Final Report

Kathryn Gromowski | Structural Option

Final Redesign

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April 7th, 2011

University Sciences Building

Northeast USA

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General Information

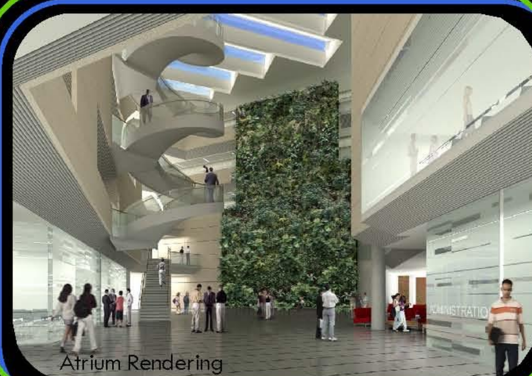
Function:	Laboratory/Classroom
Size:	138,000 SF
Height:	94 feet
Construction:	Aug. 2009-Sept. 2011
Construction Cost:	$50 Million
Delivery:	CM at Risk

Architecture

- Classrooms on first floor, labs/offices above
- Major focal point—5-story atrium
  - 4-story Biowall, the first of its kind at a US university
- Stone-aluminum honeycomb panels comprise the majority of the facade
- Windows of different sizes add interest and bring in natural light

Project Team

Owner:	Not Released
Architect:	Diamond & Schmitt
Associate Architect:	H2L2 Architects
Structural:	Halcrow Yolles
Associate Structural:	Keast & Hood
MEP:	CEL International
Civil:	Stantec Consulting



Atrium Rendering

Structure

Foundation:
- Drilled Caissons carry loads from grade beams to bedrock

Superstructure:
- Voided Filigree slabs and beams (precast/cast-in-place concrete hybrid system) comprise lower five floors
  - Uses 2" precast as leave-in formwork under C.I.P Concrete
- Mechanical penthouse is steel columns/beams

MEP Systems

Mechanical:
- Nine AHU's ranging from 6,000-42,500 CFM
- Two 620 ton capacity chillers
- VAV boxes with reheat coils throughout building

Lighting/Electrical
- 13.2 kV main switchgear
- Main power is 480/277V 3 phase, 4-wire
- 600 kW diesel emergency generator on the roof
- Uses CFL, Fluorescent, Metal-Halide & LED lighting

Fire Protection
- Largely wet pipe fire suppression system

Construction

- Sequencing of construction crucial for structural integrity
  - Begin at SW corner and build clockwise
- Architectural concrete columns require special formwork
- Varying window sizes/locations make placement difficult

CPEP Website: http://www.engr.psu.edu/ae/thesis/portfolios/2011/klg5081/index.html

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Executive Summary

This is a new, 138,000 square foot laboratory and classroom building located on an urban university campus in the Northeast USA. It has a construction cost of approximately $50 million, and has several unique architectural features, such as a biowall and a 5-story atrium through the core of the building. The main gravity system consists of voided filigree slabs and beams resting on cast-in-place columns, but the mechanical penthouse is constructed of steel. The lateral system consists of 15 shear walls scattered throughout the building, augmented above the concrete-steel transition by five braced frames.

The bulk of this report is comprised of several redesigns of the original structure. Because the existing structure was extremely efficient, the choice was made to attempt to design a viable alternative in steel, with moment frames as the lateral system. This was first done at the present site in the Northeast USA. It was found that the resulting design weighed approximately 11,800 k (about half the weight of the original structure), and was controlled by wind forces and the associated industry-standard drift limitations.

A scenario was then created in which the California State University, Northridge (CSUN) had commissioned the design of the building instead of the original owner. A geotechnical report was located for a site on CSUN's campus which was similar to the original site. The steel structure was redesigned for code minimum requirements to resist the controlling seismic forces at this new site and maintain the code-allowed drift. The resulting structure weighs approximately 12,300 k.

Finally, high-performance design was investigated by producing two designs for Immediate Occupancy criteria, as defined in ASCE's "Seismic Rehabilitation of Existing Buildings" (ASCE 41-05). The first design achieved this higher performance rating through the use of larger, stiffer steel moment frames. This structure weighed approximately 13,500 k. Then, the code-minimum frame was augmented with viscous fluid dampers on concentric steel braces in order to achieve the higher performance requirement. This design was verified with nonlinear analysis in SAP 2000. The resulting structure weighed approximately 12,500 k. Master's level coursework was integrated throughout the report in the computer modeling of the structures (AE 597A) as well as earthquake design (AE 538). However, the most direct application of mater's-level coursework can be found in the hand design of a variety of connections for the 3 structures in California (AE 534). The hand calculations for these designs can be found in Appendices E and F.

To fully compare the structures, a construction management breadth was undertaken. This used quantities from the take-offs of both structural components and some additional architectural features which were considered to determine durations for activities. Then, the existing schedule was modified to remove the existing superstructure, and the new durations for the superstructure (as well as additional architectural items) were added. These durations were used to calculate general conditions cost of the projects. The costs of the original structure and the four redesigned structures were calculated using a mix of square foot estimating, detailed estimating, and original cost data provided by Turner Construction. This analysis found that the steel structures were almost uniformly less expensive than the original structure, but they also had durations 2-3 months (10-15%) longer than the original schedule.

Finally, since the building was relocated to California, a sustainability breadth was undertaken to determine if a photovoltaic system or a green roof (neither of which were included on the original building) would be viable at the new location. Each system was designed and then evaluated with a life-cycle assessment, a payback period, a carbon footprint, and the number of additional LEED points they would earn. Each system could earn the building one additional LEED point, but the other analyses clearly indicate that the green roof is the more viable system.

Acknowledgements

I'd like to extend my gratitude to the following people and companies for their support during the completion of this report.

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Dr. David Riley, for introducing me to the world of solar power;

Johanna Harris and Paul Asp with HGA Architects and Engineers, for providing the California geotechnical report.

Building Introduction

The University Sciences Building (USB) is a new building located on an urban university campus in the Northeast USA. The site chosen was previously a parking lot serving adjacent campus buildings (See Figure 1). However, the USB provides a much more appealing image on this busy street corner. It is a departure from typical campus architecture in both material usage and architectural style. However, these differences serve as a visible indication of the university's new commitment to building sustainable, functional buildings.

Figure 1 Aerial map from Google.com showing the location of the building site.

While most other campus buildings have brick facades with narrow, strip-like windows, the USB is clad largely in a prefabricated natural stone panel with aluminum-honeycomb back-up, which enables the façade to be very light. Seemingly in homage to the surrounding buildings, the USB also utilizes tall, narrow windows. However, they are of varying widths and placement on the building, which adds interest to the façade (See Figure 2). An additional feature is the 5 story atrium that forms the core of the building. It provides significant focal points such as a sweeping spiral staircase and a four-story "biowall," the first of its kind on a US university campus (See Figure 3). The biowall is used to help mitigate air quality within the building, and it is just one of many features that will help to earn the building a LEED Gold rating upon completion.

Figure 2 Exterior rendering showing the stone façade and variation of windows on the USB.

The USB is a multi-use building, incorporating four large lecture-hall style classrooms, an auditorium, several teaching and research laboratories, and faculty offices. It locates the large classrooms and administrative functions on the ground floor of the building for easy public access, but removes the laboratories and offices to the upper four stories for additional privacy. Including the mechanical penthouse, the building stands 94'-3" above grade with a partial basement. It provides the university with 138,000 square feet of new space, and has a construction cost of approximately $50 million. Construction began in August of 2009, and has an expected completion date of September 2011.

Figure 3 Interior rendering of the atrium.

Existing Structural Overview

The University Sciences Building as it was originally designed rests on drilled concrete caissons ranging in diameter from 36" to 58" capped by caisson caps and then grade beams. The lower five floors utilize a voided filigree slab and beam system with cast-in place concrete columns. The mechanical penthouse, however, uses steel columns and floor framing. The lateral system consists of several shear walls spanning from ground to various heights. Masonry infill walls are used between columns on the lower floors to help dampen sound from the surrounding urban environment. These non-structural walls are used solely as back-up walls to support the cladding.

The importance factors for all calculations were based on Occupancy Category III. This was chosen because the USB fits the description of a "college facility with more than 500 person capacity," which requires Occupancy Category III.

Foundations

Geosystems Consultants, Inc. performed several test borings on the proposed site of the USB in October 2007. They found that the subsurface conditions consisted largely of extremely loose brick and rubble fill, followed by alluvium and finally residual soils with relatively low load-bearing capabilities. However, comparatively intact bedrock was encountered approximately 25 feet to 34 feet below the surface of the site.

In light of these conditions, traditional shallow spread footings would not be acceptable. Both driven steel H-piles and drilled caissons were considered as options for deep foundations, but H-piles were rejected due to vibration concerns within the subway station adjacent to the site, as well as noise concerns for the surrounding academic buildings. Instead, drilled caissons ranging in diameter from 36" to 58" were chosen to carry the loads from grade beams to the bedrock below. It was also recommended that the fill under the slab on grade (SOG) comprising the majority of the first floor be removed to a level of approximately 4 feet below the surface, followed by heavy compaction of subsurface materials, and then backfilled with structural fill to minimize settlement of the SOG due to the extremely poor load-bearing capacity of the brick/rubble fill.

Lastly, groundwater observation wells were installed, and groundwater was found to be present approximately 13 feet to 18 feet below the surface of the site. This is a potential concern, because some of the basement walls are 14 feet underground, and could encounter some loading due to hydrostatic pressure, particularly in seasons where the groundwater table rises due to rain.

Floor Systems

Although it may not appear so upon first glance at the very irregular shape of the building, the bay sizes are relatively consistent throughout the USB. It simply rotates the bays as necessary to accommodate the different rotations of the wings of the building. Figure 4 shows a typical floor plan with the different bay sizes highlighted with different colors. The legend lists the bay sizes with the span required for the slab first, and then the span required for the girder (if one is present).

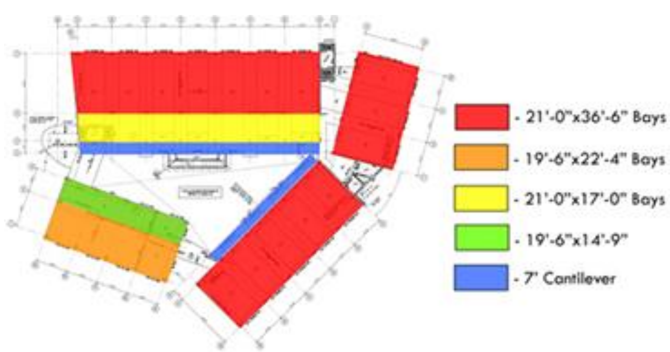


Figure 4 Floor plan from Sheet S203 showing typical bay sizes.

All of the elevated floors of the USB are a voided filigree system. This is a hybrid of precast, prestressed concrete and cast-in-place concrete. In essence, it consists of 2 ¼" of precast, prestressed concrete that functions as leave-in formwork. This is assembled and shored on site, followed by the placement of top and additional bottom reinforcing (if required, placed on rebar chairs on the bottom of the precast), and then further concrete is cast in place to unite the system. To help reduce the weight of the structure, polystyrene voids are incorporated where the concrete is not required for structural strength. Wire joists referred to as "filigree trusses" are used to transfer horizontal shear over the cold joint between precast and cast-in-place concrete.

Three separate systems were used, depending on the required spans and uses. For areas that include a span above 36 feet (typically laboratories), an 8" voided filigree slab (V.F.S.) was used to span between 18" deep voided filigree beams (V.F.B.). A schematic layout of this type of system, used in the majority of the building, is shown in Figure 5. In the Office Wing (shown in Figure 4 in green and orange), where shorter spans were allowed, the beams were removed from the system and the slab was thickened to 10 inches total depth. However, the cross section of this slab remains similar to the condition shown in the "Section 3" within Figure 5. Lastly, in the two "links" (shown in Figure 6), this flat plate is thickened to 12 inches total depth, again with a similar condition to "Section 3" in Figure 5. These links are the uniting elements in the building, and had to be cast last on every floor. These are united to the building with rebar across the cold joint rather than an official expansion joint.

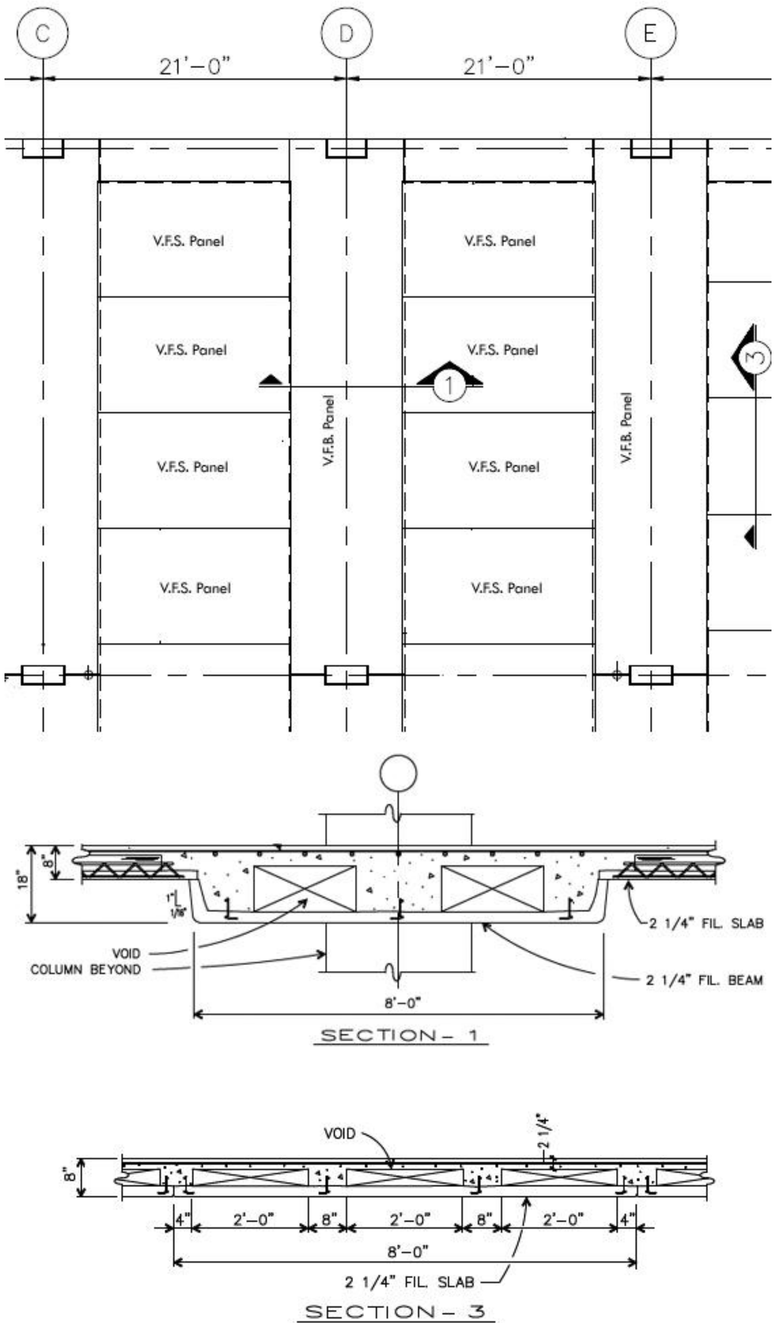


Figure 5 Typical bay with section cuts showing the condition within the beam and the slab. Modified from the filigree slab shop drawings and not to scale (NTS).

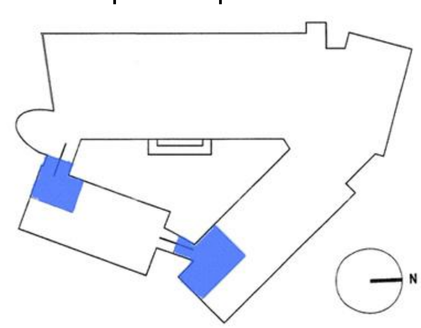


Figure 6 Modified keyplan from Sheet S202 showing the "link" areas in blue.

Framing System

The columns in the lower five stories of the USB are all cast-in-place concrete. The columns closest to the atrium on the ground floor are round columns 2 feet in diameter. Most are changed at the second level to 36"x16" rectangular columns. All other columns are 36"x16" columns, rotated as required to fit into walls. At the penthouse level, the columns change to A572 steel W-shapes. These columns range in size from W8x40 to W8x67.

Lateral System

Shear walls are the main lateral force resisting system in the USB. They are scattered throughout the building to best resist the lateral forces in the building (See Figure 7). All of these walls are 12" thick cast-in-place concrete. Most span from ground level to the roof, but since roof heights vary, they are not necessarily the same height. They are anchored at the base by grade beams that run the full length of the walls. Above the concrete-to-steel transition are also five braced frames (see Figure 7). These are extremely important in resisting the lateral forces on some of the roof levels.

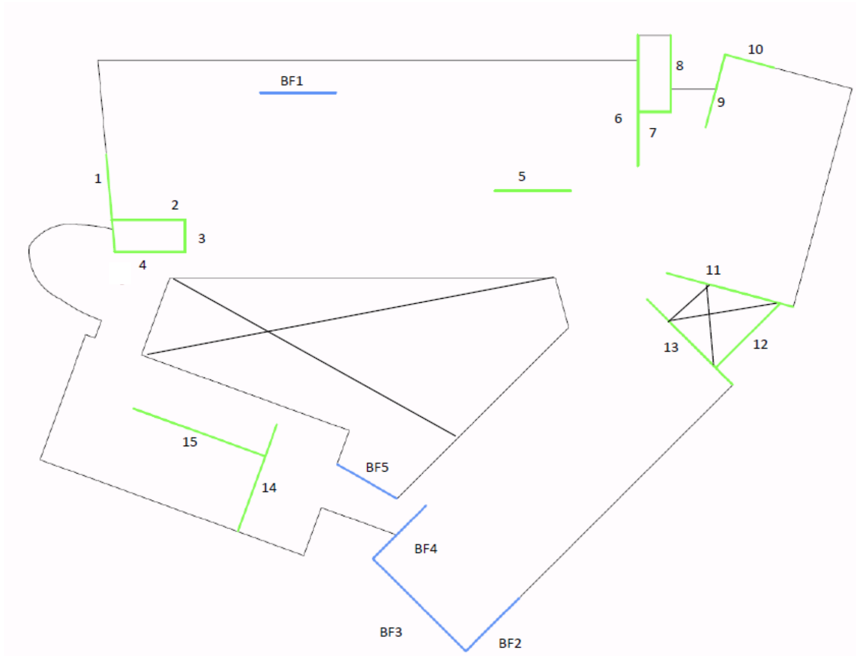


Figure 7 Typical floor plan simplified from Sheet S203. Shear walls indicated in green, braced frames indicated in blue. All elements have been labeled for ease of reference.

Roof Systems

There are six different roofs on the USB, due mostly to architectural reasons. Figure 8 shows these roofs and their heights above the ground reference elevation of 0'-0". The Office roof (shown in red) is at the same elevation as the fifth floor. Its structure is a 10" flat plate filigree slab system, similar to the office floors below it. The "Ledge" roof (shown in orange) is at the same level as the Penthouse floor, and is a continuation of the 10" V.F.S./24" V.F.B. system used in the adjacent AHU Mechanical Room. The atrium roof, 5th Level Mechanical Room roof, and AHU Mechanical Room roof (shown in yellow, green, and purple, respectively) are all 3" P2404 Canam roof deck on steel W-shape framing. The Chiller Mechanical Room roof (shown in blue) is 3" of cast-in-place concrete topping on 3" P2432 Canam composite deck (6" total depth) supported by W-shape framing. This heavier structure is necessary because this roof supports two large cooling towers and a diesel generator. This roof is also the only one with a parapet, which serves as a screen to hide the mechanical equipment and stretches from this roof level to 94'-3".

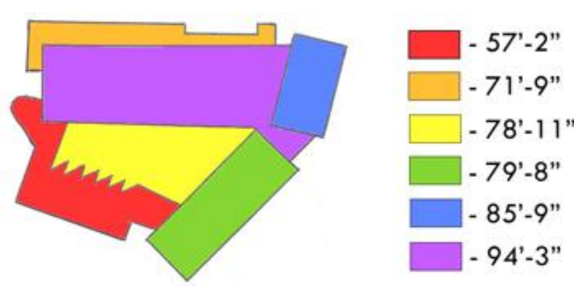


Figure 8 Modified keyplan image from Sheets S205, & S206 showing different roof heights in relation to 0'-0"

Regardless of the underlying structure, all roofs receive the same finish. This consists of sloped rigid insulation under Thermoplastic-Polyolefin (TPO) single-ply membrane.

Design Codes

According to Sheet S001, the original building was designed to comply with:

- 2006 International Building Code (IBC 2006) with Local Amendments
- 2006 International Mechanical Code (IMC 2006) with Local Amendments
- 2006 International Electrical Code (IEC 2006) with Local Amendments
- 2006 International Fuel Gas Code (IFGC 2006) with Local Amendments
- Local Fire Code based on the 2006 International Fire Code (IFC 2006) with Local Amendments.
- Minimum Design Loads for Buildings and Other Structures (ASCE 7-05)
- Building Code Requirements for Structural Concrete (ACI 318)
- Masonry Construction for Buildings (ACI 530)
- AISC Manual of Steel Construction, Load Resistance Factor Design (LRFD)

These are also the codes that were used to complete the analyses contained in this report, with heavy emphasis on the use of AISC Manual of Steel Construction and ASCE 7-05.

In addition, this report included the use of the following standards:

- Seismic Rehabilitation of Existing Buildings (ASCE 41-05)
- Seismic Provisions for Structural Steel Buildings (AISC 341-05)
- Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications (AISC 358-05)

Materials Used

Due to the variety of structural types on this project, there are also many different kinds of materials. These are listed in Table 1 below. All information was derived from Sheet S001.

Concrete		
Usage	Weight	Strength (psi)
Caissons	Normal	3000
Caisson Caps	Normal	3500
Footings	Normal	3500
Foundation Walls	Normal	4500
Shear Walls	Normal	4500
Slab-on-Grade	Normal	3500
Columns	Normal	5000
Structural Slabs/Beams	Normal	4500
Precast	Normal	5000
Housekeeping Pads	Normal	3500
Concrete on Steel Deck	Normal	3000

Steel		
Type	Standard	Grade
W-Shaped Structural Steel	ASTM A572	50
Hollow Structural Sections (HSS)	ASTM A500	C
Anchor Rods	ASTM F1554	N/A
Bolts, Washers, and Nuts	ASTM A325	N/A
3/4"x4 1/2" Long Welded Shear Studs	ASTM A496	N/A
Steel Deck	ASTM A653	A or B
Deformed Reinforcement Bars	ASTM A615	60
Welded Wire Fabric	ASTM A185	N/A

Masonry		
Type	Standard	Strength (psi)
Concrete Masonry Units	ACI 530	2175
Mortar	ASTM C270	N/A
Grout	ASTM C475	3000-5000

Miscellaneous	
Type	Strength (psi)
Non-Shrink Grout	10,000

Table 1 Summary of materials used on the USB project with design standards and strengths.

Gravity Loads

Dead, live and snow loads were all calculated and compared to loads listed on the structural drawings to verify the gravity design.

Dead and Live Loads

The structural drawings list superimposed dead loads, summarized in Table 2. Analyses found that these loads are accurate, although conservative in some cases. The ceiling and mechanical load applied is potentially higher than usual, but this can be explained by the large ductwork required to bring 100% outside air into the laboratory spaces. The uniform application of housekeeping pad loads to mechanical and electrical spaces is conservative because these pads are scattered over these spaces. However, these loads seem to be calculated by weight of concrete required for the depth of the pad specified. The masonry walls in the structure are 8" concrete masonry unit (CMU), weighing approximately 60 pounds per square foot (psf). Thus, the masonry wall load corresponds to a 14 foot high 8" CMU wall.

Superimposed Dead Loads	
Description	Load
1st Level Ceiling/Mechanical	10 psf
Other Levels Ceiling/Mechanical	15 psf
Electrical Room 4" Housekeeping Pad	55 psf
Mechanical Rooms 6" Housekeeping Pads	80 psf
Roofing	20 psf
Topping on Office Roof	36 psf
Masonry Wall	840 plf

Table 2 Summary of Superimposed Dead Loads.

Following the verification of the superimposed dead loads, estimations were made in order to calculate the overall building weight (which was also used in seismic calculations). By looking at typical sections through filigree slabs and beams, it was decided to consider the slabs 80% solid concrete and the beams 90% solid concrete.

Also considered in the building weight calculation were the weights of the columns, shear walls, superimposed dead loads, roofs, and wall loads (both exterior and interior). The exterior walls were considered to be 60 psf, as they are 8" CMU back-up walls with a cladding that weighs approximately 1 psf. The results of this calculation are summarized per level with the weights of a typical level shown in more detail in Table 3. The overall building weight was found to be approximately 25,500 k.

Live loads were also listed on the structural drawings. These were compared to the live loads in Table 4-1 in ASCE 7-05 based on the usage of the spaces, and the results are summarized in Table 4. Although many of these loads matched their ASCE 7-05 counterparts, some exceed the minimum significantly.

The large classrooms on the first floor were all designed for 100 psf, which is the design load for assembly areas with movable seating. These classrooms all have fixed seating, but it is possible that this was not yet decided at the time of the initial structural design, and therefore the more conservative load was used.

There is no provision for laboratories in classroom or research facilities, so the provision for "Hospitals – Operating Rooms, Laboratories" was used for comparison. It is possible that this was exceeded because most of these labs are to be teaching facilities, where occupant loads could exceed typical values depending on class sizes.

Weight per Level		
Level	Area (ft^2)	Weight (psf)
Ground	25,459	131.62
2nd	21,135	217.83
3rd	21,135	216.39
4th	21,135	216.39
5th	22,215	234.24
Penthouse	22,602	265.50
Roof	12,780	170.28

Weight of a Typical Floor (3rd Level)			
Description	Weight	Quantity	Total Weight (k)
8" VFS/18" VFB	127 psf	17,200 ft^2	2184.40
10" VFS	100 psf	2,890 ft^2	289.00
12" VFS	120 psf	1,045 ft^2	125.40
Superimposed DL	15 psf	21,135 ft^2	317.03
(43) 36"x16" Columns	600 plf/col	14 ft/col	361.20
Shear Wall	2100 plf	350 ft	735.00
Exterior Wall	840 plf	670 ft	562.80
		Total Weight=	4574.83 k
		Weight per Square Foot=	216.46 psf

Note: Values may differ slightly from values in "Weight per Level" table due to simplifications made in this table to allow for grouping

Table 3 Summary of building weight per level and a typical level.

The last major discrepancy was the live load on the Office Roof. This roof was accessible during construction, and was used for materials storage during this phase of the building's life. It is possible this load was increased to account for the loads associated with this, such as workers on the roof to access materials stored there.

It was also noted on the structural drawings that live load reduction was used where allowed by code. Therefore, live load was reduced wherever possible for all gravity calculations in this report.

Snow Loads

The roof snow load was calculated using the procedure outlined in Chapter 7 of ASCE 7-05, and the factors required for this calculation are summarized in Table 5. The structural drawings used a C_t of 0.8, but this does not seem to be permissible by code. Therefore, the

Live Loads			
Space	Design Live Load (psf)	ASCE 7-05 Live Load (psf)	Notes
Atrium	100	100	N/A
Large Classrooms	100	60	Fixed Seating in all
Laboratories	80	60	Based on "Hospitals - Laboratories"
Offices	50+20	50+20	Office Load+Partition Load
Links/Stairs	100	100	N/A
5th Level Lab	80+20	60+20	Based on "Hospitals - Laboratories"+ Partition Load
5th Level Mech. Room	100	N/A	N/A
Electrical Room	150	N/A	N/A
Office Roof	50	20	May be due to construction loading
AHU Mechanical Room	100	N/A	N/A
Chiller Mechanical Room	150	N/A	N/A
Other Roofs	20	20	N/A

Table 4 Summary of design live loads compared to ASCE 7-05 typical live loads.

drawings used a flat roof snow load of 20 psf, whereas 23.1 psf was calculated (and used for all subsequent calculations) in this report.

Flat Roof Snow Load Calculations	
Variable	Value
Ground Snow Load, p_g (psf)	30
Temperature Factor, C_t	1.0
Exposure Factor, C_e	1.0
Importance Factor, I_s	1.1
Flat Roof Snow Load , p_f (psf)	23.1

Table 5 Summary of roof snow load calculations.

Lateral Loads

In order to better understand the lateral systems, wind loads and seismic loads were calculated. These were calculated by hand, and then applied to a lateral model of the original structure created in ETABS. Wind load calculations can be found in Appendix B, and seismic load calculations can be found in Appendix C.

Wind Loads

Wind loads were calculated with the Method 2 Main Wind Force Resisting System (MWRFS) procedure identified in ASCE 7-05 Chapter 6. In order to be able to use this procedure, several simplifying assumptions had to be made. First, the building was modeled with a single roof height of 94'-3". Next, the surface areas were projected onto North-South (N-S) and East-West (E-W) axes, and the projected lengths were used to calculate wind pressures. However, using these projected building lengths for the calculation of L and B would be potentially unconservative. Thus, a "pseudo-footprint" was developed, and the area of the pseudo-footprint was transformed into a representative rectangle. The dimensions of this rectangle were used as L and B.

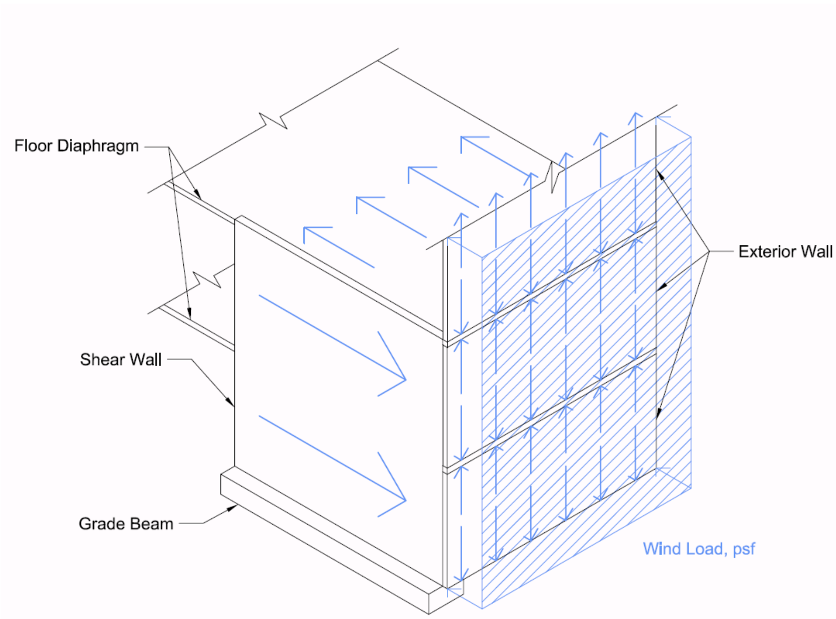


Figure 9 Diagram of the lateral load path for wind loads.

The wind loads on this building are collected by the cladding on the exterior of the building. The cladding transfers these loads to the CMU back-up walls, which are in turn anchored to the slabs with masonry dowels. This transfers the load into the slabs, which then carry the load to the shear walls. These return the loads to the foundations, and therefore to grade. This load path is illustrated in Figure 9.

Most calculations were performed using Microsoft Excel to simplify a potentially repetitive process. Wind pressures, including windward, leeward, sidewall, and internal pressure were found. These were then used to calculate the story forces at each level. It should be noted that the story forces include windward and leeward pressures, but not internal pressure, because internal pressure is effectively self-cancelling as there are no building expansion joints in the USB.

The wind pressures in both directions are listed in Tables 6 and 7. The N-S direction pressures were resolved into wind forces in the N-S direction, which are listed and diagramed in Figure 10. The resulting base shear is 281.4 k, which is about 13% less than the base shear for this wind direction listed on Sheet S001 (325 k). The E-W pressures were resolved into wind forces in the E-W direction, which are listed and diagramed in Figure 11. The resulting base shear is 407.6 k, which is about 12% less than the base shear for this wind direction listed on Sheet S001 (465 k). These discrepancies may be due to differing simplifying assumptions. However, this is not a major concern because the lateral system is controlled in both directions by seismic loads.

NE USA - Wind Pressures - N-S Direction							
Type	Floor	Distances (ft)	Wind Pressure (psf)	Internal Pressure (psf)		Net Pressure (psf)	
				(+)(GC_{pi})	(-)(GC_{pi})	(+)(GC_{pi})	(-)(GC_{pi})
Windward Walls	Ground	0.00	7.82	3.55	-3.55	4.28	11.37
	2nd	15.17	7.85	3.55	-3.55	4.30	11.39
	3rd	29.17	9.52	3.55	-3.55	5.97	13.06
	4th	43.17	10.65	3.55	-3.55	7.10	14.20
	5th	57.17	11.51	3.55	-3.55	7.97	15.06
	Penthouse	71.75	12.31	3.55	-3.55	8.77	15.86
	Roof	94.25	13.34	3.55	-3.55	9.80	16.89
Leeward Walls	All	All	-6.50	3.55	-3.55	-10.05	-2.96
Side Walls	All	All	-11.67	3.55	-3.55	-15.22	-8.13
Roof	N/A	0-47	-15.01	3.55	-3.55	-18.56	-11.46
	N/A	47-94	-15.01	3.55	-3.55	-18.56	-11.46
	N/A	94-188	-8.34	3.55	-3.55	-11.88	-4.79
	N/A	>188	-5.00	3.55	-3.55	-8.55	-1.46

Table 6 Table of wind pressures in the N-S Direction at the Northeast USA site.

NE USA - Wind Pressures - E-W Direction							
Type	Floor	Distances (ft)	Wind Pressure (psf)	Internal Pressure (psf)		Net Pressure (psf)	
				(+)(GC_{pi})	(-)(GC_{pi})	(+)(GC_{pi})	(-)(GC_{pi})
Windward Walls	Ground	0.00	7.65	3.55	-3.55	4.10	11.20
	2nd	15.17	7.67	3.55	-3.55	4.13	11.22
	3rd	29.17	9.31	3.55	-3.55	5.76	12.85
	4th	43.17	10.41	3.55	-3.55	6.87	13.96
	5th	57.17	11.26	3.55	-3.55	7.71	14.80
	Penthouse	71.75	12.04	3.55	-3.55	8.49	15.59
	Roof	94.25	13.05	3.55	-3.55	9.50	16.59
Leeward Walls	All	All	-8.15	3.55	-3.55	-11.70	-4.61
Side Walls	All	All	-11.42	3.55	-3.55	-14.96	-7.87
Roof	N/A	0-47	-17.66	3.55	-3.55	-21.21	-14.11
	N/A	47-94	-13.19	3.55	-3.55	-16.73	-9.64
	N/A	94-188	-9.65	3.55	-3.55	-13.19	-6.10
	N/A	>188	N/A	N/A	N/A	N/A	N/A

Table 7 Table of wind pressures in the E-W Direction at the Northeast USA site.

NE USA - Wind Forces - N-S Direction								
Floor Level	Elevation (ft)	Trib. Below		Trib. Above		Story Force (k)	Story Shear (K)	Accidental Moment (k-in)
		Height (ft)	Area (ft^2)	Height (ft)	Area (ft^2)			
Ground	0.00	N/A	0.00	7.59	1289.45	18.50	281.37	4296.43
2nd	15.17	7.59	1289.45	7.00	1190	37.57	262.87	8722.94
3rd	29.17	7.00	1190.00	7.00	1190	39.47	225.30	9165.94
4th	43.17	7.00	1190.00	7.00	1190	41.85	185.83	9717.15
5th	57.17	7.00	1190.00	7.29	1239.3	44.75	143.98	10392.10
Penthouse	71.75	7.29	1239.30	11.25	1912.5	61.27	99.22	14227.03
Roof	94.25	11.25	1912.50	N/A	0.00	37.95	37.95	8812.67
							Total Base Shear=	281.37 k
							Total Accidental Moment=	65,334.25 k-in

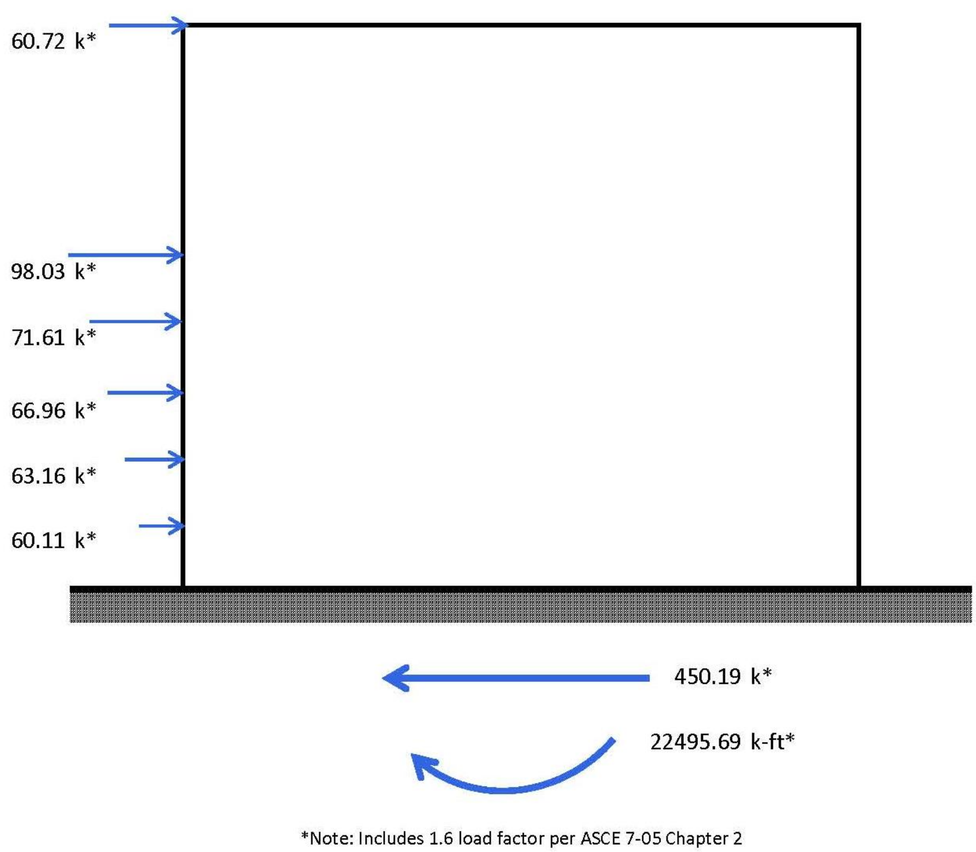


Figure 10 List and diagram of the wind forces in the N-S Direction at the Northeast USA site.

NE USA - Wind Forces - E-W Direction								
Floor Level	Elevation (ft)	Trib. Below		Trib. Above		Story Force (k)	Story Shear (K)	Accidental Moment (k-in)
		Height (ft)	Area (ft^2)	Height (ft)	Area (ft^2)			
Ground	0.00	N/A	0.00	7.59	1729.38	27.37	407.59	9853.65
2nd	15.17	7.59	1729.38	7.00	1596.00	55.24	380.22	19885.62
3rd	29.17	7.00	1596.00	7.00	1596.00	57.50	324.98	20700.17
4th	43.17	7.00	1596.00	7.00	1596.00	60.61	267.48	21820.98
5th	57.17	7.00	1596.00	7.29	1662.12	64.54	206.87	23236.09
Penthouse	71.75	7.29	1662.12	11.25	2565.00	87.94	142.32	31659.98
Roof	94.25	11.25	2565.00	N/A	0.00	54.38	54.38	19576.66
							Total Base Shear=	407.59 k
							Total Accidental Moment=	146,733.14 k-in

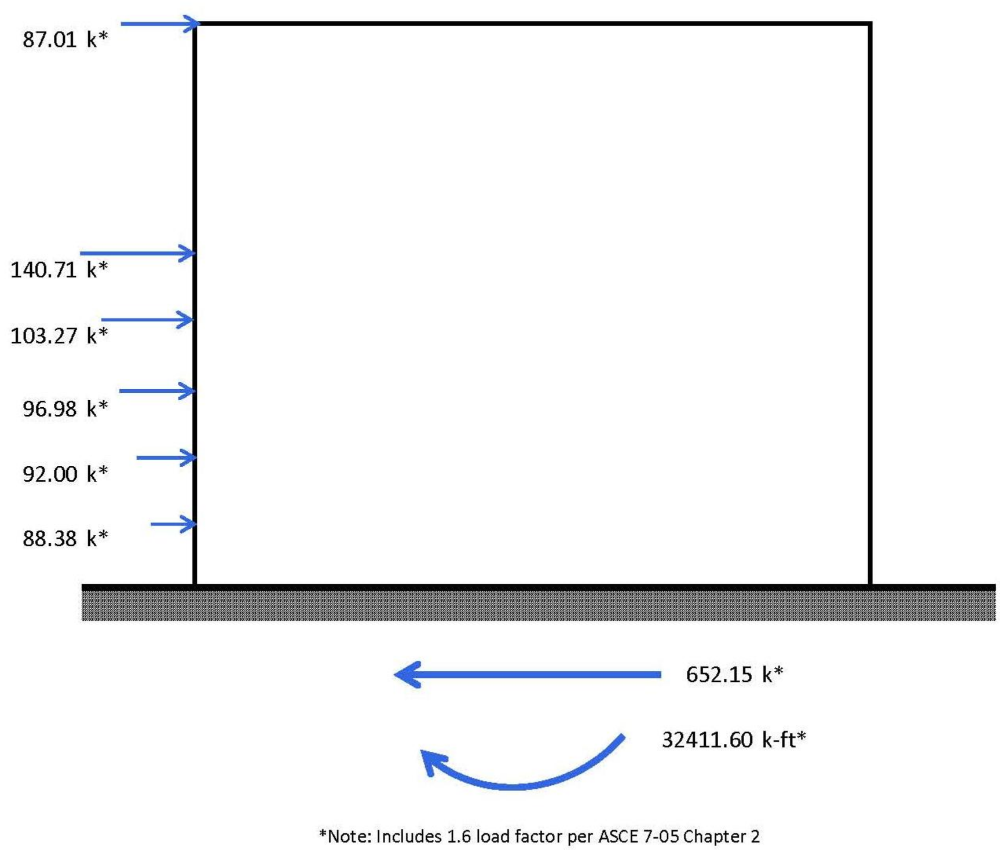


Figure 11 List and diagram of the wind forces in the E-W Direction at the Northeast USA site.

Seismic Loads

Seismic loads were first calculated with the Equivalent Lateral Force (ELF) procedure outlined in Chapters 11 and 12 of ASCE 7-05. This procedure also assumes a simple building footprint, but the simplifications required for this were much less drastic than those required for wind calculations. The approximate fundamental period for shear walls can be calculated using the generic designation of “other structures” or the specific equation for shear walls. Both were evaluated for this report, and it was determined that it was more likely that the original calculations were performed with the specific equation. Therefore, the specific solution was used for the finalization of the seismic load calculations in this technical report. To perform this specific solution, the shear walls had to be resolved onto North-South (N-S) and East-West (E-W) axes. This was accomplished with trigonometry.

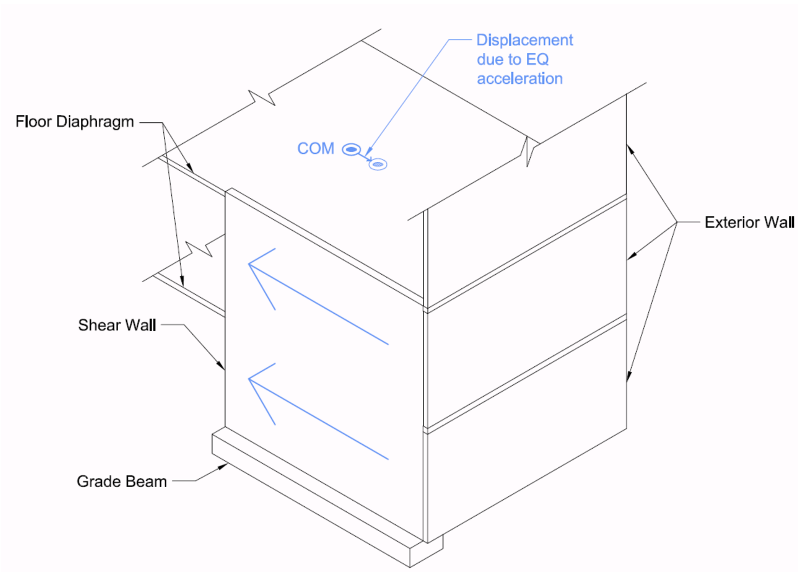


Figure 12 Diagram of the lateral load path for a seismic load.

The loads from seismic forces originate from the inertia of the structure itself, which is related to the mass of the structure. Most of the mass of the structure is locked in the slabs, which are directly connected to the shear walls. When seismic loads are generated by a ground motion, the slabs transfer the loads directly into the shear walls, which then carry the loads down to the foundations and therefore to grade. This is diagrammed in Figure 12.

The resultant ELF base shear in the N-S direction is 786.68 k, which is about 20% less than the base shear listed for this direction on Sheet S001 (955 k). This discrepancy is potentially due to the original engineer not using the Coefficient for Upper Limit on Calculated Period (C_u, ASCE 7-05 Table 12.8-1). For this building, C_u is 1.7. Assuming C_u was not incorporated, and the basic solution was used to find base shear instead of the specific solution for shear walls, base shear would be 1010 K in both directions (5-10% error).

The resultant ELF base shear in the E-W direction is 917 k, which is about 20% less than the base shear listed for this direction on Sheet S001 (1145 k). Again, this difference is probably accounted for by the same discrepancy indicated for the N-S direction.

After the lateral model was constructed in ETABS, base shears were found again using the Modal Response Spectrum Analysis (MRSA) procedure on a finite element model constructed in ETABS with the cracked section properties modeled by a 50% reduction on the modulus of elasticity for all concrete materials. This involves calculating a C_s-like quantity using the modal periods for sufficient modes to obtain 90% mass-participation in two orthogonal translational directions. This base shear is typically lower than that calculated by the ELF procedure. However, it is limited by an absolute minimum of 85% of the base shear calculated by ELF. The equations for this process are as follows:

$$C_{m,i} = min \begin{matrix} \dfrac{S_{D1}}{T_i \dfrac{R}{I}} \\ \dfrac{S_{DS}}{\dfrac{R}{I}} \end{matrix}$$

$$V_m = W(\Sigma(C_{m,i}M\%_i)^2)^{1/2} \geq 0.85V_{ELF}$$

Where M%i refers to the mass participation percentage of mode "i" in decimal form.

The MRSA seismic forces in the N-S Direction and E-W Direction are listed and diagrammed in Figures 13 and 14, respectively. This yielded base shears of 716.6 k in the N-S Direction and 936.7 k in the E-W Direction, neither of which was controlled by the 85%V_{ELF} minimum.

Rigid Diaphragm Model - Seismic Forces - N-S Direction							
Level	Story Weight, w_x (k)	Story Height, h_x (ft)	$w_x h_x^K$	C_{vx}	Story Force (k) $F_x = C_{vx} V$	Story Shear (k)	Accidental Moment (k-in)
2nd	4130.90	15.17	91575.38	0.04	29.32	716.57	4010.30
3rd	4105.92	29.17	191771.96	0.09	61.39	687.25	8398.15
4th	4105.92	43.17	299787.93	0.13	95.97	625.86	13128.42
5th	5510.78	57.17	554166.98	0.25	177.40	529.89	24268.28
Penthouse	4870.93	71.75	634590.24	0.28	203.14	352.49	27790.21
Atrium Rf.	791.87	78.92	114988.38	0.05	36.81	149.35	5035.61
Chiller Rf.	455.68	85.75	72737.75	0.03	23.28	112.54	3185.36
AHU Rm. Rf.	1568.32	94.25	278812.18	0.12	89.25	89.25	12209.85
						Base Shear [$V=C_S W$]=	716.57 k
						Total Accidental Moment=	85,816.34 k-in

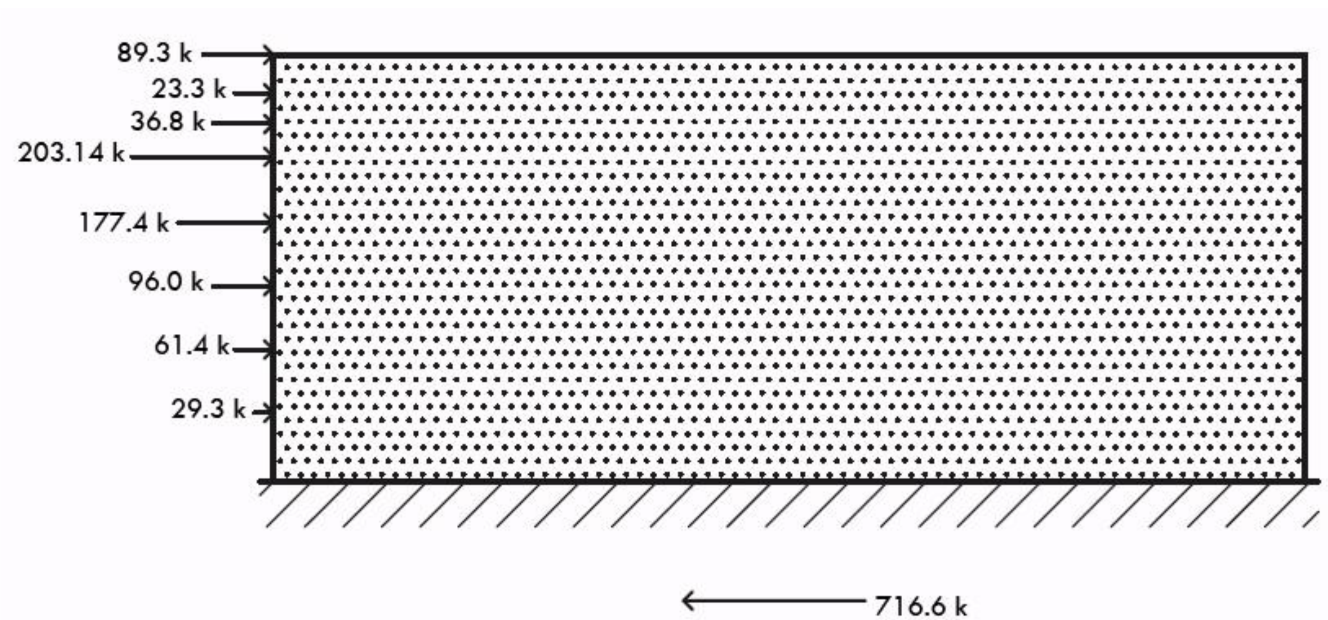


Figure 13 List and diagram of N-S direction seismic forces from the Modal Response Spectral Analysis Procedure

Rigid Diaphragm Model - Seismic Forces - E-W Direction							
Level	Story Weight, w_X (k)	Story Height, h_X (ft)	$w_X h_X^K$	C_{VX}	Story Force (k) $F_X=C_{VX}V$	Story Shear (k)	Accidental Moment (k-in)
2nd	4130.90	15.17	78772.65	0.04	41.25	936.72	5642.75
3rd	4105.92	29.17	159093.62	0.09	83.31	895.47	11396.41
4th	4105.92	43.17	243361.52	0.14	127.43	812.16	17432.81
5th	5510.78	57.17	442916.42	0.25	231.93	684.73	31727.60
Penthouse	4870.93	71.75	500851.78	0.28	262.26	452.80	35877.70
Atrium Rf.	791.87	78.92	90277.56	0.05	47.27	190.54	6466.89
Chiller Rf.	455.68	85.75	56844.48	0.03	29.77	143.27	4071.96
AHU Rm. Rf.	1568.32	94.25	216753.80	0.12	113.50	113.50	15526.81
						Base Shear [$V=C_SW$]=	936.72 k
						Total Accidental Moment=	112,616.12 k-in

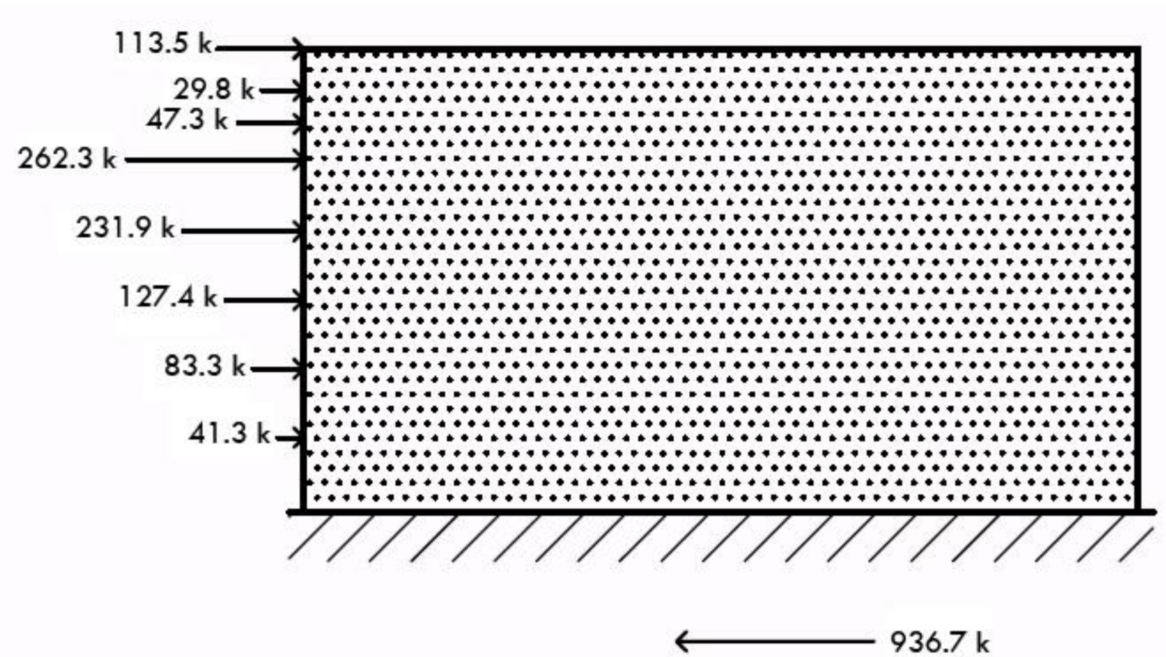


Figure 14 List and diagram of E-W direction seismic forces from the Modal Response Spectral Analysis Procedure

Problem Statement

As it is designed, there is very little that could be done to the USB that would lead to major improvements. All structural systems are adequate in strength and reasonable in comparison to typical alternatives. Redesigning the building as a different concrete system (such as the post-tensioned concrete slab with wide-shallow beams considered in Technical Report 2) would produce minimal differences. In its current location, significant reduction of building weight (such as redesigning the building in steel) would also cause wind forces to control the lateral design instead of seismic forces. The author of this report was extremely interested in investigating seismic design. Therefore, having wind forces control the lateral design was an undesirable condition.

Therefore, a scenario was created in which the California State University, Northridge (CSUN) requested the design and construction of a building identical to the University Sciences Building. The CSUN campus is essentially located on top of the Northridge fault, a fault line which produced the disastrous Northridge Earthquake in 1994. The site will have significant seismic demands which will far exceed the wind force requirements.

Also, in the last decade, a major movement in building design has been to improve the performance of buildings above the minimum design requirements without significant cost impacts. Therefore, the owner in the proposed scenario has requested the building be designed for an ASCE Structural Performance Level of “S-1 Immediate Occupancy” to target immediate access to the facilities following an earthquake with only potential minor damage to non-structural components. A comparison of the requirements for S-1 requirements and the more traditional “S-3 Life Safety” requirements can be found in Figure 15, taken from FEMA 356.

Therefore, a viable structural system must be designed to provide sufficient strength and serviceability resistance to achieve an S-1 structural performance level (as defined in ASCE 41) when resisting all dead loads, live loads, and seismic loads with as little negative impact as possible to the architecture, cost, and schedule of the building as it is currently designed.

Table C1-3 Structural Performance Levels and Damage[1, 2, 3]—Vertical Elements (continued)

Elements	Type	Structural Performance Levels: Collapse Prevention S-5	Life Safety S-3	Immediate Occupancy S-1
Concrete Walls	Primary	Major flexural and shear cracks and voids. Sliding at joints. Extensive crushing and buckling of reinforcement. Failure around openings. Severe boundary element damage. Coupling beams shattered and virtually disintegrated.	Some boundary element stress, including limited buckling of reinforcement. Some sliding at joints. Damage around openings. Some crushing and flexural cracking. Coupling beams: extensive shear and flexural cracks; some crushing, but concrete generally remains in place.	Minor hairline cracking of walls, <1/16" wide. Coupling beams experience cracking <1/8" width.
	Secondary	Panels shattered and virtually disintegrated.	Major flexural and shear cracks. Sliding at joints. Extensive crushing. Failure around openings. Severe boundary element damage. Coupling beams shattered and virtually disintegrated.	Minor hairline cracking of walls. Some evidence of sliding at construction joints. Coupling beams experience cracks <1/8" width. Minor spalling.
	Drift	2% transient or permanent	1% transient; 0.5% permanent	0.5% transient; negligible permanent
Steel Moment Frames	Primary	Extensive distortion of beams and column panels. Many fractures at moment connections, but shear connections remain intact.	Hinges form. Local buckling of some beam elements. Severe joint distortion; isolated moment connection fractures, but shear connections remain intact. A few elements may experience partial fracture.	Minor local yielding at a few places. No fractures. Minor buckling or observable permanent distortion of members.
	Secondary	Same as primary.	Extensive distortion of beams and column panels. Many fractures at moment connections, but shear connections remain intact.	Same as primary.
	Drift	5% transient or permanent	2.5% transient; 1% permanent	0.7% transient; negligible permanent

Figure 15 Comparison of performance requirements for different Structural Performance Levels for Concrete Walls and Steel Moment Frames, taken from FEMA 356 (similar to Table C1-3 of ASCE 41, which superseded FEMA 356, but was not available to the author at the time of this report).

Proposed Solution

Two solutions have been proposed for comparison, both in steel. As this is a different construction type than the original design, the gravity system shall be redesigned first. Upon completion of a suitable gravity system, the building will be designed for two lateral systems complying with the S-1 requirements:

- Traditional steel moment frame
- Traditional moment frame designed for S-3 requirements augmented with viscous fluid dampers (VFD's)

For comparison purposes, a traditional steel moment frame for the loads in the present location (Northeast USA) and S-3 requirements will also be designed.

In earthquakes, buildings are typically designed to yield at predicted locations in an expected manner, also known as "plastic design". In traditional steel moment frame design, this is most commonly accomplished by reducing the cross-section of the beam near the moment connection as shown in Figure 16, also known as "dog bones." These dog bones provide a weak location for plastic hinges to form. Although effective, plastic design can lead to permanent deformations of a building in a strong earthquake, which means a building may have to undergo expensive repairs. In keeping with the performance-based design trend in the industry, many designers are now seeking a solution which will reduce or eliminate this concern.

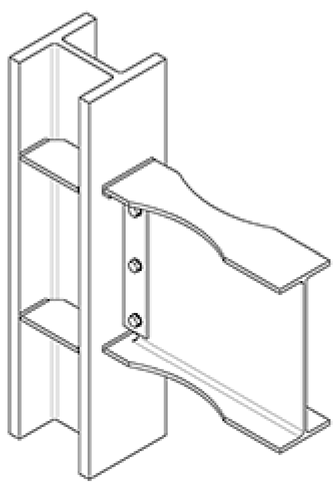


Figure 16 Image of a reduced beam section used in seismic design, from an article in Modern Steel Construction.

One such solution is the use of damping systems. These include a range of different devices which deform in response to an applied load or acceleration, thereby creating a point of energy dissipation in the structure. However, as these dampers provide some resistance to deformation, they also help to damp (or reduce) deflections caused by sudden motion, thereby decreasing both structural and non-structural component damage. Some of these dampers must be replaced following an earthquake as they will undergo permanent deformation, whereas others are able to deform without permanent damage to the damper. The most practical of these is the viscous fluid damper, or VFD, which will not undergo permanent deformation due to an earthquake provided they are designed adequately.

VFD's, an example of which can be seen in Figure 17, are similar to the closures on fire doors. The fluid inside the damper provides resistance whenever the building experiences sudden accelerations, such as those induced in an earthquake. As the piston is depressed or retracted, fluid flows through the orifices in the piston head. The pressurized fluid provides resistance to this motion, thereby reducing the distance which the piston moves. Subsequently, the displacement of any object attached to the piston is also reduced.

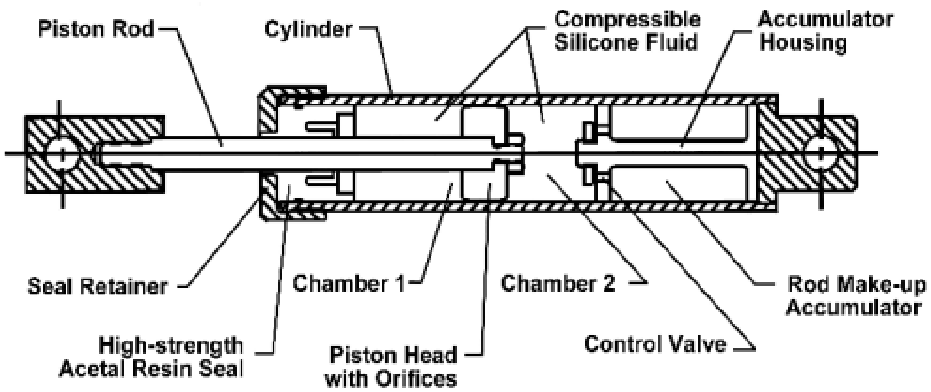


Figure 17 Image of the interior of a viscous fluid damper (VFD), taken from Taylor Device's website.

The chosen configuration for the VFD's can be seen in Figure 18. This was selected because simple static equilibrium dictates that the dampers are most effective at resisting horizontal displacements when they are placed horizontally. The top connection of concentric steel brace is designed as a sliding connection, which enables the dampers to engage when the frame deflects. The braces add negligible stiffness to the structure, instead acting purely as a connecting element to integrate the dampers into the system.

Figure 18 Image of chosen VFD configuration, taken from Taylor Device's website.

MAE Material Incorporation

Much of the calculation of the redesign drew upon material learned in MAE courses. Computer modeling techniques as taught in AE 597A – Computer Modeling were an integral tool in the completion of this redesign. Concepts such as rigid diaphragm constraints, panel zone modeling, property modifiers, and modal analysis results determination were taught for ETABS and SAP 2000. These skills were applied to ETABS and SAP as well as extrapolated to assist with modeling in RAM Structural System, which was not taught in depth in AE 597A.

The design of the steel moment frames and VFD's relied heavily on material presented in AE 538 – Earthquake Design. The limitations and requirements for a steel special moment frame and the procedures used to implement performance-based design were of particular use.

Finally, coursework from AE 534 – Steel Connections was integrated into the design of representative reduced beam section beam-to-column moment connections, lateral-torsional buckling braces for the bottom flange of special moment frame beams, damper-to-support connections, damper support-to-column connections, and concentric steel brace connections. Although the beam-to-column moment connection is the only one of the five mentioned which was specifically taught in the class, the information presented regarding typical limit states will be extrapolated to design reasonable connections for the other conditions. These designs can be found in the "Structural Depth: Steel Redesigns" section under each individual redesign subsection.

Breadth Studies

To address the integrated nature of the Architectural Engineering program, two breadth studies are also included as a part of this report. The first is a construction management breadth, which uses quantities of superstructure components and data from RS Means to determine the duration and cost of each structure. This was used to help compare the designs to determine the relative efficiency of each.

The second breadth study attempts to determine if either a photovoltaic system or a green roof would be viable now that the building is in California. Neither technology was included on the original building. However, both have the potential to earn the building additional LEED points. To fully capture the viability of each system, a life cycle assessment, payback period, carbon footprint, and a LEED analysis to determine how many additional LEED points they could achieve will be evaluated for each design. By comparing the various evaluation methods, it should be possible to conclude which system will be more beneficial to the USB.

Structural Depth: Steel Redesigns

The redesigns were done sequentially in order to create a logical design progression. First, the structure was redesigned in steel as a pure gravity system. This was accomplished by selecting typical bays in each wing (these are indicated and numbered in Figure 19) and then designing the deck and slab, infill beams, girders, and columns in these bays by hand at each level which had unique loading (these hand calculations can be found in Appendix D). The typical sizes were then used as trial sizes in a RAM Structural System model. RAM was used to full optimize the gravity structure and ensure all members were appropriate for gravity strength. Once an overall gravity frame was in place, a lateral system was added to the building for each of the designs in the processes summarized below. Each steel design is given a label which specifies its location (NE USA or CA) and its design criteria (S-3 or S-1) for ease of reference.

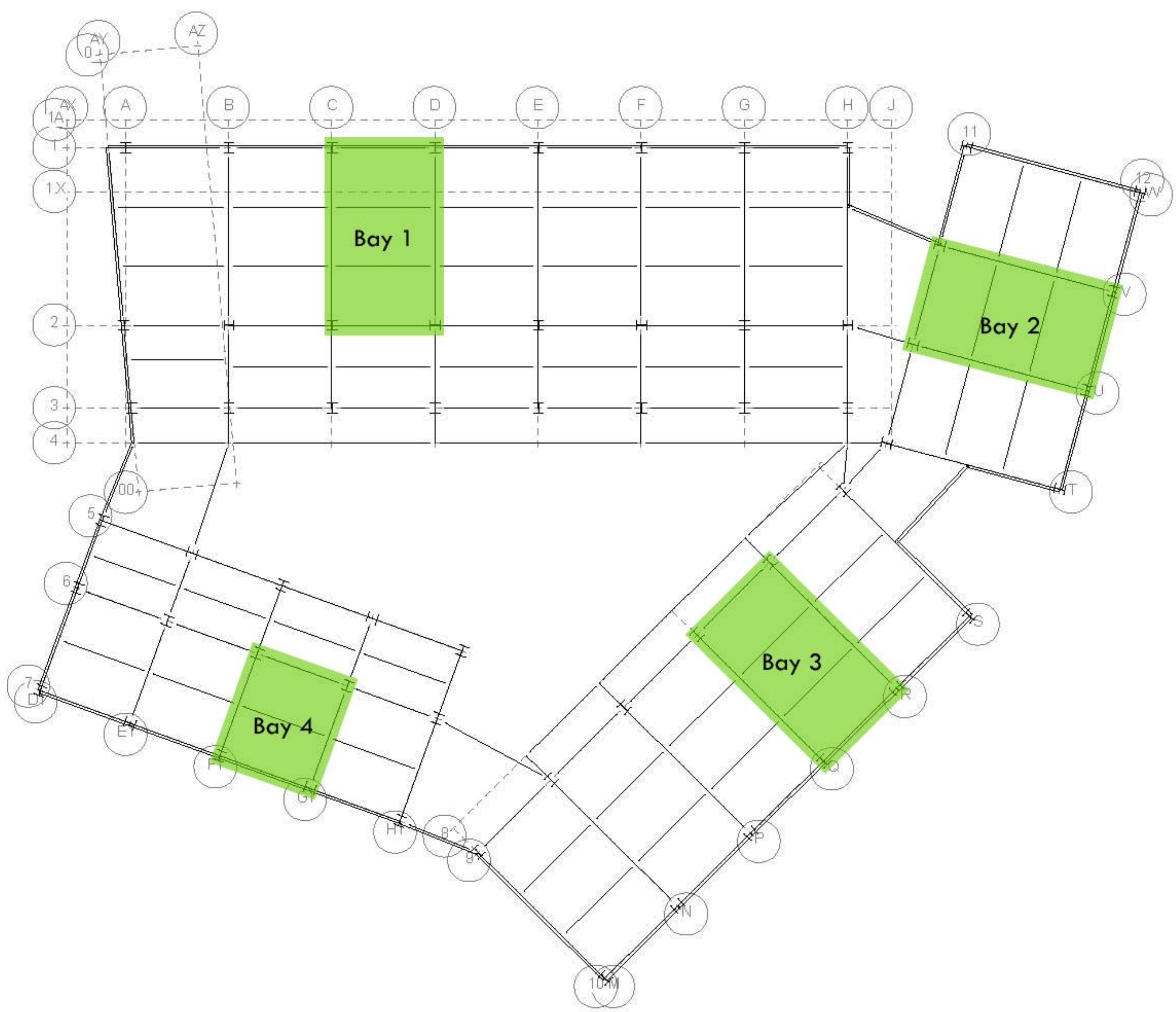


Figure 19 Floor plan indicating the typical bays used for the preliminary hand calculations for the gravity design of the steel system.

Code Minimum Moment Frame in Northeast USA (NE USA S-3)

This design was created mostly to have a baseline steel structure to serve as a logical bridge between the original concrete structure and the later designs conducted for the CSUN location. The familiarity with the loads at the original Northeast USA site made it possible for the author to become familiar with the Frame Module of RAM without being concerned that the loads applied to the model might be in error.

The lateral system was chosen to be moment frames because this is an effective stand-alone lateral system that could easily be augmented with VFD's for the final and most complicated design. Although moment frames are more expensive than braced frames or concrete shear walls, they provide additional architectural freedom. Moment frames are significantly more successful when they are longer (able to incorporate several bays in line) and are not reduced in width over the height of the building. Therefore, the moment frame layout was chosen to provide at least one line of resistance in each direction in each wing which was continuous throughout the height of the wing in which the frame was located. This layout can be seen in Figure 20. The frames were numbered for ease of reference. As can be seen in Figure 20, the columns in the frames have been oriented different ways. When a column participated in two frames, the strong axis of the column was aligned parallel to the direction which had less length of moment frame. However, when the column only participated in one moment frame, the strong axis of the column was aligned with the moment frame to provide additional strength.

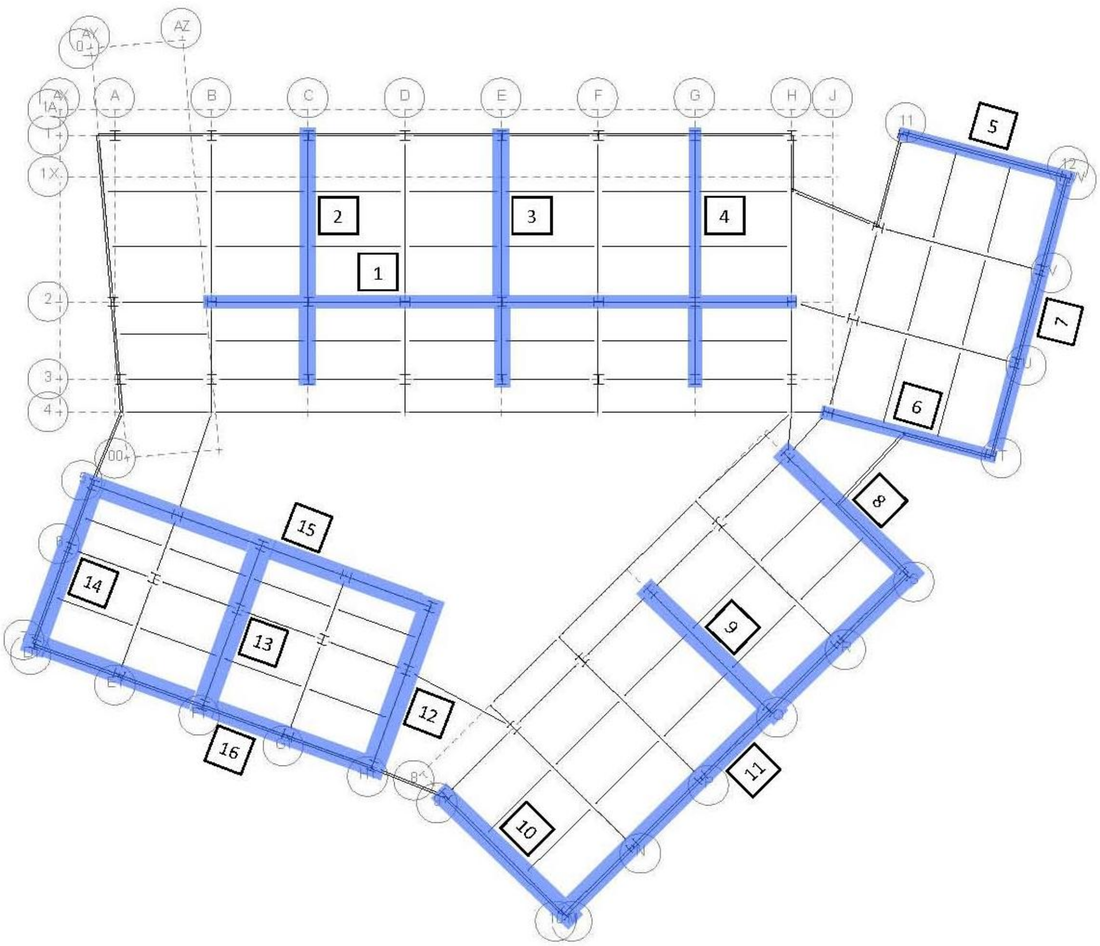


Figure 20 Floor plan with the moment frames indicated in blue. The numbers are the numbers which were assigned to the moment frames for ease of reference.

This layout was used to produce an Excel spreadsheet which was designed to calculate the forces in the beams and columns of the frame through portal frame analysis. The load on each frame was found by resolving the length of the frame into North-South (x) and East-West (y) components. The percentage of total length of frame in each direction was then multiplied by the wind force in the applicable direction, resulting in an x-force and a y-force on the frame. These were combined with Pythagorean Theorem to produce an in-plane force. Then, the percentage of in-plan load associate with each level (which is determined by the height of façade associated with that level) was calculated. The resulting story forces were used for the portal frame calculation. The spreadsheet used the portal frame column moments and the column axial loads calculated in the hand gravity calculations to perform an interaction check on the columns. All columns were assumed to be spliced every two levels for constructability purposes. Lastly, the spreadsheet did a preliminary check on the drift of the frame versus the allowable (although there is no strict requirement in the code, standard practice uses story height/400 as an allowable drift, and therefore this was used for this report). A sample column interaction check is shown in Figure 21 and a sample frame result is shown in Figure 22.

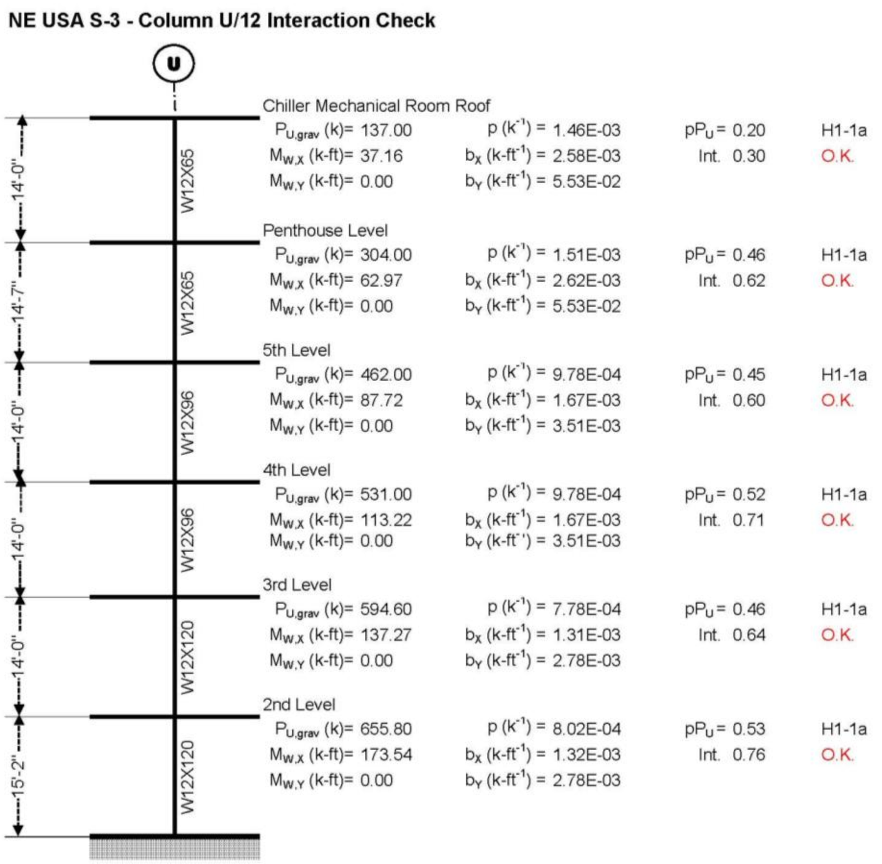


Figure 21 Example of the column interaction check performed with moments from the portal frame analysis and gravity loads from the hand calculations.

These preliminary member sizes were entered into RAM and the Frame Module was used to finalize the sizing required to meet both strength and drift requirements. For simplicity, all diaphragms were modeled as rigid. It is understood that this building has unique features which would probably require more sophisticated semi-rigid diaphragm modeling. However, this was not evaluated for this report.

It was found that although the portal frame analysis was reasonably effective at strength design, it was a very poor predictor of drift. Therefore, the Drift Control Module was used extensively to refine the model. This module uses "load pairs", which partners a unit virtual load at the level under consideration with the full load case in the appropriate direction to determine which members are significant to drift. When using the "Total/Volume" setting, the module uses colors to indicate whether or not increasing the volume of a member would significantly reduce drift. If increasing the volume of a member would significantly reduce drift, the member is indicated in a warm color (red indicating the most critical members to increase the volume of) and if increasing the volume of a member would have little impact on drift, the member is indicated in a cool color (dark blue indicates the least critical members to increase the volume of). A screenshot from the Drift Control module illustrating the usage of color is included as Figure 23. Using this information, members can be upsized only where most needed to reduce drifts. The final deflections and drifts of the building are summarized in Tables 8 and 9 below and also in the "System Comparison/Summary" section, under the "System Drifts Summary" subsection.

Once the RAM model was deemed to be adequate for both strength and serviceability, the total weight of the building was calculated for the comparison to the other systems and for use in the seismic calculations (these weights are summarized in the "System Comparison/Summary" section, under the "System Weights Summary" subsection). The overall weight for this design was found to be approximately 11,800 k (approximately half the weight of the original structure).

NE USA S-3 - N-S Drift Check		
Level	Δ (in)	δ
AHU Roof	2.51	0.002451
Chiller Roof	2.26	0.002195
Atrium Roof	2.08	0.002326
Penthouse	1.88	0.002058
5th Level	1.52	0.002381
4th Level	1.12	0.002321
3rd Level	0.73	0.002339
2nd Level	0.34	0.001852

Table 8 Summary table showing deflections and drifts in the N-S Direction for the NE USA S-3 design.

NE USA S-3 - E-W Drift Check		
Level	Δ (in)	δ
AHU Roof	2.57	0.00235
Chiller Roof	2.33	0.00232
Atrium Roof	2.14	0.00244
Penthouse	1.93	0.00223
5th Level	1.54	0.00244
4th Level	1.13	0.00244
3rd Level	0.72	0.00209
2nd Level	0.37	0.00203

Table 9 Summary table showing deflections and drifts in the E-W Direction for the NE USA S-3 design.

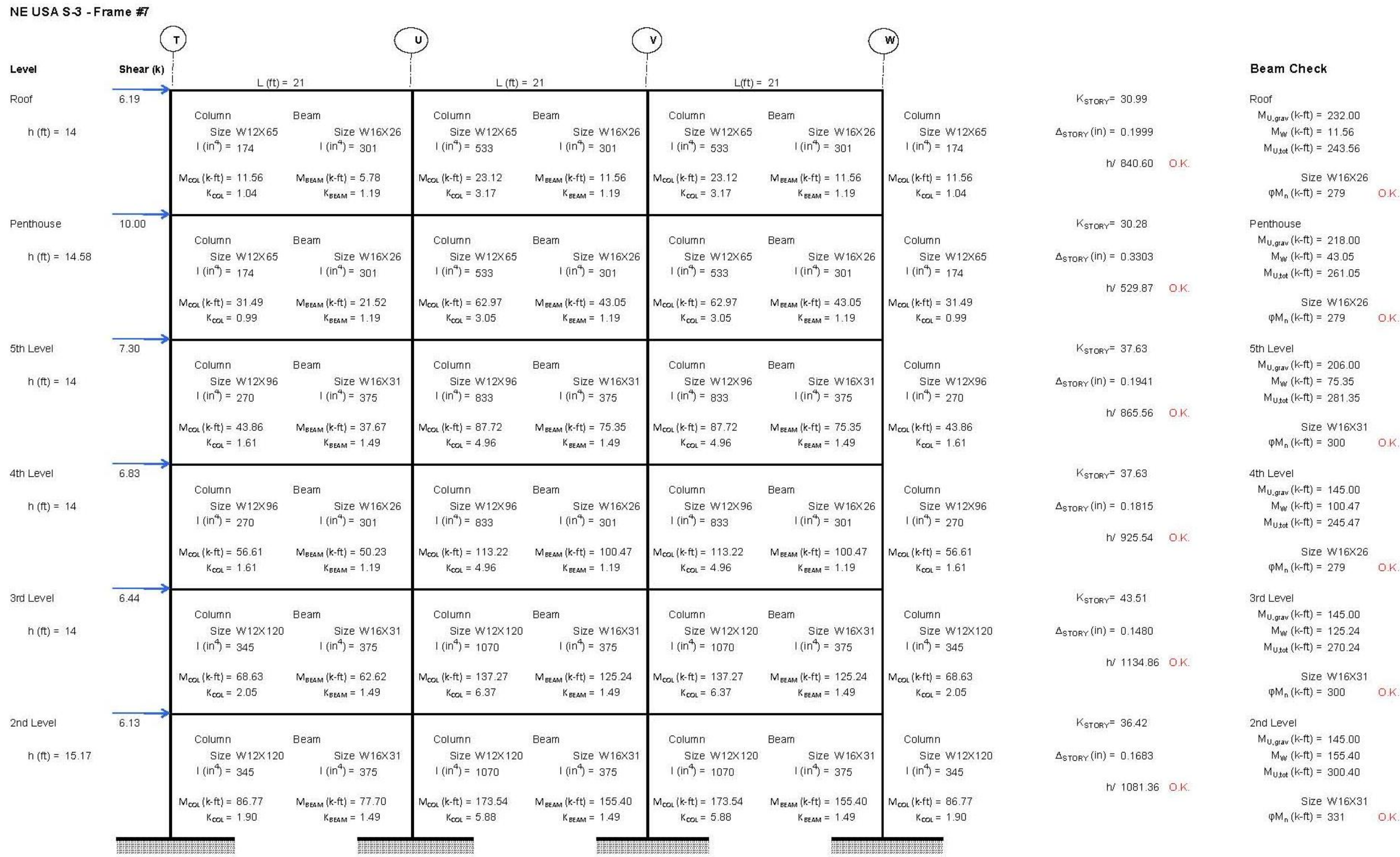


Figure 22 Example of the portal frame spreadsheet used to find moments in the frames due to wind

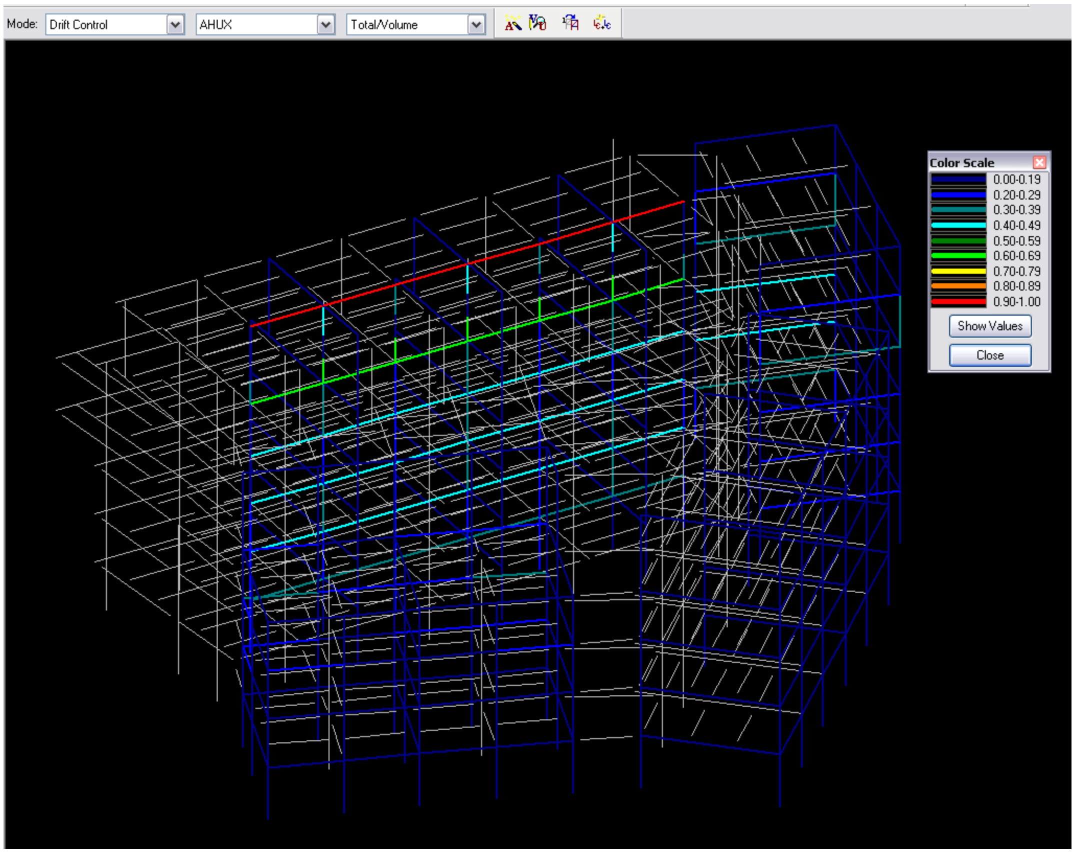


Figure 23 Screenshot from the RAM Drift Control Module showing how color is used to indicate the most effective members to upsize in order to reduce drift.

Seismic Loads

It was necessary to calculate seismic loads for this structure to verify they did not exceed the wind loads which were used for design. ELF procedure was used, since MRSA typically produces lower forces, and it was therefore conservative to use ELF. The full set of parameters used to calculate these loads can be found in Appendix C. Although the site parameters were the same for this design as they were for the original concrete structure, the R value of the NE USA S-3 structure was set as 3 because it is a steel frame which was not specifically detailed to resist seismic loads.

There is no specific equation for steel moment frames, so the basic solution was used. This resulted in identical base shears and story force distributions in both directions, which are listed and diagrammed in Figure 24. The base shear was 456.3 k. With the 1.6 load factor applied to them, the wind base shears are 450.2 k in the N-S Direction and 652.2 k in the E-W Direction. It was therefore deemed that wind could be considered to control in both directions, as was originally assumed. Therefore, this design was considered complete.

NE USA S-3 - ELF Seismic Forces - N-S Direction							
Level	Story Weight, w_x (k)	Story Height, h_x (ft)	$w_x h_x^K$	C_{VX}	Story Force (k) $F_X=C_{VX}V$	Story Shear (k)	Accidental Moment (k-in)
2nd	1793.75	15.17	55806.14	0.09	40.19	456.32	5497.33
3rd	1775.52	29.17	51786.01	0.08	37.29	416.14	5101.32
4th	1753.46	43.17	75690.96	0.12	54.50	378.85	7456.14
5th	2494.82	57.17	142620.78	0.23	102.70	324.34	14049.24
Penthouse	2219.30	71.75	159234.70	0.25	114.66	221.65	15685.84
Atrium Rf.	548.72	78.92	43303.41	0.07	31.18	106.98	4265.72
Chiller Rf.	336.95	85.75	28893.16	0.05	20.81	75.80	2846.20
AHU Rm. Rf.	810.33	94.25	76373.77	0.12	55.00	55.00	7523.40
						Base Shear [$V=C_SW$]=	456.32 k
						Total Accidental Moment=	54,901.77 k-in

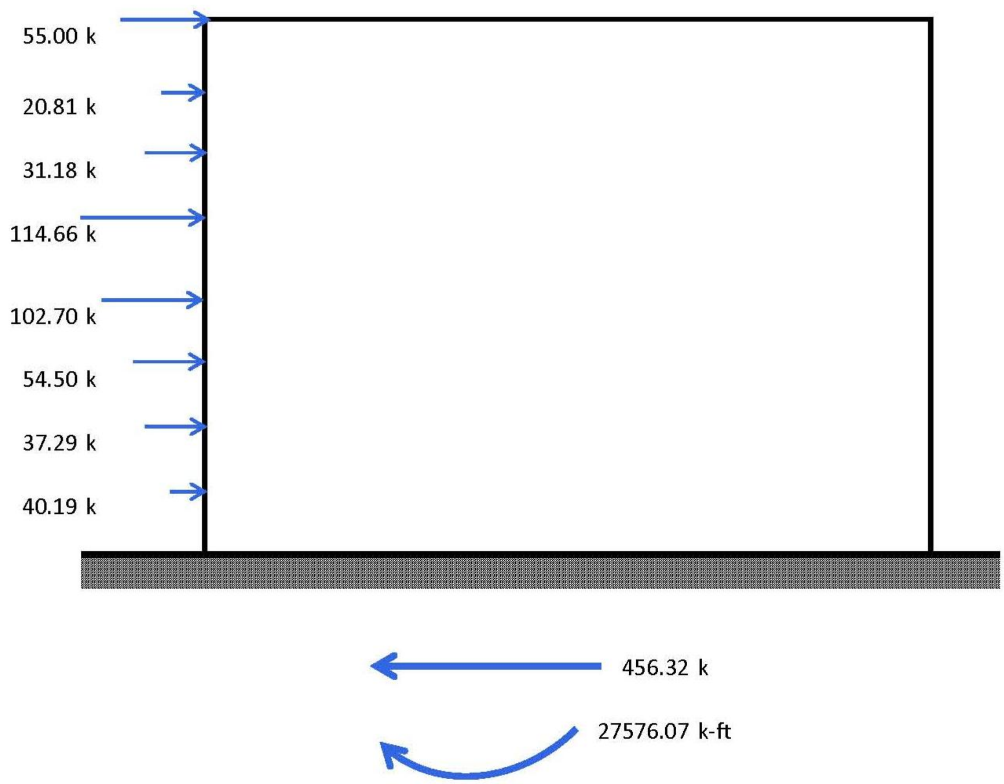


Figure 24 List and diagram of seismic forces for the NE USA S-3 design in the N-S Direction, found with the ELF procedure. Forces in the E-W Direction are identical.

California Site Overview

A geotechnical report was provided by Hammel, Green, and Abrahamson (HGA) Architects and Engineers for a site on the CSUN campus. Figure 25 shows the location of this site and the approximate footprint of the USB on the site. As can be seen, it is large enough for the building's footprint. Also, it is on the corner of two streets, similar to the original site. The orientation of the building can be preserved (the x-axis of the building can remain in the N-S direction and the y-axis can remain in the E-W direction). Lastly, it is in close proximity to the adjacent buildings, similar to the original site. The only major change in the aesthetics of the site is the fact that the corner is to the southeast of the building, as opposed to the original site where the street corner was to the southwest.

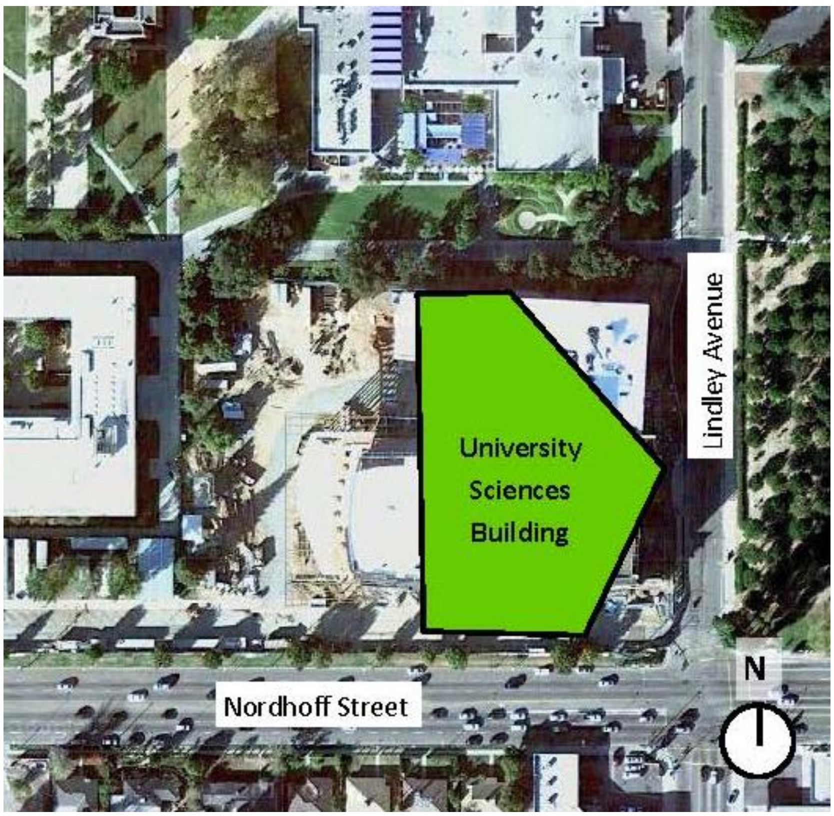


Figure 25 Image from Google Maps showing site selected on California State University, Northridge's (CSUN's) campus. The approximate footprint of the USB is shown in green.

Inspection of the geotechnical report of the California site revealed that the site was Class D, just like the Northeast USA site, which is the most crucial parameter for the production of the designs in this report. The geotechnical engineer also encountered low-quality fill materials up to 13 feet below the surface of the site, and therefore recommended the removal of all material on the site to a depth of 16 feet and subsequent compaction of the below-grade materials. This means that the excavation necessary for the California site would be similar in magnitude to that required at the Northeast USA site.

However, the underground conditions are also different in some ways. The soils at the California site are of a much higher quality, and therefore could probably support shallow foundations. Secondly, ground water is a non-issue on this site, as it was not encountered in any of the 60-foot-deep borings. Both of these differences would be important in redesigning the below-grade portions (the basement and the foundations) of the USB. However, these portions of the building were not included in the redesign, and therefore these conditions can be neglected.

Wind Load Calculations

The basic wind speed for the California site is 85 mph, as opposed to 90 mph at the Northeast USA site. This required the wind loads to be recalculated for the California site. The assumptions made for the calculation of wind loads at the Northeast USA site were also applied to the California site (see the "Wind Loads" subsection of the "Lateral Loads" section for a discussion of what these assumptions were). The full set of parameters used for the calculation of these wind loads can be found in Appendix B.

The wind pressures in both directions are listed in Tables 10 and 11. The N-S direction pressures were resolved into wind forces in the N-S direction, which are listed and diagramed in Figure 26. The resulting base shear is 251.0 k, which is 401.6 k when the 1.6 load factor is considered. The E-W pressures were resolved into wind forces in the E-W direction, which are listed and diagramed in Figure 27. The resulting base shear in this direction is 363.6 k, which is 581.7 k when the 1.6 load factor is considered. The factored base shears were used to compare to the seismic loads for each design to verify that the lateral design was controlled by seismic forces.

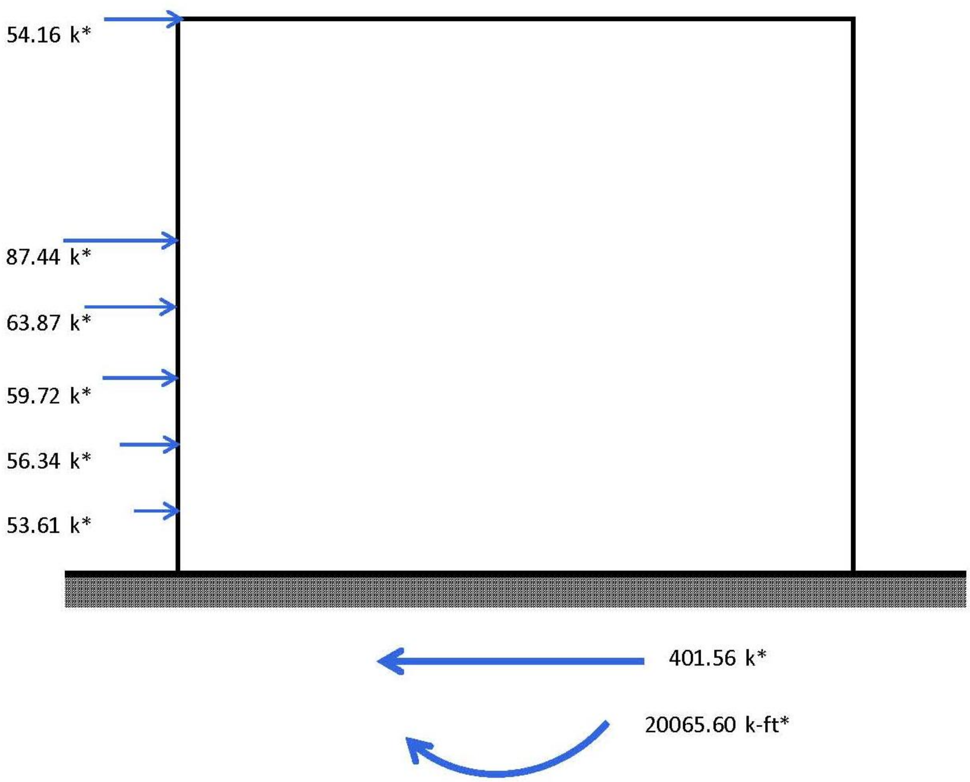
CA Wind Pressures - N-S Direction							
Type	Floor	Distances (ft)	Wind Pressure (psf)	Internal Pressure (psf)		Net Pressure (psf)	
				(+)(GC_{pi})	(-)(GC_{pi})	(+)(GC_{pi})	(-)(GC_{pi})
Windward Walls	Ground	0.00	6.98	3.16	-3.16	3.82	10.14
	2nd	15.17	7.00	3.16	-3.16	3.84	10.16
	3rd	29.17	8.49	3.16	-3.16	5.33	11.65
	4th	43.17	9.50	3.16	-3.16	6.34	12.66
	5th	57.17	10.27	3.16	-3.16	7.10	13.43
	Penthouse	71.75	10.98	3.16	-3.16	7.82	14.15
	Roof	94.25	11.90	3.16	-3.16	8.74	15.06
Leeward Walls	All	All	-5.80	3.16	-3.16	-8.96	-2.64
Side Walls	All	All	-10.41	3.16	-3.16	-13.58	-7.25
Roof	N/A	0-47	-13.39	3.16	-3.16	-16.55	-10.22
	N/A	47-94	-13.39	3.16	-3.16	-16.55	-10.22
	N/A	94-188	-7.44	3.16	-3.16	-10.60	-4.27
	N/A	>188	-4.46	3.16	-3.16	-7.63	-1.30

Table 10 Table of wind pressures in the N-S Direction at the California site.

CA Wind Pressures - E-W Direction							
Type	Floor	Distances (ft)	Wind Pressure (psf)	Internal Pressure (psf)		Net Pressure (psf)	
				(+)(GC_{pi})	(-)(GC_{pi})	(+)(GC_{pi})	(-)(GC_{pi})
Windward Walls	Ground	0.00	6.82	3.16	-3.16	3.66	9.99
	2nd	15.17	6.84	3.16	-3.16	3.68	10.01
	3rd	29.17	8.30	3.16	-3.16	5.14	11.46
	4th	43.17	9.29	3.16	-3.16	6.13	12.45
	5th	57.17	10.04	3.16	-3.16	6.88	13.20
	Penthouse	71.75	10.74	3.16	-3.16	7.58	13.90
	Roof	94.25	11.64	3.16	-3.16	8.47	14.80
Leeward Walls	All	All	-7.27	3.16	-3.16	-10.44	-4.11
Side Walls	All	All	-10.18	3.16	-3.16	-13.35	-7.02
Roof	N/A	0-47	-15.75	3.16	-3.16	-18.92	-12.59
	N/A	47-94	-11.76	3.16	-3.16	-14.92	-8.60
	N/A	94-188	-8.60	3.16	-3.16	-11.77	-5.44
	N/A	>188	N/A	N/A	N/A	N/A	N/A

Table 11 Table of wind pressures in the E-W Direction at the California site.

CA Wind Forces - N-S Direction								
Floor Level	Elevation (ft)	Trib. Below		Trib. Above		Story Force (k)	Story Shear (K)	Accidental Moment (k-in)
		Height (ft)	Area (ft^2)	Height (ft)	Area (ft^2)			
Ground	0.00	N/A	0.00	7.59	1289.45	16.50	250.98	3832.31
2nd	15.17	7.59	1289.45	7.00	1190	33.51	234.47	7780.65
3rd	29.17	7.00	1190.00	7.00	1190	35.21	200.96	8175.79
4th	43.17	7.00	1190.00	7.00	1190	37.33	165.75	8667.46
5th	57.17	7.00	1190.00	7.29	1239.3	39.92	128.43	9269.49
Penthouse	71.75	7.29	1239.30	11.25	1912.5	54.65	88.50	12690.16
Roof	94.25	11.25	1912.50	N/A	0.00	33.85	33.85	7860.68
							Total Base Shear=	250.98 k
							Total Accidental Moment=	58,276.54 k-in



*Note: Includes 1.6 load factor per ASCE 7-05 Chapter 2

Figure 26 List and diagram of the wind forces in the N-S Direction at the California site.

CA Wind Forces - E-W Direction								
Floor Level	Elevation (ft)	Trib. Below		Trib. Above		Story Force (k)	Story Shear (K)	Accidental Moment (k-in)
		Height (ft)	Area (ft^2)	Height (ft)	Area (ft^2)			
Ground	0.00	N/A	0.00	7.59	1729.38	24.41	363.56	8789.21
2nd	15.17	7.59	1729.38	7.00	1596.00	49.27	339.15	17737.48
3rd	29.17	7.00	1596.00	7.00	1596.00	51.29	289.88	18464.04
4th	43.17	7.00	1596.00	7.00	1596.00	54.07	238.59	19463.77
5th	57.17	7.00	1596.00	7.29	1662.12	57.57	184.52	20726.02
Penthouse	71.75	7.29	1662.12	11.25	2565.00	78.44	126.95	28239.92
Roof	94.25	11.25	2565.00	N/A	0.00	48.51	48.51	17461.90
							Total Base Shear=	363.56 k
							Total Accidental Moment=	130,882.34 k-in

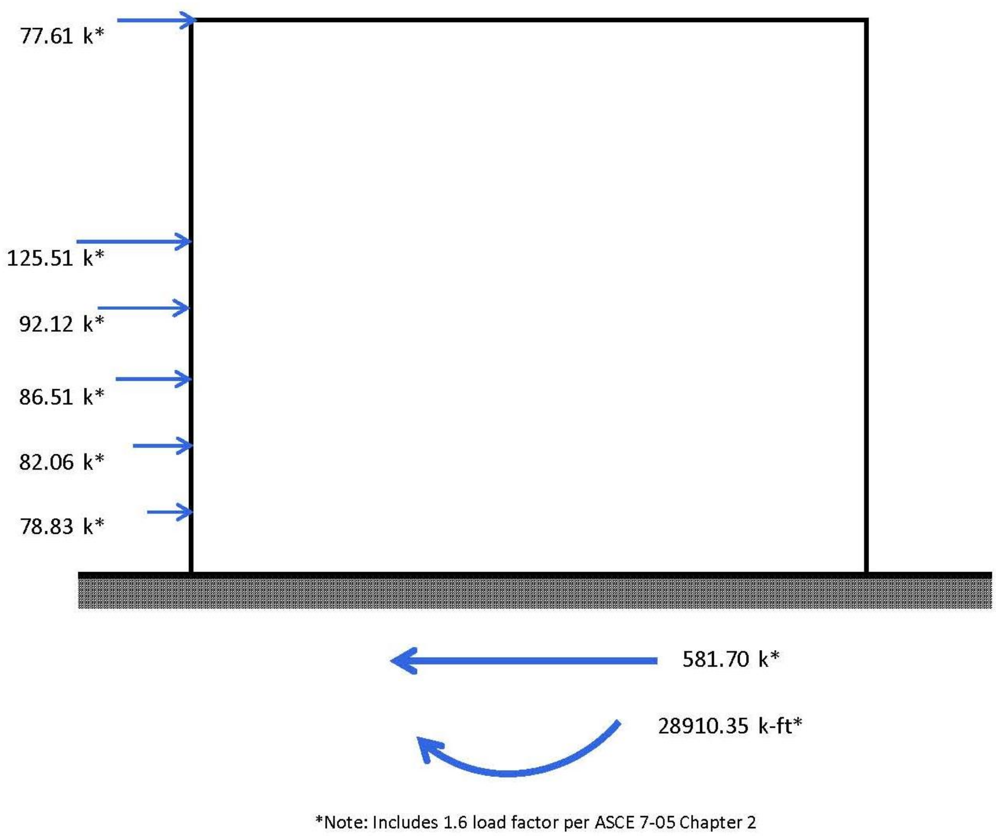


Figure 27 List and diagram of the wind forces in the E-W Direction at the California site.

Code Minimum Steel Moment Frame in California (CA S-3)

This design was created for two purposes. First, it served as a baseline to which the higher performance structures could be compared. Secondly, it was the structure to which the viscous fluid dampers were added, and therefore would have had to be designed anyways.

Seismic Loads

It was assumed that this design would be controlled by seismic forces, and therefore seismic forces (which are dependent on weight of the structure) had to be calculated for this design. In order to accomplish this, the NE USA S-3 model was used as a baseline for the design. However, it was assumed that the weight of the building would increase, since it was assumed larger members would be needed to resist the seismic loads than the Northeast USA wind loads. Therefore, the steel weight from the NE USA S-3 model was increased by 50% (multiplied by 1.5). This produced approximately a 5% increase in overall building weight. These weights and the appropriate site parameters for the California site (which can be found in Appendix C) were used to calculate estimated ELF forces. For this model, the estimated ELF base shears were 814.8 k in both directions, which far exceeds the calculated wind forces for the California location.

Upon completion of the design with the estimated forces, the overall structure was found to weigh approximately 12,300 k, which was 4.5% heavier than the NE USA S-3. This was very close to the estimated 5% increase, and therefore the estimated design was determined to be accurate for ELF forces. However, due to the irregularities in this structure (see the discussion in the "Frame Design" subsection below), the structure had to be designed for MRSA forces.

Therefore, the MRSA forces were calculated for the USB by finding a Cs-like quantity for each mode, then combining these into a single quantity using the square root of the sum of squares (SRSS) method. This calculation can be seen in Table 12. MRSA base shear is limited by code to a minimum of 85% of the ELF base shear. The base shear in both directions was controlled by this minimum, and was therefore found to be 687.3 k. These seismic forces are listed and diagrammed in Figure 28.

CA S-3 - Modal Information				
Mode	Period	UX%	UY%	$C_{m,i}$
1	2.355	51.15%	1.32%	0.049219
2	2.198	1.01%	78.35%	0.049219
3	1.966	28.13%	0.02%	0.050082
4	0.847	0.08%	1.24%	0.116284
5	0.775	7.21%	1.33%	0.127036
6	0.755	1.06%	5.99%	0.130341
7	0.607	1.17%	0.02%	0.162037
8	0.550	0.51%	0.55%	0.178869
9	0.495	0.39%	3.32%	0.198753
			$C_{m,x}$=SQRT(Σ($C_{m,i}$*UX%)^2)=	0.030389
			$C_{m,y}$=SQRT(Σ($C_{m,i}$*Uy%)^2)=	0.039972

Table 12 Modal information used to find Cm, which was used to calculate MRSA seismic forces.

CA S-3 - MRSA Seismic Forces - N-S Direction							
Level	Story Weight, w_x (k)	Story Height, h_x (ft)	$w_x h_x^K$	C_{VX}	Story Force (k) $F_X=C_{VX}V$	Story Shear (k)	Accidental Moment (k-in)
2nd	1884.64	15.17	122991.57	0.02	14.92	687.28	2040.81
3rd	1867.80	29.17	332957.28	0.06	40.39	672.36	5524.78
4th	1858.99	43.17	605303.04	0.11	73.42	631.98	10043.83
5th	2590.94	57.17	1299027.39	0.23	157.56	558.56	21554.84
Penthouse	2281.28	71.75	1621728.11	0.29	196.71	400.99	26909.43
Atrium Rf.	569.48	78.92	468612.63	0.08	56.84	204.29	7775.72
Chiller Rf.	354.13	85.75	331071.29	0.06	40.16	147.45	5493.48
AHU Rm. Rf.	818.26	94.25	884551.64	0.16	107.29	107.29	14677.42
						Base Shear [$V=C_S W$]=	687.28 k
						Total Accidental Moment=	79,342.88 k-in

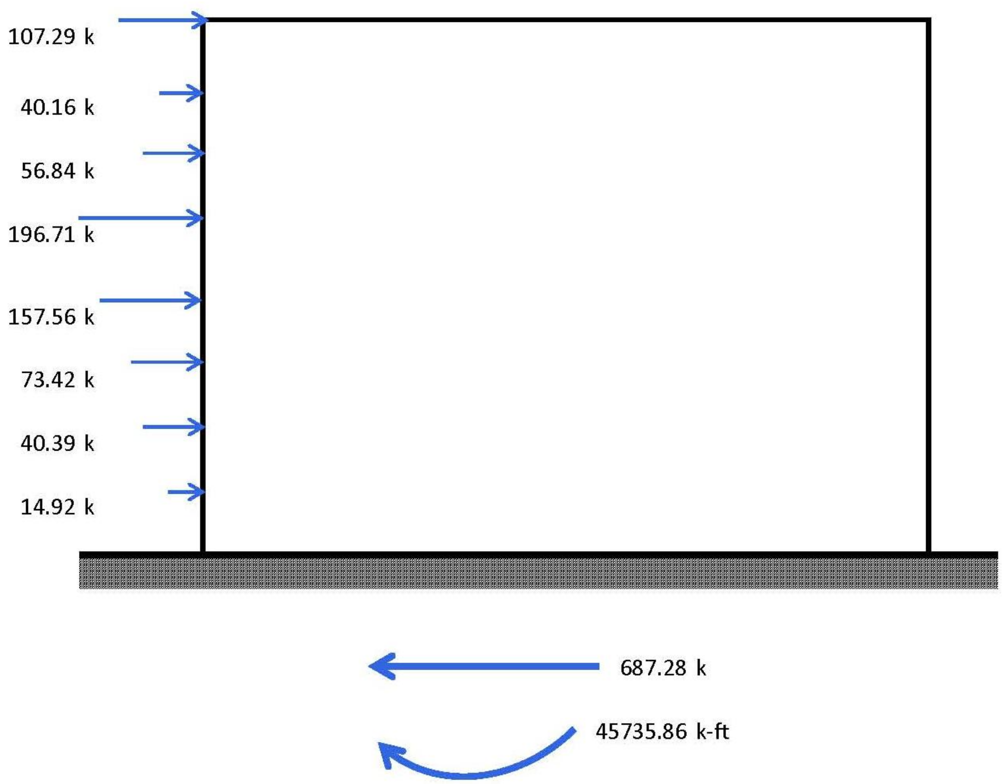


Figure 28 List and diagram of seismic forces for the CA S-3 design in the N-S Direction, found with the MRSA procedure. Forces in the E-W Direction are identical.

Frame Design

The moment frame layout created for the NE USA S-3 design was also used for the CA S-3 design (see Figure 20), and therefore the design procedure for the CA S-3 structure was similar to that used for the NE USA S-3 structure. The estimated ELF forces were entered into the portal frame spreadsheet, which was modified to also check for soft story condition and strong-column-weak-beam condition. The result for a sample frame can be seen in Figure 29. A sample column interaction check can be found in Appendix E.

Since the spreadsheet was found to be highly inaccurate at calculating drift, which is crucial in seismic design, the frame sizes were further refined using SAP. Simple 2-dimensional models consisting of a column and one-half of the bay length of the lateral beams on each side of the column were constructed. These models are commonly referred to as "drift trees," and when a response spectrum load is applied to them, they can predict the drift in the columns due to seismic loads. The column and beam sizes were adjusted as required to reduce the drift in the drift trees under the allowable limit. Due to the fact this structure is classified as Occupancy Category III, the allowable drift was found to be 1.5% of the story height. Table 13 shows the final trial member sizes for Frame 7.

Before any modeling was undertaken for the whole structure, it was necessary to determine which irregularities were applicable to it. ASCE 7-05 recognizes 5 different types of irregularities in both the horizontal and vertical directions. All were evaluated in relationship to the USB, the results of which are included in Appendix E. It was found that the USB is subject to Type 1 irregularity in both the horizontal and vertical directions as well as Type 5 horizontal irregularity. This meant that the accidental moments had to be amplified and the earthquake loads in a given direction had to be applied simultaneously with 30% of the earthquake loads in the perpendicular direction.

The final trial sizes were entered into a RAM model, which was then used to optimize the sizes for both strength and to further refine the drift of the structure. The process through which this was achieved is identical to the one used to refine the NE USA S-3 model. This model also incorporated the reduced beam sections, or dog bones, which is a feature available in RAM. In its design process, RAM considers the requirements of the Seismic Provisions for Structural Steel Buildings (AISC 341-05), which includes limitations for the frames in general, as well as the provisions in the Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications (AISC 358-05), which provides recommendations on how to design the dog bones in order to ensure proper performance.

The design of the connection was verified using hand calculations (which can be found in Appendix E) and knowledge from AE 534. A detail of the final connection can be seen in Figure 30. Additionally, when a frame undergoes seismic loads, it can experience load reversal, which results in compression occurring on the bottom flange. This can cause the beam to undergo lateral-torsional buckling in the bottom flange. Therefore, the bottom flange must be braced to prevent this from occurring. The required bracing for a single beam was designed by hand to show understanding of the provisions, the calculations for which can be found in Appendix E. A detail of the brace connection can be seen in Figure 31.

For final verification, the final design from RAM was also evaluated in ETABS. Drifts from ETABS were compared to drifts from RAM, and they were found to be comparable (ETABS drifts are summarized in Tables 14 and 15 and can be found in the "System Comparison/Summary" section, under the "System Drift Summary" subsection). Therefore, the modal properties of the building as found in ETABS (listed in Table 12) were used to calculate the MRSA forces. The MRSA forces were applied to the structure along with the amplifications to accidental moments, and it was found that the structure as it was designed for the estimated ELF forces was sufficient to resist these loads without being overdesigned. Therefore, this design was considered complete.

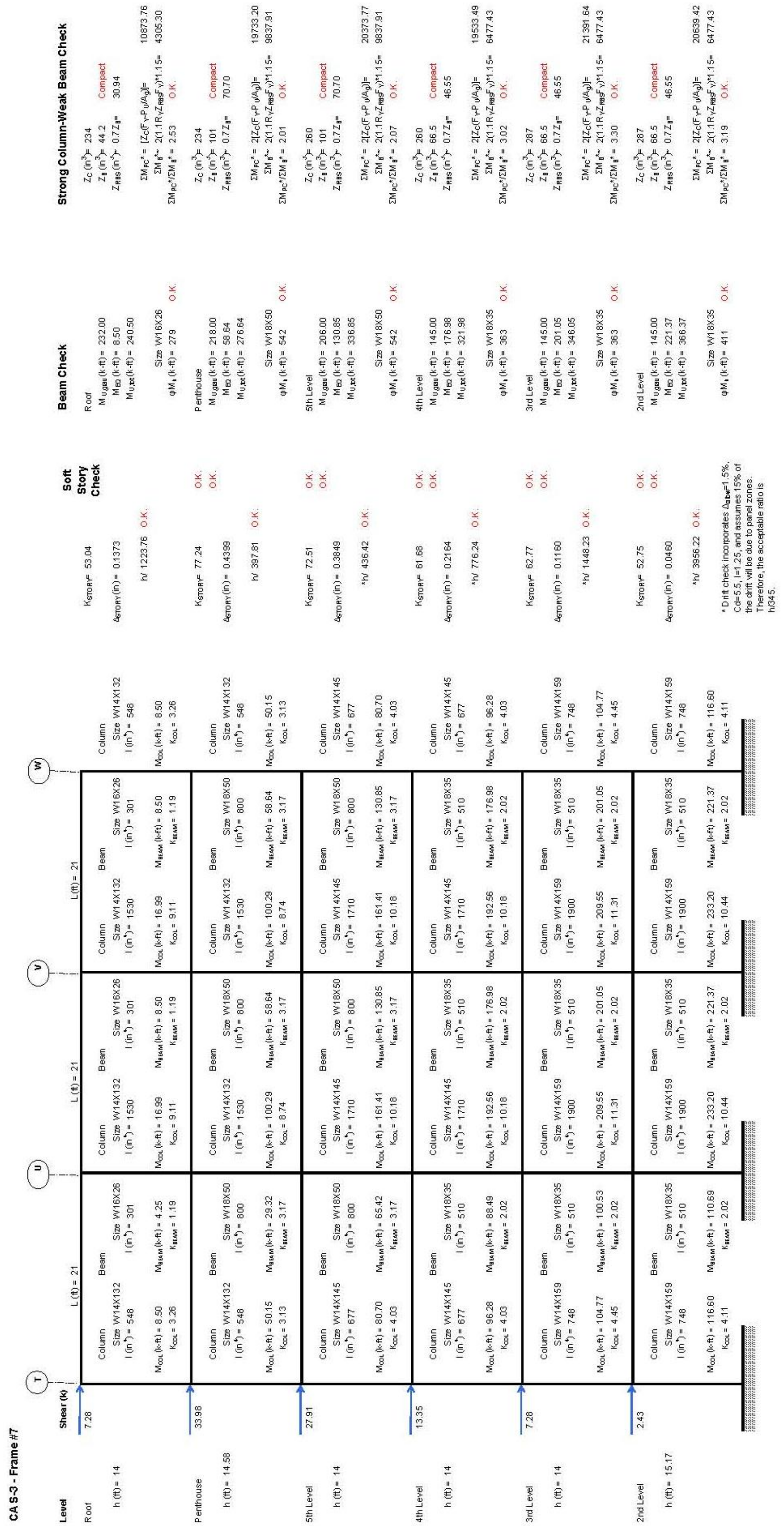


Figure 29 Example of the portal frame spreadsheet used to find moments in the frames due to the estimated ELF forces.

CA S-3 - Column U/12, Frame 7 Drift Check				
Level	Column Size	Beam Size	Shear in Monitor Col. (k)	<1.5%?
2nd Level	W14X283	W18X71	0.008	Yes
3rd Level	W14X283	W18X71	0.013	Yes
4th Level	W14X211	W18X71	0.015	Yes
5th Level	W14X211	W18X55	0.014	Yes
Penthouse Level	W14X159	W18X50	0.014	Yes
Chiller Roof Level	W14X159	W16X26	0.012	Yes

Table 13 Beam and column sizes required to reduce drift in Frame 7 below allowable limits, found using a SAP drift tree model.

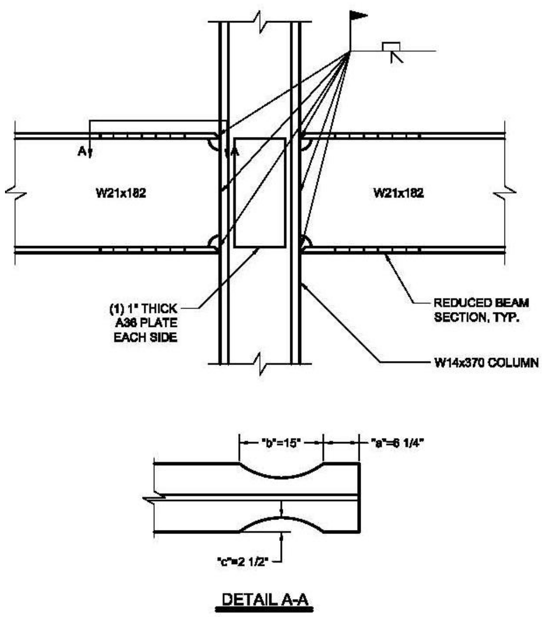


Figure 30 Final sample moment connection design for the CA S-3 structure. Hand calculations can be found in Appendix E.

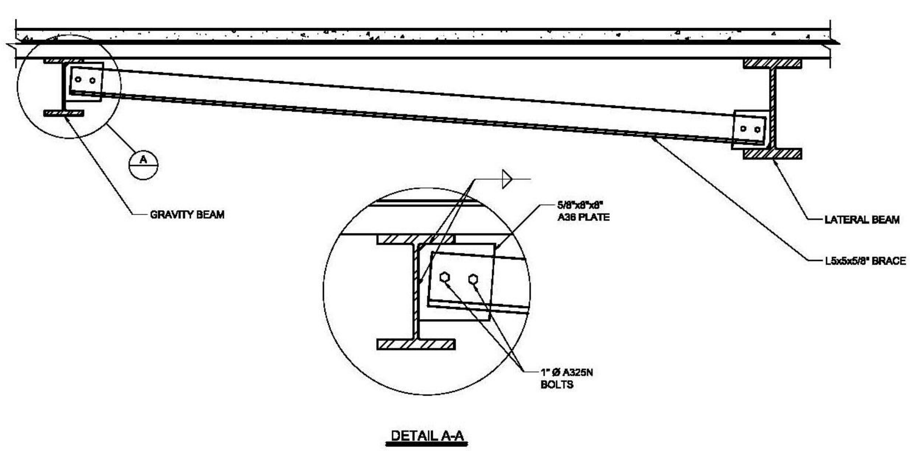


Figure 31 Final sample brace connection design for the CA S-3 structure. Hand calculations can be found in Appendix E.

CA S-3 - N-S Drift Check for Torsional Irregularity						
Level	δ_{max} *	δ_{min} *	δ_{avg}	$\delta_{max}/\delta_{avg}$	Irreg.	Ax
AHU Roof	0.012166	0.010019	0.011092	1.097	N/A	N/A
Chiller Roof	0.009209	0.002631	0.005920	1.556	1b	1.68
Atrium Roof	0.014340	0.011726	0.013033	1.100	N/A	N/A
Penthouse	0.012712	0.010507	0.011609	1.095	N/A	N/A
5th Level	0.014969	0.010657	0.012813	1.168	N/A	N/A
4th Level	0.014964	0.011075	0.013020	1.149	N/A	N/A
3rd Level	0.014547	0.009799	0.012173	1.195	N/A	N/A
2nd Level	0.009988	0.006424	0.008206	1.217	1a	1.029

* Note: All drifts are taken from the controlling ETABS MRSA load combination results and multiplied by Cd/I (5.5/1.25=4.4) per ASCE 7-05 Section 12.9.2. Per ASCE 7-05 Table 12.12-1, the USB is classified as "All Other Structures" with an Occupancy Category III, and therefore has an allowable drift ratio of 0.015.

Table 14 Drift and torsional irregularity check in the N-S Direction for the CA S-3 design.

CA S-3 - E-W Drift Check for Torsional Irregularity						
Level	δ_{max} *	δ_{min} *	δ_{avg}	$\delta_{max}/\delta_{avg}$	Irreg.	Ax
AHU Roof	0.010472	0.007058	0.008765	1.195	N/A	N/A
Chiller Roof	0.012338	0.006041	0.009189	1.343	1a	1.252
Atrium Roof	0.014164	0.011774	0.012969	1.092	N/A	N/A
Penthouse	0.012522	0.012399	0.012461	1.005	N/A	N/A
5th Level	0.013323	0.012910	0.013116	1.016	N/A	N/A
4th Level	0.014304	0.014221	0.014263	1.003	N/A	N/A
3rd Level	0.013891	0.012857	0.013374	1.039	N/A	N/A
2nd Level	0.008320	0.007841	0.008081	1.030	N/A	N/A

* Note: All drifts are taken from the controlling ETABS MRSA load combination results and multiplied by Cd/I (5.5/1.25=4.4) per ASCE 7-05 Section 12.9.2. Per ASCE 7-05 Table 12.12-1, the USB is classified as "All Other Structures" with an Occupancy Category III, and therefore has an allowable drift ratio of 0.015.

Table 15 Drift and torsional irregularity check in the E-W Direction for the CA S-3 design.

Immediate Occupancy Steel Moment Frame in California (CA S-1)

This design was the first which was undertaken in order to reach the higher performance rating. It was understood that this design was likely to be highly impractical due to the large member sizes that would be required to reduce drift below the allowable limit of 0.7% (as given in ASCE 41-05, see Figure 15). However, since the goal was to compare the efficiency of the high-tech design with the more traditional design, this was a necessary baseline.

Seismic Loads
As with the CA S-3 frame, seismic loads had to be estimated with a reasonable degree of accuracy to give a starting-point for the design. Therefore, the weight of the CA S-1 structure was estimated by increasing the steel weight of the CA S-3 model by 75% (multiplying by a factor of 1.75). This increased the overall building weight by 10%. This was used to calculate the estimated ELF forces, which resulted in a base shear in both directions of 849.0 k. This was used for all preliminary design.

Upon completion of the design with the estimated forces, the overall structure was found to weigh approximately 13,500 k, which was 9.7% heavier than the CA S-3 structure. This was very close to the estimated 10% increase, and therefore the estimated design was determined to be accurate for ELF forces. However, due to the irregularities in this structure (see the discussion in the "Frame Design" subsection of the CA S-3 design section), the structure had to be designed for MRSA forces.

Therefore, the MRSA forces were calculated for the USB by finding a Cs-like quantity for each mode, then combining these into a single quantity using the square root of the sum of squares (SRSS) method. This calculation can be seen in Table 16. MRSA base shear is limited by code to a minimum of 85% of the ELF base shear. The base shear in both directions was controlled by this minimum, and was therefore found to be 749.6 k. These seismic forces are listed and diagrammed in Figure 32.

CA S-1 - Modal Information				
Mode	Period	UX%	UY%	$C_{m,i}$
1	1.426	54.71%	1.25%	0.069044
2	1.372	7.08%	68.14%	0.071757
3	1.093	18.67%	9.82%	0.090065
4	0.551	0.11%	1.38%	0.178746
5	0.492	6.91%	0.60%	0.200164
6	0.477	0.35%	6.77%	0.206477
7	0.388	2.31%	0.05%	0.208333
8	0.359	0.49%	0.39%	0.208333
9	0.327	0.01%	2.64%	0.208333
			$C_{m,x}$=SQRT(Σ($C_{m,i}$*UX%)^2)=	0.044181
			$C_{m,y}$=SQRT(Σ($C_{m,i}$*Uy%)^2)=	0.051998

Table 16 Modal information used to find Cm, which was used to calculate MRSA seismic forces.

CA S-1 - MRSA Seismic Forces - N-S Direction							
Level	Story Weight, w_x (k)	Story Height, h_x (ft)	$w_x h_x^K$	C_{VX}	Story Force (k) $F_X=C_{VX}V$	Story Shear (k)	Accidental Moment (k-in)
2nd	2118.46	15.17	62299.81	0.03	25.58	749.59	3498.89
3rd	2087.54	29.17	138439.20	0.08	56.84	724.02	7775.04
4th	2068.62	43.17	223372.51	0.12	91.70	667.18	12545.07
5th	2772.09	57.17	424482.77	0.23	174.27	575.48	23839.85
Penthouse	2397.09	71.75	486907.54	0.27	199.90	401.21	27345.76
Atrium Rf.	637.26	78.92	145711.75	0.08	59.82	201.32	8183.48
Chiller Rf.	398.49	85.75	101029.61	0.06	41.48	141.49	5674.04
AHU Rm. Rf.	854.38	94.25	243623.87	0.13	100.02	100.02	13682.43
						Base Shear [$V=C_SW$]=	749.59 k
						Total Accidental Moment=	88,862.12 k-in

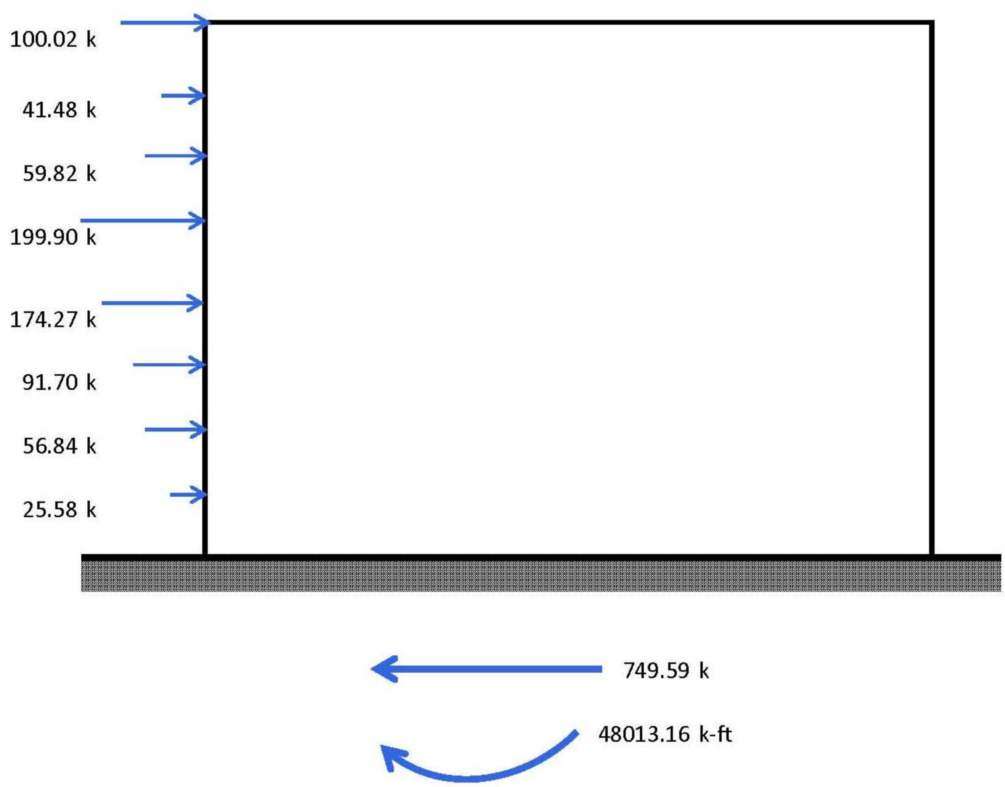


Figure 32 List and diagram of seismic forces for the CA S-1 design in the N-S Direction, found with the MRSA procedure. Forces in the E-W Direction are identical.

Frame Design

The moment frame layout created for the NE USA S-3 design was also used for the CA S-1 design (see Figure 20), and therefore the design procedure for the CA S-1 structure was similar to that used for the previous two structures. The estimated ELF forces were entered into the seismic version of the portal frame spreadsheet. The result for a sample frame and a sample column interaction check can be found in Appendix E.

Since the spreadsheet was found to be highly inaccurate at calculating drift, which is crucial in seismic design, the frame sizes were further refined using drift trees in SAP. Table 17 shows the final trial member sizes for a sample frame.

The final trial sizes were entered into a RAM model, which was then used to optimize the sizes for both strength and to further refine the drift of the structure. The process through which this was achieved is identical to the one used to refine the previous models. This model also incorporated the reduced beam sections, or dog bones, which is a feature available in RAM. As with the CA S-3 model, RAM designed the frames considering the provisions of AISC 341-05 and 358-05. The design of a connection was verified using hand calculations (which can be found in Appendix E) and knowledge gained from advanced master's level coursework in AE 534 (Steel Connection Design). A detail of the connection is shown in Figure 33. Additionally, the required bracing for a single beam was designed by hand to show understanding of the provisions, the calculations for which can be found in Appendix E. It was found that the bracing requirements were similar enough to those in CA S-3 for the brace design from the CA S-3 model to be used, so a connection detail for this brace can be seen in Figure 31.

For final verification, the final design from RAM was also evaluated in ETABS because the author is more familiar with the modal capabilities of ETABS. Drifts from ETABS were compared to drifts from RAM, and they were found to be comparable (ETABS drifts are summarized in Tables 18 and 19 and can be found in the "System Comparison/Summary" section, under the "System Drifts Summary" subsection). Therefore, the modal properties of the building as found in ETABS (listed in Table 16) were used to calculate the MRSA forces. The MRSA forces were applied to the structure along with the amplifications to accidental moments, and it was found that the structure as it was designed for the estimated ELF forces was sufficient to resist these loads without being overdesigned. Therefore, this design was considered complete.

CA S-1 - Column U/12, Frame 7 Drift Check				
Level	Column Size	Beam Size	Shear in Monitor Col. (k)	<0.7%?
2nd Level	W14X550	W24X192	0.0048	Yes
3rd Level	W14X550	W24X192	0.0067	Yes
4th Level	W14X500	W24X192	0.0069	Yes
5th Level	W14X500	W24X192	0.0063	Yes
Penthouse Level	W14X370	W24X146	0.0064	Yes
Chiller Roof Level	W14X370	W18X50	0.0058	Yes

Table 17 Beam and column sizes required to reduce drift in Frame 7 below allowable limits, found using a SAP drift tree model.

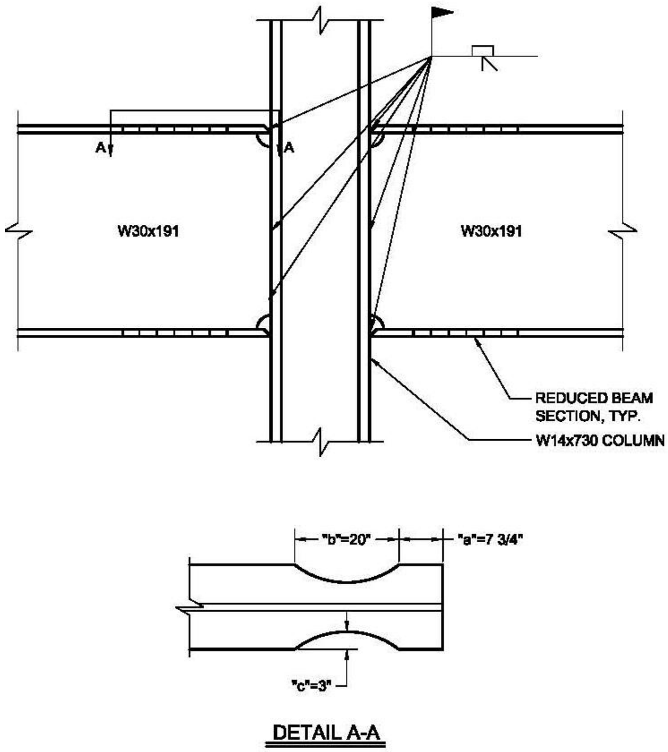


Figure 33 Final sample moment connection design for the CA S-1 structure. Hand calculations can be found in Appendix E.

CA S-1 - N-S Drift Check for Torsional Irregularity						
Level	δ_{max} *	δ_{min} *	δ_{avg}	$\delta_{max}/\delta_{avg}$	Irreg.	Ax
AHU Roof	0.005892	0.003797	0.004844	1.216	1a	1.027
Chiller Roof	0.004734	0.002059	0.003397	1.394	1a	1.349
Atrium Roof	0.006987	0.002213	0.004600	1.519	1b	1.602
Penthouse	0.006310	0.003912	0.005111	1.235	1a	1.059
5th Level	0.006662	0.003714	0.005188	1.284	1a	1.145
4th Level	0.006965	0.003991	0.005478	1.271	1a	1.123
3rd Level	0.006948	0.004017	0.005482	1.267	1a	1.115
2nd Level	0.004426	0.002728	0.003577	1.237	1a	1.063

* Note: All drifts are taken from the controlling ETABS MRSA load combination results and multiplied by Cd/I (5.5/1.25=4.4) per ASCE 7-05 Section 12.9.2. Per ASCE 41-05 Table C1-3, the allowable drift for an S-1 steel moment frame structure is 0.007.

Table 18 Drift and torsional irregularity check in the N-S Direction for the CA S-1 design.

CA S-1 - E-W Drift Check for Torsional Irregularity						
Level	δ_{max} *	δ_{min} *	δ_{avg}	$\delta_{max}/\delta_{avg}$	Irreg.	Ax
AHU Roof	0.003942	0.001087	0.002515	1.568	1b	1.707
Chiller Roof	0.006692	0.006640	0.006666	1.004	N/A	N/A
Atrium Roof	0.006948	0.001135	0.004041	1.719	1b	2.052
Penthouse	0.006930	0.004215	0.005573	1.244	1a	1.074
5th Level	0.006626	0.004176	0.005401	1.227	1a	1.045
4th Level	0.006714	0.004576	0.005645	1.189	N/A	N/A
3rd Level	0.006851	0.004501	0.005676	1.207	1a	1.012
2nd Level	0.004308	0.002873	0.003590	1.200	N/A	N/A

* Note: All drifts are taken from the controlling ETABS MRSA load combination results and multiplied by Cd/I (5.5/1.25=4.4) per ASCE 7-05 Section 12.9.2. Per ASCE 41-05 Table C1-3, the allowable drift for an S-1 steel moment frame structure is 0.007.

Table 19 Drift and torsional irregularity check in the E-W Direction for the CA S-1 design.

CA S-3 Frame Augmented with Viscous Fluid Dampers (CA S-3 with VFD)

This design uses the CA S-3 design as a baseline structure, and then proceeds to simply add viscous fluid dampers to the frame in an effort to reduce building drifts below 0.7%, which is the allowable drift for an immediate occupancy structure as given in ASCE 41-05 (see Figure 15).

Seismic Loads

Seismic loads were not calculated for this design. Dampers are designed for a target damping percentage rather than a specific force. A discussion of this process is found in the following subsections.

Damper Layout

The code recommends a minimum of 2 lines of damper resistance in each of two orthogonal directions in each story, configured to reduce torsional effects (if this is not provided, several additional provisions must be considered). Since the frame used as a base for this design was already code-compliant, this recommendation did not have to be followed. However, it seemed like a good guideline to follow whenever possible when laying out the damper frames. Since Frames 1-4 span from the ground to the very highest roof, two damper bays were placed in each direction in those bays. In the remaining wings of the buildings, one damper frame was placed in each direction in order to help reduce torsion (since the building is already severely susceptible to torsional problems).

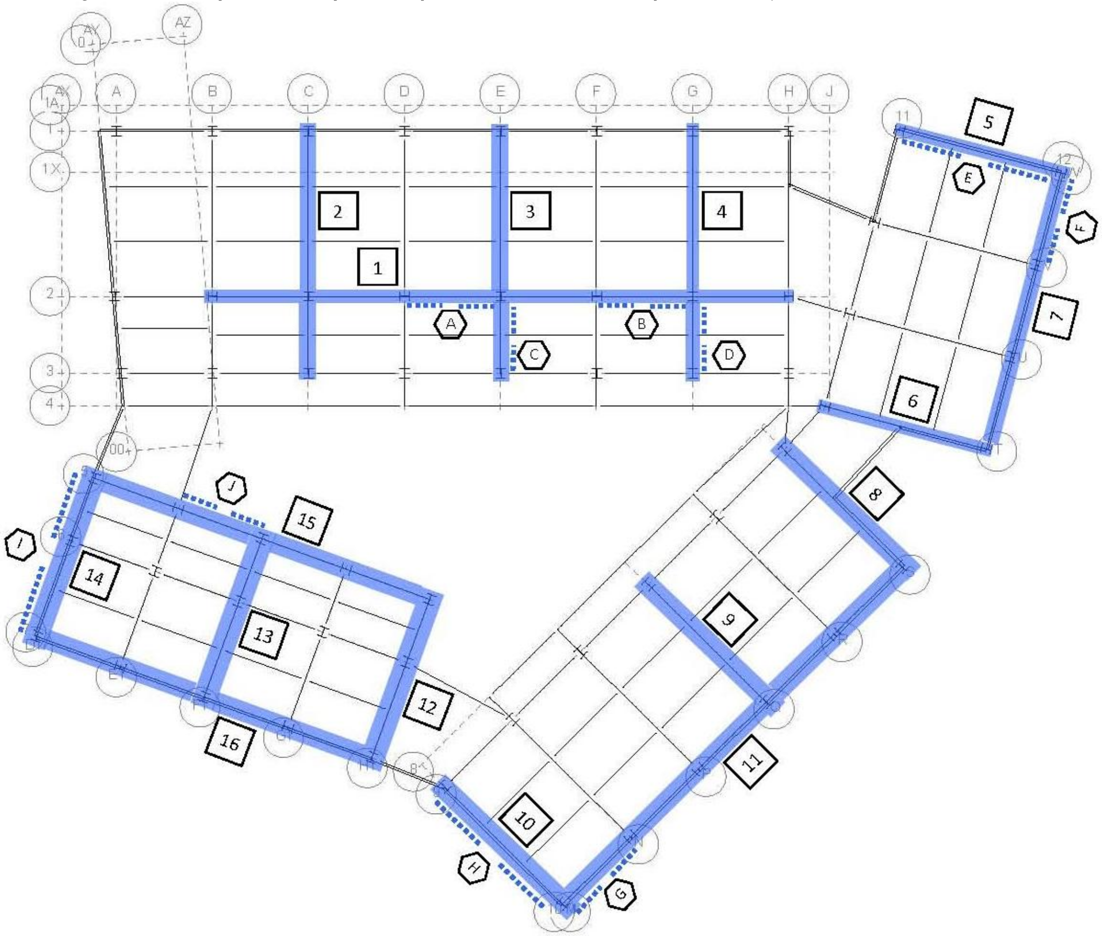


Figure 34 Floor plan showing the locations of the braced frames containing the viscous fluid dampers. The dampers are lettered, the moment frames are numbered.

The largest driving force in choosing the final locations for the damper frames was architectural. The frames were located to avoid doors, critical open spaces, areas where reduced head height would be undesirable, and to reduce the impact to the window layout on the exterior walls. Although some windows were compromised, they were largely in the rear and sides of the building, which are less crucial to the architectural impact of the overall structure. The final layout can be seen in Figure 34. The damper frames were lettered for ease of reference.

Preliminary Damper Sizing

Dampers are designed for a target damping percentage at which the Demand Response Spectrum crosses the Capacity Spectrum of the structure. The initial Demand Response Spectrum was the site-specific response spectrum for an R=1 and a damping of 5%, which was provided in the geotechnical report. The Capacity Spectrum was found using a static pushover load case on the ETABS model. The Response Spectrum was reduced for higher levels of damping using recommended equations from Chapter 15 of the Seismic Design Handbook by Farzad Naeim, shown below.

$$SR_A = \frac{3.21 - 0.68 \ln \beta_{eff}}{2.12}$$

$$SR_V = \frac{2.31 - 0.41 \ln \beta_{eff}}{1.65}$$

In these equations, βeff is the total target damping in percent form (for instance, if 30% is the total target damping, 30 should be entered into the equation, not 0.30). SR_A is multiplied by spectral acceleration and spectral displacement values during the portion of the spectrum that is approximately constant and SR_V is multiplied by spectral acceleration and spectral displacement values during the portion of the spectrum that is a parabolic curve to produce the Demand Response Spectrum reflecting the desired damping.

Although the application of these equations is slightly arbitrary, according to Kit Miyamoto and Robert Hanson (two well-respected experts in the field of viscous fluid damper design) from their paper entitled "U.S. Code Development of Structures with Damping Systems", "many design firms prefer to use a response history calculation as final verification of the combined system performance. In that case, any reasonable method for preliminary design of the seismic force-resisting system and the damping system can be used." Since exactly such a response history calculation was performed for the USB (see the "Nonlinear Analysis" subsection in this section), it was deemed this was a sufficiently reasonable method. A sample set of Demand Spectrum and Capacity Spectrum plots can be seen in Figure 35.

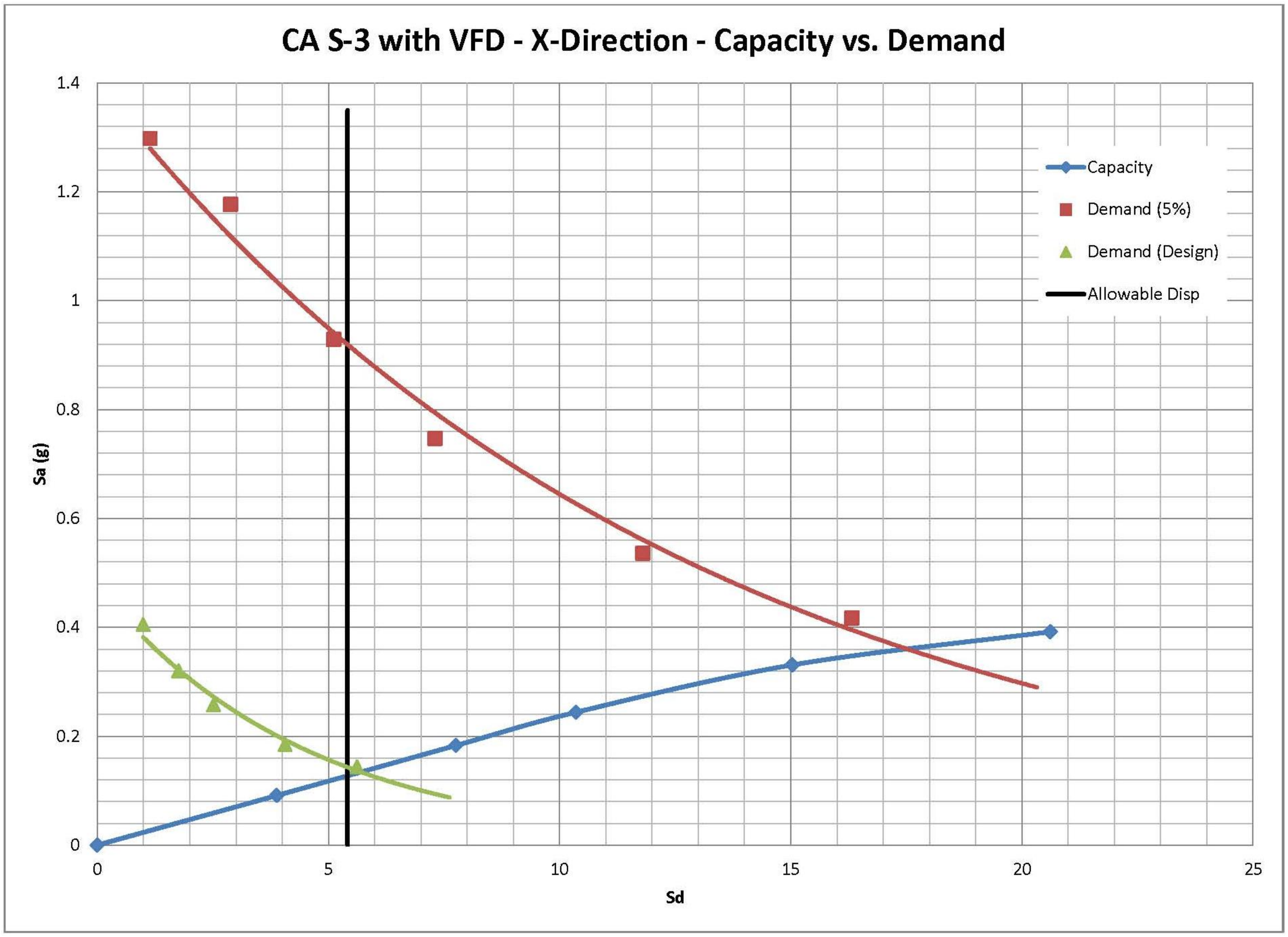


Figure 35 Sample plot of Capacity Spectrum vs. Demand Response Spectrum, both for 5% and for the design damping.

Finally, the Allowable Displacement was found by taking the highest roof height, 94'-3", multiplied by the allowable drift (0.7%, which resulted in δ=7.92 inches). This was then converted to spectral coordinates using the following equation.

$$S_D = \frac{\delta}{PF\%(\varphi)}$$

Where PF% is the percentage of mass participation in the primary mode, and ϕ is the modal displacement at the roof for the primary mode. The design damping percentage was then modified through trial-and-error until the Damped Capacity Spectrum and the Damped Demand Response Spectrum curves met approximately at the allowable spectral displacement. For both the N-S (x) and E-W (y) directions, the target damping percentage was found to be 70%.

Although there is a plethora of research available on how to find damped response spectra and capacity spectra, there is a corresponding dearth of papers on how to turn the required damping percentage into actual forces to size the dampers. However, Chapter 18 of ASCE 7-05 contains a procedure for calculating the required damping force per story per mode based on the percentage of critical damping. These can then be combined through SRSS to achieve a single required damping force per story.

However, it is important to note that the equations in Chapter 18 are circular in nature. For instance, take the following set of equations:

$$T_{1D} = T_1(\mu_D)^{0.5} \quad \text{(Equation 18.4-6)}$$

$$\mu_D = \frac{D_{1D}}{D_Y} \quad \text{(Equation 18.6-8)}$$

$$D_{1D} = \frac{g}{4\pi^2}\ \Gamma_1 \frac{S_{DS}T_{1D}^2}{B_{1D}} \geq \frac{g}{4\pi^2}\ \Gamma_1 \frac{S_{DS}T_1^2}{B_{1E}}\ if\ T_{1D} < T_S \quad \text{(Equation 18.4-12a)}$$

$$D_{1D} = \frac{g}{4\pi^2}\ \Gamma_1 \frac{S_{D1}T_{1D}}{B_{1D}} \geq \frac{g}{4\pi^2}\ \Gamma_1 \frac{S_{D1}T_1}{B_{1E}}\ if\ T_{1D} \geq T_S \quad \text{(Equation 18.4-12b)}$$

As can be seen, in order to find T_{1D}, you must first know μ_D, which requires D_{1D}, and D_{1D} requires T_{1D}. This poses a distinct problem to performing these calculations. However, by choosing a trial value for T_{1D} and D_{1D}, the calculations are able to be carried to completion. At the end, the trial values can be confirmed to be within 5% error, representing a reasonable estimate of behavior. This was accomplished in a spreadsheet, which can be found in Appendix F.

The code equations eventually produce a required damping force in each level (F) and a velocity in the direction under consideration (v). The required force was then divided by the number of dampers at the given level. In the chosen configuration, each braced frame contains two dampers. Frames that were not directly in the x- or y-direction were considered to act only in their primary direction (for instance, the dampers in Bay E were only included in the x-direction). These were then resolved onto their primary axis using trigonometry to produce an effective number of dampers in the direction under consideration. The full force required per story was then divided by the effective number of dampers to find an effective force per damper. Finally, trigonometry was applied again to find the in-plane required force per damper. A sample of this calculation is shown in Table 20, and all calculations are in Appendix F.

Once the required forces per damper were found, Taylor Devices product information was used to choose damper sizes which could achieve the required forces. Taylor Devices was chosen as the supplier because they are one of the most well-respected manufacturers of VFD's in the country, and their product information was readily available and easy to understand. An elevation of Frame 7 displaying the required forces per damper for the trial damping and the corresponding damper sizes can be seen in Figure 36.

CA S-3 with VFD - X-Direction - Damper Forces, 2nd Level						
Total Required Damping Force per Story = ΣF_2= 4,821.42 k						
Frame #	Bay	# of dampers (n_i)	θ_i (deg)	N= $n_i\cos(\theta_i)$	F_{pseduo} (k) = $\Sigma F_2/\Sigma N$	F_i (k) = $F_{pseudo}/\cos(\theta_i)$
1	D to E	2	0	2.000	453.16	453.16
1	F to G	2	0	2.000		453.16
5	11 to 12	2	-15	1.932		469.14
10	9 to 10	2	-45	1.414		640.86
11	M to N	2	45	1.414		640.86
15	E1 to F1	2	-20	1.879		482.24

Table 20 Sample calculation of the required force per damper for the 2nd Level in the X-Direction.

CA S-3 with VFD - Frame 7 - Actual Force Required and Damper Size

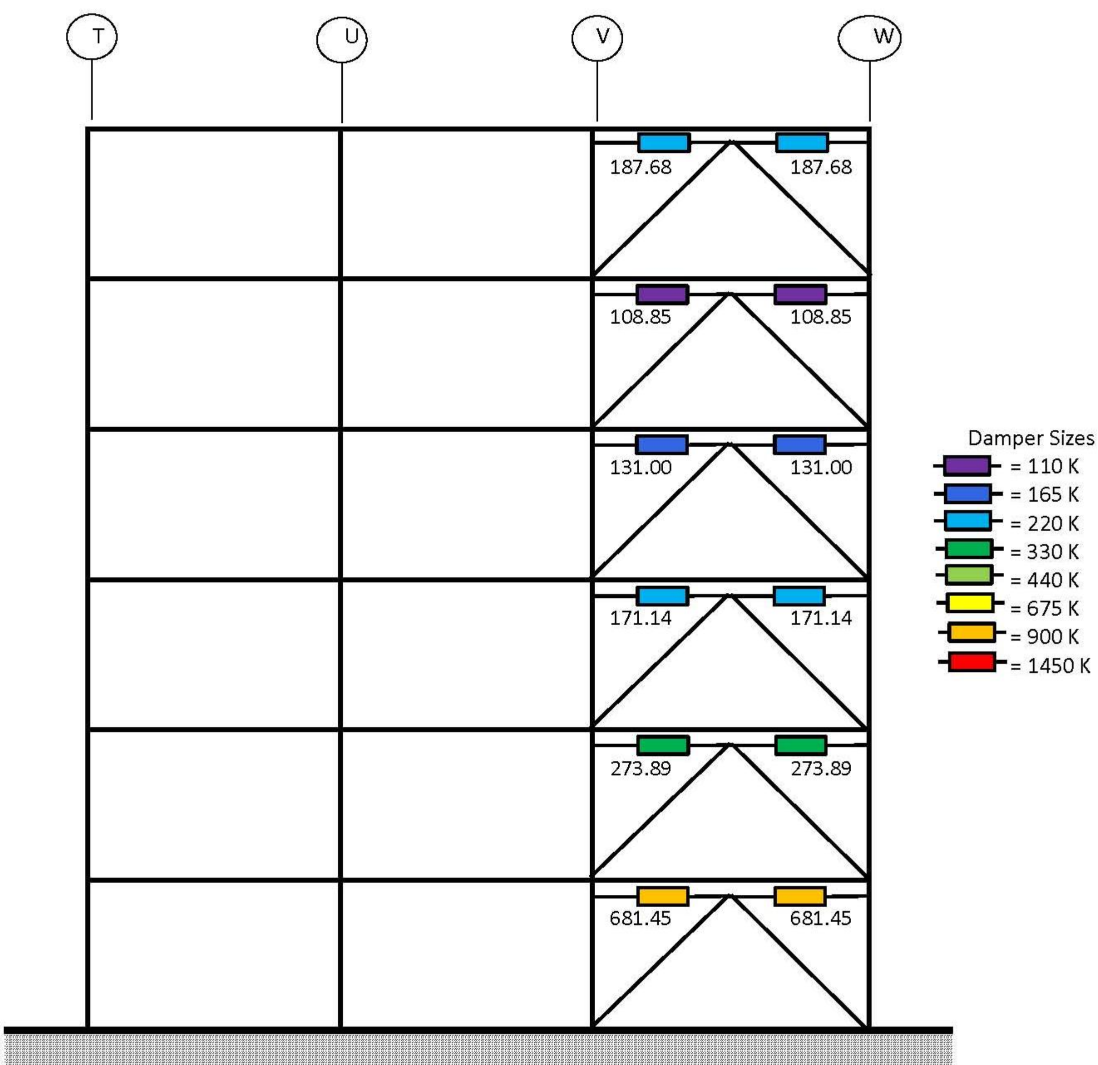


Figure 36 Elevation of Frame 7 showing required trial damper forces and the damper sizes provided.

The final step required for preliminary sizing was to calculate the required damping coefficient for the dampers. This is dictated by the equation:

$$F = Cv^{\alpha}$$

In this equation, F is the damper force, C is the damping coefficient, v is the velocity in the direction under consideration, and α is referred to as the velocity exponent. The velocity exponent is a property of the damper which is a function of how linearly it behaves. Although dampers can theoretically be designed for any value of this, it is most traditionally taken as anywhere from 0.3 to 1.0 (1.0 indicates perfectly linear behavior). Therefore, for this report, it was taken as 0.6. Knowing the force per damper and the velocity (found from the code equations) and with the assumed velocity exponent, it was possible to calculate the damping coefficient, which is also a property which the damper must be designed to achieve. A sample calculation for this process can be seen in Table 21, where the damping coefficients for Frame 7 are summarized.

CA S-3 with VFD - Frame 7 - Required Damper Properties				
Level	Size (k)	Velocity (in/s)	α	C_{req}
AHU Roof	N/A	11.114	0.6	N/A
Chiller Roof	220	11.114	0.6	51.87
Atrium Roof	N/A	11.114	0.6	N/A
Penthouse	110	11.114	0.6	25.93
5th	165	11.114	0.6	38.90
4th	220	11.114	0.6	51.87
3rd	330	11.114	0.6	77.80
2nd	900	11.114	0.6	212.18

Table 21 Calculation of damping coefficients for the dampers in Frame 7.

Earthquake Ground Motion History Record Selection and Scaling

In order to perform the nonlinear analysis to confirm the preliminary design, earthquake ground motion history records had to be selected and scaled. Although the code permits the use of as few as three records, if less than seven are used, the maximum envelope of the histories must be used. However, if seven or more records are used, the average of the records can be used, which was preferable. Due to the irregularities of this structure, motions had to be applied to multiple directions simultaneously. Therefore, a total of 14 acceleration records were selected. In order to simplify the selection of the records, recommended records from FEMA P695 were chosen. Due to the proximity of the structure to the Northridge fault line, exclusively near-field records were chosen. The ground acceleration histories for these records were then retrieved from the PEER NGA website, which is a database for ground motion records. The records for each direction were graphed, and these plots can be found in Appendix G.

Also available on the PEER NGA website are response spectra for each ground motion history. These were averaged and compared to the code-required design response spectrum. Then, the average was scaled in a two-step process. In the first step, the records are scaled to a completely arbitrary peak ground velocity (PGV) value in order to eliminate the PGV variable. This was done by selecting a trial PGV (50 in/sec for this report) and then multiplying the spectral acceleration values for each record by the ratio of the trial PGV to the PGV of the record.

The average value response spectra for the ground motion records at the calculated building period multiplied by Cu was then compared to the code response spectrum value at the calculated building period multiplied by Cu. The ratio of Code-Required Sa/Average Sa was then multiplied by the arbitrary trial PGV to produce a refined trial PGV. Then, the ratio of the second trial PGV to the PGV of the record gives the scale factor for the second step. When the second scale factor is multiplied by the spectral acceleration values of each record, it produces a correctly scaled average response spectrum. A sample result plot can be seen in Figure 37. The remaining plot and the scale factors used for each record can be found in Appendix G.

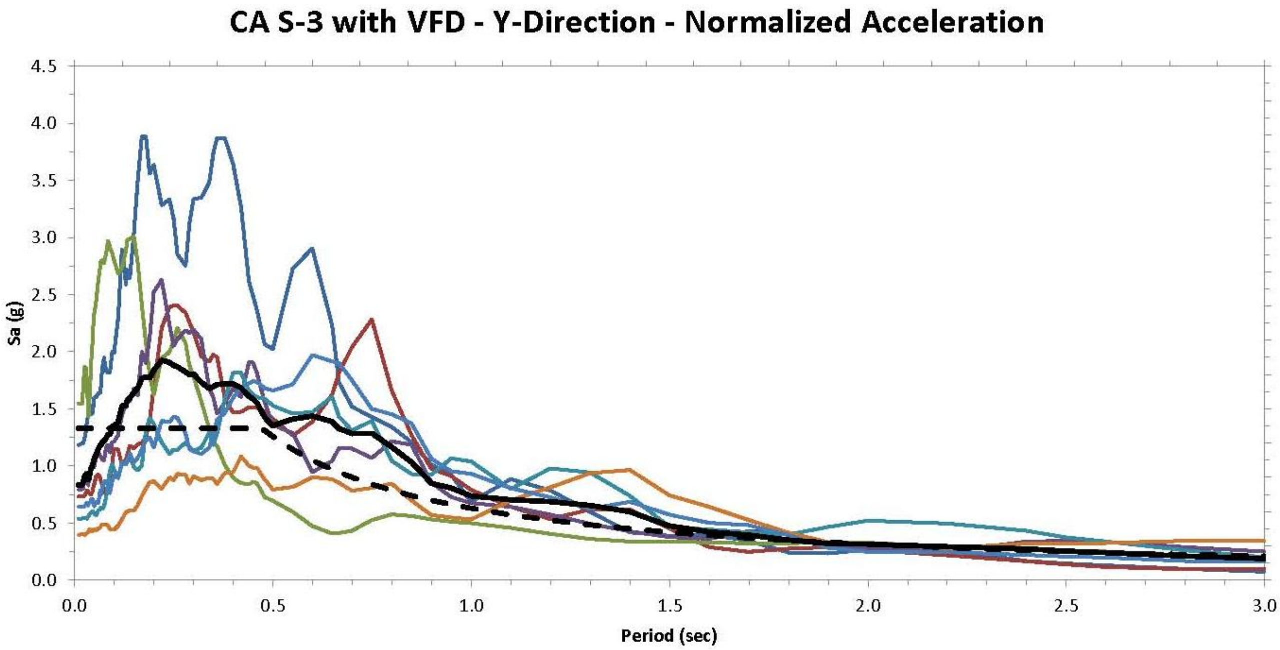


Figure 37 Plot of scaled response spectra for the earthquake records in the Y-Direction. The solid black line is the average response spectrum, the dashed black line is the code response spectrum, and the colored lines are the response spectra for the earthquake records.

Nonlinear Analysis

The nonlinear analysis was conducted in SAP due to the ease with which the damper frames could be modeled in this program. The ETABS model of the CA S-3 structure was exported to SAP, and run to verify the structure had exported correctly. Modal properties were identical between the two programs, and therefore it was determined that the SAP model was a reasonable representation of the structure as it was designed for the CA S-3 criteria.

The next step was to verify the earthquake scaling properly reflected code-level loads. This was achieved by entering the earthquake acceleration records into the SAP model as time history functions. These functions were then assigned to linear time history load cases, since this structure was considered to behave linearly. The time history functions were applied as accelerations in the U1 (x) and U2 (y) directions as applicable. The scale factors were found by multiplying the factor determined in the second step of the scaling by the acceleration of gravity (in this case, 386.4 k-s^2/in). They had to be multiplied by gravity because the acceleration records from PEER NGA are all normalized, meaning they are given as a fraction of g. Finally, the load cases in the x- and y- directions were paired (X1 with Y1 and so on) to create 7 load combinations.

Monitor columns were added to the model to easily find the drifts in each level, as SAP will not calculate this automatically (unlike ETABS). This is accomplished by setting the moment of inertia of a column placed at the four outermost corners of the structure equal to

$$I = \frac{L^2}{12E}$$

In this equation, L is the height of the column and E is the modulus of elasticity. When the model is run, the maximum shear in these columns is the maximum drift experienced in a given level. These values can easily be collected from the output of the model and then averaged to find the drift in each level due to the applied earthquake acceleration histories.

When this model was initially run, it was found that the maximum average drift for any level was only approximately 0.3%. This indicated that the earthquake records chosen for the initial scaling were not strong enough to mimic the Maximum Credible Earthquake (MCE) from the code. Therefore, all of the scale factors in the load cases were multiplied by 5 to produce a larger response from the structure. This yielded drifts that were much more reasonable in comparison to code (which are summarized in Tables 22 and 23), and therefore the final scaling was determined to be "Step 2 Factor x 386.4 x 5".

CA S-3 Linear Time History - X-Direction - Drift/Torsion Check for Average of 7 Earthquake Ground Motions										
Level	$\Delta_{x,max,avg}$ (in)	$\delta_{1,avg}$ (in)	OK?	$\Delta_{x,min,avg}$ (in)	$\delta_{2,avg}$ (in)	OK?	δ_{avg} (in)	$\delta_{max}/\delta_{avg}$	Irreg.	A_x
AHU Roof	7.970063	0.00975	O.K	6.896298	0.01198	O.K	0.01086	1.103	N/A	N/A
Chiller Roof	6.975758	0.00917	O.K	5.674243	0.00044	O.K	0.00480	1.909	1b	2.529963
Atrium Roof	6.223826	0.01410	O.K	5.638277	0.01317	O.K	0.01364	1.034	N/A	N/A
Penthouse	5.010866	0.00407	O.K	4.505253	0.00375	O.K	0.00391	1.041	N/A	N/A
5th	4.299049	0.00454	O.K	3.849111	0.00317	O.K	0.00385	1.179	N/A	N/A
4th	3.535661	0.00568	O.K	3.317314	0.00523	O.K	0.00545	1.042	N/A	N/A
3rd	2.581686	0.00769	O.K	2.439453	0.00713	O.K	0.00741	1.038	N/A	N/A
2nd	1.289369	0.00708	O.K	1.241285	0.00682	O.K	0.00695	1.019	N/A	N/A

Table 22 Summary of drifts in the X-Direction for the CA S-3 structure due to the linear application of earthauake histories.

CA S-3 Linear Time History - Y-Direction - Drift/Torsion Check for Average of 7 Earthquake Ground Motions										
Level	$\Delta_{x,max,avg}$ (in)	$\delta_{1,avg}$ (in)	OK?	$\Delta_{x,min,avg}$ (in)	$\delta_{2,avg}$ (in)	OK?	δ_{avg} (in)	$\delta_{max}/\delta_{avg}$	Irreg.	A_x
AHU Roof	7.147111	0.00694	O.K	6.656074	0.01289	O.K	0.00991	1.300	1a	1.173494
Chiller Roof	6.439212	0.01488	O.K	5.341592	0.00938	O.K	0.01213	1.227	1a	1.044878
Atrium Roof	5.219309	0.00842	O.K	4.572465	0.00357	O.K	0.00600	1.405	1b	1.3707
Penthouse	4.494925	0.00499	O.K	4.265641	0.00464	O.K	0.00481	1.036	N/A	N/A
5th	3.622267	0.00301	O.K	3.453918	0.00395	O.K	0.00348	1.135	N/A	N/A
4th	3.116252	0.00452	O.K	2.790067	0.00469	O.K	0.00461	1.018	N/A	N/A
3rd	2.356431	0.00612	O.K	2.002502	0.00480	O.K	0.00546	1.120	N/A	N/A
2nd	1.328668	0.00730	O.K	1.195751	0.00657	O.K	0.00694	1.053	N/A	N/A

Table 23 Summary of drifts in the Y-Direction for the CA S-3 structure due to the linear application of earthauake histories.

The damper frames were then modeled in SAP. Both SAP and ETABS have a very unique section property known as the "link" property. This enables the user to define elements with very specific properties. One of the link types available is a damper. When this is selected, the user can define the link properties such as mass, weight, stiffness, linearity, and damping coefficient of the damper. All dampers were modeled as nonlinear elements in the U1 (axial) direction, with a velocity exponent of 0.6 and damping coefficients as found in the preliminary design.

Once the dampers were modeled, the earthquake ground acceleration histories were entered into this model with the final scale factor (including the arbitrary coefficient of 5). These were entered with an identical procedure to the histories in the CA S-3 model, except that the load cases were nonlinear time history cases in order to incorporate the nonlinear properties of the dampers.

The final change that had to be made was to change the modal case to Ritz vectors instead of Eigenvectors. Although Eigenvector analysis is acceptable for traditional modeling, the Eigenvectors will not accurately account for the nonlinear properties of the dampers. The loads applied to find the Ritz vector modal case were the accelerations in both the U1 and U2 directions and the link properties.

With all the load cases properly established, the model was run. It was found that the average drifts at any level did not exceed approximately 0.1%. Therefore, the structure was deemed to be significantly overdesigned. At this point, trial-and-error was used to reduce the damping coefficients until the drifts were found to be much closer to the determined allowable limit of 0.7%. This occurred when the damping coefficients had been reduced by a factor of 10. The drifts resulting from the finalized design are shown below in Tables 24 and 25 as well as in the “System Comparison/Summary” section, under the “System Drifts Summary” subsection.

Also, the structure was found to have an average velocity of 31.37 in/s. Since both the damping coefficients and the velocity had changed significantly, it was decided that the dampers had to be resized. This was accomplished using the same equation as was originally used to find the damping coefficients, but this time by solving for the required force. A sample frame elevation for this process can be seen in Figure 38, and all frame elevations for the final required damper sizes can be found in Appendix F.

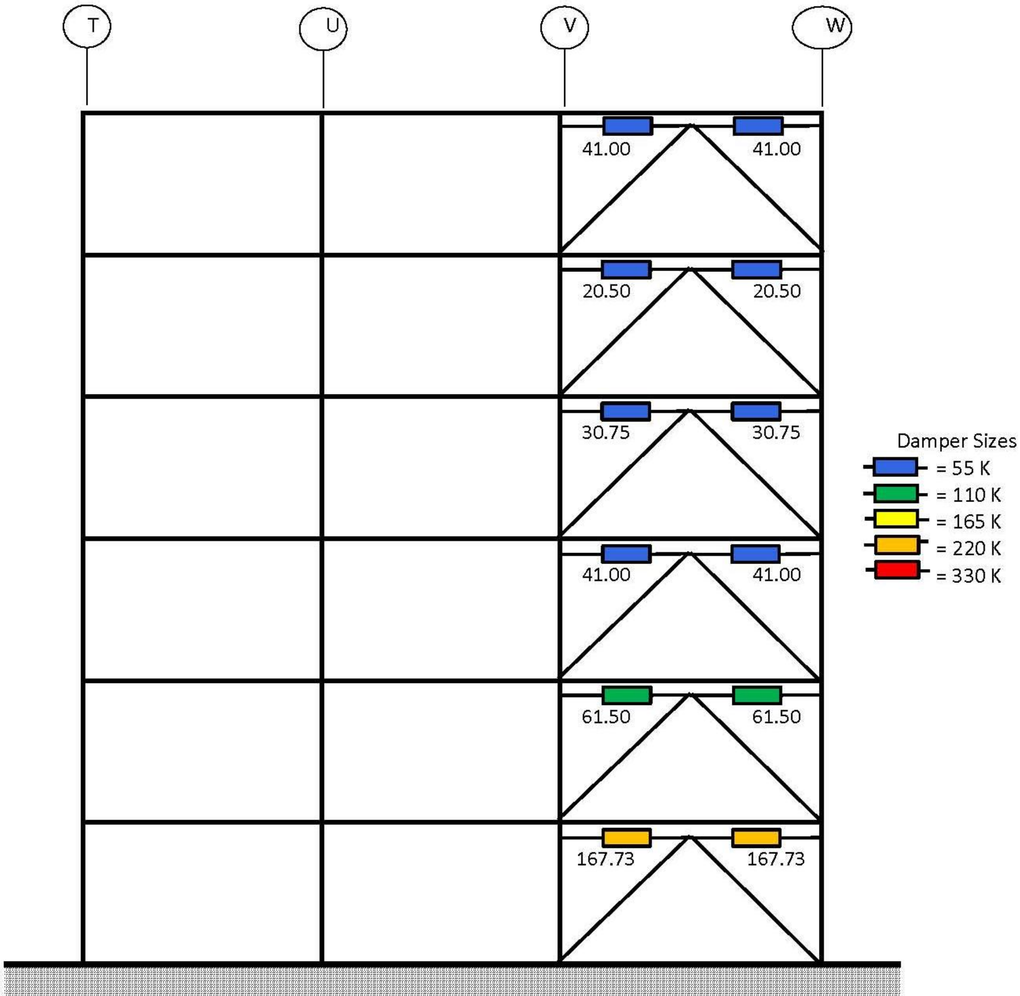
CA S-3 with VFD - X-Direction - Drift/Torsion Check for Average of 7 Earthquake Ground Motions										
Level	$\Delta_{x,max,avg}$ (in)	$\delta_{1,avg}$ (in)	OK?	$\Delta_{x,min,avg}$ (in)	$\delta_{2,avg}$ (in)	OK?	δ_{avg} (in)	$\delta_{max}/\delta_{avg}$	Irreg.	A_x
AHU Roof	5.142017	0.00182	O.K.	4.196011	0.00235	O.K.	0.00208	1.128	N/A	N/A
Chiller Roof	4.956646	0.00168	O.K.	3.956453	0.00084	O.K.	0.00126	1.335	1a	1.238142
Atrium Roof	4.819028	0.00306	O.K.	3.887943	0.00280	O.K.	0.00293	1.045	N/A	N/A
Penthouse	4.555671	0.00312	O.K.	3.647191	0.00316	O.K.	0.00314	1.006	N/A	N/A
5th	4.010369	0.00494	O.K.	3.094845	0.00419	O.K.	0.00456	1.083	N/A	N/A
4th	3.180078	0.00638	O.K.	2.39162	0.00448	O.K.	0.00543	1.175	N/A	N/A
3rd	2.108081	0.00524	O.K.	1.638506	0.00473	O.K.	0.00498	1.052	N/A	N/A
2nd	1.227945	0.00675	O.K.	0.844687	0.00464	O.K.	0.00569	1.185	N/A	N/A

Table 24 Deflections and drifts for the X-Direction for the CA S-3 with VFD design.

CA S-3 with VFD - Y-Direction - Drift/Torsion Check for Average of 7 Earthquake Ground Motions										
Level	$\Delta_{y,max,avg}$ (in)	$\delta_{1,avg}$ (in)	OK?	$\Delta_{y,min,avg}$ (in)	$\delta_{2,avg}$ (in)	OK?	δ_{avg} (in)	$\delta_{max}/\delta_{avg}$	Irreg.	A_x
AHU Roof	5.102398	0.00032	O.K.	5.014548	0.00206	O.K.	0.00119	1.729	1b	2.075431
Chiller Roof	5.069374	0.00498	O.K.	4.804066	0.00408	O.K.	0.00453	1.099	N/A	N/A
Atrium Roof	4.661154	0.00236	O.K.	4.469392	0.00270	O.K.	0.00253	1.067	N/A	N/A
Penthouse	4.458517	0.00348	O.K.	4.237551	0.00323	O.K.	0.00336	1.037	N/A	N/A
5th	3.849765	0.00481	O.K.	3.671835	0.00446	O.K.	0.00463	1.038	N/A	N/A
4th	3.042047	0.00633	O.K.	2.922983	0.00655	O.K.	0.00644	1.017	N/A	N/A
3rd	1.978374	0.00596	O.K.	1.823197	0.00523	O.K.	0.00559	1.066	N/A	N/A
2nd	0.97684	0.00537	O.K.	0.945156	0.00519	O.K.	0.00528	1.016	N/A	N/A

Table 25 Deflections and drifts for the Y-Direction for the CA S-3 with VFD design.

CA S-3 with VFD - Frame 7 - Actual Force Required and Damper Size



CA S-3 with VFD - Frame 7 - Required Damper Properties					
Level	C	Velocity (in/s)	α	F_{req}	Size (k)
AHU Roof	N/A	31.370	0.6	N/A	N/A
Chiller Roof	5.19	31.370	0.6	41.00	55
Atrium Roof	N/A	31.370	0.6	N/A	N/A
Penthouse	2.59	31.370	0.6	20.50	55
5th	3.89	31.370	0.6	30.75	55
4th	5.19	31.370	0.6	41.00	55
3rd	7.78	31.370	0.6	61.50	110
2nd	21.22	31.370	0.6	167.73	220

Figure 38 Sample frame elevation showing final required damper sizes.

System Finalization

Once the drifts were found to be adequate for the structure, hand calculations were performed to size the braces as well as design a sample connection for the supports to a damper, the central gusset plate to support the braces, and finally the moment connection of the damper support to the column. The dampers in Bay A were chosen for the sample calculations, which can be found in Appendix F. These designs drew on limit states learned in AE 534. They can be seen in Figures 39, 40, and 41, respectively.

Lastly, the weight of the frame including the bracing steel was calculated to be 12,500 k. This is a 1.5% increase over the CA S-3 design, which is sufficiently negligible. It is also a savings of 8% in steel weight over the CA S-1 structure. This can also be seen in the "System Comparison/Summary" section, under the "System Weights Summary" subsection.

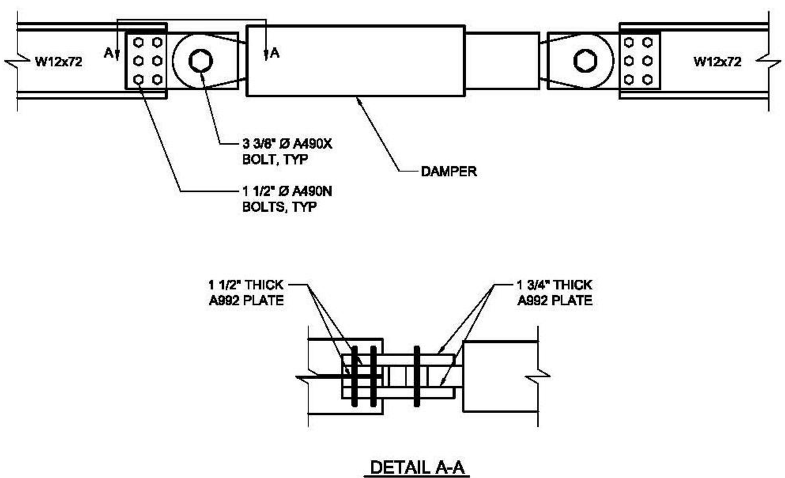


Figure 39 Damper support connection detail. See Appendix F for hand calculations used to design this connection.

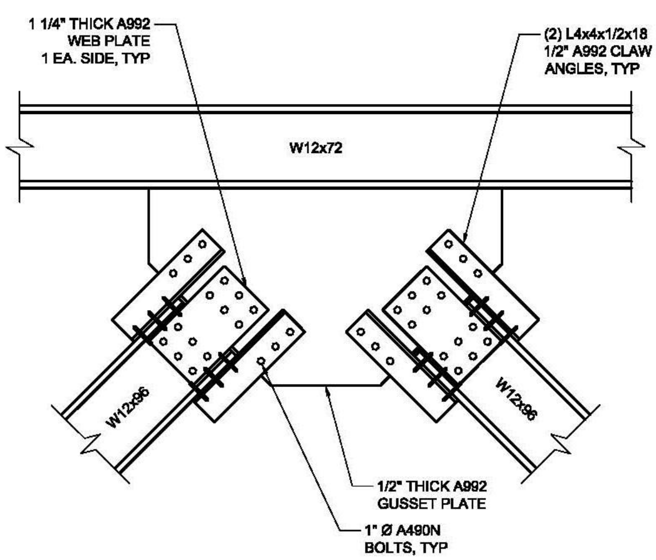


Figure 40 Central gusset connection detail. See Appendix F for hand calculations used to design this connection.

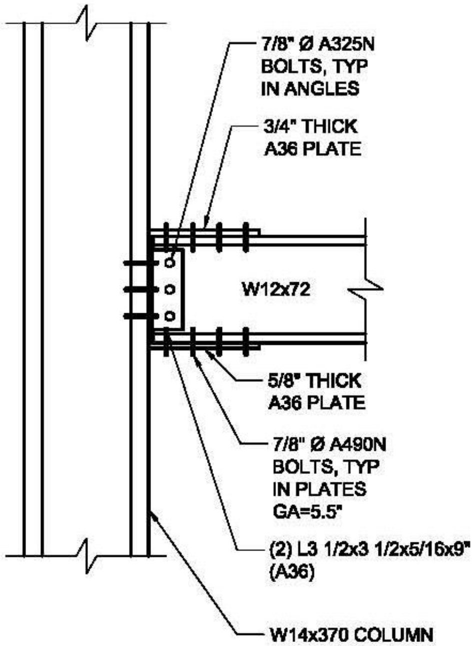


Figure 41 Damper support-to-column moment connection detail. See Appendix F for hand calculations used to design this connection.

System Comparison/Summary

This section seeks to provide a concise summary of the results of the designs which are important to comparing the overall efficiency of the structures. Final efficiency determinations are made in the "Construction Management Breadth: Cost and Schedule Analysis" section.

System Weights Summary

The weights for the steel and the deck/slab/superimposed dead loads are summarized in Table 26 below. As can be seen, the CA S-1 structure is the heaviest of the redesigns, with the CA S-3 and CA S-3 with VFD structures being approximately equal weights in the middle of the range, and the NE USA S-3 system as the lightest. This result is exactly as was expected for these designs.

Building Weights Summary										
Description		Story								
		AHU Roof	Chiller Roof	Atrium Roof	Penthouse	5th	4th	3rd	2nd	**Totals**
Deck/Slab/SDL Weight (All Designs)	Area	10,494.10	2,299.50	8,785.20	13,087.90	21,095.70	20,101.50	20,101.50	20,101.50	
	Load (psf)	73.00	133.00	55.76	158.47	108.58	78.00	78.00	78.00	
	Weight (k)	766.07	305.83	489.86	2,074.04	2,290.57	1,567.92	1,567.92	1,567.92	**10,630.13**
NE USA S-3	Beam Weight (k)	35.50	13.21	36.65	104.01	144.41	122.29	129.46	129.46	**714.98**
	Column Weight (k)	17.53	18.29	26.14	56.36	63.33	63.18	93.11	99.64	**437.56**
	Total (Beam+Col+D/S/SDL)	819.10	337.32	552.65	2,234.41	2,498.31	1,753.38	1,790.49	1,797.01	**11,782.67**
CA S-3	Beam Weight (k)	35.51	14.18	36.93	125.71	175.95	155.00	154.48	154.48	**852.23**
	Column Weight (k)	33.35	34.89	50.47	112.59	136.25	135.91	154.90	169.59	**827.96**
	Total (Beam+Col+D/S/SDL)	834.93	354.90	577.27	2,312.34	2,602.76	1,858.82	1,877.29	1,891.99	**12,310.31**
CA S-1	Beam Weight (k)	54.20	23.03	54.52	144.55	217.22	214.36	215.44	215.44	**1,138.75**
	Column Weight (k)	68.23	71.03	114.73	242.27	286.34	286.34	322.03	348.18	**1,739.14**
	Total (Beam+Col+D/S/SDL)	888.49	399.90	659.11	2,460.86	2,794.13	2,068.62	2,105.38	2,131.53	**13,508.02**
CA S-3 with VFD	Beam Weight (k)	35.51	14.18	36.93	125.71	175.95	155.00	154.48	154.48	**852.23**
	Column Weight (k)	33.35	34.89	50.47	112.59	136.25	135.91	154.90	169.59	**827.96**
	Brace Weight (k)	21.42	4.85	14.19	11.48	20.15	21.86	21.58	66.72	**182.24**
	Total (Beam +Col +Brace +D/S/SDL)	856.35	359.75	591.46	2,323.82	2,622.92	1,880.68	1,898.88	1,958.70	**12,492.55**

Table 26 Summary of the weights of each redesign system.

System Drifts Summary

The drifts for all of the designs are plotted in Figure 42 and Figure 43 for the X-Direction and the Y-Direction, respectively. These are compared to the allowable drifts for each design type. As can be seen, all drifts are below the allowable, and by far the most efficient structure in terms of deflection is the CA S-3 with VFD structure. This is the result that was expected.

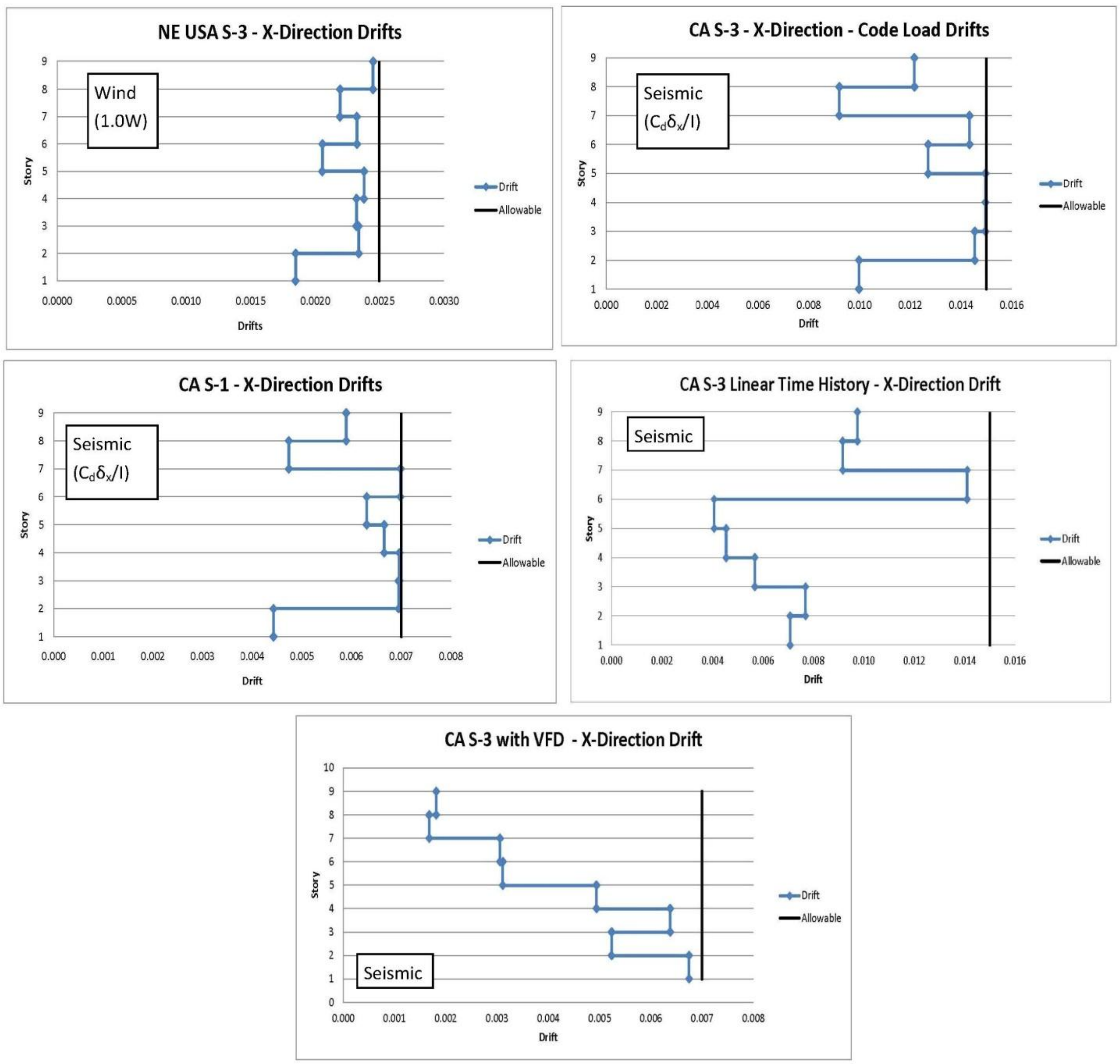


Figure 42 Plots of allowable drifts for each design and the maximum drifts experienced by the redesigned structures in the X-Direction.

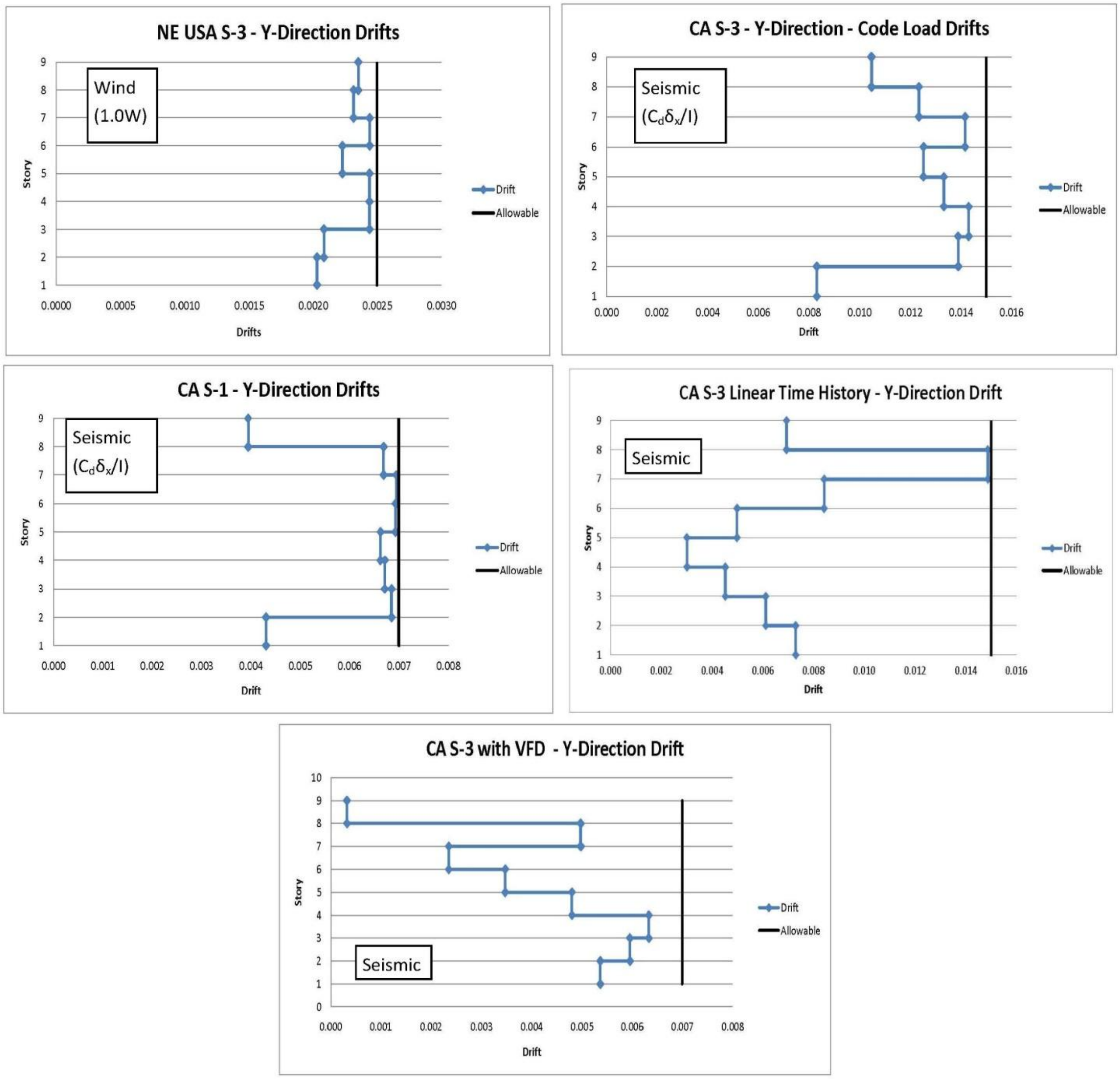


Figure 43 Plots of allowable drifts for each design and the maximum drifts experienced by the redesigned structures in the Y-Direction.

Construction Management Breadth: Cost and Schedule Analysis

The purpose of this breadth was to investigate how the changes to the superstructure will alter the building construction schedule and cost. Certain architectural features were also considered in this breadth, such as:

- The increase in façade due to the additional height of the building associated with the deeper structure (the plenum space cannot be encroached upon because of the ductwork sizes required to achieve the 100% outside air required in laboratories)
- The additional acoustical ceiling tile required in the laboratory spaces (in the original building, the structure was left exposed in these rooms due to the use of precast concrete, which would provide a smooth and aesthetically pleasing finish)
- Column covers (typically fiberglass tubes which enclose steel columns in order to hide the fireproofing) in order to preserve the aesthetics of the building (in the original design, the columns were left exposed)

Schedules

The existing schedule was provided by Turner Construction and was reduced to a simplistic 30-line schedule for ease of manipulation. This was then entered into Microsoft Project. The resulting original schedule can be found in Appendix H. It has a duration of 22 months for the main construction phase.

Quantities for the new designs were taken-off in a variety of ways. For the structures designed in RAM, the program has a reporting feature which produces take-offs for the steel frames and shear studs. One level was verified by hand, and upon finding the RAM report matched the hand take-off, the RAM reports were used for all further take-offs. Floor areas were found using an AutoCAD drawing of the structure which was produced by the author using the dimensions of the floor plans. This constituted the take-off for the decks and concrete as well as assisted with the take-offs for the shrinkage and temperature reinforcing which was integrated into the slabs. The viscous fluid dampers and their braces were taken-off by hand. Spray fireproofing, welding, acoustical ceiling, column covers, and additional façade were estimated using hand calculations.

Once these quantities were found, RS Means 2011 was used to find the production rate of these items. Then, durations for each item were calculated. Finally, the structure was sequenced in groups of two floors. This was decided by the column splices which occur every two levels. Also, this is a practical design decision because it allows decking crews to place deck on the third floor and then deck the second floor while the steel crew erects the 4^{th} and 5^{th} levels (and so on). It condenses the schedule considerably. A sample of these duration calculations can be seen in Table 27, and the similar tables for the other designs can be found in Appendix H.

From the durations, schedules were produced for each design (which can be found in Appendix H). It was found that both of the code minimum structures could be erected in 24 months, and both of the Immediate Occupancy structures could be erected in 25 months. It is possible that this increase in duration would be unacceptable to the owner, since they are on such a tight timetable to get the structure available for use for the fall semester.

NE USA S-3 Quantity, Duration, and Cost Summary									
Description	RS Means Unit Cost/Duration	Quantities/Duration/Cost per Level							
		2nd Level	3rd Level	4th Level	5th Level	Penthouse Level	Atrium Roof Level	Chiller Roof Level	AHU Roof Level
Gravity Beams									
Quantities (LF)		2,489	2,489	2,489	2,804	1,986	1,127	262	928
Duration (days)	900 LF/day	2.77	2.77	2.77	3.12	2.21	1.25	0.29	1.03
Cost	60 $/LF	$149,353.20	$149,353.20	$149,353.20	$168,257.40	$119,134.80	$67,620.00	$15,720.00	$55,686.00
Gravity Columns									
Quantities (LF)		273	252	252	252	233.28	116	87.75	110.5
Duration (days)	900 LF/day	0.30	0.28	0.28	0.28	0.26	0.13	0.10	0.12
Cost	100 $/LF	$27,306.00	$25,200.00	$25,200.00	$25,200.00	$23,328.00	$11,600.00	$8,775.00	$11,050.00
Lateral Beams									
Quantities (LF)		904	904	904	904	422	307	136	259
Duration (days)	800 LF/day	1.13	1.13	1.13	1.13	0.53	0.38	0.17	0.32
Cost	125 $/LF	$112,962.50	$112,962.50	$112,962.50	$112,962.50	$52,750.00	$38,412.50	$17,000.00	$32,375.00
Lateral Columns									
Quantities (LF)		622	574	574	574	516	244	130	111
Duration (days)	800 LF/day	0.78	0.72	0.72	0.72	0.64	0.30	0.16	0.14
Cost	200 $/LF	$124,400.00	$114,800.00	$114,800.00	$114,800.00	$103,140.00	$48,740.00	$25,960.00	$22,100.00
Welding									
Quantities (LF)		225	225	225	225	99	126	27	72
Duration (days)	30 LF/day	7.50	7.50	7.50	7.50	3.30	4.20	0.90	2.40
Cost	29 $/LF	$6,525.00	$6,525.00	$6,525.00	$6,525.00	$2,871.00	$3,654.00	$783.00	$2,088.00
Spray Fireproofing									
Quantities (SF)		36,183	36,183	36,183	37,972	23,558	15,813	4,139	18,889
Duration (days)	1250 SF/day	28.95	28.95	28.95	30.38	18.85	12.65	3.31	15.11
Cost	1.90 $/SF	$68,747.13	$68,747.13	$68,747.13	$72,147.29	$44,760.62	$30,045.38	$7,864.29	$35,889.82
Shear Studs									
Quantities (#)		1,660	1,660	1,673	2,139	1,513	0	149	0
Duration (days)	900 #/day	1.84	1.84	1.86	2.38	1.68	0.00	0.17	0.00
Cost	2.80 $/stud	$4,648.00	$4,648.00	$4,684.40	$5,989.20	$4,236.40	$0.00	$417.20	$0.00
Composite Deck									
Quantities (SF)		20,102	20,102	20,102	21,096	13,088	0	2,300	0
Duration (days)	2850 SF/day	7.05	7.05	7.05	7.40	4.59	0.00	0.81	0.00
Cost	3.21 $/SF	$64,525.82	$64,525.82	$64,525.82	$67,717.20	$42,012.16	$0.00	$7,381.40	$0.00
Roof Deck									
Quantities (SF)		0	0	0	0	0	8,785	0	10,494
Duration (days)	3600 SF/day	0.00	0.00	0.00	0.00	0.00	2.44	0.00	2.92
Cost	3.11 $/SF	$0.00	$0.00	$0.00	$0.00	$0.00	$27,321.97	$0.00	$32,636.65
Rebar									
Quantities (tons)		8.5	8.5	8.5	9.0	7.4	0.0	1.5	0.0
Duration (days)	2.9 tons/day	2.93	2.93	2.93	3.10	2.55	0.00	0.52	0.00
Cost	1900 $/tons	$16,150.00	$16,150.00	$16,150.00	$17,100.00	$14,060.00	$0.00	$2,850.00	$0.00
Slab									
Quantities (CY)		309.56	309.56	309.56	324.87	201.55	0.00	35.41	0.00
Duration (days)	110 CY/day	2.81	2.81	2.81	2.95	1.83	0.00	0.32	0.00
Cost	163 $/CY	$50,458.79	$50,458.79	$50,458.79	$52,954.43	$32,853.25	$0.00	$5,772.20	$0.00
ACT Ceiling									
Quantities (SF)		11,340	11,340	11,340	6,286	0	0	0	0
Duration (days)	380 SF/day	29.84	29.84	29.84	16.54	0.00	0.00	0.00	0.00
Cost	2.75 $/SF	$31,185.00	$31,185.00	$31,185.00	$17,286.50	$0.00	$0.00	$0.00	$0.00
Additional Façade									
Quantities (SF)		666.67	666.67	666.67	666.67	350.00	583.33	250.00	300.00
Duration (days)	300 SF/day	2.22	2.22	2.22	2.22	1.17	1.94	0.83	1.00
Cost	10 $/SF	$6,666.67	$6,666.67	$6,666.67	$6,666.67	$3,500.00	$5,833.33	$2,500.00	$3,000.00
Column Covers									
Quantities (LF)		895	895.06	826	826	826	748.98	359.7	217.55
Duration (days)	600 LF/day	1.49	1.49	1.38	1.38	1.38	1.25	0.60	0.36
Cost	60 $/LF	$53,703.60	$53,703.60	$49,560.00	$49,560.00	$49,560.00	$44,938.80	$21,582.00	$13,053.00
Steel Totals									
Quantities (LF)		4,288	4,219	4,219	4,534	3,157	1,794	616	1,408
Duration (days)		4.98	4.89	4.89	5.24	3.64	2.07	0.72	1.62
Cost		$414,021.70	$402,315.70	$402,315.70	$421,219.90	$298,352.80	$166,372.50	$67,455.00	$121,211.00
Deck/Slab Totals									
Quantities (SF)		20,102	20,102	20,102	21,096	13,088	8,785	2,300	10,494
Duration (days)		12.80	12.80	12.80	13.46	8.98	2.44	1.65	2.92
Cost		$131,134.60	$131,134.60	$131,134.60	$137,771.62	$88,925.41	$27,321.97	$16,003.60	$32,636.65
Misc. Structural Totals (Shear Studs , Welding & Fireproofing)									
Duration (days)		38.29	38.29	38.31	40.25	23.83	16.85	4.38	17.51
Cost		$79,920.13	$79,920.13	$79,956.53	$84,661.49	$51,868.02	$33,699.38	$9,064.49	$37,977.82
Misc. Architectural Totals									
Duration (days)		33.56	33.56	33.44	20.14	2.54	3.19	1.43	1.36
Cost		$91,555.27	$91,555.27	$87,411.67	$73,513.17	$53,060.00	$50,772.13	$24,082.00	$16,053.00

Table 27 Sample detailed estimate for durations and costs of the superstructure and additional architectural features.

Costs

Using the take-offs found for scheduling purposes, RS Means 2011 data could also be used to produce the costs of each structure. First, it was attempted to replicate the original costs of the building. The only information which Turner Construction was able to provide directly is summarized in Table 28.

NE USA Original Structure - Original Cost Summary		
Description	Cost	Cost per SF
Construction Cost	$50,000,000	$362.32
Total Cost	$70,000,000	$507.25
Concrete	$5,800,000	$42.03
Steel	$800,000	$5.80
Mechanical & Plumbing	$14,000,000	$101.45
Electrical	$6,000,000	$43.48

Note: These costs were provided by Turner Construction

Table 28 Cost data provided by Turner Construction.

Therefore, to fully replicate the costs, square foot estimating was used. The RS Means record for a "College Laboratory" was used as the basis for this. The summary of the base square foot estimate can be seen in Table 29.

Square Foot Estimate	
Based on RS Means 2011 M.150 - College, Laboratory	
Description	Value
Base Cost per SF	$195.20
Standard Story Height (ft)	12
Building Story Height (ft)	15.71
Adjustment per 1 LF	$4.40
Adjustment to Base Cost	$16.32
Standard Perimeter (ft)	793
Building Perimeter (ft)	800
Adjustment per 100 LF	$1.00
Adjustment to Base Cost	$0.07
Adjusted Cost per SF	$211.59
Building SF	138,000
Building Cost	$29,198,960.00
Basement Cost per SF	$33.65
Basement SF	4,000
Basement Cost	$134,600.00
Subtotal	$29,333,560.00
Location Multiplier	1.00
Time Multiplier	1.00
Total	$29,333,560.00

Table 29 Base square foot estimate.

This was then broken out using the percentages per each major assembly provided by RS Means. This was done in order to replace specific assemblies, such as the superstructure with the detailed estimates. The breakdown can be found in Table 30.

Square Foot Cost Breakdown			
Assembly	% of Total	Cost per SF	Total Cost
A Substructure	10.2%	$15.44	$2,130,949.30
B Shell			
B10 Super Structure	4.3%	$6.51	$898,341.37
B20 Exterior Enclosure	7.1%	$10.75	$1,483,307.85
B30 Roofing	4.0%	$6.06	$835,666.39
C Interiors	22.2%	$33.61	$4,637,948.48
D Services			
D10 Conveying	1.4%	$2.17	$300,000.00
D20 Plumbing	22.0%	$33.31	$4,596,165.16
D30 HVAC	13.9%	$21.04	$2,903,940.71
D40 Fire Protection	2.2%	$3.33	$459,616.52
D50 Electrical	11.7%	$17.71	$2,444,324.20
E Equipment & Furnishings	1.0%	$1.51	$208,916.60
F Special Construction	0.0%	$0.00	$0.00
Subtotal	100.0%	$151.44	$20,891,659.80
Contractor Fees	25.00%		$6,963,886.60
Construction Cost			$27,855,546.40
Architect Fees	6.00%		$1,778,013.60
Total Cost			$29,633,560.00

Table 30 Square foot cost broken out by assembly.

These costs were then partnered with the detailed cost estimates performed for the superstructure, the additional architectural elements, and the general conditions (a cost per month was estimated using detailed estimating, and can be found in Appendix H) and the original costs for the HVAC, Electrical and Plumbing as given by Turner. The combination of these three different cost-finding procedures led to the final system costs, a sample of which can be seen in Table 31. The remaining estimates can be found in Appendix H.

NE USA S-3 - Final Cost Estimate			
Assembly	% of Total	Cost per SF	Total Cost
A Substructure SF	5.73%	$15.44	$2,130,949.30
B Shell			
B10 Super Structure DET	9.27%	$24.97	$3,446,395.35
B20 Exterior Enclosure SF+DET	4.10%	$11.05	$1,524,807.85
B30 Roofing SF	2.25%	$6.06	$835,666.39
C Interiors SF+DET	13.68%	$36.84	$5,084,450.98
D Services			
D10 Conveying SF	0.81%	$2.17	$300,000.00
D20 Plumbing ORIG	37.66%	$101.45	$14,000,000.00
D30 HVAC ORIG			
D40 Fire Protection SF	1.24%	$3.33	$459,616.52
D50 Electrical ORIG	16.14%	$43.48	$6,000,000.00
E Equipment & Furnishings SF	0.56%	$1.51	$208,916.60
F Special Construction SF	0.00%	$0.00	$0.00
Subtotal	91.44%	$246.31	$33,990,802.98
General Conditions DET	8.56%	$23.05	$3,180,777.39
Subtotal with GC's	100.00%	$269.36	$37,171,580.37
Location Multiplier			1.00
Time Multiplier			1.00
Total Cost			$37,171,581.37

NOTES:

SF - Cost taken from RS Means Square Foot Estimate

DET - Cost taken from RS Means Detailed Estimate

SF+DET - Cost from RS Means Detailed Estimate added to RS Means Square Foot Estimate

ORIG - Cost taken from Original Building Cost Data, as provided by Turner Construction

Table 31 Final cost estimate which combines detailed estimates, square foot estimate, and original cost data for the NE USA S-3 system.

Summary

A summary of the systems can be found in Table 32. This information along with the system weights and drifts was used to make the final comparisons.

CM Breadth Summary		
System	Cost	Schedule Duration (months)
Original	$39.4 million	22
NE USA S-3	$37.2 million	24
CA S-3	$37.8 million	24
CA S-1	$40.1 million	25
CA S-3 with VFD	$38.3 million	25

Table 32 Summary of schedule durations and costs for all designs.

The NE USA S-3 structure and the original structure can be compared to determine which design was more efficient for the original site. It was found that the NE USA structure was 5.5% less expensive and 50% lighter than the original structure. However, due to welding requirements and additional architectural features, it would take an additional two months to construct. Since the owner needs the building open for the start of the school year, this would likely be unacceptable. However, it is also possible items on the critical path of the building could be expedited in order to reduce the schedule, therefore making this design more feasible. Therefore, it seems likely that the driving force behind the selection of the initial structural system was probably for performance requirements (it was mentioned by the construction team that the heavy structure was selected to reduce noise transmission between levels) and to keep structural depth to a minimum.

The next important comparison was to determine how much efficiency was lost in transitioning a code-minimum building from a non-seismic region to a high-seismic region. The CA S-3 structure was only 1.6% more expensive and 4.5% heavier than the NE USA S-3 structure. This indicates that there is very little penalty for constructing the same building in a high-seismic region, which is a somewhat unexpected result.

The third comparison was to see how much cost would be required to go from a code minimum design to an immediate occupancy design using a traditional method. The CA S-1 structure is 6% more expensive and 9.7% heavier than the CA S-3 structure. Alone, this comparison means little.

However, when it is partnered with the comparison between the CA S-3 structure and the CA S-3 with VFD structure, it is possible to see exactly how ineffective it is to achieve a higher performance rating using steel alone. The CA S-3 with VFD structure is only 1.5% more expensive and 1.5% heavier than the CA S-3 structure. Therefore, it accomplishes the same goal as the CA S-1 structure, but saves 4.5% of the cost and 8.2% of the weight. This means that the incorporation of the VFD is an extremely effective way to increase the performance of a building without significant added cost. This is especially true when one considers the amount of money that can be saved due to the fact that minimal or no structural repairs would be required following a major seismic event.

Sustainability Breadth: Sustainability Viability Study

This viability study attempts to address the differences between the various ways of evaluating sustainable technology and determine the viability of incorporating either a photovoltaic system or a green roof system on the USB. Neither technology was included in the original design of the USB, however now that the building is in California, one or both may be deemed viable.

Each system was evaluated based upon four different criteria. The first was a life cycle assessment, which incorporates the cost to produce, install, maintain, and eventually salvage the system over a chosen lifetime. The next was a payback period, which attempts to determine how long (typically in years) it will take for the system cost to be counteracted by how much it saves the owner in comparison to a lesser product (or in comparison to the baseline if the product were not incorporated). A carbon footprint was also determined through research, which attempts to determine how many pounds of equivalent carbon dioxide are released into the atmosphere in the production, installation, and lifetime of the product. This can sometimes be mitigated if the product also sequesters carbon (takes carbon dioxide back out of the environment), and therefore the carbon footprint calculated for this report was the net carbon footprint at the end of one year. Finally, the number of additional LEED points the system could earn the building was determined (the building already aims to achieve LEED Gold under version 2.2 by achieving 43 points). The variety of evaluations sought to provide a full profile of the true sustainability and effectiveness of each system.

Solar Photovoltaic Panels

There are many different kinds of solar cell modules which have a variety of efficiencies. Most traditional photovoltaic (PV) systems are mounted on racks to angle them to catch more sun, and therefore produce more power. However, it has been found that placing highly efficient panels horizontally on a roof does not sufficiently degrade the production capacity of the panels to counteract the ease of placement, maintenance, and the reduction in initial cost due to lack of rack systems. These panels which can be directly adhered to building surfaces are known as building-integrated photovoltaics (BIPV). There are also several different types of electrical connection methods. One of the most popular for commercial applications such as the USB is a grid-tied connection. This is where the PV system feeds energy into the building, but the building is still connected to the electrical grid. A schematic of the components of a simple grid-tied system is shown in Figure 44. Typically, grid-tied systems take advantage of net metering, where the electricity meter runs forward when the building uses power from the power company and backward when it uses electricity from the PV system. The resulting net energy use is the amount the owner is charged for electricity.

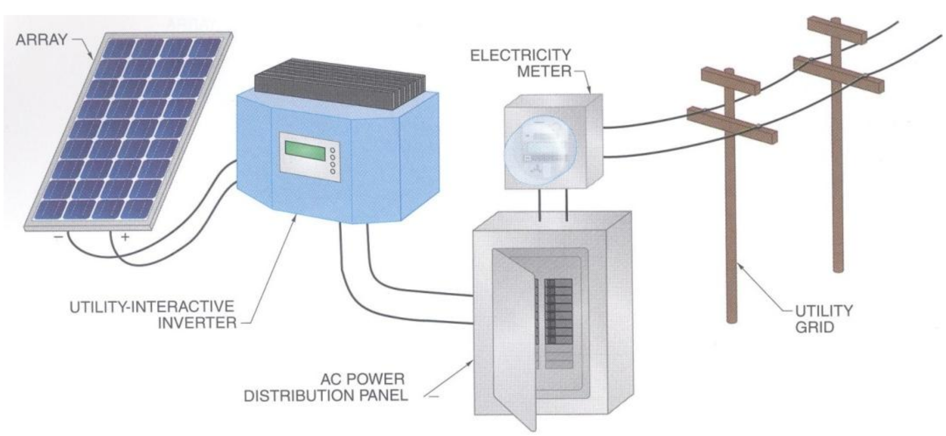


Figure 44 Schematic of a simple grid-tied system, taken from "Photovoltaic Systems" by James P. Dunlop.

For the USB, it was impractical to attempt to design a PV system which would power the entire building due to the electricity consumption requirements of the building. Therefore, it was decided that a solar shading study should be conducted to determine what location or locations would be most effective for the placement of the panels. Then, a layout would be created to accommodate the available space, and the grid-tied system could be designed for the layout.

The panels chosen for this design were Lumeta's PowerPly 400 module, which is a BIPV unit. These were selected for the quality of the panel (the solar cells are monocrystalline silicon, with an efficiency of slightly less than 14%) and the ease of placement (they are applied directly to a roofing membrane using a peel-and-stick adhesive which is certified to resist even corner-zone uplift values). They are rated at 400 W of DC power per panel. However, this value is determined in tests at 25 °C. Typically, panels are significantly hotter, which degrades power performance. At normal operating temperature, the panels produce an estimated 324 W of power. The additional properties of this panel can be found on its data sheet, which is included in Appendix I.

In order to determine where to place the panels, a solar shading study was conducted. This consisted of constructing a model of the building and the surrounding buildings at the CSUN campus site in Google Sketchup. The same program was then used to create images of the building at sunrise, sunset, and 1:00 PM (the two extreme cases of shadows and the shadows at peak production time) for an equinox and the winter and summer solstices .A sample of this can be seen in Figure 45.

Figure 45 Solar shading study from January 1st at 1:00 PM.

Due to the wide variety of roof equipment on all of the higher roofs and the shading problems they cause, it was determined that the main portion of the office roof was the best place for the PV system. This has about 2800 square feet available. The panels were laid out in this space to avoid exceeding the maximum voltage per string (600 V) and for ease of cabling the series connections, which are on the sides of the panels for this type of this panel. The final layout of panels can be seen in Figure 46, and an image of what a set of panels might look like once fully installed is included as Figure 47.

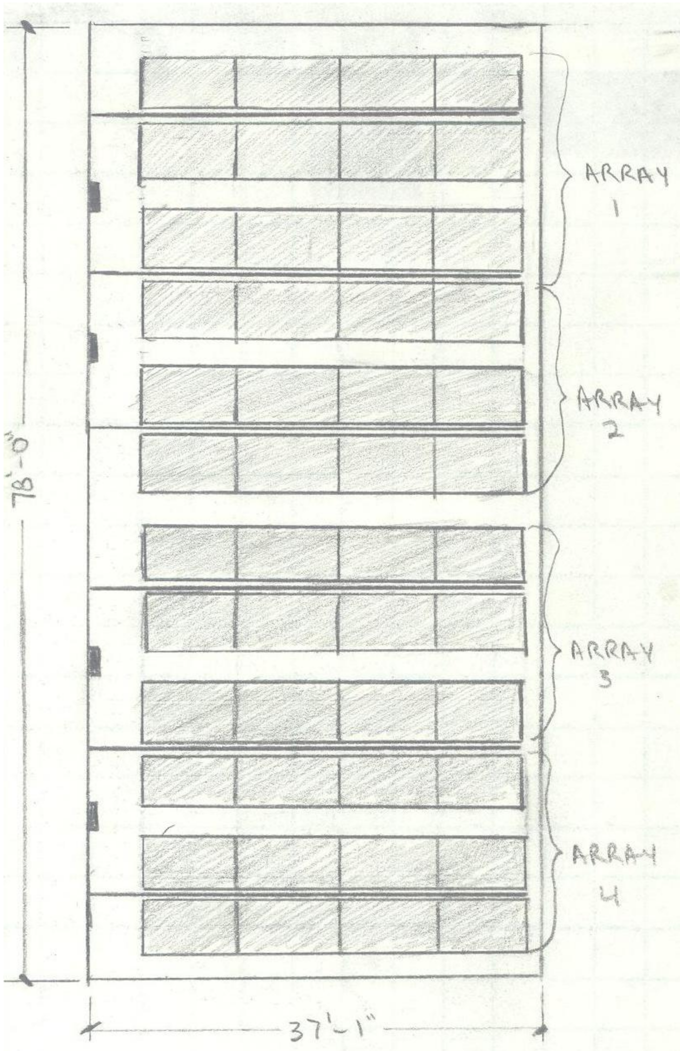


Figure 46 Final layout of solar panels. Lightly shaded rectangles are solar panels, single solid lines are conduits, and darkly shaded boxes are inverters.

Final calculations were performed to size the inverters, wires, determine how many kilowatt-hours (kWh) the system will produce, and how much the system will cost. These calculations can be found in Appendix I. The inverter is a device which converts DC power to the more typical AC power, and the model chosen was SMA's Sunny Boy 5000-US. This is a grid-tied inverter with a built-in DC disconnect switch and combiner. The DC disconnect is used to stop current from the PV panels into the inverter, and the combiner is used to connect several strings of panels in parallel. The data sheet for the inverter can be found in Appendix I.

Figure 47 Image from Lumeta's website showing how an installed system would look.

Life Cycle Assessment

As previously mentioned, the life cycle assessment of a product incorporates the cost to produce, transport, install, maintain, replace (if necessary), and eventually salvage the product over a chosen life-span evaluation. For the purposes of this report, a life span of 20 years was chosen. The installed cost of the system (which incorporates production, transportation, and installation) was determined using data from "Tracking the Sun" by Wiser, Barbose & Peterman to be $6.76 per watt of DC power. However, the federal government gives a tax incentive for 30% of the costs of a photovoltaic system. Therefore, this was deducted from the costs. In terms of maintenance, a PV system has to be inspected yearly for defects. The panels chosen are warrantied to produce peak power for 25 years, and therefore do not cost the owner to replace unless some form of damage occurs to the panels (as this cannot be accurately foreseen or predicted, the possibility of damage to the panels was neglected). However, the inverters are only warrantied for 10 years, and therefore the cost to replace the inverters at 10 years was incorporated. Finally, the salvage value of the system was taken as 20% of the initial cost of the system.

Over time, the value of money decreases. Therefore, in order to properly predict the associated costs of the system in terms of present dollars, engineering economics was used. This uses factors to represent the costs of everything in terms of present dollars. The resulting life cycle cost was approximately $72,000, as can be seen in Table 33. The rate of depreciation of value of general items was chosen as 4%, and the rate of depreciation of value of energy was chosen as 3%, based on recommendations from "Photovoltaic Systems," by James P. Dunlop.

Life-Cycle Cost - PV System					
General Rate: 0.04					
Energy Rate: 0.03					
Cost Description	Cost/year	Single Cost Year	Recurring Cost Years	Present Value Factor	Present Value
Initial Costs					
System Purchase & Installation	$105,139.63	0		1.000	$105,139.63
Incentives	-$31,541.89	1		0.962	-$30,343.30
Maintenance Costs					
Inspections	$100.00		20	13.590	$1,359.00
Repair & Replacement Costs					
Inverter Replacement	$8,000.00	10		0.676	$5,408.00
Salvage Value					
Salvage	-$21,027.93	20		0.456	-$9,588.73
				Total:	$71,974.60

Table 33 Life cycle cost of the PV system.

<u>*Payback Period*</u>

The payback period of the PV system was determined using the power rates as provided on the Los Angeles Department of Water and Power (LADWP) webpage. LADWP has six different energy rates. It first considers a high season (June-September) and a low season (October-May). Within each season, it has high peak hours (Monday-Friday, 1PM-5PM), low peak hours (Monday-Friday, 10AM-1PM and 5PM-8PM), and finally base rate hours (Monday-Friday 8PM-10AM, Saturday all day, and Sunday all day). These rates were combined as required to produce a cost per kWh for each of the 6 rates. This calculation can be found in Appendix I.

Since these rates are per kWh, the number of kWh of AC power had to be determined. A crude estimate of this was determined using recommendations from “Photovoltaic Systems” (the calculation can be found in Appendix I). A more refined estimate was then found using the free online software called PVWatts, the results of which can be found in Appendix I. The two estimates were relatively close, and therefore it was determined to use the PVWatts output. This was given per month, which enabled the calculation of AC power produced in the high season versus the low season. Next, it was determined that negligible power would be produced at the base rate, 80% would be produced at the high peak rate, and 20% would be produced at the low peak rate. The value of power produced by the PV system per year was then the total of the high season and the low season. The payback period was then determined by dividing the life cycle cost by the value of power produced per year. The resulting payback period was 33.22 years, the calculation for which can be seen in Table 34.

Payback Period - PV System		
Description	High Season	Low season
Total Power (kWh) *	8650	11456
High Peak Period Power (kWh) **	6920	9164.8
Low Peak Period Power (kWh) **	1730	2291.2
Value of High Peak Period Power	$927.33	$969.25
Value of Low Peak Period Power	$182.17	$226.70
Total Value of Power	$1,109.51	$1,195.95
Total Value per Year	$2,305.46	
Payback Period	31.22 years	

* = Found using PVWatts results
** = 80% of total power was assumed to be generated during the High Peak Period. The reamining 20% was assumed to be generated during the Low Peak Period.

Table 34 Payback period of the PV System.

Carbon Footprint

From "Emissions from Photovolatic Life Cycles" by Fthenakis, Kim, and Alsema, it was determined that the typical carbon footprint of a PV system is 35-58 g CO_{2e}/kWh. This is actually an equivalent weight of carbon dioxide, because some of the gases released by the production of solar cells are significantly more harmful to the ozone layer than carbon dioxide. Therefore, these are made into equivalent carbon dioxide weights. Typically, thin-film solar cells are on the lower side of this range, whereas monocrystalline silicon cells are on the higher side. Therefore, it was chosen to use the 58 g CO_{2e}/kWh figure to be conservative. This resulted in a carbon footprint of approximately 2,570 lb CO_{2e}. Since the PV system does not consume or sequester any carbon dioxide or greenhouse gases throughout the life of the system, the overall net carbon footprint is 2,570 lb CO_{2e}.

LEED Analysis

The original building intends to achieve LEED Gold from version 2.2 by gaining 43 points. Review of the LEED system indicates that incorporating a PV system probably could only earn one additional LEED point. This would be through the Energy & Atmosphere credit 2: On-Site Renewable Energy. The energy use for the building was not released, and the credit is based on providing on-site renewable energy for a certain percentage of the overall energy use of the building. Therefore, it was unable to be determined whether or not the power produced by the PV system is sufficient to achieve the credit.

Extensive Green Roof

Green roofs come in two main varieties, extensive and intensive. Extensive green roofs are lighter, shallower, less costly and are typically planted with only sedums (low, hardy plants). Intensive green roofs are heavier, deeper, more expensive and can be planted with a wider variety of vegetation, sometimes even trees. In general, extensive green roofs can typically be designed to not require mechanical irrigation, whereas intensive green roofs almost always require supplemental irrigation. Typically, both types of systems are built in a layered manner directly on top of the roof structure.

However, newer systems are emerging which actually provide unitized installation format. In this kind of system, the green roof is comprised of tub-like units which contain the various layers of the green roof. These are placed on the roof like living bricks to produce a green roof. These provide easy access to the roofing membrane below for maintenance purposes (simply remove the necessary units, access the roofing membrane, and then replace them with no damage to the green roof), and the units are easily replaceable should individual units have defects. These modular green roofs come in both extensive and intensive forms. However, for the USB, a basic extensive green roof was desired.

The GreenGrid 4" deep extensive system was chosen for use on the USB office roof. They have distributors all over the country, and the units are all pre-planted in local greenhouses, which eliminates the concern of invasive species. A data sheet for this system has been included in Appendix I. An image of a sample installation can be seen in Figure 48.

Figure 48 Typical installation of a 4" deep extensive GreenGrid system.

Life Cycle Assessment

To provide a comparable assessment to the PV system, a life cycle cost was found for 20 years. The manufacturer of GreenGrid was contacted, and indicated that 4" deep extensive green roofs cost $10-15/square foot for production and transportation and $3-6/square foot for installation. For a conservative estimate, $21/square foot was used as the cost for the initial investment in the system. The maintenance instructions for the system indicate that it must be inspected once every 4 weeks during the growing season each year to verify the green roof is performing appropriately. This cost was also incorporated.

The system is warrantied for 10 years. However, it seems unlikely that the entire system will fail at that time, so it was estimated that 25% of the units would need to be replaced at ten years. However, it's also important to note that this system protects the typically-exposed roof membrane from wear due to exposure to the elements. Therefore, since most built-up roofs only last about 15 years, the cost for not having to replace the built-up roof was credited to the owner. Finally, the salvage cost was determined by standard practice to be 20% of the original purchase cost.

Similar to the life cycle assessment for the PV system, the values of all of these components were put into present value in order to be able to compare them effectively. The overall life cycle cost was therefore determined to be approximately $56,600, as can be seen in Table 35.

Life-Cycle Cost - Green Roof System					
General Rate: 0.04					
Energy Rate: 0.03					
Cost Description	Cost/year	Single Cost Year	Recurring Cost Years	Present Value Factor	Present Value
Initial Costs					
System Purchase & Installation	$60,742.50	0		1.000	$60,742.50
Maintenance Costs					
Inspections	$400.00		20	13.590	$5,436.00
Repair & Replacement Costs					
25% Module Replacement	$15,185.63	10		0.676	$10,265.48
Roof Membrane Non-Replacement	-$26,032.50	15		0.550	-$14,317.88
Salvage Value					
Salvage	-$12,148.50	20		0.456	-$5,539.72
				Total:	$56,586.39

Table 35 Life cycle cost of the green roof system.

Payback Period

Using "Cost Effectiveness of Green Roofs" by Blackhurst, Hendrickson and Matthews, it was determined that green roofs save energy (by providing insulation and reducing the heating effect of a dark roof, both of which help to better maintain the temperature of the floor below the roof in both heating and cooling seasons, particularly the cooling season). They also reduce the need to process storm water run-off (since green roofs retain approximately 50% of rainwater which falls on them). Using the figures provided in this paper, it was determined that this green roof will save approximately 36,000 kWh per year in energy. One-third of this was considered high season power (since the high season is 4 months of the year) and the remaining two-thirds were considered low season power. Then (similarly to the PV system), it was determined that 80% of this was high peak period power and 20% was low peak period power. This resulted in a value of power saved per year of approximately $4,000.

The cubic feet of run-off saved were found by multiplying the approximate rainfall per year in Los Angeles (40 inches) by 50%. This was then converted to feet of rainfall, and then multiplied by the area of the roof. The Los Angeles Department of Water and Power (LADWP) rate for water was found to be approximately $0.038/CF, which was used to determine the value of the water saved.

This resulted in a total savings of approximately $4,200 per year. The payback period was then determined by dividing the life cycle cost of the green roof by the savings per year. As can be seen in Table 36, this was found to be 13.39 years.

Payback Period - Green Roof		
Description	High Season	Low season
Power Saved	12000	24000
High Peak Period Power (kWh) *	9600	19200
Low Peak Period Power (kWh) *	2400	4800
Value of High Peak Period Power	$1,286.47	$2,030.55
Value of Low Peak Period Power	$252.73	$474.94
Total Value of Power	$1,539.20	$2,505.49
Total Value per Year	$4,044.69	
Carbon Saved (lb CO_{2e})	54,000	
Run-off Saved (CF)	4,810.00	
Cost of Run-Off ($/CF)	$0.038	
Value of Run-Off Saved per Year	$182.78	
Payback Period	13.39 years	

* = 80% of total power was assumed to be generated during the High Peak Period. The reamining 20% was assumed to be generated during the Low Peak Period.

Table 36 Payback period of the green roof system.

Carbon Footprint

The carbon footprint of installing the green roof was determined using figures from "Cost-Effectiveness of Green Roofs." To produce, transport, and install the system, it costs approximately 53.4 lb CO_{2e}/square foot. This was used to determine that the cost to place the green roof system was approximately 154,500 lb CO_{2e}.

However, a green roof uses carbon dioxide in photosynthesis, essentially producing negative carbon dioxide over the course of its life. Using the figures from "Cost-Effectiveness of Green Roofs", it was determined that this green roof will remove 54,000 lb CO_{2e} per year. This results in a net carbon footprint of about 100,500 after the first year. However, toward the end of the third year, the green roof's carbon footprint will actually become negative, meaning that the green roof will actually remove more carbon dioxide from the environment than it will take to produce it.

LEED Analysis

After reviewing the LEED points already earned by the USB, it was determined that the green roof system would only earn one additional LEED point which was not originally pursued. This is achieved through Sustainable Sites credit 6.1: Stormwater Design: Quantity Control. This credit mandates retention of certain percentages of rainwater from certain storms for certain periods of time. The green roof could assist in earning this credit because it retains significant percentages of the rainwater which fall on it. However, it is possible that the green roof is insufficient in size to actually earn the credit.

Summary

Table 37 shows a summary for the two systems considered in this viability study. It also includes the weight of each system per square foot. Since the PV system is very light, it would likely have little or no impact on the overall structural system. However, the green roof system is relatively heavy, and would likely require upsizing of roof member sizes. Also, since the lateral system is designed for seismic forces, which depend on weight, this could affect the lateral system design.

Sustainability Breadth Summary		
Description	PV System	Green Roof
Life Cycle Assesment	$71,974.60	$56,586.39
Net Carbon Footprint (lb CO_{2e})	2,570.92	100,459.50
Payback Period (years)	31.22	13.39
LEED Points (Version 2.2)	1	1
Weight (psf)	2.03	18-22
Structural Impact	Minimal	Moderate

Table 37 Summary of results from the Sustainability Viability Study.

It is interesting to note that although both systems have the potential to earn only one additional LEED point, they are clearly not equal. The life cycle cost of the green roof is lower, the payback period is less than half of the payback period for the PV system, and the green roof will actually have a negative carbon footprint before even a quarter of its life cycle has ended. However, the green roof would likely have an impact on the structural system, the costs for which were not incorporated into this analysis. Even so, the results of this analysis clearly indicate that the green roof is the more viable option.

That being said, there are some additional considerations which are extremely difficult to quantify. For one, the green roof is more aesthetically pleasing, which is not really a factor here because the levels above this roof are all unoccupied mechanical levels. Also, both systems have their own potential liabilities. If improperly contained, the roots from the green roof plants can damage the roofing membrane, causing what can be immense damage. Also, the green roof must be properly drained to ensure it doesn't become significantly overweight. On the other hand, the PV system is an electrocution hazard, and is significantly more prone to damage due to the elements. Since the actual parts are only warrantied against this type of damage for 5 years, this could present a major cost if the panels were to be damaged by debris, inappropriate handling, or unforeseen weather conditions.

Conclusions

Four designs were undertaken in steel to determine whether or not steel would have been viable at the present location, what costs are associated with moving from a low seismic region to a high seismic region, how much cost is associated with designing for higher performance criteria, and which of two alternative designs, one traditional and one high-tech, is the more efficient method for achieving a higher performance.

It was found that, although the steel structure in the Northeast USA was 50% lighter and 5.5% less expensive, it also takes 2 months (about 10%) longer to construct, which would likely have been unacceptable to the owner. It was also found that the penalty to move that structure to a high-seismic region was an increase in weight of about 4.5% and an increase in cost of about 1.6%. This was significantly lower than expected. In order to increase the performance of the structure in the traditional method, the structure increases in weight by 9.7% and in cost by 6% over the basic structure in a high-seismic region. However, in order to increase the performance of the structure using the high-tech method, the structure increased in weight in 1.5% and in cost by 1.5% over the basic structure in a high-seismic region. It was therefore determined that the high-tech option is more viable.

These designs were created using a mix of hand calculations, spreadsheets, RAM Structural System, ETABS, and SAP 2000. This design process integrated master's level coursework in the modeling of the structures (AE 597A), the earthquake design (AE 538), and in the design of seven sample connections (AE 534).

The costs and schedule durations of the designs were found to constitute a construction management breadth. Using the original schedule and original cost data provided by Turner Construction, quantity take-offs for the superstructure and some additional architectural components, and data from RS Means schedules and their associated costs were developed for each design. This was used to help compare the designs.

Finally, a sustainability breadth was undertaken to determine the viability of including a photovoltaic system or a green roof on the building once it was relocated to California. These systems were designed as fully as possible, and then evaluated based on a life-cycle assessment, a payback period, a carbon footprint, and finally the number of additional LEED points each system could earn. It was found that each system had the potential to earn one LEED point. However, the life-cycle cost of the green roof was about 30% lower, the payback period of the green roof was nearly 60% less, and the net carbon footprint of the green roof will eventually become negative (meaning the green roof will remove more carbon dioxide from the atmosphere than it takes to produce it, given enough time). Therefore, it is clear that the green roof is the more viable technology, despite the fact that they are essentially equal according to the LEED system.

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