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# Analysis and testing of a flat slab concrete building

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ABSTRACT: A 14-story reinforced concrete flat slab concrete building in Southern California was assessed for earthquake risk. The mid-1960's design uses frame action between the slab and columns for lateral force resistance. Unlike other flat slab buildings damaged in past earthquakes, this building has large, deep, pyramid-shaped drop panel to reinforce the critical slab-column joint.

Preliminary linear analysis identified probable structural weaknesses and seismic demands on the structure, but the earthquake performance of the drop panel could not be assessed. Tests at the University of California, Berkeley campus investigated the ductility of the slab-column connection, and provided data for analytical model refinement. Results showed stiffness degradation as expected, but loss of strength within anticipated maximum drifts was negligible. Finally, the test data was used to calibrate an analytical model and extrapolate test results to the overall structure. This structure should perform well in moderate-to-large earthquakes, although large drifts are expected.

## 1 OVERVIEW OF THE STRUCTURE

In early 1990, we assessed the potential seismic performance of a reinforced concrete framed office tower in southern California. The structure in question consists a 14-story reinforced lightweight concrete framed office tower supported on a 3-level subterranean reinforced concrete garage. It was designed in 1962 to then-current standards.

The office tower's structural system employs a moment-resisting space frame, consisting of reinforced concrete columns framing into a reinforced concrete flat slab with large drop panels or column capitals. A modest perimeter spandrel beam frames into the exterior columns, but reinforcement details indicate that this perimeter spandrel was designed for gravity loads only, and cannot be relied upon for earthquake load resistance. Thus the lateral force resistance of the office tower relies upon the reinforced concrete flat slab working as the girder element within the moment-resisting space frame system. This type of lateral force-resisting system is no longer permitted for new construction in Uniform Building Code Seismic Zones 3 or 4.

Concerns with the flat slab/column structural system were compounded by the use of lightweight concrete in the slabs *and* the columns of the office tower.

The assessment consisted of 3 phases:

1. Preliminary Analysis: 3D linear elastic response spectrum analysis and strength assessment,
2. Laboratory Testing: half-scale model component testing, and
3. Final Analysis: 2D non-linear, inelastic time-history analysis, including P-$\partial$ effect.

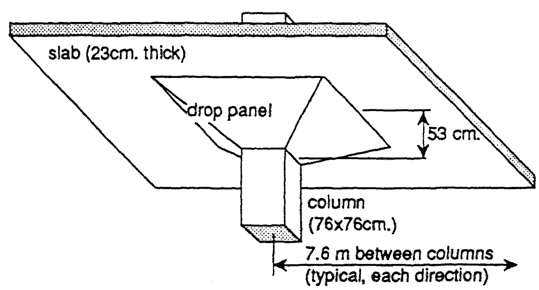


Figure 1. Typical slab/column joint with drop panel.

## 2 PRELIMINARY ANALYSIS RESULTS

The preliminary analysis consisted of the estimation of site-specific seismic hazards, a review of the original design documents, preliminary structural analysis (including response spectrum analysis), and a judgmental estimate of building performance in possible large local earthquakes.

This unique building is located in the vicinity of two large, active faults in Southern California. Other reinforced concrete flat slab buildings have been known to experience dramatic failures in past earthquakes [Esteva 1989]. However, this structure features a large (but unreinforced) drop panel which strengthens the critical column/slab joint.

The unique column/slab joint design does not lend itself to simple modeling or analysis procedures. Only through an experimental program would it be possible to realistically assess the critical performance characteristics of building strength and ductility which

On the negative side, a tall first story and other factors create a"soft" and potentially "weak" first story.

The unique column/slab joint design does not lend itself to simple modeling or analysis procedures. Only through an experimental program would it be possible to realistically assess the critical performance characteristics of building strength and ductility which must be weighed against potential earthquake structural demands. Therefore, a structural test program was planned and conducted in the structural laboratory at the University of California at Berkeley.

## 3 TEST PROGRAM

Two 1/2-scale specimens were tested, each simulating a typical interior slab/column/drop panel assemblage. The specimens were constructed of lightweight concrete and steel reinforcement which approximate the actual materials used in the actual building's construction.

The primary objective of the testing was to establish the failure mode(s), the yield and ultimate strengths, and the ductility of an actual, typical building slab/column/drop panel assemblage under simulated earthquake load cycles. A secondary objective was to estimate the effective stiffness of the assemblage, in order to improve the dynamic computer modeling of the building.

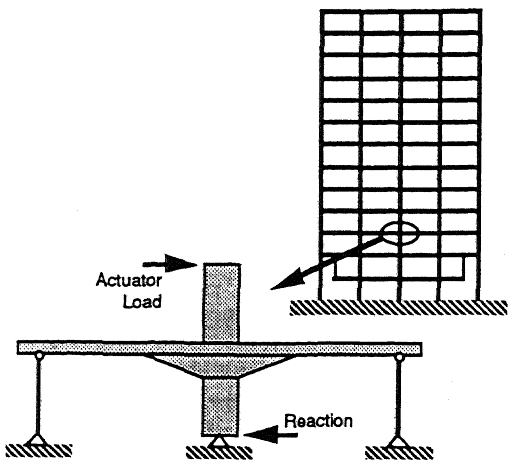


Figure 2. Test Concept

### 3.1 *Testing apparatus*

Horizontal forces are applied to the reinforced concrete column as illustrated in Figure 2. A hydraulic actuator slowly applies horizontal loads to the top of the column to force the column to displace in controlled, incremental steps. The base of the column is prevented from moving horizontally, but is allowed to rotate.

Eight vertical steel struts positioned around the perimeter of the slab simulate the slab continuity, forcing the slab in the laboratory to mimic the bending modes which occur in the corresponding section of slab in the building under earthquake loads.

A hydraulic actuator located beneath the column maintains a constant vertical force in the column as the slab sags and column punch-through progresses.

The test rig also features a 'torsional frame' to prevent twisting of the slab which can result from unsymmetrical damage or yielding of the specimen.

### 3.2 *Specimen scaling*

Scaling principles attempt to reproduce the actual state of local stresses (force per unit area) in specimen materials that have the same properties as the actual structure. In this way, the actual failure modes and mechanisms of the full-scale structure should be accurately reproduced in the laboratory specimen tests.

Complementary rules apply to scale test results back to full scale, for application to the actual structure. Drift ratios are nondimensional, and hence are unaffected by scaling.

### 3.3 *Specimen construction*

The office tower was constructed of lightweight concrete having a maximum density of 1762 kg per cubic meter (110 pounds per cubic foot). In situ building concrete compressive strength was found to be about 25,500 kPa (3700 psi). Concrete cylinder tests confirmed an average concrete compressive strength of 30,340 kPa (4400 psi) for the specimens at the time of testing. It is unlikely that this difference in concrete strength resulted in any significant impact on test results, which were largely controlled by reinforcement strength.

Reinforcement bar spacing was increased slightly to compensate for a difference in yield strength from the original reinforcement material.

### 3.4 *Column post tensioning*

In order to reproduce realistic large axial load in the columns of the test specimens, the columns were post tensioned. High strength steel stressing rods were passed through plastic ducts cast into the columns and anchored into heavy steel plates at the bottom of the columns. The rods were then post-tensioned and anchored against the column top plate.

### 3.5 *Displacement history for each specimen*

Earthquake loads on the structure were simulated by applying forces using a hydraulic actuator to produce specified displacements in a biaxial displacement pattern. A cruciform load pattern was followed, as illustrated in Figure 3. The displacement history is presented in Table 1 in terms of dimensionless drift ratios (story drift ÷ height) and the corresponding displacements at the top of the column.

After two cycles of loading at each new, higher drift level, a single cycle at the previous lower drift level was executed, to monitor stiffness degradation.

Table 1. Test displacement history

| Sequence | A | B | C | D | E |
|---|---|---|---|---|---|
| Drift Ratio | 0.25% | 0.50% | 0.25% | 1% | 0.50% |
| Displacement | 0.46cm | 0.91 | 0.46 | 1.83 | 0.91 |
| No. of Cycles | 2 | 2 | 1 | 2 | 1 |

| Sequence | F | G | H | I | J |
|---|---|---|---|---|---|
| Drift Ratio | 1.5% | 1% | 2% | 2.5% | 4% |
| Displacement | 2.74 | 1.83 | 3.66 | 4.57 | 7.32cm |
| No. of Cycles | 2 | 1 | 2 | 2 | 1 |

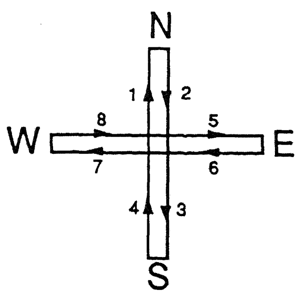


Figure 3. Displacement pattern imposed at the top of the column.

### 3.6 *Instrumentation*

Instrumentation for the slab/column specimen tests monitored the applied loads, tracked the essential specimen displacements, and detected inelastic behavior expected in slab reinforcement. Test instrumentation placement is illustrated below.

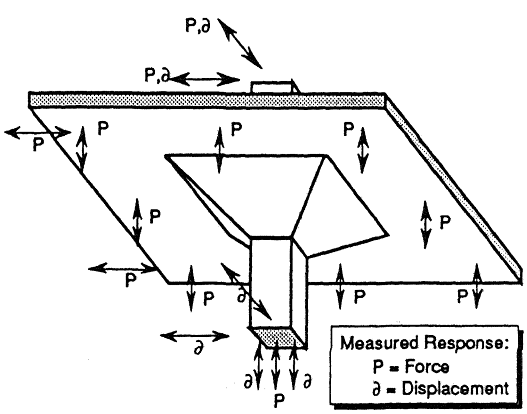


Figure 4. Test instrumentation layout

A computerized data acquisition system recorded data onto magnetic disk drive for later reduction and analysis. Displacements and loads at the top of the reinforced concrete column were monitored and plotted during the tests to guide the application of actuator displacements, and for in-test evaluation. Also, visual, photographic, and videotape records were made of the history of specimen behavior, including cracking and spalling.

## 4 OBSERVED BEHAVIOR DURING TESTS

Two tests were conducted with specimens of identical design (although the second specimen had several minor construction defects). The second test specimen carried additional lead weights to simulate floor areas in the building with high gravity loads.

Initial cracking was observed during the initial (0.25 percent drift ratio) load cycle. These cracks were well distributed hairline cracks for the most part, which closed upon load removal. Additional small cracks formed as load and displacement levels increased. At a nominal drift ratio of 1.5 percent (2.74 cm displacement of the top of the column), a lip began to form as the central portion of the slab above the drop panel began to punch through the surrounding slab. Cracking began to concentrate in this region, signaling the development of a hinge mechanism in the slab (see Figure 5). As tests proceeded to higher drift levels, the following behavior was observed:

• Some additional cracks continued to occur throughout the slab, but for the most part existing cracks opened and closed as drift cycles proceeded. Specimen damage and rotations became increasingly concentrated in the hinge region.

• As hinging continued to develop and slab rotation increased in the hinge, the deformations in other slab zones stabilized and did not significantly increase, up to the 2.5–4% drift ratio range. The progress of crack development in the drop panel also slowed in the drift ratio range from 1.5–2.5%.

• Hinge formation was accompanied by:

- gradual punch-through around perimeter of drop panel zone, and spalling and rebar buckling in the slab region immediately adjacent to the drop panel zone.

This punching and spalling behavior was more severe in the case of the second specimen, which

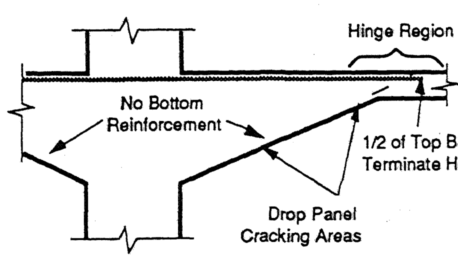


Figure 5. Hinging location & drop panel cracking

carried a higher simulated floor gravity load.

• The location of the "lip" at the top of the slab at the drop panel zone perimeter also corresponds to the cut-off point of the slab top bars. Dowel action of the terminated bars seemed to precipitate or exacerbate the spalling.

• Eventually in the higher drift ratio ranges (2.5–4%), punch-through, spalling and bar buckling became severe, and slab areas outside of the drop panel zone contributed more substantially to slab ultimate resistance. The specimen continued to resist lateral loads even after severe slab damage.

Gradual panel-zone punch-through (with the column/drop panel zone supporting the rest of the slab in catenary action), spalling and drop panel deterioration combined to produce severe slab deterioration at the 4 percent drift ratio level. Testing of each specimen was discontinued after a single cycle at 4 percent drift ratio.

No column cracking or other distress was observed in either test.

Large scale spalling of the drop panel/column capital did not occur in either specimen, although it may have been imminent at the 4% drift ratio level in the second specimen due to excessive damage and spalling of concrete.

### 4.1 *Continued vertical load-carrying capacity*

To examine the continued vertical load-carrying capacity of the heavily loaded floors in the building following a very large earthquake, the lead weights were re-distributed, and additional weights were added to the second specimen following cyclical testing. The specimen was returned to the neutral position, and the weights were re-distributed to accurately simulate the shear stresses around the critical, highly damaged drop panel perimeter under heavy gravity floor load conditions. The specimen continued to carry the imposed weights, and no further specimen deformations were observed.

### 4.2 *Load-displacement behavior*

Much of the critical engineering information obtained from the test may be seen in plots of the lateral load applied to the column versus displacement of the top of the column. Figure 6 illustrates the behavior of specimen #1 as the slab hinge mechanism developed.

• The load cycles trace out roughly oval-shaped hysteresis loops.

• Initial cycles show higher stiffness, indicated by a more vertical loop in the plot.

• The final (4% drift ratio) load cycle loop shows only minor loss of strength.

• Relatively good energy absorption is indicated.

• The loops become quite broad, but not 'pinched.'

Similar results were found for specimen #2, with a somewhat larger loss of strength in the final load cycle.

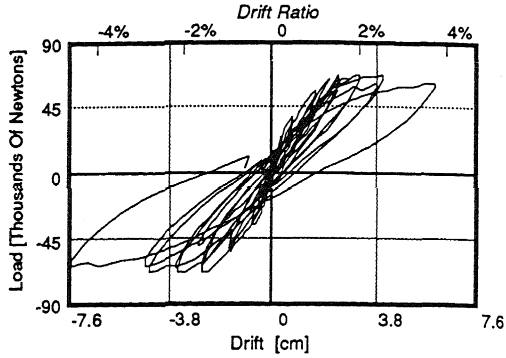


Figure 6. A load-displacement plot for specimen #1

## 5 INTERPRETATION OF TEST RESULTS

The testing established reasonable limits for typical slab/column assembly strength and ductility (i.e., capacity). The assessment of the potential for structural failure requires a comparison of this capacity with the anticipated earthquake strength and ductility demands. Failures are anticipated where estimated earthquake demands exceed component capacity.

The relatively ductile performance of the specimens presented several key questions which required evaluation before the expected building performance in moderate-to-severe earthquake ground shaking could be assessed:

• Will ductility demands be concentrated in the 'soft' and potentially weak first story of the actual building?

• Will the softening behavior of the slab-column system increase the displacements under earthquake loads, resulting in overall structural instability and collapse?

• Are there other features in the overall building structure which will result in premature failure?

In order to answer at least some of these questions, it was necessary to perform additional computer-based structural analysis, taking into account the results of the specimen tests.

## 6 POST-TEST ANALYSIS

A 2-dimensional nonlinear computer model of the actual building was developed, using a version of DRAIN-2D (Kaanan and Powell, 1973, 1975). The model was similar to the linear elastic model used in preliminary analysis, except that it incorporated the strength and stiffness characteristics found in the tests, (scaled to full size). The computer model represented an average slice of the building's structure, including one row of columns and the tributary slab area. The presumed dead loads and live loads were modeled, and the 'P-∂' effect was included. The beam members representing the reinforced concrete slab included rigid offsets for the drop panel zone, forcing plastic hinges to occur at the correct location in the slab. Variations in floor slab reinforcement were accounted for in assigning moment capacities. A simple elasto-plastic representation of slab moment-rotation behavior was used. The initial stiffness was 'softened' per the test results in selected analyses to account for stiffness degradation in floors subjected to high drift demands. The stiffening effect of nonstructural elements was excluded.

6.2 *Earthquake Time Histories*

Three earthquake time histories were used to represent large regional events which could affect the building in southern California. Two of the time histories represented earthquakes having return periods of about 475 years; the third event represented a Maximum Credible earthquake which could affect the site:

- 475-year Event #1 was taken from a 1971 San Fernando earthquake time history recorded in Van Nuys, scaled to 0.47g and spectrally modified to fit a site-specific uniform-risk response spectrum.
- 475-year Event #2 was taken from the Taft record of the 1952 Kern Co. event, scaled to 0.47g and spectrally modified to fit a Uniform Building Code spectral shape (Soil Type 1).
- The Maximum Credible time history (Figure 7) was derived from a 1979 Imperial Valley earthquake record, scaled to 0.60g and spectrally modified to fit a mean spectrum for a M7 event occurring on a fault located 1 km from the site.

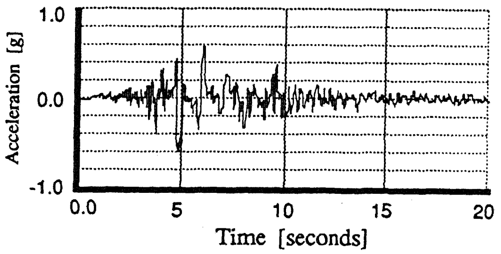

Figure 7. Maximum credible earthquake time history

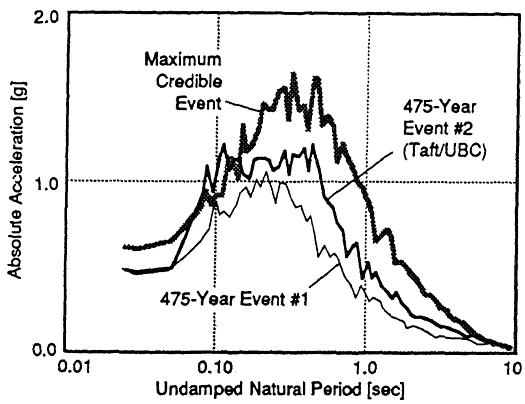

Figure 8. Response spectra for 3 earthquake events

Figure 8 shows a comparison of 5-percent damped response spectra from the 3 time histories.

6.3 *Results of Computer Modeling*

Figure 9 presents a summary of the drift ratio demands resulting from the earthquake time history analyses, plotted story-by-story over the full building's height. Based on test observations, at a drift ratio of about 1.0 percent, damage would appear to be repairable. Beyond drift ratios of 1.5 to 2 percent, the structure would likely face condemnation. For drift ratios ranging from 1 to 2 percent, the feasibility of repair is uncertain.

Figure 9 reveals a concentration of drift demands in the middle stories, which may be attributed to:

- The reduction in slab strength and drop panel size above floor 7, and
- Reduced response in the building fundamental mode of vibration (period≈4 seconds), due to structural flexibility and softening, and more significant response contributions from higher modes of vibration which fall within the dominant energy range of the ground motions (periods from 0.10 seconds to 1.0 seconds).

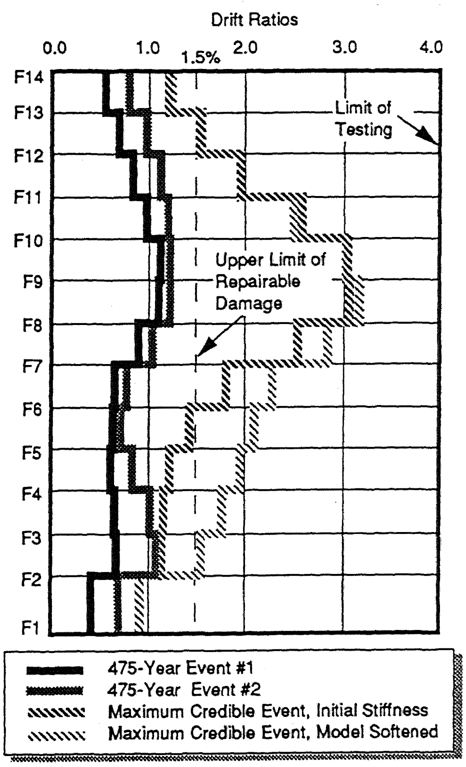

Figure 9. Drift demands from post-test analysis

## 7 CONCLUSIONS

Two 1/2-scale slab/column specimens were designed to replicate typical components in the earthquake lateral force-resisting system of a building located in southern California. The specimens were subjected to lateral displacement cycles simulating increasing earthquake displacement demands, and specimen responses were recorded instrumentally, visually and photographically.

Nonlinear dynamic computer analysis of the building structure was carried out following the tests to extrapolate the observed behavior to the overall structure. This post-test analysis leads us to believe that:

- The building should survive moderate-to-strong levels of ground shaking with limited damage, which may be repairable.
- The building should survive severe ground shaking without gross collapse, although the damage incurred may not be repairable.
- The weight and flexibility of this building will lead to larger-than-usual drifts, contributing to substantial nonstructural damage.

## 8 ACKNOWLEDGEMENT

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