EDENWALD NEW TOWER

Final Report

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Due: 9 April 2008

EDENWALD NEW TOWER
BALTIMORE, MD

General Information

Owner: General German Aged People's Home of Baltimore

Owner Representative: Matthews Development Company

Architect: SFCS, Inc.

General Contractor: Whiting-Turner

Structural Consultant: Rathgeber/Goss Associates

Estimated Cost: $52 million

Size: 253,000 sq. ft.

Project Delivery Method: GMP

Architecture

12 Story Continuous Care Retirement Community Addition

Houses 60 independent apartment units and 32 assisted living units

Amenities include an indoor pool, fitness center, enclosed walking track, outdoor terrace and more

Building envelope consists of brick veneer, pre-cast concrete panels, and glass windows and curtain walls

Mechanical

(2) 100-ton rotary screw chillers

(2) 51.7 BHP boilers

9300 CFM air handling unit

580 GMP cooling tower

Structural

Frame composed of cast-in-place columns and beams with 9" post-tensioned concrete slabs

Foundation consists of spread and strip concrete footings supported by geopiers and are designed for 4-6000 psf

WWF reinforced, 5" slab on grade

Main roof utilizes 9-16" sloped, post-tensioned concrete slabs

Penthouse is supported with a steel frame and non-composite concrete roof on metal deck

Laterally supported by ordinary reinforced concrete shear walls

Electrical

External utility transformer provides 480Y/277 power

450 kW 480/Y277 emergency transformer

(8) step down transformers to 208Y/120 power for apartment units

Lighting

Living units and circulation through building mainly illuminated with fluorescent luminaires

BRYAN HART
STRUCTURAL OPTION

http://www.engr.psu.edu/ae/thesis/portfolios/2008/bgh132/index.htm

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REFERENCES

Arthur h. Nilson, David Darwin, Charles W. Dolan, "Design of Concrete Structures, Thirteenth Edition," McGraw Hill, New York, NY, 2004.

Braja M. Das, "Principles of Foundation Engineering, Sixth Edition" Nelson, Toronto, ON, 2007.

*Building Code Requirements for Structural Concrete:*ACI 318-05, American Concrete Institute, Farmington Hills, MI, 2005.

Excerpt from 28 CFR Part 36: *ADA Standards for Accessible Design.* Department of Justice. 1994.

Jim Johnson, Jorge Rocafort, Madan Mehta, "Architectural Acoustics, Principles and Design," Prentice-Hall, Inc., Upper Saddle River, NJ, 1999.

Lighting and the Visual Environment for Senior Living: IESNA Recommended Practice-28-08, The Illuminating Engineering Society of North America, New York, NY, 2008.

Marshall Long, "Architectural Acoustics," Elsevier, Burlington, MA, 2006.

Minimum Design Loads for Buildings and Other Structures: ASCE 7-05, American Society of Civil Engineers, Reston, VA, 2006.

Linda Sanford, "The Importance of Lighting for the Elderly," Lighthouse International. Taken March 2008 from:
http://www.lighthouse.org/medical/the-importance-of-lighting-for-the-elderly/

S. K. Gosh, "Building Codes, Standards and Resource Documents: A Status Report," S.K. Ghosh Associates Inc. Taken April 2008 from:
http://www.skghoshassociates.com/sk_publication/PCI_March02_bldg_codes_stand.pdf

EXECUTIVE SUMMARY

The Edenwald New Tower is a 12-story building located in Baltimore, Maryland. Designed as an addition to an existing 15-story tower, its 253,000 square feet were designed to meet the demands of a continuous care retirement community for total project cost of $52 million. The project scope includes 60 apartments and 32 assisted living units, 4 levels of parking, and amenities.

The area of interest for the proposed thesis design revolves around the building's main lateral force resisting system: 15 ordinary reinforced concrete shear walls, eight of which form 2 separate cores. It was determined that when the building was in its initial stages of design, the code being adopted was IBC 2000. Hence, seismic analysis was performed according to that code. However, in the years since then ASCE 7-05 has become the most recently released code governing seismic design, as IBC currently directly references ASCE 7. In the latest code, changes have been made in the seismic chapters governing the S_1 and S_s values with which the seismic response coefficient, C_s, is calculated. The older code has much more stringent requirements, meaning that if the building was designed under the current code, the base shear would have been reduced, possibly allowing for the removal of some of the walls.
The goal of this thesis is to redesign the main lateral force resisting system according to loads determined from ASCE 7. At the same time, alternative wall locations will be investigated to reduce the significant amount of torsion that the current design must handle due to the high eccentricities on each floor.

The distribution of lateral forces was calculated by the computer program ETABS, with which two models were analyzed: one for serviceability requirements, and one for strength requirements. For serviceability, wall sizes and locations were determined by drift and displacement limits, and walls 3,4,6,7 and 8 were removed, while wall 1B was added (see Figures 7 and 8). Reinforcement for flexure was then determined through ETABS and PCAColumn, while shear and boundary element reinforcement requirements were calculated by hand. In anticipation of the wall reductions, which are discussed below, it was subsequently determined to include coupling beams at shear wall openings to improve the behavior of the core comprised of Walls 9, and as such their design was included in the depth study. Columns were also designed to carry gravity loads where walls were taken out. Lastly, a foundation design was performed for one of the replacement columns.

Ultimately, the proposed redesign was estimated to save the owner approximately $515,000 through the removal of walls and redesign of the foundations. These savings seem to justify the design, however the coupling beams were determined to be superfluous and unnecessary. Furthermore, torsion was not reduced due to an inability to relocate the center of rigidity.

The first of two breadth studies performed was a lighting analysis. Spaces used for the elderly are subject to more stringent lighting requirements due to the fact that the occupants are likely to have decreased or limited vision. The fifth and six floors were designed as assisted living, and so the corridor and adjacent reading/gathering area for these floors would need to have appropriate lighting, as dictated by IESNA. For this breadth, an analysis of the public spaces of these floors was conducted, using the program AGI to determine illuminance levels. The analysis proved that illumination levels were adequate, exceeding the 30 footcandle recommendation except in one area, which was addressed through the redesign. Other changes included ADA compliant wall sconces which were used to draw more attention to apartment entrances. General aesthetics and power density issues were also considered.

The second breadth study was acoustical. One of the amenities provided in the Edenwald New Tower is a chapel located on the first floor. This spaced was analyzed and redesigned accordingly for two criteria: reverberation time and sound transmission class (for the partition separating the chapel from the corridor). Those who use hearing aids are more susceptible to reverberation interference, and so it is critical that the acoustics of the space not inhibit the occupants from understanding the words being spoken. However, calculations revealed that the space was already well beneath recommended reverberation times. While churches and worship places often rate higher reverberation times to allow the sound of the music to properly develop, the fact that the elderly will probably exclusively use the chapel means that speech clarity supersedes sound quality.

Additionally, spaces such as theaters and music rooms (which are similar in function to a chapel) are desired to have the partitions separating them from adjacent corridors achieve a sound transmission class (STC) rating of 60. The STC rating is an average value given to a partition to rate its sound attenuation according to the partition's transmission loss values across 16 frequencies. The wall separating the chapel from the corridor was analyzed and found to have an STC of 43. The partition assembly was redesigned and was improved to an STC of 56.

BUILDING OVERVIEW & BACKGROUND

Completed in 1985, the original Edenwald building was built on approximately five acres of land purchased from Goucher College as a continuous care retirement community (CCRC). The existing high-rise structure is located in the heart of Towson, Maryland, and is one block away from Towson Town Center, providing residents with a great deal of opportunities within walking distance. The addition of the 12-story new tower will increase the number of residents and provide new amenities to the complex.

The tower will house 60 independent apartment units and 32 assisted living units, in addition to 4 levels of parking. Amenities provided to the residents will include a large indoor pool, a whirlpool spa, a state of the art fitness center, fully equipped locker rooms, an enclosed, air-conditioned walking track, a spacious pub & lounge, and outdoor terrace to include a nine-hole putting green, and a chapel and great room. Sample plans for the ground floor and typical tower floors are provided on pages 12 and 13.

The first floor of the tower will contain most of the amenities, including the pool, health lounge, lobby, fitness center and chapel. Additionally, offices and the mechanical room will also be located on the first floor. The track will be suspended over the pool and entered through the second floor. The entire third and fourth floors will be devoted to parking, in addition to limited parking found on the first and second floors. The apartment units will begin on floor 5, which will also contain the outdoor terrace and putting green, and continue through floor 12. Foors 5 and 6 have been exclusively set aside for the assisted living apartment units. The overall shape of the building changes at level 5, where the parking garage is discontinued and the building rises in the shape of an L. This can be seen in Figure 1.

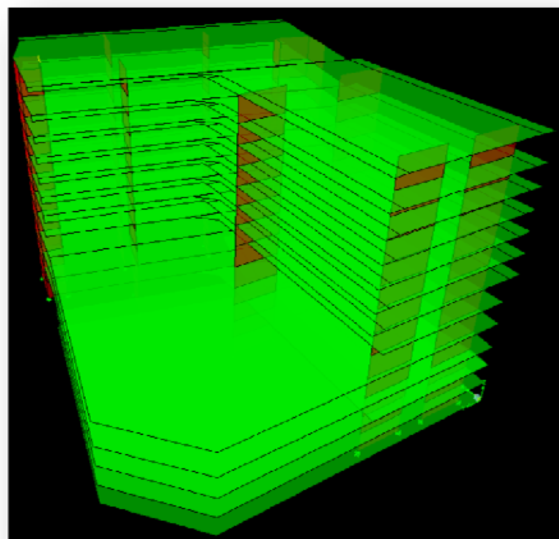

Figure 1: Building Shape Schematic

The Edenwald New Tower will be enclosed with a combination of brick veneer, precast concrete panels, and glass windows and curtain walls. The roof utilizes a Firestone rubber roof with polyiso rigid foam board placed atop a post-tensioned concrete slab. There is also a steel framed penthouse which will house mechanical equipment and also partially conceal the cooling tower.

The tower's mechanical equipment is primarily located in two places: room 129 on the first floor, and the penthouse. The first floor mechanical room has two, 100-ton rotary screw chillers and a 9300 CFM air handling unit. The air handling unit features adjustable frequency drives for supply and return fans, which help energy savings. The mechanical room also features adjustable frequency drives for the two, cold water distribution pumps.

The penthouse features two, 51.7 BHP boilers, the pumps for which also feature the energy saving adjustable frequency drives. Next to the penthouse can be found the building's 580 GPM cooling tower, provided with an 8kW electric basin heater. Also on the roof is an energy recover unit, which is estimated to recover 846.5 MBH on a winter design day, and 354.7 MBH on a summer design day.

An external utility transformer in the northeast corner of the site provides 480Y/277 power to the building while eight step down transformers provide 208Y/120 power for the apartment units. A 450 kW 480/Y277 emergency generator is located on the first floor. As for lighting, there is a great variance in the type and style of luminaires used throughout the building, though most are fluorescent.

The entire tower is sprinkled throughout, while only the 1st and 2nd floor of the garage are sprinkled. Exterior bearing walls, exits and stair enclosures, shafts and elevators, the structural frame, floor/ceiling construction are all rated are rated at 2 hours. Dwelling unit separation is rated at 1 hour. The most restrictive travel distance is 250 feet with sprinkler.

STRUCTURAL SYSTEM OVERVIEW

Foundation:

The geotechnical analysis of the sub-surface conditions prior to construction revealed great variances in soil type and depth to bedrock, ranging from 50 to 150 feet deep, making deep foundations impractical. Given two recommendations from the geotechnical engineer, it was decided by the designers to use a geopier system as opposed to an alternative of driven HP 12x74 piles. Comprised of densified "rammed" stone aggregate piers, geopiers are referred to as "intermediate foundation systems" in that they strengthen, stiffen and reinforce soil layers beneath the building. The use of this option provided the opportunity to utilize a shallow foundation system of typical spread footings. (It should be noted, however, that pre-existing utilities only discovered upon excavation in the north end of the site required the use of the HP piles, in that localized area only.) The geopiers were determined to require a 30 inch diameter, and range from 20 to 30 feet in length. The allowable bearing pressure of the strengthened soil beneath the building was then determined to be 6 ksf beneath the tower, and 4 ksf beneath the parking garage. Total settlement expected from the geopier design amounts to one inch.

All concrete used in the Edenwald New Tower is normal weight (145 pcf dry unit weight). Footings, grade beams and slabs on grade have a minimum 28-day strength of 3000 psi. Shear wall footings have a minimum 28-day strength of 4000 psi. The slab on grade is reinforced with 6x6-W2.9x2.9 WWF on a vapor barrier on 4 inches of granular subbase.

Floor System:

The typical floor system used is a 9 inch, post-tensioned concrete slab having a minimum 28-day strength of 5000 psi. In specific locations where the post tensioned system is not feasible and/or practical, reinforced one way slabs were used, ranging in thickness from 8 to 9 inches, with cast in place concrete beams, both requiring a minimum 28-day strength of 5000 psi.

Roof System:

The flat roof system is almost identical to the typical floor system. Still utilizing the post-tension reinforcement, the slab thickness reaches up to 16 inches underneath the penthouse. The penthouse is supported by a steel braced frame and is covered by 1.5 inch deep, wide rib, 20 gage galvanized metal deck. The penthouse roof is supported by a combination of steel W shapes and 12k3 joists. The columns supporting the penthouse are W8x31 shapes.

Columns:

The building is supported by rectangular concrete columns laid out in a geometric grid. The columns range in size, the most common being 22x22 and 22x36. The largest column found in the building is 22x60. Column service loads range from 203 kips in the garage to 1600 kips at the base of the tower. From the ground level to the seventh floor, the columns are required to have a minimum 28-day strength of 6000 psi. From the seventh floor to the roof, that value decreases to 5000 psi.

Lateral System:

The building is laterally supported in both the N-S and E-W directions by a total of 15 simply reinforced concrete shear walls, with thickness ranging from 12 to 14 inches. These shear walls are required to have a minimum 28-day strength of 5000 psi. Located throughout the building, the shear walls are often conveniently placed around stair and elevator shafts. All but one of the 15 shear walls run the entire height of the building. See Figure 2 for wall locations.

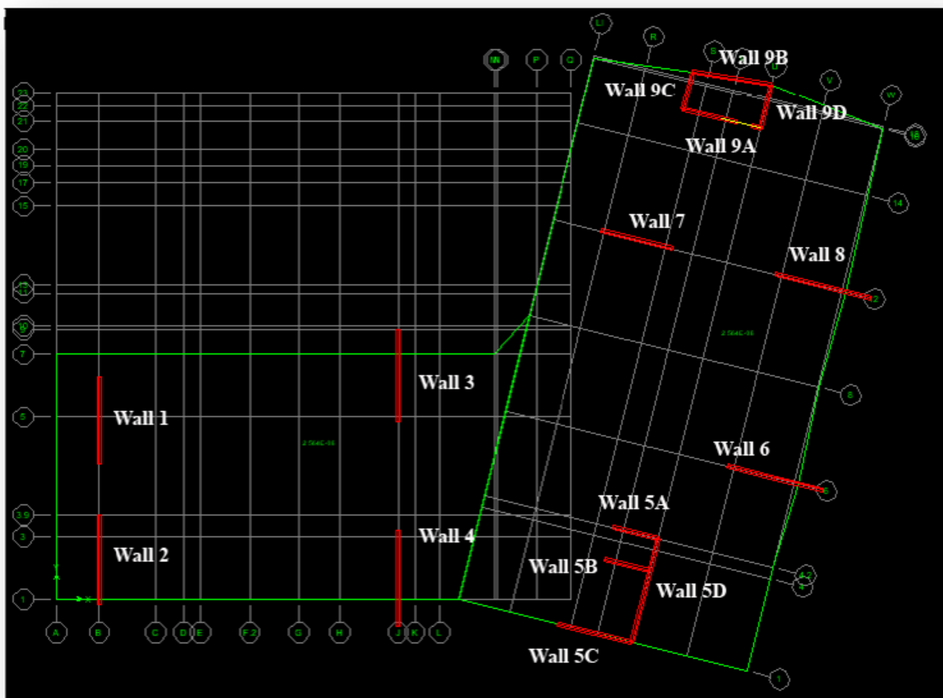

Figure 2: Typical Floor Below Level 6 with Wall Locations

SAMPLE PLANS

Figure 3: First Floor Plan, Amenities & Parking

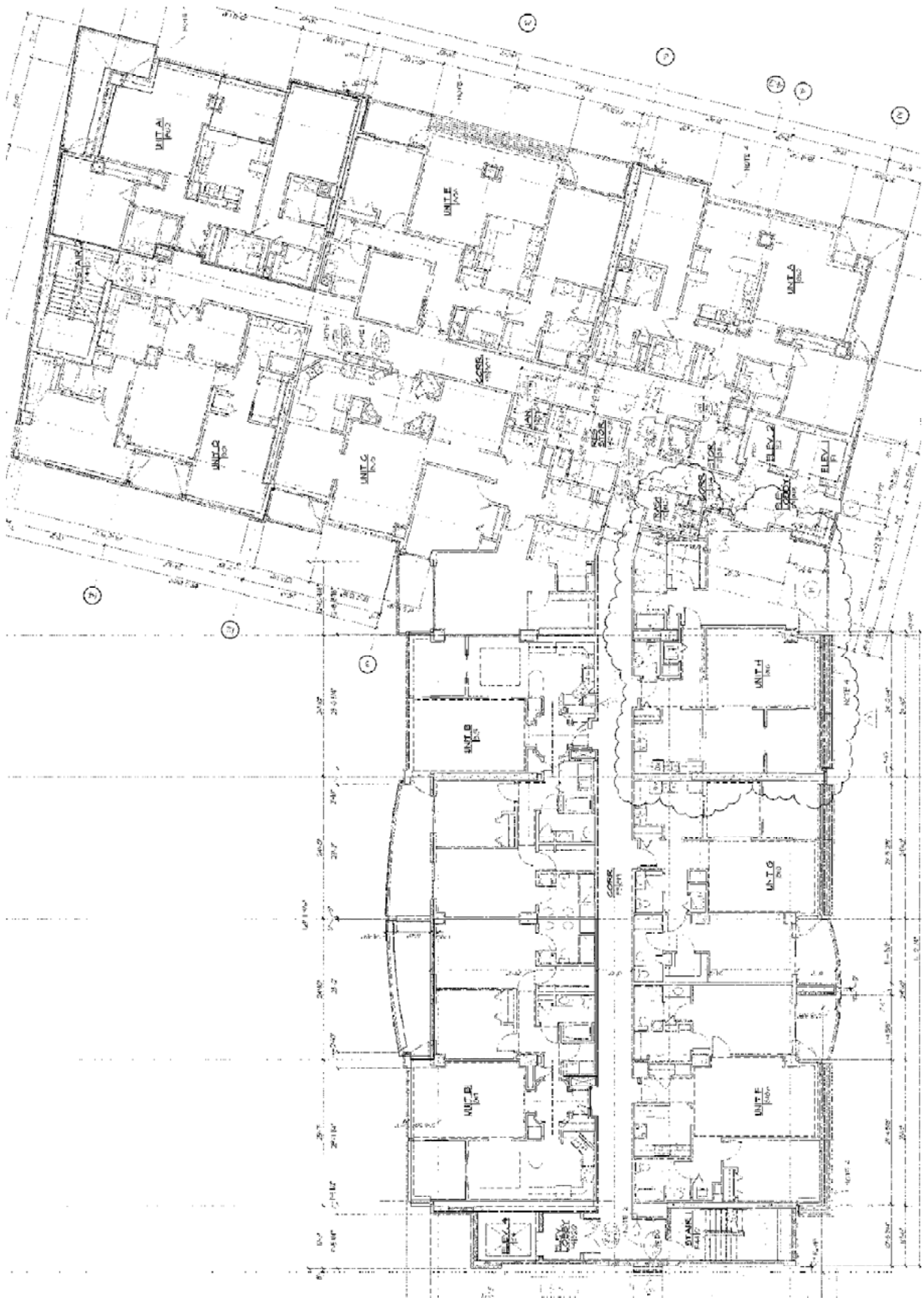

Figure 4: Typical Tower Plan, Floors 8-11, Apartments

THESIS PROPOSAL

The Edenwald New Tower was designed to withstand seismic forces calculated according to IBC 2000. When that code was released, its seismic provisions were based almost entirely on the 1997 NEHRP Provisions. However, for the release of IBC's 2003 edition, the situation changed and large portions of the seismic provisions were deleted, instead referencing the seismic sections of ASCE 7-02, which in turn were based largely on the 2000 NEHRP Provisions. Following IBC's trend, NFPA 5000-2002 then also referenced ASCE 7-02's seismic section. The reliance on ASCE 7's seismic provisions increased to its maximum when it was then determined that IBC 2006 would entirely reference ASCE 7 for seismic. Considering the fact that their document had increasingly become the governing code for seismic provisions, ASCE decided to develop a Seismic Task Group which would oversee the development of their own seismic provisions, to be released for the first time in the 05 edition.[1]

As a result, several changes were made to ASCE 7's seismic chapters. Included in those changes were the S_1 and S_s values with which the seismic response coefficient, C_s, is calculated. In the case of Edenwald's geographical location, those values saw a decrease which would in turn provide up to a 20% reduction in required seismic base shear. The original design was based off a seismic controlled base shear of approximately 1000 kips. Using the updated code, the 20% reduction would allow for significant reduction in lateral strength.

The goal of this thesis is to redesign the main lateral force resisting system according to loads determined from ASCE 7-05. The structure is suspected to be subject to severe torsion resulting from large eccentricities, and so a subsequent goal is to investigate the possibilities of relocating walls during the design process to reduce that eccentricity. It is also suspected that the structure is over-designed for even its IBC 2000 loading requirements. Thus, wall reductions are anticipated to be significant. As such, the core comprised of Wall 9A through 9D will be redesigned with coupling beams at wall openings in the case extra stiffness and energy dissipation is needed. Where walls are removed, columns will be designed to handle gravity loads no longer accounted for. Furthermore, foundations will be checked in cases where the revised design is likely to create significant changes from the conditions used in the original system.

[1] S. K. Gosh, "Building Codes, Standards and Resource Documents: A Status Report," S.K. Ghosh Associates Inc. Taken April 2008 from: http://www.skghoshassociates.com/sk_publication/PCI_March02_bldg_codes_stand.pdf

MAIN LATERAL FORCE RESISTING SYSTEM REDESIGN

ETABS Analysis & Design Overview

To begin the redesign of Edenwald's lateral system, a model of the structure was developed in the program ETABS. Ultimately, the model was used for two analyses: serviceability and strength. As the emphasis of both analyses lay on the lateral force resisting system, columns were ignored due to the large number that exist, and also due to the large number of variances. (Given the size of the shear walls in this building, the relative stiffnesses of the columns would have been negligible, making their absence insignificant for the purpose of this report.) The structural elements used within the model included:

- rigid diaphragms
- shear walls
- coupling beams to handle shear wall openings.

Lateral loads, explained in depth below, were incorporated according to ASCE 7-05. Additional masses were superimposed to ensure accurate modal response periods.

When modeling both walls and coupling beams, cracked section properties were used for the strength analysis. ACI 318-05 allows the use of fifty percent of stiffness values based on gross section properties. Accordingly, the f_{22} modifier was adjusted to 0.5. This modifier was left at 1.0 for the serviceability analysis.

Since several of the shear walls act considerably as bearing elements, namely walls 1, 2, and 5D, gravity loads were calculated for these walls by hand. (The calculations for these loads can be found in Appendix B.) In the design of these walls, additional measures were taken to ensure the inclusion of those gravity forces, which are discussed below. The remaining walls are surrounded by columns within close proximity, or are on the exterior of the building, resulting in much smaller gravity loads. It was decided when modeling these walls to neglect the gravity loads altogether. This would prove to be conservative when designing the footing connection for uplift. (ETABS does consider the gravity load induced from self-weight.)

The Edenwald New Tower has horizontal structural irregularity type 5, called Nonparallel Systems-Irregularity (ASCE 7-05 Table 12.3-1), and so the ETABS model was created to utilize three dynamic degrees of freedom, to include translation in the two orthogonal directions and rotation about the vertical axis in accordance with ASCE 7-05 12.7.3.

P Delta effects, existing where deflections magnify the effect of gravity loads, were automatically considered in the analysis.

Static Forces: Seismic

The original building was designed according to IBC 2000. However, in 2006 the International Building Code was modified to directly reference ASCE 7-05 for seismic criteria. Furthermore, it had been decided that in the eastern US, the required seismic forces were often much more severe than engineers felt appropriate, as "beefed up" lateral systems were much more costly. So, in response, the design spectral response accelerations for many parts of the Eastern US were reduced in the 2005 edition of ASCE 7. Thus, for this report, the seismic forces for Edenwald were determined in accordance with the equivalent force procedure in ASCE 7-05 Chapter 12. Table 1 summarizes the changes made in the seismic design. Using the revised base shear of 793 kips, the distribution of forces was recalculated from the original design and can be seen in Table 2. Additional seismic calculations, such as those for the building weight, are available upon request.

The forces from Table 2 were entered as manual loads and applied to each diaphragm at the center of mass. In addition to the inherent torsion created from the eccentricity between the center of mass and center of rigidity, accidental torsion was considered as 5 percent of the length of each diaphragm in both orthogonal directions.

Original Design		Thesis Design	
S_s	0.210 g	S_s	0.178 g
S_1	0.070 g	S_1	0.052 g
S_{ms}	0.336 g	S_{ms}	0.285 g
S_{m1}	0.168 g	S_{m1}	0.125 g
S_{ds}	0.224 g	S_{ds}	0.190 g
S_{d1}	0.112 g	S_{d1}	0.083 g
R	5	R	5
I	1.25	I	1.25
Period	1.2 sec	Ta	0.73 sec
Building Weight	45300 kips	Max T	1.23 sec, **controls**
		Building Weight	47000 kips
			0.047 (not greater than)
		C_s	0.017 **controls**
C_s	0.022		0.01 (not less than)
Base Shear	*996.6 kips*	*Base Shear*	*793.04 kips*
			20.43 % decrease

Table 1: Seismic Code Revision Results

Distribution of Seismic Forces per Floor

Level	w_x	h_x	$w_x h_x^k$	C_{vx}	F_x	M
*Roof	4301	119.33	12802763957	0.2520	200	23849
12	3745	107.33	8307732578	0.1635	130	13919
11	3636	98.00	6709548696	0.1321	105	10264
10	3636	88.67	5620667948	0.1106	88	7780
9	3636	79.33	4615592398	0.0909	72	5716
8	3636	70.00	3698679380	0.0728	58	4042
7	3708	60.67	2972668897	0.0585	46	2815
6	3580	50.00	1984314668	0.0391	31	1549
5	4999	39.33	2342750996	0.0461	37	1438
4	4396	28.00	1022552292	0.0201	16	447
3	4960	18.67	617893065.2	0.0122	10	180
2	3642	9.33	104778906.3	0.0021	2	15
*Includes weight of Penthouse					Overturning Moment	72014 ft-kips
					Base Shear	793 kips

Table 2: Seismic Force Distribution

Static Forces: Wind

For the calculation of wind forces, ETABS was used to produce the forces to be applied at the geometric center of each diaphragm, according to ASCE 7-02 Chapter 6. Using the input wind criteria found in Table 3 and the diaphragm extents (as well as a 2 foot parapet) to calculate the wind forces, ETABS generated the 12 possible derivations of the 4 wind cases described in Figure 6-9 in ASCE 7-02. These derivations became the static load cases used for analysis, and are shown in Table 4. (Hand calculations for the gust factor were made according to method 2 described in Chapter 6 and are available upon request.)

Basic Wind Speed	90 mph
Occupancy Category	III
Importance Factor	1.15
Exposure	B
Topographic Factor (K_{zt})	1.0
Wind Directionality Factor (K_d)	0.85
Gust Factor (both directions)	0.83
Internal Pressure Coefficient	± 0.18

Table 3: Wind Load Criteria

ETABS Static Load Case	ASCE 7 Design Wind Load Case
Wind	1
Wind-2	1
Wind-3	2
Wind-4	2
Wind-5	2
Wind-6	2
Wind-7	3
Wind-8	3
Wind-9	4
Wind-10	4
Wind-11	4
Wind-12	4

Table 4: Wind Load Cases

It is important to note that only the seismic forces were changed from the original design loads. The wind forces used in the analysis are the same as those of the original design. (To justify the use of the computer program to calculate these loads, a hand check of the wind load was performed at level 12 in one direction and found to be within 10 percent of the corresponding ETABS load. This hand check is available upon request.) The table below summarizes the original and revised *factored* lateral forces.

Conclusions for Original Design

	Wind (Factored)	Seismic (Factored)
X Direction		
Base Shear	684 k	997 k
Overturning Moment	54430 ft-k	90540 ft-k
Y Direction		
Base Shear	904 k	997 k
Overturning Moment	69320 ft-k	90540 ft-k

Conclusions for Thesis Design

	Wind (Factored)	Seismic (Factored)
X Direction		
Base Shear	684 k	793 k
Overturning Moment	54430 ft-k	72014 ft-k
Y Direction		
Base Shear	904 k	793 k
Overturning Moment	69320 ft-k	72014 ft-k

Table 5: Lateral Force Summary

From this data it is clear that the wind base shear controls in the Y direction while seismic still controls in the X direction – thus the analysis was run to include both loading conditions.

Analysis

In addition to the consideration of the static wind and seismic loads, ETABS considered P-Delta effects to account for the consequence of deflections when determining member forces and reactions. ETABS offers two methods when considering this effect, both an iterative (based on load cases) and a non-iterative (based on mass.) According to ETABS Help, the non-iterative method is described as the following:

> The load is computed automatically from the mass at each level as a story-by-story load upon the structure. This approach is approximate, but does not require an iterative solution. This method essentially treats the building as a simplified stick model to consider the P-Delta effect. It is much faster than the iterative method. It does not capture local buckling as well as the iterative method. This method works best if you have a single rigid diaphragm at each floor level, although it also works for other cases as well. The reason we provide this method is to allow you to consider P-Delta in cases where you have not specified gravity loads in your model.

Since no gravity loads were specified, and the rigid diaphragms were modeled as null areas without material properties, this approximate method was found to be acceptable.

The dynamic analysis of the structure was run to determine 12 natural modes of vibration for the structure. The three most severe modes were:

- 1st Mode: Translation in X direction; T = 2.43 sec
- 2nd Mode: Translation in the Y direction; T = 1.93 sec
- 3rd Mode: Torsion; T = 1.37 sec

Resultant forces and reactions experienced by the walls and coupling beams from the static load cases were computed in the analysis according to load combinations seen in Appendix A. To actually design the walls, the ETABS shear wall design feature was used to determine design parameters, which are discussed below.

Analysis: Existing Torsion

Since one of the goals of this report was to reduce torsion if it all possible, it was deemed appropriate to investigate the existing levels of torsion the building was designed to withstand. Thus, an analysis of the original building with the original loads was completed. The pier forces from ETABS were incorporated into a spreadsheet designed to find the maximum shear forces for each wall at each level. However, this value was the *total* shear experienced by the wall, which includes both direct and torsional shear. To isolate the portion of shear from torsion, moments were added to each diaphragm that would replicate the shear from torsion. This was accomplished by taking the story shear corresponding to the load case for the controlling shear in each wall and multiplying it by the eccentricity between the center of mass and center of rigidity (see Figure 5). (In each direction for the original design, seismic controlled, and so the geometric center's eccentricity for wind loading was not needed.)

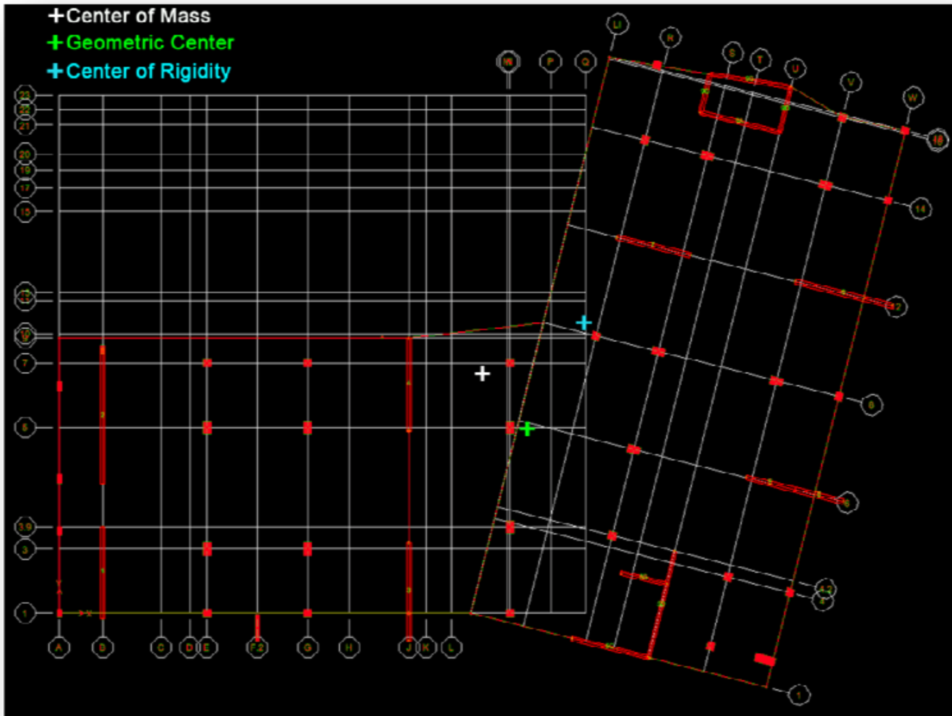

Figure 5: Building Centers

Once these moments were added to the model's diaphragms, the resultant shear in each wall was taken to represent the amount of torsion that wall was originally designed to withstand. The results can be seen in Figure 6 for wall 5C in particular. The difference between the total shear and the direct shear, or the portion in green, represents the amount of shear developed by torsion.

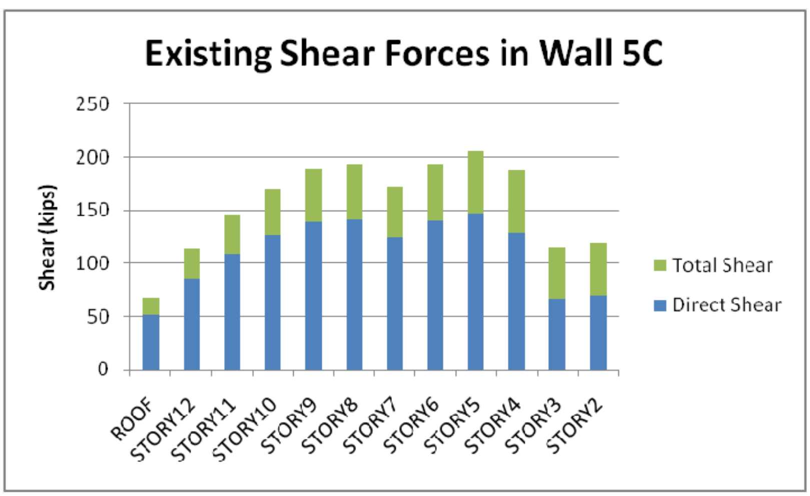

Figure 6: Existing Torsion

It is clear that a reduction in the eccentricity would greatly reduce the required shear capacity of the walls. During shear wall design, a goal was to locate the walls in a way that would bring the center of rigidity to the left.

Shear Wall Design: Serviceability

The first step in design was to perform an iterative serviceability analysis (i.e. multiple solutions were considered.) Shear walls were sized according to two serviceability criterion: total building displacement and story drift. Simultaneously, their placement was also chosen based on consideration for reduction of the eccentricity between the center of mass and center of rigidity. These procedures led to the following changes: walls 3, 4, 6, 7 and 8 were removed, wall 1B was added, and wall 9's thickness was made 16 inches for the sake of coupling beams. The original and final designs can be seen below.

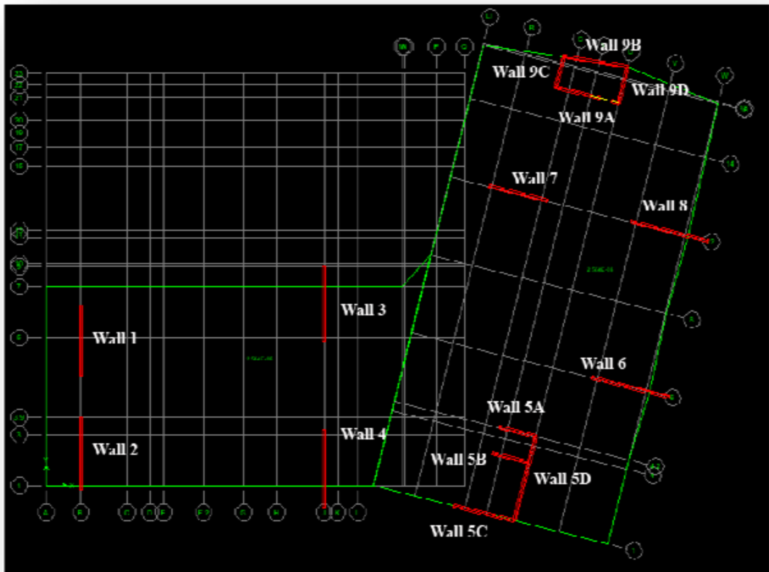

Figure 7: Original Wall Locations

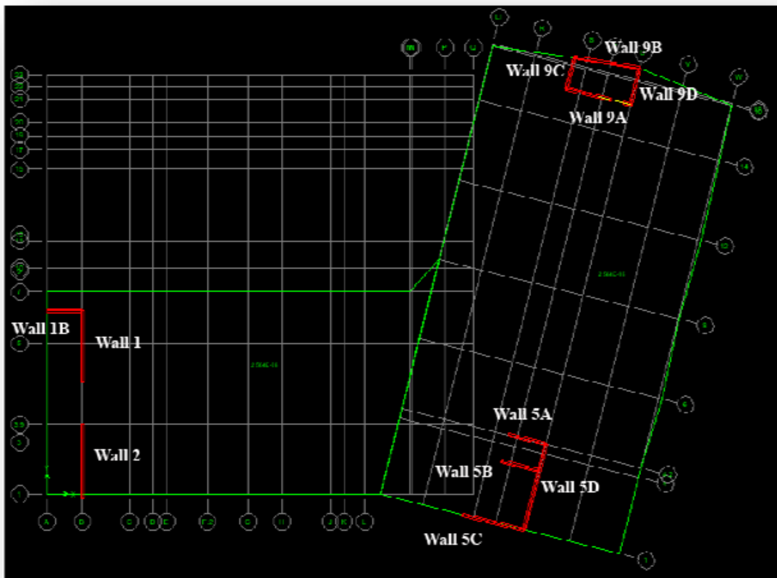

Figure 8: Revised Wall Locations

Torsion

In the effort to reduce torsion, wall placements were altered with the hope of changing the center of rigidity (COR). However, walls 5 and 9 act as core elements, which greatly increases their relative stiffness. The result was that the placement of walls 3,4,6,7, and 8 became less influential than was originally hoped, especially considering the fact that they are located closer to the center of the building. Wall 1B was added in the hope that it would increase the relative stiffness of Wall 1 by inducing core-like behavior. This change was mildly successful, but with the removal of the other walls there was a net change in the COR location of only 5 feet to the left. This is considered negligible and no further investigation of torsion reduction will be considered.

Drift

Story drift limits were determined according to ASCE 7-05 12.12.1 and building displacement was limited to the industry standard of H/400. For the seismic drift calculations, the drift value was multiplied by the deflection amplification factor C_d of 5 and divided by the importance factor of 1.15 to calculate the amplified story drift, as per ASCE 7-05 12.8.6.

Seismic X

Story	Story drift (in)	Amplified Story Drift (in)	Allowable (in)		Displacement (in)	Allowable (in)	
Roof	0.387	1.684	2.160	ok	2.660	3.600	ok
12	0.300	1.305	1.679	ok			
11	0.298	1.295	1.679	ok			
10	0.290	1.261	1.679	ok			
9	0.281	1.222	1.679	ok			
8	0.266	1.159	1.679	ok			
7	0.279	1.214	1.921	ok			
6	0.250	1.086	1.921	ok			
5	0.230	0.998	2.039	ok			
4	0.152	0.662	1.679	ok			
3	0.118	0.511	1.679	ok			
2	0.074	0.321	1.679	ok			
1	0.029	0.125	1.679	ok			

Wind X

Story	Story drift (in)	Allowable (in)		Displacement (in)	Allowable (in)	
Roof	0.135	0.360	ok	0.971	3.600	ok
12	0.105	0.280	ok			
11	0.105	0.280	ok			
10	0.102	0.280	ok			
9	0.100	0.280	ok			
8	0.096	0.280	ok			
7	0.102	0.320	ok			
6	0.093	0.320	ok			
5	0.087	0.340	ok			
4	0.059	0.280	ok			
3	0.047	0.280	ok			
2	0.030	0.280	ok			
1	0.012	0.280	ok			

Seismic Y

Story	Story drift (in)	Amplified Story Drift (in)	Allowable (in)		Displacement (in)	Allowable (in)	
Roof	0.279	1.215	2.160	ok	1.930	3.600	ok
12	0.216	0.939	1.679	ok			
11	0.213	0.925	1.679	ok			
10	0.208	0.905	1.679	ok			
9	0.200	0.871	1.679	ok			
8	0.190	0.828	1.679	ok			
7	0.200	0.868	1.921	ok			
6	0.181	0.785	1.921	ok			
5	0.165	0.715	2.039	ok			
4	0.111	0.482	1.679	ok			
3	0.083	0.360	1.679	ok			
2	0.057	0.249	1.679	ok			
1	0.027	0.117	1.679	ok			

Wind Y

Story	Story drift (in)	Allowable (in)		Displacement (in)	Allowable (in)	
Roof	0.168	0.360	ok	0.995	3.600	ok
12	0.130	0.280	ok			
11	0.130	0.280	ok			
10	0.127	0.280	ok			
9	0.124	0.280	ok			
8	0.119	0.280	ok			
7	0.126	0.320	ok			
6	0.117	0.320	ok			
5	0.108	0.340	ok			
4	0.074	0.280	ok			
3	0.058	0.280	ok			
2	0.041	0.280	ok			
1	0.018	0.280	ok			

Table 6: Drift and Displacement Tables

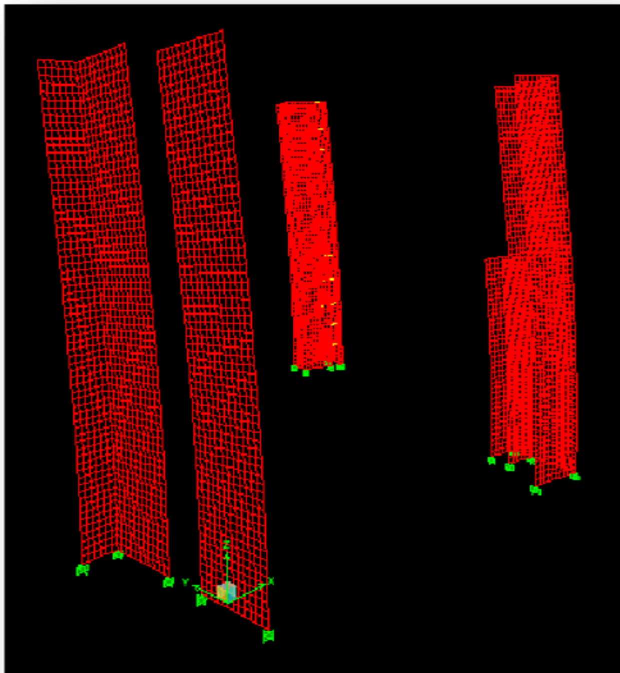

Figure 9: Deflected Shape for Seismic Loading in the Y Direction

Shear Wall Design: Strength

Three design checks were made for the strength design of each wall: flexural strength, shear strength, and boundary element requirements. The results of for each wall are summarized in the tables found in Appendix C.

Flexural Design

Flexural reinforcement was designed by ETABS according to ACI 318-02. ETABS checked wall designs against their respective P-M2-M3 interaction curves, which includes the effective flange widths for walls that intersect as dictated by the ACI Code. Walls 1,2, and 5D were rechecked in PCAColumn for additional gravity load not accounted for the ETABS model. The controlling flexural load was determined from ETABS and the additional gravity load was simply added to the axial load to determine factored forces to be input into PCAColumn. (Note: This approach is an approximation. In the case where the controlling flexural combination involves a tensile axial load, no extra gravity load was considered.)

The remaining walls were designed based on the required reinforcement ratio, , provided by the ETABS shear wall designer. For walls 1,2 and 5D, the PCAColumn interaction diagram is also included with the tables.

Shear Design

Controlling shear combinations were determined from the ETABS output which were used to design the walls. Design procedures were determined in accordance with ACI Code 11.10. According to the code, the nominal concrete shear strength is based on the lesser of

$$V_c = 3.3\sqrt{f_c'}hd + \frac{N_u d}{4l_w}$$

or

$$V_c = \left[0.6\sqrt{f'c} + \frac{l_w\left(1.25\sqrt{f'c} + 0.2N_u/l_w h\right)}{M_u/V_u - l_w/2}\right]hd$$

where N_u is the factored axial load, taken as negative for walls in tension. This value was taken to be the axial force provided by ETABS for that load case, and to it was added the factored axial forces gravity loads, as necessary. With this force, the controlling value of V_c was calculated.

The horizontal steel area required for shear reinforcing, A_v, within a vertical distance of s was determined as

$$A_v = \frac{(V_u - \phi V_c)s}{\phi f_y d}$$

The value d was approximated as $0.8l_w$ (where l_w is the length of the wall) in accordance with ACI Code 11.10.4, since the larger, more precise, value corresponding to the distance from the extreme compression fiber to the center of the force of the reinforcement in tension may only be used when determined by a strain compatibility analysis. The area of steel required was compared to the minimum allowable area required in accordance with ACI Code 11.10.9.2, which limits the ratio of horizontal shear reinforcement to gross concrete area of the vertical section, ρ_t, to 0.0025. Spacing or horizontal steel was limited to 18 inches, per 11.10.9.5.

Finally, the nominal shear strength of the wall was found as

$$V_n = V_c + V_s$$

and

$$\phi V_n \leq V_u$$

which is of the same general form as that used to determine the nominal shear strength of beams. In accordance with 11.10.3, V_n was limited to $10\sqrt{f'_c}hd$. For shear in the walls, ϕ was taken to be 0.75 unless controlled by seismic forces, in which case it was taken to be 0.6.

Boundary Element Design

Walls were determined to require a boundary element where the maximum compressive stress was found to be greater than $0.2\sqrt{f'_c}$, or 0.8 ksi, in accordance with ACI 318 Ch. 21.7.6.3. This stress was computed by considering the effects of controlling combination's factored moment and factored axial force with the equation

$$f_c = \frac{P_u}{A} + \frac{M_u c}{I}$$

Virtually all calculations of maximum compressive stress matched those produced by ETABS.

In sizing the boundary elements, the effective axial load acting on each boundary element, $P_{u,BE}$, was initially calculated by considering both the real axial load and the equivalent axial load caused by the moment according to the following equation:

$$P_{u,BE} = \frac{P_{grav}}{2} + \frac{M_u}{l}$$

where l is the length of the wall (the effective moment arm). However, this approach is conservative as it neglects the amount of axial force carried by the wall. Since Edenwald's walls are located in core shapes, large boundary element dimensions could prove to be problematic at interfaces. To reduce their sizes, $P_{grav}$was recalculated by finding the average axial stress in the wall's cross section and multiplying it by the area of the boundary element.

$$P_{u,BE} = \left(\sigma_{grav} A_{BE}\right) + \frac{M_u}{l}$$

The latter, more accurate, calculation which considers the axial capacity of the wall itself provided significantly smaller effective axial loads for the boundary elements, in some cases up to 50 percent.

The element was then designed according to its behavior as a short column. The maximum allowable axial load on short columns is taken to be

$$\phi P_{n,max} = 0.8\phi\left[0.85f'_c\left(A_g - A_{st}\right) + f_y A_{st}\right]$$

By setting $P_{n,max} = P_{u,BE}$, the area of steel, A_{st}, was determined. This value corresponds to the minimum allowable area of steel to be placed in the boundary element. The required cross sectional area of the boundary element was then calculated based on the limit of 0.06 for the steel to concrete area ratio, ρ. ACI 318 sets the depth of the boundary element as the greater of both $c/2$ and $c - 0.1l_w$. Knowing both the required gross cross sectional area and the depth, the width is easily calculated. c was taken to be $a/.85$ where

$$a = \frac{A_s f_y}{0.85f'_c A_s}$$

The steel in the boundary element was then checked for adequate tensile strength. Using the same process as for compression, the axial tensile load in the boundary element was calculated from the average axial tensile stress, and was added to the equivalent axial tensile load caused by the moment. The maximum tensile force was limited to $0.9A_s f_y$. In many cases, the boundary element reinforcing was controlled by tension due to the little gravity load experienced by the walls. For all walls, the boundary elements were designed with maximum widths of 12 inches, meaning there will be no extrusions which could conflict with architectural designs or create problems in vertical shafts at wall interfaces.

Shear Wall Design Summary

In Table 5, the summary of the shear wall design is listed. More detailed spreadsheets for each wall can be found in Appendix C.

Shear Wall Schedule

Wall	Flexural Vertical Reinf*	Shear Horizontal Reinf*	Boundary Element Length (in)	Boundary Element Width (in)	Boundary Element Reinf
1	#7 @ 12"	#5 @ 18"	14	12	(10) #9
1B	#8 @ 12"	#5 @ 18"	8	12	(4) #9
2	#5 @ 16"	#5 @ 18"	15	12	(8) #10
5A	#8 @ 12"	#5 @ 18"	4	12	(2) #10
5B	#5 @ 12"	#5 @ 18"	--	--	--
5C	#9 @ 12"	#5 @ 18"	14	12	(10) #9
5D	#6 @ 12"	#4 @ 12"	14	12	(10) #9
9A	#7 @ 12"	#6 @ 18"	12	12	(6) #9
9A2	#9 @ 12"	#6 @ 18"	6	12	(2) #9
9B	#7 @ 12"	#6 @ 18"	15	12	(8) #10
9C	#9 @ 10"	#6 @ 18"	8	12	(4) #9
9D	#8 @ 8"	#6 @ 18"	6	12	(4) #9

*Placed in both faces

Table 7: Shear Wall Schedule

As stated above, the flexural steel was designed according to ETABS wall designer and taken as uniform across the cross section of the wall. However, to better understand the available flexural strength using this design for the core comprised of Walls 9A through 9D, each of the core's four wall components were modeled as a unit in PCAColumn with the flexural reinforcing determined from the above design procedure. (See Figure 9) Then the axial loads and moments for each of the 58 load cases were summed (according to the respective axis) and entered as factored loads into PCAColumn. The results can be seen below in Table 6:

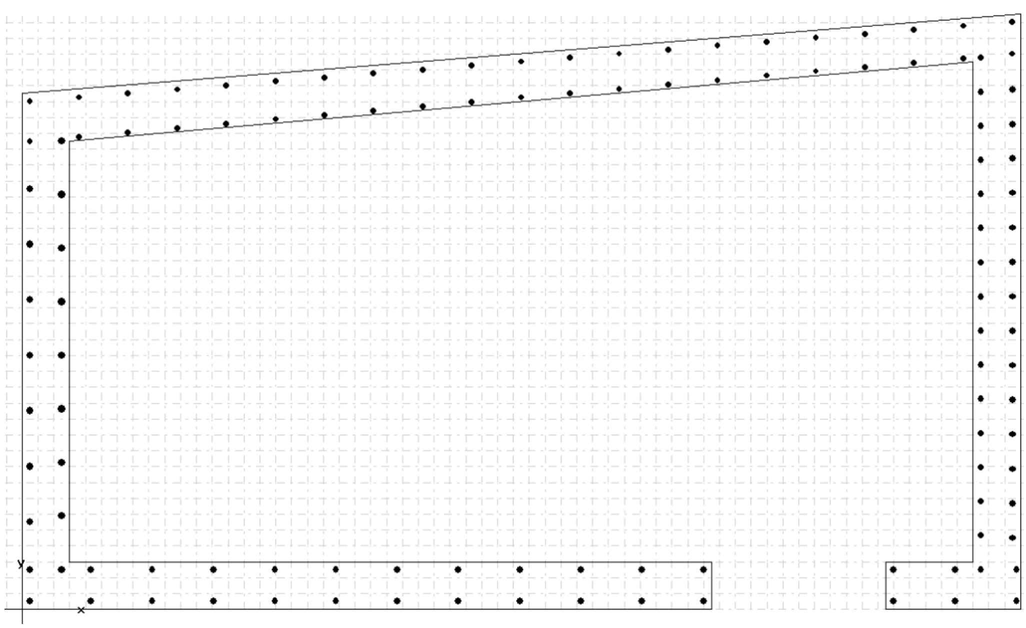

Figure 10: Core 9 Cross Section (Walls 9A-9D)

	Pu	Mux	Muy	fMnx	fMny			Pu	Mux	Muy	fMnx	fMny	
No.	kip	k-ft	k-ft	k-ft	k-ft	fMn/Mu	No.	kip	k-ft	k-ft	k-ft	k-ft	fMn/Mu
1	-1437.9	-143.1	162.7	-21995.6	25014.7	153.738	30	-1012.1	-3728.2	2485	-24070.4	16043.5	6.456
2	-1231	-146.4	214.3	-21714.3	31795.5	148.351	31	-855	967.2	9448.9	4231.9	41342.8	4.375
3	-1162.2	921.7	9530.6	3702.7	38286	4.017	32	-1120.8	3067.9	-733.5	21984.9	-5256.3	7.166
4	-1428	3022.4	-651.8	20148.7	-4345.5	6.666	33	-898.5	815.7	6346.6	5246.8	40821	6.432
5	-1205.7	770.3	6428.3	4526.5	37777.1	5.877	34	-846.2	589.1	7879	3104.7	41525.8	5.27
6	-1153.4	543.6	7960.7	2626.5	38464.9	4.832	35	-1118.8	2480.9	-1903.8	21416	-16434.5	8.632
7	-1426	2435.4	-1822.1	19706.9	-14744.7	8.092	36	-1024.5	2075	855.9	21664.5	8935.9	10.441
8	-1331.7	2029.5	937.5	19904.5	9194.9	9.808	37	-725	-1667.5	7741.4	-9154.7	42501.2	5.49
9	-1032.2	-1713	7823	-8636.7	39443	5.042	38	-1019.7	3072.3	6484.3	16960.3	35795.7	5.52
10	-1326.8	3026.8	6565.9	15649.3	33947.4	5.17	39	-829.7	-1037.2	4226.3	-10151	41361.8	9.787
11	-1136.9	-1082.7	4308	-9625.6	38298.7	8.89	40	-719.7	-1512.1	7448.2	-8647.8	42597.2	5.719
12	-1027	-1557.6	7529.9	-8177.9	39534.9	5.25	41	-1050.9	2520.8	3282.7	20076.1	26143.9	7.964
13	-1358.1	2475.3	3364.3	18468.4	25101.6	7.461	42	-940.9	2045.9	6504.5	12416.9	39476.7	6.069
14	-1248.1	2000.4	6586.2	11121.5	36616.5	5.56	43	-993.7	-1151.2	-9239.7	-5337.2	-42839.1	4.636
15	-1300.9	-1196.6	-9158	-5212.2	-39889.5	4.356	44	-728	-3251.8	942.7	-24731.1	7169.4	7.605
16	-1035.2	-3297.3	1024.3	-23225.5	7215.2	7.044	45	-950.3	-999.7	-6137.4	-7021.3	-43106	7.023
17	-1257.5	-1045.2	-6055.8	-6930.1	-40152.6	6.63	46	-1002.5	-773	-7669.8	-4318.5	-42847.4	5.587
18	-1309.7	-818.5	-7588.2	-4302.5	-39887.3	5.257	47	-729.9	-2664.8	2113	-25829.2	20480.6	9.693
19	-1037.1	-2710.3	2194.7	-24244.3	19631.6	8.945	48	-824.2	-2258.9	-646.7	-23114	-6616.9	10.232
20	-1131.4	-2304.4	-565	-21713.9	-5323.9	9.423	49	-1123.7	1483.5	-7532.2	8193.9	-41601.9	5.523
21	-1430.9	1438	-7450.5	7461.4	-38657.5	5.189	50	-829.1	-3256.3	-6275.1	-18785	-36200.3	5.769
22	-1136.3	-3301.7	-6193.4	-17928.4	-33630.1	5.43	51	-1019	853.3	-4017.1	9037.8	-42548.9	10.592
23	-1326.2	807.8	-3935.5	8133	-39623.7	10.068	52	-1129	1328.1	-7239	7632.4	-41600.7	5.747
24	-1436.2	1282.6	-7157.4	6927	-38654	5.401	53	-797.8	-2704.7	-3073.5	-21740.1	-24703.9	8.038
25	-1105.1	-2750.2	-2991.8	-20443.1	-22238.9	7.433	54	-907.8	-2229.9	-6295.3	-14439.5	-40765.4	6.475
26	-1215.1	-2275.4	-6213.7	-13938.3	-38063.5	6.126	55	-758.4	1398	14189.8	4169.3	42319.7	2.982
27	-1148.7	1359.1	14234	3669.1	38425.6	2.7	56	-1006.3	-1603.4	-13896.4	-4933.2	-42754.9	3.077
28	-1396.6	-1642.2	-13852.2	-4625.8	-39018.3	2.817	57	-1142.9	3484	-2147.3	21446.1	-13218.1	6.156
29	-1533.2	3445.1	-2103.1	19230.8	-11739.9	5.582	58	-621.8	-3689.4	2440.8	-26092.9	17262.2	7.072

Table 8: PCAColumn Output for Core 9

The significance of the PCAColumn output is most readily noticed in the column "fMnu/Mu." This ratio corresponds to the core's available strength to required strength for each loading condition. Load case 27 has the lowest value of 2.7 – thus there is significant additional strength available from the consideration of flanges and also from the uniform reinforcement, since the current design provides more than 2.5 times the required flexural strength. For this report, the reinforcement will not be optimized – but it is recognized that by locating steel near the neutral axis, this design is very conservative. It would be advantageous to locate most of the vertical steel closer to the ends of the wall, and provide minimum steel through the middle.

Detailing

Figures 11 and 12 show typical detailing for shear wall ends. Ties were determined to be No. 3 hoops for all boundary elements, since the largest bars used are No. 10 and ACI 318 Ch. 7.10.5.1 limits bars No. 10 and smaller to No. 3 ties. The spacing of the ties was limited to 12 inches for all walls (the width of the wall) which is the controlling criteria found in 7.10.5.2. Standard 90 degree hooks were then used to further develop the shear strength of the connection at wall intersection.

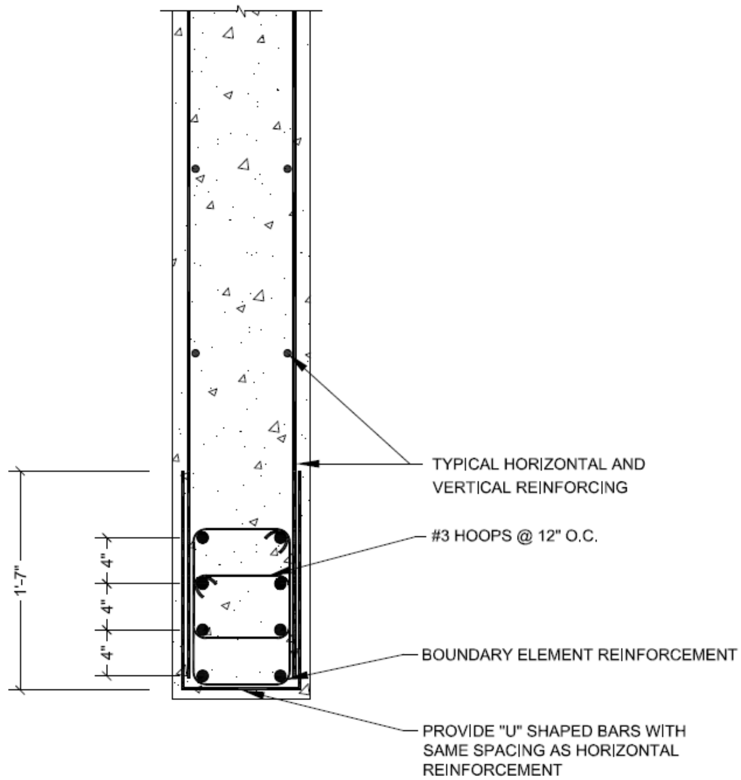

Figure 11: Shear Wall End Detail

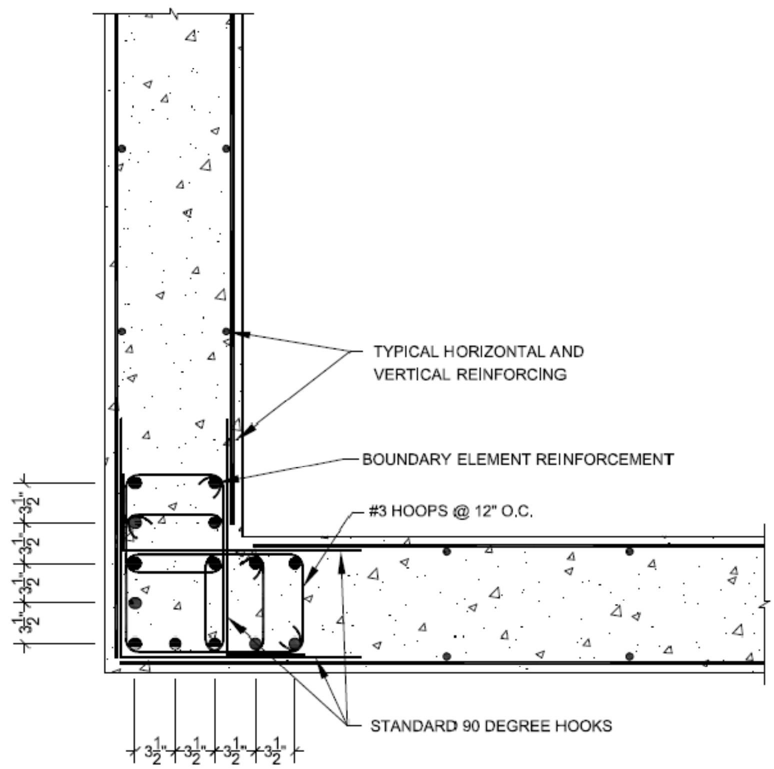

Figure 12: Shear Wall Connection Detail

Coupling Beam Design

The core at the north end of the east wing has door openings at every level which must be considered. Though it is possible to simply treat the pier as a single wall with openings which require special reinforcement at the corners (which is how the original wall was treated), it was determined advantageous to reinforce the concrete above and below the doors to behave as coupling beams. That is because, with the removal of other walls in the building, the cores will obviously take more load than expected in the original design. Their performance, therefore, should be optimized as much as possible.

Coupling beams provide that optimization by means of improved energy dissipation from seismic loading, and they can also increase the relative stiffness of the core. They additionally benefit the structure due to the fact that the ends of the beam experience inelastic rotation and behave as plastic hinges, which is necessary for the two piers to bend as one.

When modeling the beams, each end was extended well into the wall, as seen in the figure. The extended sections were then given an infinite moment of inertia to replicate a fixed end condition. As stated above, coupling beams were modeled with cracked section properties, which increases their flexural strength. As a result, the shear developed in the plastic hinges also increases. Those shear values were determined from the ETABS analysis.

The dimensions of each beam were determined by the depth of each floor, which varies throughout the building, and the thickness of the wall, which is 16 inches.

According to ACI 318 Ch. 21.7.7, beams with aspect ratios *l/h* less than 4 inches "shall be permitted to be reinforced with two intersecting groups of diagonally placed bars symmetrical about the midspan." The size of the bars was selected based on the following equation from 21.7.7.4:

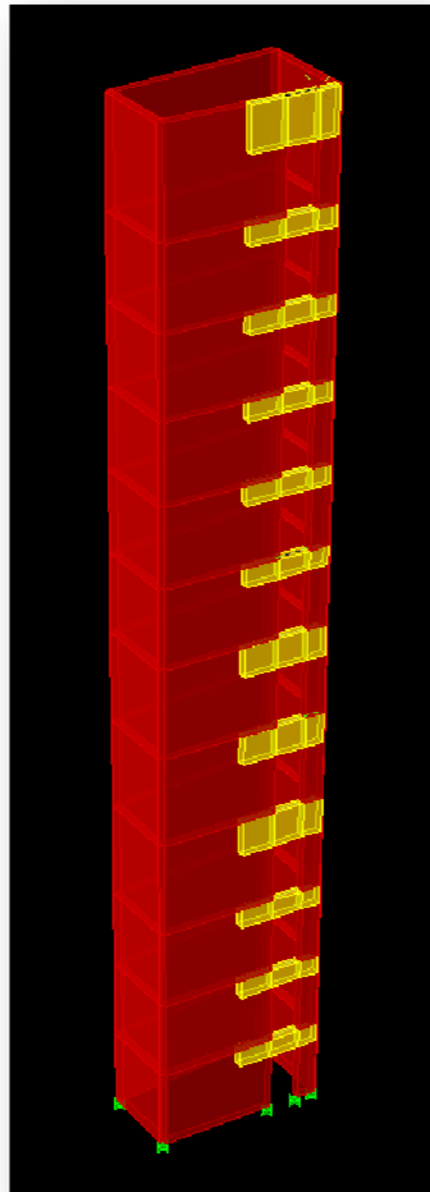

Figure 13: Coupling Beams

$$\phi V_n = \phi 2A_{vd} f_y sin\alpha \ \leq \ 10A_{cw}\sqrt{f'c}$$

so that

$$\phi V_n \leq V_u$$

Here $\emptyset$ is taken to be 0.6. Transverse reinforcement for the diagonal bars was selected according 21.4.4.1-21.4.4.3.

Longitudinal reinforcement was determined according to ACI 318 Ch. 11.8.5, which is the section on deep beams. The spacing of the bars was limited to *d/5*, where *d* was taken to be the depth from the maximum compressive fiber to the bottom bar in tension. The total area of the longitudinal bars, A_{vh}, was then taken to be not less than $0.0015b_w s_2$, where b_w is the 12 inch web thickness and s_2 is the spacing. The shear reinforcement perpendicular to the span was determined according to 11.8.4, and $A_{vh}$was taken to be not less than $0.0025b_w s_1$. The same spacing limitations apply to the vertical steel as for the horizontal.

Table displays the types of beams (the only variant being depth), their locations and controlling shear values.

Beam	Location	V_u	Load Combo
B1	Roof	32.46	321
B2	Story 12	36.02	321
B2	story 11	38.17	321
B2	Story 10	39.08	321
B2	Story 9	39.36	321
B2	Story 8	41.25	42
B3	Story 7	54.45	42
B3	Story 6	55.15	42
B4	Story 5	64.53	42
B2	Story 4	48.31	42
B2	Story 3	53.39	42
B2	Story 2	51.5	42

Table 9: Coupling Beam Forces

Coupling Beam B2

Length (ft)	Depth (ft)	L/D
3.78	2.27	1.67
V_n	ϕV_n	V_u (k)
107.71	64.63	53.39
Diagonal Bar Angle, Degrees	Diagonal Bars, per group	A_{vd} (in^2)
22.00	(4) #7	2.40
Transverse Reinf spacing (in)	Transverse Reinf for Diagonal Bars	A_{sh}, (in^2) req'd per leg
4.00	#6 Hoops	0.39
Vertical Reinf spacing (in)	Vertical Reinf	A_v, (in^2) req'd per leg
6.00	#3 Hoops	0.09
Horizontal Reinf spacing (in)	Horizontal Reinf	A_{vh} (in^2)
8.00	(8) #4	0.14

Table 11: B2 Design Summary

Coupling Beam B3

Length (ft)	Depth (ft)	L/D
3.78	3.60	1.05
V_n (k)	ϕV_n (k)	V_u (k)
102.43	61.46	55.15
Diagonal Bar Angle, Degrees	Diagonal Bars, per group	A_{vd} (in^2)
29.00	(4) #6	1.76
Transverse Reinf spacing (in)	Transverse Reinf for Diagonal Bars	A_{sh}, (in^2) req'd per leg
4.00	#6 Hoops	0.56
Vertical Reinf spacing (in)	Vertical Reinf	A_v, (in^2) req'd per leg
6.00	#3 Hoops	0.09
Horizontal Reinf spacing (in)	Horizontal Reinf	A_{vh}, req'd
7.60	(12) #4	0.14

Table 10: B3 Design Summary

Though four types exist, only the 2 most common beams were detailed for this report, and are seen in Tables 10 and 11. Their details are located on the following page. It should be noted that the transverse reinforcement for the diagonal bars is required to be #6 hoops, which is certainly not ideal. The core walls were made 16 inches thick solely for the purpose of the coupling beams, and given the size of the reinforcement, this design feature seems unsuitable as part of the proposed solution.

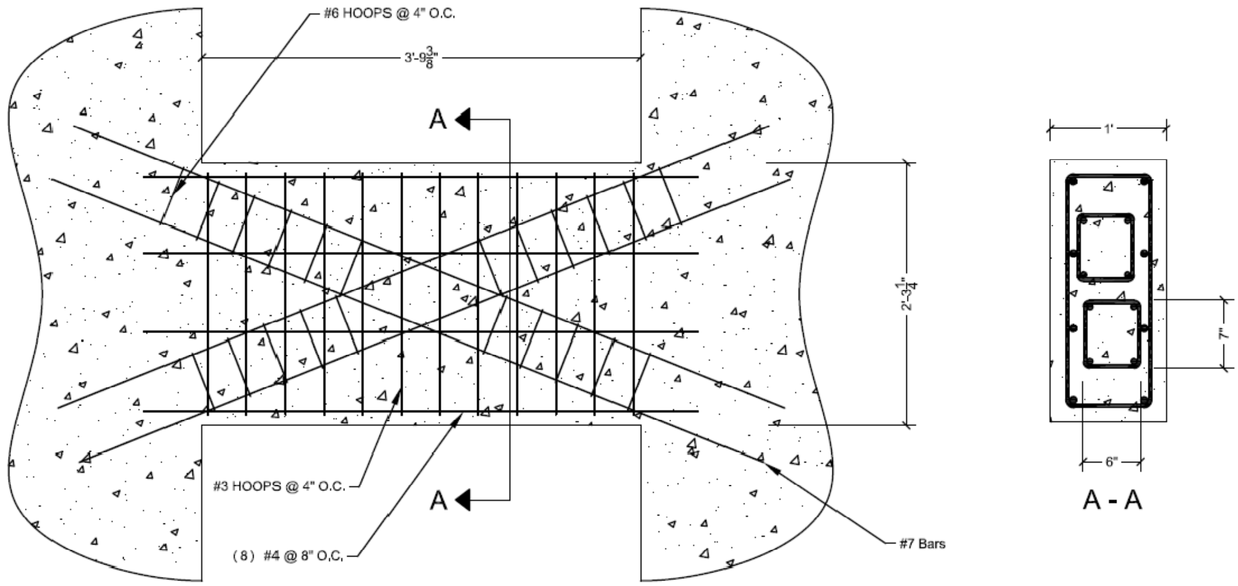

Figure 14: B2 Coupling Beam Detail

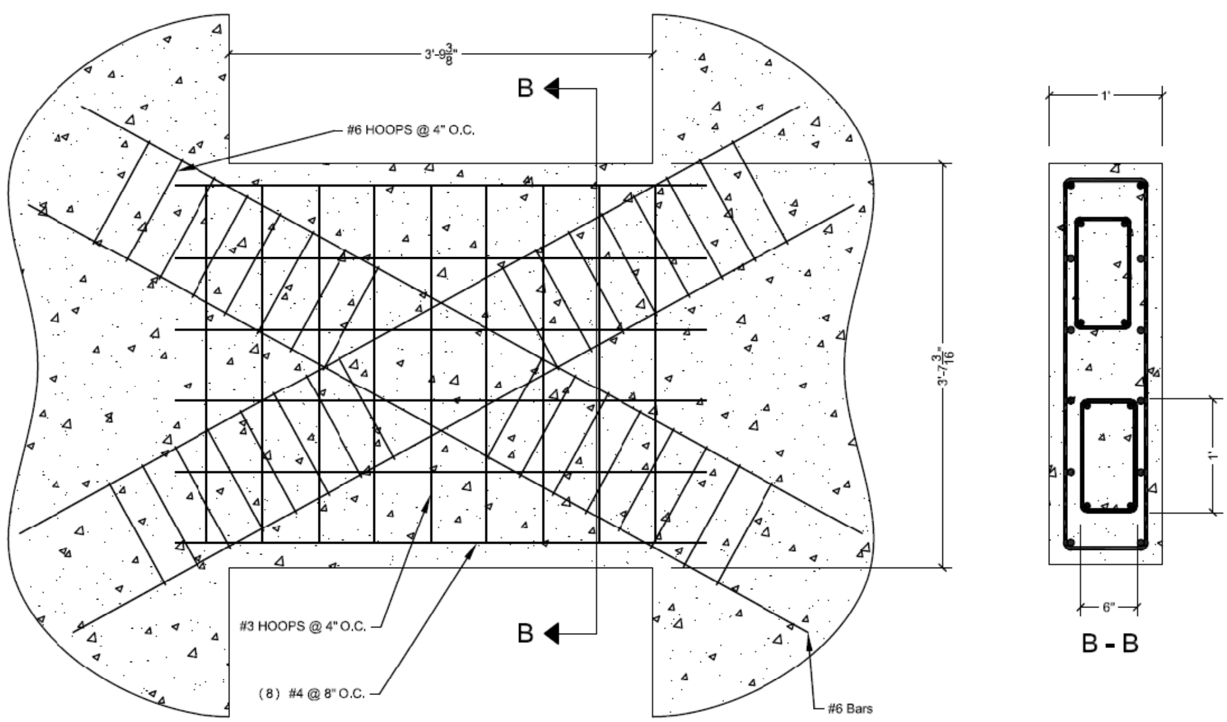

Figure 15: B3 Coupling Beam Detail

Column Design

After removing the shear walls, it became necessary to replace them with columns to carry the gravity load the walls were previously responsible for. The walls which have been removed were conveniently located in a grid of identical bays, as seen in Figure 17. Rather than optimize the columns, it made more sense to simply use the system already in place and check to ensure each has adequate strength to carry the required loads. This would be especially beneficial for the sake of limiting the required formwork.

Only four columns were designed, since several of them have identical loading conditions. Axial loads were calculated for these four columns, and can be found in Appendix D. Then, due to unbalanced moments from unequal span lengths, as seen in Figure 4, moments were calculated in each column. Since the columns are symmetric and maintain the same 28 day compressive strength at the critical sections, the resultant moment in the columns can be expected to be evenly shared between the top and bottom columns, as seen in the figure. In each case, only strong axis bending exists, as the weak axis benefits from equal span lengths.

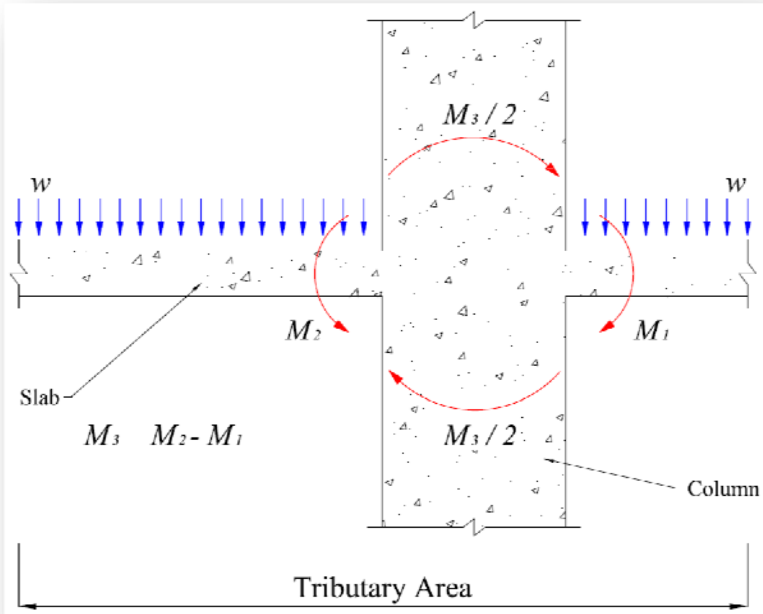

Figure 16: Unbalanced Moments in Column

The program PCAColumn was used to analyze the columns according to ACI 318-02. The base of the column was taken as the critical section for each since, at the base, the axial load was clearly the greatest, and the unbalanced moments proved to be equal or greater at this point than at every other floor.

The service dead and live axial loads as well as the service dead and live moments were entered into the program. The 28 day compressive strength was maintained at 6,000 psi. Confinement was chosen to be #3 ties for the #10 bars and #4 ties for the #11 bars. 1.5 inches of cover was provided to the transverse reinforcement. Then each column was examined for axial-bending interaction.

A summary of the column designs can be seen in Table 3, and the following figures show the axial-moment interaction for each. Appendix D provides the PCAColumn output for column V12, including load combination definitions, which are referenced within the interaction diagrams. All columns were found to be satisfactory. (For formatting purposes, the P-M diagrams have been significantly reduced in size. It was desired to simply communicate the locations of the load combinations as clearly within the interaction curve.)

Column	J1, J7	J3, J5	S12, V6, V12	R12, W6, W12
Size	22"x22"	22"x36"	22"x36"	22"x22"
Rebar	(8) #11	(8) #10	(8) #10	(8) #11

Table 12: Column Sizing and Reinforcement

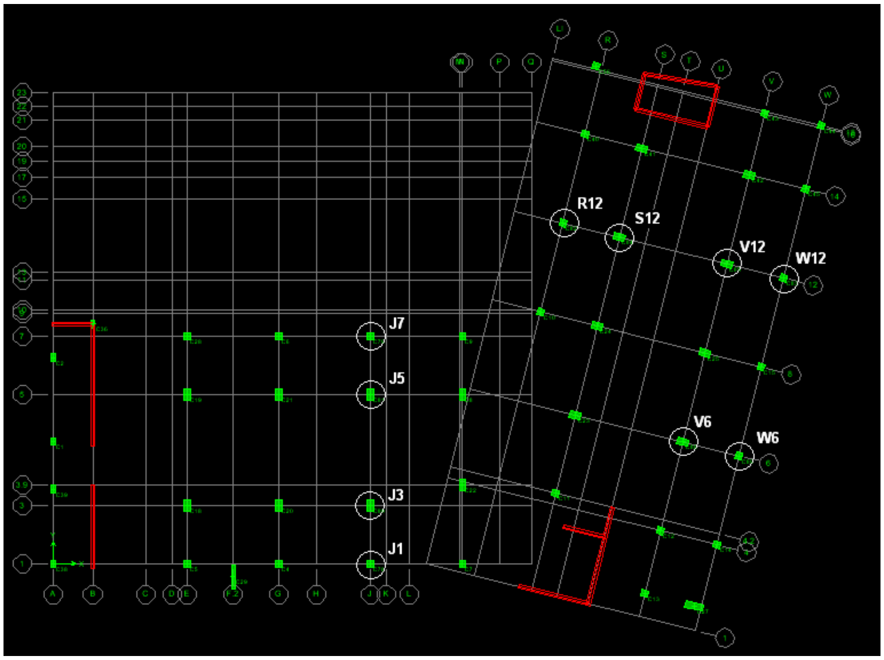

Figure 17: Replacement Column Locations

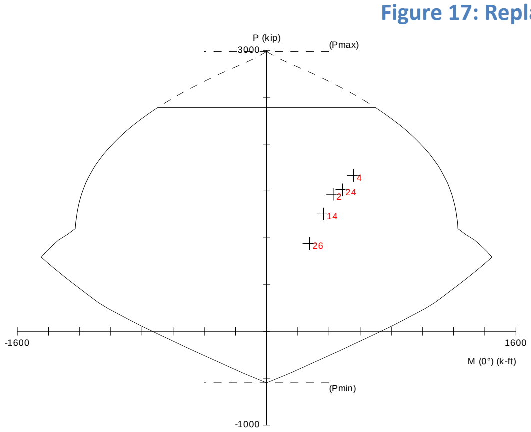

Figure 19: P-M Interaction for Column J1, J7

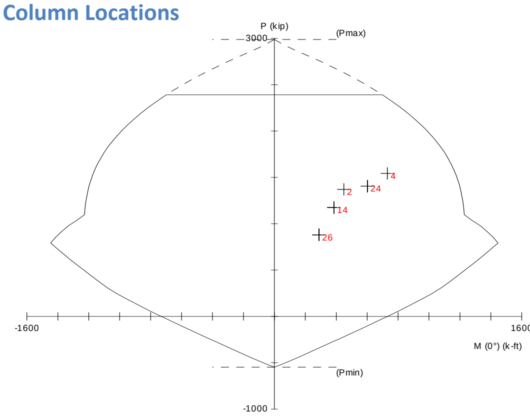

Figure 18: P-M Interaction for Column J3, J5

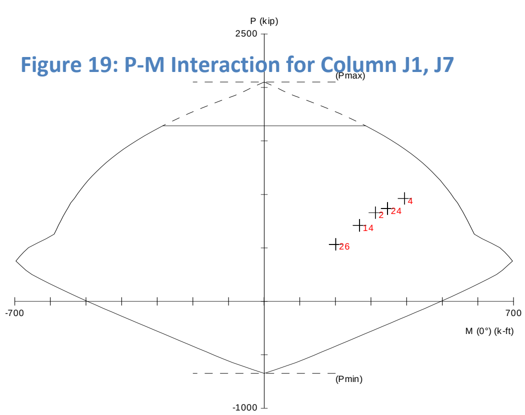

Figure 20: P-M Interaction for Column S12, V6, V12

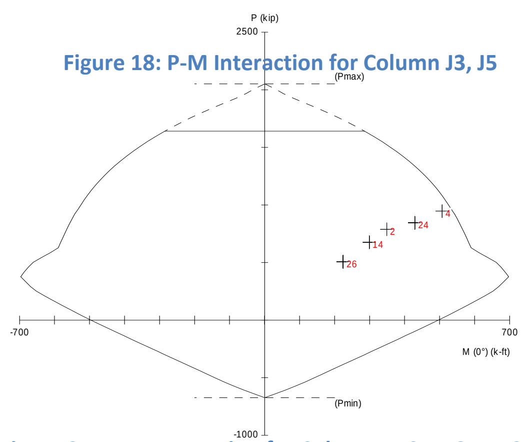

Figure 21: P-M Interaction for Column R12, W6, W12

Foundation Design

The foundations at all wall locations are already designed as large spread footings or mat foundations. It is not expected that they will need to be redesigned, and for the sake of this report they are assumed adequate. However, the columns which replaced the shear walls will need foundation designs. In the above section, column sizing was based off similar columns which have similar loading conditions, largely for the sake of formwork simplification. However, footings do not always require formwork and due to the large amount of material placed in them, it is more sensible to optimize them for realistic loads.

Below are calculations for the footing corresponding to column J5, according to loads developed by hand. Note: The column was placed in ETABS to check for uplift, however the gravity load was more than sufficient to resist both uplift and the moment caused by the lateral forces. Therefore the footing was first sized according to the load bearing capacity of the soil (which is 6 ksf, according to the geotechnical report), and accounted for the effect of the column's moment on reducing the effective area of the footing's contact with the soil. Secondly, the shear strength of the cross section was checked against the controlling shear force in the footing. Reinforcement requirements, and thus the flexural strength, were not considered.

$$P_{service} = P_D + P_L$$
$$= 570 + 152 = 622\ kips$$

$$M_{service} = M_D + M_L$$
$$= 125 + 65 = 190\ ft\text{-}\ kips$$

Assume $1.2B = L$

$$q_a \geq \frac{P}{A} + \frac{M}{S}$$
$$6ksf \geq \frac{622}{1.2B^2} + \frac{190}{\left(\frac{B(1.2B)^2}{6}\right)}$$
$$B \geq 10.1'\ \ Use\ 10.5'$$
$$L \geq 11.4'\ \ Use\ 12'$$

$$e = \frac{M}{P} = \frac{190}{622} = 0.3' \leq \frac{L}{6} = 2'$$

$$L' = L - 2e$$
$$= 12' - 2(0.3) = 11.4'$$

$$\frac{P}{A'} \leq q_a$$
$$\frac{622}{10.5(11.4)} \leq 6\ ksf$$
$$5.2\ ksf \leq$$

Use 10.5 ft x 12 ft footing

$$P_u = 1.2P_D + 1.6P_L$$
$$P_u = 808\ k$$

$$q = \frac{P_u}{A}$$
$$= 6.75\ ksf$$
$$= 46.88\ psi$$

$$v_c = \phi 4\sqrt{f_c'}$$
$$= 0.75(4)\sqrt{3000\ psi}$$
$$= 164\ psi$$

2-way shear stress controls by inspection

$$d^2(4v_c + q) + d(2v_c + q)(b + c) = q(BL - bc)$$
$$d = 21.01"$$

$$h = d + 3" + d_b$$
$$= 20.01" + 3" + 1.27" = 24.28"$$

Use depth of 25 inches.

$$d_L = 25 - 3 - 0.5(1.27) = 21.34"$$

Check wide beam shear

$$V_{uL} = 5.2\left[\frac{13 - 3}{2} - 1.78\right] = 16.74\ k$$

$$\phi V_n = 2\sqrt{f_c'}bd$$
$$= .75(2)\sqrt{3000}(12)(21.34")$$
$$= 21{,}039\ lb$$

$$\phi V_n = 21\ k\ \geq V_u = 16.74\ k$$

Use 11’ x 13’ x 25” footing

The columns of identical size and similar loading conditions to this one which are located in adjacent bays have spread footings which are sized as 14.5’ x 14.5’ x 43”. The significant change in depth could be accounted for by the engineer’s desire to minimize reinforcement, or it may also be related to the geopier design as an intermediate foundation system. However, even those dimensions from the engineer’s design were used in comparison, the footings for the replacement columns would be less than half the volume of the single, much larger wall footing currently being used.

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Cost Estimate

Any changes made to a building design would be paired with some type of cost estimate for the owner's consideration. Therefore, an approximate cost analysis of the changes recommended in this report was generated according to data taken from the RS Means CostWorks website, found at www.meanscostworks.com. For the estimate, the savings generated from the removal of the walls and foundations (to include concrete material, reinforcement, and placement costs) was compared to the costs incurred from the replacement columns, which includes the same parameters. Since formwork for similar columns and walls will already be on-site, all costs associated with formwork were ignored. (Labor for assembling forms should be considered in a more detailed estimate.) Coupling beams and changes in the reinforcement for the remaining walls were also not considered, though a truly accurate cost analysis should include them as well.

The following is a breakdown of the savings in removing the shear walls:

Concrete for walls
$106.32/C.Y. for 5000 psi concrete @ 998 cubic yards = $106,107
Wall placement with crane and bucket for 12" walls
$104.07/C.Y. + $1.99*8 [additional costs per story for crane and bucket placing over 5 stories] = $120/C.Y. @ 998 cubic yards = $119,760
Reinforcement [for over 100 ton job (includes material, labor)][2]
#3-#7: $1996.7/ton + $50.35/ton [crane cost for handling] = $2047.05/ton @ 21 tons = $40,940
#8-#18: $1726.86/ton + $50.35/ton [crane cost for handling] = $1777.21/ton @ 26.5 tons = $47,096
Spread Footing for 5+ C.Y. [includes material, placement, reinforcement]
$235.39/C.Y. @ 1451 C.Y. = $341,550
Gross Savings (approximate) = **$655,453**

The following is a breakdown of the incurred costs from the replacement columns:

Concrete
$106.32/C.Y. for 5000 psi concrete (floors 1-6) @ 215 cubic yards = $22,858
$121.38/C.Y. for 6000 psi concrete (floors 7-roof) @ 303 cubic yards = $36,778
Column placement with crane and bucket for 24" column
$59.77/C.Y. + $1.99*8 [additional costs per story for crane and bucket placing over 5 stories] = $75.69/C.Y. @ 518 cubic yards = $39,207
Reinforcement
#8-#18: $1726.86/ton + $50.35/ton [crane cost for handling] = $1777.21/ton @ 23.3 tons = $41,409
Spread Footing for 5+ C.Y. [includes material, placement, reinforcement]
$235.39/C.Y. @ 498 C.Y. = $117,224
Gross Costs (approximate) = **$140,252**

[2] Note: Existing reinforcement tonnage for each wall was calculated based on #5 bars @ 16" o.c. each way, as well as approximating boundary elements to have (20) #10 bars total.

The net approximate saving is thus estimated to be **$515,201.** While certainly to the benefit of the owner, this savings value is still not very large considering the scope of the project is $52 million. Still, at 1 percent, the owner(s) of a project on a tight budget would probably find this appealing.

Conclusions

Designing the shearwalls according to ASCE 7-05 allowed for a 20 percent reduction in seismic base shear, from 997 kips to 793 kips. In turn, a number of walls were found to be unnecessary, and a total of five were eliminated. The remaining walls were reinforced according to updated forces, and most of them included boundary elements. It is not easy to say whether seismic or wind controlled in the revised design of the Edenwald Tower, since different piers were controlled by different load combinations at different levels. It is sufficient to say that both loading conditions needed, and were given, adequate consideration.

The failure to resolve the issue of torsion is disappointing, considering the inherent torsion in some walls was seen to be up to 20 percent of the total shear. However that issue could only be resolved by a complete overhaul of the architectural design, since the cores of the building dictated nature of the structure's rigidity. That is not, however, to suggest an overhaul would have been appropriate. Engineering should serve the purposes of architectural design, as it is not ideal to construct a building where the design is dictated by unnecessary engineering constraints. It is simply being noted that from a structural perspective, the walls could have been further optimized if the architect was able to find a solution for the client which allowed for less eccentricity between the center of mass and center of rigidity.

Having completed the design, it is also the conclusion of this report that openings in the shear walls would be best handled as originally designed by the engineer. The use of coupling beams is *not recommended*. Though not included in the sections above, when the model was run without the coupling beams, there seemed to be little change in the behavior, namely the deflection, of the core. The placement of the reinforcement for coupling beams would be both highly intensive and expensive. Additionally, the entire core was thickened on the account of the beams alone, which is also expensive and probably not worth the money.

Considering the fact that this design would allow for an estimated $515,201 in savings for the owner, it is considered successful. The savings in cost would be complemented by savings in space within the floor plans. While more walls were reduced than anticipated, it should be noted that, according to RGA, there was considerably little time given to the engineer to design the system. This report had the luxury of many months to refine and optimize a single component of the building. Conversly, the engineer had a number of weeks to design an entire structure, to include more than just the lateral system.

LIGHTING BREADTH

Overview

Edenwald provides both independent and assisted living opportunities to the elderly. The addition being built includes 2 floors dedicated to assisted living apartments. The purpose of this breadth is to determine if the lighting provided in the circulation areas of these floors can be developed to better suit the needs of the elderly.

According to Linda Sanford, a lighting consultant based in Palo Alto, California, there are several things to keep in mind when designing the lighting for spaces used by the elderly.[3] There is, first of all, simply a greater need for light than for spaces used by younger people. The aging process has a detrimental effect on the eye, as pupil sizes reduce and the lens yellows and thickens. The result in both cases is that less light is transmitted to the retina. Thus, the elderly need higher light levels than do younger people.

A second issue is glare. As the lens of an older person eye thickens, it causes light to "scatter" across the retina. So while there is a need for higher lighting levels, light which is too bright can cause irritating or problematic glare. Indirect lighting is ideal in these situations, and Edenwald currently utilizes two coves per floor, which offer tremendous light levels without exposing residents to glare, which can be seen in images below.

The other two issues are contrast and color. As the ability to discern contrast decreases, the need to enhance contrast on edges such as corners and stairs increases. Furthermore, color recognition also deteriorates with age, making the need for lamps with adequate color rendering indexes essential.

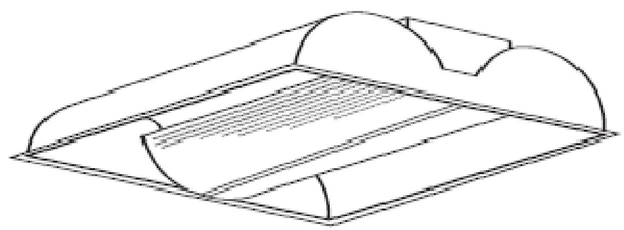

Figure 22: Day-Brite 2'x2' Split Basket

Existing Conditions

The corridors of the floors in question, 5 and 6, utilize primarily 2'x2' recessed split baskets, as seen in Figure 27. While these luminaires are ideal for their glare-free illumination, the ceilings are significantly low (only 7'-10" above the floor) and do not provide the necessary height for the light distribution to properly develop, which is noticeable in the tighter spaces which were allotted less luminaires. In two central areas of each floor exist large coves which provide indirect lighting through the use of 36 to 44, 17-watt T8 flourescent lamps rated at 1300 lumens per lamp. These coves provide excellent illumination, however sheer number of lamps is not ideal, nor is the fact that the cove is recessed through the ceiling to the bottom of the floor slab. (The slim envelopes provided for contractors to run the MEP systems on these floors have provided numerous problems during construction, and the presence of the coves have made this issue even worse.) The central cove seen in Figure 24 seems awkwardly placed with the column located off-centered within. Additionally, much of the light from the cove is directed around the column, where traffic is unlikely.

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[3] Linda Sanford, "The Importance of Lighting for the Elderly," Lighthouse International. Taken March 2008 from:
http://www.lighthouse.org/medical/the-importance-of-lighting-for-the-elderly/

Wall sconces are located in the lobby area, however they are more than 4 inches deep and thus ADA requirements restrict their height to a minimum of 6'-8". Shallow, 8-inch recessed, compact flourescent downlights are provided in selected areas, as well as limited 6-inch compact fluorescent open wall washers. The general lighting plan can be seen in Figure 23, and the lighting schedule for the luminaires in this space is summarized in Table 13.

Luminaire	Description		Lamp				
Type	Manufacturer	Catalog Number	Qty	Type	Watts	Init Lumens	Mounting
Downlights	Cooper Lighting	C8526	2	PL	26	1800	Recessed/Ceiling
Baskets	Day-Brite	2AVG2CF4ORWA12125	2	PL	38	3150	Recessed/Ceiling
Wall Washer	Cooper Lighting	C226	2	PL	26	1800	Recessed/Ceiling
Wall Sconce	Beta-Calco Inc	608126	1	PL	26	1800	Surface/Wall

Table 13: Luminaire Schedule for 6th Floor Corridor

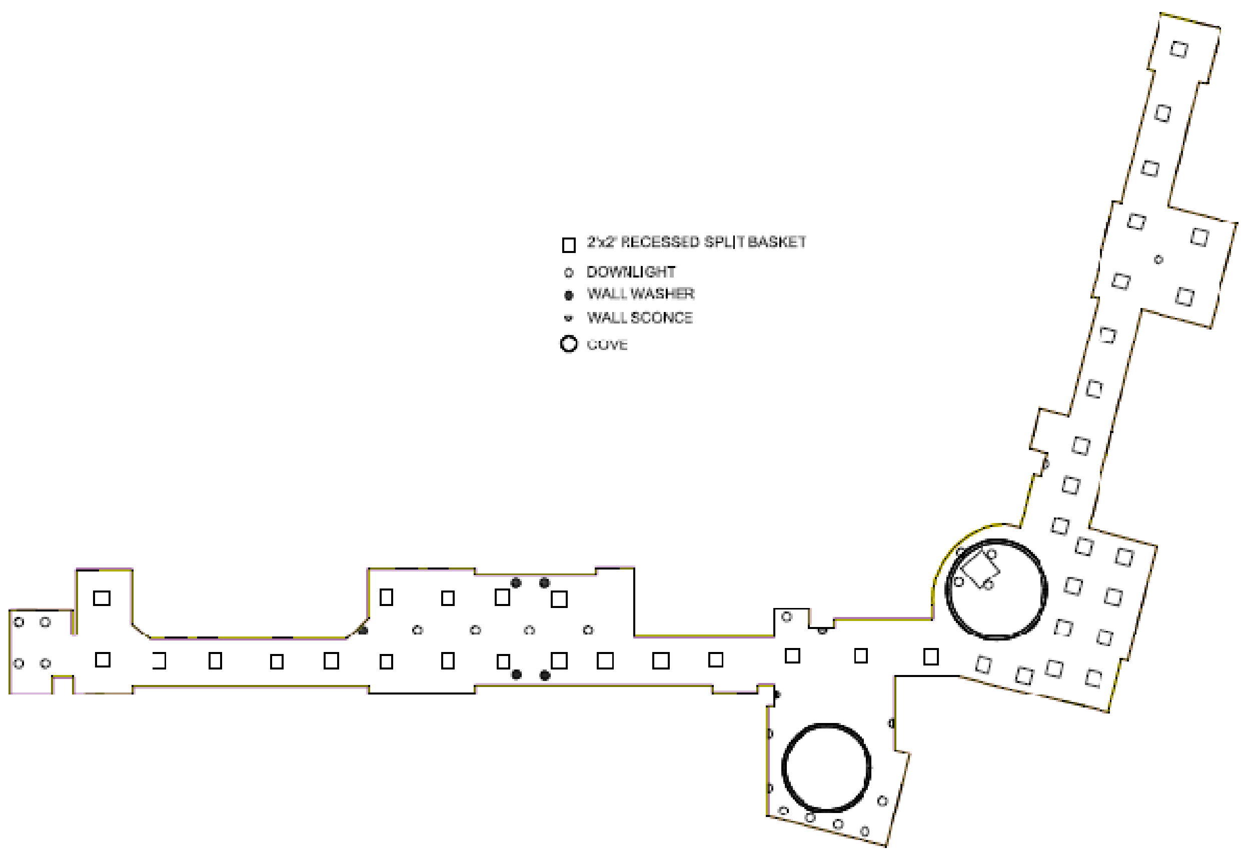

Figure 23: 6th Floor Corridor Existing Lighting Plan

Analysis

According to Table 3-3 of IESNA Recommended Practice-28-08 (Lighting and the Visual Environment for Senior Living), the controlling horizontal illuminance category for hallways during active hours is D, which corresponds to 30 footcandles. To determine the actual light levels provided by the existing design, a 3-D model of the corridor was created in AutoCAD, which was then imported into the lighting program AGI. IES files for each luminaire were downloaded from the manufacturer's websites and imported into the model to produce accurate light output. Surface colors were assumed, while surface reflectances were chosen based off typical ranges. The program was then run to both render the corridor and produce a 1'x1' calculation grid of selected surfaces, which yielded the footcandle value at each point. Renderings for sample spaces can be seen in the following figures.

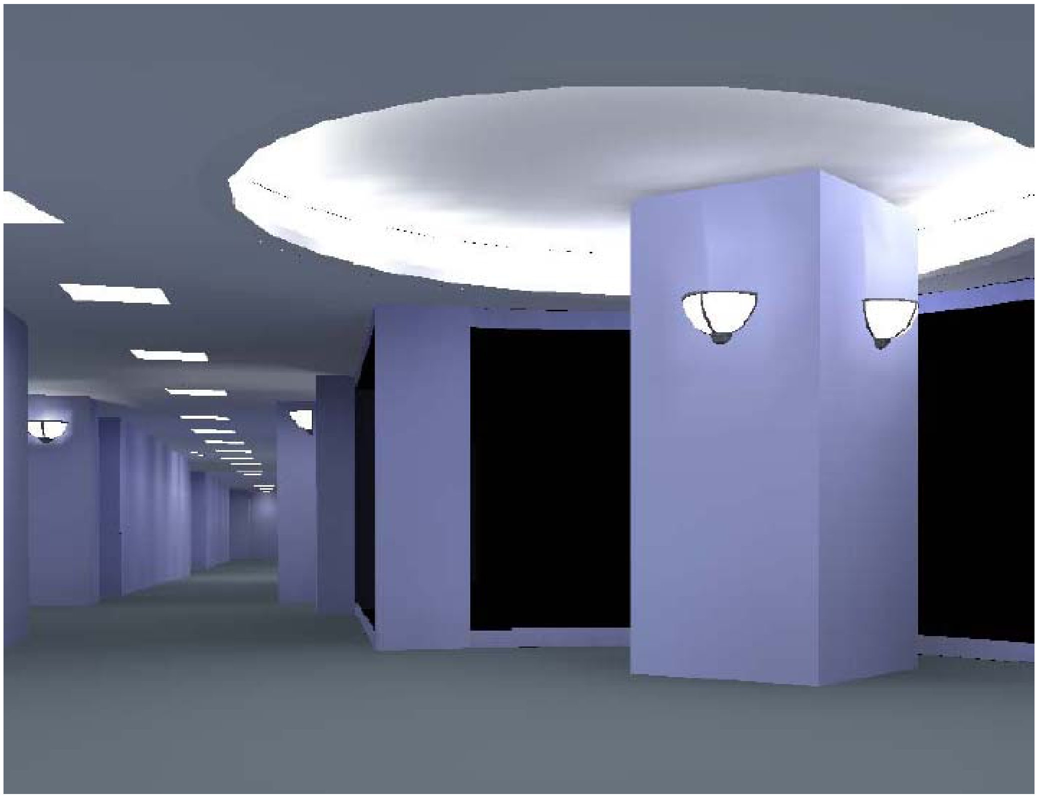

Figure 24: Existing Conditions Central Cove Rendering

Figure 25: Existing Conditions Lobby Cove Rendering

Figure 26: Existing Conditions Typical North East End of Corridor Rendering

Figure 27: Existing Conditions East Wing Rendering

As suspected, the coves provided more than adequate light levels, with the floor area registering up to 65 footcandles. However, while the central regions of the corridors averaged around 30 footcandles, the peripheries suffered slightly, particularly the North East wing, with reductions down to 13-15 footcandles. This is significant, because the reduction is occurring at apartment entrances where visibility is greatly desired. Ideally, traffic routes would be continually and uniformly lit. Figure 26 clearly displays decreased light levels at one apartment's entrance.

Revised Design

After evaluating the goals, criteria, and analyses results thus far discussed, several changes seemed reasonable. First, ADA wall sconces (those with depths not exceeding 4 inches) were added outside each apartment unit, with mounting heights lowered to 4'-6". These also replaced all existing wall sconces to allow for more appropriate mounting heights, not previously afforded by the larger fixtures. Though light output from these elements will not be significant enough to change the illumination at the floor level, they will significantly improve the amount of attention each entrance commands. The improved effect can be seen in Figure 31.

The central cove was abandoned for several reasons. The power and light it generated were not as necessary as in the south-east cove, where there is a seating and reading area. Rather, the central cove is located at the intersection of two hallways. To save energy, it was replaced with the same 2'x2' split baskets and recessed downlights found elsewhere in the lighting plan. Wall washers were used to highlight the column so the wall sconces could be removed, with downlights being placed behind the column in alignment with the glass window. The changes can be seen in Figures 29.

Lastly, the North East wing had an additional 2'x2' basket added to better illuminate the corridor in that area, which allowed for the illumination at floor level to increase from 18 footcandles to 32 footcandles, meeting IESNA standards. The improved effect can be seen in Figure 32.

Conclusions

The existing lighting design had a power density of 1.720 watts per square foot. The revised design saw an increase to 1.96 watts per square foot (the increase largely being a result of the wall sconces being placed outside every apartment entrance). Both of these values are incredibly high, as code limitations for lobbies are 1.3 watts per square foot and for corridors are 0.5 watts per square foot. (These values were taken according to LEM-1-05: IESNA Recommended Procedures for Determining Interior and Exterior Lighting Power Allowances.) Yet, several things must be noted when considering the power densities. First, the significantly low ceilings require some sort of increased height over the gathering, seating and lobby spaces to prevent a cave-like feeling. That issue, combined with the need for a minimum of 30 footcandles across the floor and the need for as little glare as possible leaves few alternatives to the energy burning coves. Furthermore, the power density issue could be resolved by considering the building as a whole, rather than considering spaces independently. There are only two floors of assisted living, and those two are the only floors with cove lighting. Thus, it is likely that if all the spaces in the building with power densities under the maximum allowed were considered, the assisted living corridors could be justified as 'borrowing' power from those spaces with power to spare. Given the unique demands on such a unique space, and the fact that they were obviously justified by the engineer on record, at least one of the coves will remain a part of this design.

The ADA wall sconces are not a necessity, and could be argued against, but considering only 2 floors of assisted living exist in the building, they could prove to be worthwhile considering the aid it could give to directing the residents to their dwellings.

Ultimately, the existing design is on the whole a satisfactory and suitable solution for the conditions of the space (aside from the aforementioned power density issues), as the recommended 30 footcandles of illumination is provided throughout the entire corridor, save a few corners and the end of the North East wing. Improvements in the wall sconces and redevelopment of the central area located at the joint of the hallways could be argued for, and have been, but the increased power density might also make strong cases against the design proposed in this report.

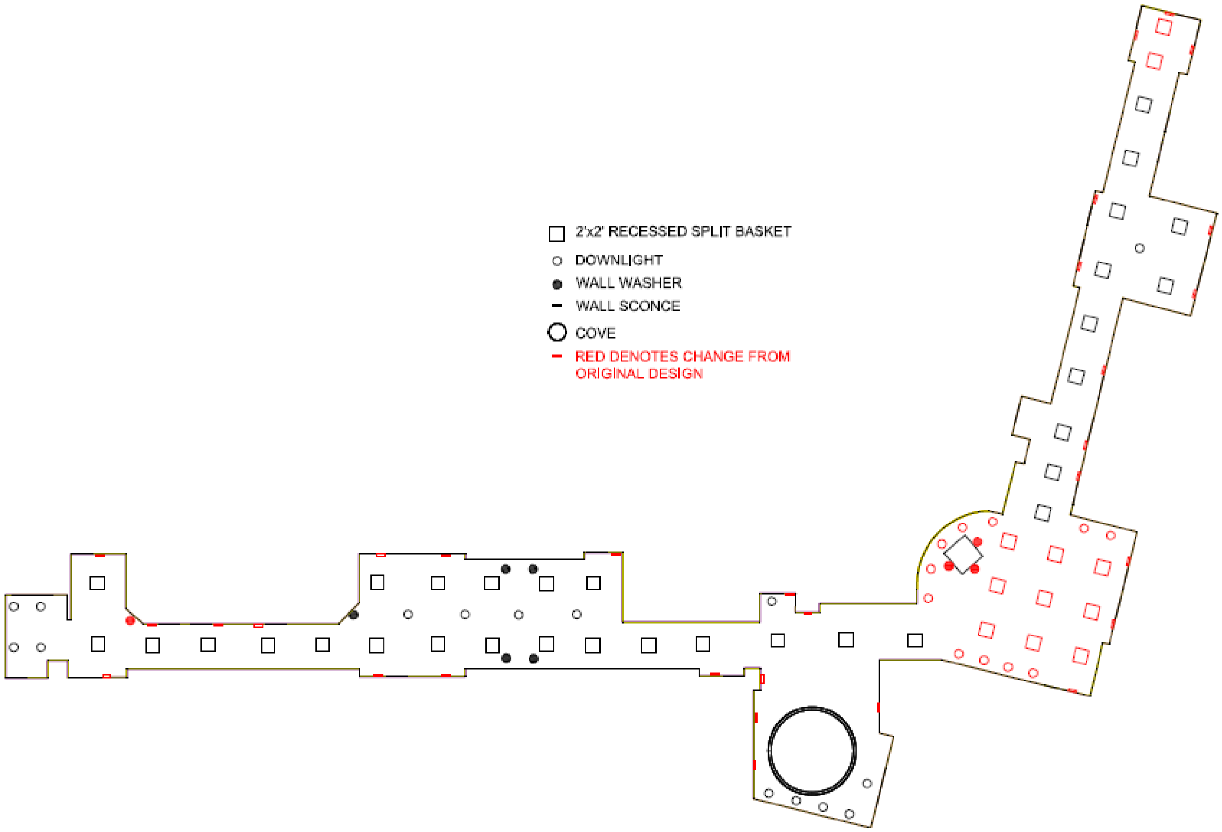

Figure 28: Revised Lighting Plan (Changes Made in Red)

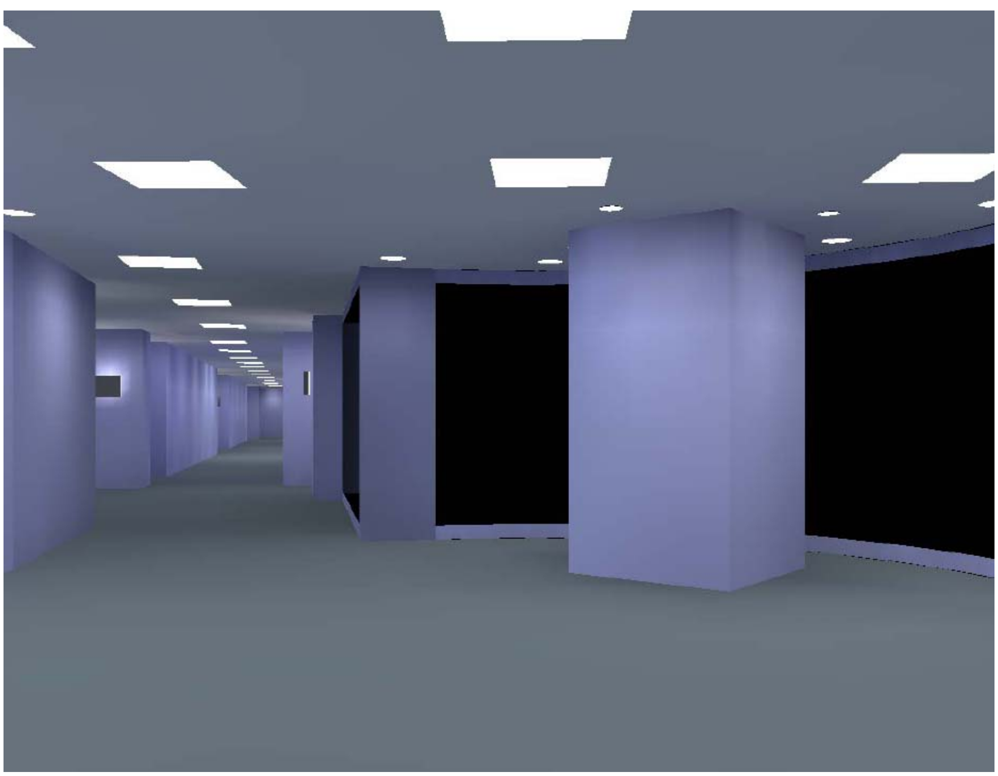

Figure 29: Redesigned Central Cove Rendering

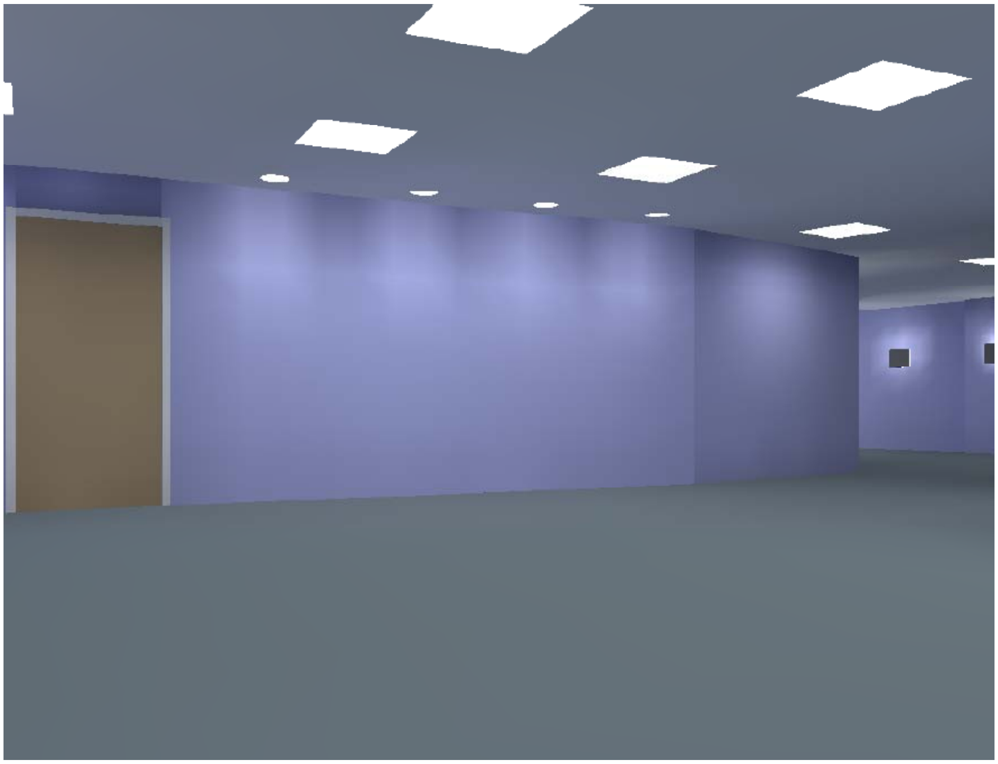

Figure 30: Redesigned Central Cove Rendering

Figure 31: Redesigned East Wing with ADA Wall Sconces Rendering

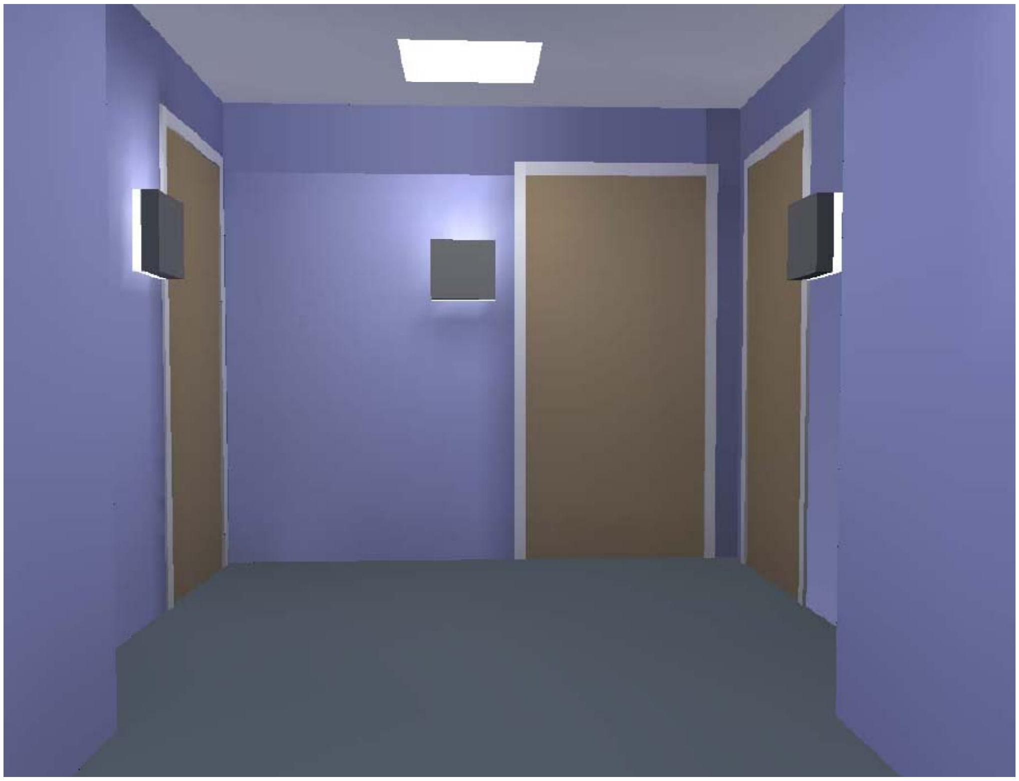

Figure 32: Redesigned Improved Illumination of North East Wing Rendering

ACOUSTICS BREADTH

Overview

The chapel located on the first floor of the Edenwald Tower (see Figure 33) was subjected to an acoustical analysis to see how the space performed with respect to both reverberation time and also to determine the sound transmission class (STC) of the partition between the chapel and the corridor.

Because the space is to be used by the elderly, reverberation times are critical, as the ability to hear decreases with age. The target reverberation time for the chapel was taken to be anywhere between 0.8 and 1.4 seconds. Though this range is on the lower end for churches and worship spaces, it is appropriate given the nature of the likely occupants.

STC can be described as the decibel (dB) reduction of sound passing through a partition. Though typically calculated by determining the sound attenuation at 16 frequencies, this report will consider only the six most common.

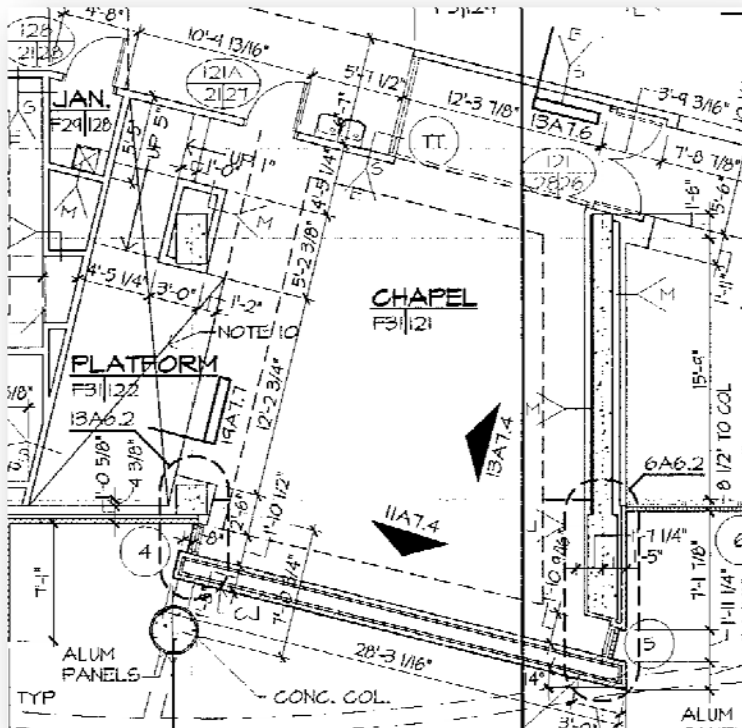

Figure 33: Chapel Plan

Reverberation Time

The chapel has hardwood floors which rest on 2”x3” wood sleepers, providing an air gap between the floor and the concrete slab. The walls surfaces are comprised of Pyrok Acoustement 40 Plaster finishing, 1/4” double paned glass, brick veneer and wood paneling, while the ceiling system is made of 5/8” gypsum wall board. To perform the reverberation analysis, absorption coefficients of each surface for the central frequencies were found from both manufacturer’s websites as well as reference data taken from acoustical textbooks and manuals and entered into a spreadsheet. The reverberation time of the space was then calculated for each frequency according to the equation

where is the absorption coefficient. The calculation was performed twice to account for different occupancy conditions. The first was based off an assumption of 60 percent of the floor area being taken up by an audience in upholstered seats, while the second was based off the assumption of 20 percent of the floor area being taken by an audience in upholstered seats. The results can be seen below.

CHAPEL Reverberation Analysis														Volume = 12218 ft^3	
60% of floor as congregation			Sound Absorption Coefficient, α						Sα						
Surface	Material	Area (sf)	125	250	500	1000	2000	4000	125	250	500	1000	2000	4000	
Walls	Brick	446.7	0.03	0.03	0.03	0.04	0.05	0.07	13.4	13.4	13.4	17.9	22.3	31.3	
Walls	Wood Sheathing	285.2	0.1	0.11	0.1	0.08	0.08	0.11	28.5	31.4	28.5	22.8	22.8	31.4	
Walls	Pyrok Acoustement 40 Plaster, 1/2"	1299.8	0.01	0.2	0.43	0.68	0.75	0.8	13.0	260.0	558.9	883.9	974.9	1039.9	
Walls	Glass Pane, 1/4"	148.8	0.15	0.05	0.04	0.03	0.02	0.02	22.3	7.4	6.0	4.5	3.0	3.0	
Walls	Wood Doors	42.0	0.19	0.14	0.09	0.06	0.06	0.05	8.0	5.9	3.8	2.5	2.5	2.1	
Floor	Wood Platform with Airspace (40%)	617.7	0.4	0.3	0.2	0.17	0.15	0.1	247.1	185.3	123.5	105.0	92.7	61.8	
Floor	Audience in auphostered seats (60%)	926.5	0.39	0.57	0.8	0.94	0.92	0.87	361.3	528.1	741.2	870.9	852.4	806.1	
Ceiling	5/8" GWB Ceiling	1494.0	0.11	0.11	0.05	0.06	0.04	0.05	164.3	164.3	74.7	89.6	59.8	74.7	
Ceiling	Glass Pane, 1/4" (sky lite)	50.2	0.15	0.05	0.04	0.03	0.02	0.02	7.5	2.5	2.0	1.5	1.0	1.0	
								ΣSα =	865.5	1198.3	1552.0	1998.6	2031.3	2051.1	
Target Reverberation Time = 0.8-1.4 Seconds								T_{60} =	0.706	0.51	0.394	0.306	0.301	0.298	

Table 14: Chapel Reverberation Analysis, 60% Occupancy

CHAPEL Reverberation Analysis														Volume = 12218 ft^3	
20% of floor as congregation			Sound Absorption Coefficient, α						Sα						
Surface	Material	Area (sf)	125	250	500	1000	2000	4000	125	250	500	1000	2000	4000	
Walls	Brick	446.7	0.03	0.03	0.03	0.04	0.05	0.07	13.4	13.4	13.4	17.9	22.3	31.3	
Walls	Wood Sheathing	285.2	0.1	0.11	0.1	0.08	0.08	0.11	28.5	31.4	28.5	22.8	22.8	31.4	
Walls	Pyrok Acoustement 40 Plaster, 1/2"	1299.8	0.01	0.2	0.43	0.68	0.75	0.8	13.0	260.0	558.9	883.9	974.9	1039.9	
Walls	Glass Pane, 1/4"	148.8	0.15	0.05	0.04	0.03	0.02	0.02	22.3	7.4	6.0	4.5	3.0	3.0	
Walls	Wood Doors	42.0	0.19	0.14	0.09	0.06	0.06	0.05	8.0	5.9	3.8	2.5	2.5	2.1	
Floor	Wood Platform with Airspace (80%)	1235.4	0.4	0.3	0.2	0.17	0.15	0.1	494.1	370.6	247.1	210.0	185.3	123.5	
Floor	Audience in auphostered seats (20%)	308.8	0.39	0.57	0.8	0.94	0.92	0.87	120.4	176.0	247.1	290.3	284.1	268.7	
Ceiling	5/8" GWB Ceiling	1494.0	0.11	0.11	0.05	0.06	0.04	0.05	164.3	164.3	74.7	89.6	59.8	74.7	
Ceiling	Glass Pane, 1/4" (sky lite)	50.2	0.15	0.05	0.04	0.03	0.02	0.02	7.5	2.5	2.0	1.5	1.0	1.0	
								ΣSα =	871.7	1031.6	1181.4	1523.0	1555.7	1575.5	
Target Reverberation Time = 0.8-1.4 Seconds								T_{60} =	0.701	0.592	0.517	0.401	0.393	0.388	

Table 15: Chapel Reverberation Analysis, 20% Occupancy

It is clear that, in both cases, the reverberation times are considerably lower than even the conservative limits established above. In essence, this is very positive for the sake of sound clarity, however it is likely the music played in the chapel will suffer in quality, since resonance will not properly develop.

STC

The second analysis performed was to determine the sound transmission class of the wall separating the chapel from the corridor. According to Mehta, the recommended STC class for a wall separating a music room or theater from a corridor or lobby is 60. Since the chapel is similar in function to that of a music room, and similar in sensitivity to that of a theater, this is an adequate recommendation for Edenwald's chapel.

Several different materials make up the corridor wall, and all were used in computing the overall STC. The wall itself is comprised of 5/8" GWB on each side of 3-5/8" metal studs spaced at 24" on center. Additionally, within the wall there is 3 inches of fiberglass acoustical insulation. A glass surface, consisting of two 1/4" panes separated by a 1/2" air space, was incorporated as a curtain wall between the two spaces. There are also 3 acoustical, wooden doors. Using sound transmission loss data for these surfaces, a composite STC of the wall was found by selecting the maximum value which satisfies standard requirements limiting the maximum deviation between the transmission loss and the STC to 8 dB, and the total deviation to 32 dB. According to this procedure, the STC of the wall is 46, well beneath the recommended 60, as seen in Table 17. (More detailed calculations are available upon request.)

To meet the desired STC rating, several changes would be required. First, it must be understood that the transmission loss of a wall is going to be largely dictated by its weakest element. Therefore, the windows and doors must either be eliminated or improved to raise the STC to the value of at least the wall.

All three elements of the partition were changed to raise the STC. First, the Marshfield acoustical doors (STC 41) were changed to Krieger acoustical doors (STC 53). Then the glazing was changed to allow for a larger air space of 4 inches, with one pane being laminated, to raise the STC to 49. Lastly, the wall was changed so that 2 layers of 5/8" GWB were placed on each side of the studs, one side being mounted with a 1/2" resilient channel to improve the sound transmission loss through the mounting. The 3 inches of acoustical insulation remained as originally designed. This combination gave an STC rating of 56, as calculated in Table 18.

The following equations were used to develop the STC rating for each wall.

$$\tau = 10^{-0.1TL}$$

$$TL_{composite} = 10\log\frac{1}{\frac{\tau A}{\sum A}}$$

Conclusions

The reverberation times are more than sufficient to reduce unwanted noise for the sake of the elderly. While the argument could be made that the quality of music could suffer from such an acoustically "dead" space, the ability for the residents to understand speech in this space takes precedence, and the existing conditions are deemed sufficient.

The partition separating the chapel from the corridor is recommended to have an STC rating of 60. The current design provides 43, suggesting that changes would be appropriate and advantageous. Those changes, listed above, raised the STC to 56, which is still short of the goal but a significant improvement. The changes in the wall are minimal, involving just two more layers of GWB on each side, and the addition of a resilient channel to improve the quality of the GWB mounting on one side. However, the changes to windows are more significant, with the air space between panes now being set at 4 inches. Additionally, one of the panes would need to be laminated. But perhaps the most significant are the changes to the doors. The improved selections would be much heavier, would likely cost the owner more money. That being said, it would be the recommendation of this report to make the change, so long as the owner approved any additional incurred costs.

Composite Transmission Loss																	Existing Partition
Material	**STC**	**125**	**160**	**200**	**250**	**300**	**400**	**500**	**630**	**800**	**1000**	**1250**	**1600**	**2000**	**2500**	**3150**	**4000**
5/8" GWB on each side of 3-5/8" studs 24" o.c., 3" fiberglass insulation	**49**	32	33	39	44	45	48	51	55	57	58	58	57	55	45	46	53
τ		0.0006	0.0005	0.0001	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Area	**460**																
Insulating Glass Unit 1/4" -- 1/2" air space -- 1/4"	**35**	22	26	18	25	25	31	32	34	36	39	40	39	35	36	46	52
τ		0.0063	0.0025	0.0158	0.0032	0.0032	0.0008	0.0006	0.0004	0.0003	0.0001	0.0001	0.0001	0.0003	0.0003	0.0000	0.0000
Area	**71**																
Marshfield Acoustical Door, STC 41	**41**	29	32	30	31	37	42	45	46	44	42	42	45	46	48	49	51
τ		0.0013	0.0006	0.0010	0.0008	0.0002	0.0001	0.0000	0.0000	0.0000	0.0001	0.0001	0.0000	0.0000	0.0000	0.0000	0.0000
Area	**43**																
ΣS 574	**ΣτA**	0.79	0.44	1.23	0.28	0.25	0.07	0.05	0.03	0.02	0.01	0.01	0.01	0.02	0.03	0.01	0.00
Max Dev: 6	**TL_{comp}**	29	31	27	33	34	39	41	43	44	47	47	47	44	42	46	53
Total Dev: -21	**TL_{adj}**	27	30	33	36	39	42	**43**	44	45	46	47	47	47	47	47	47
STC: 43	**Tl_{comp} - Tl_{adj}**	2	1	-6	-3	-5	-3	-2	-1	-1	1	0	0	-3	-5	-1	6

Table 17: Original Partition STC

Composite Transmission Loss																	New Partition
Material	**STC**	**125**	**160**	**200**	**250**	**300**	**400**	**500**	**630**	**800**	**1000**	**1250**	**1600**	**2000**	**2500**	**3150**	**4000**
(2) 5/8" GWB on each side of 3-5/8" studs 24" o.c., 3" fiberglass insulation, 1/2" resilient channel on one side	**61**	41	45	50	52	57	60	62	62	64	64	66	66	62	59	63	68
τ		0.0001	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Area	**460**																
Unsealed Glass Unit 3/4" laminated -- 4" air space -- 184"	**49**	33	31	36	38	43	44	46	47	49	50	52	52	55	59	59	58
τ		0.0005	0.0008	0.0003	0.0002	0.0001	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Area	**71**																
Krieger Acoustical Door, STC 53	**53**	34	43	49	45	49	50	49	53	55	54	54	55	55	55	57	58
τ		0.0004	0.0001	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Area	**43**																
ΣS 574	**ΣτA**	0.09	0.07	0.02	0.02	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Max Dev: 4	**TL_{comp}**	38	39	44	46	51	52	53	55	57	57	59	59	59	59	62	64
Total Dev: -22	**TL_{adj}**	40	43	46	49	52	55	**56**	57	58	59	60	60	60	60	60	60
STC: 56	**Tl_{comp} - Tl_{adj}**	-2	-4	-2	-3	-1	-3	-3	-2	-1	-2	-1	-1	-1	-1	2	4

Table 16: Redesigned Partition STC