The Loma Prieta Earthquake: Implications of Structural Damage

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Proceedings: Second International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, March 11-15, 1991, St. Louis, Missouri, Paper No. LP03

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SYNOPSIS: The Loma Prieta earthquake provides a wealth of information on the seismic response of a wide variety of structures over a large metropolitan area. Soil amplification at sites distant from the epicenter contributed significantly to the substantial damages developed during the earthquake. Because of the large shaken area, the earthquake provides much useful information for all those interested in earthquake engineering. Structural damages resulting from the earthquake are reviewed herein with emphasis on buildings and bridges. Implications for modern design and retrofit methods are highlighted. Emphasis is placed on the need to carefully consider soil conditions, to treat the structure as a system rather than as an assemblage of independent elements, to explicitly define performance expectations, and to increase efforts to retrofit older seismically hazardous structures.

INTRODUCTION

The Loma Prieta earthquake of October 17, 1989 has proven to be one of the most costly natural disasters in American history. Probably the most vivid examples of structural damages from the period immediately following the earthquake were the catastrophic collapse of the Cypress Street viaduct on Interstate 880 in Oakland, the partial collapse of a section of the San Francisco - Oakland Bay Bridge and the distress to numerous buildings in San Francisco's Marina district. As additional data was gathered during the days and weeks that followed the earthquake, it became abundantly clear that damages were far more extensive than originally suspected.

In total more than 18,000 dwellings and several thousand other structures were significantly damaged by the earthquake. Nearly a thousand of these were destroyed by the earthquake itself and 500 more have been demolished since the earthquake. Numerous buildings and bridges stand empty today as studies continue to determine the technical and economic feasibility of restoring their structural integrity.

Particularly severe damages were observed the epicentral region. The older downtown regions of Santa Cruz, Watsonville and Los Gatos (Fig. 1) were hard hit as were individual homes in the Santa Cruz mountain. Today, several square blocks in Watsonville and Santa Cruz have been demolished and stand vacant as silent reminders of the devastation caused by this earthquake. Highway and building structures were also damaged in localized regions throughout the greater San Francisco Bay Area. Significant damages were also observed in lifeline facilities (water pipelines and treatment plants in particular), telecommunication facilities, and in the architectural and mechanical components and contents of buildings. The total cost of the physical damages to structures is estimated by FEMA to be in excess of $ 6.7 billion. However, the total cost of the earthquake considering the loss in immediate and long term business revenue, damage to contents and inventories, medical and workman's compensation claims, and reallocation of resources is likely several times this amount.

The human toll of the earthquake was also substantial. Sixty seven fatalities resulted from the earthquake and more than 3,700 injuries were reported. While these numbers are large, they are no where as large as might have been expected considering severity of structural damage. This is attributable in part to the fact that on the day of the earthquake many people had gone home early to watch the World Series of baseball. For example, based on the number of vehicles expected on the Cypress Viaduct on a normal workday, estimates of fatalities made just after the earthquake exceeded 700, substantially greater than the 42 deaths that actually occurred. This is indeed fortunate, but it is clear that had the earthquake occurred under more normal circumstances, casualties in this and other structures would likely have been far greater.

In addition, the number of damaged and destroyed dwellings resulted in more than 10,000 displaced persons. Finding adequate food and shelter for these individuals contributed significantly to the recovery efforts. Fortuitously, the weather was good and with a few important exceptions the transportation infrastructure remained intact permitting relief supplies to be delivered to the needy.

The Loma Prieta is the first major earthquake to strike a major metropolitan area in the U.S. in nearly 20 years. As such, it provides the earthquake engineering profession with a unique opportunity to assess structural design and retrofit methods. As the earthquake effected an area where even larger earthquakes are expected, an examination of the damages will provide particularly valuable indications of future damage trends. Because the levels of motion was not unusually severe, and because many different common structural types and soil conditions were excited, the earthquake should also provide planners and engineers in other parts of the country with many valuable lessons.

In this paper, the overall nature of the structural damages are reviewed. After making some overall comments on the apparent severity and distribution of the damages, information regarding the specific types of structural damages observed is presented. Due to space limitations, emphasis is placed on building structures and bridges. Finally, the implications of these damages for the design of structures in seismically hazardous areas are offered.

Additional detailed information on the damages can be found in the References.

SEVERITY AND DISTRIBUTION OF DAMAGES

Ground motions were recorded at 131 sites and in 46 buildings during the earthquake. Correlation of this information with observed damage will provide a focus for research for many years to come. However, several important observations regarding the distribution and severity of structural damage can already be made.

First, ground motions recorded in the epicentral region were quite severe, ranging up to 64% of gravity. Motions greater that 30 to 40 percent of gravity were detected over a very wide area. Since these levels of motion approach those considered in the design of new structures, one might expect to see in these areas some significant damage to engineered structures, especially to older ones designed to lower force levels. However, damage in these areas with few exceptions concentrated in older unreinforced masonry and wooden structures. Other, relatively modern types of engineered structures with known seismic vulnerabilities (such as non-ductile reinforced concrete buildings, tilt-up structures, precast buildings, etc.) generally survived the motions in the epicentral region without serious damage.

While the precise reasons for this apparent anomaly are under study by many at the moment, it reconfirms the limitations of peak ground acceleration as a reliable index of earthquake damage potential. In this case, the recorded motions in the epicentral distance ranged from 6 to 15 seconds, depending on the record and the method used to determine duration. This duration is short for this magnitude event. In addition, surface displacements induced by the shaking were also relatively small in comparison with those developed during other damaging earthquakes of this magnitude.

Comparison of the motion and damage data obtained for this and other earthquakes will provide very important insight into the factors that influence damage in structures. Until the outcomes of such studies are known, it is prudent to acknowledge that the Loma Prieta earthquake did not generate particularly damaging motions in the epicentral region and that it may not be conservative to extrapolate damages occurring in this region to other areas of the Bay Area or the country where earthquakes with greater damage potential might occur.

Damage was quite serious to a wide variety of structural types in some areas outside the epicentral region. In particular, regions of San Francisco, Oakland and other areas as far as 80 to 100 km. from the epicenter suffered significant damages. Peak ground accelerations recorded on firm soil at these distances were generally around 10% of gravity or less. However, most of the structures damaged in these areas were concentrated around the San Francisco Bay on man made land overlying bay mud or on soft, deep soil deposits. In these areas motions were recorded with peak accelerations ranging between 20 and 30% of gravity.

Attenuation of peak ground acceleration at distance was substantially less severe than might be expected on the basis previous west coast earthquakes. Whether this discrepancy relates to special features of the fault mechanism and resulting directivity for the Loma Prieta earthquake or unusual, high amplification of the soft bay mud requires additional study. Regardless, the large inventories of older, seismically vulnerable, structures located over these soils resulted in substantial numbers of damaged structures. Soil effects clearly had a dominant influence on the unusual severity and distribution of damages throughout the Bay Area.

Nonetheless, serious damage still concentrated in only the most vulnerable of structures since motions still were generally significantly below current design levels. On the other hand, the damage observed possibly provides a reasonable indication of the damages that might develop over a wider area if a large local earthquake occurred in the Bay Area. In this case, peak accelerations would approach those that developed only on soft soil during the Loma Prieta earthquake. These damages also may provide a useful indication of the nature of damages that might occur other areas of the country where similar soft soil conditions exist. The 1989 Loma Prieta and 1985 Mexico earthquakes clearly point out the potential vulnerability of urban areas overlying soft deep soil deposit, even for distant earthquakes. This should have major implications for the Pacific Northwest, the central U.S. and many locations on the eastern seaboard where similar conditions exist.

Significant damages also occurred to new and older buildings in the Palo Alto area. This region was relatively close to the epicenter and had large stocks of buildings. Ground motions ranging between 30 and 40 percent of gravity were recorded. Several new concrete and steel buildings suffered damaged as did older reinforced concrete, wood and masonry structures.
Thus, it appears that structural damage is highly dependent on the nature of the building inventories present in the shaken area, the types of soil conditions and the specific characteristics of the motions generated by the fault rupture and that arrive at the site. While efforts aimed at micro-zonation must continue, this sensitivity makes it from a practical perspective all the more important to design structures in accordance with the basic lessons learned from past earthquakes.

These earthquakes have suggested the desirability of designing simple structures that are inherently insensitive to the uncertainties associated the input motions and soil conditions. Thus, emphasis should be placed on avoiding systems with limited redundancy or that tend to concentrate damage in a few locations, on providing details capable of large inelastic deformations, on selecting structural systems that are able to limit deformations to reasonable levels, and on attaching nonstructural components in such a manner that they do not adversely influence structural response and are not extensively damaged by the structural deformations.

The damages produced by the Loma Prieta earthquake must be reviewed to determine the continued soundness of these past lessons. In the following sections, the performance of a variety of structures will be reviewed with this in mind.

DAMAGE TO WOODEN BUILDINGS

As indicated previously nearly 20,000 dwelling units were damaged during the Loma Prieta earthquake and 10,000 individuals were forced to relocate as a result. A majority of these displaced people lived in wood frame houses and apartments. Many homes located in the Santa Cruz mountains, in the immediate epicentral region, were damaged. Damages ranged from fallen chimneys and porches to partial or complete collapse. Many wood frame buildings in the hardest hit areas were overwhelmed by the seismically induced inertia forces. Sadly, the majority of the damage occurred in older wood frame buildings where the structures were simply not connected to their foundations. During the earthquake, as they have in innumerable past earthquakes, the structures simply shifted off their foundation with resultant vertical dislocation (Fig. 2).

Another common type of failure that was observed related wood frame dwellings built upon short pony or cripple walls. These short walls provided a two to four foot tall access space that was used for storage, ventilation and to accommodate sloped sites. In many older buildings the framing between the ground floor and the foundation consisted of vertical studs and horizontal siding. Even though diagonal braces were provided in the upper levels to resist lateral loads, the braces were typically omitted from the pony walls. In addition, the horizontal siding covering these walls in most cases had badly deteriorated and effectively provided little or no lateral resistance (Fig. 3). The failure of these short support walls resulted in large vertical and lateral displacements of the supported structures (Fig. 4).

The unfortunate aspect of both of these forms of damage is that simple and economical retrofit procedures, if implemented prior to the earthquake, could have prevented or significantly reduced the severity of the observed damages. Repair following the earthquake was generally extremely expensive as a result of not only the required structural repairs, but because of wide spread damages to architectural, electrical and mechanical features. Pre-earthquake retrofits to these walls and foundations were seen to be effective in several instances in Santa Cruz and elsewhere.

Another form of damage to wooden structures was observed in the Marina District in San Francisco. This area was constructed on loose, fine sandy hydraulic fill. During the earthquake the area exhibited evidence of significant liquefaction with localized sand boils, lateral spreading, slumping and heaving of sidewalks and roadways.

Many wood buildings were damaged or collapsed in this area. However, a detailed inspection of the damages indicates that the collapses concentrate in three or four story apartment buildings (Fig. 5) located on the corners of blocks (or where a structure was not sandwiched between two other adjacent buildings with little or no separation). These older apartment buildings were generally constructed with relatively massive apartment floors supported over a level of garages (Fig. 6). The bottom level thus was largely open to allow for multiple garage door openings. Typically, parking stalls were several cars deep and often extended out though the back of the building to provide for even more parking. This situation resulted in a soft first story. Lateral load resistance was provided by horizontal wood sheathing (covered by brick veneer) placed over only a few walls. This sheathing was observed to have decayed severely in many cases during the life of the structure.

The amplified seismic motions in the Marina District resulted in the collapse of the lower level of these corner apartment buildings in many cases. It is significant to note that these types of apartment complexes are quit common throughout San Francisco. Some apparently identical buildings located only a few blocks away from the heavily damaged area, but on firmer ground, remained virtually undamaged following the earthquake. Nonetheless, the potential hazard posed by these structures during future more severe earthquakes should be carefully investigated.

Initial suppositions regarding to the causes of the severe structural damage in the Marina District focused on the observed soil liquefaction. However, structures did not collapse or suffer substantial damage even in the most heavily afflicted area, if the lateral load resisting system was continuous over the height of the building. Another remarkable feature was the limited damage to residential homes located along the middle of a block. These structures typically were two or three stories tall, with adjacent structures having the same floor elevations and virtually no separation. As noted in during other recent earthquakes, such structures seem (at least initially) to buttress one another and suffer only limited damage.

More extensive damage occurred in buildings located at the corners of blocks. Where liquefaction occurred near or under a structure, vertical differential settlements of a few inches were observed with resultant structural and especially architectural damage. Similar damages were also observed in other areas of San Francisco, notably in the Mission Creek area located south of Market Street.

DAMAGE TO UNREINFORCED MASONRY BUILDINGS

The older downtown areas of many well established cities in California were built of unreinforced masonry. These commercial areas suffered significant damages in Watsonville, Santa Cruz, Oakland and San Francisco. However, damage appears to be strongly influenced by local soil conditions and the intensity of ground motions. This can be seen in the following table where the approximate numbers of unreinforced masonry (URM) buildings vacated following the earthquake are compared with the total stocks in various cities. Cities are listed in order of decreasing epicentral distance.

Table 1. Damage Statistics for URM Buildings

City	Approx. Number of URM Buildings	Number Vacated
Berkeley	400	5
San Francisco	2000	252
Oakland	2000	400
South San Francisco	42	2
San Mateo	28	1
Salinas	96	16
Hollister	17	14
Palo Alto	49	0
Mountain View	20	6
Gilroy	41	2
San Jose	230	3
Los Gatos	29	11
Santa Cruz	46	36

Only limited damage was detected in Gilroy, for instance, even though motions in excess of 50 percent of gravity were recorded in the vicinity of several unreinforced masonry structures. I other cities damage only concentrated in certain areas, with severe damage in some areas and virtually none in others. While soil conditions play an important role in this, the precise interrelationships between the various factors influencing damage in this class of hazardous structure needs careful study.

The most prevalent form of damage to these structures was the collapse or dislocation of parapets. In most of these cases the masonry was tied to the floor diaphragm by means of steel anchors. The short extension of the wall above the roof fell from many buildings, even where only moderate ground motions occurred far from the epicentral region.

A particularly devastating form of damage resulted when the walls were not tied to the floor diaphragms. In many cases the walls failed out-of-plane, falling into the street (Fig. 7). In several notable cases the walls fell from upper floors of a building onto the roof of a lower adjacent structure (Fig. 8). The damage to the lower building resulted in two deaths in two separate buildings in Santa Cruz. In another instance, five casualties resulted as a wall collapsed in to the sidewalk area in front of an unreinforced masonry structure. For the level of shaking encountered, moderate amounts of tie reinforcement appeared effective in limiting thi type of damage.

In some cases in-plane shear failures of brick walls was observed (Fig. 9). In some cases thi distress resulted in local collapse of wal panels. In other buildings wide spread, incline cracks developed, stepping along weak morta joints. In many cases buildings with moderate cracking have been left un-repaired with continued occupancy or the cracks have been simply repaired with epoxy injection to insure water tightness.

Brick and stone architectural veneers fell from many buildings. Typical metal inserts embedded in the mortar joints proved insufficient when the attachment nails pulled from the wood framing. The framing under the veneer had deteriorated badly in many of these cases. In other cases the veneers were dislodged as a result of hammering against adjacent structures.

DAMAGE TO REINFORCED CONCRETE STRUCTURES

While non-ductile concrete frames have performed poorly in past earthquakes, few of these types of buildings (see later comments on concrete bridges) were located in areas subject to severe ground shakings during the Loma Prieta earthquake. In some cases severe shear cracks were observed in columns. However, the buildings in question (for example, along Mission Street near Third Street in San Francisco) were located mid-block with no space separating them from the adjacent structures. It is expected that the adjacent structures provided the required later resistance in these cases.

In Watsonville, where peak accelerations of 39% g were recorded, an old five story reinforced concrete frame structure appeared to suffer mainly architectural damage. The building had a number of broad shear walls which apparently accounted for its good behavior in spite of the sever motions recorded in the area.

Damage was observed in a number of newer reinforced concrete structures. For example, a mechanical penthouse, containing a water tank, fell from the top of a reinforced concrete multistory hotel in Burlingame. Across the street another hotel, build less than 16 months prior to the earthquake, suffered significant damages to its shear walls in the lower levels and to its diaphragms near the shear walls. Both of these hotels were constructed on fill over bay mud. Motions at the nearby San Francisco International Airport were up to 33%g.

A ten story moment frame constructed in San Jose of lightweight concrete suffered severe spalling damages in several locations and architectural damages in spite of ground motions in the area less than 11 % g. Lightweight concrete shear walls in a 15 story telecommunications building (Fig. 10) in Oakland suffered severe cracking, spalling and splitting in the lower story. This building was designed as a dual system with steel frames proportioned to carry 25% of the lateral forces along with the gravity loads.

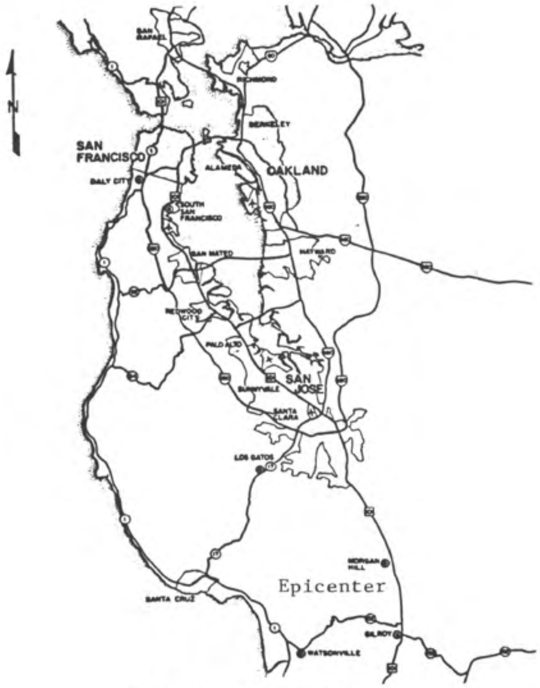

Fig. 1 Area Map

Fig. 2 Damage to House Not Bolted to Foundation

Fig. 3 Damage to "Pony" Wall

Fig. 4 Collapse of Home on "Pony" Walls

Fig. 5 Collapse of Apartment in Marina District

Fig. 6 Typcial Marina District Apartment

Fig. 7 Out-of-Plane Failure of Masonry Wall

Damage also occurred in reinforced concrete buildings used as hospitals and schools. The seven story tower of the Peralta Hospital, built in Oakland in 1927, was severely damaged and has been demolished following the earthquake. The Palo Alto Veterans Administration Hospital suffered serious damage, including shear cracking to the columns in two of six buildings. The four buildings without damage had been previously retrofit. A number of reinforced concrete buildings on the Stanford University campus, including the School of Business Administration, the Library and some residence halls suffered moderate cracking. The John O'Connell High School in San Francisco suffered serious structural cracking to its frame-wall system.

DAMAGE TO STEEL BUILDINGS

Damage to steel buildings was typically less obvious than that to masonry or concrete structures. However, there were several reports of buckled braces in several buildings along the San Francisco Peninsula near San Mateo and Palo Alto. In most cases these could be fixed within a few days. In other cases substantial effort was needed to find damage in buildings where large amounts of architectural distress suggested the presence of structural damage. Evidence of panel zone buckling, gusset plate yielding and buckling, and column buckling were found in several buildings.

DAMAGE TO RETROFIT STRUCTURES

The engineering profession has known for many years of the seismic vulnerability of many types of structural systems and has attempted to retrofit many of these structures. The Loma Prieta earthquake provides an opportunity to assess the efficacy of the procedures used.

A good example of the effectiveness of retrofits is the performance of a four story reinforced concrete telecommunications building in Watsonville. This building had been be upgraded by infilling windows and adding shear walls. During the earthquake accelerations were recorded up to 1.24g at the roof without the building suffering any significant structural damage. Another example is the retrofit reinforced concrete structures at the Veterans Administration Hospital in Palo Alto. As mentioned previously, these suffered little damage in comparison to the non-retrofit units.

On the other hand, there is ample evidence that some retrofit schemes did not work as well. This is attributable to the lack of any performance standards for retrofit work, any code to set load and detailing requirements, and virtually no research on the effectiveness of many type of retrofit procedures used in practice.

One example of this situation is a 6 story reinforced concrete building constructed in San Francisco at the order of corner and Fourth Street during the early 1980's (Fig. 11). During change of ownership a few years after it was built a number of seismic deficiencies were identified. These were remedied on the basis of protection of the life safety of the occupants by adding steel braces and shear walls. Following the earthquake, the connections in the steel braces were observed to buckle (Fig. 12), the attachments f the braces to the concrete frames had slipped (in some cases due to faulty installation of anchor bolts), extensive cracking occurred in the shear walls, and initiation of punching shear failures in a few column to flat late connections. The building has been since repaired (Fig. 13) to restore the capacity that existed at the time of the earthquake.

Another example, is the Hotel Oakland in Oakland. This is a steel frame building with extensive masonry infill panels. In retrofitting this structure new lateral load resisting elements were added and the exterior masonry elements were positively attached to the building to prevent large panels from falling into the street. As result of the earthquake nearly all of the masonry piers in the building developed distinctive x-shaped shear cracks. In addition, at a few locations masonry elements were dislodged from the building and fell to the street (Fig. 14). This building apparently preserved life safety as intended, but extensive and expensive repairs were required to restore the building fully to service.

Most of the focus of retrofit work has logically been on unreinforced masonry buildings. In a recent study by Conrad (1990) 400 of San Francisco's more than 2000 URM buildings were inspected and 69 were identified that had been retrofit to some degree. This would indicate that only about 3% of the city's URM buildings have been retrofit. Most of the retrofits had been implemented through the introduction of steel braces. Retrofit structures tended to be found in clusters where a local community was being redeveloped. The report found that 54% of the retrofit buildings suffered no damage, but no comparison with he performance with adjacent non-retrofit structures was offered. Light damage was observed in 22% of the retrofit structures and moderate damage was seen in 20%. Heavy damage was seen in three buildings (3%).

One of these structures (259 Front Street) was located at the corner of a block and it was heavily braced along two street sides (Fig. 15). The added braces at the street level buckled out of plane significantly (Fig. 16), but no damage was actually found in the masonry portions of this four story building. The second structure (1051-1075 Battery Street) was also braced but the mortar in the existing masonry walls was apparently so poor that random sections of the exterior wall fell from the building. Another building (located at Sixth Street and Bluxome Streets) had apparently been partially retrofit by addition of wall anchors. Five people were killed as the upper level wall on one side of the structure fell onto the sidewalk and street. In all of these cases it is clear that the lack of a consistent, systems approach to the design of the retrofits had a detrimental effect and that a new class of potentially hazardous buildings (i.e., the inadequately retrofit structure) needs to be investigated.

Fig. 8 Masonry Wall Fell on to Adjacent Roof

Fig. 9 In-Plane Failure of Masonry Wall

Fig. 10 Lightweight Concrete Shear Walls Failed at Base of Building

Fig. 11 Retrofit of New Reinforced Concrete Building

Fig. 12 Buckled Brace Intersection

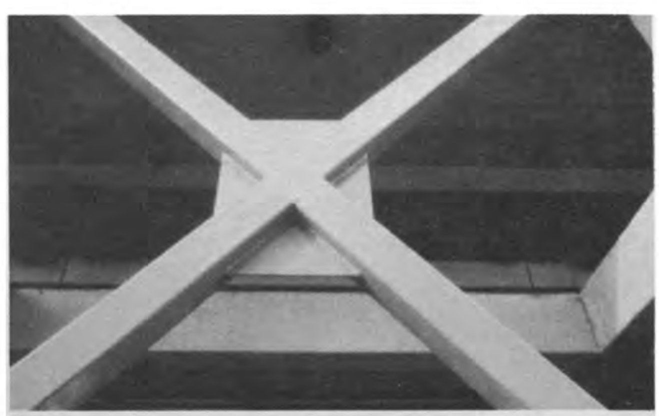

Fig. 13 Repair of Damaged Intersection

DAMAGE DUE TO HAMMERING OF ADJACENT BUILDINGS

Another important observation from the Loma Prieta earthquake is that many buildings suffered damage due to pounding. The separation of many buildings was insufficient to avoid collision of the upper stories. As a result a wide range of damage resulted, from local spalling of veneers near the impact point through cracking of vertical supporting members to failure (though not collapse) of columns. Field inspections have identified several hundred cases where pounding contributed to the observed damage. In many instances it was apparent that a few more cycles of impact could have caused the failure of one of the structures involved (Fig. 17). In many of the severely damage unreinforced masonry structures evidence suggests that pounding was a contributor to the damage.

In one instrumented building in Oakland (at the intersection of 17th and Harrison Streets) the records show clear evidence of pounding at one corner of the building. The steel moment frame building (with reinforced masonry walls along the property lines) was separated from the adjacent unreinforced masonry storefront by about an inch. However, no significant damage was observed in either building as a result of the pounding.

Damage due during the 1985 Mexico and other earthquakes suggest that pounding is a serious problem that needs to be addressed by the engineering profession. A number of investigators are currently studying this problem as it relates to the Loma Prieta earthquake.

DAMAGE TO BRIDGE STRUCTURES

More than 1500 bridges exist in the area affected by the Loma Prieta earthquake. Eighty of these suffered some relatively minor damage during the earthquake, ten required shoring (though traffic continued to flow over them during the repairs) and another ten were closed due to the severity of the damage. Three bridges suffered collapse of one or more spans. These were the double deck Cypress Street Viaduct, the truss section of the San Francisco - Oakland Bay Bridge and the Struve Slough bridge west of Watsonville. The collapsed span on the Bay Bridge was replaced within about a month. The Cypress Street Viaduct has been demolished, and six other double deck viaducts in and near San Francisco remain closed a year after the earthquake awaiting the results of engineering studies to determine whether they can be economically retrofit. The Struve Slough bridge was removed and rebuilt within a few months using modern design practices.

Virtually all of the bridges in the area had undergone the Phase I retrofit program, initiated by the California Department of Transportation (Caltrans) following the 1971 San Fernando earthquake. These retrofits consisted of installing cable restrainers at bridge expansion joints which are intended to prevent bridge deck members from sliding from their seats. These retrofits proved to be generally quite effective during the earthquake. In addition, at the time of the earthquake several bridges in the Bay Area, including the double deck Interstate 480 Embarcadero viaduct, were under design review for Phas II retrofits. The focus of this new retrofi effort was on tall single columns bents tha failed catastrophically during the 1971 Sa Fernando event.

DAMAGE TO STEEL BRIDGES

A number of long span steel bridges are used ir the Bay Area to cross the bay and to span ove the Sacramento River. Damage was reported to three of these: the San Francisco - Oakland Bay Bridge on Interstate 80, the San Mateo - Haywarc Bridge on Highway 92 and the Carquinez Bridge where Interstate 80 crosses the Sacramento River at Carquinez Straits.

The portion of the San Francisco - Oakland Bay Bridge east of Yerba Buena Island consists of a double level freeway supported on truss spans. A large steel tower (numbered E9) is providec near the center of this segment of the bridge. This tower resists longitudinal loads from a single 506-ft. span to the west and two 290-ft. spans to the east. The two concrete roadways extend across the tower on sets of 50-ft. long, simply supported steel beams running parallel to the bridge's longitudinal axis. Each of these beams was bolted to a seat angle on the east, but the seat was allowed to slide freely at the west end. During the earthquake the beams slid from the free end with the west end of the upper deck resting on the lower one (Figs. 18 and 19). The lower deck was also unseated and came to rest several feet lower on an electrical transformer housing and the tower braces.

In addition, the bottom chords of the trusses were attached to the tower by means of twenty 1-in. diameter bolts. During the earthquake the bolts in the connection on the east side of the tower sheared completely and the bearing plate moved approximately 5-10 inches to the east. This was probably the proximate cause of the roadway beams slipping from their seats.

Additional damage to the eastern portions of the bridge were also identified. These damages ranged from spalling of concrete in support piers to shifting of bearing plates and breakage of anchor bolts.

The bridge was repaired by jacking the eastern segments back into place and providing new roadway beams and supports. The support seats were constructed about 50% longer than in he original design. A more extensive study is currently under way by Prof. A. Astaneh and his colleagues at the University of California, Berkeley and by Caltrans to assess more reliable long term retrofits.

The San Mateo - Hayward Bridge is a steel orthotopic box girder bridge with a main span of 750-ft. During the earthquake one of the bearing assembles on an approach pier shifted. The cap screw used to retain the 5-1/2-inch diameter bearing pin stripped off. The upper portion of the bearing assembly shifted with respect to the lower potion by about 2-3/4 inches. The bridge was jacked back into position and repaired without interruption of traffic.

Fig. 14 Damage to Retrofit Steel Frame Building with Masonry Infilled Walls

Fig. 15 Retrofit URM Building with Steel Braces

Fig. 16 Buckled Brace in URM Building

Fig. 17 Pounding Damage

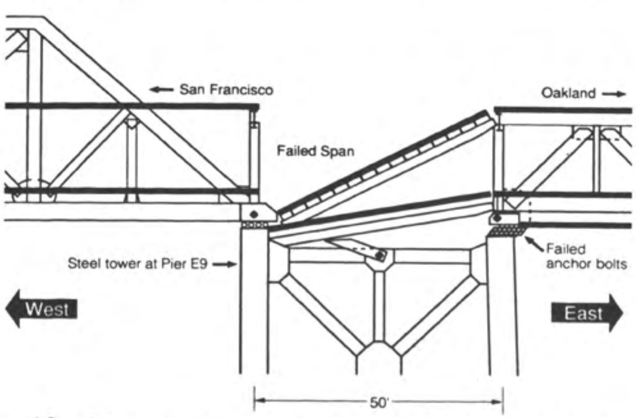

Fig. 18 Detail at Collapsed Section of Bay Bridge

Fig. 19 Detail of Seat Connection on Bay Bridge

The Carquinez Bridge consists of a pair of steel cantilever truss spans. Bearings supporting steel plate girder on the approach to the bridge reportedly tipped over. These were repaired by jacking the girders up and re-setting the girders. In addition, the piers supporting the truss spans shifted in some locations as much at 2-inches.

The closure of the San Francisco - Oakland Bay Bridge and the damage to the other bridges crossing the bay clearly indicated how vital these arteries were to transport within the Bay Area. The Governor of the State of California has since signed a proclamation mandating that such critical bridge structures be designed to remain operable following a major earthquake. This is a significant departure from the emphasis placed on life safety in the design of such structures.

DAMAGE TO REINFORCED CONCRETE BRIDGES

The most spectacular damage to reinforced concrete bridges was the catastrophic collapse of nearly a mile long portion of the upper deck of the Cypress Street Viaduct located in west Oakland on poor soil reclaimed from the bay (Fig. 20). This structure was designed in the early 1950's and construction was completed in 1957. The death toll for his collapse was 42, though as mentioned earlier a thousand fatalities or so casualties might have been expected had the earthquake occurred during congested rush hour traffic. A variety of factors contributed to the failure of the structure. These include the amplification of the ground motions in the vicinity due to the soft soil conditions, traveling wave effects, and variable soil conditions. The structure also was designed at a time when earthquake resistant design procedures were not highly developed. Thus, while the bridge appears to have been conservatively designed relative to the structural requirements in force at the time of its construction, it has a number of major deficiencies with respect to current construction practices.

This bridge represented in many respects a substantial advance in the state of the art at the time it was built. Not only did it have two levels, it was one of the first major bridge structures in the U.S. to employ post-tensioning and large diameter reinforcing bars. In most cases the structural system incorporated a significant number of flexural hinges in order simplify the future addition of access ramps, to reduce secondary forces resulting from deformations associated with post- tensioning and foundation settlement, to reduce computational effort and to control moments that could be transferred to the foundations. This resulted in a structure that was nearly statically determinant. In addition, detailing did not incorporate features needed to impart ductility to the elements (shear reinforcement sufficient to develop the flexural capacity of members, confinement of potential plastic hinge regions and inadequate development length on reinforcement), and joints were neither confined nor designed to resist the shear forces associated with realistic lateral loadings. Details of a typical transverse bent are shown in Fig. 21. In addition, in the longitudinal direction, frame action depended on the bending forces developed in the decks being transferred to the columns by torsional forces developed by shear friction. A relatively massive and brittle structure resulted.

Detailed studies by Nims et al (1989) and others have indicated that any of a number of structural deficiencies could have triggered the collapse The lack of redundancy (needed to redistribute forces in the event of the failure of an element, and the brittle details utilized made the final catastrophic outcome a certainty, regardless of the actual trigger of the failure and input details.

The dominant features of the typical collapse mechanism are shown in Fig. 22. Shear cracks in the pedestal supporting the hinged upper level columns were able to extend downward into the joint region following the projection of downward bending hooks on the end of the transverse girder's (cap's) top level reinforcement. The lack shear reinforcement in the joint allowed this inclined crack to act like a slide, with the resultant pancake collapse of the upper level.

The sudden and catastrophic nature of this failure mechanism was demonstrated convincingly during the demolition of the Viaduct. A wrecking ball was used to initiate failure in a single column. Almost immediately progressive collapse occurred throughout the upper level not only extending transversely across the roadway, but also longitudinally for six bays to the end of the remaining standing portion of the viaduct. Clearly, such brittle structures are not compatible with seismic resistant design.

Upon construction of the Cypress Street Viaduct a series of six other double deck viaducts were designed and constructed in the Bay Area. All of these suffered damages during the earthquake. Each suffered characteristic distress in the lower joint regions similar to that developed in the Cypress Street structure. An example of this is the Southern Freeway portion of Interstate 280 located near the southern border of San Francisco (Fig. 24).

A bent in a curved connector structure between Interstates 880 and 980 near the Cypress Viaduct also suffered damage. A knee joint extending transversely from the side of the deck developed severe shear cracks and a #18 bar fractured at the hook (Fig. 25). The apparently inadequate shear capacity of such joints is of concern since the structure was only about two years old at the time of the earthquake. Similar damages were seen on several older single level viaducts (e.g., where Interstate 280 crosses Mission Creek). Improved design criteria and retrofit procedures for such knee joints are under investigation by Caltrans.

Two short bridges crossed over Struve Slough near Watsonville. These consisted of multiple, skewed T-beam spans supported on monolithic pile bents. During the earthquake the supporting piles failed in shear, resulting in the deck on one of the bridges collapsing and the piles punching up through the deck.

RETROFIT TESTS OF CYPRESS VIADUCT

The need for rapid retrofit of the damaged double deck freeways in San Francisco led to a series of field tests on undamaged portions of the Cypress Street Viaduct. These tests consisted of forced vibration studies to determine the dynamic characteristics of the structure and static lateral load tests in the transverse direction to determine force and deformation capacities. Tests were performed on a relatively unmodified segment of the structure, as well as on the same segment after it was retrofit. The three bents used in the tests are shown in Fig. 26. The results of these tests have been reported by Moehle and Mahin (1990).

Tests on the original structure indicate that the upper column pedestal and the adjacent lower joint developed cracking characteristic of the damage leading to the failure of the structure (Fig. 22) at a displacement of about 3/4-in. at the upper deck level. The total lateral load of 1400 kips applied to the three bents corresponds to an equivalent base shear coefficient of about 0.32 assuming a 2:1 distribution of loads to the upper and lower deck levels. A simple elastic time history analysis of the test structure has shown that it would have required a base shear coefficient of at least 61% to have remained elastic for a ground motion recorded nearby. The working stress base shear coefficient required in the original design was 0.06.

Three different types of retrofits for the lower joints were investigated. In addition, upper and lower bent caps (girders) were externally post-tensioned to help compensate for inadequate bar development lengths, and to provide increased strength and confinement in the joints. Most of the columns were reinforced in shear by adding exterior post-tensioning.

The retrofits were able to maintain the vertical load integrity of the test structure. However, significant damage was observed. In particular, the critical failure zone moved away from the lower joint to the upper joints, which began to fail and disintegrate during the tests. Shear cracks in the upper joints began at displacements at the upper deck level less than 1-in. In addition,the non-retrofit columns began to fail in shear at an upper deck level displacement of about 1.5-in. and an equivalent base shear coefficient of about 0.61.

After strengthening the shear damaged columns, the structure was again cycled and lateral displacements of 9.8-in. were achieved at its top along with equivalent base shear coefficients of 0.91. These values are about 13 and 3 times greater, respectively, than the values that could have been developed by the original structure.

One of the bents had heavy steel wide flange sections post-tensioned vertically along the face of the columns like splints. During the tests composite action could not be maintained and the relatively long lengths of unbounded post-tensioning steel used resulted in large cracks and spalling in the lower joints and in gaps between the steel and the concrete (loss of confinement) over large portion of the columns and joints.

Another bent had inclined rock anchors grouted in the lower joints. This bonded reinforcement proved effective in resisting perpendicularly oriented shear cracks. However, towards the end of the tests spalling and flexural cracking and joint shear cracking had begun. The continued integrity of this retrofit scheme under these conditions was not fully addressed.

The third retrofit consisted of a heavy steel collar placed around the pedestal and the base of the upper level column. This collar was post-tensioned to the sides of the lower joint. During the test the collar was observed to rotate about the lower joint and, upon its removal following the tests, fully developed sets of inclined cracks were observed in the joint along with much pulverized concrete.

These tests indicate the effectiveness of retrofitting existing bridge structures. However, they point out the need to consider a structure as a system and not to simply fix the damaged or overstressed portions. The limitations of the tests must also be recognized. Loading of the test structure was halted when the test site had to be vacated. However, it was not believed safe to continue testing at that time. The deformation history imposed may not be conservative for the most severe ground motions expected in the Bay Area. In addition, no attempt was made to retrofit or test the structure in the longitudinal direction and this mode of behavior must be carefully considered.

LOSS OF FUNCTIONALITY

An important observation regarding the damage to structures has been the public's reaction to them. Most structures performed generally in conformance with the design professions expectations. New structures and most engineered structures did not collapse, and life safety was protected, in regions of severe shaking. However, these motions were not the most severe or damaging that might be expected in the Bay Area. Moreover, the public reaction to these damages was generally that they were excessive.

There was much nonstructural damage to partitions, ceilings, cladding and contents (Fig. 28) in structures with little or no structural damage. This damage not only contributed a significant life safety threat in many cases, it disrupted the functionality of the structures and added substantially to repair costs. One ten story building in Oakland reportedly had $6 million in water damages due to the breakage of a fire sprinkler line.

Structural damages were repaired quickly in many cases, but in other cases considerable delays were encountered as owners await financing or as technical difficulties are encountered. In many cases the resulting repairs were cosmetic or intended to simply restore the structure to its pre-quake condition.

Large institutional owners or ones with significant capital investments have increasingly developed heightened awareness of the need to control damage. In addition to consideration of life safety for a structure's occupants, they have

Fig. 20 Collapse of Cypress Street Viaduct

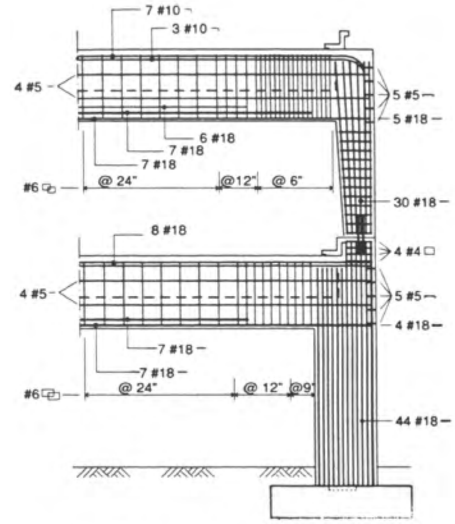

Fig. 21 Detail of Typical Bent [from Housner (1990)]

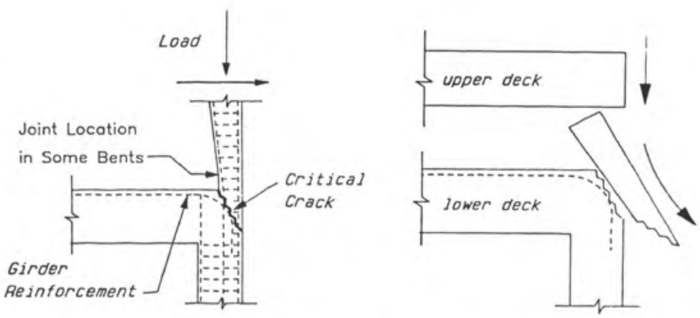

Fig. 22 Postulated Failure Mode

Fig. 23 Failure Mode Observed during Demolition

Fig. 24 Damage to Southern Freeway

Fig. 25 Damaged Knee Joint in New Freeway

become concerned over institutional life safety. For example, Stanford University has developed much more stringent criteria since the earthquake for retrofitting their seismically hazardous buildings. Having had to relocate several departments and instructional and research facilities following the earthquake, they now stipulate that buildings should be designed or retrofit to permit repairs to be made within a few days or weeks of the earthquake (depending on the occupancy of the building). Other institutions that cannot similarly just pick up and move have come to similar conclusions. Similarly, the Governor's proclamation also indicates that California state buildings should be built or retrofit so that they would remain operable following a major earthquake. This is a major departure relative to the design of buildings.

IMPLICATIONS FOR EARTHQUAKE RESISTANT DESIGN

Few new lessons have been developed as a result of the Loma Prieta earthquake. However, as the first major urban earthquake in the U.S. in nearly two decades, the earthquake stands as a compelling reminder of many important past lessons. We have again seen the vulnerability of several major classes of buildings designed in earlier eras when our knowledge of earthquake resistance was not fully developed. The damages to the numerous unreinforced concrete buildings, wood dwellings with inadequate foundation attachments, soft story structures (e.g., Marina District apartment houses) and non-ductile concrete frames (e.g., double deck viaducts) are reminders of well-known past lessons.

Similarly, the important effects of soil liquefaction and site amplification have been seen in many past earthquakes, such as the 1985 Mexico, 1967 Venezuela and 1964 Nigatta, Japan earthquakes. The Loma Prieta earthquake indicates that these soft soil sites should be carefully considered in design as they will be sensitive not only to close earthquakes, but also to any number of earthquakes generated on other relatively distant faults.

The special need to have a safe and dependable transportation system has been again demonstrated. It has been seen that certain structures may be more critical than others, and greater levels of conservatism should be considered in their design.

At the same time some past phenomena were not seen. However, the Loma Prieta earthquake appears to be an unusual earthquake in many respects and its motion in the epicentral region appears not to be as damaging as other typical west coast earthquakes have been. Thus, the good behavior of some types of structures during this earthquake should not lead to complacency in their retrofit given the seismicity of the Bay Area.

The damages provide a clear warning to the Bay Area and to other seismically active areas to increase preparedness activities. In particular, the substantial hazard posed by our older seismically vulnerable structures must be remedied. This can only be done through reliable evaluation and retrofit criteria developed and validated on the basis of laboratory research and quantitative investigations of the seismic performance of buildings during earthquakes.

The earthquake has shown a greater value being placed by the public, government and corporate entities on reducing economic as well as life hazards. Control of damage during earthquakes need not involve great expense, but additional attention to the selection of the structural system and to detailing of nonstructural components. Additional research is needed to provide a reliable design basis for such considerations.

Finally, it must be recalled that the timing and location of the Loma Prieta earthquake were fortuitous. The epicenter was in a sparsely populated region relatively far from urban centers. It also occurred at a time when many people were away from the most vulnerable structures. This set of favorable circumstances can not be expected to occur for all future earthquakes.

ACKNOWLEDGMENTS

Many of the research observations made in this paper were made possible by the generous research support provided by the National Science Foundation and the California Department of Transportation. In addition, the author would like to thank his many colleagues, especially Jack Moehle, Hasan Astaneh and Roy Stephen, for sharing their observations with him. The findings and conclusions of this paper, however, must be blamed solely on the author alone.

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Fig. 26 Cypress Street Test Structure

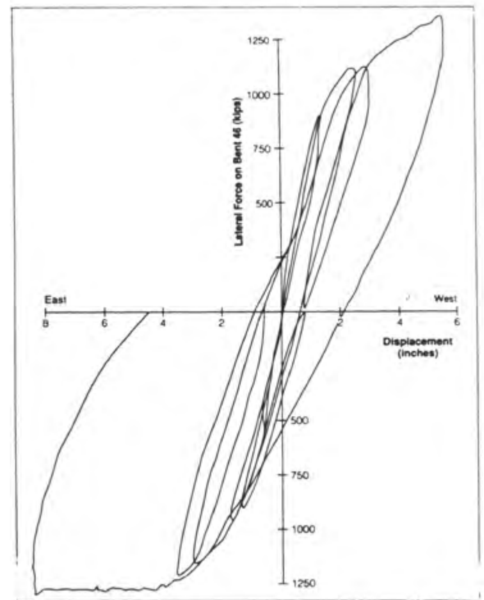

Fig. 27 Hysteretic Plots for Retrofit Viaduct

Fig. 28 Damage to Building Contents