Best Design Practices for Seismic Evaluation and Preservation of Historic Buildings

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Abstract

Two historic and distinctive building complexes, located in downtown Sacramento, were targeted for seismic evaluation and rehabilitation as part of the city's seismic safety and downtown revitalization program. This evaluation provided a unique opportunity to examine new seismic systems and perform benefit-cost analyses. Complex 1 was originally constructed in the late 19th century and comprised of seven buildings. Several of the buildings suffered fires contributing to the abandonment of the entire complex. However, the tall concrete grain silos and some of the historic features were intact. Hence, the complex is listed on the city's register of historic places and required preservation. Advanced analysis showed that the undamaged buildings had sufficient capacity to resist seismic loading; only minor seismic upgrade of these structures was necessary. A fire-damaged unit was seismically retrofitted using concrete shearwalls. The other units were demolished. Two new buildings were constructed using a unique structural system to provide open living spaces. The project provides 146 housing units and a recreation center. Complex 2 is a replacement of a two-story 1950s lightly reinforced concrete building. Detailed structural investigations showed that the cost of preservation would be prohibitive. A modern and aesthetically pleasing steel building was engineered as a replacement. Its many un-common features—the complex uses multi-directional sloped roofs, sloped columns, and oval interior openings—necessitated comprehensive seismic design and detailing. Upon completion, the building will serve as an interactive learning center, including a theater, used to educate the audience about the diverse and rich history and cultural heritage of the state.

Introduction

Seismic assessment and upgrade of historic buildings present structural engineers with unique challenges. On one hand, these structures use seismic design and detail practices that use earlier editions of building codes or no code at alls. Many of such practices are inadequate to meet the current code provisions, could lead to damage and failure, and require thorough investigation. On the other hand, many such structures possess unique architectural features, have historic significance, and require preservation.

The authors recently had the unique opportunity to evaluate two such structures. The relevant findings are presented in this paper. Both structures are located in Sacramento, CA, and have deteriorated with age. As part of the city's renovation program, both were scheduled for re-use. Conventional code approach and performance based design were utilized to evaluate the structures.

Seismic assessment showed that two historically buildings of the first complex could be preserved. The remaining buildings had to be demolished and replaced. Advanced analyses showed that the structure composing the second complex would require extensive and costly retrofit below grade. Since the replacement cost was similar, this building was demolished and replaced.

Building Complex 1

Overview

This building complex was comprised of seven distinct structures. Seismic and gravity analyses showed that one structure could be preserved with minimal change. Another building was retrofitted for earthquake design using conventional methods and thus was preserved. The remaining units were demolished and replaced with two new five-story buildings.

Introduction

This building complex is a city designated historic landmark. The construction on the complex began in the late 1880's and continued into the 1920's. A historic photograph of the site from the 1930's (California State Library collection, 2007) is shown in Figure 1. In this figure the Mill building and its water tower are facing the west.

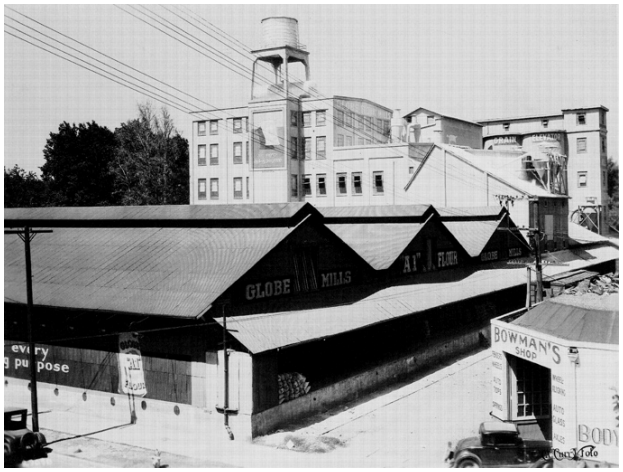


Figure 1. Globe Mills Complex, 1933

It is a former grain and cereal mill complex and operated until the 1970's. The complex footprint approximately measures 170 x 150 ft. The original site plan for the complex is shown in Figure 2 and consisted of seven adjacent structures. Cast-in-place concrete wall structures were used for the silos, Main Mill, and Crockery structures.

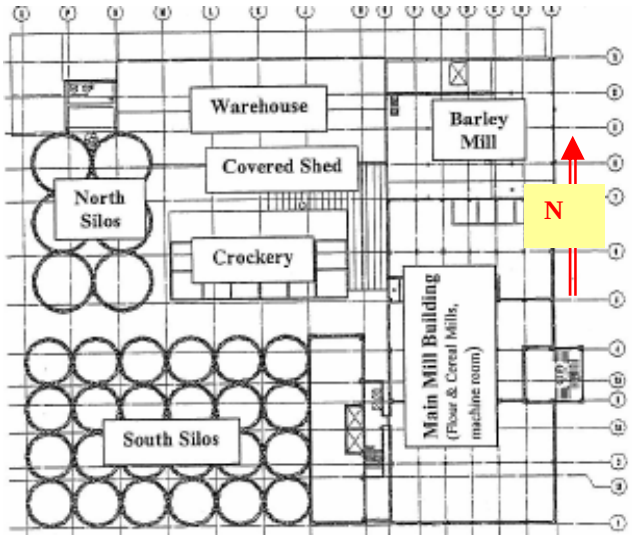


Figure 2. Original Site Plan

The mill's ceased operation in the 1960's. The sporadic use of the site since 1970 had left the complex in a severely dilapidated condition; see Figure 3. This picture was taken looking west and shows the Mills building on the left side.

Figure 3. West Elevation of Globe Mills Complex, 2006

In 1995, a large fire struck the wood flooring of several buildings and destroyed them (see Figure 4 for typical floor damage in the Mill building). The city contemplated razing the complex in the late 1990's and until the current rehabilitation was planned, the complex had been derelict in 2000's.

Figure 4. Interior View of Fire-Damaged Mill Building

In the 2000's, as part of the city's rehabilitation plan, an adaptive reuse and development plan for the site was initiated. The project scopes were to preserve the historic character of the complex and provide affordable housing for active seniors. The architectural rendering of the rehabilitated complex is shown in Figure 5 (Applied Architecture, 2008).

Figure 5. Architectural Rendering

The project consisted of preservation of the South Silos and converting the head house unit above the silos to an activity center for seniors. The Mill building was retrofitted. The barley building, constructed of wood, was a safety hazard and was demolished and its site converted to an open space community area. All other structures were demolished and, in their place, two new buildings, hereafter referred to as NB1 and NB2, were constructed providing 146 mixed income units of housing including 100 units for seniors. The project includes tenant-serving retail and common area facilities. The new site plan is shown in Figure 6. The construction cost was approximately $40 million. This is one of the largest rehabilitation and re-uses projects in the greater downtown Sacramento area.

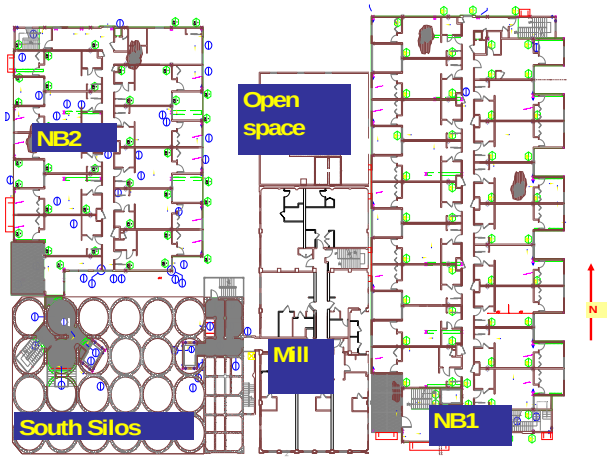


Figure 6. New Site Plan

Conventional code procedures and performance-based design were used in design and evaluation of the buildings. Geotechnical investigations were conducted (JP Singh and Associates, 2006) and site-specific response spectrum (Figure 7), and acceleration records were prepared for this purpose. Both the site-specific response spectrum and design spectrum constructed following FEMA 356 (NEHRP, 2000) guidelines were used in evaluation. Note that in the short period, the design spectrum governs response, whereas, in longer periods, the site-specific spectrum has higher spectral acceleration ordinates.

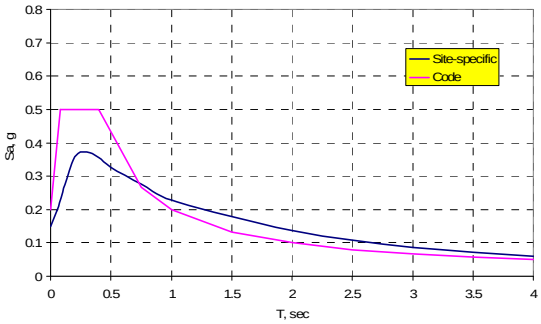


Figure 7. Code and Site-Specific Response Spectra

Evaluation of South Silos

This unit consists of 24 interconnected annular silos, a rectangular silo, and a stairway. A comprehensive investigation was undertaken to assess the structural performance of the silos in the new configuration. Figure 8 presents a photograph of the structure, and Figure 9 depicts the plan layout of the silos with the removed areas identified.

Figure 8. South Silo Structure

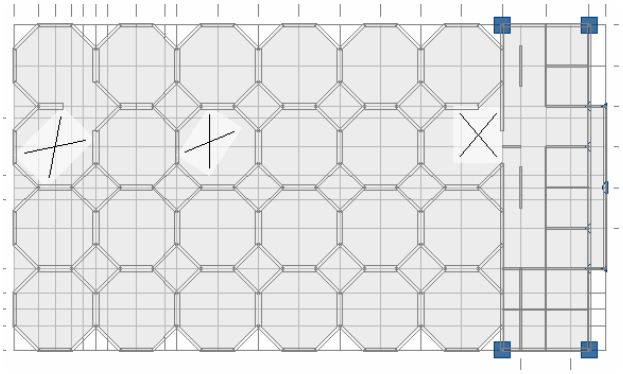


Figure 9. South Silo Plan Layout

The total height of the structure is approximately 127 ft above the basement slab. Reinforced concrete shearwalls provide the main support for both vertical and lateral loading. Each annular silo has a diameter of 15 ft and a wall thickness of 6 in. The rectangular silo measures 16 ft wide and 60 ft long, and has a perimeter 6 1/4-in. thick wall. The stairway segment measures 3 ft by 22 ft. The Structure is supported on a 13 1/2-in thick concrete slab, which in turn, is supported by a series of 12 in. thick concrete walls and a 9-in. thick perimeter wall. The height of the silos is nearly 73 ft, where they are capped by a 5 in. slab. At the top, a head-house is situated which will be used as a community center. Above this level, a portion of the rectangular silo structure extends for three additional floors.

The silos are "connected" to (via a seismic gap), and provide access to the Mills building at five levels. Two stairways and an elevator shaft have been added to the existing structure. This necessitated removing sections of some of the silos. Figure 10 depicts some of the openings.

Figure 10. Openings in the Silo

As part of the investigations, a testing and inspections consultant (Krazan, 2005) performed extensive field evaluations to determine concrete and reinforcement strength, reinforcement distribution, and member sizes. Core samples were taken from 22 locations. For the annular silos, concrete strength of over 6 ksi was measured. A detailed three-dimensional mathematical model of the structure was prepared using the computer program SAP (CSI, 2007).

Figure 11 presents a schematic representation of the model. For all members, centerline dimensions per as-built drawings were used. Annular walls were modeled as an assemblage of eight concrete wall segments in an octagonal pattern. Analysis of a single silo modeled as an annular ring and an octagon showed that this approximation was accurate. Concrete walls, slabs, beams, and columns were sized using available data from plans, field measurements, and from the field evaluation report. Concrete material properties were specified using conservative test data. Except for the concrete strength of the annular silo walls, field data for concrete walls, slabs, and columns were within range of FEMA 356 (NEHRP, 2000) recommended lower-bound compressive strength values. Cracked properties were used. Live loads of 20 psf for roofs and 100 psf for corridors and assembly areas were considered. In addition to member self-weight, a load of 50 psf was placed on floor members to account for equipment and additional seismic mass. The total inertial mass of the structure was computed as approximately 14,000 kips. Per FEMA 356 recommendation, a 5% offset in center of mass was included in analysis to account for accidental torsion.

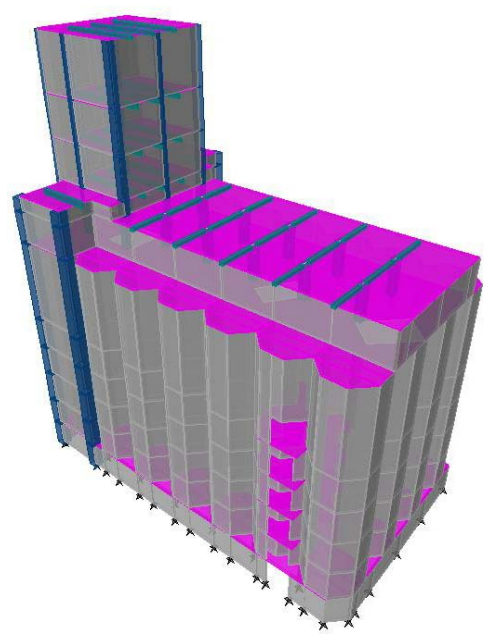


Figure 11. Mathematical Model of South Silos

Sufficient modes were used in analyses to account for close to 100% of the seismic mass in each orthogonal direction. Table 1 presents the first six modes of the structure. It is noted that response in the x-direction is uncoupled. The extension of the rectangular silo above the head-house on one side introduces some coupling in torsional and translational response in the y-direction. The structure is stiffer than individual silos due to the contribution of interconnected walls.

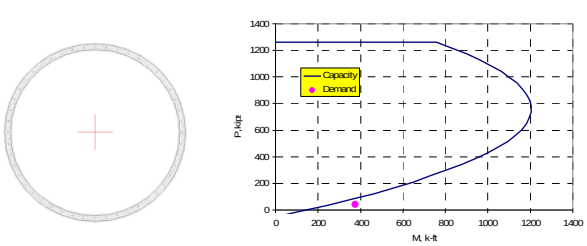
Table 1. Dynamic Properties of the South Silos

Mode	Period, sec	Mass participation factor, %		
		x-	y-	torsion
1	0.22	0	45	15
2	0.19	45	0	0
3	0.13	0	19	51
4	0.11	32	1	1
5	0.09	0	10	10
6	0.05	1	7	0

The linear dynamic analysis procedure (LDP) of FEMA 356 was used for evaluation. The performance target was chosen to correspond to Life Safety (LS) at the design earthquake (DE). DE is defined as either 2/3 of maximum considered earthquake (MCE) or the design bases earthquake (DBE). The computed base shear coefficient is approximately 0.28g. The maximum elastic story drift (Δs) was 0.06%, resulting in inelastic drifts (Δm) below the 1% limiting value.

The reinforcement for the annular walls consisted of #4 bars at 12 in. on center vertically and #5 bars at 18 in. on center horizontally. The reinforcement ratio in each direction is approximately 0.29%, which is larger than the minimum ratio of 0.25% for concrete shear walls (ACI, 2005).

The design module of the program was used to check the concrete walls. The critical wall for the annular silos is the exterior wall segment for the corner silo. The computed maximum shear demand to capacity ratio (DCR) was 0.35. A separate mathematical model of a typical wall segment was prepared; see Figure 12. Nominal dimensions and measured material properties were specified for the wall segment. An axial load-bending moment interaction diagram for the wall segment was developed. The computed DCR was 1.6, less than FEMA 356 minimum *m*-factor of 2.0; as such, the flexural design is adequate. The floor slabs were checked for the gravity loading and were found to be adequate. Hence, the south silo capacity was adequate and no retrofitting was necessary.



a. Cross section b. Interaction curve

Figure 12. Strength Evaluation for an Individual Silo

Barley Building

This is the oldest building in the complex, originally constructed in the 1880's. This wood building had severely deteriorated (Figure 13) and presented a life-safety hazard due to severe loss of capacity of its gravity-resisting members (Figure 14). It was not feasible to rehabilitate this building and, as such, it was demolished.

Figure 13. Existing Condition of the Barley Building

Figure 14. Failure of Existing Columns

Mill Building

The 50 x 100 ft multi-story Mill building used cast-in-place reinforced perimeter walls. The wood floors of this unit were destroyed in past fires. This is the third generation of the building on the same foundations. The building had rotated on its foundation and had a permanent skew. The building skeleton prior to retrofit is shown in Figure 15.

Figure 15. Mill Building Prior to Retrofit (Leaning Left)

Seismic assessment showed that the perimeter walls were adequate in the short (EW) direction. However, the building had insufficient lateral load capacity in the long (NS direction). To address this deficiency, full height concrete walls were added in this direction. A typical wall segment is shown in Figure 16.

Figure 16. Concrete Wall Retrofit

The new walls were supported on 36-in. thick concrete footings. The existing building footings were also thickened and the result was a solid concrete mat at the building ground level. Dowels were used to connect the existing and new foundations. This design mitigated concerns with the permanent tilt in the building that was previously mentioned.

A typical floor plan for the building is shown in Figure 17. A system of steel gravity beams and columns was used to frame between the walls. 3W steel deck topped with 2.5 in. of normal weight concrete was used for floors.

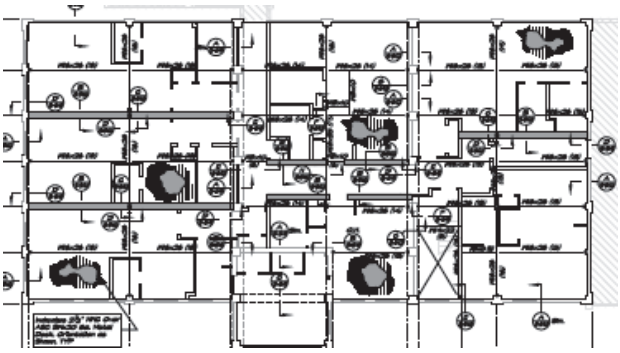


Figure 17. Plan Layout of Mill Building

The existing water tower on the Mill building is one of the historically significant features of this building (see Figure 1 and Figure 3) and hence required preservation. However, its framing was structurally deficient and required retrofit. The structural upgrade consisted of providing lateral stability to the supporting concrete framing by adding 1-in diameter rod bracing and copious concrete patching. Figure 18 shows the water tower during retrofit.

Figure 18. Retrofit of Water Tower

New Buildings NB1 and NB2

These new five-story structures were designed and constructed to replace some of the existing units on the complex. NB1 measures 82 x 156 ft, and is shown in Figure 19. NB2 measures 80 x 90 ft and is shown in Figure 20.

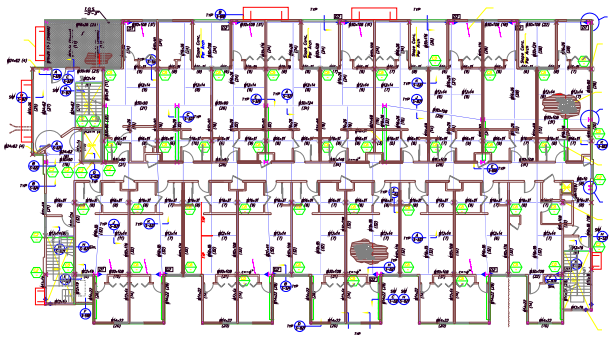


Figure 19. Plan View of NB1

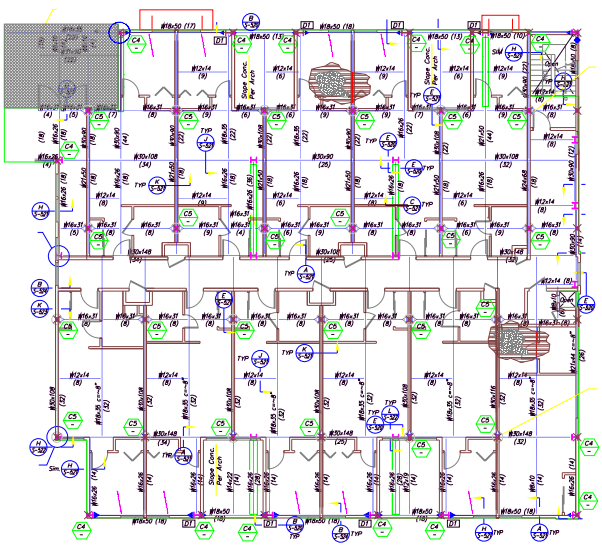


Figure 20. Layout of NB2

Due to architectural constraints, the columns on the ground floor could not align with the upper story columns. This lack of direct continuity in the load path necessitated careful analysis of these buildings and required designing a system for the transfer of upper level seismic forces to the lower story columns at the podium level. In conventional design, building codes, usually require amplification of forces at the discontinuous members by the overstrength factor. Three distinct design alternatives were examined. The option that was the most cost-effective and simplest to construct was selected.

The three investigated alternatives all used a system of concrete/steel podium with steel ordinary moment resisting frames (RMRF) in one direction and concrete shear walls in the perpendicular direction. Reinforced concrete walls were 12 in. thick, and the second level floor consisted of a composite concrete slab of a 2.5 in topping over W3 steel decking. Concrete grade beams (see Figure 21) were used to provide fixity at the base of podium columns. Typical grade beams were 2 ft wide and 3 ft deep.

Figure 21. Foundation Construction for Podium

The following three options for the four stories above the podium were investigated:

- Metal stud bearing/shear walls
- Steel ordinary concentric braced frames (OCBFs)
- Truss moment frames with Hambro joists (Hambro, 2008)

Option 1 was not selected due to its prohibitive cost. Very heavy (14-gage) studs and built-up sections were required for structural design. Furthermore, heavy gage metal deck was needed to span between the stud walls. Heavy shear and hold-down connections were needed for the walls to account for the code's overstrength (Ω_o) factor. At the time of design, the cost of light-gage framing was growing exponentially, driving up costs dramatically, and pricing out this alternative.

In option 2, heavy wide flange transfer girders were required. Detailed three-dimensional mathematical models of NB1 and NB2 were prepared using the computer program ETABS (CSI, 2005); see Figure 22. Only pertinent lateral load resisting

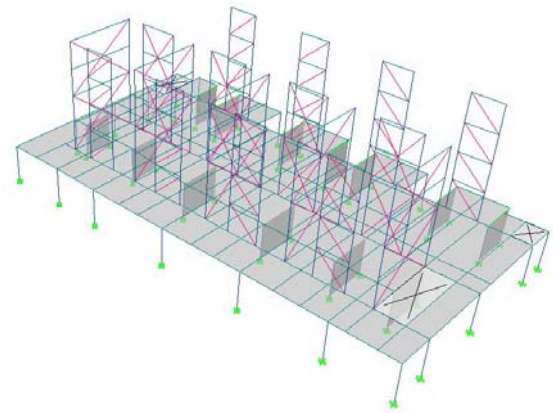
members were included in the model. For all members, centerline dimensions were used. Loading from upper story bearing walls was simulated as dead- and live- line loading at the second floor level. The seismic mass at each level was lumped at floor center-of-mass. Both translational and rotational mass were included in the model. Total mass of NB1 and NB2 was approximately 4,500 and 3,000 kips, respectively. Rigid diaphragms were placed at each level.

A three-step procedure was used in design. Conventional code-based design and performance-based engineering (PBE) were employed. Initially the steel members were sized using site-specific response spectrum loading. Next, static nonlinear (pushover) analyses of the structures were performed to verify the design and assess the response of the structure at the target displacement, or performance point. Finally, acceleration history analyses were conducted to verify results. Analytical investigation of NB1 and NB2 showed that the seismic performance of the buildings would be essentially linear, with minor nonlinearity in a few braces and elastic behavior in the steel transfer beams and moment frames at the podium.

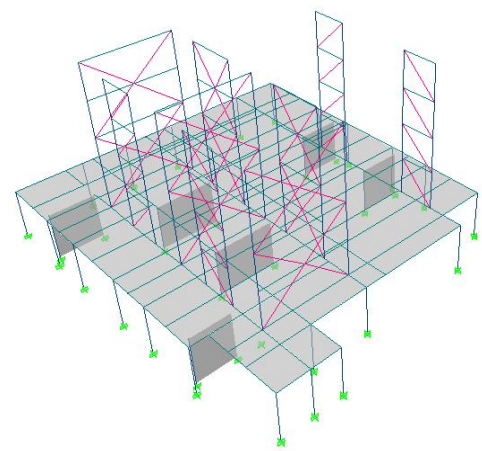
Conventional code linear dynamic procedure (LDP) was used to design the structural member sizes. Seismic loads were based on the reduced response spectrum demands. All members and connections were designed at this step. This design required using the overstrength factor.

In lieu of using the code's overstrength factor (Ωo), PBE was used to ensure that members supporting discontinuous columns remained elastic. The PBE analysis consisted of pushover analysis to assess the performance at the expected seismic displacement and nonlinear response analysis to verify the accuracy of pushover results.

Nonlinear static procedure (NSP) or pushover was used to determine the expected performance of the building in a seismic event. Nonlinear hinges were added to the model. Axial (P) hinges were placed at midspan of braces; flexural (M) hinges at midspan of second floor beams; flexural hinges near the supports of OMRF beams and PMM hinges near the supports for the OMRF columns. Initially NB1 and NB2 were preloaded with gravity load (1.2D + 0.5L), then incrementally loaded laterally. Non-orthogonal effects were taken into account by using load combinations of (1.00X + 0.30Y) and (1.00Y + 0.30X), and directionality effect was accounted for by alternating the direction of applied loading. The amplitude of applied lateral loading at each floor was selected as the displacements at the floor obtained from response spectrum analyses.



a. NB1



b. NB2

Figure 22. Mathematical Models of NB1 and NB2

FEMA 356 (NEHRP, 2000) coefficient method and ATC 40 (ATC, 2000) performance point procedure were utilized. The performance point was selected to provide Life Safety (LS) for the site-specific design earthquake (DE). Figure 23 presents the spectral demand-spectral capacity curves for one of the analyses. The target displacement for this case was 2.4 in (with a single degree of freedom spectral displacement of 1.7 in, as shown in the figure). Note, the building response is linear up to the performance point. The yield displacement is estimated at 3 in. (spectral displacement of 2.5 in.). Similar analyses showed that there was little or no nonlinearity at the performance points for either NB1 or NB2. No yielding of beams was observed from analyses; in other words, the members supporting discontinuous columns remained elastic.

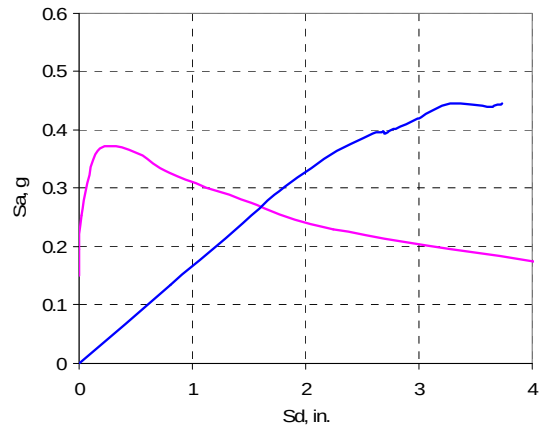


Figure 23. Typical Pushover Data and Performance Point

Finally, nonlinear dynamic procedure (NDP) was used to analyze NB1 and NB2 and to verify the computed displacements from conventional code analysis and NSP. Three pairs of spectrum-compatible histories oriented at 0- and 90-degrees (6 total cases) were used in analysis. Figure 24 presents the roof displacement response for buildings subjected to a typical acceleration history input. The linear limit is shown by the dashed line. Note that the response is below the yield value.

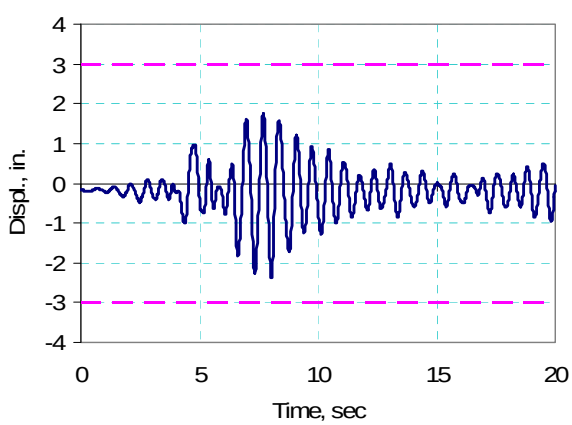


Figure 24. Roof Displacement Response

Table 2 presents the computed roof displacements for NB1 and NB2 at the roof level. Note the similarity between conventional design, pushover, and linear dynamic analysis results. Comparison of analytical data for linear dynamic and nonlinear static analyses confirmed that the response of the structure at the site seismic loading was expected to be essentially elastic and as such the lateral load design methodology employed herein was adequate.

Table 2. Computed Roof Displacements From Analyses

Analysis	NB1		NB2	
	x-, in.	y-, in.	x-, in.	y-, in.
LDP	2.3	2.5	1.8	2.6
NSP	2.6	2.9	1.9	3.0
NDP (max)	2.3	2.5	1.7	3.0

Although a reasonable design was achieved, there were several challenges that made this design undesirable. These included:

- Space design was extremely aggressive, making it difficult to make room for braces. At these locations, fitting of braces would have required major architectural compromise.

- Although PBE was used to optimize member sizes, nonetheless, large steel members were required at the podium. The size of these members supporting the columns of braced frames and the connection costs drove the cost of steel at the podium level significantly upward. This was primarily because the space and architectural design necessitated using narrow and frequent braced frame bays resulting in high vertical forces. In addition, column locations had been fixed, in the planning phase, prior to the structural design, thus requiring the placement of many transfer girders to transfer loading to the ground level columns and foundation.

- Due to the presence of many transfer beams at the podiums of NB1 and NB2, gravity load deflection of these beams resulted in complex loading to the braced frames. This created large pre-loads in braces that were difficult to predict and could potentially lead to overstressing of these braces under combinations of gravity and earthquake loading

The third option was presented by a design-build steel subcontractor. The authors designed the first floor podium and the upper floors were contracted as design-build. This system was ultimately selected.

In this alternative, a system of truss moment frames (see Figure 25), steel joists, and Hambro framing (see Figure 26) for resisting gravity loading were used. The truss moment frames were typically 18 in deep.

Figure 25. Typical Truss Moment Frames

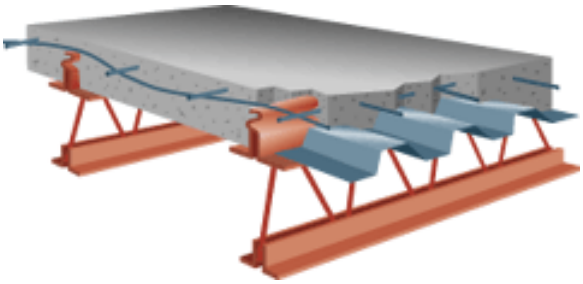


Figure 26. Hambro Joist System

The truss moment frames consisted of small tubular columns and light truss frame elements. The columns were spaced at approximately 14-ft on center (Figure 27 depicts the size and spacing of columns).

Figure 27. Columns of Truss Moment Frames

Such small column spacing greatly reduced the lateral forces for podium design. Thus, typical moment frame beams were W30 and frame columns were W12. Figure 28 shows a photograph of the partially completed podium.

Figure 28. Partially Completed Podium

Photographs of NB1 and NB2 under construction are presented in Figure 29 and Figure 30, respectively.

Figure 29. NB1 Under Construction

Figure 30. NB2 Under Construction

Building Complex 2

Overview

This historic two-story structure was selected for seismic assessment due to the change of occupancy and proposed major structural re-configuration. Such changes necessitated evaluating the existing lateral load components. Evaluation revealed that seismic retrofit would be possible but not feasible economically. As such, the building was razed and a new structure will be erected on the site.

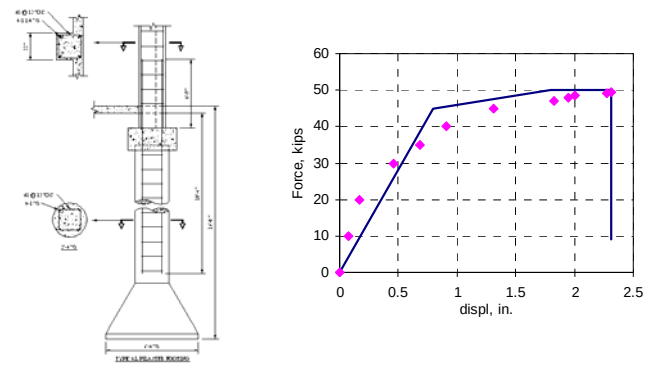
Seismic Assessment of the Existing Building

The original 25-ft tall, 17,000 ft^2, structure was constructed in the 1940's with two stories. It measured 136 feet in the North South and 61 feet in the East West direction. The existing structure was composed of cast-in-place concrete slab supported by a system of interior columns and perimeter walls and pilasters. Columns were supported on 16 ft deep cylindrical shafts terminated in belled caissons; see Figure 31a (CUC, 2006).

It was proposed to add a steel-framed third floor to the existing two-story building and convert it to educational occupancy. This option also involved removing a number of the existing shear walls and a large portion of the floor slabs for an atrium and new exterior windows. This initiated a comprehensive evaluation of the building.

The provisions of FEMA 356 (NEHRP, 2000) were used to model the building and assess its seismic performance. Two seismic levels were defined: The 500-year event or the Design Level Earthquake (DE) and the 2,500-year event as the Maximum Credible Event (MCE). Three performance levels were considered: Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP). For DSA projects, The California Code (CBC, 2001) requires that buildings designed for the Division of State Architect (DSA) meet two performance targets: LS-0.33 (LS-IO) for the DE level and CP for MCE level. For this building, the DE requirement governed response. Nonlinear static and dynamic analyses were used in evaluation.

A detailed geotechnical investigation of the site was conducted by Wallace-Kuhl, (2006) to prepare site-specific design spectrum and acceleration records and by JP Singh and Associates, (2007) to model the soil-structure interaction and calculate soil spring properties. The soil-structure interaction was based on the soil data obtained from site borings. The lack of confinement in the shaft and inadequate lap splices of longitudinal reinforcement from the pile to the columns were included. Pile lateral capacity of only around 45 kips were computed, see Figure 31b.



a. Foundation details b. Soil spring properties

Figure 31. Foundation Detail for Pilasters

Computer analysis program SAP (CSI, 2007) was used to model the structure. Nonlinear springs accounted for the soil-structure interaction. Nonlinearity was modeled as user-defied plastic hinges in pushover analysis or by using nonlinear elements for response history analysis. Figure 32 presents the deformed shape of the ground slab and piles from the pushover analysis. The building nonlinear behavior was concentrated at the soil springs and that the nonlinear demands on members exceeded the allowable values.

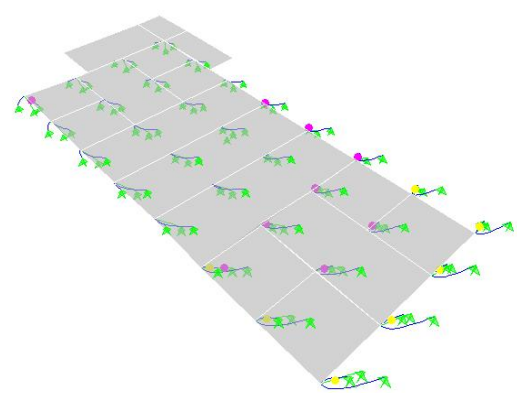


Figure 32. Plastic Hinge Pattern

Analytical investigations and detailed component ductility evaluation (ACI, 2005) showed that the existing building had major seismic deficiencies as cited below.

- Capacity of concrete shear walls
- Anchorage of slab reinforcement into the existing walls
- Splice of longitudinal reinforcement
- Lateral capacity of the shafts
- Vertical capacity of the slabs after addition of atrium

To mitigate these deficiencies, a comprehensive retrofit methodology was investigated. It comprised of: shotcreting

the walls, adding grade beams below grade at the perimeter shear walls, adding FRP to slabs, using mechanical splices or welds at splice locations in columns.

The retrofit cost was estimated at $11 million. The replacement cost of the building was estimated at $12 million As such, the owner, in agreement with the structural engineers, agreed to pursue replacement of the structure. The decision to replace rather than retrofit was primarily based on the following reasons:

- A new structure would perform better during seismic events, providing enhanced life safety.
- There will be a better understanding of materials and their properties for a new building and hence the behavior of the structure can more accurately be determined.
- There is more accuracy in cost estimation for a new structure compared to retrofit of an existing structure. This could reduce the number and amount of contract change orders (CCOs) during construction.
- There will be more options for the architect to integrate interior planning as well as exterior features.

This decision and the foundation concerns, raised during analysis evaluations, were validated during demolition. As the contractor attempted to extract some of the shafts, they broke at the shaft-caisson joint. The demolition of the building is shown in Figure 33.

Figure 33. Photograph of Building During Demolition

Seismic Design of the New Building

The new structure is a 3-story building. Figure 34 presents an architectural rendering of the structure. As seen in the figure, the top floor has sloped roof and is comprised of several segments at different elevations. The building has a relatively large atrium at the entry to the building. Figure 35 depicts the second floor plan. The atrium area is shaded.

Figure 34. Architectural Rendering

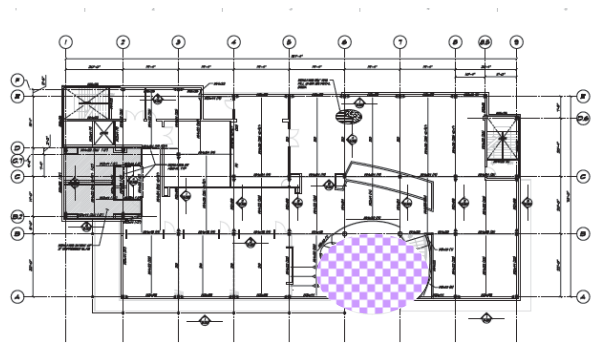


Figure 35. Floor Plan

The Vertical loading components consist of self-weight, nonstructural components and live load. The planned occupancy consisted of exhibit space, teaching space, office space, and auditorium. For future flexibility for use of space, the structure was designed for live load of 100 psf. The lateral resisting system consists of ordinary steel moment resisting frames (OMRF). Since the roof structure has several different levels, wide flange columns were not an option since it would have resulted in moment frame connections to the weak axis of the columns. HSS tube columns were utilized in order to avoid weak axis columns connections at the moment frames; see Figure 36. Additionally, the architect could use the tube columns as architectural features throughout the structure. Moment connections at the floor and roof levels were created by cutting the tube column, adding continuity plates at the top and bottom flanges, and reinforcing the beam flanges to have the strength to resist the combined forces with over strength level seismic forces. The structure required a one-hour fire rating; concrete with metal deck was provided with adequate fire rating supported by composite steel beams and columns to resist gravity loading. All of the moment frame columns have been fixed at the base. The moment at the base of the columns is transferred to the foundation through reinforcing welded to continuity plates at the column, which develop into the grade beam.

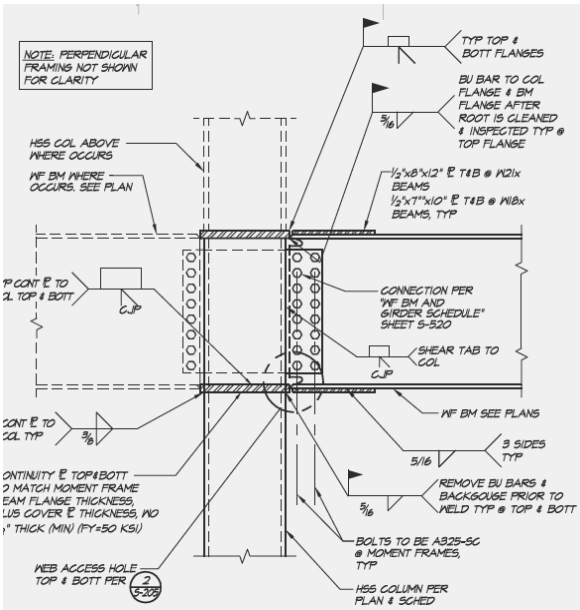


Figure 36. Tubular Column-wide Flange Beam Connection Detail

The columns are supported on end bearing auger cast piles with a tip elevation approximately 30 feet below finished grade. Figure 37 shows the steel columns supported on the augur pile. The grade beam is used to provide fixity at the base of columns.

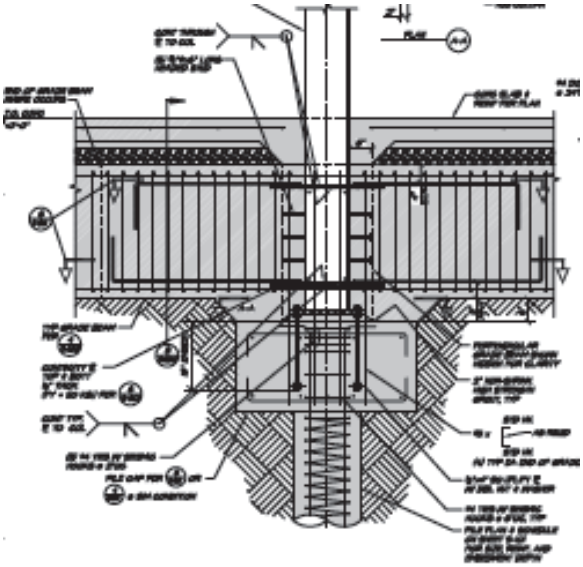


Figure 37. Foundation Detail

The piles are designed for both supports of gravity loads as well as lateral resistance of forces due to seismic loads. The piles were designed as fixed at the top. The longitudinal reinforcing from the piles are developed into the pile cap and reinforcing dowels are provided from the pile cap to the grade beam to transfer moments. A single pile was adequate for gravity loading at all columns; however, additional piles were required for lateral resistance.

Augur piles are one type of drilled shaft and are installed by rotating a continuously flighted hollow stem augur (Figure 38) into the ground to the specified design depth. No steel casings are used. High-strength concrete or grout is then injected into the augur shaft (stem) under pressure as the augur is withdrawn.

Figure 38. Hollow-Stem Augur (FHWA 2007)

The high pressure is continuously maintained and monitored. This pressure acts upwardly on the soil-filled augur flights and laterally on the surrounding soil as the augur is withdrawn. The result is the formation of a concrete column in the drilled hole. Figure 39 presents the schematics of the pile placement (FHWA, 2007)

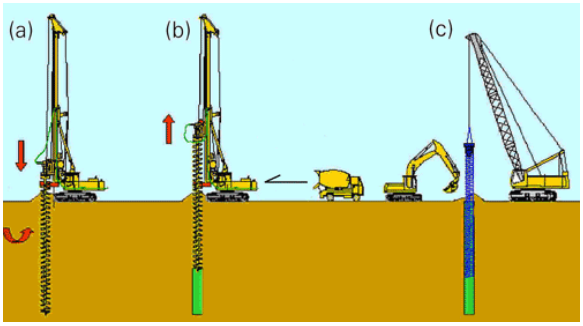


Figure 39. Augur Cast Pile Installation

Reinforcing steel is placed in the center of the hollow stem and/or in the concrete column prior to hardening (FHWA, 2007), see Figure 40. The result is the augur cast pile.

Figure 40. Placement of Reinforcement Cage

Since the pile is drilled in one continuous operation, the time required to drill the hole is significantly less than conventional drilling and thus construction savings are realized. Furthermore, the piles do not generate the noise and vibrations common to driven piles.

Due to the rapid rate of construction, many contractors prefer using augur cast piles. However, good construction QC/QA and an experienced foundation contractor are essential in obtaining an acceptable pile. The difficulties in obtaining a good quality augur pile can be attributed to its particular installation.

In drilled shafts, concrete is poured using a drop chute into the dry holes, reinforcement cage is placed and maintained prior to the pour, and access tubes are placed for future nondestructive tests. As such, there is good observation/inspection for these piles. By contrast, .the placement of grout in augur cast piles can only be monitored indirectly by monitoring the volume delivered through the auger. Additionally, the grout must maintain consistency to allow the placement of the reinforcement cage after the pour. As such, the quality of the finished augur pile system is highly dependent upon operator control.

To address these issues, the authors worked closely with foundation engineers and developed a comprehensive specification for this task. The specifications addressed the quality control/Quality assurance during installation and spelled out test requirements to verify the adequacy of finished piles.

Prior to this project, DSA (DSA, 2008) had not accepted the use of auger cast piles. Auger cast piles were approved based on the engineer of record providing design for both the longitudinal and confinement reinforcing. The piles are required to be tested for two compression piles near the center of the building footprint to verify construction methods and pile capacity. All piles will be required to be tested using nondestructive low-strain dynamic testing methods. A minimum of 10 percent of the piles will be tested using a nondestructive high strain testing methods. The testing of piles is required to verify that there are no sizable voids or inconsistencies created in the pile created during pile or reinforcement installation. Piles that do not meet the required testing criteria will need to be repaired or replaced (Wallace Kuhl, 2008).

The code's linear static procedure (LSP) could be used for this building. However, due to presence of sloping roof, change in roof elevation, and large openings in the diaphragm for the atrium, linear dynamic procedure (LDP) was used instead. The unreduced site-specific design spectrum is shown in Figure 41. In analysis, the seismic demand was scaled to ensure that the strength calculations met the code's minimum specified values as percentage of values computed by linear static procedure. The code does not address drift calculations utilizing linear dynamic procedures. The drift calculated with linear dynamic procedure and that calculated with linear static procedures without the limit based on the period per code were within 5 percent of each other; therefore, linear static procedures were used in computing drifts.

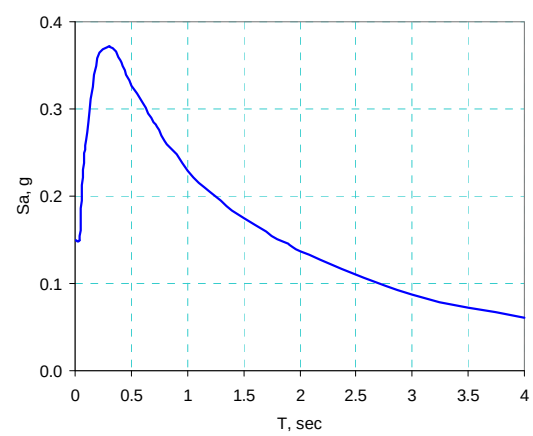


Figure 41. Design (DE) Response Spectrum

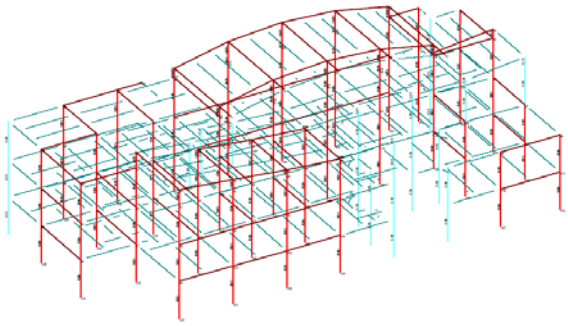
Detailed mathematical models of the building were prepared using commercially available programs RAM-Steel (Bently, 2007) for analysis and design of the main structure and RISA (2007) for the design and analysis of the atrium. SAP (CSI, 2007) was used for analysis and verification of RAMSTEEL and RISA results. Figure 42 depicts the mathematical models of the structure.

The new building consists of a complicated and sloped roof system. A detailed mathematical model of the building was prepared using the program RAMSTEEL. This model has been used in sizing the members for gravity loads and for seismic design using the CBC 2001 code. The loads from the atrium as determined in the RISA model were added to the RAMSTEEL model as nodal masses. To assess the adequacy of the design methodology for seismic design, a second independent model was prepared using the program SAP to incorporate both the atrium and the main building structure. The models are similar and they model the pertinent geometry, member articulation, gravity loading, inertial mass, and stiffness components closely. Although similar, the models are not identical. Since the SAP program was not intended for design and instead was used as a check, it did not include all detailed elements, which are not considered of great consequence for the lateral analysis and design of the structure. The seismic mass of the building was approximately 2,900 kips.

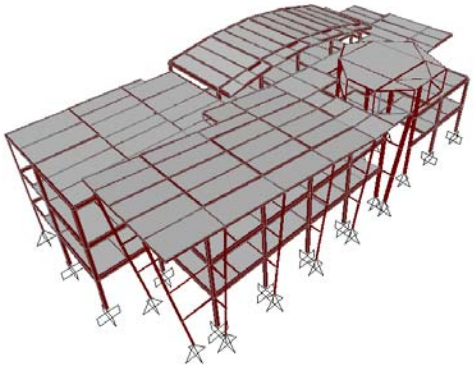
DSA was concerned with the atrium and the main structure having different responses. The SAP model was created to show that the results from the RAMSTEEL and RISA models correctly represented the response and force distribution for the structures.

Analysis of the two models resulted in similar mass for the structure with approximately a 5% variation between the models. The models also had similar periods, differing by less than 10%. As such, the models are dynamically equivalent. Thus, they will have similar global performance for seismic analysis. Additionally, the frame lateral forces (Vu) were compared between the two models utilizing a response spectrum analysis. The two models predicted similar values of seismic demand for the columns in each frame, and for the frames overall. They would yield similar seismic design values since the models were dynamically equivalent. The choice of RAM Steel for design was based on the program being a more convenient design abased on the application for steel structures.

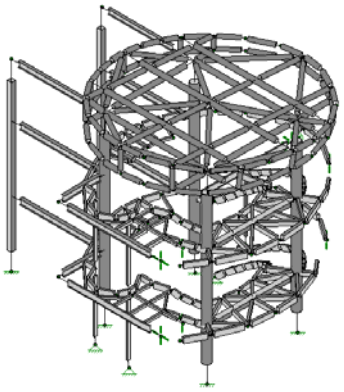
Table 3 presents the modal properties for the building. Note that the response is dominated by the first mode, the mode shapes are uncoupled, and the fundamental period is approximately 1 sec in both lateral directions. The computed base shear coefficient for the building was 0.15g.



a. RAM-Steel model of the building



b. SAP model of the building



c. RISA model of atrium

Figure 42. Mathematical Models

Table 3. Modal Properties

		T, sec	Mn, %
Mode	Dir	SAP	SAP
1	x	1.1	78
2	y	1.0	73
3	rotation	0.9	
4	x	0.4	9
5	y	0.35	11
6	rotation	0.3	

Table 4 lists the computation for the torsional irregularity. In both principal building directions and all levels, the computed displacements at one edge of diaphragm were similar to the values on the opposite edge; hence the building was not torsionally irregular.

The maximum elastic drift for the building was 0.55% resulting in an inelastic drift (for R of 5.5) of 2.2%, which is less than the CBC limiting value of 2.5%.

Table 4. Building Irregularity Check

Level	Dir.	u max	u min	u max/u avg
Roof	x-	2.76	2.51	1.05
	y-	2.59	2.81	1.04
Third	x-	1.87	1.70	1.05
	y-	1.89	1.99	1.02
Second	x-	0.75	0.69	1.04
	y-	0.80	0.76	1.03

For seismic design, an importance factor of 1.15 was used. The redundancy (ρ) factor equaled 1. Since the building roof is sloped, it and will produce a higher ρ value of 1.5. However, the code (CBC, 2001) recommends that ρ calculations be based on the bottom 2/3 of the structure and this produced a redundancy factor of 1.

Since RAM-steel modules do not account for the code upper limit values on drifts when LDP is used, the building drifts were calculated using LSP with a ρ of 1.0.

Load combinations with wind loading were also investigated. Although the height of the trees is below the new roof level of the new building, this does not affect the exposure criteria for this building. In accordance with the code, Exposure B requires that the site have "terrain with buildings, forest, or surface irregularities, covering at least 20 percent of the ground level area extending one mile from the site". Since the site is located in an urban area with buildings covering 20 percent, a wind exposure B is used for this building. The wind importance factor of 1.15 was used in design.

The interaction between the atrium and the main structure was investigated by examining the relative stiffness of the structures. A unit lateral load in each direction was placed on the structure and the deflections were computed. The atrium and the building are tied together. The columns at the atrium were modeled in RISA for loads from the main building as well as an additional load and deflection added to the columns to account for the drift of the main building structure. In addition to the drift being added to the structure, the reactions from the atrium were accounted for in the main building beams connected to these columns. The beams and their connections were designed to transfer the axial loads back to the main building structure. To assess the relative stiffness of the building and the atrium, a unit load in each lateral direction was applied at the connection points between the atrium and the building. Such loading resulted in deflections of 0.1 in for the building and 0.8 in for the atrium. Thus, the building is approximately eight times stiffer laterally than the atrium. This implies that the direction of transfer of the lateral loads is primarily from the building to the atrium and not vice versa. Such assumption was incorporated in analysis.

Summary and Conclusions

Conventional and performance-based engineering were used to assess the seismic performance of two historic structures in California. Evaluations showed that:

- For historic buildings, the first priority is preservation. For building complexes comprised of multiple structures, it might be feasible to rehabilitate some of the units, and preserve the unique historical features.

- For some structures, replacement is the prudent alternative. Although, the building can be seismically retrofitted to meet its performance goals, the associated costs and inherent uncertainties with any retrofit could make replacement a more attractive alternative.

- A combination of conventional and advanced analysis can be used to optimize the knowledge regarding the expected performance of the buildings.

- The code procedures alone might lead to unrealistically conservative results. For example, Performance based engineering can be used to assess whether the code mandated overstrength factor is warranted for the particular situation being investigated.

- For unique structures, it might be warranted to investigate in detail several design options at the type selection stage. The optimal design solution then can be selected based on cost, ease of construction, preliminary analysis results, and architectural constraints.

- Augur-cast piles present a cost-effective foundation solution. These piles are simpler to drill than conventional drilled shafts. However, the construction quality depends on the experience of the foundation engineers and a comprehensive test program.

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