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SEISMIC DESIGN AND ANALYSIS OF CALIFORNIA ACADEMY OF SCIENCES BUILDING ALLOWING FOUNDATION UPLIFT (ROCKING)

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ABSTRACT

This paper discusses the innovative seismic design approach employed for the new California Academy of Sciences (CAS) Building in San Francisco. The new CAS will incorporate three new 3-story buildings; one existing 2-story building; and an Exhibit Area, which includes two large spheres housing the Planetarium and the Rainforest. The piazza floor slab is fully contiguous and interconnects the four buildings. The entire structure will be enclosed by a “green” undulating grass-covered roof. Once completed, the 370,000 square foot CAS building will be the only museum in the US to house a planetarium, aquarium, rainforest and research facilities under one roof.

A standard code approach required the use of ground anchors to prevent the building from overturning during a seismic event. This paper details the methods used by Arup engineers to validate the removal of such anchors and allowing the building to rock during a seismic event. Using fundamental design principals, it was determined that the undulating roof, supported by the four main building components, would remain stable when subject to earthquake forces. Non-linear dynamic time history analyses were employed to prove that the global behavior of the building was not significantly affected by permitting limited uplift of the foundation. The paper further discusses the performance benefits of adopting this innovative approach and the significant reduction in foundation costs which resulted from the elimination of ground anchors.

Introduction

Located in the heart of Golden Gate Park, San Francisco, the California Academy of Sciences (CAS) is nestled amid 100-year old trees and expansive lawns. Founded in 1853 by a group of naturalists studying the Californian resources, the Academy is now one of the 10 largest natural history museums in the world. The Academy’s mission - to explore, explain and protect the natural world - requires the development of a unique building which combines a public museum and international research facility.

The original Academy buildings were constructed between 1916 and 1990. The need for seismic upgrade and repair of concrete suffering deterioration (due to saltwater exposure in the aquarium) was seen as a unique opportunity to celebrate the 150th anniversary of the academy with a new home, constructed on the original site.

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The new three-story Academy shown in Fig. 1 extends over 410,000 sq ft. An undulating grass roof shelters the new building while being an exuberant exhibit itself, echoing the surrounding landscape. Tucked beneath the rolling green roof are four 'wings' that house galleries and research space. In the open central plaza two spherical bolas house the planetarium and rain forest. Below grade, one full and one partial basement level house the building's power, a central utility plant, and aquaria water treatment facility. Large expanses of glazing will link the interior with the surrounding park and publicly accessible laboratory areas will involve visitors in the Academy's research. The plan above plaza level is shown in Fig. 2.

Figure 1. Architectural Rendering of the new Academy.

Deliberate design decisions bind every aspect of the building to the surrounding environment. The intention to be integrated with the environment whilst pushing the boundaries of conventional design can be seen in the green roof, platinum LEED accreditation goal, and overall seismic system.

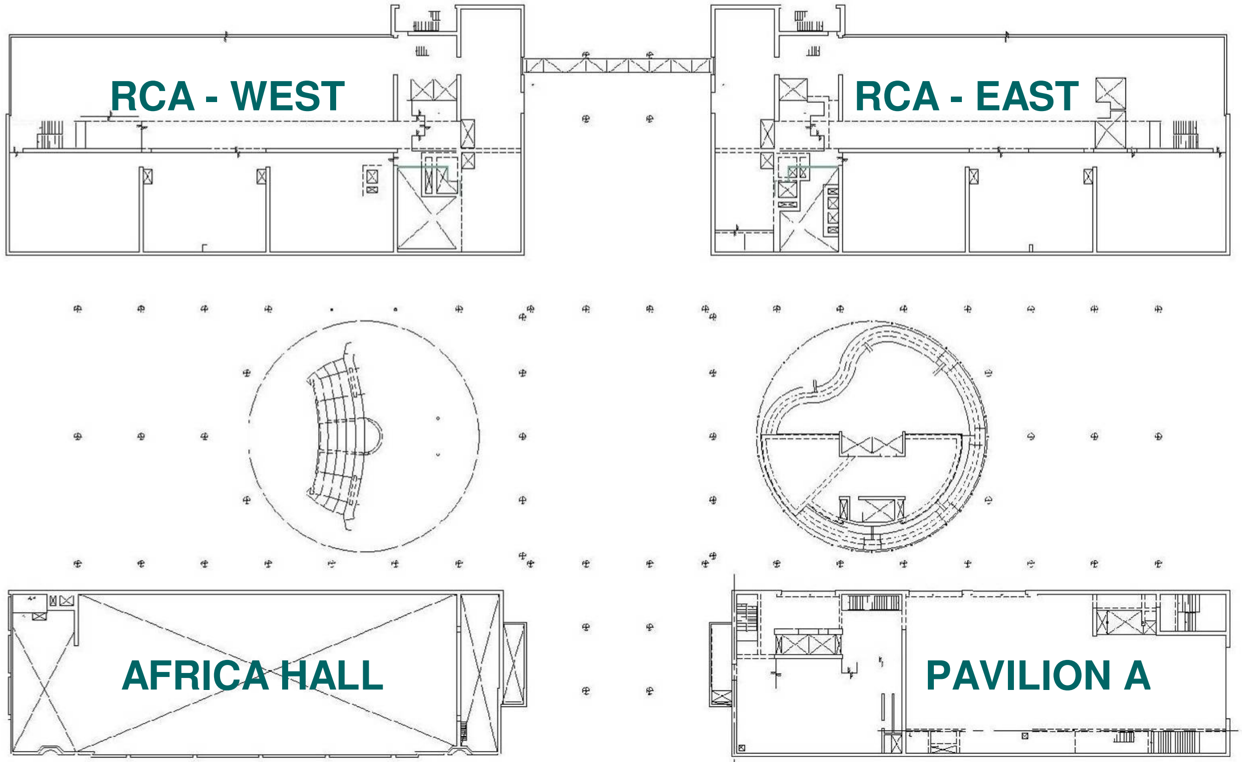

Figure 2. Plan of the Academy.

Seismic Design

Code Based Approach

The strong architectural aesthetic called for a uniform grid structure for the roof, with a prominent expression of grid lines in the North-South direction. This immediately provided a challenge as it precluded the most obvious structural system around the domes - a ring beam to gather the arch thrust forces.

Various design concepts were explored early on, from base isolation to roof isolation to tying the four wings together. There was much experimentation and debate among the design team about how to deal with the undulating roof. Many studies were undertaken to understand and optimize the structural system, which was heavily driven by the strong architectural aesthetic.

The seismic concept agreed upon can be considered similar to the structure of a table. The four wings act as table legs and the roof as the table top. The roof and ground floor slab tie the table legs together to ensure the building acts as a whole. This roof comprises a 5” concrete slab over steel beams typically supported on a 48 x 24 ft column grid, except in the dome areas, where beams arch up to 96ft intervals in the north-south direction. During an earthquake, lateral forces are transferred through the roof and floor slabs in to 18” reinforced concrete shear walls in the four wings and basement.

The interaction between the four wings and the roof was key to the seismic design of the new building. The roof structure is reasonably flexible largely due to the curved dome structures with glazed penetrations and the large central space for the elegant piazza glass canopy. The living green expanse of the roof must roll and expand with an earthquake to accommodate any differential movements between the wings without compromising its stability.

Working in close collaboration with the Academy and design team performance criteria were established for the building based upon the acceptability of damage. The basic design was to the California Building Code (CBC) 2001 incorporating the San Francisco Building Code (SFBC) 2001 amendments.

The site lies within 8km of the San Andreas Fault, so is likely to be subject to very strong ground shaking. The soil is sandy and classified as type S_D per CBC classification. The likely ground motion was predicted through a site-specific seismic hazard assessment by Rutherford & Chekene (2004). This provided the site-specific response spectra corresponding to the design basis earthquake (DBE) (1 in 475 year return period earthquake) and the ultimate basis earthquake (UBE) (1 in 950 year return period earthquake) which are shown below in Fig. 3. The performance criteria for the DBE and UBE are life safety and collapse prevention, respectively.

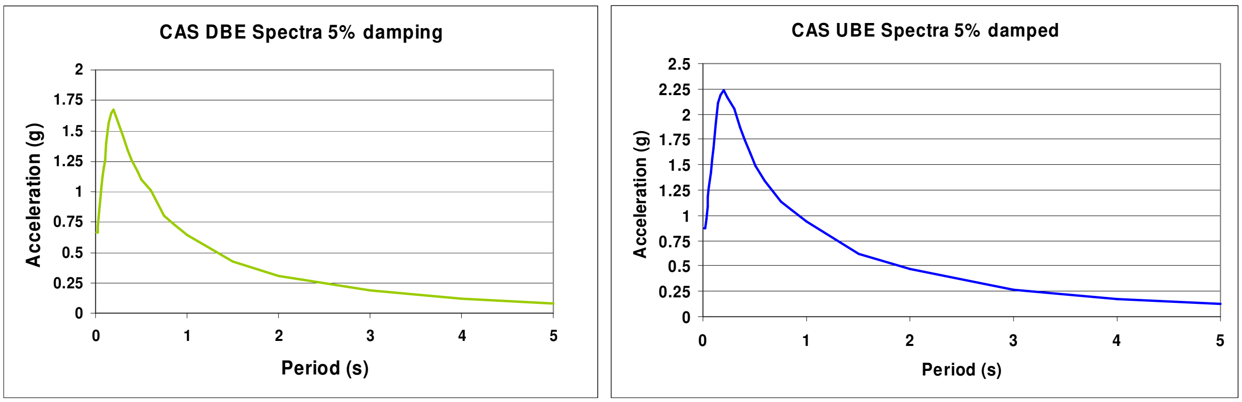

Figure 3. Site-specific spectra for DBE and UBE

Code Based to Rocking Approach

The code compliant analyses and design was completed using ETABS8 (2004). This included linear response spectra analyses.

To completely capture the complex interaction of the four wings and the roof structure during an earthquake a number of sensitivity analyses were performed in order to bracket the design. These analyses provided a range of design scenarios and eventualities for which the building was ultimately designed. These included the following:

- Imposing a 1” differential displacement on the roof in both directions to account for any differential movement between wings. This resulted in providing additional redundant truss elements over the north and south entry areas.
- The wings were checked independently to make sure they could carry their own attributed seismic mass
- The steel beams supporting the roof were designed to resist load in the unlikely event that all of the concrete on the roof cracked and was not contributing to the strength of the roof.
- The construction sequence of the building will significantly influence the initial stresses that are locked in to the structure. In order to limit additional stresses and movements it was decided to prop the domes during construction.

The new Academy is a very stiff and squat building, so has a short natural period (0.12 seconds in the longitudinal (East-West) direction and 0.20 seconds in the transverse (North-South) direction). The seismic weight is approximately 121,000 kips.

The comparison of code level and unreduced site specific base shears are given in Table 1. For base design the site specific spectra were scaled to code level.

Table 1. Base shear from ETABs response spectrum analyses.

Earthquake	EW Base Shear Coefficient	NS Base Shear Coefficient
97UBC	0.31	0.31
Site Specific DBE	0.99	1.02

The results from these analyses indicated that a large number of the shear walls were in tension (trying to uplift) during an earthquake and the code dictates that these walls must be held down which meant that $1.5m worth of ground anchors were required.

In keeping with the sympathetic nature of the building's design whilst pushing the boundaries of conventional design, it was decided that instead of aggressively resisting an earthquake by tying it down, the building should work with the seismic forces, dissipating the energy in an elegantly simple manner – through rocking.

This is not a new idea, many engineers have hypothesized the benefits of allowing buildings to "rock", but few have implemented it. During an earthquake, anchoring anything down tends to put more force in to it, so allowing it to rock a little can potentially reduce forces within the building. Potentially the building performance could be improved and savings made on foundation costs by eliminating the anchors. Obviously, the building needs to be detailed appropriately to allow for this to occur, but the additional cost of this is small compared to the potential overall benefits.

This idea was keenly embraced as it was clearly in tune with the green philosophy of the building design. The idea of the building "dancing with nature" was appealing, but first it had to be thoroughly investigated.

Initial studies exploring the local and global effects of allowing the building to rock involved static hand calculations, non-linear static pushover and linear time history (with non-linear springs). Early indications suggested the building would uplift around 1.5" during the DBE, but this was anticipated to reduce with more detailed (non-linear dynamic) analyses.

The design of the building was completed based on the results from the fixed base ETABS analyses but was verified for rocking behavior using more sophisticated non-linear software which is detailed in the next section.

Verification of Rocking Approach

Finite Element Analysis Model & Input

A performance-based approach was undertaken to validate this design concept and quantify the performance of the new Academy without ground anchors. Non-linear dynamic time history analyses were performed on the 3-dimensional finite element model shown in Fig. 5 using advanced simulation software, CEAP (2004), which is an Arup developed version of LS-DYNA. Essentially the new Academy was being "virtually tested" for an earthquake. The objectives of this study were:

a) Quantify the performance of the new building
b) Verify results of response spectrum and ETABS time history analysis
c) Verify the removal of ground anchors
d) Quantify global and local effect of allowing uplift on building

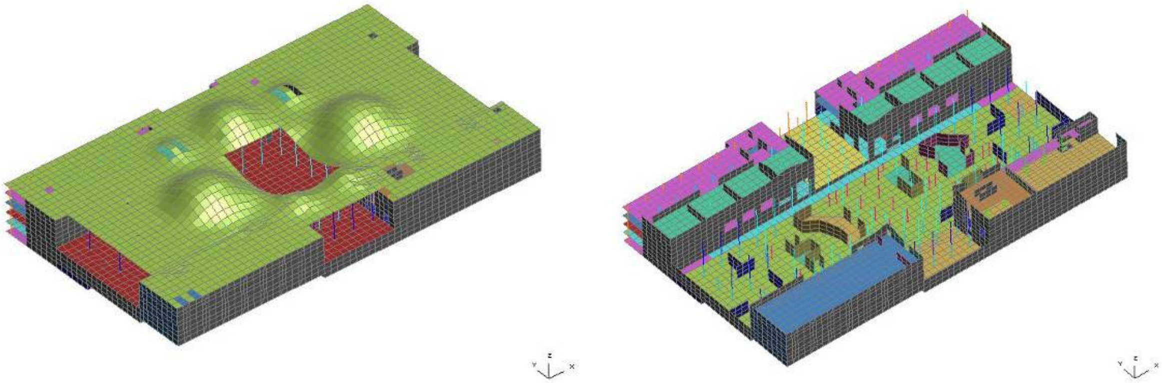

Figure 5. Non linear dynamic analysis model.

All the significant walls, beams, column and foundations of the building were included in a fully nonlinear finite element model. All elements were modeled with nonlinear material models. The shear walls and slabs capture nonlinear tensile and shear behavior – the non-linear shear behavior is illustrated in Fig. 6(a). The beams and columns are modeled using a nonlinear element which replicates the bending/axial interaction failure surface of the actual element, as illustrated in Fig. 6(b).

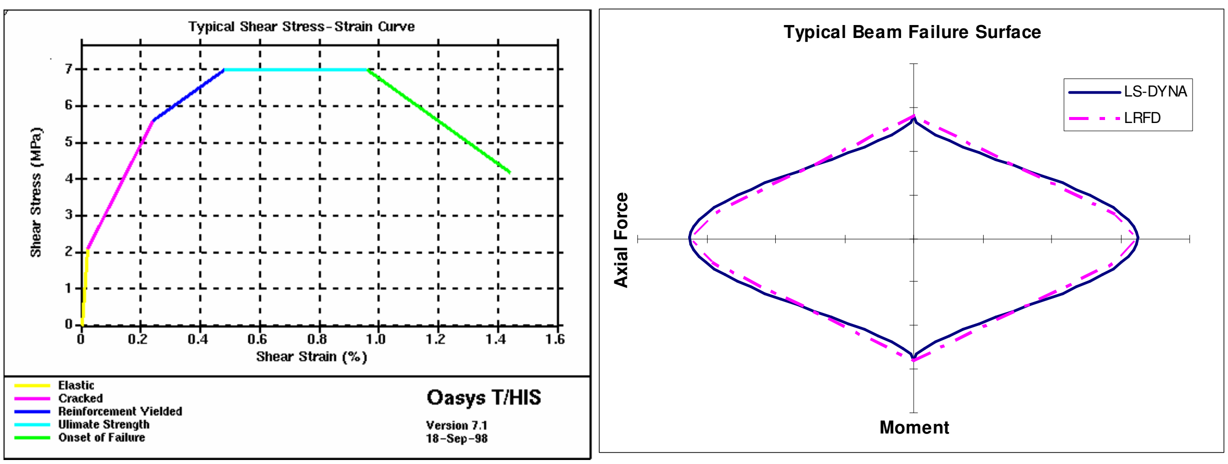

Figure 6. (a) Non-linear shear stress-strain relationship for concrete (b) Slice through typical steel beam failure surface.

The foundation was modeled by a set of nonlinear spring elements with appropriate compression stiffness but zero tension stiffness (i.e. allow uplift). These were included to determine the consequences of removing the ground anchors. The sensitivity of variation in compression stiffness was explored, along with assumptions on roof connection stiffness.

A suite of 7 tri-directional time history records were generated to match the DBE and UBE site specific response spectra. The proximity of the San Andreas Fault meant that it was important to include directivity effects (direction of rupture) in the analysis. The real earthquake time history records selected for matching are given in Table 2.

Table 2. Earthquake records used for spectral matching.

Time History	Distance (km)	Magnitude (M_w)	Directivity	DBE	UBE
Duzce 375	8.2	7.1	Average	X	X
Duzce 1062	13.3	7.1	Average	X	X
Landers MVH	19.3	7.3	Average	X	X
Chi Chi TCU046	14.3	7.6	Average	X	X
Landers CLW	21.2	7.3	Average	X	X
Imperial Valley ELC	8.3	6.3	Fault Normal	X	X
Loma Prieta LGP	6.1	6.9	Fault Normal	X	X

These spectrum compatible earthquakes were applied to the Academy to quantify its performance. The results were averaged over the seven records at DBE and UBE levels.

Results

The removal of the ground anchors allows the shear walls to uplift if necessary during a large seismic event. The maximum expected uplifts during the DBE and UBE events are 0.78" and 1.38" respectively.

Fig. 7 illustrates the amount of supports uplifting vs the amount of uplift during the DBE analyses. This shows that the majority of the supports are uplifting less than 0.2" with just a small number responsible for the maximum uplift result. This is a relatively insignificant amount of uplift.

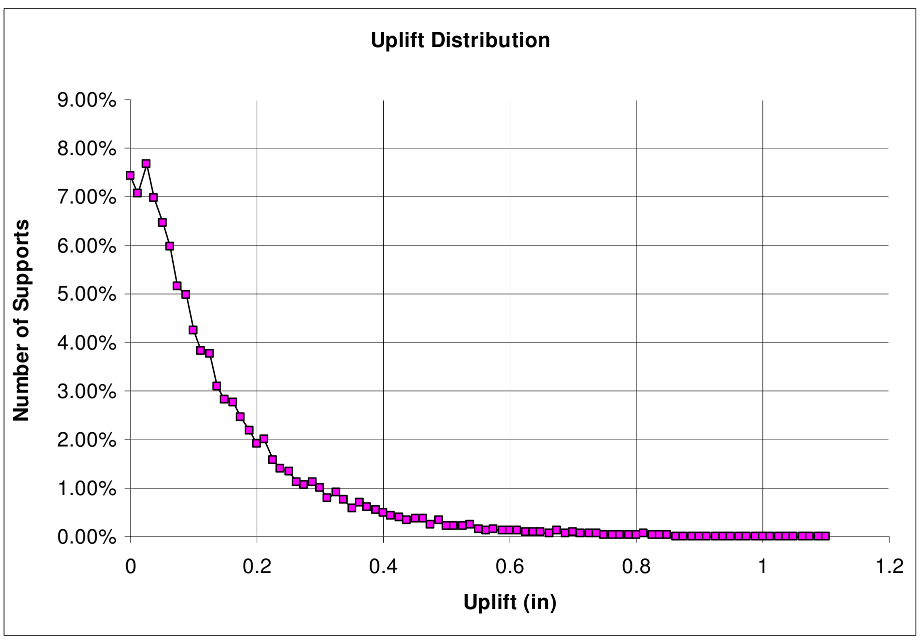

Figure 7. Distribution of number of supports vs uplift (note these numbers are unaveraged).

The maximum horizontal roof displacement in the north-south (NS) direction is 1.39" and in the east-west (EW) direction is 1.44". This is well within allowable code values. These values are compared with displacements from the ETABS response spectrum (RS) analysis and the CEAP fixed based non-linear dynamic time history (NLTH) analysis in Table 3. As would be expected, the global building horizontal displacements increase due to rocking and this is more noticeable in the shorter NS direction. The ETABS (fixed base) response spectrum analysis gives smaller displacements than the fixed base CEAP non-linear dynamic time history analysis. This could be partly due to the code's assumption of equal displacements for the ratio of elastic to inelastic response spectra in the short period range (<0.33 s). For structures in the short period range, such as the Academy, the ratio of elastic to inelastic spectra is better based on equal energy which would result in larger displacements than those given in Table 3.

The stress level in the roof is heavily influenced by the differential behavior of each of the building legs, as well as the flexibility of the contoured roof. The maximum relative displacements between the building legs is therefore an important indicator of the rigidity of the roof system and the overall behavior of the building and quantifying this actual displacement would validate the assumptions made for the code design of the building.

Table 3. Comparison of horizontal roof displacements at DBE level.

Corner Location	East-West Roof Displacement (in)			North-South Roof Displacement (in)		
	ETABS RS	CEAP NLTH		ETABS RS	CEAP NLTH	
	Fixed base	Fixed base	Rocking	Fixed base	Fixed base	Rocking
SE	0.70	0.97	1.14	0.64	0.96	0.99
SW	0.66	1.27	1.44	0.55	0.81	1.22
NE	0.17	0.07	0.14	0.31	0.18	0.49
NW	0.13	0.25	0.30	0.54	0.96	1.39

The results also predicted a maximum N-S differential building displacement of 0.76" and a maximum E-W differential building displacement of 0.73". This shows that the application of 1" differential movement in the ETABS study was appropriate and conservative.

To further illustrate the difference in global performance, a comparison between base shears for the fixed and rocking building during one of the DBE events is shown below. Fig. 8 shows twenty percent reduction in peak base shear for rocking in the North-South direction. The East-West direction showed a reduction of about ten percent compared to the fixed based case.

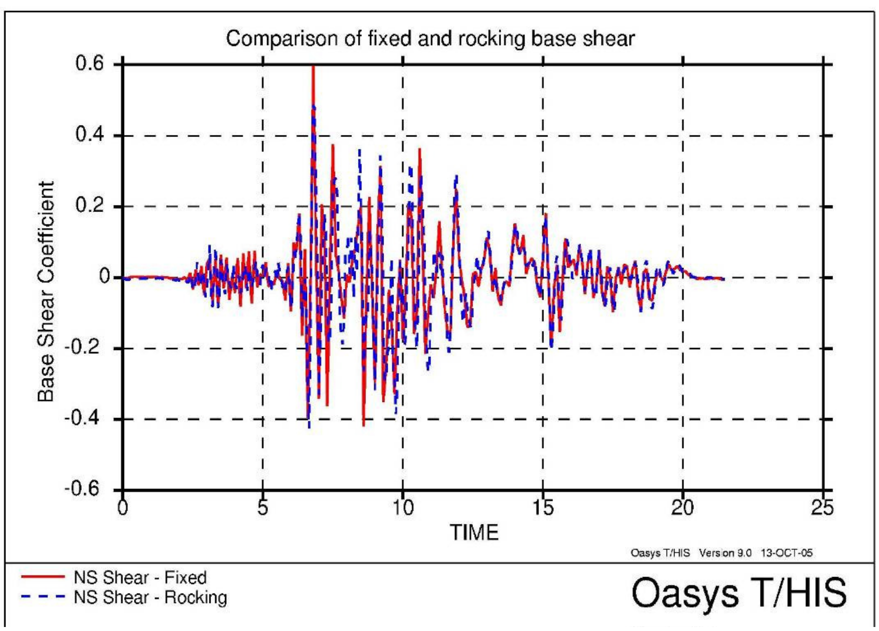

Figure 8. DBE base shear for fixed and rocking base NL TH analyses in NS direction.

In terms of element performance, the roof showed some cracking and limited yielding in the concrete, but no damage to roof beams during the DBE. The walls were shown to crack with slight yielding during the DBE and more extensive yielding during UBE. Larger grade beams were provided at foundation levels to robustly tie the building together and enable uniform rocking across the building. Fig. 9 illustrates the damage to the shear walls during the DBE. The colors correspond to the position on the non-linear shear curve; blue is elastic, green is concrete cracking, yellow is reinforcement yielding, red is ultimate strength and pink is onset of failure. Fig. 10 similarly illustrates the damage in the roof during the DBE.

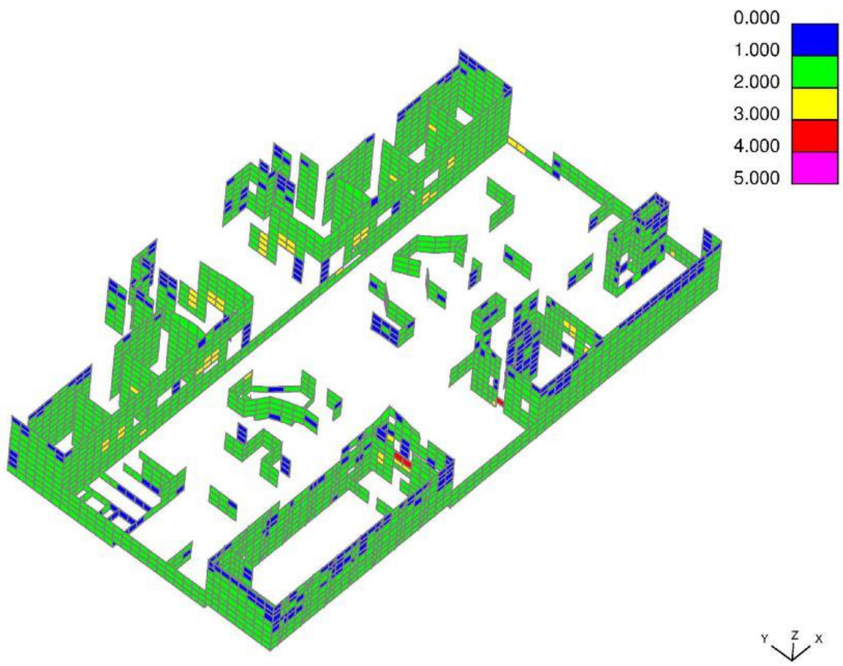

Figure 9. Damage to shear walls during DBE.

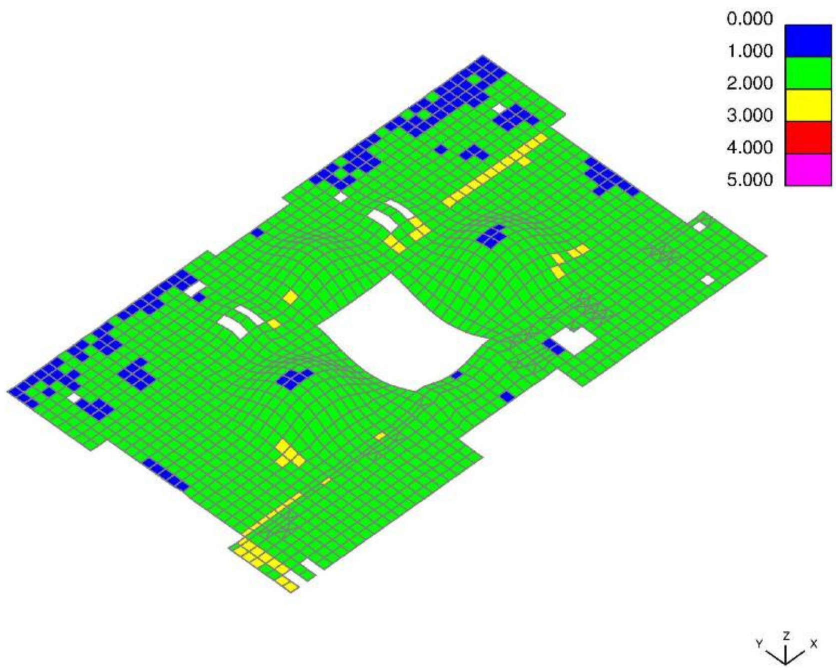

Figure 10. Damage to roof during DBE.

The results from the non-linear dynamic time history analyses verified that the code design approach and bracketed analysis performed using ETABS produced an acceptable design for the building elements, although was conservative in its requirement for ground anchors. The performance of the building was shown to meet code level standards and was improved through allowing the building to rock at its foundation.

Conclusions

Non-linear dynamic time history analyses were instrumental in verifying the “rocking” design concept.
The performance based rocking approach produced a number of benefits for the California Academy of Sciences which included:

- Significant cost saving with removal of ground anchors ~ $1.5m
- Improved performance
- Integration of seismic design concept with Academy’s “natural” concept

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