Ingleside AT KING FARM

An Architectural Engineering Senior Thesis

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STEPHEN A. TAT

Structural Option 2008-2009

- Introduction
- Thesis Statement
- Existing Condition
- Breadth Study I
- Breadth Study II
- Structural Depth
- Conclusion

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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.

Project Profile

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 Match 2009
Delivery method: CM Agency
Bid Cost: GMP of $97 Million

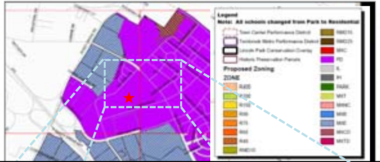
- Building usage
- Rising Demand for CCRC

Building Usage

- The building site is located between a residential and commercial zone.
- The building is continuous care retirement center (CCRC)

Features: (living units and rec. rooms)

- 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



- Building usage
- Rising Demand for CCRC

Rising Demand for CCRC

- Baby Boom generation has just begun to reach retirement age.
- According to Future Age magazine March 2009, approximately 78 million baby boomers will reach retirement age.
- People aged 50 and over made up 12% of the U.S. population and many (37% in a survey) have desired and demanded a luxurious lifestyle according to Fall 2007 Land Development, National Association of Home Builders.
- Boomers have $21 Trillion in spending power according to Money Magazine statistic in 2005.

- Over 4,000 CCRCs exists in the U.S. today.
- However, the supply of CCRCs falls short of the demand side for the Boomers.

- Thesis Statement
- The 1994 Northridge Earthquake

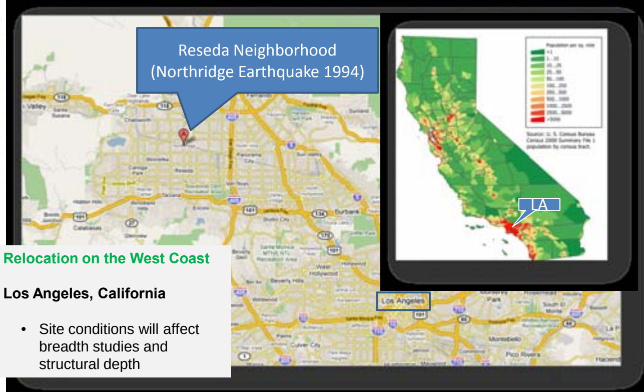
Thesis Statement

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 on the west coast of the United States.

Other Goals

Integrate my structural depth with my two breadth studies

- Utilizing emerging design principles such as green built
- Design of an alternative building envelope to adapt to the region's climate



Relocation on the West Coast

Los Angeles, California

- Site conditions will affect breadth studies and structural depth

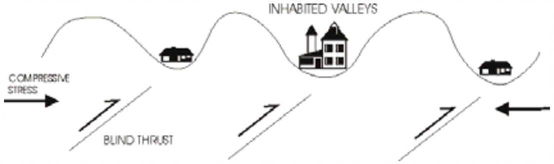
- Thesis Statement
- The 1994 Northridge Earthquake

The 1994 Northridge Earthquake

- Occurred on Jan. 17, 1994
- Lasting for about 20 seconds
- Moment magnitude of 6.7
- $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



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- Expansion Joints
- Two-way Post-tension Flat Plate System
- Gravity System
- Shear Walls
- Foundations

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Expansion Joints

There are three true 2-inch expansion joints built into the building.

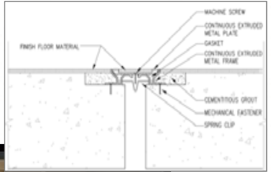
To reduce pre-stress losses in the tendons due to the shortening of the concrete slab caused by shrinkage or cooling, which will induce cracks around restraining boundaries (such as walls and beams)..

Another reason is for better constructability and the utilization time for faster construction.

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

Typ. Floor Exp. Joint Detail



- Expansion Joints
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Two-way Post-tension Flat Plate System

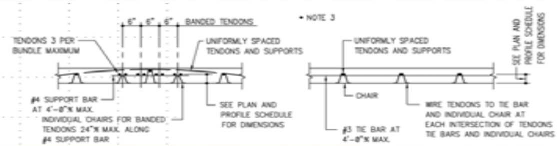
270 ksi unbonded ½ diameter 7 wire tendons.

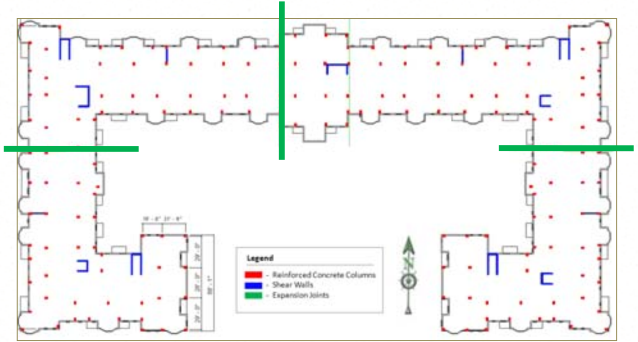
Concrete slabs are 8 inches thick for typical floors with a compressive strength of 4500 psi.

Irregular column gird of the building

Bays range from 15 feet to 29.5 feet.

Typical Tendon Support Bars





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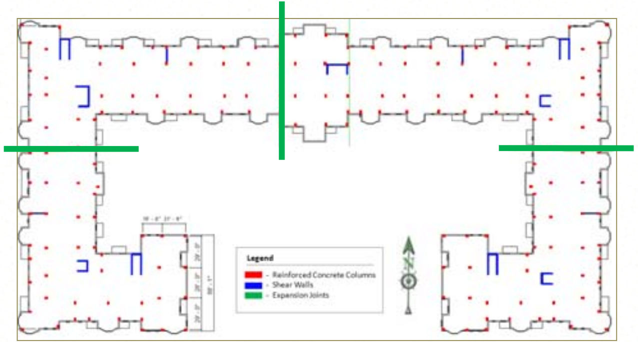
Gravity System

Over 140 reinforced columns, are either 18" x 30" or 12" x 30".

Shear Walls

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.

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.



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Foundations

- The loads from above are transferred down by 30” x 18” reinforced concrete columns with 10 #8 bars to spread footings of various sizes.
- Beneath the spread footings is 3 feet of compact fill and soil with a bearing capacity of 50 ksf.
- 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.
- 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;
- 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.

FOOTING SCHEDULE

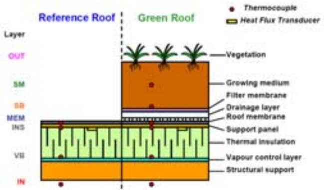
MARK	SIZE	REINFORCING
F1	4'-6" x 4'-6" x 14"	5#5 E.W. BOTTOM
F2	5'-0" x 5'-0" x 15"	4#6 E.W. BOTTOM
F3	5'-6" x 5'-6" x 17"	5#6 E.W. BOTTOM
F4/F4T	6'-0" x 6'-0" x 18"	6#6 E.W. BOTTOM *
F5/F5T	6'-6" x 6'-6" x 20"	5#7 E.W. BOTTOM *
F6	7'-0" x 7'-0" x 21"	6#7 E.W. BOTTOM
F7/F7T	7'-6" x 7'-6" x 23"	7#7 E.W. 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

- Green Design – Green Roof
- Green Façade Material
- Building Envelope

Breadth 1: Green Roof

Executive Order S-20-04:

- Green Building Initiative reducing the energy consumption.



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

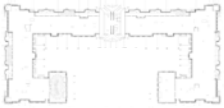
ealth and community benefits:

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

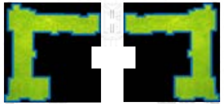
Comparison of Extension and Intensive Green Roof Systems

	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.

Existing Roof Membrane: White PVC Single Ply System



Placement of Green Roof on Prototype Design



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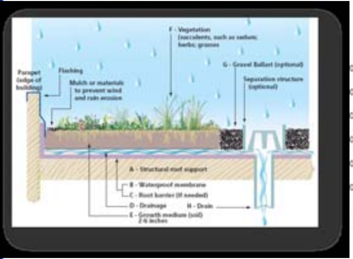
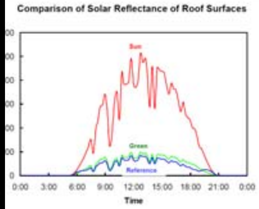
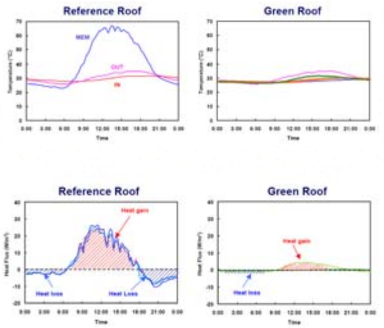
Breadth 1: Green Roof





A typical sunny summer day



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Breadth 1: Green Roof

Life Cycle Cost Analysis

Initial Cost

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

If all of the rain water 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

- Green Design – Green Roof
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Breadth 1: Green Façade Material

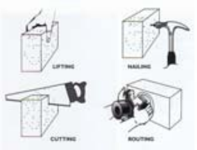
Use AUTOCLAVED AERATED CONCRETE (AAC) - green material

Seismic design

- The light weight of AAC in relation to its strength reduces the base shear.
- Fire-resistant characteristics provide further advantage against fires commonly associated with earthquakes. (4" thick panel gives UL fire rating of 4 hours)

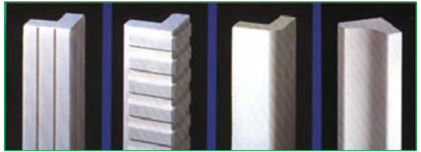
Building Envelope Benefits

- High durability = less maintenance, good life cycle cost
- Rapid construction and good workability = reduced labor
- Excellent thermal insulation
- Excellent sound absorption
- Lighter than light weight concrete (~80% air)




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)




- Green Design – Green Roof
- Green Façade Material
- Building Envelope

Breadth 1: Green Façade Material

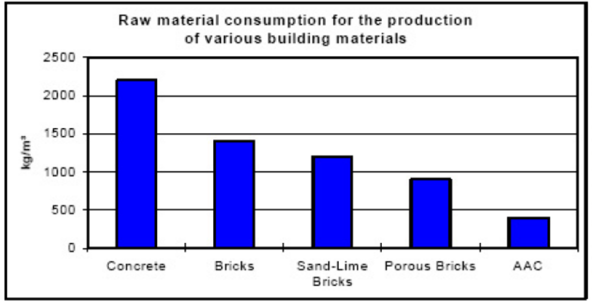
How is it made?

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.



- Green Design – Green Roof
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Building Envelope:

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 with an assembly consisting of 8” post tension slab and membrane roof water proofing system.

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.

On the 6th floor, the exterior veneer brick is replaced by a light-beige stucco with a reinforcing mesh behind it.

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.

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.

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- Building Envelope Redesign
- Building Envelope Analysis

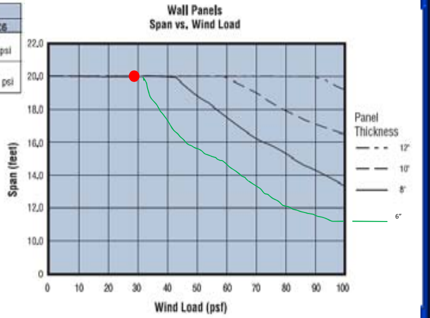
Breadth 2: Building Envelope Redesign

- Use light weight precast architectural panels 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..
- Address structural integrity and code compliances such as cladding issues and strength issues.
- Analyze air and moisture permeability, and thermal comfort provided by the recommended envelope assembly.

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

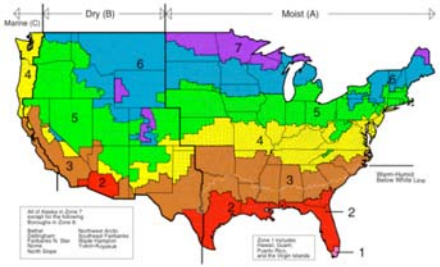
Deflection

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



- Building Envelope Redesign
- Building Envelope Analysis

Breadth 2: Building Envelope Redesign



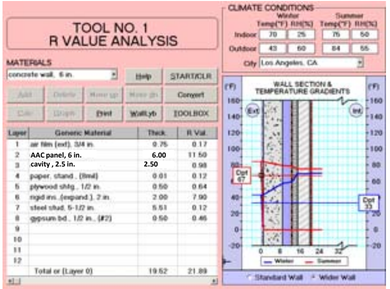
The following cavity wall assembly was designed for the prototype:

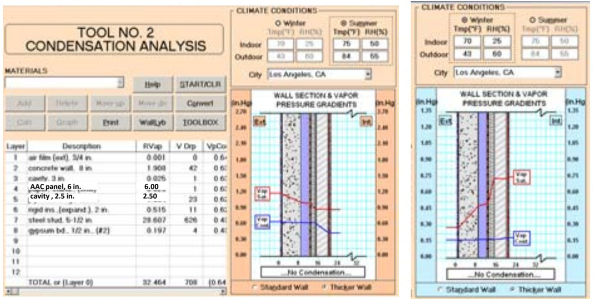
- Exterior Autoclaved Aerated Concrete Panels - 6.0 inch thick
- Air space and drainage plane - 2.5 inch
- Paper stand - 8 mil
- Plywood sheathing ½ inch
- Rigid insulation - 2 inch
- Steel Studs -5 ½ inch
- And gypsum board - ½ inch

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.

- Building Envelope Redesign
- Building Envelope Analysis

Breadth 2: Building Envelope Analysis





- Building Envelope Redesign
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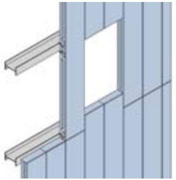
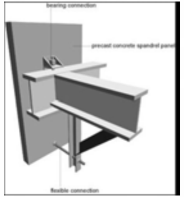
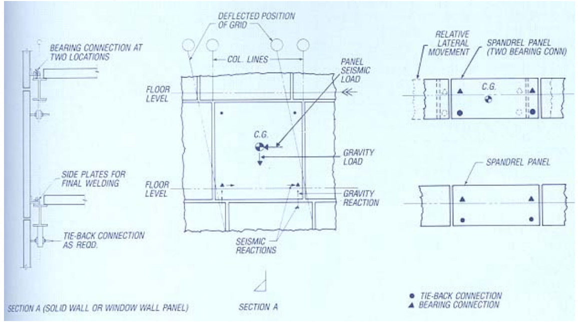


Breadth 2: Building Envelope Redesign

The most common moisture protection system used with precast concrete wall systems is a barrier system incorporating an adequate joint seal. In some cases where additional moisture protection is needed, the application of a sealer or a concrete coating is also used

Seismic codes require that heavy panels accommodate movement either by sliding or ductile connections. In high seismic zones 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.





- Goals and Criteria
- Placement of Expansion Joints
- Expansion Joints Specified by Code
- Floor System Design
- Lateral Design
- Seismic Analysis

Goals and Criteria

Goals:

- Utilize a steel structural system
- Preserve existing architectural design (Plans, story heights, aesthetics)
- Optimize lateral system

Codes and Standards:

- Meet seismic design criteria
- Strength design
- Serviceability requirements
- State/local codes (LOS ANGELES REGIONAL UNIFORM CODE PROGRAM (LARUCP))

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.*

Load Combinations:

All are based on LRFD design method

- $1.4(D + F)$
- $1.2(D + F + T) + 1.6(L + H) + 0.5(Lr \text{ or } S \text{ or } R)$
- $1.2D + 1.6(Lr \text{ or } S \text{ or } R) + (L \text{ or } 0.8W)$
- $1.2D + 1.6W + L + 0.5(Lr \text{ or } S \text{ or } R)$
- $1.2D + 1.0E + L + 0.2S$
- $0.9D + 1.6W + 1.6H$
- $0.9D + 1.0E + 1.6H$

- Goals and Criteria
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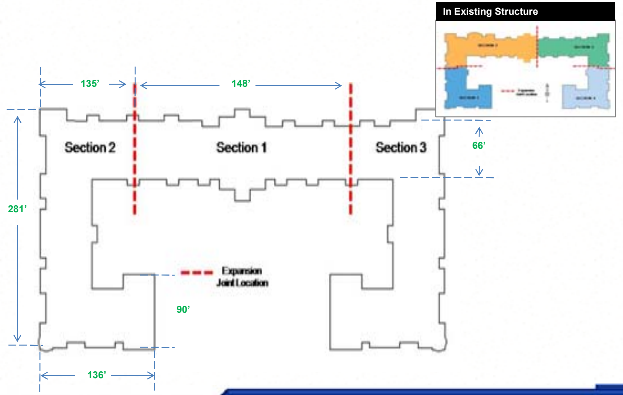
Placement of Expansion Joints

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.

Re align the column grid for more consistency in bays



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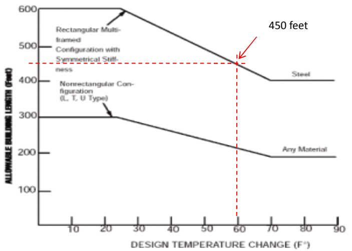
- Expansion Joints Specified by Code



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."

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.

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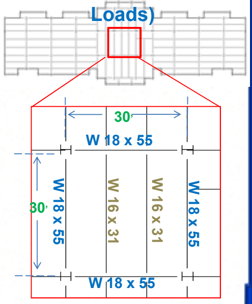
Floor System Design

Typical Floor System Loads

			Spaces		Façade
			Residential	Public	
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	-	-

Composite Beam and Formed Metal Deck	
Slab Design	
Use 18 gauge 3 inch formed deck fy=50ksi Max unshored span = 10.7 ft	Slab depth = 6.25" Light Weight Conc: f'c=3 ksi
Required Moment for Composite Beam	
Mu=1.2 (65.3) + 1.6 (112) = 258 ft kips	W 18 x 40 or W 16 x 31
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	W16x31 is ok for beam
Serviceability Criteria	
Live Load Deflection	L/360
Total Deflection Limit	L/240
Construction Load Deflection	L/360

Section 1 (Bay under Public Loads)



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Structural Depth

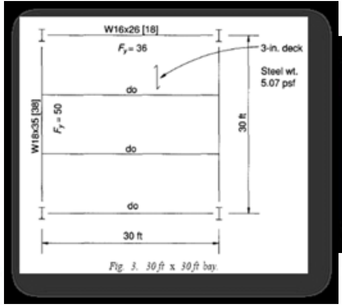
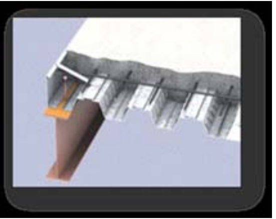


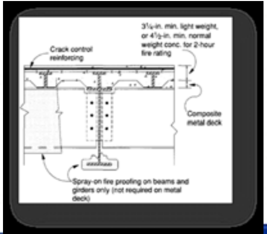
Fig. 3. 30 ft x 30 ft bay.

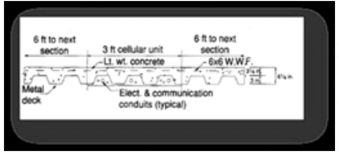


From AISC Steel Design Guide: Low-and Medium-Rise Steel Buildings

Percentage Comparison Square Foot Costs

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





Ingleside AT KING FARM

An Architectural Engineering Senior Thesis

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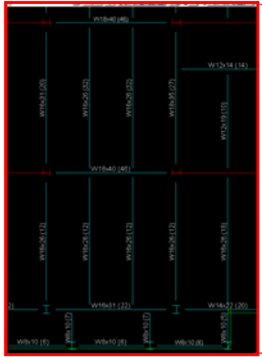
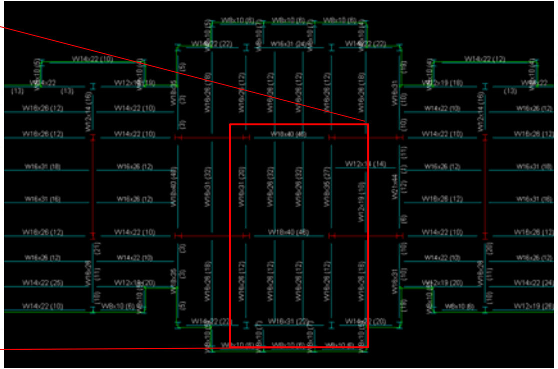
Structural Depth

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





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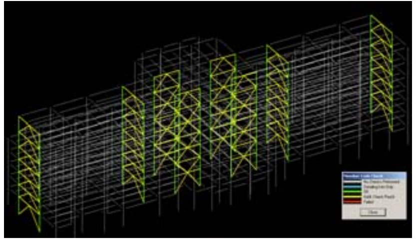
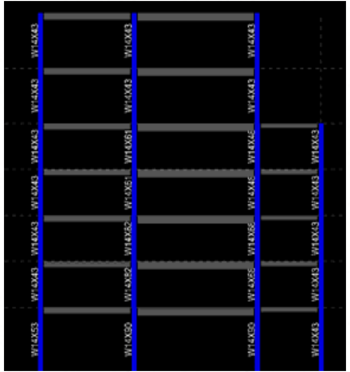
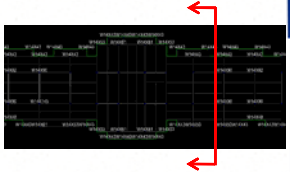
Floor System Design

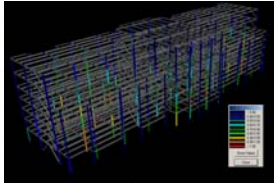
Column Members Sized

- Typically W 14 x 43 and W 14 x 90









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Seismic Parameter

Used in Equivalent Lateral Force Procedure

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

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 Cd	5	ASCE 7-05 Table 12.2-1 # B3
Over strength Factor Ω_0^g	2	ASCE 7-05 Table 12.2-1 # B3

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

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Structural Depth

Check for Irregularities Criteria

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 (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

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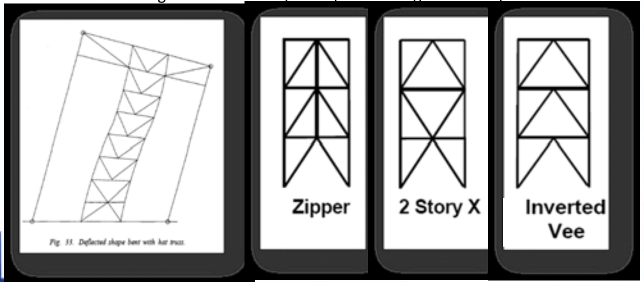
Lateral Design

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.

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.

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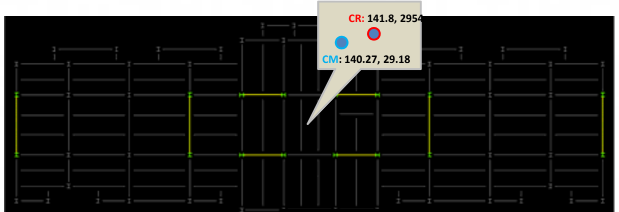
Lateral Design



Center of Mass for SCBF

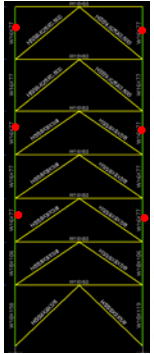
Story	X Direction	Y Direction
Roof	140.05	23.19
7	140.05	23.19
6	140.78	29.18
5	140.78	29.18
4	140.78	29.18
3	140.78	29.18
2	140.78	29.18

Center of Rigidity for SCBF

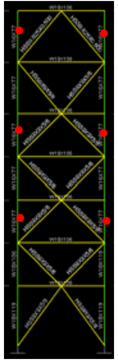
Story	X Direction	Y Direction
Roof	142.67	29.73
7	142.67	29.73
6	141.8	29.5
5	141.8	29.5
4	141.8	29.5
3	141.8	29.5
2	141.8	29.5

Redundancy Factor $\rho = 1.3$

Use Equivalent Lateral Force Procedure.



N-S Direction

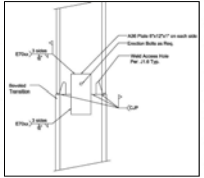


E-W Direction

Frame 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



X-braces are deemed better performers because of the buckling of the braces and excessive beam flexure during an earth quake.

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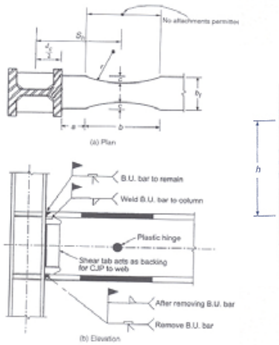
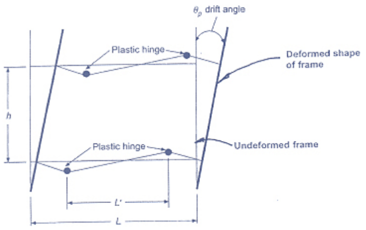
Lateral Design

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.

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. 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.





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Seismic Analysis

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

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[illegible]	$0.007h_{sx}$	$0.007h_{sx}$
All other structures	[illegible]	$0.015h_{sx}$	$0.010h_{sx}$

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

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Seismic Analysis

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

Other Checks: Inherent Torsion, Amplification Factor Ao, Accidental Torsion

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

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2007 LOS ANGELES REGIONAL UNIFORM CODE PROGRAM (LARUCP)

2007 LARUCP 16-10. Section 1614, 1614.1 and 1614.1.7 is added to Chapter 16 of the 2007 California Building Code to read as follows:

SECTION 1614
MODIFICATION TO ASCE 7.

1614.1 General. *The text of ASCE 7 shall be modified as indicated in this Section.*

1614.1.7 ASCE 7, Section 12.12.3. *Replace ASCE 7 Section 12.12.3 as follows:*

12.12.3 Minimum Building Separation. *All structures shall be separated from adjoining structures. Separations shall allow for the maximum inelastic response displacement (Δ_M). Δ_M shall be determined at critical locations with consideration for both translational and torsional displacements of the structure as follows:*

$$\Delta_M = C_d \delta_{max} \qquad \text{(Equation 16-45)}$$

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

REASONS FOR AMENDMENT/INTERPRETATION/CLARIFICATION:
Section 12.12.3 of ASCE 7-05 including Supplement No. 1 does not provide requirements for separation distances between adjacent buildings. Requirements for separation distances between adjacent buildings, not structurally connected, were included in previous editions of the IBC and UBC. However, when ASCE 7-05 was adopted by reference for IBC 2006, these requirements were omitted. In addition, ASCE 7-05 defines (δ_x) in Section 12.8.6 to refer to the deflection of Level x at the center of mass. The actual displacement that needs to be used for building separation is the displacement at critical locations with consideration of both the translational and torsional displacements. These values can be significantly different.

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2006 IBC / 2007 CBC
ICC LA Basin Chapter • Structural Code Committee

FINDINGS:
Local Geological Conditions – The greater Los Angeles/Long Beach region is a densely populated area having buildings constructed over and near a vast array of fault systems capable of producing major earthquakes, including but not limited to the recent 1994 Northridge Earthquake. The seismic separation is necessary to permit adjoining buildings, or parts thereof, to respond to earthquake ground motion independently and preclude possible structural damage due to pounding between buildings and other structures. The need to incorporate this modification into the code will help to assure that new buildings and additions to existing buildings are designed and constructed in accordance with the scope and objectives of the International Building Code.

- Goals and Criteria
- Placement of Expansion Joints
- Expansion Joints Specified by Code
- Floor System Design
- Lateral Design
- Seismic Analysis

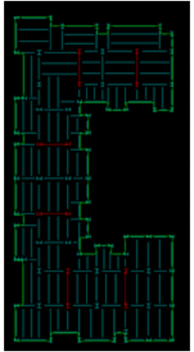
Width of Seismic Analysis

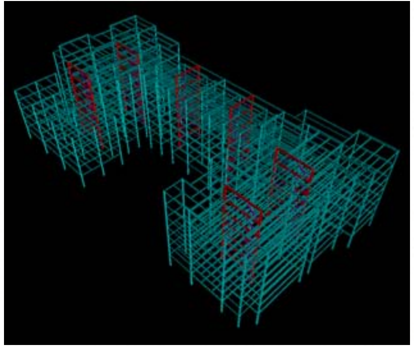
Drift and Displacement Calculations for SCBF E-W Direction For Section 1	
Story	Story Displacement (in)
Roof	1.055
7	0.849
6	0.673
5	0.475
4	0.312
3	0.150
2	0.000

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.





- Structural Systems Comparison
- Façade Material Comparison
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Structural Systems Comparison

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.

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

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Façade Material Comparison:

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

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

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Green Roof Retrofit Comparison:

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.

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 Service Life	9.5	50
Maintenance	Medium to High	Low

Introduction
Thesis Statement
Existing Condition
Breadth Study I
Breadth Study II
Structural Depth
Conclusion

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Final Works:

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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Ingleside AT KING FARM
An Architectural Engineering Senior Thesis

STEPHEN A. TAT
Structural Option 2008-2009

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Thank you…

Questions?