



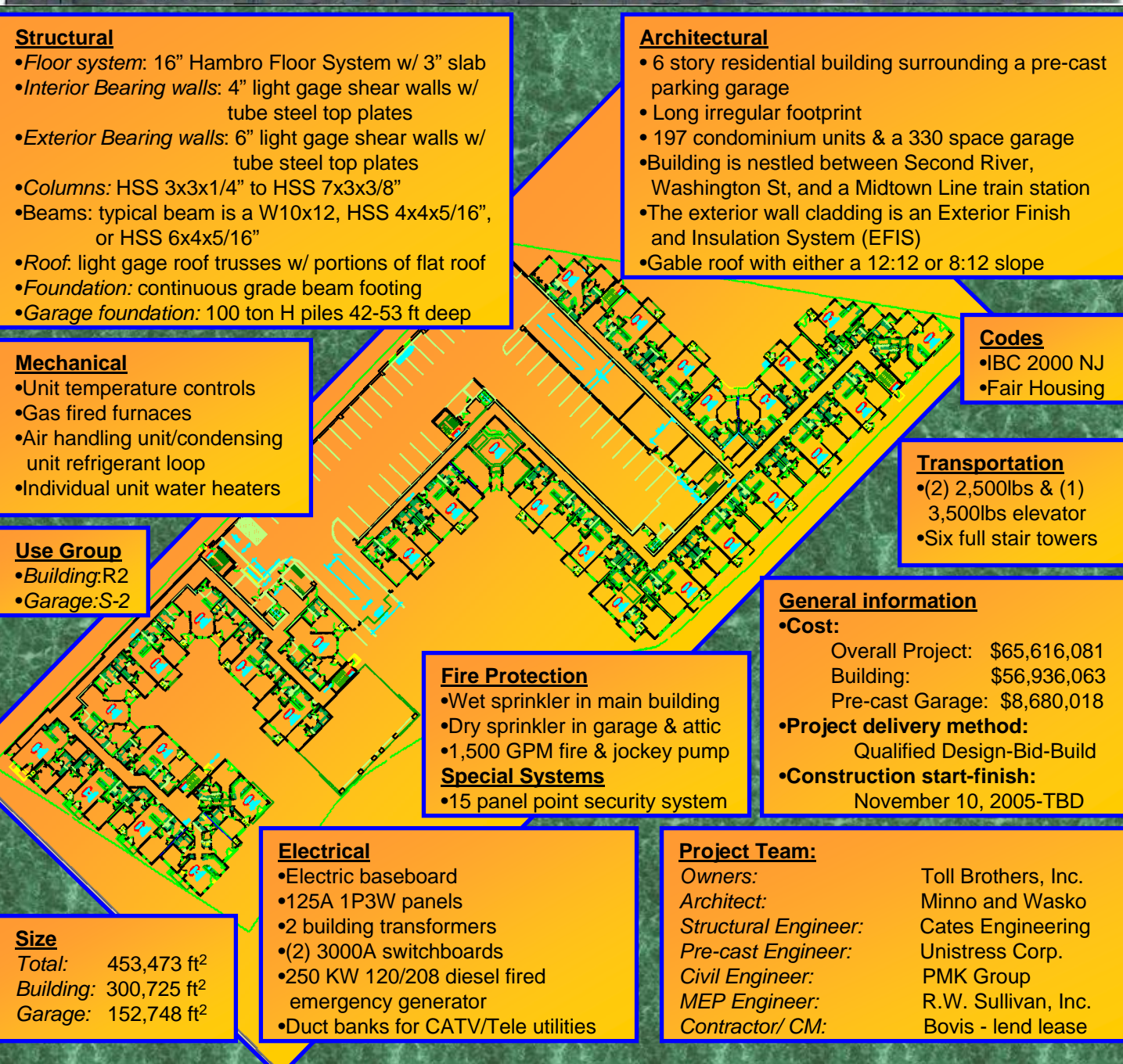
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Senior in Architectural Engineering
Structural Option
MAE/ BAE
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Senior Thesis Project:
Parkview at Bloomfield Station
Spring 2006

Faculty Consultant:
Professor Parfitt

Robert Whitaker

Robert S. Whitaker
Structural Option
Parkview at Bloomfield Station
Bloomfield, NJ
www.arche.psu.edu/thesis/eportfolio/current/portfolios/rsw153/



Structural

- Floor system: 16" Hambro Floor System w/ 3" slab
- Interior Bearing walls: 4" light gage shear walls w/ tube steel top plates
- Exterior Bearing walls: 6" light gage shear walls w/ tube steel top plates
- Columns: HSS 3x3x1/4" to HSS 7x3x3/8"
- Beams: typical beam is a W10x12, HSS 4x4x5/16", or HSS 6x4x5/16"
- Roof: light gage roof trusses w/ portions of flat roof
- Foundation: continuous grade beam footing
- Garage foundation: 100 ton H piles 42-53 ft deep

Architectural

- 6 story residential building surrounding a pre-cast parking garage
- Long irregular footprint
- 197 condominium units & a 330 space garage
- Building is nestled between Second River, Washington St, and a Midtown Line train station
- The exterior wall cladding is an Exterior Finish and Insulation System (EFIS)
- Gable roof with either a 12:12 or 8:12 slope

Mechanical

- Unit temperature controls
- Gas fired furnaces
- Air handling unit/condensing unit refrigerant loop
- Individual unit water heaters

Codes

- IBC 2000 NJ
- Fair Housing

Use Group

- Building: R2
- Garage: S-2

Transportation

- (2) 2,500lbs & (1) 3,500lbs elevator
- Six full stair towers

Fire Protection

- Wet sprinkler in main building
- Dry sprinkler in garage & attic
- 1,500 GPM fire & jockey pump

Special Systems

- 15 panel point security system

General information

- **Cost:**
 Overall Project: \$65,616,081
 Building: \$56,936,063
 Pre-cast Garage: \$8,680,018
- **Project delivery method:**
 Qualified Design-Bid-Build
- **Construction start-finish:**
 November 10, 2005-TBD

Electrical

- Electric baseboard
- 125A 1P3W panels
- 2 building transformers
- (2) 3000A switchboards
- 250 KW 120/208 diesel fired emergency generator
- Duct banks for CATV/Tele utilities

Size

- Total: 453,473 ft²
- Building: 300,725 ft²
- Garage: 152,748 ft²

Project Team:

Owners:	Toll Brothers, Inc.
Architect:	Minno and Wasko
Structural Engineer:	Cates Engineering
Pre-cast Engineer:	Unistress Corp.
Civil Engineer:	PMK Group
MEP Engineer:	R.W. Sullivan, Inc.
Contractor/ CM:	Bovis - lend lease

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Parkview at Bloomfield Station



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Senior Thesis Final Report

By Robert Whitaker



Executive Summary

This thesis presents the process taken to create a new design for Parkview at Bloomfield Station, a residential building located in Bloomfield, New Jersey. The summary of the original design is introduced and the initial design requirements are laid out. From here, the new structural system is created and analyzed based on structural and architectural requirements. Finally, two breadth topics are introduced to determine the new buildings cost advantage and cladding as compared to the original design.

Original Design

The original structural system is composed of light gage roof trusses, panelized bearing light gage walls, 16" deep D500 Hambro® floor joists, and 38 shear walls in the main lateral force resisting system. The precast garage at the center of the building is structurally separate, and only the 4" building separation is considered for story drift.

New Structural Design

The analysis of a new steel braced frame design is conducted to replace the current light gage bearing wall system. The most recent codes are used in the design analysis, updating the codes used in the original design. From the two different framing orientations analyzed, the 20K9 bar joist floor system spanning 38'-0" was concluded to be the most efficient and compatible design. Furthermore, the use of the braced frame system requires less lateral frames than the original system, creating the use of gravity frames at some unit separation locations. This helps to preserve the architecture of the living units while allowing for changes in future use. The foundation system in the new design is composed of spread footings that replace the original strip footings. In addition to these structural issues, a vibration analysis on the bar joist system shows that the floor is over the design limits but can still be considered acceptable.

Breadth Overview

This section investigates two breadth topics that effect large portions of Parkview at Bloomfield Station. First, the effect of changing from a Hambro® system to the new bar joist on steel frame system had on the cost and schedule is analyzed to show that the new system has cost benefits and better sequencing flexibility than the original system. Secondly, an analysis of the current Exterior Insulation and Finish System (EIFS) is studied and shown that a drainable EIFS system is the best solution.

Introduction

Architectural Engineering is a five year program at Penn State University that develops engineers to become well versed in a main area of study and sufficiently adept in three other areas pertinent to the building industry. The four areas of study offered are structural engineering, mechanical engineering, lighting/electrical design engineering, and construction management.

I have elected to study structural engineering as my main area of study. However, this report does not cover all areas of training that I have received in my five years at Penn State, but focuses in on the areas of knowledge that were pertinent to the building presented. Similarly, the breadth of topics included at the end of this report represent a small portion of the accumulated knowledge learned from the mechanical, lighting/electrical, construction management and other various courses taken at Penn State.



Acknowledgements/ Credits

I would like to thank the following people for their help and support with my senior thesis:

Professor Parfitt *Professor Hanagan*
Professor Memari *Professor Schneider*
Professor Geschwindner *Professor Bowers*
All the PSU Faculty and Staff

Cates Engineering
Michael Stansbury

Toll Brothers, Inc.
Minno & Wasko

Lauren Whitaker ~ my wife
My Family
Jesus



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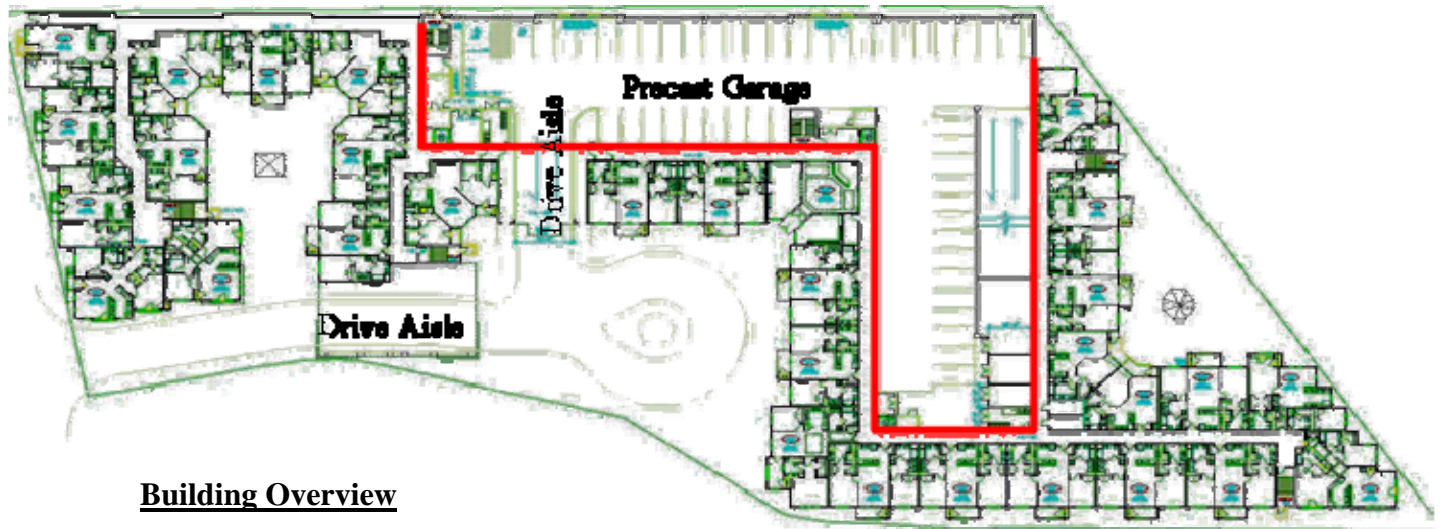
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Section I
Existing Building Description

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Building Overview

Parkview at Bloomfield Station is a uniquely shaped six story residential condominium building located in Bloomfield, NJ. The building is most noticeable for its long sprawling irregular footprint. Part of the reason for its irregular shape is due to the shape of the lot and the fact that the building wraps around a precast parking garage. The parking garage is only visible from the train station side of the building as seen in the rear elevation below. The building is nestled between the Second River to the south, Washington St. to the west, a tree filled lot in the east, and a train station for the Midtown Line to the north. There are 197 condominium units and 330 parking spaces included in the design of this building. Numerous storage facilities are located in the parking garage and an exercise room is also included, located above the lobby area. A drop off circle, located just off of the tree lined entry drive, allows for easy access for visitors and taxi services. An outdoor gazebo and portico area is included in the project for residential enjoyment.

<i>Garage:</i>	<i>152,748 ft²</i>
<i>Building:</i>	<i>300,725 ft²</i>
<i>Per Floor:</i>	<i>50,121 ft²</i>
<i>Total:</i>	<i>453,473 ft²</i>



Rear Elevation

Building Envelope

The roof system consists of asphalt shingle roofing over metal deck and fire rated plywood. The light gage roof trusses have either a 12:12 slope or 8:12 slope for gable roofs, with portions in the middle acting as a flat roof. Standing seam copper roof is used in some areas. The main body of the roof consists of hip roof conditions accented with gable roof sections over balcony areas. The exterior wall cladding is an Exterior Finish and Insulation System (EFIS) over light gage walls. Anderson vinyl windows and entry doors also make up parts of the wall system. The wall along the train line consists mainly of the open precast garage panels.

Type of Construction

Site work includes existing building demolition, grading, importing fill and locating new utilities. It will also include installation of fountains, gazebo and a portico. The construction of the parking garage will be completed first, using a crane to lift the precast double T shapes into place. Upon completion of the parking garage the panelized light gage walls will be lifted into place and anchored with the floor system. The delivery method is a qualified design-bid-build for both the building and the parking garage. The total building cost is \$65,616,081, not including a CM bond.

Electrical System

The electrical panels are sized at 125A 1P3W, and there are two building transformers. Secondary service includes feeders in GRC conduit from the utility company transformers to the (2) 3000 amp switchboards. Apartment services are fed from meter centers located on each floor at 2 locations, with 125 amp feeders to each apartment load center with 3C#1 armored interlock cables. The emergency generator is a 250 KW 120/208, diesel fired with an 800 amp ATS. The HVAC equipment in the units is based on (1) 20/3p feed to the HWH/HVAC unit, and the electric range and dryer are each supplied with 50/2P feeds. Each unit is accessed with (1) ¾" conduit from the Tele/Com closet and (1) 1" pull wire conduit for CATV requirements. The electrical system includes the incoming duct banks for Electrical/CATV/Tele utilities, all to be concrete encased with pulled wires. Also included in the outfitting of the building and garage area are the lighting, HVAC, FA, telephone and CATV systems.

Lighting System

There is lighting located exteriorly at the fountain, gazebo, and portico. In addition, site lighting includes pole fixtures, and the access point of the existing (2) 5" PVC utility line. Interior lighting consists of MC cabling and EMT within garage areas. Temporary lighting and power is included.

Mechanical System

The HVAC system includes sheet metal ducts covered in 1 ½" insulation, insulation board on the exterior walls, unit temperature controls, refrigerant piping, water heaters, condensate drain piping, grd's testing and balancing, and rigging. Electric base board, air handling units, gas fired furnaces and air cooled condensing units compose the HVAC system. There is a refrigerant loop between air handling units and condensing units. Hot water from hot water heaters is sent to air handling units for use in heating air. The exhaust fan is a Nutone LS80 (with no lights).

Fire Protection and Plumbing

The fire protection system includes a fire pump, wet sprinkler system and a dry sprinkler system in the attic area. The garage will have a dry sprinkler system. The wet fire protection system is connected to a 6" combination water main, installed with a 1,500 GPM fire pump and jockey pump. There is a fire department connection on the west side of the building to the fire protection room. The dry systems in the attic and garage are on a connected system and branched from this room. The dry system will be installed with galvanized steel pipe. The floor mains and branches will be orange CPVC with sprinkler heads concealed in the plaster ceiling.

The building plumbing includes a sanitary system, natural gas, domestic water with booster pump, plumbing fixtures, gas hot water heaters, washing machine indirect waste and water heater indirect waste. The main utility room is located outside the 2nd floor trash room. The garage includes storm water and oily water drains, non-freeze wall hydrants connected to the condo building and domestic water. The sanitary system is composed of schedule 40 PVC pipe and drainage fittings. The natural gas system is based on a load of 70 CFH per hot water heater. It is supplied from a 10" line from the source and branches off into schedule 40 black steel pipes varying in size from 8" to 3" with 2" valve taps for single risers. Domestic water is brought in through a 6" pipe with water meter and backflow preventer. Once inside, it is split into (2) 4" mains for north and south halves. Main sizes vary from 4" down to 2" based on 14.5 WSFU per condo and velocity less than 8 FPS. A Triplex water booster pump is used to maintain the water pressure to upper floors. Plumbing fixtures are attached with type DWV copper piping. The hot water heater is a direct vent with a 75 gallon storage capacity and a 100 GPH recovery.

Other Building Systems

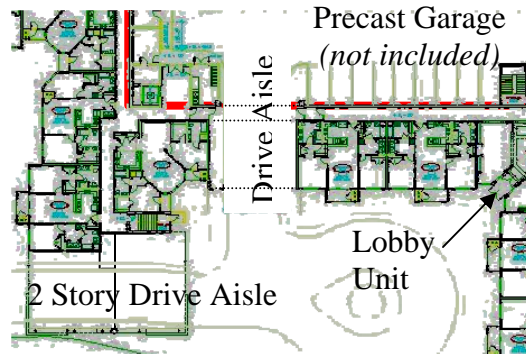
Telephone and CATV outlets are provided for each living unit. There are six stair wells that have access to all six levels of the building, and access to all floors from the 6 story attached parking garage. In addition, there are (2) 6-stop 2,500lbs elevators and (1) 6-stops 3,500lbs elevator. Finally, a security system terminal is located at the main entrance as well as fourteen additional locations with panel points. Further, there is one overhead electric security parking door at the garage.

Original Structural System

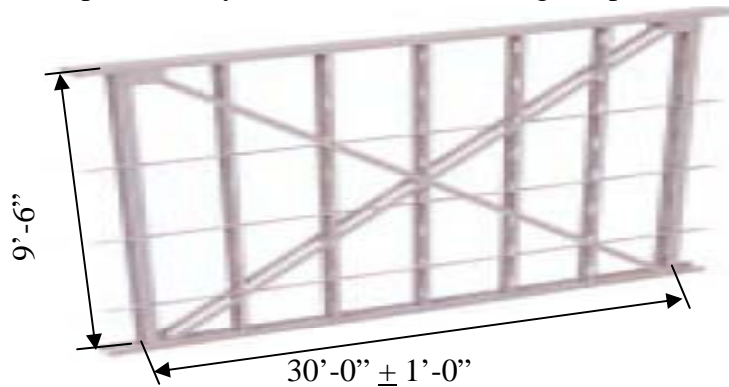
Building Framing

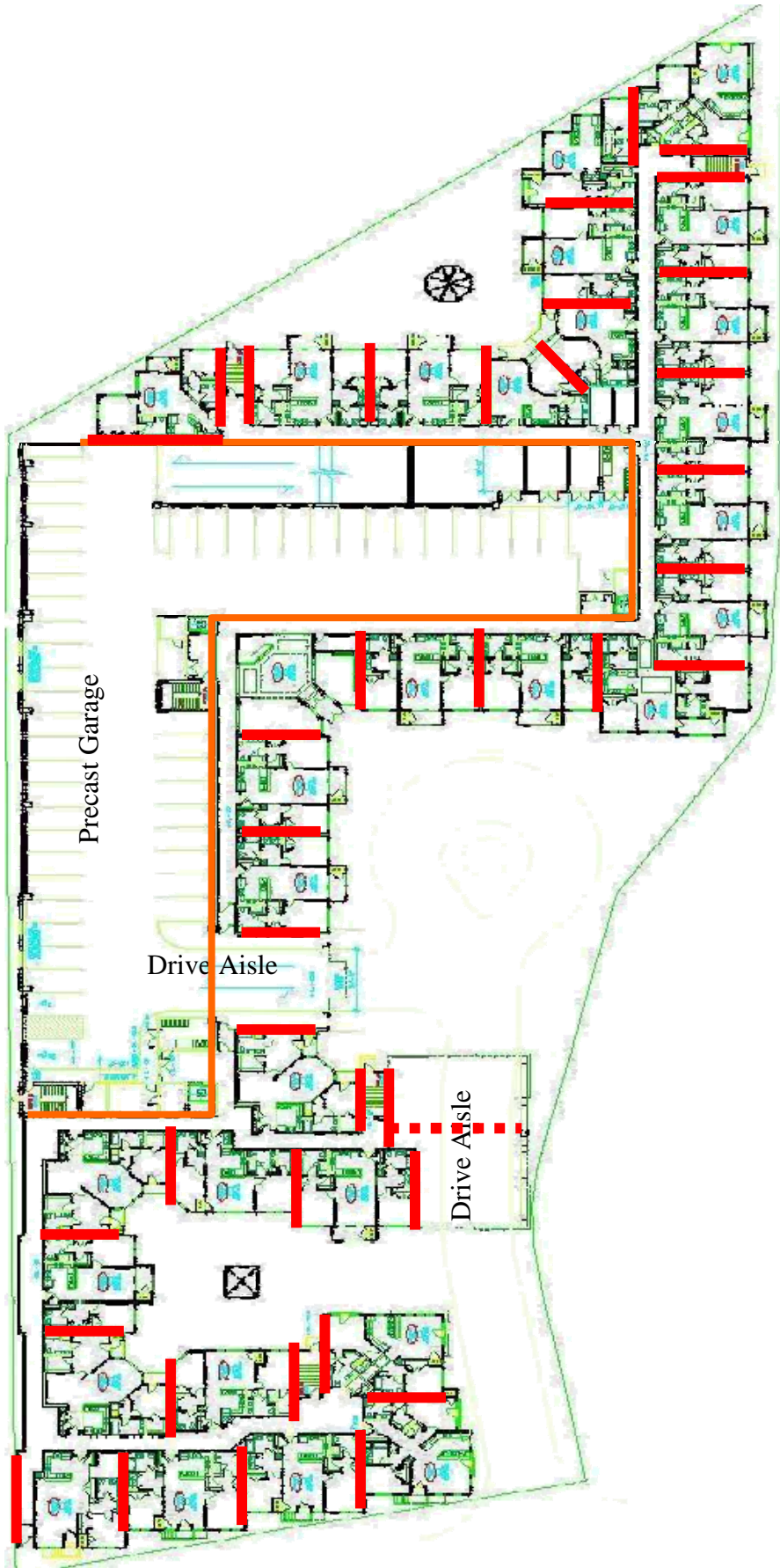
The structural system for Parkview at Bloomfield Station is composed of a light gage roof spaced 2' on center (oc) spanning front to back with some hip conditions incorporated, bearing on exterior and corridor walls, and girder trusses at hip roof conditions. Beams and transfer beams provide bearing points for the floor system, columns, and roof trusses. The bearing walls are panelized bearing light gage steel stud walls 4" and 6" wide continuously capped with a steel tube, HSS 4x4x5/16" and HSS 6x4x5/16" respectively, for load distribution purposes.

Along with the bearing light gage walls, there are two braced frame systems at the drive aisles that pass under the building. The upper floors in these sections are supported by a series of one or two story columns that are part of this W18 braced frame system. All six floors of the building have mainly the same floor plans with the exception of four locations: an entry/lobby unit, a two story drive aisle, a one story drive aisle, and a 1st floor exit route. In these areas, transfer beams are utilized requiring much larger beam sizes. The two story braced frame system used in the two story drive aisle consists of nineteen W18 columns placed along bearing lines. There is a similar system at the one story drive aisle consisting of twelve columns.



While these braced frames act as the lateral force resisting system in these two unique areas, the main lateral force resisting system for the building is a shear wall system. This resisting system is provided by thin steel cross bracing straps attached to the light gage shear walls, as seen in the image on the right. There are eighteen shear walls in the N-S direction of the building and 20 shear walls in the E-W direction. The placement of these shear walls throughout the building is shown on the next page.



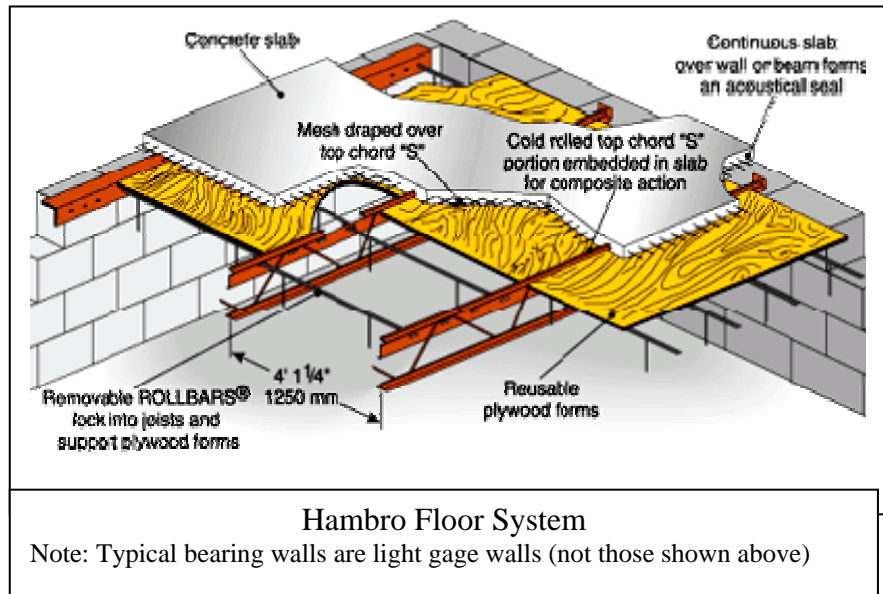


Shear Wall Locations

Hambro® Floor Framing

A 16" deep Hambro® D500™ floor system makes up the composite rigid floor diaphragm and consists of joists spaced at 4'-1¼" oc connected to a 3" concrete floor (3000psi). The 4'-1¼" joist spacing is based on the standard dimensions of a plywood panel. This method of formwork is discussed in more detail in Section III: Breadth Studies ~ Cost Advantages. The bottom chord (Fy = 50,000psi min.) acts as a tension member in the concreting stage and during the service life of the floor. The Hambro web system tying the top and bottom chords consists of bent rods (Fy = 44,000psi min.) and resists vertical shear in a conventional truss manner. The patented 13 gage top chord (Fy = 50,000psi min.) acts as a compression member during the non-composite stage.

In the composite stage, the top chord (including an "S" shape extension that is embedded in the concrete) functions as a continuous shear connector. The concrete slab is supported during the concrete pour by reusable plywood panel forms. The forms are located between joists and braced by ROLLBARS® that are held in place by holes in the top chord of the joist. The concrete slab is reinforced with a



6x6 welded wire mesh system. The "S" on the top chord functions as a high chair for this wire mesh, developing the negative moment capacity in the composite system which produces the effect of a continuous one-way reinforced slab over the joists. The 16" Hambro joists span the short direction of the living units (typically 30' ± 1'-0") and Hambro RTC joists (top cord only joists) span and support the corridor (typically 6'). The total ceiling to floor depth is 21" including the drop ceiling depth. Also, the joist system allows the mechanical duct work to pass through the open webs of the joists.

The precast garage, located at the center of the building, consists of precast double-T planks bearing on precast load bearing elements. The vertical elements in the garage transfers its' load to pile caps encompassing 100 ton H piles drilled to bedrock (ranging from 42-53 feet below the slab-on-grade surface). The precast garage is structurally separated from the main building by a 4" air gap and by 4" expansion joints at building connection points. Because of this the garage will not be considered in the new building design and will remain the same.

Site and Foundations

The site where the building sits is in Bloomfield, NJ which is located along the east coast near New York City. This site is close to Second River and contains a modest level of top soil that has been deposited by the river over the course of years. This layer needs to be scraped off prior to the placement of the foundation. The soil below this top soil level has an allowable bearing capacity of 3000 psf. The frost depth for this area is 4'-0" and will further necessitate the removal of upper layers of soil. The site is located at the center of an east coast seismic epicenter. Furthermore, since the site is located near the coast line it experiences greater wind speeds.

Finally, continuous 2'-6" wide footings make up most of the building bearing wall support under the 4" slab-on-grade foundation. However, larger spread footings (typically 4'x 4') are utilized below leaning column point loads. The spread footings supporting the one and two story drive aisle columns merge together and resemble larger single spread footings. Yet, out of the entire building there are only 43 isolated spread footings including the drive aisle columns. This small number of isolated footings is partially due to the fact that the continuous footings support the smaller tube steel column loads. The foundation of the parking garage encompasses a deep foundation system rather than the main buildings shallow footing system. Since the deep foundation will not be relying on the same soil as the main building and the footings of the precast garage are separate from that of the main building's foundation, they were ignored in the initial building design.

Building Parameters

Original Design Theory

The design theory used in the original analysis of Parkview at Bloomfield Station was Allowable Stress Design (ASD). The beam calculations were designed using American Institute of Steel Construction (AISC) 9th Edition ~ ASD and designed using the Enercalc[®] program (ASD based). The tube steel columns were also designed based on the column tables in chapter 3 of the AISC 9th Edition ~ ASD manual.

Building Code References

The original design of the structure was in accordance with the International Building Code (IBC) 2000 with New Jersey amendments, the New Jersey Uniform Construction Code, and local county and township requirements. IBC 2000 used design loads specified in ASCE 7 for both gravity and lateral loadings. Furthermore, the New Jersey amendments to IBC 2000 did not create any changes to the structural code requirements of IBC 2000, but focused more on non-structural issues. In addition, no changes to the structural design requirements were added by the New Jersey Uniform Construction Code or any of the local requirements.

The live loads and dead loads used in the initial design were taken from ASCE-7 Table 4-1, and are seen in the table below. The only exception is that the original designers added 10 psf on the top and bottom chord of the roof trusses. The additional 10 psf on the top chord was added to account for snow drift and the 10 psf added to the bottom chord accounts for any light attic storage.

Gravity Design loads				IBC 2000 NJ ~ ASCE 7				
Location	Live Load	Dead Load	Total Load	Wall Type	Live Load	Dead Load	Wall Height	Total Load
<i>Roof</i>	40 psf	17 psf	57 psf	<i>Single Light Gage Wall</i>	-	11 psf	9'-6"	105 plf
<i>Unit/Balcony</i>	40 psf	45 psf	85 psf	<i>Double Light Gage Wall</i>	-	15 psf	9'-6"	143 plf
<i>Corridor</i>	100 psf	45 psf	145 psf	<i>8" CMU Wall</i>	-	60 psf	9'-6"	570 plf
<i>Storage</i>	125 psf	45 psf	170 psf					

The lateral loads were based on the design criteria of ASCE-7. Since Bloomfield, NJ is located at the center of an east coast seismic epicenter, seismic loads had a much larger affect on the lateral analysis. Similarly, Bloomfield is located near the coast line, so it also experiences greater wind speeds (basic wind speed of 110 mph). It was determined that the affects of seismic loading, while close to the loading incurred by wind, created larger forces to be resisted in the shear walls.

The design criteria for the lateral and snow loads used in the initial design of Parkview at Bloomfield Station are listed in the figure below. The full load calculations for these loads are found in Technical Assignment #3 (not included in this document).

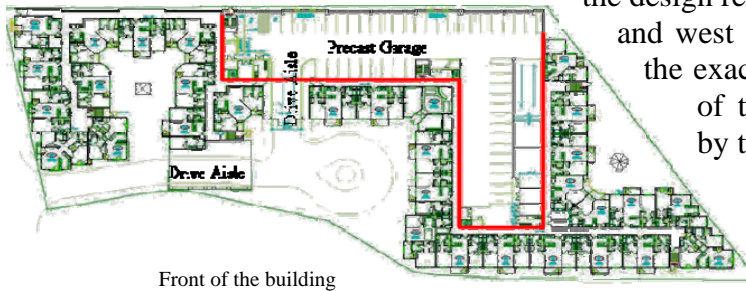
Snow and Lateral Load Overview

<p>SNOW LOADS: ASCE 7-98</p> <p>GROUND SNOW LOAD (P_g) = 30 PSF EXPOSURE FACTOR (C_e) = 0.8 (TABLE 7-2, TERRAIN D) THERMAL FACTOR (C_t) = 1.1 (TABLE 7-3) IMPORTANCE FACTOR (I) = 1.00 (TABLE 7-4) ROOF SLOPE FACTOR (C_s) = 1.0 WITH ROOF PITCH < 8:12 (FIG 7-2) FLAT ROOF SNOW LOAD ($P_f=0.7 C_e C_t I P_g$) = 21 PSF SLOPED ROOF SNOW LOAD ($P_s=C_s P_f$) = 21 PSF</p> <p>WIND LOAD: WIND LOADS ARE EVALUATED IN ACCORDANCE WITH PROVISIONS OF SECTION 1609 OF THE 2000 INTERNATIONAL BUILDING CODE BASED ON THE FOLLOWING PARAMETERS: - BASIC WIND SPEED 110 MPH (3-SECOND GUST WIND SPEED) - WIND LOAD IMPORTANCE FACTOR: 1.0 - WIND EXPOSURE CATEGORY: EXPOSURE D</p>	<p>EARTHQUAKE LOADS: EARTHQUAKE LOADS ARE EVALUATED IN ACCORDANCE WITH PROVISIONS OF SECTION 1615 OF THE 2000 INTERNATIONAL BUILDING CODE BASED ON THE FOLLOWING PARAMETERS: - MAX. EARTHQUAKE SPECTRAL RESPONSE ACCELERATION AT SHORT PERIODS, $S_s = 0.43g$ - MAX. EARTHQUAKE SPECTRAL RESPONSE ACCELERATION AT 1 SECOND, $S_1 = 0.095g$ - SITE CLASS = F - EARTHQUAKE LOAD IMPORTANCE FACTOR: 1.00 - MAX. CONSIDERED EARTHQUAKE SPECTRAL RESPONSE ACCELERATION AT SHORT PERIODS, $S_{ms} = 0.77g$ - MAX. CONSIDERED EARTHQUAKE SPECTRAL RESPONSE ACCELERATION AT 1 SECOND, $S_{M1} = 0.20g$</p>
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Building and Site Restrictions

During the original architectural design of Parkview at Bloomfield Station it was observed that the building does not have any height restrictions due to local county and township requirements. A floor to ceiling height of 9'-0" was established based on IBC 2000 code and normal practices. This coupled with the ceiling to floor height of 21" from the Hambro floor system creates a typical story height of 10'-9". The roof trusses have a maximum height of 24'- 9" and give the building a total height of 89'-3" above the ground.

The site outline for Parkview at Bloomfield Station can clearly be seen by the dark line outlining the building in the image below. Due to the irregular site shape, the building designers were forced to create an unusually shaped building to accomplish all the design requirements. The units on the east and west side of the building step back at the exact slope of the site. At the front of the building, the site is bounded by the shape of the Second River and this led to a building that provides views of and complements the shape of the river. On the back of the site, the boundary line is very straight due to the Midtown train line. The placement of the parking garage and a few units facing the north was due to the location of the train line on that side of the building.



Front of the building

the boundary line is very straight due to the Midtown train line. The placement of the parking garage and a few units facing the north was due to the location of the train line on that side of the building.

Finally, since the building complements the site fully and utilizes a large portion of it, the building is just below the impervious percentage limit for the site. This played an important role in the extension of balconies and paved areas. It also necessitated the use of the two drive aisles that were forced to pass under the building footprint to save on green space on the site.



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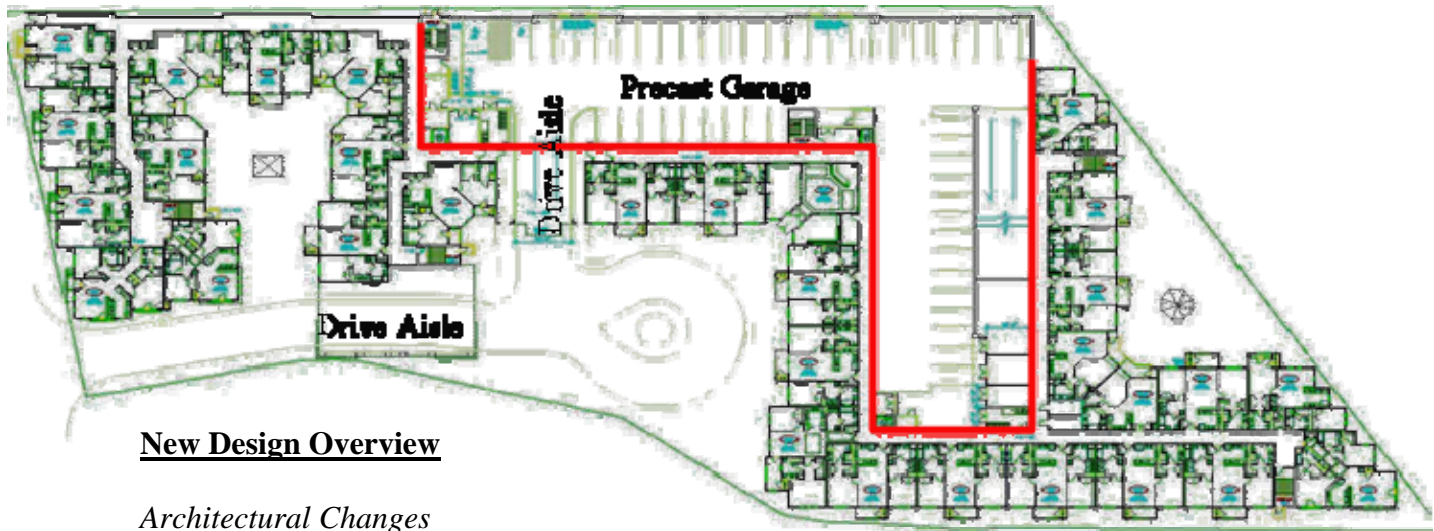
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Section II
Structural Depth Analysis

Faculty Consultant:
Professor Parfitt

Senior Thesis Final Report

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New Design Overview

Architectural Changes

The new floor plan above reflects the changes made to the architectural layout of Parkview at Bloomfield Station due to the new structural system. It is nearly identical to the original design and has only changed within the walls between units and within the floor cavity.

The first change to the architecture was due to switching from the bearing light gage wall system to a braced frame structure. This structural change imposes the need for wide flange columns to be placed within the space of the original 11" wall depth. This will generally not create a difference in wall size but in a few cases creates the need for larger wall sections at column locations. The increase in wall size ultimately deducts small amounts of square footage from the units that are affected. This change to the architecture of the units will be discussed in greater detail in the lateral system discussion in the Review of Design Criteria sub-section, later in this section.

The other change to the original architecture was that the ceiling to floor depth increased from 21" to 25". The original design had a floor to ceiling height of 9'-0" that was established based on IBC 2000 code and normal practices. This height combined with the ceiling to floor height of 21" from the Hambro floor system, produced a typical story height of 10'-9". The roof trusses have a maximum height of 24'-9" and produce a total height of 89'-3" above the ground. The new system uses the same floor to ceiling

height requirements and roof trusses, but the new building will have a total height of 91'-3", a difference of 2 feet. The typical floor to floor height is 11'-1".

In some buildings the non-modular 11'-1" floor to floor height would cause a problem with claddings such as brick, but the new design incorporates a drainable Exterior Insulation and Finish System (EIFS) system. This system can be tailored to fit any dimension and will not have a problem with the non-modular nature of the structure. The difference between the new drainable EIFS and the original designs typical EIFS cladding is discussed in greater detail in Section III: Breadth Topics.

Gravity Structural Changes

The new structure, while it still resembles the original system, is in fact quite different. The first of these differences is that the original system used Hambro joists to span the 30 foot depth of the structure. This system needed intermediate bearing locations in some larger units and in some corner units. However, the new design spans the long direction of the unit, 38 feet. This enables the joists to span any unit without the need for intermediate framing locations while using the same joist depth. This allows for partition walls that can be placed at any location and can be completely removed if the use changes in the future.

Furthermore, since the new structure is not supported by the wall systems between the units, the walls can be erected at a later time quickly, not needing the tube steel cap required for the original design. This saves on field welding and coordination between trades. The floor system can also be placed sooner and by the same crew that erects the steel frame. The wall placement is not as critical a step to the building sequencing in the new design.

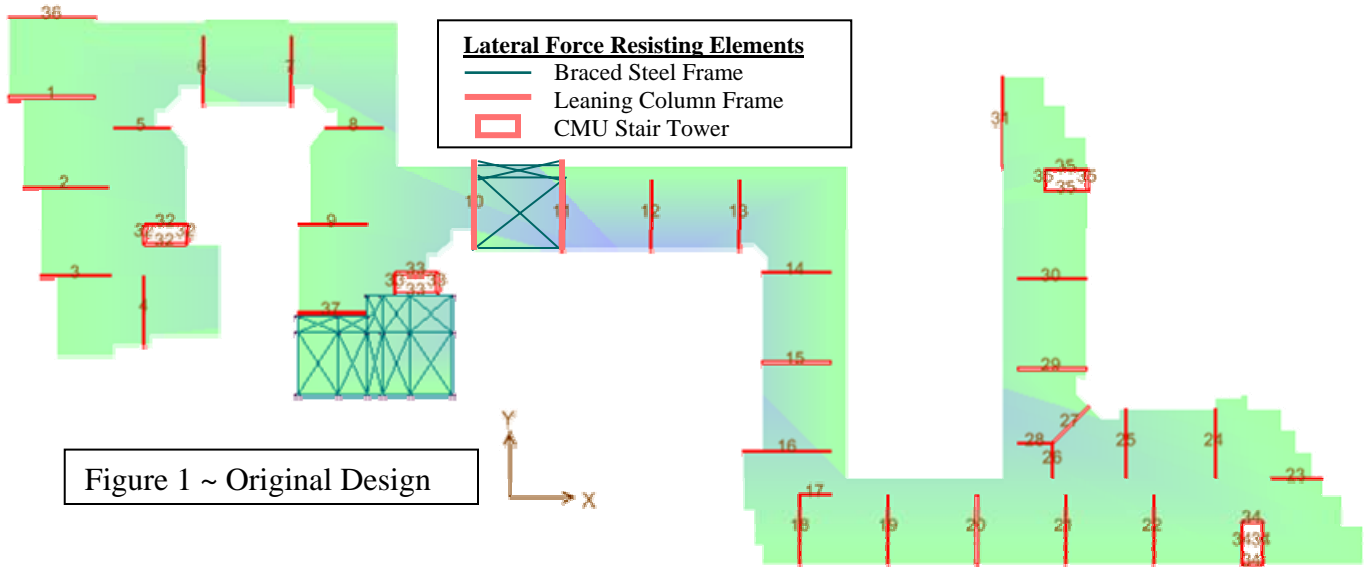
The foundation in the new design looks significantly different than that of the original design. The new design relies totally on spread footings to support the structure where as the original was predominately composed of strip footings that outlined a majority of the building. The main drawback for the new foundation system is that formwork will be required for the spread footings to ensure the proper coverage, as compared to the strip footings, used originally, that can be poured against the earth in most cases.

In addition, since both systems have advantages and disadvantages the difference in the new design and other structural issues that were affected by the new design are further compared in Section III: Breadth Topics ~ Cost Advantages.

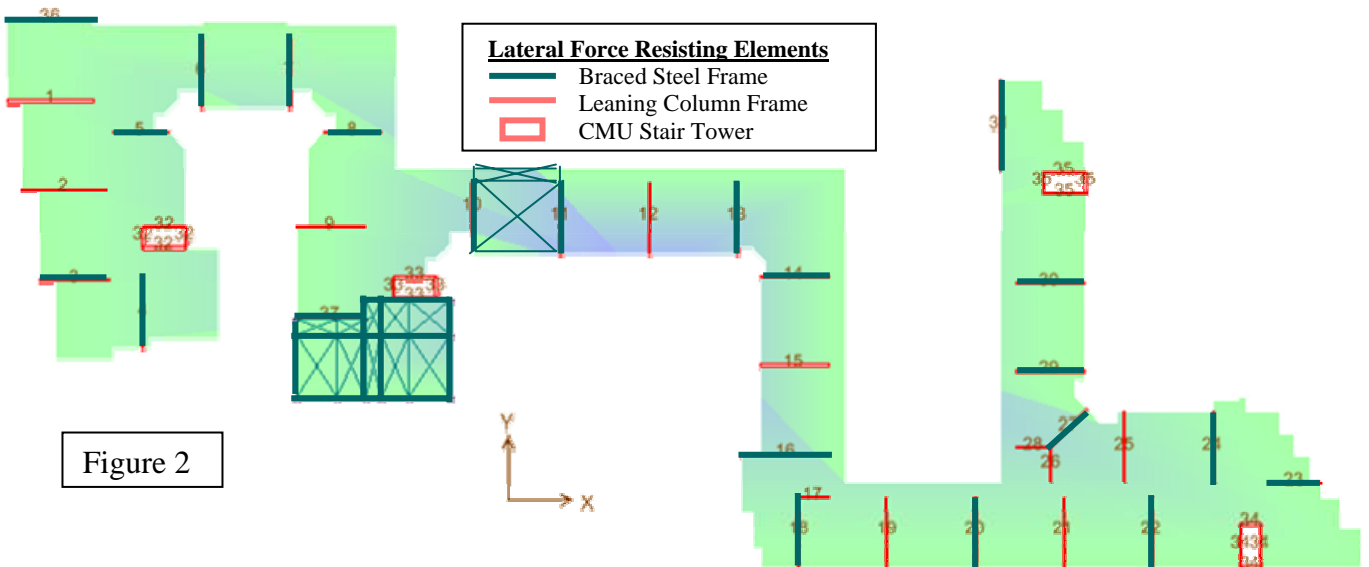
Lateral Changes

The final difference in the new design with respect to the original design is the lateral force resisting system. In this design the lateral forces, earthquake and wind forces, are resisted by braced frames made from wide flange shapes. These frames will

occupy similar locations throughout the building as the existing shear wall system. However, this new frame system will utilize less lateral resisting elements throughout the building while still maintaining the same architectural layout. The braced steel frame will allow more lateral strength capacity with less lateral force resisting members. The red lines in Figure 1 below are the original shear wall locations and the dark blue lines represent the braced frame.



The dark blue lines on Figure 2 below represent the new steel braced frame locations in the building. The other unit separation locations (the remaining red lines) are locations that were shear walls in the original design but are frames consisting of leaning columns in the new design. The four concrete masonry unit (CMU) stair towers are the same in both designs.



New Design

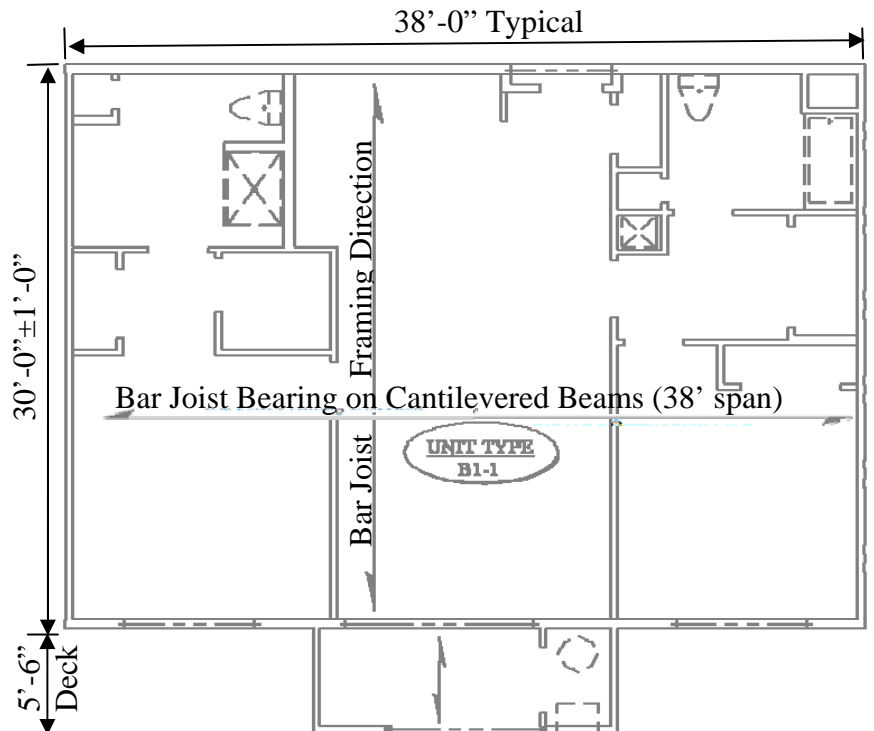
Design Criteria

As part of the new design criteria, the most recent code provisions available were used in the design. The most significant change was utilizing ASCE 7-05 code instead of the ASCE 7-02 version that was used for the original design. This provision had the largest affect since it deals with structural issues such as the lateral loading. The other code update change that was made was the use of the International Building Code (IBC) 2003 rather than IBC 2000.

The second thing used as a basis for the new design criteria was to try to maintain architectural integrity. Because of the building's initial nearness to the impervious percentage limitation of the site and to the actual site boundary lines, there should be no extensions to the overall building footprint. The overall square footage of rentable space should be kept as close as possible to the original design in order to maintain the profitability of the project. Finally, the overall appearance of the project should be maintained, but minute changes to some details are aloud.

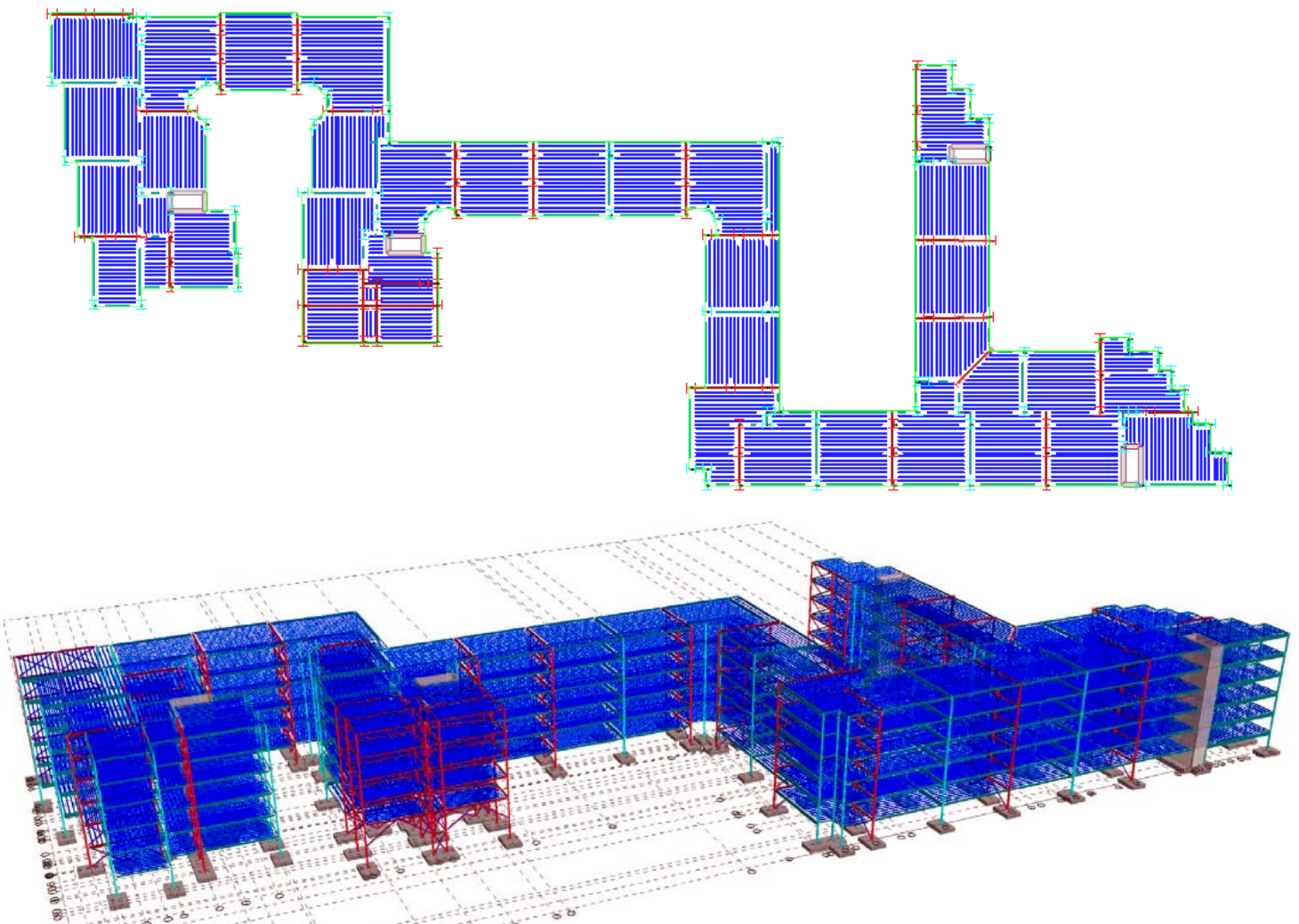
Structural Analysis

The new design for Parkview at Bloomfield Station analyzes steel braced frames that are a replacement for the original light gage bearing wall system. The two different framing orientations that were investigated are labeled below, and each was analyzed for efficiency and compatibility with the redesigned system. The required joist depth for the 30' span layout could be accomplished with a joist of just 16" deep, but the 38' span requires a 20" deep joist. Furthermore, since both systems use 2'-0" oc (on center) joist spacing the floor decking for both systems will be the same, ½" deck depth with 3" cover. This creates an overall difference of 4" in the ceiling to floor depth. The 30' joist layout works well for the typical unit, but requires intermediate bearing locations in some corner units. After considering the disadvantages of introducing extra depth into the architectural look of the



building and the interference to future architectural remodeling, it was decided that the bar joist system that spans the 38' direction fits with the building design criteria more adequately. This decision was based on this floors ability to span all of the units in the building without the need for any intermediate framing locations or any change in structural depth. This not only allows for an open plan feel in the units, but the consistency in structural depth and the ability to pass mechanical ducts through the open web joists will greatly help in the mechanical design of the building.

Furthermore, the use of the braced frame system will require less braced frames throughout the building than the original bearing wall system. The remaining original bearing wall locations, unit separation wall locations, are replaced by the creation of leaning column frames. The use of the leaning column frames allows for the removal of the non-bearing infill unit separation walls below the frame, if needed for future renovations. The ability to remove these walls to create expanded two unit long rooms is a benefit not available in the original design. There are a total of 22 braced frames, shown in red in the images below, in the new design of the building: twelve in the north-south direction, and ten in the east-west direction, along with four concrete masonry unit (CMU) stair towers that act as shear walls. Further analysis of the building was conducted using the *RAM Structural System Program, Version 10*.

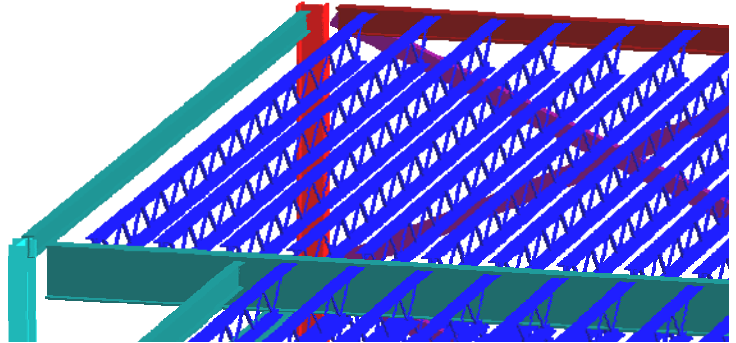


RAM Model

The bulk of the structural analysis was performed using the *RAM Structural System Program, Version 10*. Within this model the columns, beams, foundations and lateral braced frames were sized and checked for meeting code requirements.

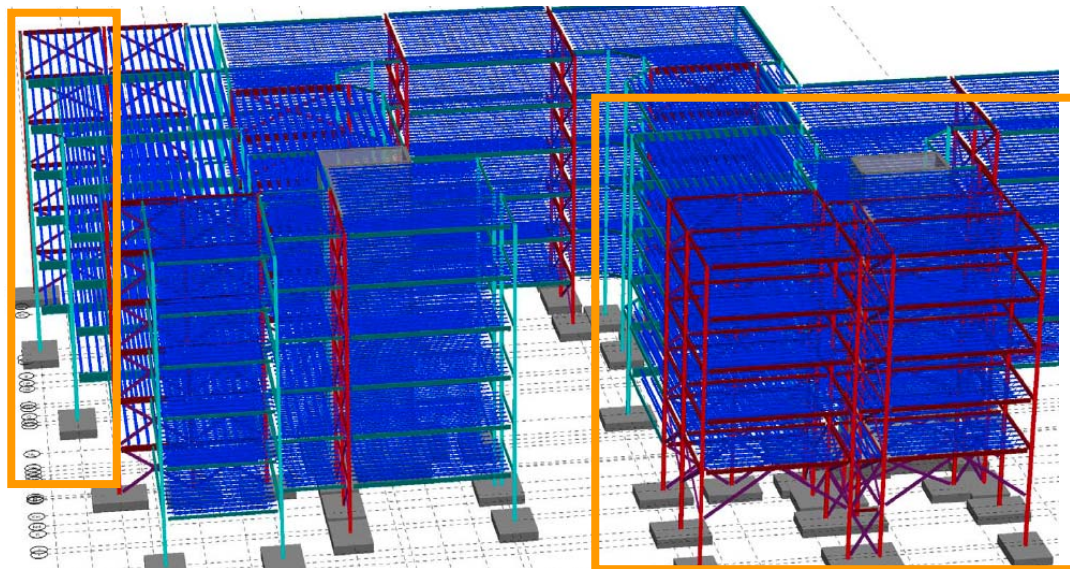
Bar Joist on Steel Girders

As explained earlier in this section, the selected bar joist system spans the 38' dimension of the unit. The decking used to support the floor loads over these bar joists at 2' oc was a 30" wide 0.6C28 CSV Conform deck with 3½" total slab depth. The structural bar joist needed for this deck design was calculated by hand to be a 20K9 at 2'-0" oc with 3 rows of bridging. At this point, the bar joist size and decking were input into a RAM model. RAM confirmed these sizes for all but a few units where a 20K10 joist was needed to span the slightly larger 40'-0" bays. The typical bay framing configuration is shown on the right with the joists seat bearing on the flange of the wide flange girders. This same attachment method is used for the joists in the lateral braced frames and the leaning column frames. For ease of construction, a smaller wide flange beam runs parallel to the joists and supports the slab edge between columns.



Steel columns

In the new design, all of the columns in the building run continuously the entire height of the building. This difference is shown mainly in the northwest unit and the drive units over the drive aisle, shown boxed below. The steel



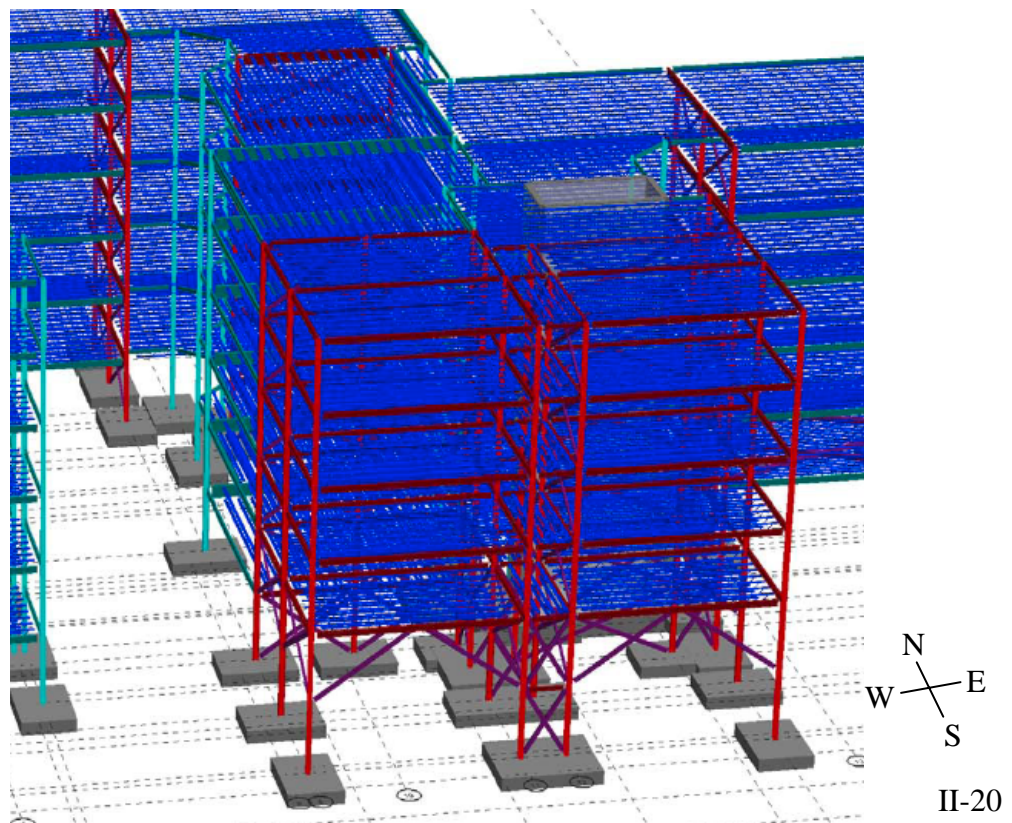
columns throughout the building are composed of wide flange sections spliced at every 3rd story. This makes the lower column, generally a larger size than the upper, have a length of 38'-3" including a four foot length to the splice above the fourth floor and the one foot extension to the foundation stem. The upper column section has a length of 29'-3" and terminates where the beams support the roof trusses.

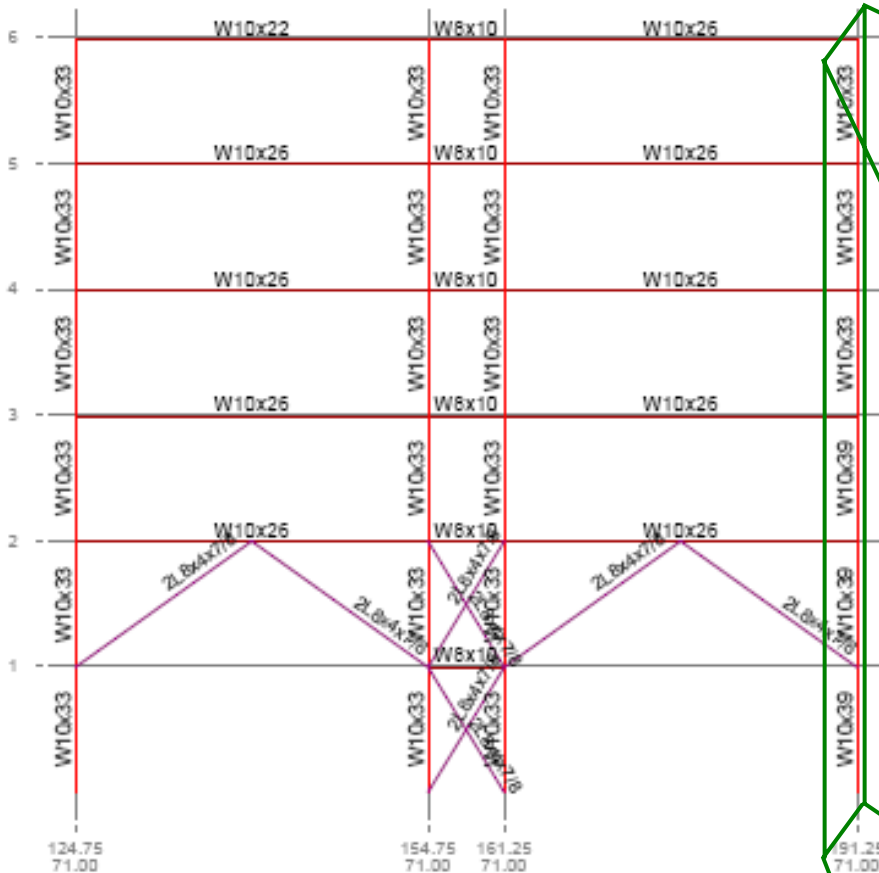
There are a total of 84 gravity columns and 73 lateral columns in the building. The gravity columns range in size from W10x33 to W12x65 with a typical column size of W10x33. The upper column sections of the gravity frames are almost entirely composed of W10x33 sections, leaving the majority of the column variation in the lower gravity column sections. Finally, the lateral columns are discussed in detail in the next sub-section.

Lateral Frames

The new braced frame system requires the use of less lateral elements than the original system. There are 29 lateral frames located in Parkview at Bloomfield Station. Eight of these lateral frames are used in the drive aisle, pictured below.

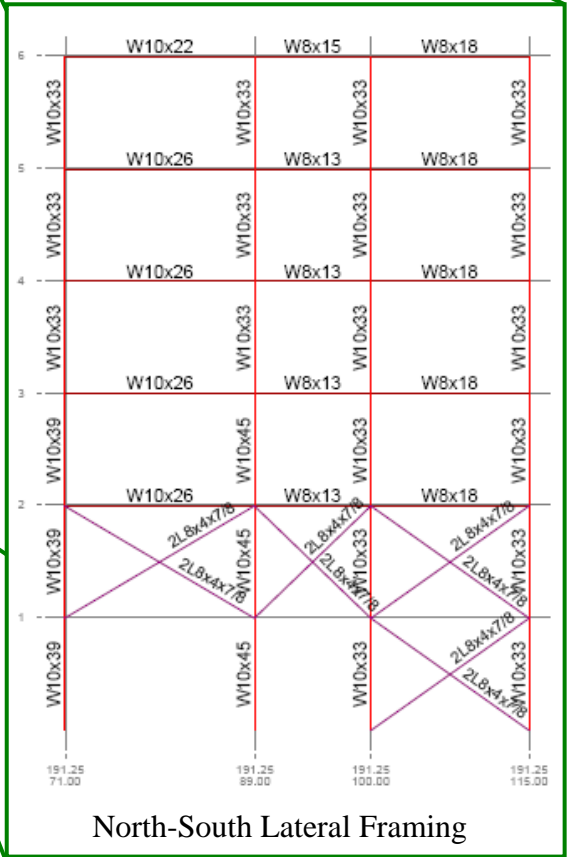
These eight frames have unique design considerations that must be adhered to and are special only to these frames in the building. The frames must be designed to allow for vehicular and pedestrian traffic below while supporting the upper stories of the building. Further limits are placed on these frames due to architectural restrictions on the bracing layouts.





East-West Lateral Framing members at the drive aisle

These frames support the main architectural detailing that is visible when approaching the building from the drive aisle. Since this is a highly visible location, there are several important architectural details in this area.



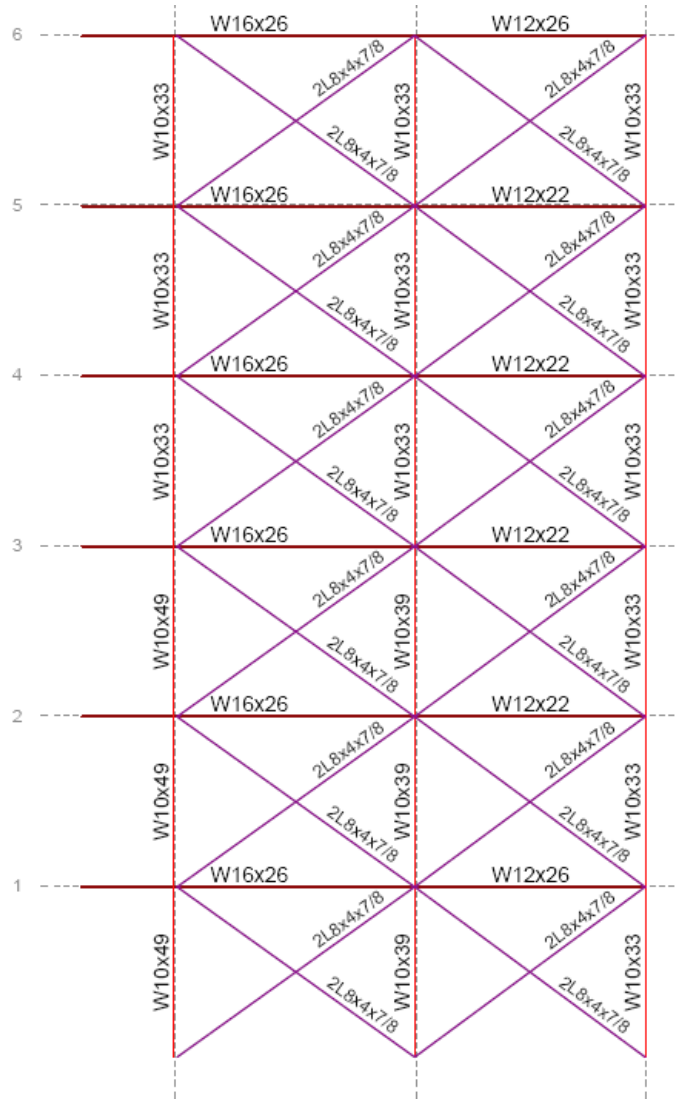
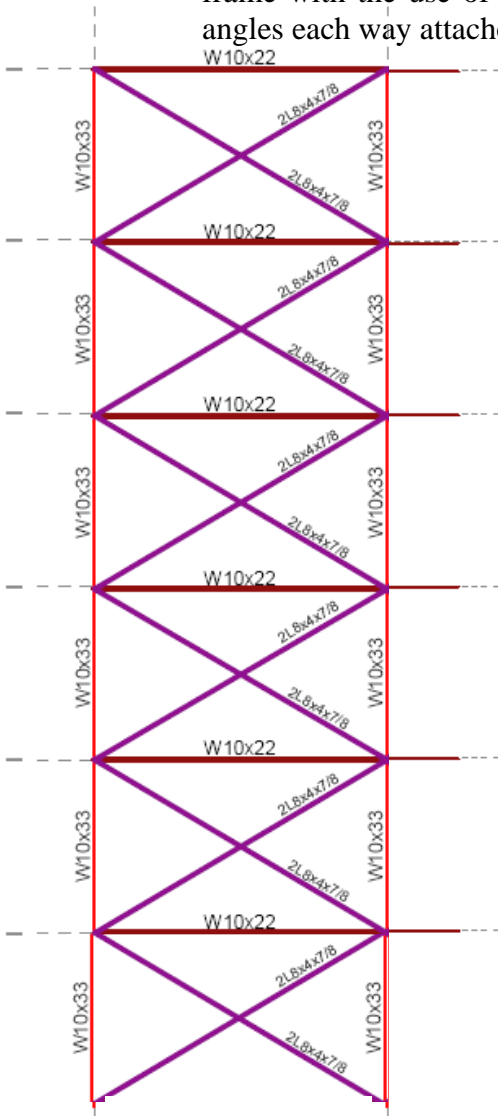
North-South Lateral Framing

The first of these details is recognized in the North-South Lateral Framing Section above. The bottom left bay of this frame is the drive aisle location and passes below the W10x22 beam. The bay to the right of that is where a pedestrian walkway passes under the building. This area is adorned with an arch, and due to the narrow width of the arch, it is unable to have any bracing members. The only cross brace in this frame is found in the next bay over and supports a decorative wall. The bays above this level are the exterior walls of the units above and contain many window openings that do not allow for any cross bracing members to be present.

The frame shown above that spans the east-west direction is the front of the building and is required to support the highlight of the façade of the building. The right and left bays on the bottom story need to support graceful arches to adorn the front of the building. The architectural detailing only allowed for the

use of chevron bracing members because they could be fully hidden behind the arch. The middle bay is allowed to use smaller cross bracing members because it supports a continuous vertical pier with elaborate detail. Furthermore, the bays on the upper floors are the exterior walls of the units above and have too many window openings to allow for bracing.

The remaining 21 braced frames in the new building design resemble the two frames on this page. The frame on the right is the most common frame found in the building and spans the 38'-0" typical unit. The cross bracing is accomplished in the frame with the use of two 8"x4"x7/8" angles each way attached back to back.



The reason for using the double angles is so the frame will fit within the wall assembly without the need to make the wall thicker. This aspect of the lateral frame system will be addressed in the Review of Design Criteria sub-section below.

In addition, four of the building frames have a single bay braced frame configuration that spans 24'-6", as shown on the left. Since these frames span less distance the need to break up the cross bracing was not needed. In the other frames, the bracing was broken into two separate bays in order to reduce the total stress in the diagonal members and prevent failure in those members. Breaking the frame into multiple bays also allowed for shallower beams to be used, rather than the large beams that would be required to span the full 38 feet. This change also benefited the vibration analysis of the floor structure, which is discussed in more detail in the Vibrations sub-section.

Most of the typical beams in the lateral frames are W12x26 and W16x26, but the beams range in size from W8x13 up to W18x40. The extra depth in the W16x26 is needed because this beam in the frame, like all the typical frames, needs to be cantilevered out six feet to support the corridor. This corridor cantilever is utilized in the gravity frames throughout the building as well.



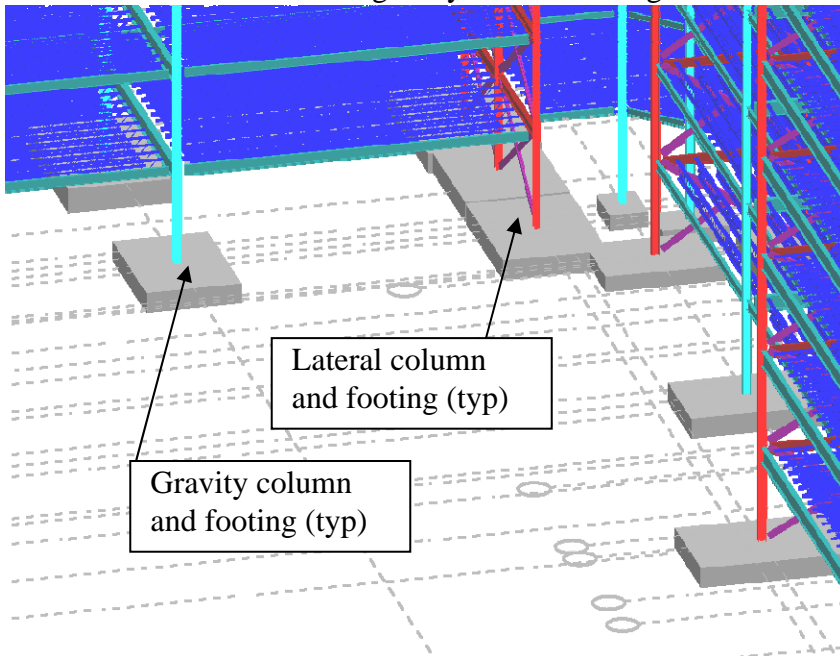
Additionally, the lateral frame column sizes do not change dramatically throughout the building. The columns in the project consist of a range from W10x33 up to W10x54. The typical lateral column size was the W10x33 which is similar to the gravity columns; however there is one lateral frame where a W14x398 was required. The location of this column is shown boxed in the image on the left. The size of this column was so large because of the building torsion that was introduced into that wing of the building and therefore a need to restrain the story deflection in this wing to less than 4" to avoid contact with the parking garage. This wing runs parallel to the parking garage and extends 172'-0" (four unit widths) while only being just over 36'-3" deep (unit and corridor). Since the section is long and narrow, it does not have

as much lateral stiffness as most of the building, and requires larger frames to support this building wing.

Spread Footings

The foundation for Parkview at Bloomfield Station is composed of nearly all spread footings. These spread footings range in size and depth significantly depending on the column type. The gravity columns have relatively square footings, while the lateral load bearing column footings blend together into a long footing that runs parallel with the frame.

The gravity loaded footings in the building range in size from 4'x4'x1½'



deep up to 14'x14'x2' deep footings. The gravity footings carry much smaller loads and generally remained isolated, as can be seen in the image on the left. The smaller footings in the project are very similar in size to the leaning (gravity loaded only) columns in the original design.

Conversely, the laterally loaded footings, as seen in the image on the left, tended to be

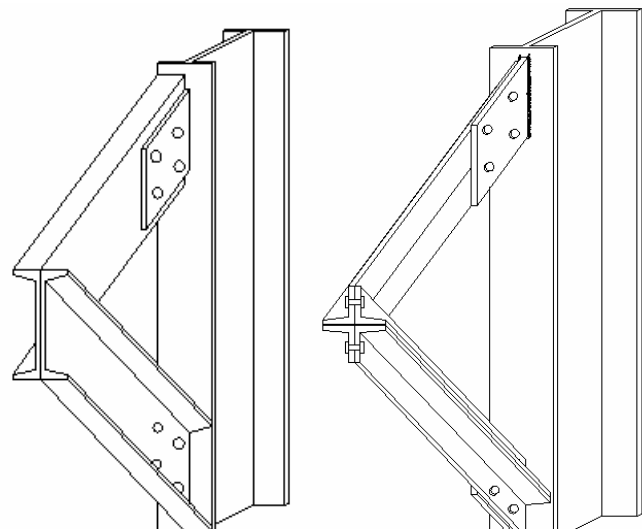
much longer in the direction of the frame in order to resist the overturning moment. Since, the length in this direction is so great, these footings are poured and designed as a long spread footing with three column point loads. Due to this fact, the combined spread footings for the lateral frames range in size from 9'x30'-6"x1'-2" deep for the single bay frames up to 14'x36'x3' deep for the double bay frames. Nevertheless, there are some lateral column foundations that remained isolated in the drive aisle locations. These spread footings range in size from 5'x5'x1½' deep up to 8'x14'x2' deep. A further look into the affect that the new spread footings have on the building's cost is analyzed in Section III: Breadth Topics ~ Cost Advantages.

Review of Design Criteria

In order to meet the new design criteria, the most recent code provisions available were used in the design of Parkview at Bloomfield Station. ASCE 7-05 code was utilized along with the use of Load and Resistance Factor Design (LRFD) in all calculations. Gravity floor loads and lateral loads were calculated from chapters 3, 4, 6, 7, and 9 of ASCE 7-05. The analysis of the building's structure was performed by using the current RAM program version (Version 10). This program performed lateral load calculations, including seismic and wind from three directions, and computed the loads for the lateral frames in the building in accordance with the ASCE 7-05 code. A building drift limit was set at 4" and members were computed using the RAM program, based on sizes from the Manual of Steel Construction ~ 3rd Edition. Other allowable changes that were made to the architecture were verified by the International Building Code (IBC) 2003 and the New Jersey provisions to this code.

The second design criterion for the new design was to try to maintain the initial architectural intent for the building. Furthermore, no extensions to the overall building footprint were allowed due to the limitations on impervious percentage and the building's proximity to the site boundary lines.

As mentioned earlier, the new lateral frame columns in some instances were larger than the 11" allowable wall cavity. Efforts were taken to keep the braced frame within this 11" limit, and two options were developed as can be seen on the right. While both bracing systems, C-channel and double angles, fit within the wall cavity, the steel angles were selected due to the lower weight of the system. Furthermore, the columns of most of the braced frames fit within the wall. Yet, a few frames require an architectural change by requiring the



Section: Braced Frame (2 Options)

columns to be boxed out beyond the wall width. This column size requirement unfortunately reduces the amount of rentable space within the building and interferes with the second design criteria. The second design requirement states that the overall square footage of rentable space should be kept as close to the original design in order to maintain the profitability of the project.

In order to solve this problem, the additional room needed for these walls, to frame out around the columns, will be taken out of the 6' corridor spaces. The existing corridors are 1'-0" greater than required by the IBC 2003 and therefore a reduction of up to 1'-0" is allowable if necessary. This design decision to shift the units down slightly should be first checked with the owner and architect to see if this change is even necessary, or if the loss of an average of 2 square feet of living space per apartment unit is an allowable loss.

Finally, the floor to floor height is taller than the original design by 4", creating a new story height of 11'-1". This height increase did not interfere with any of the local height code restrictions, but did create a building that is taller than the original by two feet. Yet, even though the overall appearance of the project was stretched vertically by an unnoticeable 4" per floor, the design was still able to maintain all other architectural detail requirements.

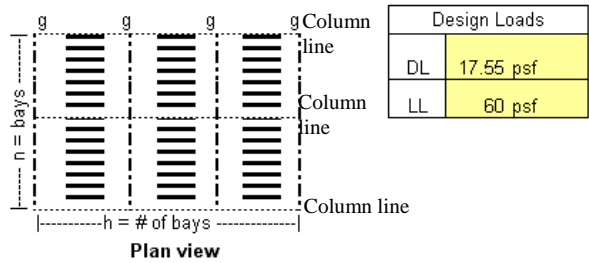
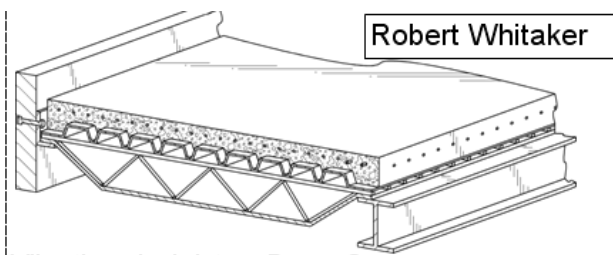
Vibration Analysis

Vibrations in building floors occur for many reasons but do not usually correspond to unsafe structures. Vibrations are caused by a floor system that has a natural frequency close that that of the load applied, due to people walking across the floor, a machine starting up or other mechanical equipment. Furthermore, a floor can also encounter vibration problems if its natural frequency is greater than 9-10 Hz and if it does not have sufficient stiffness. Since vibration is not a structural safety issue but a serviceability issue, it is normally ignored for most structures. However, adapting a floor system to adequately handle and control vibration is needed in cases where sensitive equipment is being used, such as microscopes and rooms where surgeries are conducted. Efforts to Control vibrations are also deemed necessary in some offices, residences, churches and other such areas where people will feel any excessive vibrations. These areas are analyzed on a case by case basis as decided by the owner or engineer. Guidelines for these areas are listed on Table 4.1 from AISC Steel Design Guide 11, shown here.

Table 4.1 Recommended Values of Parameters in Equation (4.1) and a_o/g Limits			
	Constant Force P_o	Damping Ratio β	Acceleration Limit $a_o/g \times 100\%$
Offices, Residences, Churches	0.29 kN (65 lb)	0.02-0.05*	0.5%
Shopping Malls	0.29 kN (65 lb)	0.02	1.5%
Footbridges—Indoor	0.41 kN (92 lb)	0.01	1.5%
Footbridges—Outdoor	0.41 kN (92 lb)	0.01	5.0%
* 0.02 for floors with few non-structural components (ceilings, ducts, partitions, etc.) as can occur in open work areas and churches, 0.03 for floors with non-structural components and furnishings, but with only small demountable partitions, typical of many modular office areas, 0.05 for full height partitions between floors.			

Due to the light weight nature of the new bar joist floor system, vibrations are a concern for the structure and were analyzed to determine vibration severity. In order to analyze the impact of vibrations on the structure, the design procedure set out in AISC Steel Design Guide 11 was utilized.

Using the typical 30 foot by 38 foot bay and girder sizes established from the RAM model (W16x31, W18x35, and W21x62), an excel spread sheet was created to determined the adequacy of the floor system. The vibration values were then compared with the values found on Table 4.1 from the design guide. This table sets a floor acceleration limit of 0.5%g ((a_o/g)*100%) for residential buildings with non-structural components and furnishings (an open floor plan with removable partitions). The floor damping ratio used in the calculations was 0.03 and a constant load of 65 pounds. The input for the typical bay can be seen below.



Vibrations in Joist on Beam System

Based on AISC Steel design guide 11 ex 4.6 & 6.2

20K9		W16x31	
W _{self} (plf)	10.8	W _{self} (plf)	31.0
W _{total allow} (plf)	279.0	A (in ²)	9.13
W _{joist design} (plf)	233.1 ok	d (in)	15.70
d (in)	20.0	I _x (in ⁴)	375.0
M _{allow} (ft-k)	49.48	f _{allow} (k)	30 kip
A _{bottom} (in ²)	1.04	ρ _{conc} (pcf)	145 pcf
A _{top} (in ²)	1.30	E _s (ksi)	29000 ksi
A _{cord} (in ²)	2.34	f _c (ksi)	3 ksi
I _{cord} (in ⁴)	208.9	E _c	3024 ksi
I _{comp} (in ⁴)	466.0	n	7.10
y _c (in)	8.94		
t _{conc}	3.00 in	*update Ws+d value	
t _{deck}	0.50 in		
t _{tot}	3.50 in	t _{eff}	3.25 in

building	
n (# bays)	2 bays
h (# bays)	3 bays

LOADS		
Ws+d	39 psf	<== look up value in deck manual
DL	4.0 psf	<== 4 psf typ office service load
LL	11.0 psf	<== 11 psf typ office service load

Length	
Girder (L _g)	15 feet
Joist (L _j)	38 feet
Joist Spaci	2 feet

joist		girder			
L min =	24	182.4	L min =	72	456
L _{eff} =>	24 in		L _{eff} =>	72 in	

Based on these inputs into the program, it was shown that the natural frequency of the floor (f_n = 4.3 Hz) was well below the 9-10 Hz limit, so there is no problem with the floor stiffness, but the walking evaluation of the floor failed by nearly 0.6%. Attempts were made to increase the joist size and slab thickness in order to decrease the total joist deflection. These attempts showed that a slab and deck thickness of eight inches and a

Stiffness analysis		(fn ok, no need to check stiffness analysis)	
using a	0.224 kip load		
Δ _j applied	0.04282 in		
Δ _j pannel	0.00714 in		
Δ _g Pannel	0.00145 in	(fn= 4.30 Hz)	
Δ _{total}	0.00786 in	fn ok	
K _{floor}	28.5 kip/in	>5.7kip/in limit ok	

Walking Evaluation		(fn= 4.30 Hz)	
W _{PANELtot}	43.9 kips		
β	0.030	Res._mid low damp	table 4.1
β _W	1316.4 #		
P _o	65.0 #	table 4.1	compare with table 4.1
a _p /g =	0.01098	=	1.098% g fails > 0.5% fails
Fails, need to increase joist size or slab thickness (delta j controls)			

joist of 28 inches deep was the smallest design that would pass the 0.5% acceleration limit for the 38 foot span. With the eight inch slab and 28 inch joist, the floor to floor height would jump from 11'-1" to 12'-0" and the weight of the building would increase greatly.

28k12		W16x31		building	
W _{self} (plf)	17.1	W _{self} (plf)	31.0	n (# bays)	2 bays
W _{total allow} (plf)	461.0	A (in ²)	9.13	h (# bays)	3 bays
W _{joist design} (plf)	319.1 ok	d (in)	15.70	LOADS	
d (in)	28.0	lx (in ⁴)	375.0	Ws+d	82 psf <== look up value in deck manual
M _{allow} (ft-k)	81.76	f _{allow} (k)	30 kip	DL	4.0 psf <== 4 psf typ office service load
A _{bottom} (in ²)	1.21	ρ _{conc} (pcf)	145 pcf	LL	11.0 psf <== 11 psf typ office service load
A _{top} (in ²)	1.51	Es (ksi)	29000 ksi	Length	
A _{board} (in ²)	2.73	fc (ksi)	3 ksi	Girder (Lg)	15 feet
I _{board} (in ⁴)	490.5	Ec 3024 ksi		Joist (Lj)	38 feet
I _{comp} (in ⁴)	1570.9	n 7.10		Joist Spaci	2 feet
γ _c (in)	12.50			joist	
t _{conc}	5.00 in	*update Ws+d value		L min =	24 182.4
t _{deck}	3.00 in			Leff =>	24 in
t _{tot}	8.00 in	teff = 6.50 in		girder	
Stiffness analysis (fn < 9 Hz, no need to check stiffness analysis)			Walking Evaluation (fn = 4.82 Hz)		
using a	0.224 kip load			W _{PANELtot}	80.4 kips
Δ _{j applied}	0.01626 in			β	0.030 Res_mid low damp table 4.1
Δ _{j pannel}	0.00208 in			βW	2410.9 #
Δ _{gPannel}	0.00104 in	(fn = 4.82 Hz)		P _o	65.0 # table 4.1 compare with table 4.1
Δ _{total}	0.00260 in	fn ok		a _{p/g} =	0.00500 = 0.500% g < 0.5% ok
K _{floor}	86.1 kip/in	>5.7kip/in limit ok			

Because of the vibration problem with the 38' joist configuration, a check was conducted for vibrations if the joists only needed to span 30 feet. The joist length was reduced to 30 feet and the slab and deck were reset to their original 3 1/2" design depth. This time the joists had a higher natural frequency, yet still within the limits, but the walking evaluation for the system adequately passed. When the joists needed to span a shorter length, the deflection in the members reduced and created less vibrations.

Girder		Transformed Girder properties based on unit width	
γ _g	5.24 in	D _j	246.80 in ⁴ /ft
I _{gcomp}	2466 in ⁴	D _g	29.92 in ⁴ /ft
I _{gnon-comp}	375 in ⁴	Transformed moment of inertia per unit of width in x direction	
I _{gred}	898 in ⁴	Joist connected to girder web?	flr length B _g calc
W _g	3043 plf	C _g	1.6 no ==> B _j = 60 ft or 40.67 ft
Δ _g	0.133 in	B _g	40.67 ft <2/3 * floor length
f _g	9.69 hz	W _g	61.9 kips
		Δ _{g'}	0.100 in L _g < B _j
Stiffness analysis (fn ok, no need to check stiffness analysis)		Walking Evaluation (fn = 5.67 Hz)	
using a	0.224 kip load	W _{PANELtot}	60.8 kips
Δ _{j applied}	0.01521 in	β	0.030 Res_mid low damp table 4.1
Δ _{j pannel}	0.00196 in	βW	1824.1 #
Δ _{gPannel}	0.00105 in	P _o	65.0 # table 4.1 compare with table 4.1
Δ _{total}	0.00248 in	a _{p/g} =	0.00490 = 0.490% g < 0.5% ok
K _{floor}	90.4 kip/in	>5.7kip/in limit ok	

Since the walking evaluation for this system passed the acceleration limit, it was then examined against the vibration criteria for sensitivity equipment. This is the second

check that can be done for vibrations in a system. The criteria for this check are found in Table 6.1 from the design guide and are shown on this page. The floor vibration velocity is based on the speed of impacts across the floor. A fast walk is when a 185 pound test subject takes 100 steps per minute. A slow walk is when a 185 pound test subject takes 50 steps per minute. The moderate walk is in the middle of these two and is based on 75 steps per minute.

For slow walk, the 30' system has an acceptable vibration velocity for laboratory robots, computer systems, operating rooms, and microscopes up to 100x magnification.

MODERATE WALK		SLOW WALK	
W person	185 #	W person	185 #
step/min	75 step/min	step/min	50 step/min
Fm/W	1.5 (table 6.2)	Fm/W	1.3 (table 6.2)
Fm	277.5 #	Fm	240.5 #
$U_v =$	5500 # Hz ²	$U_v =$	1500 # Hz ²
f_0	2.5 hz (figure 6.5)	f_0	1.4 hz (figure 6.5)
f_n/f_0	2.269 >>0.5 use eq 6.4b	f_n/f_0	4.05 >>0.5 use eq 6.4b
$T_0=1/f_0$	0.4 sec	$T_0=1/f_0$	0.7143 sec
f_n*T_0	2.269 > 0.5	f_n*T_0	4.05 > 0.5
A_m	0.097	A_m	0.030
X max	267 in x 10 ⁻⁶	X max	73 in x 10 ⁻⁶
V	9,598 x 10⁻⁶ in /sec	V	2,618 x 10⁻⁶ in /sec
	compare with table 6.1 values		compare with table 6.1 values

While the value is close to the acceptable mid-span velocity, none of the activities are allowable at the moderate walking level. With these results as a base mark, the 38' design can have added clarity by comparing these two systems. The slow walk vibration velocity for the 38' design was calculated to be 11,690 x 10⁻⁶ in/sec, which is close to the moderate walk level for the 30' system.

Facility Equipment or Use	Vibrational Velocity*	
	(μ in./sec)	(μm/sec)
Computer systems; Operating Rooms**; Surgery; Bench microscopes at up to 100x magnification;	8,000	200
Laboratory robots	4,000	100

* Value of V for Figure 6.1.
** Criterion given by solid curve of Figure 6.1 corresponds to a standard mean whole-body threshold of perception (Guide 1974).

With this new comparison, the 38' design could be justified as being acceptable even though it is outside the given acceleration limit (resonance response). Yet, converting the acceleration limit (a/g=.005) to a mid-span velocity shows that the maximum acceptable mid-span velocity is 71,545 micro-inches per second for the floor. This value is much lower than the calculated value of 157000 micro-inches per second from the a/g=.011 calculated. These values were determined using the equation $V=a/(g*\omega)$ where $\omega = 2*\pi*fn$. With this conversion showing that the resonance response controls the vibration of the floor system and that there are slight discrepancies in the code analysis between the transient response and the resonance response analysis for floors with a natural frequency around 4Hz. Alternative floor spans and stiffness may need to be used to correct this floor system. The full results of the vibration calculations are in the appendix of this document.

A judgment to accept the floor system as designed could be based on the fact that the vibration issue is a service issue, and therefore based on the occupant's opinion. The designed floor system is adequate structurally, and does not need to be converted to smaller bay sizes based on load. However, the final say for this decision is the owners.



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Section III
Breadth Topics

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Senior Thesis Final Report

Breadth Topics

By Robert Whitaker



Breadth Topics

This section investigates two breadth topics that effect large portions of Parkview at Bloomfield Station. First, the effect of changing from a Hambro[®] on bearing wall system to the new bar joist on steel frame system has on the cost and schedule is analyzed. Secondly, an analysis of the current Exterior Insulation and Finish System (EIFS) is studied.

Cost Advantages

The bar joist structure bearing on a wide flange structure shows a \$493,400 savings over the original Hambro on light gage bearing wall structure. In addition to the cost savings, there is also increased flexibility in the building sequencing such as allowing secondary crews to have access to an area quicker, not waiting on stripping formwork for reuse, and the fact that the structural progress is not dependant on the assembly of detailed light gage shear walls and tube steel. Therefore, the new design is the better option based on the assemblies' level of detail, but this benefit must be weighed against the structural benefits of the original Hambro system.

Cladding Analysis

An analysis of the original Exterior Insulation and Finish System (EIFS) façade is checked for weather resistance and wall performance based on its uses in other buildings. After checking previous uses it is my recommendation that drainable EIFS is used at Parkview at Bloomfield Station rather than the original design for conventional EIFS. While the drainable EIFS cladding system is more expensive than the original system, it will ultimately pay dividends to the building owner in the long run. Both systems will fully work if attention to detail and installation is adhered to; yet, the drainable EIFS system allows a factor of safety for any minor mistakes that can be made during installation in the field or during design in the office through the inclusion of a water resistant membrane and drainage holes. Yet, in order for the drainable EIFS system to be effective for Parkview at Bloomfield Station, qualified laborers must be located and supervised to ensure a good product.

Cost Advantages

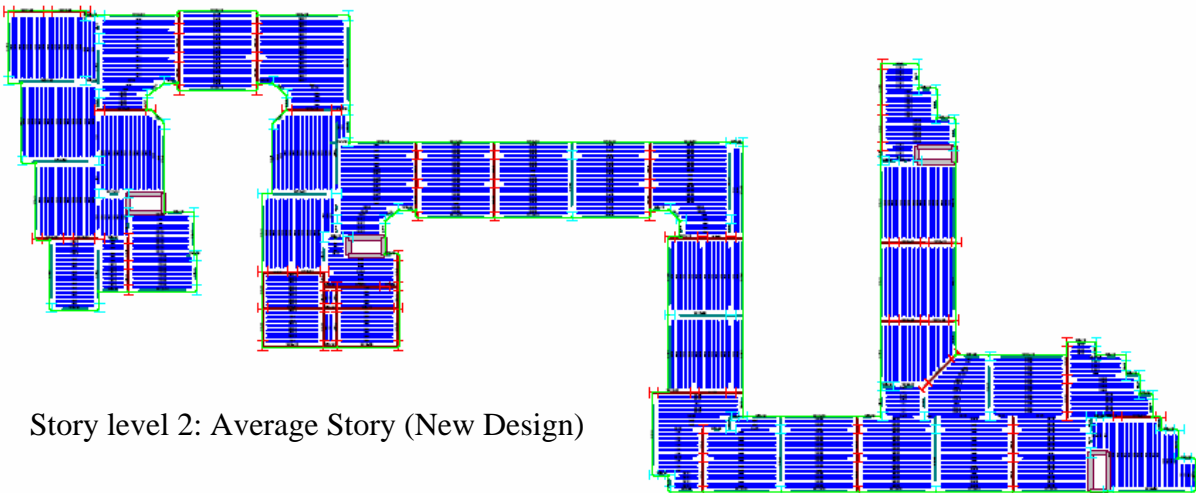
Breadth Topic



Changing Structural Systems

The new structural system for Parkview at Bloomfield Station, while similar to the original system, incorporates regular k-series bar joists and metal decking rather than the specialized Hambro system that had no permanent decking requirements. The new system also relies on a wide flange structure and columns supported by larger spread footings. This is in stark contrast with the original system that utilized light gage bearing walls capped with steel tubes and supported by continuous footings. With these changes made to the structural systems, there needs to be a change made to the overall cost of the building. The new structural system will inevitably have a different effect on cost than the original system.

The building costs were calculated using RS Means Assembly Cost Data 31st ED 2006 for both the new and the original system, in order to establish a good comparison. The cost data was further simplified by using story level 2, shown below, and obtaining the total building cost by multiplying that story's values by six stories. The only exception to this procedure was for the foundation systems that were based on the lowest floor; however, this floor aligned primarily with the 5 floors above and did not create any major difference.



Footings

The new foundation consists of primarily spread footings to support the point loads from the steel columns. There are 157 wide flange columns (84 gravity & 73 Lateral frame columns) incorporated in the new design. The break down of footing capacity is illustrated in the spreadsheet output below and corresponds with the column output to be discussed later. The new design also includes a small amount of strip footings that support the four CMU stair towers throughout the building. These footings account for 232 feet of strip footing and show up in both the new and original designs.

In addition to the four CMU stair towers incorporated in both designs, the original design relied heavily on strip footings for support. Since the load was transferred over a longer area (the length of the bearing walls) the overall capacity needed for the wall was low in most areas. Furthermore, ninety-five point loads from small tube steel columns were even incorporated into the normal strip footings load, as shown shaded below. The strip footings under the shear walls were more sizeable due to not only resisting gravity loads but also lateral loads.

While the main foundation for the original design relied on strip footings, there was a fair number of spread footings used to pick up columns located away from the bearing walls. The majority of these were located at the 1 and 2 story drive aisles to support the upper floors. These wide flange columns were much larger than the typical tube steel columns used elsewhere in the project and account for the largest 30 spread footing sizes listed below.

NEW CONDITIONS						ORIGINAL CONDITIONS					
*All values based on RS Means Assembly Cost Data 31st ED 2006						*All values based on RS Means Assembly Cost Data 31st ED 2006					
NEW FOUNDATION						ORIGINAL FOUNDATION					
<i>Spread footing (3 ksf soil)</i>						<i>Spread footing (3 ksf soil)</i>					
capacity	costs (per spread footing)			spread quantity	total	capacity	costs (per spread footing)			spread quantity	total
	material	installation	total				material	installation	total		
700k	4075	3100	7175	12	\$86,100	700k	4075	3100	7175	18	\$129,150
500k	2575	2063	4638	31	\$143,778	500k	2575	2063	4638	8	\$37,104
300k	1075	1025	2100	37	\$77,700	300k	1075	1025	2100	4	\$8,400
200k	585	625	1210	33	\$39,930	200k	585	625	1210	1	\$1,210
100k	214	282	496	35	\$17,360	100k	214	282	496	12	\$5,952
50k	107	166	273	9	\$2,457	50k	107	166	273	0	\$0 on strip ftg
Total per floor					\$367,325	Total per floor					\$181,816
Total for the 6 story building					\$367,325	Total for the 6 story building					\$181,816
NEW FOUNDATION						ORIGINAL FOUNDATION					
<i>Strip footing (3 ksf soil)</i>						<i>Strip footing (3 ksf soil) total length 2480 feet</i>					
capacity	costs (per foot of footing)			strip length (feet)	total	capacity	costs (per foot of footing)			strip length (feet)	total
	material	installation	total				material	installation	total		
20klf	70.00	53.50	123.50	0	\$0	20klf	70.00	53.50	123.50	310	\$38,291
15klf	45.00	40.00	85.00	0	\$0	15klf	45.00	40.00	85.00	868	\$73,792
10klf	20.00	24.00	44.00	0	\$0	10klf	20.00	24.00	44.00	95	\$4,180
5.1klf	12.15	18.80	30.95	232	\$7,180	5.1klf	12.15	18.80	30.95	232	\$7,180
2.6klf	5.90	11.30	17.20	0	\$0	2.6klf	5.90	11.30	17.20	1302	\$22,398
Total per floor					\$7,180	Total per floor					\$145,841
Total for the 6 story building					\$7,180	Total for the 6 story building					\$145,841

The foundation systems both support the loads from the building, but the new foundation has slight cost and time ramifications. This is due to the fact that spread footings have a higher installation cost due to the need for formwork and deeper holes, causing them to be more expensive in most cases. Furthermore, the foundation for the new structural system has a higher cost by nearly \$47,000 due to the larger number of spread footings required and larger quantities of concrete needed. However, since both structural systems use this type of footing the delay in the new system's schedule is not a major concern.

Columns

As mentioned in the previous section, there are many more columns in the new structural system than in the original system. This lead to increased costs in columns but less cost incurred in the wall systems, as will be seen later. The columns in the table below correspond with the spread and strip footings just discussed. While the large columns in the original design were located at the critical drive aisle areas, the large columns in the new design correspond to columns located in the braced frames. These braced frames replaced the shear walls and counteract the lateral forces using fewer frames. This lead to larger lateral loads being placed on frames and required higher capacities. In the new system there are the same large columns around the drive aisles, but they encounter less loading due to an increased number of columns in that area.

NEW CONDITIONS							ORIGINAL CONDITIONS						
*All values based on RS Means Assembly Cost Data 31st ED 2006							*All values based on RS Means Assembly Cost Data 31st ED 2006						
NEW COLUMN LAYOUT (unsupported 10 feet)							ORIGINAL COLUMN LAYOUT (unsupported 10 feet)						
Wide flange columns (84 gravity & 73 Lateral frame columns)							Tube steel columns (walls act as lateral frame)						
Type A capacity	costs (per column)			weight (plf)	column quantity	total	Type F,D capacity	costs (per column)			weight (plf)	column quantity	total
	material	installation	total					material	installation	total			
700k	173.00	8.90	181.90	136	12	\$2,183	700k	131.00	8.90	139.90	103.3	12	\$1,679
500k	125.00	8.90	133.90	136	31	\$4,151	500k	96.00	8.90	104.90	103.3	14	\$1,469
300k	77.50	8.90	86.40	61	37	\$3,197	300k	61.00	8.90	69.90	47.9	4	\$280
200k	57.00	8.90	65.90	45	33	\$2,175	200k	48.00	8.90	56.90	37.7	1	\$67
100k	44.50	6.65	51.15	40	35	\$1,790	100k	24.00	8.90	32.90	19.02	19	\$625
50k	20.50	8.90	29.40	16	9	\$265	50k	15.50	8.90	24.40	12.21	91	\$2,220
Total per floor						\$13,760	Total per floor						\$6,329
Total for 6 story building						\$82,560	Total for 6 story building						\$37,976

The columns in the new structural system, while greater in number and nearly \$45,000 more expensive, will be able to be erected quickly. This system will be quicker than the tube steel capped bearing walls to assemble due to the ability to be fabricated in shop controlled settings and the ability to know where every piece goes in the project. This braced frame system will require slightly more coordination, but since Parkview at Bloomfield Station is a large project, this will also allow for other trades to begin work on portions of the building that have been pieced together. This time advantage will be further explored in the next section.

Floor System

The floor systems are very similar in appearance but quite different in application. The original floor system, Hambro Joists, is a composite joist system that does not employ decking but utilizes reusable plywood forms that lock into place between the joists. Since the joists are composite and can be spaced at further distances (4'-1 3/8"), they work well for this system and are spaced at the exact dimensions needed for the plywood forms. This means that no time is wasted in cutting the forms to fit. However, in order to achieve the composite action welded wire mesh must be draped over the S extension, creating an extra task in the installation of the floor system. While the overall approach saves on materials, as seen below, it involves more installation cost and time due to having to return to sections after they are dry and strip the forms to be reused in a different area.

The new system has the time and cost advantage in the floor system even though it uses more materials. Since the bar joists require less labor to be put in place (most work done by cranes lifting materials into place) the system has a lower installation cost. Since the joists are placed at 2 foot on center, the placement of end seats is much quicker than for that of the unusual 4'-1 3/8" spacing needed for the Hambro system. The increased speed is also due to the fact that the decking is not reused but rather left in place, allowing crews to place a bay and then move on while the concrete is being poured. This will also allow secondary crews to begin their work uninterrupted below and around these areas sooner.

NEW CONDITIONS						ORIGINAL CONDITIONS					
*All values based on RS Means Assembly Cost Data 31st ED 2006						*All values based on RS Means Assembly Cost Data 31st ED 2006					
Steel joists on beams (30x35 feet ² bays)						Hambro joists on bearing walls (30x35 feet ² bays)					
capacity	costs (per column)			ft ² quantity	total	capacity	costs (per column)			ft ² quantity	total
	material	installation	total				material	installation	total		
148 psf	12.40	5.25	17.65	50120	\$884,618	173 psf	11.00	8.50	19.50	50120	\$977,340
Total per floor					\$884,618	Total per floor					\$977,340
Total for the 6 story building					\$5,307,708	Total for the 6 story building					\$5,864,040

The flooring is the area where most of the cost difference between the two systems takes place. This is due primarily to the large square footage encompassed in the costs. There is almost a \$556,000 difference between these two floor systems; however, a difference of just under \$2 per square foot would place the Hambro floor system even with the steel joist system. This just reinforces that the main difference between is the time involved in installation of system.

The Hambro system uses less material but costs nearly double to install, creating the cost and time difference with the new system. The cost and time savings for the new structural system in this area makes up for the delays in placing the columns and footings. This cost difference is nearly 10 times that of the cost differences seen to this point, making it nearly impossible for the original system to end up with a lower price tag. However, it is important to note that while the system may be overall more expensive it does carry along with it special benefits such as making the building shorter, lighter, and with composite floors.

Wall System

The last system that changed with the new design was the wall systems. The original design had tube steel continuously capping the light gage bearing walls for load distribution purposes. This detail required field welding in order to attach the tube steel, and required careful attention to detail at corner locations. If the walls could be shop fabricated, it would require a crane or a large workforce to be able to tilt these walls into place and attach to the floor.

In the new structural system, the light gage walls are non-bearing and only need to be continuously capped by a light gage cap. This makes the wall system lighter and easier to field fabricate; furthermore, it since the structure is not dependant on its erection, it can be constructed at any point in the building sequence. These walls can be built and tilted into place with far fewer workers and much less time. This can be seen below in the cost difference of nearly \$171,000 between the new system and original system.

NEW CONDITIONS						ORIGINAL CONDITIONS							
*All values based on RS Means Assembly Cost Data 31st ED 2006						*All values based on RS Means Assembly Cost Data 31st ED 2006							
non-bearing light gage studs (24 inches on center with light gage cap)						Bearing light gage studs (12 inches on center with tube steel cap)							
item	costs (per foot of wall length)			length (feet)	total	item	costs (per foot of wall length)			length (feet)	total		
	material	installation	total				material	installation	total				
wall	2.65	6.2	8.85	4931	\$43,638	wall	3.10	6.73	9.83	4931	\$48,470		
cap	0.27	0.62	0.89	4931	\$4,388	cap	4.80	0.89	5.69	4931	\$28,056		
Total per floor					\$48,026	Total per floor					\$76,526		
Total for the 6 story building					\$288,156	Total for the 6 story building					\$459,156		
totals						\$6,052,930	totals						\$6,688,830
Savings of \$635,900													

Conclusion

The bar joist structure bearing on a wide flange structure shows a \$635,900 savings over the original Hambro on light gage bearing wall structure. In addition to the cost savings, there is also increased flexibility in the building sequencing such as allowing secondary crews to have access to an area quicker, not waiting on stripping formwork for reuse, and the fact that the structural progress is not dependant on the assembly of detailed light gage shear walls and tube steel. More exact savings calculations could be done in the future on a per item basis to determine exact time and cost benefits, but the results should be similar to those found in this report. In conclusion, the new design is the better construction management option based on the assemblies' level of analysis, but this benefit must be weighed against the structural benefits of the original system.

Cladding Analysis

Breadth Topic



EIFS Background

The exterior of Parkview at Bloomfield Station consists of a large number of windows and the use of Exterior Insulation and Finish Systems (EIFS). EIFS is a wall cladding system that acts as a barrier wall rather than a cavity wall. The purpose of this barrier system is to stop water from penetrating the surface outermost layer, therefore keeping all of the backing materials dry¹. This system is desirable because of its ability to keep the entire structure insulated without the need to infill between studs. This also prevents any of the usual thermal breaks due to the framing members.

Developed in Europe in the 1950s, EIFS were introduced in the U.S. in the 1970s. They were initially used on commercial buildings and later introduced into residential markets. Today, EIFS account for nearly 30% of the U.S. commercial exterior wall market. EIFS typically consist of the following components:

- Insulation board, made of polystyrene or polyisocyanurate foam, which is secured to the exterior wall surface with a specially formulated adhesive and/or mechanical attachment
- A durable, water-resistant base coat, which is applied on top of the insulation and reinforced with fiber glass mesh for added strength
- A durable finish coat (lamina) - typically utilizing acrylic copolymer technology - which is both colorfast and crack-resistant throughout the entire depth².

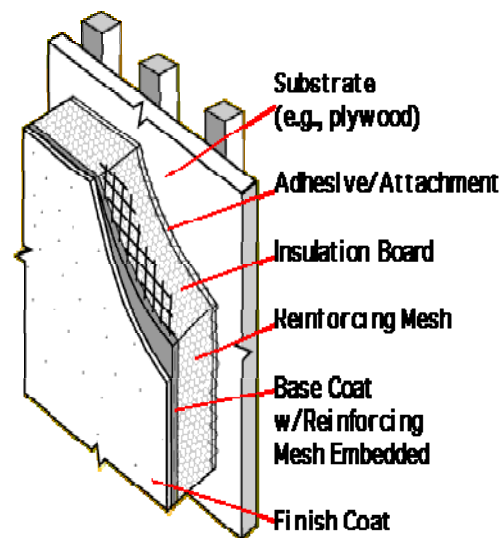


Image 1: Typical EIFS assembly
© 2002 EIFS Industry Members Association²

¹ "History and Development of EIFS -from the Original Concept to Present Day Activities"

² <http://www.EMIA.com/>

Benefits

EIFS provides superior energy efficiency and offers much greater design flexibility than other cladding products. When combined with standard wall insulation, the R-value for the wall is increased by at least five times³. It never needs to be painted and is designed to be totally water resistant. This water resistance is not affected by minor scratches and dents in the exterior layer and even the color remains intact after scratching because of its uniformity throughout the thickness. The finish lamina coatings appear to offer substantial resistance to water penetrations and act in tandem with the reinforced base coat to provide water penetration resistance⁴. Even if there is a breach in the lamina coating, the reinforced base coat will prevent excessive moisture infiltration, allowing time for repairs to be made to the lamina layer. Finally, with EIFS, skilled applicators can create a variety of exterior architectural detailing that would often be too expensive using conventional construction methods; some examples are cornices, arches, columns, keystones, special moldings and decorative accents⁵.

Problems

In the existing EIFS system, once the water enters and becomes trapped between the wood sheathing and the foam insulation, rotting begins⁶, as seen in Image 2⁷. This water usually enters the system through failed sealant at joints, around openings that are not properly flashed, and in areas where the lamina has been cracked or punctured. Test results also indicate that there is a direct relationship between the thickness of the EIFS lamina and the ability of the lamina to resist water penetration. Laminas that were thinner than the manufacturer's recommended minimum (1.6 mm) failed to prevent water penetration. Water was absorbed in thin lamina areas in a few minutes, while water penetration was prevented for hours only a few inches away⁸. Furthermore, it appears that the composition and porosity of the base and finish coats, as well as aggregate sizes, will also affect the lamina's water resistance⁹.



Image 2: Rotten corner post

³ <http://www.EMIA.com/>

⁴ *EIFS Resistance to Water Penetration and Evaluation in Accordance with EMIA Method 101.02*

⁵ <http://www.EMIA.com/>

⁶ http://www.askthebuilder.com/242_The_Barrier EIFS_Nightmare_-_It_is_Real_.shtml

⁷ <http://www.rtbullard.com/stucco/progress/progress34a.htm>

⁸ *EIFS Resistance to Moisture: Face-sealed Barrier Performance*

⁹ *EIFS Resistance to Moisture: Face-sealed Barrier Performance*



Image 3: Rotten sheathing below the insulation



Image 4: Rotten sheathing below the insulation

While the reinforced base coat appears to work with the lamina layer to provide water resistance, the glass fiber mesh has been observed in tests to be weakened by moisture, especially in a high alkaline environment. The lamina base coat must prevent prolonged moisture penetration to the mesh in order to maintain its structural integrity, as seen in Image 3¹⁰. Adequate base coat thickness, base coat primers, and proper mixing are all important for reducing water permeability to the mesh¹¹. Along with these exterior precautions, regular maintenance checks of the system should be performed to correct any surface problems before water has a chance to penetrate the reinforced base coat.

While all these problems appear to be caused by numerous different areas in the system, they can all be traced back to one general cause: workmanship. The improper attention to openings, flashing details, lamina thicknesses, and manufacturer's guidelines ultimately lead to problems in the system. Since barrier EIFS homes rarely communicate any early visible warning signs that massive wood rotting is taking place just inches away, it is important to catch the workmanship problems early during the application process.¹² This creates the final problem for the EIFS: by the time it is known that something is wrong with the system, it is already out of control and the entire system may need to be replaced. , as seen in Image 4¹³.

Possible Solutions

Precautions should be taken at Parkview at Bloomfield Station to protect the EIFS system around any openings: doors, window frames, lighting fixtures, and the areas where flashing is needed such as corners and overhangs. All these areas must be sealed to prevent water from seeping behind the EIFS. Furthermore, the gutters should be kept clean and positioned to drain away from the building, foam insulation should not extend

¹⁰ <http://www.rtbullard.com/stucco/progress/progress34a.htm>

¹¹ *Factors Affecting the Performance of EIFS Cladding*

¹² http://www.askthebuilder.com/242_The_Barrier EIFS_Nightmare_-_It_is_Real_.shtml

¹³ <http://www.rtbullard.com/stucco/progress/progress34a.htm>

below grade, and any items that penetrate the lamina must be properly sealed¹⁴. There are also newer means of removing the water when it does penetrate the system. Drainable EIFS are now commonly a viable option and allows for drainage of the system, as seen in Image 5¹⁵.

However, the main influence on the effectiveness of the EIFS is the workmanship, both in the field and in the design office. In the field quality control needs to be better supervised by trained professionals and attention to detail is critical. More stringent requirements should also be used for the design of unique conditions such as overhangs, changes in wall height, corners, deck projections, and openings where minimum lamina thickness values don't adequately protect against failure. In these areas specialized detailing and/or more accurate thicknesses could be derived based on available weather data¹⁶.

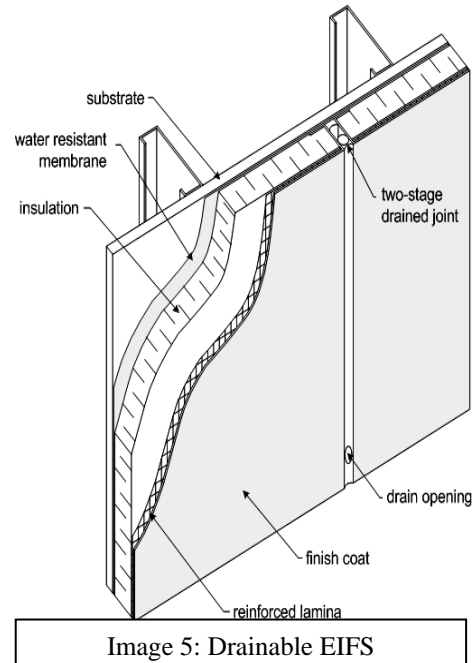


Image 5: Drainable EIFS

Further weather considerations should be taken into account for Parkview at Bloomfield Station due to the unusual shape of the building. The stepped wall, as shown in image 6, will create circulating wind conditions that could cause water to be forced upwards towards the wall cladding. This necessitates that the top and bottom joints in these areas are designed for this condition. In addition to the extra protection needed at the top and bottom of the walls, efforts need to be taken to ensure that the decks do not allow any water to penetrate behind the lamina layer of the system. A drip edge on the decks and proper wall flashing details where the deck meets the EIFS wall should be adequate to prevent this problem.



Image 6: South-East wing

Finally, the orientation of the building must be taken into account in the EIFS design. Due to the fact that the southern side of the building will be warmed by the sun's rays, any water that may be covering the EIFS surface will be evaporated before it gets a chance to penetrate the lamina. Since this is not the case on the northern side of the building, thicker laminas should be used on this side in order to obtain better water penetration prevention characteristics. Thicker laminas should also be employed in areas where water may exist for longer portions of time such as around decks and other projections through the cladding system.

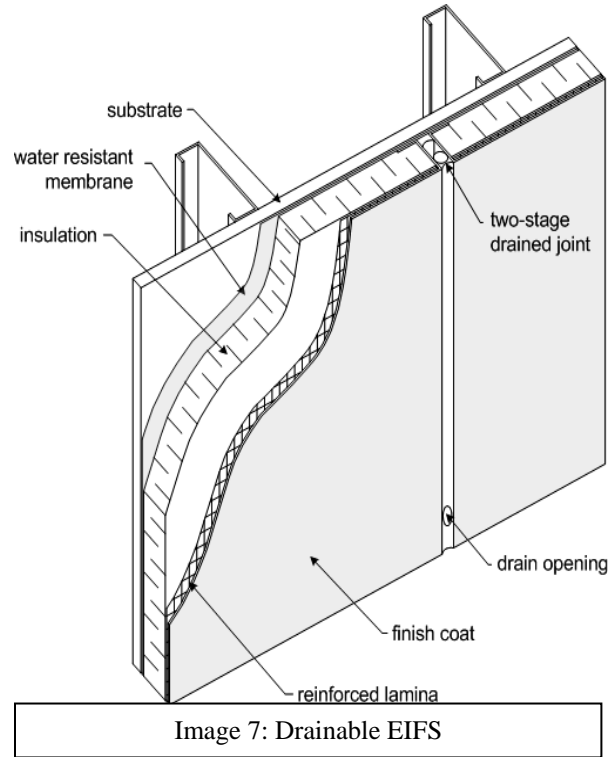
¹⁴ http://homebuying.about.com/cs/syntheticstucco/a/eifs_facts.htm

¹⁵ http://www.civil.uwaterloo.ca/BEG/Drawings/Enclosure_Drawings.htm

¹⁶ *EIFS Resistance to Water Penetration and Evaluation in Accordance with EMIA Method 101.02*

Conclusion

It is my recommendation that drainable EIFS is used at Parkview at Bloomfield Station rather than the original design for conventional EIFS. While the drainable EIFS cladding system is more expensive than the original system, it will ultimately pay dividends to the building owner in the long run. Both systems will fully work if attention to detail and installation is adhered to; yet, the drainable EIFS system allows a factor of safety for any minor mistakes that were made during installation in the field or during design in the office. This added protection can mean the difference of having to replace the entire system in 1-5 years if a problem did occur to having an effective system for more than 20 years with proper maintenance.



The drainable system also employs the use of a water resistant membrane over the substrate material that was not included in the original EIFS design. This extra membrane adds redundancy to the exterior water resisting lamina system. The added membrane forces the water to exit through the drain openings rather than being absorbed into the substrate as was the case during water penetration in typical EIFS designs.

Throughout many tests, typical EIFS and drainable EIFS have been proven to be effective wall systems when properly installed. Yet, in order for the drainable EIFS system to be effective for Parkview at Bloomfield Station, qualified laborers must be located and supervised to ensure a good product. If this is not done, it generally will lead to faulty areas in the wall assemblies that create moisture penetration and eventually rot and mold. Once rot and mold are present, entire areas of wall must be replaced to prevent further spread of the problem areas.

To sum it up, the viability of the system as a successful wall product is ultimately up to the design professionals, the quality controllers in the field, and the workers. Problems with EIFS products have been avoided in many projects and can be avoided in Parkview at Bloomfield Station through proper attention to the design detailing, and proper installation of the materials.



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Parkview at Bloomfield Station
Spring 2006

Section IV
Conclusion

Faculty Consultant:
Professor Parfitt

This thesis presented and proved the process taken to create a new design for Parkview at Bloomfield Station, a residential building located in Bloomfield, New Jersey. The original design criteria was introduced and played an important role in how the new design was laid out. From these design criteria, the new structural system was created and analyzed based on meeting and exceeding the structural and architectural requirements. Finally, the two breadth topics that were introduced to determine the new buildings cost advantage and cladding showed that compared to the original design there were advantages in both designs.

The original structural system, composed of light gage roof trusses, panelized bearing light gage walls, 16" deep D500 Hambro® floor joists, and 38 shear walls in the main lateral force resisting system, was the more complex of the two building systems. Yet, with this complexity came the benefit of a composite floor system, less weight, and more redundancy throughout the building.

The analysis of the new steel braced frame design also showed benefits that were not part of the original design. One such benefit is that the overall system is less complex and has a structure that can progress rapidly. This design also created a skeleton that can allow the change of uses at a later point more readily than the original design.

This design process also showed that using the most recent codes in the design analysis did not have a tremendous difference on the overall result because only minor changes occurred between ASCE7-02 and ASCE7-05. Furthermore, from the two different framing orientations analyzed, the 20K9 bar joist floor system spanning 38'-0" was concluded to be the most efficient and compatible design, but it encountered slight problems with vibrations. A vibration analysis on the bar joist system shows that the floor is over the design limits but can still be considered acceptable.

Furthermore, the use of the braced frame system requires less lateral frames than the original system making it less redundant, but creating gravity frames at some unit separation locations allowed for more savings and architectural advantages. This helps to preserve the architecture of the living units while allowing for quicker structural assembly. The foundation system in the new design, composed of spread footings that replace the original strip footings, was more expensive but had the same amount of material. This concentration of material helped out the lateral frames by providing greater downward force to resist the overturning moment and uplift. In addition to these structural issues, the new structural design was able to support the architectural features of the building. This is most evident at the drive aisles where the bracing patterns needed to mimic the decorative arches.

While the precast garage at the center of the building is structurally separate and did not have much impact on the building design, it did show what a building separation of 4" made an effective building separation distance.

The breadth topics were the final issues to be dealt with in the structure. In these sections two topics that effected large portions of Parkview at Bloomfield Station were investigated. In the first topic section, it was seen that the effect of changing from the original system to the system had nearly a half-million dollar cost advantage and that the sequencing schedule was more flexible than the original system. It also made note that the original system had structural benefits that did not have a price tag but were important nonetheless.

In the second topic section, the current Exterior Insulation and Finish System (EIFS) were studied for its benefits and shortfalls. After learning about past mistakes made with the system, it was shown that the most important factor in getting an EIFS to function properly was the quality of workmanship put into the project. With this in mind, the recommendations for adequate and complete details along with proper supervision were advised. In addition, a method to compensate for any error in the design or supervision is to use a drainable EIFS. Even though this system is more expensive, it is the best solution to guarantee a proper building envelope and is used in the new design of the building.

Throughout the design comparison between the new design and the old design, there have been many benefits and disadvantages shown for both systems, but in the end both are equally viable solutions to the same problem



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Section V
Appendices

Faculty Consultant:
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Section V
Building Summary

Faculty Consultant:
Professor Parfitt

General Building Statistics:

- ❖ **Building Name:** Parkview at Bloomfield Station
- ❖ **Location and Site:** Washington St, Bloomfield Township, NJ 07003
- ❖ **Building Occupant Name:** Toll Brothers, Inc.
- ❖ **Occupancy or Function Types:**
 - **Primary Occupancy:** Condominiums (use group R-2)
 - **Accessory Occupancy:** Pre cast parking garage (use group S-2)
- ❖ **Code:** IBC 2000 NJ (no structural changes noted in the NJ version)
Fair housing Act
- ❖ **Zoning:** Residential (There are no special or historical issues for the building)
- ❖ **Size:**
 - **Total Sq. Ft.:** 453,473 ft²
 - **Building:** 300,725 ft²
 - **Pre cast Garage:** 152,748 ft²
- ❖ **Number of Stories Above Grade:** 6 stories
- ❖ **Project Team:**
 - **Owner:** Toll Brothers, Inc.
www.tollbrothers.com
Forest City Residential Group
www.fceinc.com
 - **Architect:** Minno and Wasko
www.minnowasko.com
 - **Structural Engineer:** Cates Engineering
www.cateseng.com
 - **Pre cast Engineer:** Unistress Corporation
www.unistresscorp.com
 - **Civil Engineer:** PMK Group
www.pmkgroup.com
 - **MEP Engineer:** R.W. Sullivan, Inc.
www.rwsullivan.com
 - **Contractor/ CM:** Bovis - lend lease
www.bovislendlease.com
- ❖ **Dates of Construction:**
 - **Project Startup:** March 27, 2005
 - **Design Development:** July 29, 2005
 - **Final Specs:** August 5, 2005
 - **Permit Set:** October 12, 2005
 - **Planned construction start:** November 10, 2005
- ❖ **Cost:**
 - **Overall Project:** \$65,616,081
 - **Building:** \$56,936,063
 - **Pre cast Garage:** \$8,680,018
- ❖ **Project delivery method:** Qualified Design-bid-build (to be determined exactly)

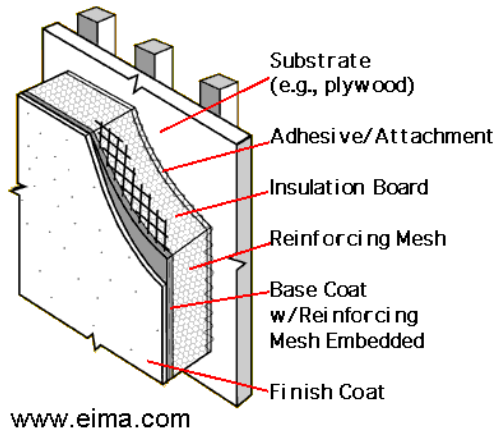
Building System Information:

Architecture:

Parkview at Bloomfield Station is a unique 6 story residential condominium building located in Bloomfield Township, NJ. The building's footprint is irregular and wraps around a pre-cast parking garage. The building is nestled between Toney's Brook, Washington St, and a train station for the Midtown Line. There are 197 condominium units and 330 parking spaces included in the design of this building. There is an exercise room located above the lobby area for residential use. There is a drop off circle in the front of the building and a outdoor gazebo and portico area for resident enjoyment.

Building Envelope:

The roof system consists of asphalt shingle roofing over metal deck and fire rated plywood. The light gage roof trusses have either a 12:12 slope or 8:12 slope for gable roofs, with portions in the middle acting as a flat roof. Standing seam copper roof is used in some areas. The main body of the roof consists of hip roof conditions accented with gable roof sections over balcony areas.



Exterior Finish & Insulation System

The exterior wall cladding is an Exterior Finish and Insulation System (EFIS) over light gage walls. Anderson vinyl windows and entry doors also make up parts of the wall system. The wall along the train line consists mainly of the open pre cast garage panels. The building is separated from the pre cast garage by an air gap and expansion joint. The

building foundation is a 2'-6" wide continuous footings with 3'-0" column spread footings. The pre-cast garage has an extensive pile foundation consisting of 29 pile groups; each consisting of (5) 50' long "H" steel piles of 100 ton capacity each. There are 10" retaining walls on the site also, each with a footing of at least 2'-6" wide.

Structural:

The structural system for Parkview @ Bloomfield Station is a Hambro® and bearing panelized light gage wall system. The typical exterior bearing wall consists of a 6" light gage wall, while the typical interior bearing wall consists of a 4" light gage wall. All 6 floors have the same floor plans with the exception of 4 locations: an entry/lobby unit, a 2 story drive aisle, a 1 story drive aisle, and at a 1st floor exit route. The typical floor system is a 16" deep joist Hambro composite floor system with 3" slabs (3000 psi conc.). The ground level floor is composed of a 4" slab on grade system. The Hambro system is supported on either wide flange beams or tube steel distribution plates located on the light gage bearing walls. The typical beam is

W10x12, HSS 4x4x5/16", or HSS 6x4x5/16". Column sizes range from HSS 3x3x1/4" to HSS 7x3x3/8".

The foundation system of Parkview is continuous footings, spread footings, and piles. The Garage is on 100 ton H piles. The piles are drilled to bedrock (ranging from 42-53 ft). The Garage consists of double T shapes attached to pre cast verticals. The lateral force resisting frame consists of strapped light gage shear walls and cross bracing system over drive aisles. Interior walls are panelized non-bearing light gage infill walls. The roof consists of light gage roof trusses at 48" oc, and includes girder trusses to accommodate end hip conditions.

Construction:

Site work includes existing building demolition, grading, importing fill and locating new utilities. It will also include installation of fountains, gazebo and a portico. The construction of the parking garage will be completed first, using a crane to lift the pre cast double T shapes into place. Upon completion of the parking garage the panelized light gage walls will be lifted into place and anchored into the Hambro floor system. The delivery method is a qualified design-bid-build for both the building and the parking garage. The total cost of completing the building is \$65,616,081, and that does not include a CM bond.

Electrical:

Panels are sized at 125A 1P3W, and there are 2 building transformers. Secondary service includes feeders in GRC conduit from the utility company transformers to the (2) 3000 amp switchboards. Apartment services are fed from meter centers located on each floor at 2 locations, with 125 amp feeders to each apartment load center with 3C#1 armored interlock cables. The emergency generator is a 250 KW 120/208, diesel fired with an 800 amp ATS. The HVAC equipment in the units is based on (1) 20/3p fed to the HWH/HVAC unit an electric range and dryer, each with 50/2P feeds. Each unit is accessed with (1) 3/4" conduit from the tele/com closet and (1) 1" pull wire conduit for CATV requirements. The electrical system includes the incoming duct banks for electrical/CATV/Tele utilities, all to be concrete encased with pull wires. Also included is the fit out of building and garage area with lighting, HVAC, FA, telephone and CATV.

Lighting:

There is lighting located exteriorly at the fountain, gazebo, and portico. In addition, site lighting includes pole fixtures, and the access point of the existing (2) 5" PVC utility line. Interior lighting consists of MC cabling and EMT within garage areas. Temporary lighting and power is included.

Mechanical:

The HVAC includes sheet metal ducts covered in 1 1/2" insulation, insulation board on the exterior walls, unit temperature controls, refrigerant piping, water heaters, condensate drain piping, grd's testing and balancing, and rigging. Electric base board, air handling units, gas fired furnaces and air cooled condensing units compose the HVAC system. There is a refrigerant loop between air handling units and

condensing units. Hot water from hot water heaters is sent to air handling units for use in heating air. The exhaust fan is Nutone LS80 (no lights).
I will be getting more information in this area.

Fire Protection:

The fire protection system includes a fire pump, wet sprinkler system and a dry sprinkler system in the attic area. The garage will have a dry sprinkler system. The wet fire protection system is connected to a 6" combination water main, installed with a 1,500 GPM fire pump and jockey pump. There is a fire department connection on the west side of the building to the fire protection room. The dry systems in the attic and garage are on a connected system and branched from this room. The dry system will be installed with galvanized steel pipe. The floor mains and branches will be orange CPVC with sprinkler heads concealed in the plaster ceiling.

Plumbing:

The building includes sanitary system, natural gas, domestic water with booster pump, plumbing fixtures, gas hot water heaters, washing machine indirect waste and water heater indirect waste. The main utility room is located outside the 2nd floor trash room. The garage includes storm water and oily water drains, non freeze wall hydrants connect to condo building and domestic water. The sanitary system is composed of schedule 40 PVC pipe and drainage fittings. The natural gas system is based on a load of 70 CFH per hot water heater. It is supplied from a 10" line from the source and branches off into schedule 40 black steel pipes varying in size from 8" to 3" with 2" valve taps for single risers. Domestic water is brought in through a 6" pipe with water meter and backflow preventer. Once inside, it is split into (2) 4" mains for north and south halves. Main sizes vary from 4" down to 2" based on 14.5 WSFU per condo and velocity less than 8 FPS. A Triplex water booster pump is used to maintain the water pressure to upper floors. Plumbing fixtures are attached with type DWV copper piping. The hot water heater is a direct vent with 75 gallon storage and 100 GPH recovery.

Telecommunications:

Telephone and CATV outlets are provided for each living unit.

Transportation:

There are (2) 6-stop 2,500lbs elevators and (1) 6-stops 3,500lbs elevator. There are also six stair wells that access to all 6 levels of the building, and access to all floors from the 6 story attached parking garage.

Special Systems:

There is a security system terminal at the main entrance, 14 additional locations have panel points, and there is (1) overhead electric security parking door at the garage.



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Section V
***Calculations, Figures, Tables,
and Charts Index***

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Section V
Calculations Section

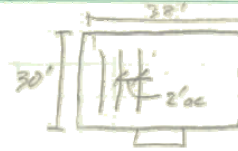
Faculty Consultant:
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Initial Joist Design

Floor Redesign #1 | Bar Joist System | ASD

Span = 31' > 30' try to cover all cases

@ 2'-0" oc w/ 2" slab



$$LL = 40 \text{ psf}$$

$$DL = 17.55 + \frac{3.5}{12} (150 \text{ psf}) + 8 \text{ plf} \left(\frac{1}{2' \text{ oc}} \right) = 65.3 \text{ psf} \quad \left. \begin{array}{l} \\ \end{array} \right\} TL = 105.3 \text{ psf}$$

∴ try 16K5 from New Columbia Joist Company
allowable plf @ 2' oc

$$TL = 228 \text{ plf} \geq 105.3 (2') = 210.6 \text{ plf} \quad \therefore \text{ok}$$

$$LL = 114 \text{ plf} \geq 40 (2') = 80 \text{ plf} \quad \therefore \text{ok}$$

needs 3 rows of bridging

16K5 steel joists @ 2'-0" oc w/ 3 rows of bridging

Decking

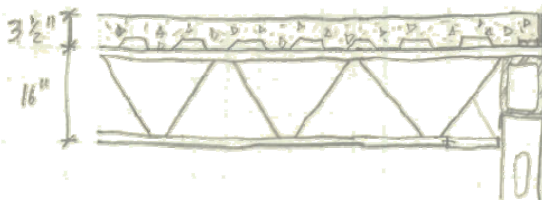
Try Vulcraft 0.6C, CSV Conform decking, 28 gage, non-composite
clear span = 2'-0" ^{3.5" total slab depth, 1/8" deep deck (t=1.5" total)}
_{Fire rating min.}

$$\text{Actual Load} = (40 \text{ psf}) + (47.6 \text{ psf}) = 87.6 \text{ psf}$$

3 span @ 2' spacing
0.6C28 w/ 3.5" slab or 6x6-41.4x41.4: 400 psf ≥ 92.8 psf ∴ ok
deck only: 186 psf = 240 ≥ 92.8 psf ∴ no shearing needed
Allowable span: 2'-7" ≥ 2'-0" ∴ ok
w1 = 177 psf > 41 psf slab + deck weight ∴ ok

doesn't meet
T+S req.

30" wide 0.6C28 w/ 3.5" slab w/ 6x6-41.4x41.4



UL 3hr rating G523

22-141 50 SHEETS
22-142 100 SHEETS
22-144 200 SHEETS



New Joist Design

Span = 38' > 38' typ to cover all cases

@ 2'-0" oc w/ 2" slab



$$LL = 60 \text{ psf}$$

$$DL = 17.55 + \frac{3.5''}{12} (150 \text{ psf}) + 8 \text{ plf} \left(\frac{1}{2' \text{ oc}} \right) = 65.3 \text{ psf} \quad \left. \vphantom{DL} \right\} TL = 125.3 \text{ psf}$$

∴ try 20K9 from New Columbia Joist Company
allowable plf @ 2' oc

$$TL = 228 \text{ plf} \geq 125.3 (2') = 250.6 \text{ plf} \quad \therefore \text{ok}$$

$$LL = 114 \text{ plf} \geq 60 (2') = 120 \text{ plf} \quad \therefore \text{ok}$$

needs 3 rows of bridging

20K9 steel joists @ 2'-0" oc w/ 3 rows of bridging

Decking

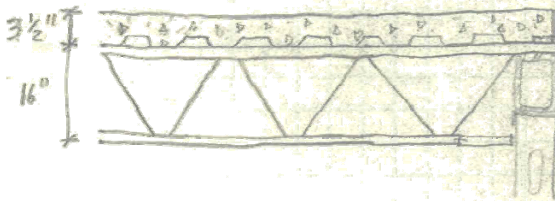
Try Vulcraft 0.6C, CSV Conform decking, 28 gage, non-composite
clear span = 2'-0", 3.5" total slab depth, 1/8" deep deck (t = 1.5" total)
Fire rating min.

$$\text{Actual Load} = (40 \text{ psf}) + (47.6 \text{ psf}) = 87.6 \text{ psf}$$

3 span @ 2' spacing
0.6C28 w/ 3.5" slab or 6x6-W14x14: 400 psf \geq 92.8 psf ∴ ok
deck only; 186 psf = 240 \geq 92.8 psf ∴ no shoring needed
Allowable span; 2'-7" \geq 2'-0" ∴ ok
W1 = 177 psf > 41 psf slab + deck weight ∴ ok

doesn't meet
T + S req.

30" wide 0.6C28 w/ 3.5" slab w/ 6x6-W14x14



UL 3hr rating G523

Joist Size

K-SERIES ECONOMY TABLE

Joist Designation	26K5	16K9	24K7	18K9	26K6	20K9	26K7	22K9	28K6	24K8				
Depth (in.)	26	16	24	18	26	20	26	22	28	24				
Approx. Wt (lbs./ft.)	9.8	10	10.1	10.2	10.6	10.8	10.9	11.3	11.4	11.5				
Span (ft.)														
16		550												
17		550												
18		550		550										
19		550		550										
20		550		550		550								
21		550		550		550								
22		550		550		550		550						
23		550		550		550		550						
24		550	550	550		550		550		550				
25		514	550	550		550		550		550				
26	542	474	550	538	550	550	550	550		550				
27	502	439	550	498	547	550	550	550		550				
28	466	408	521	463	508	517	550	550	548	550				
29	434	380	485	431	473	482	527	532	511	536				
30	405	355	453	402	441	450	492	497	477	500				
31	379	332	424	376	413	421	460	465	446	468				
32	356	311	397	353	387	395	432	436	418	439				
33	334	299	373	332	364	371	406	410	393	413				
34	315	283	351	312	343	349	382	386	370	388				
35	297	267	331	294	323	329	360	364	349	366				
36	280	253	313	278	305	311	340	344	330	346				
37	265	236	296	262	289	294	322	325	312	327				
38	251	221	281	247	274	279	305	308	296	310				
39	238	211	266	232	260	265	289	292	280	294				
40	227	200	253	219	247	251	275	278	266	280				
41	215	187	241	207	235	239	262	264	253	266				
42	205	178	229	195	224	228	249	252	241	253				
43	196	169	219	185	213	217	238	240	230	242				
44	187	160	209	175	204	208	227	229	220	231				
45	179	151	199	165	194	198	217	210	220	231				
46	171	142	191	157	186	190	207	201	211	221				
47	164	133	183	149	178	182	199	192	202	212				
48	157	124	175	141	171	175	190	184	194	204				
49	150	115	165	131	164	168	183	177	187	197				
50	144	106	157	123	157	161	175	170	180	190				
51	139	97	149	115	151	155	168	163	173	183				
52	133	88	141	106	145	149	162	157	167	177				
53														
54														
55														
56														

Deck Size



0.6 C, CSV CONFORM



MAXIMUM CONSTRUCTION CLEAR SPANS (S.D.I. CRITERIA)

Total Slab Depth	Deck Type	Weight PSF	NW Concrete N=9 145 PCF			Weight PSF	LW Concrete N=14 110 PCF		
			1 Span	2 Span	3 Span		1 Span	2 Span	3 Span
2" (t=1 1/2")	0.6C28	23	2-3	2-10	2-11	17	2-4	3-0	3-0
	0.6C26	23	2-8	3-5	3-5	18	2-9	3-6	3-7
	0.6C24	23	3-4	4-3	4-4	18	3-6	4-6	4-7
	0.6C22	23	3-10	5-0	5-1	18	4-1	5-4	5-4
2 1/2" (t=2")	0.6C28	29	2-2	2-9	2-10	22	2-3	2-10	2-11
	0.6C26	29	2-6	3-3	3-4	22	2-8	3-5	3-6
	0.6C24	29	3-2	4-1	4-2	22	3-4	4-4	4-4
	0.6C22	29	3-8	4-9	4-10	23	3-11	5-1	5-2
3" (t=2 1/2")	0.6C28	35	2-1	2-8	2-8	27	2-2	2-10	2-10
	0.6C26	35	2-5	3-2	3-2	27	2-7	3-4	3-4
	0.6C24	35	3-0	3-11	4-0	27	3-2	4-2	4-2
	0.6C22	35	3-6	4-7	4-7	27	3-9	4-10	4-11
3 1/2" (t=3")	0.6C28	41	2-0	2-7	2-7	31	2-1	2-9	2-9
	0.6C26	41	2-4	3-0	3-1	31	2-6	3-3	3-3
	0.6C24	41	2-10	3-9	3-10	32	3-1	4-0	4-1
	0.6C22	42	3-4	4-5	4-5	32	3-7	4-8	4-9
4" (t=3 1/2")	0.6C28	47	1-11	2-6	2-7	36	2-1	2-8	2-8
	0.6C26	47	2-3	2-11	3-0	36	2-5	3-2	3-2
	0.6C24	47	2-9	3-8	3-8	36	3-0	3-11	3-11
	0.6C22	48	3-2	4-3	4-3	36	3-5	4-6	4-7
4 1/2" (t=4")	0.6C28	53	1-10	2-5	2-6	40	2-0	2-7	2-8
	0.6C26	53	2-2	2-10	2-11	40	2-4	3-1	3-1
	0.6C24	53	2-8	3-6	3-7	41	2-10	3-9	3-10
	0.6C22	54	3-1	4-1	4-2	41	3-4	4-5	4-5
5" (t=4 1/2")	0.6C28	59	1-10	2-5	2-5	45	1-11	2-6	2-7
	0.6C26	59	2-1	2-9	2-10	45	2-3	3-0	3-0
	0.6C24	59	2-7	3-5	3-6	45	2-10	3-8	3-9
	0.6C22	60	3-0	3-11	4-0	46	3-3	4-3	4-4

NON-COMPOSITE

REINFORCED CONCRETE SLAB ALLOWABLE LOADS

Total Slab Depth	Reinforcement		Superimposed Uniform Load (psf) — 3 Span Condition												
			Clear Span (ft.-in.)												
			2-0	2-3	2-6	2-9	3-0	3-3	3-6	3-9	4-0	4-6	5-0		
2" (t=1 1/2")	6X6-W1.4XW1.4	0.028*	194	153	124	103	86	74	63						
	6X6-W2.1XW2.1	0.042	285	225	183	151	127	108	93						
	6X6-W2.9XW2.9	0.058	384	304	246	203	171	146	125						
2 1/2" (t=2")	6X6-W1.4XW1.4	0.028*	268	212	172	142	119	102	88	76	67	53			
	6X6-W2.1XW2.1	0.042	396	313	254	210	176	150	129	113	99	78			
	6X6-W2.9XW2.9	0.058	400	400	344	284	239	204	176	153	134	106			
3" (t=2 1/2")	6X6-W1.4XW1.4	0.028*	342	271	219	181	152	130	112	97	86	68			
	6X6-W2.1XW2.1	0.042*	400	400	325	268	226	192	166	144	127	100			
	6X6-W2.9XW2.9	0.058	400	400	400	366	307	262	226	197	173	137			
3 1/2" (t=3")	6X6-W2.1XW2.1	0.042*	400	400	396	327	275	234	202	176	155				
	6X6-W2.9XW2.9	0.058*	400	400	400	400	375	320	276	240	211				
	4X4-W2.9XW2.9	0.087	400	400	400	400	400	400	400	353	310				
4" (t=3 1/2")	6X6-W2.1XW2.1	0.042*	400	400	400	384	322	275	237	206	181				
	6X6-W2.9XW2.9	0.058*	400	400	400	400	400	372	321	280	246				
	4X4-W2.9XW2.9	0.087	400	400	400	400	400	400	400	400	358				
4 1/2" (t=4")	6X6-W2.9XW2.9	0.058*	400	400	400	400	400	400	359	313	275				
	4X4-W2.9XW2.9	0.087	400	400	400	400	400	400	400	400	400				
	4X4-W4.0XW4.0	0.120	400	400	400	400	400	400	400	400	400				
5" (t=4 1/2")	6X6-W2.9XW2.9	0.058*	400	400	400	400	400	400	396	345	303				
	4X4-W2.9XW2.9	0.087*	400	400	400	400	400	400	400	400	400				
	4X4-W4.0XW4.0	0.120	400	400	400	400	400	400	400	400	400				
			0.6C28	0.6C26	0.6C24	0.6C22									

- NOTES:
- * As does not meet A.C.I. criterion for temperature and shrinkage.
 - Recommended conform types are based upon S.D.I. criteria and normal weight concrete.
 - Superimposed loads are based upon three span conditions and A.C.I. moment coefficients.
 - Load values for single span and double spans are to be reduced.
 - Superimposed load values in bold type require that mesh be draped. See page 19.
 - Vulcraft's painted or galvanized form deck can be considered as permanent support in most building applications. See page 19. If uncoated form deck is used, deduct the weight of the slab from the allowable superimposed uniform loads.

Maximum Foundation Load



RAM Manager v10.0
DataBase: Bloomfield

Foundation Load Summary

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Col	Level	Case	V x	V y	P	M xx	M yy	T
		W4	1.49	1.51	6.34	0.10	0.15	0.00
		W5	4.97	5.07	19.18	0.51	0.34	0.00
		W6	3.60	3.67	13.66	0.37	0.23	-0.00
		W7	4.87	4.96	19.18	0.46	0.37	-0.00
		W8	-2.48	-2.54	-8.97	-0.30	-0.12	0.00
		W9	3.67	3.74	14.42	0.35	0.28	-0.00
		W10	4.85	4.94	19.14	0.46	0.37	0.00
		E1	0.47	0.48	2.58	0.03	0.05	-0.00
		E2	0.57	0.57	3.06	0.04	0.06	0.00
		E3	0.71	0.73	3.43	0.08	0.05	0.00
		E4	0.47	0.48	2.24	0.05	0.03	-0.00
130	1	D	63.68	0.00	144.47	0.01	5.50	-0.00
		Lp	30.33	0.00	44.43	0.00	1.39	-0.00
		W1	6.87	-0.00	16.44	-0.00	0.30	-0.00
		W2	-0.76	0.02	-2.38	0.17	-0.03	-0.00
		W3	5.96	-0.00	14.24	-0.00	0.26	-0.00
		W4	6.07	-0.00	14.54	0.00	0.26	0.00
		W5	-0.26	0.02	-1.01	0.17	-0.01	0.00
		W6	-1.08	0.01	-3.16	0.13	-0.04	-0.00
		W7	4.58	0.01	10.54	0.13	0.20	-0.00
		W8	5.73	-0.01	14.12	-0.13	0.24	0.00
		W9	3.66	0.01	8.30	0.10	0.16	-0.00
		W10	4.36	0.01	10.15	0.13	0.19	0.00
		E1	2.42	-0.00	6.39	-0.00	0.10	-0.00
		E2	2.48	0.00	6.58	0.00	0.10	0.00
		E3	0.02	0.00	0.02	0.02	0.00	0.00
		E4	-0.14	0.00	-0.46	0.02	-0.00	-0.00
133	1	D	56.22	0.00	454.96	0.03	54.15	-0.01
		Lp	24.23	0.00	103.19	0.01	11.91	-0.00
		W1	4.41	-0.00	31.35	-0.01	8.39	0.00
		W2	-0.18	0.04	-3.36	1.80	-0.58	-0.10
		W3	3.87	-0.00	27.43	-0.04	7.35	0.00
		W4	3.85	0.00	27.42	0.02	7.33	-0.00
		W5	-0.24	0.04	-2.98	1.76	-0.59	-0.10
		W6	-0.08	0.03	-2.89	1.39	-0.43	-0.08
		W7	3.17	0.03	20.99	1.34	5.86	-0.07
		W8	3.44	-0.03	26.03	-1.36	6.73	0.07
		W9	2.85	0.02	18.41	1.02	5.19	-0.06
		W10	2.70	0.03	18.33	1.33	5.06	-0.07
		E1	1.27	-0.00	12.88	-0.01	2.92	0.00
		E2	1.25	0.00	12.88	0.03	2.90	-0.00
		E3	-0.04	0.01	-0.52	0.33	-0.10	-0.02
		E4	-0.01	0.01	-0.51	0.24	-0.07	-0.01

Typical Column Sizes



RAM Steel v10.0
 DataBase: Bloomfield
 Building Code: IBC

Gravity Column Design Summary

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 Steel Code: AISC LRFD

Column Line S3 - SR

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
6	12.3	1.4	3.9	10	0.11 Eq H1-1b	90.0	50	W10X33
5	25.2	0.7	1.8	1	0.08 Eq H1-1b	90.0	50	W10X33
4	36.0	0.7	1.7	1	0.12 Eq H1-1b	90.0	50	W10X33
3	46.6	0.7	1.7	1	0.16 Eq H1-1b	90.0	50	W10X33
2	57.1	0.6	1.6	1	0.19 Eq H1-1b	90.0	50	W10X33
1	67.2	0.6	1.7	10	0.26 Eq H1-1a	90.0	50	W10X33

Column Line S4 - SH

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
6	27.5	16.4	0.1	1	0.16 Eq H1-1b	90.0	50	W10X33
5	51.2	7.6	0.1	1	0.17 Eq H1-1b	90.0	50	W10X33
4	74.3	7.4	0.1	1	0.30 Eq H1-1a	90.0	50	W10X33
3	97.1	7.2	0.1	1	0.37 Eq H1-1a	90.0	50	W10X33
2	119.7	7.1	0.1	1	0.45 Eq H1-1a	90.0	50	W10X33
1	142.2	7.1	0.1	1	0.52 Eq H1-1a	90.0	50	W10X33

Column Line S4 - SJ

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
6	29.1	15.7	1.6	1	0.19 Eq H1-1b	90.0	50	W10X33
5	53.9	7.2	0.8	1	0.18 Eq H1-1b	90.0	50	W10X33
4	78.0	7.0	0.8	1	0.32 Eq H1-1a	90.0	50	W10X33
3	101.7	6.8	0.8	1	0.40 Eq H1-1a	90.0	50	W10X33
2	125.2	6.7	0.7	1	0.47 Eq H1-1a	90.0	50	W10X33
1	148.6	6.6	0.7	1	0.55 Eq H1-1a	90.0	50	W10X33

Column Line 92.25ft - 98.00ft

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
6	29.0	13.1	3.9	1	0.21 Eq H1-1b	90.0	50	W10X33
5	53.9	6.0	1.9	1	0.18 Eq H1-1b	90.0	50	W10X33
4	78.1	5.8	1.8	1	0.33 Eq H1-1a	90.0	50	W10X33
3	102.0	5.6	1.8	1	0.41 Eq H1-1a	90.0	50	W10X33
2	125.7	5.6	1.8	1	0.49 Eq H1-1a	90.0	50	W10X33
1	149.1	5.5	1.8	1	0.56 Eq H1-1a	90.0	50	W10X33

Column Line S5 - SH

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
6	23.6	14.1	0.1	1	0.14 Eq H1-1b	90.0	50	W10X33
5	43.8	6.5	0.1	1	0.15 Eq H1-1b	90.0	50	W10X33
4	63.4	6.3	0.1	1	0.25 Eq H1-1a	90.0	50	W10X33
3	82.8	6.1	0.1	1	0.32 Eq H1-1a	90.0	50	W10X33
2	101.9	6.0	0.1	1	0.38 Eq H1-1a	90.0	50	W10X33
1	120.9	6.0	0.1	1	0.44 Eq H1-1a	90.0	50	W10X33

Some Non-typical Column Sizes



RAM Steel v10.0
 DataBase: Bloomfield
 Building Code: IBC

Gravity Column Design Summary

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 Steel Code: AISC LRFD

Column Line S00 - SR

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
6	83.6	3.2	42.6	1	0.86 Eq H1-1a	90.0	50	W12X40
5	160.4	1.6	20.4	1	0.74 Eq H1-1a	90.0	50	W12X40
4	236.0	1.6	20.0	1	0.95 Eq H1-1a	90.0	50	W12X40
3	311.3	1.6	22.8	1	0.70 Eq H1-1a	90.0	50	W12X58
2	388.5	1.6	22.7	1	0.82 Eq H1-1a	90.0	50	W12X58
1	466.4	1.6	22.7	1	0.96 Eq H1-1a	90.0	50	W12X58

Column Line S0 - SM2

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
6	88.0	3.2	44.5	1	0.88 Eq H1-1a	90.0	50	W10X39
5	168.9	1.6	21.3	1	0.78 Eq H1-1a	90.0	50	W10X39
4	248.7	1.6	20.9	1	0.99 Eq H1-1a	90.0	50	W10X39
3	328.1	1.6	24.0	1	0.70 Eq H1-1a	90.0	50	W10X60
2	410.3	1.6	24.0	1	0.83 Eq H1-1a	90.0	50	W10X60
1	492.6	1.6	24.0	1	0.96 Eq H1-1a	90.0	50	W10X60

Column Line S1 - SG

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
6	37.1	19.5	2.6	1	0.25 Eq H1-1b	90.0	50	W10X33
5	69.5	9.1	1.3	1	0.31 Eq H1-1a	90.0	50	W10X33
4	101.1	8.8	1.2	1	0.41 Eq H1-1a	90.0	50	W10X33
3	132.2	8.6	1.2	1	0.52 Eq H1-1a	90.0	50	W10X33
2	163.1	8.5	1.2	1	0.62 Eq H1-1a	90.0	50	W10X33
1	193.9	8.4	1.2	1	0.72 Eq H1-1a	90.0	50	W10X33

Column Line 37.75ft - 199.75ft

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
6	96.5	0.0	40.8	10	0.76 Eq H1-1a	90.0	50	W12X45
5	192.8	0.0	19.5	4	0.72 Eq H1-1a	90.0	50	W12X45
4	283.9	0.0	19.1	4	0.94 Eq H1-1a	90.0	50	W12X45
3	378.0	0.0	24.8	4	0.67 Eq H1-1a	90.0	50	W12X65
2	472.7	0.0	24.8	4	0.80 Eq H1-1a	90.0	50	W12X65
1	564.4	0.0	24.8	10	0.93 Eq H1-1a	90.0	50	W12X65

Column Line 37.75ft - 232.00ft

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
6	9.7	0.2	4.9	1	0.11 Eq H1-1b	90.0	50	W10X33
5	19.1	0.1	2.4	1	0.08 Eq H1-1b	90.0	50	W10X33
4	27.2	0.1	2.3	1	0.09 Eq H1-1b	90.0	50	W10X33
3	35.2	0.1	2.2	1	0.12 Eq H1-1b	90.0	50	W10X33
2	43.0	0.1	2.1	1	0.14 Eq H1-1b	90.0	50	W10X33
1	50.8	0.1	2.1	1	0.17 Eq H1-1b	90.0	50	W10X33



Gravity Column Design Summary

Column Line 161.25ft - 132.25ft

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
6	21.4	2.6	0.0	1	0.07 Eq H1-1b	90.0	50	W10X33
5	39.5	1.2	0.0	1	0.13 Eq H1-1b	90.0	50	W10X33
4	57.0	1.2	0.0	1	0.19 Eq H1-1b	90.0	50	W10X33
3	74.2	1.8	0.0	2	0.26 Eq H1-1a	90.0	50	W10X33
2	91.3	1.7	0.0	2	0.32 Eq H1-1a	90.0	50	W10X33
1	108.2	1.1	0.0	1	0.37 Eq H1-1a	90.0	50	W10X33

Column Line 161.25ft - 145.00ft

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
6	136.8	51.4	20.5	10	0.64 Eq H1-1a	0.0	50	W10X49
5	267.9	24.5	9.6	4	0.71 Eq H1-1a	0.0	50	W10X49
4	400.1	24.4	9.6	4	0.97 Eq H1-1a	0.0	50	W10X49
3	533.9	25.7	9.8	4	0.68 Eq H1-1a	0.0	50	W10X88
2	667.8	25.7	9.8	4	0.82 Eq H1-1a	0.0	50	W10X88
1	798.7	25.7	9.8	10	0.96 Eq H1-1a	0.0	50	W10X88

Column Line 166.75ft - 124.00ft

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
6	7.6	4.4	0.0	1	0.04 Eq H1-1b	0.0	50	W10X33
5	15.2	2.2	0.0	1	0.05 Eq H1-1b	0.0	50	W10X33
4	21.8	2.1	0.0	1	0.07 Eq H1-1b	0.0	50	W10X33
3	28.0	2.0	0.0	1	0.09 Eq H1-1b	0.0	50	W10X33
2	34.2	2.0	0.0	1	0.11 Eq H1-1b	0.0	50	W10X33
1	40.2	1.9	0.0	1	0.13 Eq H1-1b	0.0	50	W10X33

Column Line S8 - SN

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
6	12.4	1.4	2.9	6	0.09 Eq H1-1b	90.0	50	W10X33
5	27.9	0.7	1.0	1	0.09 Eq H1-1b	90.0	50	W10X33
4	40.8	0.7	1.0	1	0.14 Eq H1-1b	90.0	50	W10X33
3	53.1	0.7	1.0	1	0.18 Eq H1-1b	90.0	50	W10X33
2	65.3	0.7	1.3	2	0.24 Eq H1-1a	90.0	50	W10X33
1	76.2	0.7	1.3	6	0.28 Eq H1-1a	90.0	50	W10X33

Column Line 166.75ft - 185.75ft

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
6	67.6	37.0	1.7	8	0.48 Eq H1-1a	90.0	50	W10X33
5	129.6	17.6	0.8	2	0.56 Eq H1-1a	90.0	50	W10X33
4	190.3	17.2	0.8	2	0.76 Eq H1-1a	90.0	50	W10X33
3	250.7	17.4	0.8	2	0.69 Eq H1-1a	90.0	50	W10X45
2	310.8	17.3	0.8	2	0.83 Eq H1-1a	90.0	50	W10X45
1	372.1	17.2	0.8	1	0.97 Eq H1-1a	90.0	50	W10X45



Gravity Column Design Summary

Column Line S8 - ST

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
6	80.8	43.6	5.1	1	0.62 Eq H1-1a	90.0	50	W10X33
5	154.1	20.7	2.5	1	0.68 Eq H1-1a	90.0	50	W10X33
4	226.1	20.2	2.4	1	0.92 Eq H1-1a	90.0	50	W10X33
3	297.7	20.3	2.8	1	0.69 Eq H1-1a	90.0	50	W10X49
2	368.9	20.1	2.8	1	0.83 Eq H1-1a	90.0	50	W10X49
1	439.8	20.0	2.8	1	0.97 Eq H1-1a	90.0	50	W10X49

Column Line S9 - SI

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
6	15.0	5.1	1.6	12	0.09 Eq H1-1b	90.0	50	W10X33
5	30.6	2.2	0.8	1	0.10 Eq H1-1b	90.0	50	W10X33
4	43.8	2.1	0.7	1	0.15 Eq H1-1b	90.0	50	W10X33
3	56.7	2.0	0.7	1	0.19 Eq H1-1b	90.0	50	W10X33
2	69.5	2.1	0.7	3	0.26 Eq H1-1a	90.0	50	W10X33
1	82.2	1.9	0.7	1	0.30 Eq H1-1a	90.0	50	W10X33

Column Line S10 - SK

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
6	11.7	1.5	3.1	10	0.09 Eq H1-1b	90.0	50	W10X33
5	24.4	0.7	1.4	1	0.08 Eq H1-1b	90.0	50	W10X33
4	35.0	0.7	1.3	1	0.12 Eq H1-1b	90.0	50	W10X33
3	45.2	0.7	1.3	1	0.15 Eq H1-1b	90.0	50	W10X33
2	55.3	0.7	1.2	1	0.19 Eq H1-1b	90.0	50	W10X33
1	65.0	0.7	1.3	10	0.24 Eq H1-1a	90.0	50	W10X33

Column Line 276.75ft - 134.00ft

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
6	78.5	39.3	0.6	6	0.51 Eq H1-1a	90.0	50	W10X33
5	152.4	18.7	0.3	2	0.63 Eq H1-1a	90.0	50	W10X33
4	224.0	18.3	0.2	2	0.87 Eq H1-1a	90.0	50	W10X33
3	295.2	18.3	0.3	2	0.66 Eq H1-1a	90.0	50	W10X49
2	368.5	18.3	0.3	2	0.80 Eq H1-1a	90.0	50	W10X49
1	441.5	18.3	0.3	6	0.94 Eq H1-1a	90.0	50	W10X49

Column Line 276.75ft - 164.00ft

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
6	117.5	0.0	0.0	1	0.33 Eq H1-1a	90.0	50	W12X40
5	225.4	0.0	0.0	1	0.63 Eq H1-1a	90.0	50	W12X40
4	331.6	0.0	0.0	1	0.92 Eq H1-1a	90.0	50	W12X40
3	437.4	0.0	0.0	1	0.62 Eq H1-1a	90.0	50	W12X65
2	546.8	0.0	0.0	1	0.77 Eq H1-1a	90.0	50	W12X65
1	656.3	0.0	0.0	1	0.92 Eq H1-1a	90.0	50	W12X65



Gravity Column Design Summary

Column Line 354.75ft - 170.00ft

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
6	63.1	1.5	25.1	8	0.65 Eq H1-1a	0.0	50	W10X33
5	122.7	0.6	11.9	2	0.62 Eq H1-1a	0.0	50	W10X33
4	180.1	0.5	11.7	2	0.81 Eq H1-1a	0.0	50	W10X33
3	237.2	0.5	11.5	2	0.71 Eq H1-1a	0.0	50	W10X45
2	293.9	0.5	11.4	2	0.84 Eq H1-1a	0.0	50	W10X45
1	351.5	0.2	11.4	1	0.98 Eq H1-1a	0.0	50	W10X45

Column Line 360.50ft - 36.00ft

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
6	23.6	2.2	10.0	8	0.25 Eq H1-1b	0.0	50	W10X33
5	44.1	1.0	4.6	2	0.17 Eq H1-1b	0.0	50	W10X33
4	63.7	1.0	4.4	2	0.29 Eq H1-1a	0.0	50	W10X33
3	82.9	1.0	4.3	2	0.36 Eq H1-1a	0.0	50	W10X33
2	102.0	1.0	4.2	2	0.42 Eq H1-1a	0.0	50	W10X33
1	120.8	0.9	4.1	1	0.48 Eq H1-1a	0.0	50	W10X33

Column Line S13 - SN

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
6	18.3	4.2	5.9	1	0.17 Eq H1-1b	90.0	50	W10X33
5	35.7	2.1	2.9	1	0.13 Eq H1-1b	90.0	50	W10X33
4	51.5	2.0	2.7	1	0.17 Eq H1-1b	90.0	50	W10X33
3	67.1	2.0	2.6	1	0.28 Eq H1-1a	90.0	50	W10X33
2	82.6	2.0	2.6	1	0.33 Eq H1-1a	90.0	50	W10X33
1	97.9	2.0	2.5	1	0.38 Eq H1-1a	90.0	50	W10X33

Column Line S14 - S0A

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
6	79.3	39.8	0.6	6	0.52 Eq H1-1a	90.0	50	W10X33
5	153.9	18.9	0.3	2	0.64 Eq H1-1a	90.0	50	W10X33
4	226.4	18.5	0.2	2	0.87 Eq H1-1a	90.0	50	W10X33
3	298.4	18.6	0.3	2	0.66 Eq H1-1a	90.0	50	W10X49
2	372.4	18.5	0.3	2	0.81 Eq H1-1a	90.0	50	W10X49
1	446.3	18.5	0.3	6	0.95 Eq H1-1a	90.0	50	W10X49

Column Line 379.25ft - 30.00ft

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
6	107.7	0.0	0.0	1	0.30 Eq H1-1a	90.0	50	W10X39
5	206.2	0.0	0.0	1	0.58 Eq H1-1a	90.0	50	W10X39
4	303.1	0.0	0.0	1	0.85 Eq H1-1a	90.0	50	W10X39
3	399.5	0.0	0.0	1	0.64 Eq H1-1a	90.0	50	W10X60
2	498.0	0.0	0.0	1	0.80 Eq H1-1a	90.0	50	W10X60
1	597.7	0.0	0.0	1	0.96 Eq H1-1a	90.0	50	W10X60



Gravity Column Design Summary

Column Line 354.75ft - 170.00ft

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
6	63.1	1.5	25.1	8	0.65 Eq H1-1a	0.0	50	W10X33
5	122.7	0.6	11.9	2	0.62 Eq H1-1a	0.0	50	W10X33
4	180.1	0.5	11.7	2	0.81 Eq H1-1a	0.0	50	W10X33
3	237.2	0.5	11.5	2	0.71 Eq H1-1a	0.0	50	W10X45
2	293.9	0.5	11.4	2	0.84 Eq H1-1a	0.0	50	W10X45
1	351.5	0.2	11.4	1	0.98 Eq H1-1a	0.0	50	W10X45

Column Line 360.50ft - 36.00ft

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
6	23.6	2.2	10.0	8	0.25 Eq H1-1b	0.0	50	W10X33
5	44.1	1.0	4.6	2	0.17 Eq H1-1b	0.0	50	W10X33
4	63.7	1.0	4.4	2	0.29 Eq H1-1a	0.0	50	W10X33
3	82.9	1.0	4.3	2	0.36 Eq H1-1a	0.0	50	W10X33
2	102.0	1.0	4.2	2	0.42 Eq H1-1a	0.0	50	W10X33
1	120.8	0.9	4.1	1	0.48 Eq H1-1a	0.0	50	W10X33

Column Line S13 - SN

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
6	18.3	4.2	5.9	1	0.17 Eq H1-1b	90.0	50	W10X33
5	35.7	2.1	2.9	1	0.13 Eq H1-1b	90.0	50	W10X33
4	51.5	2.0	2.7	1	0.17 Eq H1-1b	90.0	50	W10X33
3	67.1	2.0	2.6	1	0.28 Eq H1-1a	90.0	50	W10X33
2	82.6	2.0	2.6	1	0.33 Eq H1-1a	90.0	50	W10X33
1	97.9	2.0	2.5	1	0.38 Eq H1-1a	90.0	50	W10X33

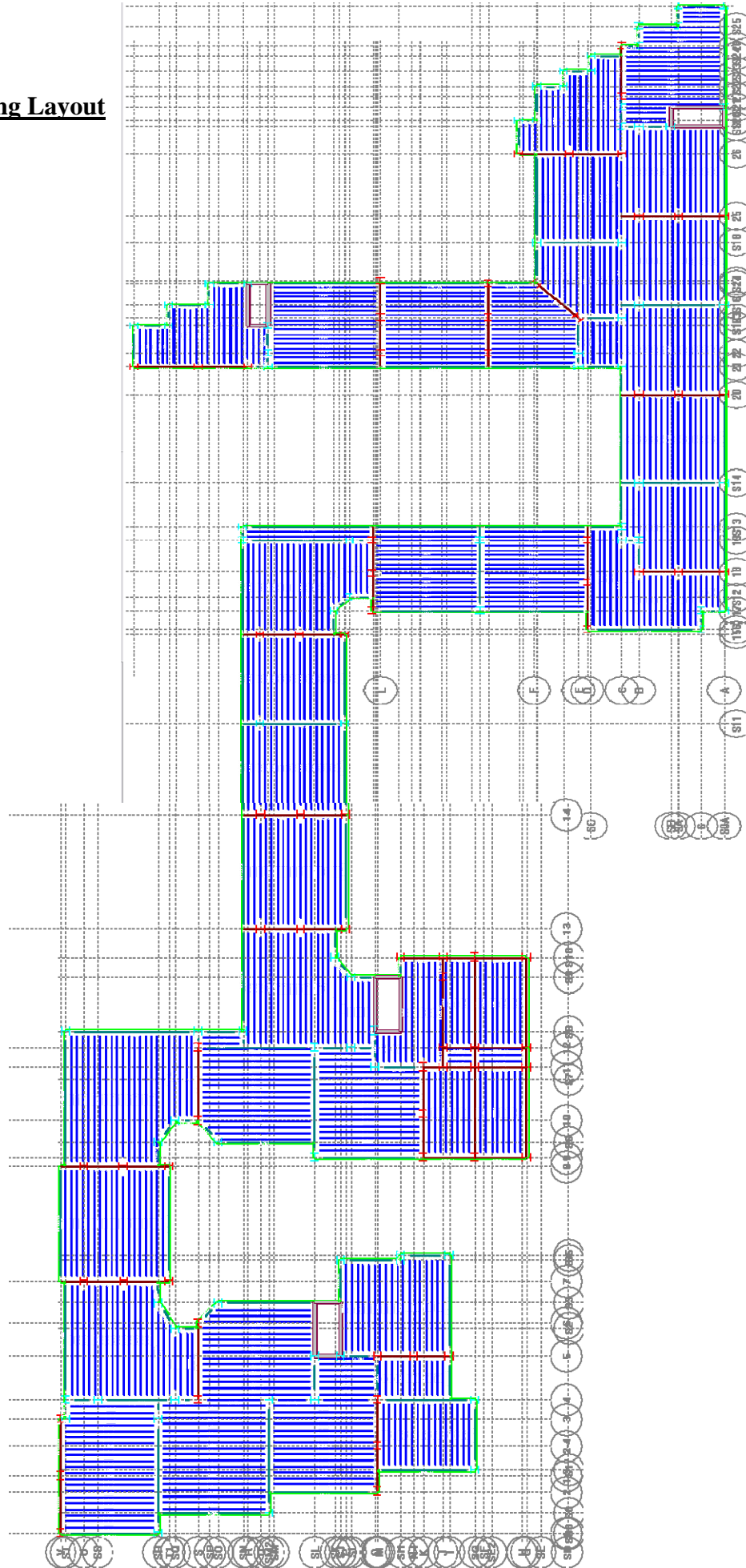
Column Line S14 - S0A

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
6	79.3	39.8	0.6	6	0.52 Eq H1-1a	90.0	50	W10X33
5	153.9	18.9	0.3	2	0.64 Eq H1-1a	90.0	50	W10X33
4	226.4	18.5	0.2	2	0.87 Eq H1-1a	90.0	50	W10X33
3	298.4	18.6	0.3	2	0.66 Eq H1-1a	90.0	50	W10X49
2	372.4	18.5	0.3	2	0.81 Eq H1-1a	90.0	50	W10X49
1	446.3	18.5	0.3	6	0.95 Eq H1-1a	90.0	50	W10X49

Column Line 379.25ft - 30.00ft

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
6	107.7	0.0	0.0	1	0.30 Eq H1-1a	90.0	50	W10X39
5	206.2	0.0	0.0	1	0.58 Eq H1-1a	90.0	50	W10X39
4	303.1	0.0	0.0	1	0.85 Eq H1-1a	90.0	50	W10X39
3	399.5	0.0	0.0	1	0.64 Eq H1-1a	90.0	50	W10X60
2	498.0	0.0	0.0	1	0.80 Eq H1-1a	90.0	50	W10X60
1	597.7	0.0	0.0	1	0.96 Eq H1-1a	90.0	50	W10X60

Framing Layout



Drift at end of North-East Wing



RAM Frame v10.0
 DataBase: Bloomfield2_v10
 Building Code: IBC

Drift

04/03/06 18:35:50
 Steel Code: IBC

CRITERIA:

Rigid End Zones: Ignore Effects
 Member Force Output: At Face of Joint
 P-Delta: Yes Scale Factor: 1.00
 Diaphragm: Rigid
 Ground Level: Base

LOAD CASE DEFINITIONS:

D	DeadLoad	RAMUSER
Lp	PosLiveLoad	RAMUSER
W1	wind	Wind_IBC00_1_X
W2	wind	Wind_IBC00_1_Y
W3	wind	Wind_IBC00_2_X+E
W4	wind	Wind_IBC00_2_X-E
W5	wind	Wind_IBC00_2_Y+E
W6	wind	Wind_IBC00_2_Y-E
W7	wind	Wind_IBC00_3_X+Y
W8	wind	Wind_IBC00_3_X-Y
W9	wind	Wind_IBC00_4_CW
W10	wind	Wind_IBC00_4_CCW
E1	EQ	EQ_IBC00_X_+E_F
E2	EQ	EQ_IBC00_X_-E_F
E3	EQ	EQ_IBC00_Y_+E_F
E4	EQ	EQ_IBC00_Y_-E_F

RESULTS:

Location (ft): (429.274, 208.352)

Story	LdC	Displacement		Story Drift		Drift Ratio	
		X in	Y in	X in	Y in	X	Y
6	D	-0.0191	0.0169	-0.0040	0.0044	0.0000	0.0000
	Lp	-0.0087	0.0095	-0.0018	0.0024	0.0000	0.0000
	W1	0.0417	-0.0006	0.0078	-0.0001	0.0001	0.0000
	W2	0.0033	0.1265	0.0005	0.0245	0.0000	0.0002
	W3	0.0375	-0.0019	0.0070	-0.0004	0.0001	0.0000
	W4	0.0355	0.0008	0.0066	0.0001	0.0001	0.0000
	W5	-0.0043	0.1205	-0.0010	0.0233	0.0000	0.0002
	W6	0.0101	0.1008	0.0018	0.0195	0.0000	0.0002
	W7	0.0338	0.0944	0.0062	0.0183	0.0000	0.0001
	W8	0.0288	-0.0953	0.0055	-0.0185	0.0000	0.0001
W9	0.0357	0.0741	0.0066	0.0143	0.0001	0.0001	
W10	0.0234	0.0910	0.0042	0.0176	0.0000	0.0001	



Drift

Story	LdC	Displacement		Story Drift		Drift Ratio	
	E1	0.0183	-0.0004	0.0037	-0.0001	0.0000	0.0000
	E2	0.0169	0.0015	0.0034	0.0003	0.0000	0.0000
	E3	-0.0022	0.0229	-0.0005	0.0048	0.0000	0.0000
	E4	0.0013	0.0181	0.0002	0.0038	0.0000	0.0000
5	D	-0.0151	0.0126	-0.0043	0.0040	0.0000	0.0000
	Lp	-0.0069	0.0070	-0.0019	0.0022	0.0000	0.0000
	W1	0.0340	-0.0005	0.0081	-0.0001	0.0001	0.0000
	W2	0.0028	0.1020	0.0005	0.0251	0.0000	0.0002
	W3	0.0305	-0.0015	0.0073	-0.0004	0.0001	0.0000
	W4	0.0289	0.0007	0.0069	0.0002	0.0001	0.0000
	W5	-0.0034	0.0972	-0.0009	0.0239	0.0000	0.0002
	W6	0.0083	0.0813	0.0019	0.0200	0.0000	0.0002
	W7	0.0276	0.0761	0.0065	0.0187	0.0001	0.0001
	W8	0.0234	-0.0768	0.0057	-0.0189	0.0000	0.0001
	W9	0.0291	0.0598	0.0069	0.0147	0.0001	0.0001
	W10	0.0191	0.0734	0.0045	0.0181	0.0000	0.0001
	E1	0.0145	-0.0003	0.0038	-0.0001	0.0000	0.0000
	E2	0.0134	0.0012	0.0035	0.0003	0.0000	0.0000
	E3	-0.0017	0.0181	-0.0005	0.0048	0.0000	0.0000
	E4	0.0010	0.0143	0.0003	0.0038	0.0000	0.0000
4	D	-0.0108	0.0086	-0.0039	0.0035	0.0000	0.0000
	Lp	-0.0050	0.0048	-0.0018	0.0020	0.0000	0.0000
	W1	0.0259	-0.0003	0.0081	-0.0001	0.0001	0.0000
	W2	0.0023	0.0769	0.0006	0.0247	0.0000	0.0002
	W3	0.0233	-0.0011	0.0073	-0.0004	0.0001	0.0000
	W4	0.0220	0.0005	0.0069	0.0002	0.0001	0.0000
	W5	-0.0024	0.0733	-0.0009	0.0235	0.0000	0.0002
	W6	0.0064	0.0612	0.0019	0.0197	0.0000	0.0002
	W7	0.0211	0.0574	0.0065	0.0184	0.0001	0.0001
	W8	0.0177	-0.0579	0.0056	-0.0186	0.0000	0.0001
	W9	0.0223	0.0451	0.0069	0.0145	0.0001	0.0001
	W10	0.0147	0.0554	0.0045	0.0178	0.0000	0.0001
	E1	0.0108	-0.0002	0.0036	-0.0001	0.0000	0.0000
	E2	0.0099	0.0009	0.0034	0.0003	0.0000	0.0000
	E3	-0.0013	0.0133	-0.0004	0.0046	0.0000	0.0000
	E4	0.0008	0.0105	0.0003	0.0036	0.0000	0.0000
3	D	-0.0069	0.0050	-0.0032	0.0028	0.0000	0.0000
	Lp	-0.0032	0.0028	-0.0015	0.0016	0.0000	0.0000
	W1	0.0178	-0.0002	0.0076	-0.0001	0.0001	0.0000
	W2	0.0017	0.0522	0.0006	0.0225	0.0000	0.0002
	W3	0.0160	-0.0008	0.0068	-0.0003	0.0001	0.0000
	W4	0.0152	0.0004	0.0064	0.0002	0.0000	0.0000



Drift

Story	LdC	Displacement		Story Drift		Drift Ratio	
	W5	-0.0016	0.0497	-0.0008	0.0215	0.0000	0.0002
	W6	0.0045	0.0415	0.0018	0.0180	0.0000	0.0001
	W7	0.0146	0.0390	0.0061	0.0168	0.0000	0.0001
	W8	0.0121	-0.0393	0.0052	-0.0170	0.0000	0.0001
	W9	0.0154	0.0306	0.0065	0.0132	0.0001	0.0001
	W10	0.0102	0.0376	0.0042	0.0163	0.0000	0.0001
	E1	0.0071	-0.0002	0.0032	-0.0001	0.0000	0.0000
	E2	0.0066	0.0006	0.0030	0.0003	0.0000	0.0000
	E3	-0.0008	0.0087	-0.0004	0.0040	0.0000	0.0000
	E4	0.0005	0.0069	0.0002	0.0032	0.0000	0.0000
2	D	-0.0037	0.0022	-0.0037	0.0022	0.0000	0.0000
	Lp	-0.0017	0.0012	-0.0017	0.0012	0.0000	0.0000
	W1	0.0102	-0.0001	0.0102	-0.0001	0.0000	0.0000
	W2	0.0011	0.0296	0.0011	0.0296	0.0000	0.0001
	W3	0.0092	-0.0004	0.0092	-0.0004	0.0000	0.0000
	W4	0.0087	0.0002	0.0087	0.0002	0.0000	0.0000
	W5	-0.0008	0.0282	-0.0008	0.0282	0.0000	0.0001
	W6	0.0027	0.0236	0.0027	0.0236	0.0000	0.0001
	W7	0.0085	0.0221	0.0085	0.0221	0.0000	0.0001
	W8	0.0069	-0.0223	0.0069	-0.0223	0.0000	0.0001
	W9	0.0089	0.0174	0.0089	0.0174	0.0000	0.0001
	W10	0.0059	0.0213	0.0059	0.0213	0.0000	0.0001
	E1	0.0039	-0.0001	0.0039	-0.0001	0.0000	0.0000
	E2	0.0036	0.0003	0.0036	0.0003	0.0000	0.0000
	E3	-0.0004	0.0047	-0.0004	0.0047	0.0000	0.0000
	E4	0.0003	0.0037	0.0003	0.0037	0.0000	0.0000
1	D	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	Lp	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	W1	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	W2	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	W3	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	W4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	W5	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	W6	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	W7	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	W8	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	W9	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	W10	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	E1	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	E2	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	E3	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	E4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

Foundation Summary



RAM Manager v10.0
DataBase: Bloomfield

Foundation Load Summary

Date: 03/07/06 16:09:47

Forces on Gravity Members from RAM Steel.

Gravity Column Loads

Forces on Gravity Members from RAM Steel.

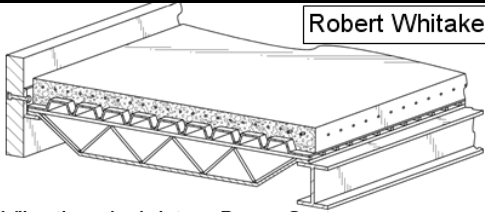
Col	Level	Dead kip	Self kip	+Live kip	-Live kip	+Roof kip	-Roof kip	Min Total kip	Max Total kip
1	1	309.52	3.15	56.98	0.00	0.00	0.00	312.67	369.65
3	1	327.11	3.19	60.13	0.00	0.00	0.00	330.30	390.44
6	1	125.43	2.13	25.49	0.00	0.00	0.00	127.56	153.06
8	1	367.78	3.53	76.12	0.00	0.00	0.00	371.32	447.44
9	1	28.53	2.13	8.77	0.00	0.00	0.00	30.66	39.43
11	1	151.74	2.13	31.06	0.00	0.00	0.00	153.87	184.94
13	1	102.97	2.13	24.73	0.00	0.00	0.00	105.10	129.83
14	1	330.98	3.53	68.15	0.00	0.00	0.00	334.51	402.66
16	1	169.01	2.13	36.34	0.00	0.00	0.00	171.15	207.48
17	1	181.61	2.13	38.54	0.00	0.00	0.00	183.74	222.28
18	1	103.34	2.13	22.16	0.00	0.00	0.00	105.47	127.63
22	1	40.27	2.13	12.23	0.00	0.00	0.00	42.40	54.63
23	1	102.97	2.13	24.73	0.00	0.00	0.00	105.10	129.83
24	1	50.60	2.13	13.87	0.00	0.00	0.00	52.73	66.60
26	1	53.69	2.13	11.89	0.00	0.00	0.00	55.82	67.71
27	1	39.10	2.13	11.21	0.00	0.00	0.00	41.23	52.44
31	1	89.36	2.13	20.26	0.00	0.00	0.00	91.49	111.75
32	1	93.19	2.13	21.38	0.00	0.00	0.00	95.32	116.70
33	1	95.09	2.13	20.30	0.00	0.00	0.00	97.22	117.52
34	1	74.91	2.13	17.80	0.00	0.00	0.00	77.04	94.83
41	1	325.26	2.80	60.50	0.00	0.00	0.00	328.06	388.56
42	1	54.81	2.13	12.14	0.00	0.00	0.00	56.94	69.08
43	1	28.00	2.13	8.36	0.00	0.00	0.00	30.13	38.50
45	1	58.75	2.13	15.83	0.00	0.00	0.00	60.88	76.71
51	1	62.32	2.13	16.76	0.00	0.00	0.00	64.45	81.20
55	1	64.90	2.13	17.35	0.00	0.00	0.00	67.03	84.38
56	1	520.35	4.42	107.47	0.00	0.00	0.00	524.77	632.24
58	1	21.10	2.13	7.72	0.00	0.00	0.00	23.23	30.95
59	1	49.07	2.13	9.94	0.00	0.00	0.00	51.21	61.15
60	1	247.70	2.53	44.89	0.00	0.00	0.00	250.22	295.12
61	1	294.53	2.65	51.96	0.00	0.00	0.00	297.18	349.14
64	1	48.36	2.13	13.51	0.00	0.00	0.00	50.50	64.00
69	1	37.85	2.13	10.85	0.00	0.00	0.00	39.99	50.83
76	1	289.73	2.65	57.15	-2.68	0.00	0.00	289.70	349.52
77	1	434.59	3.38	81.71	0.00	0.00	0.00	437.97	519.68
81	1	152.32	2.13	31.11	0.00	0.00	0.00	154.45	185.56
83	1	29.55	2.13	7.91	0.00	0.00	0.00	31.69	39.60
84	1	29.96	2.13	9.07	0.00	0.00	0.00	32.10	41.17
85	1	289.89	2.65	57.15	-2.48	0.00	0.00	290.06	349.69



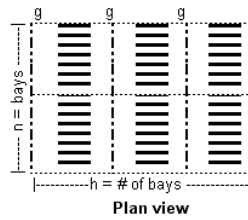
Foundation Load Summary

Col	Level	Dead	Self	+Live	-Live	+Roof	-Roof	Min Total	Max Total
87	1	38.12	2.13	10.92	0.00	0.00	0.00	40.25	51.17
88	1	49.03	2.13	13.72	0.00	0.00	0.00	51.16	64.88
94	1	45.81	2.13	13.40	0.00	0.00	0.00	47.94	61.34
95	1	48.17	2.13	13.95	0.00	0.00	0.00	50.30	64.25
97	1	427.35	3.38	80.60	0.00	0.00	0.00	430.73	511.33
99	1	180.07	2.13	37.79	0.00	0.00	0.00	182.20	219.99
100	1	232.07	2.53	43.74	0.00	0.00	0.00	234.59	278.33
101	1	74.65	2.13	17.93	0.00	0.00	0.00	76.78	94.72
102	1	62.47	2.13	12.75	0.00	0.00	0.00	64.60	77.35
103	1	293.70	2.65	57.15	-2.11	0.00	0.00	294.24	353.50
104	1	393.95	3.19	75.72	0.00	0.00	0.00	397.14	472.86
108	1	59.60	2.13	15.04	0.00	0.00	0.00	61.73	76.77
109	1	83.30	2.13	18.16	0.00	0.00	0.00	85.43	103.59
110	1	67.20	2.13	14.57	0.00	0.00	0.00	69.33	83.90
114	1	79.27	2.13	20.18	0.00	0.00	0.00	81.40	101.58
117	1	81.24	2.13	20.60	0.00	0.00	0.00	83.37	103.97
118	1	54.55	2.13	15.29	0.00	0.00	0.00	56.68	71.97
119	1	41.52	2.13	11.41	0.00	0.00	0.00	43.66	55.07
120	1	42.04	2.13	10.59	0.00	0.00	0.00	44.17	54.76
121	1	91.69	2.13	22.61	0.00	0.00	0.00	93.82	116.43
125	1	298.56	2.65	57.15	-2.07	0.00	0.00	299.13	358.36
126	1	372.33	3.00	78.10	0.00	0.00	0.00	375.33	453.43
127	1	60.82	2.13	15.69	0.00	0.00	0.00	62.95	78.64
128	1	53.43	2.13	14.02	0.00	0.00	0.00	55.56	69.58
131	1	54.15	2.13	14.01	0.00	0.00	0.00	56.28	70.29
132	1	63.94	2.13	16.45	0.00	0.00	0.00	66.07	82.52
134	1	237.15	2.53	49.82	0.00	0.00	0.00	239.68	289.50
135	1	253.88	2.65	47.84	0.00	0.00	0.00	256.52	304.37
142	1	46.24	2.13	13.54	0.00	0.00	0.00	48.37	61.91
143	1	75.47	2.13	19.48	0.00	0.00	0.00	77.60	97.08
144	1	35.35	2.13	11.13	0.00	0.00	0.00	37.49	48.61
145	1	26.89	2.13	7.61	0.00	0.00	0.00	29.02	36.63
146	1	23.06	2.13	6.51	0.00	0.00	0.00	25.19	31.70
147	1	198.63	2.33	37.55	0.00	0.00	0.00	200.95	238.50
148	1	35.84	2.13	11.25	0.00	0.00	0.00	37.97	49.22
150	1	48.32	2.13	13.59	0.00	0.00	0.00	50.45	64.05
151	1	43.06	2.13	11.48	0.00	0.00	0.00	45.19	56.68
152	1	43.36	2.13	12.43	0.00	0.00	0.00	45.49	57.92
153	1	41.12	2.13	11.89	0.00	0.00	0.00	43.25	55.14
154	1	65.01	2.13	15.90	0.00	0.00	0.00	67.14	83.04
155	1	37.39	2.13	11.14	0.00	0.00	0.00	39.52	50.66
157	1	28.80	2.13	7.96	0.00	0.00	0.00	30.93	38.89
158	1	42.19	2.13	11.44	0.00	0.00	0.00	44.32	55.76
159	1	172.87	2.13	33.72	0.00	0.00	0.00	175.00	208.72
160	1	30.00	2.13	8.04	0.00	0.00	0.00	32.13	40.17

Vibration calculation based on new framing layout of the 30'x38' typical bay



Robert Whitaker



Design Loads	
DL	17.55 psf
LL	60 psf

Vibrations in Joist on Beam System

Based on AISC Steel design guide 11 ex 4.6 & 6.2

20k9		W16x31	
W _{self} (plf)	10.8	W _{self} (plf)	31.0
W _{total allow} (plf)	279.0	A (in ²)	9.13
W _{joist design} (plf)	233.1 ok	d (in)	15.70
d (in)	20.0	I _x (in ⁴)	375.0
M _{allow} (ft-k)	49.48	f _{allow} (k)	30 kip
A _{bottom} (in ²)	1.04	ρ _{conc} (pcf)	145 pcf
A _{top} (in ²)	1.30	E _s (ksi)	29000 ksi
A _{cord} (in ²)	2.34	f _c (ksi)	3 ksi
I _{cord} (in ⁴)	208.9	E _c	3024 ksi
I _{comp} (in ⁴)	466.0	n	7.10
y _c (in)	8.94		
		*update Ws+d value	
t _{conc}	3.00 in	t _{eff} =	3.25 in
t _{deck}	0.50 in		
t _{tot}	3.50 in		

building	
n (# bays)	2 bays
h (# bays)	3 bays

LOADS		
Ws+d	39 psf	<== look up value in deck manual
DL	4.0 psf	<== 4 psf typ office service load
LL	11.0 psf	<== 11 psf typ office service load

Length	
Girder (L _g)	15 feet
Joist (L _j)	38 feet
Joist Spaci	2 feet

joist		girder			
L min =	24	182.4	L min =	72	456
L _{eff} =>	24 in		L _{eff} =>	72 in	

Joist	cord type	angle	
y _j	0.555 in		
I _j	445 in ⁴		6 < L _j /d = 22.8 < 24
C _r	0.896		therefore use Eq 3.16
γ	0.116		
I _{eff}	356 in ⁴		Eq 3.16
I _{mod}	187 in ⁴		Eq 3.15
W _j	119 plf		
Δ _j	0.539 in		
f _j	4.82 hz		

Transformed Joist properties based on unit width		
D _s	4.833 in ⁴ /ft	Transformed moment of inertia per unit of width in x direction
D _j	178.15 in ⁴ /ft	
Joist parallel to an interior edge? flr width Bj calc		
C _j	2	no ==> B _j = 20 ft or 30.84 ft
B _j	20.00 ft	<2/3 * floor width
W _j	45.1 kips	

Girder	
y _g	4.29 in
I _{gcomp}	1472 in ⁴
I _{gnon-comp}	375 in ⁴
I _{gred}	649 in ⁴
W _g	2288 plf
Δ _g	0.138 in
f _g	9.50 hz

Transformed Girder properties based on unit width		
D _j	178.15 in ⁴ /ft	Transformed moment of inertia per unit of width in x direction
D _g	17.08 in ⁴ /ft	
Joist connected to girder web? flr length B _g calc		
C _g	1.6	no ==> B _j = 76 ft or 43.13 ft
B _g	43.13 ft	<2/3 * floor length
W _g	39.0 kips	
Δ _{g'}	0.104 in	L _g < B _j

Stiffness analysis (fn ok, no need to check stiffness analysis)

using a	0.224 kip load	
Δ _{j applied}	0.04282 in	
Δ _{j pannel}	0.00714 in	
Δ _{gPannel}	0.00145 in	(fn = 4.30 Hz)
Δ _{total}	0.00786 in	fn ok
K _{floor}	28.5 kip/in	>5.7kip/in limit ok

Walking Evaluation (fn = 4.30 Hz)	
W _{PANELtot}	43.9 kips
β	0.030 Res_mid low damp table 4.1
β _W	1316.4 #
P _o	65.0 # table 4.1 compare with table 4.1
a _{p/g}	= 0.01098 = 1.098% g fails > 0.5% fails
Fails, need to increase joist size or slab thickness (delta j controls)	

Midspan Flexibility

f _n	4.30 hz		
d _e	3.25 in		
N _{eff} (# joists)	6.00 >1.0 ok eq 4.7		
0.018 ≤ d _e /S _j =	0.135 ≤	0.208 ok, use eq 4.7	0.135417
4.5E+6 ≤ L _j ⁴ /I _j =	121.4E+6 ≤	257.0E+6 ok, use eq 4.7	121.4E+6
2 ≤ L _j /S _j =	19 ≤	30 ok, use eq 4.7	19
Δ _{o j}	191.2E-6 in/lb mid span flexibility		
Δ _{g P}	3.2E-6 in/lb mid span flexibility		
Δ _p	33.5E-6 in/lb mid span flexibility		

Table 4.1 Recommended Values of Parameters in Equation (4.1) and a _p /g Limits			
	Constant Force P _o	Damping Ratio β	Acceleration Limit a _p /g × 100%
Offices, Residences, Churches	0.29 kN (66 lb)	0.02-0.05*	0.5%
Shopping Malls	0.29 kN (66 lb)	0.02	1.5%
Footbridges—Indoor	0.41 kN (92 lb)	0.01	1.5%
Footbridges—Outdoor	0.41 kN (92 lb)	0.01	5.0%

* 0.02 for floors with few non-structural components (ceilings, ducts, partitions, etc.) as can occur in open work areas and churches.
0.03 for floors with non-structural components and furnishings, but with only small demountable partitions, typical of many modular office areas.
0.05 for full height partitions between floors.

MODERATE WALK		<-----		SLOW WALK	
W person	185 #			W person	185 #
step/min	75 step/min			step/min	50 step/min
Fm/W	1.5	(table 6.2)	Uv= 5500 # Hz*2	Fm/W	1.3
Fm	277.5 #			Fm	240.5 #
f_0	2.5 hz	(figure 6.5)		f_0	1.4 hz
f_n/f_0	1.718 >>0.5	use eq 6.4b		f_n/f_0	3.07 >>0.5
$T_0=1/f_0$	0.4 sec			$T_0=1/f_0$	0.7143 sec
f_n*T_0	1.718 > 0.5			f_n*T_0	3.07 > 0.5
Am	0.169			Am	0.053
X max	1573 in x 10 ⁻⁶			X max	428 in x 10 ⁻⁶
V	42,862 x 10 ⁻⁶ in /sec	compare with table 6.1 values		V	11,690 x 10 ⁻⁶ in /sec
					compare with table 6.1 values

**Table 4.1
Recommended Values of Parameters in
Equation (4.1) and a_o/g Limits**

	Constant Force P_o	Damping Ratio β	Acceleration Limit $a_o/g \times 100\%$
Offices, Residences, Churches	0.29 kN (65 lb)	0.02–0.05*	0.5%
Shopping Malls	0.29 kN (65 lb)	0.02	1.5%
Footbridges—Indoor	0.41 kN (92 lb)	0.01	1.5%
Footbridges—Outdoor	0.41 kN (92 lb)	0.01	5.0%

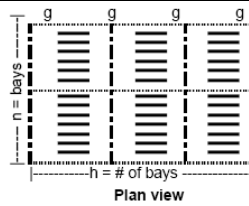
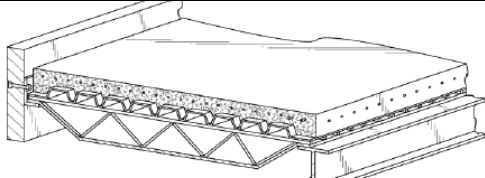
* 0.02 for floors with few non-structural components (ceilings, ducts, partitions, etc.) as can occur in open work areas and churches,
0.03 for floors with non-structural components and furnishings, but with only small demountable partitions, typical of many modular office areas,
0.05 for full height partitions between floors.

**Table 6.1
Vibration Criteria for Sensitive Equipment**

Facility Equipment or Use	Vibrational Velocity*	
	(μ in./sec)	(μ m/sec)
Computer systems; Operating Rooms**; Surgery; Bench microscopes at up to 100x magnification;	8,000	200
Laboratory robots	4,000	100
Bench microscopes at up to 400x magnification; Optical and other precision balances; Coordinate measuring machines; Metrology laboratories; Optical comparators; Microelectronics manufacturing equipment—Class A****	2,000	50
Micro surgery, eye surgery, neuro surgery; Bench microscopes at magnification greater than 400x; Optical equipment on isolation tables; Microelectronics manufacturing equipment—Class B***	1,000	25
Electron microscopes at up to 30,000x magnification; Microtomes; Magnetic resonance imagers; Microelectronics manufacturing equipment—Class C***	500	12
Electron microscopes at greater than 30,000x magnification; Mass spectrometers; Cell implant equipment; Microelectronics manufacturing equipment—Class D****	250	6
Microelectronics Manufacturing equipment—Class E****; Unisolated laser and optical research systems	130	3

* Value of V for Figure 6.1.
** Criterion given by solid curve of Figure 6.1 corresponds to a standard mean whole-body threshold of perception (Guide 1974)
*** Class A: Inspection, probe test, and other manufacturing support equipment.
Class B: Aligners, steppers, and other critical equipment for photolithography with line widths of 3 microns or more.
Class C: Aligners, steppers, and other critical equipment for photolithography with line widths of 1 micron.
Class D: Aligners, steppers, and other critical equipment for photolithography with line widths of 1/2 micron; includes electron-beam systems.
Class E: Aligners, steppers, and other critical equipment for photolithography with line widths of 1/4 micron; includes electron-beam systems.]

Vibration calculation based on new framing layout of the 30'x38' typical bay



Design Loads	
DL	17.55 psf
LL	60 psf

Robert Whitaker

Vibrations in Joist on Beam System

Based on AISC Steel design guide 11 ex 4.6 & 6.2

20K9		W16x31	
W _{self} (plf)	10.8	W _{self} (plf)	31.0
W _{total allow} (plf)	450.0	A (in ²)	9.13
W _{joist design} (plf)	315.1 ok	d (in)	15.70
d (in)	20.0	I _x (in ⁴)	375.0
M _{allow} (ft-k)	49.51	f _{allow} (k)	30 kip
A _{bottom} (in ²)	1.04	ρ _{conc} (pcf)	145 pcf
A _{top} (in ²)	1.30	Es (ksi)	29000 ksi
A _{coord} (in ²)	2.35	f _c (ksi)	3 ksi
I _{coord} (in ⁴)	209.0	E _c	3024 ksi
I _{comp} (in ⁴)	1011.1	n	7.10
Y _c (in)	8.94		
t _{conc}	5.00 in	*update Ws+d value	
t _{deck}	3.00 in		
t _{tot}	8.00 in	t _{eff}	6.50 in

building	
n (# bays)	2 bays
h (# bays)	3 bays

LOADS		
Ws+d	80 psf	<== look up value in deck manual
DL	4.0 psf	<== 4 psf typ office service load
LL	11.0 psf	<== 11 psf typ office service load

Length	
Girder (L _g)	15 feet
Joist (L _j)	30 feet
Joist Spacing	2 feet

joist		girder			
L min =	24	144	L min =	72	360
L _{eff} =>	24 in		L _{eff} =>	72 in	

Joist		cord type:	angle
y _j	0.739 in		
I _j	716 in ⁴		
C _r	0.884		
γ	0.132		
I _j = I _{eff}	494 in ⁴	Eq 3.16	
I _{mod}	185 in ⁴	Eq 3.15	
W _j	201 plf		
Δ _j	0.256 in		
f _j	6.99 hz		

6 < L_j/d = 18 < 24
therefore use Eq 3.16

Transformed Joist properties based on unit width		
D _s	38.662 in ⁴ /ft	Transformed moment of inertia per unit of width in x direction
D _j	246.80 in ⁴ /ft	
Joist parallel to an interior edge?		flr width B _j calc
C _j	2	no
B _j	20.00 ft	=> B _j = 20 ft or 37.75 ft
W _j	60.2 kips	<2/3 * floor width

Girder	
y _g	5.24 in
I _{gcomp}	2466 in ⁴
I _{gnon-comp}	375 in ⁴
I _{gred}	898 in ⁴
W _g	3043 plf
Δ _g	0.133 in
f _g	9.69 hz

Transformed Girder properties based on unit width		
D _j	246.80 in ⁴ /ft	Transformed moment of inertia per unit of width in x direction
D _g	29.92 in ⁴ /ft	
Joist connected to girder web?		flr length B _g calc
C _g	1.6	no
B _g	40.67 ft	=> B _g = 60 ft or 40.67 ft
W _g	61.9 kips	<2/3 * floor length
Δ _{g'}	0.100 in	L _g < B _j

Stiffness analysis (fn ok, no need to check stiffness analysis)	
using a	0.224 kip load
Δ _j applied	0.01521 in
Δ _j pannel	0.00196 in
Δ _g pannel	0.00105 in
Δ _{total}	0.00248 in
K _{floor}	90.4 kip/in

(fn = 5.67 Hz)
fn ok
>5.7kip/in limit ok

Walking Evaluation (fn = 5.67 Hz)	
W _{PANELtot}	60.8 kips
β	0.030
β _{VW}	1824.1 #
P _o	65.0 #
α _p /g	= 0.00490 = 0.490% g < 0.5% ok

Midspan Flexibility	
fn	5.67 hz
d _e	6.5 in
N _{eff} (# joists)	7.78 >1.0 ok
0.018 ≤ d _e /S _j =	0.271 ≤ 0.208 fail, max of 0.208
4.5E+6 ≤ L _j ⁴ /I _j =	34.0E+6 ≤ 257.0E+6 ok, use eq 4.7 34.0E+6
2 ≤ L _j /S _j =	15 ≤ 30 ok, use eq 4.7 15
Δ _{oj}	67.9E-6 in/lb mid span flexibility
Δ _{gP}	2.3E-6 in/lb mid span flexibility
Δ _p	9.9E-6 in/lb mid span flexibility

	Constant Force P _o	Damping Ratio β	Acceleration Limit α _p /g x 100%
Offices, Residences, Churches	0.29 kN (65 lb)	0.02-0.05*	0.5%
Shopping Malls	0.29 kN (65 lb)	0.02	1.5%
Footbridges—Indoor	0.41 kN (92 lb)	0.01	1.5%
Footbridges—Outdoor	0.41 kN (92 lb)	0.01	5.0%

* 0.02 for floors with few non-structural components (ceilings, ducts, partitions, etc.) as can occur in open work areas and churches.
0.05 for floors with non-structural components and furnishings, but with only small demountable partitions, typical of many modular office areas.
0.05 for full height partitions between floors.

MODERATE WALK <-----		SLOW WALK	
W person	185 #	W person	185 #
step/min	75 step/min	step/min	50 step/min
Fm/W	1.5 (table 6.2)	Fm/W	1.3 (table 6.2)
Fm	277.5 #	Fm	240.5 #
f_0	2.5 hz (figure 6.5)	f_0	1.4 hz (figure 6.5)
f_n/f_0	2.269 >>0.5 use eq 6.4b	f_n/f_0	4.05 >>0.5 use eq 6.4b
$T_0=1/f_0$	0.4 sec	$T_0=1/f_0$	0.7143 sec
f_n*T_0	2.269 > 0.5	f_n*T_0	4.05 > 0.5
Am	0.097	Am	0.030
X max	267 in x 10 ⁻⁶	X max	73 in x 10 ⁻⁶
V	9,598 x 10 ⁻⁶ in /sec compare with table 6.1 values	V	2,618 x 10 ⁻⁶ in /sec compare with table 6.1 values

**Table 4.1
Recommended Values of Parameters in
Equation (4.1) and a_o/g Limits**

	Constant Force P_o	Damping Ratio β	Acceleration Limit $a_o/g \times 100\%$
Offices, Residences, Churches	0.29 kN (65 lb)	0.02–0.05*	0.5%
Shopping Malls	0.29 kN (65 lb)	0.02	1.5%
Footbridges—Indoor	0.41 kN (92 lb)	0.01	1.5%
Footbridges—Outdoor	0.41 kN (92 lb)	0.01	5.0%

* 0.02 for floors with few non-structural components (ceilings, ducts, partitions, etc.) as can occur in open work areas and churches,
0.03 for floors with non-structural components and furnishings, but with only small demountable partitions, typical of many modular office areas,
0.05 for full height partitions between floors.

**Table 6.1
Vibration Criteria for Sensitive Equipment**

Facility Equipment or Use	Vibrational Velocity*	
	(μ in./sec)	(μ m/sec)
Computer systems; Operating Rooms**; Surgery; Bench microscopes at up to 100x magnification;	8,000	200
Laboratory robots	4,000	100
Bench microscopes at up to 400x magnification; Optical and other precision balances; Coordinate measuring machines; Metrology laboratories; Optical comparators; Microelectronics manufacturing equipment—Class A****	2,000	50
Micro surgery, eye surgery, neuro surgery; Bench microscopes at magnification greater than 400x; Optical equipment on isolation tables; Microelectronics manufacturing equipment—Class B***	1,000	25
Electron microscopes at up to 30,000x magnification; Microtomes; Magnetic resonance imagers; Microelectronics manufacturing equipment—Class C***	500	12
Electron microscopes at greater than 30,000x magnification; Mass spectrometers; Cell implant equipment; Microelectronics manufacturing equipment—Class D****	250	6
Microelectronics Manufacturing equipment—Class E****; Unisolated laser and optical research systems	130	3

* Value of V for Figure 6.1.
** Criterion given by solid curve of Figure 6.1 corresponds to a standard mean whole-body threshold of perception (Guide 1974)
*** Class A: Inspection, probe test, and other manufacturing support equipment.
Class B: Aligners, steppers, and other critical equipment for photolithography with line widths of 3 microns or more.
Class C: Aligners, steppers, and other critical equipment for photolithography with line widths of 1 micron.
Class D: Aligners, steppers, and other critical equipment for photolithography with line widths of ½ micron; includes electron-beam systems.
Class E: Aligners, steppers, and other critical equipment for photolithography with line widths of ¼ micron; includes electron-beam systems.]



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Section V
References and Bibliography

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Bibliography ~ Existing Building Description

American Institute of Steel Construction “Manual of Steel Construction ~ Allowable Stress Design 2nd Edition” copyright 1997

American Society of Civil Engineers (ASCE) 7-02 “Minimum Design Loads for Buildings and Other Structures” copyright 2003

International Building Code (IBC 2000)

Bibliography ~ Structural Depth Analysis

American Institute of Steel Construction “Manual of Steel Construction ~ Load and Resistance Factor Design 3rd Edition” copyright 2001

American Society of Civil Engineers (ASCE) 7-05 “Minimum Design Loads for Buildings and Other Structures” copyright 2006

International Building Code (IBC 2003)

Bibliography ~ RAM Model

RAM Structural System Program, Version 10

Bibliography ~ Vibrations

American Institute of Steel Construction Steel Design Guide 11 “Floor Vibrations due to Human Activity” copyright 2003 ex 4.6 & 6.2

New Columbia Joist Company “Steel Joists and Joist Girders” Nicholas J. Bouras, Inc. copyright 2002

Vulcraft “Steel Roof and Floor Deck” Nucor Corporation 1998

Bibliography ~ Cost Advantages

RS Means Assembly Cost Data 31st ED 2006

Bibliography ~ EIFS Recommendations

- #1 Zwyer, G. L., “*EIFS: When It Works, When It Does Not*” Pg. 21-25 of Exterior Insulation Finish Systems (EIFS): Materials, Properties, and Performance, ASTM STP 1269, Peter E. Nelson and Richard E. Kroll, Eds., American Society for Testing and Materials, 1996.
- #2 Remmele, T. E., “*EIFS Resistance to Water Penetration and Evaluation in Accordance with EMIA Method 101.02*” Pg. 177-185 of Exterior Insulation Finish Systems (EIFS): Materials, Properties, and Performance, ASTM STP 1269, Peter E. Nelson and Richard E. Kroll, Eds., American Society for Testing and Materials, 1996.
- #3 Tamburrini, V., “*History and Development of EIFS –from the Original Concept to Present Day Activities*” Pg. 3-12 of Development, Use, and Performance of Exterior Insulation and Finish Systems (EIFS), ASTM STP 1187, Mark F. Williams and Richard G. Lampo, Eds., American Society for Testing and Materials, Philadelphia, 1995.
- #4 Piper, R.S. and Raab, S., “*Factors Affecting the Performance of EIFS Cladding*” Pg. 149-161 of Development, Use, and Performance of Exterior Insulation and Finish Systems (EIFS), ASTM STP 1187, Mark F. Williams and Richard G. Lampo, Eds., American Society for Testing and Materials, Philadelphia, 1995.
- #5 Williams, M. F. and Williams, B. L., “*EIFS Resistance to Moisture: Face-sealed Barrier Performance*” Pg. 175-191 of Development, Use, and Performance of Exterior Insulation and Finish Systems (EIFS), ASTM STP 1187, Mark F. Williams and Richard G. Lampo, Eds., American Society for Testing and Materials, Philadelphia, 1995.
- #6 <http://www.EMIA.com/> © 2002 EIFS Industry Members Association
- #7 http://homebuying.about.com/cs/syntheticstucco/a/eifs_facts.htm
- #8 http://www.askthebuilder.com/242_The_Barrier EIFS_Nightmare_-_It_is_Real_.shtml

Pictures

- #1 http://www.civil.uwaterloo.ca/BEG/Drawings/Enclosure_Drawings.htm
- #2 <http://www.rtbullard.com/stucco/progress/progress34a.htm> (6 year old EIFS clad house)