Performance of bridges in the 1994 Northridge earthquake

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Abstract: This paper describes damage to bridges caused by the 1994 earthquake at Northridge, California. A description of the damage and the probable causes are presented for the seven bridges that suffered some form of collapse. The majority of damage was due to column shear or combined shear and flexure failures, but restrainer failure was the cause of collapse in one bridge. The lack of damage to other bridges and to foundations is discussed.

Key words: Northridge, earthquake, seismic, bridges, damage, performance.

Résumé : Cet article décrit les dommages causés à sept ponts lors du tremblement de terre de 1994 de Northridge et analyse les causes probables. La plupart des dommages est attribuable au cisaillement des poteaux ou à des défaillances causées par l'effet combiné du cisaillement et de la flexion. Toutefois, l'effondrement de l'un des ponts est attribuable à une défaillance des dispositifs de retenue. L'absence de dommages aux autres ponts et aux fondations fait également l'objet d'une discussion.

Mots clés : Northridge, tremblement de terre, sismique, ponts, dommages, performance.
[Traduit par la rédaction]

Introduction

The area around Los Angeles has long been known as a region of high seismicity, with the famous San Andreas fault lying a few tens of kilometres to the northeast. The January 17, 1994, Northridge earthquake occurred on a hidden fault northwest of downtown Los Angeles very near to the location of the 1971 San Fernando earthquake. This area has many seismic monitoring stations, some located in the immediate epicentral region, which recorded peak ground accelerations of over $1g$ (>10 m/s^2). See the accompanying paper by Finn et al. (1995) for a fuller discussion on the seismicity and recorded ground motions.

Previous earthquakes in California have caused considerable damage to bridge structures. Most notably, the 1971 San Fernando earthquake exposed a number of design deficiencies, including low design lateral force levels, inadequate confining reinforcement, lack of longitudinal restraint across expansion joints, and insufficient seat widths at joints and supports. This led to a number of changes in seismic design provisions, and to the introduction of an extensive bridge retrofitting program in California, of which the initial phase was the provision of restrainers across expansion joints. More recently, and especially after the 1989 Loma Prieta earthquake, the pace of retrofitting has increased and many structures have been retrofitted with steel jackets added to columns to enhance flexural ductility and shear resistance. At the time of the Northridge earthquake there were over 100 bridges in the Los Angeles basin that had been retrofitted, and the only reported damage to these was minor spalling of the superstructure at control joint locations (Priestley et al. 1994). A more extensive survey of some of these bridges is being undertaken to see if there is damage to the concrete beneath the steel jackets.

Figure 1 shows a location map of the six sites where collapse or damaged occurred, along with recordings of peak horizontal ground acceleration taken at the California Strong Motion Instrumentation Program (CSMIP) sites in the area. Initial estimates of the peak ground acceleration at the bridge sites where collapses occurred ranged from $0.80g$ at the State Route 118/Mission-Gothic interchange to $0.30g$ at the Interstate 10/Venice-La Cienega and Interstate 10/Fairfax-Washington interchanges (Priestley et al. 1994).

Most of the soils in the region are described as loose to dense silty sands or lightly consolidated cohesionless material. Depth to bedrock is generally unreported but is likely to be considerably in excess of 25 m. The exception to this may be the soils in the I-10/Venice-La Cienega and I-10/Fairfax-Washington region. It is reported (Priestley et al. 1994) that these structures traverse an old river bed. Damage to these bridges was extensive, as was damage to a nearby parking

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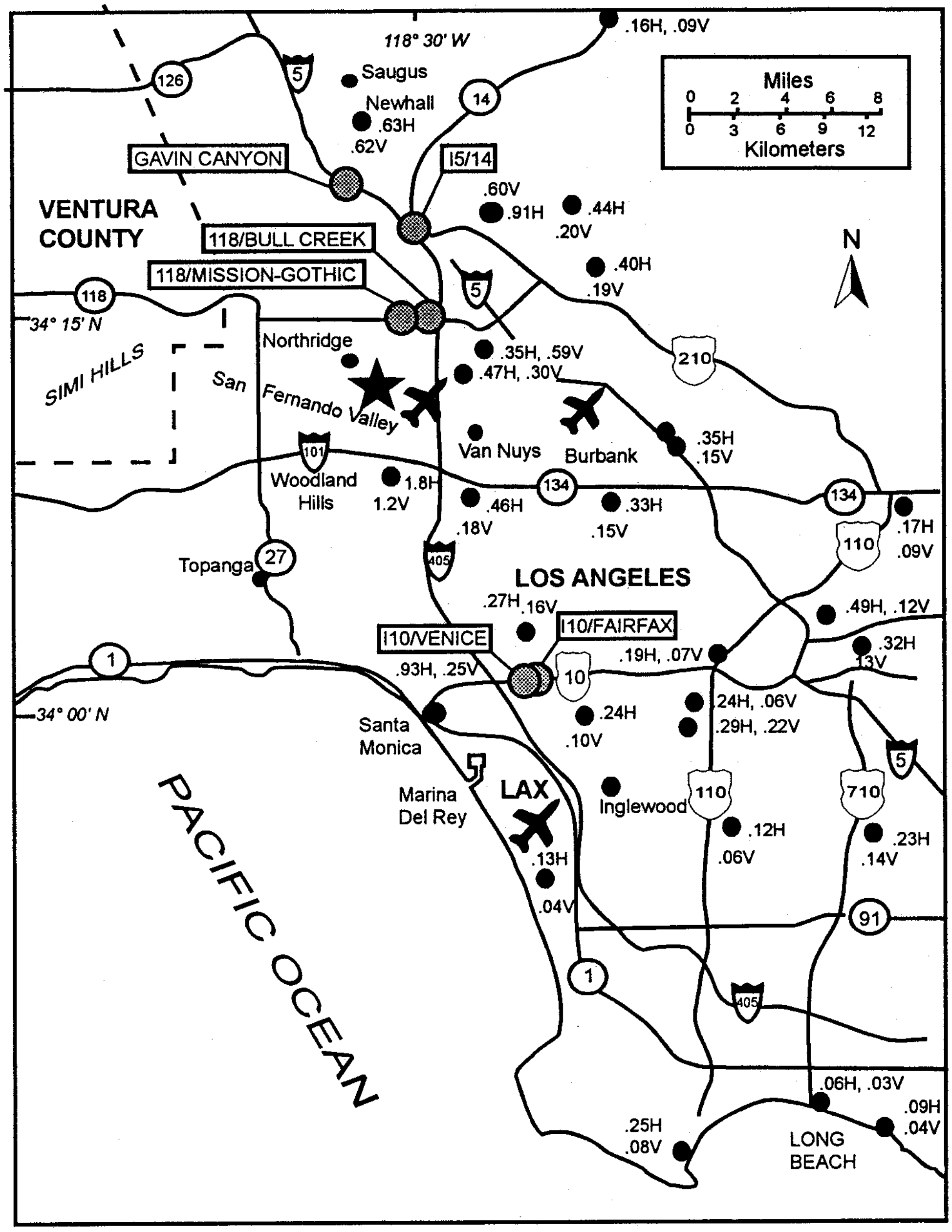
Fig. 1. Sites of bridge failures, Northridge earthquake (after CSMIP). Epicenter is denoted by the star; notation beside black dots represent peak ground acceleration in units of g in the horizontal (H) and vertical (V) directions.



garage. However, only a short distance away there were similar bridge structures that showed no damage, including lack of any spalling at the abutments. The evidence would seem to indicate that there was soil amplification at this location and that the structures were subjected to larger peak ground accelerations than the $0.30g$ which is estimated from the initial attenuation relations.

With the exception of the Gavin Canyon failure where loss of span is suspected, all the other collapses are attributed to column failures, although at the I-5/14 interchange there was evidence of several restrainer failures, nearly allowing unseating of decks at expansion joints. A brief description of the structures and column details is given for six structures that suffered collapses attributed to column failures. This is followed by a description of the loss of span failure at Gavin Canyon, and by comments on foundation retrofits.

Damage and failure of columns

Six bridge structures that suffered column failure are discussed below. Details of the columns and a discussion of the types of failure are presented. A seventh bridge which suffered damage but did not collapse is briefly mentioned.

I-5/14 South connector

Figure 2 gives an overall view of the failure of the I-5/14 South connector. The structure was damaged, while under construction, in the 1971 San Fernando earthquake, and was

Fig. 2. Overall view of the I-5/14 South connector failure.

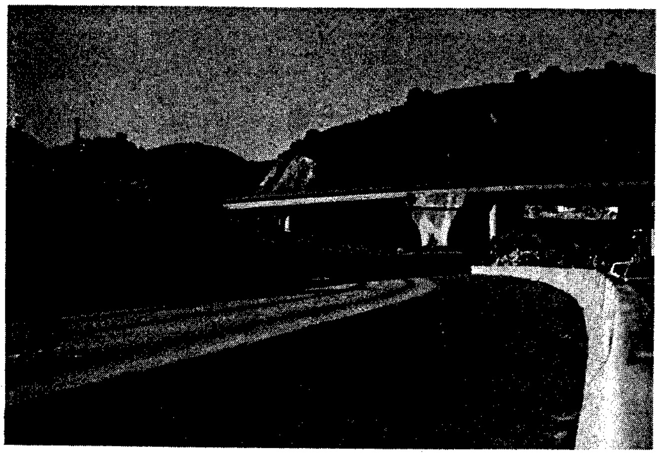
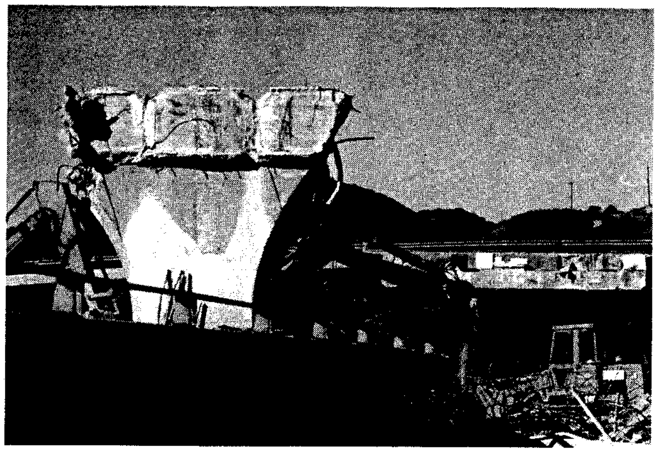


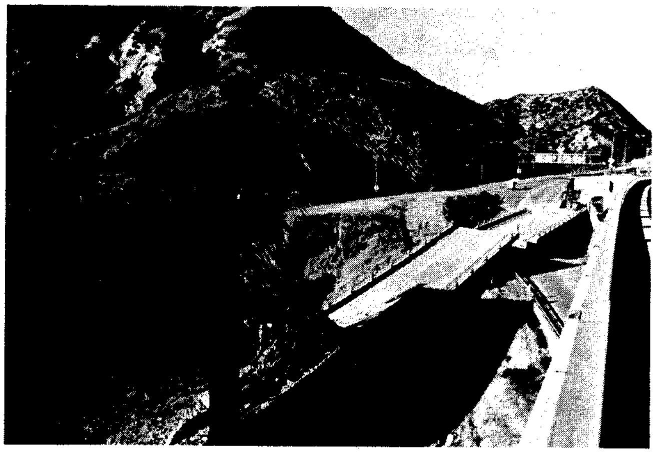
Fig. 3. Flared column of the I-5/14 South connector.



completed in 1974 without any substantial modifications being made to the original design. The structure consists of cast-in-place post-tensioned box birders supported by either 3.6 × 1.2 m or 3.6 × 1.8 m octagonal columns. These octagonal columns flared to a width of 8 m at the underside of the superstructure.

The significant movements of the superstructure during the earthquake resulted in a shear failure at the short column closest to the abutment, failure of the reinforced concrete wing wall and shear keys at the abutment with subsequent loss of support of the superstructure. This was followed by failure of the superstructure at the diaphragm over the continuous support, leaving the flared column standing (see Fig. 3). The column that failed contained 20 #18 (57 mm dia.) longitudinal bars and #5 (16 mm dia.) ties at a spacing of 305 mm in the 3.6 × 1.2 m section. This amount of confinement and shear reinforcement is inadequate to ensure ductile behavior and to prevent a shear failure.

Fig. 4. Collapse of the I-5/14 North connector.



I-5/14 North connector

The I-5/14 North connector overcrossing structure consists of ten spans and a 2.4 m deep hollow box superstructure, some spans of which were post-tensioned. The structure was designed in 1968 and was completed after the 1971 San Fernando earthquake. Figure 4 shows the collapse that occurred in the first two spans as the column closest to the abutment failed in shear (CALTRANS 1994; Priestley et al. 1994). The single column supports consisted of 1.2 × 2.4 m octagonal columns. The column that failed contained 42 #18 (57 mm dia.) longitudinal bars with #4 (12.7 mm dia.) confining bars spaced at 305 mm as shown in Fig. 5. The clear heights of the columns (measured from the top of the footings to the underside of the superstructure) varied from 8 to 23 m, with the failed column having a clear height of 8 m. The shorter length of the failed column and the lack of horizontal restraint at the adjacent abutment most likely resulted in larger shears being attracted by this column. In addition, the low amounts of confinement and shear reinforcement in this short column contributed to the shear failure.

State Route 118/Mission-Gothic

The Mission-Gothic undercrossing on State Route 118 was designed in 1973, with construction completed in 1976. The bridge comprises two structures separated by a longitudinal joint, the superstructure consisting of cast-in-place post-tensioned box girders. The twin column bents consist of 1.8 m octagonal columns with a 3.6 m long flare at their tops (see Fig. 6). The octagonal columns contain 45 #11 (35.8 mm dia.) bars confined with #5 (16 mm dia.) spiral reinforcement having a pitch of 89 mm. The bases of the columns were pinned during the post-tensioning opration and were later clamped by cast-in-place reinforced concrete collars to provide some fixity at their bases. Figures 7 and 8 show the failures that occurred in the columns in regions just below the flares. Calculations by Priestley et al. (1994) suggest that the reinforcing details were sufficient to prevent brittle shear failure prior to flexural hinging, but that shear failure occurred after flexural plastic hinging had caused a reduction in the shear capacity of the concrete and (or) fractured the spiral reinforcement. The column flares forced the plastic hinges to develop at the bottom of the flares rather than at the deck level, resulting in higher shear forces when the columns formed a mechanism. This is contrary to the design assumption which had the flair concrete spalling off and the plastic hinge located at the top of the column. The reinforced concrete shear collar failed around one column (CALTRANS 1994), resulting in large column base movements as indicated by the ground heaving and large gaps

Fig. 5. Details of failed column of the I-5/14 North connector.

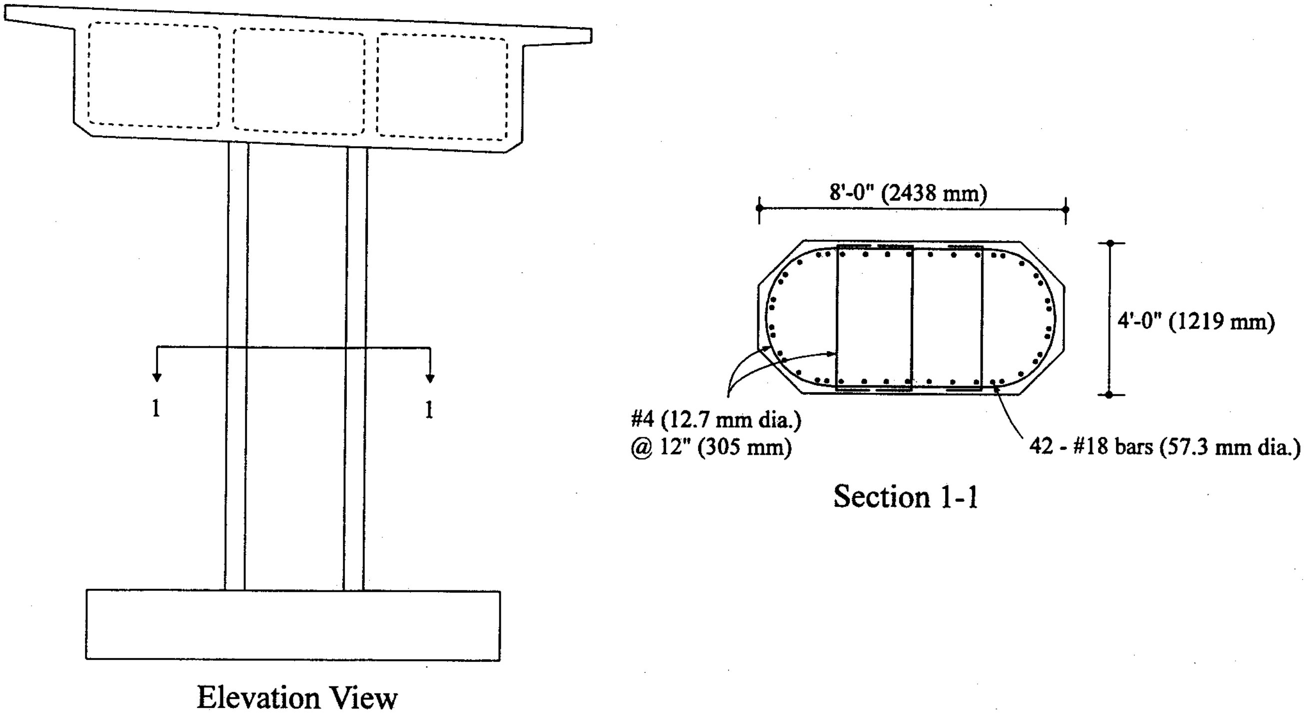


Fig. 6. Details of columns in 118/Mission-Gothic undercrossing.

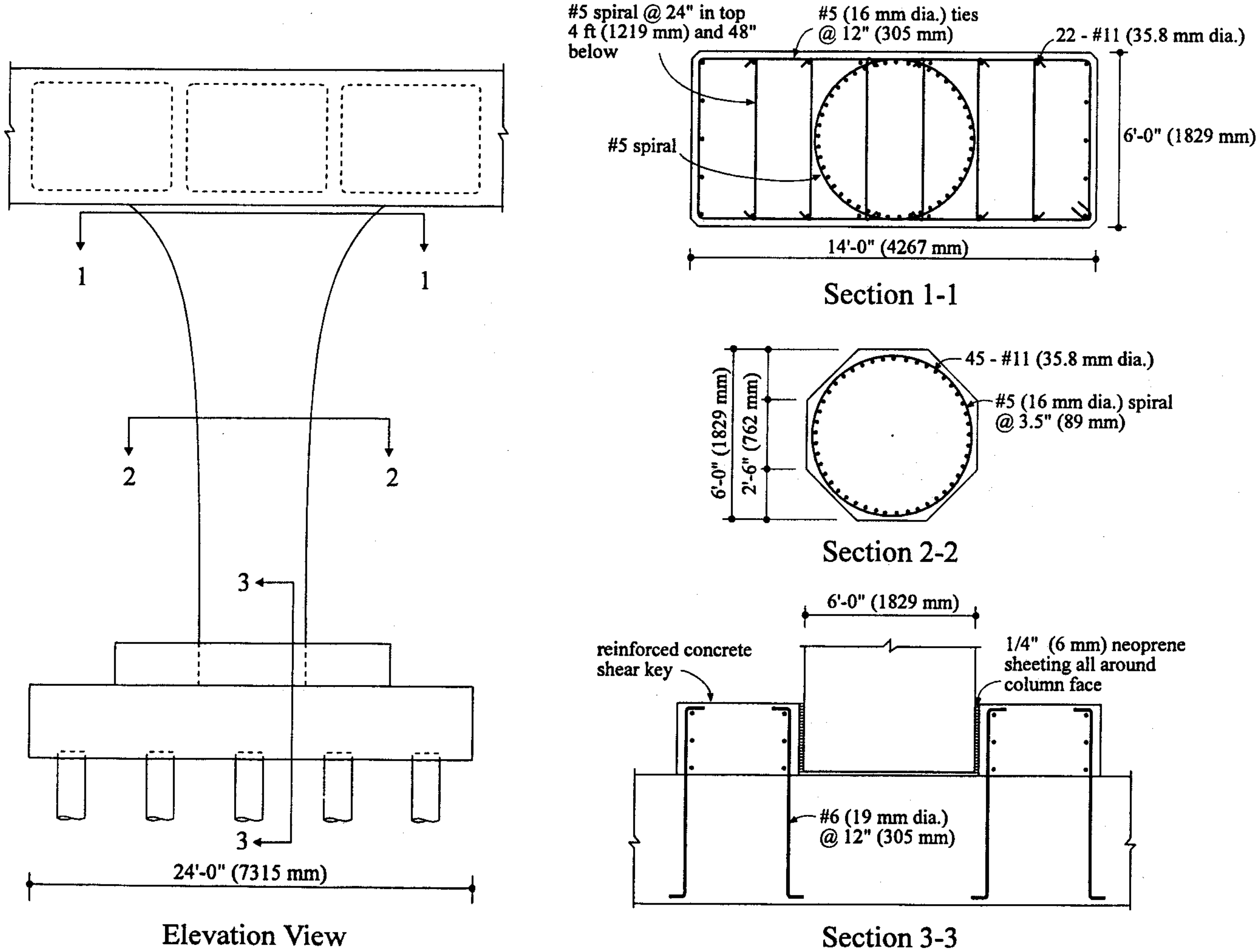


Fig. 7. Shear failures in columns of 118/Mission-Gothic undercrossing.

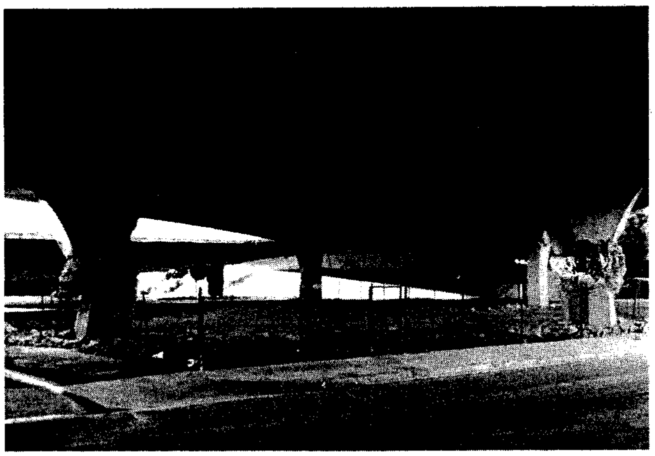


Fig. 8. Shear failure of column of 118/Mission-Gothic undercrossing.

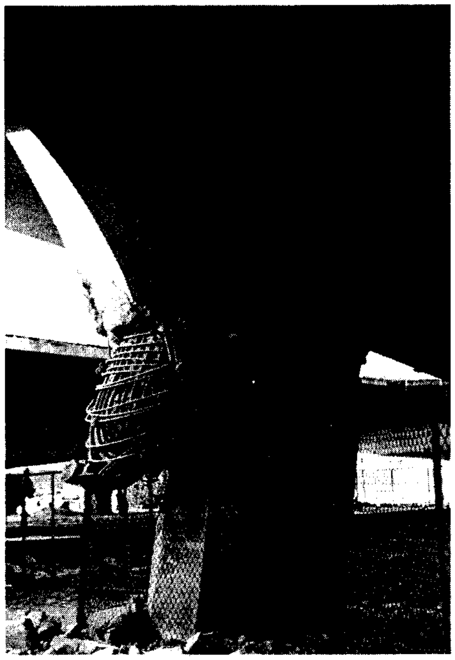
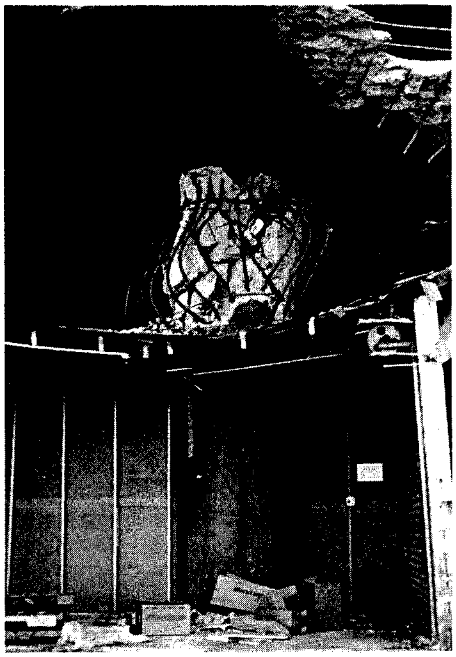


Fig. 9. Evidence of significant movement at base of column of 118/Mission-Gothic undercrossing.

Fig. 10. Failure of column of I-10/Venice-La Cienega undercrossing.



between the column and the soil (see Fig. 9). The failure of the shear collar resulted in a smaller shear being attracted to this column and may have contributed to a significant torsional eccentricity in the bridge structure.

Interstate 10/Venice-La Cienega

This structure on Interstate 10 (Santa Monica Freeway) which crosses La Cienega and Venice boulevards suffered severe damage to the columns. Built in 1964, it is supported by prismatic 1.2 m diameter circular columns reinforced with 12 to 48 #11 (35.8 mm dia.) bars and confined by #4 (12.7 mm dia.) hoops with 460 mm lap splices at 305 mm spacing. Some columns suffered severe damage at their tops (see Fig. 10) while others were damaged near their bases. The inadequate amounts of confinement and shear reinforcement provided by the widely spaced hoops, combined with failure of the lap splices of these hoops, led to the flexure–shear failure of the columns.

Interstate 10/Fairfax-Washington

This bridge on Interstate 10, crossing Fairfax and Washington streets, was also constructed in 1964. The bridge has skew supports with columns supported on pile footings. Some columns are fixed at their bases while others are pinned (see Fig. 11). The prismatic 1.2 m diameter circular columns contained 12 to 62 #11 (35.8 mm dia.) bars with #4 (12.7 mm dia.) lap spliced hoops at 305 mm spacing. The failure of some columns near their tops was probably due to the inadequate amount of poorly anchored hoop reinforcement which became ineffective once the concrete cover spalled off and led to the buckling of the longitudinal bars. The result was a series of failures (see Figs. 12 and 13) due to a combination

Fig. 11. Details of columns of I-10/Fairfax-Washington undercrossing.

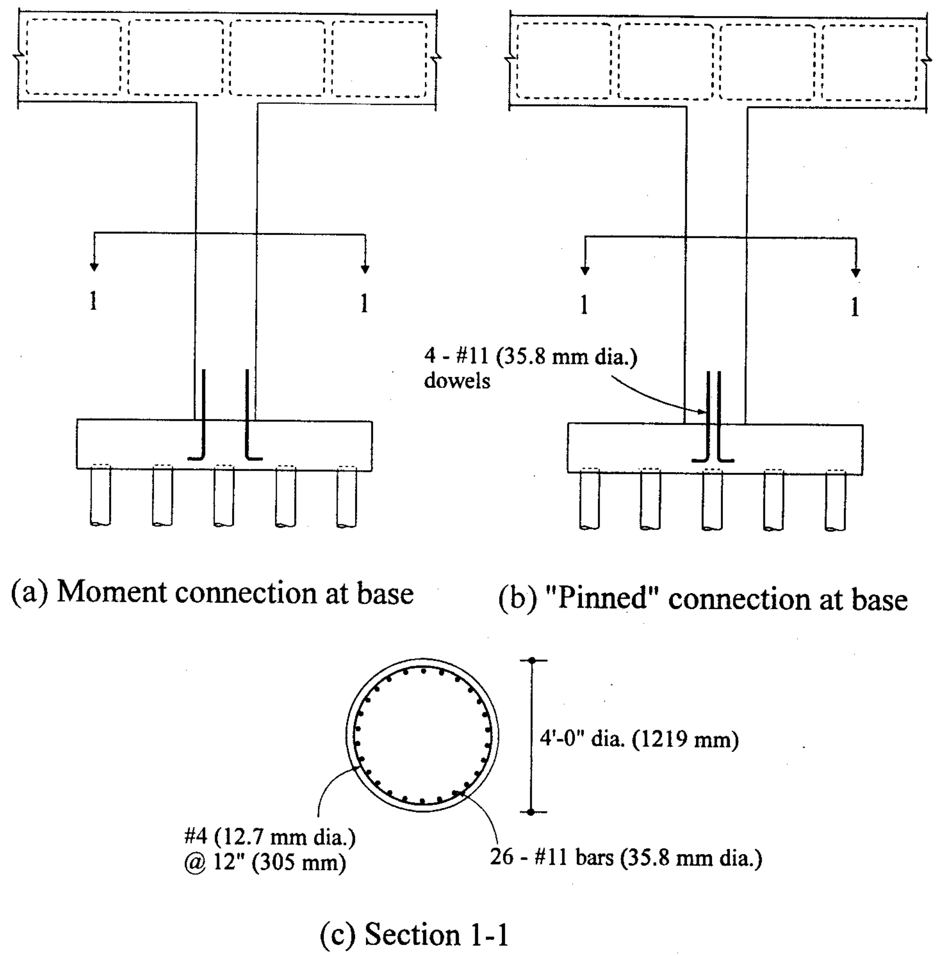


Fig. 12. Shear failures of columns of I-10/Fairfax-Washington undercrossing.

of shear, flexure, and compression.

State Route 118/Bull Creek

This three-span bridge on State Route 118, crossing Bull Creek, was built in 1976 and has skew supports. The multi-column bents consist of prismatic 1.2 m octagonal columns supported on pile footings. The vertical column bars are continuous from the bottom of the footing into the bent cap. The transverse reinforcement consists of #5 (16 mm dia.) spirals at 305 mm with a closer spacing of 75 mm over lengths of 1.2 m just above the footing and just below the connection with the superstructure. Figures 14 and 15 show the failures that occurred at the bottoms of some columns and at the tops of other columns. A typical failure at the top of a column occurred at the location where the spacing of the transverse reinforcement changed from 75 to 305 mm. The failures near the bottom of the columns were just above a culvert wall which provided some lateral restraint in the region where the spacing of the transverse reinforcement was 305 mm. These failures were flexure–shear failures with loss of confinement and buckling of the longitudinal bars.

Ruffner Avenue

The Ruffner Avenue overcrossing is on State Route 118 and is supported by two column bents. These octagonal columns contained #14 (43 mm dia.) longitudinal bars confined with #5 (16 mm dia.) spirals having a pitch of 90 mm. Only minor damage occurred to the columns with spalling of the shell concrete below the column flare (see Fig. 16). Although this column had a smaller amount of confinement reinforcement

Fig. 13. Shear failure of column of I-10/Fairfax-Washington undercrossing.

Fig. 14. Failure of columns of 118/Bull Creek Bridge.

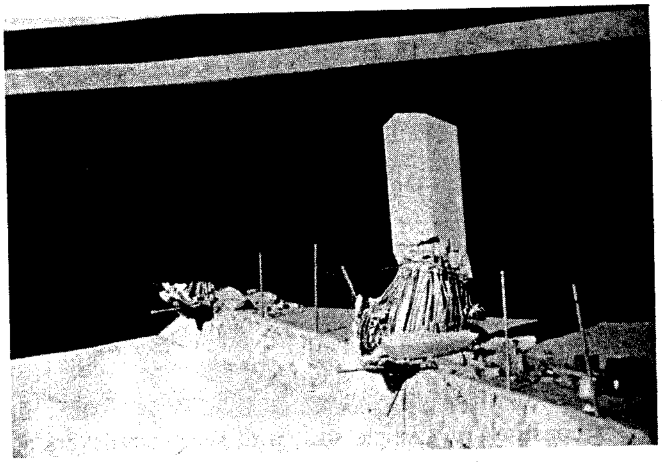


Fig. 15. Failure of columns of 118/Bull Creek Bridge.

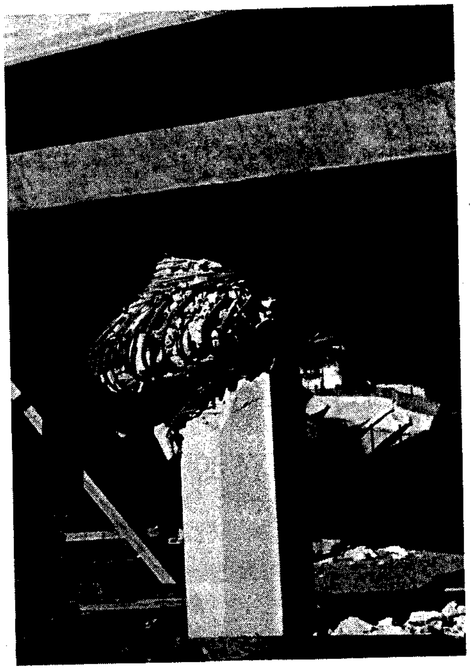
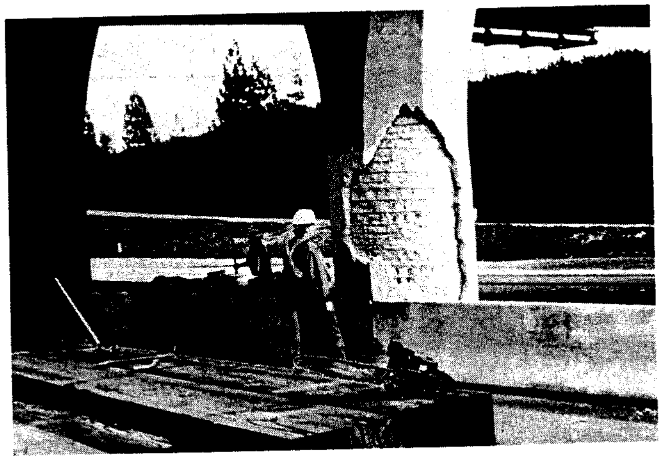


Fig. 16. Spalling of concrete shell in column of 118/Ruffner Avenue overcrossing.



than that required by current design procedures, the core concrete remained intact and only minor repair would be necessary for the bridge (CALTRANS 1994).

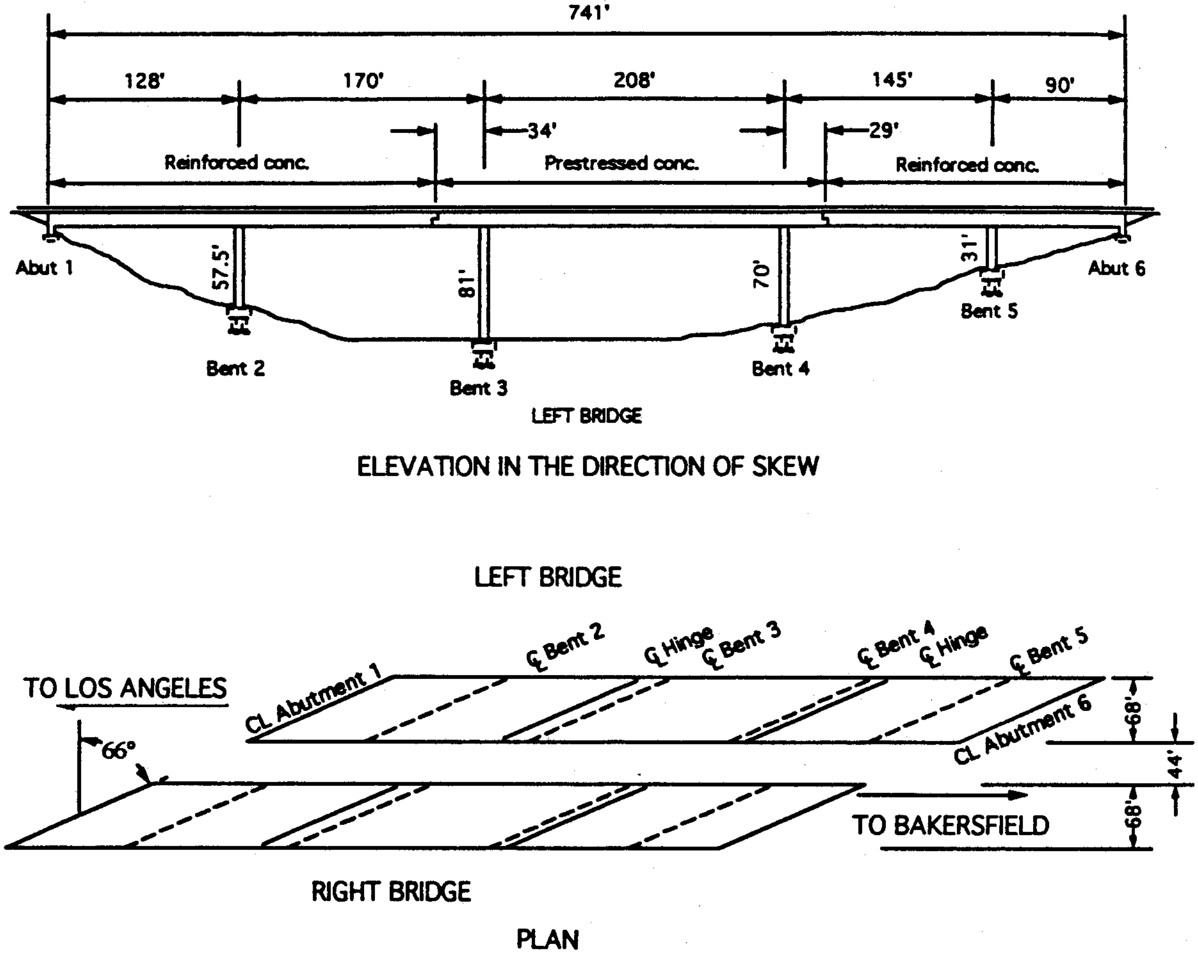
Current design and detailing requirements for columns

In the survey of damage carried out by CALTRANS (1994), it was concluded that there was no need to consider major revisions to the CALTRANS (1990) specifications for new bridges. However, the report did identify the effects of vertical acceleration and the effects of column flares as requiring detailed evaluation, and these may result in changes to the specifications. Preliminary analyses of column failures carried out by Priestley et al. (1994) cited shear failures and lack of flexural ductility, accentuated by buckling of the longitudinal bars, as possible causes of the failures rather than the influence of vertical accelerations. The columns that failed had been designed by earlier codes and consequently lacked the improved details in the current standard (CALTRANS 1990) for confinement, had inadequate shear strength to develop flexural hinging, and lacked the improved anchorage details for reinforcement. All of these requirements were developed following the 1971 San Fernando earthquake (CALTRANS 1988; AASHTO 1983), and bridge columns designed using these specifications suffered at most only minor damage.

Performance of restrainer retrofits

Very shortly after the San Fernando earthquake of 1971, during which numerous bridges collapsed due to unseating at expansion joints, CALTRANS initiated Phase I of its seismic rehabilitation program (Housner 1990). This phase, completed in 1987, entailed the provision of seismic restrainers at vulnerable hinges and expansion joints of nearly 1300 bridges, at a total cost of $55 million (Roberts 1990). Restrainers have proved effective in preventing unseating in numerous recent earthquakes (Roberts 1993).

Fig. 17. General layout of Gavin Canyon Bridge (after Priestley et al. 1994).



The value of restrainers was again evident in the Northridge earthquake, although there were some failures. The collapse of the Gavin Canyon bridge was attributable to unseating even though it had been retrofitted with restrainers, while a small number of minor problems caused by faulty restrainer design was also noted at other bridges. These are discussed below. Apart from these few cases, all of which related to very early restrainer designs, joint restrainers performed satisfactorily, and again proved to be an inexpensive but effective retrofit measure in preventing loss of span failures.

Gavin Canyon

This bridge, constructed in 1964, carried Interstate 5 across a canyon approximately 15 km north of the epicentre. The peak ground acceleration in this area, estimated from early attenuation relations, is thought to have been around 0.44*g* (Priestley et al. 1994), although there were higher recorded peak ground accelerations further to the north at Newhall. Initial estimates from CALTRANS were 0.6*g* (Yashinsky 1994). The bridge consisted of two nearly identical structures, carrying northbound and southbound traffic, oriented at a very high skew angle of 66°. Each structure consisted of a five-span box girder deck, continuous over flared twin-column bents and built in at the abutments (Fig. 17). Movement joints were located in the second and fourth spans, thus splitting the deck into three separate sections. The central section consisted of a single span with short cantilevers at each end, while the outer sections were two-span decks built into the abutment at one end, resting on the central section cantilevers at the other, and continuous over the intermediate bent. The two bents of the central span had column heights in excess of 20 m, while those adjacent to the abutment were considerably shorter.

The deck joints were oriented in the direction of the skew, and supported on a seat width of only 200 mm. The joints were retrofitted with cable restrainers in 1974. Five restrainers were added to the joint in the second span; four at the other joint, which supported a shorter suspended span. Each restrainer consisted of seven 19 mm (3/4 in.) diameter cables installed in a single 150 mm (6 in.) diameter hole in the diaphragm, which was cored parallel to the longitudinal axis of the bridge. The cables passed through the joint, wrapped around a cable drum on the back side of the diaphragm of the cantilever span, and then went back through the joint a second time to be anchored against the back side of the diaphragm of the suspended span. The anchorage consisted of a threaded swaged fitting connected to a 50 × 254 × 254 mm (2 × 10 × 10 in.) bearing plate seated on a bolster attached to the skewed diaphragm. In addition to the cable restrainers, the original construction drawings show a 38 mm (1.5 in.) diameter bolt installed parallel to the bridge at the centre of each box cell, giving six bolts across each joint. The bolt was

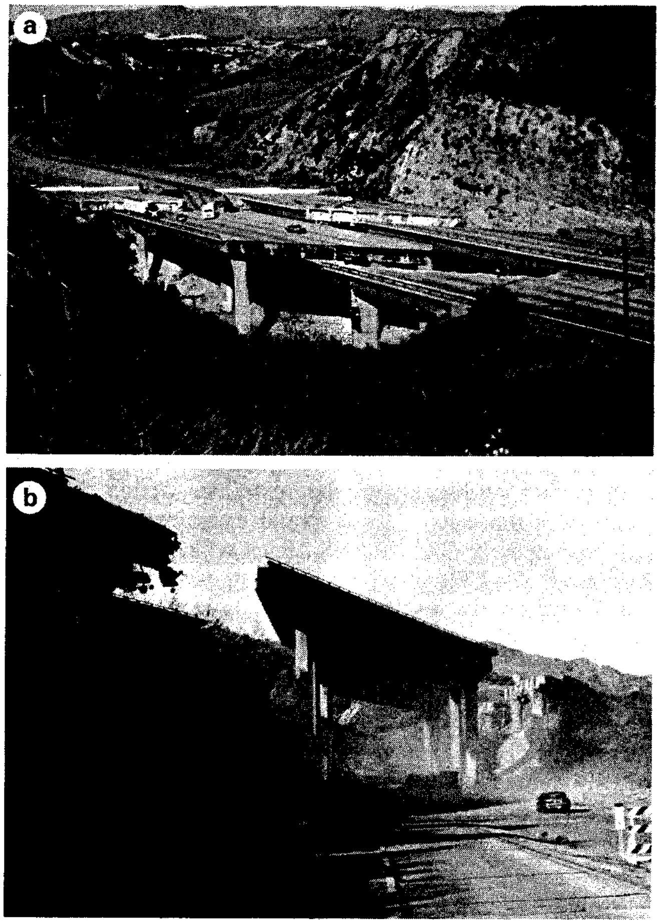


Fig. 18. Collapse of second and fourth spans of the Gavin Canyon Bridge: (*a*) overall view of damaged bridges; (*b*) view of surviving third span, dust cloud from demolition work.

cast into the diaphragm of the cantilever and then passed through a 51 mm (2 in.) diameter hole in the diaphragm of the suspended span and anchored using a 32 × 152 × 152 mm (1.25 × 6 × 6 in.) expanded polystyrene washer plus nut.

During the earthquake, all but one of the second and fourth spans of both bridges collapsed owing to unseating at the expansion joints (Fig. 18). The large skew and the unsymmetric lateral load support system most likely caused large torsional movements of the deck during the earthquake, leading to relative lateral motion between the deck sections at the skew joints, and hence loss of seating. The longitudinal restrainers would have provided little resistance to this motion. The restrainers appeared to have been pulled sideways, causing ripping of the longitudinal holes, and then to have failed by punching through the end diaphragms of the suspended deck sections as they fell. Intact restrainers with bearing plates still in place could be seen hanging from the undamaged central cantilever section of deck (Figs. 19*a* and 19*b*). It would appear that the bolt restrainers pulled out of the cantilever end, as is evidenced by what looks to be freshly exposed concrete at the bolt hole locations situated above the cable restrainer holes.

Skew decks are particularly prone to unseating problems,

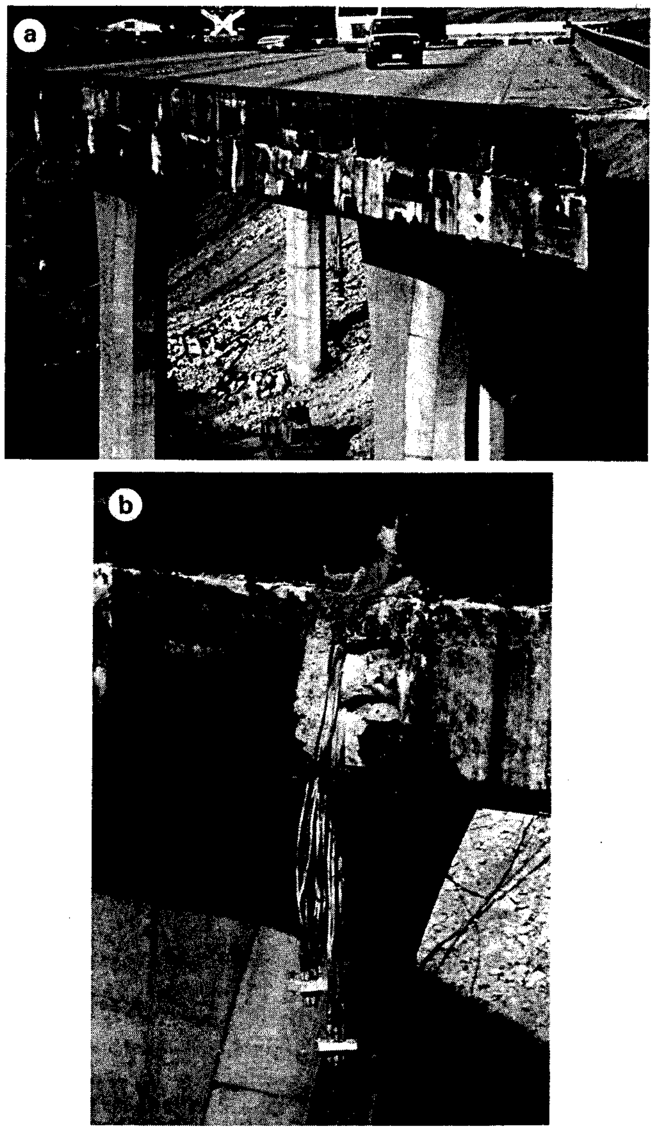


Fig. 19. Views of expansion joint face of the Gavin Canyon left bridge: (*a*) overall view showing five restrainer locations; (*b*) restrainers with failed end anchorages, note evidence of sideways pull by spalling of concrete on left side of restrainer holes.

since opening of the joints can be caused by twisting and lateral movements as well as longitudinal movement. The very large skew of this bridge made the problem particularly acute. Clearly, it would be preferable to construct highly skewed bridges without expansion joints along their length if at all possible. Where this is not possible, extra-large seat widths are needed.

Seismic design guidelines typically include a requirement for a minimum bearing support width. The current CALTRANS (1990) guidelines recommend the following equation for calculating support width:

$$[1] \quad N = (305 + 2.5L + 10H)\left(1 + \frac{S^2}{8000}\right) \geq 760 \text{ mm}$$

where N is the minimum required support width in milli-

Fig. 20. I-5/14 North connector overcrossing: badly damaged expansion joint with one span bearing on approximately 50 mm of concrete.

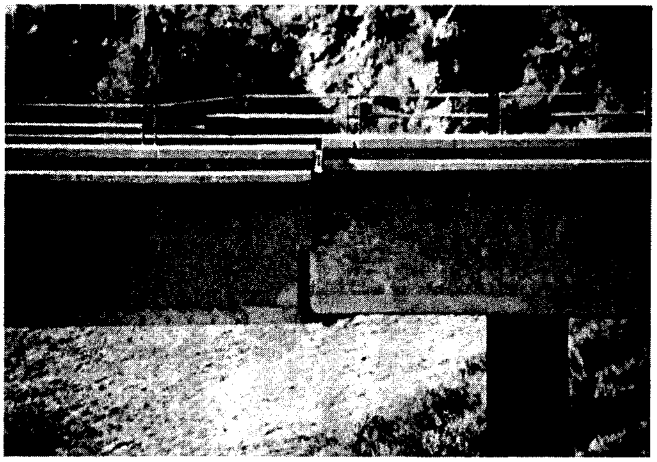
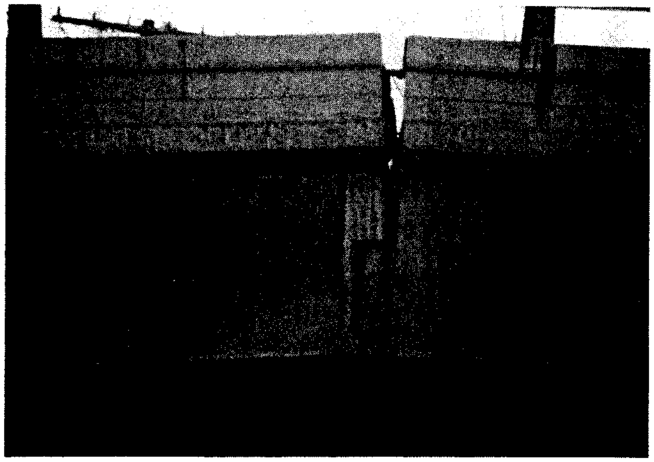


Fig. 21. I-10/Fairfax-Washington: spans tied together by seismic restrainers possibly preventing further collapse.



metres, L is the length in metres to the next adjacent expansion joint or abutment, H is the average height in metres of columns or piers supporting the bridge deck, and S is the skew of the abutment in degrees. The effect of skew and the lower limit of 760 mm are modifications made by CALTRANS from the AASHTO (1991) requirements. For columns or piers, the values calculated using the above equation for the spans on each side of the hinge must be added. The bearing support widths provided must then be at least equal to the minimum value obtained using the above equation, or the displacements calculated from an elastic seismic analysis, whichever is greater.

Applying [1] to the Gavin Canyon bridge gives a minimum required seat width of just over 2.0 m, which is ten times the value actually provided. (Without the skew the required seat width would be 1.3 m.) Except over bents, it would not be easy to provide for such a long seat width and this may be one of the reasons that new bridges do not have as many inspan movement joints.

With regard to restrainers, it is clear that their orientation should be perpendicular to the joint if they are to provide effective resistance to unseating, especially if lateral shear keys are not provided. It is possible that the wording of the various design guidelines has caused some confusion over this point. For example, CALTRANS' Bridge Design Specifications (CALTRANS 1990) specifies that *longitudinal* restraints must be provided, and AASHTO's Standard Specifications for Seismic Design of Highway Bridges (AASHTO 1991) requires that a *longitudinal* linkage force be considered in the design process. It appears that this wording has resulted in some restrainers being designed and installed parallel to the roadway of skewed bridges rather than perpendicular to the joint. In the Gavin Canyon bridge there was not sufficient lateral room inside the cells of the box girder to core holes perpendicular to the end diaphragms, and so to retrofit the structure with perpendicular restrainers would have been very difficult. Some more recent joint retrofits by CALTRANS have used heavy pipe sections placed in holes cored through the diaphragms to provide shear restraint or to act as seat length extenders, and these may have been of help for this bridge.

A further problem with the restrainer design for this bridge was the use of seven-strand restrainer assemblies. This was an early CALTRANS design which has since been abandoned; CALTRANS now prefers to use smaller assemblies which impose less severe punching loads on the bridge diaphragms (Yashinsky 1994).

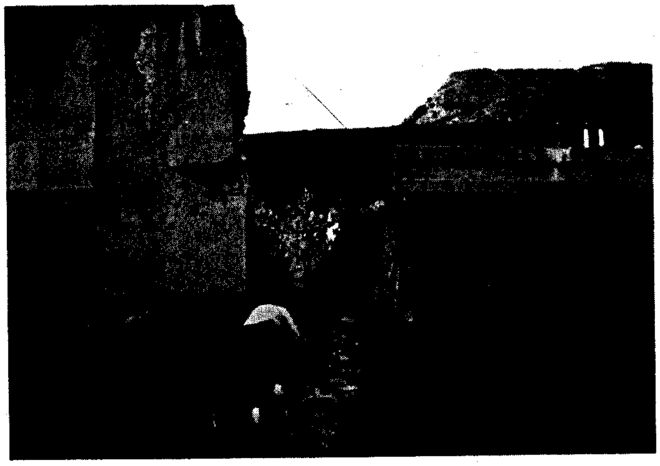
Other bridges

Several restrainer failures were noted in bridges at the I-5/14 interchange. The two major collapses are thought to have been initially caused by column failures, followed by restrainer failure and unseating at the adjacent supports. However, in these cases the restrainers could not be expected to prevent unseating. In other cases at joints close to the collapsed sections, for example, at the north connector overcrossing where the first pier failed and two spans collapsed, the next hinge very nearly unseated and ended up bearing on no more than 50 mm of concrete (Fig. 20). This could be considered a restrainer failure as the movements were excessive; on the other hand, if the restrainers were not there, it is likely the span would have fallen. Other restrainer failures were observed in other parts of this interchange, but did not lead to collapse. Some of these failures occurred because the restrainres were grouted inside their cored holes; this prevents overall elongation of the restrainer cables and leads to fracture. In other cases, restrainers pulled out of their diaphragm anchorage (CALTRANS 1994). As with the Gavin Canyon bridge, these were early restrainer designs, dating from 1974, which would not comply with current guidelines. Nevertheless, in spite of their deficiencies, restrainers in this complex interchange sufficiently enhanced the behaviour during the strongest shaking to prevent additional collapses.

In numerous other instances, restrainers worked very well, preventing unseating at expansion joints, and in some cases even keeping various spans tied together to prevent collapse when part of the span suddenly dropped due to severe column damage (see, for example, Fig. 21).

Damage to abutment shear keys was also visible at the I-5/14 interchange. Shear keys at seat abutments are often used in California to provide lateral resistance to the seismic forces. A large number of internal and external shear key designs exist, but most tend to be of a brittle rather than duc-

Fig. 22. I-5/14 South connector: damaged exterior shear key wing-wall at abutment.



tile nature. Therefore, the CALTRANS bridge design specification (CALTRANS 1990) requires that seismic forces in shear keys and other fixed restraining devices be designed for 125% of the calculated forces, and that these forces should preferably be determined by dynamic analysis. An example of a damaged wingwall designed to act as an external shear key is shown in Fig. 22. This damage, probably produced when the span collapsed owing to other structural deficiencies, illustrates well the brittle nature of typical shear key designs.

Performance of other bridges

There were several thousand bridges in the region affected by the earthquake. It is estimate that 1600 bridges were subjected to a peak ground acceleration of more than $0.25g$. Of about 40 targeted for detailed inspection by CALTRANS immediately after the earthquake, 9 suffered major damage or collapse, 2 had moderate damage, 17 had minor damage, and the remainder had no damage (CALTRANS 1994). Among those with major damage, almost all had been retrofitted with span restrainers. Some of these were early retrofits (after the 1971 San Fernando earthquake) and were not up to current CALTRANS standards. Several were scheduled for further retrofit, especially to substructures.

There were 120 bridges in the region that had column and footing retrofits. Of these, it is estimated that 20 were subjected to a peak ground acceleration greater than $0.5g$, and 60 to a peak ground acceleration greater than $0.25g$. Some 60 of these structures have been inspected and little or no damage was found.

The State of California has about 3200 bridges. Of these, there are roughly 1000 bridges that CALTRANS have identified as critical structures and retrofit to these bridges should be completed by the end of 1994. There are another 1500 bridges classified as less critical but still requiring retrofit. Local governments have some 3000 bridges with 1000 slated for retrofit. The cost of retrofitting these structures is in the \$3 billion to \$4 billion range.

It is important to note that while the California experience is by far the best source of information on the seismic behaviour of actual bridges, these bridges are quite different from most Canadian bridges. The California freeway bridges are virtually all of the same construction type. They consist of heavy cast-in-place concrete box superstructures, with relatively long spans, often on a curved and (or) skewed alignment. There are many inspan expansion joints, more in the older structures than in recent construction. The bridge seats at expansion joints were typically less than 150 mm in the older bridges, and have been increased to much larger values in the 1980s and later. Restrainer retrofits have been made to these bridges to preclude spans falling from the narrow seats. The substructures usually consist of single- or multiple-column bents on spread or pile footings, and due to the multiplicity of traffic ways in any particular interchange, there are often outrigger bents, with external knee joints subjected to bending, shear, and torsion.

In Canada, outrigger bents, a source of many problems, are much less common. Typical two-column bents usually have cantilever ends, permitting far better anchorage of flexural steel in the cap beam, although in many old designs the bottom steel may be discontinuous at the columns, and may have only limited anchorage in the column. Anchorage or lap problems are also quite often found at column bases.

Superstructures vary, but include many steel stringer and precast prestressed concrete systems, on a variety of pier types. Spans are probably smaller for overpasses, but spans and bridge types vary more because a greater proportion of the bridges are for terrain such as river crossings, rather than for highway interchanges and overpasses. There is a greater proportion of steel bridges, and superstructures are generally lighter. The bridges generally have fewer lanes, hence the difference between lateral wind loads and seismic loads, while remaining large, is not as dramatic as for California's wide freeway bridges. However, for large bridges in the Vancouver and Montreal areas, regions where the authors have some experience, most have required retrofit. Many of the details that have proved unsuitable in the bridges in California exist in older Canadian bridges.

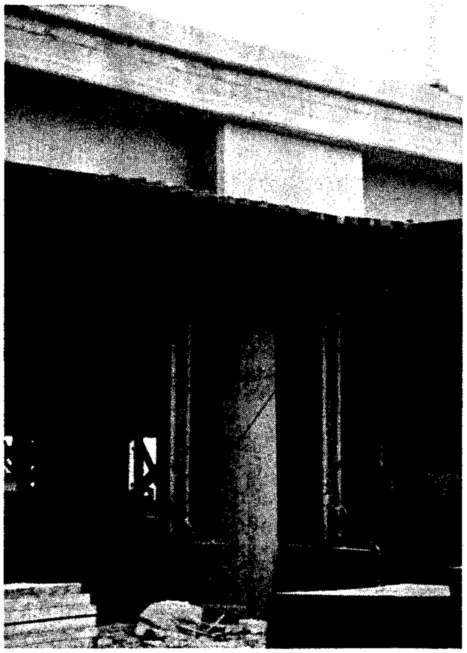
Foundations

Recent seismic retrofit programs by CALTRANS have included significant expenditures on foundation retrofits. The retrofits have been carried out in conjunction with retrofits to columns and piers, and have been required for several reasons related to the increased lateral demands on the system.

Under many design circumstances involving soft soil, CALTRANS has elected to discount friction on piles to resist uplift. Because of this, the foundations are very sensitive to even small increases in lateral load. When uplift is found, this necessitates widening the footings and adding piles at the perimeter of the widened footing. To carry out this work, expensive excavations have to be made, often encroaching into the traffic lanes, especially where the work is required on highway overpass structures where there is almost always a pier in the highway median.

In addition to widening, the increased lateral loads often result in the need to improve the shear and moment capacities of the footing, and to provide some capacity for negative moment in footings that were originally constructed with only a bottom mat of steel. This is done by dowelling into the existing footing, adding a new layer of concrete, and adding top bars in the new concrete. This technique also

Fig. 23. I-10/Fairfax-Washington undercrossing, view of repaired deck and supporting formwork.



assists in improving development length in the vertical column steel, which typically has starter bars protruding up from the original footing, by providing confinement around the lap area.

A third reason for requiring footing retrofit has been the preference to ensure that hinging and damage occur where it can be detected visually. Thus design standards for new structures, using capacity design concepts, have tried to ensure that the foundations are stronger than the columns.

It is apparent that footing retrofits are extremely costly. They involve excavation and much disruption in areas in or adjacent to traveled highway lanes. It is therefore interesting to examine the Northridge experience to determine whether foundation retrofits were effective or were necessary.

Soil conditions at most of the bridge sites in the affected area are generally similar, consisting of loose sands near the surface, underlain by dense sands and then sandstone. There were no soft soil sites identified at locations where bridges were damaged (CALTRANS 1994).

Of the structures that collapsed, foundation failure or footing failure was not identified as a cause of any of these structures. In several cases, footings were excavated and inspected. The I-10/Venice-La Cienega undercrossing, which suffered extensive damage, had only a bottom mat of steel in the pile footings. Six of the footings were excavated, exposing piles, footings, and columns. There was no sign of damage to any of these (CALTRANS 1994). The I-10/Fairfax-Washington undercrossing also had extensive damage. Two of the footings were excavated and no damage was noted (CALTRANS 1994).

Although footing damage is difficult to detect, it is significant that none was observed in those locations where CALTRANS anticipated the greatest likelihood of finding damage.

We would conclude that due to the high cost of footing retrofits, it will be very important to develop improved understanding of the older style footings in order to avoid unnecessary retrofits.

Replacement of the collapsed Santa Monica Freeway bridges

The Santa Monica Freeway reopened to traffic on April 12, 1994, 84 days after the Northridge earthquake. Crews worked around the clock and in all weather conditions to make reopening of this freeway possible two and a half months ahead of the June 24 deadline set to the contractor, thanks to an extraordinary $200 000 per day bonus incentive for each day cut from that deadline. It was reported by some, but denied by others, that the contractor would collect a $14.6 million bonus for this early completion. Considerable time was cut from the original deadline by using deck formwork capable of carrying traffic loads, as shown in Fig. 23, and by using rapid setting concrete for the deck slab.

Summary and conclusions

Essentially all the damage to bridges in the Northridge earthquake occurred to older structures that had not been constructed to modern design standards, or which had not been seismically upgraded in the most recent retrofit program. The column failures that did occur could have been predicted, and the retrofit methods currently being employed most likely would have prevented collapse.

Short columns, heavily reinforced to carry vertical or lateral loads, do not perform well if the lateral confinement and shear capacity are not adequate to resist the shears that are developed when plastic hinges form in the column. This deficiency is exacerbated if the plastic hinge rotations are high enough to degrade the shear normally thought to be carried by the concrete. Features that may be missed in the design that lead to high shear forces are such as the column flares, restraint from walls or the roadway, or other features that tend to reduce the height between plastic hinges; such features need to be carefully guarded against.

The danger from loss of span, either at movement joints or at piers, must be minimized as the consequences are so great. Current CALTRANS designs require a seemingly very large seat length and the issue is still a contentious one. Restrainers provide a reasonably economical retrofit and, if properly designed, can reduce the seat length requirements considerably. However, the current design methods are questionable, mainly because the problem is a difficult one to analyze and involves the response of the entire bridge, taking into account possible nonlinear action at the joints and noncoherent ground motion inputs at the supports.

Although the Northridge earthquake caused considerable damage and disruption to the freeway system of Los Angeles, it should be realized that there were many bridges left essentially undamaged and able to carry full traffic loads. Given another year or two on the retrofit program being carried out by CALTRANS, many of the problems would have been eliminated.

The current Canadian code covering the design of concrete buildings, CSA-A23.3-94, provides requirements for the detailing of reinforcement that will result in ductile response. The bridge design code, CSA-CAN-S6-84, does not specifically cover ductile seismic design of concrete members.

Shear failures should be avoided at all cost, and in this respect the concept of capacity design should be followed such that flexural yield will occur before the shear capacity is reached. This applies even to members that may not be considered part of the seismic load resisting system if it leads to failure of the gravity load structure.

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