HEARST MEMORIAL MINING BUILDING SEISMIC IMPROVEMENTS, UNIVERSITY OF CALIFORNIA, BERKELEY

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SUMMARY

This paper describes the programming, planning, design, and analysis for the seismic improvements to a four-story brick masonry building originally constructed in 1907. The building is an historic landmark on the UC Berkeley campus, and is located very close to the Northern Segment of the Hayward fault. The selected system for seismic improvements is the installation of a high-damping rubber isolator system, in combination with fluid viscous dampers, and includes complete replacement of the foundation system and a new first floor system. The analytical procedures used, in combination with careful consideration of anticipated ground motions and reasonable performance goals, allowed the historic fabric of the structure to maintain its original appearance visually unencumbered by a new lateral force resisting system.

INTRODUCTION

The Hearst Memorial Mining Building was constructed for $675,000 and dedicated in 1907. Acknowledged as among the most significant buildings architecturally and historically on the Berkeley campus, the building occupied a prominent site and anchored the principal East-West axis of the new developing campus. The Hearst Memorial Mining Building is listed on the National Register of Historic Places. As a monument to the memory of George Hearst, the building was constructed as a laboratory and workshop to teach technical mining processes. With time, the building became a more general center for civil engineering and minerals engineering research, eventually housing the Department of Materials Science, responsible for some of the very significant research and teaching breakthroughs for materials in the semiconductor, medical, aerospace, and bio-engineering industries. See Fig. 1 for building arrangement. The building area is approximately 134,000 gsf.

Immediately following the Loma Prieta earthquake, in 1989, the campus decided to evaluate the structure for seismic safety, and developed a program not only to address serious seismic deficiencies, but also to completely redevelop the existing laboratory, office, and classroom spaces with a completely new infrastructure to support the academic program of the future. A critical goal was to minimize intervention into the historic fabric of this well-known and highly esteemed building, either from structural improvements, or from new architectural/infrastructure additions.

BUILDING DESCRIPTION

The exterior punched walls are of unreinforced brick masonry construction with a finished exterior of Sierra White granite blocks. Interior structural bearing walls are also unreinforced brick masonry with some localized steel elements to carry the floor system, which is concrete on steel framing. Floor–to-floor heights are typically 16 feet and one floor is below grade. The original construction includes an areaway around much of the building perimeter and tall masonry chimneys; these are major elements of the building's historic character.

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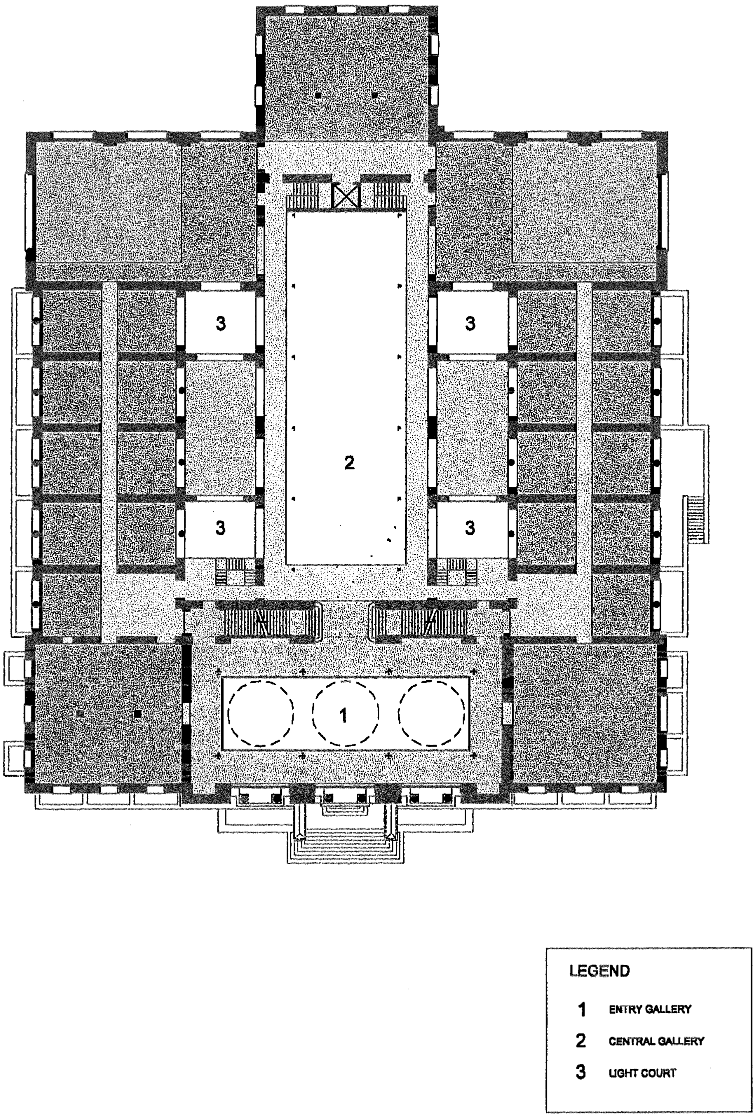

Figure 1: Second Floor Plan

The seismic studies we completed after the Loma Prieta earthquake identified a number of major deficiencies in the structure, such as overstressed shear walls, poorly constructed diaphragms, poor wall-to-diaphragm connections, and significant falling hazards including the very tall chimneys. A subsequent study was conducted to assess the feasibility and cost of conventional fixed-base schemes and seismic isolation. In 1994, based on this study and because of the building's historic importance to the University and the Berkeley community, the base isolation scheme was selected. Concurrent with these building studies, Rutherford & Chekene also carried out several ground-motion studies to develop site-specific ground motion criteria, considering the proximity of the Hayward fault which has been physically located only 800 feet away. The northern branch of the Hayward fault has been identified as one of the faults most likely to lead to a major earthquake (Mˆ7.0) in the next several decades. Concurrent with developing realistic design ground motions work was completed on suitable performance objectives for the building, for both the Design Basis Earthquake (DBE) (repairable damage or better) and the Maximum Considered Earthquake (MCE) (collapse prevention).

Typical brick masonry construction consists of multi-wythe brick in running bond lay-up. The exterior walls are typically 30 inches thick with 6- to 11-inch thick granite veneer. Interior masonry walls range from 25 ½ to 30 inches in thickness. Several interior walls step to 13 inches thick above the fourth floor. In general, the quality of the masonry wall construction is very good. The configuration of both interior and exterior masonry construction includes both solid walls with limited door and window openings, walls with many openings, and slender pier elements. The existing foundations are continuous strip footings beneath the walls, and individual spread footings beneath existing columns, which are few in number. As you enter the building, the Memorial Hall atrium rises three stories to circular skylights above an elegant Guastavino tile vaulted ceiling.

Extensive testing was completed to verify the properties of the existing masonry construction, including exterior veneer attachment, and the concrete floor diaphragms. Brick testing included cyclical and unidirectional in-place shear and compression tests. Investigations indicated that the granite veneer was well anchored, and that the quality of the existing construction was very good. In contrast, investigation of concrete floors revealed minimally reinforced weak concrete that was placed in two and three lifts.

GROUND MOTION

The design process for seismically-isolated buildings required by the 1997 Uniform Building Code and other design guidelines includes analysis at two site-specific levels of ground shaking. For this project, earlier studies by Rutherford & Chekene developed three earthquake levels, roughly corresponding to probabilities of 50% in 50 years (Level 1), 10% in 50 years (Level 2), and 10% in 250 years (Level 3). The University's Seismic Review Committee, whose members include eminent faculty from the College of Engineering, worked with Rutherford & Chekene to select the appropriate spectra to define the DBE and MCE events for use on the UC Berkeley campus and the Hearst project specifically. Consistent with current codes, a 10% in 50 years DBE and 10% in 100 years MCE were selected.

Based upon this thorough review, the DBE was chosen as the Level 2 event, which is a postulated $M = 7.0$ earthquake on the nearby northern segment of the Hayward fault using median estimates of the ground motion attenuation. This deterministic estimate of ground shaking yields a response spectrum that is similar to the response spectrum that would be developed using uniform hazard methods, and the probabilistic definition of the DBE.

Similarly, the MCE is based on the deterministic definition of ground shaking due to a Moment magnitude $M_w = 7.0$ earthquake resulting from fault rupture on the northern segment (only) of the Hayward fault. Median estimates of attenuation (random horizontal component) were increased by a factor of 1.5 to account for spatial variation of ground shaking and again by 1.5 to account for potential increase in the dominant horizontal direction of shaking. The selection of the moment magnitude Mw = 7.0 event for this scenario is consistent with the "characteristic" moment magnitude Mw = 7.05 associated with the northern segment of the Hayward fault and used by the USGS as part of their most recent national hazard mapping. The response spectrum for the MCE yields ground motion levels similar to those derived probabilistically using uniform risk methods and a 10% chance of exceedance in 100 years. The response spectra for both the DBE and MCE are shown in Fig. 2.

After reviewing concurrence from the University Seismic Review Committee and the independent Peer Review Panel, the next step was to select appropriate time history records consistent with the geotectonic setting (near-source /rock site) that could be nominally scaled to match the DBE and MCE design spectra. The UBC requires that one suite of seven time histories be used for DBE analysis and another seven for MCE analysis. However considering the limited number of appropriate near-source records available to choose from, a slight alteration to the more conventional approach for selecting time history records was adopted as follows: (1) A single set of seven time histories is selected (2) The average response of the set of time histories is appropriate for DBE analysis and (3) the maximum response of the set of time histories is appropriate for MCE analysis.

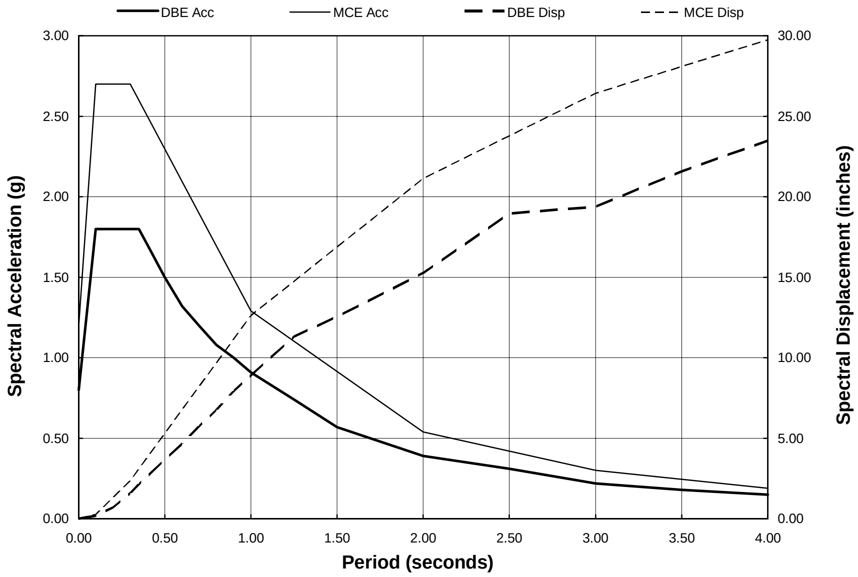

Figure 2: Response Spectra

Seven source ground motions were selected, as shown in Table 1. The records were first oriented in their fault normal and fault parallel directions prior to scaling. Then using a scaling procedure, each record (and direction) was scaled with the resultant scaling factors varying from 1.0 to about 2.0 with an overall scale factor of 1.3. No records were scaled downward (less than 1.0) and the three components (two horizontal and one vertical) were scaled with the same factor. To verify that the suite of records complies with UBC requirements for average and maxima for the 5% damped spectra, the records were processed and yielded the plots of Figure 3, showing that both the average and the maximum would envelope the accepted respective DBE and MCE spectra in the period range of 1.5 to 3.5 seconds.

Table 1: Source Motions

No.	Year	EARTHQUAKE	STATION	M	Distance (km)	Scaling Factor
1	1995	Kobe, Japan	Kobe University	6.9	0.2	2.03
2	1992	Landers, California	Lucerne Valley	7.3	1.1	1.11
3	1989	Loma Prieta, California	Corralitos	6.9	5.1	1.93
4	1989	Loma Prieta, California	UC Santa Cruz, LGPC	6.9	6.1	1.01
5	1978	Tabas, Iran	-	7.4	3.0	1.00
6	1992	Cape Mendocino, California	-	7.1	8.5	1.0
7	1994	Northridge, California	Sylmar Converter, East	6.7	6.1	1.0

The manner in which the ground motion records were processed, and their specific suitability for the project were the subject of intense discussions with both the Peer Review panel and University's Seismic Review Committee. One of the major topics of discussions in this process concerned the fact that the existing structure is somewhat stronger in the fault-normal direction, and that characteristically near-fault ground motion tends to be stronger in the fault-normal direction and weaker in the fault parallel direction. In the suite of seven records, two records, while seeming to be anomalous (i.e., stronger fault-parallel motion), were actual records from recent earthquakes and were being used with a scale factor of 1.0 to match the design spectra. Given the inherent uncertainties in ground motion prediction, particularly at a site very near the source, the limited number of near-source records, and the fact that the building is practically right on top of the fault considering the probable depth of fault rupture, it was the recommendation of the design team that we not reduce the design spectra for the fault parallel direction in order to reduce the demand on the isolation system. Underestimating ground motions was also particularly undesirable given the proposal to utilize the brick walls to resist lateral forces while essentially providing no new shear walls, together with the performance objective of collapse prevention in an MCE event.

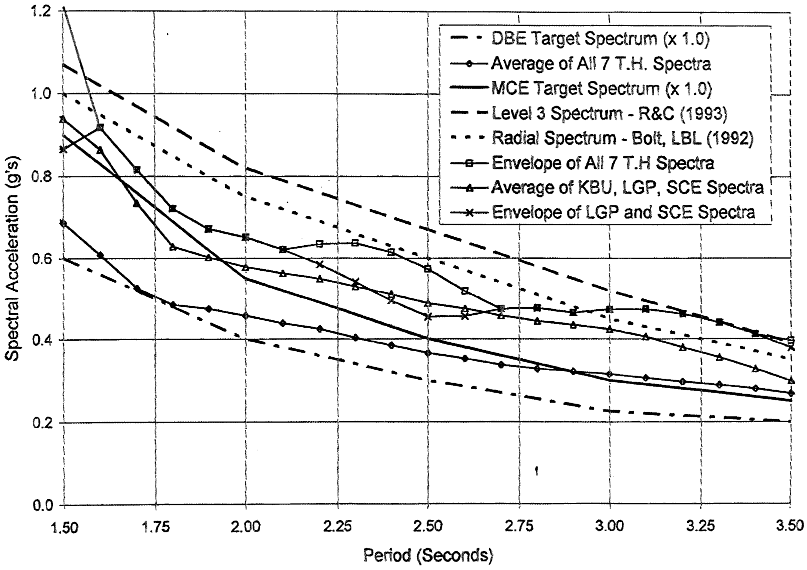

Figure 3: Comparison of Target Spectra and 5%-Damped Resultant (X,Y) Response of Scaled Time Histories

ANALYSIS AND DESIGN

The principal analytical tools that allowed the project to succeed with minimal impact on the building's historic appearance were the use of pushover analysis to assess the strength of the existing structure and non-linear time history analysis, considering the superstructure and the isolation system as two separate non-linear components, to determine the lateral force demands.

The pushover analysis utilized the latest analytical approach, adapted from FEMA 273, to evaluate the lateral force resisting capacity of the existing brick masonry walls. Based on pushover results, acceptable building drifts and base shears, for the DBE and MCE, were chosen to assure that the brick would perform acceptably. A three-dimensional model was developed for the analysis using the SAP 2000 program with the model consisting of both shell and frame elements. The interactive graphics modeling capabilities of the SAP 2000 program were extremely useful in creating the complex assemblage of building elements and visualizing the behavior of the structure during analyses. The walls and diaphragms were modeled as shell elements, and the columns, trusses and selected beams as frame elements. The initial interior brick wall stiffness used in the analysis was based on a modulus of elasticity, E, of 1000 ksi, based on cyclic compression tests, while the modulus of exterior walls was determined considering an E of the brick equal to 1000 ksi and that of the granite equal to 2000 ksi (FEMA 273) for calculating a composite modulus value. New concrete elements at the first floor level were modeled with an E of 3600 ksi. The masonry shear strength was based on 22 in-situ bed joint shear tests. Using FEMA 273 terminology, the average bed joint shear strength was v_{te} = 155 psi.

Multiple pushover analyses were performed to consider the behavior under different directions of loading and to account for the effects of the anticipated high vertical accelerations due to near field effects. Vertical acceleration was considered by varying the gravity load from 60% to 140% of the actual calculated gravity loads. Each pushover analysis was performed as a series of step-wise linear analyses. An initial lateral load was applied to the model and then scaled such that, when added to the gravity loads, a single element or group of elements yields. The properties of the yielded elements were then revised by adjusting the Modulus of Elasticity, thereby reducing the element stiffness. The process was repeated until a global collapse mechanism was reached. The initial pushover curves were developed assuming the brick piers behave in an essentially elastic-perfectly-plastic manner as soon as yielding occurs by adjusting the E to a nominal value. Several sawtooth pushovers were performed to consider the structure's reduced capacity when some piers yield in a brittle force-controlled mode such as toe crushing. The sawtooth curve pattern develops when these brittle piers are removed from the analysis and the pushover is restarted from the beginning. Because of the time consuming nature of this approach, subsequent pushover curves were based on the elastic-perfectly-plastic pier behavior and were then adjusted by approximate methods consistent with the previous sawtooth analyses. Typically diaphragms are assumed to remain elastic and are then later checked under DBE forces. The wall spandrels are also assumed to behave elastically. This assumption is based on extensive studies of demand–capacity ratios for spandrels and adjacent piers, with the conclusion that the piers are much more vulnerable. We typically used FEMA 273 criteria, with some modification, to calculate brick pier capacities. FEMA 273 pier rocking and toe crushing capacities for piers adjacent to perpendicular walls were adjusted by increasing the axial load due to the effect of the cross wall, acting as an anchor, to delay onset of rocking and/or toe crushing. See Fig. 4 for a sample pushover curve.

Figure 4: Sample Pushover Curve

The results of the pushover analysis indicated that in the North-South direction (essentially fault-parallel) the building would remain elastic up to a base shear of 0.2g, and a mechanism developed at the second story level at 0.25g. In the East-West direction, the building is essentially elastic up to base shear of 0.35g, with some pier rocking. A story mechanism developed in this direction at 0.6g. Based on earlier preliminary time history analyses of the building, considering various seismic isolation systems (with and without supplemental damping), we expected that an isolation system could be developed to limit base shear to allow the structure to remain largely elastic requiring limited spot strengthening of brick piers and spandrels, the addition of diaphragm chords, collectors, and wall to diaphragm ties. With pushover analysis complete, we were then able to complete time history analyses to confirm this hypothesis and determine which isolation systems could provide this optimal performance.

The next step in the design process was to select an isolation system, which might include some amount of supplemental damping. It was the opinion of the design team and the University that an open competition should be held in order to select a system that would meet the performance objective for the least cost. A request for information (RFI) was sent to all known vendors of the different isolation technologies, such as high-damped rubber, lead-rubber, friction-pendulum, sliders, and combinations of same. The same RFI was also sent to vendors of viscous damping devices.

Eleven responses to the RFI were submitted, including nine from isolator vendors and two from vendors of fluid viscous dampers. In order to evaluate the estimated performance of each proposed system fairly and realistically, several concurrent studies were conducted: the first to evaluate the performance of the various technologies, and the second to consider the cost and associated collateral cost (i.e. other related construction costs) of different isolation systems. Less important design considerations were also compared such as vendor warranties, aesthetic impact of the required moat clearance, maintenance requirements, etc. Properties furnished by each vendor for their respective proprietary technology were separately incorporated into a simplified non-linear stick model of the building, again using the SAP 2000 program. Using the "stick" model, each technology (some including supplemental fluid viscous dampers) was extensively studied and compared under various earthquake scenarios. The "stick" model was composed of 5 lumped masses to represent each floor level, non-linear elements to represent the isolation system, and a non-linear "stick" for the superstructure based on stiffnesses derived from the final pushover analyses. See Fig. 5 for a schematic diagram. Numerous parametric studies of trial systems were run, in three different rounds of analysis. For the cost study, the direct cost of the isolation technology was combined with the estimated collateral construction costs based on conceptual details developed for each system.

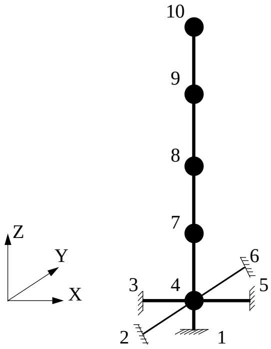

Figure 5: Stick Model

Eventually, through these numerous parametric studies of the various systems, four systems were found to provide acceptable performance. From these four the final system selection was made on the basis of lowest cost, least displacement, and optimal building performance in both the DBE and MCE events. All four systems were either elastomeric or lead-rubber, with one high damping rubber system including a limited number of supplemental dampers.

The final system selected is a high-damping elastomeric system supplied by Andre, combined with 12 viscous dampers installed in both orthogonal directions. See Figure 6 for layout of the isolators and dampers. There is a total of 134 isolators in the final system. Through a hard bid process Taylor Devices Inc. was selected to supply the fluid viscous dampers.

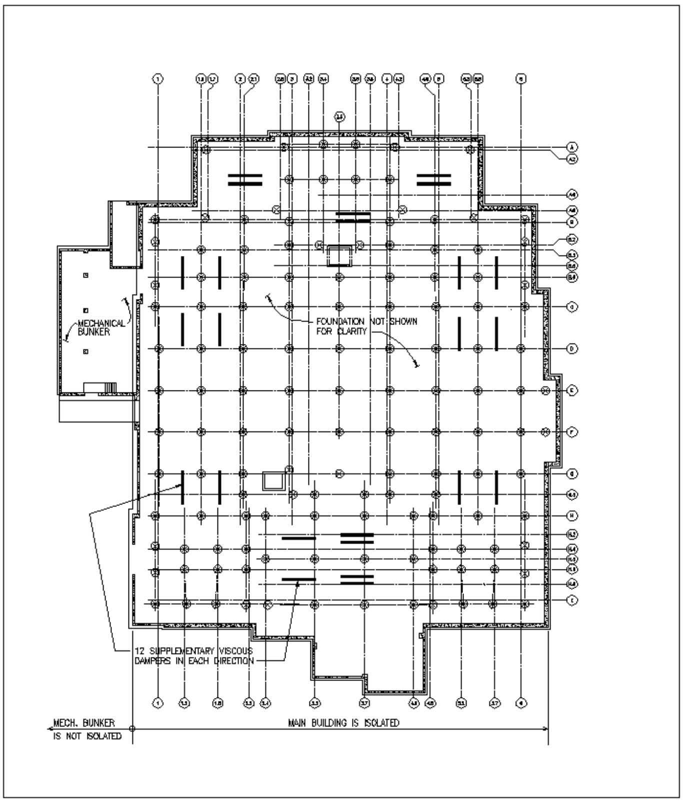

Figure 6: Isolator & Damper Layout

The design and construction of the isolation system, which includes the foundation, isolator/damper installation, and first floor construction is very complex, compared with most existing buildings where seismic isolation has been installed, due to two principal factors. First, the existing first floor elevation must be maintained to preserve the first floor height for programming purposes, and to minimize the impact on existing architectural/historical elements. Second, the Hearst Memorial Mining Building is primarily a masonry bearing wall structure rather than a steel frame structure, which requires more complex temporary shoring techniques.

Since the first floor elevation must be maintained, the entire foundation, isolator space, and first floor construction is pushed down below the current slab-on-grade. The depth of the new first floor and foundations is particularly great, since these elements must resist the large moments induced by isolators that are expected to displace as much as 27 inches in an MCE event. The bottom of new footings will be placed as much as 11 feet below the original first floor elevation. This is generally well below the bottom of existing foundations, which means that the existing foundation must be completely removed and replaced with a new foundation system. This also requires a two-step load transfer sequence, first to transfer the building loads to a temporary support system while foundations are removed and replaced, and second to transfer the loads back to the new foundations.

These complexities also require a very complex sequential construction procedure to install temporary shoring, perform load transfer, construct new foundation, install isolation bearings and dampers, construct the new concrete first floor system, and retransfer the load to the new foundations, all of which is too complex to describe fully here. The exact system is a joint design effort by Rutherford & Chekene and the General and Sub-Contractors, which is an ongoing and daily activity.

One of the most interesting elements of the project, mentioned previously, is the Guastavino ceiling in the Memorial Gallery. This system is among the most significant of historic elements. This vaulted ceiling is a brittle system composed of alternating layers of clay tiles and mortar which has significant capacity under gravity load but is considered a serious falling hazard in the event of an earthquake. This proprietary system was designed and installed by the "Guastavino Fire Proof Construction Company" that employed their unique system in over a thousand buildings located across the United States over a span of thirty years around the turn of the century. Due to the hazard posed by this system and due to its unique construction and historic importance, a unique system has been developed jointly by Rutherford & Chekene and William Krysler and Associates to strengthen the ceiling system in a non-intrusive way. The final selected system utilizes shop fabricated fiberglass ribs, a wire mesh/insulating foam backing system, and polymer pins connecting the facing tiles to the backing system. Extensive finite element analysis has been performed to develop and validate the system. Additionally testing has been performed to verify the visual appearance of the face tile attachment to assure an acceptable visual appearance and architectural quality.

The construction process is now under way, and in a short period of time the load-transfer system will be installed. The overall construction period is expected to be approximately 36 months, and the construction cost is planned at approximately $49,000,000. The first shipment of elastomeric bearings has arrived in the U.S.. The project architect is NBBJ Architects and the General Contractor is Turner Construction.

CONCLUSIONS

This project demonstrates the efficacy of seismic isolation as a strategy for the seismic strengthening of major unreinforced masonry buildings, even in a near-fault location. Faced with the requirements for acceptable performance and protection for the existing structure in a DBE event, it is necessary to utilize not only the latest analytical tools, like SAP 2000, but it is equally necessary to make use of current research and guidelines, such as FEMA 273, that make it possible not only to design an effective structural system, but also to preserve and respect the architectural heritage of important buildings such as the Hearst Memorial Mining Building.

The process of analysis, design, and isolator selection used in this project also demonstrates that it is possible to design and select an effective isolation system while concurrently considering the building performance, construction cost, and architectural and programmatic impacts. This project demonstrates that even in a public institution environment it is possible to achieve both minimal architectural intervention and an appropriate level of seismic risk reduction.

REFERENCES

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