11. Opinion Paper
Unexpected Stress Failures during Earthquakes

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At present it is the consensus that structures should be designed to undergo ductile deformations if subjected to very strong ground shaking, so structures designed without consideration of large deformations can be expected to experience undesirable damage. However, in most strong earthquakes some unexpected ground motion or unexpected structural failure occurs which deserves closer examination. In both the Northridge and the Kobe earthquakes there were unexpected ground motions as well as unexpected structural damage. The reason such failures are unexpected is that their possibility was not identified during the design process.

Structural failures are valuable sources of information for engineers for it is important to know why and how failures occur so that similar failures can be avoided in the future. An unexpected type of failure occurred during the Northridge earthquake when welded steel members cracked in many structures. Also, an unusual type of failure occurred during the 1995 Kobe earthquake in which an intermediate story of a building disappeared and the building was one story less in height than before the earthquake. This happened to a number of buildings so the cause was not an accidental defect in construction but must have been a common feature in the design of these buildings. I was told when I was in Kobe that the failures occurred where there was a change in type of construction from steel to concrete but the precise nature of the situation was not explained. In my opinion the failures of the columns resulted from an unrecognized stress distribution in the columns that was built-in by the type of construction. The usual method of repair of these structures was to remove the portion of the building above the collapsed story and then build a top story and roof to replace it. For example, a ten-story building whose fourth story collapsed is now a four-story building.

Engineers are well aware that buildings have many different kinds of built-in stresses, that is, stresses that are developed not by the application of external actions such as gravity, earthquake, and wind but are produced by such things as concrete shrinkage, concrete creep, temperature expansion, unequal footing settlement, etc., and in the case of steel structures the internal stresses and strains produced by the production process, fabrication methods and welding procedures. An illuminating example of built-in stresses is the former Southern California Edison Company building in Los Angeles. This ten-story steel frame building was

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erected in the late 1920's and its joints were welded to produce an earthquake resistant frame. When the welding was completed it was found that the columns in the exterior walls of the building had raised off the foundation by a significant amount. The problem was solved by placing the nuts on the anchor bolts and screwing them down until the column base plate was back in contact with the foundation, which now experienced an uplift force exerted by the column. This would have put appreciable forces and stresses in the frame. If the nuts had been fastened down tightly at first, no doubt similar stresses would have been produced by the welding. In addition to such built-in stresses there are also stress-risers of various types, which could also be called built-in, such as sharp corners, local defects, etc., which can interact with the stresses produced by gravity, earthquake and wind.

The preceding discussion highlights the difference between actual stresses in a building and the stresses calculated according to the code requirements. The calculated stresses involve a number of simplifying assumptions and, in addition, neglect the existence of built-in stresses and stress risers. So far as I know there have been no measurements of the true stresses in a building as compared with the calculated stresses even though such information could be of value in seismic design. It seems that such stresses are not taken into account in the design calculations primarily because of lack of knowledge, normally the built-in stresses are not considered because experience has shown that they do not seem to affect the performance of the building, but, as the Northridge and Kobe earthquakes have shown, the built-in stresses and the stress-risers can have adverse effects.

During the Northridge earthquake numerous cracks were developed in welded joints in buildings, in steel bridge girders, and in column base plates. These cracks appear to have originated at a stress-riser point in or adjacent to a weld and then propagated through the material. This behavior was very clear in the case of the girder bridges and the column base plates because they were relatively simple compared to the welded joints which form a very complicated structural component. In the case of the bridge girders, one or more cracks originated at the lower end of the welding that attached a stiffener to the web (Figure 1), and the cracks propagated in the web of the girder for considerable distances. The stiffener was part of the cross-bracing between girders, so it was a stressed member. In the case of the column base plates in the Cal State Northridge University Library Building, large 4 inch thick plates were welded to the columns and were anchored to the foundation. The columns formed the chords of vertical trusses and during the earthquake a column pulled up and pushed down alternately as the building vibrated. In a number of instances a crack originated at the weld and then propagated through the 4 inch thick plate to its edge (Figure 2). It is clear that as a result of the welding a stress-riser was formed that initiated the crack.

In the case of the Kobe building collapses, as additional reports on the earthquake have come out of Japan the situation has become clearer and I think that the cause of the failures was an unusual type of stress distribution resulting from the nature of the construction. These were not the usual weak-story failures that we have seen in the past where the bending moments at the top and bottom of the columns exceeded the strength and the story failed as a mechanism. In the Kobe failures the upper parts of the buildings came almost straight down and were clearly not the result of a mechanism type of failure. Now from various reports we do know the nature of the steel and concrete construction in the collapsed stories.

To understand the failures it is first necessary to explain a special type of combined structural-steel and concrete design used in Japan in the 1950's and 1960's. When in Japan in those days I saw this type of construction underway for three and four-story buildings. Steel columns were fabricated in the shop out of four angles and latticing (Figure 3) and the first story columns were transported to the site and erected. The beams of the second floor were fabricated similarly with four angles and latticing and were then erected at the site. Wood forms were then placed around the columns and concrete was poured resulting in a square concrete column with axial reinforcing of four steel angles and shear reinforcing of lattices. Then wood forms were suspended from the steel beams and concrete was poured for the beams and reinforced floor slab. At this stage there was a reinforced concrete first story and second floor, and the procedure was then repeated for the second story and third floor. This was an ingenious method of constructing a reinforced concrete building but, apparently, is not used anymore. Recently I saw a two-story building under construction in Kyoto with steel I-beams and columns which presumably reflects the fact that the relative cost of labor has increased.

When taller buildings with ten stories were constructed, the heavy loads on the lower columns were accommodated by enlarging the concrete columns so that they had a combined steel-angle/reinforcing-bar system (Figure 4). The structural steel core in the column stopped at the floor level and above was a normal R/C column with reinforcing bars and column ties. It is clear that the four steel angles with enclosed concrete would form a relatively rigid core whose effective modulus of elasticity would be greater than that of concrete. Figure 5 shows the transition point where the more rigid core of the column ends at the floor level. Due to the greater rigidity of the core, and the concrete creep over the years, there would result a higher compressive stress over the end of the core than over the surrounding concrete of the column. Just above the floor there is then a transition zone in which the axial stress distribution changes from non-uniform to uniform with increasing distance above the floor. In the transition zone there would be shearing stresses in addition to axial compression stresses and these built-in stresses would make the bottom of the column susceptible to a special type of brittle failure. I believe that when the bending moments and shear forces produced by the earthquake were superposed on the existing stresses they triggered the failure of all the columns in the story. This would cause the bottom of the columns to disintegrate because of the transition stresses and let the upper part of the building come down with a small lateral offset. Some research would be necessary, of course, to determine the true transition stresses and their effect on the failure mechanism.

Although this failure analysis seems reasonable for the way the buildings were constructed there is no hard data to corroborate it. I believe, however, that there is value in putting this analysis of the problem on record as it, and also the crack starters in welded steel assemblies, bring to the attention of engineers the hazard of built-in stresses which can have a significant effect on the performance of buildings during earthquakes. Finally, what about retrofitting Kobe-type columns to improve their performance during earthquakes? Probably, using the Caltrans type of retrofitting concrete columns would be appropriate, that is, provide circumferential containment around the bottom of the column to suppress brittle shear-stress failure.

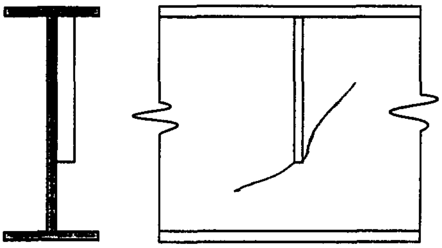

Figure 1. Cracks in welded steel bridge girder.

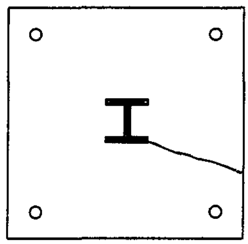

Figure 2. Crack in 4-inch thick base plate.

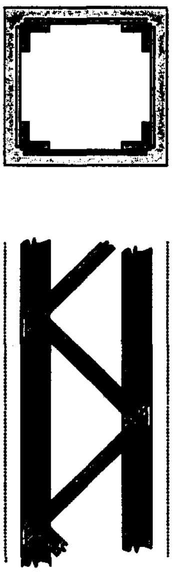

Figure 3. Structural steel reinforced concrete column.

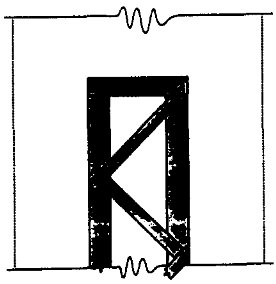

Figure 4. Steel core in concrete column whose rigidity produces a concentration of stress over the top of the core.

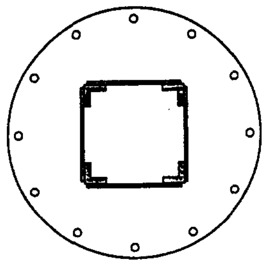

Figure 5. Column with combined reinforcing.