Calibration of Various Nonlinear Analysis Procedures with Earthquake Damage in a Reinforced Concrete Shear Wall Building

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SUMMARY:
Analytical prediction of earthquake damage in buildings, even after damaging events provide ample physical data of how buildings actually behaved, is typically fraught with difficulty. Though performance-based procedures for doing so have now been standardized and popularized, there is little evidence that any of these formal procedures are capable of generating reliable predictions, generally about building response, and more specifically about earthquake damage, even when wielded by some of the most able earthquake engineering professionals and academicians. After more than a quarter century of effort, earthquake damage prediction remains a holy grail of earthquake engineering even if the problem of designing earthquake-resistant structures has been largely solved. This paper describes a case study in which two six-story reinforced concrete shear wall buildings, damaged by two earthquakes off the coast of the Big Island of Hawaii, were subjected to two different analysis methods utilizing soil-structure interaction, and damage predictions generated by each were compared.

Keywords: Rocking, Reinforced Concrete, Nonlinear Modal Time History, Nonlinear Static

1. SIGNIFICANCE OF THE STUDY

Analytical prediction of earthquake damage in buildings, even after damaging events provide ample data of how buildings actually behaved, is typically fraught with difficulty. Though performance-based procedures for doing so have now been standardized and popularized, for example the US Standard ASCE 41 (ASCE 2006), there is little evidence that any of these formal procedures are capable of consistently generating reliable predictions of earthquake damage even when wielded by experienced earthquake engineering professionals and academicians. Blind predictions typically fare even worse.

Recent studies reveal the extent of the difficulties in accurately predicting building performance during strong shaking even when using the most modern detailed methods and state-of-the-practice modeling techniques and software. Maison *et al.* (2009) employed both ASCE 41 and FEMA 351, (FEMA 2000), in parallel efforts to try, after the fact, to predict the results of E-Defense shake table collapse tests of a full-scale four-story welded steel moment frame building. Using the formalized ASCE 41 nonlinear static and nonlinear dynamic methodologies, the Maison *et al.* study found that Collapse Prevention (CP) acceptance criteria were exceeded when the structure was still essentially elastic in real life. In other words, the ASCE 41 Standard *predicted* that the structure was a collapse hazard and required seismic strengthening even though when loaded on the shake table, it remained essentially undamaged at the published CP limit state. The results of the parallel analyses using FEMA 351 were just as incorrect but in the *opposite* sense. Using the FEMA 351 nonlinear procedures to compare the acceptance criteria with the test building performance, Maison *et al.* found the actual test building on the shake table to be at the CP limit state (i.e., on the verge of collapse) with drifts that were well below the “allowable” capacities defined by the document for local and global CP criteria.

Bayhan *et al.* (2011) conducted even more recent studies using ASCE 41 nonlinear static and nonlinear dynamic analysis methods, as well as ATC-40 defined performance levels (Applied

Technology Council 1996), to correlate analysis predictions with the well-documented performance of three nearly identical buildings in different cities in Turkey that were each damaged by different strong earthquakes. The buildings studied are reinforced concrete frames with deep spandrels and shear-critical captive columns. Nearby near-fault strong motion records were available for each of the buildings. The study found wide discrepancies between the predicted and documented performance of each of the buildings. The study reported that the building that experienced the most severe earthquake shaking and that was predicted to fare the worst, actually fared *far better* than the other two, and actually continued to be occupied after the earthquake as an emergency service facility. In contrast, the building that was predicted by the nonlinear dynamic procedure to have experienced only minor damage actually experienced damage that was judged to be moderate, and the building that was predicted to have experienced moderate damage actually experienced the heaviest damage of the three. Data in the paper suggests that this building was very close to experiencing or perhaps even experienced a partial collapse.

Indeed, after more than a quarter century of development of performance-based evaluation methods, earthquake damage prediction remains a holy grail of earthquake engineering even if the problem of designing earthquake-resistant structures has been largely solved. Given the oft dismal success of the profession in predicting damage, case studies in which predictive methods are employed to predict documented damage are necessary to improve our collective understanding of the strengths and weaknesses of existing methods. The balance of this paper is devoted to the presentation of a case study in which an assortment of nonlinear analysis methods was used to generate earthquake response damage predictions. In each case, the analysis methods appear to generate strikingly consistent results, the reasons for which will also be explored.

2. CASE STUDY BACKGROUND

2.1 Seismological background

On October 15, 2006, a pair of significant earthquakes, including an M6.7 earthquake shook the Big Island of Hawaii and caused damage to two buildings that are the subject of this case study. At approximately 39 km, the earthquake hypocenter of the main Kiholo Bay event was unusually deep for the region. The distance between the building complex and the epicenter was 20 km for the larger earthquake and 55 km for the smaller event that followed. A relatively sparse network of instruments, including one within about 1 km from the site of the buildings, recorded the characteristics of ground shaking across the island. This nearby instrument was located in a single-story building situated on similar soils and was taken to be the best available source for ground shaking data. The recorded peak ground acceleration (PGA) and peak ground velocity (PGV) at this station due to the M6.7 earthquake were 0.27g and 19 cm/sec, respectively. Tri-Net ShakeMap (Wald *et al*. 1999) data was also accessed and mined for relevant data, but the utility of that data for this particular earthquake and this particular study was limited for reasons that are beyond the scope of this paper.

2.2 Structural Background

The subjects of this paper are two nearly identical six-story reinforced concrete shear wall towers that rise from a common base. They were subjected to moderate ground shaking during the two 2006 events. The towers were originally constructed in 1975, meaning that the shear walls do not have all of the reinforcing detailing that would be required according to current code in similar new buildings to provide for ductile response. The typical walls are of constant dimension over the height of the building and are typically 20 cm thick although some wall segments are 35 cm thick. Stacked doorways penetrate a number of these walls, leaving typical lightly reinforced link beams between wall segments on either side of the openings. The typical floor slabs are 20 cm thick. The walls and the balance of the vertical elements in the building are founded on spread footings that bear directly on rock. The plan of the typical floor, which illustrates the shear wall and column layout, is provided in Figure 1. Typical wall elevations are provided in Figure 2.

After the earthquake, portions of the towers were stripped of finishes, and damage to the shear walls and link beams was documented. Inclined cracking, with widths typically on the order of 0.125 to 0.6 mm, was documented, primarily in the lower stories. Link beams experienced a range of damage from spalled concrete and buckled reinforcing to hairline cracking. A photo of a damaged link beam is shown in Figure 2.

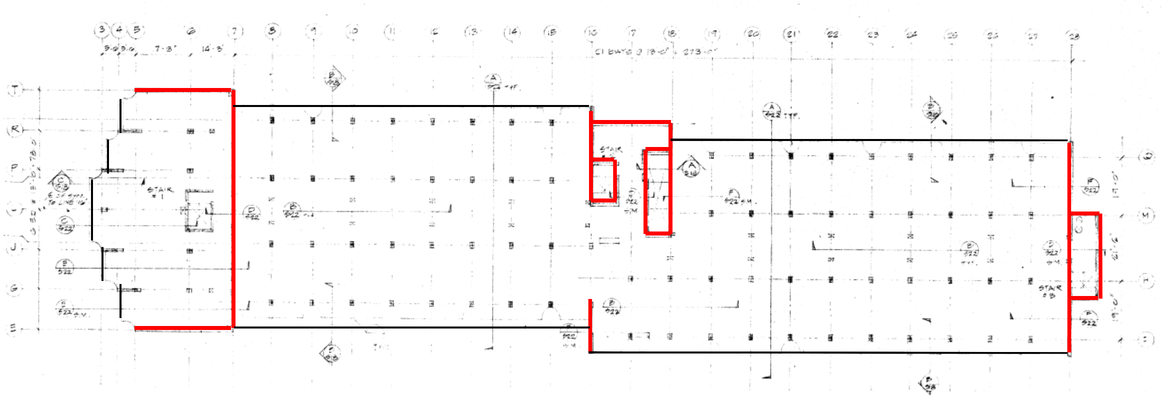

Figure 1. Typical floor plan showing shear wall locations in red

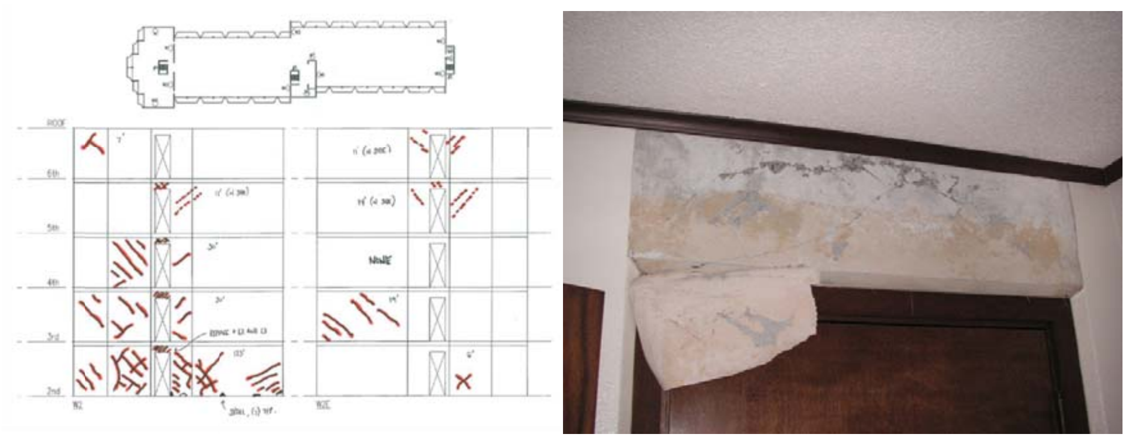

Figure 2. Typical shear wall elevations with crack mapping (left) and damaged link beam (right)

3. DESCRIPTION OF ANALYSIS METHODS

With the extent and severity of physical damage catalogued, the towers were subjected to two different types of analysis, nonlinear dynamic and nonlinear static, each explicitly considering the limited ability of the shear walls to resist uplift, and using the aforementioned strong motion records located within a kilometer from the building. The analyses included soil-structure interaction following recommendations from FEMA 440 (2005) and allowance for uplift. Documented damage to the shear walls was employed as the benchmark for assessing the validity of the analysis results, wall by wall and story by story.

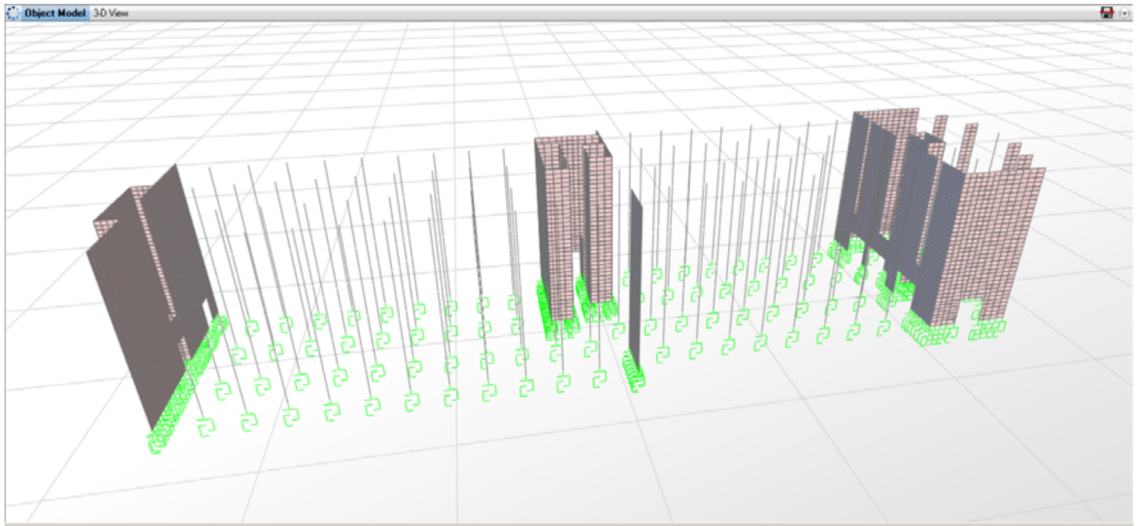

Figure 3. Computer model of building without slabs to show locations of shear walls

3.1 Soil-Structure Interaction

In each of the models, the effects of both foundation flexibility and, to a certain extent, soil-structure interaction, were incorporated using the recommendations provided in FEMA 440. Three basic sources of soil-structure interaction were considered: foundation flexibility, so-called "kinematic effects" related to filtering of seismic motion input into the structure, and foundation damping. These sources are implemented into the numerical analyses on both the capacity side -- by changing properties of the model itself -- and on the demand side by reducing the seismic input using an equivalent damping parameter.

The first source of soil-structure interaction, foundation flexibility, was implemented using two approaches. In both analysis models, the stiffness of the entire foundation system was modeled: not just the spread footing system, but also the 15 cm slab-on-ground that interconnects the bottom of the columns and shear walls. Additionally, gap elements were introduced underneath the footings. In compression, these elements behaved as stiff springs, representative of a concrete footing cast on bedrock, but allowed for uplift in the event that overturning moment exceeded the resisting moment associated with self-weight. Nonlinearity of the foundation related to uplift of the gap elements was considered in the pushover and nonlinear time-history analyses.

The second source of effective damping related to soil-structure interaction, the kinematic effect, is primarily related to embedment of the foundation and to base-slab averaging. In the structures modeled in this paper, the spread footings were cast directly onto bedrock, so the effects of embedment were neglected. However, because of the large size of the slab-on-ground (21 m by 100 m), base-slab averaging provided a significant reduction to the earthquake demands. The averaging was only performed at the slab located at the footprint of each tower even though a common slab connects the towers. The equivalent viscous damping related to kinematic effects was computed to be 7% and 10% in the longitudinal and transverse directions, respectively.

The final component of damping derived from soil-structure interaction is presented in FEMA 440 as foundation damping. Foundation damping is idealized as being a function of, among other things, the first translational periods of vibration of the structure. However, structures that exhibit rocking behavior have translational periods of vibration that are a function of the amount of uplift -- they are nonlinear. So, the equivalent damping related to foundation flexibility was computed using an effective linear-elastic period of vibration corresponding to the lateral drift (and foundation uplift) expected when subjected to 0.065g and 0.18g in the longitudinal and transverse directions. These lateral accelerations are equivalent to the damped demand event described in the Seismology section of this paper. The resulting equivalent damping related to foundation flexibility was computed to be 3.5% of critical in both the longitudinal and transverse directions.

The combination of all three of the effects describe above -- foundation flexibility, kinematic effects, and foundation damping -- resulted in an overall reduction in the seismic demands input into the structural models. FEMA 440 recommends incorporating these demand reductions by assigning equivalent damping values, which were computed as 22% and 13% in the longitudinal and transverse directions of the building. An important point to be noted is that although these damping values fall well outside the realm of what is normally considered for elastic analysis, a damping coefficient of 5% of the critical value was also used to characterize structural damping, which is common to US building codes. The balance of the effective damping may be better thought of as a reduction in the seismic input as it transitions from the native soil to the base of the structure, rather than an actual damping inherent to the structure itself.

3.2 Nonlinear Analyses

The building was modeled in the structural analysis program SAP 2000 Version 14 (Computers and Structures, Inc. 2009) using shell, frame, and link elements. Based on observations of the building after the earthquake, very few of the reinforced concrete elements experienced yielding, or behavior that would lend itself to nonlinear modeling techniques. Therefore the nonlinearity in the model was focused at the foundation level, with the implementation of stiff nonlinear link elements providing for compression-only behavior at the base of each shear wall and each column. The links were located at meshed nodes at a spacing of around 0.5 meters, allowing more local behavior to be balanced with computational efficiency. All of the reinforced concrete columns in the building support flat slabs and were modeled using frame elements. Although the flat-slab frames were not expected to contribute in a large way to the transmission of lateral forces between floors, and indeed the stiffness of the shear walls severely limits the story drifts that would affect the forces in the columns, the support conditions at the base of the frames were left unreleased based on the small magnitude of forces encountered at the earthquake considered.

Our first analyses considered the use of nonlinear elements to model the link beams -- if any above-grade portion of the building experienced nonlinear behavior based on damage observed after the 2006 earthquake, it was the link beams with their 1970s detailing. However, when an appropriate nonlinear element combination was used in these locations it only dramatically increased the run time without providing a meaningful effect on the output. It was therefore decided to model the link beams using frame elements with an effective section reduced to 20% of the actual section to account for the local nonlinear behavior. The shear walls were modeled as shell elements with 50% bending stiffness reduction.

The nonlinear analyses performed in SAP 2000 included both Nonlinear Static and Nonlinear Modal Time History. Multiple analyses were performed using each of the methods in order to gain an understanding of the effects of parameters such as the stiffness modifiers, foundation uplift, or various soil-structure interaction relationships. A scaled deformed plot of the shear wall with link beams on Line 7 of the building is shown in Figure 3.

We conducted the Nonlinear Static, or pushover, analyses in two primary modes in each direction, but found limited influence from higher modes and subsequently conducted the analyses in the fundamental modes only. The building response was analytically and graphically compared to the earthquake demands by use of the Capacity Spectrum Method (Mahaney *et al.* 1993). In addition, to gain an understanding of the "period shift" in the structure due to rocking, the fundamental period in each direction was calculated at various levels of earthquake loading using story forces, masses, and analysis-provided displacements using the Rayleigh method:

$$T = 2\pi \sqrt{\frac{\sum m\delta^2}{\sum F\delta}}$$

Due to the nonlinearity created solely by rocking, the calculation allows a comparison of the elongation of the building period response on a S_A versus T scale as shown in Figure 4. Also shown is the seismic demand as recorded at the nearby recording station and the demand as-reduced by soil-structure interaction. The pushover analyses were not continued until a postulated "collapse", but stopped prematurely and used only to gain an understanding of the behavior of the building during the 2006 earthquakes, which were only slightly to moderately damaged.

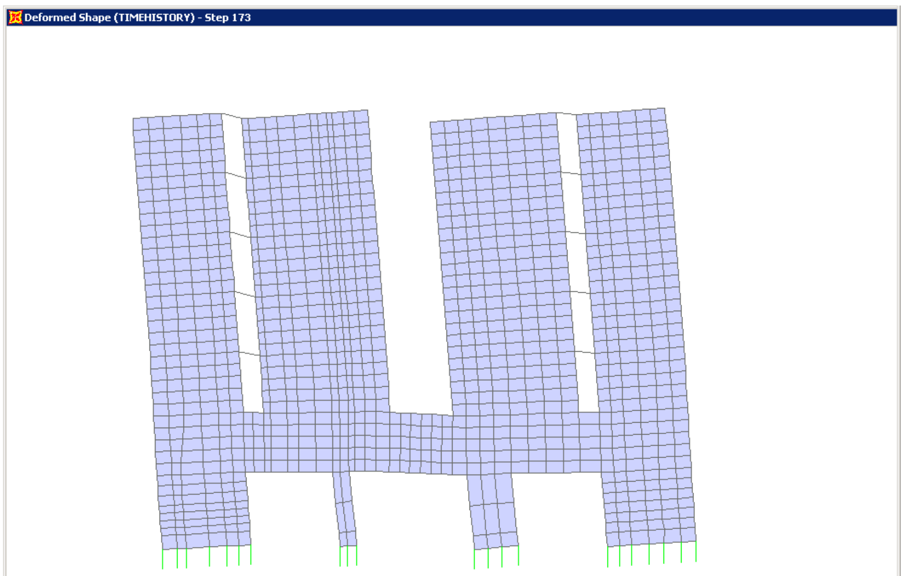

Figure 3. Deformed shape of coupled shear walls on Line 7 showing uplift

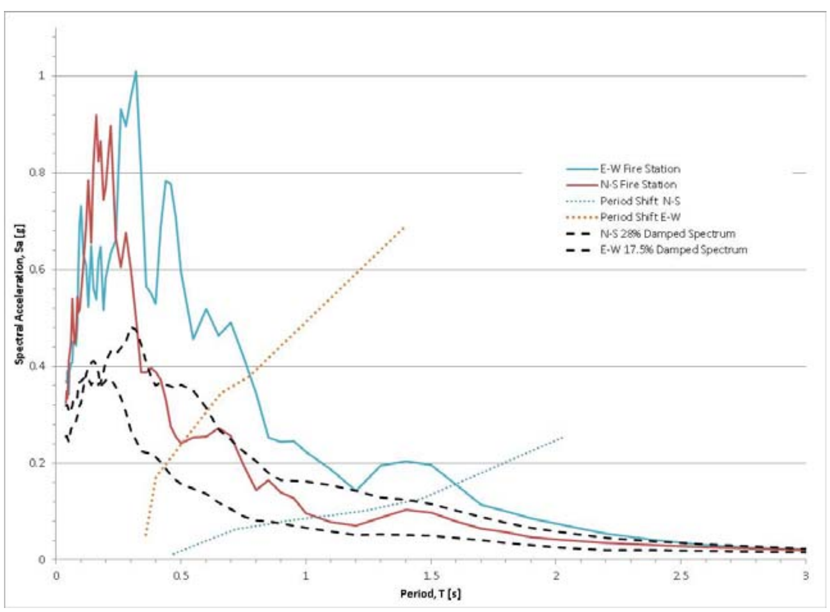

Figure 4. Rayleigh period shift of building shown versus recorded and damped response spectra

Nonlinear Modal Time History (also called Fast Nonlinear Analysis, or FNA) is an efficient method to analyze structures that are predominately linear-elastic but that have a limited number of predefined nonlinear link/support elements (Computers and Structures 2009). The response of a structure using FNA is dependent on being able to adequately represent nonlinear forces by modal forces and requires an appropriate number of modes be used to ensure that the static modal load participation ratio of each nonlinear degree of freedom near 100 percent. The number of nonlinear degrees of freedom in our model was small, and the fundamental behavior was mainly confined to the uplifting of several key shear walls. In our case, FNA was also useful in that different damping values were input for the fundamental modes of building response. The analysis would then damp out the fundamental modes based on soil-structure interaction, while leaving the higher modes with the traditional damping of 5%. The analyses generated results that compared quite well with the amount and location of damage actually documented. The link beams connecting shear walls provided a particularly good benchmark of performance since the damage to these elements was well documented. The Nonlinear Static analysis at the performance point predicted a shear stress in a heavily damaged link beam corresponding to 1.3 MPa; an envelope of the FNA response predicted a shear stress in this same beam of 1.4 MPa. Damage, or lack of damage, in many other locations within the building was also predicted by the models, showing the effectiveness of properly accounting for rocking in building behavior. Parallel analyses that did not incorporate appropriate releases at the bases of the shear walls generated damage predictions that far exceeded anything actually observed.

4. DISCUSSION

The very narrow range of spectral acceleration response predicted by the two nonlinear methods used to study the building is notable, particularly in light of the limited success reported in other studies that have attempted to benchmark nonlinear analysis results against documented earthquake damage. The authors have concluded that the most likely explanation for this success is that the building nonlinearities in the current studies were concentrated at the base of the shear walls, as rocking, rather than distributed through a large number of elements as structural damage. Moreover, the nonlinearities in the current studies, occurring at the base of the shear walls, are bilinear, and are thus substantially simpler to model than multi-linear hysteretic behavior inherent in degrading structural members.

The analyses demonstrate that rocking of the shear walls during the 2006 event was a dominant behavior mode that prevented the building from responding at accelerations high enough to cause significant structural damage. The data that supports this conclusion is clear and unambiguous. Modeled with a fixed base, the five-percent-damped spectral acceleration demands of the earthquake on the building approached 1g, whereas the yield strength of the walls is only approximately 0.25g. While a discrepancy of this magnitude can often be explained as a function of ductility, damping, or energy dissipation, severe damage indicative of a highly damped structural response could not be found after the earthquake. The limited areas of concentrated damage observed in the small, lightly reinforced door headers are insufficient to have provided the energy dissipation necessary to reduce the demands from 1g to below 0.25g. Simply put, had the shear walls actually responded as fixed-base structures, substantial yielding would have been required to accommodate the seismic demands, and substantial damage to the walls --- far greater than the relatively narrow cracking documented in the field of most of the walls --- would have been observed. To reduce the response demands from 1g to below the yield threshold for the walls --- but without yielding --- requires that the walls responded predominantly by rocking.

To explain the limited cracking and the absence of more significant structural damage, wall supports modeled with nonlinear gap elements were necessary to prevent the walls from picking up more load than possible in reality. In contrast, the earthquake response of the building would not have been predicted with any degree of accuracy by any of the analysis methods employed had the model incorporated fixed-base supports for the shear walls. As usual, boundary conditions and modeling assumptions played a major role in the ability of finite element software to adequately predict behavior.

From a historical perspective, seismic design provisions in most building codes have generally, if inadvertently, communicated that rocking is an undesirable mode of seismic response. Codes typically contain provisions that invoke the unfortunately alarming term “overturning”, in which the stability of a seismic resisting elements are explicitly checked by comparing the base “overturning moment” arising from code level seismic forces with a resisting moment supplied by gravity plus any positive connection of the element being designed to other elements, most commonly an anchored foundation. For this purpose, resisting moment is usually required to be very conservatively estimated, inappropriately suggesting that loss of stability and collapse are the necessary and undesirable result when “overturning moment” exceeds resisting moment. Such logic, however, is spurious because in as much as seismic design relies on response modification factors (the R-factor in US codes), lateral forces experienced by buildings during large earthquakes will often result in available restoring moments being exceeded. In other words, and contrary to the somewhat frightening implications of the term “overturning”, there exists in most codes an unstated implication that the lateral systems will uplift or rock during design events.

Interestingly, most codes do not have any provisions for explicit determination of the consequences of such uplifting. While building codes do typically contain requirements for checking displacement compatibility at drift levels that are set forth as consistent with those expected during response to design motions, these drift levels are normally estimated by simplistic algebraic formulae and tabulated factors, making it relatively unlikely that they accurately quantify the drift that will occur during the design event. In any case, and while uplift and rocking of lateral systems is clearly expected, it is not clear whether, generically, such rocking will or will not have straightforward structural consequences for structures responding to any particular earthquake. For the structure in this study, however, rocking clearly occurred and the effects of rocking were clearly positive. Rocking provided a ductile mode of response that dissipated more than ample amounts of energy. This corresponds to the authors’ expectations that rocking and uplift are generally advantageous to the performance of structures subjected to large ground motions, provided, of course, that the dimensions of the rocking elements are sufficiently large to preclude global instability, that the effects of rocking do not result in especially severe local crushing of structural elements or other undesirable behaviors, and that displacement compatibility has been accounted for.

5. CONCLUSIONS

Through consideration of soil-structure interaction effects, and using the simple modeling technique of allowing foundation uplift through compression-only springs, two nonlinear analysis methods were employed that accurately predicted seismic damage in a reinforced concrete shear wall building with link beams. Both analysis methods utilized explicit nonlinearity at the foundation level only and demonstrated levels of damage commensurate with observed distress. Models that failed to include the nonlinear rocking behavior at the foundation dramatically over-predicted the structural response and the resulting damage. In many structures, uplift and rocking play a major role in the behavior of the structure under significant lateral loads, and it is important that finite element models include these phenomenon if a good understanding of the behavior of the structure is to be obtained.

REFERENCES

American Society for Civil Engineers (2006). ASCE 41: Seismic Rehabilitation of Existing Buildings, ASCE, Reston, VA.

Applied Technology Council (1996). Seismic Evaluation and Retrofit of Concrete Buildings, Redwood City, California.

Bayhan, B., Gülkan, P. (2011). Buildings Subjected to Recurring Earthquakes: A Tale of Two Cities. *Earthquake Spectra*, Volume 27:3, 6365-659.

Computers and Structures, Inc. (2009). CSI Analysis Reference Manual for SAP2000, ETABS, SAFE and CSiBridge, Berkeley, California.

Earthquake Engineering Research Institute (2006). Compilation of Observations of the October 15, 2006 Kiholo

Bay (Mw=6.7) and Mahukona (Mw=6.0) Earthquakes, Hawai'i, Oakland, CA.
Federal Emergency Management Agency (2005). Improvement of Nonlinear Static Seismic Analysis Procedures. Washington, District of Columbia.
Mahaney, J.A. Freeman, S.A., Paret, T.F., Kehoe, B.E. (1993). The capacity spectrum method for evaluating structural response during the Loma Prieta earthquake, *Proceedings 1993 National Earthquake Conf., Memphis*, 501-510.
Maison, B.F., Kasai, K., Deierlein, G. (2009). ASCE-41 and FEMA-351 Evaluation of E-Defense Collapse Test. *Earthquake Spectra*, Volume 25:4, 927-953
Munson, C.G., and Thurber, C.H. (1997). Analysis of the attenuation of strong ground motion on the island of Hawaii. *Bulletin of the Seismological Society of America*. Volume 87: 4, 945-960.
Wald, D.J., Quitoriano, V., Heaton, T.H., Kanamori, H., Scrivner, C.W., and Worden, C.B. (1999). "TriNet 'ShakeMaps': Rapid Generation of Instrumental Ground Motion and Intensity Maps for Earthquakes in Southern California," *Earthquake Spectra*, Vol. 15, No. 3, pp. 537-556.
Wyss, M. and Koyanagi, R.Y. (1992) Isoseismal Maps, Macroseismic Epicenters and Estimated Magnitudes of Historic Earthquakes in the Hawaiian Islands, *U.S.G.S. Bulletin.* 2006: 93.