Ingleside at King Farm

Final Thesis Report

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Department of Architectural Engineering
Senior Thesis 2008-2009

Ingleside
AT KING FARM

STATISTICS

Location and Site: 701 King Farm Blvd. Rockville, MD 20852
Building Occupant Name: Elderly Residents and Nurses
Occupancy or function types: CCRC (Continuous Care Retirement Center)
Size: 790,000 SF
Height: 103 feet, 7 above grade, 1 below grade.
Construction Dates: Nov 1, 2006 to Jan 15, 2009
Delivery method: CM Agency
Bid Cost: GMP of $97 Million

ARCHITECTURE

Features:
- 244 Independent living units
- 43 Assisted living units
- 16 Skilled nursing units
- 10 Dementia units
- A theater room
- A swimming pool
- A tennis court
- Underground parking
- Roof gardens

Sustainable Elements:
- High-efficiency pluming utilities
- Low E glass
- High-efficiency HVAC equipments
- Plantings over the plaza
- A feature pond on the project's north side
- Low VOC coatings

Building Aesthetics:

The base of the building consist of cast stones, which gives it a more solid and rustic appearance than the rest of the building. The mid-portion of the building consist of brick veneer and light-beige stucco. The mansard roof has a darker colored metal shingles with a well defined soffit line.

There is uniformity both in the proportioning of the building's geometry and in the facades. Rhythm and harmony is well developed. Windows are all proportional and are evenly spaced apart. Keystones, dormers, lintels and wrought iron shutters are used to give dept to windows and doors.

PROJECT TEAM

Owner:
Ingleside Presbyterian Retirement Community
Architect and Landscape Architect:
Cochran, Stephenson & Donkervoet, Inc.
General Contractor:
Turner Construction Company
Construction Manager:
Turner-Konover

Structural Engineer:
Morabito Consultants, Inc.
Mech/Electrical Engineer:
Siegel, Rutherford, Bradstock & Ridgway, Inc.
Civil Engineer:
Loieder Soltesz Associates Inc.

STRUCTURE

Foundation:
- Spread footings and geopiers on soil with 5000 psf bearing capacity

Columns:
- 140 concrete reinforced columns (typical: 18" x 30")

Floor system:
- 8 inch two-way flat plate post-tension concrete slabs (normal weight concrete with f'c=5,000 psi)

Lateral system:
- Reinforced concrete shear walls

Framing:
- Roof - Light gauge metal framing (typical: 6" x 4" x 5/16" galvanized angles)
- Walls - 6" metal studs

MEP SYSTEMS

Mechanical:
- Water source heat pumps in each living unit
- Constant Volume Air System
- Cooling towers with plate and frame heat exchanger
- Gas-fired A/C Units on roof top
- Gas-fired forced draft hot water boilers

Electrical:
- Residential: 120/208V 3 phase 4 wire system
- Public Areas: 277/480V 3 phase 4 wire system
- Two Pepco 700kw transformers
- Diesel fuel back up generator

STEPHEN A. TAT

Structural Option 2008-2009

CPEP Website: http://www.engr.psu.edu/ae/thesis/portfolios/2009/dpt5001/

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1. Executive Summary

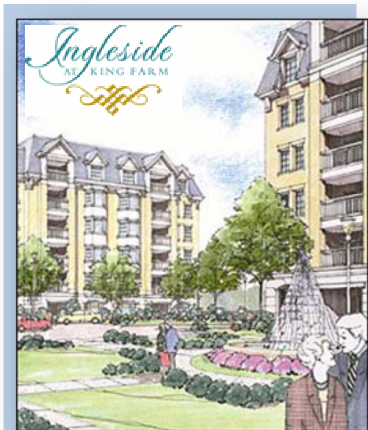
This final report summarizes the design of a prototype Continuous Care Retirement Community (CCRC) for construction on the west coast of the United States using the existing Ingleside at King Farm King design as a model. The prototype design will be relocated to a high seismic zone in California utilizing steel frame construction, slab on metal deck system, and special concentric braced frames.

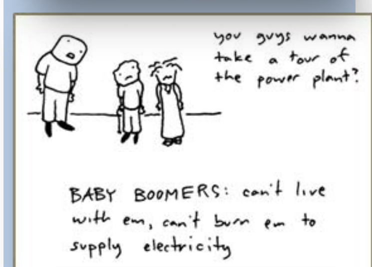
The prototype CCRC will have to be redesigned to resist the high seismicity of the west coast region. Not only will seismic activity affect the outcome of the design, but local codes, and the Southwestern U.S. climate will affect it as well. Both the State of California and Los Angeles City has their own design requirements. Such as energy consumption by new buildings, and amendments to ASCE-07 to design for a more conservative allowable building drift and seismic expansion joint separations. The amendments are to account for abnormally large earthquakes due to the cities proximity to blind thrust faults and soft basin soil resulting in a magnified earthquake due to direct shear impacts upon the buildings.

The first breadth study includes the research and analysis of implementing an extensive green roof and the usage of Autoclaved Aerated Concrete for precast architectural panels to appeal to California’s energy conservation codes. These two design decisions are integrated into the second breadth study focusing on the evaluation of the building facade's thermal and moisture resistance performance. The design parameters as a result of the breath studies are integrated into the structural depth study. Such as how the cladding system and reduced building weight affects the energy dissipation of the building during an earthquake, and how the loads from the roof gardens affects possibilities of soft stories and member sizes in a seismic analysis.

Better performance usually comes with a cost, however there are paybacks that out weights the dollar amount. In the case of retrofitting a building for seismic resistance, the reward could be the reduction in lives lost, medical costs, loss of tenants, loss of assets within the building, and loss of building functions. Other benefits include reduction in insurance premiums, increase in property value, and higher income from tenants.

Redesigning a prototype design of Ingleside at King Farm for Los Angeles, California will be costly due to the special requirements by codes to make the building safer during and right after a seismic event. Indirect damage includes fires caused by seismic activity, which can weaken the structural system and cause structural failures. In the case of extremely high seismic activity, such as the Northridge Earthquake in 1994 due to a combination of direct shear and poor soil conditions, retrofitting the building design and to resist seismicity can result in significant savings due to decrease in damages and delayed building functions, and more importantly, increasing the safety and survival rate of the occupants.





2. Introduction

2.1 Building Usage

Ingleside at King Farm is owned by the Ingleside Presbyterian Retirement Community and was designed by Cochran, Stephenson & Donkervoet, Inc. (CSD). The building was constructed under a guaranteed max price of $97 million, which covers construction only with a CM contract by general contractor Turner Construction Company of Baltimore, MD and construction manager Turner-Konover of Rockville, MD. Morabito Consultants, Inc. is serving as the engineering firm. Construction of the 103 feet, seven- story and 790,000 square footage post-tension concrete building began on November 1, 2006 and was completed in late March 2009.

The building site is located between a residential and commercial zone. The building itself is a mixed-use continuous care retirement center (CCRC) designed with several roof gardens, independent living units, assisted living units, and nursing units for the top seven floors. In addition, the first floor consist of many public servicing areas including but not limited to a theater, Olympic size swimming pool and a market place is the first floor. All the floor plans are identical with the exception of the first floor having an extended floor area for the swimming pool and market place.

2.2 Rising Demand for CCRC

The Baby Boom generation, born between 1946 and 1964, has just begun to reach retirement age. In 10 to 15 years, Baby Boomer seniors will comprise a large, unprecedented population seeking the amenities and lifestyle that continuing care retirement community (CCRC) living provides. According to Future Age magazine March 2009 issue, approximately 78 million baby boomers will reach retirement age, and will want to remain independent and productive, to live in the community and continue to contribute. Additionally, in difficult times people seek out family as a source of comfort and support. A CCRC is more than just four walls and a roof, it is a community – a large, extended family that looks out after their own.

During bad economic time, people often do nothing and try to wait out the financial crisis. Many seniors have decided that waiting is detrimental to their long-term well-being. However, one investment they should seriously consider for their health, wealth and quality of life is a fully refundable and guaranteed deposit in a continuing care retirement community (CCRC), which are regulated by State Government and the Department of Insurance. CCRCs offer service and housing packages that create an independent but secure lifestyle not achievable in regular housing, with immediate access to assisted living and skilled nursing in the event these services are needed.

The U.S. Census Bureau had estimated that 57.8 million Boomers will still be alive in 2030 between the ages 66 to 84, Over 4,000 CCRCs exists in the U.S. today. In a member survey conducted by the American Association of Retired Persons (AARP) in 2004, 37% of respondents voiced curiosity about life care systems such as CCRCs. According to MetLife Mature Market Institute, Boomers have $21 Trillion in spending power, and according to Money Magazine statistic in 2005, people aged 50 and over made up 12% of the U.S. population and many have desired and demanded a luxurious lifestyle (Fall 2007 Land Development, National Association of Home Builders). However, the supply of CCRCs falls short of the demand side of the Boomers.

3. Thesis Statement

Due to the large Baby Boomer Population, the demand for CCRCs is increasing faster than the amount of CCRCs being established. Thus, the Ingleside at King Farm CCRC design will be used as a model for a new prototype design for establishment on the west coast of the United States.

The prototype design for the west coast will be required to met seismic design criteria. The structural system selection will depend on the availability of type of building materials and labor/trades associated with the west region. The structural design will abide to required building codes for strength design, and serviceability requirements.

Other considerations include the design of an alternative building envelope to adapt to the region's climate, and utilizing emerging design principles such as sustainability. These two topics are usually affected by local codes, and will be addressed in the breadth studies.

4. Building Statistics

4.1 General Building Data

Location..701 King Farm Blvd. Rockville, MD 20852
Building Occupant Name……………..…. Elderly Residents and Nurses
Occupancy.. CCRC (Continuous Care Retirement Center)
Size.. 790,000 SF
Height.. 103 feet, 7 above grade, 1 below grade
Construction Dates.............................. Nov 1, 2006 to Jan 15, 2009
Delivery method.................................. CM Agent
Bid Cost.. GMP of $97 Million

4.2 Project Team

Owner
Ingleside Presbyterian Retirement Community
3050 Military Road NW, Washington, DC 20015

Architect & Landscape Architect
Cochran, Stephenson & Donkervoet, Inc.
323 West Camden Street, Suite 700, Baltimore, MD 21201

General Contractor
Turner Construction Company
250 West Pratt Street Suite 620, Baltimore, MD 21201

Construction Manager
Turner-Konover
1623 Piccard Dr. Unit A , Rockville, MD 20850

Structural Engineer
Morabito Consultants, Inc.
952 Ridgebrook Road Suite 1700, Sparks, MD 21152-9390

Civil Engineer
Loiederman Soltesz Associates Inc.
2 Research Place, Rockville, MD 20850

4.3 Architecture

Features:

- 244 Independent living units
- 43 Assisted living units
- 16 Skilled nursing units
- 10 Dementia units
- A theater room
- A swimming pool
- A tennis court
- Underground parking
- Roof gardens

Sustainable Elements:

- High-efficiency pluming
- Low E glass
- High-efficiency HVAC equip.
- Plantings over the plaza
- A feature pond
- Low VOC coatings

Building Aesthetics:

The base of the building consist of cast stones, which gives it a more solid and rustic appearance than the rest of the building. The mid-portion of the building consist of brick veneer from the 2nd to 5th floor, and light-beige stucco for the 6th floor. The 7th floor consist of a mansard roof construction with metal shingles that gives it a well defined soffit line.

There is rhythm and harmony in the proportioning of the building's geometry and the facades. The appearance echoes that of the surround residential buildings. Windows are all proportional and are evenly spaced apart. Keystones, dormers, lintels and wrought iron shutters are used to give dept to windows and doors.

Building Envelope:

The building envelope consists of three primary wall assemblies. The exterior façade at the base consist of 16x24 cast stones. It is followed by an air space, ½" sheathing, masonry veneer ties at 16" O.C., 6" steel studs at 16" O.C., 6" batt insulation at an R value of 19 and 5/8" foil face gypsum board.

The mid section of the building (2nd to 5th floor) is similar to the base section except that masonry brick is used in place of the cast stones. On the 6th floor, the exterior veneer brick is replaced by a light-beige stucco with a reinforcing mesh behind it. The 7th floor building envelope consist of a sloped roof assemble (mansard roof style) characterized by dark colored metal shingles on plywood roof sheathing and 4" metal stud framing.

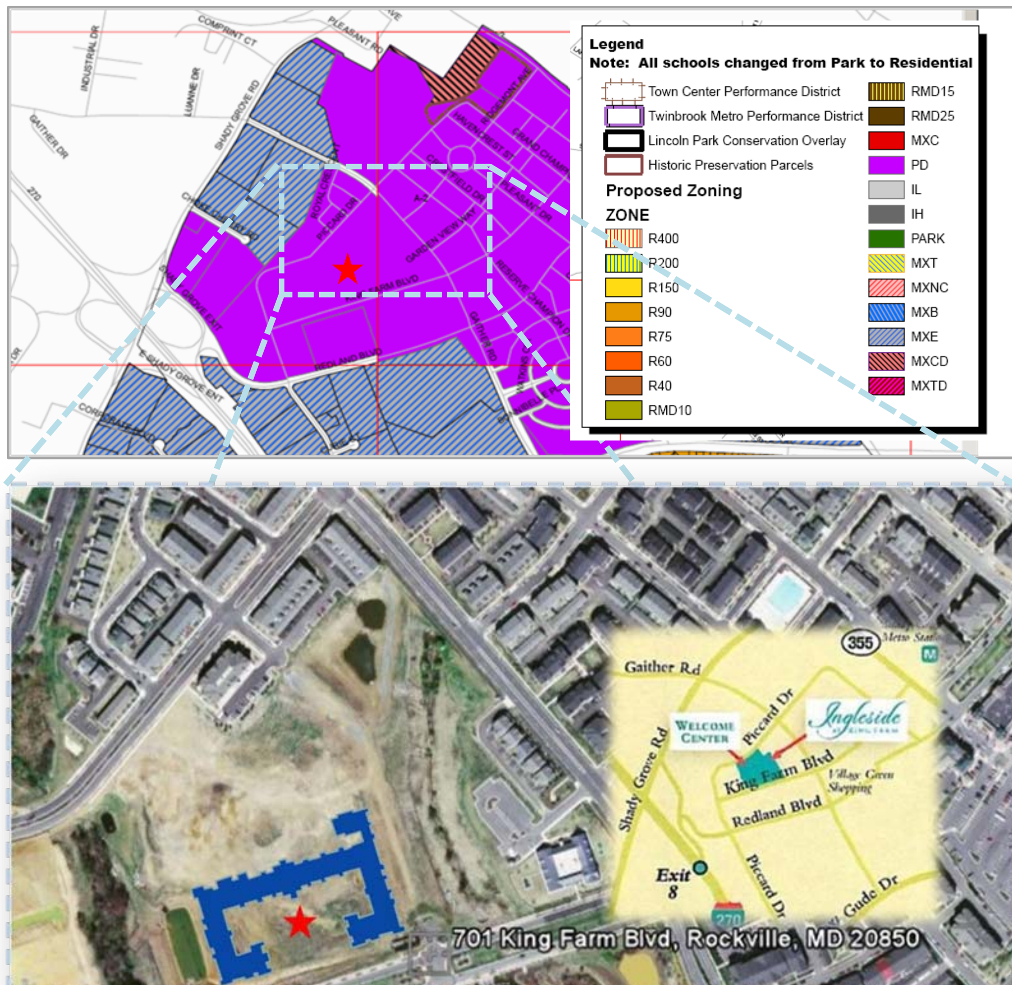
The roof membrane is a 3" rigid insulation on 1 ½" x 20 gauge galvanized metal deck supported by either 26 k12 or 28 k12 joists depending on the roof loads. There are also a low roof areas (mainly for the roof gardens and penthouse) with an assembly consisting of 8" post tension slab with a membrane roof water proofing system.

Major National Model Codes:

- 2003 International Building Code
- 2003 International Residential Code (with amendments)
- 1997 International Plumbing Code
- 1996 International Mechanical Code
- 1996 National Electrical NFPA-1
- 2003 NFPA 1, 101, 13, 72

Zoning:

Planned development zone - The design and construction is in compliance with Chapter 5 of the Rockville City Code.



Major Historical Requirements of Building: None

Due to the building site's proximately of 0.30 miles from King Farm Farmstead Park Historic District, the architectural design is rather conservative and is designed in context with the existing buildings in the community, but with no historical requirements. There is no unique style to describe its architecture. Although it resembles the Victorian style it follows (by choice and not as a requirement) the Architectural Design Guidelines for the Exterior Rehabilitation of Buildings in Rockville's Historic Districts adopted in 1977.

4.4 Primary Engineering Systems

Construction:

The developer Penrose Group hired Turner Construction and Konover in a joint venture contract to deliver the Ingleside at King Farm project with a CM Agent delivery method. The goal of the project was to deliver affordable living to senior citizens in Rockville. Penrose Group had helped finance this project. Construction of the 790,000 square foot complex began in November 2006. The complex is a mixed use building: Type I construction. It will consist of living units, office spaces, a multi use theater space, Olympic size swimming pool (under a different contract), a market place, and various of public spaces for the seniors.

Due to the enormous size of the complex, there are four expansion joints in the building dividing it into five sections. Dividing the building into sections help increase the constructability and site logistic planning of the project. It helps decrease the lag time of the construction.

Structural:

The primary structural system present in Ingleside at King Farm is a two-way post-tension flat plate system. Slab thickness for all the floors are 8 inches with 7-wire strands 1/2 inch diameter tendons. All post-tension floor slabs utilizes normal weight concrete with f'c of 4,500 psi; except for the structural floor slab holding up the court yard that is a two-way post-tension flat plate with 10 inch thick drop panels and a 12 inch thick slab utilizing normal weight concrete with an f'c of 6,000 psi. The bay sizes (being a two-way system) range from 20 feet to 30 feet. The sub level of the building is mainly used as parking garage and houses most of the building's mechanical rooms. The loads from above are transferred down by 30" x 18" reinforced concrete columns to spread footings 2 feet below grade on soil with a 5000 psf bearing capacity. Slab on grade slabs, which are 5 inches thick and has an f'c of 4,000 psi are reinforced with 6" x 6" wielded wire fabric over a vapor barrier and a 4 inch porous fill.

Vertical supporting elements consist of reinforced concrete columns, tubular columns and W shape. The column grid for the building is irregular with column offsets within 10 percent of its span. There are over 140 reinforced concrete columns (typical size 18" x 30") each with 10 #8 reinforcing bars rated at 60 ksi located on the sub grade level to the 6th floor. The 6" x 6" x 3/8" tube columns are located on the 1st floor at where the market place is, which only support the roof loads from above. The steel columns on the 7th floor supporting the roof are typically W 8 x 31. Because of these steel columns on the 7th floor, which most of them are offset from the alignment of the concrete columns on the 6th floor, 10 inch thick drop panels are required on the 6th floor where ever the steel columns on the 7th floor are offset.

There are eleven ordinary reinforced concrete shear walls throughout the building to resist lateral loads. They utilize normal weight concrete with an f'c of 5,000 psi, and are located symmetrically about a line of symmetry (North-South) through the center of the building. These shear walls run from the sub level to the 6th floor. The lateral resistance for the 7th floor is provided by moment frames with W 8 x 31 columns and typically W 12 x 14 girders. These moment frames utilizes a seated frame connection as specified in Table VIII in the AISC (13th edition).

The framing system for the exterior walls and partition walls are 6" steel studs at 16" O.C. for floors 1 to 6. The 7th floor, which utilizes a sloped roof assembly, is supported by light gauge metal framing of varying sizes and W 8 x 31 steel columns. Flat roof areas; typically for green gardens or pent houses uses 8 inch post-tension slabs. Flat roof areas that have no particular usage utilize a metal deck and 26 k 12 or 28 k 12 joist system.

Lighting:

The living units utilize a mixture of incandescent lighting and fluorescent lighting. Bathrooms and hallways use down-light incandescent and compact fluorescents. Living and dining rooms consists of incandescent chandeliers. Walk in closets uses long fluorescent acrylics and the balcony areas use wall mounted incandescent. All nursing units use either long fluorescent or compact fluorescent lighting. The hallways connecting the living units are lit by down-light compact fluorescents.

Interior public areas such as office spaces and office corridors utilize 2' x 2' or 2' x 4' fluorescent lighting. Compact fluorescent lighting is used in the library, multi-purpose rooms and corridors connecting these social spaces. Incandescent lighting is used for the lobby spaces and roof gardens. As for exterior lighting, high intensity discharge (HID) lighting is utilized in the canopy area, walkways, site, landscaping, and parking garage. For exterior exits, compact fluorescents are used.

All emergency lighting uses LED with power provided by the emergency power system. All lighting for living units and social areas runs on 120 V while the exterior HID lighting and office lighting runs on 227 V.

Electrical:

Electrical service provided by the local utility company PEPCO enters the building from two locations; one on the west side of the building and one on the east side. The service voltages are transformed down to a 480Y/277V secondary service that is a 3-phase 4 wires system. Each service then feeds to a 4000 Amp main switchboard located in an electrical room in the sub grade level. There are 3 electrical rooms on the northwest end of the sub grade level serving the public spaces and office spaces, two on the second level (one in each wing of the building), and two on the fifth level. There are 500 KV transformers in each electrical room on the floors above grade converting the voltage down to a 208Y/120 V service. The 208Y/120 service is used for receptacle loads, incandescent lighting, and much of the living units. The main emergency power system for the building is a 750 KW diesel generator. Power is distributed from the generator to emergency lighting, fire pumps, elevators, door controls with an automatic transfer switch.

The minimum branch circuit wires for 20 amp circuits are #12 AWG. Circuit length up to 75 feet uses a # 12 AWG wiring for both 120 V and 277 V. For Circuit length between 75 feet and 150 feet uses a # 10 AWG wiring for 120 V and a # 12 wiring for 277 V. For over 150 feet of circuit length, a # 8 and # 10 AWG wiring is used for the 120 V and 277 V respectively.

Mechanical:

A majority of the mechanical and boiler rooms are located in the northwest end of the sub level garage. A positive pressure in the corridors is maintained by 15 Gas-Fired Rooftop A/C Units to keep a constant volume air system throughout the living units and prevent cross-contamination between them. The rooftop units have at least 15 tons of nominal cooling capacity (over 4000 cfm). Economizer units include air dampers, air filters, barometric relief controls and system controls capable of introducing up to 100% outdoor air.

A water source heat pump unit (with 175 PSIG and 3500 RPM) is fitted into each living units, offices and storage areas. These heat pumps are linked together in a heat pump loop served by 2 Induced Draft Cooling Towers located on the roof top, which are linked to 2 Plate and Frame Heat Exchangers (for cooling) located in the garage. The hot water source for these heat pump units are provided by 10 Gas-Fired Forced Draft Boilers in boiler room. For additional heat when needed, an electric baseboard is used. There are 12 Electric Heaters to provide heat for public areas. Memory assist living units are served by a Ductless Split System. The sub level garage is heated with small individually controlled electric Unit-Heaters and the exhaust gases are removed with large exhaust fans (5000 cfm) on the north side of the building.

The minimum branch circuit wires for 20 amp circuits are #12 AWG. Circuit length up to 75 feet uses a # 12 AWG wiring for both 120 V and 277 V. For Circuit length between 75 feet and 150 feet uses a # 10 AWG wiring for 120 V and a # 12 wiring for 277 V. For over 150 feet of circuit length, a # 8 and # 10 AWG wiring is used for the 120 V and 277 V respectively.

4.5 Additional Engineering Systems

Fire Protection:

Ingleside at King Farm utilizes a wet and dry automatic sprinkler system. The wet system is typically used throughout the living units and public areas while the dry system is used in the garage due to freezing conditions in the winter. The sprinkler head that is utilized is a chrome pendent type for areas with a suspended ceiling, and a standard upright brass for areas without a suspended ceiling. The main fire alarm control panel is located in the water service room on the garage level, and is linked to fire alarm terminal cabinets on each floor. The terminal cabinets are linked to smoke detectors throughout each floor. A combination of fire and smoke dampers are also used in certain areas of the building.

National Fire Protection Association Pamphlet 101 (NFPA 220 – 2003) has determined the fire construction type as Type I – 322 constructions. This means that both the exterior walls and structural frame are 3-hour rated, and the floor construction has to be at least 2-hour rated. Due to the steel frame construction on the 7th floor, asbestos is used as the fire proofing material for the steel frames.

Transportation:

Transportation throughout the building is handled by six OTIS Gen2 Machine Roomless elevations. They are designed to be cost, energy, installation, and space efficient. Five of the six elevators are designated for public usages while the remaining one is for service usage for housing keeping located at the northwest section of the building. The five public elevators do not require a machine room adjacent to it. However, they still require a remote space for power distribution and controls. There are four main fire exits throughout the building and two main entrances to the building.

Telecommunications:

All the telecommunications, phone and internet (CAT5E) and cable TV (CATV) services provided by the CATV Company and Verizon enter by the east side of the building through 4" conduits. The lines are run into the telecommunication office room on the first floor, where the terminal boards are located in. Each living unit is provided with hard-wire internet connection, telephone and TV.

Security System:

The surveillance system is a closed-circuit television (CCTV) system consisting of video cameras (surface mounted), digital video recorders, monitors, interface hardware, and cabling. A majority are installed in the sub level garage. A series of electric and magnetic locks and card readers are installed throughout the building for office usage.

FOOTING SCHEDULE

MARK	SIZE	REINFORCING
F1	4'-6" x 4'-6" x 14"	5#5 E.W. BOTTOM
F2	5'-0" x 5'-0" x 15"	4#6 E.W. BOTTOM
F3	5'-6" x 5'-6" x 17"	5#6 E.W. BOTTOM
F4/F4T	6'-0" x 6'-0" x 18"	6#6 E.W. BOTTOM *
F5/F5T	6'-6" x 6'-6" x 20"	5#7 E.W. BOTTOM *
F6	7'-0" x 7'-0" x 21"	6#7 E.W. BOTTOM
F7/F7T	7'-6" x 7'-6" x 23"	7#7 E.W. BOTTOM *
F8/F8T	8'-0" x 8'-0" x 24"	6#8 E.W. BOTTOM *
F9/F9T	8'-6" x 8'-6" x 26"	6#8 E.W. BOTTOM *
F10/F10T	9'-0" x 9'-0" x 27"	7#8 E.W. BOTTOM *
F11/F11T	9'-6" x 9'-6" x 28"	8#8 E.W. BOTTOM *
F12/F12T	10'-0" x 10'-0" x 29"	9#8 E.W. BOTTOM *
F13/F13T	10'-6" x 10'-6" x 30"	10#8 E.W. BOTTOM *
F14T	11'-0" x 11'-0" x 32"	11#8 E.W. TOP AND BOTTOM
F15/F15T	11'-6" x 11'-6" x 33"	9#9 E.W. BOTTOM *
F16/F16T	12'-0" x 12'-0" x 34"	10#9 E.W. BOTTOM *
F17T	12'-6" x 12'-6" x 35"	11#9 E.W. TOP AND BOTTOM
F18T	13'-0" x 13'-0" x 36"	12#9 E.W. TOP AND BOTTOM
F19T	13'-6" x 13'-6" x 37"	10#10 E.W. TOP AND BOTTOM
F20T	12'-0" x 15'-0" x 34"	12#10 S.W. 10#9 L.W. TOP AND BOTTOM
F21T	15'-0" x 15'-0" x 41"	12#10 E.W. TOP AND BOTTOM
F22T	9'-6" x 12'-6" x 28"	11#9 S.W. 8#8 L.W. TOP AND BOTTOM
F23T	15'-0" x 32'-0" x 40"	28#10 S.W. 13#10 L.W. TOP AND BOTTOM
F24T	20'-0" x 28'-0" x 30"	22#9 S.W. 16#9 L.W. TOP AND BOTTOM
F25T	20'-0" x 32'-0" x 32"	26#9 S.W. 16#9 L.W. TOP AND BOTTOM
F26T	25'-0" x 25'-0" x 48"	22#11 E.W. TOP AND BOTTOM
F27T	26'-0" x 34'-0" x 41"	28#10 S.W. 22#10 L.W. TOP AND BOTTOM
F28T	35'-0" x 35'-0" x 42"	30#10 E.W. TOP AND BOTTOM
F29T	37'-0" x 37'-0" x 49"	31#11 E.W. TOP AND BOTTOM

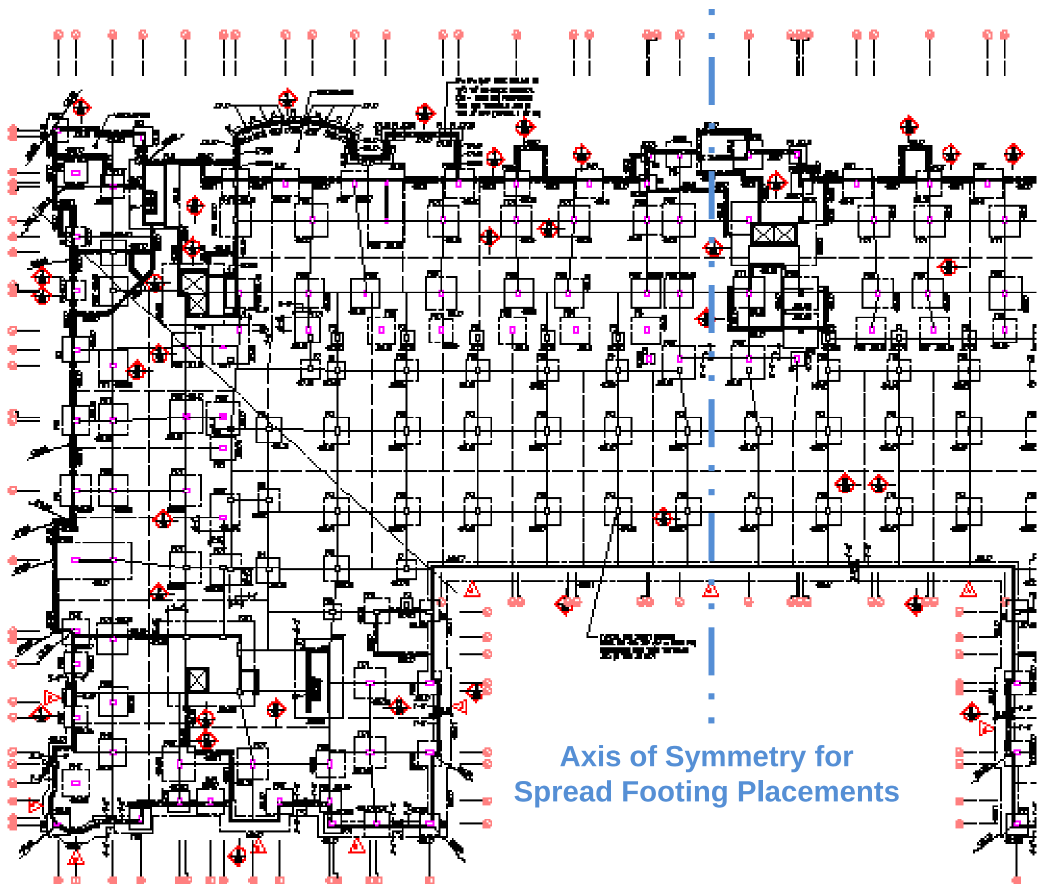
5.1 Foundations

The sub level of the building is mainly used as a parking garage and also houses most of the building's mechanical rooms. The loads from above are transferred down by 30" x 18" reinforced concrete columns with 10 #8 bars to spread footings of various sizes.

Approximately 40% of the footings are sized 10'- 6" x 10'- 6" x 30" with 10 # 8 E. W. reinforcing bars at the bottom; 20% of the footings are sized 12' x 12' x 34" with 10 # 9 E. W. reinforcing bars at the bottom; about another 20% of the footings sized 12' - 6" x 12' - 6" x 35" with 11 # 9 E. W. reinforcing bars located at both the top and bottom; and the remaining percentage of footages are of various sizes with the smallest being 4' - 6 x 4' - 6" x 14" with 5#5 E.W. bottom reinforcing bars and the largest spread footing being 37' x 37' x 49" with 31 # 11 E.W. reinforcing bars top and bottom.

All shear walls are designed with much larger spread footings typically 20' x 28' x 30" with 22 # 9 S.W. and 16 # 9 L.W. top and bottom.

Beneath the spread footings is 3 feet of compact fill and then soil with a bearing capacity of 50 ksf. The 30" x 18" reinforced columns extends all the way to either the 6th or 7th floor. The structural slab in the foundation and sub level parking garage is a 5" concrete slab on grade reinforced with 6" x 6" W2.9 / W2.9 welded wire fabric over a vapor barrier and a 4" porous fill. It utilizes standard weight concrete with a 28 day minimum compressive strength of 4000 psi.



Columns Viewed From Exterior

5.2 Gravity System

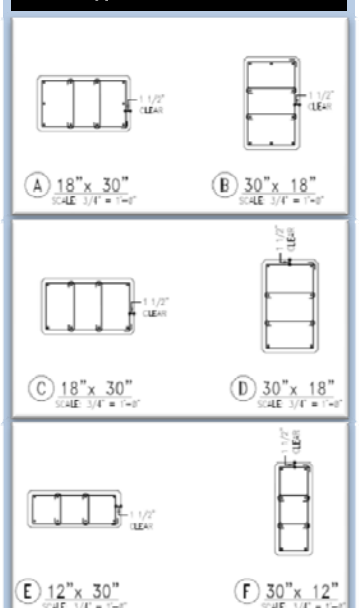
The building contains over 140 reinforced columns, which are either 18" x 30" or 12" x 30". Due to the building's irregular column grid, some columns are miss-counted for in the column schedule. These reinforced concrete columns extend from the sub level to the 6th floor.

All 7th floor columns are W 8 x 31 steel rolled. There are approximately 152 of these steel columns and 33 of them are offset from the concrete reinforced concrete columns below. Thus, 5' x 5' x 10" drop panels are present on the 6th floor to aid with the load transfer and punching shear resistance for the offset columns.

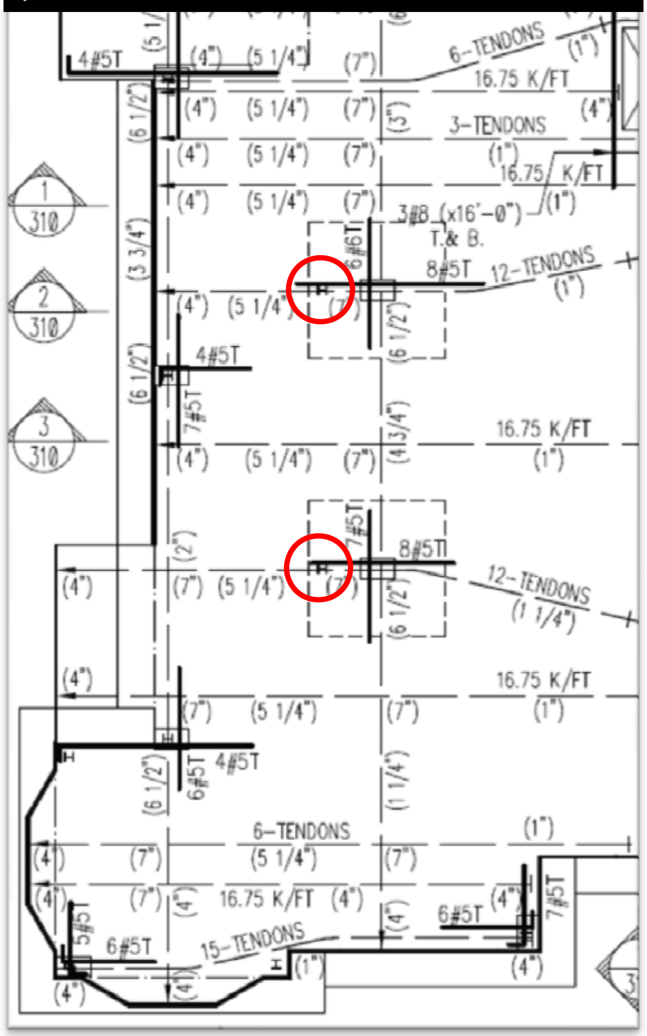
The column schedule also does not account for the 6" x 6" x 3/8" steel tubular columns that are located in section two of the building where a majority of the public areas are found. These HSS columns support the gravity loads of areas whose roof line is at the first floor and second floor level.

Columns Viewed From Interior

Typical Column Sizes



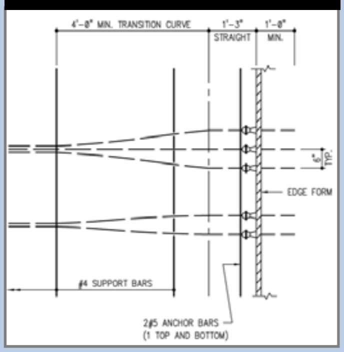
7th Floor structural drawing showing steel columns offset from reinforced concrete column from the 6th floor and the usage of drop panels for the 7th floor.



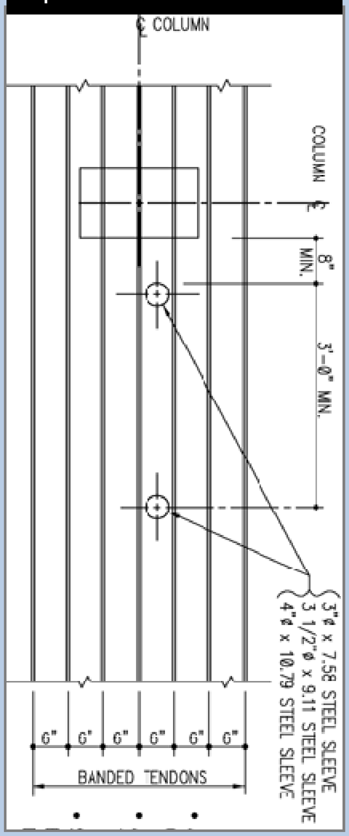
Photos showing the transition from reinforced concrete gravity system at the bottom to steel columns on the 7th floor.

Banded and Uniformly Spaced Tendons

Typ. Plan Detail-Flair at Tendon Ends



Pipe Sleeves in Concrete Slab

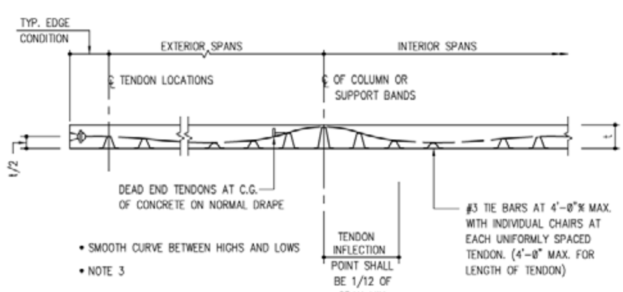


5.3 Two-way Post-tension Flat Plate System

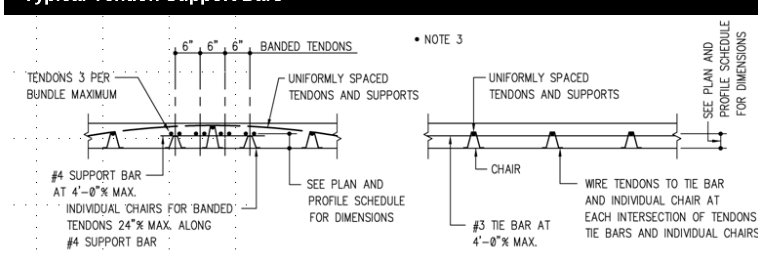
Ingleside at King Farm's primary structural system is a two-way flat plate post-tension concrete structure with 270 ksi unbonded ½ diameter 7 wire tendons. The post-tension concrete slabs are 8 inches thick for typical floors with a compressive strength of 4500 psi. All Concrete used in this building's construction is normal weight. There are no drop panels or beams supporting these typical slabs. The only drop panels in the building are found on the sub level columns holding up the 12 inch thick slab (f'c=6000 psi) that is supporting the weight of the court yard, and the 6th floor columns supporting the 7th floor loads due to the offset W 8 x 31 wide flange columns found on the 7th floor. All the drop panels are 5' x 5' x 10".

Due to the irregular column gird of the building, bays range from 15 feet to 29.5 feet. For the analysis of alternative floor systems, a bay area of 30' x 30' is utilized for a more conservative design, which is the typical interior bay area for the building

Typical Tendon Profile

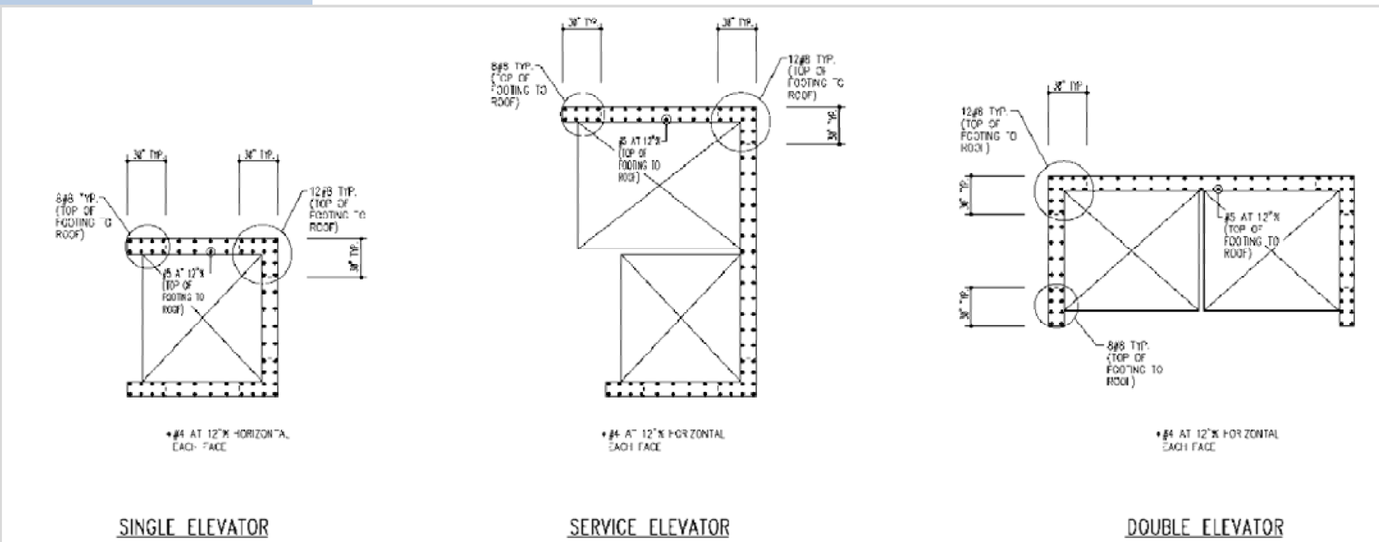


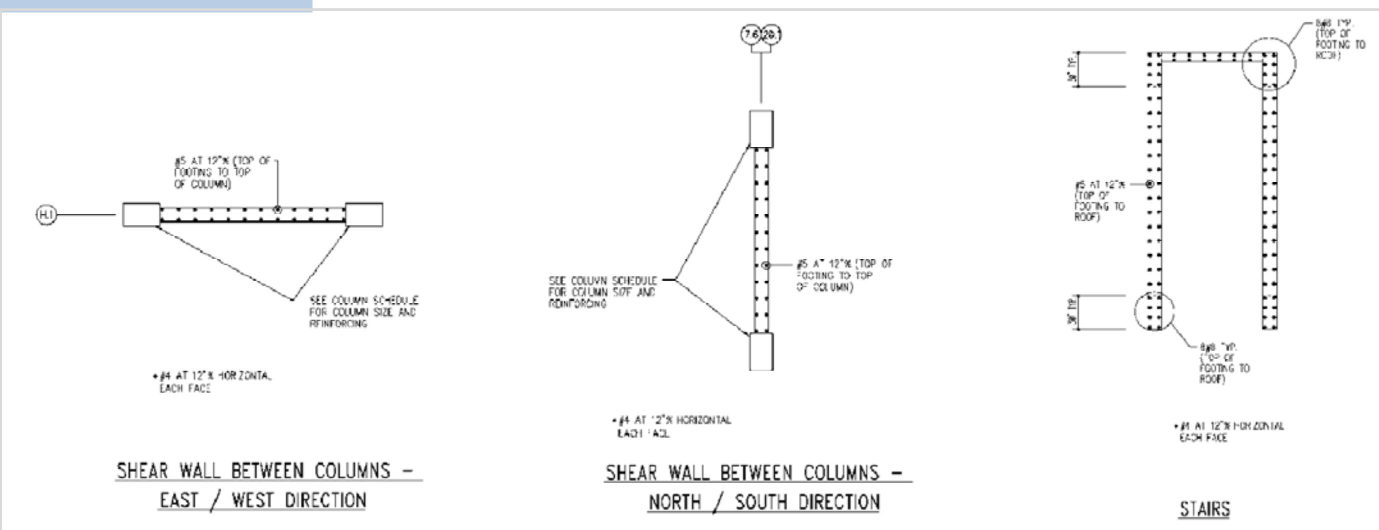
Typical Tendon Support Bars



5.4 Shear Walls

Ingleside at King Farm has eleven shear walls to resist lateral loads from the sub level up to the 7th floor. Seven of the walls are ordinary reinforced concrete shear walls located at stairwells and elevator shafts with #4 horizontal reinforcing bars and #8 vertical reinforcing bars. Typical spacing of these bars is 12 inches. All these walls have a compressive strength of 5000 psi. The remaining four reinforced concrete shear walls have boundary elements and are 15 feet in length; two in east/west direction and two in north/south direction. Spacing of vertical and horizontal reinforcements is 30 inches and 12 inches respectively. Typical clear cover is 1 ½ inches for the reinforcements.

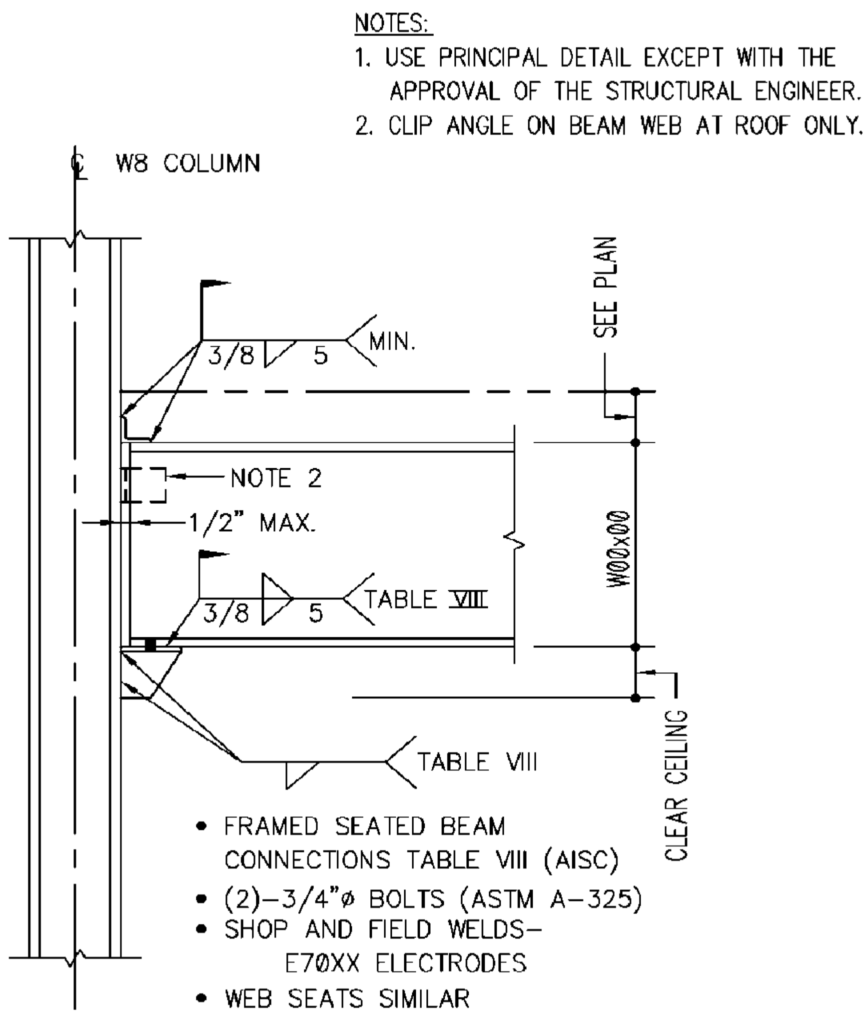




Photos showing the transition from reinforced concrete gravity system at the bottom to steel columns on the 7th floor.

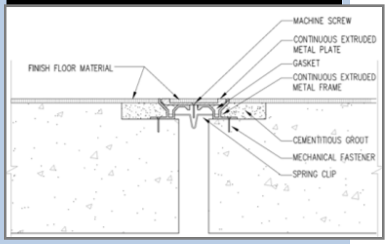
5.5 Moment Frames

On the 7th floor, in addition to the shear walls, there are also (welded/bolted) moment connections to resist the lateral loads. Based on lateral load analysis in technical report one, it was discovered that the loads were largest at the 7th floor roof line. Thus, these moment connections (framed seated beam connection) justify the high wind loads that were calculated in technical report one.

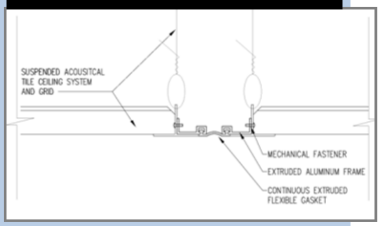


GIRDER TO COLUMN CONNECTIONS
PRINCIPAL DETAIL

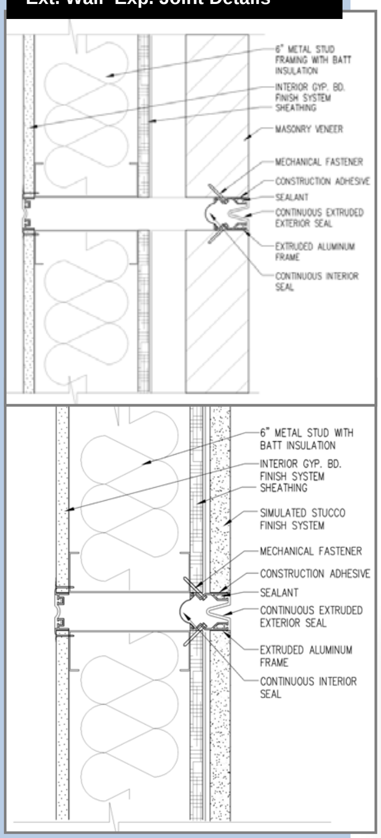
Typ. Floor Exp. Joint Detail



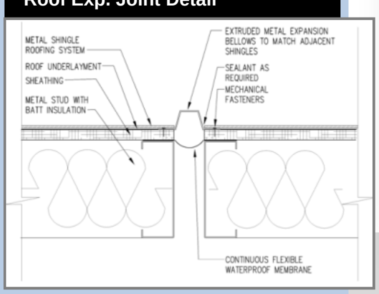
Typ. Ceiling Exp. Joint Detail



Ext. Wall Exp. Joint Details



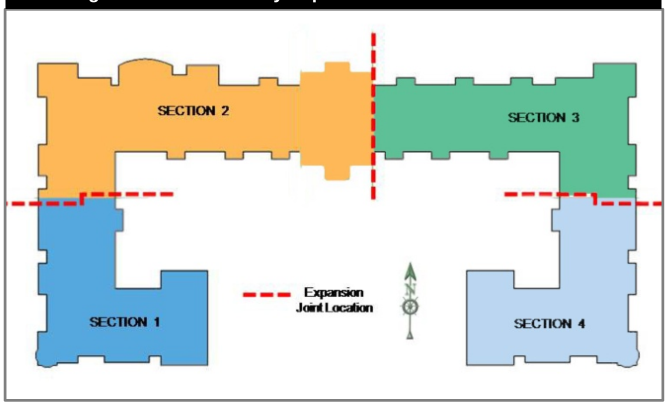
Roof Exp. Joint Detail



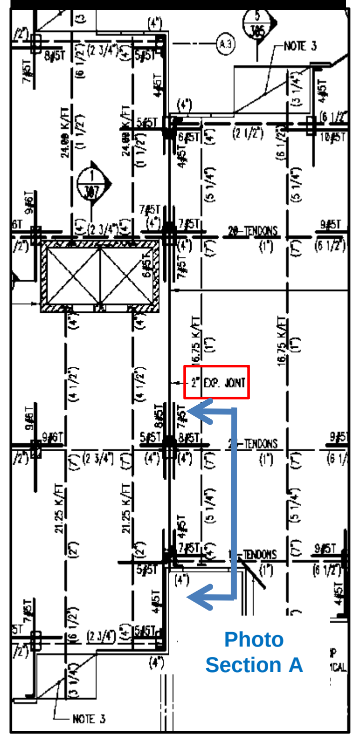
5.6 Expansion Joints

There are three true 2-inch expansion joints built into the building. The primary reason for these expansion joints is to reduce pre-stress losses in the tendons due to the shortening of the concrete slab caused by shrinkage or cooling, which will induce cracks around restraining boundaries (such as walls and beams).. Another reason is for better constructability and the utilization time for faster construction. While one section of the placed slab is left to cure, another section can be worked on. Where there exist a 2" true expansion joint in the building, there is a row of double 12" x 30" columns as oppose to the typical 24" x 30" columns on each side of the joint.

Building Sections Created by Expansion Joints



Exp. Joint Between Sec. 2 & 3



Exp. Joint Between Sec. 3 & 4

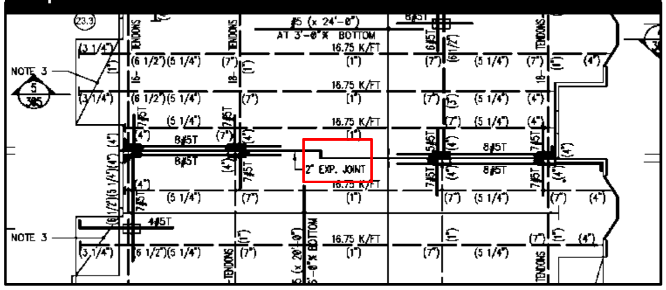
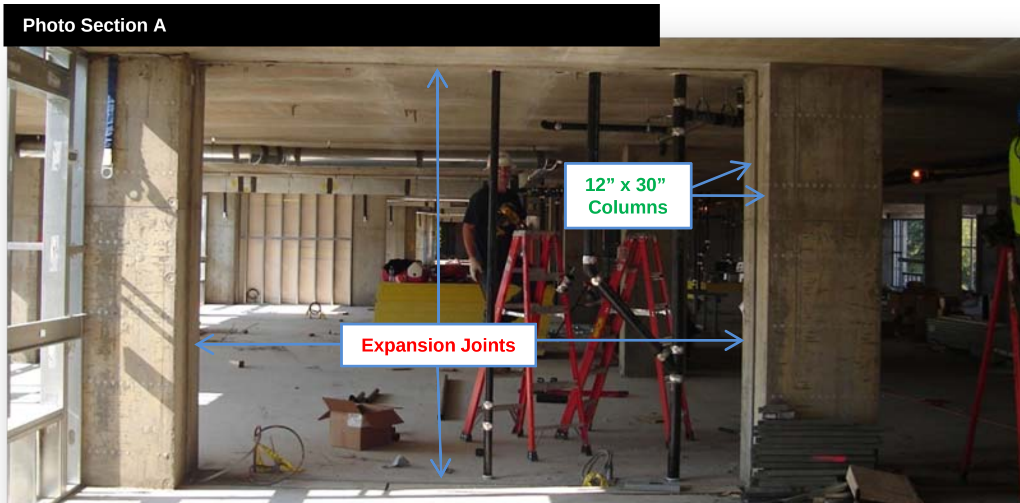
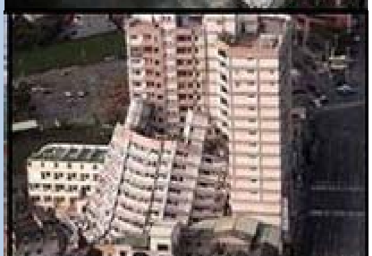


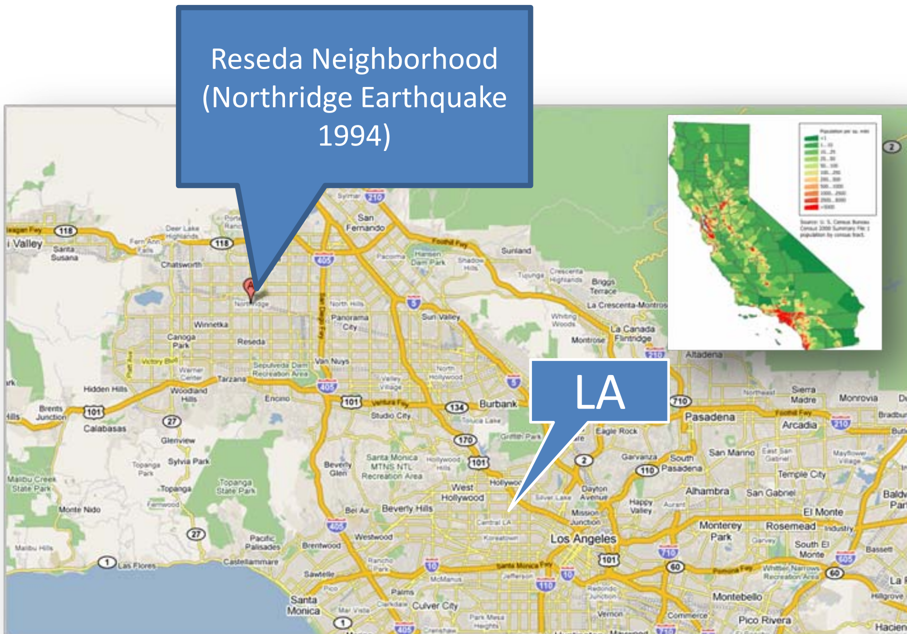
Photo Section A





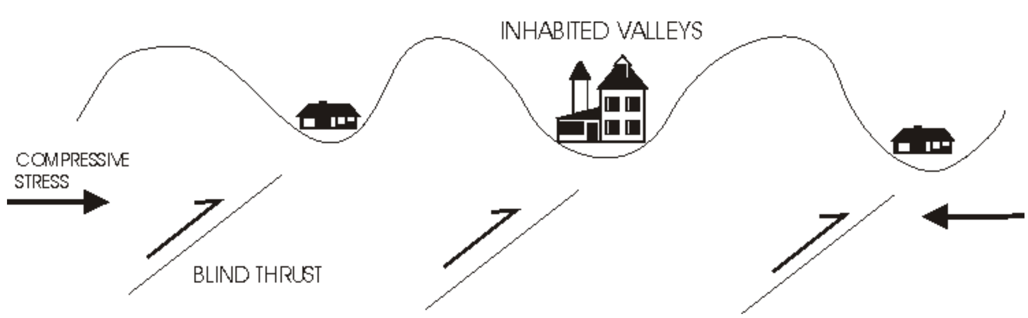
6. Site Selection

The site's specifications that I propose to have my prototype designed to is Los Angeles, California. The reason being is due to the high seismic activity on the west coast. The Northridge Earthquake occurred on Jan. 17, 1994 in L.A. lasted for about 20 seconds with a moment magnitude of 6.7 and did $20 billion in damage.



Blind thrust faults exist near tectonic plate margins, in the broad disturbance zone. They form when a section of the earth's crust is under high compressive stresses, due to plate margin collision, or the general geometry of how the plates are sliding past each other.

Although usually of magnitude 6 to 7 compared to the largest magnitude 9 earthquakes of recent times, it was especially destructive because the seismic waves are highly directed, and the soft basin soil of the valley can amplify the ground motions tenfold or more



7. Proposed Solutions and Tasks

7.1 Breadth 1: Green Design

- Implement green building materials into the new prototype (Autoclaved Aerated Concrete)
- Use light weight Autoclaved Aerated Concrete panels (a green building material) to reduce building weight, which in turn reduce the base shear of the building, but still conserve the aesthetics of the original cavity wall design.
- Green Roof Design for Ingleside at King Farm

7.2 Breadth 2: Building Envelope Redesign

- Redesign the building envelope to adjust to the climates in Los Angeles, California.
- Utilize a cladding system for the entire façade to reduce building interfaces of the existing building design.
- Use light weight Autoclaved Aerated Concrete panels as the material will provide excellent thermal insulation.
- Address structural integrity and code compliances such as cladding issues.
- Analyze thermal and moisture permeability of the building façade.
- Perform cost analysis.

7.3 Structural Depth

- Relocate building site to Los Angeles, California.
- Relocate Seismic Expansion joints to redefine the shape of the building sections/wings to remedy differential vibrations between the sections by creating more symmetrical building sections with uniform mass and stiffness distribution so that the behave in a predictable manner.
- Redefine the center of rigidity and mass of each building sections to reduce torsion by locating seismic-resisting elements at the extremity of the wings or tie building sections together at lines of stress concentration.
- Redesign structural system from reinforced concrete to steel.
- Re-align gravity elements (columns) to create a more uniform grid.
- Replace post-tension slabs with composite steel formed deck.
- Redesign lateral system with special concentric braced frames (SCBF), moment Frames utilizing reduced beam section (RBS) connections based on the Northridge earthquake 1994, and a combination of outriggers and belt truss.
- Establish width of seismic expansion joints to allow for the estimated inelastic deflection of adjacent wings to prevent pounding.
- Utilize the extra height from the 7th floor and Roof Screen, and retrofit the existing steel trusses as a Rooftop Tuned Mass Damper Frame. Isolate the floor containing the mechanical units to create the suspended pendulous mass that will increase the fundamental period, which will result in a decrease in seismic acceleration response of the building section.

8. Design Criteria and Goals

8.1 Codes

Codes, Standards, and Guides	
Codes and Standards in Original Design	**Codes and Standards used for Prototype Design**
International Building Code 2003	International Building Code 2006
ASCE 7-98: Minimum Design Loads For Buildings and other Structures.	ASCE 7-05: Minimum Design Loads For Buildings and other Structures.
Rockville, MD City Codes: Local amendments.	American Institute of Steel Construction (AISC) 13th Edition
	AISC Seismic Design Manual
	AISC –LRFD 1999, Load and Resistance Factor Design Specification for Structural Steel Buildings
	Vulcraft Steel Roof and Deck Catalog
	2007 California Building Code Section 1614
	ACI 318-08 *Building Code Requirements for Structural Concrete*
	PCI Design Handbook - *Precast and Prestressed Concrete*
	Architectural Precast Concrete (2nd ed.)
	IBC 2006 *Structural/Seismic Design Manual: Building Design Examples for Steel and Concrete*.

8.2 Material Properties

Material Strength Summary in Existing Structure	
Structural Steel	
Wide Flange Shapes	Fy=50 ksi
Hollow Structural Steel (HSS)	Fy=46 ksi
Anchor Rods	Fy=55 ksi
Channels	Fy=36 ksi
Angles	Fy=36 ksi
Concrete	
Structural Slab Supporting Court Yard	F'_c = 6000 psi, Normal wt.
Slab on Grade/Foundation	F'_c = 4000 psi, Normal wt.
Floor Slab	F'_c = 4500 psi, Normal wt.
Cast-in-place Columns	F'_c = 5000 psi, Normal wt.
Cast-in-place Walls	F'_c = 5000 psi, Normal wt.
Shear Walls	F'_c = 5000 psi, Normal wt.
Reinforcements	
Deformed Bars	ASTM A615, Fy=60 ksi
Welded Wire Fabric	ASTM A18, Fy=70 ksi
Post-Tension Tendons	ASTM A-416-74, 270 ksi

Material Strength Summary in Prototype Design	
Structural Steel	
Wide Flange Shapes	ASTM A572, Grade 50
Hollow Structural Steel (HSS)	Fy=36 ksi
Metal Decking 3.5" composite deck	Fy = 40 ksi, 18 gage

8.3 Design Loads

Ground Floor System Loads

			Spaces		
			Residential	Public	Façade
Load Type	Material / Usage	Reference	Load	Load	Load
Dead Load	Normal Weight Concrete	ACI 318 - 08	150 pcf		-
	Partition Walls	WDG	15 psf		-
	Miscellaneous (M/E/P)	ACI 318 - 08	10 psf		-
	Cold-formed, light gauge steel stud walls with insulation and 5/8" gypsum board	WDG	-	-	5 psf
	6" Precast Concrete Panels (Autoclaved Aerated Concrete - AAC)	MSJC	-	-	34 pcf
Live Load	Public Corridors/Theater/or Retail Spaces	ASCE 7 - 05	-	100 psf	-
	Living Units	ASCE 7 - 05	40 psf		-

Typical Floor System Loads

			Spaces		
			Residential	Public	Façade
Load Type	Material / Usage	Reference	Load	Load	Load
Dead Load	Light Weight Concrete	ACI 318 - 08	110 pcf (30 psf - 3.25" above flute)		-
	Steel Deck	ASCE 7 - 05	3 psf		-
	Partition Walls	WDG	15 psf		-
	Miscellaneous (M/E/P)	ASCE 7 - 05	10 psf		-
	Cold-formed, light gauge steel stud walls with insulation and 5/8" gypsum board	WDG	-	-	5 psf
	6" Precast Concrete Panels (Autoclaved Aerated Concrete - AAC)	MSJC	-	-	34 pcf
Live Load	Corridors/Theater/or Retail Spaces	ASCE 7 - 05	-	100 psf	-
	Living Units	ASCE 7 - 05	40 psf	-	-

Roof System Loads

			Type	
			Non-accessible	Accessible
Load Type	Material / Usage	Reference	Load	Load
Dead Load	Light Weight Concrete	ACI 318 - 08	110 pcf (30 psf - 3.25" above flute)	
	Steel Deck	ASCE 7 - 05	3 psf	
	Cold-formed, light gauge steel frame	AISC 13th ed.	by shape (Self weight)	
	Insulation, and waterproofing membrane	AISC 13th ed.	8 psf	
	Metal Shingles		3 psf	-
	Roof Garden (wet)		50 psf	
Live Load	Ordinary flat, pitch, and curved roofs	ASCE 7 - 05	20 psf	-
	Roof Garden	ASCE 7 - 05	-	100

8.4 Deflection Criteria

Deflection Type	Minimum Criteria
Live Load Deflection	L/360
Total Deflection Limit	L/240
Construction Load Deflection	L/360

8.5 Load Combinations

These load combinations are based on LRFD design method used in generating the computer model analysis. A few of the equations are modified combinations per ASCE 7-05 and AISC 341-05. The Seismic Design Category analyzed was Category D.

Adjusted per Section 12.4.2.3

1) $1.4(D + F)$
2) $1.2(D + F + T) + 1.6(L + H) + 0.5(Lr \text{ or } S \text{ or } R)$
3) $1.2D + 1.6(Lr \text{ or } S \text{ or } R) + (L \text{ or } 0.8W)$
4) $1.2D + 1.6W + L + 0.5(Lr \text{ or } S \text{ or } R)$
5) $(1.2 + 0.2SDS)D + \rho QE + L + 0.2S$
6) $0.9D + 1.6W + 1.6H$
7) $(0.9 - 0.2S_{DS})D + \rho Q_E + 1.6H$

where: $S_{DS} = 1.08$

$\rho = 1.3$ *(per ASCE 7-05 Section 12.3.4.2)*

Q_E *= effects of horizontal seismic forces from V or Fp.*

8.6 Seismic Irregularities

ASCE 7-05 Section 12.3 code gives the limitations for diaphragm flexibilities and also determines the type of structural irregularity on both the horizontal and the vertical planes of the building.

Irregularities that needs to be check includes:

- Horizontal Irregularity
- Vertical Irregularity
- P-delta Effects
- Inherent Torsion
- Accidental Torsion
- Overall building Torsion

8.7 Seismic Drifts

ASCE 7-05

$$\delta_x = \frac{C_d \, \delta_{xe}}{I} \qquad (12.8\text{-}15)$$

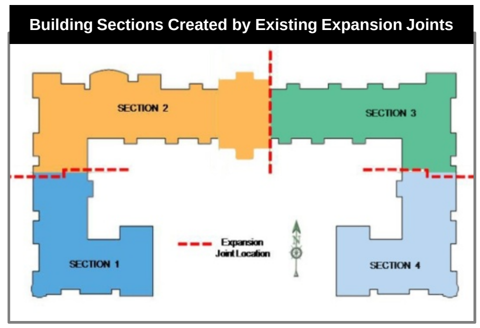
$$\Delta_a = 0.020 h_{sx} \qquad (12.12\text{-}1)$$

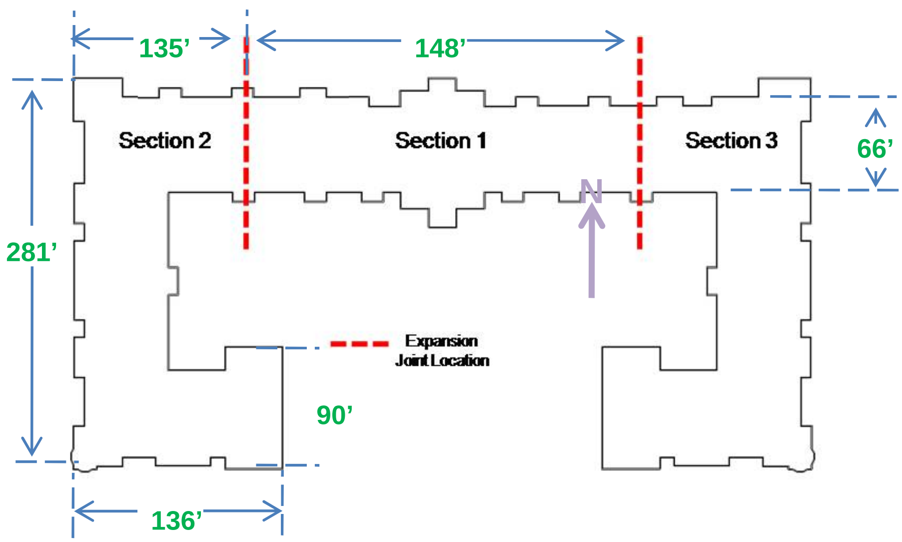
2007 California Building Code

$\Delta M = C_d \, \delta_{max}$ (equation 16-45).

9. Structural Depth

9.1 Placement or Expansion Joints





The relocation of expansion joints is needed due to the irregular configuration of the building. Two main problems related to seismic performance that may result are:

1. Different vibrations between different wings may result in a local stress concentration at reentrant corners

2. Torsion may result because of the center of rigidity and center of mass not coinciding.

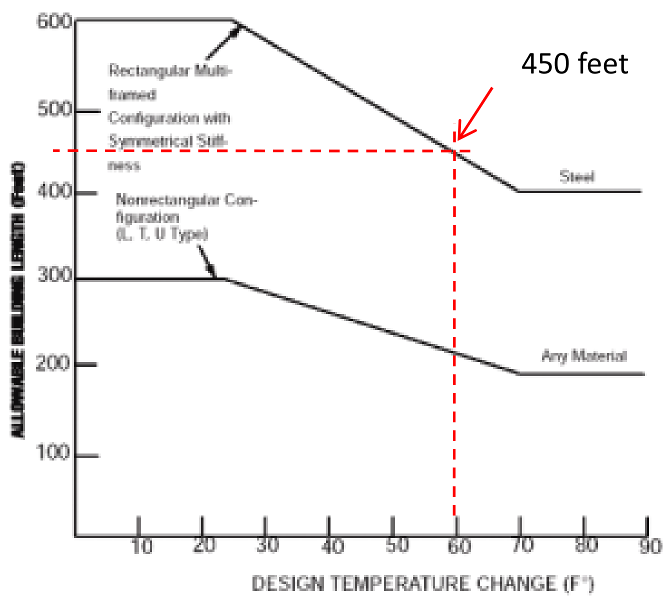
The building sections are then designed as independent structures. This report will cover analysis for only section 1 and section 2, because section 2 is similar to section 3 due to symmetry. Expansion joints are costly, thus I proposed to reduce the amount to only two. The new location of the expansion joints are based on the idea that we will consider pounding between the building sections only in the East-West direction during an earthquake. In this case, it will mean less damage to the entire building during an earthquake.

9.2 Expansion Joints Specified by Code:

LRFD Specification (AISC) lists expansion and contraction as a serviceability issue and provides the statement in Section L2, "Adequate provision shall be made for expansion and contraction appropriate to the service conditions of the structure."

ASCE 7-05 *Minimum Design Loads for Buildings and Other Structures* states, "Dimensional changes in a structure and its elements due to variations in temperature, relative humidity, or other effects shall not impair the serviceability of the structure."

The major temperature difference in LA, California between the record high temperature and record low temperature is approximately 60° F (From Almanac). As for normal temperature change, the average daily change is approximately 20° F. According to a graph in the AISC Steel Construction Manual, the allowable building length for a steel constructed building is approximately 450 feet for a temperature change of 60 ° F. The max distance of a building section as a result of my proposed expansion joint placements is no greater than 281 feet. This meets the design length criteria.



As for minimum building separation (of adjoining structures), L.A , California had modified ASCE 7 in Section 1614 in the **2007 California Building Code** to allow for the maximum inelastic response displacement :

$$\Delta M = C_d\, \delta_{max} \text{ (equation 16-45).}$$

Where δ_{max} is the calculated maximum displacement at Level x as define in ASCE 7 Section 12.8.4.3.

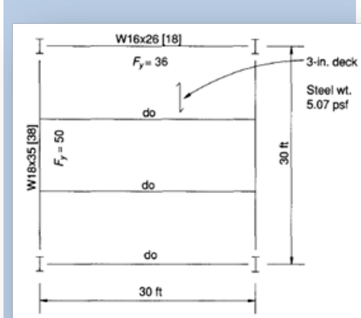


Fig. 3. 30 ft x 30 ft bay.

9.3 Floor System Design

The cost of a filler beam and/or girder beam consists of the cost of the mill material, the cost of fabrication, and the cost of erection. The cost of fabrication and erection for a single beam is about the same for a heavy beam . Thus, beams should be spaced as far apart as practical to reduce the number of pieces that has to be made and erected on site.

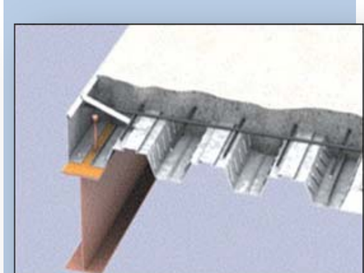
Another consideration is the bay sizes. For steel buildings, smaller bay size may not reduce costs. For economy, it is important to reduce the number of pieces to be fabricated and erected. Since the cost of fabrication and erection for a small beam is essentially the same as for a large beam, the savings involved in reducing the member weight is primarily savings in the cost of mill material. When the number of pieces is reduced, the actual cost of fabrication.

Before redesigning the gravity system, the entire column grid for the building is realigned. Most of the existing concrete columns are offset by approximately 10% of its span (as permissible by ACI). Spans are then limited to as fewer lengths as possible to reduce the number of fabrication variations and to make possible to order the materials in bulk quantity for cheaper prices.

A RAM model was constructed to aid in the realignment of the column grid and spacing of spans. The maximum and typical span is 30 feet x 30 feet, and will serve as the typical bay size for the design of the floor system. This will also be a conservative design as smaller bay sizes will eventually result with smaller sized beams as the 30 feet x 30 feet bays will be supporting public loads that are larger than the residential loads. Smaller bays will consist typically 25 x 25 feet.

From AISC Steel Design Guide: Low-and Medium-Rise Steel Buildings
Percentage Comparison Square Foot Costs

Bay Size	Mill Material	Fabrication & Delivery	Erection & Studs	Composite Deck	Total
25 x 25 ft	21%	14%	34%	31%	100%
30 x 30 ft	25%	14%	32%	32%	103%
30 x 30 ft (alt.)	31%	16%	35%	31%	113%
30 x 40 ft	31%	13%	33%	32%	109%

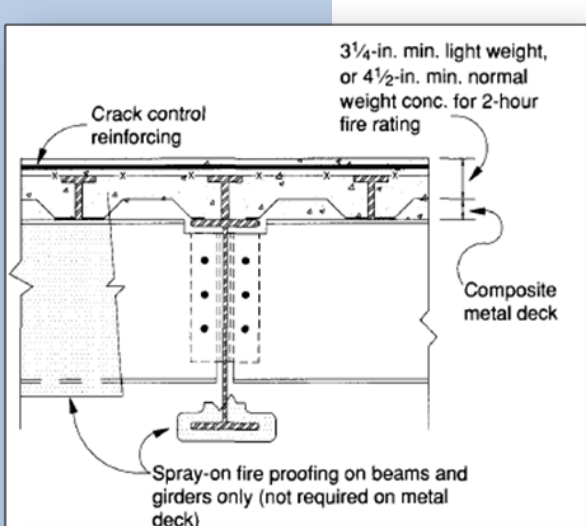


Composite floor systems, consisting of composite metal deck with concrete fill, steel filler beams, and girders made composite by using headed stud connectors, have become a standard type of construction, and are considered by many (engineers and architects) to be the highest quality type of construction. The floors are stiffer and more serviceable than open web joist systems. Fire resistance ratings may be obtained by providing a coat of fireproof material on the structural shape only. The combination of the concrete slab (light weight or normal weight) and composite steel deck require no additional protection when the proper slab thickness is used for a required fire rating.

The use of the ultimate strength design procedure with the LRFD Specification often results in some saving of mill material. In this case, the use of LRFD results in a savings of about 20% in the cost of mill material to the fabricator.

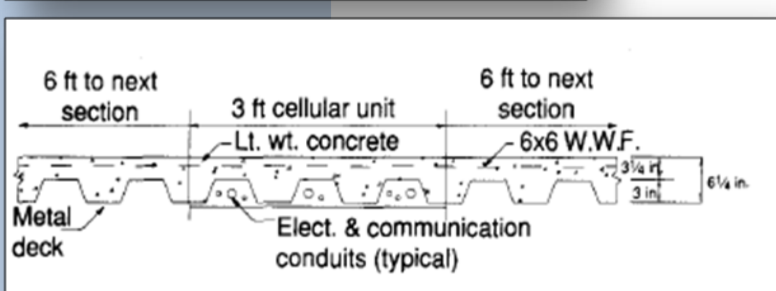
Unshored construction will be used to simplify the work of the contractor. The floor beams and girders must be designed to support the wet load condition loads as non-composite sections. Serviceability considerations includes vibrations, floor deflection, and crack control.

Composite Beam and Formed Metal Deck	
Slab Design	
Use 18 gauge 3 inch formed deck	Slab depth = 6.25"
fy=36ksi	Light Weight Conc: f'c=3 ksi
Max unshored span = 10.17 ft	with 26 to 30 ¾-in dia. Headed studs
Required Moment for Composite Beam	
Mu=1.2 (65.3) + 1.6 (112) = 258 ft kips	Use a W 16 x 31 or W 18 x 40
Required Strength for bare steel beam under dead load plus construction live	
Mu=1.2(65.3) + 1.6(40) = 142 ft kip	
Mp = 203 ft kip > 142 ft kip	Use a W16 x 31 is ok for beam
Serviceability Criteria	
Live Load Deflection	L/360
Total Deflection Limit	L/240
Construction Load Deflection	L/360

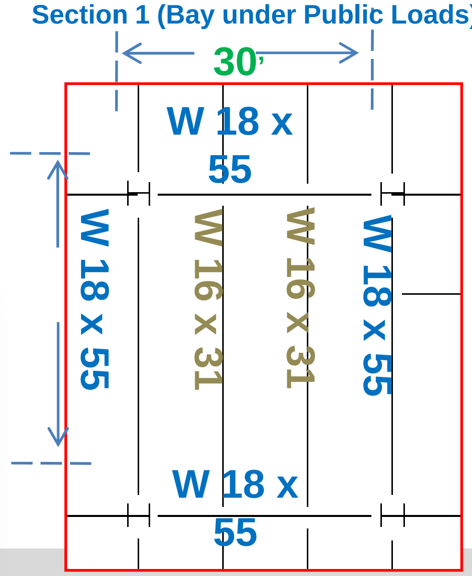


A typical 30' by 30' was chosen for design. The loads that this bay is subjected to are all from public areas.

A composite deck design was selected with a max unshored span of 10.7 feet before designing the girders and beams.



Section 1 (Bay under Public Loads)

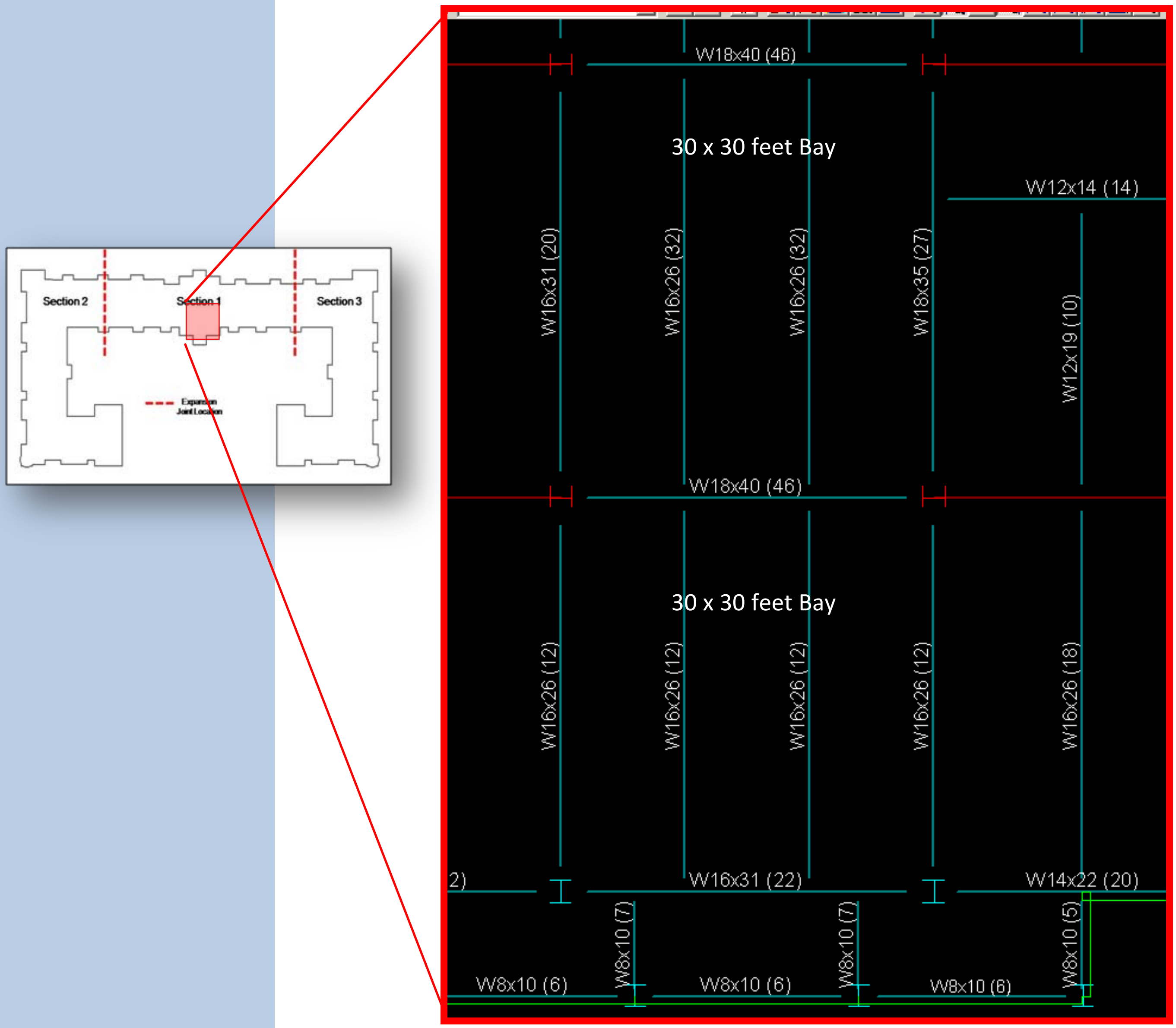


Floor System Design Summary

Members Sized in RAM

- Girders: W 18 x 40
- Infill Beams: W 16x26, W 16 x 31
- Beams supporting façade: W 8 x 10, W 14 x 22
- Typically W 14 x 43 and W 14 x 90

Comparing the members that were sized in RAM, they are the same



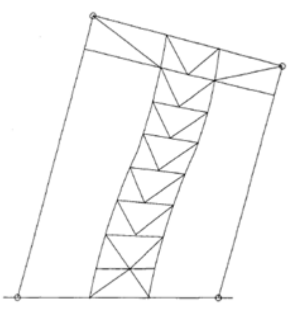
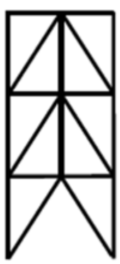
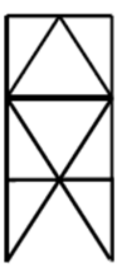
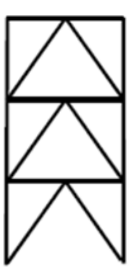


Fig. 33. Deflected shape bent with hat truss.







9.4 Lateral Design

System Selection

Braced frames are often the most economical method of resisting lateral loads in multi-story buildings. However, the use of bracing bents alone can result in uplift forces even in moderately low high-rise buildings (10-15 stories). This may not be a problem if deep foundations which can resist uplift are used or a combinations of other systems such as hat or belt trusses can be very. In L.A., California, the governing load combinations consist of seismic loads. In addition, Ingleside at King Farm is from 6 to 7 stories. Uplift will not be a controlling factor in the structural design of this project.

A belt truss system was considered as the 6.25 feet roof parapet surrounding the building will hide the truss located on the roof top. However, due to the irregular geometry of the building, and architectural plans, it is difficult to design the outrigger or have a core system coinciding with the center of mass and rigidity of each building sections without compromising the architectural layout.

A Special Concentric Braced Frame (SCBF) is recommended in areas of high seismicity. The difference between SCBF and OCBF is mainly due to the design of the connections to give more ductility in response to high seismicity. Ductility is of high demand for a structural steel system for resisting seismic loads. The SCBF is considered to be a better system than the Ordinary Concentric Braced Frame (OCBF) due to the better ductility of the system achieved through individual brace member design and gusset plate design. The brace when axially loaded in compression will eventually buckle, and the direction of brace buckling depends upon the brace shape orientation and the end restraints of the brace connections of beam to column members. The preferred but difficult to achieve behavior of a SCBF is the in-plane buckling of the braces.

Due to poor performance during past earthquakes of chevron bracing (both V and inverted V braces), only X bracing or chevron braced frame with a zipper column is recommended for high seismic loads.

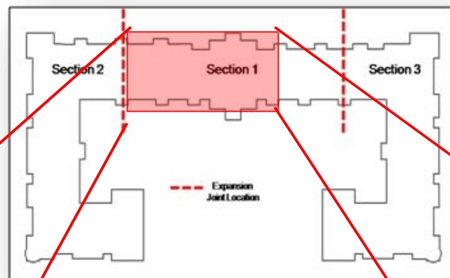
Based on past research, zipper frame or X bracing configurations resulted in simultaneous buckling of the braces at all story levels and hence a well distributed energy dissipation along the height of the frame during an earth quake. Both V and inverted V alone results in the buckling of bracings and excessive flexure of the beam at mid span where the braces intersect the beam. I proposed to utilize a combination of an inverted V and 2 story X brace system for the prototype design. The 2 story X brace system will be utilized where ever possible without architectural obstructions, such as hallways. Where ever there exists a hallway, the inverted V shall be used.

Brace Frame Locations

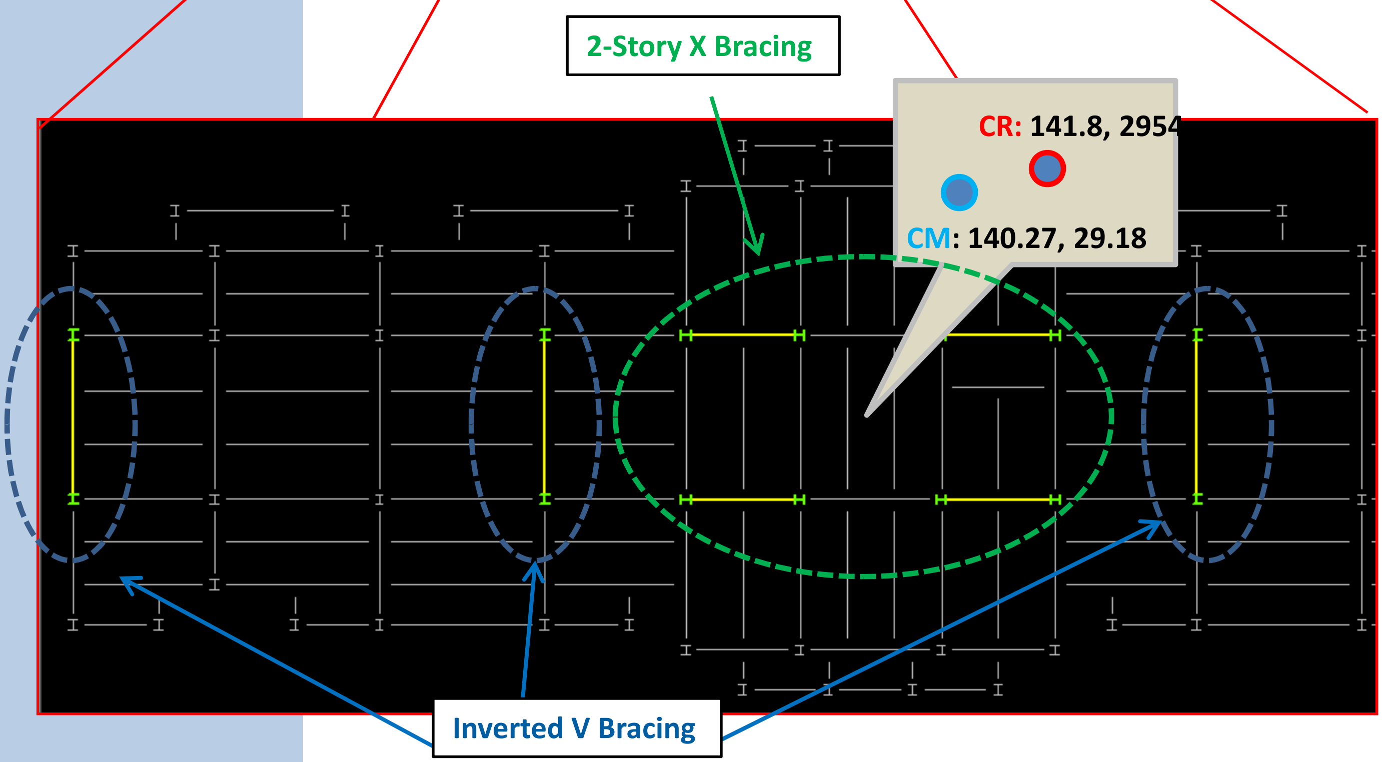
The placement of the brace frames will focus on the matter of having the center of mass and center of rigidity coinciding with each other as much as possible to reduce the effects of torsion due to lateral forces. It is recommended by code to place the braces at the exterior perimeter of the building so that the redundancy factor ρ can be equated to 1.0. However, due to the numerous fenestrations of the building envelope, it is not a feasible decision to locate the brace frames at the perimeter.

Redundancy is based on ASCE section 12.3.4. It was concluded that no more than one frame takes 33% of the shear when is removed. However, there must be a minimum of 2 frames in each direction along the perimeter, which is not possible with this design. As a result, the **redundancy factor ρ = 1.3**.

Center of Mass for SCBF		
Story	X Direction	Y Direction
Roof	140.05	23.19
7	140.05	23.19
6	140.78	29.18
5	140.78	29.18
4	140.78	29.18
3	140.78	29.18
2	140.78	29.18



Center of Rigidity for SCBF		
Story	X Direction	Y Direction
Roof	142.67	29.73
7	142.67	29.73
6	141.8	29.5
5	141.8	29.5
4	141.8	29.5
3	141.8	29.5
2	141.8	29.5



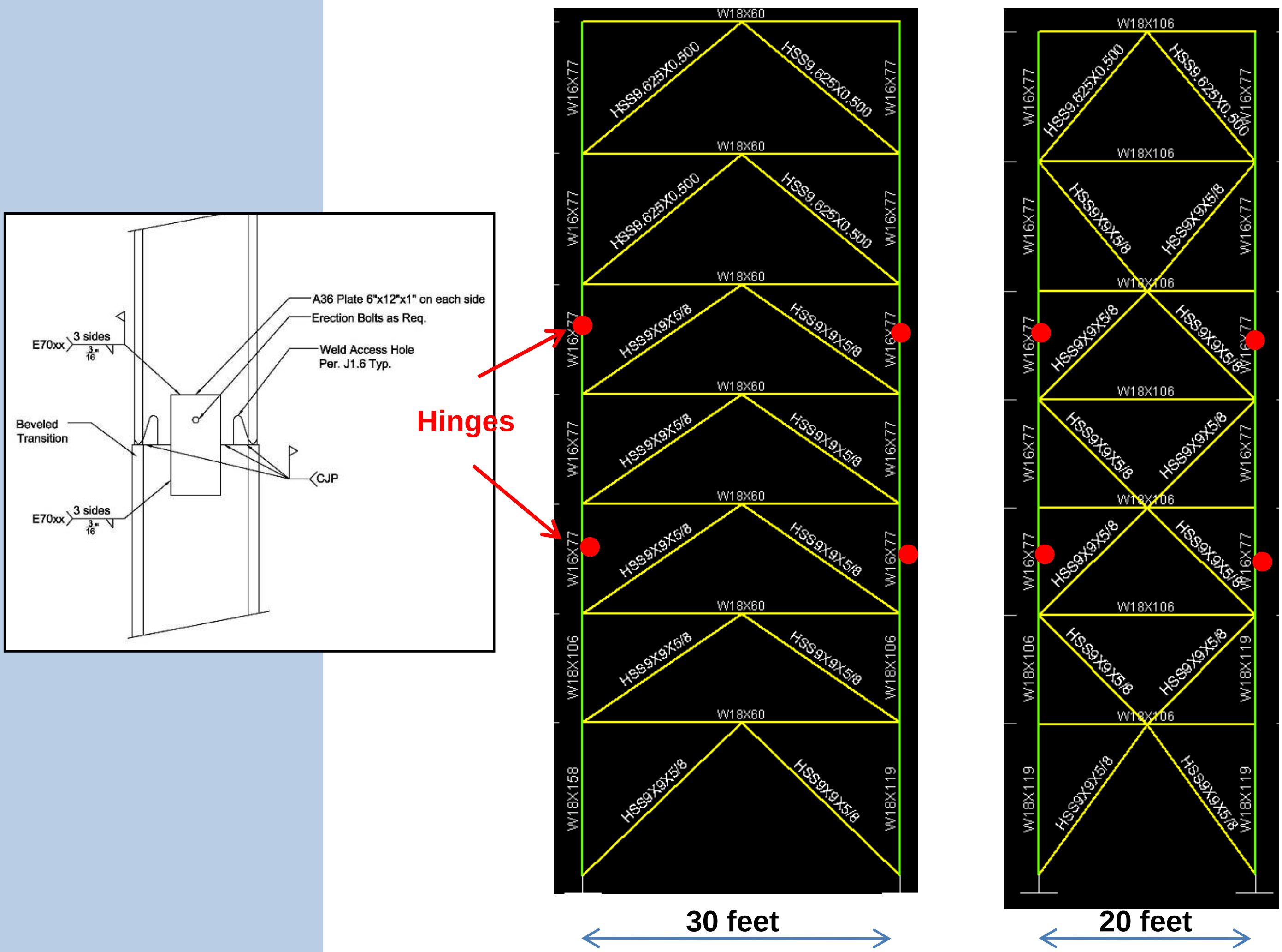
Brace Frame Design

Sizing of the frame members and braces were done using excel spreadsheet and was checked with RAM .

Final Designed members:

- Columns: W 16x77, W18x119
- Beams: W18 x 106
- Braces: HSS 9x9x5/8

AISC 341 requires splices be located in the columns to prevent story mechanisms. All column splices were located at the mid-height of the clear column at every other story starting with the 3rd story. A plate is bolted on each side of the splice to carry the shear requirement and a CJP was used to carry the flexural capacity of the members.

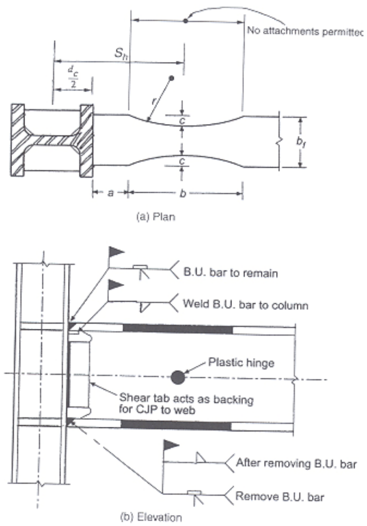


N-S Direction E-W Direction

Moment Frame Design

Moment Frames

After the earthquake of Northridge 1994, it was discovered that steel moment frame buildings experience brittle fractures of beam-to-column connections. It had demonstrated that brittle fractures was initiated within connections at very low levels of plastic demand and also while the structures remained elastic. Fractures at complete joint penetration weld between the beam and bottom flange and column flange can progress along numerous paths. Due to this event, FEMA 350 prequalified several connection types. One is the welded flange plate (WFP), the other being a reduced beam section (RBS).

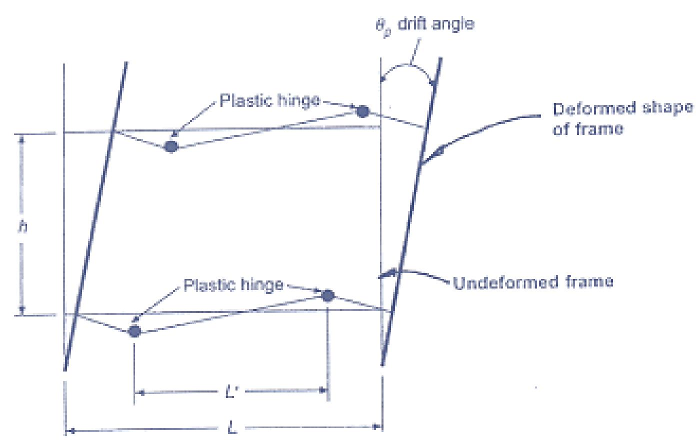


The RBS connection utilizes less material, and thus is the preferred choice in the prototype design of Ingleside at King Farm. This connection utilizes circular radius cuts in both top and bottom flanges of the beam to reduce the flange area over a length of the beam. RBS also has no reinforcing other than the weld metal is used to joint the flanges of the beam to the column. This is so that plastic hinges will form at these reduced section areas of the beam. The formation of plastic hinges at the beam-column interface during seismic event results in large inelastic strain demands at the connection leading to brittle failure. The reduced section of the beam at the desired location of the plastic hinge can remedy this issue.

Strong Column-Weak Beam

The purpose of a strong column-weak beam concept is to ensure the frame stability. The formation of plastic hinges in a column can cause failure. Large inelastic displacements are produced in the columns as the result of the formation of plastic hinges. This inelastic displacement can result in the increase of the P-delta effect leading to failure. This concept can be achieved in accordance with the requirement

$$\Sigma M^*_{pc}/\Sigma M_c > 1.0$$



Beam Buckling

AISC-Seismic specifies the use of sections with a maxium flange width-to-thickness ration of

$$B_f/2t_f = 52/(Fy)^{0.5}$$

To proved for adequate web stability, the height-to-thickness ration of the web shall not exceed

$$h_c/t_w = 418/(Fy)^{0.5}$$

Column Design

When the ration of column moments to beam moments is $\Sigma M^*_{pc}/\Sigma M_c < 2.0$, columns shall comply with slenderness requirements.

9.5 Seismic Analysis

Structural Irregularities

Section 12.3 of the ASCE 7-05 code determines and dictates the limitations for diaphragm flexibilities, structural irregularity for both horizontal and the vertical planes of the building. Table 12.6-1 of ASCE 7 gives the permitted analytical procedures for each design class along with the limitations due to a structural irregularity.

Horizontal Structural Irregularities (Table 12.3.1 ASCE)			
Type	Irregularity	Varification	Status
1a	Torsional	Checked with RAM Model	ok
2	Reentrant Corner	All over the building - concentration forces at corners	NG
3	Diaphragm Discontinuity	None by inspecting drawings	ok
4	Out of Plane Offsets	None by inspecting drawings	ok
5	Non Parallel System	All lateral resisting systems are parallel to major axis	ok

Vertical structural irregularities determined according to Section 12.3.2.2.

Vertical Structural Irregularities (Table 12.3.2 ASCE)			
Type	Irregularity	Varification	Status
1a	Stiffness-Soft Story	Level 6 is a soft story due to varying heights	NG
2	Weight (Mass)	calculated weight of each story and is fine	ok
3	Vertical Geometric	(66/51)=1.29 < 1.3	ok
4	In-Plane Discontinuity of Vertical Lateral Force Resisting Elements	No by drawing speculations	ok
5a, 5	Discontinuity on Lateral Strength	Lateral system runs continuously	ok

Upon Analyzing the structure and the limiting factors that governs the analytical procedure as determined by Section 12.6, Two irregularities exists for section 1 of Ingleside at King Farm. One irregularity is the reentrant corners, the other is a stiffness-story (in the E-W direction only). The diaphragm connections need a 25% increase in their force as permitted by the Equivalent Lateral Force Procedure (ELFP) and since T<3.5Ts, then it is permitted to use the **Equivalent Lateral Force Analysis**. This procedure is also more simple to use as oppose to modal analysis.

Seismic Design Parameters

Seismic calculations were performed using excel spreadsheet following the procedure prescribed in ASCE 7-05.

Criteria	Value	Code Reference
Occupancy Category	I	Table 1.1
Importance Factor	1	Table 11.5-1
Spectral Acceleration for Short Periods (Ss)	1.656	www.usgs.org
Spectral Acceleration for 1 Second Periods (S1)	0.59	www.usgs.org
Site Coefficient, Fa	1	ASCE 7-05 Table 11.4-1
Site Coefficient, Fv	1.5	ASCE 7-05 Table 11.4-2
Site Class	D	Assumed
Seismic Design Category	D	ASCE 7-05 Section 11.6
R Factor (SCBF)	6	ASCE 7-05 Table 12.2-1 # B3
SMS	1.66	ASCE 7-05 Equation 11.4-1
SM1	0.89	ASCE 7-05 Equation 11.4-2
SDS	1.104	ASCE 7-05 Equation 11.4-3
SD1	0.393	ASCE 7-05 Equation 11.4-3
Deflection Amplification C_d	5	ASCE 7-05 Table 12.2-1 # B3
Over strength Factor Ω_0^g	2	ASCE 7-05 Table 12.2-1 # B3

Criteria	Value	Code Reference
x	0.75	ASCE 7-05 Table 12.8.2
C_t	0.02	ASCE 7-05 Table 12.8.2
h_u	94	
$Ta=C_t h_u^x$	0.6038	
C_u	1.4	ASCE 7-05 Table 12.8.1
$T=Ta*C_u$	0.845286478	
T_l	8	
$(T>T_l)$ CS	0.733370513	
W	7585.80	
k	2	

Vertical Force Distribution for SCBF

Distribution of the seismic based shear is calculated using ASCE-07 Section 12.8.3. Cs factor was determined by Section 12.8.1.1 and the overall weight of the building was calculated before calculating the seismic base shear, distribution factor and forces and shear per story. Calculations were computed for both E-W and N-S directions due to an adjustment in the directions when the actual period was inputted after the structure was initially designed.

Vertical Force Distribution N-S Direction					
Floor	Height (Ft.)	Weight (Kips)	Cvx	Fx (kips)	Story Shear
Roof	12.00	915.40	0.13	76.69	76.69
7	12.00	930.90	0.13	77.99	154.68
6	10.00	1142.80	0.14	79.78	234.46
5	10.00	1143.50	0.14	79.83	314.29
4	10.00	1144.30	0.14	79.89	394.18
3	10.00	1147.10	0.14	80.08	474.26
2	14.00	1161.80	0.19	113.55	587.81
Total Weight		7585.8 Kips			

Seismic Base Shear	587.81	kips
Overturning Moment	6641.69	kip-ft

Vertical Force Distribution E-W Direction					
floor	Height (Ft.)	Weight (Kips)	Cvx	Fx (kips)	Story Shear
Roof	12.00	915.40	0.13	90.05	90.05
7	12.00	930.90	0.13	91.57	181.62
6	10.00	1142.80	0.14	93.68	275.29
5	10.00	1143.50	0.14	93.74	369.03
4	10.00	1144.30	0.14	93.80	462.83
3	10.00	1147.10	0.14	94.03	556.86
2	14.00	1161.80	0.19	133.33	690.19
Total Weight		7585.8 kips			

Seismic Base Shear	690.19	kips
Overturning Moment	7798.48	kip-ft

Seismic Drift and Soft Story Checks for SCBF N-S Direction (Building Section 1)

Drift is a serviceability requirement and should be limited as much as possible, especially building with seismic expansion joints to prevent pounding of the sections. Story drift for each floor was calculated using ASCE 7 chapter 12, equation 12.8-15 and 12.12-1. Story displacement values were obtained from RAM to calculate the overall deflections. The tables below summarizes the calculations

Calculations were preformed to see if vertical irregularities existed. Since there were non, **Equivalent Lateral Force Procedure** was possible

TABLE 12.12-1 ALLOWABLE STORY DRIFT, $\Delta_a{}^{a,b}$

Structure	Occupancy Category		
	I or II	III	IV
Structures, other than masonry shear wall structures, 4 stories or less with interior walls, partitions, ceilings and exterior wall systems that have been designed to accommodate the story drifts.	$0.025h_{sx}{}^{c}$	$0.020h_{sx}$	$0.015h_{sx}$
Masonry cantilever shear wall structures [d]	$0.010h_{sx}$	$0.010h_{sx}$	$0.010h_{sx}$
Other masonry shear wall structures	$0.007h_{sx}$	$0.007h_{sx}$	$0.007h_{sx}$
All other structures	$0.020h_{sx}$	$0.015h_{sx}$	$0.010h_{sx}$

Drift and Displacement Calculations for SCBF N-S Direction						
Story	Height (Ft.)	Story Displacement (in)	δxe (in)	δx (in)	Δa (in)	Final Results
Roof	12	0.684	0.104	0.475	2.880	ok
7	12	0.580	0.117	0.534	2.880	ok
6	10	0.463	0.101	0.461	2.400	ok
5	10	0.362	0.101	0.461	2.400	ok
4	10	0.261	0.096	0.438	2.400	ok
3	10	0.165	0.084	0.384	2.400	ok
2	14	0.081	0.081	0.370	3.360	ok

Soft Story Check for SCBF N-S Direction					
Story Drift	Drift Ratio	0.7x the Story Drift Ratio	0.8x the Story Drift Ratio	Avg. Story Drift Ratio of Next 3 Stories	Soft Story Issue
0.104	0.0087	0.0061	0.0069	-	No
0.117	0.0097	0.0068	0.0078	-	No
0.101	0.0101	0.0071	0.0081	-	No
0.101	0.0101	0.0071	0.0081	0.0095	No
0.096	0.0096	0.0067	0.0077	0.0100	No
0.084	0.0084	0.0059	0.0067	0.0099	No
0.081	0.0058	0.0041	0.0046	0.0094	No

Seismic Drift and Soft Story Checks for SCBF E-W Direction (Building Section 1)

Upon checking for soft story in the E-W direction, there appears to be a soft story issue.

Drift and Displacement Calculations for SCBF E-W Direction						
Story	Height (Ft.)	Story Displacement (in)	δxe (in)	δx (in)	Δa (in)	Final Results
Roof	12	1.055	0.206	0.988	2.880	ok
7	12	0.849	0.176	0.844	2.880	ok
6	10	0.673	0.198	0.950	2.400	ok
5	10	0.475	0.164	0.784	2.400	ok
4	10	0.312	0.162	0.775	2.400	ok
3	10	0.150	0.150	0.719	2.400	ok
2	14	0.000	0.000	0.000	3.360	ok

Soft Story Check for SCBF E-W Direction					
Story Drift	Drift Ratio	0.7x the Story Drift Ratio	0.8x the Story Drift Ratio	Avg. Story Drift Ratio of Next 3 Stories	Soft Story Issue
0.206	0.0172	0.0120	0.0137	-	No
0.176	0.0147	0.0103	0.0117	-	No
0.198	0.0198	0.0139	0.0158	0.0106	Yes
0.164	0.0164	0.0114	0.0131	0.0172	No
0.162	0.0162	0.0113	0.0129	0.0169	No
0.150	0.0150	0.0105	0.0120	0.0174	No
0.000	0.0000	0.0000	0.0000	0.0158	No

Torsion Effects

Inherent Torsion

Inherent torsion generally happens when the center of mass and the center of rigidity are not coinciding near each other. ASCE 7-05 Section 12.8.4.1 was used to analyze this issue. It was determined that the building had a rigid diaphragm with COM and COR almost over lapping each other on all floors. The torsional moments in both X and Y-axis are small and negligible.

Accidental Torsion

According to ASCE 7-05 Section 12.8.4.2, where earthquake forces are applied concurrently in two orthogonal directions, the required 5 percent displacement of the center of mass need not be applied in both of the orthogonal directions at the same time, but shall be applied in the direction that produces the greater effect.

This is done by adding a torsional moment at each floor equal to the story force multiplied by 5% of the floor dimension perpendicular to the direction of force. This method is equivalent to moving the center of mass by 5% of the plan dimension in a direction perpendicular to the force. If the lateral deflection at either end of the building is more than 20% greater than the average deflection, then the building is classified as torsionally irregular and accidental eccentricity must be amplified using the formula:

$$A_x = \left(\frac{\delta_{max}}{1.2\delta_{avg}}\right)^2 \qquad (12.8\text{-}14)$$

where

δ_{max} = the maximum displacement at Level x (in. or mm) computed assuming $A_x = 1$

δ_{avg} = the average of the displacements at the extreme points of the structure at Level x computed assuming $A_x = 1$ (in. or mm)

For a conservative analysis, I assumed the lateral deflection at one end of the building section to be at least 20% due to building section 1' length being 148 feet in the X-direction (E-W) resulting in a large shift in the center of mass. The torsional moment due to forces in the Y-direction (N-S) will likely be greater than the X-direction because of the buildings length.

Accidental Torsion Continued

After calculations, it resulted in the amplification factor being less than 1.0 for both directions. As for accidental torrsional calculations, there is no torsion irregularity. Torsional shears may be subtracted from direct shears if the torsional shear is reduced by the effects of accidental torsion. Like wise, torsional shears that are increased by the effects must be added to the direct shears.

Amplification Factor, Ax in the N-S Direction for SCBF						
Story	δA (in)	δB (in)	δavg (in)	δmax (in)	Ax	Torsion Irreg.
Roof	0.684	0.8208	0.7524	0.8208	0.83	No
7	0.58	0.696	0.638	0.696	0.83	No
6	0.463	0.5556	0.5093	0.5556	0.83	No
5	0.362	0.4344	0.3982	0.4344	0.83	No
4	0.261	0.3132	0.2871	0.3132	0.83	No
3	0.165	0.198	0.1815	0.198	0.83	No
2	0.081	0.0972	0.0891	0.0972	0.83	No

Accidental Torsion in the N-S Direction for SCBF					
Story	Width Bx (Ft)	5% Bx (Ft)	Story Force (K)	Ax Factor	Torsion (Ft-K)
Roof	148	7.4	76.69	0.83	469.0
7	148	7.4	77.99	0.83	477.0
6	148	7.4	79.78	0.83	487.9
5	148	7.4	79.83	0.83	488.2
4	148	7.4	79.89	0.83	488.6
3	148	7.4	80.08	0.83	489.7
2	148	7.4	113.55	0.83	694.4

Amplification Factor, Ax in the E-W Direction for SCBF						
Story	δA (in)	δB (in)	δavg (in)	δmax (in)	Ax	Torsion Irreg.
Roof	1.055	1.266	1.1605	1.266	0.83	No
7	0.849	1.0188	0.9339	1.0188	0.83	No
6	0.673	0.8076	0.7403	0.8076	0.83	No
5	0.475	0.57	0.5225	0.57	0.83	No
4	0.312	0.3744	0.3432	0.3744	0.83	No
3	0.15	0.18	0.165	0.18	0.83	No
2	0	0	0	0	0.00	No

Accidental Torsion in the E-W Direction for SCBF					
Story	Width By (Ft)	5% By (Ft)	Story Force (K)	Ax Factor	Torsion (Ft-K)
Roof	50	2.5	90.05	0.83	186.05
7	50	2.5	91.57	0.83	189.19
6	66	3.3	93.68	0.83	255.49
5	66	3.3	93.74	0.83	255.65
4	66	3.3	93.8	0.83	255.82
3	66	3.3	94.03	0.83	256.45
2	66	3.3	133.33	0.00	0.00

Seismic Expansion Joints Widths

As indicated in this report previously, minimum building separation (of adjoining structures), L.A , California had modified ASCE 7 in Section 1614 in the **2007 California Building Code** to allow for the maximum inelastic response displacement:

$\Delta M = C_d \, \delta_{max}$ (equation 16-45).

Where δ_{max} is the calculated maximum displacement at Level x as define in ASCE 7 Section 12.8.4.3.

The story displacement for both building section 1 and 2 were obtained from the RAM model. Cd is equal to 5.0 as defined using ASCE-07 when defining the seismic parameters previously in this report.

Drift and Displacement Calculations for SCBF E-W Direction For Section 1	
Story	Story Displacement (in)
Roof	1.055
7	0.849
6	0.673
5	0.475
4	0.312
3	0.150
2	0.000

Drift and Displacement Calculations for SCBF E-W Direction For section 2		
Story	Story Displacement (in)	ΔM (in)
Roof	1.270	6.35
7	1.090	5.45
6	0.779	3.895
5	0.615	3.075
4	0.459	2.295
3	0.306	1.53
2	0.167	0.835

Comparing the story displacements of Building Section 1 and 2, Building Section 2 express a greater displacement of 1.27 inches at the roof level. This results in ΔM = 6.35. Since this value only accounts for section 2, this value must be multiplied by 2 giving $\Delta M_{overall}$ = 12.7 inches.

It was concluded that a seismic expansion joint of 2 feet is required for the separation of the two building sections.

10. Breadth 1: Green Design

Governor Schwarzenegger's Green Building Initiative (Executive Order S-20-04), resulted in California being a leading example in reducing the amount of electricity, natural gas, water and other resources that state facilities consume on a daily basis.

> "That the California Public Utilities Commission (CPUC) is urged to apply its energy efficiency authority to support a campaign to inform building owners and operators about the compelling economic benefits of energy efficiency measures; improve commercial building efficiency programs to help achieve the 20% goal; and submit a biennial report to the Governor commencing in September 2005, on progress toward meeting these goals."

Calculation of increased R values and resulting energy reductions will be preformed.

10.1 Benefits of Green Buildings:

Environmental benefits:

- Enhance and protect ecosystems and biodiversity
- Improve air and water quality
- Reduce solid waste
- Conserve natural resources

Economic benefits:

- Reduce operating costs
- Enhance asset value and profits
- Improve employee productivity and satisfaction
- Optimize life-cycle economic performance

Health and community benefits:

- Improve air, thermal, and acoustic environments
- Enhance occupant comfort and health
- Minimize strain on local infrastructure
- Contribute to overall quality of life

10.2 Green Roofs

Green roofs are thin layers of living vegetation installed on top of conventional flat or sloping roofs. Green roofs protect conventional roof waterproofing systems and are a powerful tool in combating the adverse *impacts of land development* and the loss of open space.

Green roofs are divided into two categories:

1) **Extensive green roofs**, which are 6 inches or shallower and are frequently designed to *satisfy specific engineering and performance goals (The preferred choice in this thesis)*

2) **Intensive green roofs**, which may be quite deep and merge into more familiar on-structure plaza landscapes with promenades, lawn, large perennial plants, and trees.

The challenge in designing extensive green roofs is to replicate many of the benefits of green open space, while keeping them light and affordable.

The most common vegetated roof cover in temperate climates is a single un-irrigated 3- to 4-inch layer of lightweight growth media vegetated with succulent plants and herbs. In most climates, a properly designed 3-inch deep vegetated roof cover will provide a durable, low maintenance system that can have many benefits.

Design Factors

There are many interactive factors that must be taken into account for optimal performance in each setting:

- Climate, especially temperature and rainfall patterns
- Strength of the supporting structure
- Size, slope, height, and directional orientation of the roof
- Type of underlying waterproofing
- Drainage elements, such as drains, scuppers, buried conduits, and drain sheets
- Accessibility and intended use
- Visibility, compatibility with architecture, and owner's aesthetic preferences
- Fit with other "green" systems, such as solar panels
- Cost of materials and labor

Benefits

- Controlling storm water runoff
- Improving water quality
- Mitigating urban heat-island effects
- Prolonging the service life of roofing materials
- Conserving energy
- Reducing sound reflection and transmission
- Creating wildlife habitat, and
- Improving the aesthetic environment in both work and home settings.

http://www.wbdg.org/resources/greenroofs.php

Controlling Storm Water Runoff

The runoff of storm water from paved areas and roofs can cause flooding, erosion, pollution, and habitat destruction. The capacity of green roofs to moderate this runoff through both retention (water holding) and detention (flow-slowing). Green roofs share many engineering features with conventional storm water management basins, and compared to many at-grade storm water management practices, vegetated roof covers are unobtrusive, low maintenance, and reliable. They can be designed to achieve specified levels of storm water runoff control, including reductions in both total annual runoff volume (reductions of 50 to 60 percent are common) and peak runoff rates.

Improving Water Quality

By reducing both the volume and the rate of storm water runoff, green roofs benefit cities with combined sewer overflow (CSO) impacts. However, the research also shows that the correct choices of growing medium and plant types are essential for success. In cities with combined storm and waste water sewer systems, storm water dilutes the sanitary waste water, rendering treatment less efficient.

In urban areas, up to 30% of total nitrogen and total phosphorus released into receiving streams is derived from dust that accumulates on rooftops. This can result in ecological damage and human health hazards.

Due to the lesser amount of rain fall in LA, California compared with Rockville Maryland, controlling storm water runoff is not crucial in this thesis.

Mitigating Urban Heat-Island Effects

Covering dark conventional roofs with green roofs can significantly reduce the temperature above the roof, and have been shown to out-perform white or reflective roof surfaces in reducing the ambient air temperature. If sufficient urban surfaces utilizes extensive green roofs, *this cooling and improvement of air quality can have significant positive effects on human health, especially for the young and elderly in congested urban areas.*

Prolonging the Service Life of Roofing Materials

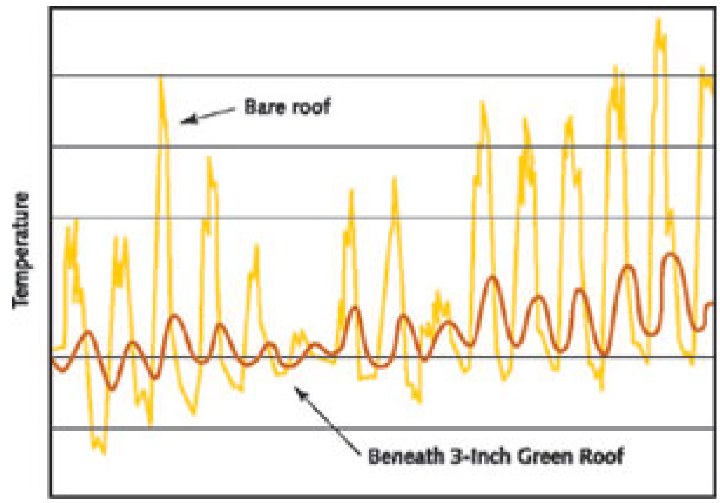
- The multiple layers of the green roof can protect the underlying roof materials from the elements:
- Protecting from mechanical damage (walking on roof top, wind-blown dust and debris, and animals)
- Shielding from ultraviolet radiation by buffering temperature extremes, minimizing damage from the daily expansion and contraction of the roof materials.

A roof assembly that is covered with a green roof can be expected to outlast a comparable roof without a green roof by a factor of at least two, and often three. Researchers expect that they will last 50 years and longer before they require significant repair or replacement.

Due to the huge surface area of exposed white roof, an extensive green roof will provide Ingleside at King Farm a beneficial life cycle cost for the roof envelope.

Conserving Energy

Not all benefits will be equally important in every project or climate. For instance, the capacity of green roofs to reduce heat flow, and therefore energy demand in buildings, is mostly a warm season phenomenon. As a result, this benefit will be realized most fully in warm climates like in L.A., California where energy expenditures on air conditioning is an important concern. Energy-related benefits will also be less important in multi-story buildings, due to the low ratio of roof area to the total of exposed building skin. Because green roofs are more complex than simple insulators, project-specific building envelope analysis is required to predict energy conservation under specific project conditions.



Reducing Sound Reflection and Transmission

Green roofs can absorb a portion of the sound that bounces off hard roofing surfaces. A 3-inch deep vegetative cover can be expected to reduce sound transmission by a minimum of 5 decibels. Sound abatement of up to 46 decibels has been measured on thicker roofs.

Creating Wildlife Habitat

Green roofs can be used to create wildlife habitats to supplement or replace diminishing open space in developing areas.

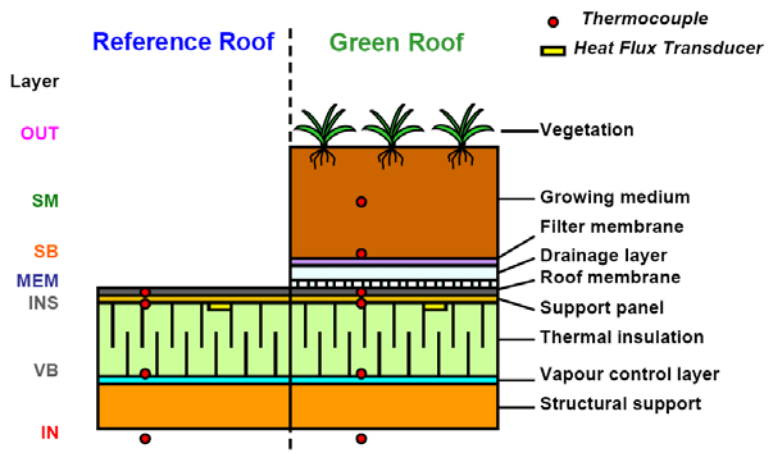
Improving the Aesthetic Environment

Green roofs offer interesting new opportunities for architectural design. A green roof can allow a structure to merge with the surrounding landscape, provide a dramatic accent, or reinforce the defining aspects of the structure's geometry. However, due to the 6' - 6" roof parapet, green roof aesthetics is not a major concern with Ingleside at King Farm.

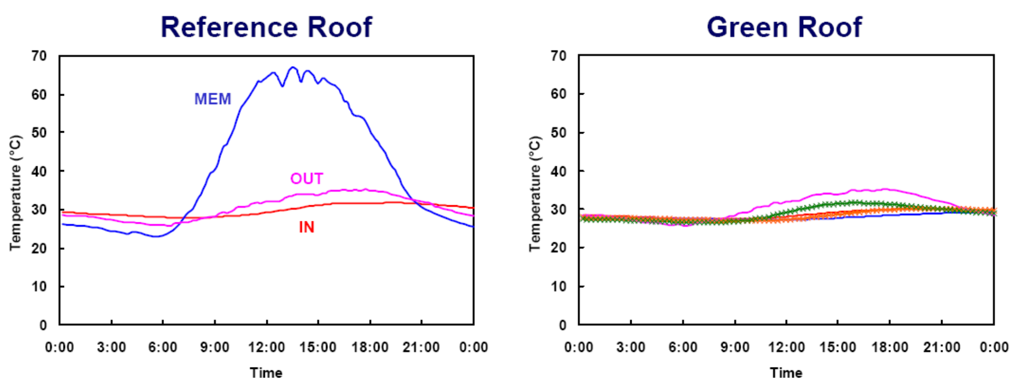
Choosing the Right System

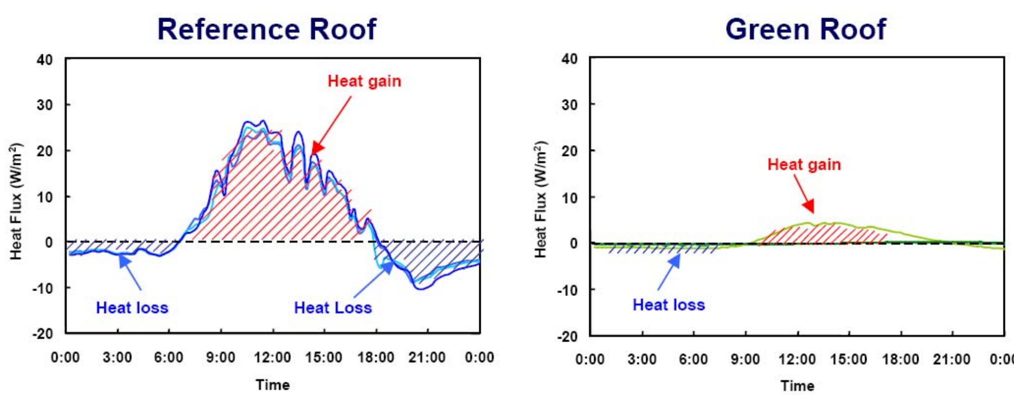
Comparison of Extension and Intensive Green Roof Systems		
	EXTENSIVE GREEN ROOF	INTENSIVE GREEN ROOF
	Thin growing medium; little or no irrigation; stressful conditions for plants; low plant diversity	Deep soil; irrigation system; more favorable conditions for plants; high plant diversity; often accessible
Advantages	• Lightweight; roof generally does not require reinforcement. • Suitable for large areas. • Dutiable for roofs with 0 - 30° (slope). • Low maintenance and long life. • Often no need for irrigation and specialized drainage systems. • Less technical expertise needed. • Often suitable for retrofit projects. • Can leave vegetation to grow spontaneously. • Relatively inexpensive. • Looks more natural. • Easier for planning authority to demand as a condition of planning approvals.	• Greater diversity of plants and habitats. • Good insulation properties. • Can simulate a wildlife garden on the ground. • Can be made very attractive visually. • Often accessible, with more diverse utilization of the roof. i.e. for recreation, growing food, as open space. • More energy efficiency and storm water retention capability. • Longer membrane life.
Disadvantages	• Less storm water retention benefits. • More limited choice of plants. • Usually no access for recreation or other uses. • Unattractive to some, especially in winter.	• Greater weight loading on roof. • Need for irrigation and drainage systems requiring energy, water, materials. • Higher capital & maintenance costs. • More complex systems and expertise.

Green Roof Design Analysis

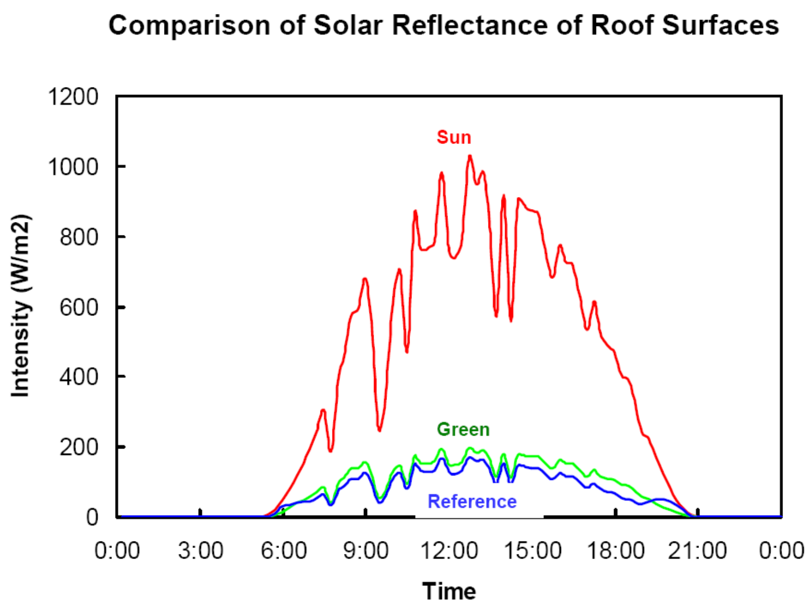


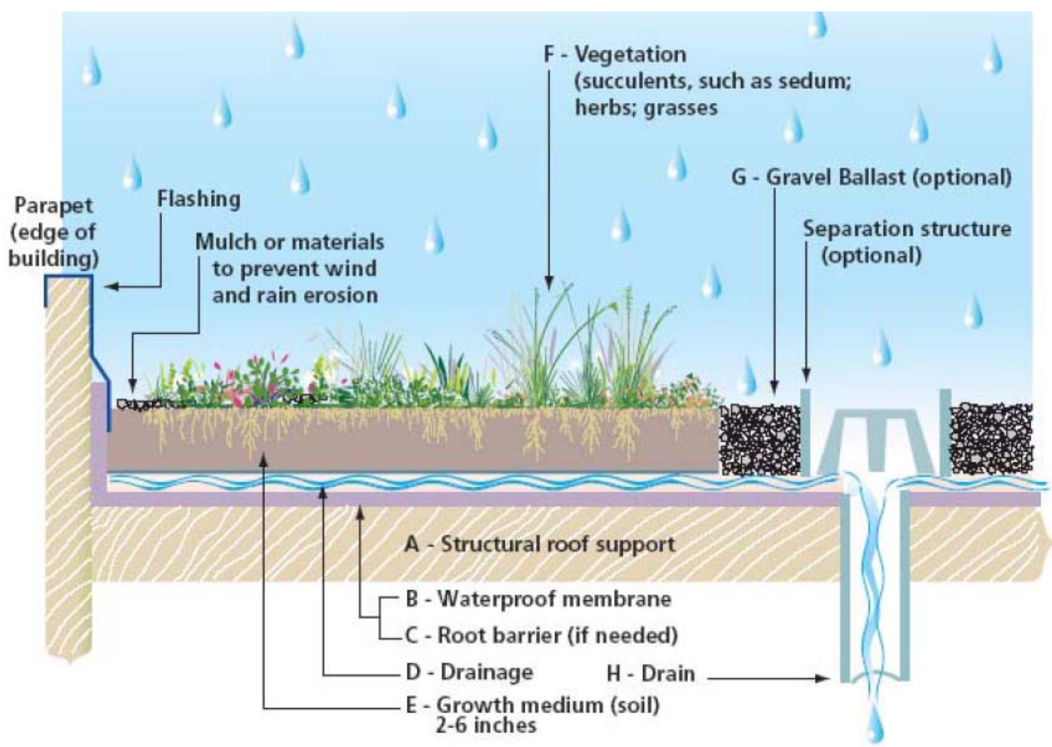
A typical sunny summer day





Reflectance





Life Cycle Cost Analysis

Initial Cost

According to a consultant on the project, the green roof on the GAP headquarters in San Bruno, CA, cost approximately $24 per square foot because it was one of the first green roof projects by an American corporation. It was completed in 1995. The costs of green roofs have declined, and the GAP green roof would probably only cost $11 to $14 per square foot ($120 to $150/sq m) today (EAD, Los Angeles, CA).

Maintenance

A green roof does have higher maintenance costs than a conventional roof. Maintenance activities that must be performed on a green roof are weeding, replanting, and inspections of the waterproof membrane. The green roof can also be divided into distinct compartments which can be moved for inspections or, when the time comes, after 30 to 50 years, for the replacement of the membrane. Electronic leak detection services are also available. Conducting several annual plant inspections and an annual inspection of the roof membrane entails an annual expense of approximately $1 per square foot.

Irrigation

Climate data for 1971 to 2000 shows an annual average of 15 inches of precipitation at the Los Angeles Civic Center. Using a procedure for estimating landscape water needs developed by the University of California Cooperative Extension, it is estimated that a green roof in Los Angeles will require 0.9 cubic feet of additional water per square foot annually (6.7 gallons per square foot annually). The approximate annual cost of this water assuming a price of $2.20 per hundred cubic feet of water (EAD, Los Angeles, CA) would be $0.020 per square foot ($0.22 per square meter) or about $200 per year for a 10,000 square foot (930 square meters) green roof.

Even greater water efficiency can be achieved if captured rainwater or gray water can be used for irrigation. A green roof can capture between 10 and 100 percent of incidental rainfall. Adopting the midpoint of those values (55 percent), under the average annual precipitation in Los Angeles of about 15 inches, a 10,000 square foot green roof would yield 6,250 cubic feet of runoff annually. If all of that were captured, it would supply 70 percent of the estimated annual water needs of the roof (EAD, Los Angeles, CA).

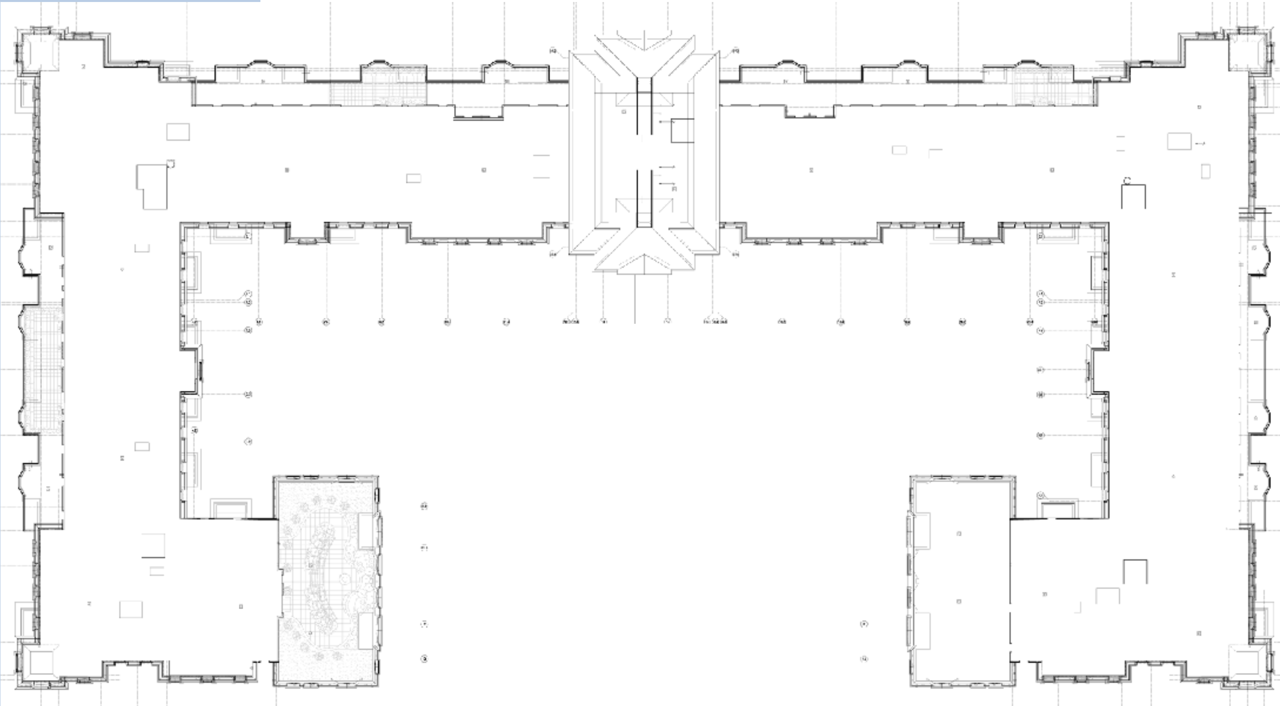
Summary of Costs

The benefits provided by a green roof depends on many factors. The direct benefits that can result from a green roof, such as the decreased cooling expenses is just one of many. Taking into consideration the many benefits provided by green roofs undoubtedly would yield a much higher total value. Such as the energy savings and improved air quality to have a present value (assuming a 20 year project life) of approximately $0.72 per square foot of cool roof (EAD, Los Angeles, CA).

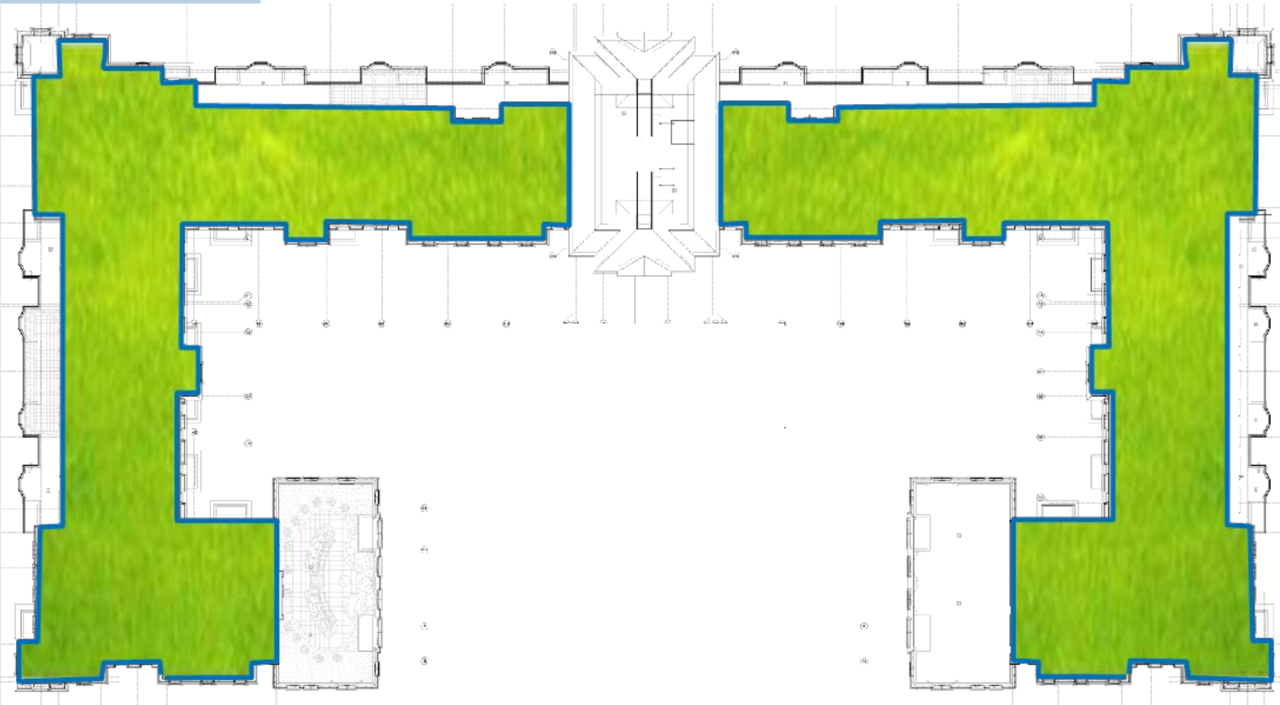
	Reroof	New Roof
Anticipated Life (yrs)	35 - 40	35 - 40
Annualized Initial Cost (per sf)[3]	$1.35	$0.84
Maintenance Cost (per sf)	$1.00	$1.00
Irrigation Cost (per sf)	$0.02	$0.02
Total Annual Cost (per sf)	$2.37	$1.86

Placement of Green Roof on Prototype Design

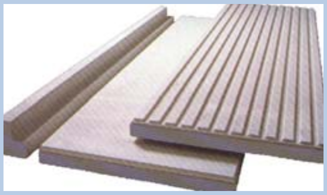
Existing Roof Membrane: White PVC Single Ply System

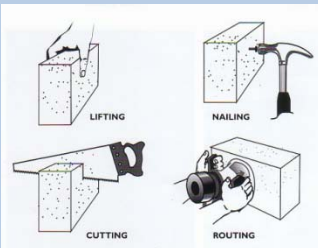


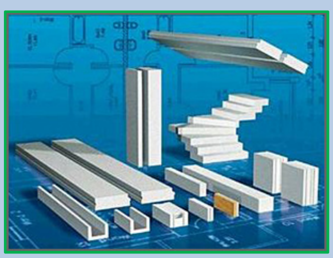
Prototype Roof Membrane: Extensive Green Roof Usage



10.3 Choices of Green Materials







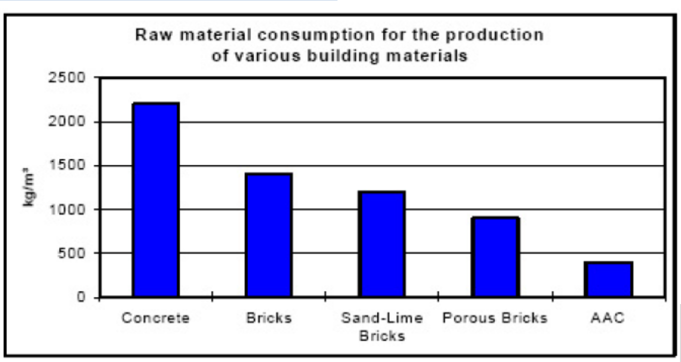
I proposed to use AUTOCLAVED AERATED CONCRETE (AAC). The material consists of approximately 80 percent air. Due to its low consumption of readily available raw materials, excellent durability, energy efficiency compared to manufacturing CMUs and concrete, relative cost effectiveness, produces no pollution, and ability to be recycled, AAC has a “green” designation. The light weight of AAC in relation to its strength reduces the seismic base shear, and its fire-resistant characteristics provide further advantage against fires commonly associated with earthquakes. (4” thick panel gives UL fire rating of 4 hours). Other benefits which are more towards building envelope relevance of high durability resulting in less maintenance, good life cycle cost, rapid construction and good workability resulting in reduced labor cost, excellent thermal insulation and sound absorption. It is also a low shrinkage material compared to concrete and its reduced weight also lowers shipping costs and cost about the same as timber construction.

The material is available in masonry units and precast panels. The usage of a single material with various appearance can reduce the amount of façade interfaces that is in the existing design of Ingleside at King Farm. This will decrease the chances of infiltration and moisture penetration into the structure and conditioned spaces.

Material	R-value**	Load capacity, kN/m (klf)	STC
AAC 4.0 (203 mm, [8-in.])	1.66 (11.5)	800 (56)	50
CMU (8 in., hollow)	0.32 (2.2)	164 (11.3)	45
CMU (8 in., foamed cores)	0.81 (5.6)	164 (11.3)	45
Normal-weight concrete (152 mm [6 in.])	0.06 (0.4)	5250 (360)	57
Lightweight concrete (6 in.)	0.22 (1.5)	3150 (216)	N/A

Strength Class	Specified Compressive Strength	Nominal Density
AAC 2.0	2.0 MPa (290 psi)	400 to 500 kg/m³ (25 to 31 pcf)
AAC 4.0	4.0 MPa (580 psi)	500 to 800 kg/m³ (31 to 50 pcf)
AAC 6.0	6.0 MPa (870 psi)	700 to 800 kg/m³ (44 to 50 pcf)

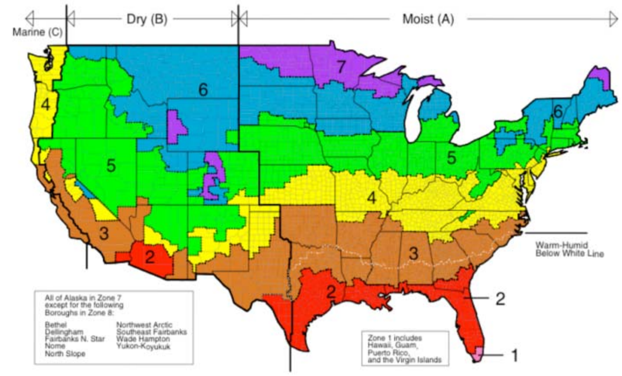
Ingredients used to make AAC include Portland cement mixed with lime, silica sand, or recycled fly ash (a byproduct from coal-burning power plants), water, and aluminum powder or paste and the mixed is poured into a mold. The reaction between aluminum and concrete causes microscopic hydrogen bubbles to form, expanding the concrete to about five times its original volume. After evaporation of the hydrogen, the now highly closed-cell, aerated concrete is cut to size and formed by steam-curing in a pressurized chamber (an autoclave). The result is a non-organic, non-toxic, airtight material.



Panels are available in thicknesses of between 8 inches to 12 inches, 24-inches in width, and lengths up to 20 feet.

Blocks come 24”, 32”, and 48” inches long, between four to 16 inches thick, and eight inches high.

11. Building Envelope Redesign



11.1 Aesthetics

The existing façade is a cavity wall of 4 different assemblies as defined in the existing condition section of this report. Due to this reason, the multiple interfaces allows a greater chance for infiltration and moisture penetration. As specified by IBC 2006 U.S. climate zone map, a vapor barrier is climate zone 1, 2, 3 ,4 is not required. However, the building site location had been moved from Rockville, MD (a heating climate) to L.A., Ca (a cooling climate). The cavity wall must be designed to prevent moisture penetration and provide well thermal insulation.

11.2 Choosing an Assembly

The following cavity wall assembly was designed for the prototype:

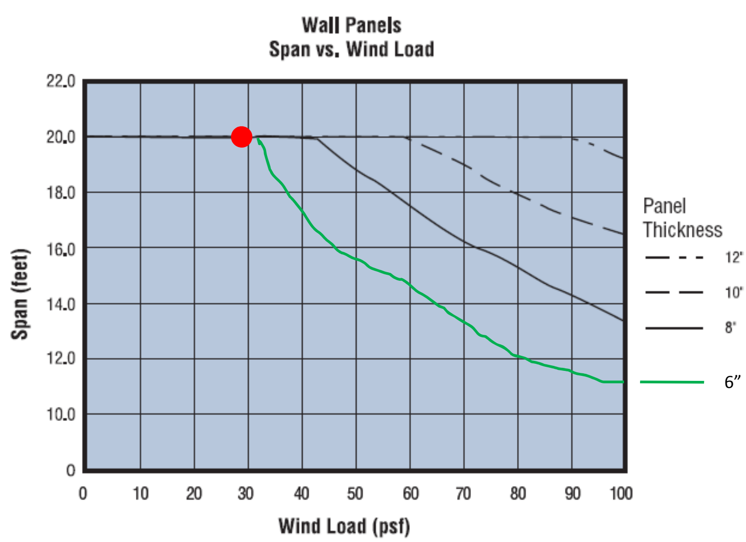
- Exterior Autoclaved Aerated Concrete Panels - 6.0 inch thick
- Air space and drainage plane - 2.5 inch
- Paper stand - 8 mil
- Plywood sheathing ½ inch
- Rigid insulation - 2 inch
- Steel Studs -5 ½ inch
- And gypsum board - ½ inch

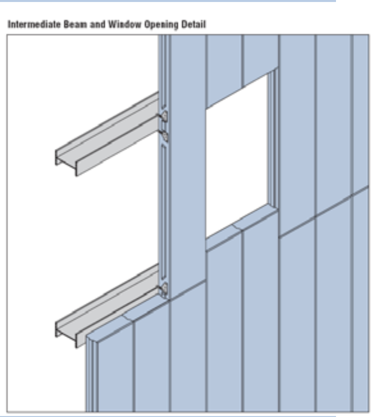
The material is available in masonry units and precast panels. The usage of a single material with various appearance can reduce the amount of façade interfaces that is in the existing design of Ingleside at King Farm. This will decrease the chances of infiltration and moisture penetration into the structure and conditioned spaces.

Bearing Stress Condition	Strength Class	
	AC4	AC6
Allowable Bearing Stress without a Bearing Pad	60 psi	85 psi
Allowable Bearing Stress with a Bearing Pad	100 psi	130 psi

Deflection

The allowable lateral deflection of AERCON wall panels due to lateral load is L/240. In most cases, an 6" thick wall panel is sufficient to resist the design loads in L.A. as wind is not a factor (85 mph per ASCE-07)

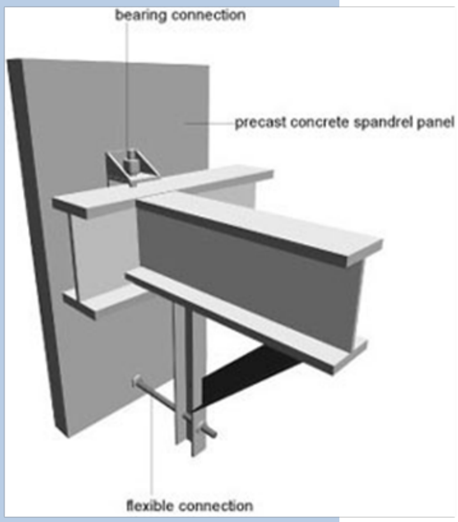


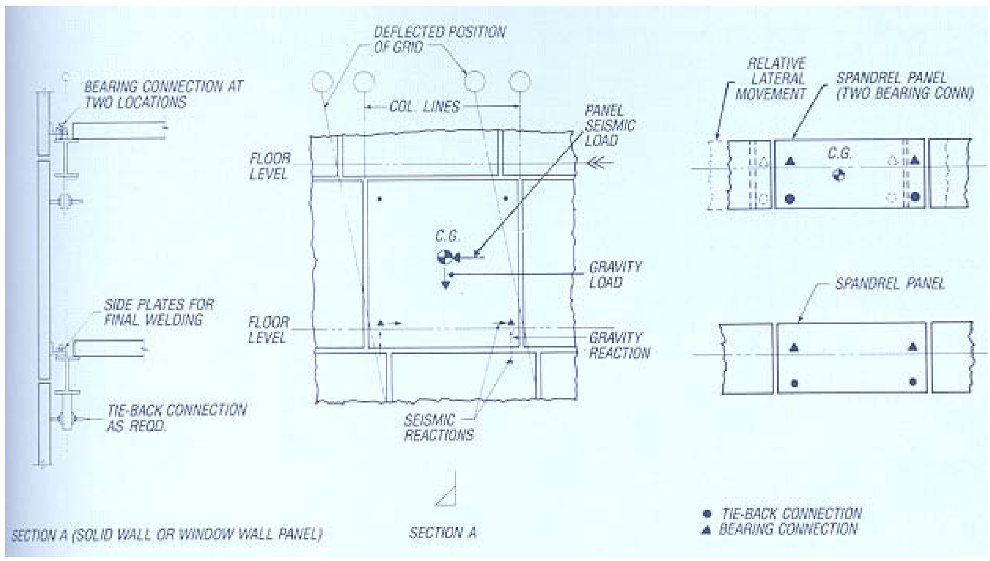


11.3 Cladding and Anchors

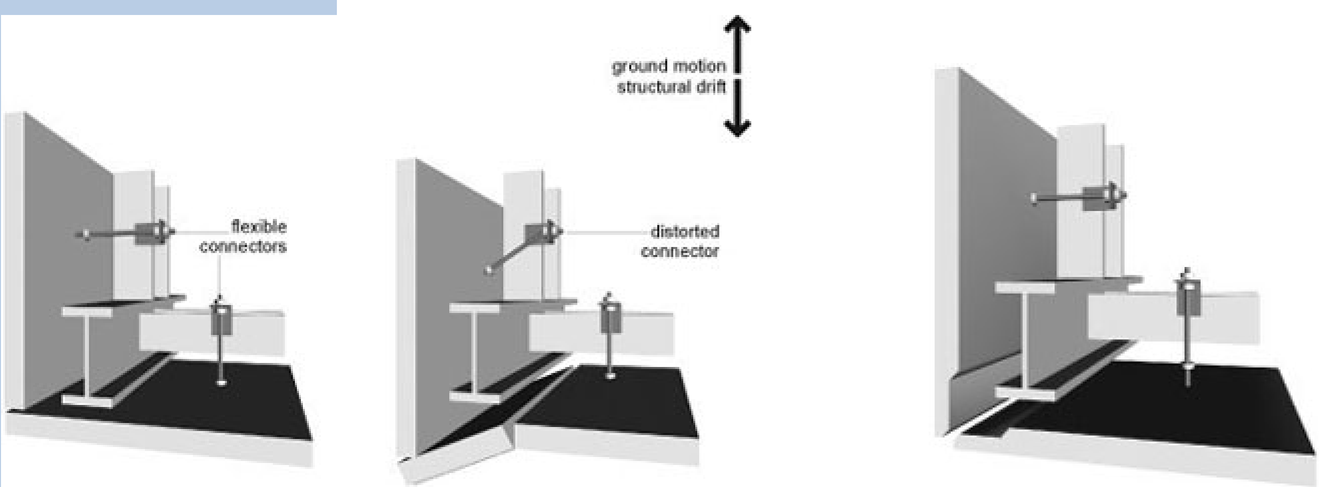
Seismic codes require that heavy panels accommodate movement either by sliding or ductile connections. In high seismic zones, sliding connections is not a good choice, because of the possibility of incorrect adjustments when bolts are used, jamming or binding due to unwanted materials left after installation and jamming due to geometrical change of the structural frame under horizontal forces.

A ductile connection will be utilized in the prototype design of Ingleside at King Farm. One type of ductile connection is a "Push-pull" - with Bearing connection at top, tieback connection at bottom.





There is a problem however, with the connections of corner panels and connections. They must be designed to permit panels to slide past one anther with minimum damage.

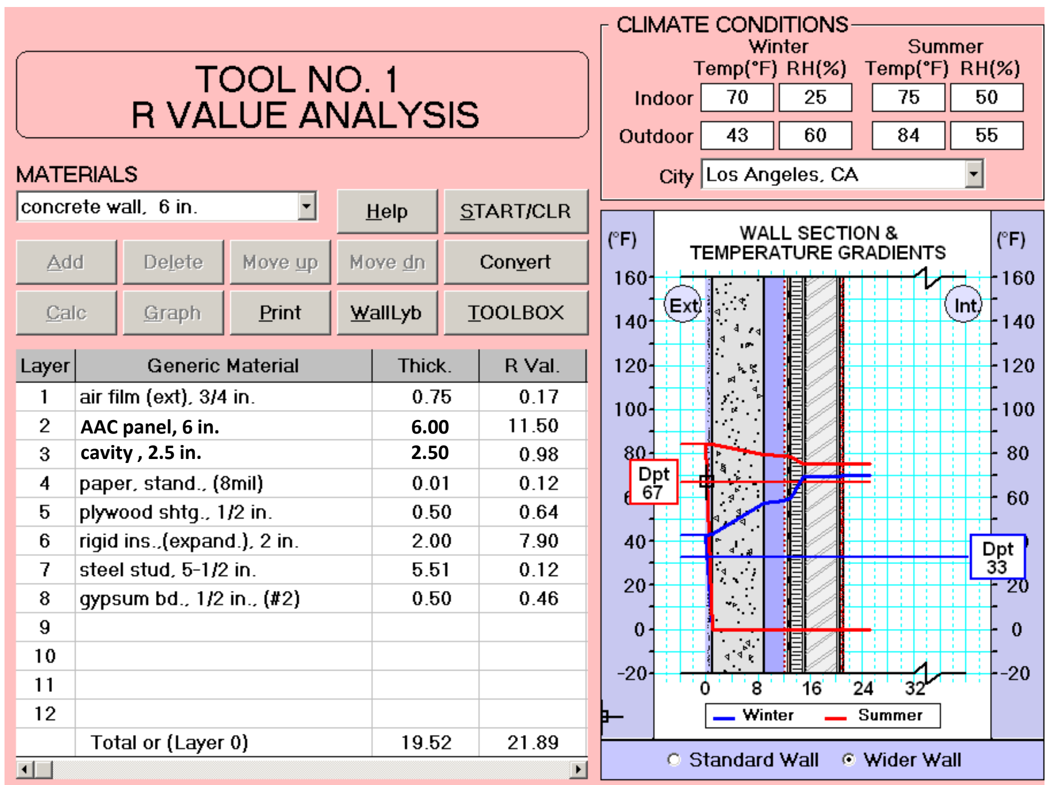


11.4 Envelope Performance Evaluation

R-Value Analysis

After performing a R-value analysis using HAM software and Excel spreadsheet using each materials' thermal conductivity, and thickness to obtain R values, both results were compared to and revealed similar results. Analyzing the overall R value of the system, which R=21.89 it is more than adequate as compared to a typical R value of 19 for residential homes.

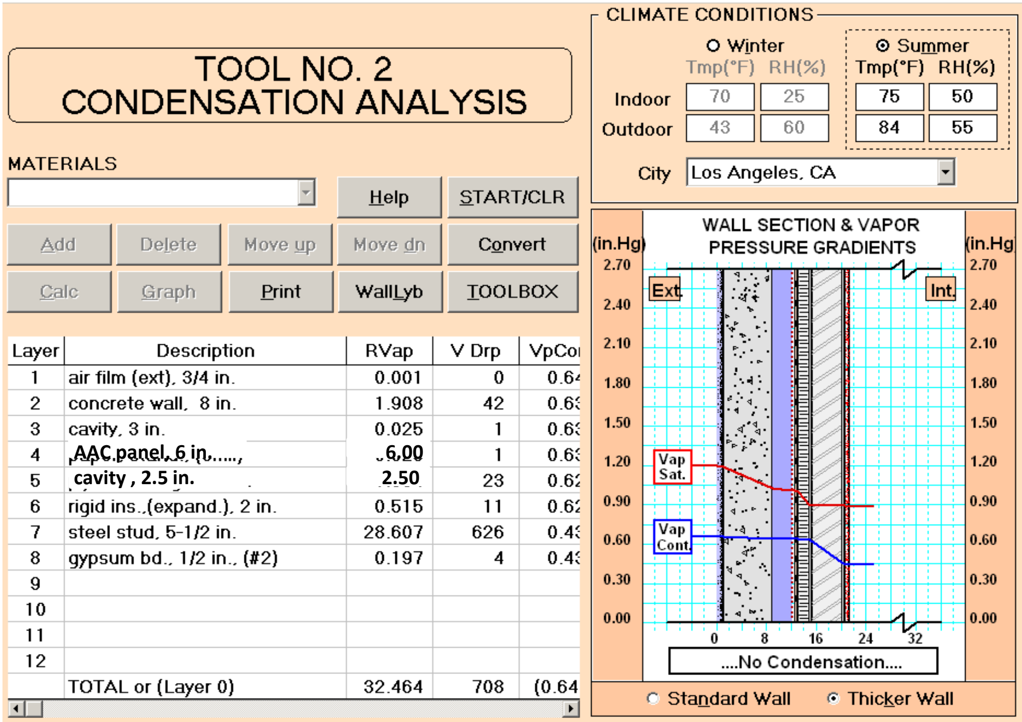
The dew point appears on the exterior side of the concrete panels, which is a desirable condition. In the event that moister or water does penetrate the concrete panels, a 2.5 inch air gap and drainage plane provides the next line of defense.



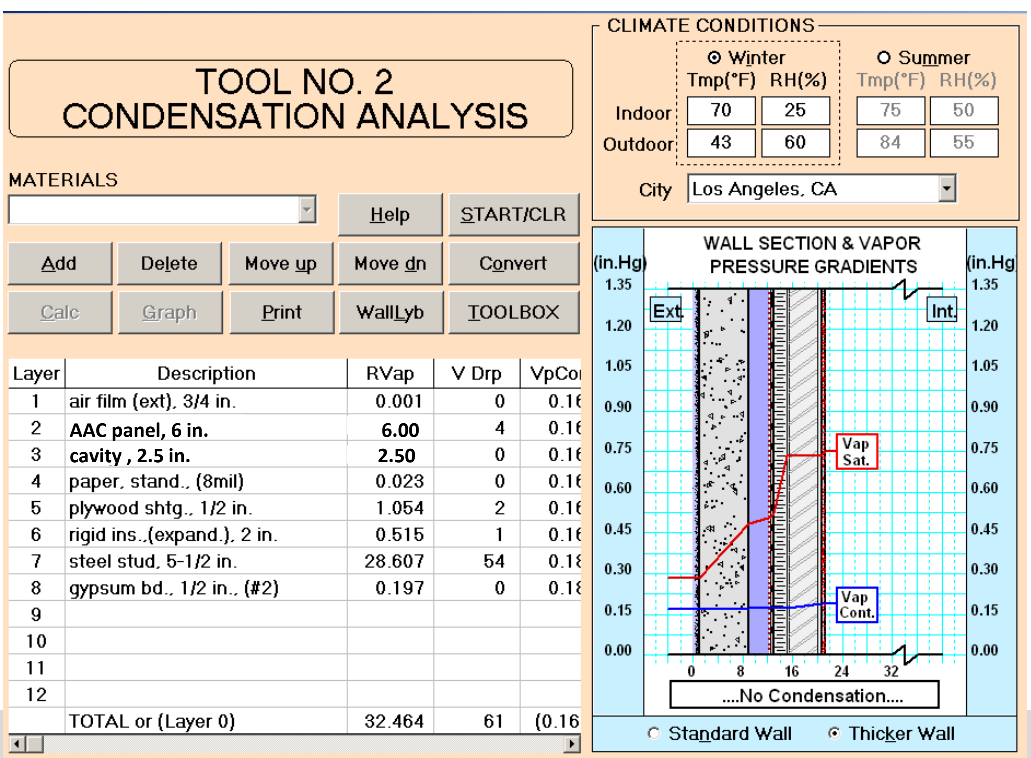
Layer	Generic Material	Thick.	R Val.
1	air film (ext), 3/4 in.	0.75	0.17
2	**AAC panel, 6 in.**	**6.00**	11.50
3	**cavity , 2.5 in.**	**2.50**	0.98
4	paper, stand., (8mil)	0.01	0.12
5	plywood shtg., 1/2 in.	0.50	0.64
6	rigid ins.,(expand.), 2 in.	2.00	7.90
7	steel stud, 5-1/2 in.	5.51	0.12
8	gypsum bd., 1/2 in., (#2)	0.50	0.46
9			
10			
11			
12			
	Total or (Layer 0)	19.52	21.89

Condensation Analysis

The wall system was analysis for condensation during the summer and winter as well. As seen with the graph, no condensation occurs within the system in both conditions.



Layer	Description	RVap	V Drp	VpCo
1	air film (ext), 3/4 in.	0.001	0	0.64
2	concrete wall, 8 in.	1.908	42	0.63
3	cavity, 3 in.	0.025	1	0.63
4	AAC panel, 6 in.	6.00	1	0.63
5	cavity , 2.5 in.	2.50	23	0.62
6	rigid ins.,(expand.), 2 in.	0.515	11	0.62
7	steel stud, 5-1/2 in.	28.607	626	0.43
8	gypsum bd., 1/2 in., (#2)	0.197	4	0.43
9				
10				
11				
12				
	TOTAL or (Layer 0)	32.464	708	(0.64



Layer	Description	RVap	V Drp	VpCo
1	air film (ext), 3/4 in.	0.001	0	0.16
2	AAC panel, 6 in.	6.00	4	0.16
3	cavity , 2.5 in.	2.50	0	0.16
4	paper, stand., (8mil)	0.023	0	0.16
5	plywood shtg., 1/2 in.	1.054	2	0.16
6	rigid ins.,(expand.), 2 in.	0.515	1	0.16
7	steel stud, 5-1/2 in.	28.607	54	0.18
8	gypsum bd., 1/2 in., (#2)	0.197	0	0.18
9				
10				
11				
12				
	TOTAL or (Layer 0)	32.464	61	(0.16

12. Comparison of Existing and New Prototype Design

12.1 Structural Systems Comparison:

Structural Systems Comparison		
	Existing System: Two-way Flat Plate Post Tension	Prototype System: Composite Steel
Cost	$17.18/sq ft	29.28/sq ft
Structural Depth	8" slab	3 1/2 " slab 18" girder
Structural Weight	100 psf	54 psf
Fireproofing	2 hr (spray on)	2 hr
Effect of Column Grid	Must Re-align	-
Construction Difficulty	Difficult (West Coast)	Easy
Lead Time	Short	Long

Although the prototype system cost almost twice the amount of the existing system, its building structural weight is reduced by about 50%. The prototype system will require 8 inches of extra ceiling height due to the depth of the girders, resulting in an increase of approximately 5 feet in the overall building height. The decrease in building weight can reduce the base shear of the building during a seismic event, which can help reduce the amount of damage received by the building. Concrete material is replaced with steel, which results in less material usage and less waste. As post-tension is not a common practice on the west coast, labor cost may be more expensive. The new prototype system is the better choice for its location in Los Angeles, California.

12.2 Façade Material Comparison:

Facade Material Comparison		
	Existing System: Face Brick Veneer	Prototype System: 6" Autoclaved Aerated Concrete Panels
Material Cost	$2.75/sq foot	$2.30/sq foot
R-Value	0.8/ inch	1.25/ inch
Thickness	4"	6"
Structural Weight	38.7 psf	17 psf
Fireproofing	1.25 hr	4 hr
Construction Difficulty	Medium	Easy

AAC is cheaper and provides speedy construction and a reduced labor cost. AAC consumes 50% to 20@ less energy than that needed to produce concrete and CMUs. Its thermal efficiency can significantly reduce the cooling loads for the building to comply with California energy conservation codes. There is also no construction waste as the material is 100% recyclable. Its usage can also reduce the building weight compared with the brick veneer, and reduce the number of façade interfaces of the existing design to reduce the chances of moisture penetration and infiltration. AAC's high URL fire rating can also help prevent seismic fire related damage. The use of the AAC panels does result in an increase in the thickness of the exterior walls up to 3 inches, but it will deliver a better building envelope performance resulting in energy cost savings, and a worthy investment for a building in a high seismic zone. Another disadvantage would be the cost of anchoring connections

12.3 *Green Roof Retrofit Comparison*

Roof Retrofit Comparison		
	Existing System: PVC Single Ply System	Prototype System: Green Roof Retrofit
Cost	$ 3.75/sq ft	15$/sq ft
R-Value	10.75	23.4
Structural Weight	40 psf	50 psf
Reflectivity	95%	-
Emittance	80%	-
Solar Reflectance index	110	-
Average Survice Life	9.5	50
Maintenance	Medium to High	Low

The usage of an extensive green roof can contribute to the reduction of cooling loads and thus energy consumption and cost by the building. In a life cycle cost analysis, it can increase the service life of the roof membrane, and can help increase the revenue of the residential building. Environmental improvements includes improved water and air quality, which is an emerging issue in Los Angeles due to traffic and air pollutions. It can also reduce reflection and transmission of heat and glare to surrounding buildings, and mitigate urban heat-island effects. It can be used to control storm water runoff and improve the aesthetic environment. Although the initial cost at first may be expensive, it will pay off in a least two years mainly from revenues and the reduction of mechanical loads. With such a vast roof surface area, Ingleside and King Farm can significantly benefit from the implementing a green roof system. Its extra dead load bears no burden to the structural system as demonstrated in the design calculations.

13. Conclusion

Better performance always comes with a cost, however there are paybacks that out weights the dollar amount. In the case of retrofitting a building for seismic resistance, the reward could be the reduction in lives lost, medical costs, loss of tenants, loss of assets within the building, and loss of building functions. Other benefits include reduction in insurance premiums, increase in property value, and higher income from tenants.

Redesigning a prototype design of Ingleside at King Farm for Los Angeles, California will be costly due to the special requirements by codes to make the building safer during and right after a seismic event. Indirect damage includes fires caused by seismic activity, which can weaken the structural system and cause structural failures. In the case of extremely high seismic activity, such as the Northridge Earthquake in 1994 due to a combination of direct shear and poor soil conditions, retrofitting the building design and to resist seismicity can result in significant savings due to decrease in damages and delayed building functions, and more importantly, increasing the safety and survival rate of the occupants.

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15. Appendix

15.1 LRFD Composite Beam Design

L = 30 ft Spacing = 10 ft b = (30/8)2 = 7.5 ft f_c = 3.5 ksi F_y = 36.0 ksi 3¼-in. slab + 3-in. deck

Loads

LL	100	(1.6) =	160
DL	60	(1.2) =	72
Total	160	(1.45) =	232 psf

A_i = (10)(30)(2) = 600 sq. ft
$L = L_o(0.25 + 15/\sqrt{A_i}) = 100(0.25 + 15/\sqrt{600}) = 86$ psf
Reduction = 100 − 86 = 14 psf
W = 10 [232 − 14(1.6)] / 1000 = 2.1 kip/ft
$M_u = 2.1(30)^2/8 = 236.3$ kip-ft

Preliminary Beam Section

$$\text{Beam weight} = \left[\frac{M_u(12)}{(d/2 + Y_{con} - a/2)\phi F_y}\right]$$

For M_u = 236.3 kip-ft, Y_{con} = 3 + 3.25 = 6.25. Assume a = 1½ in.

Preliminary Beam Selections

Section Nom. Depth	$\frac{M_u(12)(3.4)}{\phi F_y}$	$d/2$	$Y_{con} - a/2$	Wt.
14	315.1	1	5.5	25.2
16	315.1	8	5.5	23.3
18	315.1	9	5.5	21.7

Try W16x26, F_y = 36, $Y2$ = 5.5, M_u (Req'd) = 236.5

From pg. 4-23 LRFD Manual (see next page), for ΣQ_n = 242 kip, ϕM_n = 248 kip-ft

$$a = \frac{242}{(.85)(3.5)(7.5 \times 12)} = 0.90 \text{ in.}$$

$$Y2 = 3 + 3.25 - 0.90/2 = 5.8 \text{ in.}$$

For ¾-in. dia. headed stud with f'_c = 3.5 ksi

Conc. wt. = 115 pcf from LRFD Manual, pg. 4-7.
Q_n = 19.8 kips/stud
No. of studs req'd = (242/19.8)2 = 26

Use: W16x31, F_y=36ksi with twenty-six ¾-in. dia. headed studs.

LRFD Composite Beam Design Continued

Deflection Summary

Assume 10 psf ambient LL for long term deflection
76 psf LL for short term deflection
$M_{LL} = 85.5$ kip-ft $M_{DL} + 10$ psf $= 78.8$ kip-ft

Shored Construction

$$\Delta_{LL} = \frac{ML^2}{161\ I} = \frac{85.5\ (30)^2}{161\ (1400)} = .34 \text{ in.}$$

$$\Delta_{DL\ long\ term} = \frac{78.8\ (30)^2}{161\ (1185)} = \underline{.37 \text{ in.}}$$

Total Defl. = .71 in.

Unshored Construction

(Assume camber overcomes dead load deflection.)

Long term load causing deflection 9 psf DL + 10 psf LL = 19 psf $M = 21.4$ kip-ft

$$\Delta_{LL} = .34$$

$$\Delta_{DL\ +\ 10\ psf\ LL} = \frac{21.4\ (30)^2}{161\ (1185)} = \underline{.10}$$

Total Defl. = .44 in.

15.2 Inverted V-Brace Design

Parameters

ρ=	1.3	
S_{DS}=	1.104	
DL=	40	kips
S=	0	
H=	0	
LL=	20	kips
QE=	90	Kips

ASTM A 500, Grade B, Fy=46 ksi, Fu (minimum tensile stress) = 58 kisi

$P_{uc}=1.2D + 0.5L + 0.2S + \rho Q_E + 0.2S_{DS}D$

P_{uc}= 183.832 kips, compression This Governs

$P_{ut}=0.9D + 1.6H + 0.2S - \rho Q_E - 0.2S_{DS}D$

P_{uc}= -89.832 kips, tension

Unbraced Length

l=H/sinϴ H=0.5*span of 30 ft

(Brace angle) ϴ= 20

l=15/sin20= 16.430339 **use unbraced length 17 feet**

K=1K*l=1*17=17

Use HSS 9x9x5/8

$\phi_c P_n$ = 607 kips

> 183.8 kips ok

Slenderness Ratio required by AISC-Seismic

$l/9=1000/(fy)^{0.5}$

= 154.30335

l/9= 22.666667

< 154.3 ok

Thickness Ratio required by AISC-Seismic

$(b \text{ or } h)/t_w=0.64(Es/Fy)^{0.5}$

= 16.069415

9/0.58= 15.517241

< 16.06 ok

15.2 Inverted V-Brace Design Continued

Given

Frame Column: W14 x 342	Grade 50	Fu=65	ksi
Frame Beam: W18x60	Grade 50	Fu=65	ksi
Center to Center span of frame=	30 ft		
Factored Axial Load on column P_{uc}=	820kips		
DL= 2.3kip/ft			
LL= 1.04Kip/ft			

Beam Critical Parameters

1. Beam depth = 18"< 6" ok FEMA Table 350
2. Span-depth-ratio = 360/18= 20 > 7 permitted minimum for SMRF - ok
3. Weight of 60 psf < 200psf less than the minimum permitted - ok
4. $b_f/2t_f$ = 7.56/(2*0.695)= 5.44
   Max permitted= $52/(Fy)^{0.5}$= 7.35 ok
5. Thickness of flange t_f=0.695 < 1.75 ok
6. Beam material A572 Grade 50 permitted by FEMA 350 ok
7. Flange reduction will be within FEMA guidelines ok

Column Critical Parameters

$$M^*_{pc} = \Sigma Z_c(F_{yc}-(P_{uc}/A_g))$$

M^*_{pc} = 2*672*(65-(820/101.6))= 76448.32 kip-in
= 6370.69 kip-ft

M_c = 2 x 2373 = 4746 kip-ft

M^*_{pc}/M_c = 6370/4746= 1.34
> 1.0 ok

Connection Design

1. Determine length and location of beam flanges
   a= $(0.5 \text{ to } 0.75)b_f$ = 0.5*11.5 5.77in.
   b= $(0.65 \text{ to } 0.85)d_b$ = 0.75*33.5 25.13in.

2. Determine depth of flange reductio, c

   Assume c=0.2 bfb= 0.2*11.5 = 2.33 in

Z_{RBS}= $Z_{zb}-2ct_{fp}(d_b-t_{fb})$

= 514-2*2.33*0.96(33.6-0.96)= 367.9in^3

15.3 Moment Connection Design

$M_{pr} =$ $C_{pr}R_yZ_bF_y$
=1.15*1.1*367.98*50 =23274.74 kip-in
=1939.56 kip-ft

$V_{gravity} =$ (1.2D+0.5L)*L'/2
=(1.2*2.3+0.5*1.04)*27.9/2 =45.76 kips

$V_{seismic} =$ 2Mpr/L'
=2*1939.6/27.9 =139.04 kips

$V_p = V_{gravity} + V_{seismic}$
=45.76+139 =184.76 kips

$M_f =$ 1939.6+184.8*42/12 **=2586.40** kip-ft

$M_{pc} =$ $C_{pr}R_yZ_bF_y$
=1.15*1.1*514*50 =32510.50 kip-in
=2709.21 kp-ft

M_f=2586 is less than Mpc=2709 ok

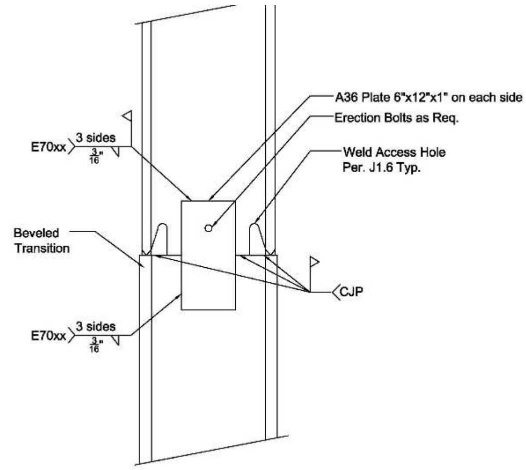
3. $M_c = M_{pr}+V_p(x+d_c/2) = M_{pr} + V_pS_h$
=1939+184.8*(27.5/12) =2362.5 kip-ft

4. Calculate shear at column face
$V_f=2M_f/(L - d_c) + V_g$
=2*2586/(30-17.5) =413.76 Kips

Bolt capacity ϕR_n =43.5 ϕ=0.75
43.5/58 =58 kips
No. of bolts=413.76/58 7.121

Use a 8 1-in diameter A325 bolt as an alternative to welding

15.4 Column Splice Design



Column Splice Design AISC 341, 13.5 use W16×77 $F_y = 65$ ksi

$V_u = V_n = 0.6 F_y A_w = 0.6 F_y (d - 2t_f) t_w$

$= 0.6(65\text{ ksi})[(16.5) - 2(0.76)]0.455 = 265.8$ kips

$M_u = \frac{1}{2} M_n = \frac{1}{2} F_y Z = \frac{1}{2}(65)(134\text{ in}^3) = 4355$ kip-in

Check splice-plate shear strength

$\phi V_n = \phi 0.6 F_y A = 0.9(0.6)(6\,1/2\text{ in})\,5/8\text{ in} = 110\text{k}$ per plate ok

Check splice weld strength

$C_1 = 1.0$ for E70

$L = 12$ in

$kL = 12$ in, $k = 1$ $x = 0.33$

$L = aL + xL$

$ex = aL = L - xL = 0.667L$

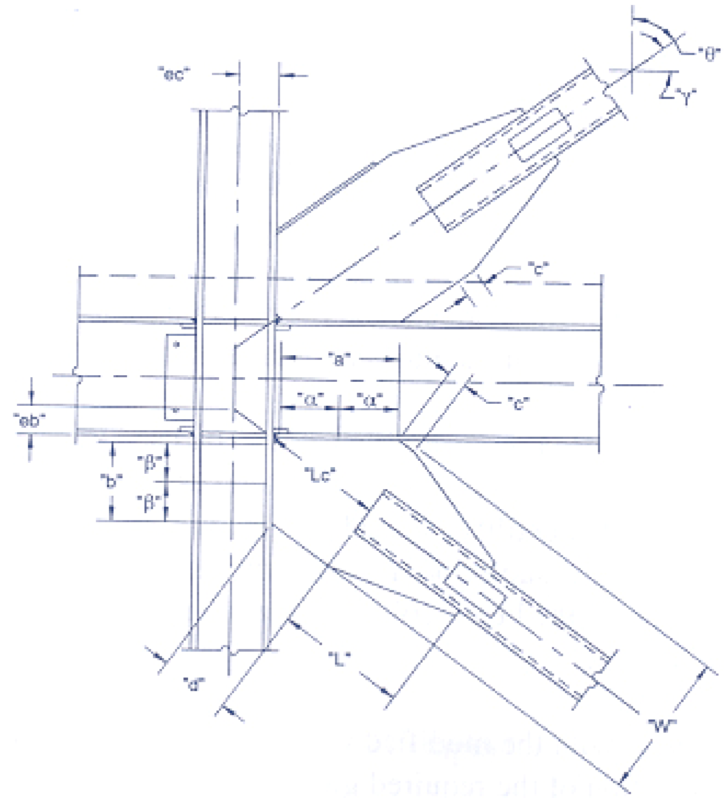
$a = 0.667$

$D_{min} = 1/16'' \cdot \frac{P_u}{C C_1 L} = \frac{1}{16}'' \frac{(1/2 \times 183^k)}{7.03^{k/in}(1)(12'')} = \frac{4.6}{36.36} in < \frac{3}{8} in$

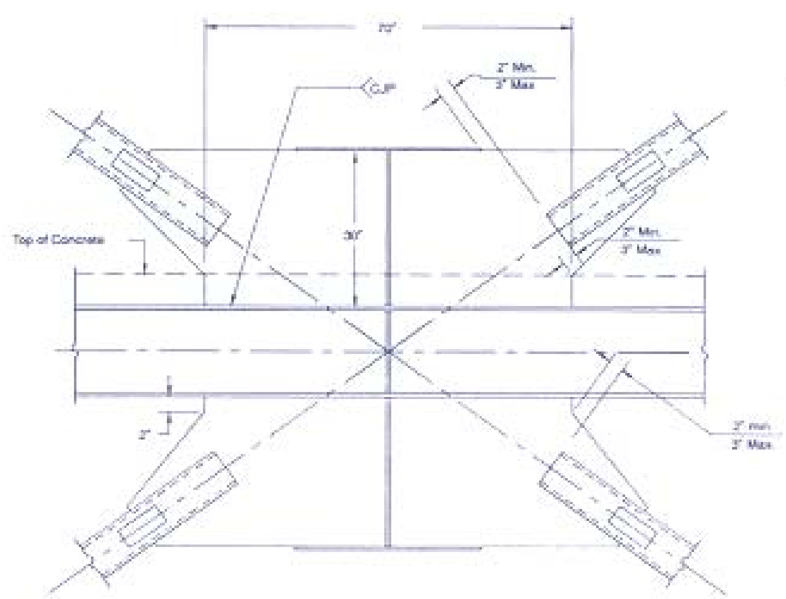
ok ✓

use 3/16" is fine

15.5 Brace-Beam-Column Connection Diagram



15.5 Brace-to-Beam Midspan Connection Diagram



15.6 Brace-Beam-Column Connection Design

Brace - beam - colum connection design (HSS 9 x 9 x 5/8), $F_y = 42$ ksi

$P_u = R_y F_y A_g = 1.4 (42 \text{ ksi})(18.7) = 1099.6$ kips   AISC 341 13.3a

$W_{min} = \dfrac{R_y F_y A_g}{\phi F_t t} = \dfrac{1099}{0.9(50)(0.58)} = 24 \text{ in}^2/t$

Assume $t = 7/8$ in

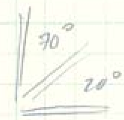
$W_{min} = \dfrac{24}{7/8} = 27.9$ in $\Rightarrow$ 28 in

$W_{min} = \dfrac{1099}{0.75(65)(7/8 \text{ in})} + W_{(hole)} = 25.7 + 1.5 = 27.3$ in

- Gusset Dimensions

$a = \dfrac{W_{min}}{2\cos\theta}$ $b = \dfrac{W_{min}}{2\sin\theta}$ where $\theta \cong 70°$, $\gamma \cong 20°$

$a = 39.9$ in $b = 14.5$ in



use $a = 40$ in
$b = 16$ in

- Horizontal + vertical forces to the beam & column

$V_{ub} = V_{uc} = \dfrac{P_u}{2}\sin(\gamma) = \dfrac{1099}{2}\sin 20° = 187.9$ kips

$H_{ub} = H_{uc} = \dfrac{P_u}{2}\cos(\gamma) = \dfrac{1099}{2}\cos 20° = 516.3$ kips

- Check vertical section of gusset plate   [AISC 360, J4-1]

$P_u = \sqrt{H_{uc}^2 + 3V_{uc}^2} = \sqrt{516.3^2 + 3(187)^2} = 609$ kips

$\phi P_n = \phi b t F_y = (0.9)(16)(7/8 \text{ in})(50 \text{ ksi}) = 630$ kips

$630 > 609$ kip   ok ✓

Brace-Beam-Column Connection Design Continued

• Check horizontal section of gusset plate

$$P_u = \sqrt{V_{ug}^2 + 3H_{ug}^2} = \sqrt{(187)^2 + 3(516.3)^2} = 549 \text{ kips}$$

$$\phi P_n = \phi a t F_y = 0.9(40)(7/8)(50) = 1575 \text{ kips}$$

$$1575 > 549 \text{ kips} \quad \text{ok} \checkmark$$

• Check column web for yielding [AISC 360, J10-2] (W16x77 grade 50)

$$R_u = H_{uc} = 516.3 \text{ kips}$$

$$\phi R_n = \phi(b + 5k) F_y t_w = 1[16'' + 5(1.5)]50(0.45'') = 528 \text{ kips}$$

$$\phi R_n > R_u \quad \text{--- ok} \checkmark$$

• Check column web for crippling at max compression force

$$P_{max} \le R_y F_{cr} A_{br} = (1.4 \times 33.6 \times 18.7) = 879.6 \text{ kips}$$

$$R_u = H_{uc} \frac{P_{max}}{R_y F_y A_{br}} = \frac{516.3\,(879.6)}{1099} = 413 \text{ kips} \quad \text{[AISC 360 J10-4]}$$

$$\phi R_n = \phi\left[0.8 t_w^2 \left[1 + 3\frac{b}{d}\left(\frac{t_w}{t_f}\right)^{3/2}\right]\sqrt{\frac{E F_y t_f}{t_w}}\right]$$

$$\phi R_n = 0.75\left[0.8(0.45)^2\left[1 + \frac{3 \times 16}{9}\left(\frac{0.45}{0.76}\right)^{3/2}\right]\sqrt{\frac{29000(50)(0.45)}{0.76}}\right] = 1210 \text{ kips}$$

$$\phi R_n > R_u \quad \text{--- ok} \checkmark$$

Brace-Beam-Column Connection Design Continued

- check beam web for yielding (W18×60 grade 50)

$R_u = V_{ub} = 187.9$ kips

$\phi R_n = \phi(a+5k)F_y t_w = 1.0[16+5(1.15)]50 \times 0.415 = 451$ kips

$\phi R_n > R_u$ ... ok ✓

- Check beam web for crippling

$R_u = V_{ub} \cdot \dfrac{P_{max}}{R_y F_y A_{br}} = 187.9\dfrac{(879)}{1099} = 149.8$ kips

$N = a = 40$ in

$N = \dfrac{d}{2} = \dfrac{18}{2} = 9$ in

$\phi R_n = \phi\left[0.8 t_w^2\left[1+3\frac{a}{d}\left(\frac{t_w}{t_f}\right)^{3/2}\right]\sqrt{\frac{EF_y t_f}{t_w}}\right]$

$\phi R_n = 0.75\left[0.8(0.45)^2\left[1+3\frac{40}{18}\left(\frac{0.45}{0.695}\right)^{3/2}\right]\sqrt{\frac{29000(50)0.695}{0.45}}\right]$

$= 813.4$ kips

$\phi R_n > R_u$ ... ok ✓

- Design brace to gusset welds

$L_{min} = \dfrac{(1/4)P_u}{\phi\frac{\sqrt{2}}{2}(s)0.6F_{EXX}} = \dfrac{(0.25)(1099)}{0.75\left(\frac{\sqrt{2}}{2}\right)(1/2)(0.6)(70)}$

$L_{min} = 24.7$ in $> 2(9) = 18$ in

width of brace

$24.7 > 18$

∴ no shear lag. ok ✓

Brace-to-Beam Midspan Connection Design

Brace-to-beam mid-span connection design

- Calculate max horizontal forces

$$V_{con} = (R_y F_y A_g - P_{max}) \sin(\gamma) = 1099 - 880 \sin 20 = 798 \text{ kips}$$

$$H_{con} = (R_y F_y A_g - P_{max}) \cos(\gamma) = 1099 - 880 \cos 20 = 272 \text{ kips}$$

$$M_{con} = H_{con}\left(t_{slab} + \frac{d_b}{2}\right) = 272(6.25 + 9\text{ in}) = \underline{287.25 \text{ k-in}}$$

- Calculate eccentricity and effective width

$$e = \frac{M_{con}}{V_{con}} = \frac{287.25}{798} = 0.359 \text{ in}, \quad W = 70 \text{ in}$$

$$W_{ef} = \sqrt{4e^2 + W^2} - 2e = \sqrt{4(0.359)^2 + 70^2} - 2(0.359)$$

$$W_{ef} = 6.95 \text{ in}$$

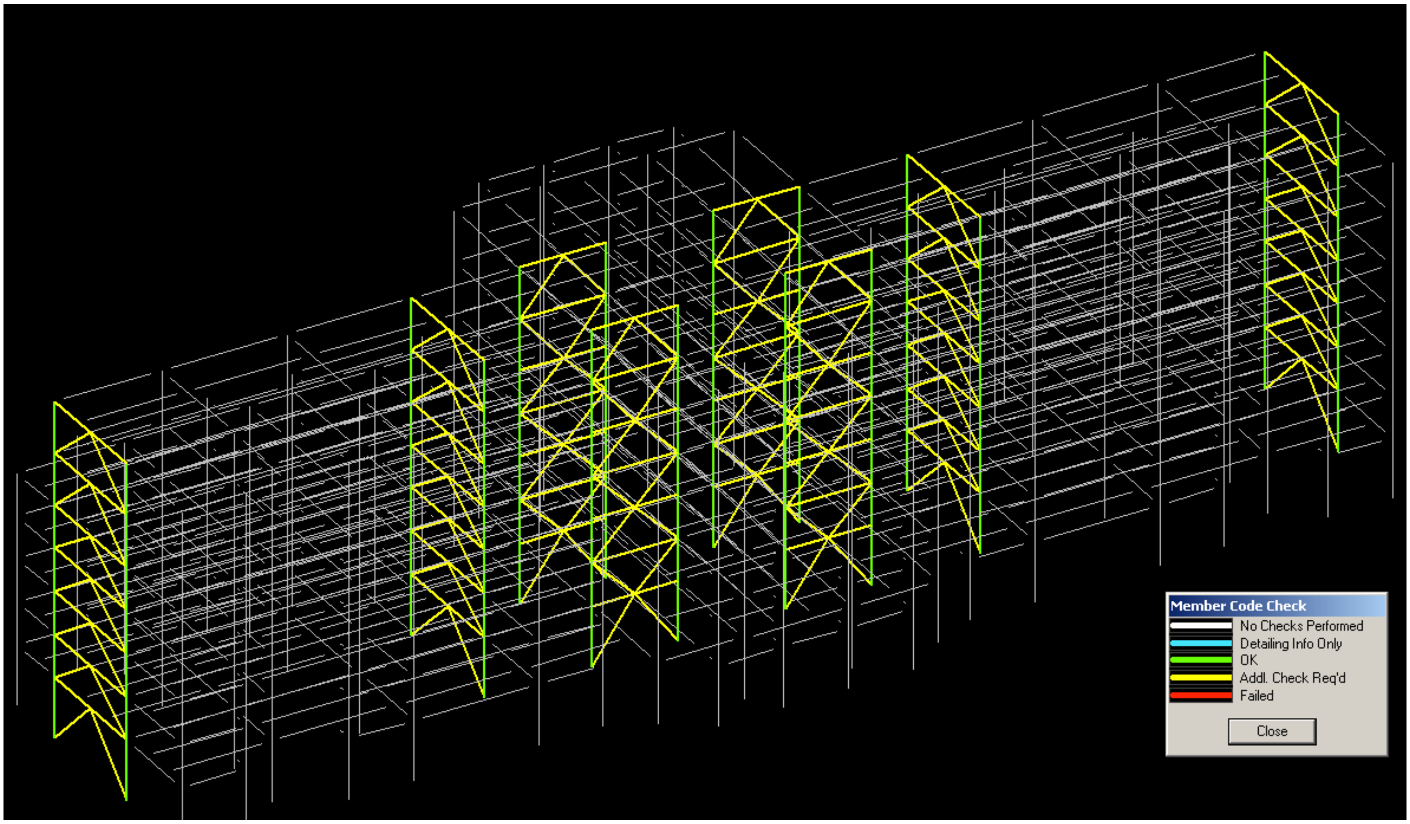
- Check combined stresses

$$\sqrt{\left(\frac{V}{\phi F_y L_{ef} t}\right)^2 + 3\left(\frac{H}{\phi F_y L t}\right)^2}$$

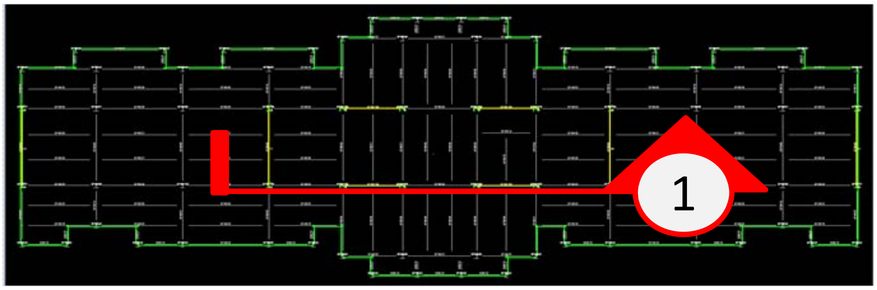
$$= \sqrt{\left(\frac{798}{0.9(50)(6.96)\,7/8}\right)^2 + 3\left(\frac{272}{.9(50)(70)\,7/8}\right)^2}$$

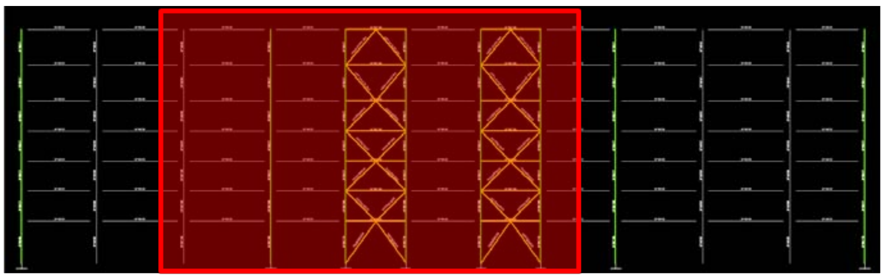
$$= \sqrt{10.15 + 0.029} = 3.19$$

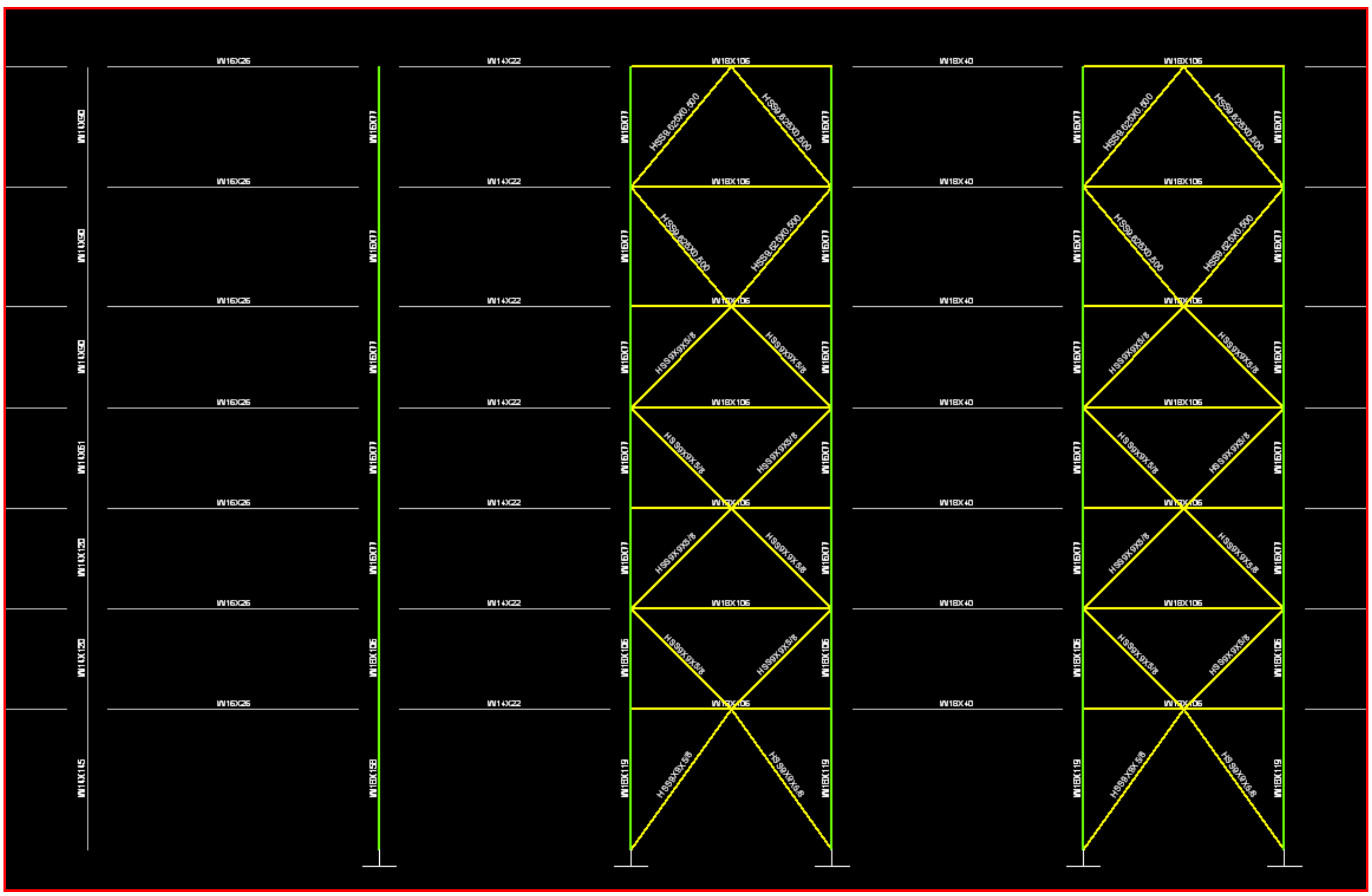
RAM Diagram: Section 1 Isometric View



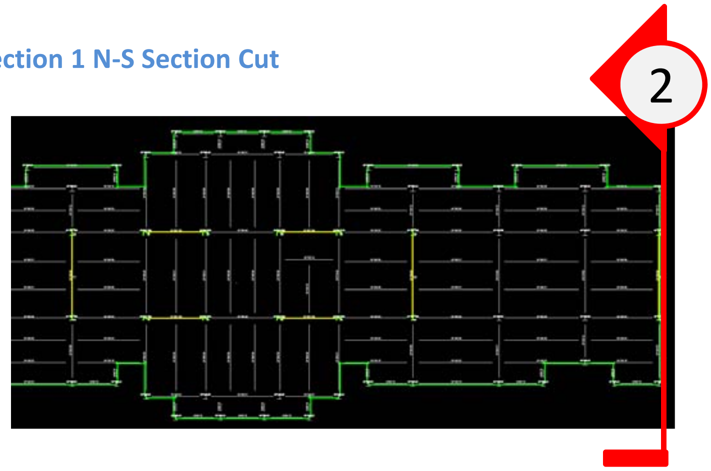
RAM Diagram: Section 1 E-W Section Cut

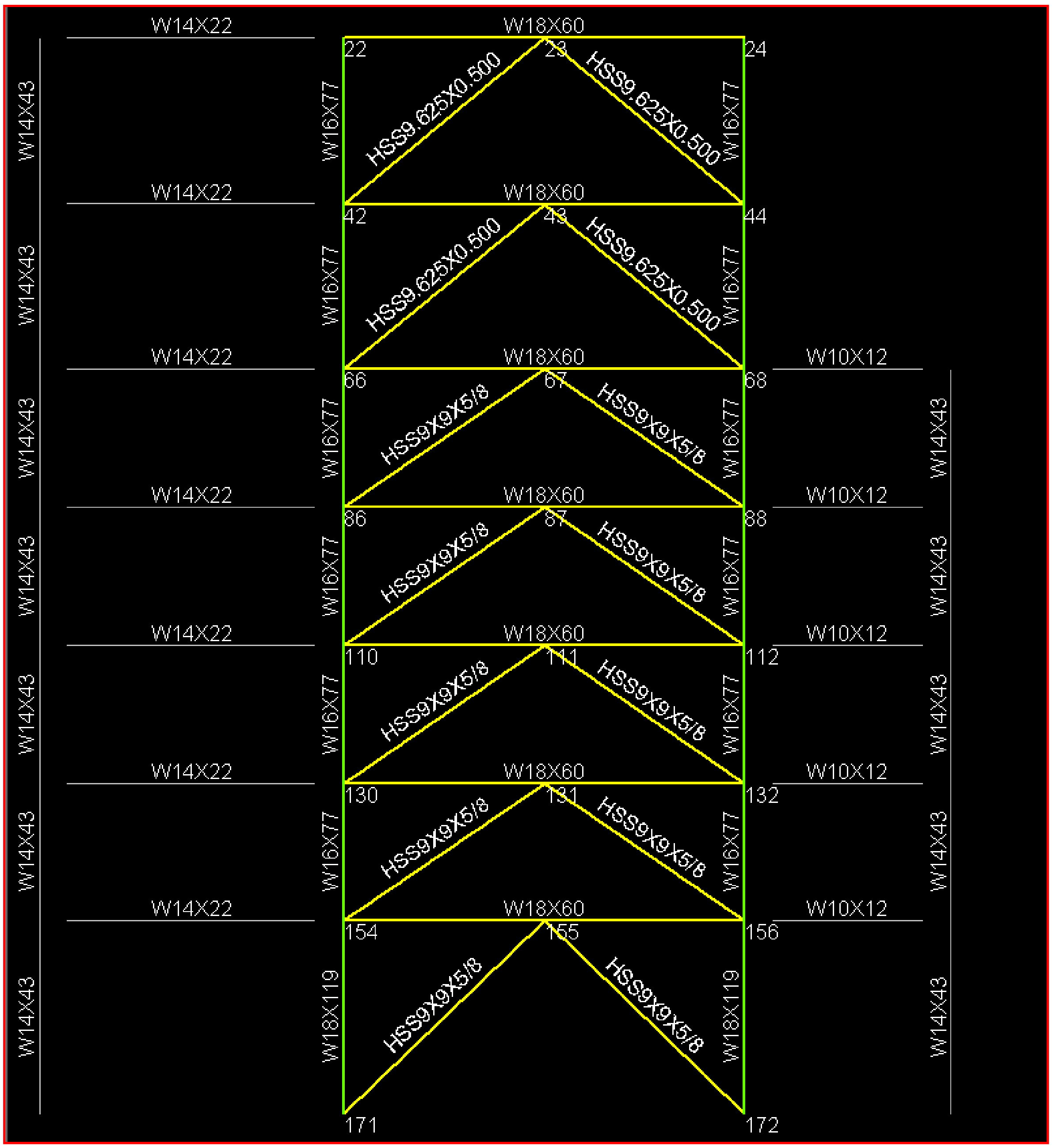




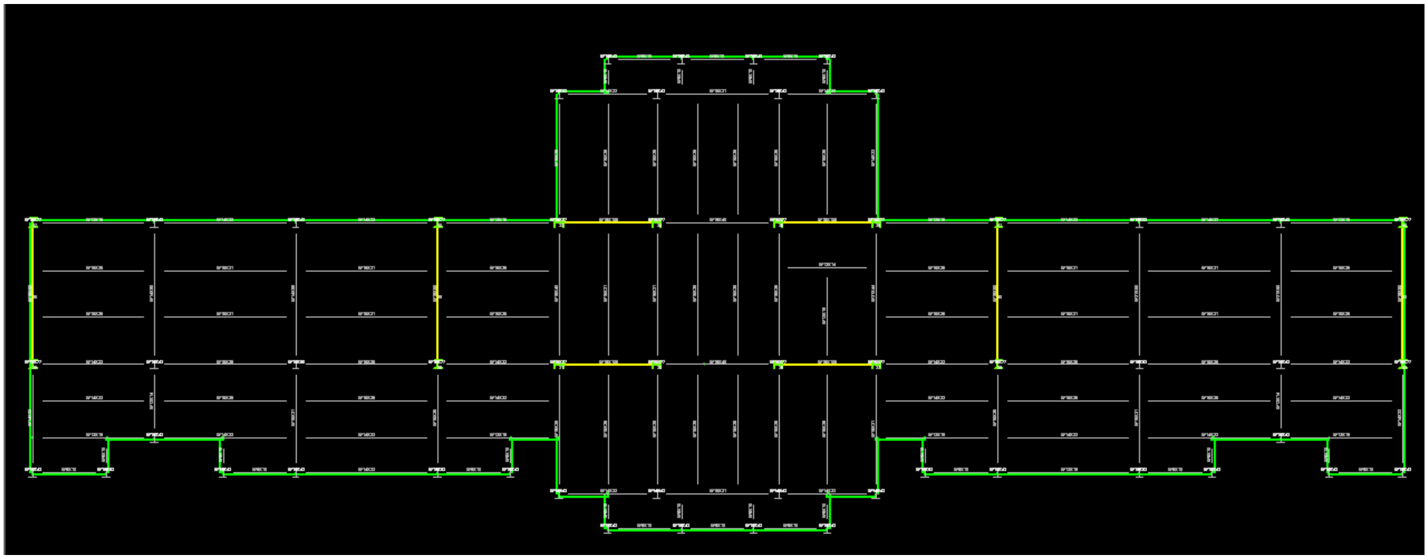


RAM Diagram: Section 1 N-S Section Cut

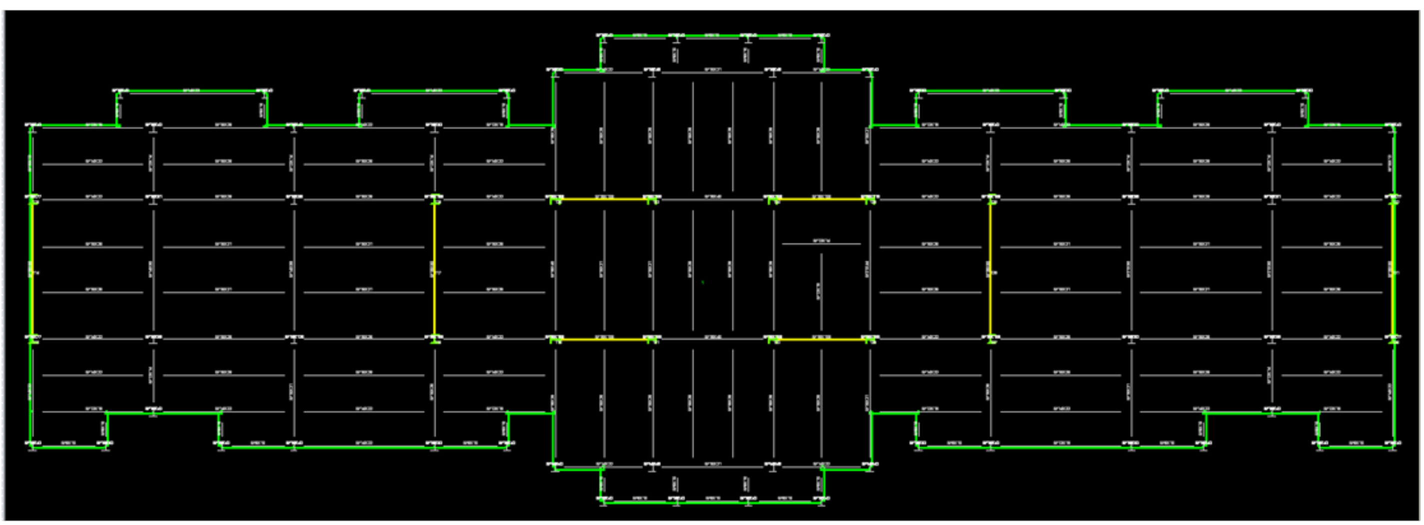




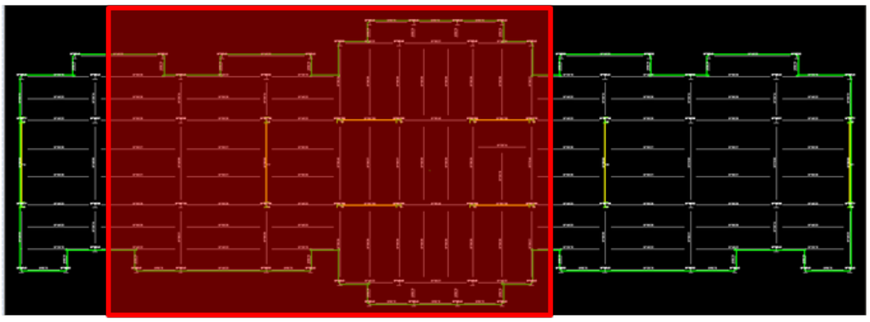
RAM Diagram: Section 1 7th Floor Plan

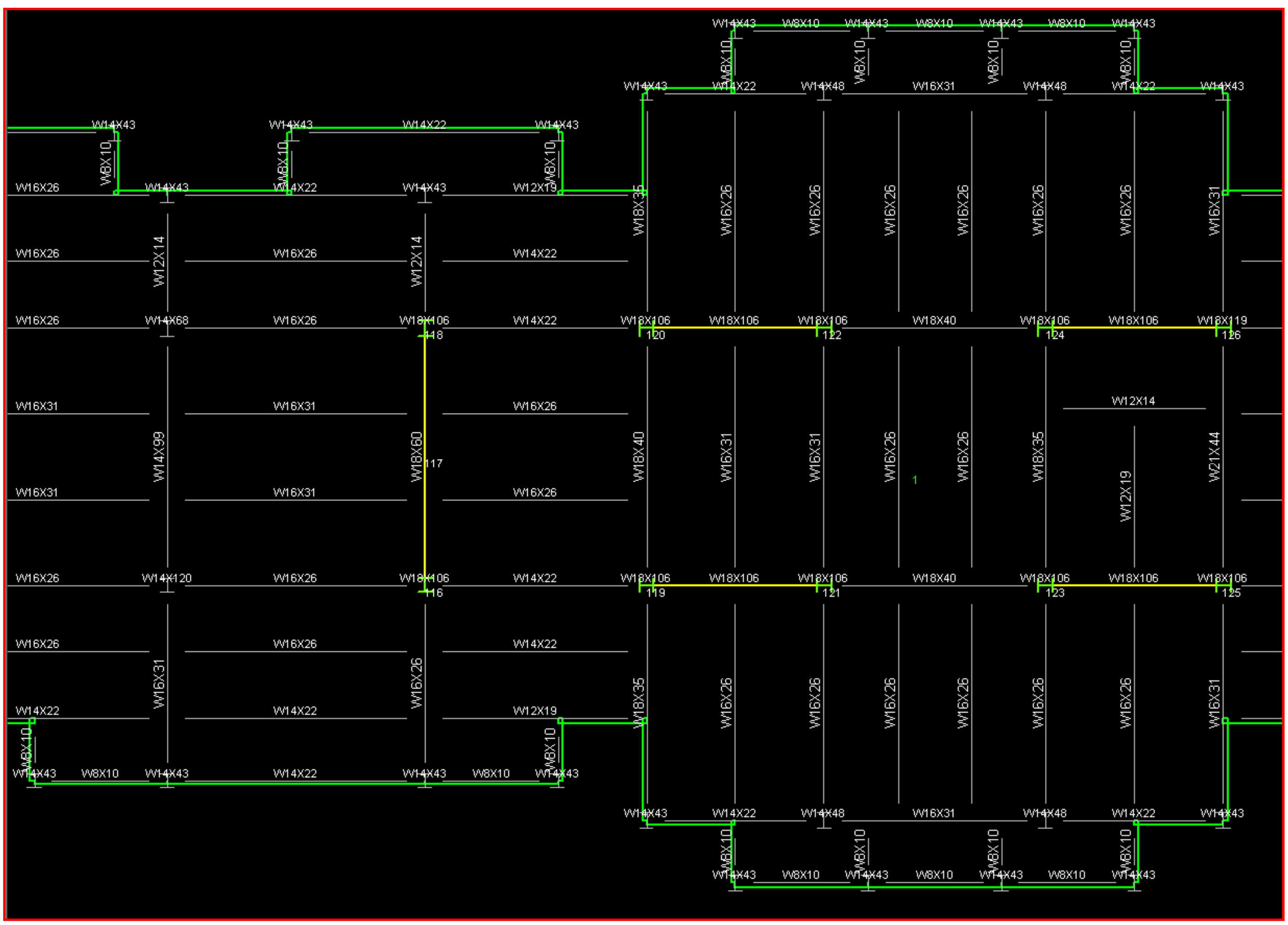


RAM Diagram: Section 1 7th Typical 2nd -6th Floor Plan

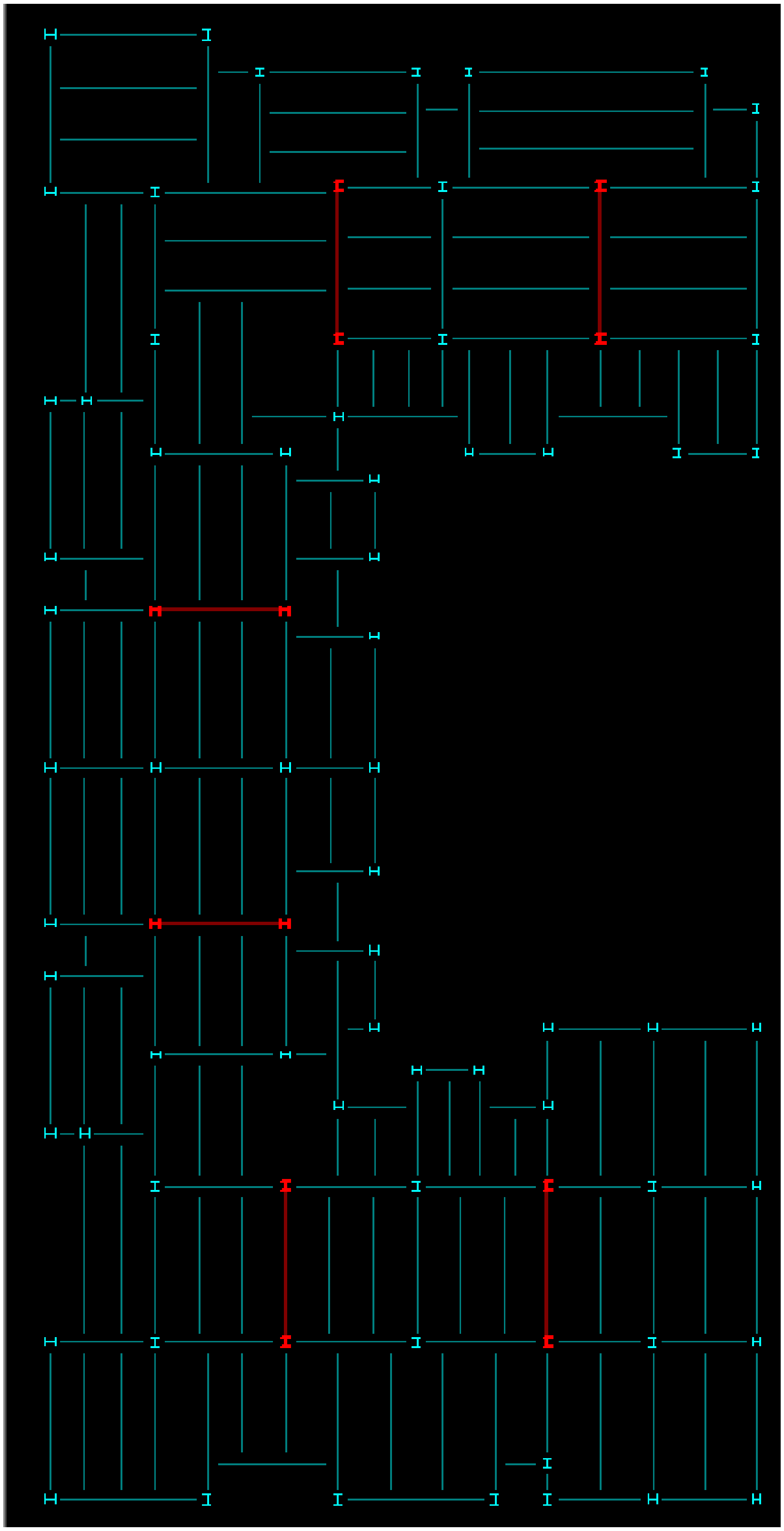


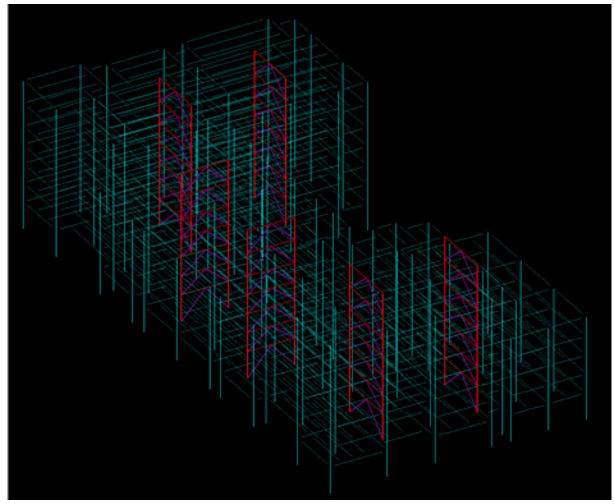
RAM Diagram: Section 1 Enlarged Floor Plan View





RAM Diagram: Section 2 Plan and Isometric View





Note:

Structural members were not manually designed for section 2 or 3 of Ingleside at King Farm. Although a RAM Model was constructed to obtain drifts and displacements easily.