PROGRESS AND TECHNIQUES USED IN EARTHQUAKE RETROFITTING CALIFORNIA HIGHWAY BRIDGES

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ABSTRACT

Following the San Fernando, California earthquake of February 9, 1971 about 1100 structures of the 13,000 in the highway system were found to be in need of retrofitting work to increase their seismic resistance. Retrofitting is now complete for over half of this number. All identified seismic retrofitting on State Highway System structures should be completed by 1988.

The initial goal of the retrofitting program was to tie bridge superstructure components together at hinges and bearings and to tie superstructures to substructures at bearing supports. Provision for vertical restraint at steel bearings was also deemed an important consideration, since the San Fernando Earthquake (Richter Magnitude 6.6) had demonstrated that appreciable vertical motions as well as the more commonly anticipated horizontal motions could be expected with even a relatively low magnitude event.

Highway bridge structures should be made seismically resistant to the extent that while they may sustain localized damage, they will not collapse catastrophically. Techniques and hardware for accomplishing this purpose are discussed.

A priority system has been developed for the remainder of the retrofit program, which takes into account the expected bedrock acceleration at the structure site, the estimated cost to retrofit the structure to withstand the expected bedrock acceleration, the cost of structure replacement in the event of loss, the ratio of the replacement cost to the retrofit cost, the length and availability of detours, and the average daily traffic on the main highway, as well as several other minor factors. The total cost for seismic retrofitting of California's State Highway bridges is expected to be about fifty million dollars.

INTRODUCTION

The initial goal of the retrofitting program was to tie bridge superstructures together at hinges and bearings and to tie superstructures to substructures at bearing supports, in the highly seismic areas of the State. Provision for vertical restraint was also deemed an important consideration, since the February 1971 San Fernando Earthquake (Richter Magnitude 6.6) had demonstrated that we could expect appreciable vertical motions as well as the more commonly anticipated horizontal motions, with even a relatively low magnitude event.

The main purpose of restrainers is to prevent spans from separating at hinges or falling off their bearing supports and to make structures seismically resistant to the extent that while they may sustain localized damage, they will not collapse catastrophically. It is also deemed desirable that highway structures be rendered capable of carrying emergency traffic with quickly performed temporary repairs so as to provide transportation lifelines for a stricken community immediately after a disaster.

GENERAL

As a minimum hinge restrainers should be capable of resisting a longitudinal force equal to 25% of the dead load of the lighter segment of superstructure framing into the hinge, for Working Stress Design, or 33% for Load Factor Design. If a dynamic analysis reveals higher seismic loading greater restrainer capacities should be provided. An ideal restrainer should resist anticipated seismic forces and remain elastic, restrict unduly large movements of bridge segments, dissipate energy and return the structure segments to their relative pre-earthquake positions. An economically feasible, ideal restrainer has not been developed to date.

Steel cables and rods acting in direct tension are probably the most economical and suitable restrainers for most bridges. Since even the most sophisticated dynamic analyses are rough approximations at best, a designer should not hesitate to supplement the results of such analyses with imagination and judgement.

Cables and rods acting in tension do not dissipate any significant amount of energy. They store energy as they are stretched but impart it back to the structure as the segments move back together. Cables tensioned repeatedly within the elastic range will store more energy than an equivalent number of high strength steel rods of the same length. Rods will absorb more energy than an equivalent number of cables of the same length if both are tensioned beyond the yield strength. However, considering all of the unknowns involved, it is not prudent to depend on a restrainer acting beyond its elastic limit, so dissipation of energy has not been attained to any great degree in systems developed to date.

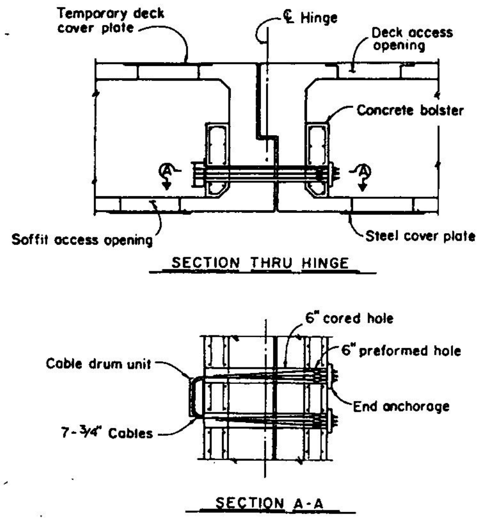
Adequate lengths of cables or rods should be used in order to assure sufficient deformation. Insufficient ability to stretch can permit non-elastic excursions and premature failure of the restrainers. The results of dynamic analyses for complex structures should be carefully reviewed to assure that superstructure movements at joints are kept within tolerable limits, permitting restrainers to function within the elastic range. Excessive stretching could lead to large superstructure displacements with resultant column failures and could be especially critical in permitting superstructures with narrow hinge or bearing seats to become dislodged. Cable restrainers may permit the faces of hinges or the ends of girders to be damaged by battering on the compression cycle but the damage should be repairable and not extensive enough to allow the spans to drop.

Restrainers should also have redundancy. There is always a chance that a single unit may have a defect due to faulty fabrication, installation, adjustment or maintenance. A system of restrainer units therefore should have enough capacity to pick up a proportionately increased share of the load if one unit should fail prematurely. When a restrainer assembly is subjected to ultimate load, failure should be ductile rather than brittle.

Attaching devices for restrainers should not fail under any conditions of seismic loading. Restrainer brackets and connections should be at least 25% stronger than the cables, rods or primary restraining devices. They should be designed so that they will not fail or cause failure of the portion of structure they are attached to if some component part or parts of the unit are misadjusted or fail prematurely. (See appendix for additional data on mounting devices.)

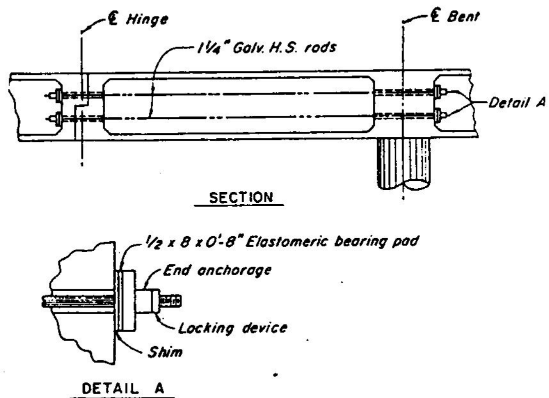
HINGE IN CONCRETE SUPERSTRUCTURES

In California we concentrated first on the hinges in our continuous structures and developed a fairly simple hinge restrainer unit for use in concrete box girder bridges. (Fig. 1) Components were easily fabricated from standard, available hardware and our retrofit contractors quickly developed the knack of installing them in structures at a reasonable price. Concrete bolsters are generally required to spread out the concentrated forces of the restrainers so that they don't overload the hinge diaphragms. One 7-cable (428 kip) unit placed in each exterior cell at each hinge is generally considered to be a minimum requirement in order to provide optimum resistance to transverse bending of the entire superstructure. This 7 cable unit, however, does not have high enough load capacity for certain superstructure configurations in highly seismic areas.



Cable Restrainer
Figure 1

As a result we have developed the high-strength rod restrainer (Fig. 2). A unit with four symmetrically placed 1¼ high strength rods is rated at 600 kips design load. Units using one, two or four rods have been successfully installed. They possess the additional advantage of requiring smaller size holes to be cored through existing concrete elements. (See appendix for cable and rod data.)



HIGH STRENGTH ROD RESTRAINER
Figure 2

CORING EXISTING CONCRETE

It is desirable that holes cored through concrete do not cut through reinforcing steel, hinge hardware or prestressing tendons. Although coring through steel is usually more of a construction nuisance than a structural problem, it is desirable to avoid it whenever possible. Holes should be located where they will not interfere with the above elements and special care should be taken to avoid structurally critical reinforcement.

Coring through a few pretensioned strands near the end of a precast-prestressed concrete girder may be unavoidable and is usually not structurally serious. However, special consideration should be given to locating holes to be cored in bridge members post-tensioned with rods or large multi-strand tendons.

Before locating holes to be cored in existing prestressed members, the designer should determine the method of prestressing which was used. The as-built plans, shop plans, or construction photos may be consulted. If it cannot be determined otherwise, it should be assumed that prestressing was accomplished by means of rods or large tendons, and measures to avoid them should be taken accordingly.

HINGES IN STEEL SUPERSTRUCTURES

Steel superstructure hinges pose similar problems to hinges in concrete superstructures.

It can generally be assumed that any seat type hinge (Fig. 3) used with steel girders will need additional transverse, longitudinal, and vertical restraint in even moderate seismic areas.

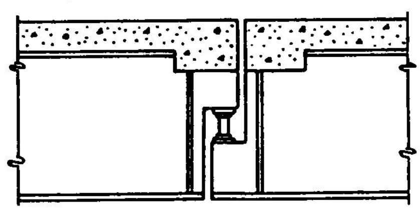


Figure 3

Hanger type hinges (Fig. 4) generally have more seismic resistance than the seat type shown in Figure 3, but are still subject to damage under seismic loading particularly in the transverse direction. These hinges often have steel bars or angles that bear against the web, or lugs attached to the flanges, which are designed to keep the girders aligned transversely for wind forces. Those devices are usually structurally inadequate for even moderate seismic loading and are generally much too short to be effective. Consideration should be given to complete replacement or adding supplemental longitudinal and transverse restraint.

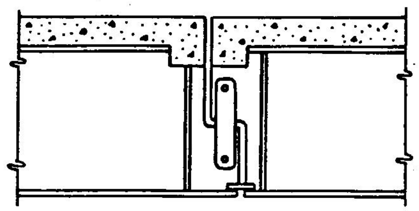


Figure 4

DESIGN CONSIDERATIONS FOR BEARING SUPPORTS

Bridge bearings historically have been among the most seismically sensitive details, and their ability to resist earthquake forces cannot be reliably evaluated with any great degree of precision. They are often the primary cause of complete seismic failures. There are many cases where bearings have been damaged by minor quakes and would undoubtedly have permitted spans to drop if the shaking had been a little stronger or lasted a little longer. Bearings at the Alamo River Bridge which were moved laterally by the Imperial Valley quake of Oct. 15, 1979 are a case in point. Strong shaking at this location only lasted for four or five seconds with a total earthquake duration of thirteen seconds. The Richter Magnitude was 6.4. The bearing rockers were displaced about six inches transversely and the keeper plates were destroyed.

One should be very cautious about assuming that keeper plates, bearing plate anchor bolts, keeper plate bolts or welds, and similar details have any significant effect in keeping a bridge superstructure on its supports during a major earthquake. All of the bearings at the end of a span do not always resist horizontal forces simultaneously. Because keeper plates or other devices are not set during construction with exactly the same clearances, only a portion of the bearings will initially resist a horizontal force in one direction.

It is not uncommon for a line of bearings at one end of a span to be damaged to varying degrees after an earthquake. The Eureka offshore earthquake of November 8, 1980 provided an example of variable bearing damage at the south Abutment of the Fields Landing O.H. The Richter magnitude was estimated to be between 6.6 and 7.1. The concrete support pedestals, were destroyed progressively across the abutment allowing the bearing rockers to fall and causing collapse of the superstructure.

Anchor bolts placed too close to the edge of the bearing seat will spall off the concrete and tear out when subjected to horizontal loads.

Anchor bolts are frequently threaded below the top surface of the pier or abutment seat. This practice gives a reduced area for shear and minimal resistance to bending before failure occurs due to notch sensitivity at the root of thread.

Grout pads under bearing masonry plates have traditionally posed problems during and after construction and have also been one of the main trouble spots in minor quakes. Failure of a grout pad will allow the bearing assembly to move and subject the anchor bolts to combined bending and shear.

Bearing rollers allow normal movements between the sole plate and the superstructure plate. Transverse sliding due to seismic loading takes place on both contact surfaces. Transverse sliding obviously does not start until the horizontal force exceeds the vertical dead load on the bearing multiplied by the coefficient of friction. This action, once initiated, results in repeated impacts on the end keeper plates leading to a progressive bending failure of the plates.

The simplest way to retrofit bearings of two adjacent simple spans resting on a single bent or pier is shown in Figure 5.

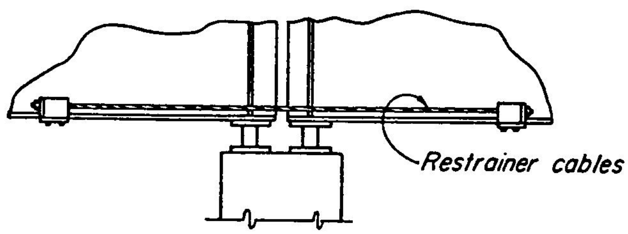


Figure 5

Connecting the ends of girders together in adjacent simple spans is generally satisfactory for short structures with relatively wide bent caps where it is extremely unlikely that the ends of girders will drop off the bents. For new construction our present minimum distance from centerline cap to edge is 24 inches. The Applied Technology Council (ATC-6) requirements for cap widths for new structures vary in accordance with length of contributing span and seismicity, but are generally more conservative than our requirements. Many of our older structures have much less edge distance and in these cases we now replace the bearings with new higher capacity assemblies.

The detail in Fig. 6 is generally preferred to that in Figure 5, since a vertical component in the cable is provided in addition to a horizontal restraint. The bent should be sufficiently strong to accept this force. With this detail, vertical clearances under the structure near the bent should also be considered. In some areas the visibility of the cable may be aesthetically objectionable. All of which brings us to the most effective, yet most costly method of retrofitting bearings developed to date - complete replacement.

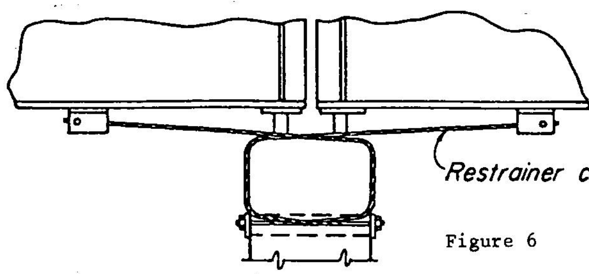


Figure 6

Following the field review of bearing failures after the four most recent earthquakes in California, it was decided, where possible, to replace high pedestals or bearing bars with elastomeric pads. In order to maintain the same elevation of superstructure, fabricated steel bearing assemblies are installed together with elastomeric pads in lieu of the rocker bearings. The units are made from A-36 steel. Jacking from the piers or abutments permits removal of the rockers and insertion of the new bearing system. Details at an abutment are shown in Figure 7.

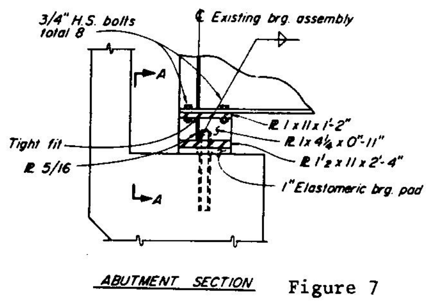
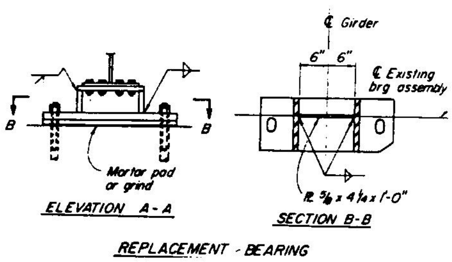
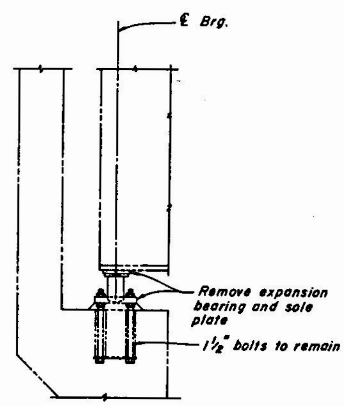


Figure 7

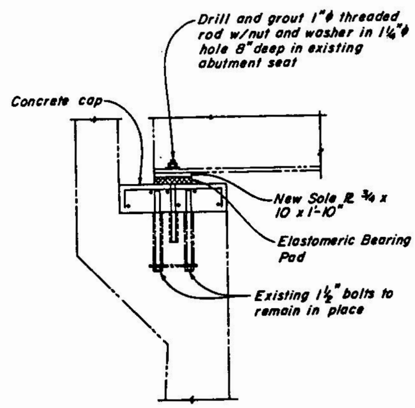


A bearing assembly replacement using a cast-in-place reinforced concrete cap was accomplished at the Middle Fork of the Smith River on Highway 199 near Crescent City. The bridge is a two-span composite steel girder with expansion bearings at the abutments and fixed bearings at the center pier. The new concrete cap at the abutments provides an enlarged bearing area for the new elastomeric bearing pads. It is secured to the existing concrete with #6 rebar grouted into one inch holes, eight inches deep. One inch threaded rods secure the girders through the sole plates, the new concrete cap and into the existing abutment seat. The original abutment bearing is shown in Figure 8 and the modified bearing is shown in Figure 9.



EXISTING EXPANSION ROCKER BEARING AT ABUTMENT

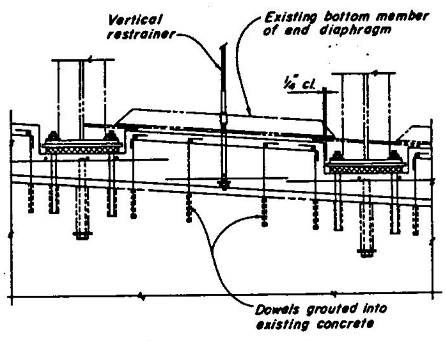
Figure 8



REPLACEMENT BEARING AT ABUTMENT

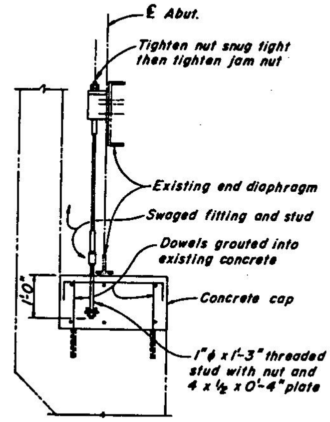
Figure 9

Transverse shear keys are provided by configuring the new concrete cap as shown in Figure 10. Our present earthquake criteria calls for hold-down devices (vertical restrainers) at all supports. The minimum design force for a hold-down device is 10% of the dead load reaction. For this installation vertical restrainers are provided between the girders. The restrainer anchorages are imbedded in the new concrete cap; the upper end is secured to the end diaphragm by means of a special bracket and four 1 inch high strength bolts, Figure 11.



EXPANSION ROCKER BEARING
PART ELEVATION

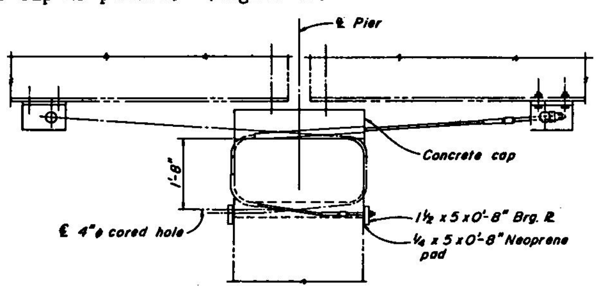
Figure 10



VERTICAL RESTRAINER
AT ABUTMENT

Figure 11

A measure of vertical restraint is provided by the looped configuration of the longitudinal restrainer cables at the pier. Longitudinal restrainers are placed and tensioned before the concrete in the pier top concrete cap is placed. (Figure 12)



PART ELEVATION
LONGITUDINAL CABLES

Figure 12

The preceding examples briefly cover the present state-of-the-art of seismic retrofitting in California's highway bridges.

Restrainer Materials Data

3/4 inch cable, galvanized:

Minimum breaking strength = 46 kips

A_s = 0.222 in ^{2}

E = 18,000,000 psi (after initial stretching)

Working Strength Design:

Allowable working strength = 0.5 x 46 = 23 kips

Allowable seismic load per cable - 1.33 x 23 = 30.6 kips

Load Factor Design:

Assume yield strength = 85% x 46 = 39.1 kips

High strength rods; galvanized: ASTM A-722 with supplementary requirements (the supplementary requirements specify a minimum elongation of 7% in 10 rod diameters.)

E = 30,000,000 psi

Diameter inches	Cross Section Area inches	Ultimate Strength ksi	Yield Strength ksi	Yield Strength kips	Working Strength kips
1	0.85	150	120	102	60
1¼	1.25	150	120	150	90
1 3/8	1.58	150	120	190	115

Brackets and Mounting Devices

The following ultimate loads should be assumed for designing connections and determining the adequacy of supporting members:

3/4" cables (6x19. Federal Spec. RR-w-410c)

F_u = 53 kips

1¼" H.S. rods (ASTM A-722 with Supplemental Requirements)

F_u = 188 kips

(use 53 x 1.25 = 66.2 kips and
188 x 1.25 = 235.0 kips per cable
and rod, respectively)

Bolted Connections shall be designed as a bearing type:

H.S. Bolts (A325)	Allowable Shear (F_v=0.6 Fu Ø Ar)	Allowable Tension (Ft = Ø F_u)
3/4"	20.1 kips	34.1 kips
7/8"	27.7	47.1
1"	36.3	61.8
1 1/8"	45.8	68.1