REHABILITATION OF THE CALIFORNIA STATE CAPITOL

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Abstract

Structural evaluation, in 1974, of the historic California State Capitol identified a number of deficiencies in the 100-year old unreinforced masonry structure with respect to the seismic hazard at the site. Extensive structural and functional rehabilitation of the building was performed while retaining the historic exterior of the building and the interior rotunda. The preliminary structural design was in accordance with the California State Building Code, Title 24. The results of a site-specific seismicity study by the California Department of Transportation were utilized to perform soil-structure interaction analyses to obtain ground motion at the foundation level. Linear dynamic analyses with this motion provided close correlation with the preliminary design.

Introduction

Early in 1972, John Blume, president of URS/John A. Blume and Associates, received a call from OSA inviting him to visit the Capitol with two of their engineers that had identified several areas of concern in the historic structure (Figure 1). John asked the author and the late Don Teixeira to go with him. Our visit confirmed the fact that there was cause for concern regarding the integrity of the unreinforced masonry walls and the inner dome and tension ring in the rotunda area. After OSA issued their report in June of 1972, the firm of VTN was asked to do a more comprehensive investigation, including testing of the brick and mortar. Their report, issued in March of 1973, also confirmed the vulnerability of the building and the Legislature closed the Capitol to the public.

Since the Capitol was badly in need of functional as well as structural rehabilitation, the State Legislature retained the firm of Welton Beckett and Associates to develop alternative concepts for the rehabilitation of both the east and west wings of the Capitol. We;lton Beckett retained URS/Blume as structural consultants for the project. The WBA report was issued in October of 1974 and the Legislature selected the recommended concept for implementation. Approximately $40,000,000 was appropriated for the program of which $15,000,000 was earmarked for the structural rehabilitation.

At this point the Legislature did something that was very unusual for a public agency-- the Joint Rules Committee, representing the state Legislature negotiated design and construction contracts concurrently for what turned out to be a very successful experiment in partnering. Welton Beckett was awarded the design contract and Continental Construction the construction contract. Again URS/Blume was the structural engineering consultant to Welton Beckett. John Worsley, a former State Architect, was appointed as Project Manager.

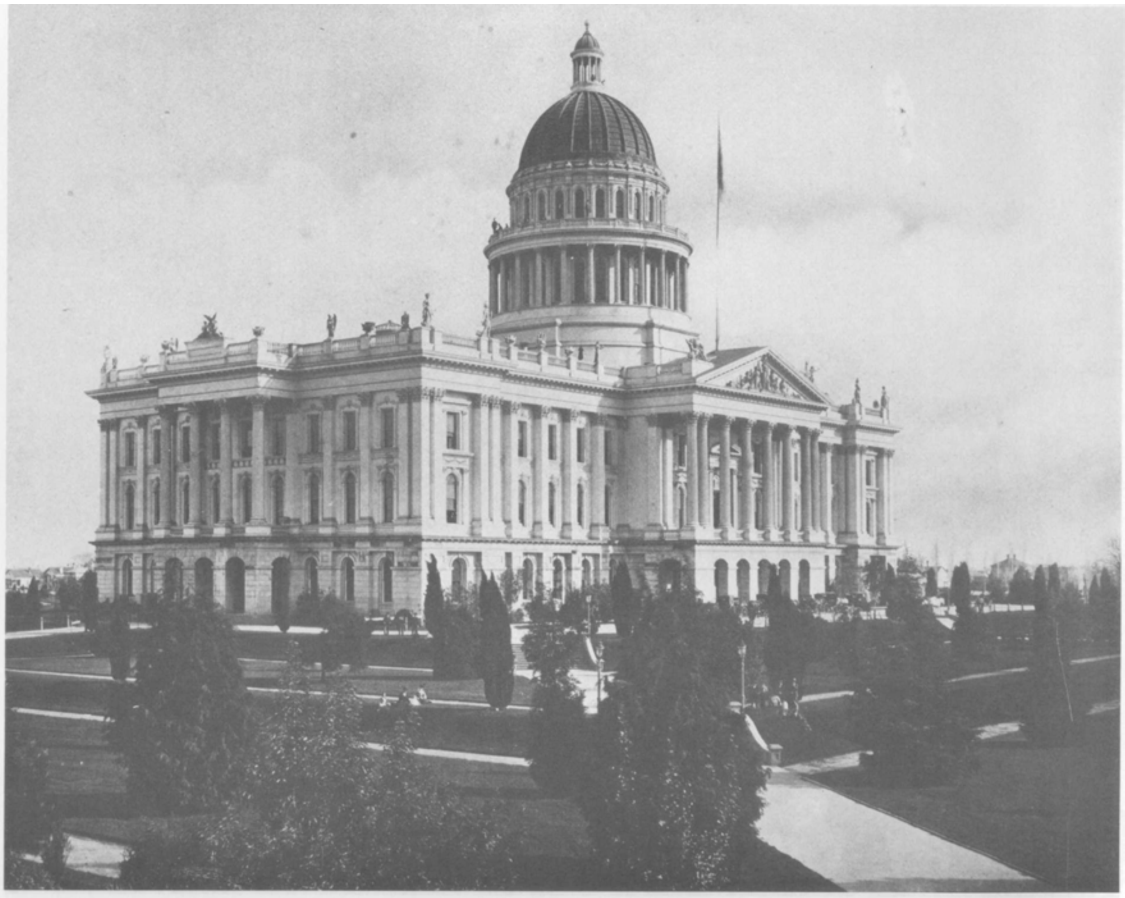

Figure 1: California State Capitol

Description of Project

The original construction of the West wing of the State Capitol was completed in 1874. The building, consisting of a basement and four stories, was constructed with massive unreinforced masonry walls and brick arch slabs supported on wrought iron beams. The URM walls of the rotunda extend 120 ft above the main roof (Figure 2). An unreinforced masonry inner dome was constructed with a springline about 10 ft above the main roof. The upper dome consisted of wrought iron trusses and wood framing surmounted by a small cupola. The walls are supported on continuous unreinforced concrete footings about 3 ft thick and up to 14 ft wide. The basement floor was a slab on grade and there was evidence of moisture seepage during the rainy season.

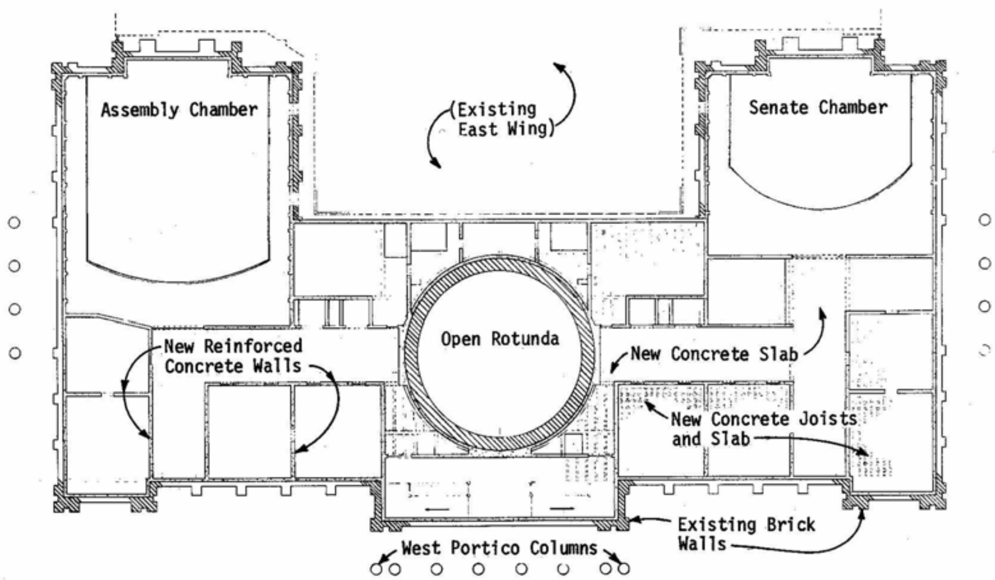

Figure 2: Third Floor Framing Plan

Because of the many structural and nonstructural alterations that had taken place over the years to accommodate the changing functional requirements, it was decided that only the external appearance and the original materials of the outer shell of the building and the interior rotunda were of primary historical significance. Actually, considerable effort was expended to remove, restore, and replace many of the original materials, including interior door and window frames, tile and terrazzo floors, and even ornamental plaster.

Since the approved concept included the removal and replacement of the interior unreinforced masonry walls and slabs except in the Rotunda area, the contractor immediately started the installation of temporary steel buttresses to support the exterior walls as he commenced demolition of the interior walls and slabs. This provided a little lead time for the structural design of the retrofit and it was managed to stay slightly ahead of the construction throughout the project. The Project Manager and the representative of thr Joint Rules Committee held weekly progress meetings at the site with the project architect and engineer to discuss and resolve any potential problems.

Two wythes of brick were removed from the interior face of the exterior walls and replaced with 12 in. of pneumatically placed reinforced concrete (Figure 3). Similarly, 12 in. of concrete was place against the outer face of the Rotunda walls. All new interior walls and floor systems were cast-in-place reinforced concrete. A new reinforced concrete ring beam was provided in the lower Colonnade and connected to the new concrete of the inner dome with reinforced concrete needle beams (Figure 4). The outer dome was replaced with new steel trusses and wood with copper sheathing. The original cupola was reinstalled on top.

The original building had four porticos, one on each side, but the east portico was removed when the East Wing was constructed in the 1960s. The portico columns as well as all the exterior window and door frames are cast iron (Figure 5). To strengthen the porticos, 12 in. of reinforced concrete was pneumatically place on the inside face of the walls, the columns were removed, filled with reinforced concrete, and replaced as a portion of new reinforced columns extending from the roof to the foundations.

Geotechnical Investigations

In October of 1974, Caltrans issued a report on foundation and seismic investigations that they had performed for the Capitol. The report contained:

- Results of a seismicity study
- Analysis of soil bearing capacities
- Ground response analyses
- Evaluation of liquefaction and settlement potential
- Estimates of dewatering requirements for groundwater

Soil borings by Caltrans at the site disclosed that the upper 5 to 10 ft contained sand and silt with some boulders and rubble. The next 15 to 30 ft was a clayey silt underlain by an additional 8 to 15 ft of sand and gravel. Alternate layers of clayey sand, sand, and silt extended to a competent sand and gravel layer at a depth of 120 ft. Rock under the site was expected to be at depths of 250 to 350 ft.

Analysis of Soil-Structure Interaction

The seismic site response analysis performed by Caltrans was based on postulated 7.0 earthquake on the Midland Fault at a distance of 24 miles and an 8.0 on the San Andreas Fault at a distance of 80 miles. The ground motion from both events was attenuated to the site and this free field spectrum was proposed for design (Figure 6). URS/Blume suspected that a free field time history had been used at the rock level to generate this free field spectrum so that the short period acceleration was effectively filtered out twice. To compensate for this, it was proposed to envelope the Caltrans spectrum with the standard 1 sigma spectral shape the firm had developed for the AEC for design of nuclear power plants.

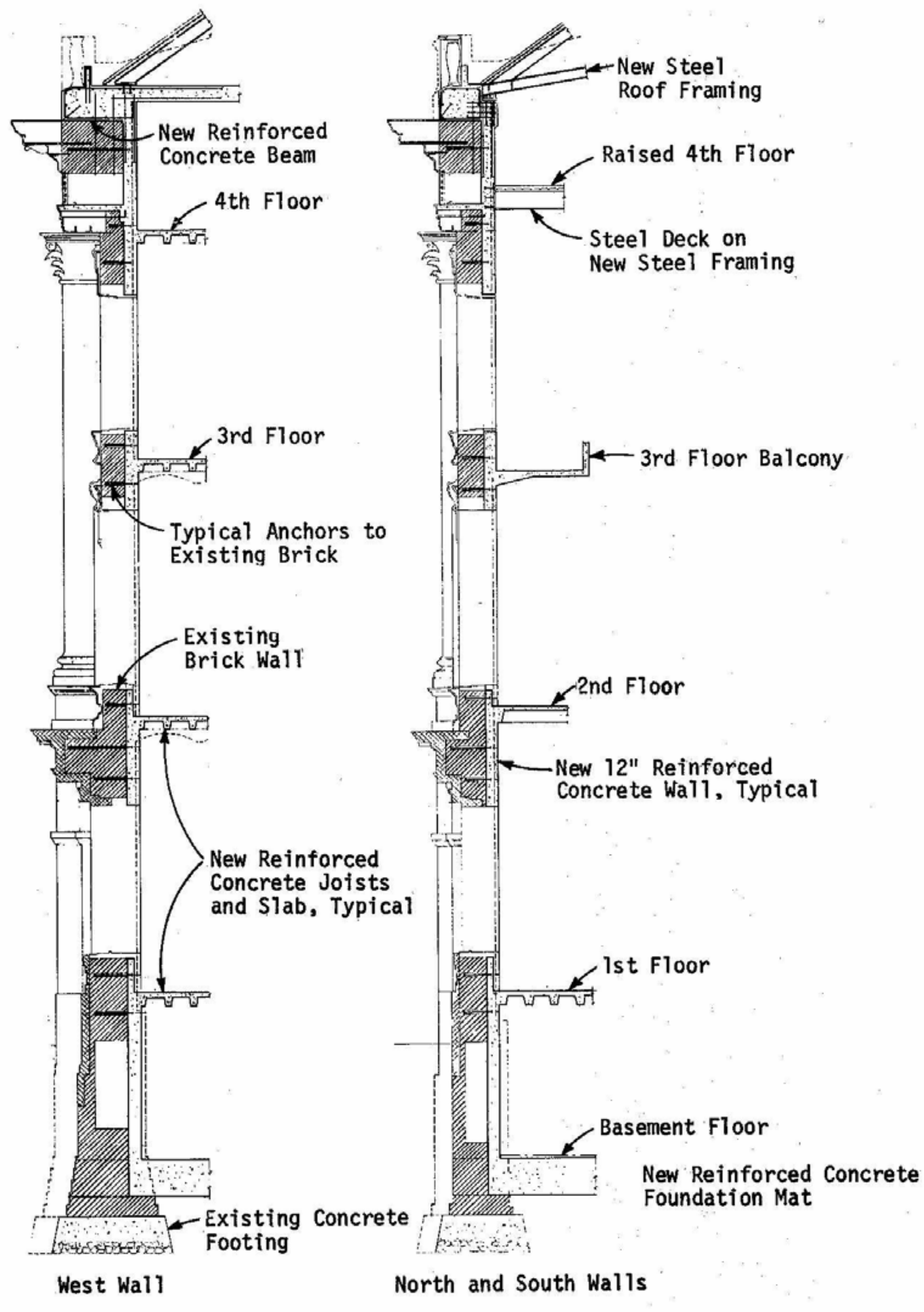

Figure 3: Typical Wall Sections

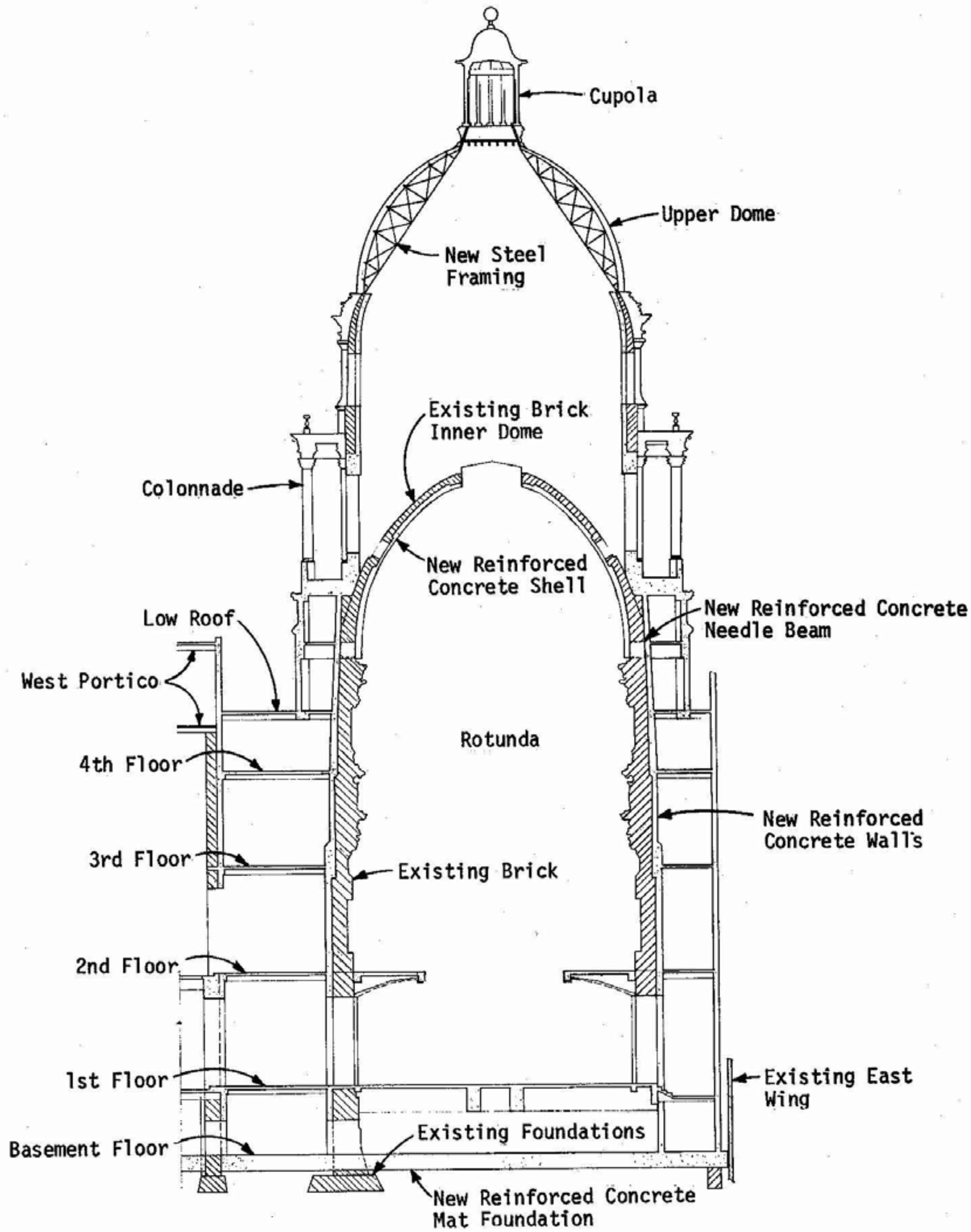

Figure 4: Typical Section Through Rotunda and Domes

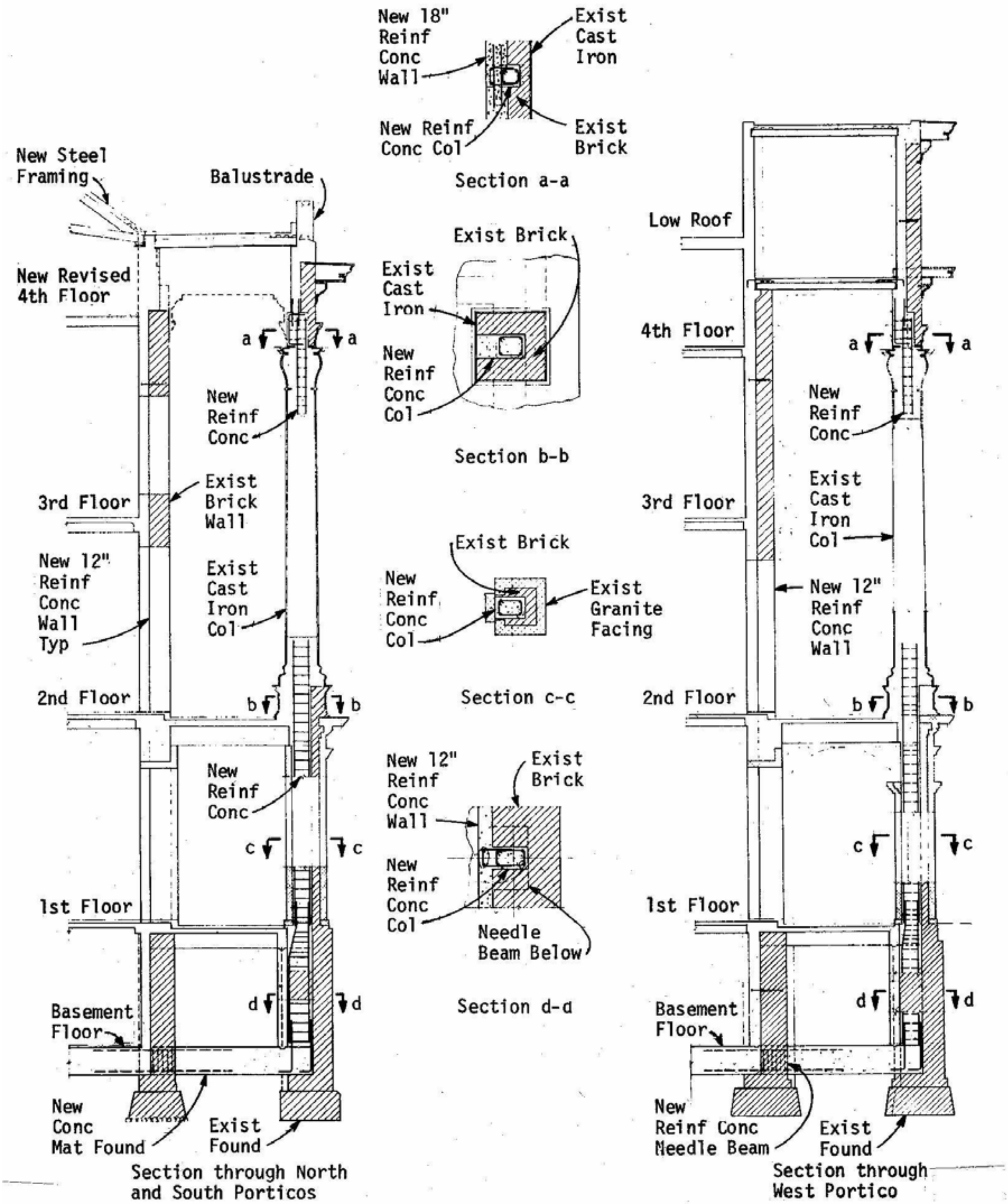

Figure 5: Typical Sections Through Porticos

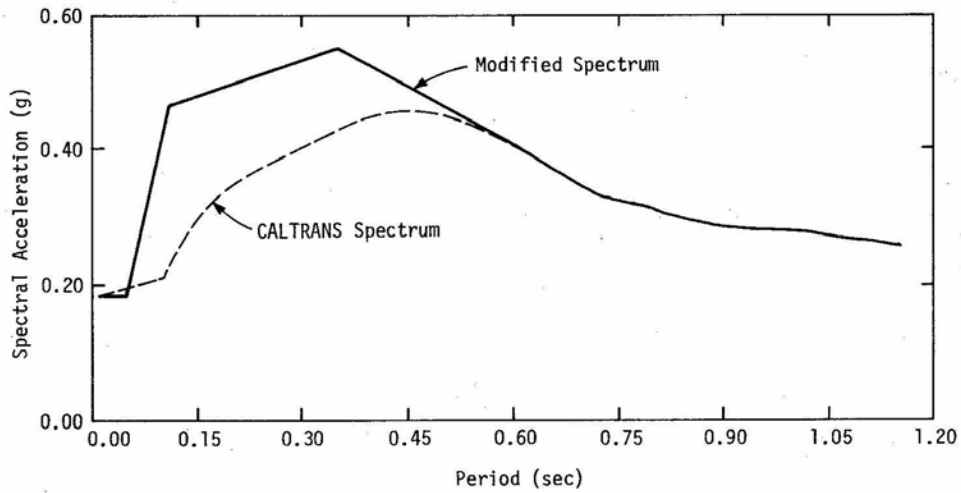

Figure 6: Modified Free-Field Spectrum

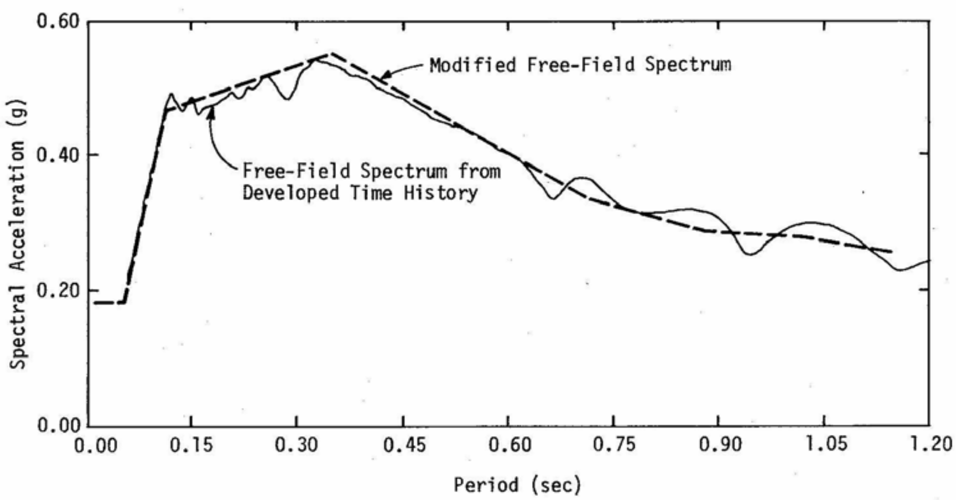

Figure 7: Free-Field Spectrum From Developed Time History

Caltrans agreed to this modification and this modified spectrum was used as the target spectrum for the soil- structure interaction analysis. Using the in-house program SMSPC, a time history was developed to match the target spectrum (Figure 7). A finite model of the soil column was developed with 28 layers and assigned these dynamic properties to each layer (Table 1). We now deconvoluted the time history down through the soil column and obtained a time history at the rock level. A lumped mass model for each direction of the retrofitted building was developed with the appropriate stiffness between each mass point and with the appropriate width of the foundation mat to detect any tendency for rocking. For the east-west analysis, the mass and width of the adjacent East Wing was included to detect its effect on the response of the West Wing (Figure 8). The effect turned out to be negligible. This soil-structure model was now subjected to the time history at the rock level and a new time history and response spectrum was generated at the foundation level. This spectrum which was used to design the retrofit, turned out to be 80 to 85 percent lower than the free field spectrum.

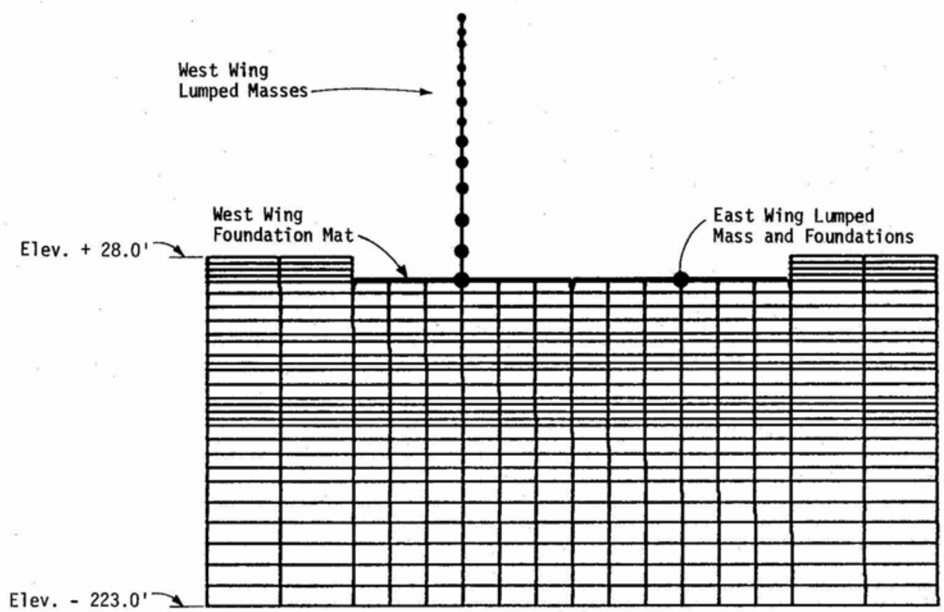

Figure 8: Idealized Soil-Structure Model, East-West Direction

Table 1: Dynamic Soil Properties for Finite Element Model

Material	Elevation (ft)	Layer	Type	Thickness (ft)	In-Situ Shear Modulus (ksf)	Effective Shear Strain (%)	Effective Shear Modulus (ksf)	Effective Damping (%)
	+28.0							
		1	1	4.5	2380	0.00257	1428	3.3
Clayey Silt		2	1	4.5	2380	0.01213	896	5.0
		3	1	4.5	2380	0.01488	1180	5.4
		4	1	4.5	2380	0.01599	1487	5.5
	+2.0	5	1	8.0	7300	0.01145	2805	4.9
		6	2	10.0	12300	0.00456	9499	4.0
Silty Sand		7	2	10.0	13800	0.00536	10329	4.3
	-28.0	8	2	10.0	14700	0.00631	10651	4.6
Clayey Silt	-33.0	9	1	5.0	6400	0.05767	1312	7.9
Silty Sand	-43.0	10	2	10.0	9700	0.01462	5689	7.0
		11	1	5.0	7570	0.05885	1535	8.0
		12	1	5.0	7570	0.06375	1479	8.2
		13	1	10.0	15400	0.02076	4808	5.9
Sandy Clay		14	1	10.0	15400	0.02336	4586	6.0
		15	1	5.0	6630	0.12153	924	10.0
		16	1	5.0	6630	0.12664	906	10.2
		17	1	5.0	6630	0.13069	892	10.3
	-93.0	18	1	5.0	6630	0.13363	883	10.4
		19	2	10.0	14000	0.01459	8207	7.0
Sand and Gravel		20	2	10.0	14000	0.01528	8085	7.2
		21	2	10.0	14800	0.01492	8612	7.1
	-133.0	22	2	10.0	14800	0.01592	8436	7.4
		23	2	15.0	17600	0.01346	10582	6.7
Silty Sand with Gravel		24	2	15.0	17600	0.01494	10246	7.1
		25	2	15.0	18000	0.01599	10258	7.4
	-193.0	26	2	15.0	18000	0.01755	9946	7.7
		27	2	15.0	23500	0.01288	14325	6.6
Rock	-223.0	28	2	15.0	23500	0.01391	13990	6.9

Rehabilitation of the Inner Dome

The new inner dome was designed to resist all of the vertical loads and to act as a diaphragm to resist the lateral loads at that level. The preliminary analysis was performed with the AXIDYN program to determine tentative concrete thickness and reinforcement. With this information this 3-dimensional finite element model was developed for analysis with the SAP IV program (Figure 9). The results generally confirmed the AXIDYN analysis, but provided more capability to define the boundary conditions and the penetrations more realistically.

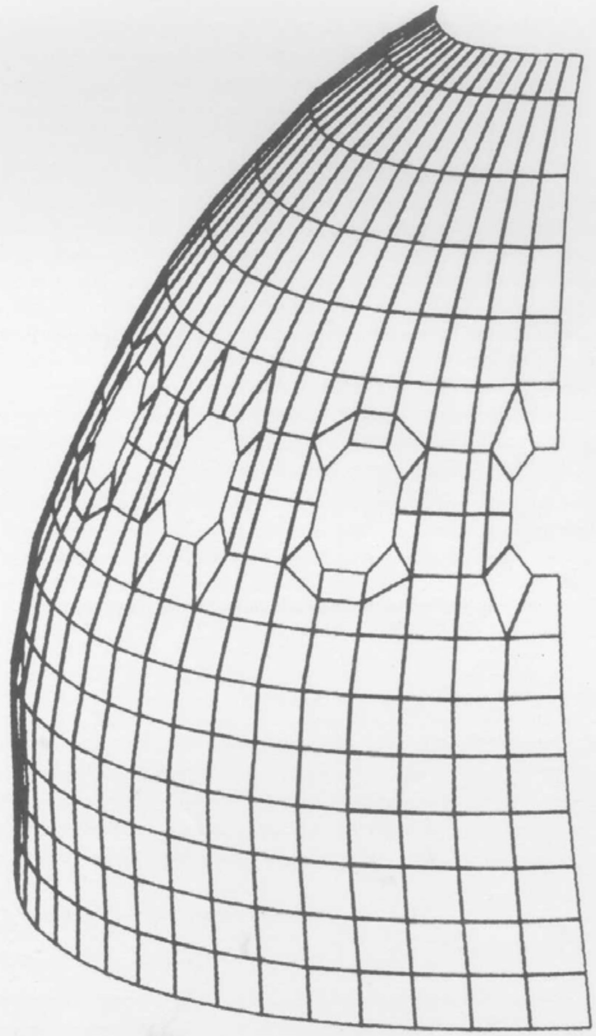

Figure 9: Finite Element Model of Concrete Shell at Inner Dome (Quarter Symmetry)

Applicable Code Provisions

The 1973 Uniform Building Code (UBC) was the applicable building code during this project. Pertinent seismic provisions of that code are summarized in Table 2.

Table 2 - 1973 UBC

V = ZKCW
Z = Zone coefficient, for Sacramento in zone 3, Z = 1.0.
K = 1.33 for load bearing shear wall building.
C = Response factor, $0.05/T^{1/3}$.

V = 0.116W N-S
= 0.110W E-W

U = 1.4(DL +LL) + 1.4E
= 0.9D + 1.4E
2.8E for shear and torsion

The 1974 recommendations of the Structural Engineers Association of California, which became the seismic provisions of the 1976 UBC are summarized in Table 3.

Table 3 - 1974 SEAOC

V = ZIKCSW
Z = 0.75 for zone 3.
I = 1.5 for essential facility
K = 1.33
C = $1/15T^{1/2}$
S = Soil Factor, $1.0 + T/T_s - 0.5(T/T_s)^2$. = 1.57.

V = 0.203W N-S
= 0.206W E-W

U = 1.4(DL + LL) + 1.4E
= 0.9D + 1.4 E
2.0 E for shear and torsion

The California State Building Code, Title 24, is applicable to schools, hospitals, and state-owned public buildings. The two alternative seismic analysis provisions permitted by this document at the time are indicated in Table 4.

Table 4 - California Code, Title 24

Method A. Dynamic analysis based on ground motion prescribed for the site in a geotechnical report The report shall consider the seismic event that may be postulated with a reasonable confidence level within a 100-year period.

Method B. Static analysis that may be used in lieu of Method A for structures that are less than 160 ft in height and that do not have highly irregular shapes, large differences in lateral resistance or stiffness between adjacent stories, or other unusual structural features.

The calculation of base shear and story forces specified under Method B is the same as for the 1973 UBC, except that a K coefficient of 3.00 is to be used for all buildings with the product KC limited to 0.25. The Code further prescribes that the base shear resulting from a Method A analysis shall not be less than 80 percent of that calculated by Method B.

Comparison of Design Criteria

The initial design, prior to the soil-structure interaction analysis, was in accordance with Method B of Title 24 using a linear static analysis with the ETABS program. The design was also checked with an ETABS dynamic analysis using the foundation response spectrum. This analysis complied with Method A in Title 24. In the design for this analysis a load factor of 1.4 for dead and live loads was used but, because of the deterministic seismic analysis, only 1.0 for seismic loads was used with 1.5 for shear and torsion.

Table 5 compares the results of the various criteria. It should be noted that the building codes permit a one-third increase for load combinations with seismic forces while no increase was taken for our spectral response analysis. When this is taken into account, our analysis compares very favorably with Title 24 and is substantially more conservative than the 1976 UBC.

Table 5: Comparison of East-West Seismic Story Shear With Various Code Provisions

	1973 *UBC*			1973 *UBC* SEAOC Setback Provisions			1976 *UBC* I = 1.5			California Code for Hospital Facilities Method B			Spectral Response (RSS)	
	U=1.0	U=1.4	U=2.8	U=1.0	U=1.4	U=2.8	U=1.0	U=1.4	U=2.0	U=1.0	U=1.4	U=2.1	U=1.0	U=1.5
12	176	264	493	246	344	689	549	769	1,098	650	910	1,365	678	1,017
11	358	501	1,002	501	701	1,402	1,159	1,623	2,318	1,371	1,919	2,879	1,431	2,147
10	535	749	1,498	749	1,049	2,097	1,680	2,352	3,360	1,989	2,785	4,177	2,075	3,113
9	671	939	1,879	939	1,315	2,629	2,149	3,009	4,298	2,543	3,560	5,340	2,653	3,980
8	717	1,003	2,008	1,003	1,404	2,808	2,300	3,220	4,600	2,722	3,811	5,716	2,840	4,260
7	1,040	1,456	2,912	1,455	2,037	4,074	3,550	4,970	7,100	4,201	5,881	8,822	4,383	6,575
6	1,150	1,610	3,220	1,609	2,253	4,505	4,281	5,993	8,562	5,066	7,092	10,639	5,286	7,929
5	2,986	4,180	8,360	3,445	4,823	9,646	6,931	9,703	13,862	8,202	11,483	17,224	8,558	12,837
4	4,668	6,535	13,070	5,127	7,178	14,356	9,511	13,315	19,022	11,255	15,757	23,636	11,744	17,616
3	5,962	8,347	16,694	6,421	8,989	17,979	11,503	16,104	23,006	13,613	19,058	28,587	14,204	21,306
2	6,926	9,696	19,393	7,385	10,339	20,678	13,006	18,208	26,012	15,391	21,547	32,321	16,059	24,089
1	--	--	--	--	--	--	--	--	--	--	--	--	16,672	25,008
B	--	--	--	--	--	--	--	--	--	--	--	--	--	--
C_B*	0.110	0.154	0.307	0.117	0.164	0.328	0.206	0.288	0.412	0.244	0.341	0.512	0.254	0.382

*Base shear coefficient at 1st floor level.

In conclusion, by today's standards for historic buildings we probably would have been forced into more restoration and less reconstruction. Perhaps base isolation would have helped, but probably some reconstruction could not have been avoided, particularly in the rotunda and dome area. Judging from cost estimates made for similar monumental historic buildings of unreinforced masonry, base isolation, while providing more opportunity for preservation and restoration, has generally resulted in a significant increase in the cost of rehabilitation.