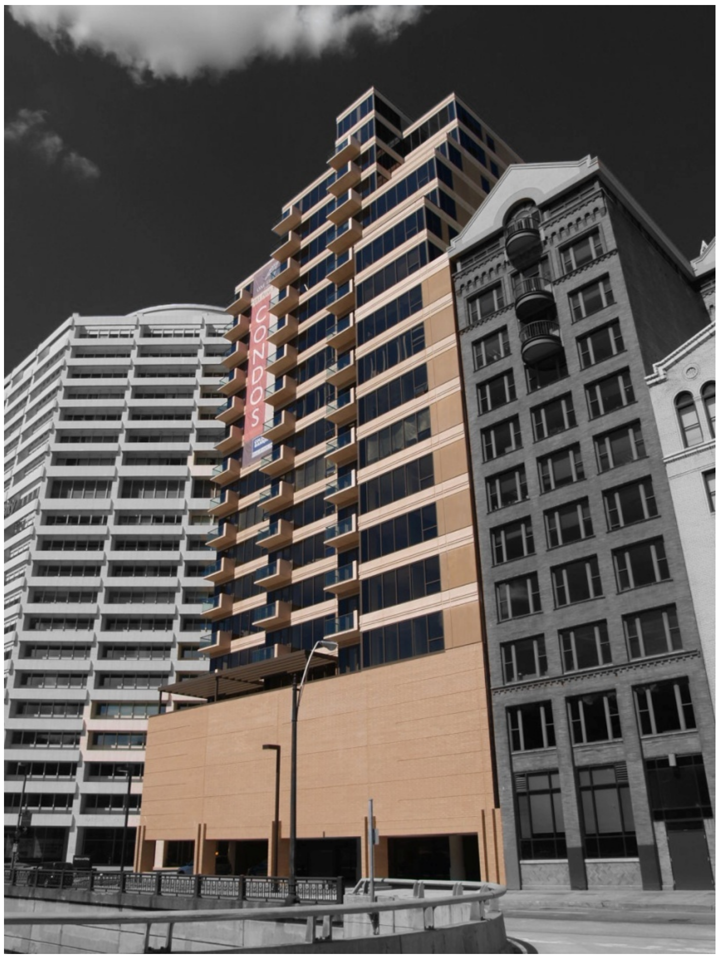
151 First Side

Final Report

April 9th, 2008



William J. Buchko

Structural Option

AE 481w – Senior Thesis
The Pennsylvania State University

Thesis Advisor: Kevin Parfitt

151 First Side

Pittsburgh, PA

www.151firstside.com

PRIMARY PROJECT TEAM
Owners: EQA Landmark Communities
Ralph A. Falbo, Inc.
Zambrano Corporation
Architect: Indovina Associates Architects
Structural Engineer: The Kachele Group
MEP Engineer: RAY Engineering
Electrical Consultant: Caplan Engineering Company
Contractor: Zambrano Corporation

GENERAL PROJECT INFO
18 story condominium including parking
233,000 SF
$24M construction cost

CONSTRUCTION TYPE
Design-Build

ARCHITECTURE
82 units ranging from 1,000 to 4,000 SF
Open floor plan with large windows gives view of river
Set backs allow for large balconies on upper floors

STRUCTURAL
Floor is Hambro system consisting of steel joists, steel decking, and concrete slab
Columns are steel W shapes
Exterior walls are 8" CMU with 4" veneer
Lateral bracing is a combination of braced frames and moment connections
Foundation is caisson system

MECHANICAL
Mechanical system is\ an AAON RN040 36.7 Ton roof top unit with individual heat pumps per unit.

ELECTRICAL
Main and secondary systems are 120/208 volt 3 phase 4 wire. Main switch is 1800A

Lighting
Most lighting is recessed indirect troffers with fluorescent downlights with wall washers in corridors

William J. Buchko - Structural Option
http://www.engr.psu.edu/ae/thesis/portfolios/2008/wjb170/

Project Abstract

Table of Contents

Executive Summary

Report Summary:

The purpose of this final report is to document the findings of the year long thesis project. Throughout the year 151 First Side was analyzed from the ground up. After initial comparisons, 2 changes in structural systems were proposed. In addition, acoustical changes were also proposed.

The structural breadth includes an analysis of the floor system, as well as the lateral system. A composite beam system was found to be a suitable alternative to the Hambro composite joist system originally used. While three lateral systems were analyzed, none were found to be suitable alternatives.

The floor system proposed was checked for its acoustical qualities. It was found to equal or better all of those found in the original floor system. The mechanical system was also explored to see if the sound level on the penthouse terrace could be reduced. It was found that by moving the roof top unit to the opposite side of the mechanical room, the sound level could be drastically reduced to below the accepted level.

The cost and scheduling of each proposed system was also considered. The proposed floor system was able to reduce cost and labor time, though it was not able to reduce the length of the critical path. The proposed acoustical changes were found to have no effect on scheduling and a negligent effect on cost.

Building Overview

Architecture

Architecture:

151 First Side is an 18 story 82 unit condominium with units ranging from 1,000-4,000 SF. It features an open and adjustable floor plan to allow customization by the resident. The first three floors are resident parking with a central entrance. The 4th level is a terrace level with levels 5 through the Penthouse consisting of one to four living spaces per floor. The upper levels are set back to allow large outdoor terraces.

Building Envelope:

The exterior walls consist of 8" CMU covered with a 4" veneer. The roof system is comprised of Hambro joists with 1½" steel deck topped with 3¼" normal weight concrete.

Building Systems.

Mechanical System: The building temperature is controlled by a 36.7 ton roof top unit by AAON. Each unit as well as each major common space also has its own heat pump with wall mounted thermostat. Hot water for the building is provided by three boilers located in the sub-basement.

Electrical System: The main power system provided by the Duquesne Electric vault is a 120/208 3 phase system. The main switch is rated at 1800A. Heating and cooling equipment run at 208V. while the boilers and general building uses 120V.

Lighting System: The units are primarily lit by incandescent downlights. Corridors contain both fluorescent downlights as well as wall washers. Offices and general areas contain recessed indirect troffers with electronic ballasts. The parking area has surface mounted fixtures with magnetic ballasts. The outdoor canopy lighting is provided by recessed metal-halide downlights with electronic ballasts.

Construction Details: The owner is a cooperation of three individual companies, Zambrano Corp., Ralph A. Falbo, Inc., and EQA Landmark Communities. The largest of these companies, Zambrano Corp., is also the general contractor. This building was completed as a design-build project. Physical construction was typical, with crane tie-ins on the 8th and 16th floors. A vertical survey had been preformed and designs changed to accommodate an older building which was leaning 3" into the property.

Structural System

Foundation:

The foundation was designed based on soil reports prepared by Engineering Mechanics, Inc. and Ackenheil Engineering, Inc., dated April, 2002 and July 1, 2005 respectively. Due to the close proximity of the Monongahela River pressure injected auger cast piles, 18" in diameter were used. Pile tips were placed at an elevation of 674'-0", which gives an average length of 52'. Each pile has a capacity of 120 tons. Pile caps are made of concrete with a 28 day strength of f'_c = 3000psi.

Slab on Grade:

The sub-basement and basement floors consist of slab on grade at elevations 725'-0" and 728'-0" respectively. Slabs are made from 5" of concrete with a 28 day strength of f'_c = 4000psi and are reinforced with 6x6 w2.1 x w2.1 welded wire fabric. Concrete was placed above 4" of AASHTO 57 well graded compacted granular stone.

Structural Frame:

The structural framing is made of steel W shapes. Beams range from W10 to W16 with the most common size being a W14x61. The columns are W12 shapes with weights ranging from 40 to 336 pounds per linear foot. Common column splices occur at every second floor.

Floor and Roof System:

The parking levels on the first three stories as well as the terrace level have poured concrete floors. All parking floors are 4" of light weight concrete on a 2" 20ga. galvanized composite metal deck with the exception of some highly loaded areas of the ground floor in which there is a 6" slab. The 4" sections on the parking levels are reinforced with #4 rebar spaced at 12" in both the bottom and the top of the slab with the top bars continuing for ¼ of the span length past the supports. The 6" sections contain 6x6-W2.9xW2.9 welded wire fabric while the terrace level has 6x6-W1.4xW1.4 welded wire fabric for its reinforcement.

The residential and mechanical levels, as well as the roof, contain an MD200 composite floor joist system provided by Hambro. A typical floor plan can be found in figure 1. There is a 3¼" thick slab made from concrete with a 28 day strength of f'_c=4000psi. Reinforcing within the concrete is a 6x6-W2.9xW2.9 welded wire mesh. The concrete is supported by 22ga. 1½" galvanized steel deck. Joist depth is 16" unless otherwise noted. The top chord is an "S' shape piece of cold-rolled, ASTM A 1008, Grade 50, 13ga. steel which works as both a compressive member as well as a shear connector while the bottom chord is made of two steel angles. Both chords have a minimum F_y=50,000psi. The web is formed from 7/16" hot-rolled steel bars with an F_y=44,000psi. The roof is also topped with a waterproof membrane.

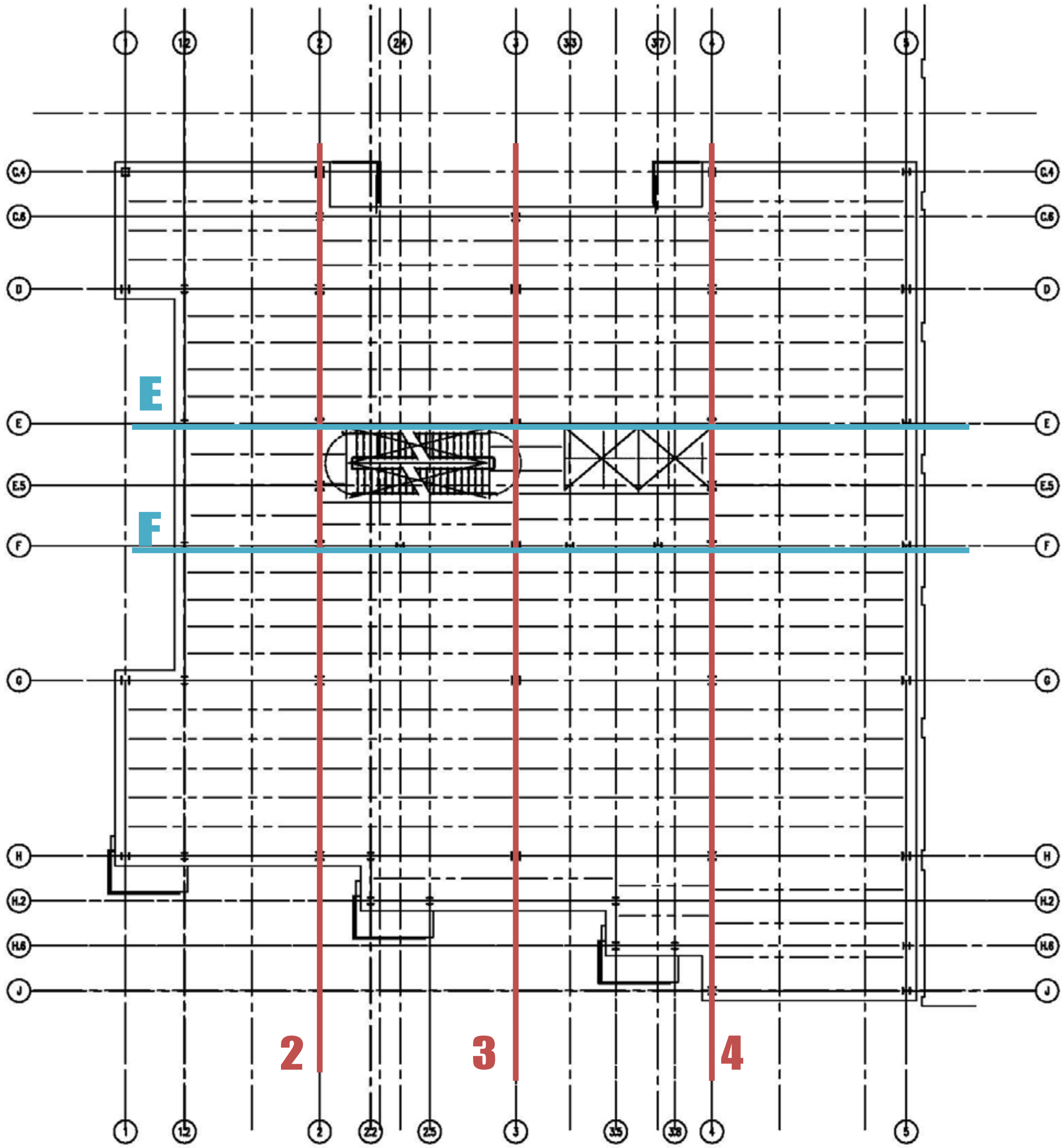


Figure 1

Lateral System:

The lateral system is composed of both braced frames as well as special moment frames. Lateral bracing is provided on column lines E and F (Figure 2) and column lines 2, 3, and 4 (Figure 3). Each of these column lines contain both moment connections and braced frames made of W12's or back to back channels.

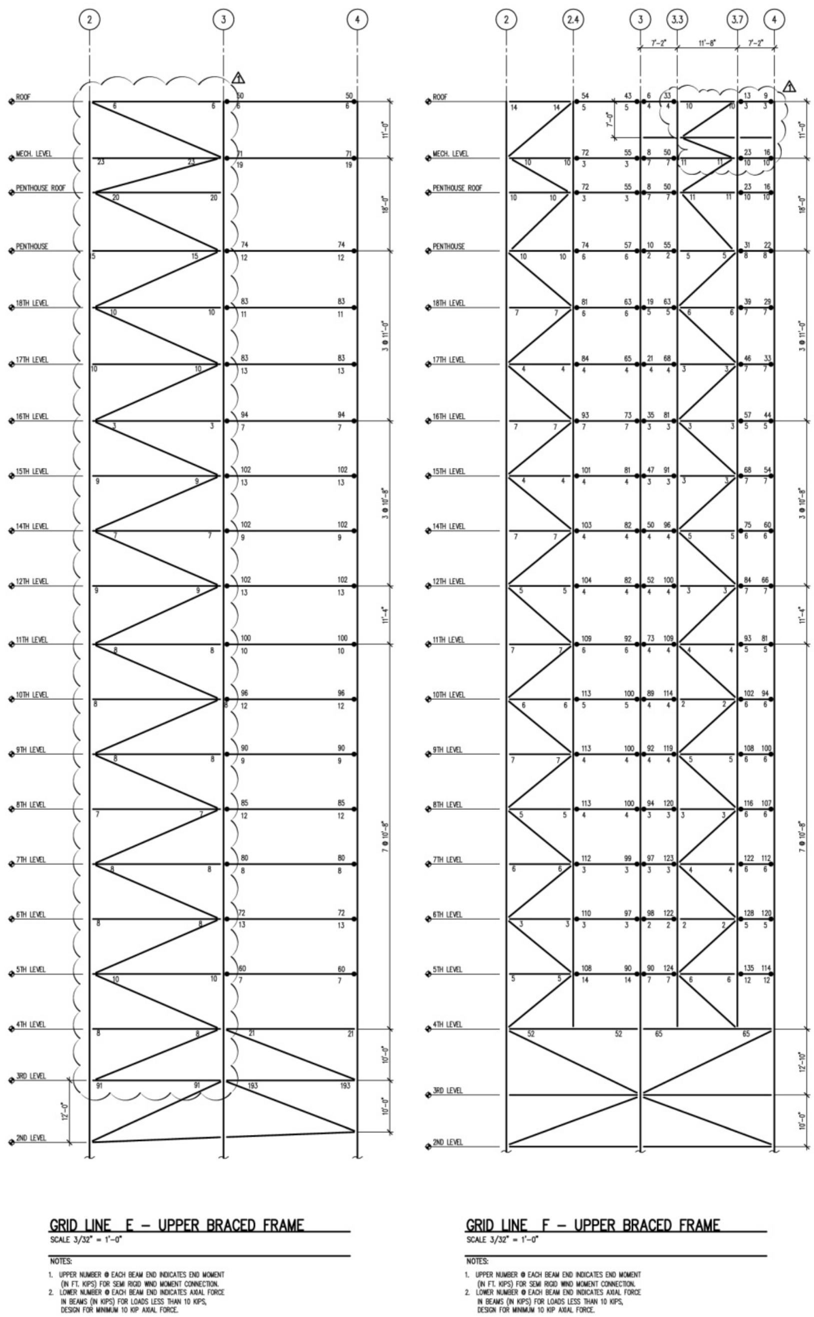


Figure 2

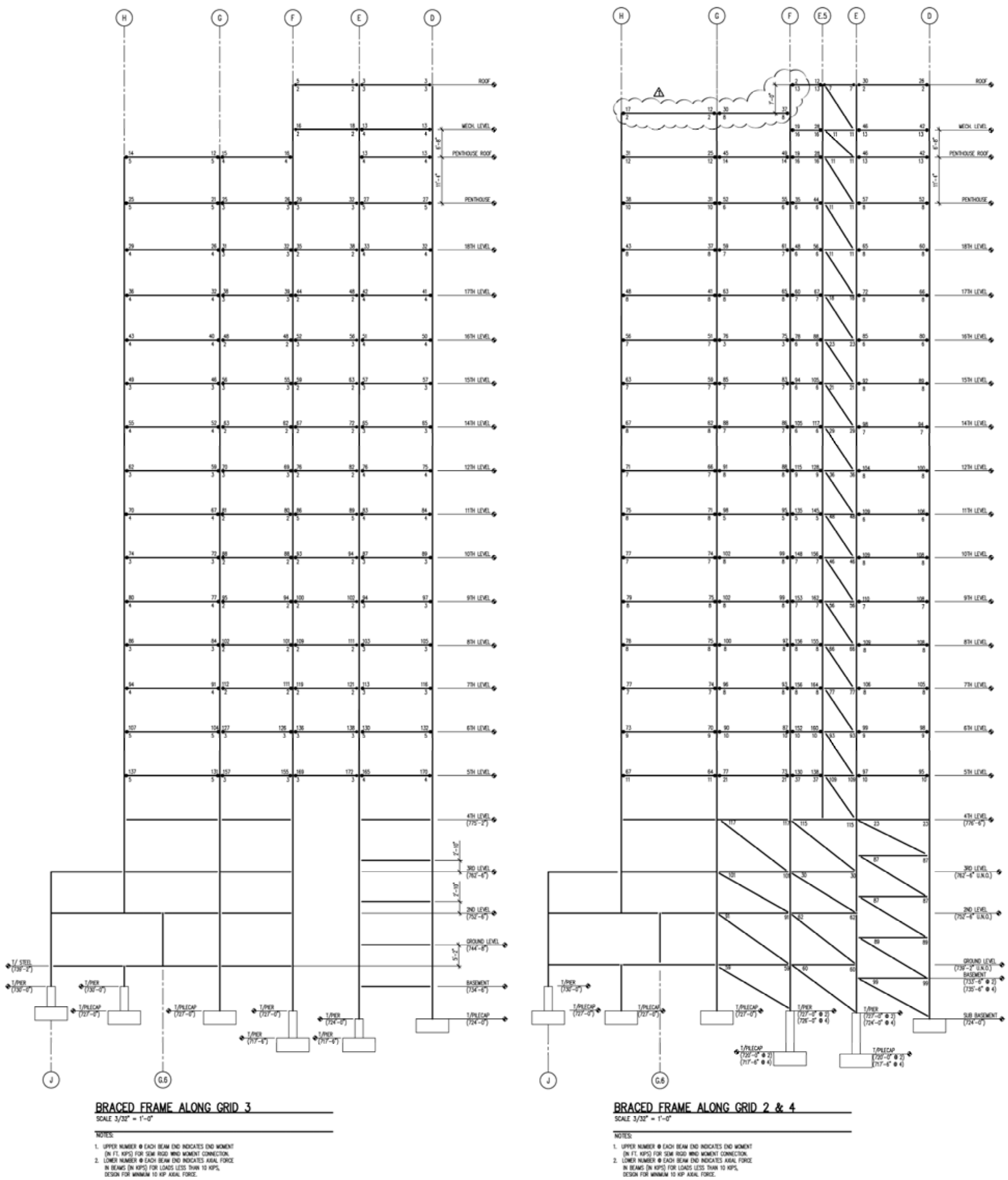

BRACED FRAME ALONG GRID 3
SCALE 3/32" = 1'-0"
NOTES:
1. UPPER NUMBER @ EACH BEAM END INDICATES END MOMENT (IN FT. KIPS) FOR SEMI RIGID WIND MOMENT CONNECTION.
2. LOWER NUMBER @ EACH BEAM END INDICATES AXIAL FORCE IN BEAMS (IN KIPS) FOR LOADS LESS THAN 10 KIPS, DESIGN FOR MINIMUM 10 KIP AXIAL FORCE.
BRACED FRAME ALONG GRID 2 & 4
SCALE 3/32" = 1'-0"
NOTES:
1. UPPER NUMBER @ EACH BEAM END INDICATES END MOMENT (IN FT. KIPS) FOR SEMI RIGID WIND MOMENT CONNECTION.
2. LOWER NUMBER @ EACH BEAM END INDICATES AXIAL FORCE IN BEAMS (IN KIPS) FOR LOADS LESS THAN 10 KIPS, DESIGN FOR MINIMUM 10 KIP AXIAL FORCE.

Figure 3

Codes

Building Code:

International Building Code (IBC), 2003 edition

Structural Concrete:

Building Code Requirements for Reinforced Concrete (ACI 318, latest edition)

Specifications for Structural Concrete (ACI 301, latest edition)

Steel Design:

Specifications for the Design, Fabrication, and Erection of Structural Steel for Buildings (AISC, 9^{th} Edition)

Code of Standard Practice for Steel Buildings and Bridges (with exception of Section 4.2)

Building Design Loads:

ANSI/ASCE-7 2002

Design Loads

General Loads:

Floor Live Loads

Load Area	**Design Load**	**Minimum Load (ASCE 7-05)**
Common Areas	100 psf	100 psf
Corridors	100 psf	100 psf
Parking	40 psf	40 psf
Residential	40 psf	40 psf
Mechanical	150 psf	n/a
Partition Allowance	20 psf where applicable	n/a

Dead Loads

Item	**Design Value**
Superimposed Dead Loads	
Mechanical , Electrical, Sprinkler	20 psf
Ceiling Finishes	5 psf
Floor Finishes	5 psf
Structure	Varies
Other Dead Loads	Where Applicable

Wind Loads:

The wind pressures and resulting base shear and overturning moment were calculated based on an exposure category B. The following spreadsheets give a detailed view of the pressure applied to each height level, and the corresponding floors. See the Appendix for my original calculations and diagrams regarding wind.

Pressure				
Wind from the North/South				
Windward		Leeward		Total
h (ft)	P (psf)	h (ft)	P (psf)	
0-15	6.72	0-15	-9.43	16.15
20	7.31	20	-9.43	16.74
25	7.78	25	-9.43	17.21
30	8.25	30	-9.43	17.68
40	8.96	40	-9.43	18.39
50	9.55	50	-9.43	18.98
60	10.02	60	-9.43	19.45
70	10.49	70	-9.43	19.92
80	10.96	80	-9.43	20.39
90	11.32	90	-9.43	20.75
100	11.67	100	-9.43	21.10
120	12.26	120	-9.43	21.69
140	12.85	140	-9.43	22.28
160	13.32	160	-9.43	22.75
180	13.79	180	-9.43	23.22
200	14.15	200	-9.43	23.58
250	15.09	250	-9.43	24.52

Pressure				
Wind from the East/West				
Windward		Leeward		Total
h (ft)	P (psf)	h (ft)	P (psf)	
0-15	6.68	0-15	-9.26	15.94
20	7.26	20	-9.26	16.53
25	7.73	25	-9.26	16.99
30	8.20	30	-9.26	17.46
40	8.91	40	-9.26	18.17
50	9.49	50	-9.26	18.75
60	9.96	60	-9.26	19.22
70	10.43	70	-9.26	19.69
80	10.90	80	-9.26	20.16
90	11.25	90	-9.26	20.51
100	11.60	100	-9.26	20.86
120	12.19	120	-9.26	21.45
140	12.77	140	-9.26	22.03
160	13.24	160	-9.26	22.50
180	13.71	180	-9.26	22.97
200	14.06	200	-9.26	23.32
250	15.00	250	-9.26	24.26

Wind from the North/South							
Floor	**Height (Ft.)**	**Story Height (Ft.)**	**Trib. Area (Sf.)**	**P-total (psf)**	**Story Force (Kip)**	**Total Shear (Kip)**	**Overturning Moment (Ft.-Kip)**
1 (ground)	0	0	0	16.15	0.00	**473.61**	**556969.93**
2	13.33	13.33	1242.50	16.15	20.07	473.61	6314.85
3	23.33	10.00	1215.88	17.21	20.93	453.55	10582.79
4	192.83	12.83	1251.38	18.39	23.01	432.62	83424.05
5	180.00	10.67	1136.00	18.98	21.56	409.61	73729.99
6	169.33	10.67	1136.00	19.45	22.10	388.05	65710.08
7	158.67	10.67	1136.00	19.92	22.63	365.96	58065.11
8	148.00	10.67	1136.00	20.39	23.17	343.33	50812.23
9	137.33	10.67	1136.00	20.75	23.57	320.16	43968.57
10	126.67	10.67	1136.00	21.69	24.64	296.59	37568.25
11	116.00	10.67	1171.50	21.69	25.41	271.95	31546.44
12	105.33	11.33	1171.50	22.28	26.10	246.54	25969.16
14	94.00	10.67	1136.00	22.28	25.31	220.44	20721.62
15	83.33	10.67	1136.00	22.75	25.84	195.13	16261.16
16	72.67	10.67	1153.75	22.75	26.25	169.29	12301.69
17	62.00	11.00	1171.50	23.22	27.20	143.04	8868.53
18	51.00	11.00	1171.50	23.22	27.20	115.84	5907.65
Penthouse	40.00	11.00	1544.25	23.58	36.41	88.63	3545.26
Mech. Level	29.00	18.00	1544.25	24.52	37.86	52.22	1514.52
Roof	11.00	11.00	585.75	24.52	14.36	14.36	157.98

North/South Direction:

Base Shear: 473.61 Kip

Overturning Moment: 556969.93 Ft.-Kip

Wind from the East/West							
Floor	Height (Ft.)	Story Height (Ft.)	Trib. Area (Sf.)	P-total (psf)	Story Force (Kip)	Total Shear (Kip)	Overturning Moment (Ft.-Kip)
1 (ground)	0	0	0	15.94	0.00	**468.27**	**550854.54**
2	13.33	13.33	1242.50	15.94	19.81	468.27	6243.61
3	23.33	10.00	1215.88	16.99	20.66	448.47	10464.19
4	192.83	12.83	1251.38	18.17	22.73	427.80	82494.47
5	180.00	10.67	1136.00	18.75	21.30	405.07	72912.39
6	169.33	10.67	1136.00	19.22	21.84	383.77	64984.40
7	158.67	10.67	1136.00	19.69	22.37	361.93	57426.38
8	148.00	10.67	1136.00	20.16	22.90	339.56	50255.38
9	137.33	10.67	1136.00	20.51	23.30	316.66	43488.44
10	126.67	10.67	1136.00	21.45	24.36	293.36	37159.44
11	116.00	10.67	1171.50	21.45	25.13	269.00	31203.98
12	105.33	11.33	1171.50	22.03	25.81	243.87	25688.08
14	94.00	10.67	1136.00	22.03	25.03	218.06	20497.85
15	83.33	10.67	1136.00	22.50	25.56	193.03	16086.03
16	72.67	10.67	1153.75	22.50	25.96	167.47	12169.50
17	62.00	11.00	1171.50	22.97	26.91	141.51	8773.53
18	51.00	11.00	1171.50	22.97	26.91	114.60	5844.52
Penthouse	40.00	11.00	1544.25	23.32	36.02	87.69	3507.53
Mech. Level	29.00	18.00	1544.25	24.26	37.46	51.67	1498.52
Roof	11.00	11.00	585.75	24.26	14.21	14.21	156.31

East/West Direction:

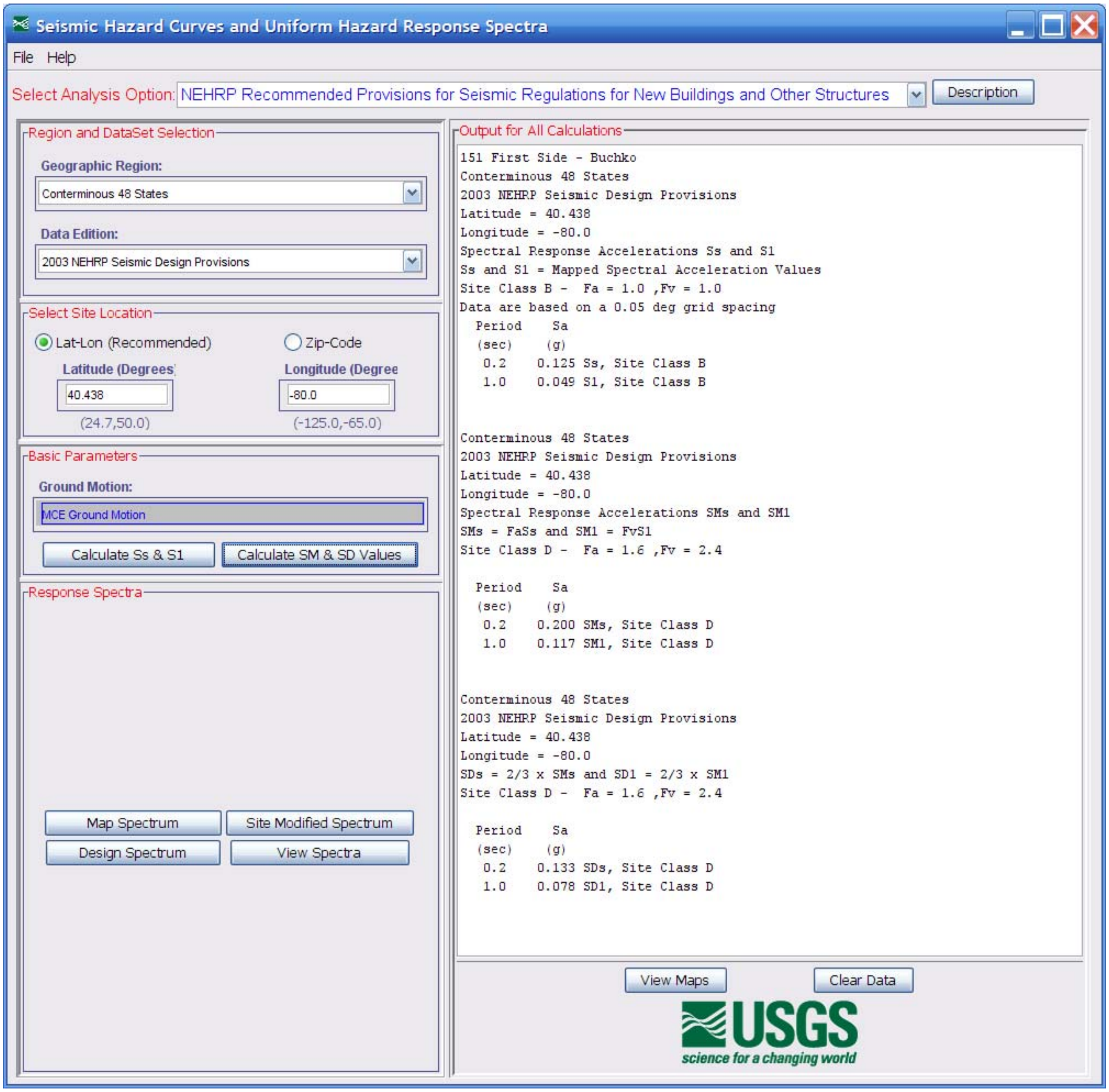
Base Shear: 468.27 Kip

Overturning Moment: 550854.54 Ft.-Kip

Seismic Loads:
Even though Pittsburgh is not known for its seismic activity, a simplified check has been performed to ensure that wind loading is indeed the controlling case. The building has been analyzed as a seismic design category B with ordinary concentric braced framing as its main seismic force resisting system. I have used software from the USGS website as an aid in calculating the required data. I have also preformed a vertical distribution of the seismic load. A sketch of the resultant loads can be found within the Appendix.

When I checked my value for the design base shear with that of the designer I noticed that mine was almost 1% off. When I investigated this further I found that the designer and I had started with different values for spectral response acceleration (S_1 and S_s). This can be accounted for based on the method of obtaining these values. I determined these values based on the output of the USGS software after inputting the longitude and latitude. It seems that the designer had used the then-current generic values for south eastern Pennsylvania. This discrepancy does not affect the overall design as both values are still less than the wind loads.

The following pages include a print out of the USGS website displaying the values that I have used for my analysis in addition to a spreadsheet showing the vertical distribution of the seismic load and final base shear.


Seismic Hazard Curves and Uniform Hazard Response Spectra
File Help
Select Analysis Option: NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures
Description
Region and DataSet Selection
Geographic Region:
Conterminous 48 States
Data Edition:
2003 NEHRP Seismic Design Provisions
Select Site Location
Lat-Lon (Recommended)
Zip-Code
Latitude (Degrees
Longitude (Degree
40.438
-80.0
(24.7,50.0)
(-125.0,-65.0)
Basic Parameters
Ground Motion:
MCE Ground Motion
Calculate Ss & S1
Calculate SM & SD Values
Response Spectra
Map Spectrum
Site Modified Spectrum
Design Spectrum
View Spectra
Output for All Calculations
151 First Side - Buchko
Conterminous 48 States
2003 NEHRP Seismic Design Provisions
Latitude = 40.438
Longitude = -80.0
Spectral Response Accelerations Ss and S1
Ss and S1 = Mapped Spectral Acceleration Values
Site Class B - Fa = 1.0 ,Fv = 1.0
Data are based on a 0.05 deg grid spacing
Period Sa
(sec) (g)
0.2 0.125 Ss, Site Class B
1.0 0.049 S1, Site Class B
Conterminous 48 States
2003 NEHRP Seismic Design Provisions
Latitude = 40.438
Longitude = -80.0
Spectral Response Accelerations SMs and SM1
SMs = FaSs and SM1 = FvS1
Site Class D - Fa = 1.6 ,Fv = 2.4
Period Sa
(sec) (g)
0.2 0.200 SMs, Site Class D
1.0 0.117 SM1, Site Class D
Conterminous 48 States
2003 NEHRP Seismic Design Provisions
Latitude = 40.438
Longitude = -80.0
SDs = 2/3 x SMs and SD1 = 2/3 x SM1
Site Class D - Fa = 1.6 ,Fv = 2.4
Period Sa
(sec) (g)
0.2 0.133 SDs, Site Class D
1.0 0.078 SD1, Site Class D
View Maps
Clear Data
USGS
science for a changing world

Vertical Distribution of Seismic Load					
K=1.67 Vb=304.7					
Level	wx (Kip)	hx (Ft.)	wxhx^1.67	Cvx	Fx (Kip)
Roof	1304.04	216.17	10336846.93	0.1342	40.88
Mech. Level	1304.04	205.17	9473474.13	0.1230	37.47
Penthouse	1304.04	187.17	8126668.00	0.1055	32.14
18	1304.04	176.17	7344860.53	0.0953	29.05
17	1304.04	165.17	6595099.13	0.0856	26.08
16	1304.04	154.17	5878073.59	0.0763	23.25
15	1304.04	143.50	5214751.14	0.0677	20.62
14	1304.04	132.83	4583674.00	0.0595	18.13
12	1304.04	122.17	3985675.73	0.0517	15.76
11	1358.64	110.83	3529424.99	0.0458	13.96
10	1358.64	100.17	2980658.20	0.0387	11.79
9	1358.64	89.50	2469726.52	0.0321	9.77
8	1358.64	78.83	1998066.39	0.0259	7.90
7	1358.64	68.17	1567363.51	0.0203	6.20
6	1358.64	57.50	1179640.56	0.0153	4.67
5	1358.64	46.83	837396.93	0.0109	3.31
4	1358.64	36.17	543850.54	0.0071	2.15
3	1473.20	23.33	283650.10	0.0037	1.12
2	1473.20	13.33	111406.21	0.0014	0.44
1 (ground)	1473.20	0.00	0.00	0.0000	0.00
Totals	27025.08			1.00	304.70

Seismic Loading:

Base Shear: 304.7 Kip

Lateral Force Distribution

151 First Side achieves its lateral force resistance through a combination of ordinary concentric braced framing and moment connections. The building was originally designed to only use ordinary concentric braced framing, but due to a change in architectural plan the framing was altered to its current state. The parking levels rely solely on two sets of braced frames. Moment connections were used in many areas of the residential levels so that none of the rentable space would have a diagonal brace within it. This resulted in diagonal braces near the central core with three sets of moment connections in the N-S direction and two sets in the E-W direction.

Lateral loads are transferred from the façade to the framing and into the floor system. Since the Hambro floor system creates a rigid diaphragm, the loads are taken from the floor and applied to the lateral frames as both a moment at the moment connections and as an axial compression force at the braced frames. These loads are carried through the columns and distributed through the foundation to the surrounding soil.

Due to the somewhat complex nature of this dual system, a RAM Structural System model was created to further analyze the distribution of lateral forces and the effects they have on the building. The original design documents were converted into a 3d computer model which could be analyzed using RAM Frame.

Initial Comparison Overview

Systems Analyzed:

Hambro Composite Joist System (Current)
Steel Composite System

Design Criteria:

Live Load: 40psf + 20psf partition allowance (except common areas)
Superimposed Dead Load: 30psf
Self Weight: Varies

Deflection:
- Steel:
  - Total = L / 240
  - Live = L / 360

Fire Rating: 2 Hours

Area of Design:

The area being analyzed is the residential levels as these contain the typical framing system of the building and provide the most opportunity for change. Depending on the system being analyzed, either a single worst case bay or a worst case frame will be used. I will then use these values to determine general properties for the entire system. These values will be conservative due to the methods used to obtain them, but this will allow for special details and situations which will not be discussed in this section. Note that only gravity loads were considered in the preliminary analysis.

Hambro Composite Joist System (Current)

Overview:

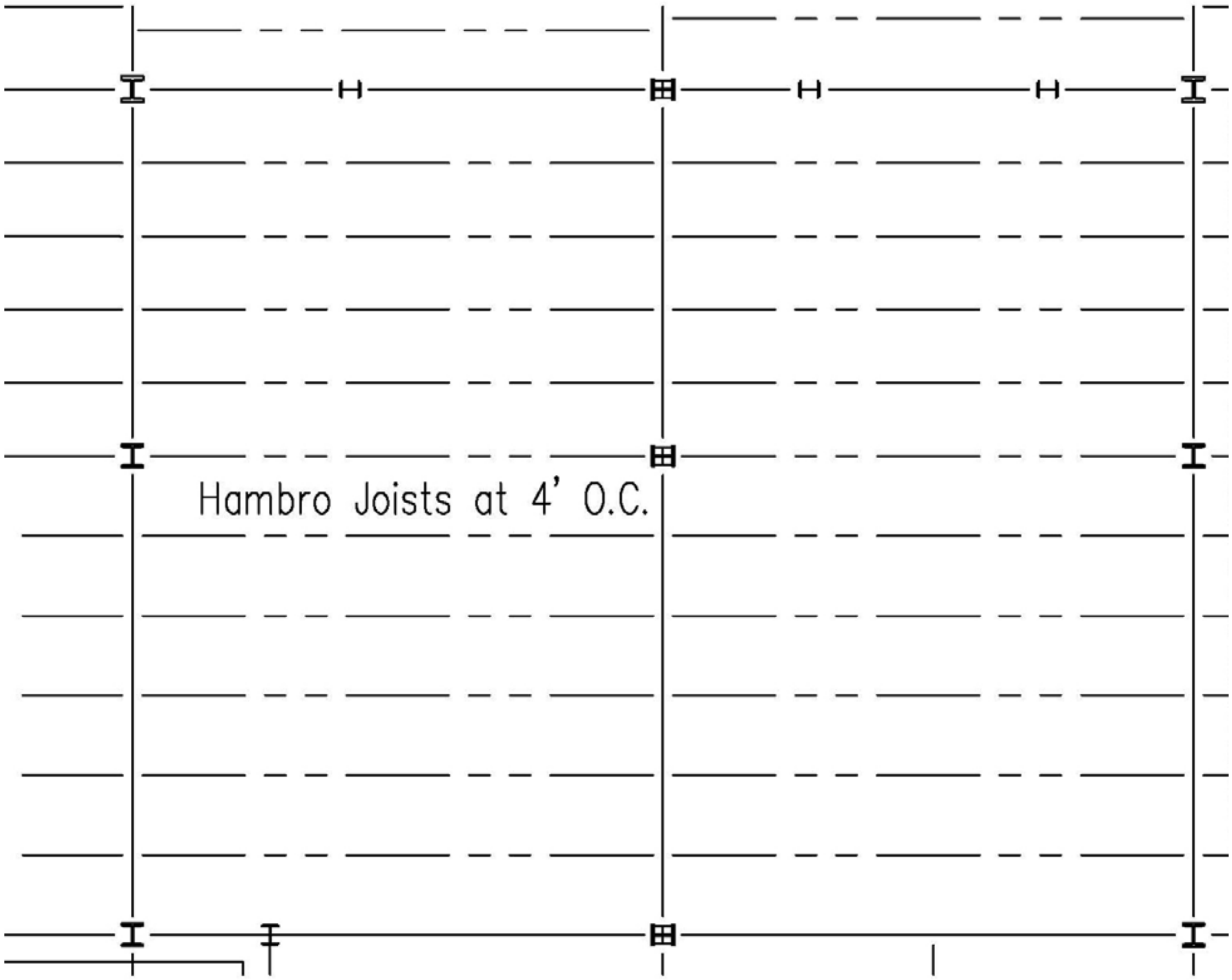
The current floor system is a MD2000 Hambro system which contains proprietary composite joists. It is comprised of a 3¼” slab with 16” composite joists resting on W14x61. These values are higher than what the Hambro design guide recommends. After discussion with a Hambro representative, I have found that the concrete slab was increased in depth by ½” for both vibration and acoustical reasons. The deeper joists were used due to slightly higher loads than what the design guide is written for, the need for larger mechanical openings, as well as the ability to hang the ceiling from the joists without interference from the beams. More information can be found in the Appendix on pages 47 and 48.

Advantages:

The Hambro system has many advantages. Since the lateral conditions are controlled by wind loading, the lighter weight of the joist is desirable. The open webs of the joist also allow for easy penetrations of mechanical, fire protection, and electrical equipment. The composite action of the joist also allows for a smaller system depth. This system is also relatively quick and easy to install.

Disadvantages:

Joist systems do have some inherent disadvantages. Because of the relative flexibility of the joists, the system can have problems with deflection and sound transmission. This has been taken into consideration in 151 First Side and the slab was made thicker to compensate. Also, more work is needed to obtain the required fire rating of 2 hours. Typical methods include spray-on fire protection or a fire rated suspended or gypboard ceiling, both of which can be costly and/or time consuming.



Typical bays H2-F4 for the Hambro System

Steel Composite System

Overview:

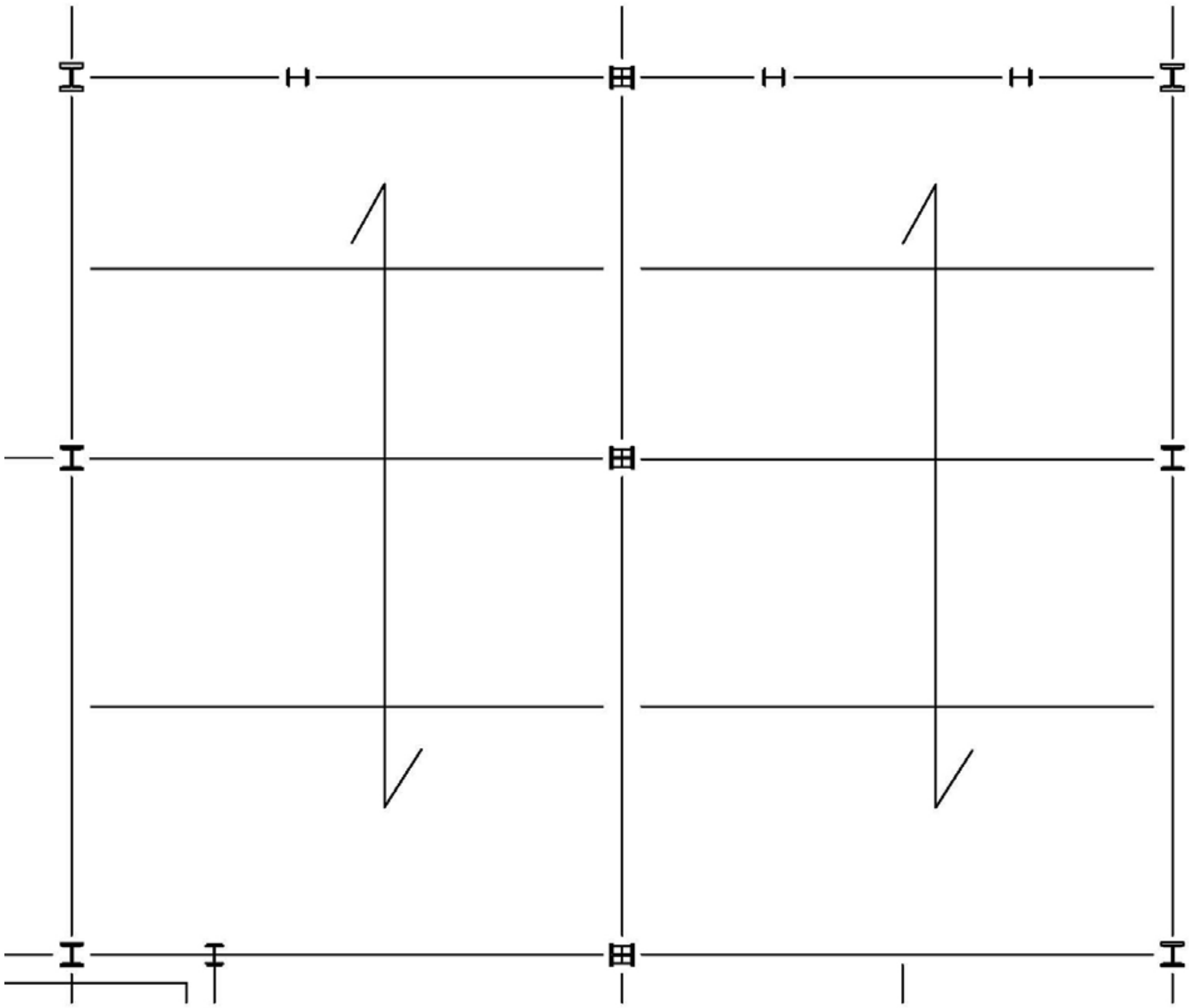
I chose to analyze a more conventional steel framing system consisting of composite beams and composite steel deck. Using the United Steel Deck design manual I have determined that a USD 2" Lok-Floor with 2½" of concrete would be the best choice in decking without requiring shoring. Using a RAM computer model, I have found that the majority of the beams would be W14x22 shapes with an average of 10 studs per beam.

Advantages:

Conventional steel systems are used often because of their many advantages. For 151 First Side the column grid would not need to be adjusted as the beams and decks could be adapted to fit the current layout. The floor would not need any extra fire protection and the beams could be quickly protected by a simple spraying process. Construction is also relatively quick with conventional steel framing, especially when the floor does not require any shoring. In addition, most of the materials that are needed will be readily available for quick delivery.

Disadvantages:

The obvious disadvantage of conventional steel framing is the extra labor involved in placing more beams as well as creating composite action. Another disadvantage is the closed webs. Penetrations may have to be made for mechanical equipment as well as sprinkler systems.



Potential typical bays H2-F4 for the Steel Composite System

Depth Topics and Proposal

In the second technical report, it was determined that a composite steel floor system would be a viable option with the possibility of cutting costs. This type of system has the potential to cost less in raw materials, as well as provide savings in fireproofing. During my research for the third technical report, I found that the building was initially designed with concentrically braced frames as the sole lateral support. It was later decided by the architect that the planned location of braced frames would be too intrusive in the open-floor plan. Because of this, the braced frames in those locations were changed to moment frames. While converting the previous design to the current design may have provided economical benefits in terms of engineering man hours, I feel that with further study a system can be found that will provide the required lateral stability while reducing material and installation costs.

Breadth Topics and Proposal

In addition to my proposed structural redesign I will consider its affect on other systems in the building. I will also be exploring some of the primary concerns of the owner and engineer in regards to serviceability. From these two topics, I have decided on two topics for my breadth studies.

My first breadth study will be an acoustical analysis. The current floor system design had an extra ½"of concrete added to help in both sound transmission and vibration. I will be looking at the effects of my proposed floor system on the acoustical properties of the residential areas. I will also look at possible ways to reduce the noise from the rooftop mechanical unit as the most common complaint from people touring the building is that sound carries from the unit to the 1,000 SF outdoor terrace of the Penthouse.

The second area I will investigate is within the construction management field. Since this project was designed with cost and schedule as major components of the design process, I will be analyzing the effect of my proposals on both of these criteria. Using RS Means, computer software, and information obtained by the contractor and owner, I will perform a cost analysis and schedule impact between the current system and the proposed floor system, including acoustical additions.

Structural Depth

The structural depth covers two topics which were chosen since the original designs were unconventional. The original design for the floor system uses the MD2000 Hambro system, which is a proprietary composite joist system. The lateral system that was used during construction consisted of a mix of braced frames as well as moment connections. Alternative designs were assessed and analyzed for both of these topics. All original design guidelines as well as owner and architect applied criteria were acknowledged and followed in the analysis of each of these alternatives.

As an aid in analysis a previously designed RAM model was used. It was found during the 3rd technical report that RAM can give wrong information when a framing column is ended at a transfer girder instead of continuing down to the support. To solve this issue the RAM model was modified so that all columns within the lateral framing system extended down to the base supports. In the areas where there is no actual column, the added column was modified so that it had a cross sectional area of 0.01 in^2 and a moment of inertia of 0.01 in^4. Also the yield strength was reduced to 0.01 ksi. This fulfilled the need for columns to extend to base supports while not affecting the actual design.

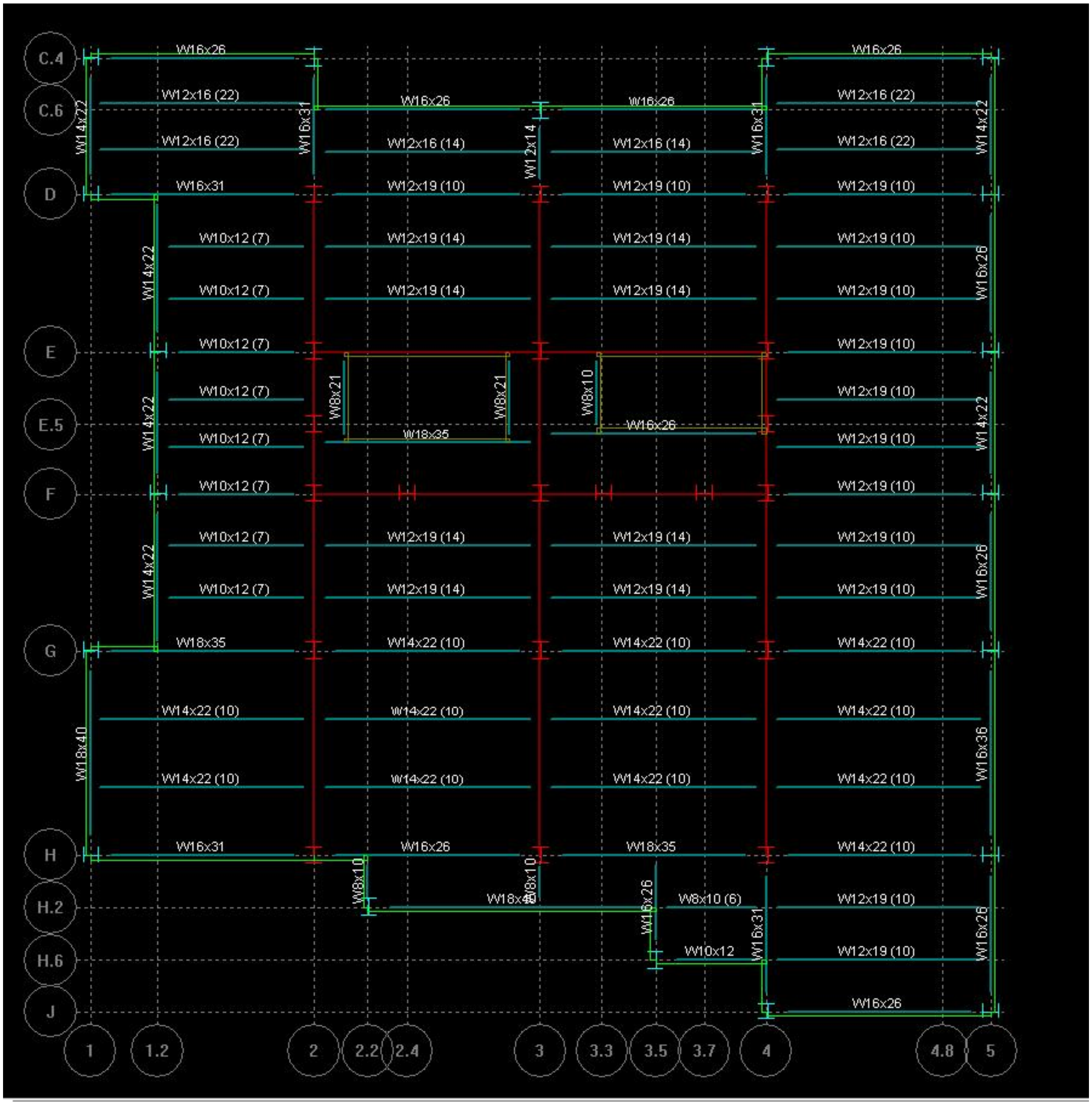
Floor System:

151 First Side was designed with a composite joist system by Hambro. The original idea was that a proprietary system, though possibly more costly, would provide a good floor system that met and surpassed the serviceability needs for the residential levels of the condominium. As part of the structural depth, alternative floor systems were analyzed. During the second technical report it was decided that a good alternative may be a composite steel system.

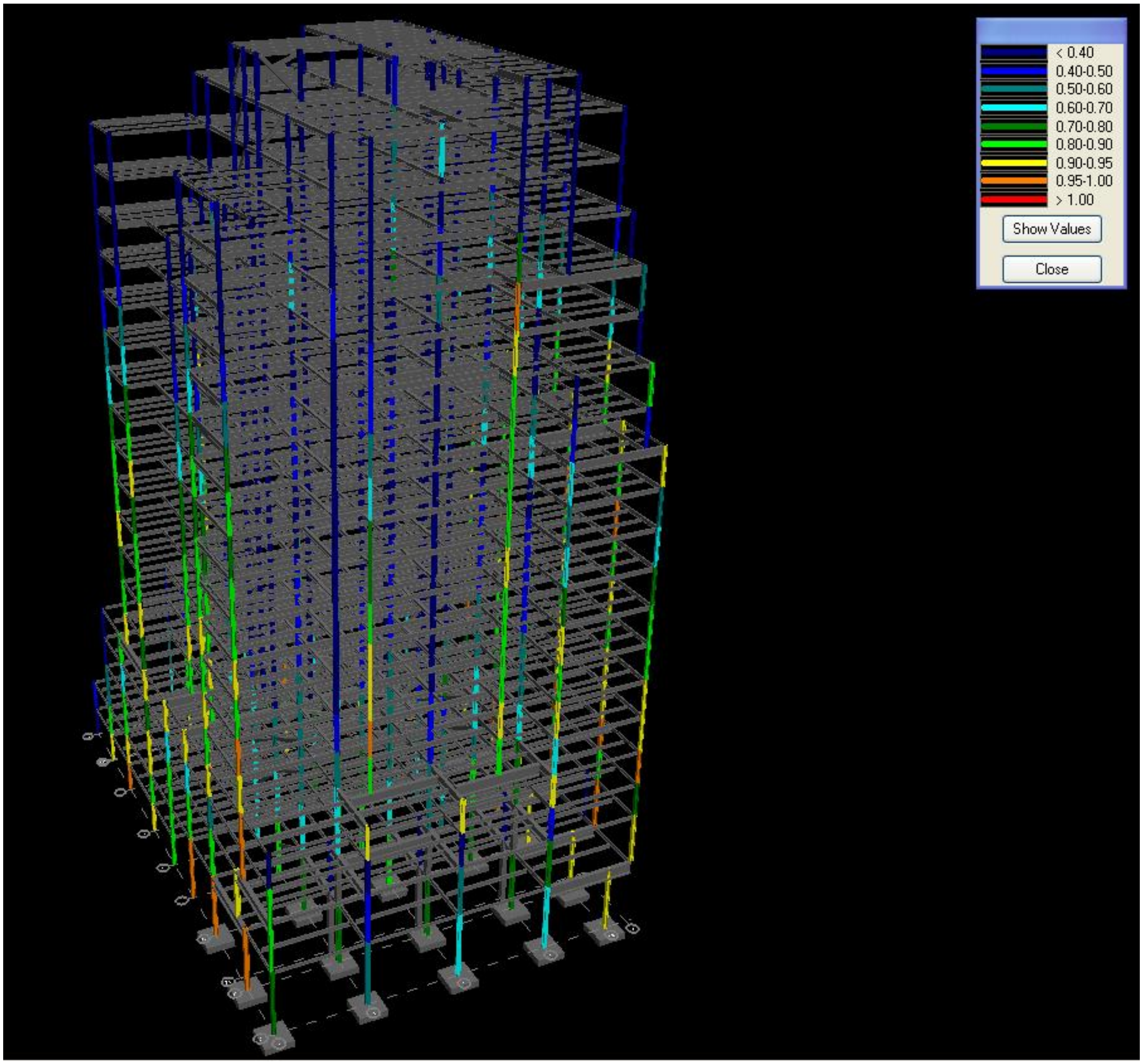
Due to acoustical considerations that will be discussed in the Acoustic Breadth section, it was decided that light weight concrete would be the best decision. It was found that a suitable deck system would be a 4” total depth of light weight concrete on top of B-LOK decking with 1 stud per foot. Most bays have been split into 3 equal sections to allow easy installation and provide small enough spans as to not require any shoring which will save time during construction. A typical floor plan can be seen on page 29.

The 4” of light weight concrete will actually weigh less than the 3¼ ” of normal concrete used in the current Hambro system. A takeoff was performed to see if the addition of beams added to the overall weight of the structural steel. Columns were also resized using the RAM model, which can be seen on page 30. The final takeoff including all gravity and lateral structural steel came to 1,167 Tons of steel. This is actually less than the estimated weight of structural steel for the Hambro system which was 1,308 Tons. These numbers were close enough to the original design that they will have little to no effect on the lateral system design. Also, the original structural engineer confirmed that the same foundation could be utilized with little to no change.

Due to the mass and moment of inertia of the beams, there will be less of a vibration problem which can be found with a joist system. Also, since the spacing of the beams is not always uniform due to the different size bays, the beams themselves vary in size. While this may not be as cheap as a system with all the same beams, it is helpful in dealing with vibration. According to the AISC Design Guides for serviceability and vibration, having beams or joists of the same size can causes a “wave” effect which sends a vibration along the deck perpendicular to the beams or joists. The difference in moment of inertia from the different sized beams, as well as the different effective width from the composite action with unequal spacing will cause the “wave effect” to disappear completely.



Typical Floor Beam Design



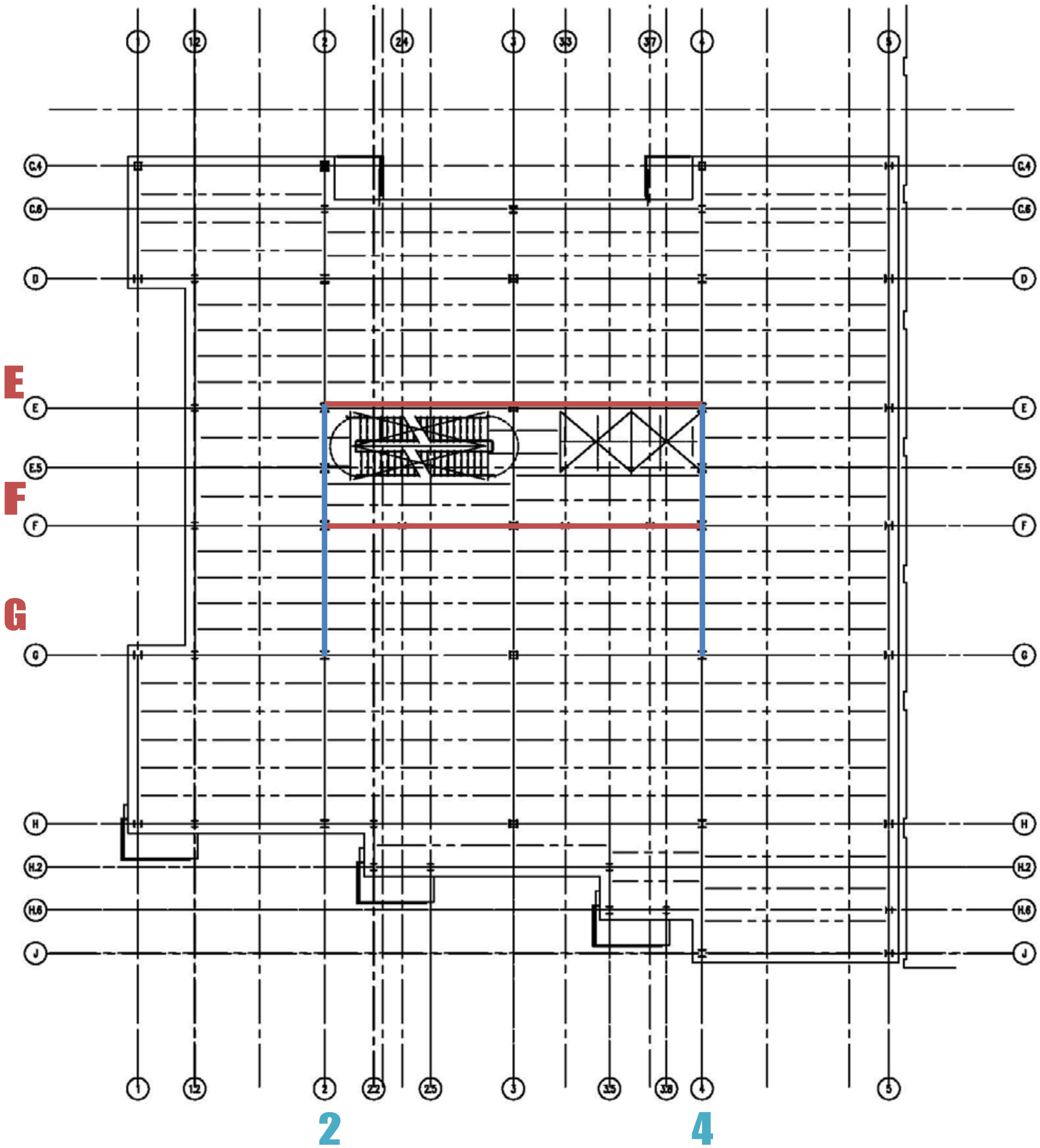
Column Redesign Using RAM Model

Lateral System:

Due to a change in architectural requirements, the lateral system of 151 First Side was modified to its current complex combination of braced framing and partially restrained moment frames.. As part of the structural depth research, multiple alternatives have been considered. The primary alternative systems examined were a system consisting of a concrete core, one consisting of only braced frames, and one consisting of only moment connections.

The first system looked at was the concrete core. This system has the advantage of keeping an open floor plan while providing a rigid central core that also doubles as the required fire protection for the stairwell. However, this system was quickly discarded after discussions with the owner/contractor. The owner/contractor was firm in his position to not mix different trades whenever possible. Because of this position, it would unfeasible to have a steel framing system while using concrete shear walls.

The second lateral system considered was a set of braced frames running the height of the building. It was found that a suitable configuration would be concentrically braced frames along grid lines 2 and 4 between gridlines E and G for the north-south direction. In the east-west direction braced frames could be placed along grid lines E and F between grid lines 2 and 4 as seen on page 32. This system has the advantage of low torsion forces due to its relative symmetry around the center of mass of the building. This idea was discussed with the architect and the owner. It was determined that, while this system would adequately meet all of the structural and serviceability needs, it would not be sufficient in this situation since the diagonal bracing needed between grid lines F and G do not comply with the open floor plan.



Braced Frame Lateral System Layout

The third lateral system considered was one that consisted solely of moment connections to resist the lateral loading. After further research, it was determined that a system of partially restrained moment connections would not be suitable for a building taller than 10 stories. It was also decided that a system of fully restrained moment connections would not be a feasible alternative. This is due not only to the high cost of making a fully restrained connection, but also to the increased cost due to larger columns. Many columns are part of the lateral framing in both the north-south and east-west. Because of the large moments applied by a fully restrained connection, the columns would need to be increased so that they would not fail in the weak direction.

Because of these issues it has been determined that none of these systems would be an intelligent alternative.

Acoustic Breadth

One of the concerns during the initial design of 151 First Side was sound level and sound transmission. In the original design the floors were adjusted to improve their acoustical qualities. This helped sound transmission from one floor to another. While each floor can be sold as multiple units, the partition walls are not part of the original design and are to be custom made and constructed as per the tenant's needs. This allows for the tenant to have walls with high acoustical qualities if that is what they desire.

The acoustic breadth is being performed for two reasons. First, the proposed floor system will be analyzed and compared to the current Hambro system to ensure that the same acoustic qualities can be met or bettered. Second, the mechanical system will be considered to see if the sound level on the penthouse terrace can be lowered.

Floor System:

As discussed in the structural depth, a composite system utilizing light weight concrete has been chosen as an alternative floor system. The 4" of light weight concrete has slightly less mass than the 3¼" of normal weight concrete. While less mass would normally indicate a lower STC, the difference is very small. As a benefit, however, the lower density light weight concrete can actually outperform the more massive normal weight concrete in its absorption of low end noises.

The introduction of steel beams in place of the steel joists helps with the overall structure born sound by reducing the susceptibility to vibration. The IIC of this system would be comparable to that of the Hambro composite joist system. The IIC could easily be improved by adding a thicker padding between the concrete floor and the floor covering.

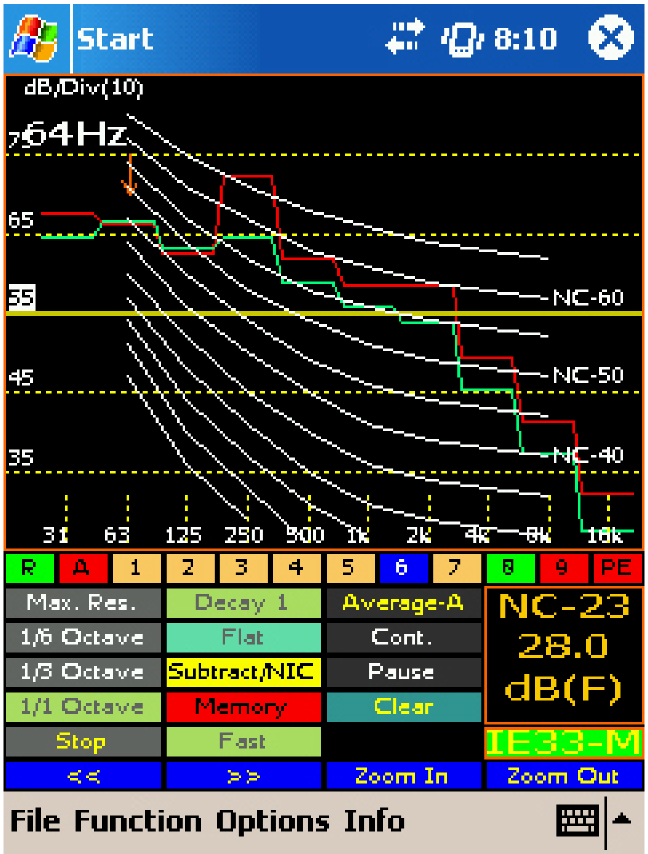
Overall the system should achieve an STC of approximately 51 and an IIC of 35 without considering additional floor coverings or ceiling treatments.

Mechanical System:

151 First Side is serviced by a 36.7 ton AAON RN series rooftop unit. The current location of this unit is above the penthouse near approximately 1,000sf of outdoor terrace. Unfortunately this unit is in direct line of sight of the terrace. One of the most common complaints by engineers, construction workers, and potential tenants was that the rooftop unit was loud and distracting while on the penthouse terrace.

Since the perception of loud is quite subjective, a representative of the manufacturer was contacted regarding acoustical data on the specific unit. The representative was unable to provide any relevant data on this unit so another method had to be used to find the sound level.

The Electrical Engineering West building on the University Park campus of Penn State has a 40 ton RK series unit. The RK series is a predecessor to the RM series, which is similar to the RN series used in 151 First Side. Using a Pocket PC equipped with an IVIE IE-33 Real Time Audio Jacket the sound levels of this unit were obtained at 10' and 20' away from the unit. In the figure below the red line shows an average over time from 10' away and the green line shows an average over time from a distance of 20'. As can be seen, the maximum sound level occurs at a frequency of 250Hz at approximately 73dB from 10' away. During the testing, the Real Time Sound Analysis showed a peak sound level of 83dB from 10' away.

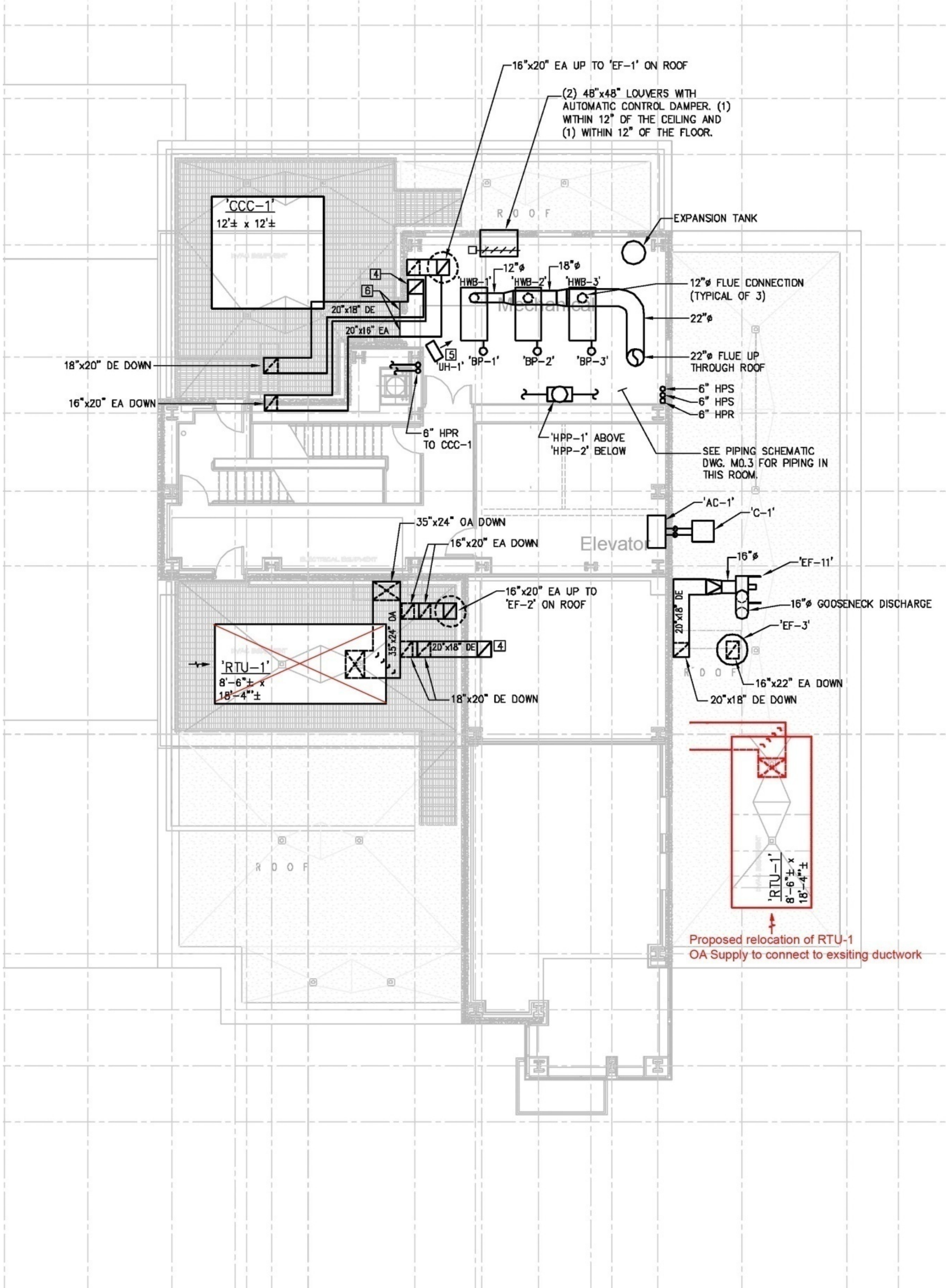


IVIE IE-33 Graph

These values have been compared with values obtained by a 3rd party acoustician. Unfortunately the chart of values obtained by the acoustician is not to be published as the project for which they were obtained is still under construction and is on a secure site. While these values concern a different manufacturer, they are extremely close to those found by the IVIE program, confirming that the values obtained are believable. The values obtained by the acoustician will be available for personal discussion and verification.

The original proposal to limit the noise level on the terrace was to install acoustical shielding. Acoustical shielding can theoretically lower the sound level by as much as 17dB for a semi-infinite sound barrier according to Architectural Acoustics. In practice, this value is usually closer to 14dB or 15dB. When installed on a rooftop in an urban area, as is the case with 151 First Side, this reduction is limited to around 6dB due to reflection and refraction of the sound as well as the finite length available on the roof. While this reduction would be welcomed, it does not bring the noise level down to an acceptable level.

To lower the sound level even more, alternative locations have been examined. It was found that the rooftop unit could be placed on the other side of the mechanical room with little effect on the mechanical system. The proposed layout can be seen on page 37. While this would place the unit in direct line of sight with a balcony, this would be preferable to its current location near the much larger, and more likely used terrace. This would lower the noise level in two ways. First, the unit will be 30 feet further away which would reduce the noise level by approximately 15dB if the unit produced sound in a non-directional way. Since the unit produces more sound from the supply end, and this end will now be facing away from all balconies, an additional decrease of 3dB to 5dB will occur. Second, the mechanical room will block a portion of the sound by providing multiple transitions in sound transport mediums. This will easily produce a transmission loss of 20dB which brings the overall sound level on the outdoor terrace to under 40dB which is well within acceptable levels.



Mechanical Level 19th Floor Mechanical Plan
SCALE: 1/8"=1'-0"
1
M1.8

Construction Management Breadth

A main part of any project is cost and scheduling. 151 First Side is no different and both of these played a large role in the original design. It was determined that in addition to meeting all of the original criteria, any alternative designs should be analyzed to see if they could meet or better the scheduling and cost of the original design.

Schedule:

It was found that the original design schedule was controlled by the placement of the structural steel. The placement was scheduled at 177 days. After discussions with both the contractor and the Hambro joist representative it was learned that the steel joists from the Hambro proprietary system were considered part of the structural steel. These joists are installed quicker than steel beams, but are placed closer together. Because of this Hambro recommends scheduling their placement within the same time frame that it would take to erect a conventional steel frame.

The pouring of the floor system for the Hambro composite joist is quite time consuming. The Hambro system must be poured in smaller sections, installing a proprietary composite top chord to each joist. The original schedule allowed for 3 days per floor. A composite beam system can be installed in as little as half of the time it takes to install the Hambro system. A conservative estimate of 2 days per floor was used. Unfortunately, since the structural steel still controlled the critical path, the overall project length was not shortened. There are, however, cost savings as will be discussed in the next section.

Another benefit of using a steel beam design over a steel joist design is fireproofing. It was estimated that a conservative 10 days of the original 130 days could be saved due to the easier application of fireproofing to a beam over a joist. Once again, while this may not affect the critical path, it will save money through labor.

While the proposed braced frame lateral system was not found to be a suitable alternative, such a change would have affected the critical path. Based on information provided by the engineers and the contractors, an estimated 5 days could have been saved on the project. However, since this design does not fit the criteria set forth by the architect and the owner, this is a moot point.

In the original thesis proposal, it was proposed that an acoustical shield be placed around the rooftop HVAC unit. This would have added another task to the schedule. However, after research it was determined that a more economical and effective approach was to move the unit. This move, including extra ductwork, does not increase the scheduling.

A gnatt chart for the original design can be found in the appendix on page 57. One for the proposed floor system is also within the appendix on page 64.

Cost:

As with most things in life, cost was a major factor in the design of 151 First Side. Therefore, a cost analysis was performed on the proposed changes in design to see how they would affect the overall budget. All of the values are either from RSMeans, sample projects that were provided by a contractor and estimator, or given values from representatives.

The Hambro composite joist system, for a building the size of 151 First Side, is approximately $2.41/SF for decking materials only. The materials used for the composite beam deck system are approximately $1.79/SF. This is approximately 35% cheaper than the Hambro system. However, this system uses light weight concrete and has a thicker slab. The slab thickness required is 17% larger than the composite joist system. Light weight concrete also costs an estimated 15% more than normal weight concrete. When combined, these add an additional 35% to the cost of the system. Therefore there is virtually no change in the cost to the floor system.

The real savings, however, come with the lower amount of steel in the project. As discussed in the structural breadth section, the redesign of the beam and column system that support the new floor system would result in a decrease in steel by approximately 131 Tons. This results in approximately $228,000 worth of savings in material alone.

In addition to saving on materials, there is savings in labor as was discussed in the scheduling section of the construction management breadth. The savings in labor can be conservatively estimated at $30,000 over the course of the project. It is important to note, however, that these total savings of $258,000 are partially based on the original internal steel estimates. Actual savings may not be as high if the original design was over estimated.

Conclusions

While 151 First Side was designed to meet and exceed all codes and criteria, it may be possible to improve upon the original design. The two main topics explored in this thesis are the structural depth and the acoustics breadth. Of these two sub categories were also analyzed.

Within the structural breadth both the floor system and lateral system were considered. During the analysis of the floor system, it was found that a composite beam system with light weight concrete could be used in place of the current Hambro composite joist system with normal weight concrete. By implementing this system and redesigning the supporting beams and columns, approximately 131 Tons of steel could be saved, in addition to much labor.

During the lateral system analysis 3 separate styles of systems were examined. Unfortunately the concrete core and braced framing systems were unable to fulfill the criteria put forth by the architect and contractor/owner. The third system consisting of only moment frames would be possible, but due to the high cost of fully restrained moment connections this system is not a suitable alternative. Therefore, the existing system consisting of both braced frames and partially restrained moment connections is still recommended.

With the recommendation of a new floor system, the acoustical effects were analyzed. The results showed equal or better acoustical qualities than the original design. Additionally, the mechanical system's acoustical qualities were analyzed. It was found that a drastic improvement in sound level on the penthouse terrace could be achieved by relocating the rooftop unit to the opposite side of the mechanical room.

In addition to the structural depth and the acoustics breadth, the scheduling and cost of each proposed system was analyzed. Each proposed system was found to be either of equal or even potentially lesser cost than the original design.

Acknowledgements

There are many people who should be acknowledged for their contributions to this report as well as the thesis experience as a whole.

Rob Sklarsky, John Moore, and Zambrano, Inc. for owner permission as well as information regarding cost, scheduling, original design, and design criteria

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Kevin Parfitt for his help with the RAM structural model in addition to all of his general thesis advice

The Thesis Mentors for their help and input throughout the entire thesis process

Without each of these individuals and companies this thesis would not have been successful. Thank you to each of you.

Appendix

151 First Side | Wind Loads pg 1/3 | William Buchko

Design Wind Speed $V = 90$ mph
Wind Importance Factor $I = 1.0$
Wind Exposure Category B
Building Category II
$GCP_i = \pm 0.18$

Exposure Coefficients K_h and K_z

height	components cladding	MWFRS
0-15'	0.7	0.57
20'	0.7	0.62
25'	0.7	0.66
30'	0.7	0.70
40'	0.76	0.76
50'	0.81	0.81
60'	0.85	0.85
70'	0.89	0.89
80'	0.93	0.93
90'	0.96	0.96
100'	0.99	0.99
120'	1.04	1.04
140'	1.09	1.09
160'	1.13	1.13
180'	1.17	1.17
200'	1.20	1.20
250'	1.28	1.28

$K_{zt} = 1.0$ Topigraphical factor
$K_d = 0.85$ wind direction factor

$T = 0.1(18) = 1.8$
$n = \frac{1}{T} = 0.55$ ∴ assume flexible structure

$$G_f = 0.925 \left(\frac{1 + 1.7 I_z \sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7 g_v I_z} \right)$$

$g_Q = 3.4$
$g_v = 3.4$
$g_R = \sqrt{2 \ln(3600(1.8))} + \frac{0.577}{\sqrt{2 \ln(3600(1.8))}}$
$g_R = 4.33$

$L_z = l \left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}}$
$= 320 \left(\frac{128}{33}\right)^{1/3.6}$

$L_z = 502.78$

$I_z = c \left(\frac{33}{128}\right)^{1/6}$
$I_z = 0.239$

$\alpha = 7.0$
$z_g = 1200$
$\hat{a} = 1/7$
$\hat{b} = 0.84$
$\bar{\alpha} = 1/4.0$
$\bar{b} = 0.45$
$c = 0.30$
$l = 320$
$\bar{\epsilon} = 1/3.0$
$z_{min} = 30$
$\bar{z} = .6(213.33) = 128'$

$\bar{V}_z = \bar{b} \left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V \left(\frac{88}{60}\right)$
$\bar{V}_z = 0.45 \left(\frac{128}{33}\right)^{1/4.0} (90) \left(\frac{88}{60}\right)$
$\bar{V}_z = 83.36$

$n_h = 4.6 n_1 h / \bar{V}_z = 4.6(0.55)(213.33)/83.36 = 6.47$
$n_B = 4.6 n_1 B / \bar{V}_z = 4.6(0.55) \begin{smallmatrix}106.5\\138.33\end{smallmatrix} / 83.36 = 3.23$ N-S, 4.2 E-W
$n_L = 15.4 n_1 L / \bar{V}_z = 15.4(0.55) \begin{smallmatrix}138.33\\106.5\end{smallmatrix} / 83.36 = 14.06$ N-S, 10.82 E-W
$N_1 = n_1 L_z / \bar{V}_z = 0.55(502.78)/83.36 = 3.32$

$R_n = \frac{7.47 N_1}{(1 + 10.3 N_1)^{5/3}} = \frac{7.47(3.32)}{(1 + 10.3(3.32))^{5/3}} = 0.066$

$R_h = \frac{1}{6.47} - \frac{1}{(6.47)^2(2)} (1 - e^{-2(6.47)}) = 0.143$

$R_B = \frac{1}{\begin{smallmatrix}3.23\\4.2\end{smallmatrix}} - \frac{1}{2\left(\begin{smallmatrix}3.23\\4.2\end{smallmatrix}\right)^2} \left(1 - e^{-2\left(\begin{smallmatrix}3.23\\4.2\end{smallmatrix}\right)}\right) = 0.262$ N-S, 0.21 E-W

$R_L = \frac{1}{\begin{smallmatrix}14.06\\10.82\end{smallmatrix}} - \frac{1}{2\left(\begin{smallmatrix}14.06\\10.82\end{smallmatrix}\right)^2} \left(1 - e^{-2\left(\begin{smallmatrix}14.06\\10.82\end{smallmatrix}\right)}\right) = 0.069$ N-S, 0.088 E-W

$R = \sqrt{\frac{1}{\beta} R_n R_h R_B (0.53 + 0.47 R_L)} = 0.103$ N-S, 0.0967 E-W

151 First Side | Wind Loads Pg 2/3 | William Buchko

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_z}\right)^{0.63}}} = 0.824 \text{ N-S},\ 0.816 \text{ E-W}$$

$$G_f = 0.925\left(\frac{1+1.7(0.239)\sqrt{3.4^2(Q^2)+4.33^2(R^2)}}{1+1.7(3.4)(0.239)}\right) \begin{array}{l} = 0.836 \text{ N-S} \\ = 0.831 \text{ E-W} \end{array}$$

Building is Enclosed
$GC_{pi} = \pm 0.18$

C_p windward = 0.8, use with q_z
C_p leeward = −0.5 N-S use with q_h
−0.494 E-W from interpolation

$q_z = 0.00256\, k_z (1.0)(0.85)(90^2)(1.0) = 17.6256(k_z)$

See spreadsheet for results

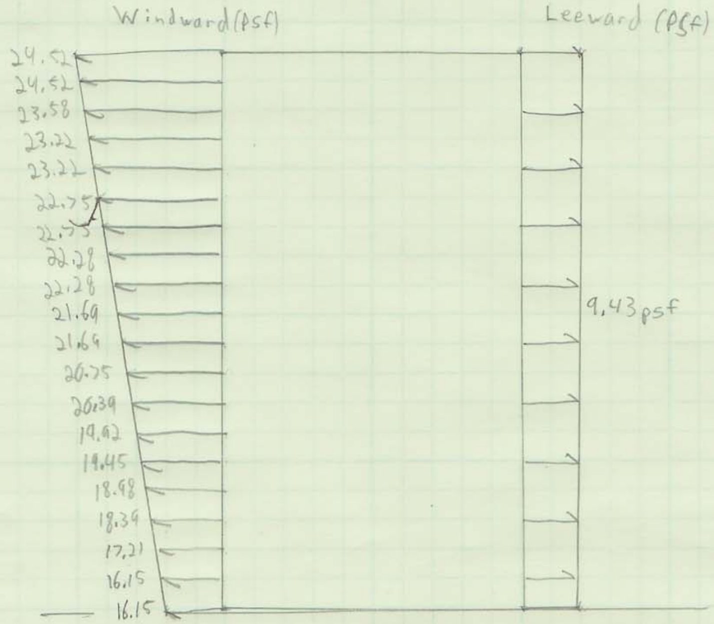
$q_h = 0.00256(1.28)(1.0)(0.85)(90^2)(1.0) = 22.56$

$p = qGC_p$ ignore internal pressure

for N-S use q from spreadsheet, $G_f = 0.836$, $C_{pww} = 0.8$, $C_{pLw} = -0.5$
for E-W use q from spreadsheet, $G_f = 0.831$, $C_{pww} = 0.8$, $C_{pLw} = -0.494$

151 First Side | Wind Loads pg 3/3 | William Buchko

Pressure Diagram: wind from N-S



Note: windward distribution is not linear

151 First Side | Seismic Design pg 1/2 | William Buchko

Seismic Site Class ; D

Seismic Weight:
total Dead Load
25% Live Load for Storage
include Partition Load (20 psf)
Equipment Operating Weight
20% flat roof snow Load (if $P_f > 30$ psf

Using Software from USGS Website for Lat = 40.438 and Long = -80.0
(note that values are constant for all of zip code 15222

$S_S = 0.125$
$S_1 = 0.049$

From 11.4 (ASCE 7-05)
$F_a = 1.6$
$F_v = 2.4$

Note: Due to the complex combination of moment and braced frames, I will use the value of R provided by the designer which is Based off of ASCE 7-02.

From USGS Software
$S_{MS} = 0.200$
$S_{M1} = 0.117$
$S_{DS} = 0.133$
$S_{D1} = 0.078$

$I = 1.0$ (11.5-1)
$R = 5.5$ (Construction Documents)
SDC = B

C_t and X based on braced steel frame as per construction documents
$C_t = 0.03$
$x = 0.75$
$T_a = 0.03(213.33)^{.75}$
$T_a = 1.67s$

$C_u = 1.4$
$T = C_u T_a = 2.34s$
$T_L = 12$

$C_s \geq S_{DS}/(R/I) = 0.133/(5.5/1) = 0.0242$
$\geq S_{D1}/(T(R/I)) = 0.078/(2.34(\frac{5.5}{1})) = 0.0061$
$\geq \frac{S_{D1} T_L}{T^2(R/I)} = \frac{0.078(12)}{(2.34)^2(\frac{5.5}{1})} = 0.031$

$C_s = 0.01$ min.

Weight: Dead load + partition = 100 psf + 20 psf = 120 psf for Residential Levels
Dead load = 100 psf for parking Levels

$W = 120[9(10887) + 8(11322)] + 100(3(14732)) = 27025080$ lbs
$V_B = C_s W = 0.01(27025080) = \boxed{270.37k}$

Note: Designer used different C_s (0.019) assumed to be based on older values of Ss and S_1

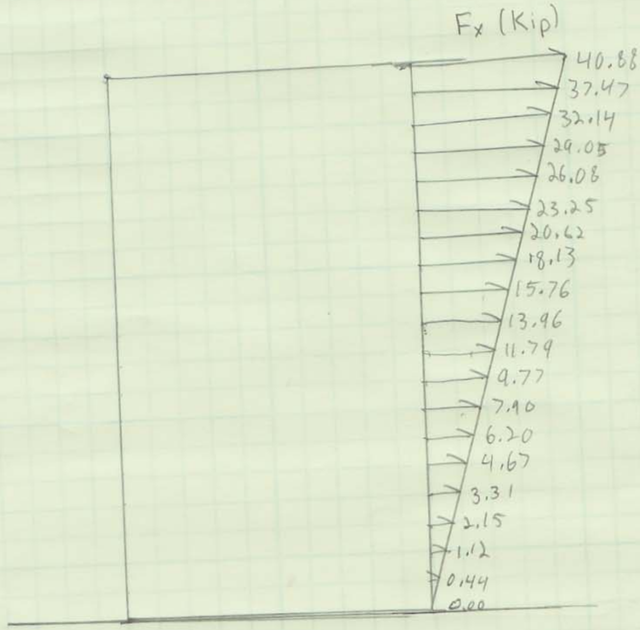
151 First Side | Seismic Design pg 2/2 | William Buchko

Vertical distribution of Seismic forces

$F_x = C_{vx} V$

$C_{vx} = \frac{w_x h_x^k}{\Sigma w h^k}$

$k = 1.67$ by interpolation



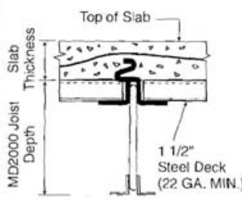
Note: Fx distribution is not Linear

HAMBRO SPAN TABLES

TABLE 8: MD2000® Clear Span Table

	Residential				Commercial	
Slab Thickness	2 3/4"	3"	3 1/4"	3 1/2"	2 3/4"	3 1/4"
Joist Depth*	LL = 40 psf DL = 68 psf	LL = 40 psf DL = 71 psf	LL = 40 psf DL = 74 psf	LL = 40 psf DL = 77 psf	LL = 50 psf DL = 68 psf	LL = 50 psf DL = 74 psf
8"	18' - 0"	18' - 0"	18' - 0"	18' - 0"	18' - 0"	18' - 0"
10"	22' - 6"	22' - 6"	22' - 6"	22' - 6"	22' - 6"	22' - 6"
12"	27' - 0"	27' - 0"	27' - 0"	27' - 0"	27' - 0"	27' - 0"
14"	31' - 6"	31' - 6"	31' - 6"	30' - 10"	31' - 6"	31' - 6"
16"	35' - 11"	35' - 0"	34' - 1"	33' - 2"	35' - 11"	34' - 1"
18"	38' - 7"	37' - 5"	36' - 5"	35' - 7"	38' - 7"	36' - 5"
20"	41' - 0"	39' - 11"	38' - 10"	37' - 9"	41' - 0"	38' - 9"
22"	43' - 0"	42' - 3"	41' - 0"	39' - 11"	43' - 0"	41' - 0"
24"	43' - 0"	43' - 0"	43' - 0"	42' - 1"	43' - 0"	43' - 0"

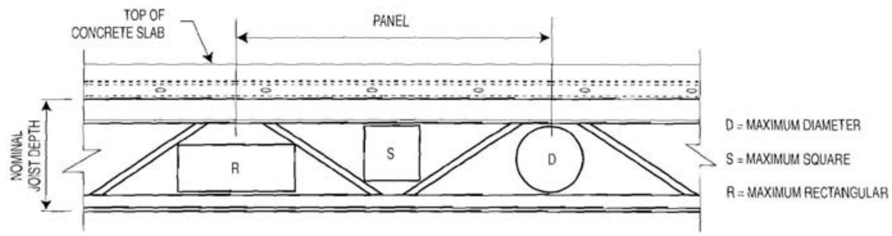
*Total floor depth = MD2000® Joist depth plus slab thickness



NOTES:

- Minimum slab thickness = *2 3/4"*
- Minimum top chord cover = *1 1/4"*
- $f'_c = 3{,}000\ psi$, $F_y = 50\ ksi$
- Joist spacing: *4'-1 1/4"*
- Table reflects uniform loads only.
- Metal deck standard: *1 1/2", 22 ga* (Galvanized)
- Nominal slab thickness = *slab thickness + 1/2" (Concrete in Deck)*
- Live load deflection design standard: *L / 360*
- Design clear spans, other than those shown in the above table, require additional structural review.

Maximum Duct Openings



DEPTH (in.)	PANEL (in.)	D (in.)	S (in.)	R (in. x in.)
8	20	4	4	6 x 3
10	20	6	5	7 x 4
12	24	8	6	9 x 5
14	24	9	7	9 1/2 x 6 11 x 5
16	24	10	8	10 1/2 x 6 1/2 13 x 5
18	24	11	8 1/2	11 x 7 12 1/2 x 6
20	24	11 1/2	9	12 x 7 13 x 6
22	24	12	9 1/2	12 x 8 14 x 6
24	24	12 1/2	10	13 x 8 14 x 7

NOTE: For other configurations, the maximum limits will be defined by the joist geometry.

FIRE PROTECTION - CLEAR SPAN TABLE

HAMBRO®
Proven Concrete Floor System

MD2000® Fire Protection

Floor/ceiling assemblies using Hambro® have been tested under restrained and unrestrained conditions by independent laboratories. Fire resistance ratings have been issued by Underwriters Laboratories Inc. (UL) and by Underwriters Laboratories of Canada Inc. (ULC) covering gypsum board, acoustical tile and spray on protection systems. Reference to these published listings should be made in detailing ceiling construction. Check your UL and ULC directory for the latest update of these listings.

ULC/CUL Design No.	Rating (hr)	Slab Thickness* (in.)	Slab Thickness* (mm)	Ceiling	Beam Rating (hr)
I522	2	3	75	Gypboard 1/2" (12.7 mm)	1 1/2
I800	1 1/2 - 2	2 1/2 - 2 3/4 - 3 - 3 1/2	65 - 70 - 76 - 89	suspended or panel	1
G003	2	2 3/4	70	suspended or panel	-
G213	2 - 3	3 - 4	75 - 100	suspended or panel	3
G227	2	2 3/4	70	suspended or panel	3
G228	2	3 1/4	83	suspended or panel	2
G229	2 - 3	3 - 4	75 - 100	suspended or panel	2 - 3
G401	4	2 1/2	65	Plaster	-
G524	2 - 3	2 3/4 - 3 1/2**	70 - 90	Gypboard 1/2" (12.7 mm)	2 - 3
G525	3	3 1/4	83	Gypboard 5/8" (15.9 mm)	3
G702	1 - 2 - 3	Varies**	Varies**	Direct spray on	-
G802	1 - 2 - 3	Varies**	Varies**	Direct spray on	-

* Slab Thickness = concrete above decking
** Normal and lightweight concrete

MD2000® Clear Span Table

Slab Thickness	Residential 2 3/4" (70 mm)	Residential 3" (75 mm)	Residential 3 1/4" (83 mm)	Residential 3 1/2" (90 mm)	Commercial 2 3/4" (70 mm)	Commercial 3 1/4" (83 mm)
Joist Depth	LL = 40 psf (1.9 kPa) DL = 68 psf (3.2 kPa)	LL = 40 psf (1.9 kPa) DL = 71 psf (3.4 kPa)	LL = 40 psf (1.9 kPa) DL = 74 psf (3.5 kPa)	LL = 40 psf (1.9 kPa) DL = 77 psf (3.7 kPa)	LL = 50 psf (2.4 kPa) DL = 68 psf (3.2 kPa)	LL = 50 psf (2.4 kPa) DL = 74 psf (3.5 kPa)
8" (200 mm)	18' - 0" (5 485 mm)	18' - 0" (5 485 mm)	18' - 0" (5 485 mm)	18' - 0" (5 485 mm)	18' - 0" (5 485 mm)	18' - 0" (5 485 mm)
10" (250 mm)	22' - 6" (6 860 mm)	22' - 6" (6 860 mm)	22' - 6" (6 860 mm)	22' - 6" (6 860 mm)	22' - 6" (6 860 mm)	22' - 6" (6 860 mm)
12" (300 mm)	27' - 0" (8 230 mm)	27' - 0" (8 230 mm)	27' - 0" (8 230 mm)	27' - 0" (8 230 mm)	27' - 0" (8 230 mm)	27' - 0" (8 230 mm)
14" (350 mm)	31' - 6" (9 600 mm)	31' - 6" (9 600 mm)	31' - 6" (9 600 mm)	30' - 10" (9 400 mm)	31' - 6" (9 600 mm)	31' - 6" (9 600 mm)
16" (400 mm)	35' - 11" (10 945 mm)	35' - 0" (10 670 mm)	34' - 1" (10 390 mm)	33' - 2" (10 110 mm)	35' - 11" (10 945 mm)	34' - 1" (10 390 mm)
18" (450 mm)	38' - 7" (11 760 mm)	37' - 5" (11 405 mm)	36' - 5" (11 100 mm)	35' - 7" (10 845 mm)	38' - 7" (11 760 mm)	36' - 5" (11 100 mm)
20" (500 mm)	41' - 0" (12 495 mm)	39' - 11" (12 165 mm)	38' - 10" (11 835 mm)	37' - 9" (10 505 mm)	41' - 0" (12 495 mm)	38' - 9" (11 810 mm)
22" (550 mm)	43' - 0" (13 105 mm)	42' - 3" (12 880 mm)	41' - 0" (12 495 mm)	39' - 11" (12 165 mm)	43' - 0" (13 105 mm)	41' - 0" (12 495 mm)
24" (600 mm)	43' - 0" (13 105 mm)	43' - 0" (13 105 mm)	43' - 0" (13 105 mm)	42' - 1" (12 825 mm)	43' - 0" (13 105 mm)	43' - 0" (13 105 mm)

Notes: • Table reflects uniform loads only.
• Design clear spans, other than those shown in the above table, require additional structural review.

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HAMBRO®

4

Frame Takeoff

RAM Frame v11.2
DataBase: New-Floor
Building Code: IBC

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Columns:

Wide Flange:

Steel Grade: 50

Size	#	Length ft	Weight lbs	UnitWt psf
W12X58	55	606.7	35093	
W12X79	27	300.0	23683	
W12X87	4	45.0	3920	
W12X106	2	20.0	2123	
W12X120	88	943.6	113343	
W12X136	16	170.7	23170	
W12X152	2	25.0	3803	
W12X170	2	20.0	3403	
W12X190	78	826.0	156825	
W12X210	43	463.0	97360	
W12X230	26	291.3	67112	
W12X252	54	577.0	145284	
W12X279	2	30.0	8360	
W12X336	3	35.0	11767	
	402		695245	3.34

Beams:

Wide Flange:

Steel Grade: 50

Size	#	Length ft	Weight lbs	UnitWt psf
W10X12	4	32.7	393	
W12X14	6	78.0	1104	
W12X16	1	18.0	288	
W12X19	2	44.0	834	
W14X22	44	515.2	11377	
W14X30	20	319.7	9626	
W14X26	1	16.3	427	
W14X61	240	3725.7	226925	
W14X68	35	910.0	61929	
W14X38	1	8.3	318	
W16X26	17	301.1	7868	
W16X36	3	50.7	1827	
W16X31	3	70.5	2190	
W18X35	6	124.5	4363	
W24X68	5	130.0	8891	
W24X131	2	52.0	6812	
W24X250	2	32.7	8170	

Frame Takeoff

RAM Frame v11.2
DataBase: New-Floor
Building Code: IBC

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Size	#	Length	Weight	UnitWt
W30X90	1	16.3	1467	
W33X387	12	139.2	54016	
	405		408829	1.96

Braces:

Wide Flange:
Steel Grade: 50

Size	#	Length ft	Weight lbs	UnitWt psf
W10X33	8	130.7	4320	
W10X49	2	55.7	2730	
W10X68	2	56.2	3825	
W10X77	7	195.5	15033	
W10X88	3	84.1	7408	
W10X100	1	28.1	2811	
W12X40	18	358.7	14281	
W12X45	5	110.6	4930	
W12X53	6	153.7	8160	
W12X58	1	28.1	1626	
W12X50	40	644.3	32011	
Channel:				
C10X30	19	259.8	7789	
	112		104924	0.50

Note: Length and Weight based on Centerline dimensions.

Gravity Beam Design Takeoff

RAM Steel v11.2
DataBase: New-Floor
Building Code: IBC

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Steel Code: ASD 9th Ed.

Total Number of Studs = **1554**

Floor Type: basement
Story Level 1
Steel Grade: 50

SIZE	#	LENGTH (ft)	WEIGHT (lbs)
W8X10	12	92.83	935
W10X12	1	12.50	151
W12X14	2	42.08	596
W12X16	1	16.33	262
W12X19	12	236.25	4478
W8X21	1	16.17	339
W14X22	27	683.75	15100
W16X26	12	273.75	7154
W14X30	1	18.00	542
W18X35	7	165.25	5792
W21X83	1	25.75	2129
W36X160	1	25.75	4118
W36X328	1	25.75	8447
W40X431	2	51.50	22256
W40X503	1	25.75	12968
	-----		-------------
	82		**85266**

Total Number of Studs = **763**

TOTAL STRUCTURE GRAVITY BEAM TAKEOFF

Steel Grade: 50

SIZE	#	LENGTH (ft)	WEIGHT (lbs)
W8X10	157	1475.83	14865
W8X24	2	34.00	819
W8X21	31	332.41	6968
W10X12	151	2713.26	32683
W8X15	3	60.25	910
W10X22	2	21.78	481
W12X14	104	1952.92	27645
W12X16	95	2287.25	36658
W12X19	280	6930.42	131355
W14X22	451	10767.31	237785
W14X43	1	26.00	1115
W14X30	1	18.00	542

Gravity Beam Design Takeoff

RAM Steel v11.2
DataBase: New-Floor
Building Code: IBC

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Steel Code: ASD 9th Ed.

SIZE	#	LENGTH (ft)	WEIGHT (lbs)
W14X26	1	26.00	680
W14X53	1	15.67	832
W14X34	1	26.00	885
W14X38	1	26.00	991
W16X26	242	5524.67	144378
W16X31	73	1512.34	46984
W16X36	7	164.50	5933
W18X35	69	1718.42	60228
W18X40	29	817.00	32805
W21X48	2	49.50	2375
W21X44	4	94.83	4195
W21X83	1	25.75	2129
W24X55	3	87.25	4839
W24X62	1	25.75	1603
W24X68	1	18.00	1231
W24X76	1	25.75	1963
W27X84	1	25.75	2173
W30X90	1	33.00	2964
W33X118	2	43.75	5166
W33X130	4	87.50	11404
W33X141	2	51.50	7290
W36X160	2	51.75	8276
W33X387	1	26.00	10086
W36X328	1	25.75	8447
W40X167	1	23.50	3934
W40X199	5	121.42	24169
W40X431	2	51.50	22256
W40X503	3	77.25	38904
W44X230	3	73.00	16817
W44X262	2	49.25	12938
	-----		-------------
	1745		**978703**

Total Number of Studs = **12751**

Gravity Column Design TakeOff

RAM Steel v11.2
DataBase: New-Floor
Building Code: IBC

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Steel Code: ASD 9th Ed.

Steel Grade: 50

I section

Size	#	Length (ft)	Weight (lbs)
W12X40	113	2027.3	80711
W12X45	15	287.3	12808
W12X50	17	358.6	17818
W12X53	17	305.3	16207
W12X58	11	227.0	13131
W12X65	25	463.3	30113
W12X72	4	71.3	5122
W12X79	12	197.7	15604
W12X87	4	65.7	5720
W12X96	5	105.0	10076
W12X106	6	115.7	12280
W12X120	3	30.0	3604
W12X136	3	45.0	6110
W12X170	4	70.0	11910
W12X252	1	20.0	5036
W12X279	1	15.0	4180
	241		250428

Structural Steel Cost

Project A		**150,000 SF**			$/Ton	$/SF		#/SF		
	Structural Steel		454	Tons						
	Contract	$	818,865.00		$ 1,803.67	$ 5.46		6.05		
	Vescom Joist, Joist Gi	$	822,463.00			5.48				
	Erection	$	508,000.00			3.39				
		$	**2,149,328.00**			**14.33**				
Project B		**224,000 SF**			$/Ton	$/SF		#/SF		
	Structural Steel		1,308	Tons						
	Contract	$	2,275,052.00		$ 1,739.34	$ 10.16		11.68		
	Hambro Joist, Joist Gi	$	538,948.00			2.41				
	Erection	$	970,000.00			4.33				
		$	**3,784,000.00**			**16.89**				
Project C		**231,895** SF								
	Structural Steel		231,895	SF	12	#/SF		1,391	Tons	
			1,391	Tons	$ 2,000.00	/Ton		2,782,740		
	Beam Penetartions							50,000		
	Hambro Joist & Decking							939,995		
	Erection	$	231,895	SF	4.33	/SF		1,004,188		
							$	**4,776,923**	**$ 20.60**	
Project D		**231,895** SF								
	Structural Steel		231,895	SF	12	#/SF		1,391	Tons	
			1,391	Tons	$ 2,000.00	/Ton		2,782,740		
	Beam Penetartions							50,000		
	Vescom Joist & Deck							674,635		
	Erection	$	231,895	SF	4.33	/SF		1,004,105		
							$	**4,511,480**	**$ 19.45**	
Project D (Alternative)		**231,895** SF								
	Structural Steel		231,895	SF	6.25	#/SF		725	Tons	
			725	Tons	$ 2,000.00	/Ton		1,449,344		
	Beam Penetartions							50,000		
	Vescom 16" deep Joists & Decking							675,635		
	Vescom 16" deep Joists Girders							266,216		
	Erection		231,895	SF	4.00	/SF		927,580		
	Garage Level Structura	$	52,042	SF	20.15	/SF		1,048,646		
							$	**4,417,421**	**$ 19.05**	

05 21 Steel Joist Framing

05 21 23 – Steel Joist Girder Framing

05 21 23.50 Joist Girders		Crew	Daily Output	Labor-Hours	Unit	2008 Bare Costs Material	Labor	Equipment	Total
7104	20 to 29 tons, add					20%			
7106	10 to 19 tons, add					30%			
7107	5 to 9 tons, add					50%	25%		
7108	1 to 4 tons, add					75%	50%		
7109	Less than 1 ton, add					100%	100%		
8000	Trusses, 40-ton job lots, shop fabricated WT chords, shop primer, average	E-5	11	7.273	Ton	4,950	310	154	5,414
8100	For less than 40-ton job lots								
8102	For 30 to 39 tons, add					10%			
8104	20 to 29 tons, add					20%			
8106	10 to 19 tons, add					30%			
8107	5 to 9 tons, add					50%	25%		
8108	1 to 4 tons, add					75%	50%		
8109	Less than 1 ton, add					100%	100%		

05 31 Steel Decking

05 31 13 – Steel Floor Decking

05 31 13.50 Floor Decking		Crew	Daily Output	Labor-Hours	Unit	Material	Labor	Equipment	Total
0010	**FLOOR DECKING** R053100-10								
3200	Open decking, 3" deep, wide rib, 22 gauge, galvanized, under 50 squares	E-4	3600	.009	S.F.	2.21	.39	.04	2.64
3250	50-500 squares		3800	.008		1.77	.37	.03	2.17
3260	over 500 squares		4000	.008		1.59	.35	.03	1.97
3300	20 gauge, under 50 squares		3400	.009		2.58	.41	.04	3.03
3350	50-500 squares		3600	.009		2.06	.39	.04	2.49
3360	over 500 squares		3800	.008		1.85	.37	.03	2.25
3400	18 gauge, under 50 squares		3200	.010		3.32	.44	.04	3.80
3450	50-500 squares		3400	.009		2.66	.41	.04	3.11
3460	over 500 squares		3600	.009		2.39	.39	.04	2.82
3500	16 gauge, under 50 squares		3000	.011		4.39	.46	.04	4.89
3550	50-500 squares		3200	.010		3.51	.44	.04	3.99
3560	over 500 squares		3400	.009		3.16	.41	.04	3.61
3700	4-1/2" deep, long span roof, over 50 squares, 20 gauge		2700	.012		4.13	.52	.05	4.70
3800	18 gauge		2460	.013		5.30	.57	.05	5.92
3900	16 gauge		2350	.014		3.98	.59	.06	4.63
4100	6" deep, long span, 18 gauge		2000	.016		7.60	.70	.07	8.37
4200	16 gauge		1930	.017		5.70	.72	.07	6.49
4300	14 gauge		1860	.017		7.30	.75	.07	8.12
4500	7-1/2" deep, long span, 18 gauge		1690	.019		8.35	.82	.08	9.25
4600	16 gauge		1590	.020		6.25	.88	.08	7.21
4700	14 gauge		1490	.021		8.05	.93	.09	9.07
4800	For painted instead of galvanized, deduct					2%			
5000	For acoustical perforated, with fiberglass, add				S.F.	1.09			1.09
5200	Non-cellular composite deck, galv., 2" deep, 22 gauge	E-4	3860	.008		1.53	.36	.03	1.92
5300	20 gauge		3600	.009		1.69	.39	.04	2.12
5400	18 gauge		3380	.009		2.15	.41	.04	2.60
5500	16 gauge		3200	.010		2.69	.44	.04	3.17
5700	3" deep, galv., 22 gauge		3200	.010		1.67	.44	.04	2.15
5800	20 gauge		3000	.011		1.86	.46	.04	2.36
5900	18 gauge *CN*		2850	.011		2.29	.49	.05	2.83
6000	16 gauge		2700	.012		3.06	.52	.05	3.63

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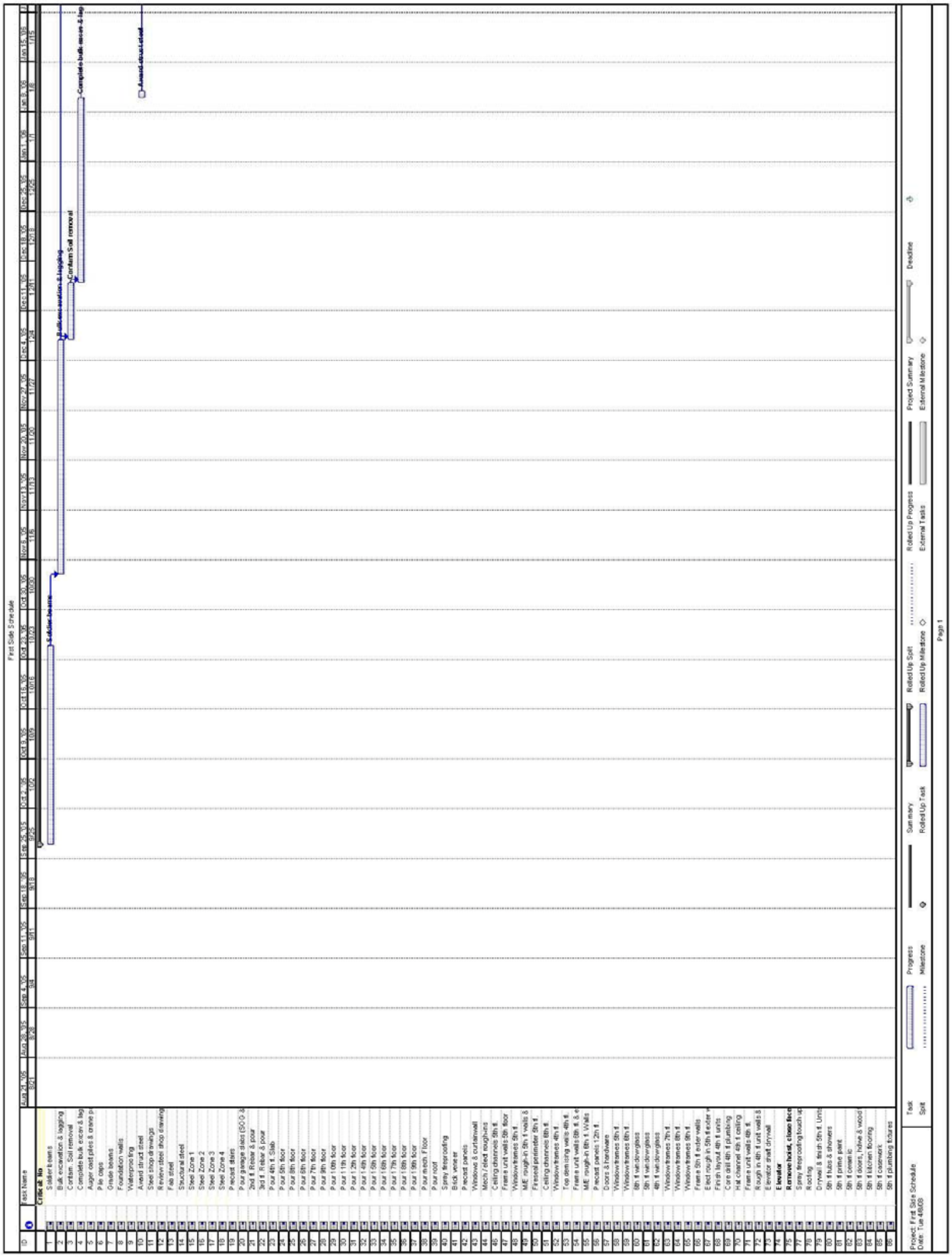
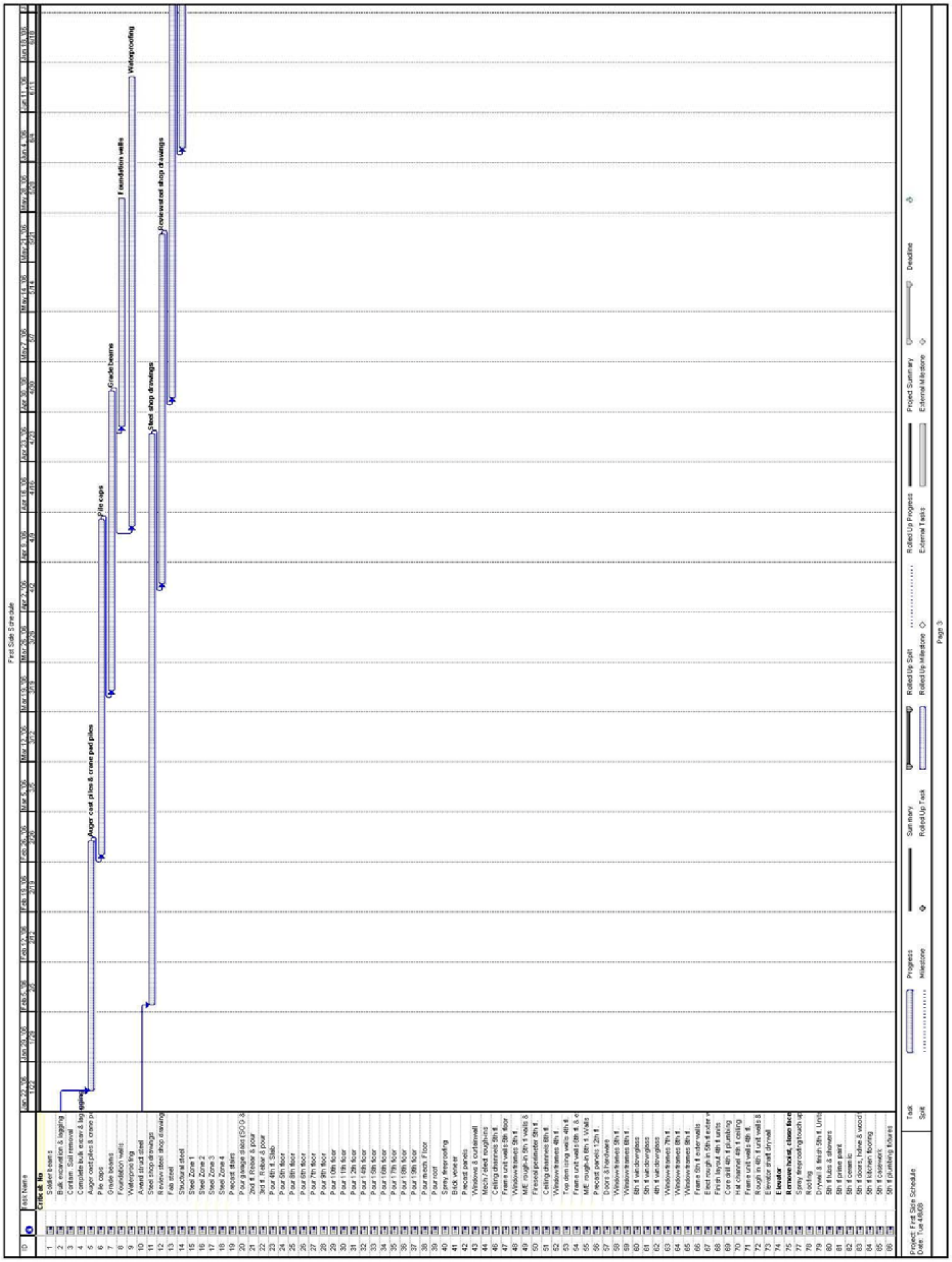
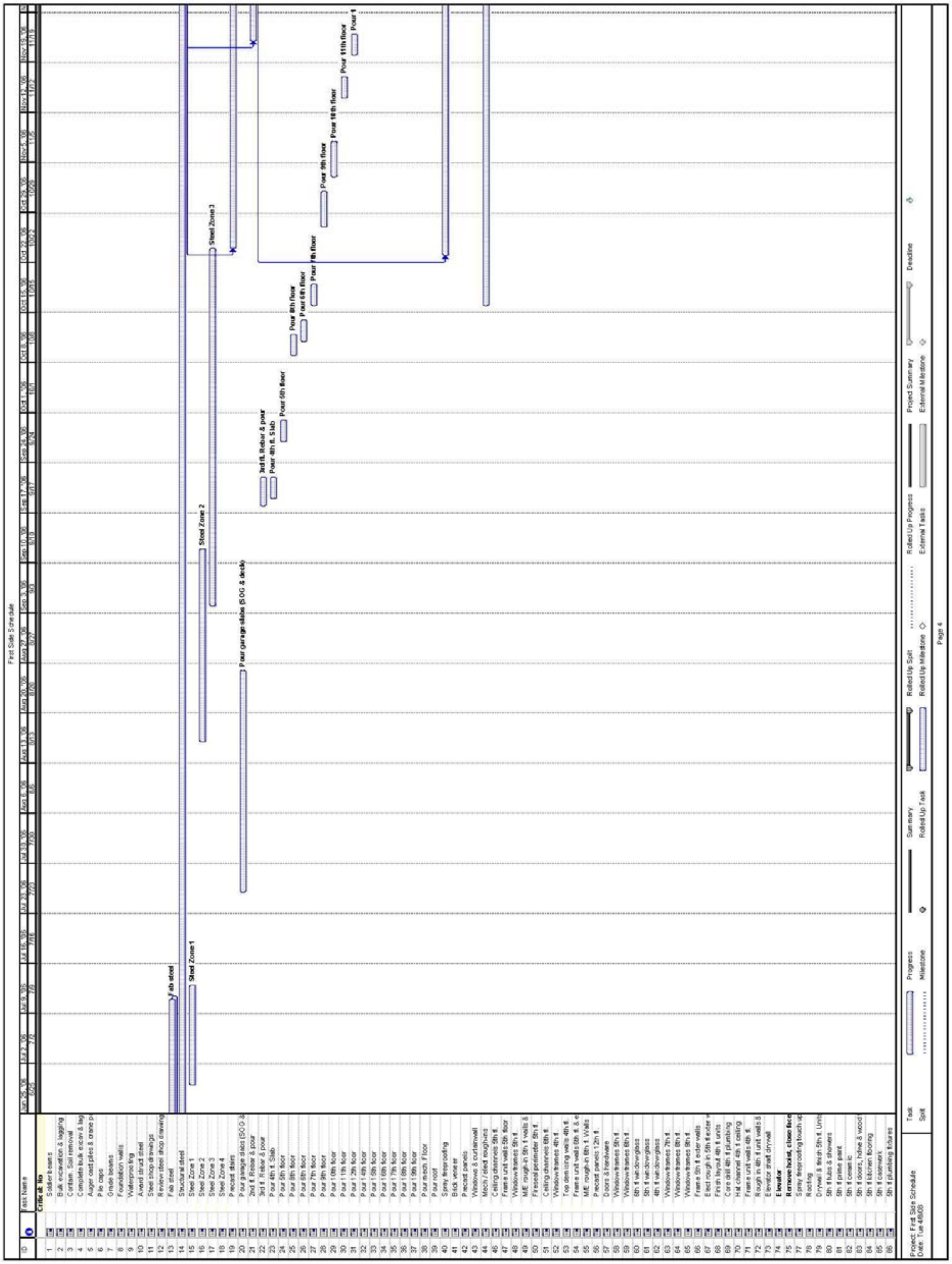
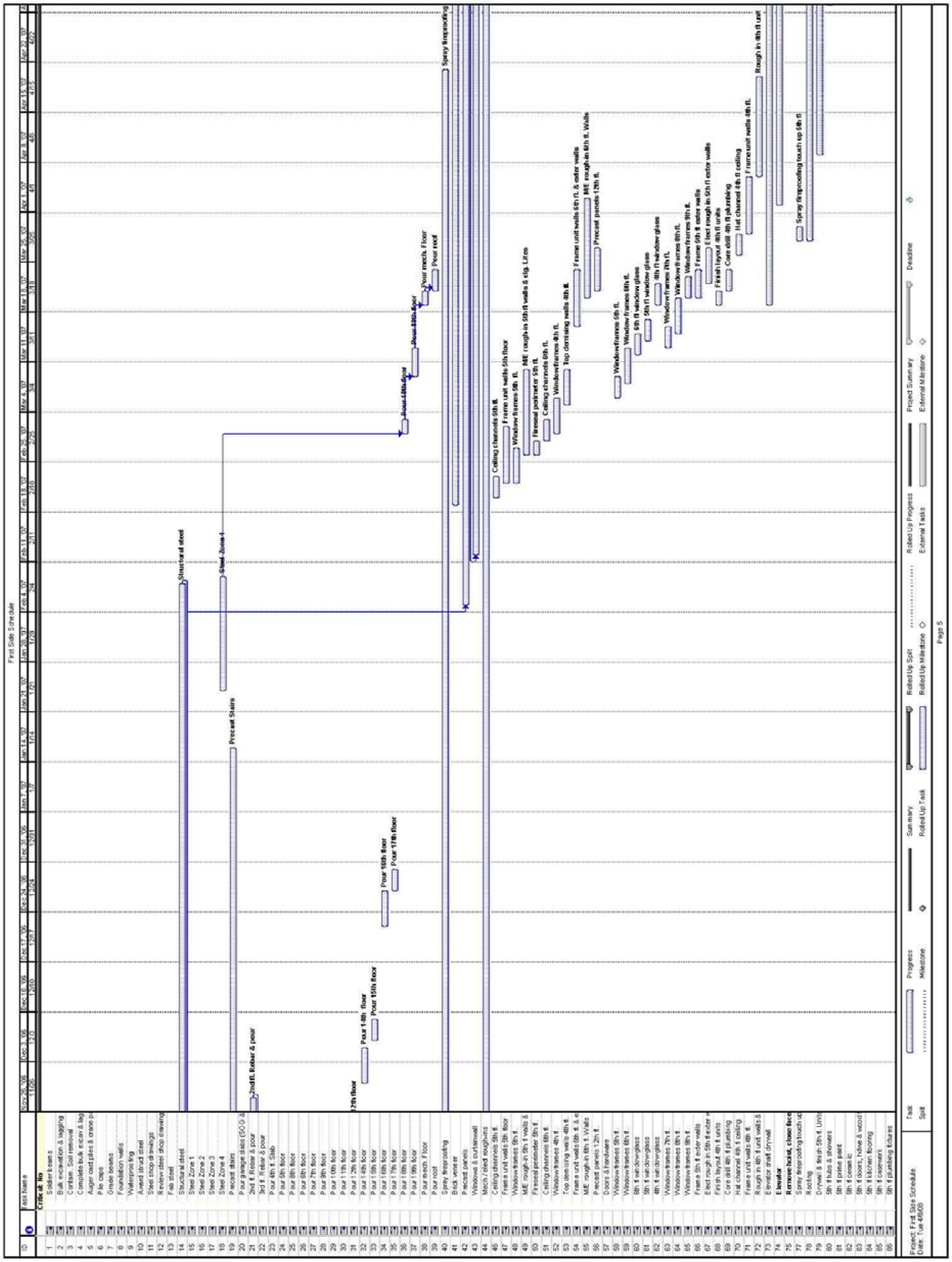
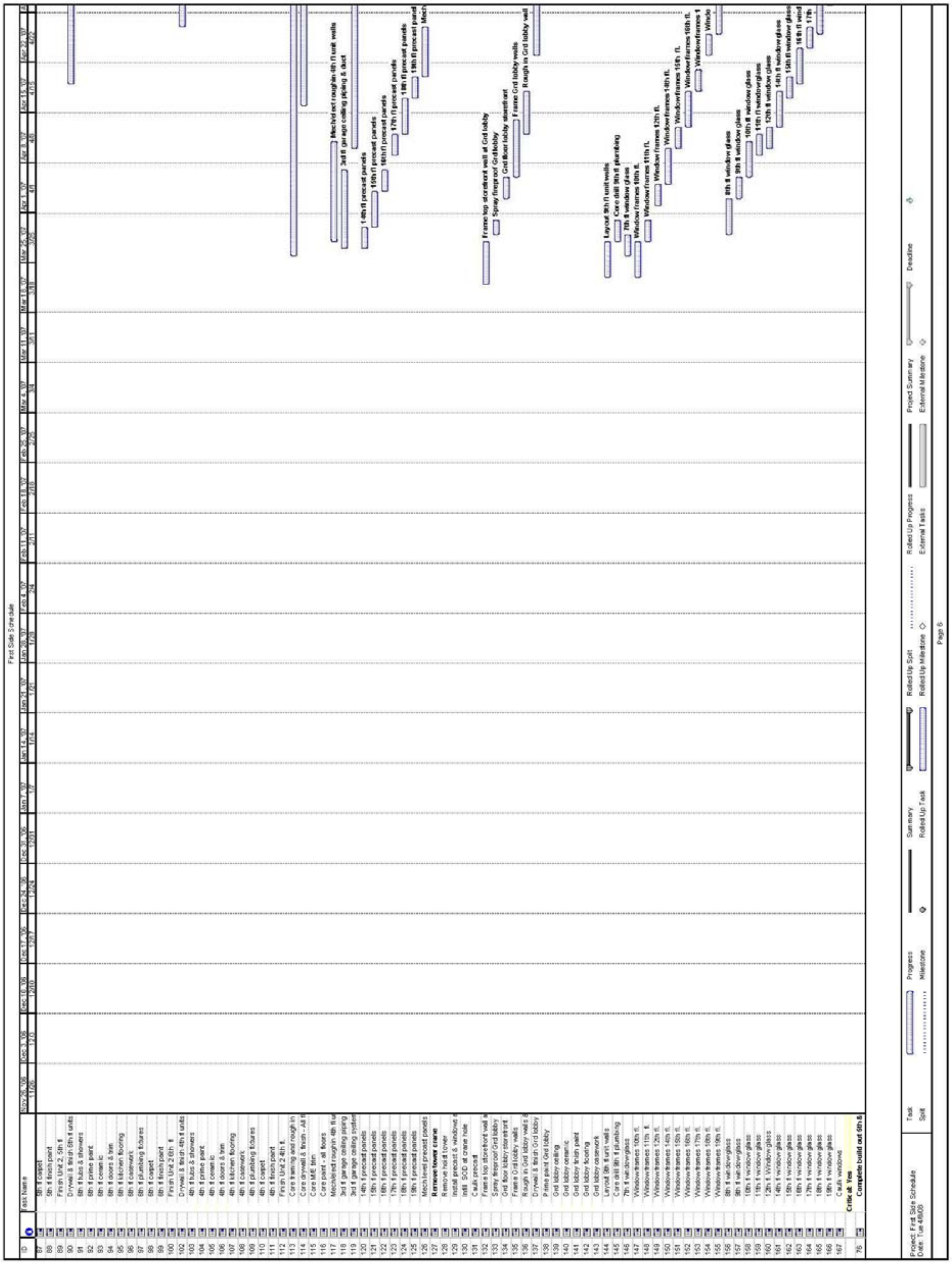
Crew No.	Bare Costs		Incl. Subs O & P		Cost Per Labor-Hour	
Crew D-9	**Hr.**	**Daily**	**Hr.**	**Daily**	**Bare Costs**	**Incl. O&P**
3 Bricklayers	$39.15	$939.60	$59.55	$1429.20	$34.88	$53.05
3 Bricklayer Helpers	30.60	734.40	46.55	1117.20		
48 L.H., Daily Totals		$1674.00		$2546.40	$34.88	$53.05
Crew D-10	**Hr.**	**Daily**	**Hr.**	**Daily**	**Bare Costs**	**Incl. O&P**
1 Bricklayer Foreman	$41.15	$329.20	$62.60	$500.80	$37.96	$57.61
1 Bricklayer	39.15	313.20	59.55	476.40		
1 Bricklayer Helper	30.60	244.80	46.55	372.40		
1 Equip. Oper. (crane)	40.95	327.60	61.75	494.00		
1 S.P. Crane, 4x4, 12 Ton		611.00		672.10	19.09	21.00
32 L.H., Daily Totals		$1825.80		$2515.70	$57.06	$78.62
Crew D-11	**Hr.**	**Daily**	**Hr.**	**Daily**	**Bare Costs**	**Incl. O&P**
1 Bricklayer Foreman	$41.15	$329.20	$62.60	$500.80	$36.97	$56.23
1 Bricklayer	39.15	313.20	59.55	476.40		
1 Bricklayer Helper	30.60	244.80	46.55	372.40		
24 L.H., Daily Totals		$887.20		$1349.60	$36.97	$56.23
Crew D-12	**Hr.**	**Daily**	**Hr.**	**Daily**	**Bare Costs**	**Incl. O&P**
1 Bricklayer Foreman	$41.15	$329.20	$62.60	$500.80	$35.38	$53.81
1 Bricklayer	39.15	313.20	59.55	476.40		
2 Bricklayer Helpers	30.60	489.60	46.55	744.80		
32 L.H., Daily Totals		$1132.00		$1722.00	$35.38	$53.81
Crew D-13	**Hr.**	**Daily**	**Hr.**	**Daily**	**Bare Costs**	**Incl. O&P**
1 Bricklayer Foreman	$41.15	$329.20	$62.60	$500.80	$36.76	$56.05
1 Bricklayer	39.15	313.20	59.55	476.40		
2 Bricklayer Helpers	30.60	489.60	46.55	744.80		
1 Carpenter	38.10	304.80	59.30	474.40		
1 Equip. Oper. (crane)	40.95	327.60	61.75	494.00		
1 S.P. Crane, 4x4, 12 Ton		611.00		672.10	12.73	14.00
48 L.H., Daily Totals		$2375.40		$3362.50	$49.49	$70.05
Crew E-1	**Hr.**	**Daily**	**Hr.**	**Daily**	**Bare Costs**	**Incl. O&P**
1 Welder Foreman	$45.00	$360.00	$81.20	$649.60	$41.92	$71.90
1 Welder	43.00	344.00	77.55	620.40		
1 Equip. Oper. (light)	37.75	302.00	56.95	455.60		
1 Welder, gas engine, 300 amp		132.20		145.42	5.51	6.06
24 L.H., Daily Totals		$1138.20		$1871.02	$47.42	$77.96
Crew E-2	**Hr.**	**Daily**	**Hr.**	**Daily**	**Bare Costs**	**Incl. O&P**
1 Struc. Steel Foreman	$45.00	$360.00	$81.20	$649.60	$41.86	$72.30
4 Struc. Steel Workers	43.00	1376.00	77.55	2481.60		
1 Equip. Oper. (crane)	40.95	327.60	61.75	494.00		
1 Equip. Oper. Oiler	35.10	280.80	52.95	423.60		
1 Lattice Boom Crane, 90 Ton		1567.00		1723.70	27.98	30.78
56 L.H., Daily Totals		$3911.40		$5772.50	$69.85	$103.08
Crew E-3	**Hr.**	**Daily**	**Hr.**	**Daily**	**Bare Costs**	**Incl. O&P**
1 Struc. Steel Foreman	$45.00	$360.00	$81.20	$649.60	$43.67	$78.77
1 Struc. Steel Worker	43.00	344.00	77.55	620.40		
1 Welder	43.00	344.00	77.55	620.40		
1 Welder, gas engine, 300 amp		132.20		145.42	5.51	6.06
24 L.H., Daily Totals		$1180.20		$2035.82	$49.17	$84.83
Crew E-4	**Hr.**	**Daily**	**Hr.**	**Daily**	**Bare Costs**	**Incl. O&P**
1 Struc. Steel Foreman	$45.00	$360.00	$81.20	$649.60	$43.50	$78.46
3 Struc. Steel Workers	43.00	1032.00	77.55	1861.20		
1 Welder, gas engine, 300 amp		132.20		145.42	4.13	4.54
32 L.H., Daily Totals		$1524.20		$2656.22	$47.63	$83.01

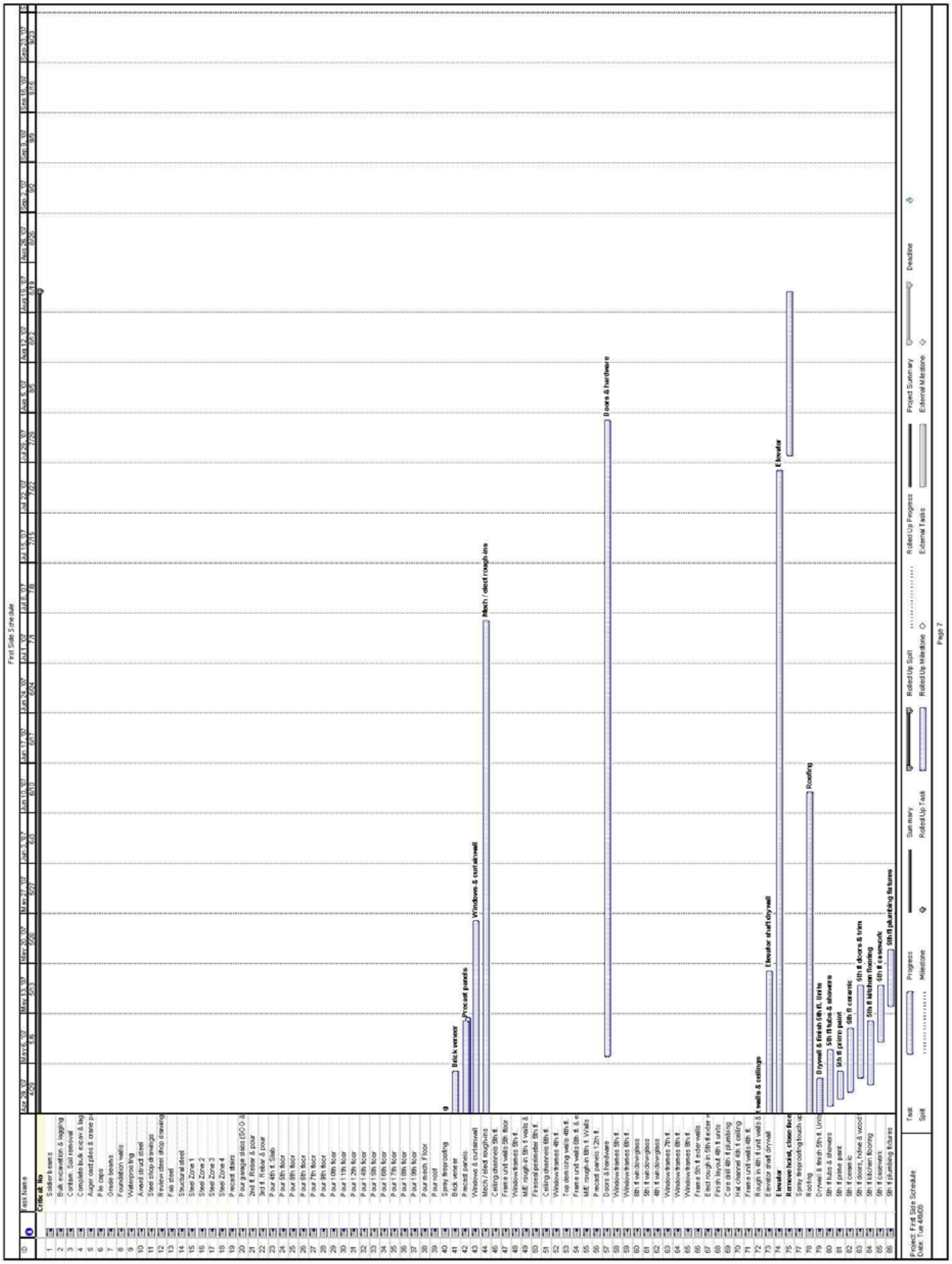


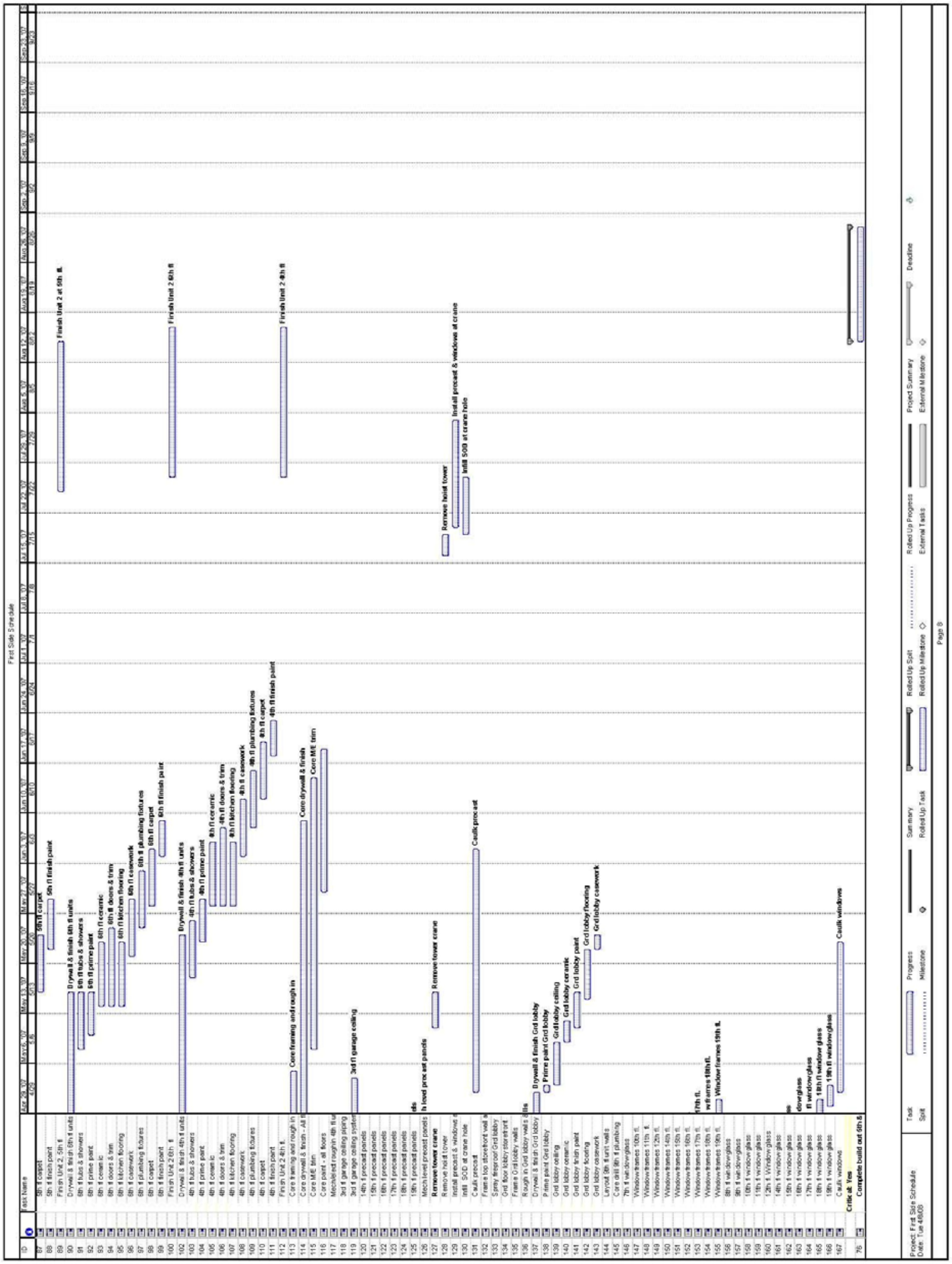


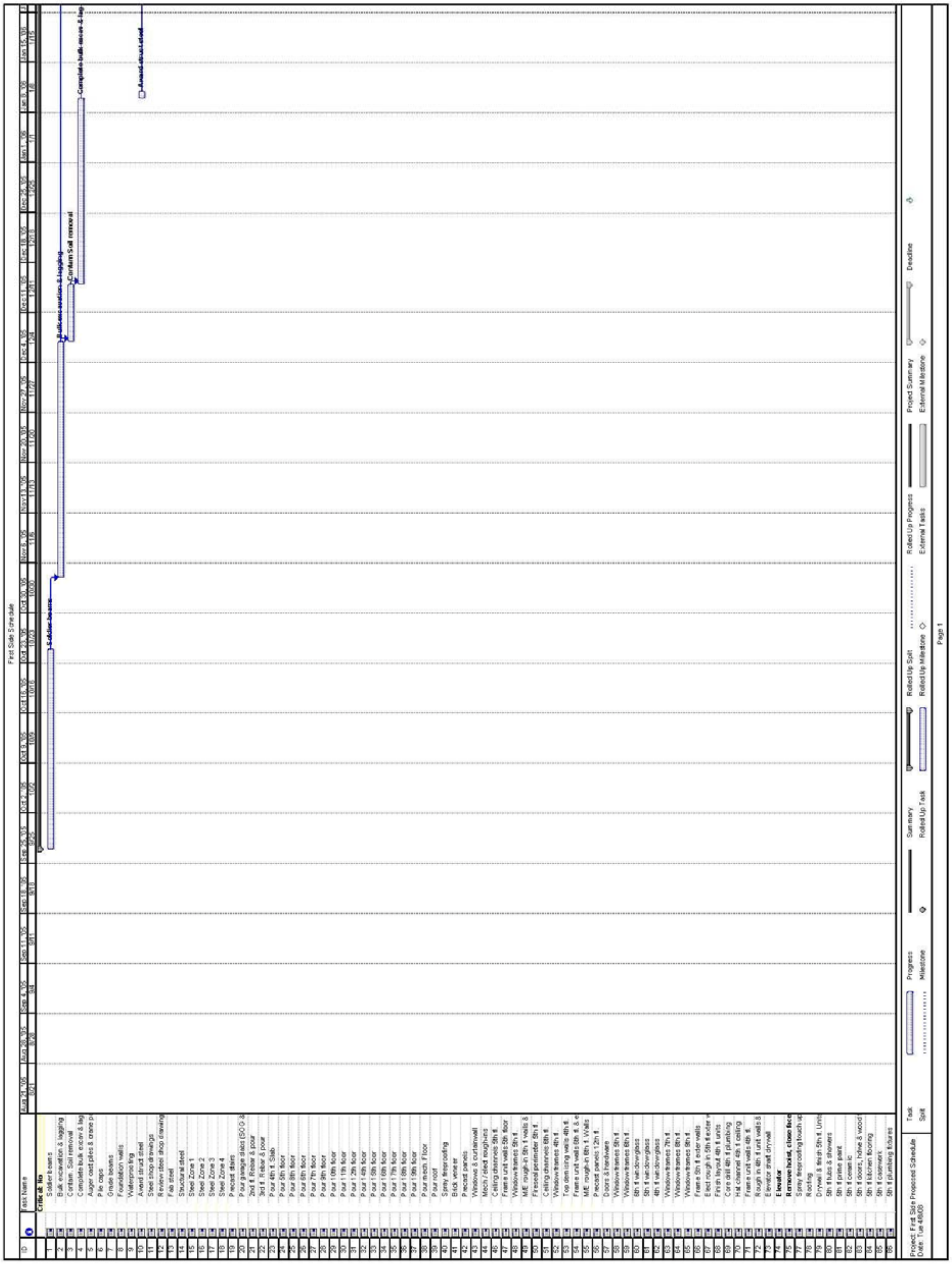


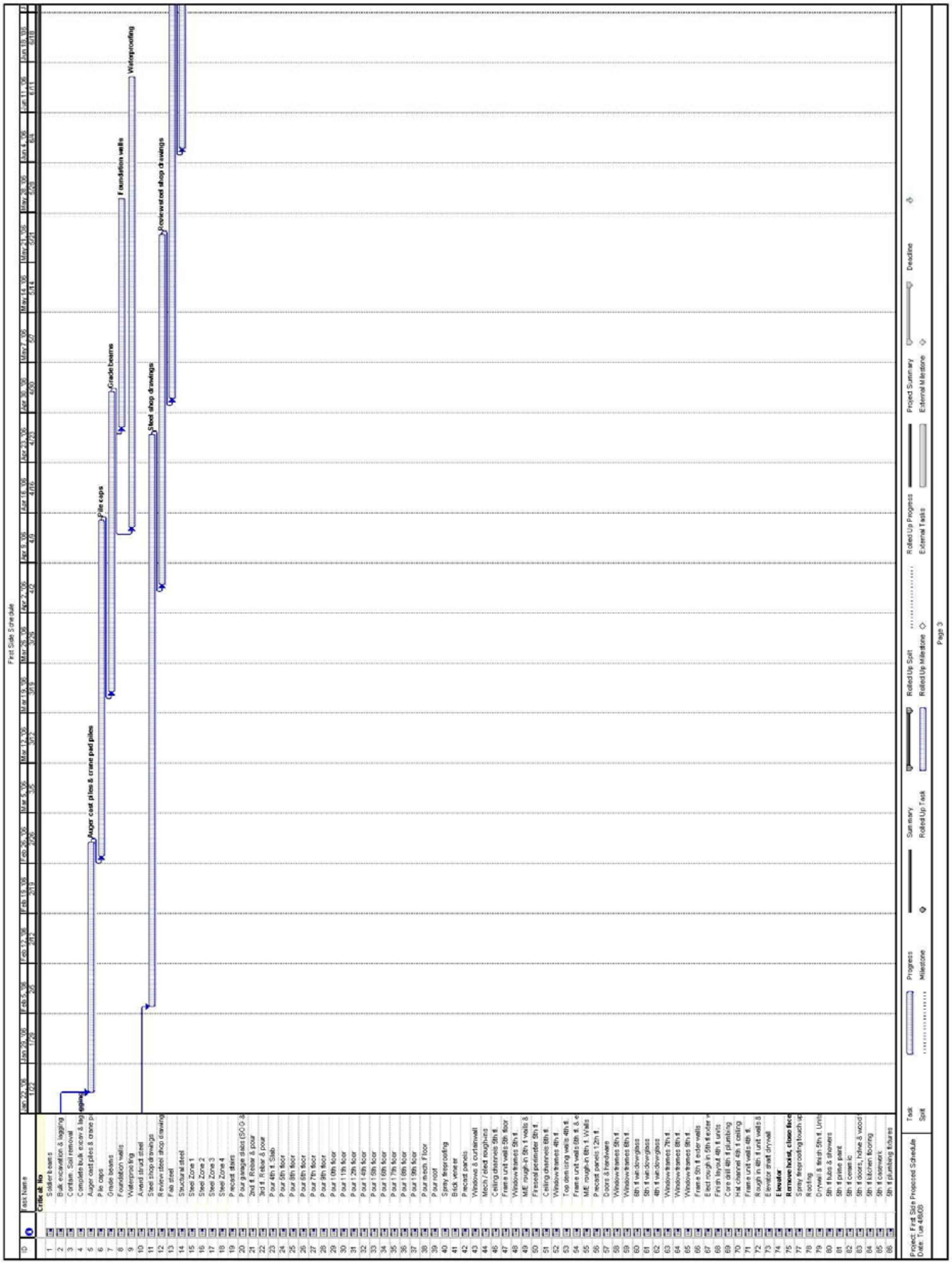









First Side Schedule


First Side Schedule
Auger cast piles & crane pad piles
Pile caps
Grade beams
Foundation walls
Waterproofing
Steel shop drawings
Reviewed shop drawings
Page 3