Earthquake Damage Assessment of Reinforced Concrete Hotel Buildings in Hawaii

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SUMMARY:
This paper presents an evaluation of damage to mid-rise reinforced concrete hotel buildings from the 2006 earthquakes on the island of Hawaii. Designed in the 1960s and 1970s with limited consideration for ductile detailing, the structural system of the hotels consisted of reinforced concrete shear walls, some of which transition to tall columns at the first two stories, providing a large open area for the lobby and the promenade. On-site damage investigation revealed that the buildings had sustained, among other structural damage, substantial cracking of the reinforced concrete shear walls and tall columns during the 2006 Hawaii earthquakes. Visual observations and concrete coring through epoxy injected cracks in the walls revealed the extent and the size of the cracks. Finite element analyses of the buildings revealed the amount and the nature of the cracking damage directly attributable to the earthquakes and differentiated it from cracking due to shrinkage. Numerically predicted regions of overstressed concrete walls due to earthquake loads and those due to shrinkage correlated well with the location, pattern, and direction of observed wall cracks. The locations of observed column cracks also corresponded well with the location of predicted moment demands and strains in the column elements.

Keywords: Reinforced concrete, buildings, damage investigation.

1. INTRODUCTION

This paper contains the results of a structural engineering investigation to determine the effects of the 2006 Hawaii earthquakes on reinforced concrete hotel buildings located on the island of Hawaii. The purpose of the evaluation was to determine the cause and extent of damage to the structural components and recommend the repairs required to restore structural strength.

A major issue of damage that was observed during the field investigation is the diagonal cracking of reinforced concrete shear walls and the tall columns that support the walls. The paper focuses on the reasons for the damage to the shear walls and the tall columns; however, evaluation of the earthquake related structural damage in other parts of the buildings is also discussed.

2. BUILDING DESCRIPTION

The hotels are located along the west coast of the island of Hawaii and comprised of multiple buildings that were constructed in the 1960s and 1970s. Figures 1 and 2 show overall elevation views of three buildings from three different hotel sites. The buildings typically consist of a 4- to 6-story tall, reinforced concrete structure with cast-in-place concrete flat slabs and reinforced concrete divider walls between hotel rooms and along hotel corridors. The floor slabs are typically 200 mm (8 inches) thick and supported by the divider walls, also typically 200 mm (8 inches) thick, which serve as both bearing and shear walls. Some of the buildings such as those shown in Figure 1 and Figure 2b have tall first stories where the shear walls in the upper floors transition to reinforced concrete transfer beams and tall columns, thus providing a large open space for the lobby, the promenade, and other common areas of the hotels. The tall concrete cruciform columns shown in Figure 1 vary in height between 3.7

m (12 feet) and 10.4 m (34 feet). The gravity and the lateral loads imposed on the buildings are resisted by and transferred from the roof and floor diaphragms to the shear walls, transfer beams and tall columns, and down to the reinforced concrete foundation. The buildings' properties were determined from on-site observations and available building plans.

Figure 1. Overall view of hotel building with tall columns at the first story.

Figure 2. Overall views of hotel buildings with shear walls.

3. EARTHQUAKE GROUND MOTION AT THE SITE

The hotel sites are located between 19 km (12 miles) and 55 km (34 miles) from the epicenters of the two 2006 Hawaii earthquakes, namely the Kiholo Bay and Mahukona earthquakes. The magnitude 6.7 Kiholo Bay and the magnitude 6.0 Mahukona earthquakes occurred on October 15, 2006 at 7:07 am and 7:14 am, respectively. The epicenters were located along the northwest coast of the island of Hawaii and the hypocenters were located at depths of 38 km (24 miles) and 19 km (12 miles), respectively.

The ground motions at the site of the hotels were estimated using a spatially weighted averaging technique (King et al. 2004) for the recorded ground motion response spectra at the two closest stations on similar soil conditions and the USGS-produced ShakeMap ground motion parameters. The ground motion recording stations are all located within approximately 16 km (10 miles) of the building sites.

The effective peak acceleration (EPA) values of the estimated earthquake ground motion at the hotel sites are between 0.18 g and 0.23 g in the North-South direction, between 0.18 g and 0.26 g in the

East-West direction, and 0.12 g in the vertical direction. Figure 3 shows the 5% damped acceleration response spectra from the estimate of the ground motion experienced at the building sites during the 2006 Kiholo Bay earthquake.

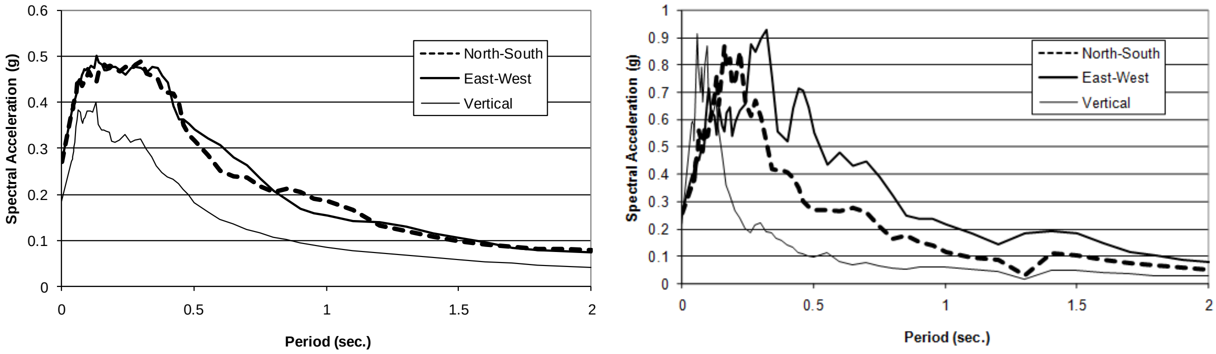

Figure 3. Estimated site response spectra for the hotels for the 2006 M6.7 Kiholo Bay earthquake.

4. ON-SITE DAMAGE INVESTIGATION

Visual investigation and destructive testing of the buildings' components was performed to identify damages to various structural elements. Numerous cracks ranging in width from hairline to 1 mm (40 mils) were observed in the reinforced concrete shear walls and tall columns.

In many of the buildings that were investigated, cracks were observed in the shear walls on all the floors of the building. The wall cracks at the lower floors were primarily diagonal with a few vertical cracks (Figure 4). The wall cracks at the upper floors were fewer and primarily vertical with some diagonal cracks (Figure 5). The cracks in the tall cruciform columns were primarily diagonal (Figures 6 and 7a). Cracking and spalling was observed in other columns (Figure 7b). Buildings in which coupling beams were used between adjacent shear walls sustained damage in the form of crushing of concrete and de-bonding of the reinforcing steel bars from the concrete in the coupling beams (Figure 8); the adjacent shear walls also sustained diagonal cracks.

Cracks observed in the shear walls and columns were mapped and photographed; crack width measurements were taken. It was determined that the crack width measurements taken at the surface of the walls and columns did not represent the actual crack width due to the elastomeric paint finish on the wall surfaces. Concrete samples taken by coring through the cracks in the 200-mm (8-inch) thick walls and crack width measurements on the cores also did not represent the actual crack width because the concrete cores would come apart at the crack while being removed from the wall. Measurement of the crack size from inside the core opening in the wall was also not possible because the edges of the cracks in the core opening would be disturbed and worn out by the rotating bit of the core drill.

Therefore, the best way to measure the size of the cracks in the walls was to epoxy inject the cracks first and core through the cracks that had been epoxied. The epoxy penetrates the cracks, fills all the voids, and bonds the concrete together. Once the core is removed from the wall, the size of the crack is determined by measuring the width of the epoxy filling the crack. All core samples indicated that the cracks penetrate the entire thickness of the walls and the cracks are wider inside the walls than observed on the surface of the walls through the wall finish. Measurements of the cracks through the wall finishes underestimate the actual size of the cracks. This is illustrated in Figures 9a and 9b in which the crack width was measured as 0.25 mm (10 mils) on the wall finished surface and 0.76 mm (30 mils) along the concrete core, respectively. Figure 10 shows that a wall crack measured to be 0.08 mm (3 mils) on the finished surface was measured as 0.25 mm (10 mils) on the bare concrete surface without the painted finish.

Concrete core samples were taken also for petrographic examination to determine the relative age of the observed cracks. Most of the cracks contained a shallow depth of carbonation and no discoloration indicating that they were due to a recent event, while some of the other cracks were determined to have formed earlier, possibly due to shrinkage, but were exacerbated recently indicated by carbonation to a larger depth but no carbonation deeper in the crack.

While damages to the shear walls and columns were structurally the most critical, other concrete components in the buildings including beams, balconies, floor slabs, roof trellises, pedestrian bridges, and their connections also sustained damage in the form of cracking and spalling. Damage to other components in the buildings included loosened, detached, and collapsed lava rock veneers on some of the exterior concrete walls due to the out-of-plane response of the walls and the veneers during the earthquakes. The above listed damages and the potential hazard resulting from the damaged components in the event of an aftershock contributed to the closure of some of the hotels due to the concerns for life safety.

Figure 4. Typical cracking on lower floor shear walls.

Figure 5. Typical cracking on upper floor shear walls.

Figure 6. Typical cracking in cruciform columns.

Figure 7. a) Typical cracking in cruciform column; b) Concrete column spalling.

Figure 8. Coupling beam damage.

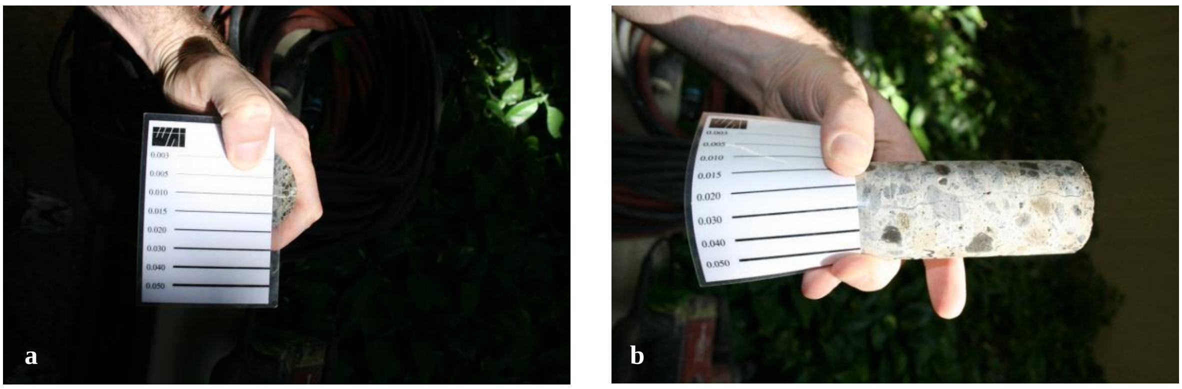

Figure 9. Crack width on wall surface (10 mils) versus inside the wall along concrete core (30 mils).

Figure 10. Crack width on finished wall surface versus bare concrete surface.

5. STRUCTURAL ENGINEERING ANALYSIS

5.1. Models

Three-dimensional computer models of various hotel buildings were developed using the SAP2000 software (Figure 11). In some cases, only a representative portion of the building was developed. These “substructure” models as well as the full building models included all the floors in the building, the concrete floor slabs, shear walls, columns, and beams. The floor slabs, shear walls, and the tall cruciform columns were modeled as shell elements; regular columns and beams were modeled as frame elements in SAP2000. The expected strength value for concrete and the effective stiffness values for concrete elements were used based on the specifications of ASCE 41 (ASCE, 2007).

The building models were analyzed for stresses caused by gravity loads (dead and live), earthquake loads (both horizontal and vertical), long-term shrinkage, and the combinations of these loads. Dead loads included the self-weight of the structural elements and the superimposed dead loads for the weights of the non-structural elements such as the floorings and the partitions. Live load was considered to be 25% of the design live load per ASCE 41 (ASCE, 2007) to represent the loads on the existing buildings at the time of the earthquakes. The estimated site-specific response spectra discussed in Section 3 were used as input for the earthquake loads modified for an estimated viscous damping of 3%. Two factors significantly influence shrinkage effects: Relative humidity of the environment and age of the concrete. Average relative humidity for the analyses was taken as 69% representing a typical coastal environment on the Hawaiian Islands. Age of the concrete was taken as the approximate time between the construction of the buildings and the occurrence of the 2006 Hawaii earthquakes.

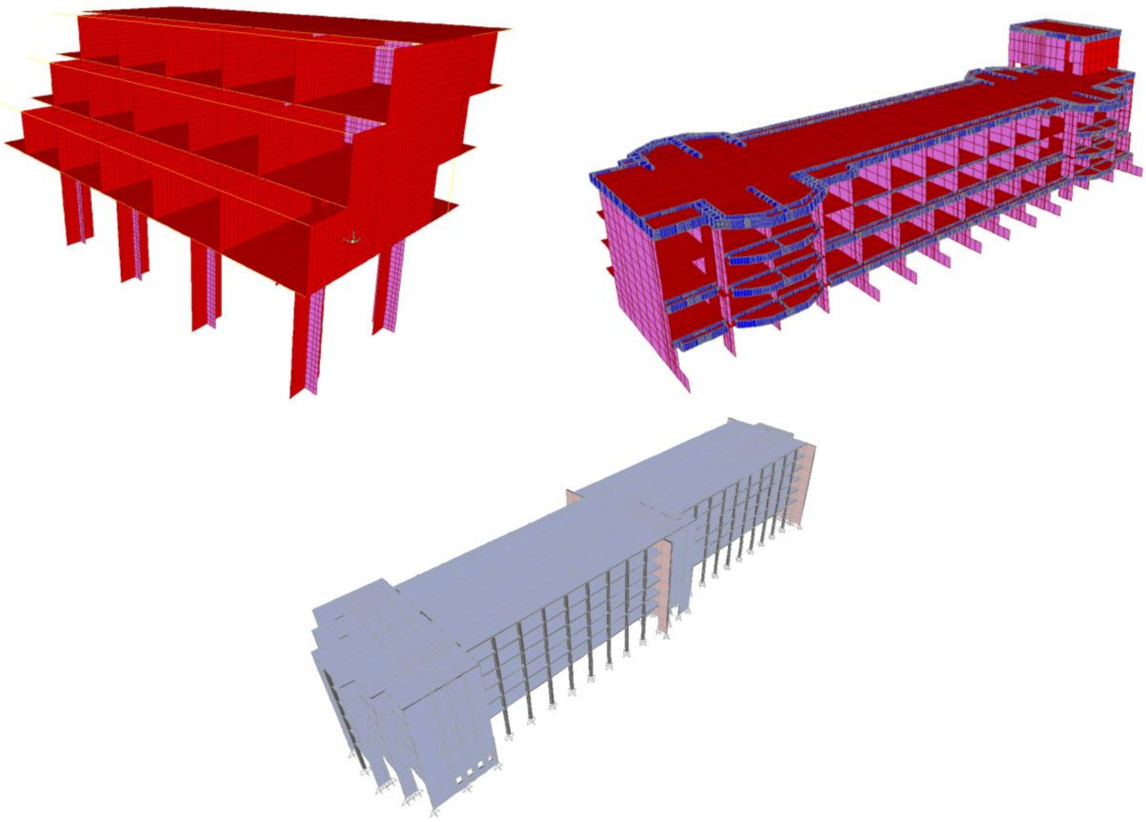

Figure 11. Structural engineering analysis models for various hotel buildings.

5.2. Results

Modal analyses were performed with sufficient number of modes of vibration that contributed to more than 90% of the mass participation in both orthogonal directions and the vertical direction. The periods of vibration for the first three modes for one of the buildings are shown on the site-specific response spectra of the Kiholo Bay earthquake in Figure 12, which illustrates that these three modes with the largest mass participation coincide with the period range where the peaks of the response spectra occur. Modal analyses showed similar results for many of the other hotel buildings also. Proximity of the periods of vibration to the peaks of the response spectra results in amplified structural response due to the earthquake.

Based on the gravity and earthquake load analyses, contours of principal stresses in the shear walls and their corresponding directions were obtained for various load combinations to determine the location and magnitude of the earthquake induced stresses in the shear walls (Figure 13). Stress contours in Figure 13 are drawn such that the purple color indicates the stresses are smaller than the tensile strength of concrete and all the other colors indicate the stresses exceed the tensile strength of concrete. The direct tensile strength of concrete is calculated from equations in ACI 209R-92 (ACI, 1997). Directions of principal stresses are shown by arrows in Figure 13; the length of the arrow indicates the magnitude of the stress exceeding the tensile strength of concrete.

Figure 14a shows the location and direction of the expected cracks in the shear walls based on stress contours and arrows such as those shown in Figure 13. Cracking is typically along the direction perpendicular to the principal tensile stress direction. Figures 13 and 14a show that the stresses are greater and more widespread and the cracks are more severe in the lower floor walls than the upper floors. Stresses in the shear walls were sufficiently large to cause yielding of the reinforcing steel bars on the lower floors in some of the buildings. Figure 14b compares the pattern and direction of the observed cracks in a shear wall with the results of the computer analysis, indicating that most of the cracking is attributable to the earthquakes.

Figures 15a and 15b show stress contours and stress directions, respectively, for a series of wall piers along the building's longitudinal direction subjected to gravity and earthquake loads. Figure 15c shows the deformation of the same walls due to shrinkage effects such that the exterior sides of individual wall piers have tensile deformations while their interior sides undergo compression. Figure 15d shows the location and direction of the expected cracks in the walls under the combined effects of seismic and shrinkage. The cracks are longer and more numerous at the exterior sides of individual wall piers because the shrinkage effects intensify them, and they are less pronounced at the interior sides of the piers due to opposite shrinkage effects.

Analyses results also showed that the shear demand on the tall columns for various load combinations was at or near the design capacity of the columns during the earthquakes, i.e. the demand was large enough to cause cracking of the concrete and yielding of the steel reinforcement in the columns.

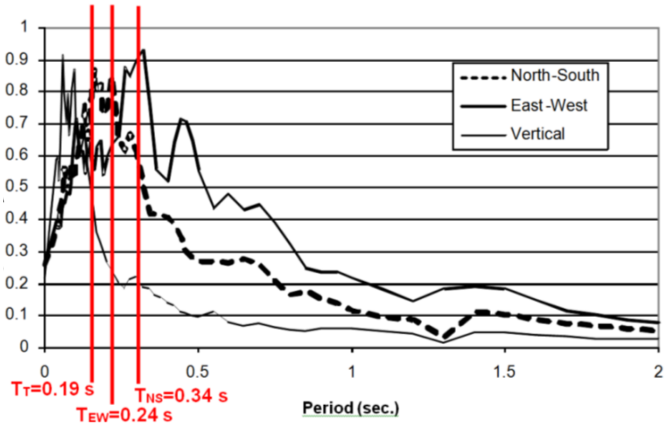

Figure 12. Periods of vibration overlaid on site-specific response spectra.

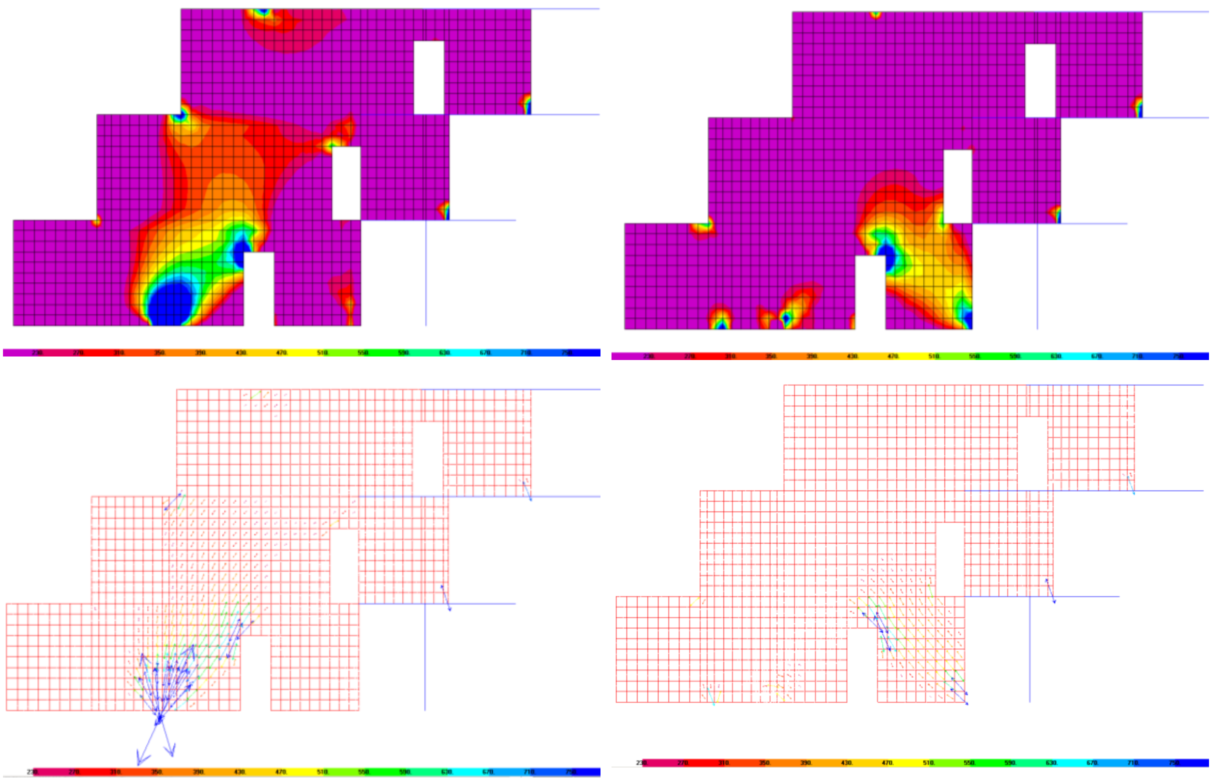

Figure 13. Stress contours and stress directions in walls.

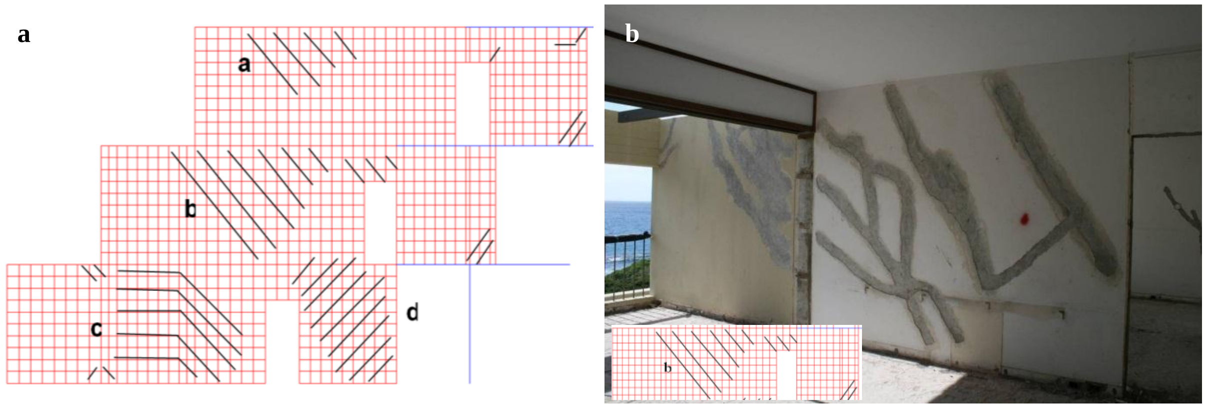

Figure 14. a) Location and direction of expected cracks; b) Comparison of analysis results to observed damage.

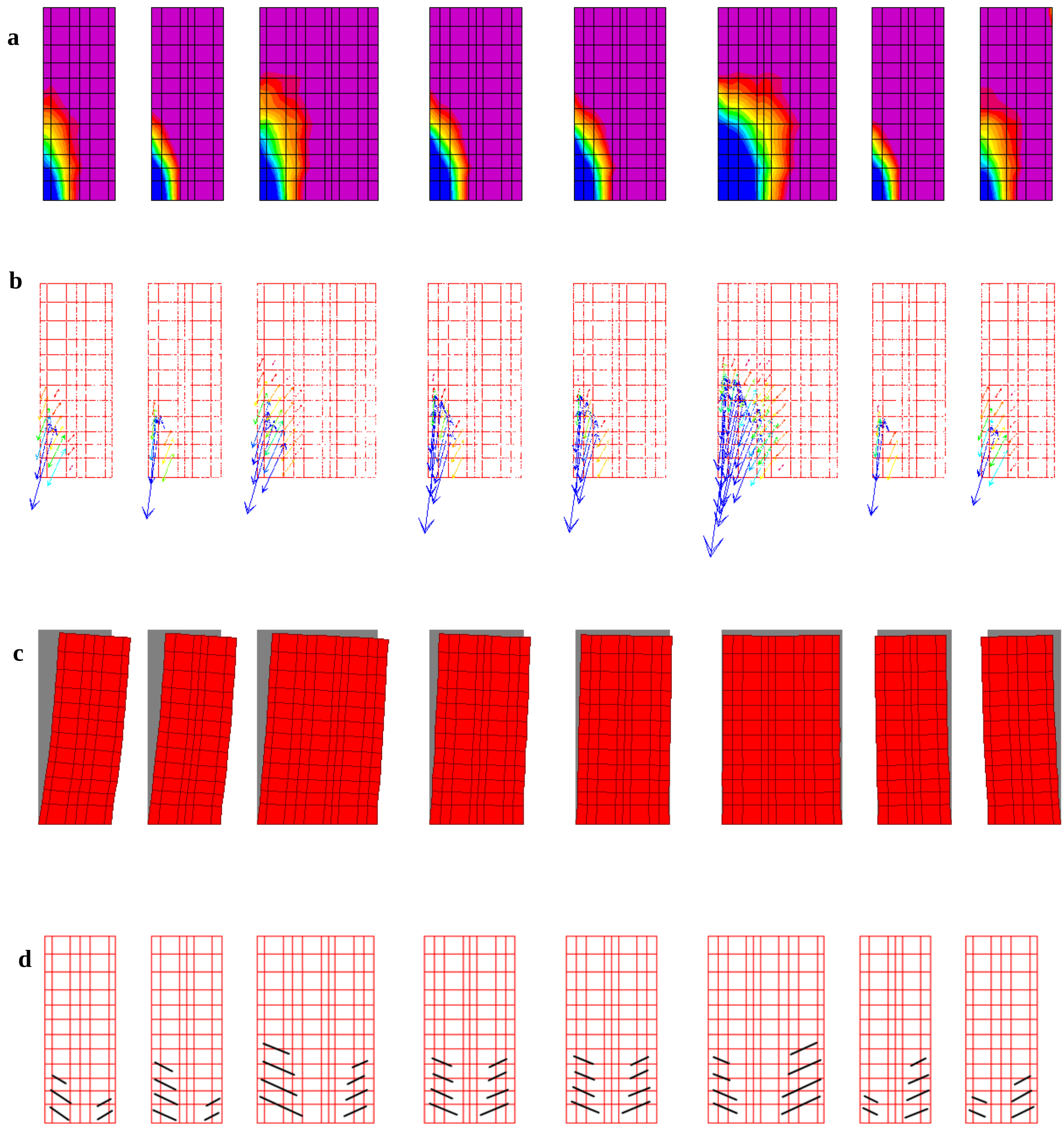

Figure 15. a) Stress contours; b) Stress directions; c) Shrinkage deformations; d) Expected cracks.

6. CONCLUSIONS

Field observations of structural damage were validated with expected structural behavior, analyses, and calculations. Analyses assisted in distinguishing the damage caused by shrinkage and earthquakes. Analyses also helped identify wall cracks that are otherwise hidden from view due to the presence of wall finishes or building contents.

The pattern and direction of the cracks observed in the shear walls and columns correlate well with the results of the computer model analyses, indicating that most of the cracks in the walls and columns (mainly diagonal shear cracks) were attributable to the earthquake ground motions. If in some locations the cracks were pre-existing, the size of these cracks was significantly increased in width and length as a result of the earthquakes. The cause of the vertical cracks in the walls was attributable to the shrinkage of the concrete; however, the earthquakes exacerbated the sizes of the cracks.

Assessment of the ground motions at the sites during the 2006 Hawaii earthquakes, visual observations of damage, evaluation of existing conditions, and structural engineering calculations and analyses lead to the conclusion that the earthquakes were strong enough to cause the observed substantial damage to the buildings' structural and non-structural components, necessitating pervasive repairs and in some cases the temporary closure of the hotel.

Repair recommendations for the shear walls and columns included epoxy injection of cracks, and use of fiber reinforced polymer (FRP) sheets on the reinforced concrete walls in some of the buildings in addition to epoxy injection. Repairs are necessary to recover the strength and the stiffness of the shear walls and columns that was lost due to cracking during the earthquakes and to prevent the long term maintenance issue of cracks re-appearing through the wall finishes over time or during future seismic events. Recommendations were also made to replace the coupling beams and repair other reinforced concrete components and connections that were damaged.

Cracks were repaired with epoxy repair procedure outlined in FEMA 308 (1999). Crack widths as small as 2 mils can be epoxy injected according to FEMA 308; however, epoxy injection of crack widths as small as 3 mils was found to be feasible at the site during the repairs.

Coring through epoxy injected cracks was essential in accurately determining the crack widths. Petrographic examination assisted in determining the relative age of the cracks.

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