still completed by a single crew. The additional crew allowed the formwork to be placed in 17 days and reduced the original system's schedule to 23 days. The proposed system is scheduled to complete a single floor in 9 days. By changing to a composite floor system the project could be completed 36 days earlier. By completing the project earlier the owner will be able to occupy the building and begin charging tenants earlier. Although early completion is a significant improvement to a design, in this case it was assumed that the early completion did not offset the increase in cost. The complete breakdown and schedule can be located in Appendix E.

Description	Daily Output	Schedule (Days)
In Place Forms (2 crews)	931.0	17
In Place Reinforcing	2.9	7
Placing Concrete	180.0	3

Table 8: Current System Task Duration

Description	Daily Output	Schedule (Days)
Structural Steel W16x31	900	1
Structural Steel W18x76	900	1
Structural Steel W10x12	600	1
Structural Steel W12x16	880	1
Metal Decking	2850	6
Weld Shear Conn	910	1
Placing Concrete	180	1

Table 9: Alternative System Task Duration

Architectural Impact

The new layout of the shear walls for the residential floors can be seen in figure 9 and 10. After examining the new shear walls and the current architectural layouts for the residential floors it was determined that the new layout will only have a minor impact on the architecture. Figure 12 and 11 show the only walls that impact the layout of floors 5 through 6. Figures 13 through 16 show the impact of the shear walls on the layout of the 11th floor.

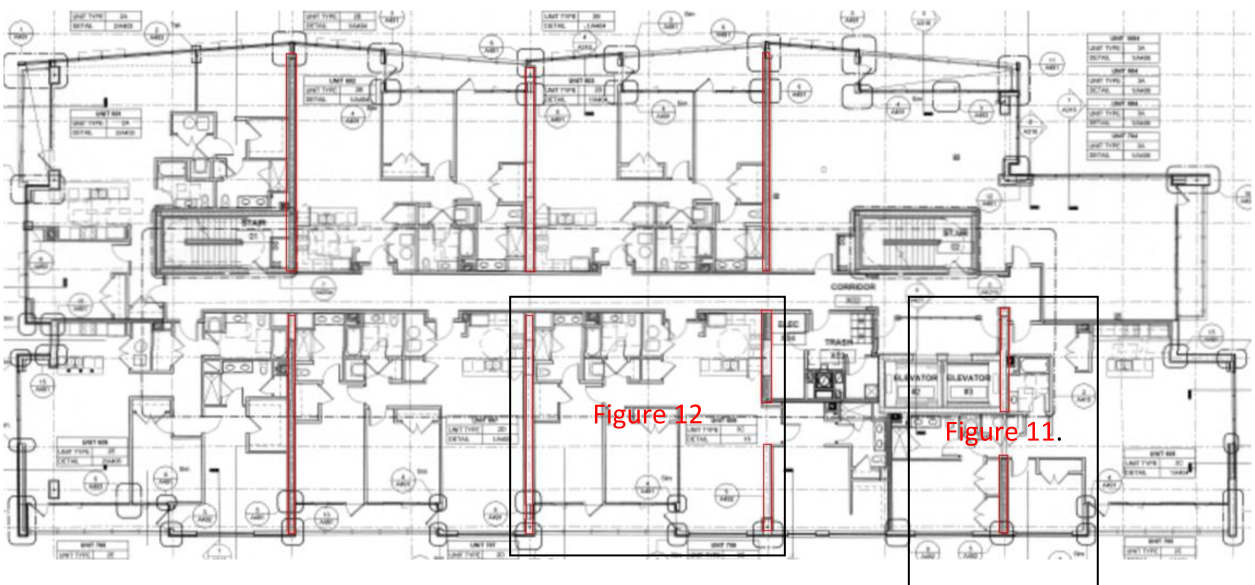


Figure 9: New Layout for Floors 5-10

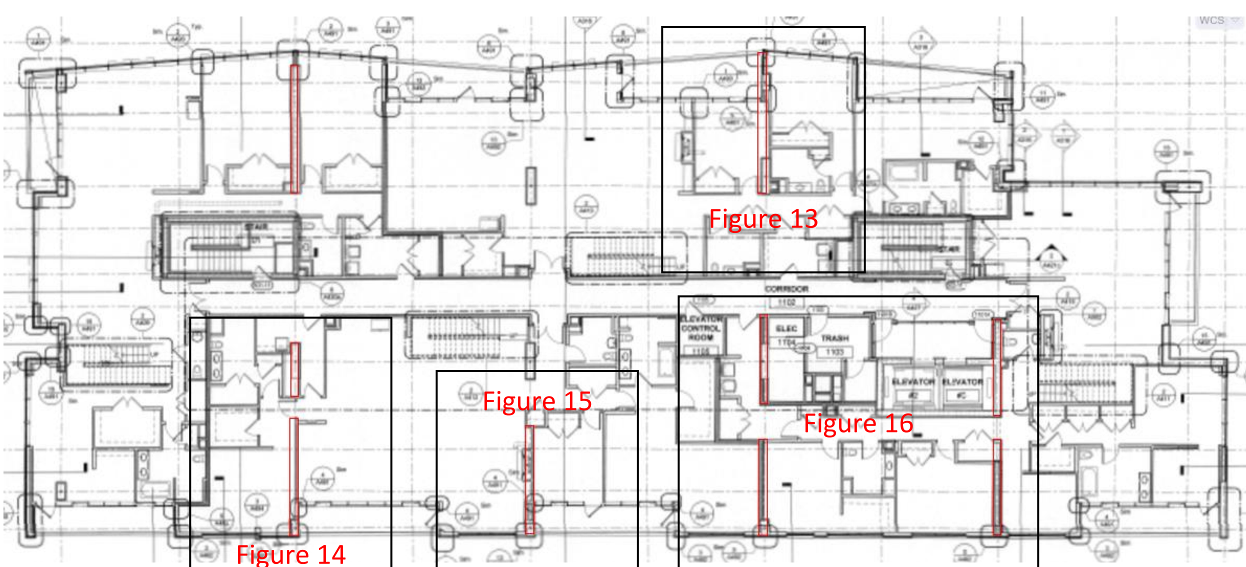


Figure 10: New Layout for 11th Floor

As shown in Figure 11, there is no architectural impact from the new shear walls at that location. The shear walls within the area marked for Figure 12 will have a slight impact on the architecture of the Living/Dining Room. The width of the southern part of the room will become 1'-2 ¼" shorter, but this will also create a more regular space by removing the setback in the eastern wall.

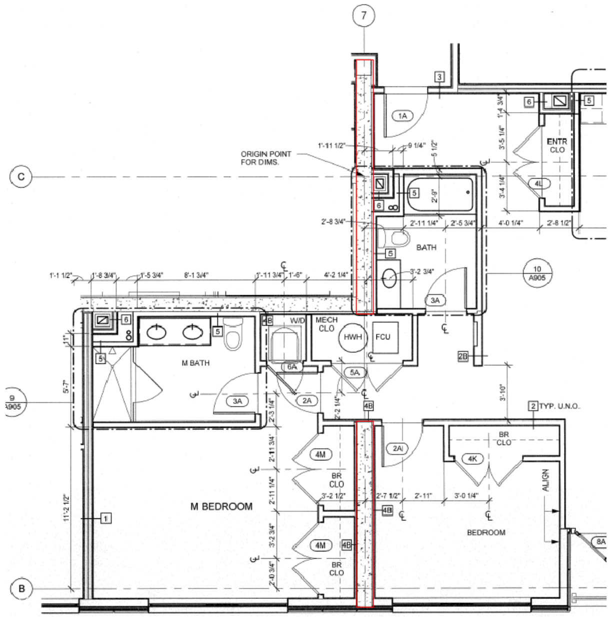


Figure 11: New Layout for 11th Floor

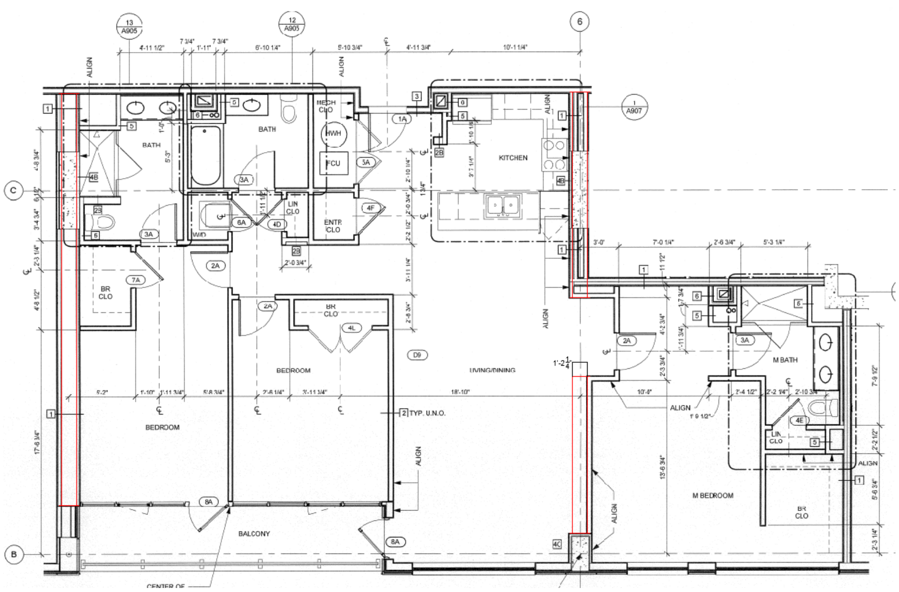


Figure 12: New Layout for 11th Floor

The architecture of the eleventh floor will need to be altered more than the lower levels in order to accommodate the new shear walls. The bedroom shown in Figure 13 will become a rectangular room 9'-6" wide when the east wall is brought out 1'-1 ½". In Figure 14 the bedroom and dining room will both be affected. The wall will encroach into the bedroom 6" and the dining room 3 ½". The 6" in the bedroom will result in the partition forming an entryway being shortened by 6". In the living room shown in Figure 15 the wall on the eastern side will become straight after the 1' setback is removed. Figure 16 shows how the width of the bedroom in that area is only shortened by 9" which will change the room from being more or less square to a possibly more interesting rectangle. As can be seen from the Figures the new shear layout will have a minimal impact on the current layout and architecture of the residential floors.

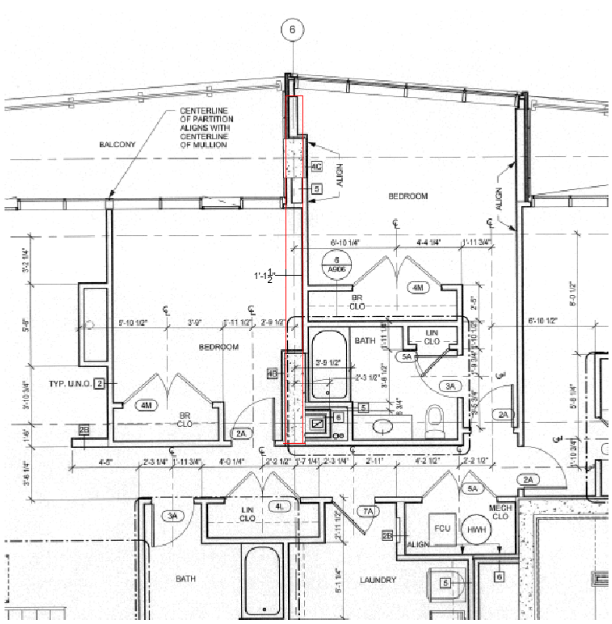


Figure 13: Architecture Impact for 11th Floor

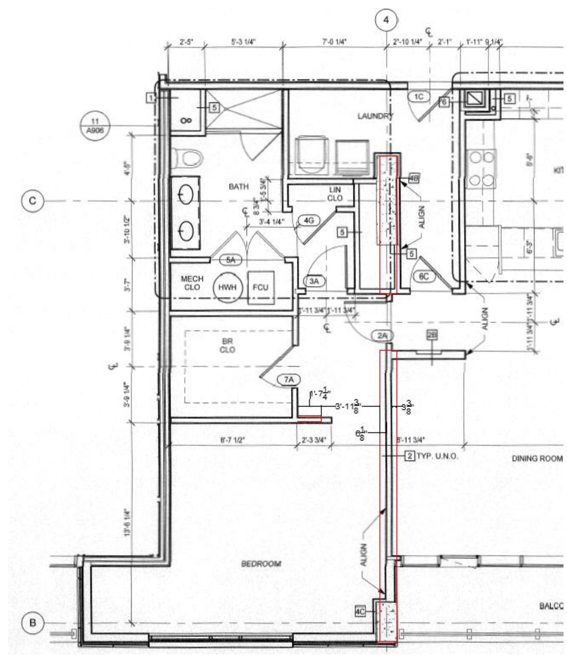


Figure 14: Architecture Impact for 11th Floor

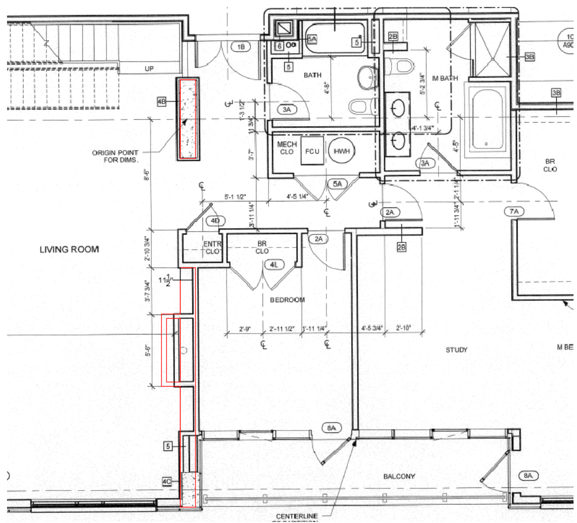


Figure 15: Architecture Impact for 11th Floor

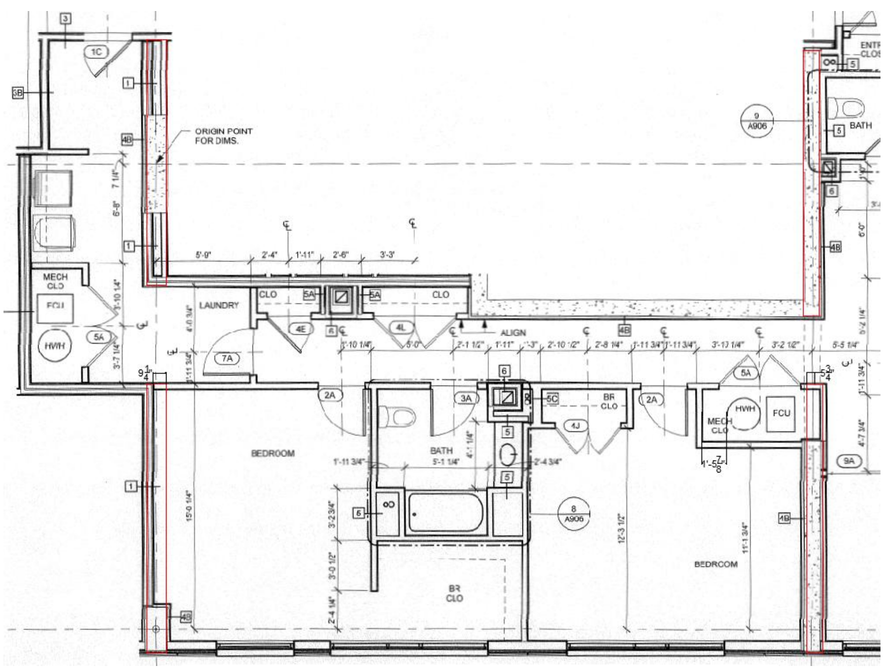


Figure 16: Architecture Impact for 11th Floor

Summary and Conclusions

Using a composite beam system for Fraser Centre does not seem to be a viable alternative to the current system. The composite floor system allowed the project to be completed over a month earlier, and although this is a very large positive it is negated by the additional $4.7 million per floor cost involved in constructing the system. The cost of using the steel deck was the single most influential factor in the new floor system. The composite system provided a lighter floor for seismic design while increasing the depth of the floor to 13". This increase in depth and additional cost of this system is why it is not a viable alternative. The new shear wall layout resulted in less torsion and resulted in 14" thick shear walls in the residential floor. These additional walls will have a minimal impact on the architecture of those floors. In conclusion, the composite floor system and shear wall layout is not a recommended alternative to the current system.

References

International Building Code (IBC) 2006

American Concrete Institute (ACI) Building Code 318-08

American Institute of Steel Construction (AISC) 13th Edition

American Society of Civil Engineers (ASCE) 7-10

RS Means "Building Construction Cost Data" 2005 Edition

ETABS (Computer modeling program)

Appendix A: Composite Floor Design

composite floor page 1 of

loads

live loads - 40 psf
superimposed dl - 20 psf

$4\frac{1}{2}$" concrete - 2 hr rating

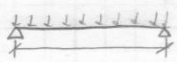
3" VLI vulcraft composite deck 18 gage ⇒ weight = 75 psf

max unshored clear span 12'-0" ⇒ 3 span

clear span = 10'-0"

live load + dead load = 40 psf + 20 psf + 75 psf = 135 psf < 181 psf ∴ ok

beam design B1



20 ft

$V_u = \frac{w_u l}{2} = \frac{1.87(20)}{2}$

$V_u = 18.7$ kips

$M_u = \frac{w_u l^2}{8} = \frac{1.87(20)^2}{8}$

$M_u = 93.5$ 'k

tributary width : 13.33'

approx selfweight

DL = 75(1.1) + 20 = 102.5 psf

LL = 40 psf

q_u = 1.2(102.5) + 1.6(40) = 187 psf

W_u = 187(10) = 1.87 k/ft

assume a = 1.5"

Y = 7.5"

$Y_2 = 7.5 - \frac{1.5}{2} = 6.75$

use Y_2 = 6.5 (to be conservative)

try W10 × 12 ⇒ ϕM_p = 47.3

$Y1 = 0.210$" ⇒ ϕM_n = 110 'k

ΣQ_n = 93.6 k

\# studs = $\frac{\Sigma Q_n}{Q_n} = \frac{93.6}{17.2} = 5.4$ ⇒ 6 studs ⇒ 12 studs across entire beam

3/4" studs; weak; ⊥ deck

$M_{u_b} = \frac{0.012(20^2)}{8} = 0.6$ 'k

M_u = 94.1 'k < 110 'k ∴ ok

check assumption

$b_{eff} = \min \begin{cases} 10' \\ 20/4 = 5' \end{cases}$

b_{eff} = 5'(12) = 60"

$a = \frac{\Sigma Q_n}{0.85 f'_c b_{eff}} = \frac{93.6}{0.85(5)(60)}$ = 0.38" < 1.5" ∴ conservative

check construction load

DL = 82.5(10) + 12 = 837 plf ⇒ 0.837 k/ft

$M_c = \frac{0.837(20^2)}{8}$ = 41.85 'k < 47.3 'k ∴ ok composite floor   page 2 of 4

check deflection

$\Delta_{cons} = \frac{5wl^4}{384EI} = \frac{5(0.837)(20^4)(1728)}{384(29000)(53.8)} = 1.93''$

$\Delta_{limit} = \frac{l}{360} = \frac{20(12)}{360} = 0.67'' < 1.93''$ ∴ no good with ½" camber $I_{req'd} = 88.8 in^4$

use W12×16 ⇒ $I = 103 \Rightarrow \Delta_{cons} = 1.01'' \Rightarrow$ ½ in camber $\Rightarrow \Delta_{cons} = 0.51 in < 0.67 in$ ∴ ok

$\Delta_{live} = \frac{5(0.04)(10)(20)^4(1728)}{384(29000)(103)} = 0.48'' < 0.67 = l/360$ ∴ ok

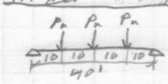
check shear

$V_u = 18.7^k$

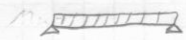
$\phi V_n = 79.1^k$

$V_u < \phi V_n$ ∴ ok

girder design G1



$P_u = 2(18.7) = 37.4^k$ ⇒



$w_u = \frac{37.4(3)}{40} = 2.8^{k/ft}$

$M_u = \frac{w_u l^2}{8} = \frac{2.8(40)^2}{8} = 560^{'k}$

$V_u = \frac{2.8(40)}{2} = 56^k$

AISC table 3-10

unbraced length = 10ft

- economical use W24×68 ⇒ $\phi M_n = 591^{'k} > 560^{'k}$ ∴ ok

$\phi V_n = 295^k > 56^k$ ∴ ok

check deflection

$l/360 = 1.33''$

$\Delta_{live} = \frac{5w_u l^4}{384EI} = \frac{5(0.04)(20)(40^4)(1728)}{384(29,000)(1830)} = 0.868'' < 1.33''$ ∴ ok

use ⇒ - architectural use W18×76 ⇒ $\phi M_n = 600^{'k} > 560^{'k}$ ∴ ok

$\phi V_n = 232^k > 56^k$ ∴ ok

$\Delta_{live} = \frac{5(0.04)(20)(40^4)(1728)}{384(29,000)(1330)} = 1.19'' < 1.33''$ ∴ ok

22.2 inches deep
10.2 deeper than current system

Beam B1 ⇒ W12×16

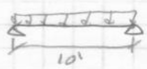
Girder G1 ⇒ W18×76

Beam B2 ⇒ W10×12

Girder G2 ⇒ W16×31

composite floor page 3 of

Beam B2



$q_u = 187\,psf$
$W_u = 1.87\ ^k/_{ft}$

$V_u = \frac{w_u l}{2} = \frac{1.87(10)}{2}$

$V_u = 9.35\ k$

$M_u = \frac{w_u l^2}{8} = \frac{1.87(10)^2}{8}$

$M_u = 23.38\ ^{'k}$

assume $a = 1.5''$
$Y = 7.5''$
$Y2 = 7.5 - \frac{1.5}{2} = 6.75$
use $Y2 = 6.5$ (conservative)

Try W10x12 $\Rightarrow \phi M_p = 47.3\ ^{'k}$

$Y1 = 0.210'' \Rightarrow \quad \phi M_n = 110\ ^{'k}$
$\Sigma Q_n = 93.6\,k$
$\#\,studs = \frac{93.6}{17.2} = 5.4 \Rightarrow 6$ per side $\Rightarrow$ 12 studs across entire beam

$M_{ub} = \frac{0.012(10)^2}{8} = 0.15\ ^{'k}$

$M_u = 23.38 + 0.15$

$M_u = 23.53\ ^{'k} < 110\ ^{'k}\ \therefore$ ok

check assumption

$b_{eff} = \min \left| \begin{array}{l} 10' \\ \frac{10}{4} = 2.5' \end{array} \right. = 2.5(12) = 30$

$a = \frac{\Sigma Q_n}{0.85 f'_c b_{eff}} = \frac{93.6}{0.85(5)(30)} = 0.73'' < 1.5''\ \therefore$ conservative

check construction load

$DL = 82.5(10) + 12 = 0.837\ ^k/_{ft}$

$M_c = \frac{0.837(10)^2}{8} = 10.46\ ^{'k} < 47.3\ ^{'k}\ \therefore$ ok

check deflection

$\Delta_{cons} = \frac{5wl^4}{384EI} = \frac{5(0.837)(10^4)(1728)}{384(29000)(53.8)} = 0.12''$

$\Delta_{limit} = \frac{l}{360} = \frac{10(12)}{360} = 0.33''$

$\Delta_{cons} = 0.12'' < 0.33'' = \Delta_{limit}\ \therefore$ ok

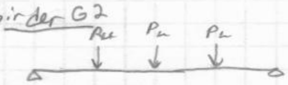
$\Delta_{live} = \frac{5(0.04)(10)(10^4)(1728)}{384(29000)(53.8)} = 0.06''$

$\Delta_{live} = 0.06'' < 0.33'' = \Delta_{limit}\ \therefore$ ok

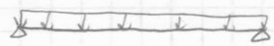
check shear
$V_u = 9.35\,k < 79.1\,k = \phi V_n\ \therefore$ ok $\Rightarrow$ use W10x12

comp floor page 4 of 4

Girder G2



$P_u = 9.35k \Rightarrow$



$w_u = \frac{9.35(3)}{40}$

$w_u = 0.7\,k$

$M_u = \frac{0.7(40)^2}{8} = 140^{1k}$

unbraced length = 10'-0"

$V_u = 14^k$

AISC table 3-10 $\Rightarrow$ W14x30 $\Rightarrow \phi M_n = 144^{1k} > 140^{1k} \therefore ok$

$\phi V_n = 110^{1k} > 14^k \therefore ok$

check deflection

$l/360 = 1.33"$

$\Delta_{live} = \frac{5(0.04)(5)(40^4)(1728)}{384(29000)(291)} = 1.37"$

$\Delta_{live} > l/360 \therefore$ not ok

try W16x31 $\Rightarrow \Delta_{live} = 1.37\left(\frac{291}{375}\right) = 1.06"$

$\Delta_{live} \not> l/360 \therefore ok$

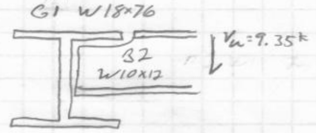
use W16x31

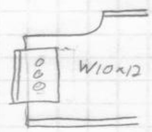
compfloor page 5 of

Steel Connections

Design connection C1: G1 B2 welded / bolted single angle connection

7/4" ∅





AISC table 10-11

2 bolts in vert. row
3/4 in ∅ bolt A325N
L4x3x3/8

(B2) $\phi V_n = 31.8k > 9.35^k \therefore ok$

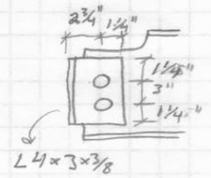
(B1) $\phi V_n = 31.8^k > 18.7^k \therefore ok$

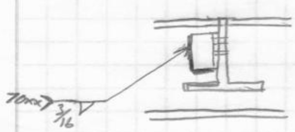
5 1/2" length
3/16" weld

(B2) $\phi V_n = 25.3^k > 9.35^k \therefore ok$

(B1) $\phi V_n = 25.3 > 18.7^k \therefore ok$

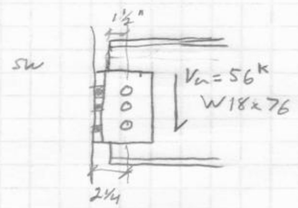
min $t_w = 0.286"$
$t_w = 0.425$





comp floor 6 of

Design connection C2



double angle connection
bolted/bolted

Table 10-1

A325N bolts ¾" Φ

$\left.\begin{array}{l}\text{3 rows of bolts}\\ \text{¼" angle}\end{array}\right\}$ bolt and angle $\phi V_n = 76.4^k$

$L_{eh} = 1\tfrac{1}{2}"$ uncoped $\Rightarrow \phi V_n = 263^k$

$\phi V_n = 76.4^k > 56^k \therefore$ ok

Appendix B: Lateral System Output and Calculations

ETABS v9.7.2 File:LATERAL Units:Kip-in March 28, 2011 17:24 PAGE 1

SUMMARY OUTPUT DATA - UNIFORM REINFORCING PIER SECTIONS - DESIGN (UBC97)

Story Label	Pier Label	Sta Loc	Edge Bar	End Bar	Edge Spacing	Required Ratio	Current Ratio	Pier Leg	Shear Av in^2/ft	B-Zone Length
STORY11	P4	Top	N12	N12	12.000	0.0025	0.0023	T 1	0.420	Not Needed
								T 2	0.420	Not Needed
								T 3	0.420	Not Needed
		Bot	N12	N12	12.000	0.0025	0.0023	B 1	0.420	Not Needed
								B 2	0.420	Not Needed
								B 3	0.420	Not Needed
STORY10	P4	Top	#4	#4	12.000	0.0025	0.0025	T 1	0.420	Not Needed
								T 2	0.420	Not Needed
		Bot	#4	#4	12.000	0.0025	0.0025	B 1	0.420	Not Needed
								B 2	0.420	Not Needed
STORY9	P4	Top	#4	#4	12.000	0.0025	0.0025	T 1	0.420	Not Needed
								T 2	0.420	Not Needed
		Bot	#4	#4	12.000	0.0025	0.0025	B 1	0.420	Not Needed
								B 2	0.420	Not Needed
STORY8	P4	Top	#4	#4	12.000	0.0025	0.0025	T 1	0.420	Not Needed
								T 2	0.420	Not Needed
		Bot	#4	#4	12.000	0.0025	0.0025	B 1	0.420	Not Needed
								B 2	0.420	Not Needed
STORY7	P4	Top	#4	#4	12.000	0.0025	0.0025	T 1	0.420	Not Needed
								T 2	0.420	Not Needed
		Bot	#4	#4	12.000	0.0025	0.0025	B 1	0.420	Not Needed
								B 2	0.420	Not Needed
STORY6	P4	Top	#4	#4	12.000	0.0025	0.0025	T 1	0.420	Not Needed
								T 2	0.420	Not Needed
		Bot	#4	#4	12.000	0.0025	0.0025	B 1	0.420	Not Needed
								B 2	0.420	Not Needed
STORY5	P4	Top	N12	N12	12.000	0.0025	0.0022	T 1	0.420	Not Needed
								T 2	0.420	Not Needed
								T 3	0.420	Not Needed
		Bot	N12	N12	12.000	0.0025	0.0022	B 1	0.420	Not Needed
								B 2	0.420	Not Needed
								B 3	0.420	Not Needed

Story Label	Pier Label	Sta Loc	Edge Bar	End Bar	Edge Spacing	Required Ratio	Current Ratio	Pier Leg	Shear Av in^2/ft	B-Zone Length
STORY11	P5	Top	#5	#5	12.000	0.0025	0.0039	T 1	0.420	Not Needed
		Bot	#5	#5	12.000	0.0025	0.0039	B 1	0.420	Not Needed
STORY10	P5	Top	#4	#4	12.000	0.0025	0.0025	T 1	0.420	Not Needed
								T 2	0.420	Not Needed
		Bot	#4	#4	12.000	0.0025	0.0025	B 1	0.420	Not Needed
								B 2	0.420	Not Needed
STORY9	P5	Top	#4	#4	12.000	0.0025	0.0025	T 1	0.420	Not Needed
								T 2	0.420	Not Needed
		Bot	#4	#4	12.000	0.0025	0.0025	B 1	0.420	Not Needed
								B 2	0.420	Not Needed
STORY8	P5	Top	#4	#4	12.000	0.0025	0.0025	T 1	0.420	Not Needed
								T 2	0.420	Not Needed
		Bot	#4	#4	12.000	0.0025	0.0025	B 1	0.420	Not Needed
								B 2	0.420	Not Needed
STORY7	P5	Top	#4	#4	12.000	0.0025	0.0025	T 1	0.420	Not Needed
								T 2	0.420	Not Needed
		Bot	#4	#4	12.000	0.0025	0.0025	B 1	0.420	Not Needed
								B 2	0.420	Not Needed
STORY6	P5	Top	#4	#4	12.000	0.0025	0.0025	T 1	0.420	Not Needed
								T 2	0.420	Not Needed
		Bot	#4	#4	12.000	0.0025	0.0025	B 1	0.420	Not Needed
								B 2	0.420	Not Needed
STORY5	P5	Top	#4	#4	12.000	0.0025	0.0025	T 1	0.420	Not Needed
								T 2	0.420	Not Needed
		Bot	#4	#4	12.000	0.0025	0.0025	B 1	0.420	Not Needed
								B 2	0.420	Not Needed

Story Label	Pier Label	Sta Loc	Edge Bar	End Bar	Edge Spacing	Required Ratio	Current Ratio	Pier Leg	Shear Av in^2/ft	B-Zone Length
STORY11	P6	Top	N12	N12	12.000	0.0025	0.0023	T 1	0.420	Not Needed
								T 2	0.420	Not Needed
								T 3	0.420	Not Needed
		Bot	N12	N12	12.000	0.0025	0.0023	B 1	0.420	Not Needed
								B 2	0.420	Not Needed
								B 3	0.420	Not Needed
STORY10	P6	Top	N12	N12	12.000	0.0025	0.0022	T 1	0.420	24.300
								T 2	0.420	22.050
								T 3	0.420	Not Needed
		Bot	N12	N12	12.000	0.0025	0.0022	B 1	0.420	24.300
								B 2	0.420	22.050
								B 3	0.420	Not Needed
STORY9	P6	Top	N12	N12	12.000	0.0025	0.0022	T 1	0.420	Not Needed
								T 2	0.420	Not Needed
								T 3	0.420	Not Needed
		Bot	N12	N12	12.000	0.0025	0.0022	B 1	0.420	Not Needed
								B 2	0.420	Not Needed
								B 3	0.420	Not Needed
STORY8	P6	Top	N12	N12	12.000	0.0025	0.0022	T 1	0.420	Not Needed
								T 2	0.420	Not Needed
								T 3	0.420	Not Needed
		Bot	N12	N12	12.000	0.0025	0.0022	B 1	0.420	Not Needed
								B 2	0.420	Not Needed
								B 3	0.420	Not Needed
STORY7	P6	Top	N12	N12	12.000	0.0025	0.0022	T 1	0.420	Not Needed
								T 2	0.420	Not Needed
								T 3	0.420	Not Needed
		Bot	N12	N12	12.000	0.0025	0.0022	B 1	0.420	Not Needed
								B 2	0.420	Not Needed
								B 3	0.420	Not Needed
STORY6	P6	Top	N12	N12	12.000	0.0025	0.0022	T 1	0.420	Not Needed
								T 2	0.420	Not Needed
								T 3	0.420	Not Needed
		Bot	N12	N12	12.000	0.0025	0.0022	B 1	0.420	Not Needed
								B 2	0.420	Not Needed
								B 3	0.420	Not Needed
STORY5	P6	Top	N12	N12	12.000	0.0025	0.0022	T 1	0.420	Not Needed
								T 2	0.420	Not Needed
								T 3	0.420	Not Needed
		Bot	N12	N12	12.000	0.0025	0.0022	B 1	0.420	Not Needed
								B 2	0.420	Not Needed
								B 3	0.420	Not Needed

Story Label	Pier Label	Sta Loc	Edge Bar	End Bar	Edge Spacing	Required Ratio	Current Ratio	Pier Leg	Shear Av in^2/ft	B-Zone Length
STORY11	P7	Top	N12	N12	12.000	0.0025	0.0023	T 1	0.420	Not Needed
								T 2	0.420	Not Needed
		Bot	N12	N12	12.000	0.0025	0.0023	B 1	0.420	Not Needed
								B 2	0.420	Not Needed
STORY10	P7	Top	N12	N12	12.000	0.0025	0.0023	T 1	0.420	Not Needed
								T 2	0.420	22.050
		Bot	N12	N12	12.000	0.0025	0.0023	B 1	0.420	24.300
								B 2	0.420	22.050
STORY9	P7	Top	N12	N12	12.000	0.0025	0.0023	T 1	0.420	Not Needed
								T 2	0.420	Not Needed
		Bot	N12	N12	12.000	0.0025	0.0023	B 1	0.420	Not Needed
								B 2	0.420	Not Needed
STORY8	P7	Top	N12	N12	12.000	0.0025	0.0023	T 1	0.420	Not Needed
								T 2	0.420	Not Needed
		Bot	N12	N12	12.000	0.0025	0.0023	B 1	0.420	Not Needed
								B 2	0.420	Not Needed
STORY7	P7	Top	N12	N12	12.000	0.0025	0.0023	T 1	0.420	Not Needed
								T 2	0.420	Not Needed
		Bot	N12	N12	12.000	0.0025	0.0023	B 1	0.420	Not Needed
								B 2	0.420	Not Needed
STORY6	P7	Top	N12	N12	12.000	0.0025	0.0023	T 1	0.420	Not Needed
								T 2	0.420	Not Needed
		Bot	N12	N12	12.000	0.0025	0.0023	B 1	0.420	Not Needed
								B 2	0.420	Not Needed
STORY5	P7	Top	N12	N12	12.000	0.0025	0.0023	T 1	0.420	Not Needed
								T 2	0.420	Not Needed
		Bot	N12	N12	12.000	0.0025	0.0023	B 1	0.420	Not Needed
								B 2	0.420	Not Needed

$$\left.\begin{aligned} \phi V_n &\geq V_u \\ V_n &= V_c + V_s \end{aligned}\right\} p155$$

$$V_n \leq 10\sqrt{f'_c}\, h d$$

h = thickness of wall

$d = 0.8\, l_w$

$$V_c \leq 2\lambda\sqrt{f'_c}\, h d$$

$$V_c = \min \left| \begin{array}{l} 3.3\lambda\sqrt{f'_c}\, h d + \dfrac{N_u d}{4 l_w} \\ \left[0.6\lambda\sqrt{f'_c} + \dfrac{l_w\left(1.25\lambda\sqrt{f'_c} + 0.2\dfrac{N_u}{l_w h}\right)}{\dfrac{M_u}{V_u} - \dfrac{l_w}{2}}\right] h d \end{array} \right.$$

$$V_s = \frac{A_v f_y d}{s}$$

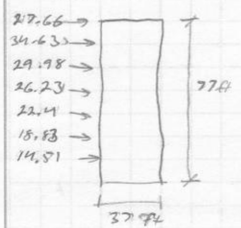
$$\rho_t \geq 0.0025$$

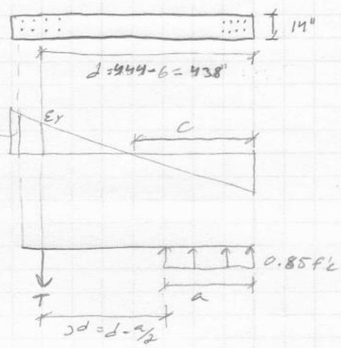
$$S_{horz} \leq \min \left| \begin{array}{l} l_w/5 \\ 3h \\ 18in \end{array} \right.$$

$$\rho_l \geq \max \left| \begin{array}{l} 0.0025 + 0.5\left(2.5 - \frac{h_w}{l_w}\right)(\rho_t - 0.0025) \\ 0.0025 \end{array} \right.$$

$$S_{vert} \leq \min \left| \begin{array}{l} l_w/3 \\ 3h \\ 18in \end{array} \right.$$

Shear wall 5C





$V = 174.24\ k$

$M = 8531'k$

flexure

$\phi M_n = \phi A_s f_y (d - a/2) = A_s f_y\, jd$ let $jd = 0.9\,d = 0.9(438) = 394.2"$

$C = T \Rightarrow 0.85 f'_c\, a \cdot b = A_s f_y$

$8531(1200) = 0.9 A_s (60{,}000)(394.2)$

$A_s = 0.48\ in^2$

$0.85(4000)\,a\,(14) = 0.48\,(60{,}000)$

$a = 0.61$

$jd = 438 - \frac{0.61}{2} = 437.7"$

$A_s = \frac{8531(1200)}{0.9(60{,}000)(437.7)} = 0.43\ in^2$

try $\boxed{(4)\text{-}\#4 \text{ bars}}$ $A_s = 0.4\ in^2$

$C = T \Rightarrow a = \frac{A_s f_y}{0.85 f'_c b} = \frac{0.4(60{,}000)}{0.85(4000)(14)} = 0.50\ in$

$c = a/\beta_1 = \frac{0.5}{0.85} = 0.591\ in$

$\varepsilon_t = \varepsilon_{cu} \frac{d_t - c}{c} = 0.003\left(\frac{209 - 0.59}{0.59}\right) = 0.066$

$0.066 > 0.005 \therefore$ tension controlled $\therefore$ ok shear wall 5c

max permitted shear

$\phi V_{n\,max} = \phi 10\sqrt{f'_c}\, h\, d$

$d = 0.8 l_w = 0.8(37.\ 5\times 12)$

$= 355.2''$

$\phi V_{n\,max} = 0.75(10)\sqrt{4000}(14)(355.2)/1000$

$= 2357^k > V_u = 174^k \therefore ok$

shear strength concrete

$a = \begin{cases} \frac{l_w}{2} = \frac{37}{2} = 18.5' \leftarrow \text{governs} \\ \frac{h_w}{2} = \frac{77}{2} = 38.5' \end{cases}$

$V_c \le 3.3\sqrt{4000}(14)(201)/1000$

$\le 1280\,kips$

$V_c \le \left[0.6\sqrt{4000} + \dfrac{3557(1.25)\sqrt{4000}}{\frac{8533(12)}{174} - \frac{357}{2}}\right](14)(359)/1000$

$\le 528.1\,kips \leftarrow$ governs

$V_u = 91.3\,kips < \frac{1}{2}\phi V_c = \frac{1}{2}(0.75)(528) = 198\,kips \therefore$ use minimum

use $\boxed{\#4 @ 12''}$

$\rho_\ell = \frac{A_v}{sh} \ge 0.0025 + 0.5\left(2.5 - \frac{77}{37}\right)(0.005 - 0.0025)$

$= 0.0034 < 0.0025 \therefore$ use 0.0025

max $s = 12''$

$A_v\ \rho_\ell sh = 0.0025(12)(14)$

$= 0.42$

try (2) #4 $\Rightarrow A_v = 0.42 \Rightarrow \rho_\ell = \frac{0.42}{12(14)} = 0.0025$

$S = \frac{0.42}{0.0025(14)} = 12'' \Rightarrow 18''$ use $\boxed{(2)\ \#4 @ 12''\ \text{for vert reinforcing}}$

$\boxed{A_v/ft = 0.42\,in^2/ft}$

Appendix C: Wind Calculations

Wind calcs

$V = 90$ mph
$K_d = 0.85$

Exposure Category: B

$K_{zt} = 1.0$

$K_z = 1.13$

$q_z = 0.00256\, K_z K_{zt} K_d V^2$
$q_h = 0.00256\, K_h K_{zt} K_d V^2$

$p = q\,GC_p - q_i\,(GC_{pi})$

$q = q_z$ for windward walls
$= q_h$ for leeward, side, and roof

$q_i = q_h$

Gust effect

$L_{eff} = \frac{\Sigma h_i L_i}{\Sigma h_i} \Rightarrow h_z \le 300$
$h_z < 4 L_{eff}$

$n_a = 75/h \Rightarrow$ flexible building

$$G_f = 0.925\left(\frac{1 + 1.71 I_z \sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.71 g_v I_z}\right)$$

$g_Q = 3.4$
$g_v = 3.4$
$g_R = \sqrt{2\ln(3600 n_a)} + \frac{0.577}{\sqrt{2\ln(3600 n_a)}}$

$R = \sqrt{1/\beta\, R_n R_h R_B (0.53 + 0.47 R_L)}$
$\beta = 0.02$
$R_n = \frac{7.47 N_1}{(1 + 10.3 N_1)^{5/3}}$

$N_1 = \frac{n_a L_{\bar{z}}}{\bar{V}_{\bar{z}}}$

$\bar{V}_z = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} \cdot \left(\frac{88}{60}\right) V$ T26.9-1

$R_\ell = \frac{1}{\eta} - \frac{1}{2\eta^2}\left(1 - e^{-2\eta}\right)$

$R_\ell = R_h$: $\eta = 4.6 n_a h / \bar{V}_{\bar{z}}$
$R_\ell = R_B$: $\eta = 4.6 n_a B / \bar{V}_{\bar{z}}$
$R_\ell = R_L$: $\eta = 15.4 n_a L / \bar{V}_{\bar{z}}$

$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6}$
$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}}$
$\bar{z} = 0.6h$

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

C_p from fig 27.4-1, 27.4-2, 27.4-3

GC_{pi} from Table 26.11-1

North/South Pressure

L/B (1-4)		0.630
L/B (5-R)		0.374
V		90
K_d		0.85
Exposure Category B		
K_{zt}		1
GC_{pi}	+/-	0.18
α		7.0
n_1		0.475
G		0.8604

Level	Z(ft)	K_z	q_z (psf)	q_h (psf)	$C_{p\ WW}$	$C_{p\ LW}$	$C_{p\ Side}$	P_{ww}	P_{LW}	P_{side}	B (width)	Story Height	Area	Force (kips)	Story Force	Story Shear	Overturning Moment
Roof	158.5	1.126	19.846	19.846	0.8	-0.500	-0.7	13.66	-8.54	-11.95	203	2.75	558.25	12.4	24.7	24.7	3911.7
	155.8	1.111	19.573	19.846	0.8	-0.500	-0.7	13.47	-8.54	-11.95	203	2.75	558.25	12.3			
11	153	1.095	19.300	19.846	0.8	-0.500	-0.7	13.28	-8.54	-11.95	203	8.2	1664.6	36.3	72.4	97.1	18762.9
	144.8	1.082	19.069	19.846	0.8	-0.500	-0.7	13.13	-8.54	-11.95	203	8.2	1664.6	36.1			
10	136.6	1.068	18.815	19.846	0.8	-0.500	-0.7	12.95	-8.54	-11.95	203	6.15	1248.45	26.8	53.4	150.5	39323.1
	130.5	1.054	18.573	19.846	0.8	-0.500	-0.7	12.78	-8.54	-11.95	203	6.15	1248.45	26.6			
9	124.3	1.040	18.331	19.846	0.8	-0.500	-0.7	12.62	-8.54	-11.95	203	5.95	1207.85	25.6	50.9	201.4	64359.2
	118.4	1.026	18.088	19.846	0.8	-0.500	-0.7	12.45	-8.54	-11.95	203	5.95	1207.85	25.4			
8	112.4	1.013	17.846	19.846	0.8	-0.500	-0.7	12.28	-8.54	-11.95	203	5.9	1197.7	24.9	49.7	251.1	92582.1
	106.5	0.999	17.604	19.846	0.8	-0.500	-0.7	12.12	-8.54	-11.95	203	5.9	1197.7	24.7			
7	100.6	0.984	17.344	19.846	0.8	-0.500	-0.7	11.94	-8.54	-11.95	203	5.95	1207.85	24.7	49.2	300.3	122793.8
	94.65	0.968	17.053	19.846	0.8	-0.500	-0.7	11.74	-8.54	-11.95	203	5.95	1207.85	24.5			
6	88.7	0.951	16.762	19.846	0.8	-0.500	-0.7	11.54	-8.54	-11.95	203	5.925	1202.78	24.1	48.1	348.4	153693.9
	82.78	0.935	16.471	19.846	0.8	-0.500	-0.7	11.34	-8.54	-11.95	203	5.925	1202.78	23.9			
5	76.85	0.914	16.110	19.846	0.8	-0.500	-0.7	11.09	-8.54	-11.95	203	5.925	1202.78	23.6	46.9	395.3	184069.6
	70.93	0.892	15.722	19.846	0.8	-0.500	-0.7	10.82	-8.54	-11.95	203	5.925	1202.78	23.3			
4	65	0.870	15.334	19.846	0.8	-0.500	-0.7	10.55	-8.54	-11.95	262	16.5	4323	82.5	161.5	556.8	220261.2
	48.5	0.803	14.145	19.846	0.8	-0.500	-0.7	9.74	-8.54	-11.95	262	16.5	4323	79.0			
3	32	0.712	12.549	19.846	0.8	-0.500	-0.7	8.64	-8.54	-11.95	262	10.5	2751	47.3	91.8	648.6	241017.3
	21.5	0.632	11.139	19.846	0.8	-0.500	-0.7	7.67	-8.54	-11.95	262	10.5	2751	44.6			
2	11	0.570	10.047	19.846	0.8	-0.500	-0.7	6.92	-8.54	-11.95	262	5.5	1441	22.3	44.5	693.2	248642.1
	5.5	0.570	10.047	19.846	0.8	-0.500	-0.7	6.92	-8.54	-11.95	262	5.5	1441	22.3			
1	0	0.570	10.047	19.846	0.8	-0.500	-0.7	6.92	-8.54	-11.95	262	0	0	0.0	22.3	715.4	248642.1
															Total=	**715.43**	**248642.1**

East/West Pressure

L/B (1-4)		1.588
L/B (5-R)		2.671
V		90
K_d		0.85
Exposure Category B		
K_{zt}		1
GC_{pi}	+/-	0.18
α		7.0
n_a		0.475
G		0.8926

Level	Z(ft)	K_z	q_z (psf)	q_h (psf)	C_p WW	C_p LW	C_p Side	P_{ww}	P_{LW}	P_{side}	B (width)	Story Height	Area	Force (kips)	Story Force	Story Shear	Overturning Moment
Roof	158.5	1.126	19.846	19.846	0.8	-0.267	-0.7	14.17	-4.73	-12.40	203	2.75	558.25	10.6	21.0	21.0	3327.7
	155.8	1.111	19.573	19.846	0.8	-0.267	-0.7	13.98	-4.73	-12.40	203	2.75	558.25	10.4			
11	153	1.095	19.300	19.846	0.8	-0.267	-0.7	13.78	-4.73	-12.40	203	8.2	1664.6	30.8	61.4	82.3	15927.1
	144.8	1.082	19.069	19.846	0.8	-0.267	-0.7	13.62	-4.73	-12.40	203	8.2	1664.6	30.5			
10	136.6	1.068	18.815	19.846	0.8	-0.267	-0.7	13.44	-4.73	-12.40	203	6.15	1248.45	22.7	45.1	127.5	33342.3
	130.5	1.054	18.573	19.846	0.8	-0.267	-0.7	13.26	-4.73	-12.40	203	6.15	1248.45	22.5			
9	124.3	1.040	18.331	19.846	0.8	-0.267	-0.7	13.09	-4.73	-12.40	203	5.95	1207.85	21.5	42.8	170.3	54514.0
	118.4	1.026	18.088	19.846	0.8	-0.267	-0.7	12.92	-4.73	-12.40	203	5.95	1207.85	21.3			
8	112.4	1.013	17.846	19.846	0.8	-0.267	-0.7	12.74	-4.73	-12.40	203	5.9	1197.7	20.9	41.6	212.0	78340.1
	106.5	0.999	17.604	19.846	0.8	-0.267	-0.7	12.57	-4.73	-12.40	203	5.9	1197.7	20.7			
7	100.6	0.984	17.344	19.846	0.8	-0.267	-0.7	12.38	-4.73	-12.40	203	5.95	1207.85	20.7	41.1	253.1	103798.9
	94.65	0.968	17.053	19.846	0.8	-0.267	-0.7	12.18	-4.73	-12.40	203	5.95	1207.85	20.4			
6	88.7	0.951	16.762	19.846	0.8	-0.267	-0.7	11.97	-4.73	-12.40	203	5.925	1202.78	20.1	39.9	293.0	129787.1
	82.78	0.935	16.471	19.846	0.8	-0.267	-0.7	11.76	-4.73	-12.40	203	5.925	1202.78	19.8			
5	76.85	0.914	16.110	19.846	0.8	-0.267	-0.7	11.50	-4.73	-12.40	203	5.925	1202.78	19.5	38.7	331.7	155278.8
	70.93	0.892	15.722	19.846	0.8	-0.267	-0.7	11.23	-4.73	-12.40	203	5.925	1202.78	19.2			
4	65	0.870	15.334	19.846	0.8	-0.382	-0.7	10.95	-6.77	-12.40	262	16.5	4323	76.6	149.5	481.2	186557.9
	48.5	0.803	14.145	19.846	0.8	-0.382	-0.7	10.10	-6.77	-12.40	262	16.5	4323	72.9			
3	32	0.712	12.549	19.846	0.8	-0.382	-0.7	8.96	-6.77	-12.40	262	10.5	2751	43.3	83.8	565.0	204637.4
	21.5	0.632	11.139	19.846	0.8	-0.382	-0.7	7.95	-6.77	-12.40	262	10.5	2751	40.5			
2	11	0.570	10.047	19.846	0.8	-0.382	-0.7	7.17	-6.77	-12.40	262	5.5	1441	20.1	40.2	605.2	211294.1
	5.5	0.570	10.047	19.846	0.8	-0.382	-0.7	7.17	-6.77	-12.40	262	5.5	1441	20.1			
1	0	0.570	10.047	19.846	0.8	-0.382	-0.7	7.17	-6.77	-12.40	262	0	0	0.0	20.1	625.3	211294.1
															Total=	**625.25**	**211294.1**

Appendix D: Seismic Calculations

Seismic Calcs

Seismic Importance factor: $I = 1.25$

latitude: 40.793°

Risk Category: III

longitude: -77.862°

Seismic Design Category A

Site Class B

$S_s = 0.147g$

$S_1 = 0.049g$

S_s & S_1 obtained from USGS Ground motion Parameter calculator

$S_{MS} = F_a S_s$

$S_{M1} = F_v S_1$

$S_{DS} = \frac{2}{3} S_{MS}$

$S_{D1} = \frac{2}{3} S_{M1}$

$T_a = C_t h_n^x$

$T = C_u T_a$

$$C_s = \min \begin{cases} \frac{S_{DS}}{R/I} \\ \frac{S_{D1}}{T(R/I)} \\ \frac{S_{D1} T_L}{T^2 (R/I)} \end{cases}$$

$V_b = C_s W$

distribution of story shears

$F_x = C_{vx} V_b$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$$

$$V_x = \sum_{i=x}^{n} F_i$$

	N-S	E-W
W(kips)=	65036	
F_a=	1.0	
F_v=	1.0	
S_s=	0.147	
S_1=	0.049	
R (Table 12.2-1:A2)=	4.0	4.0
I=	1.25	
$S_{ms}=F_aS_s$	0.147	
$S_{m1}=F_vS_1$	0.049	
$S_{DS}=2/3S_{ms}$	0.0980	
$S_{D1}=2/3S_{m1}$	0.0327	
$T_a=C_th_n^x$	0.900	
$T=C_uT_a$	1.260	
C_s= $S_{DS}/(R/I)$	0.0306	0.0306
$S_{D1}/[T(R/I)]$	0.0081	0.0081
$S_{D1}T_L/[T^2(R/I)]$	0.0386	0.0386
$V_b=C_sW$	1991.7	1991.7

			k=1.38			Story Shear
Level	h_x (ft)	W_x (kips)	$W_xh_x^k$	$C_{v,x}$	F_x	V_x
Roof	158.45	2533	2750974	0.1190	236.93	0.00
11	152.95	3295	3408268	0.1474	293.54	236.93
10	136.6	2979	2636286	0.1140	227.05	530.46
9	124.25	2939	2282068	0.0987	196.54	757.51
8	112.4	2939	1987271	0.0859	171.15	954.05
7	100.55	2939	1704068	0.0737	146.76	1125.21
6	88.7	2939	1433290	0.0620	123.44	1271.97
5	76.85	3056	1222762	0.0529	105.31	1395.41
4	65	7915	2513456	0.1087	216.47	1500.72
3	32	25244	3014846	0.1304	259.65	1717.19
2	11	6316	172808	0.0075	14.88	1976.84
1	0	1942	0	0	0	1991.73

Appendix E: Cost and Schedule Estimates

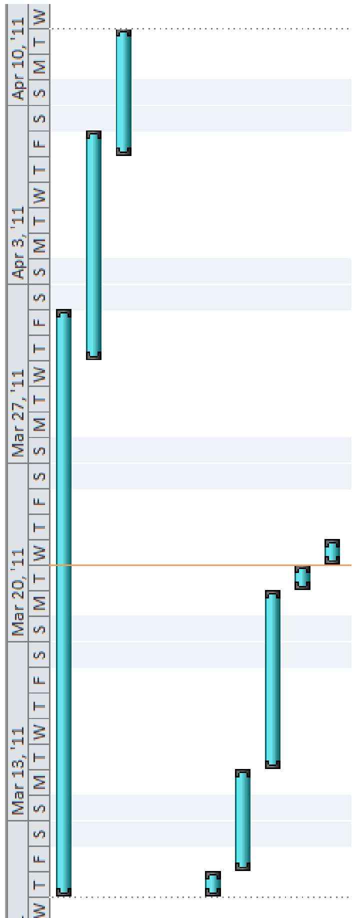
Current System

Number	Description	Crew	Daily Output	Labor-Hours	# Units	Unit	Bare Cost	Total Cost (O+P)	City Cost Index	Cost Estimate	Schedule (Days)
03110-420-2100	In Place Forms	(2) C-2	931	0.09	15400	SF	7.37	11.29	93.8	2,212,344	16.54
03210-600-0400	In Place Reinforcing	4 Rodm	2.9	11.03	20.34	Ton	1270	1625	93.8	3,432	7.01
03310-220-0300	4 ksi Ready Mix	--	--	--	571	CY	84	92.5	93.8	60,492	
03310-700-1500	Placing Concrete	C-20	180	0.356	571	CY	14.38	20.5	93.8	37,658	3.17
									Total	2,313,926	

2011 Cost Index= 100
2005 Cost Index= 81.9

Proposed System

Number	Description	Crew	Daily Output	Labor-Hours	# Units	Unit	Bare Cost	Total Cost (O+P)	City Cost Index	Cost Estimate	Schedule (Days)
05120-640-2900	Struct. Steel W16x31	E-2	900	0.062	336	LF	33.77	38.5	102.2	260,361	0.37
05120-640-0603	Struct. Steel W18x76	E-5	900	0.089	672	LF	77.9	88	102.2	829,142	0.75
05120-640-0600	Struct. Steel W10x12	E-2	600	0.093	264	LF	17.21	21	102.2	74,389	0.44
05120-640-1100*	Struct. Steel W12x16	E-2	880	0.064	756	LF	19.24	22.63	102.2	333,575	0.86
05310-300-5900	Metal Decking	E-4	2850	0.011	15400	SF	2.56	3.12	102.2	5,450,667	5.40
05090-840-0800	Weld Shear Conn	E-10	910	0.018	454	Ea.	1.48	2.11	102.2	66,410	0.50
03310-220-0300	4 ksi Ready Mix	--	--	--	191	CY	84	92.5	93.8	20,235	
03310-700-1500	Placing Concrete	C-20	180	0.356	191	CY	14.38	20.5	93.8	12,597	1.06
									Total	7,047,374	



Task Name	Duration	Start	Finish
Place Forms (2 crews)	17 days	Thu 3/10/11	Fri 4/1/11
Place Rebar	7 days	Thu 3/31/11	Fri 4/8/11
Place Concrete	3 days	Fri 4/8/11	Tue 4/12/11
Place Girders (W18 & 16)	1 day	Thu 3/10/11	Thu 3/10/11
Place Beams (W12 & 10)	2 days	Fri 3/11/11	Mon 3/14/11
Place Metal Decking	5 days	Tue 3/15/11	Mon 3/21/11
Weld Shear Connections	1 day	Tue 3/22/11	Tue 3/22/11
Place Concrete	1 day	Wed 3/23/11	Wed 3/23/11