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Structural Details to Accommodate Seismic Movements of Highway Bridges and Retaining Walls

by

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Technical Report NCEER-97-0007

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NOTICE

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This report describes detailing for structural movements for bridges and retaining walls for new construction in the western and eastern U.S. Bridge retaining devices such as longitudinal joint restrainers, vertical motion restrainers, shear keys, and integral superstructure to substructure connections are described. Many of these details are traditional methods that have been used in new bridge construction to limit displacements for seismic events. Sacrificial elements, which include abutments and joints, are also described. These types of details have been used in new seismic designs within the last two decades. An introduction to passive energy dissipating devices and isolation bearing systems is provided as well as recommendations for detailing. Both devices are relatively new as a method to limit displacements in bridges within the U.S. In fact, isolation bearing systems have just emerged for new bridge construction within the last few years. The minimum support length requirements are reviewed. The current practice for designing earth retaining systems for seismic displacements is reviewed and some recommendations for detailing are provided. The effects of substructure flexibility on the isolation system is documented. An example study and comparison is given to illustrate the impact of substructure flexibility.

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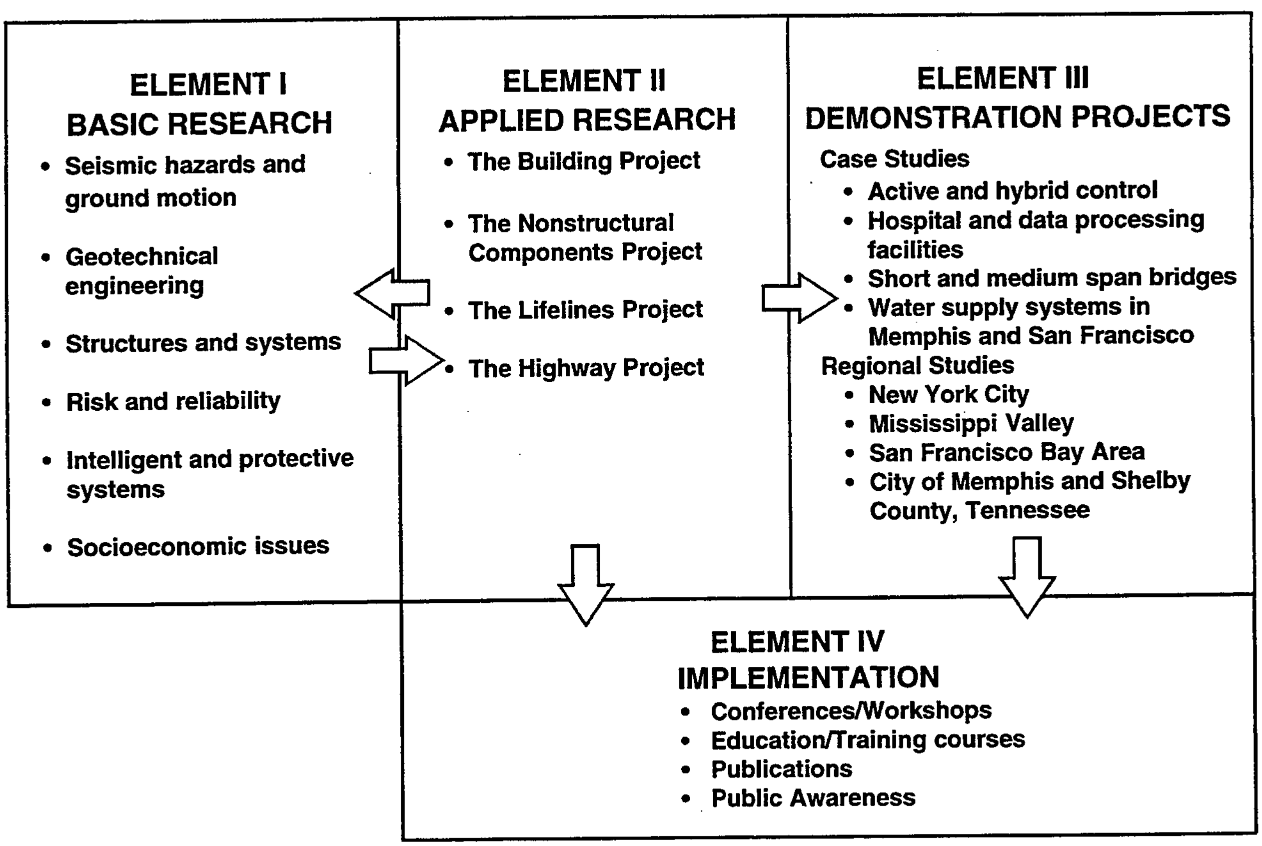
PREFACE

The National Center for Earthquake Engineering Research (NCEER) was established in 1986 to develop and disseminate new knowledge about earthquakes, earthquake-resistant design and seismic hazard mitigation procedures to minimize loss of life and property. The emphasis of the Center is on eastern and central United States *structures*, and *lifelines* throughout the country that may be exposed to any level of earthquake hazard.

NCEER's research is conducted under one of four Projects: the Building Project, the Nonstructural Components Project, and the Lifelines Project, all three of which are principally supported by the National Science Foundation, and the Highway Project which is primarily sponsored by the Federal Highway Administration.

The research and implementation plan in years six through ten (1991-1996) for the Building, Nonstructural Components, and Lifelines Projects comprises four interdependent elements, as shown in the figure below. Element I, Basic Research, is carried out to support projects in the Applied Research area. Element II, Applied Research, is the major focus of work for years six through ten for these three projects. Demonstration Projects under Element III have been planned to support the Applied Research projects and include individual case studies and regional studies. Element IV, Implementation, will result from activity in the Applied Research projects, and from Demonstration Projects.



Research under the **Highway Project** develops retrofit and evaluation methodologies for existing bridges and other highway structures (including tunnels, retaining structures, slopes, culverts, and pavements), and develops improved seismic design criteria and procedures for bridges and other highway structures. Specifically, tasks are being conducted to: (1) assess the vulnerability of highway systems and structures; (2) develop concepts for retrofitting vulnerable highway structures and components; (3) develop improved design and analysis methodologies for bridges, tunnels, and retaining structures, with particular emphasis on soil-structure interaction mechanisms and their influence on structural response; and (4) review and improve seismic design and performance criteria for new highway systems and structures.

Highway Project research focuses on one of two distinct areas: the development of improved design criteria and philosophies for new or future highway construction, and the development of improved analysis and retrofitting methodologies for existing highway systems and structures. The research discussed in this report is a result of work conducted under the new highway construction project, and was performed within Task 112-D-5.3, "Detailing for Structural Movements - Bridges and Retaining Walls" of the project as shown in the flowchart.

The overall objective of this task is to develop recommended design details for bridges and earth retaining systems that can accommodate movements associated with structural response during earthquakes. This report provides a review of current practice for bridge restraining devices, sacrificial elements, minimum support length requirements and the design of earth retaining systems for seismic displacements. Passive energy dissipation devices and isolation bearings, which are now being considered in the design of new bridges, are also described by type and potential application.

SEISMIC VULNERABILITY OF NEW HIGHWAY CONSTRUCTION
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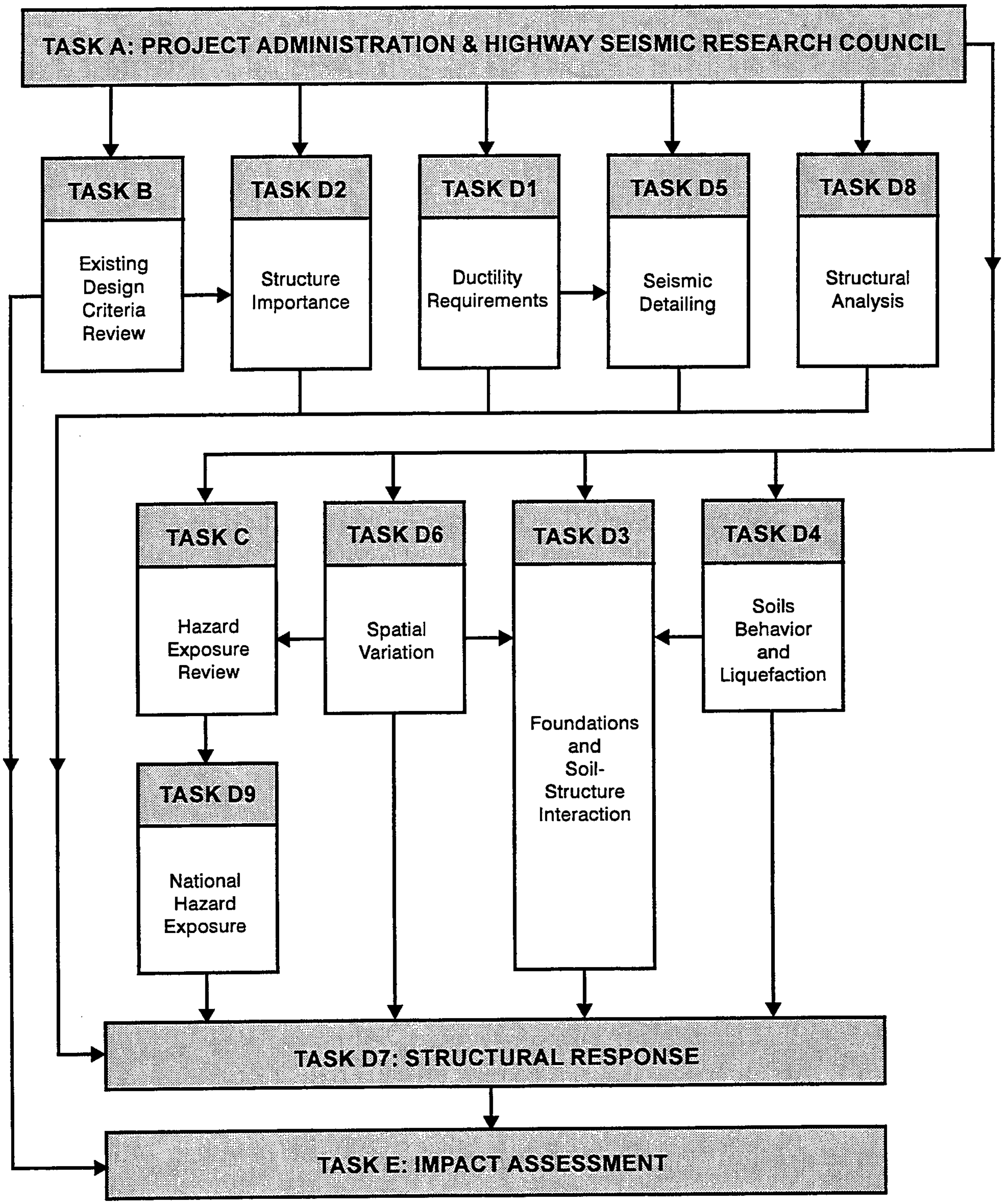

ABSTRACT

This report describes detailing for structural movements for bridges and retaining walls for new construction in the western and eastern U.S. Bridge retaining devices such as longitudinal joint restrainers, vertical motion restrainers, shear keys, and integral superstructure to substructure connections are described. Many of these details are traditional methods that have been used in new bridge construction to limit displacements for seismic events. Sacrificial elements, which include abutments and joints, are also described. These types of details have been used in new seismic designs within the last two decades. An introduction to passive energy dissipating devices and isolation bearing systems is provided as well as recommendations for detailing. Both devices are relatively new as a method to limit displacements in bridges within the U.S. In fact, isolation bearing systems have just emerged for new bridge construction within the last few years. The minimum support length requirements are reviewed. The current practice for designing earth retaining systems for seismic displacements is reviewed and some recommendations for detailing are provided. The effects of substructure flexibility on the isolation system is documented. An example study and comparison is given to illustrate the impact of substructure flexibility.

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LIST OF ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation Officials
Caltrans	California Department of Transportation
EERI	Earthquake Engineering Research Institute
FHWA	Federal Highway Administration
PennDOT	Pennsylvania Department of Transportation

SECTION 1

INTRODUCTION

In the summer of 1993, the National Center for Earthquake Engineering Research initiated a research program directed at developing improved seismic analysis and design procedures for highway infrastructure. The research program is sponsored by the Federal Highway Administration of the U.S. Department of Transportation and consists of a series of special studies, each focused on the seismic analysis or design of specific highway system components (e.g., bridges or tunnels) and structural elements (e.g., foundations or substructures).

As a basis for developing improved bridge design standards, a task within this program was conducted to identify and establish detailing for structural movements – bridges and retaining walls in use throughout the U.S. The task was divided into two parts, one focused on collecting and establishing details in the western U.S. conducted by Imbsen & Associates, Inc. (Task 112-D5.3(a)) and the other concerned with eastern and central U.S. practice conducted by Modjeski and Masters, Inc. (Task 112-D-5.3(b)). Many structural details have been developed in the western U.S., because of the larger magnitudes and recurrence rates of earthquakes in that region. Once the states of the central and eastern U.S. began implementing seismic details into their bridge components, it was only natural that tried and proven details already developed in the western states would be adopted. In addition, many details for new bridges, such as passive energy dissipating devices and isolation bearing systems, are being introduced in designs today. The report combines the effort from Task 112-D5.3 (a) and 112-D5.3 (b).

This report includes these topics: bridge restraining devices, sacrificial elements, passive energy dissipating devices and isolation bearing systems, minimum support length requirements, earth retaining systems, and the effects of column flexibility when using isolation. This report is a summary compilation of existing details. It also includes energy dissipation devices and isolation systems. Many states are using these new devices currently and modifying existing proven details that orginated from the west coast. The equations in Appendix A are not meant to be a substitutue for code requirements, rather they are presented on a selected basis to demonstrate the general processes.

SECTION 2

BRIDGE RESTRAINING DEVICES

2.1 Longitudinal Joint Restrainers

AASHTO recommends to designing generous support lengths at joints or hinges in new bridges as a first line of defense to prevent loss of support during a seismic event (AASHTO, 1991a). This has been a design practice in the western U.S. since the 1971 San Fernando earthquake. In addition, longitudinal restrainers have been incorporated in new bridge designs and retrofitting to limit relative displacements at joints and hinges. Longitudinal joint restrainers have been used extensively by the California Department of Transportation (Caltrans) since 1971, but their use in the eastern and central United States is relatively recent.

Restrainer demand and stiffness can be determined from a response spectrum analysis of the structure, but results from such an analysis may not be realistic because they result from elastic column forces which will not develop. Restrainers are usually designed only for tensile capacity and with joints that have a finite size gap; however, in an elastic analysis it is difficult to model this condition. Simplified procedures for designing restrainers have been developed which take into account joint behavior (FHWA, 1995).

In no case should the restrainer force capacity be less than the acceleration coefficient times the weight of the lighter of two adjoining spans or parts of the structure (AASHTO, 1991). In no case shall the restrainer force capacity be less than that required to resist an equivalent static load of 0.35 times the dead load of the bridge (FHWA, 1987). When two bridge segments are tied together, the New York Department of Transportation requires the minimum restrainer capacity to be the maximum of the two capacities obtained by considering each section independently (FHWA, 1987).

Connections of the restrainer to the superstructure and/or the substructure should be designed to resist 125 percent of the ultimate restrainer capacity. Any existing bridge elements subject to brittle failure should also be capable of resisting 125 percent of the ultimate restrainer capacity. Because of eccentricities caused by variations in the restrainer forces, it is recommended that the restrainer connections and existing bridge elements be capable of resisting variations in the restrainer forces of at least 10% of the nominal restrainer capacity.

Longitudinal restrainers should be oriented along the principle direction of expected movement. If piers are rigid in the transverse direction of the bridge, the expected movement of the superstructure will be along the longitudinal axis of the bridge. Restrainers should be placed to oppose such movement (see Figure 2-1).

The recommended analysis for longitudinal restrainer design is the equivalent static analysis method, developed at Caltrans (Caltrans, 1994a, b and FHWA, 1995). This method is currently used by Caltrans and is briefly summarized as follows:

1. Verify seat width by comparing it to the maximum restrainer deflection and limit the displacement to the bridge seat width.
2. Compute the longitudinal earthquake deflections on both sides of the superstructure joint under consideration by simplified methods. For curved bridges, compute the joint opening resulting from a lateral earthquake.

3. Compare the earthquake deflections to the allowable restrainer deflections.
4. Compute the number of restrainers required.
5. Check the deflections of the restrained system and revise the restrainer and/or column assumptions if required.

New design procedures for hinge restrainer design for multiple frame bridges have been proposed and compared to the current Caltrans' procedure (Desroches, 1996). This procedure was presented at the Caltrans' seismic research workshop in July 1996. The Caltrans' procedure has generally worked well; however, recent studies have shown that the procedure may be conservative or unconservative when estimating the relative displacement of the hinge. The purpose of the study was to develop a restrainer design procedure to better predict relative displacements between adjacent frames and determine the number of restrainers required to limit hinge displacement. A parameter study was performed to investigate the new procedure. Evaluation of the proposed procedure by linear and nonlinear time history analysis showed its ability to limit hinge displacement to a prescribed value. This procedure is under review by Caltrans.

Caltrans' design procedures require that cable and rod restrainers and all their associated steel hardware be galvanized (Caltrans, 1992). Caltrans has tested both $1^1/_4$" diameter bars and $^3/_4$" diameter cables to determine load versus elongation relationships. Graphs displaying the test results are shown in Figure 2-2 (FHWA, 1992). Many eastern states, such as New York, have a stated preference for galvanized steel wire rope for restrainers.

Another advantage of using longitudinal restrainers, other than limiting relative displacements at joints, is 1) their ability to transfer load through the hinge to other portions of the structure such as adjacent columns and abutments and 2) cable and rod restrainers are economical because the material is relatively inexpensive and they are relatively easy to install.

Some details of longitudinal restrainers are provided in Figures 2-3 to 2-5. The prestressed I-girder detail for new bridges shown in Figure 2-3 could be adapted for steel bridges, which are used frequently in eastern states. This detail is used in California and has been adopted in Pennsylvania. New box girder bridge designs in California use restrainers in combination with generous support widths at intermediate hinge span locations, see Figure 2-4. In addition, restrainers have been incorporated in new bridge designs of prestressed I-girders as shown in Figure 2-5. Caltrans has introduced a new detail on the end of swaged fittings on restrainers (see Figure 2-6). Disc springs are recommended for installation load indicators (Sahs, 1995). After the 1994 Northridge earthquake, the inspection of hinge restrainers for damage was given a high priority. Some observations were made that the amount of sag in the cables from Type 2 restrainers shown in Figure 2-4 varied from bay to bay and bridge to bridge. Also, there was no way for inspection teams to determine if the cables yielded or were installed with incorrect slack. This new detail is now recommended to eliminate these problems encountered in the field.

As an alternative to longitudinal restrainers, linkage slabs are being used as restraining devices. The linkage slab is designed to transmit horizontal seismic forces, yet produces little joint movement under live loading (FHWA, 1983). An example of a linkage slab is provided in Figure 2-7.

For the design of new steel girder bridges in New York State, displacement limiting devices such as restrainers are generally not used. Movement is accommodated by designing seat widths for the expected seismic movement, and providing superstructure continuity over the support to eliminate unseating. For transverse movement at the pier cap, adequate room is provided between the edge of the pier cap and the fascia girder bearing. Transverse shear keys are generally not required unless there is a possibility of a girder falling off the cap for an existing bridge. For new designs, elastomeric and pot bearings are considered. Pot bearings are used for long spans where thermal movements would require excessively thick elastomeric pads (Malik, 1995).

Seismic retrofits in New York state attempt to eliminate joints where possible instead of using restrainers to limit movement and prevent unseating. A standard design procedure converts simple span steel girders into a continuous superstructure. A 5 foot portion of the concrete deck on each side of the pier joint is removed, the steel girders are joined by bolted splice plates, see Figure 2-8, and a continuous deck is repoured. The two rocker bearings that supported the steel girders are replaced by one elastomeric bearing pad.

Unlike New York state, New Jersey still allows the use of steel rocker bearings as long as they can handle or are modified to handle the seismic movement. Modifications to steel bearings may include:

1. Increase size, number or embedment of anchor bolts,
2. Increase the outer diameter of the pin head,
3. Increase the width of the expansion rocker, and
4. Increase the top and bottom dimension of the pintle detail.

In lieu of restrainers, generous girder support lengths are provided for new design.

The state of Pennsylvania does not typically recommend cable restrainers, especially for retrofitting bridges. Steel rockers and roller bearings are not permitted on new bridges and should be replaced on rehabilitated projects. Providing superstructure continuity in new bridges, wherever possible, is recommended to eliminate a series of simply supported structures (PennDOT, 1994).

2.2 Vertical Motion Restrainers

Experience from past earthquakes has shown that vertical movement can take place at the bearings. This can lead to the displacement of bearings and a possible increase in the chance of a loss of support failure (FHWA, 1987). Vertical motion restrainers are usually not economically justified unless a bridge is classified in Seismic Performance Category D (SPC-D). Since accelerations in the eastern U.S. are usually lower, vertical motion restrainers are not typically needed there.

AASHTO seismic design specifications require that hold-down devices be provided at all supports or hinges in continuous structures where the vertical seismic force due to the longitudinal horizontal seismic load opposes and exceeds 50% of the dead load reaction.

If the vertical seismic force (Q) is less than 100% of the dead load reaction (DL), the minimum net upward force for the hold-down device shall be 10% of the dead load downward force that would be exerted if the span were simply supported.

If the vertical seismic force (Q) due to the longitudinal horizontal seismic load opposes and exceeds 100% of the dead load reaction (DL), the net upwards force for the hold-down device shall be 1.2(Q-DL), but it shall not be less than that specified in the previous paragraph (for $Q < 100\%$ DL) (AASHTO, 1991a).

Caltrans' design specification requires hold-down devices to be provided at all supports and intermediate hinges where the vertical seismic force opposes and exceeds 50% of the dead load reaction. In this case, the minimum seismic design force of the hold-down device shall be the greater of:

(a) 10% of the dead load reaction or
(b) 1.2 times the net uplift force (Caltrans, 1994c).

Vertical restrainer material is inexpensive, but they are usually difficult to install in box girder hinge seats congested with reinforcement. Therefore, it is not recommended to specify vertical ties as a standard if they are not required. Some example details of vertical restrainers are provided in Figures 2-4 to 2-5. Future vertical response studies may result in design specifications which could revise vertical restrainer policies.

2.3 Shear Keys

Current AASHTO specifications imply that shear keys transmit the entire force across a joint. This philosophy will result in the direct transfer of the full seismic force from the superstructure to the substructure, requiring substructure elements such as piles be designed to withstand the full seismic force. AASHTO design procedures (AASHTO, 1991a) require that shear keys be designed with a Response Modification Factor of R=0.80, or a magnification of 1.25 times the seismic loading. Such magnification is necessary due to the brittle failure nature of the shear key and the redistribution of forces to connections as plastic hinges form in columns.

Current Caltrans procedures deviate from this policy at abutments to protect subsurface structural members. Abutment shear keys are designed transversely to withstand the ultimate shear capacity of one wingwall plus 75% of the ultimate shear capacity of the piles. This philosophy will limit the force transmitted through the shear key, thus reducing the likelihood of damage to the piles. Likewise, the backwall key is designed to fuse at a force which is expected to prevent damage below the seat. (Caltrans, 1994b).

Proper design of a reinforced concrete shear key should recognize the deep beam or simple truss action of these short shear span members (Harper, 1994). There are two basic failure modes as shown in Figure 2-9, that must be addressed in design: 1.) a direct shear failure at the horizontal interface between the key and the abutment seat and 2.) separation between the key and the abutment stem. In the second mode of failure, the failure plane can be anywhere between vertical to approximately 45 degrees below the horizontal plane. This was the most common type of key failures seen in the 1994 Northridge earthquake.

In summary:

Force Concept (AASHTO design philosophy)

- Key transmits total seismic force.
- Requires adjacent component match.

Fuse Concept (Caltrans' abutment design philosophy)

- Key transmits service loads.
- Key fails before adjacent component match.

Some example details of shear keys from California are provided in Figures 2-10 to 2-12. The state of Massachusetts recently updated some standard details that control seismic movements for precast box beam superstructures (Massachusetts, 1994). Abutment curtain walls are designed as seismic shear keys. Continuity is achieved over the support through a superstructure closure pour that "keys" into the pier. Prestress strands from the butted box beams anchor into the closure segment. The details are shown in Figure 2-13. Some shear key details used by Penn DOT are shown in Figures 2-14 and 2-15. A unique shear key design in combination with an elastomeric bearing which has been implemented on a seismic retrofit in Michigan is shown in Figure 2-16. This concept could be extended to new bridge designs. The State of New York places lateral restraining angles on the elastomeric bearing in the transverse direction of the bridge (see Figure 2-17) (Malik, 1995).

Gapped shear keys designed to engage after some relative displacement are commonly referred to as "Stopper Blocks." These blocks provide restraint against the unanticipated movement of the superstructure relative to the substructure. They are common on bridges that use isolation bearings which have large expected displacements (i.e., 12" or greater). The gap is an important consideration in the design. As the gap becomes smaller, the stopper block will act as a shear key which transmits the total force to the substructure. Their use in isolation bearings are treated as a backup system in the event that the bearing fails. Gaps should be greater than the anticipated isolation displacement (at least 25% greater) to ensure that the bearing will function properly. An example of a stopper block, as shown in Figure 2-18, is being used on the Benicia-Martinez Bridge in California. This is intended as a backup system in the transverse direction of the bridge.

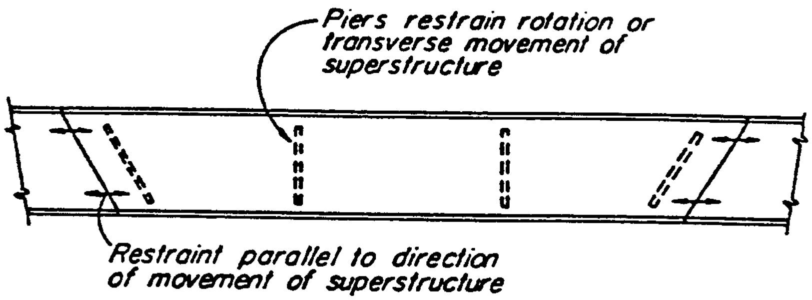
The Japanese have investigated stopper blocks that minimize impact to the substructure (Ozaki, 1992). A shock absorber type device made up of anchor bars and vertically placed elastomeric bearing pads enclosed in a steel shell provided some flexibility of restraint. The theory is to supply a device that prevents unseating yet if engaged will not transmit the full seismic shear to the substructure.

2.4 Integral Superstructure to Substructure Connections to Limit Seismic Displacements

The majority of bridges outside the western region of the United States use precast concrete or steel girders positioned on bent caps. The superstructure to substructure connection consists of some bearing type that would be modeled as a "pin" connection in a computer analysis. If a plastic hinge formed at the column base, instability would result. If a "fixed" type of connection could be detailed at the superstructure to substructure interface, seismic displacements would be greatly reduced.

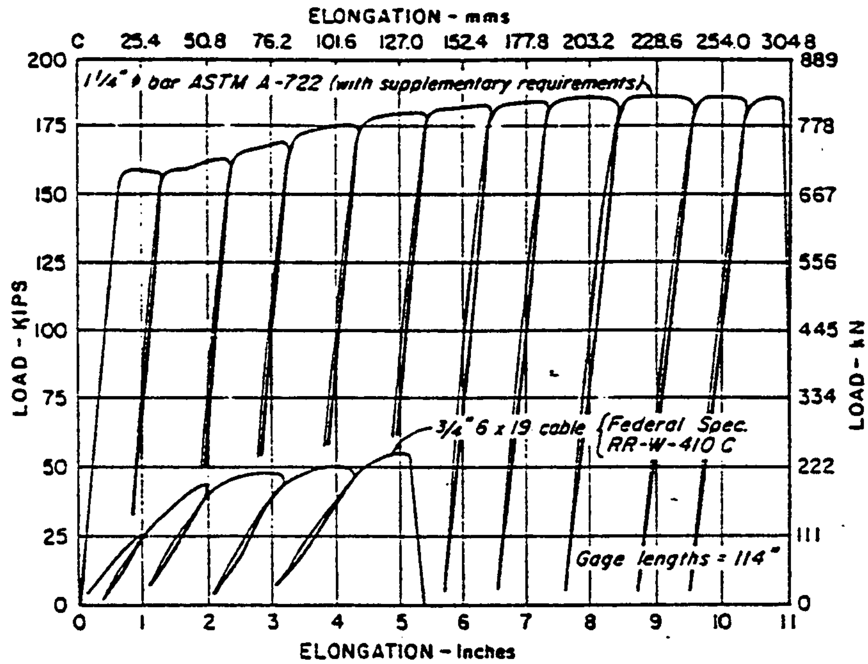
Precast concrete bridges of this type have been routinely constructed in the southeastern United States for reasons other than seismic. A research project currently underway at the University of California, San Diego is exploring the seismic benefits of an integral bent cap connection. Transverse post-tensioning across the bent cap provides a "clamping" force to the girders, thereby reducing the need for joint shear reinforcement and distributing the column plastic moment to all the girders.

Steel girder bridges have also been built with integral bent caps. Transverse post-tensioning across the bent cap is similar to the precast girder detail, see Figure 2-19. The seismic benefit is a "fixed" top and "fixed" bottom column connection which forces the column to rotate in double curvature for lateral displacement. This makes the column four times stiffer than a "pin" connection at the column top with corresponding reductions in displacement.

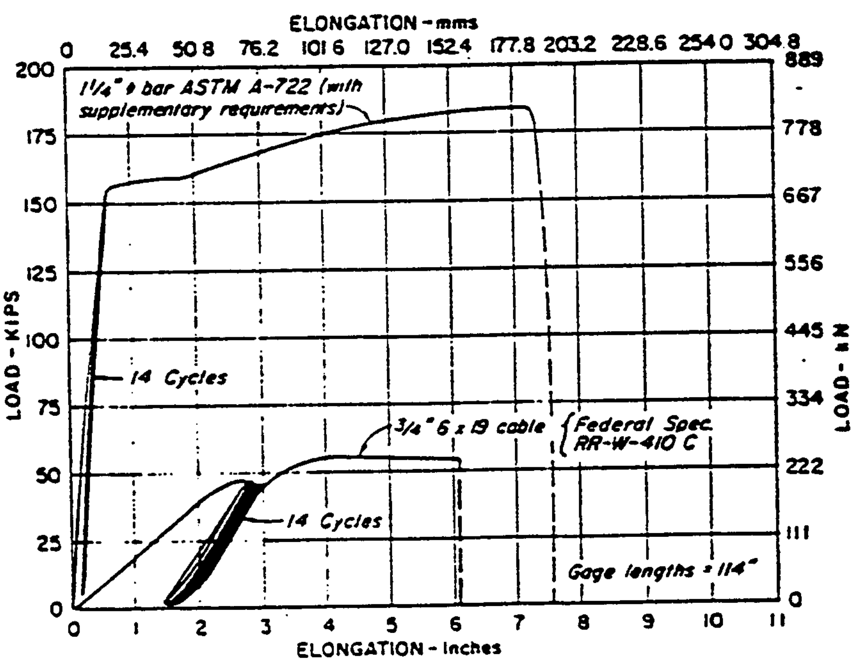


Restrainer Orientation – Transversely Rigid Supports

FIGURE 2-1: Restrainer Orientation



Cyclic Test of Restrainer Cables Beyond Yield



Cyclic Tests of Restrainer Cables to Yield

FIGURE 2-2: Cyclic Tests of Restrainer Cables and Bars

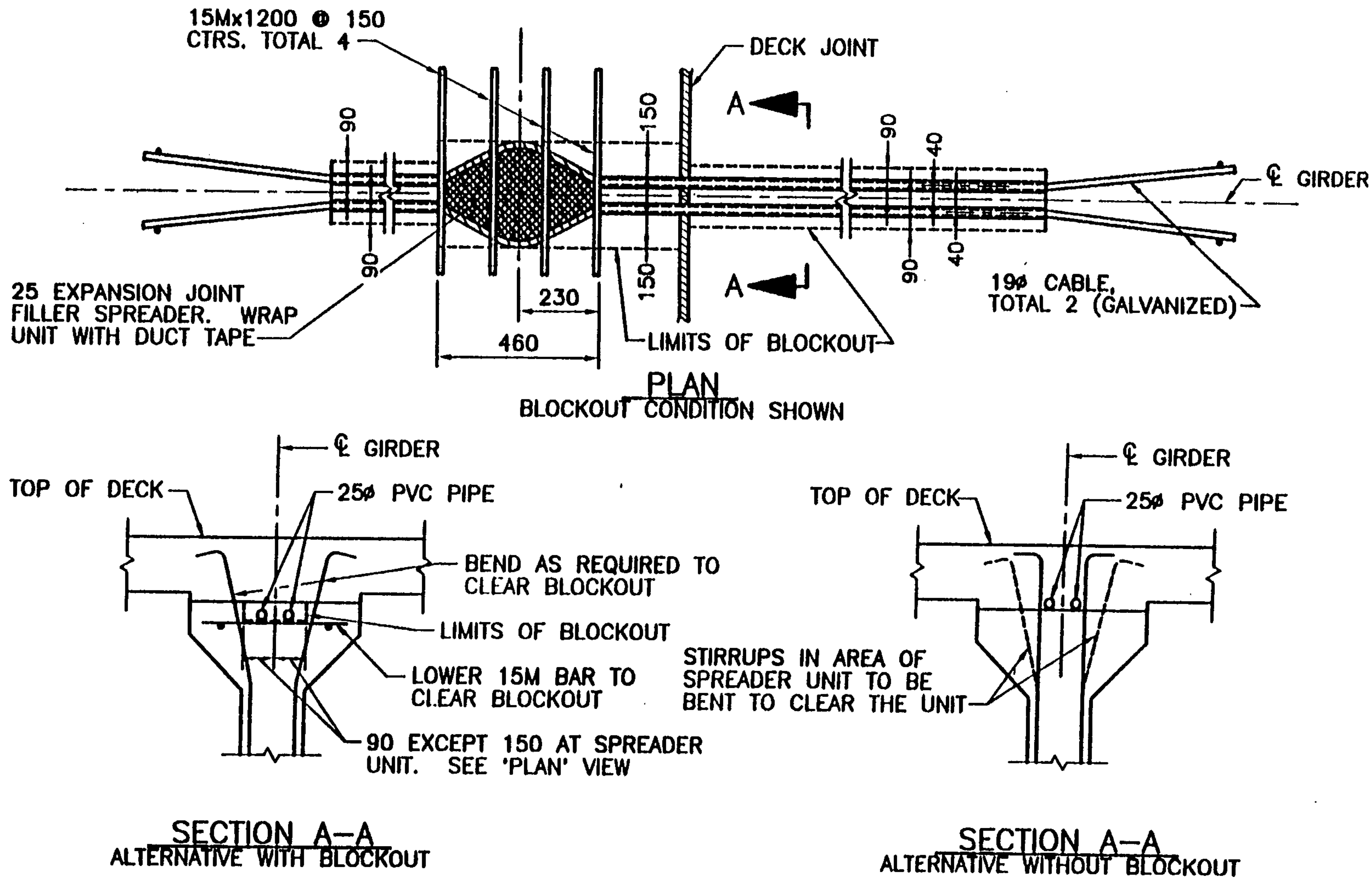
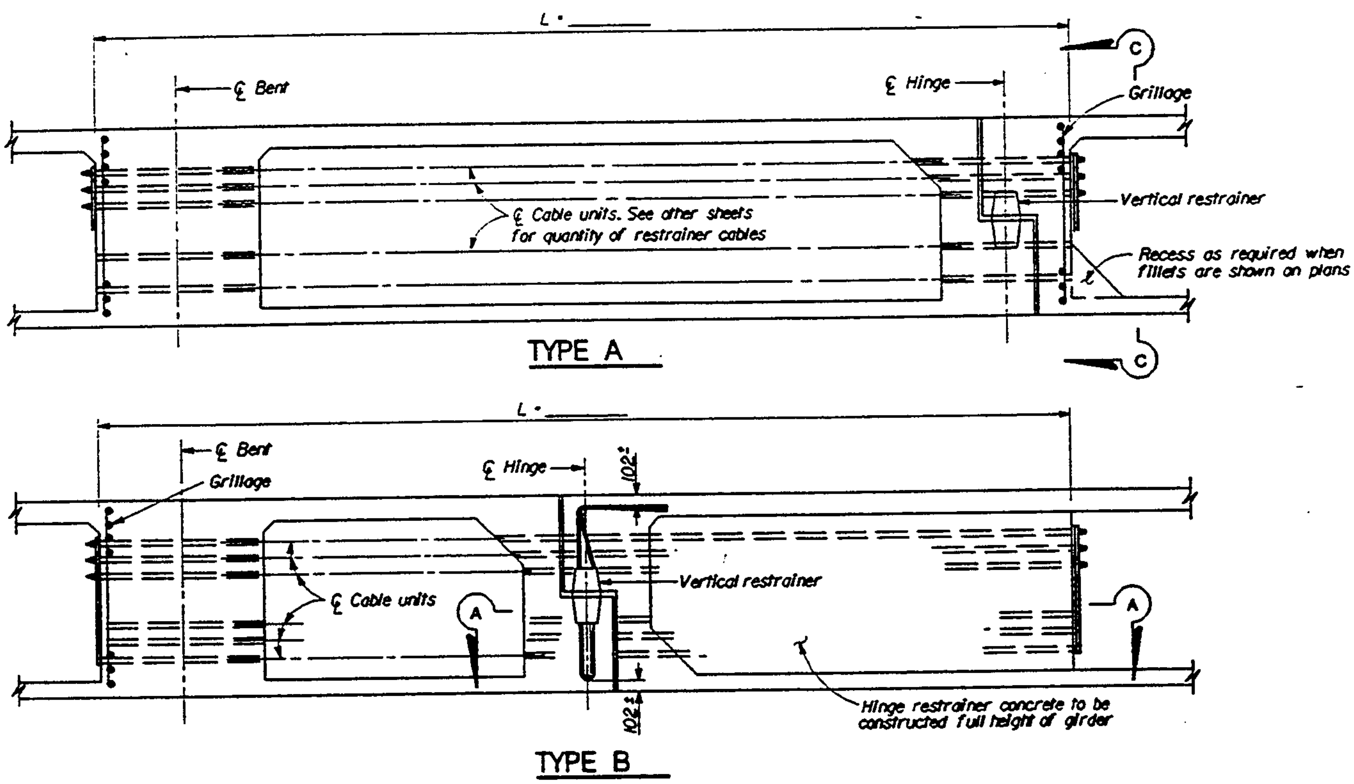


FIGURE 2-3: Precast Concrete I-Girder Longitudinal Restrainer Details



(Optional Type B – Use Determined by Design Specification Loads Section at Caltrans)

FIGURE 2-4: Cast-In-Place Box Girder Longitudinal and Vertical Restrainer Details (Type 2)

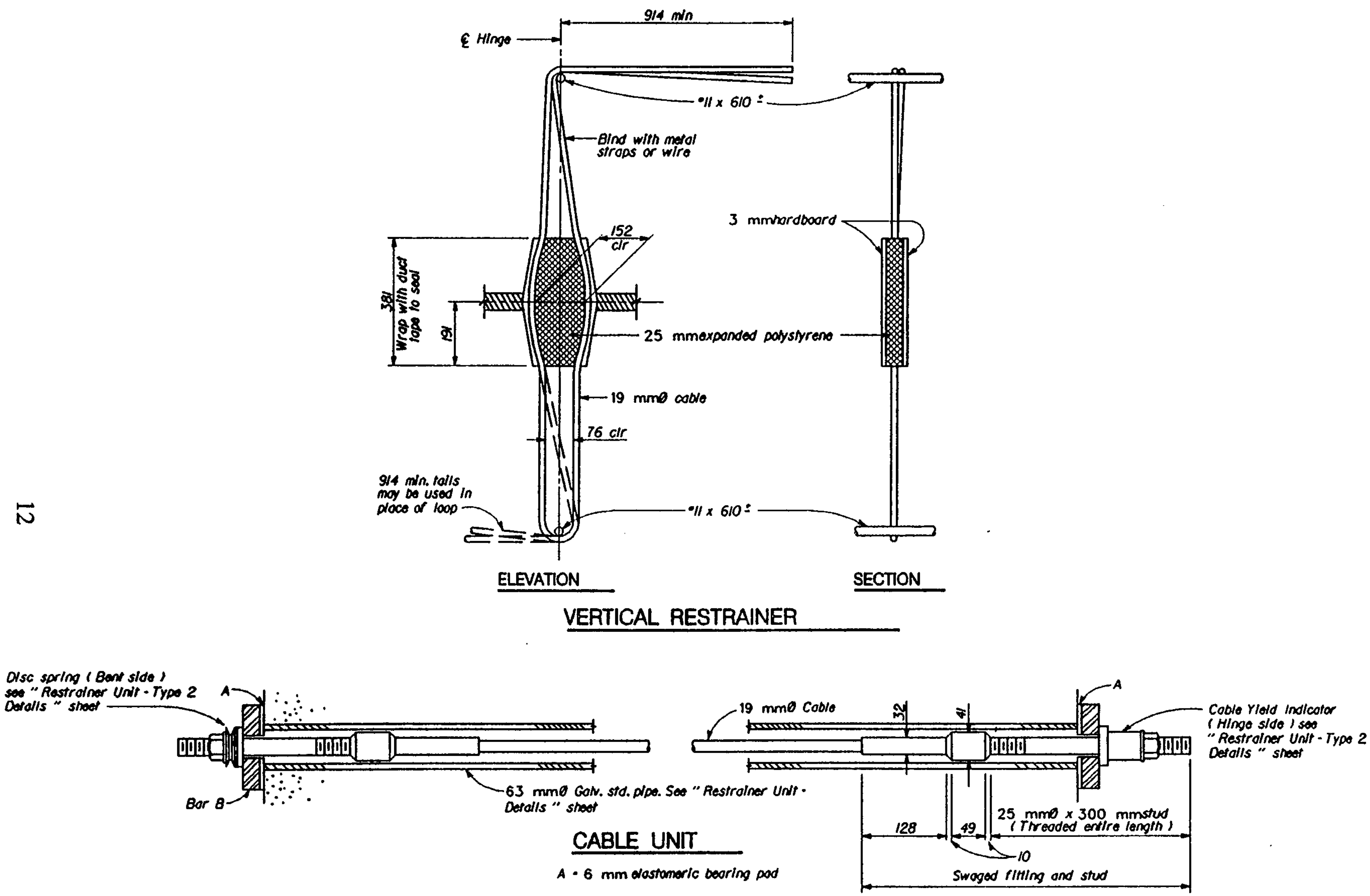
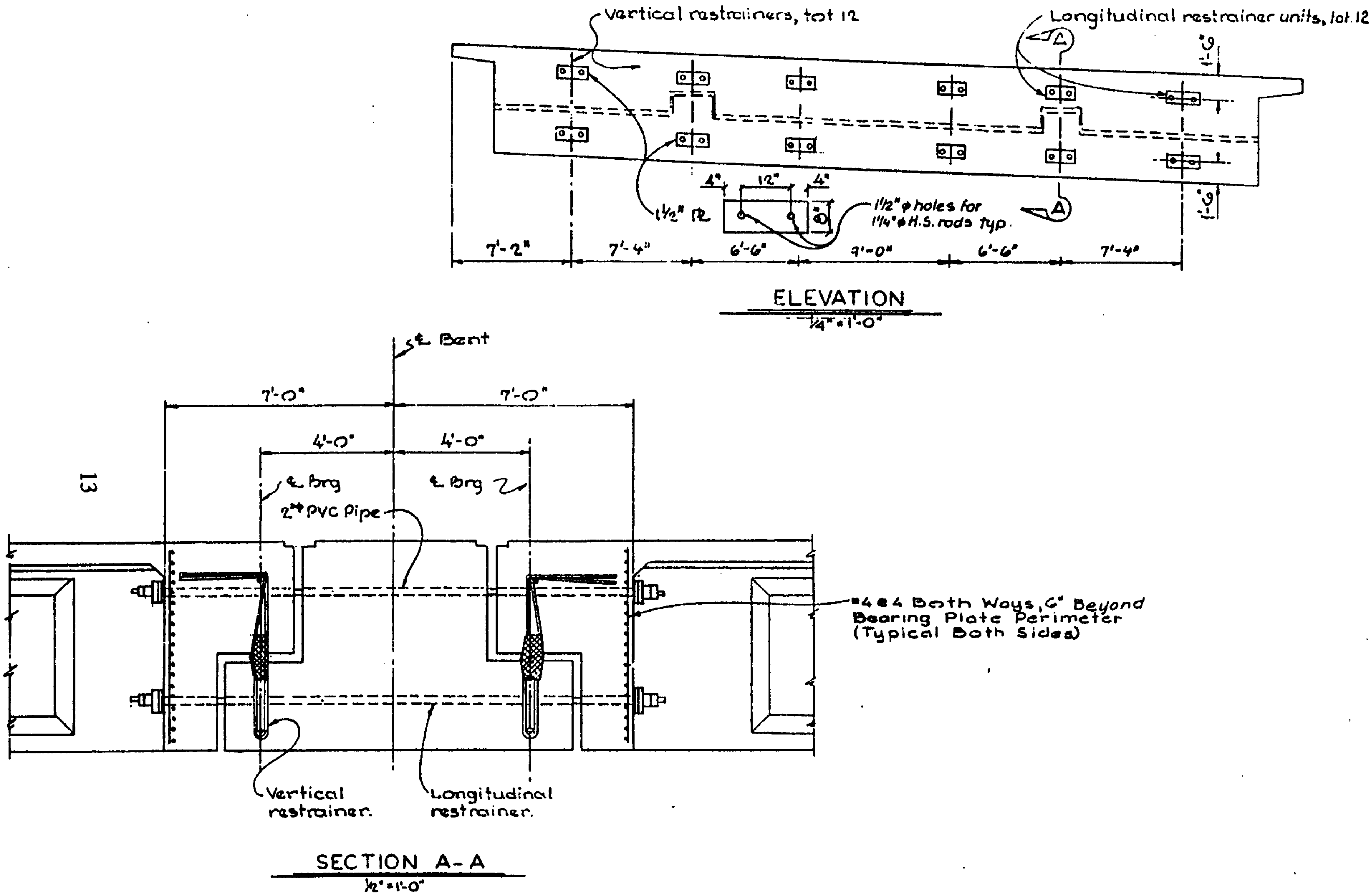


FIGURE 2-4: Cast-In-Place Box Girder Longitudinal and Vertical Restrainer Details (cont.)



FFIGURE 2-5: Precast Concrete I-Girder Longitudinal and Vertical Restrainer Details

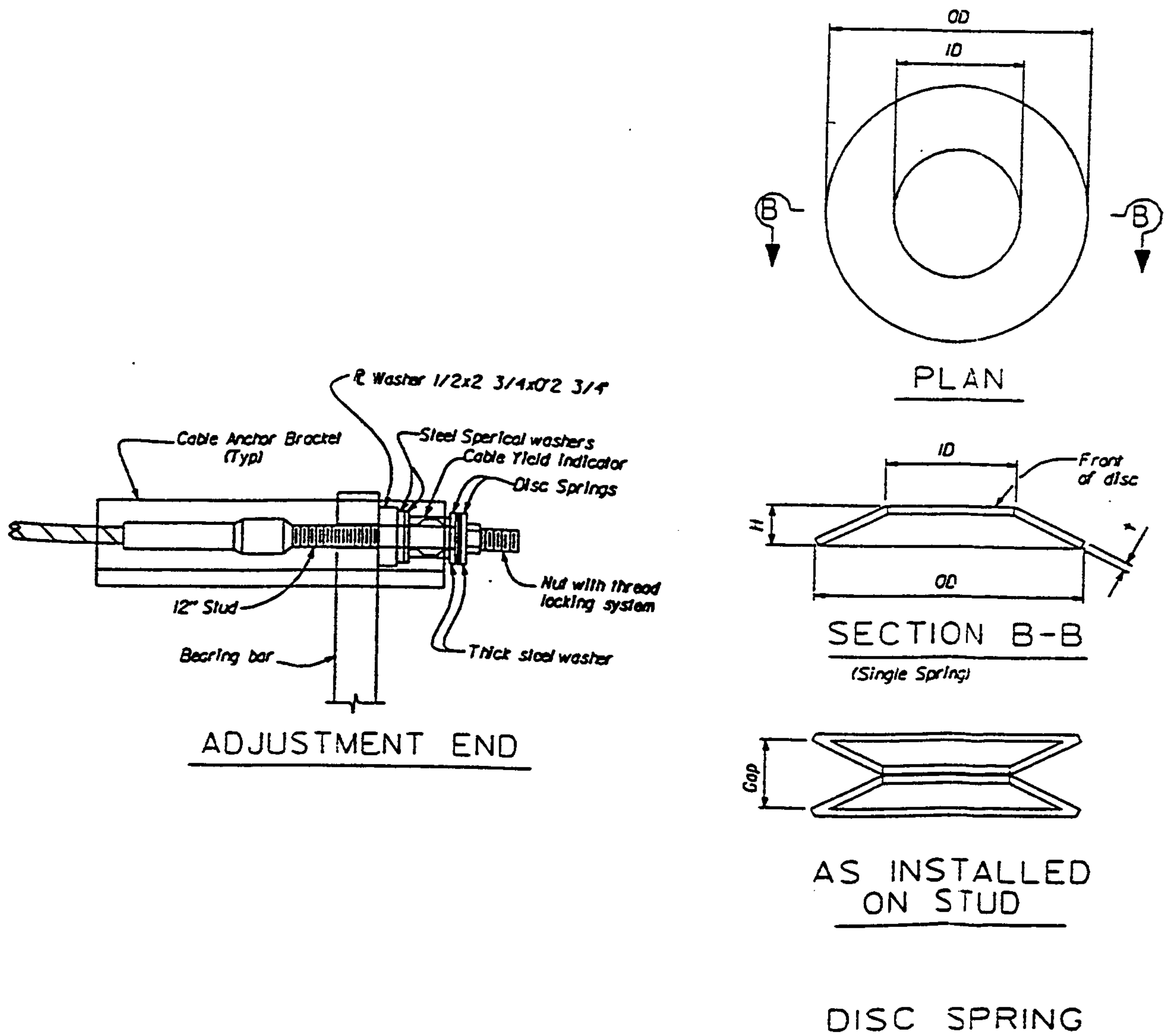
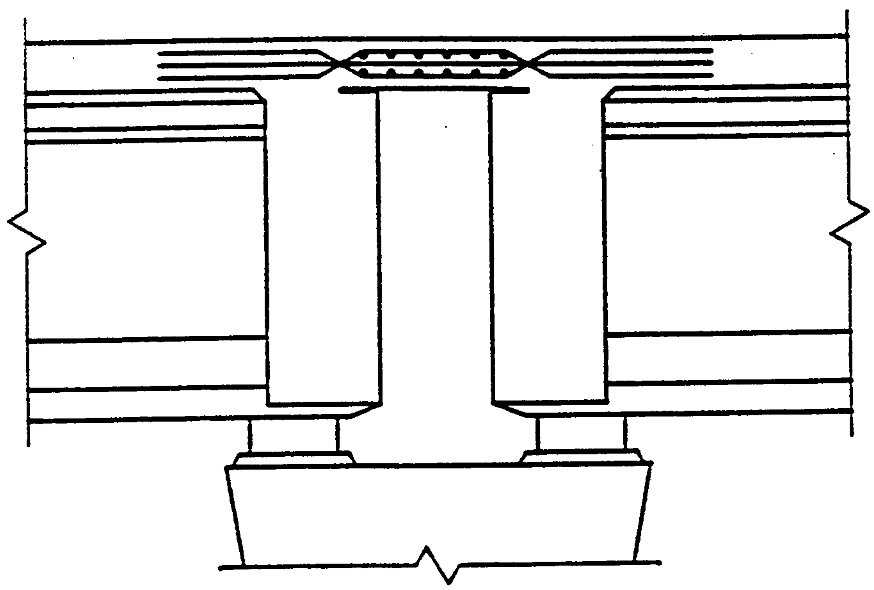
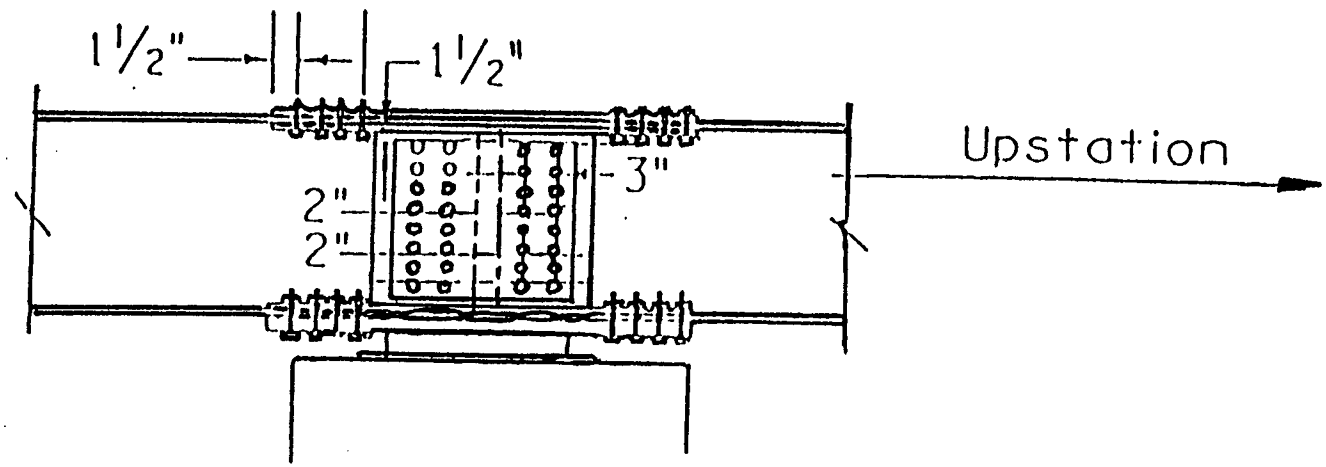


FIGURE 2-6: New Detail used on Swaged Fittings



Note: Linkage slab designed to transmit horizontal seismic forces, but superstructure acts simply supported under live load.

FIGURE 2-7: Reinforced Concrete Linkage Slab at Pier



TYPICAL ELEVATION
AT PIERS

FIGURE 2-8: Girder Splice Plate Details

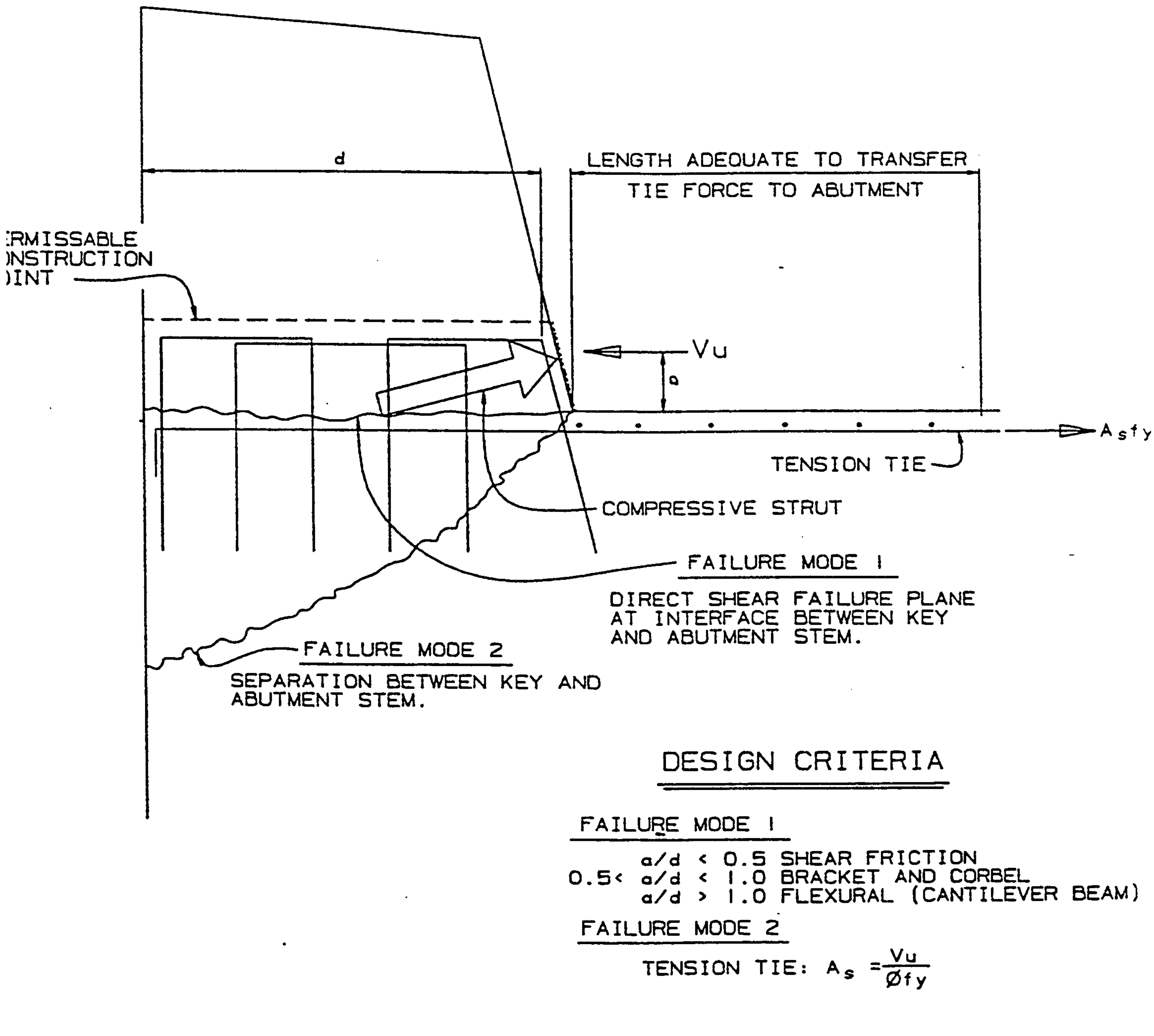


FIGURE 2-9: Shear Key Failure Modes

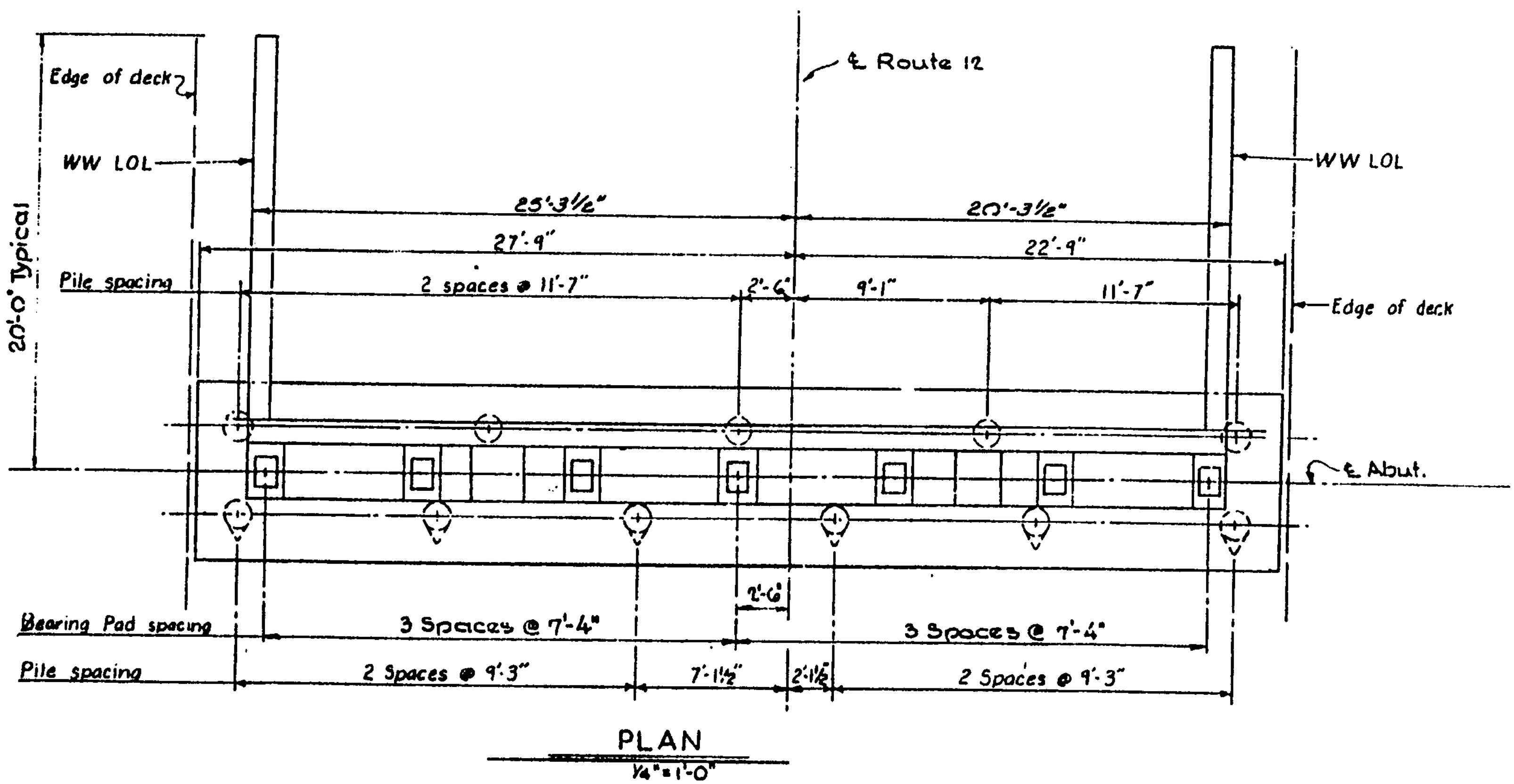
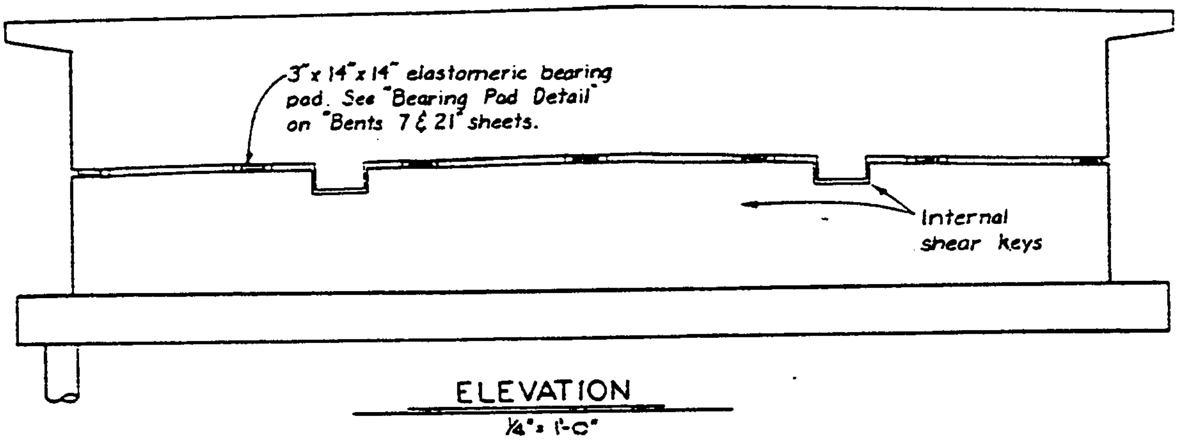


FIGURE 2-10: Precast Concrete I-Girder Depressed Shear Key at Abutment Details



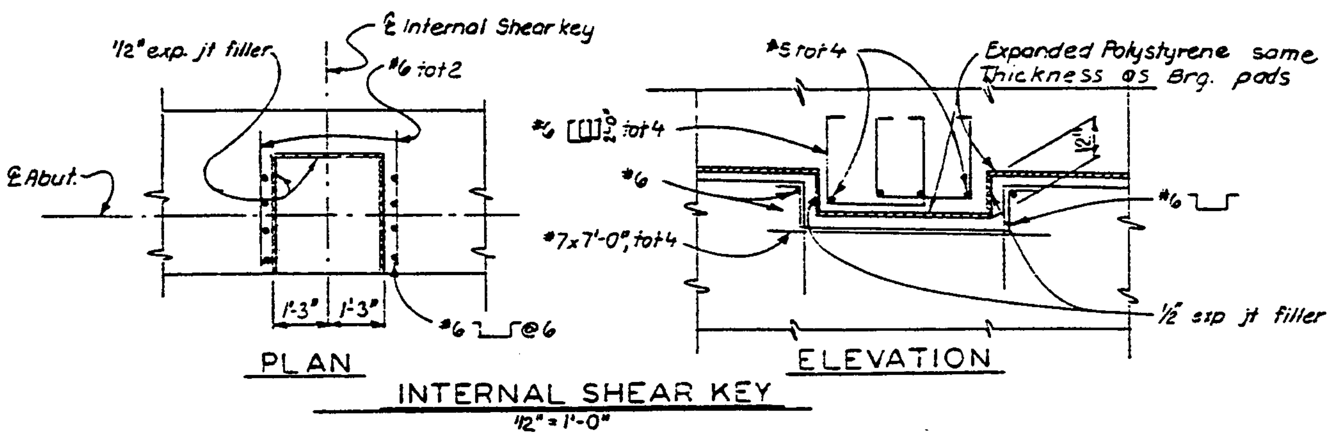
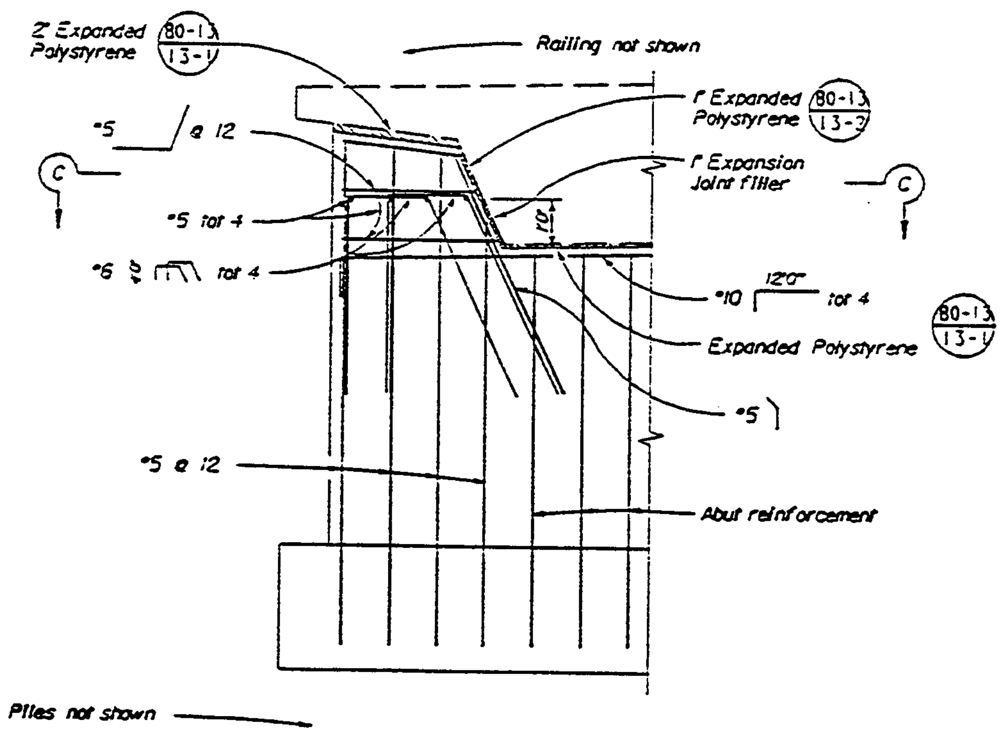


FIGURE 2-10: Precast Concrete I-Girder Depressed Shear Key at Abutment Details (cont.)



PART ELEVATION

½"=1'0"

Note: Caltrans' policy requires keys to shear (fuse) at the abutments for large earthquakes to prevent foundation damage.

FIGURE 2-11: Cast-In-Place Box Girder External Shear Key at Abutment Details

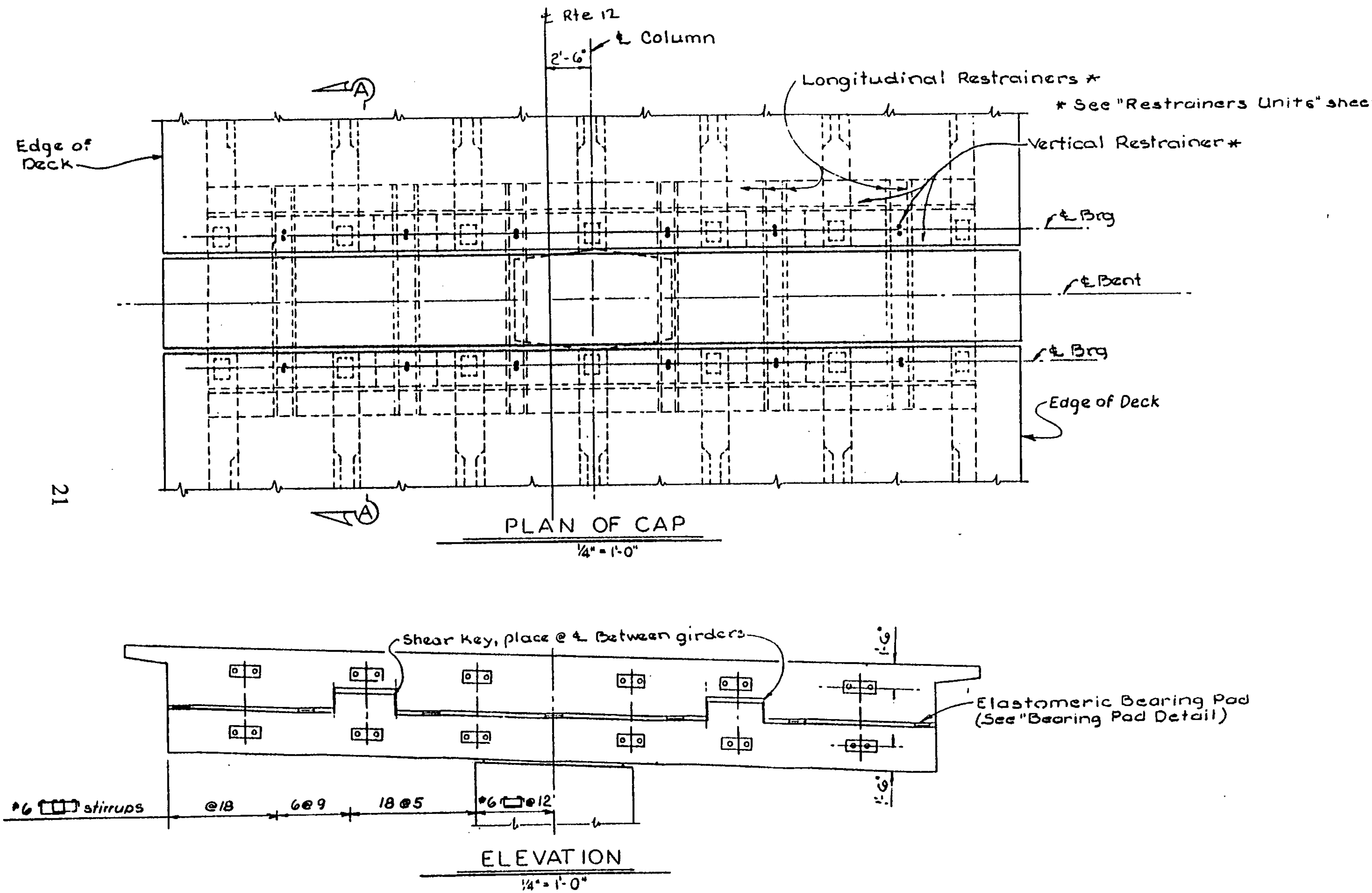


FIGURE 2-12: Precast Concrete I-Girder Raised Shear Key Details

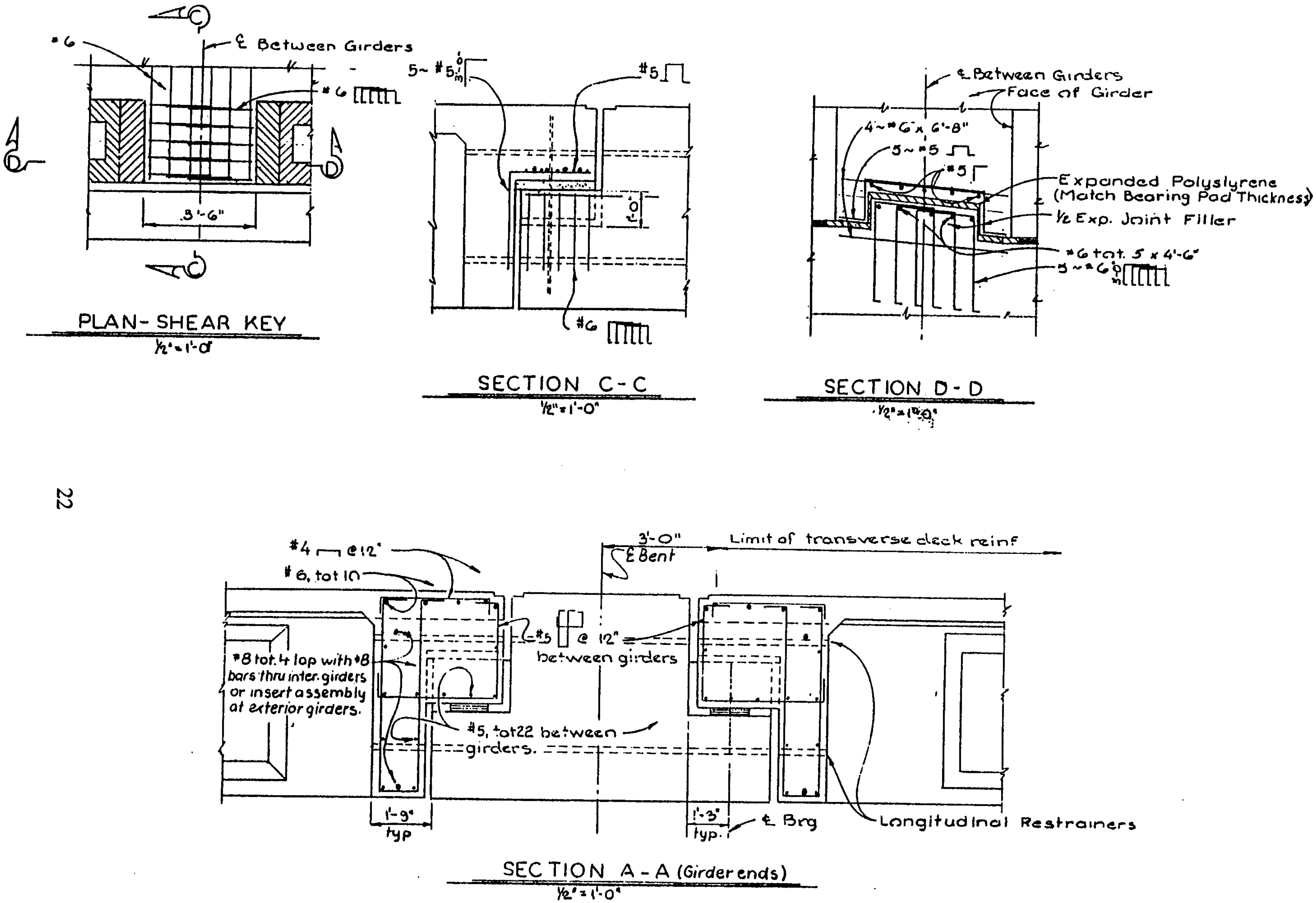


FIGURE 2-12: Precast Concrete I-Girder Raised Shear Key Details (cont.)

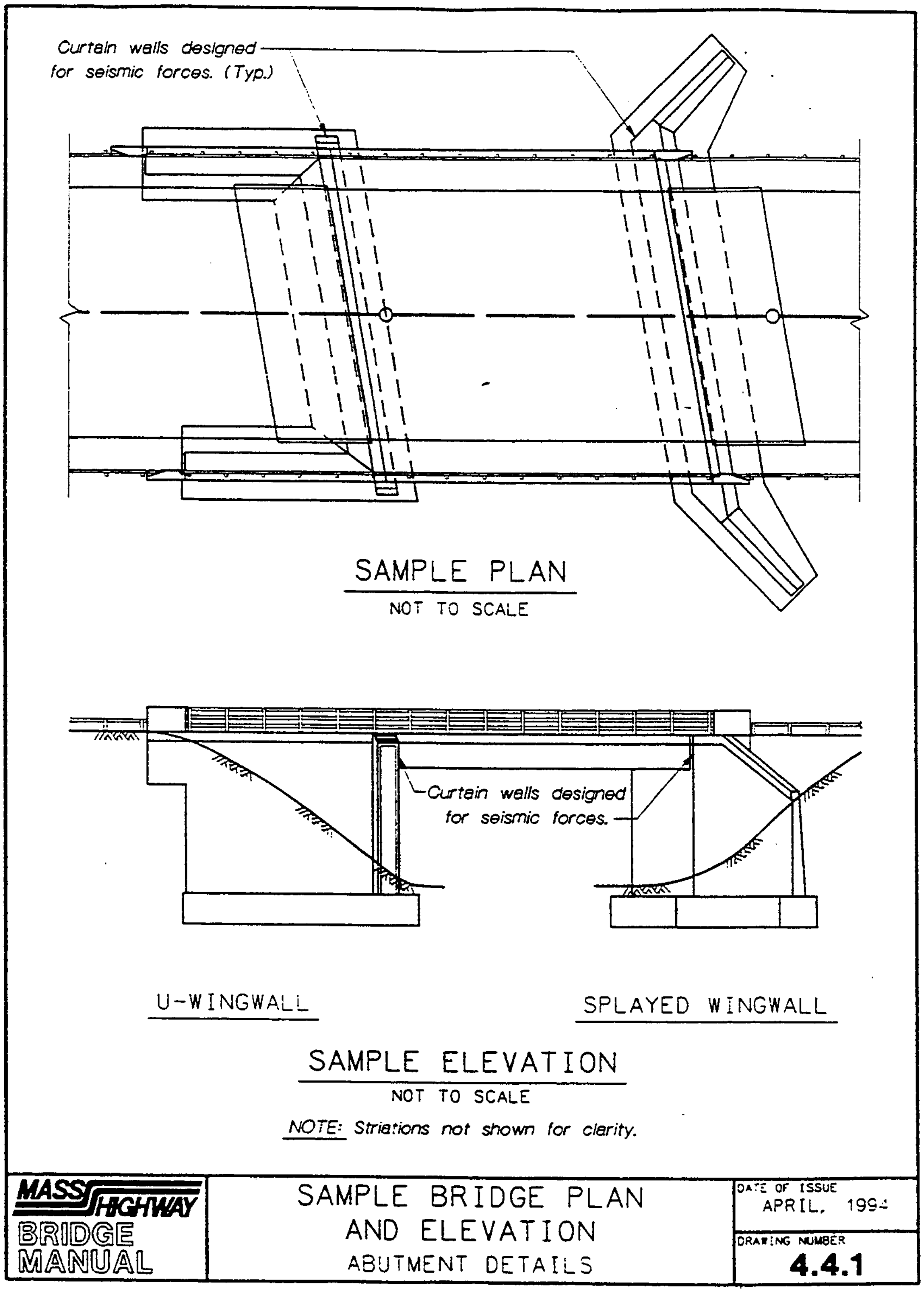
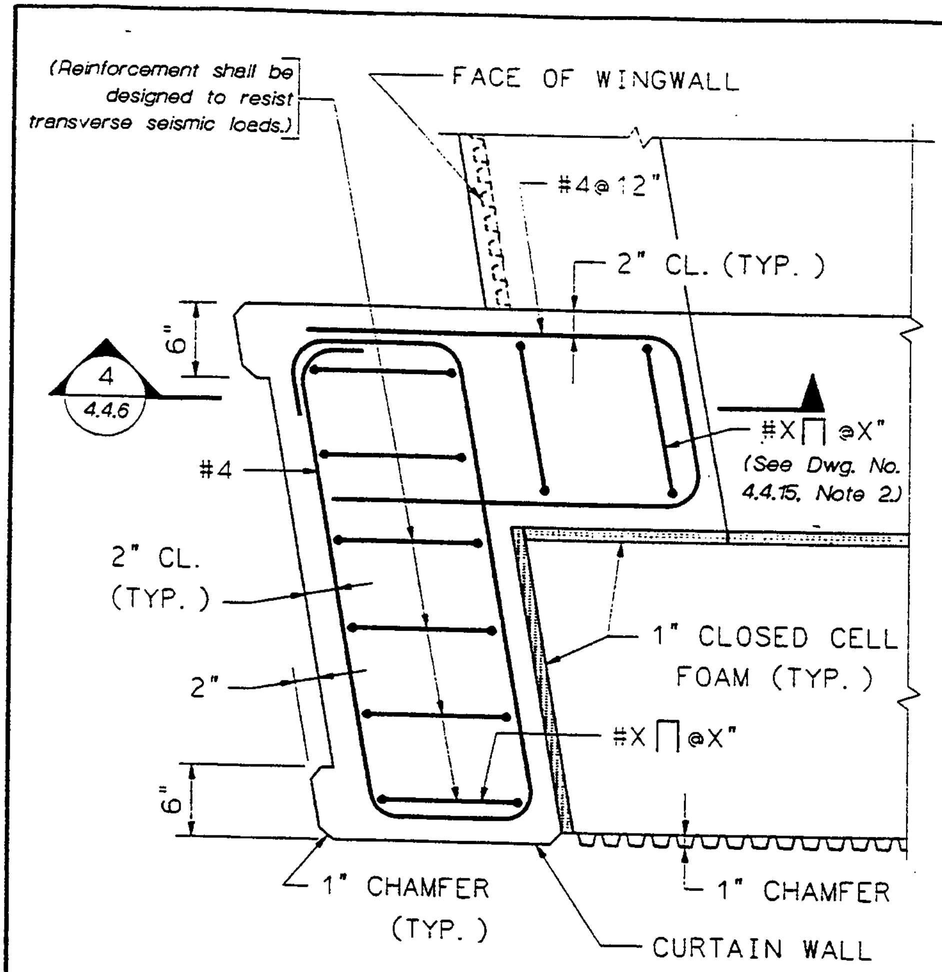


FIGURE 2-13: Abutment Curtain Walls Designed for Seismic Forces; (a) Overview



NOTE:

ATTACH CLOSED CELL FOAM TO THE BACK AND SIDE OF THE EXTERIOR PRECAST BEAM PRIOR TO PLACING THE CONCRETE FOR THE BACKWALL AND CURTAIN WALL.

SECTION 1

SCALE: 1"=1'-0"

MASS HIGHWAY BRIDGE MANUAL	PLAN SECTION @ ABUTMENT END U-WINGWALLS ABUTMENT DETAILS	DATE OF ISSUE APRIL, 1994 / DRAWING NUMBER 4.4.5

FIGURE 2-13: Abutment Curtain Walls Designed for Seismic Forces; (b) Details

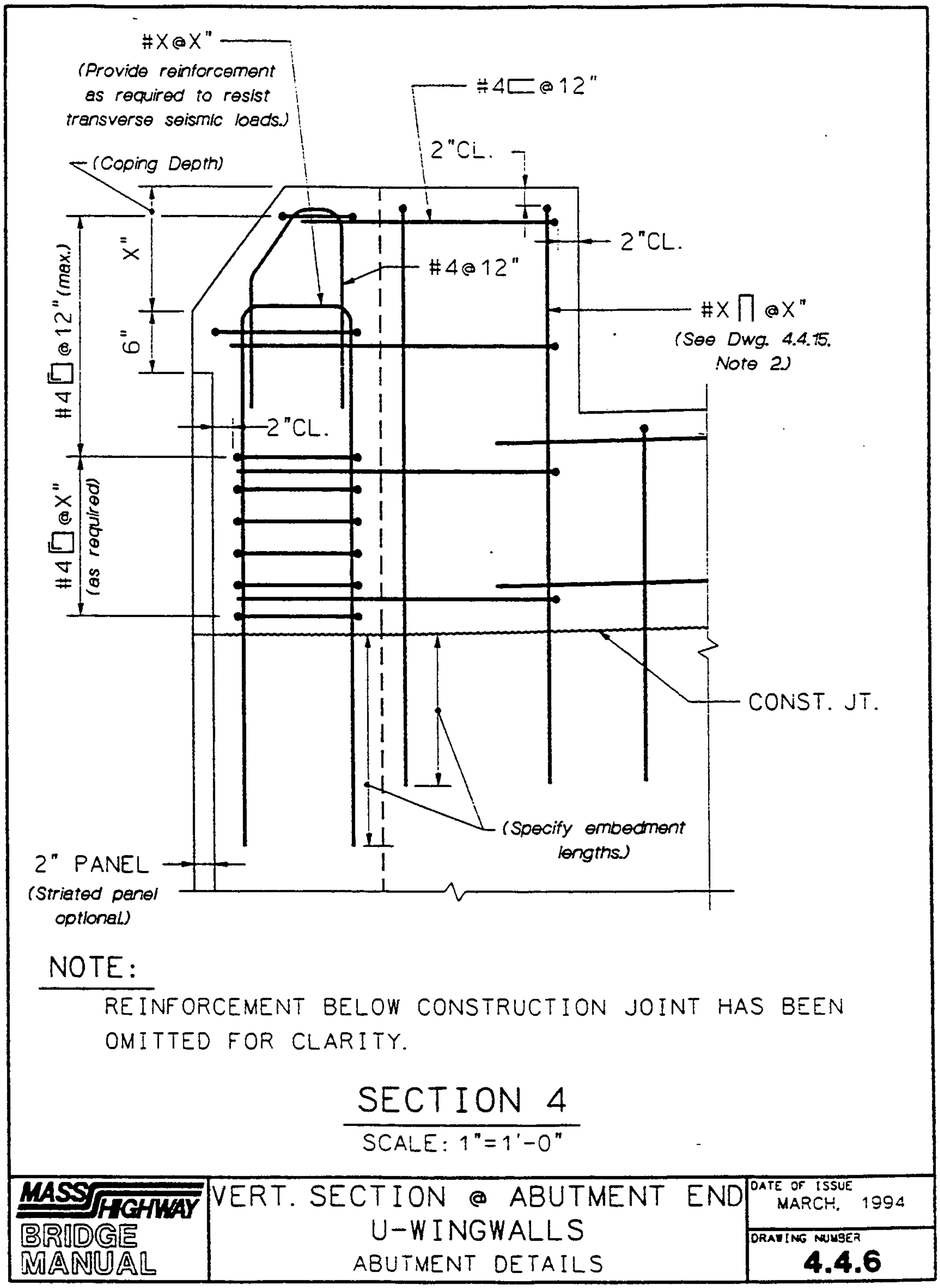


FIGURE 2-13: Abutment Curtain Walls Designed for Seismic Forces; (c) Details

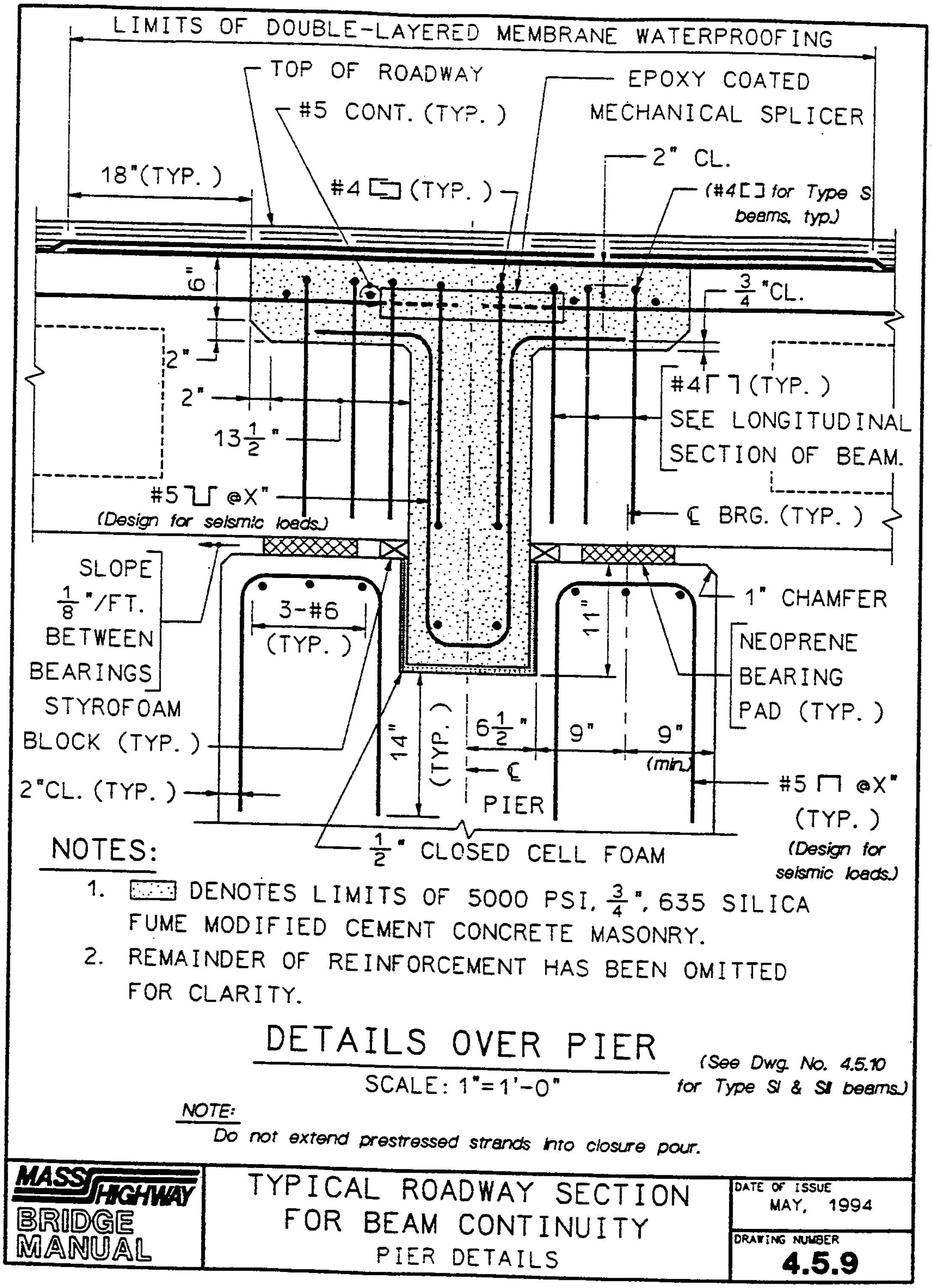


FIGURE 2-13: Abutment Curtain Walls Designed for Seismic Forces; (d) Typical Roadway Section for Beam Continuity

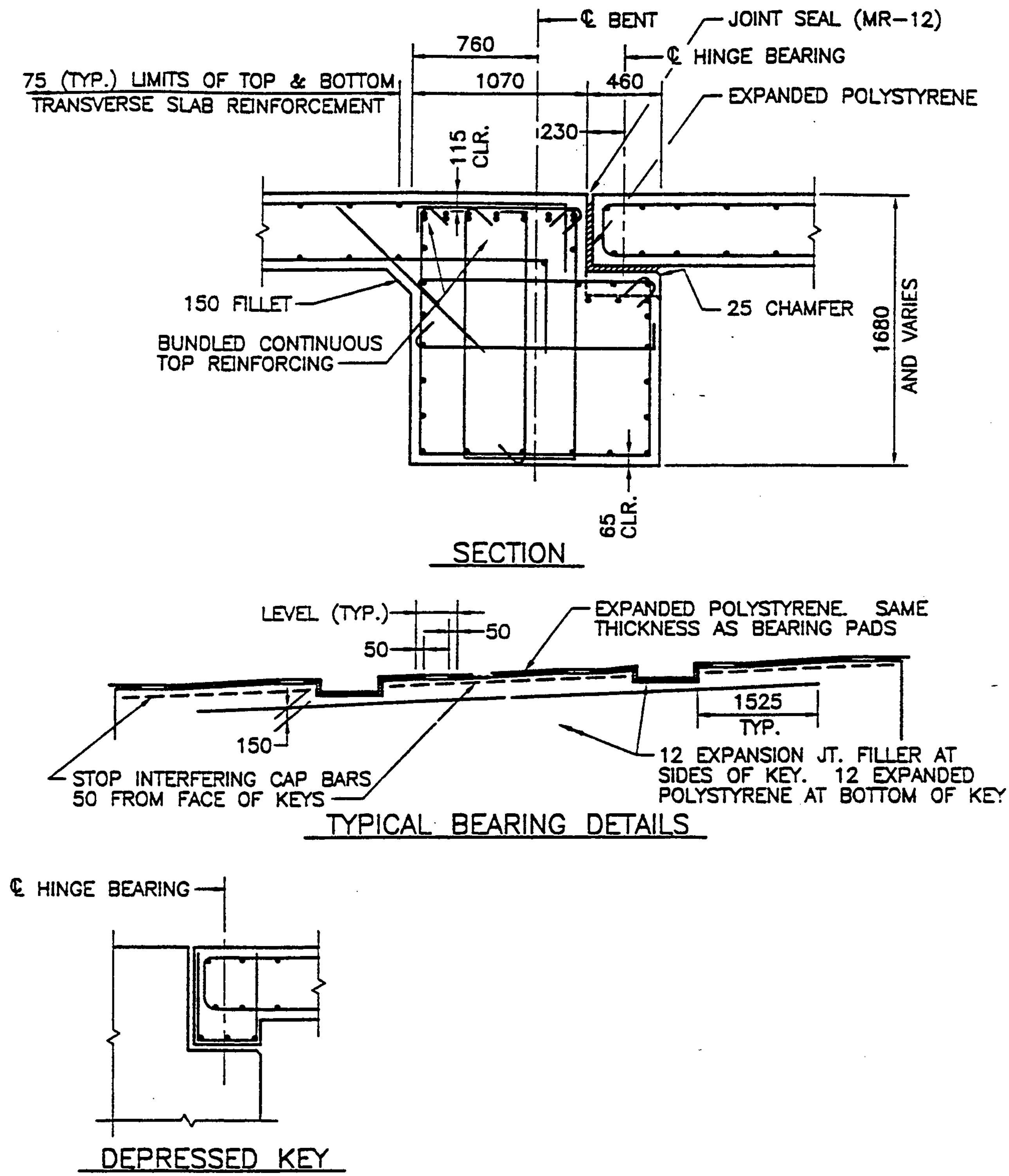


FIGURE 2-14: Cast-In-Place Slab Depressed Shear Key Details

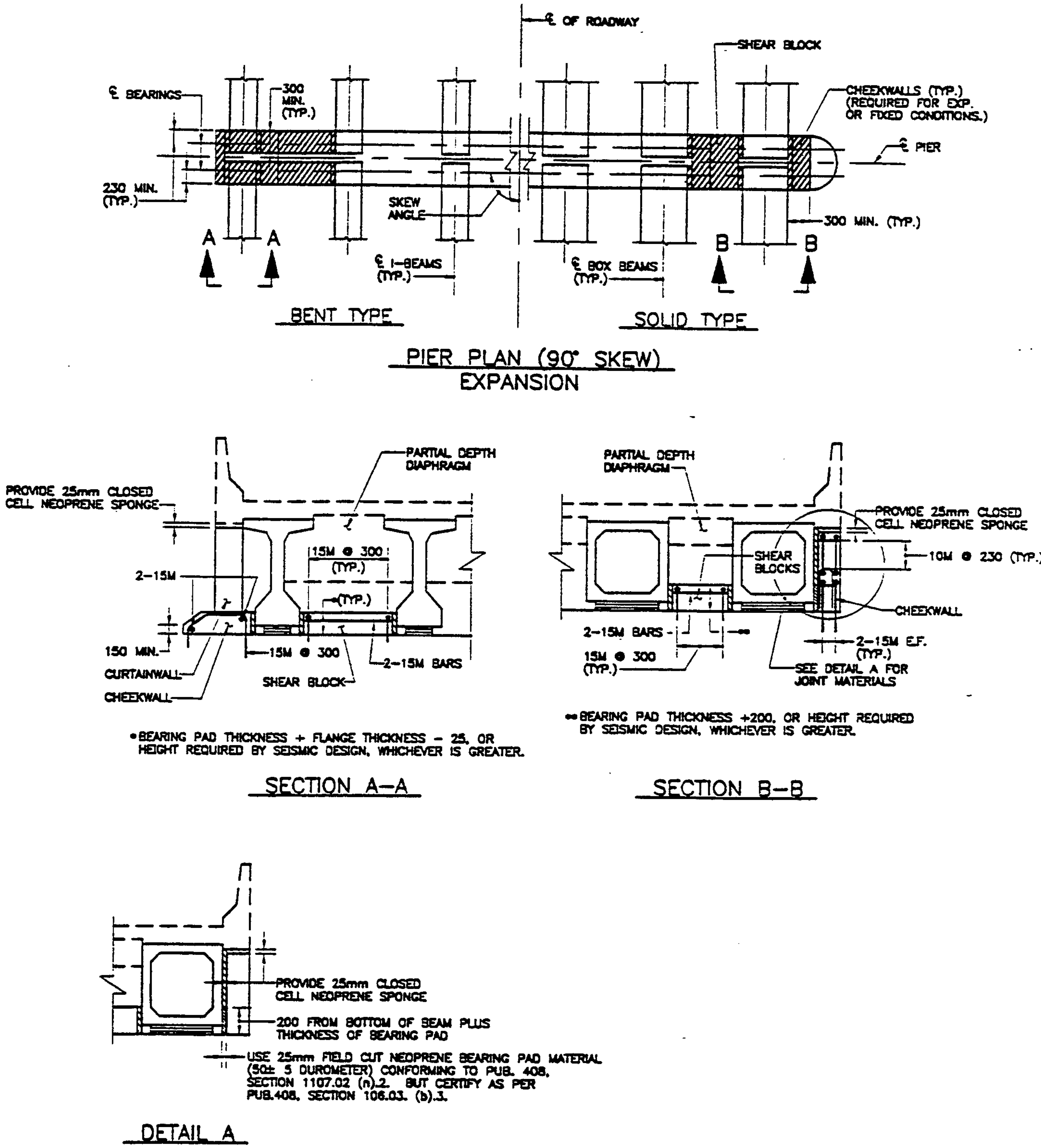


FIGURE 2-15: Cast-In-Place Shear Keys

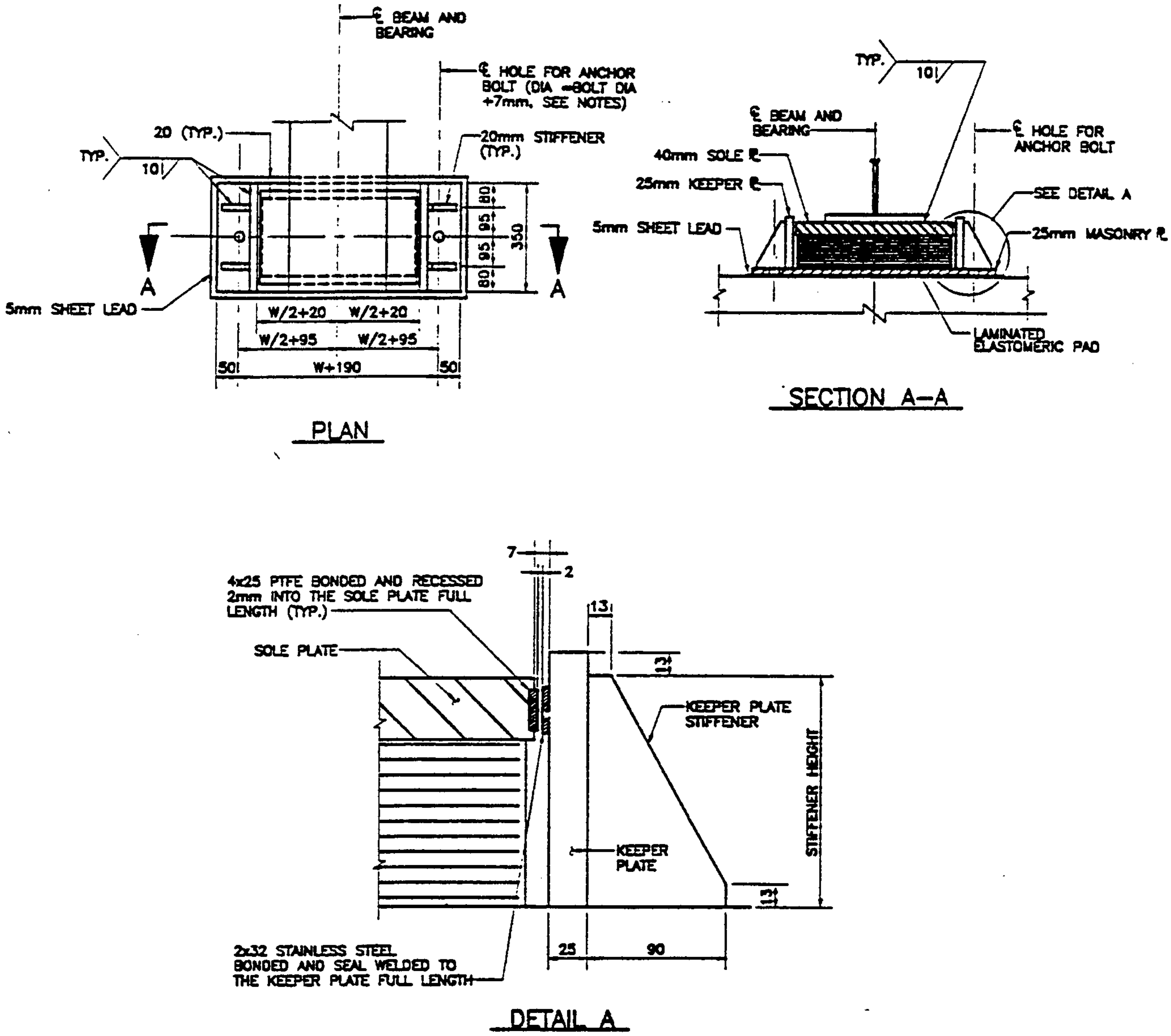


FIGURE 2-16: Shear Brackets at Expansion Bearings

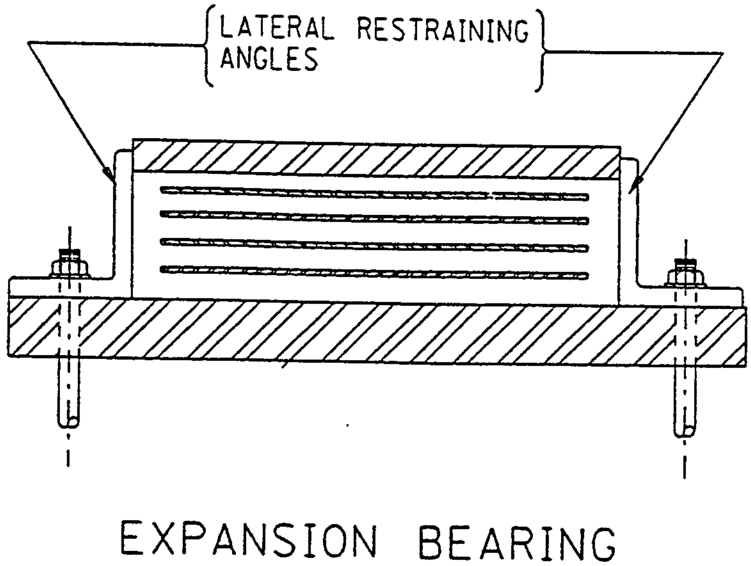


FIGURE 2-17: Lateral Restraining Angles

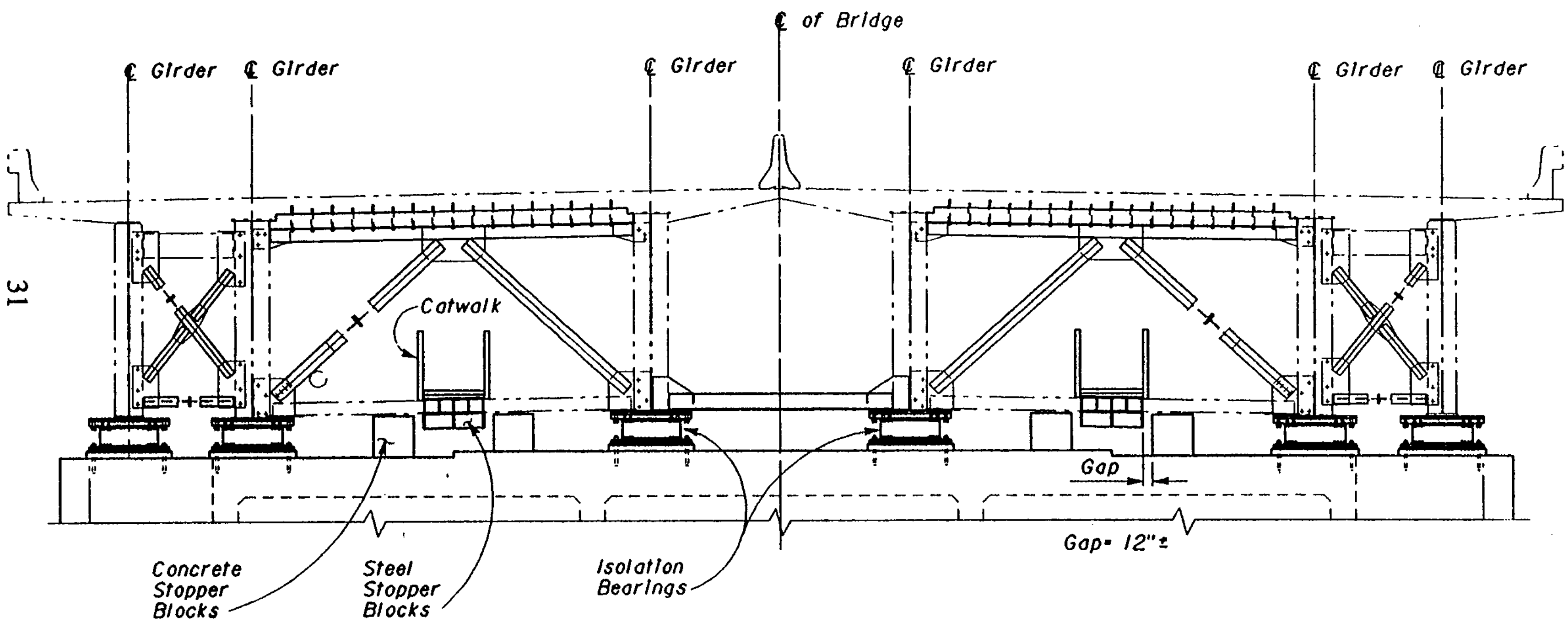


FIGURE 2-18: Use of Stopper Blocks in Seismic Isolation

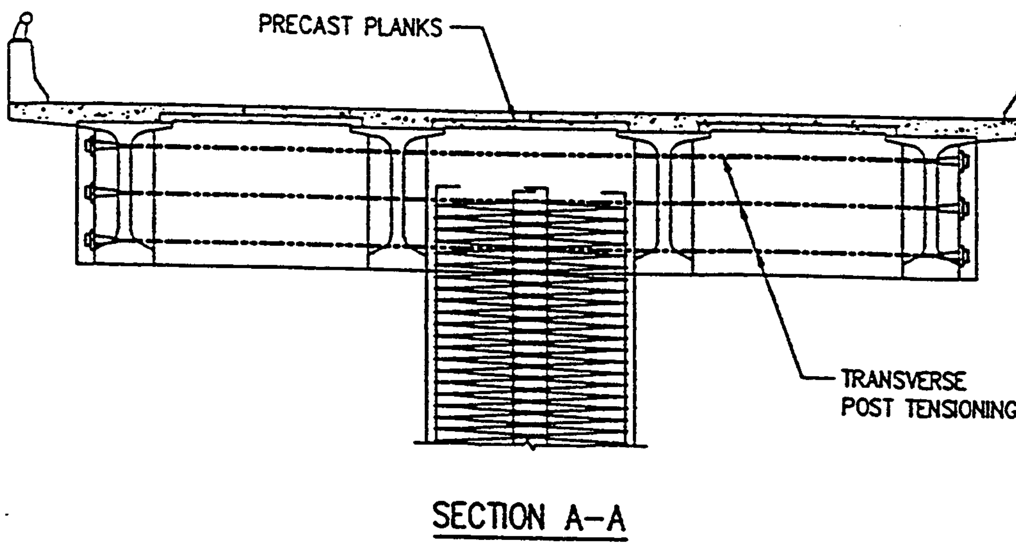


FIGURE 2-19: Integral Pier Cap Details

SECTION 3

SACRIFICIAL ELEMENTS

3.1 Abutments

Caltrans' design philosophy takes advantage of the energy dissipating capabilities of the short height and high cantilever seat type abutments. The longitudinal earthquake force required to mobilize the backfill for the full height of the abutment is generally much larger than a practical size backwall can be designed to resist. Therefore, the backwall is designed to fail before forces can be transmitted to the substructure portion of the abutment (Caltrans, 1994b). Adjacent bents are required to take additional load due to the reduction in abutment stiffness.

A disadvantage resulting from the failure of an abutment backwall due to a major seismic event would be closing the bridge to traffic while repairs are completed. On the positive side, little or no damage is expected during smaller seismic events, and smaller, less expensive expansion joint assemblies can be used.

Some example details of seat type abutment details are provided in Figures 3-1 and 3-2.

An alternative to the typical Caltrans seat type abutment backwall are knock-off devices which utilize a preformed joint located at the top of the abutment backwall. The main advantage of having the joint near the top of the superstructure, as opposed to the bottom of the soffit, is any damage that may occur during a seismic event is readily detectable and repairable, thus reducing the amount of time the bridge is closed to traffic. On the negative side, the damage will occur during smaller moderate seismic events, or large, expensive expansion joint assemblies must be used. More movement usually results in greater column ductilities and damage, or the installation of expensive slide bearings at bents. Some example details of knock-off devices are provided in Figures 3-3 and 3-4.

3.2 Joints

An alternative to preventing abutment backwall damage in bridges is to provide a sacrificial joint element. Seismically isolated bridges require large movements that alter the structural period, thereby reducing demands on the substructure. Design of deck expansion joints to accommodate these movements, especially in high seismic zones, is difficult at best. One problem in designing deck joints for seismic movements is providing for motion in the transverse direction of the bridge. Most expansion joint systems, such as traditional modular assemblies or finger joints are manufactured for longitudinal movement only. Any transverse movement, usually required when seismic isolation bearings are used, will most likely result in joint assembly failure.

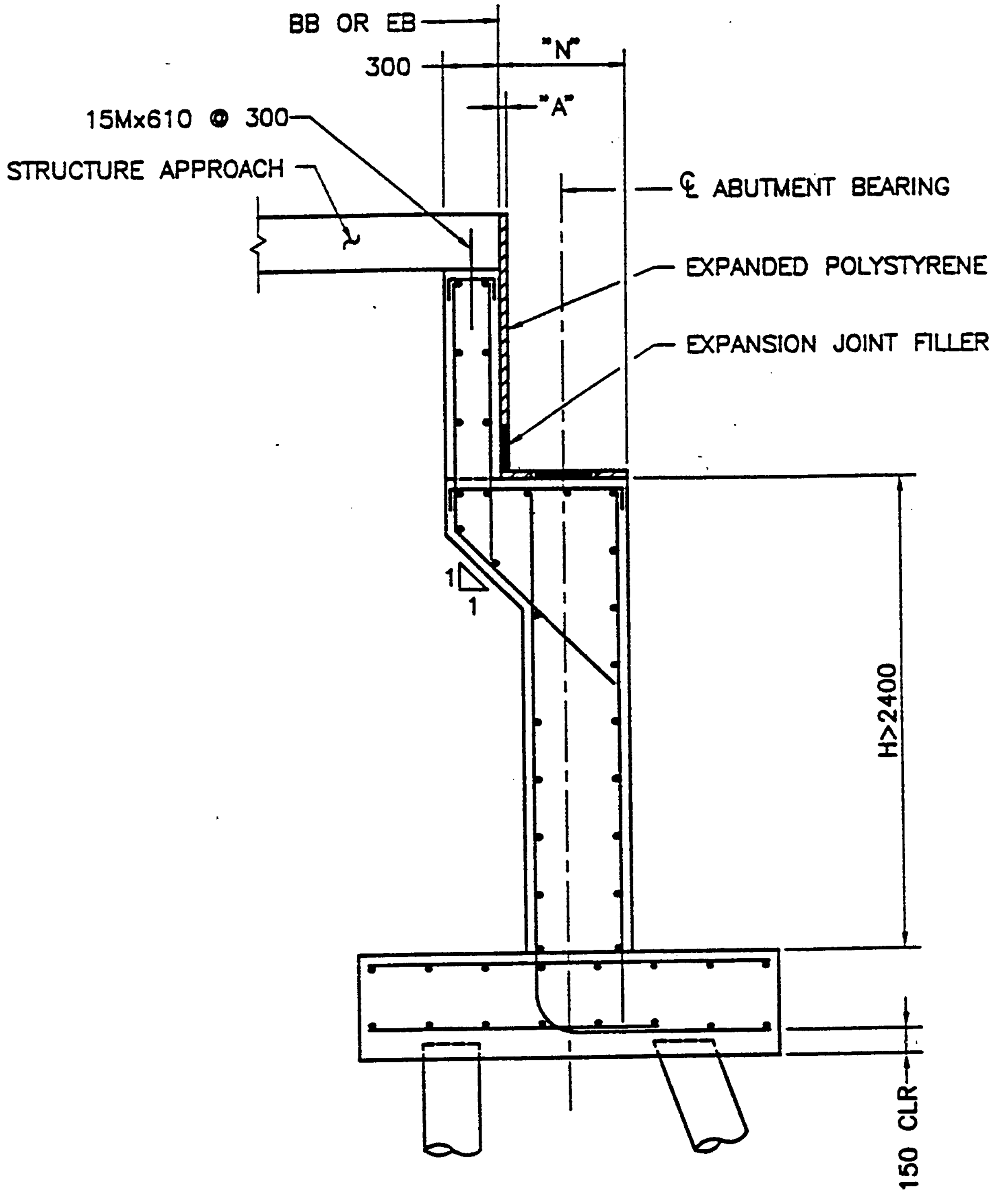
A bridge constructed Walnut Creek, California, in 1991 required an expansion joint as the superstructure changed from a continuous built up plate girder to a concrete box girder (see Figure 3-5). To facilitate the 9" longitudinal seismic movement, concrete breakaway detail with a 9" void covered by steel plate formed a unique joint assembly. A modular unit was installed to handle the 4" movement rating due to thermal loading in conjunction with this voided area that would allow free movement of the isolated steel structure.

The Golden Gate Bridge retrofit project is incorporating large sacrificial joints to accommodate both longitudinal and transverse displacements at the abutments of the North Viaduct, a steel deck truss structure. In addition, the Benicia-Martinez retrofit project has a similar detail at the abutments of the north and south approaches, which are simply supported steel I-girder spans. For this design, shear keys are used to prevent movement in the transverse direction. The seismic performance goal for both projects was to prevent collapse and provide serviceability after a major seismic event. Following a major event, it is likely that the bridge will have to be closed for a short time to allow repairs to the joint element. Although these details are being used to retrofit bridges, they could be extended to new bridge designs. These retrofits utilize isolation bearings between the substructure and superstructure. In order to accommodate the large longitudinal movements expected (±12 to 15 inches), a sacrificial joint is used at the abutments. A sample detail of those joints is shown in Figures 3-6 and 3-7. An alternative detail using a swivel joint, which is intended to remain serviceable after an earthquake, was also investigated as an alternative for the North Viaduct. This joint was designed for both longitudinal and transverse displacements. See Figure 3-8 for an example of a swivel joint developed by the D. S. Brown Company.

The Japanese have designed joints for both longitudinal and transverse movement (Kemishima, 1992). An expansion joint developed for a three-span 106 m bridge used a traditional finger joint for longitudinal movement (see Figure 3-9). The joint was supported by two steel "I" beams placed perpendicular to the bridge centerline. Between the two steel beams was a guide block to prevent uplift and any longitudinal movement of the joint due to live load impact. Teflon sheets were placed over the support beams to provide a "sliding" surface. The joint performed well in laboratory tests, but concerns were raised over noise due to live load impact and development of adequate drainage equipment.

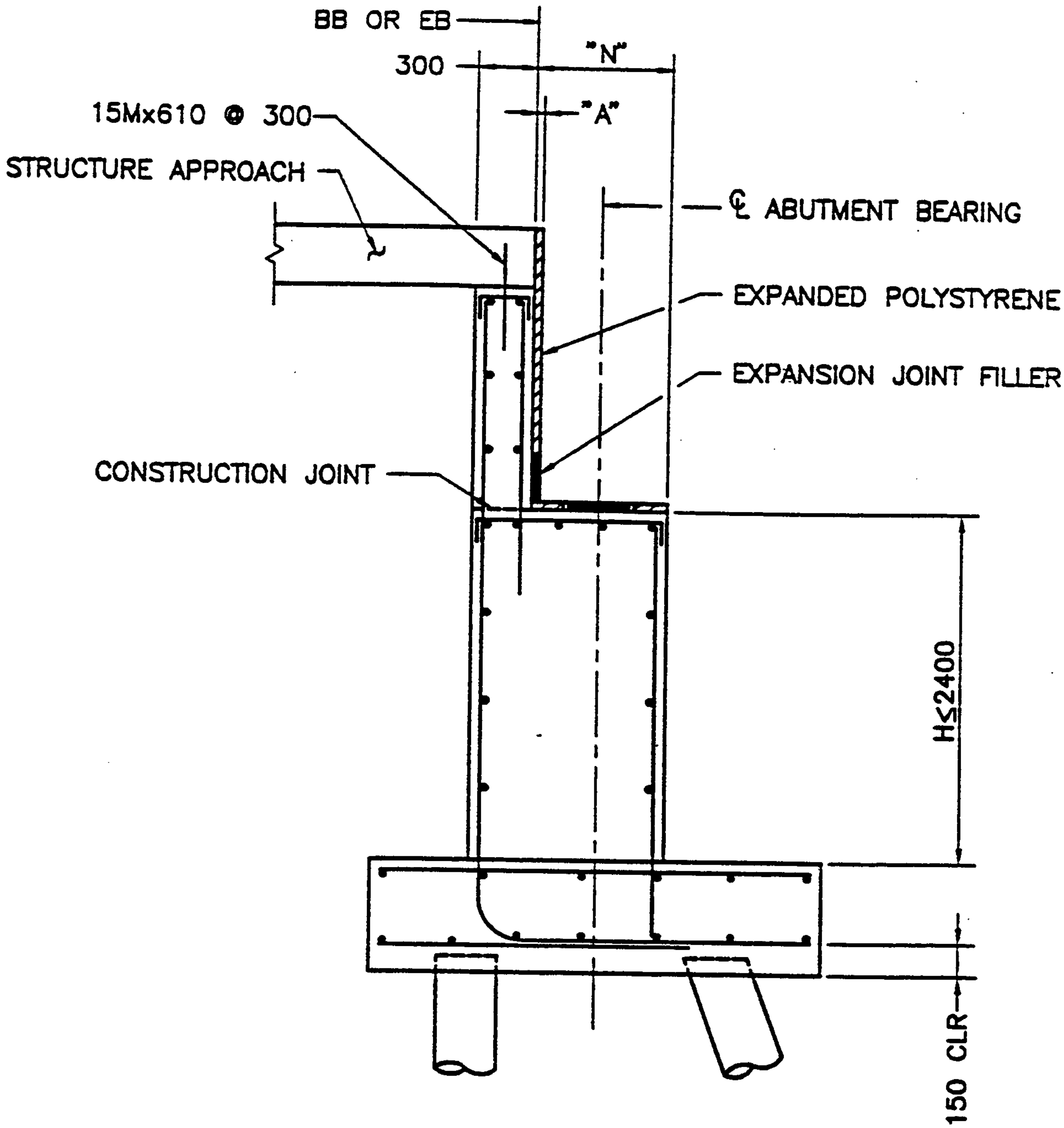
Several bridges were reconstructed after the Northridge earthquake in 1994. Two such bridges were the North and South Connector Ramps at the 14/5 Interchange (Roberts, 1995). The new North Connector Ramp is 1532' long with columns ranging from 25' to 75' in length so it was not possible to eliminate all the deck hinge and expansion joints. However, a new detail was developed for these hinges which is shown in Figure 3-10. The hinge joint is centered between two columns approximately 40' apart and the two deck elements cantilever from the adjacent bents. Since neither side supports the other, there is no risk of a deck collapse, even in a major seismic event with large ground movement. The joint may separate, but a steel plate can be placed over that joint and the bridge can remain in service during repair. This detail has been utilized extensively in this interchange on three of the longest connector ramps.

An additional feature of the improved bridge design is shown in Figure 3-11. That is for the abutments at the ends of the 1500' long Northbound Route 14 ramp that was constructed without an intermediate deck joint. The large displacements are handled at the abutment by isolating the superstructure and supporting the end span on 6' diameter pile shaft, placed adjacent and in front of the abutment. This detail allows for both transverse and longitudinal displacements up to 4' at the abutments. It is expected that the deck joint expansion material will be damaged when such large movements occur. The resulting damage will probably cause closure of the bridge until a temporary plate can be installed over the gap. This type of damage repair can be completed within hours of the event. Traffic will remain in service while the permanent repairs are being completed.



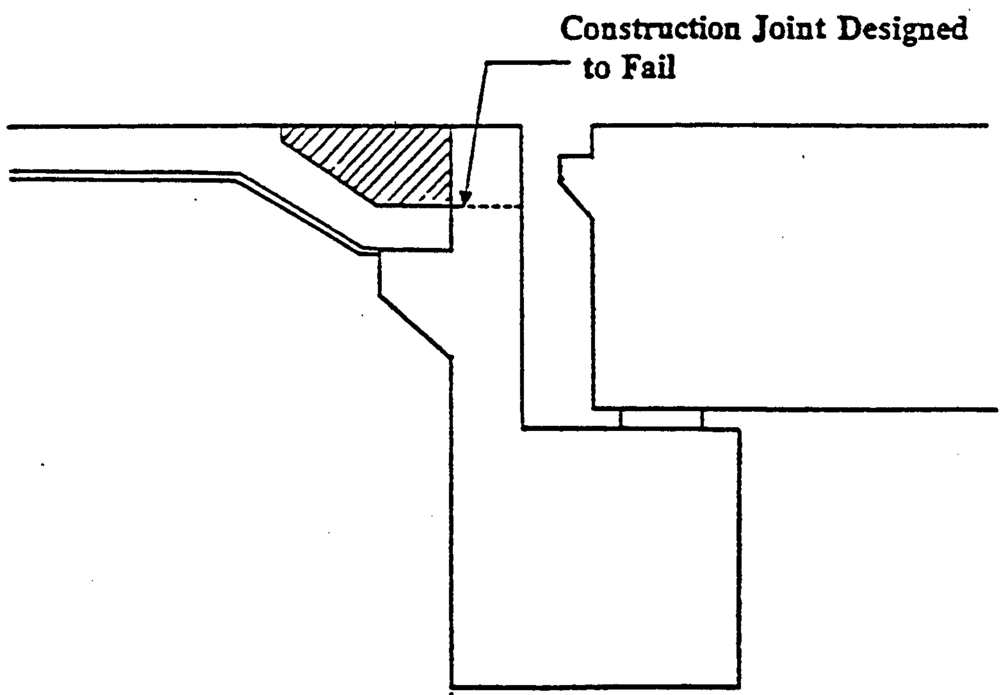
Note: Vertical ties in footing not shown.

FIGURE 3-1: Reinforced Concrete Seat Type "High Cantilever" Abutment



Note: Vertical ties in footing not shown.

FIGURE 3-2: Reinforced Concrete Seat Type "Short Height" Abutment



Top of Backwall Preformed Joint

(Note: This detail to be used where damage will be easily visible.)

FIGURE 3-3: Abutment Backwall Knock-off Device

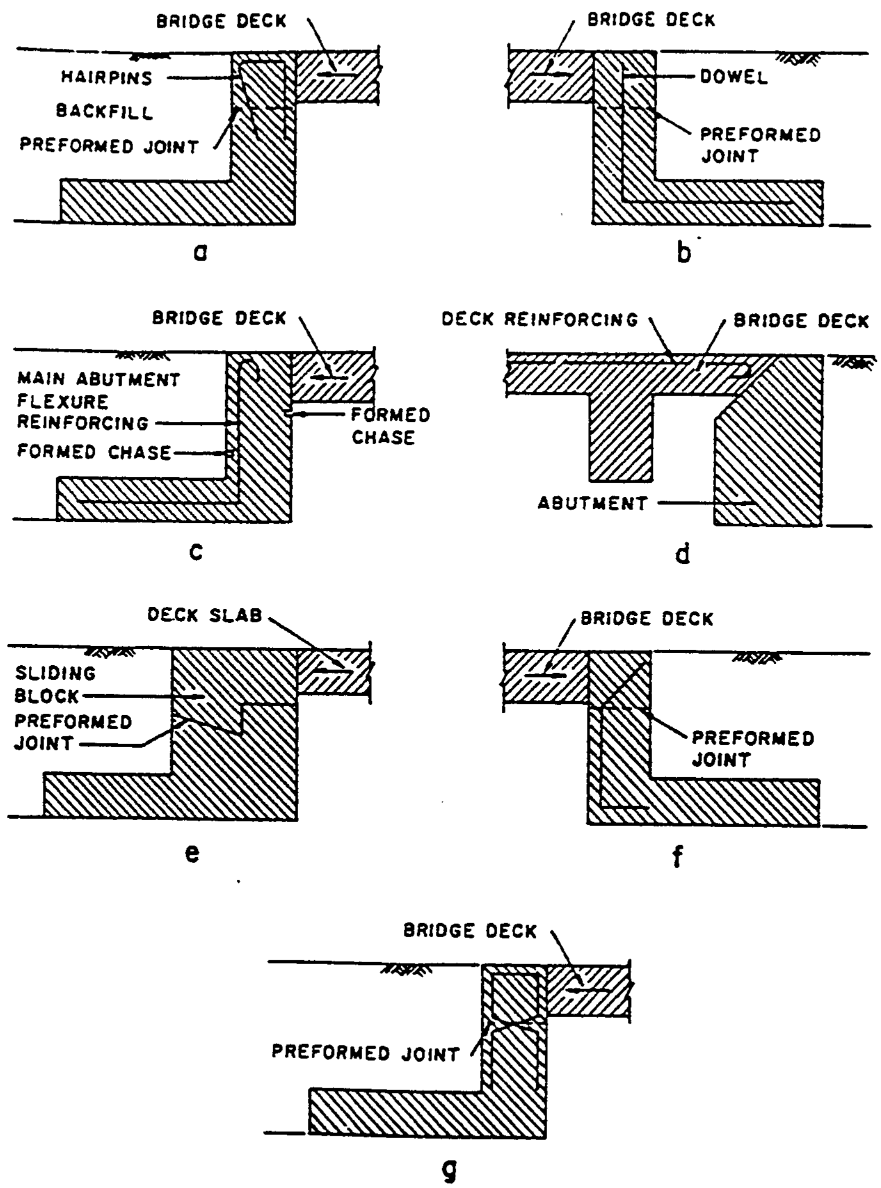
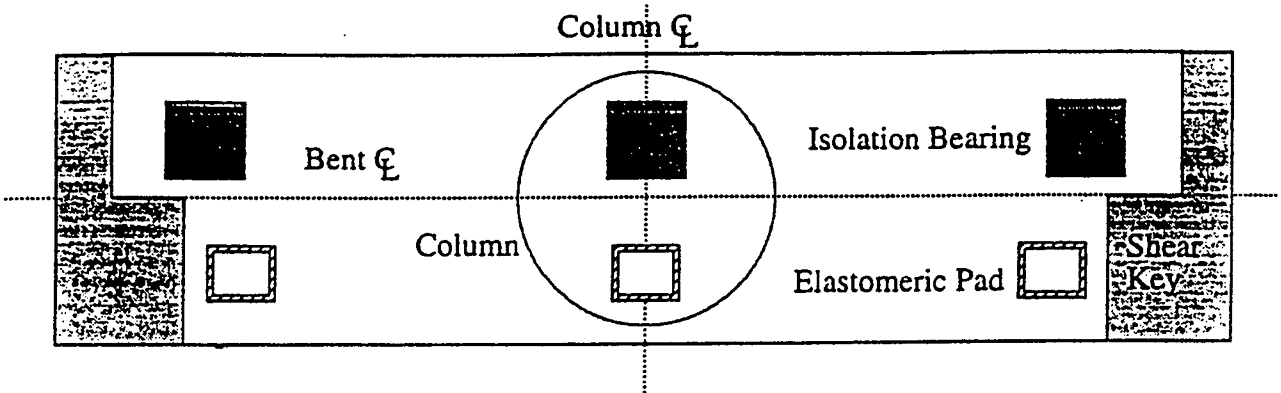
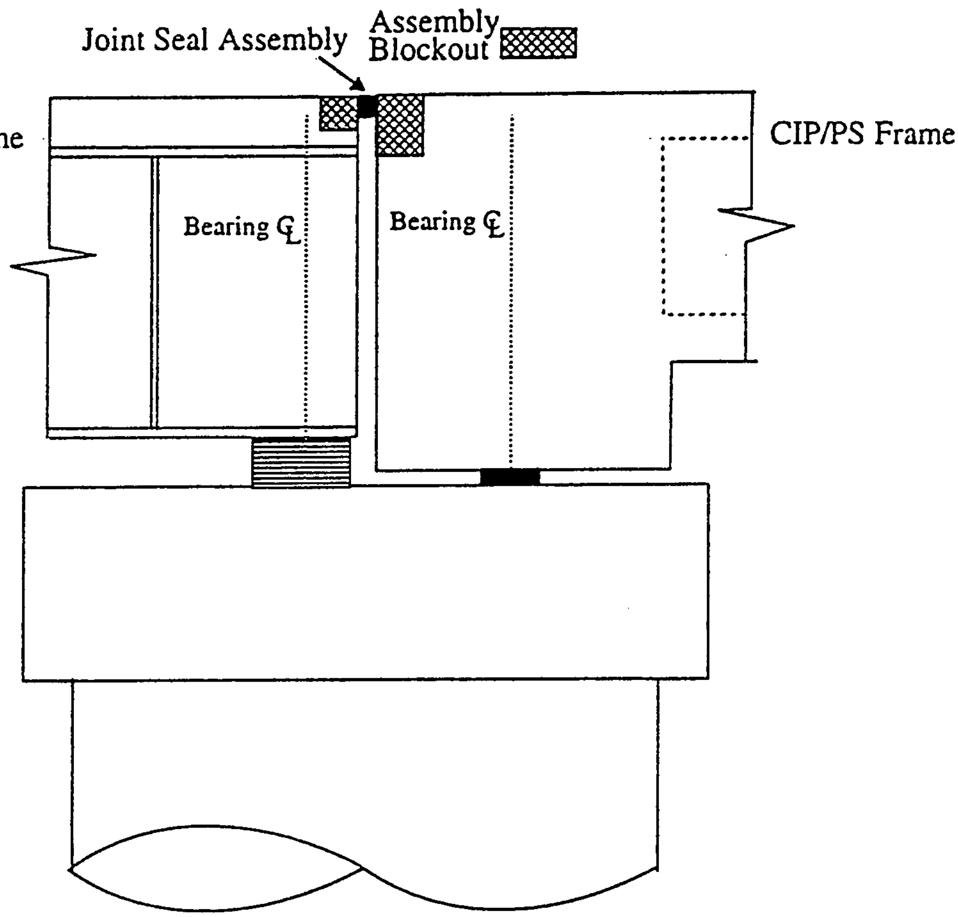


FIGURE 3-4: Abutment Backwall Knock-off Devices



(a) Plan



(b) Longitudinal View

(No Scale)

FIGURE 3-5: Expansion Joint Details

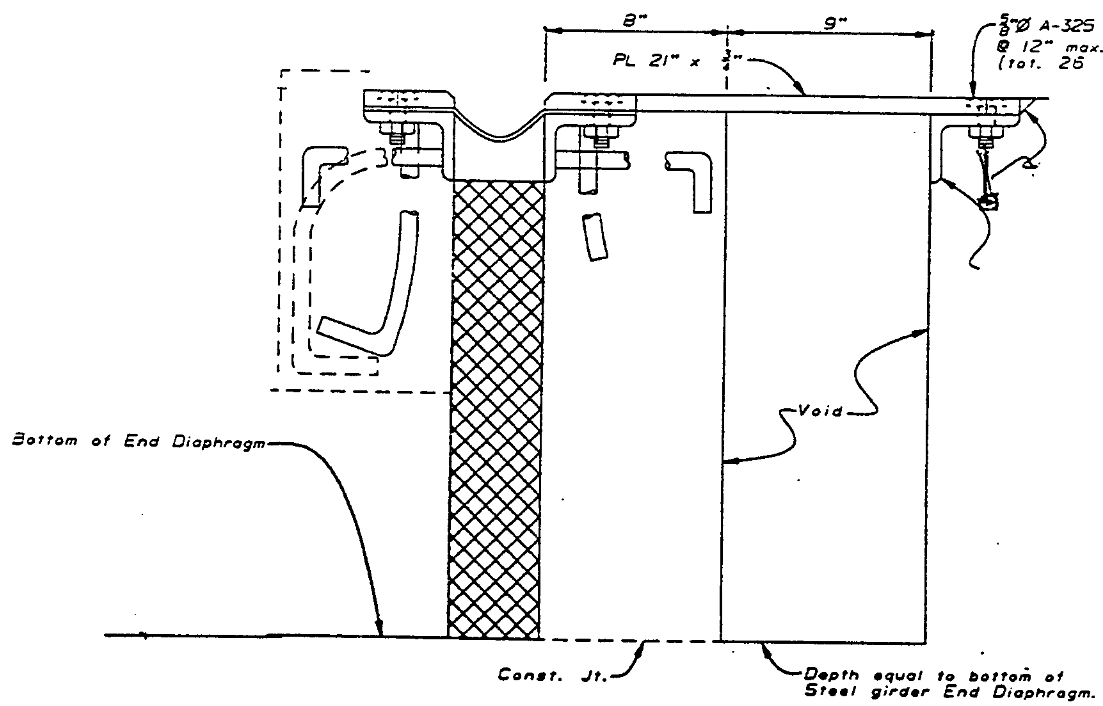


FIGURE 3-5: Expansion Joint Details (cont.)

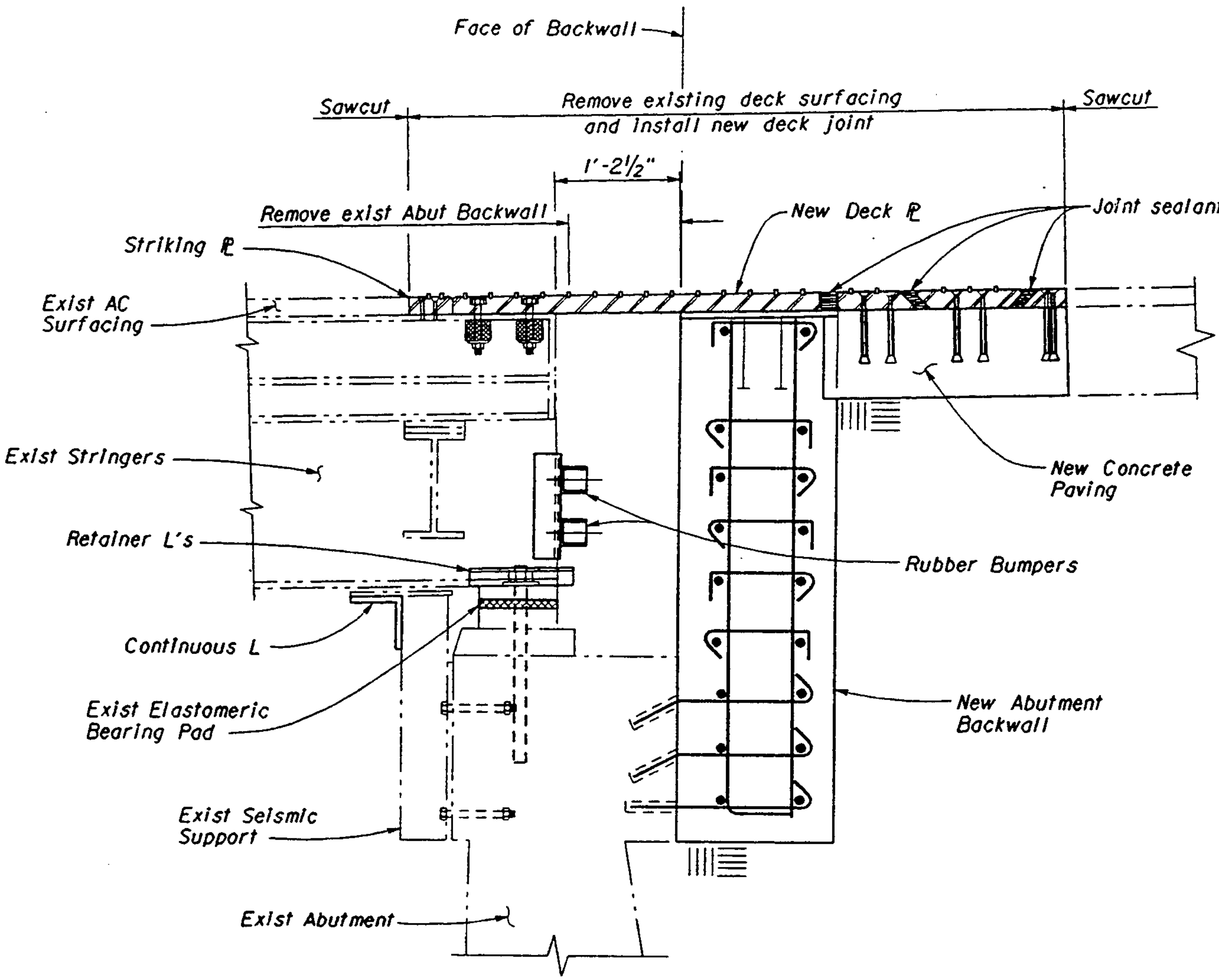


FIGURE 3-6: Sacrificial Expansion Joint at an Abutment – Steel (North Viaduct – Golden Gate Bridge)

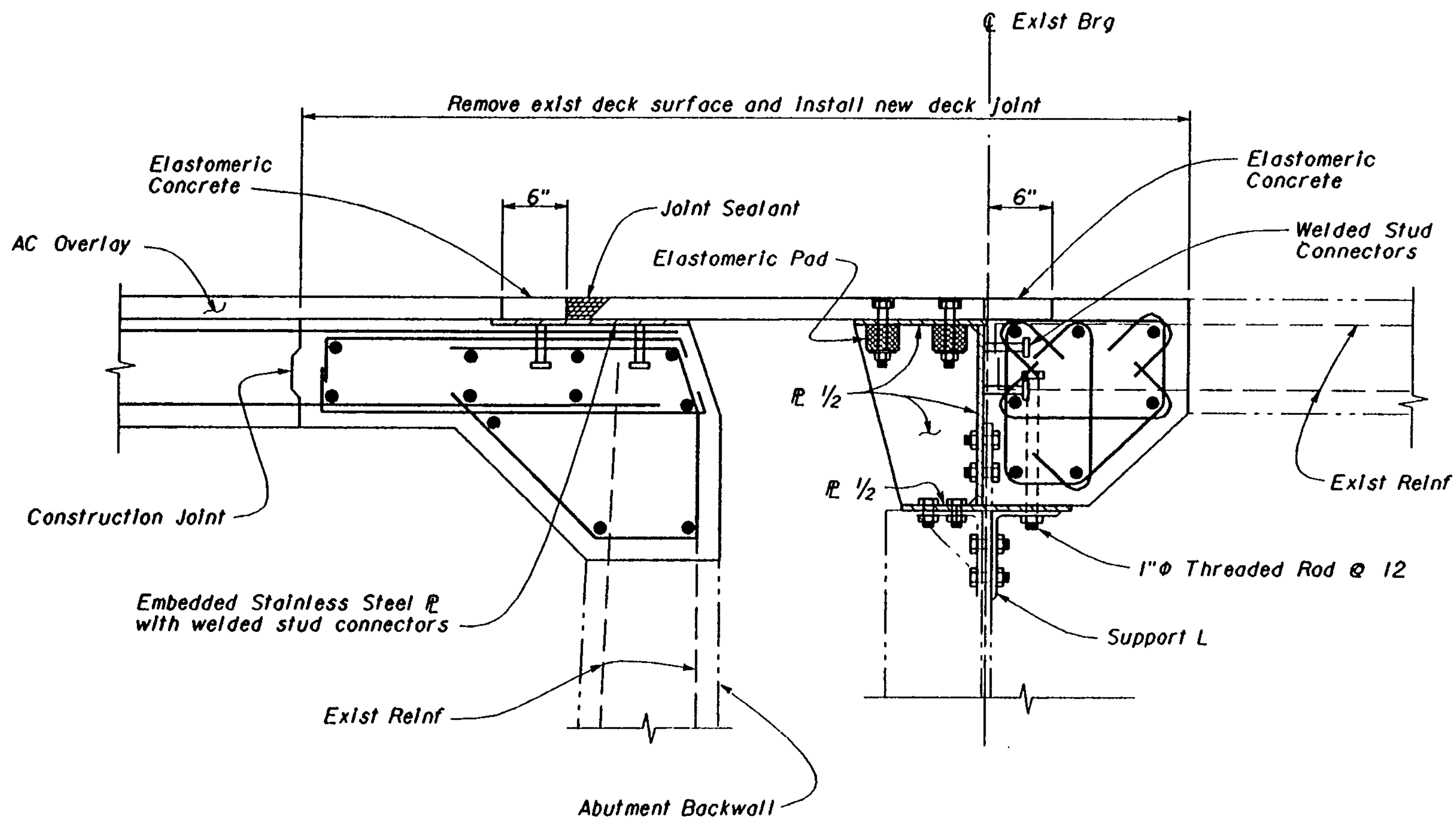


FIGURE 3-7: Sacrificial Expansion Joint at an Abutment – Steel (Benicia-Martinez Approach Spans)

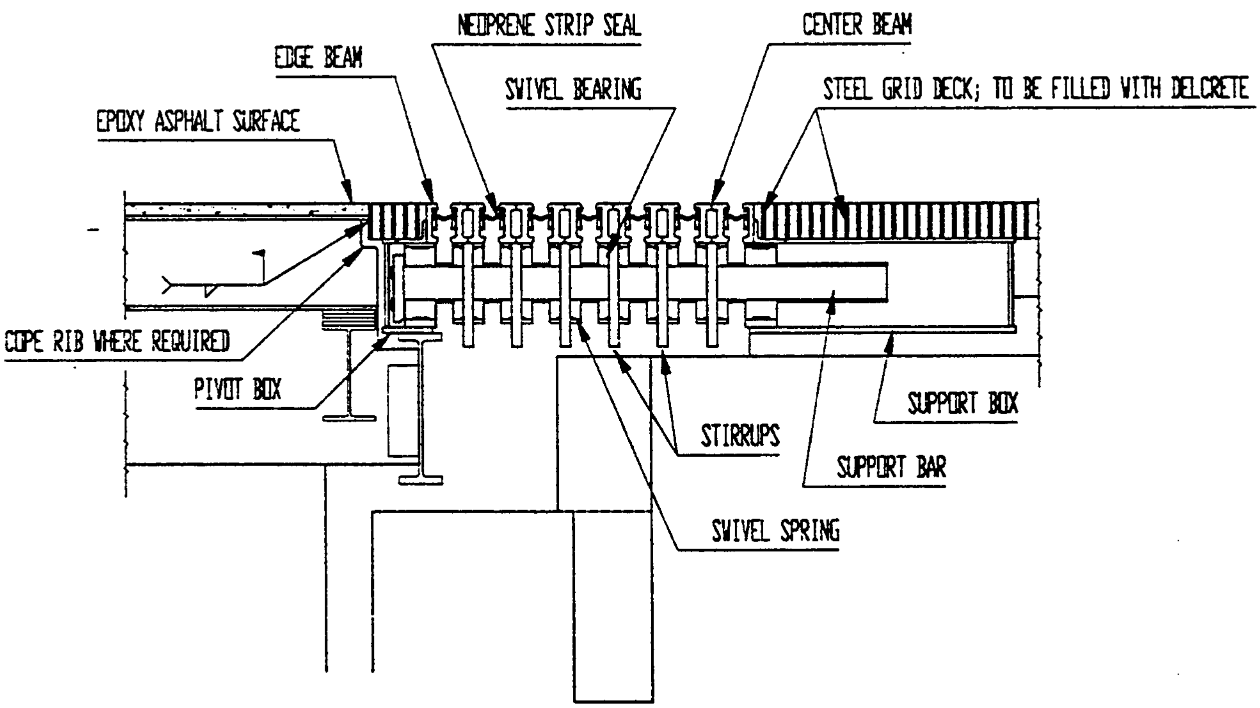


FIGURE 3-8: Serviceable Expansion Joint at an Abutment – Swivel (An Alternative design for the North Viaduct – Golden Gate Bridge)

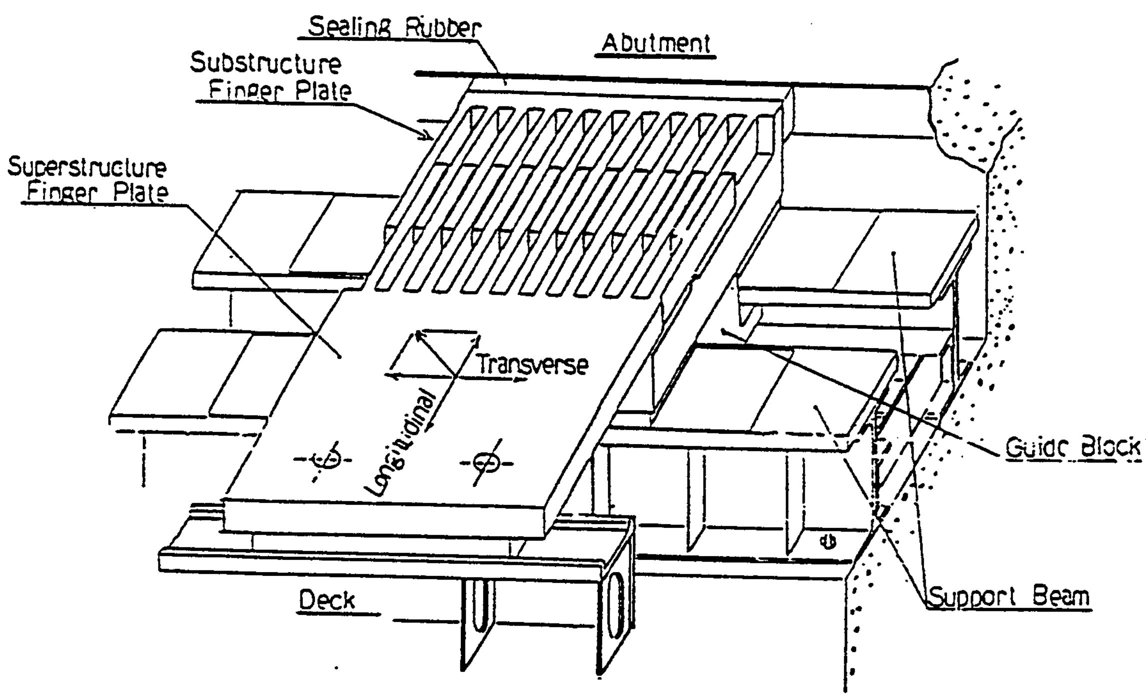


FIGURE 3-9: Structural Outline (Showing 1 Block of Finger Plate)

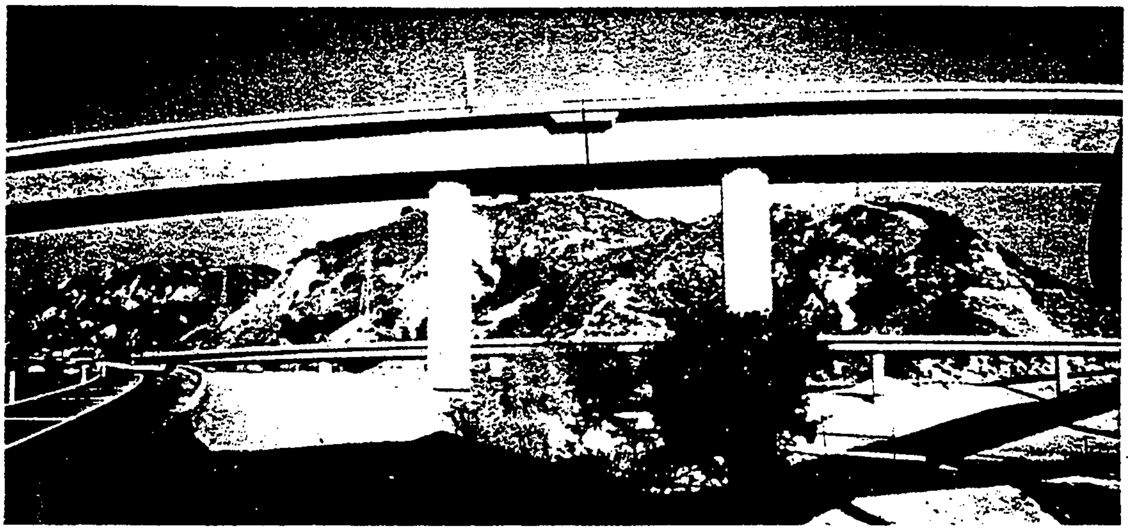


FIGURE 3-10: Hinge Detail on North Connector

FIGURE 3-11: Abutment Detail for Large Displacement

SECTION 4

PASSIVE ENERGY DISSIPATING DEVICES AND ISOLATION BEARING SYSTEMS

Several energy dissipating devices and isolation bearing systems have emerged in the last two decades. They are capable of reducing the movement and the responses of bridges during earthquakes by adding damping and limiting the structure force below damage level. These devices can be classified by their mechanics and their way of dissipating energy such as yielding steel dampers, lead-extrusion dampers, friction dampers, hydraulic dampers, viscoelastic dampers, and isolation bearing systems. Traditionally, two philosophies have been used in the past to seismically design a bridge: strength or ductility. The incorporation of energy dissipating devices is relatively new to bridge construction. Background information will be presented first in Section 4.1 and their application to bridges will be discussed in Section 4.2.

4.1 Classification

4.1.1 Yielding Steel Devices

Yield steel devices improve the behavior of the bridge by increasing it's stiffness, strength and damping. The energy is dissipated mainly through the yielding of the mild-steel which can be designed to yield at a predetermined force level. By forcing these devices to yield the forces going into the structure can be limited below the damage level; therefore, reducing excessive ductility demand in the structure. Several yielding steel devices are shown in Figure 4-1 and their possible uses in bridges are listed in Table 4-1.

TABLE 4-1: Application of Yielding Steel Devices in New Bridges

Yielding Steel Devices	Application in Bridges					
	Span Hinge	Superstructure -Substructure Connection	Column	Tower-Deck Connection*	Superstructure -Abutment Connection	Steel Truss Diagonals
Shear Panel Damper						X
Added Damping and Stiffness (ADAS)	X	X		X	X	
Steel Ring Damper						X
Tapered Steel Ring Damper (T-SRD)						X
Bell Damper	X	X		X	X	
Tapered Column (T-CD)	X	X		X	X	
Honeycomb	X	X		X	X	
Flexural Beam		X				
Torsional Beam			X			
Multi-Directional Crescent Moon-Shaped Damper		X		X	X	
Italian W-Shaped Damper		X		X	X	
Italian E-Shaped Damper		X		X	X	

*Suspension, Cable-Stay Bridges

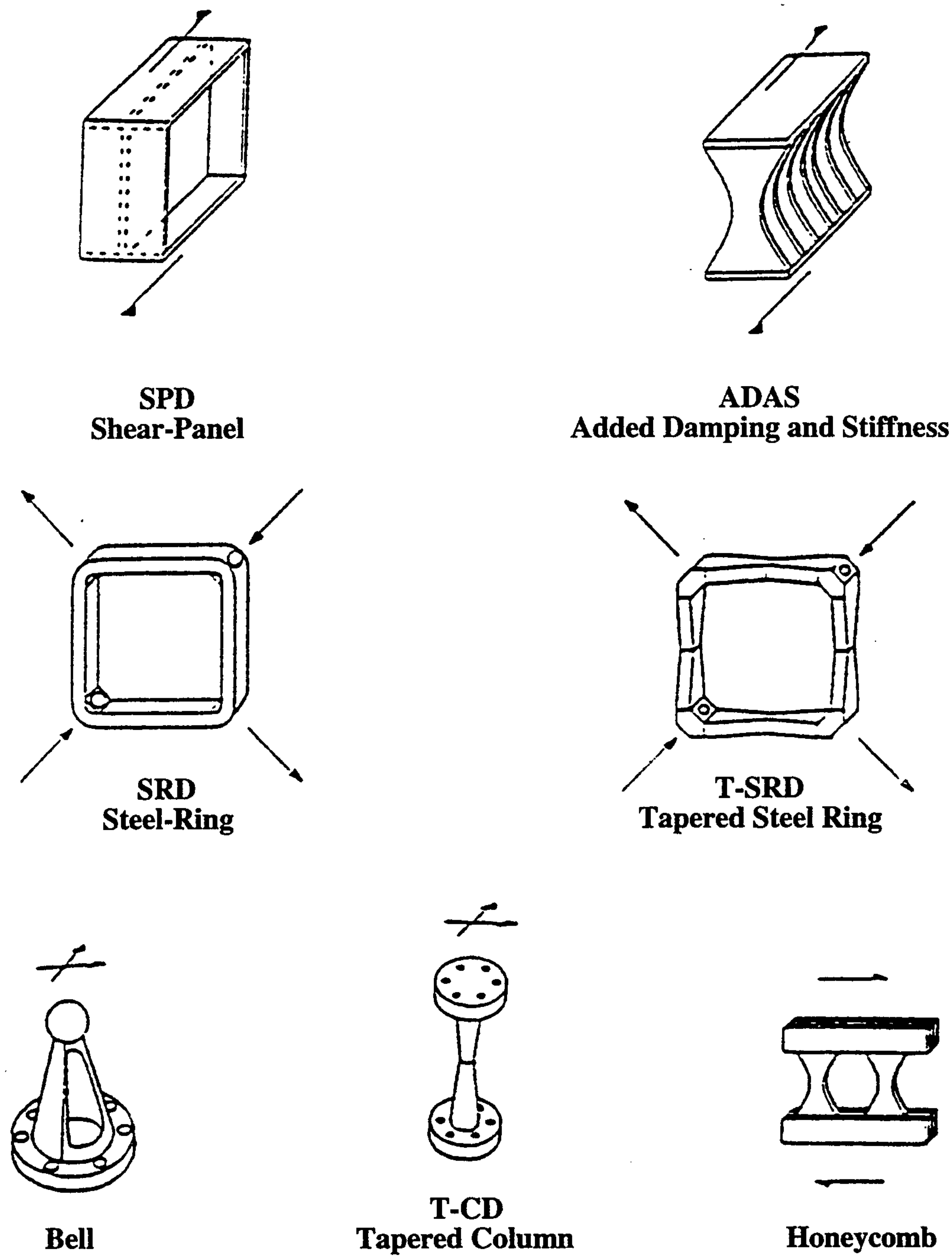


FIGURE 4-1: Yielding Steel Devices; (a) SPD, ADAS, SRD, T-SRD, Bell, T-CD, and Honey Comb (Dorka)

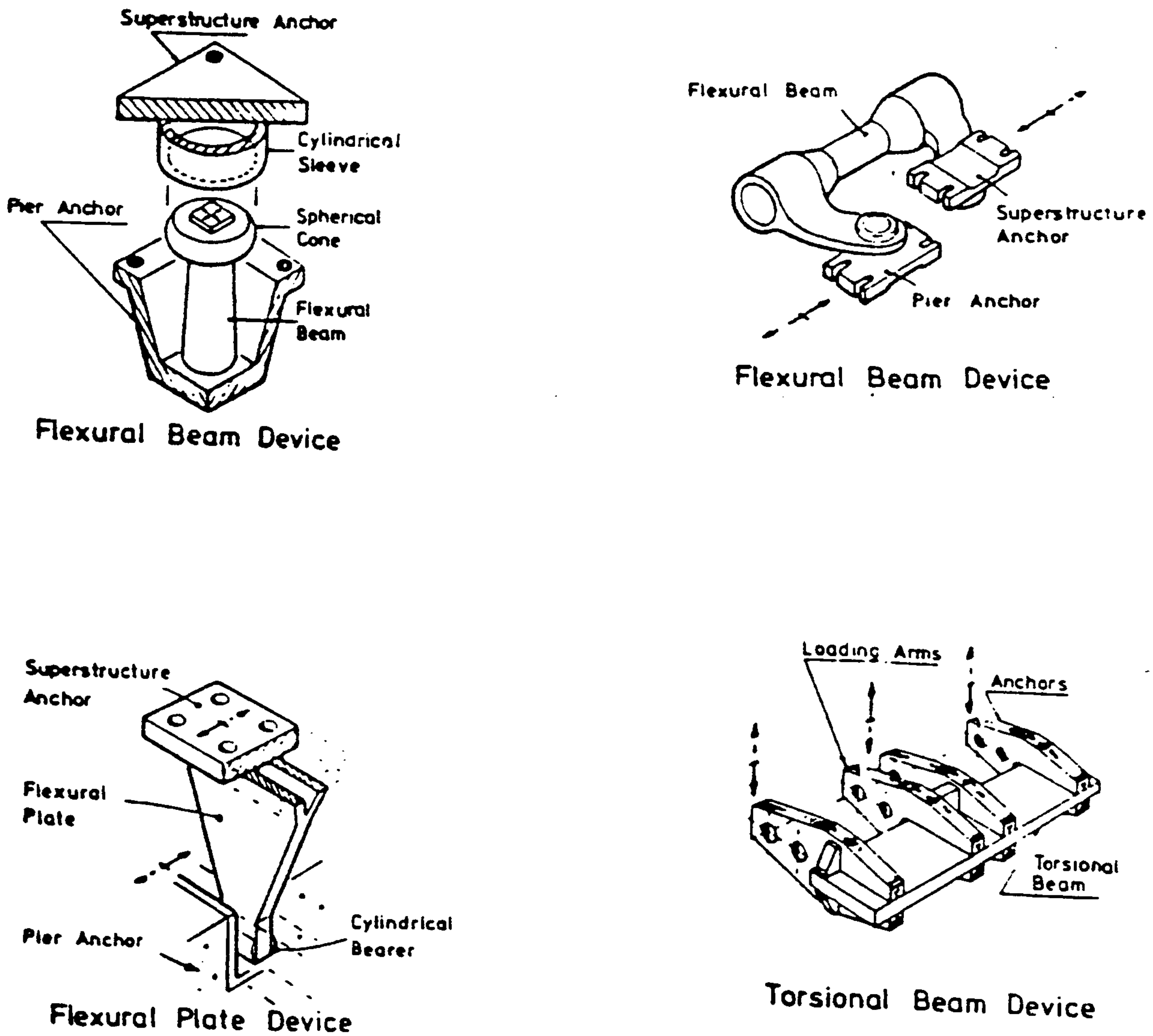


FIGURE 4-1: Yielding Steel Devices; (b) Flexural Beam, Flexural Plate, and Torsional Beam (Buckle, 1990)

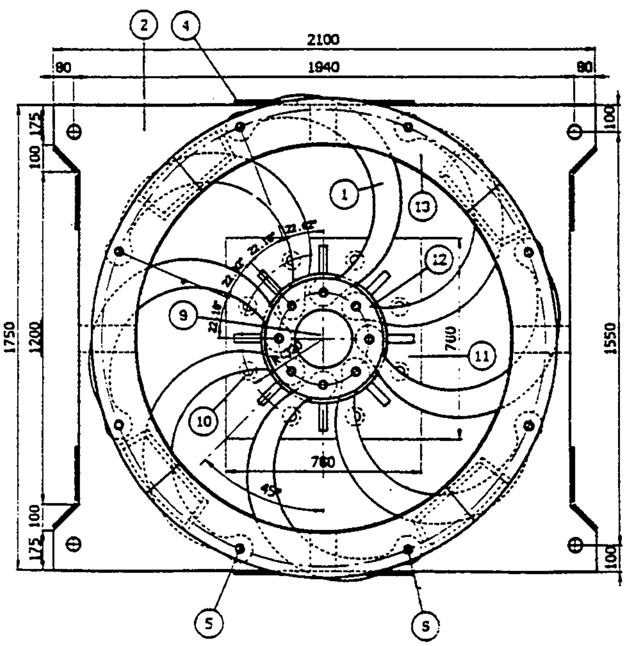


FIGURE 4-1: Yielding Steel Devices; (c) Multi-directional crescent moon-shaped steel damper (Courtesy of ALGA) (Priestley, 1996)

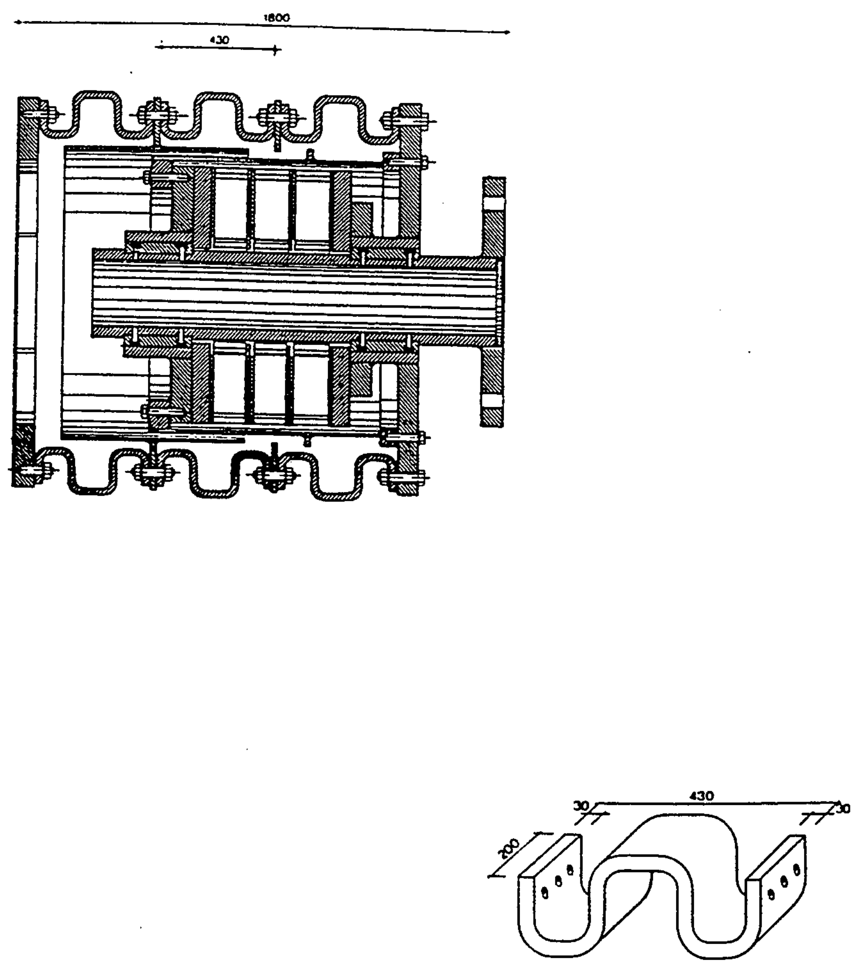


FIGURE 4-1: Yielding Steel Devices; (d) Italian W-shaped damper (Buckle, 1990)

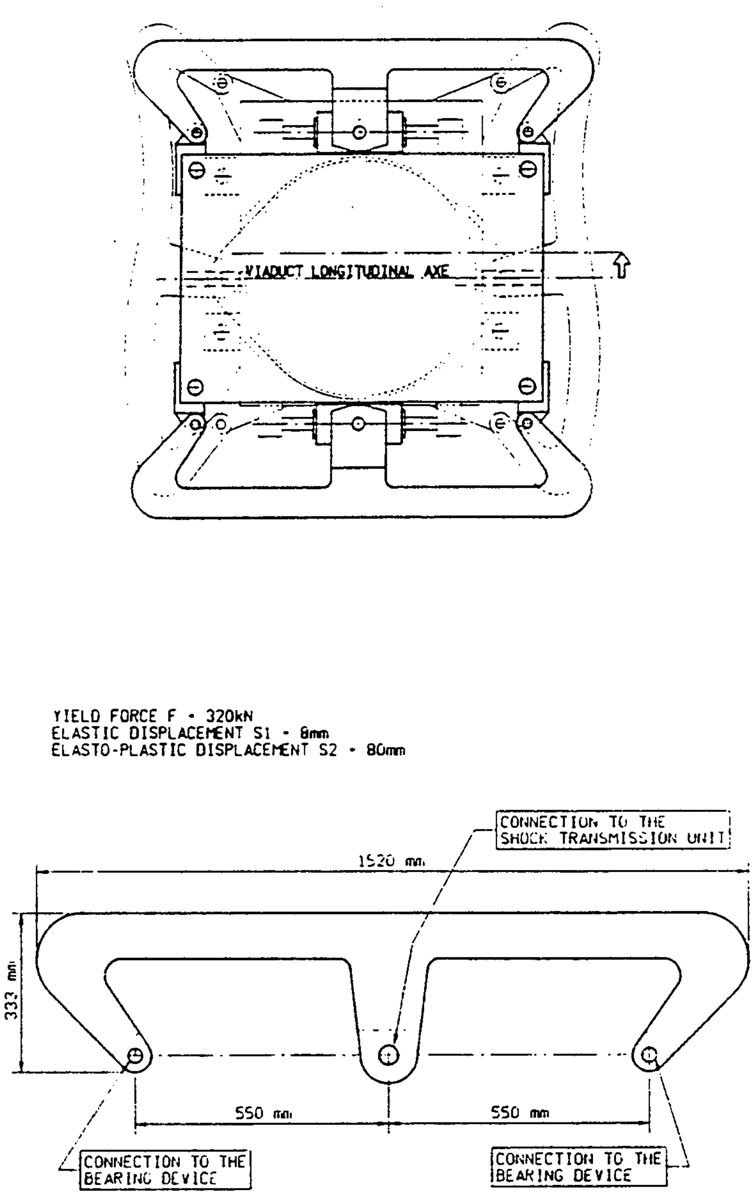


FIGURE 4-1: Yielding Steel Devices; (e) Working scheme of a bearing device under dynamic transversal movement and E-shaped two-bays portal frame hysteretic dampers (Marioni, 1991)

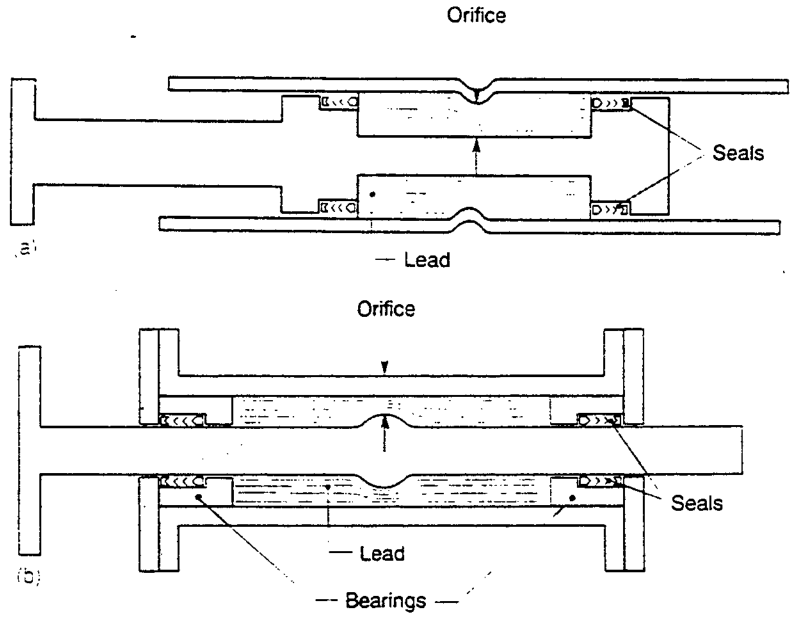
4.1.2 Lead-Extrusion Dampers

The concept of the lead-extrusion dampers are very similar to hydraulic dampers in functionality (see Figure 4-2). The lead goes into a liquid state under high temperature and pressure and flows through orifices (Priestley, 1996). At the end of an earthquake event the lead recrystalizes, and returns to the original state. Their behavior is velocity and temperature dependent; therefore, their yield level may not be well defined which is a disadvantage when used as a structural fuse (Dorka). Lead extrusion devices are shown in Figure 4-2 and their possible use in bridges are listed in Table 4-2.

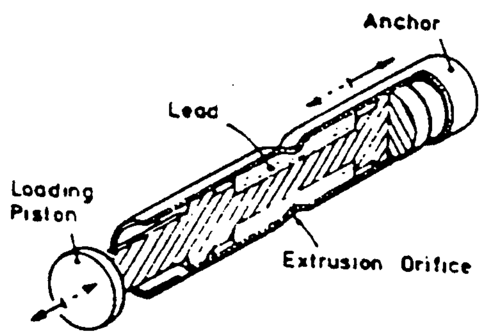
TABLE 4-2: Application of Lead Extrusion Devices in New Bridges

Lead Extrusion Devices	Span Hinge	Superstructure -Substructure Connection	Column	Tower-Deck Connection*	Superstructure -Abutment Connection	Steel Truss Diagonals
Lead Extrusion	X	X		X	X	X

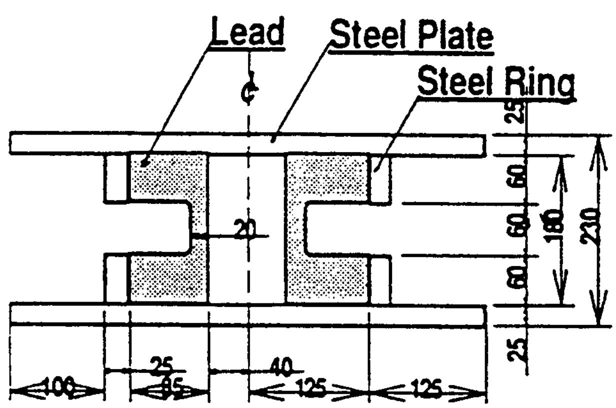
*Suspension, Cable-Stay Bridges



(a) Longitudinal sections (Priestley, 1996)



(b) Lead Extrusion Device (Buckle, 1990)



(c) Lead Joint Damper (Sakurai, 1992)

FIGURE 4-2: Lead Extrusion Devices

4.1.3 Friction Dampers

Friction dampers have long been used as energy dissipation devices for thermal loading of bridges and to reduce the response of buildings and bridges under wind vibration loading. Recently, many friction dampers have been proposed for seismic loading to dissipate energy and control the amount of force going into the structure as shown in Figure 4-3. Most of the friction dampers possess a perfect elasto-plastic force-displacement hystetric loop (Aiken, 1990). The force level can easily be adjusted even after a damper has been installed. Their behavior is not affected by frequency or cyclic loading. The coefficient of friction and the sliding interface material are susceptible to environmental influence and aging (Dorka). Table 4-3 lists several friction dampers and possible uses in bridges.

TABLE 4-3: Application of Friction Devices in New Bridges

Friction Devices	Span Hinge	Superstructure -Substructure Connection	Column	Tower-Deck Connection*	Superstructure -Abutment Connection	Steel Truss Diagonals
Energy Dissipation Restraint (EDR)	X	X		X	X	X
Sumitomo Friction Damper	X	X		X	X	X
Displacement Control Device (DCD)	X	X		X	X	X
Pall Friction Device						X
Slotted Bolted Connection (SBC)	X	X		X	X	X

*Suspension, Cable-Stay Bridges

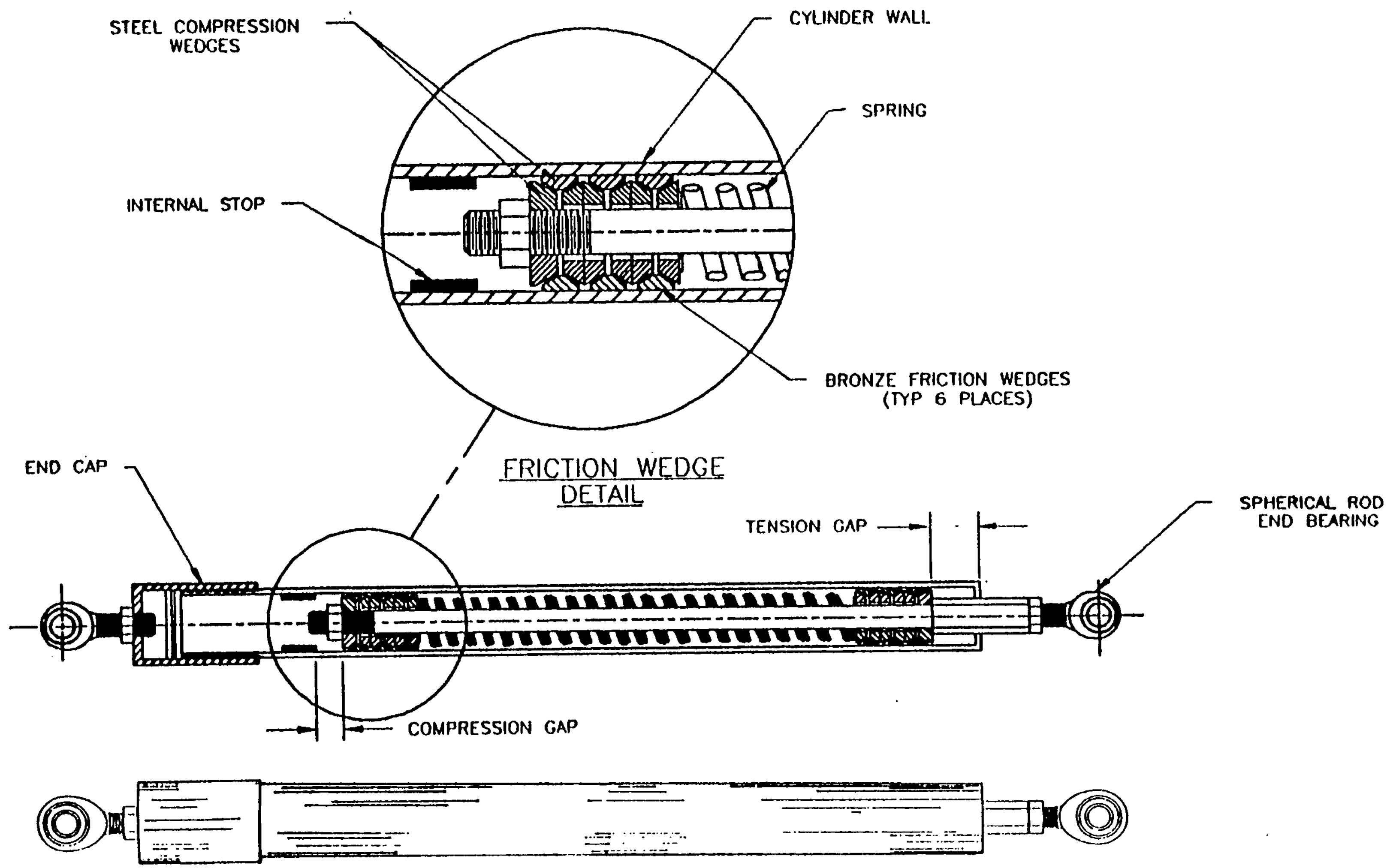


FIGURE 4-3: Friction Dampers; (a) External and Internal Views of Energy Dissipating Restraint (Nims, 1993)

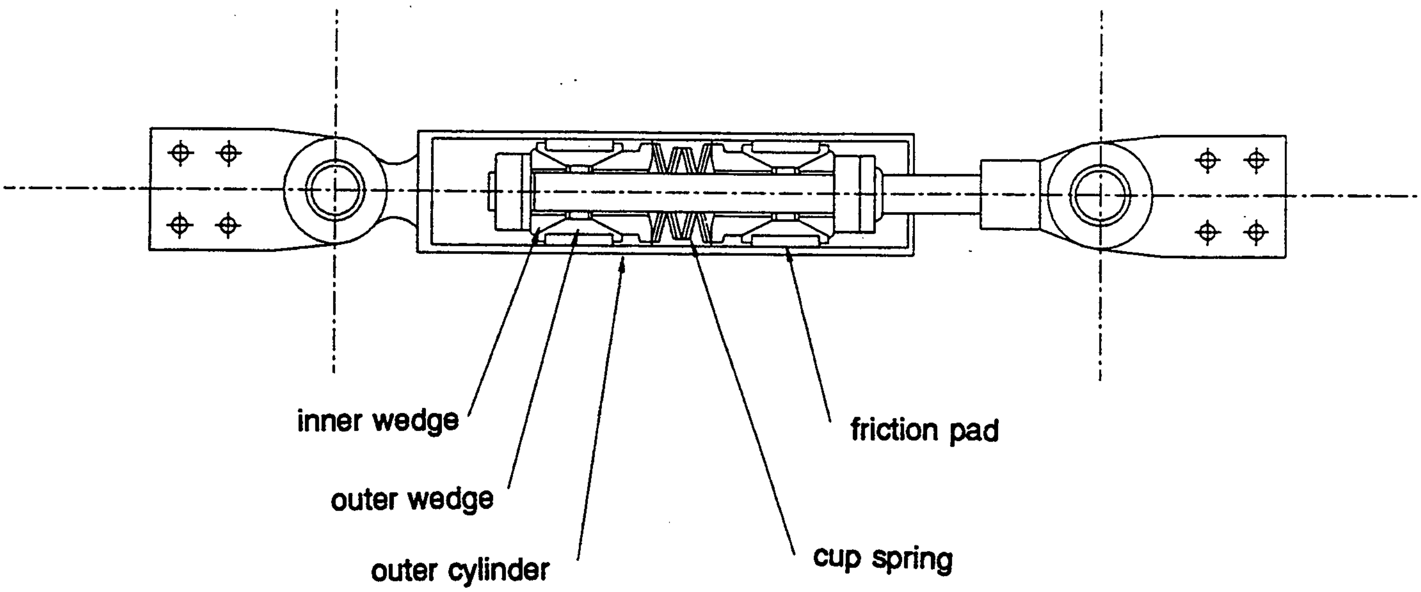
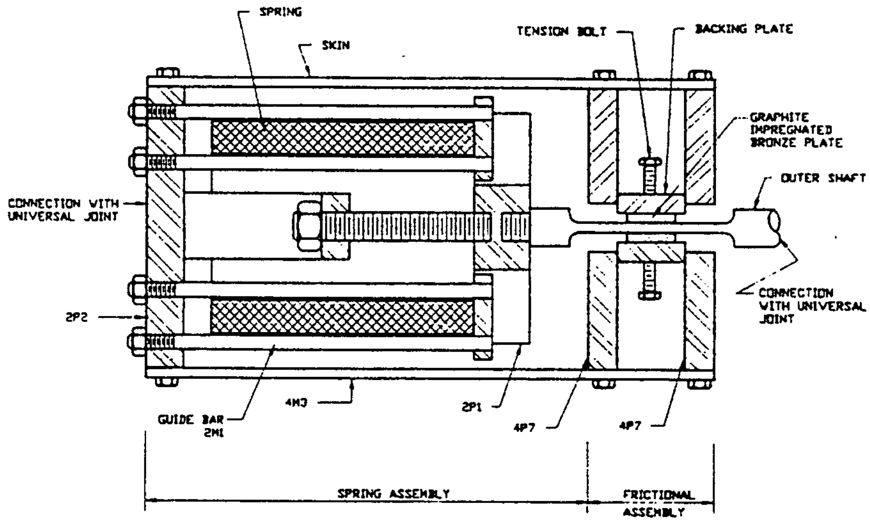
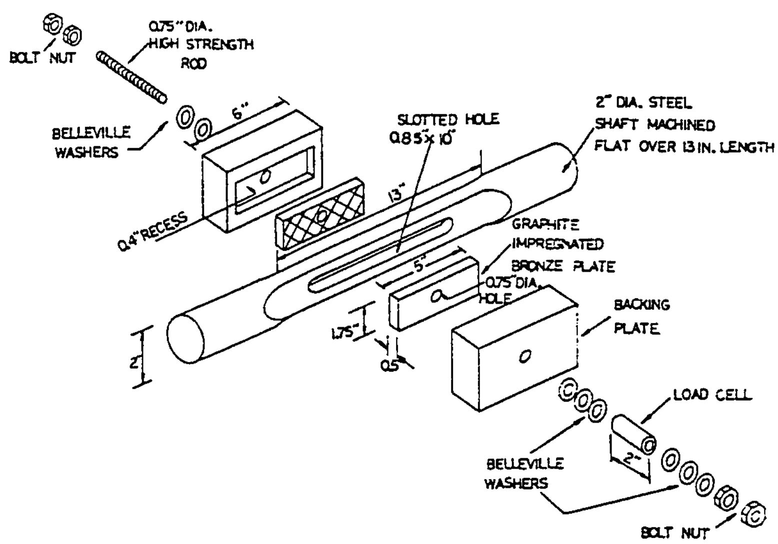


FIGURE 4-3: Friction Dampers; (b) Sectional Views of a Sumitomo Friction Damper



Displacement control device



Frictional assembly in displacement control device

FIGURE 4-3: Friction Dampers; (c) Displacement Control Devices (Constantinou, 1991)

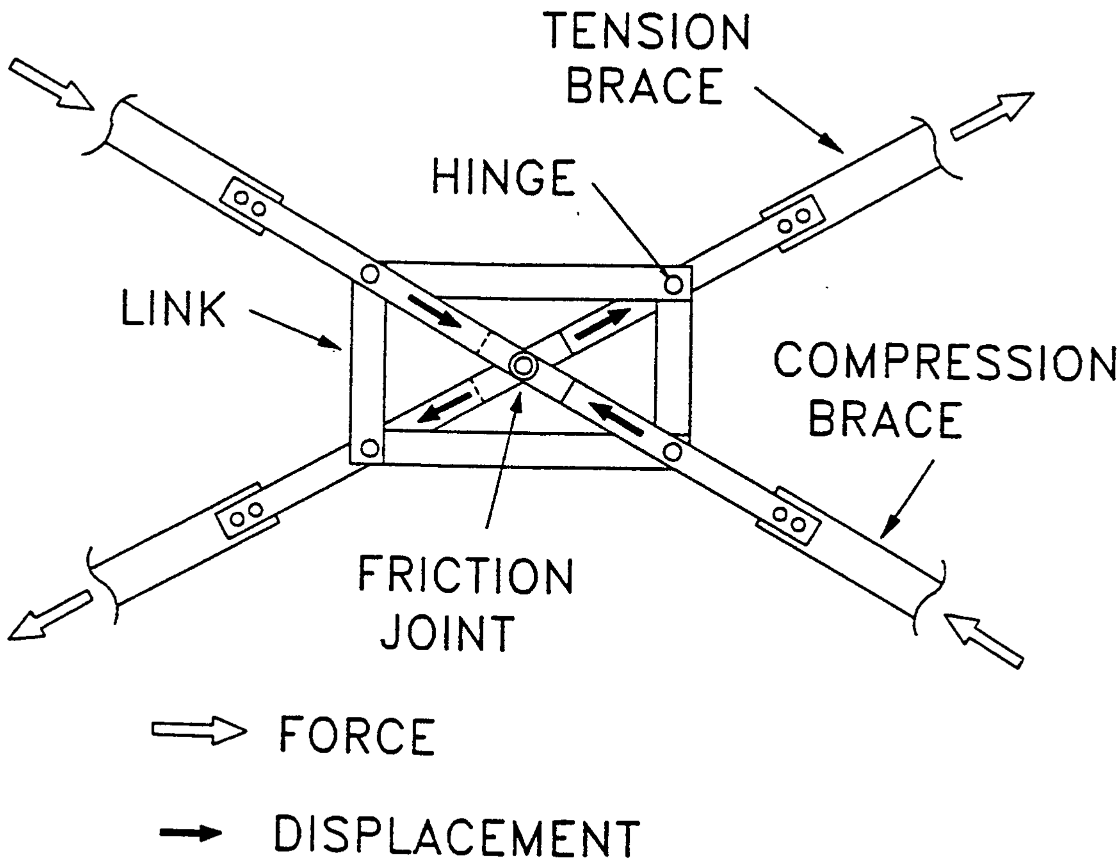


FIGURE 4-3: Friction Dampers; (d) Friction Damping Device (Pall, 1982)

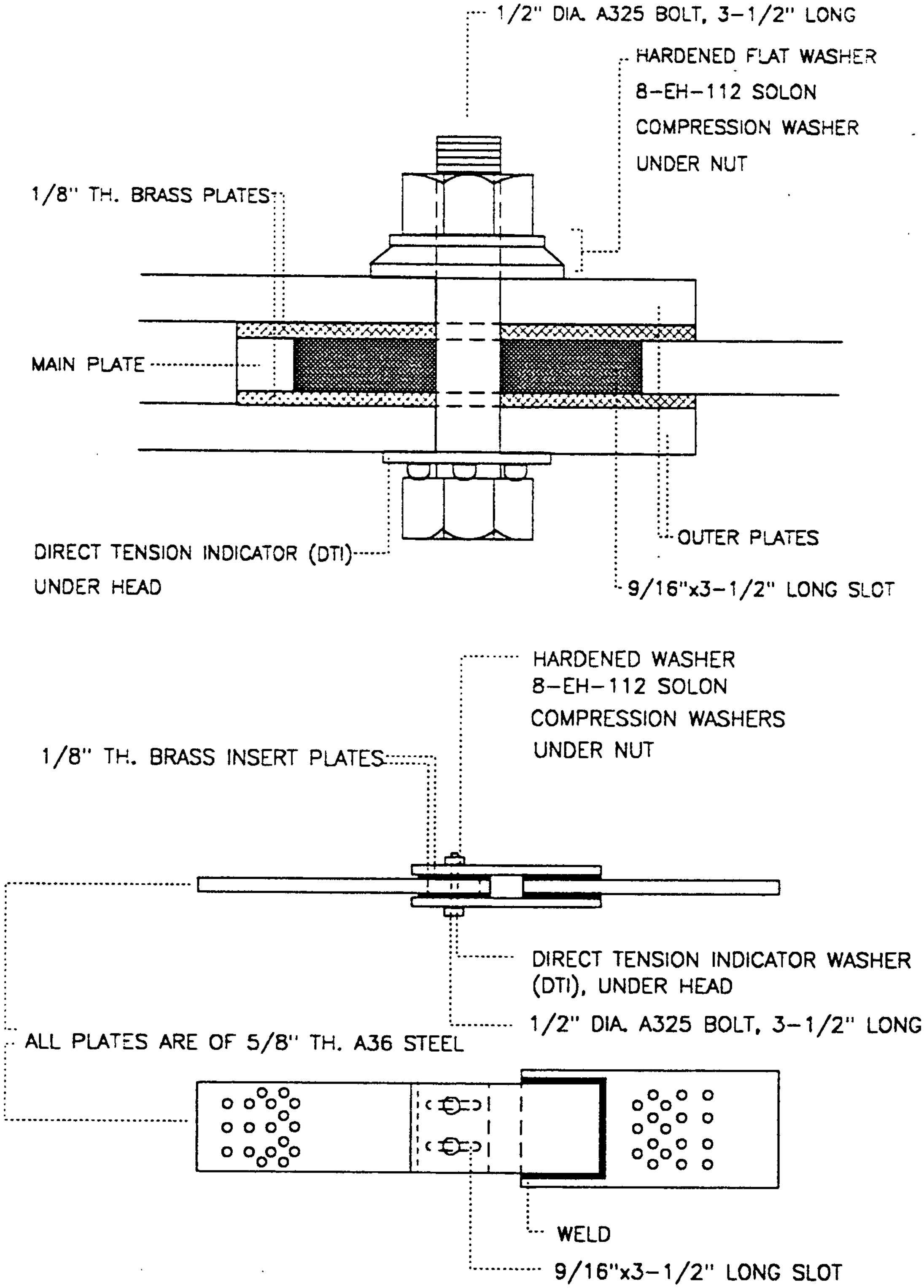


FIGURE 4-3: Friction Dampers; (e) Slotted Bolted Connection (SBC) (Grigorian, 1993)

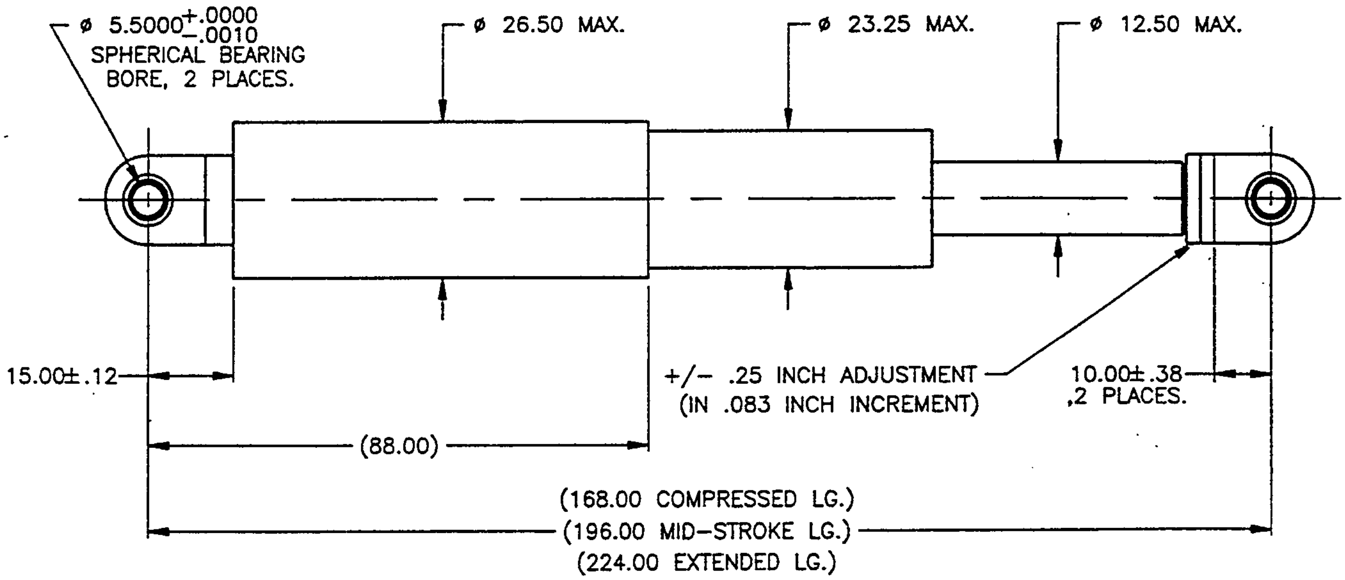
4.1.4 Hydraulic Dampers

Hydraulic dampers (fluid viscous dampers) have been used for many years in the automobile, military and aerospace industry to minimize vibration shock. The Italians have extended their use to energy dissipation and as shock transmitters in the past two decades for highway bridges. Recently, in the United States, a few bridges and buildings have been seismically retrofitted using fluid viscous dampers. Their damping force is velocity dependent; however, it is out-of-phase with the displacement which is a very desirable feature for passive damping, and relatively insensitive to temperature change. Some hydraulic dampers are shown in Figure 4-4 and their possible uses in bridges are listed below in Table 4-4.

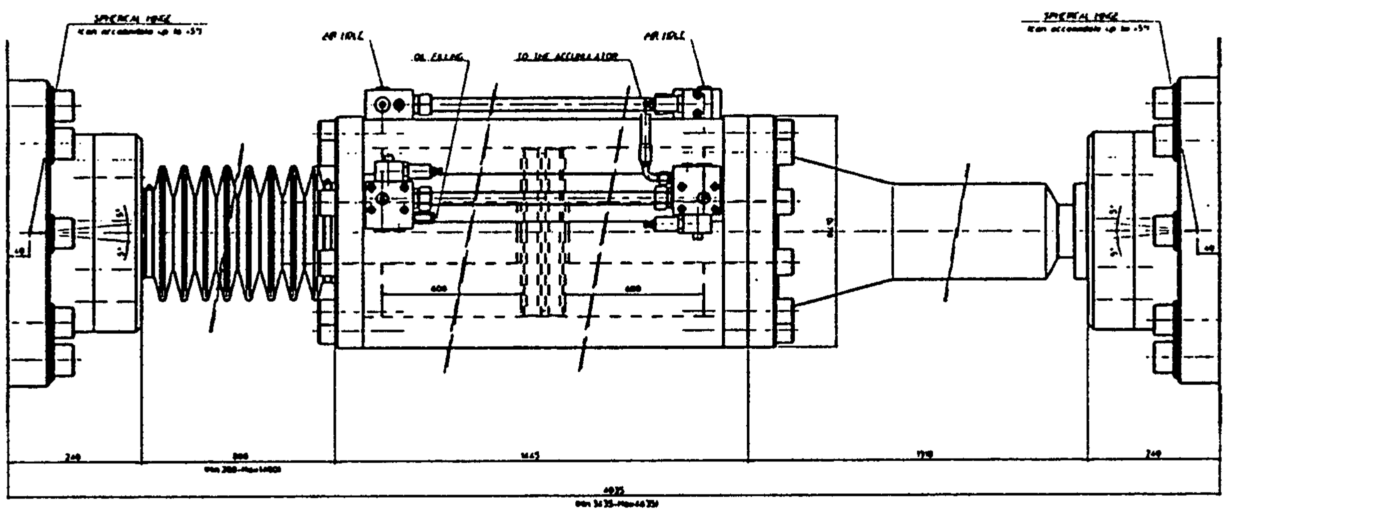
TABLE 4-4: Application of Hydraulic Devices in New Bridges

Hydraulic Devices	Span Hinge	Superstructure -Substructure Connection	Column	Tower-Deck Connection*	Superstructure -Abutment Connection	Steel Truss Diagonals
Taylor Devices	X	X		X	X	X
FIP Devices	X	X		X	X	X
Enidine Devices	X	X		X	X	X
Lisega Devices	X	X		X	X	X

*Suspension, Cable-Stay Bridges

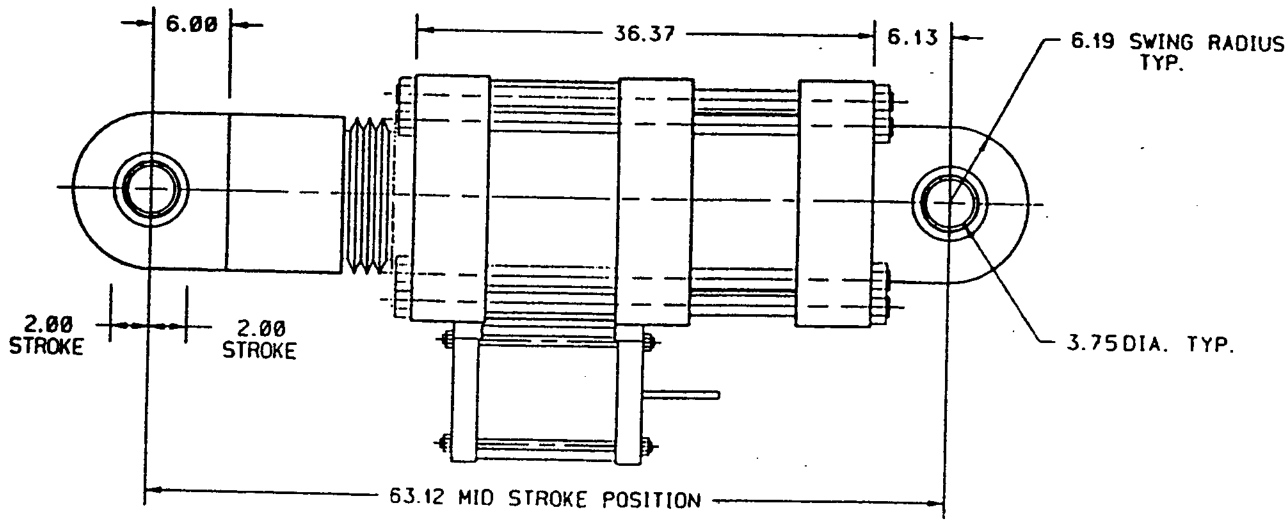


(a) Taylor device

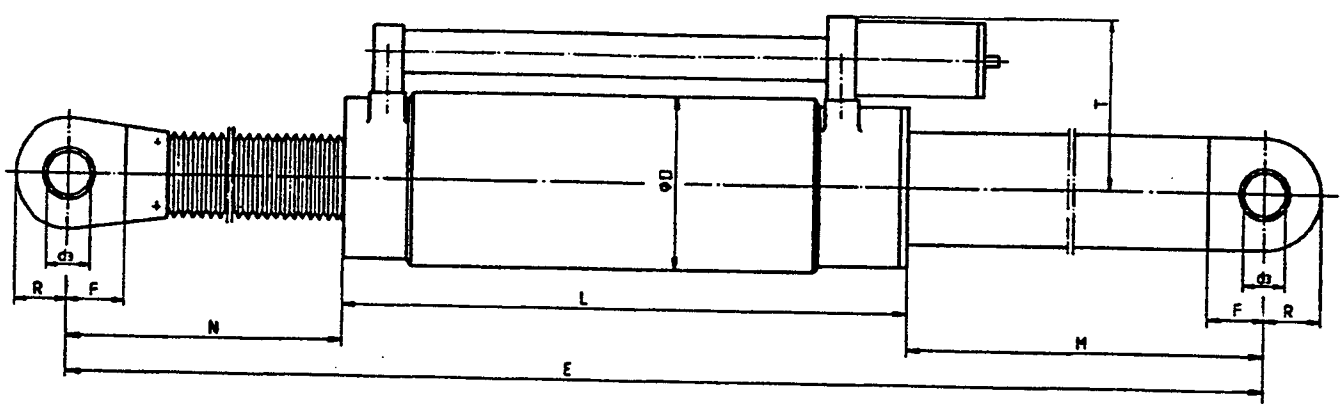


(b) FIP device

FIGURE 4-4: Fluid Viscous Dampers



(c) Enidine device



(d) Lisega device

FIGURE 4-4: Fluid Viscous Dampers (cont.)

4.1.5 Viscoelastic Dampers

Viscoelastic dampers have been developed by the 3M Company and were used in several tall buildings for wind control vibration applications. The viscoelastic dampers are composed of two bonded viscoelastic layers (acrylic polymers) as shown in Figure 4-5. Recently, their use has been extended to resist earthquake loadings (Yakota, 1992). The viscoelastic damper material properties, such as shear loss modulus and shear storage modulus, are frequency and temperature dependent (Mahmoodi, 1969). Their possible application in new bridges is shown in Table 4-5.

TABLE 4-5: Application of Viscoelastic Devices in New Bridges

Viscoelastic Devices	Span Hinge	Superstructure -Substructure Connection	Column	Tower-Deck Connection*	Superstructure -Abutment Connection	Steel Truss Diagonals
3M						X

*Suspension, Cable-Stay Bridges

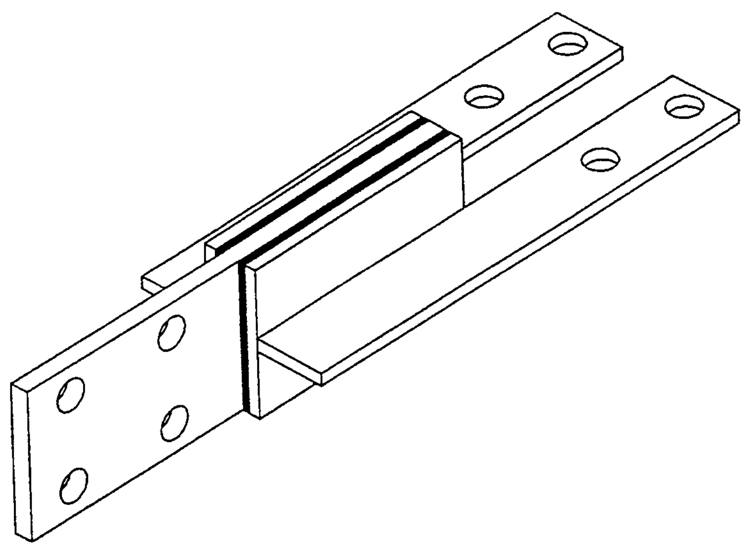


FIGURE 4-5: Viscoelastic Damper (Aiken, 1990)

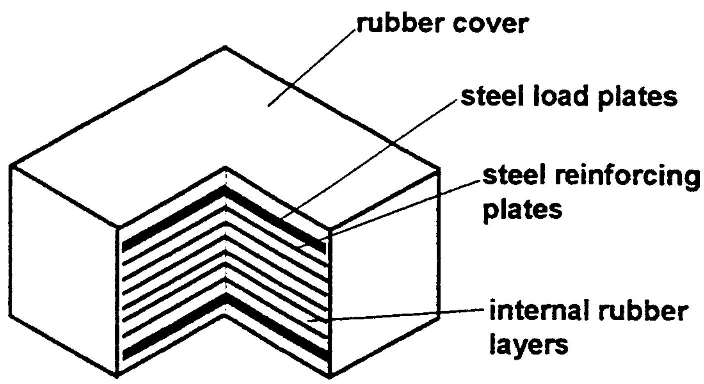
4.1.6 Isolation Bearing System

Seismic isolation systems can be categorized into two categories, either elastomeric or sliding bearings. Elastomeric and sliding bearings are two ways of introducing flexibility into a bridge. They lengthen the period of the structure, therefore, reducing the amount of force transmitted to the substructure and increase the bearing relative displacement. Reduction of bearing displacement can be accomplished by using high damping rubber or by use of additional energy dissipating devices like yield steel dampers, lead extrusion dampers, friction dampers, hydraulic dampers, and lead plugs in the elastomeric bearing (Buckle, 1990). In the last two decades, many isolation bearing systems have been adopted all over the world as a seismic isolation system in bridges and buildings. Some of the isolation bearing systems are shown in Figure 4-6 and their possible uses in bridges are listed below in Table 4-6. The effect of the substructure flexibility was investigated in details as shown in Appendix A. The effect of the substructure flexibility on the isolation system response is almost negligible in the case of a stiff substructure. However, in the case of a flexible substructure, the bearing displacement can increase substantially and one should incorporate it in the determination of the isolation system response.

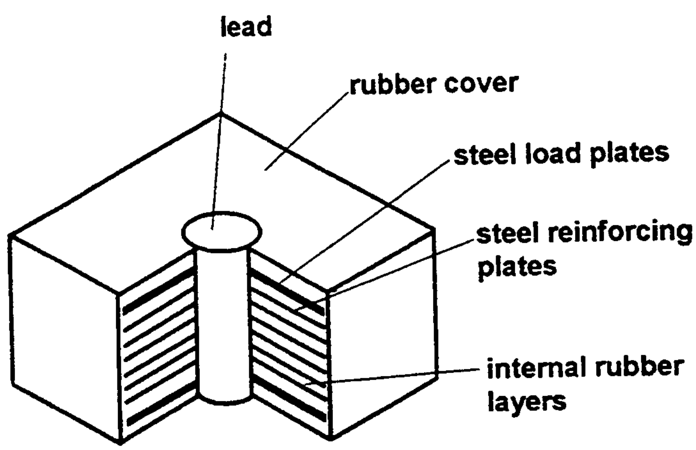
TABLE 4-6: Application of Isolation Bearing Devices in New Bridges

Isolation Bearing Devices	Span Hinge	Superstructure -Substructure Connection	Column	Tower-Deck Connection*	Superstructure -Abutment Connection	Steel Truss Diagonals
Lead Rubber Bearing (LRB)		X		X	X	
High Damping Rubber Bearing (HDB)		X		X	X	
Friction Pendulum Sliding Bearing (FPS)		X		X	X	
PTFE Flat Sliding Disc Bearing		X		X	X	
Resilient Frictional Base Isolation (RFBI)		X		X	X	
Elastomeric Bearing and Sliding Plate		X		X	X	

*Suspension, Cable-Stay Bridges

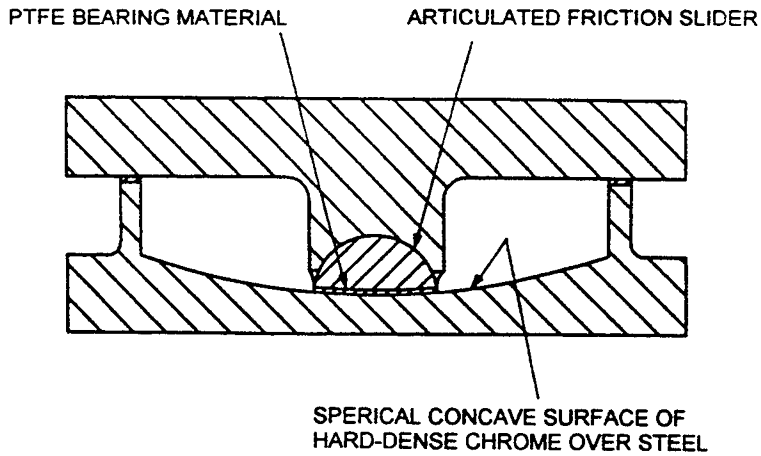


(a) Laminated-rubber bearing

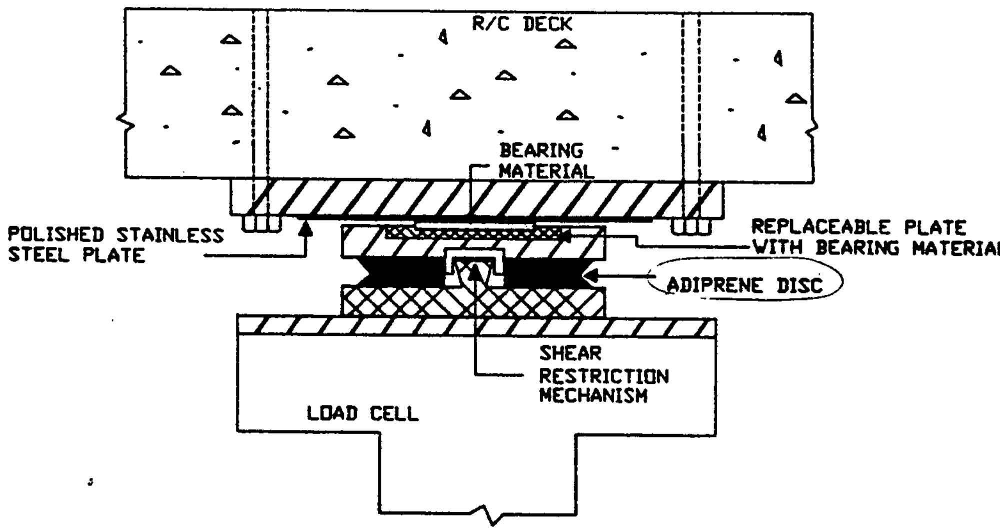


(b) Lead-rubber laminated bearing

FIGURE 4-6: Isolation Bearing Systems (Priestley, 1996)

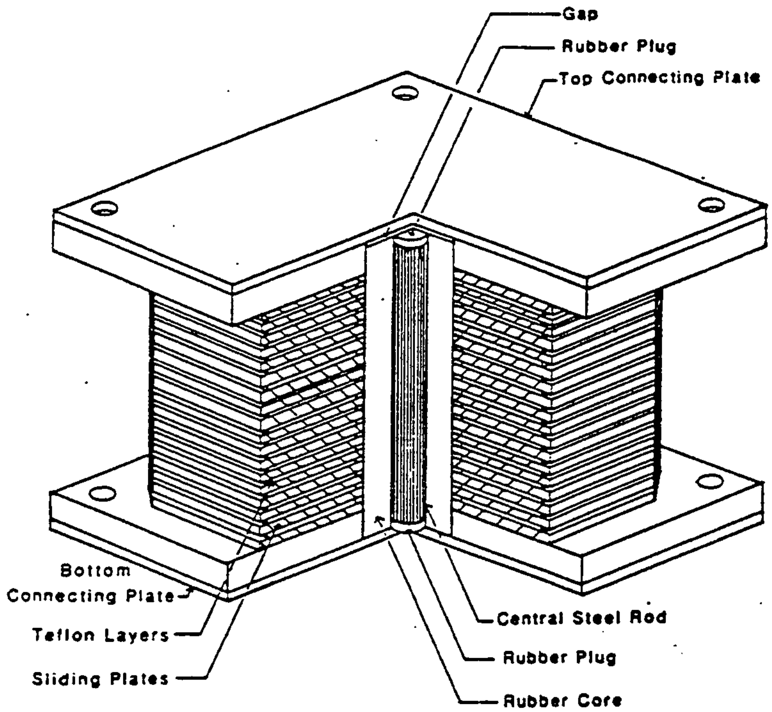


(c) Section view of a friction pendulum device

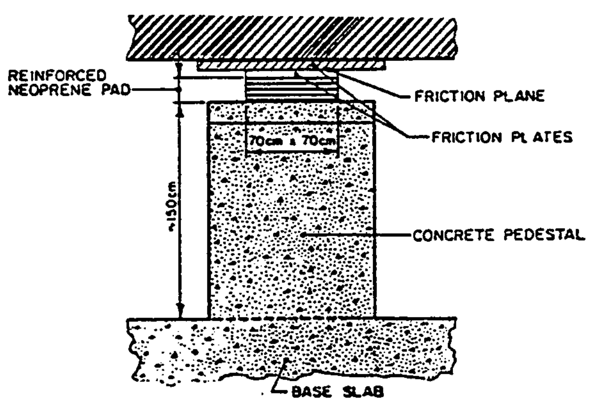


(d) Sliding disc bearing (Constantinou, 1991)

FIGURE 4-6: Isolation Bearing Systems (cont.)



(e) The R-FBI bearing



(f) Elastomeric bearing and friction plates

FIGURE 4-6: Isolation Bearing Systems (cont.)

4.2 Application to Bridges

Many passive energy dissipating devices have been used successfully to retrofit bridges; however, their use can be extended to new bridge construction. In Figures 4-7 to 4-11, their application to new bridge design is shown schematically in structural form diagrams. Traditionally, two philosophies have been used in the past to seismically design a bridge: strength or ductility. The incorporation of energy dissipating devices is relatively new to bridge construction. Their use in new bridges may be beneficial depending on the seismic performance goal, the bridge type and configuration. The intent of this section is to provide some ideas for bridge designs where energy dissipating devices could be used.

Several energy dissipating devices are currently being used in New Zealand which are sacrificial or utilize sacrificial components as follows:

1. A torsional beam energy dissipator is present on the South Rangitikei Rail Bridge in New Zealand. The twin concrete columns are designed to "step" by alternatively lifting off their elastomeric bearing supports during transverse seismic shaking. Lift-off induces torsional action in the two torsional beam dissipators at the base of each column. A vertical shear key member is provided at the base of each column with a stop to limit the extent of vertical stepping in case of a major seismic event (EERI, 1990). Some example details of the torsional beam energy dissipator are provided in Figure 4-12.

2. A flexural beam energy dissipator is composed of a set of cast steel loading arms and a mild steel circular beam. Loads are applied to the ends of the loading arms, which induce alternating tensile and compressive loads on the device. These loads induce bending stresses in the circular beam as the beam is forced to deform. Six of these devices were placed in the Cromwell Bridge in New Zealand. The devices were connected to a fixed abutment to dissipate any energy that might result from the longitudinal movement caused by large earthquakes. Analytical studies indicated that the advantage of incorporating the devices was a reduction in superstructure displacement (similar to the longitudinal joint restrainer previously discussed), which resulted in both a cost savings for the abutment joint details and a greater degree of protection against yielding in the piers (FHWA, 1983). An example detail of a flexural beam energy dissipator is provided in Figure 4-13.

3. The Bannockburn Bridge, a 148 meter long four span steel truss bridge (see Figure 4-14) on tall reinforced concrete piers, was completed in 1989 and spans a lake created by a power development project. The bridge was built before the lake was filled. The bases of the piers are now submerged and are therefore inaccessible for convenient repair. The seismic design approach adopted therefore was that the piers should remain elastic under the design earthquake conditions. Preliminary analysis demonstrated the need for slab piers to provide adequate transverse stiffness and for energy dissipators to be incorporated at the abutments to provide longitudinal stiffness and damping. Lead-rubber bearings are located at the piers and footing. Lead extrusion devices were adopted for use at the abutments. The adopted energy dissipating devices provide yield forces, represented as a percentage of the superstructure weight, of 20% longitudinally and 10% transversely. The maximum design displacements at the abutment deck joints and across the bearings are less than 100 mm. With the avoidance of yielding of the pier stems, limited ductility design procedures were adopted for the pier stem reinforcement (EERI, 1990).

4. Similar lead extrusion devices have also been used in two bridges (see Figure 4-15) crossing a motorway in Wellington. Both of these bridges are on a steep grade and are supported on

sliding bearings while being restrained longitudinally by the lead extrusion dissipators. In these bridges, the dissipators must resist not only significant traction and braking forces from vehicles, but must also provide longitudinal seismic restraint with a limited force applying to the abutment (EERI, 1990).

Energy dissipation devices are being used in the U.S. to retrofit some large span bridges. The Marquan Bridge in Portland, Oregon incorporated shock transmission devices at the joints of a drop in span adjacent Pier 3 and Pier 4 (see Figures 4-16 and 4-17). The Golden Gate Bridge proposed retrofit is utilizing a damper system between the tower and deck connection at the North (Marin) Tower and Pylon N1 (see Figures 4-18 to 4-20). Both systems are used to limit impacting to structural components. A fluid viscous damper system was proposed for the Martin Luther King Bridge (Route I-70) in St. Louis, Missouri. The existing bridge is a multi-girder steel viaduct on steel rocker bearings. The design alternative proposed to incorporate fluid viscous dampers with new elastomeric bearings. The purpose of the retrofit was to reduce the seismic forces to the two column bents. An example detail for this proposed retrofit is shown in Figures 4-21 and 4-22.

Several new bridge designs in the U.S., which include the states of Illinois, California, Oregon, New Hampshire, Kentucky, and Missouri, have incorporated lead core rubber bearings for seismic isolation (Mayes, 1995) as follows:

1. The Sexton Creek Bridge is located in Alexander County in southern Illinois, in the New Madrid fault zone. It is the first new bridge in North America to be isolated. The 3-span continuous bridge with spans of 120'-154'-120' and 5 lines of slightly curved, steel girders is supported on lead-rubber isolation bearings. For seismic loads, the acceleration coefficient, A, was 0.20 and the soil profile was Type III. One goal of the design was to reduce the seismic forces by a factor of 3, the other was to evaluate the range of possible lateral load distributions by using bearings of a constant height, but varying the distribution of the lead core for the purpose of minimizing lateral forces on the piers.

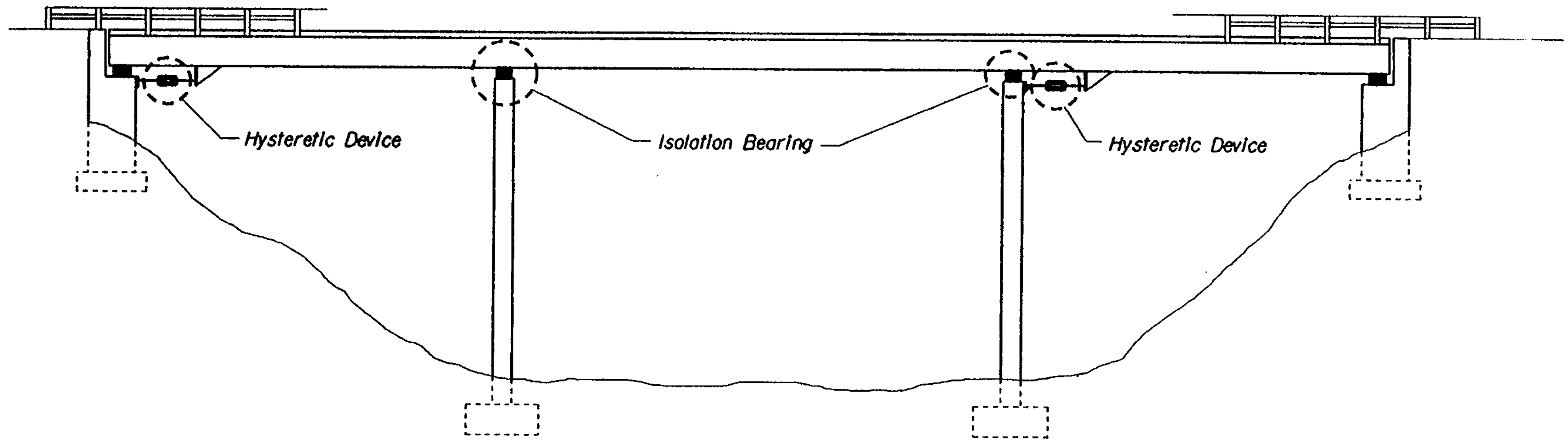
2. Olympic Boulevard Separation is part of the reconstruction of Route 24/I-680 Interchange in Walnut Creek, east of Oakland, California. This 700 ft.-long, 4-span continuous, curved, composite-steel plate-girder bridge is Caltrans' first new bridge to be seismically isolated. It was built as part of a temporary flyover to handle southbound traffic on I-680 until late in the project schedule, when the superstructure and its isolation bearings will be moved over to its final, permanent alignment. Caltrans chose isolation design to ensure serviceability and avoid the cost of repair after a 0.6g seismic event, and to simplify removal and reuse of the components in the permanent structure. The design reduced the seismic forces by a factor of 6.5. Compared to conventional design, this reduced the cost of the piled foundations by 38%.

3. McLoughlin Boulevard was seismically upgraded as part of reconstruction of the Tacoma Street Interchange southeast of Portland, Oregon. This upgrade required the replacement of an existing ramp with a new, 1005-ft.-long, 8-span structure which curves through an 80 degree change in direction. For aesthetic and maintenance reasons, Oregon DOT's Bridge Section selected a conventional, continuous, post-tensioned concrete box girder with integral cross beams and monolithically connected columns supported on piled footings. The conventional design for a 0.3g event resulted in large and expensive footings, which would have encroached on the root system of adjacent redwood trees. By incorporating isolation design, the period of the structure was lengthened from 0.33 sec. to 2.1 sec., producing a force reduction sufficient to cut the footing size in half and reduce the number of piles by 30%.

4. The Squamscott River Bridge is a river crossing located in Rockingham County in the south-east corner of New Hampshire. The seismic requirement for the area is Category B, based on an acceleration coefficient of 0.15. The subsurface conditions very closely match those of AASHTO Soil Profile Type III. The bridge, spanning the Squamscott River and adjoining wetlands, is a 52 ft.-8 in. wide, 874 ft.-long, 6-span, continuous steel plate-girder structure with a composite concrete deck slab. It was originally designed in the traditional manner, utilizing steel rocker bearings, with the center pier (Pier 3) fixed and expansion joints at each abutment. This required Pier 3 to resist all the longitudinal lateral loads, while both abutments and the five piers resisted the transverse lateral loads. As a result of this lateral load distribution and the poor foundation material, the original Pier 3 design resulted in a heavily reinforced, oversize footing (54'x43'x8') with 76 H12x53 piles and a heavily reinforced 6 ft.-thick solid-concrete shaft. The revised design resulted in Pier 3 having a 48'x15'x4' footing with thirty H12x53 piles and a 4 ft.-thick, solid-concrete shaft similar to Piers 4 and 5.

5. The US-51 Bridge over Minor Slough in Kentucky is the first prestressed concrete I-girder structure in North America to be isolated. It is situated in Ballard County on the Wickliffe-Cairo Road adjacent to the Mississippi River, well within the New Madrid fault zone. The design acceleration coefficient was 0.25 and the soil condition was AASHTO Type II. The structure is 47 ft.-wide and 371 ft.-long, and crosses Minor Slough at a 45 degree skew. There are three 121 ft. simple spans with six lines of girders. Continuity was effected by casting the pier diaphragms monolithically with the deck slab. The piers are 4-column bents on piled footings, and the abutments are pile-supported end bents. The overall seismic load on the bridge was reduced by a factor of 3.5. The resulting lateral forces on the substructures were then redistributed to favor the piers. This was accomplished by eliminating the lead cores from the pier isolators and increasing the size of the lead cores in the abutment isolators.

6. A segment of the Metrolink Light Rail System in Missouri includes seven bridges in the section of the new, double-track, light-rail line from Taylor Avenue to the St. Louis International Airport. Each superstructure is a concrete or steel box girder supporting a single track on a concrete-ballasted deck. Haunched and constant-depth, simple and continuous spans are used, some tangent, some curved. Seat or wall-type abutments are used, some founded on piling. The piers, some founded on piling, are 2-column bents with the columns closely spaced at the base, curving upward and outward to connect at the ends of the bent cap which supports the superstructures for both tracks. The project site lies within the influence of the New Madrid Fault, and the seismic criteria are AASHTO Category B with an acceleration coefficient of 0.1 and Soil Type I. While seismic protection to keep the operation functional after a seismic event was a major consideration, the main motivation for pursuing isolation design was to distribute lateral forces evenly among all the substructures in any given structure, instead of resisting them at one fixed pier. The high longitudinal forces - up to 18% of dead load - were the chief concern. Large lead cores were required to provide the necessary high, initial stiffness to resist these forces at each support. In some instances, the required lead core diameter would exceed availability. In these cases, clusters of smaller bearings with smaller lead cores were installed . The number of bearings per cluster varied from two to eight, depending on longitudinal force requirements. A nominal seismic force reduction was also achieved.



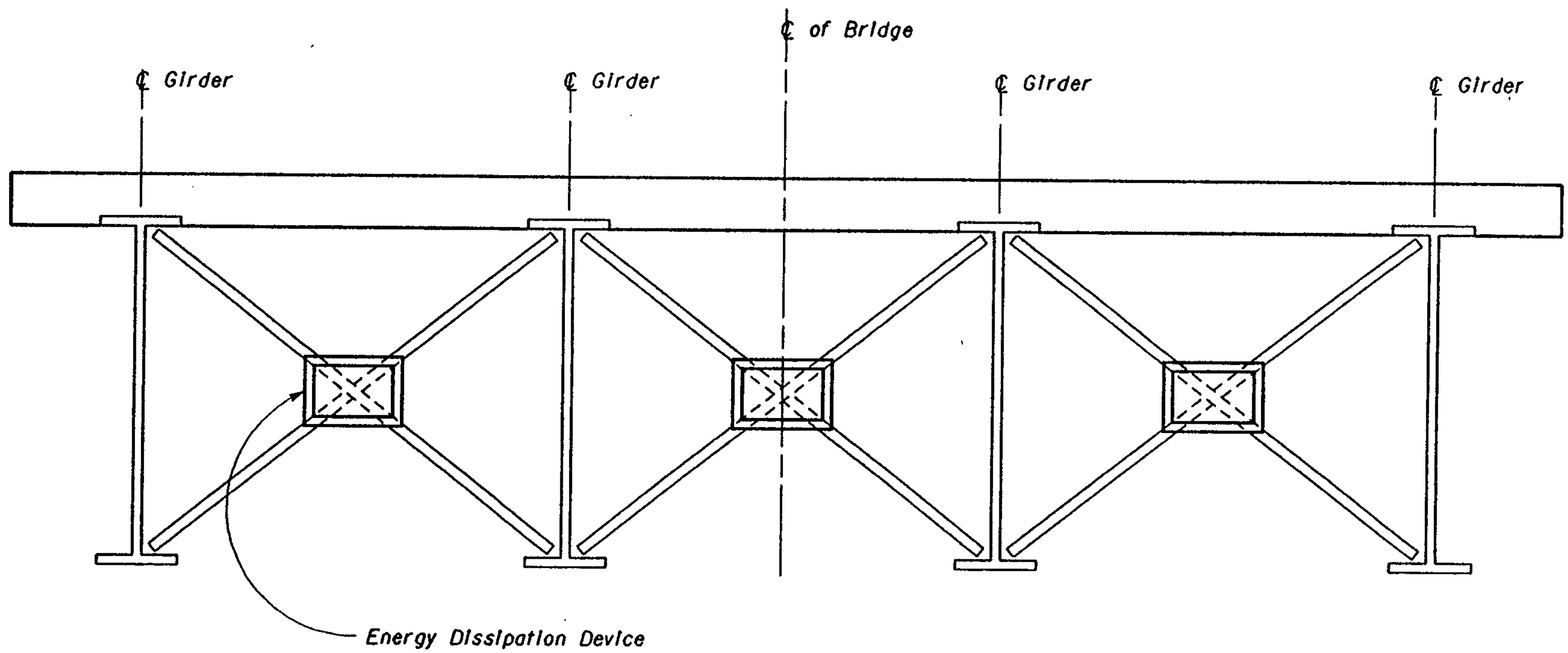
DESCRIPTION

Steel I-Girder Bridge
Seat Type Abutments
Energy Dissipating Devices
- fluid devices
- friction devices
- lead extrusion devices
Isolation Bearings
- lead core
- elastomeric
- sliding

COMMENTS

- Continuous superstructure
- Force and displacement limited in substructure
- Column designed elastic and reduces footing size

FIGURE 4-7: Conventional Girder Type Bridge



DESCRIPTION
Steel I Girder

COMMENTS
- Provides energy dissipation in superstructure
- Limits forces to substructure to prevent plastic hinging occuring

FIGURE 4-8: Bracings at Abutments or Piers

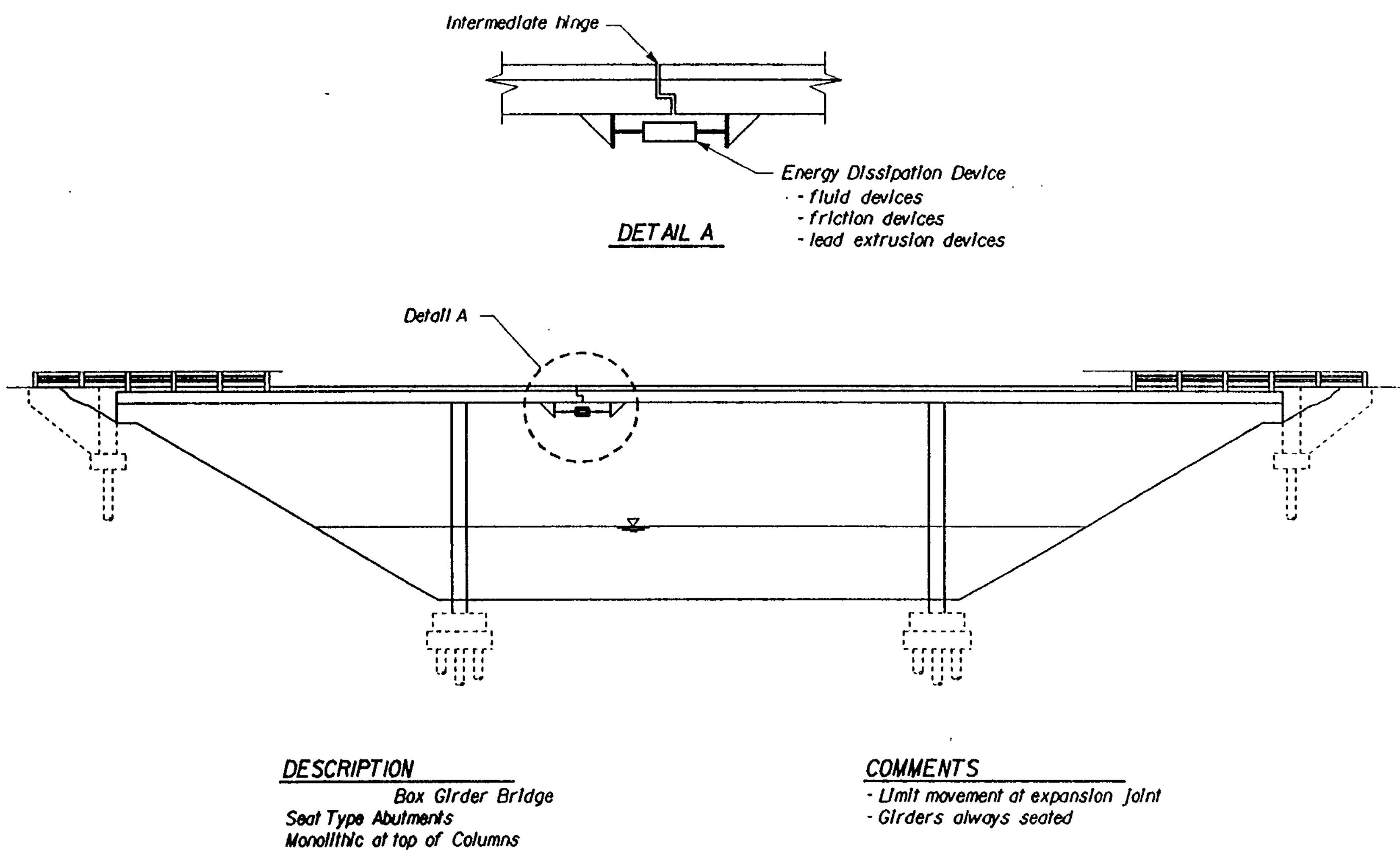
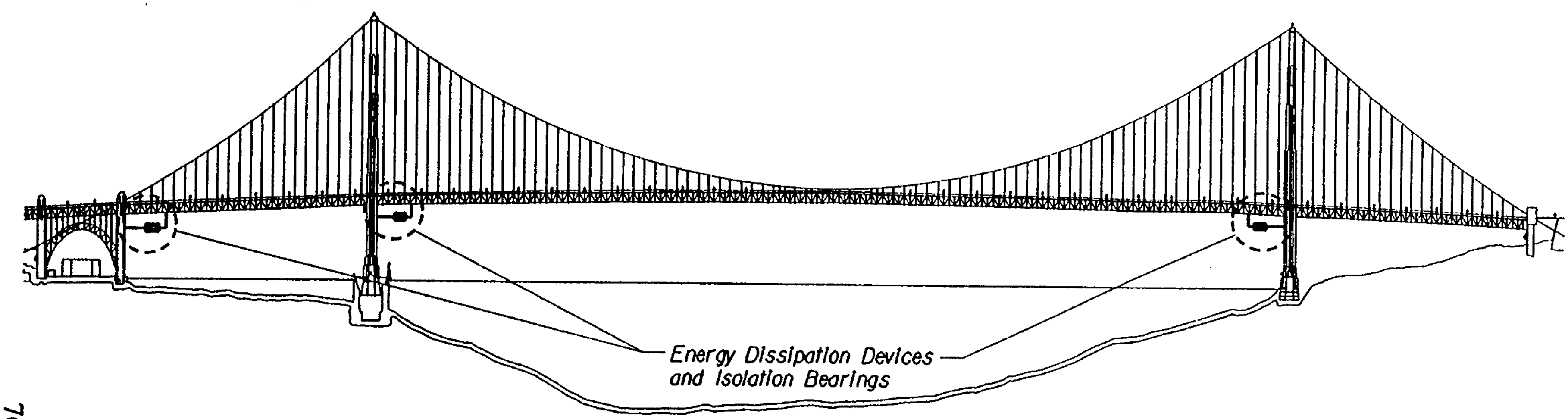


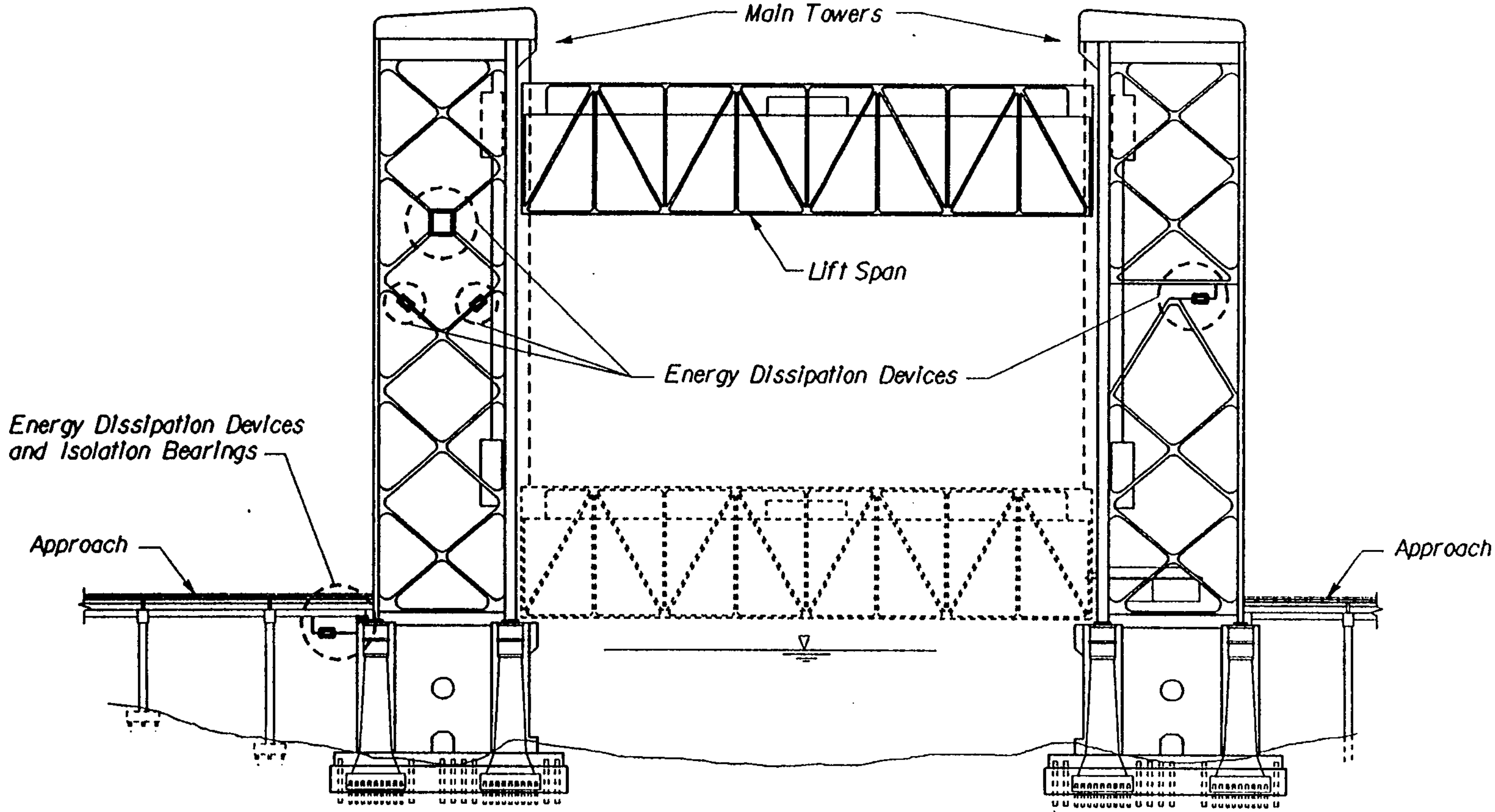
FIGURE 4-9: Conventional Girder Type Bridge



DESCRIPTION
Suspension Bridge
Energy Dissipation Devices
- fluid devices
- friction devices
- lead-extrusion devices
Isolation Bearings
- elastomeric
- sliding

COMMENTS
- Tower-deck interaction limited
- Tower designed elastic
- Smaller tower footing

FIGURE 4-10: Suspension Bridge



DESCRIPTION
Movable Lift Span Bridge
Energy Dissipation Devices
- yielding steel devices
- fluid devices
- friction devices
- visco-elastic devices
- lead extrusion devices
Isolation Bearings
- elastomeric
- sliding

COMMENTS
- Eliminate diagonal buckling in braces in towers
- Reduce force in the tower legs
- Reduce force interaction between approach and main tower
- Smaller tower foundation

FIGURE 4-11: Moveable Bridge

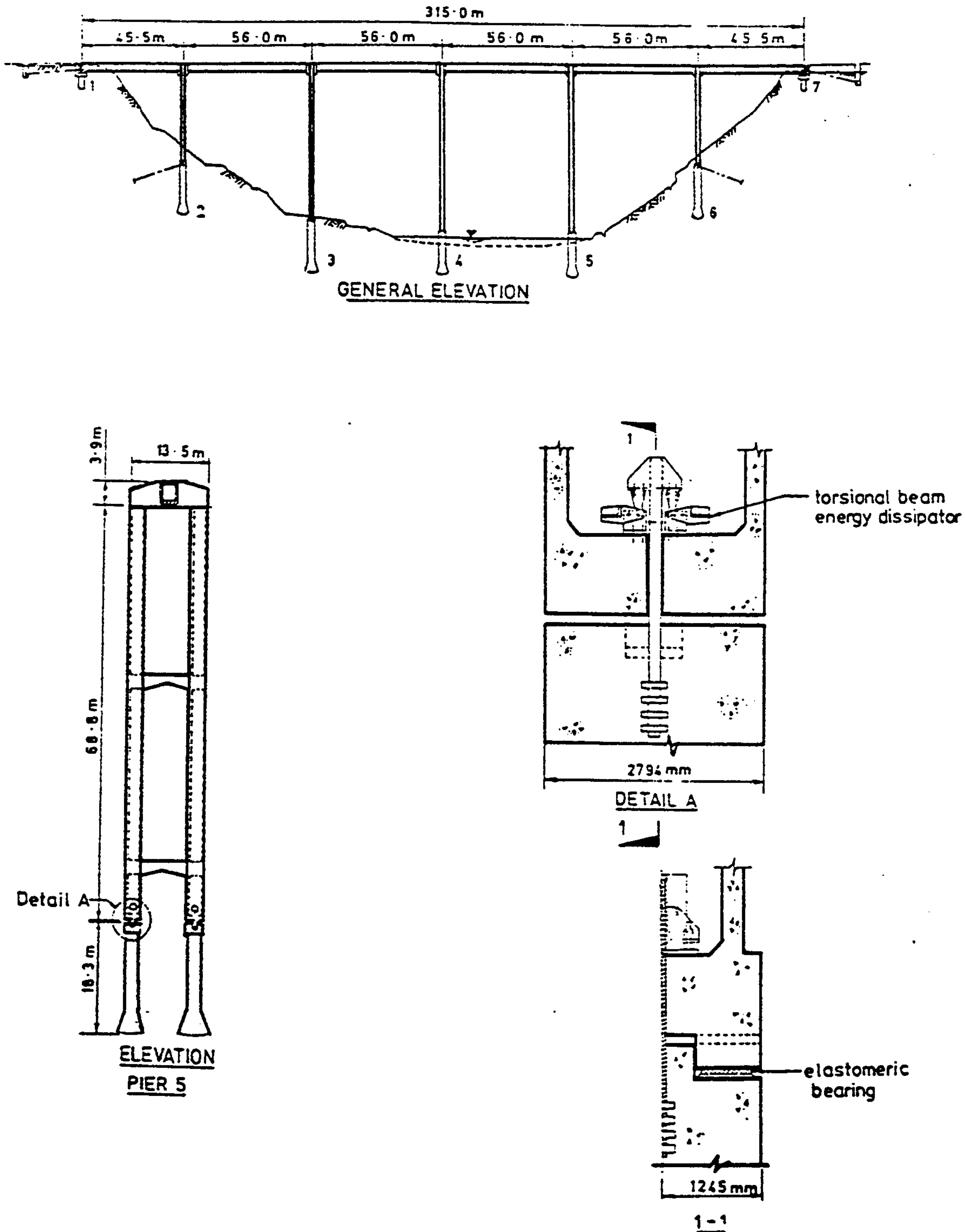
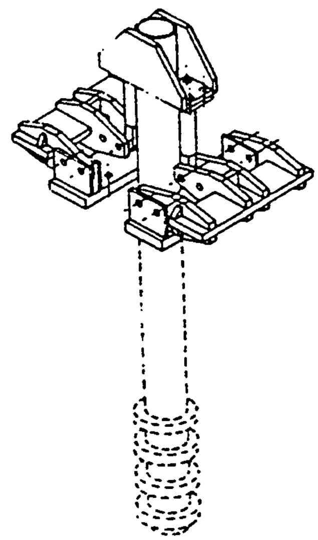
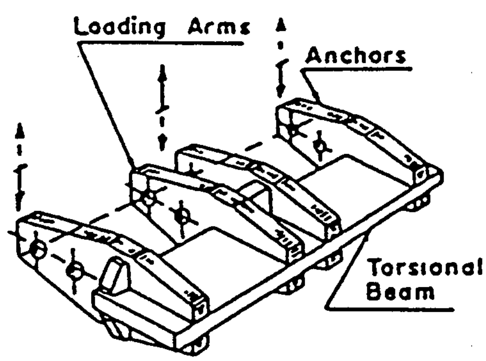


FIGURE 4-12: Torsional Beam Energy Dissipator

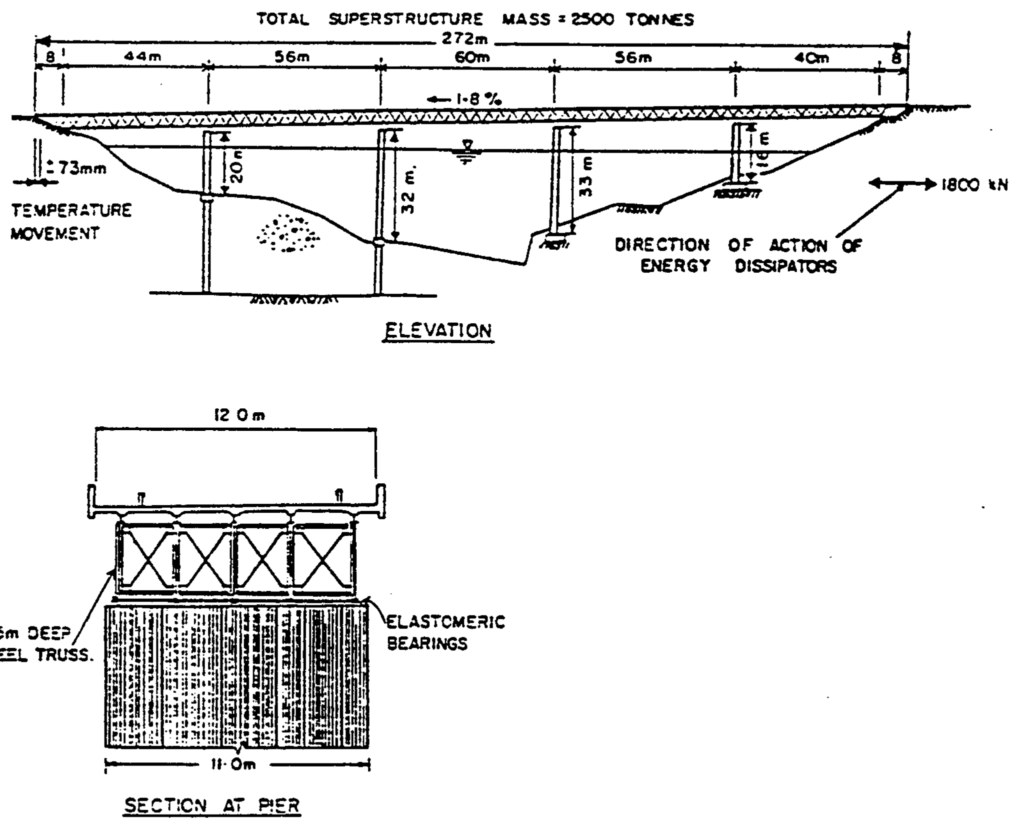


Detail



Torsional Beam Device

FIGURE 4-12: Torsional Beam Energy Dissipator (cont.)



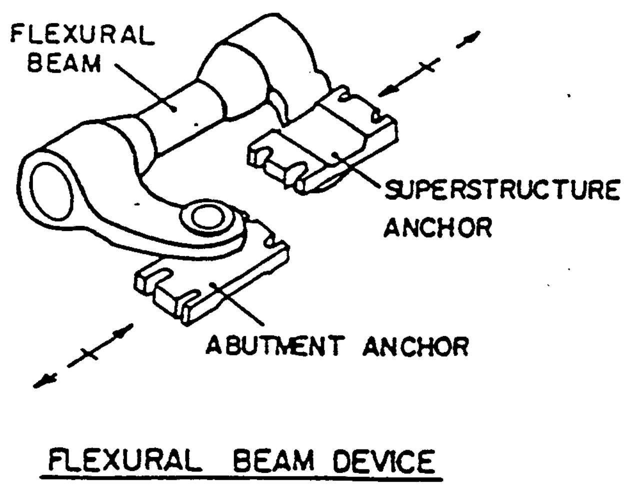


FIGURE 4-13: Flexural Beam Energy Dissipator

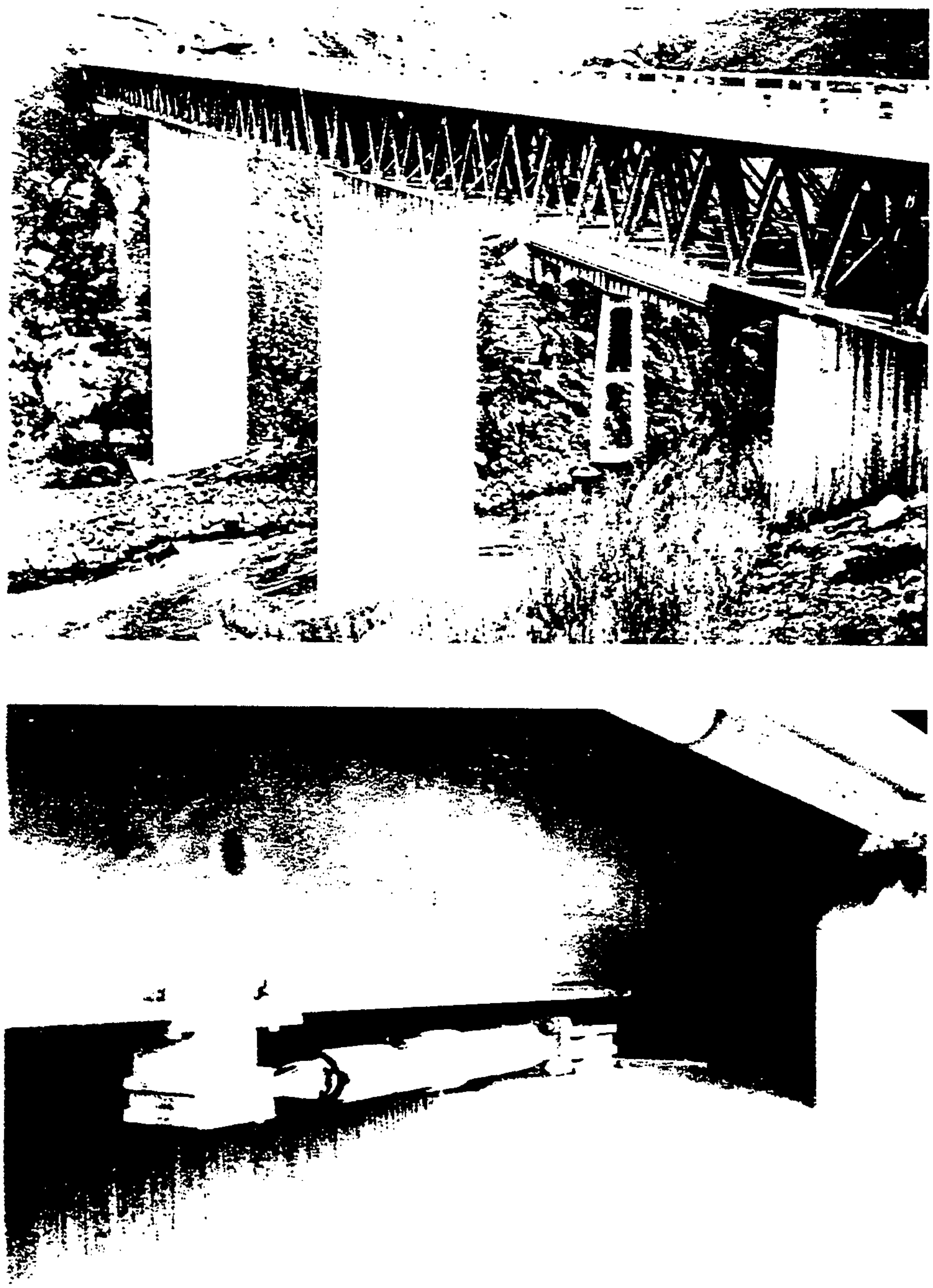


FIGURE 4-14: Bannockburn Bridge – General View and Detail of 150 kN Lead Extrusion Dissipator at Abutment (4 per Abutment)

FIGURE 4-15: Bolton Street and Aurora Terrace Bridges, Wellington

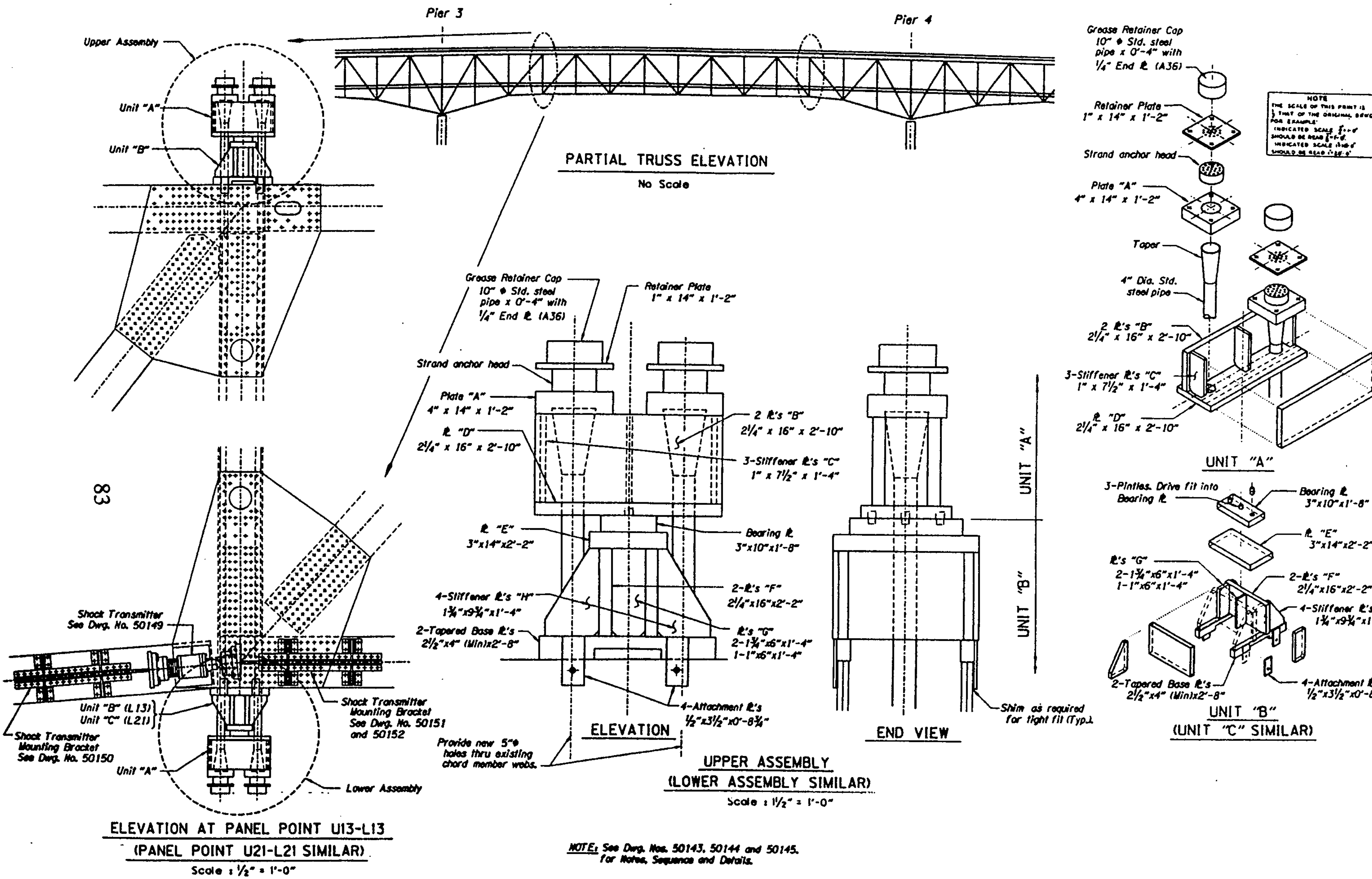


FIGURE 4-16: Shock Transmitter Layout on the Marquam Bridge

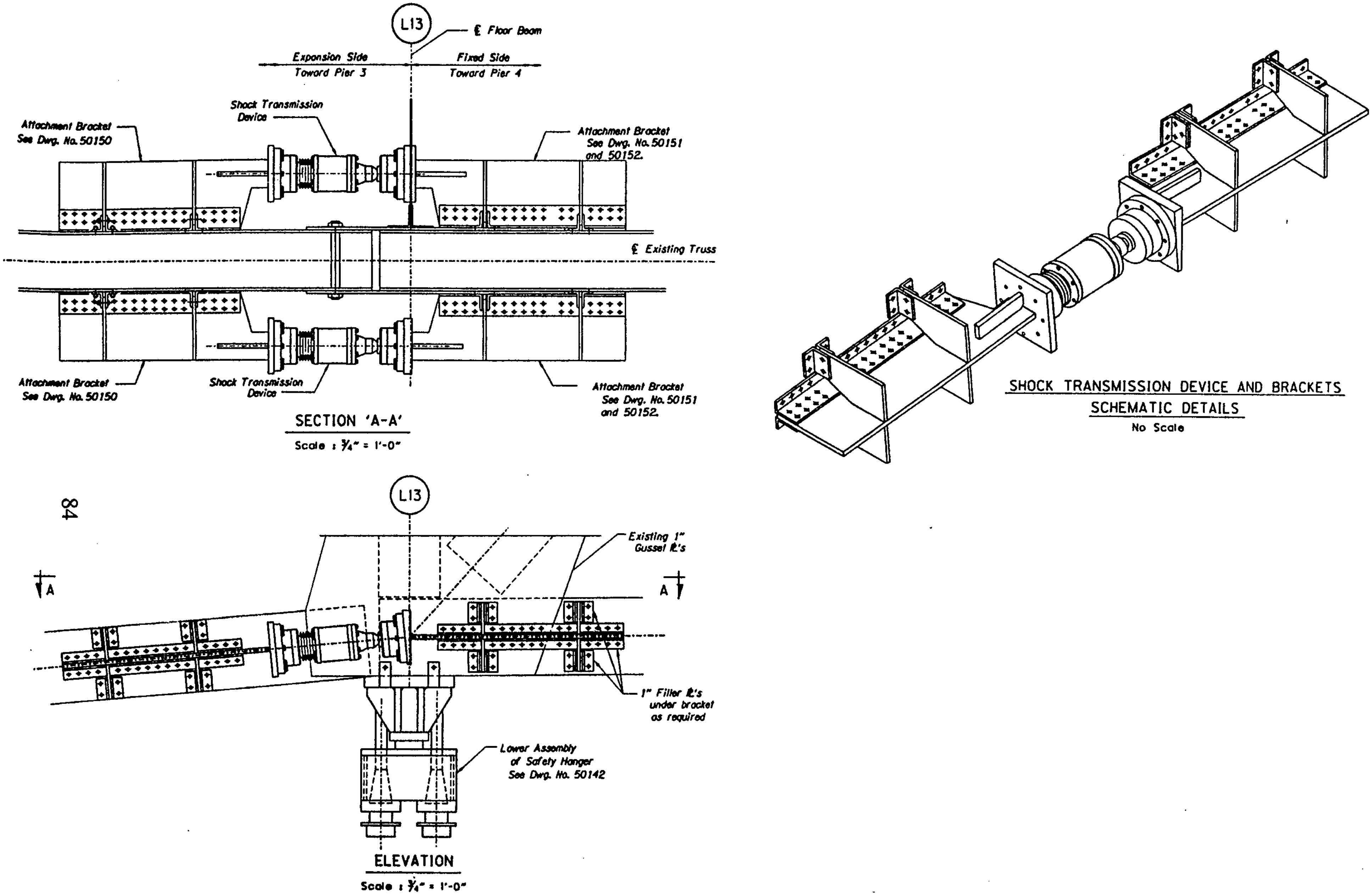


FIGURE 4-17: Shock Transmission Devices on the Marquam Bridge

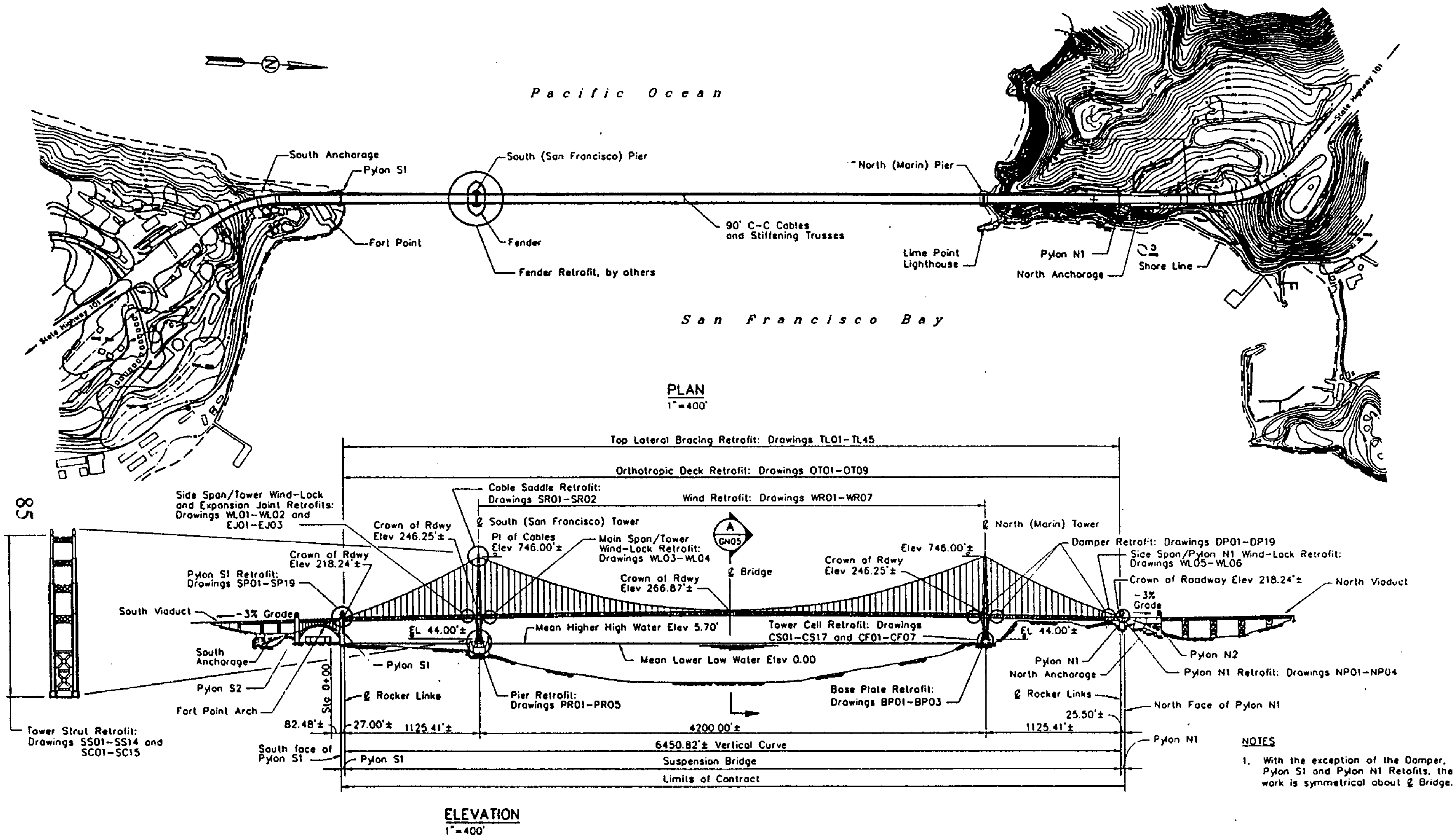


FIGURE 4-18: General Plan of the Golden Gate Bridge Retrofit

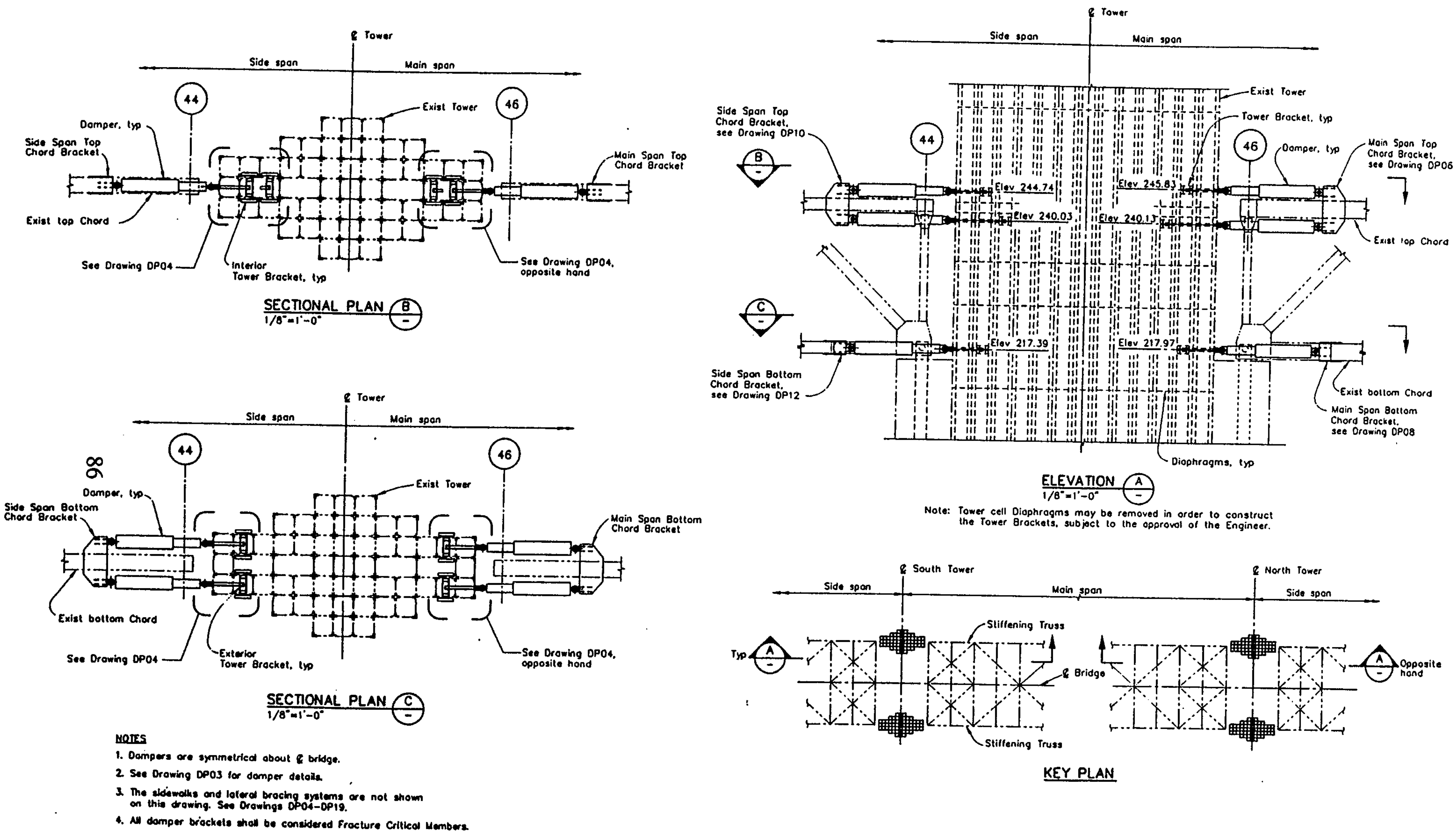


FIGURE 4-19: Dampers – Tower Layout on the Golden Gate Bridge

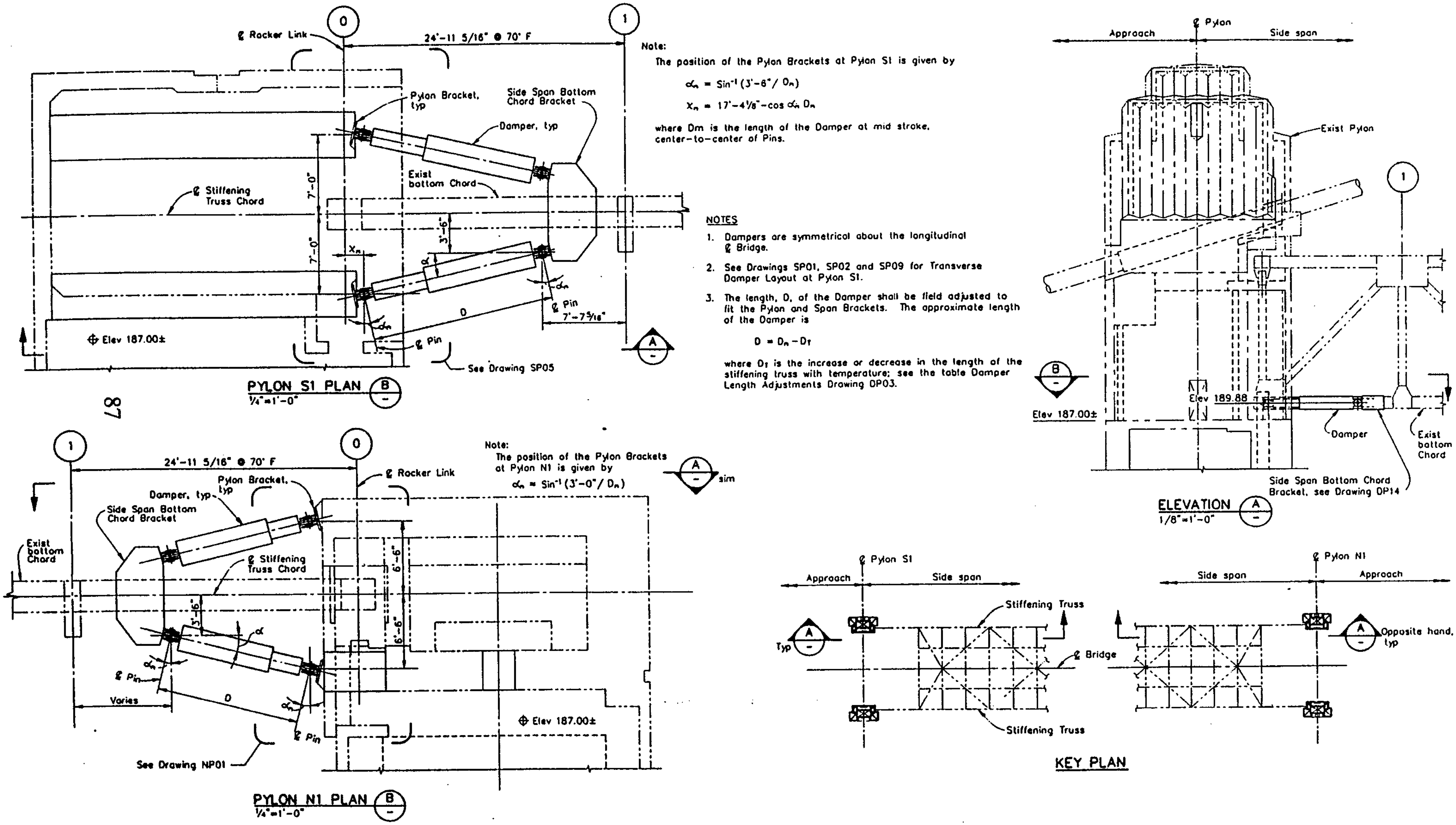


FIGURE 4-20: Damper – Pylon Layout on the Golden Gate Bridge

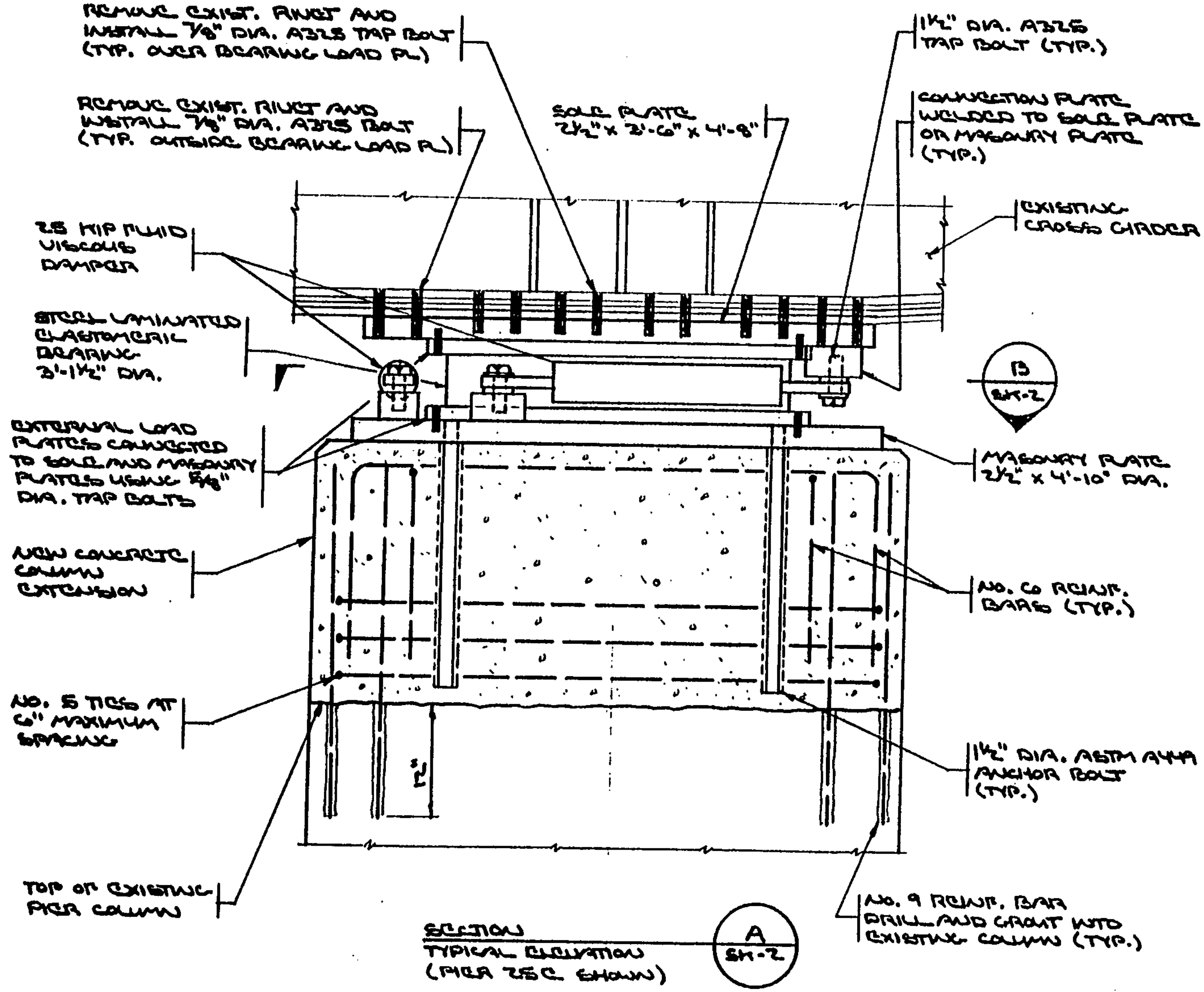


FIGURE 4-21: Martin Luther King Bridge, Elevation of Proposed Viscous Fluid Damper System and Elastomeric Bearing

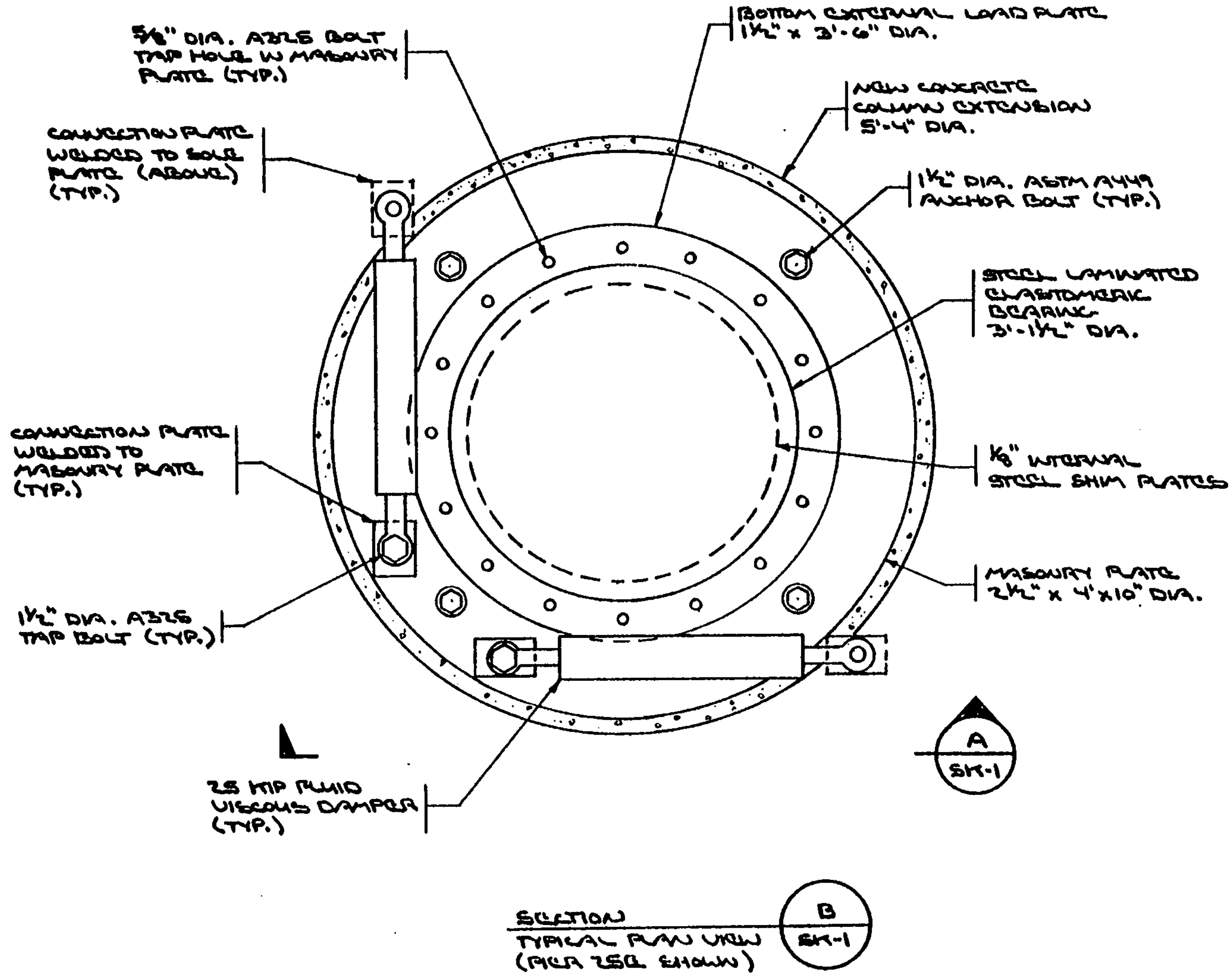


FIGURE 4-22: Martin Luther King Bridge, Plan View of Proposed Viscous Fluid Damper System and Elastomeric Bearing

SECTION 5

MINIMUM SUPPORT LENGTH REQUIREMENTS

5.1 Longitudinal Direction

The minimum support length in the longitudinal direction should be equal to the maximum unreduced earthquake displacement from a dynamic analysis or the nominal seat width N:

where

$N = (8 + 0.02L + 0.08H)$ (inches)

as defined in the AASHTO Seismic Code (AASHTO, 1996b) for Categories A and B, and

$N = (12 + 0.03L + 0.12H)$ (inches)

as defined in the AASHTO Seismic Code for Categories C and D. Caltrans, as well as AASHTO, has suggested the formula should be modified to account for the skew effect with the factor $(1+S^2/8000)$, where S is the angle of the skew in degrees. L is the length in feet of the bridge deck from the abutment to the adjacent expansion joint and H is the average height in feet of columns or piers. Several states are adopting this formula in their seismic design procedures.

5.2 Transverse Direction

There is currently no definition in the design code for the minimum support length which should be provided in the transverse direction. Adequate support length is usually provided by extending the bent cap or shear keys are provided to limit displacements when conventional type bearings are used. However, in the case of isolated bridges a gapped shear key (stopper blocks) is used in the transverse direction. They are treated as a back-up system if the bearing fails or displaces more than the ultimate bearing displacement capacity. See Section 2.3 for further information.

SECTION 6

EARTH RETAINING SYSTEMS

The design of earth retaining structures for seismic load is a subject on which there are few guidelines. In fact, most highway departments in the central and eastern United States do not design retaining walls for seismic loads. Instead they assume, based on previous performance, that static design is adequate. Many feel that the factor of safety provided in the design of walls for static pressures may be adequate to prevent damage or detrimental movements during many earthquakes. Thus, where backfill and foundation soils remain stable, it is only in areas where very strong ground motions might be expected, for walls with sloping backfills or heavy surcharge pressures and for structures which are very sensitive to wall movements, that special seismic design provisions for lateral pressure effects may be necessary (Seed & Whitman, 1970). Even the most detailed seismic design codes, such as the recommendations from the French Association for Seismic Engineering published in 1990, contain only a few rather simplistic rules for the design of retaining walls for seismic loading.

These assumptions appear to be adequate in the case of reinforced earth type structures on the basis of performance. Post earthquake condition inspections of reinforced earth structures following the 1976 Friuli, Italy earthquake (6.4 Richter magnitude, four walls – no damage), the 1983 Akita, Japan earthquake (7.7 Richter magnitude, 20 reinforced earth structures – no damage) and the 1994 Northridge, California earthquake (6.7 Richter magnitude, 21 walls and two abutments – superficial damage to one wall) revealed little difference in the performance characteristics between walls and/or abutments designed for special seismic provisions and those that were not.

Contacts with DOT's in the central and eastern United States reveal that those states that are designing retaining structures for seismic loading are using the pseudo-static Mononabe-Okabe method as outlined in the AASHTO Standard Specifications for Highway Bridges (AASHTO, 1991).

Structural damage to earth retaining structures from past earthquakes has been primarily limited to those walls that were an integral part of the bridge substructure or in close proximity to the bridge. These components were designed to dissipate energy and act as fuses to protect other non-ductile elements. In contrast free standing earth retaining structures have performed very well, suffering only cosmetic damage. The ability of earth retaining structures to act as flexible elements during earthquakes is the key in limiting structural damage. Providing details that allow free unrestricted movement of the structure will minimize repairs.

The earth retaining structures to be discussed will be classified, based on AASHTO Bridge Design Specifications, into four basic groups:

1. Conventional Walls
2. Anchored Walls
3. Mechanically Stabilized Earth Walls
4. Prefabricated Modular Walls

6.1 Conventional Walls

Conventional walls include gravity walls, semi-gravity walls, cantilevered walls, counterfort walls, and buttressed walls. These walls move in phase with the surrounding ground during earthquakes until some critical lateral force occurs resulting in permanent displacement. These displacements can accumulate during the event resulting in an overall permanent displacement.

Earth retaining structures such as bridge wingwalls suffered extensive damage in previous earthquakes. The 1971 San Fernando earthquake caused simple span propped wingwalls to completely separate from their abutments. This allowed backfill material to escape resulting in excessive settlement of the bridge approach. The 1994 Northridge Earthquake caused widespread spalling and cracking to horizontally cantilevered bridge wingwalls. Freestanding cantilever walls performed well except for one instance where the wall translated out 2 inches. It was apparent that the footing embedment did not satisfy minimum design requirements and, therefore, there was a deficiency in passive soil resistance. Another problem associated with the seismic performance of conventional walls is the settlement of material behind the wall. While undesirable, it is primarily caused by the densification of the backfill and, in some cases, the foundation material coupled with small outward movements of the wall.

Currently Caltrans provides standard design drawings for cantilever retaining walls up to 36 feet. The conservative Working Stress Method of design is used with no provision for seismic loading. The wall design has not been updated partly due to the wall's acceptable earthquake performance. As Caltrans transitions to Load and Resistance Design (LRFD), the design will be updated and seismic forces included. Seismic design is required for walls that support vehicular barrier railing or soundwalls. External stability is checked using the Working Stress Design method while internal stability under seismic loading is checked by the Load Factor Design method. The active earth pressure coefficient (Kae) due to earthquake is determined using the Mononobe-Okabe analysis method. The plastic moment capacity of the stem is taken as a factor of 1.3 times the nominal moment capacity. This moment is used to determine the plastic shear at the base of the stem and is checked against the available stem shear capacity. This same shear is also applied directly above the footing along with the stem, barrier, footing, and earth dead load to determine if the resultant force assures overall stability.

Research conducted at the University of California, Irvine, (Haroun, 1994) on structural pier walls may explain the good behavior of the cantilever retaining wall. Physical testing of scaled pier walls showed excellent results in ductility and drift capacity. The slender nature of a pier wall (height to width ratio) resulted in a primarily flexural response when loaded in the weak direction. The geometry of the reinforcement, with two parallel rows at the front and back face of the wall strained all the bars equally. This resulted in a long plastic hinge length that allowed large transverse displacements with little structural degradation. The cantilever retaining wall is also a slender structural element with similar reinforcement geometry, and its behavior should be very close to the pier walls.

Minor detailing modifications to these walls could substantially reduce the cosmetic damage suffered in past earthquakes. Caltrans Standard Plan B3-8, July 1992, shows a 1/2" expansion joint between adjoining wall segments. Widening this joint to 3/4" would reduce the spalling that occurs from the walls banging into each other (see Figure 6-1). The waterstop located at this joint could also be improved to permit greater movements.

An aesthetic detail modification such as a horizontal offset could be added to retaining walls that connect to bridge abutment wingwalls or return walls to limit the visual effect of the permanent offsets and deformations that accompany large earthquakes (see Figure 6-2).

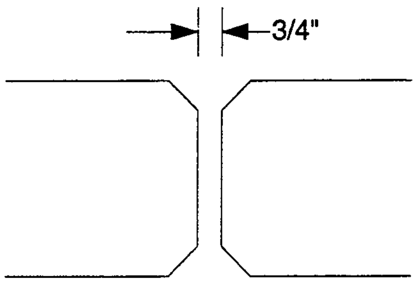


FIGURE 6-1: Plan of Wall with Expansion Joint

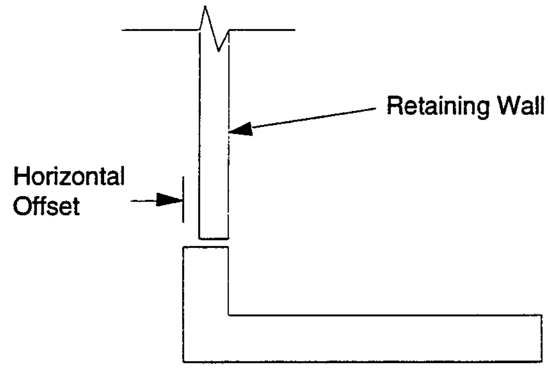


FIGURE 6-2: Plan of Abutment with Horizontal Offset

6-2 Anchored Walls

Anchored walls include active anchors such as prestressed tiebacks or passive anchors such as soil nails. There has been minimal reports of damage from past earthquakes to anchored walls.

The main concern during earthquakes with actively anchored walls is structural damage to the anchorage which could lead to failure. Current Caltrans specifications for prestressed tieback anchors requires wedge lockoff at 150% of the design load to fully seat the strand wedges. The 8" to 12" cast-in-place concrete wall poured over the anchorage area prevents the wedges from unseating during large earthquakes. If no concrete wall is to be placed over the anchorage, a retainer plate can be used to prevent wedge unseating. This retainer plate is routinely used in bridge seismic retrofits that require tensile resistance in column footings that resist overturning from seismic loading. Bolts hold the retainer in place and are torqued to a force greater than the unseating force (see Figure 6-3).

For tieback design loads, the Mononobe-Okabe analysis method is used to determine the earthquake active pressure coefficient. Caltrans typically designs stand-alone tiedback walls for static earth pressures and, where warranted, checks global stability considering horizontal seismic acceleration. One construction step that should be considered is to seat the permanent strand wedges at the level of test load specified.

Soil nail walls have been an economical alternative to active tieback anchors. Many soil nail walls were in close proximity to severe shaking during the Loma Prieta earthquake with no visible damage. The walls had flexible thin facings (4" thick) that appeared to articulate with the earthquake ground motion. For corrosion protection, Caltrans has increased the thickness of the walls to 12" which may cause problems as flexibility is sacrificed. The current cast-in-place wall facing is designed to resist the yield strength of the soil nail in punching shear.

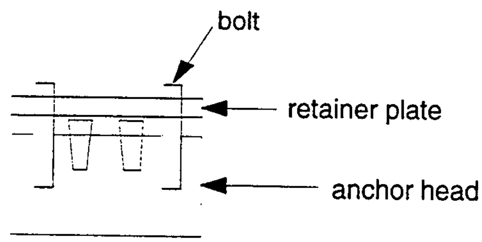


FIGURE 6-3: Retainer Plate for Strand Wedges

6.3 Mechanically Stabilized Embankment (MSE) Walls

This soil retaining system uses either strip or grid-type metallic tensile reinforcement in the soil mass connected to a modular precast concrete facing. These walls have performed well in past earthquakes as they articulate with the movement of the soil. Damage has been limited to superficial spalls and cracks to the precast concrete facing. One MSE wall during the Northridge Earthquake suffered moderate damage at the bottom of the wall which was restricted from lateral movement and rotation by the abutting concrete pavement. There has been no evidence of pull-out or yielding of the embedded tensile reinforcement.

Research conducted at the University of California, Davis, concluded that MSE walls are a viable alternative to conventional walls in seismic environments.(Romstad, 1992) These results were based on comprehensive centrifuge modeling of both the MSE wall and the concrete cantilever retaining wall. It was found that the MSE wall was much more sensitive to the backfill material type than the concrete cantilever. The acceleration force for soundwalls mounted on top of a cantilever wall is twice the acceleration force for the soundwalls mounted on top of MSE wall; however, the permanent displacement for the soundwalls mounted on MSE wall is two to four times that of soundwalls mounted on a cantilever wall. Soundwalls mounted on top of the MSE wall were supplemented by a pile anchor system.

The flexibility of the MSE wall system in an earthquake may produce densification of the reinforced soil mass and the underlying foundation material. Caltrans requires that piling support a bridge abutment that uses MSE walls to retain earth.

For face panels that are located in close proximity of an abutment footing in an area of significant seismic activity, special precautions and design considerations need to be taken. Displacement of an abutment relative to face panels can induce high force resisting demands of the connection of the soil reinforcement to the panel. Failure of these connections could result in a hazard of a falling face panel.

A recommended detail modification would be to provide a clearance between the wall and any element that may restrict movement, such as roadway pavement or concrete slope paving (see Figure 6-4).

6.4 Prefabricated Modular Walls

Prefabricated modular walls have not had significant damage in past earthquakes, but their inability to tolerate large differential displacements is of concern. Caltrans standard details for Crib walls use interlocking Header and Stretcher precast beams and are not detailed for ductility.

Damage in past earthquakes included densification and lateral spreading of the wall. The interlocking precast concrete elements tended to clamp down on one another resulting in cracking.

The height of these walls should be limited in high seismic zones and alternative retaining structures considered for tall walls. The precast elements should be ductility detailed in the area where the header and stretcher interlock.

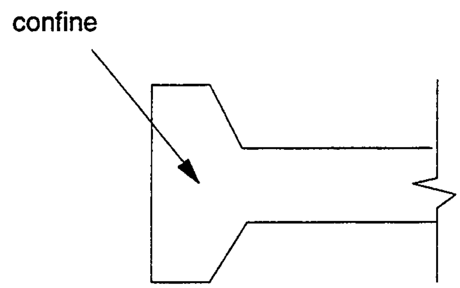


FIGURE 6-4: Elevation – Crib Wall Header

SECTION 7

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APPENDIX A

EFFECT OF SUBSTRUCTURE FLEXIBILITY ON THE ISOLATION SYSTEM RESPONSE

The AASHTO Guide Specifications for Seismic Isolation Design – Draft Report (AASHTO, 1996a) suggested that the substructure flexibility should be included in the calculation of the effective isolation stiffness as shown in Figure A-1 and Figure A-2. The effect of the stiffness (flexibility) of the substructure in the isolation system will lengthen the effective period of the isolation system, which leads to an increase in the displacement of the isolation bearing system. However, if the flexibility of the substructure is not considered the isolation bearings system displacement may be underestimated. In order to account for the substructure flexibility, two cases will be studied and compared.

A.1 Combined System (Isolation System on a Flexible Substructure)

In order to study this case, the following assumptions will be made:

- The substructure force-displacement behavior is linear $\left(K_u^{sub} = K_d^{sub}\right)$ as shown in Figure A-2a.
- The substructure mass is neglected.
- The isolation system force-displacement behavior is bilinear as shown in Figure A-2b.

The initial stiffness of the combined system (K_u^c) (substructure and isolation system) as shown in Figure A.2c is

$$\frac{1}{K_u^c} = \frac{1}{K_u^{sub}} + \frac{1}{K_u^{is}} \text{ (two springs in series)} \tag{A.1}$$

which can be written also as

$$K_u^c = \frac{K_u^{sub} \cdot K_u^{is}}{K_u^{sub} + K_u^{is}} \tag{A.2}$$

and the post-elastic stiffness of the combined system (K_d^c) is

$$K_d^c = \frac{K_d^{sub} \cdot K_d^{is}}{K_d^{sub} + K_d^{is}} \tag{A.3}$$

define

$$\alpha = \frac{K_u^{sub}}{K_d^{is}} = \frac{K_d^{sub}}{K_d^{is}} \tag{A.4}$$

and

$$ß = \frac{K_u^{is}}{K_d^{is}} \tag{A.5}$$

where the superscripts are defined as "sub = substructure" and "is = isolation". Substituting Equations A.4 and A.5 into Equations A.2 and A.3, we obtain,

$$K_u^c = \frac{\alpha\ ß}{\alpha + ß} K_d^{is} \tag{A.6}$$

and

$$K_d^c = \frac{\alpha}{1+\alpha} K_d^{is} \tag{A.7}$$

The effective stiffness of the combined system as shown in Figure A.2c $\left(K_{eff}^c\right)$ is equal to the maximum bearing force, (F_{max}), divided by the maximum isolation displacement, d_{is},

$$K_{eff}^c = \frac{F_{max}}{d_{is}} \tag{A.8}$$

where

$$F_{max} = K_u^c\, d_y + K_d^c (d_{is} - d_y) \tag{A.9}$$

let

$$\mu = \frac{d_{is}}{d_y} \tag{A.10}$$

Substitute Equations A.6, A.7 and A.10 into Equation A.9, F_{max} becomes

$$F_{max} = \left[\frac{\alpha ß}{\alpha + ß}\frac{1}{\mu} + \frac{\alpha}{1+\alpha}\left(\frac{\mu - 1}{\mu}\right)\right] K_d^{is}\, d_{is} \tag{A.11}$$

and Equation A.8 becomes

$$\frac{K_{eff}^c}{K_d^{is}} = \frac{\alpha ß}{\alpha + ß}\frac{1}{\mu} + \frac{\alpha}{\alpha + 1}\left(\frac{\mu - 1}{\mu}\right) \tag{A.12}$$

A.2 Non-Combined System (Isolation System on a Rigid Substructure)

If the effect of substructure flexibility is not considered, then the effective stiffness will be

$$\frac{K^{is}_{eff}}{K^{is}_{d}} = \frac{\beta}{\mu} + \frac{\mu - 1}{\mu} \tag{A.13}$$

To quantify the effect of substructure flexibility on the Isolation Effective Period

let

$$T^{c}_{eff} = 2\pi\sqrt{\frac{M}{K^{c}_{eff}}} \tag{A.14}$$

$$T^{is}_{eff} = 2\pi\sqrt{\frac{M}{K^{is}_{eff}}} \tag{A.15}$$

Substituting Equations A.12 and A.13 into Equations A.14 and A.15 and taking the ratio of the two, we obtain,

$$\frac{T^{c}_{eff}}{T^{is}_{eff}} = \sqrt{\frac{K^{is}_{eff}}{K^{c}_{eff}}} = \sqrt{\frac{\beta + (\mu - 1)}{\frac{\alpha\beta}{\alpha + \beta} + \frac{\alpha}{1 + \alpha}(\mu - 1)}} \tag{A.16}$$

Equation A.16 represents the ratio by how much the period of the combined system has been lengthened by including the effect of the substructure flexibility. Different combinations of α and β have been used in Equation A.16 to see the effect of the substructure flexibility on the isolation system as shown in Table A-1 and Figure A-3.

A.3 Verification Problem (Example)

In order to verify the effect of substructure flexibility on the isolation system response and validate our assumptions used in deriving Equation A.16, a verification problem is presented. A fully base-isolated bridge shown in Figure A-4, which is located in Benicia, California, will be used in the study. This toll bridge is being retrofitted to a maximum credible earthquake. Although this is for a retrofitted structure its application for substructure flexibility can be extended to new bridge designs. The superstructure of the multi-span bridge is composed of six steel girders seated on six isolation bearings per bent as shown in Figure A-5. It is assumed stiff in the longitudinal and transverse directions compared with the stiffness of the isolation bearings and substructure (bents). The substructure is composed of two-column bents founded on piles. The bent's rigidity varies from very stiff (i.e., Pier 16) to very flexible (i.e., Pier 13). Three different bents, Pier 16, Pier 14 and Pier 13, were used in this study, with the same section properties, but with different column heights as shown in Table A-2. Each pier (bent) is modeled as a simplified model in the transverse direction with a superstructure tributary weight equal to 650 kips per bearing. The bent is modeled as a linear beam element and the isolation bearings as bilinear spring elements with the properties summarized in Table A-2. A time history analysis was conducted using a site specific time history acceleration input as shown in Figure A-6. The maximum displacement responses are summarized in Table A-3 for the three bents. The time history displacement is plotted in Figure A-7 to Figure A-9. As we can see from Table A-3, the effect of the substructure on the isolation response is minimum for the case of Pier 16 (stiff pier); however, it is substantial in the case of Pier 13 (flexible) where the bearing displacement increased by about 1.5 times ($\frac{21.3}{14.2} = 1.5$).

In order to verify the validity of Equation A-16, the effective stiffnesses and their associated periods of the isolation system for the three bents were determined (see Table A-4) using the maximum relative bearing displacement obtained from the time history analysis as shown in Table A-3. By comparing them to the effective isolation periods calculated based on Equation A-16 as shown in Table A-5. Equation A.16 predicted quite well the period lengthening of the isolation system due to the substructure flexibility.

A.4 Conclusion

The effect of the substructure flexibility was investigated and the following conclusions can be derived.

The effect of the substructure is almost negligible for the case of the stiff substructure on the isolation system responses. However, in the case of a tall column bent the effect of the substructure flexibility can be substantial and one should incorporate it in the determination of the isolation system responses.

The derived Equation A.16 gives a good indication by how much the effective isolation period can be lengthened due to the flexibility of the substructure.

TABLE A-1 Effect of Substructure Flexibility on the Isolation System

$\mu = \frac{d_{is}}{d_y}$ (Eqn. 10)	$\alpha = \frac{K_u^{sub}}{K_d^{is}}$ (Eqn. 4)	$\beta = \frac{K_u^{is}}{K_d^{is}}$ (Eqn. 5)	$\frac{\alpha}{\beta} = \frac{K_u^{sub} = K_d^{sub}}{K_u^{is}}$ (Eqn. 4)/(Eqn. 5)	T_{eff}^c / T_{eff}^{is} * (Eqn. 16)
1	1	1	1	1.4142
2	3	1	3	1.1547
5	10	1	10	1.0488
10	50	1	50	1.0100
20	80	1	80	1.0062
30	100	1	100	1.0050
1	20	20	1	1.4142
2	60	20	3	1.1462
5	200	20	10	1.0406
10	1000	20	50	1.0070
20	1600	20	80	1.0033
30	2000	20	100	1.0022
1	100	100	1	1.4142
2	300	100	3	1.1528
5	1000	100	10	1.0468
10	5000	100	50	1.0091
20	8000	100	80	1.0052
30	10000	100	100	1.0039

*T isolation (flexible substructure)/T (isolation rigid substructure)

TABLE A-2 Simplified Model Properties

		Column Properties		Bent Cap Properties		Single Bearing Property			
Bent	**Col. Height (ft)**	**A (ft^2)**	**$I=0.5I_g$ (ft^4)**	**A (ft^2)**	**Ig (ft^4)**	**Ku (K/in)**	**Kd (K/in)**	**Fy (Kip)**	**Dy (in)**
16	33	64.5	179.2	110	916	72.6	7.1	43	0.59
14	61.5	64.5	179.2	110	916	72.6	7.1	43	0.59
13	123	64.5	179.2	110	916	72.6	7.1	43	0.59

TABLE A-3 Simplified Model Time History Results

		Maximum Displacement			
Bent	**Col. Height (ft)**	**Bent (in)**	**Deck (in)**	**Bearing (in)**	**Comments**
16	33	0.7	14.3	14.2	Rigid Substructure
14	61.5	2.5	16.3	16.9	
13	123	16.2	25.5	21.3	Flexible Substructure

TABLE A-4 Simplified Model Natural Period

Bent	Col. Height (ft)	SDOF Natural Period (sec)		Simplified Model, Natural Period (sec)		Teff/Teff * Model
		Bent Alone (w/out deck)	Deck on Isolation (rigid substructure)	Bent Mode	Deck on Isolation Mode	
16	33	0.2	2.63	0.21	2.63	1.00
14	61.5	0.48	2.63	0.46	2.79	1.06
13	123	1.49	2.63	1.14	3.61	1.37

*T isolation (flexible substructure)/T isolation (rigid substructure)

TABLE A-5 Effect of Substructure Flexibility (Eqn. 16)

Bent	Col. Height (ft)	Alpha (Eqn. 4)	Beta (Eqn. 5)	Max. Disp. (time history)	Max. Bear. Ductility (Eqn. 10)	Teff/Teff * (Eqn. 16)
16	33	90	10.25	14.2	24	1.00
14	61.5	19	10.25	16.9	29	1.06
13	123	2.9	10.25	21.3	40	1.27

*T isolation (flexible substructure)/T isolation (rigid substructure)

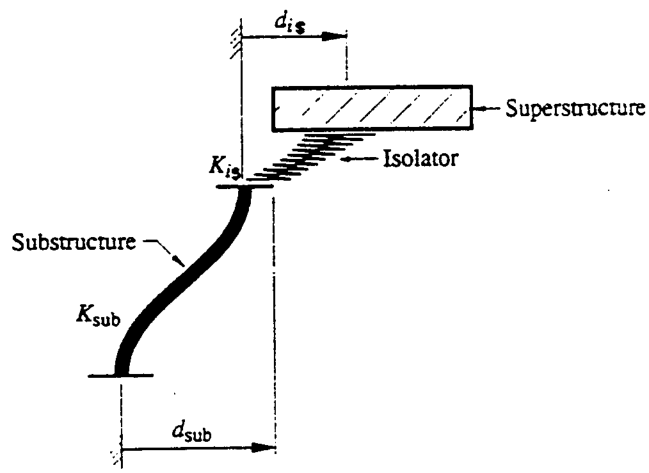


FIGURE A-1: Substructure Flexibility included in Isolation System

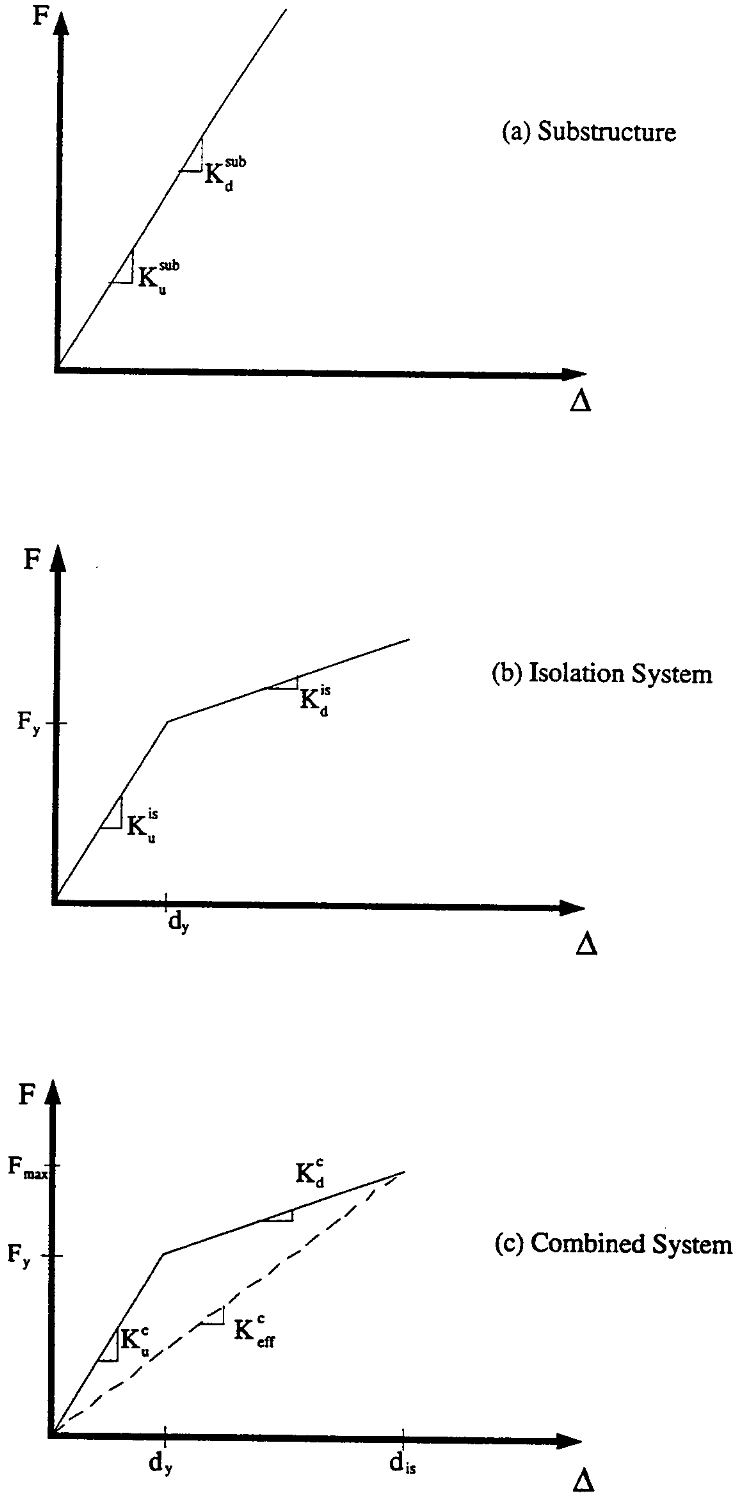


FIGURE A-2: Force – Displacement Characteristics

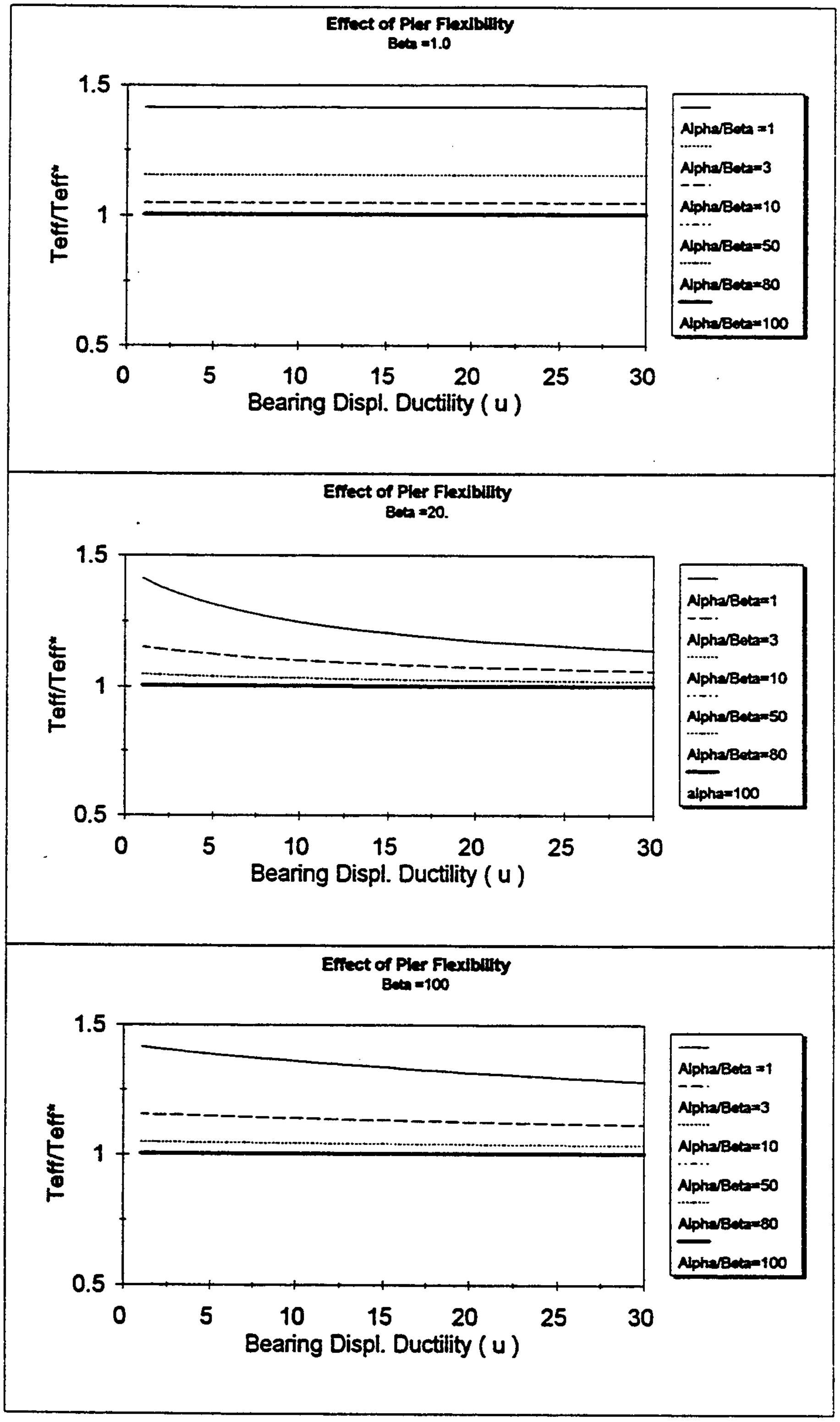


FIGURE A-3: Effective Isolation Period vs. Bearing Ductility Displacement

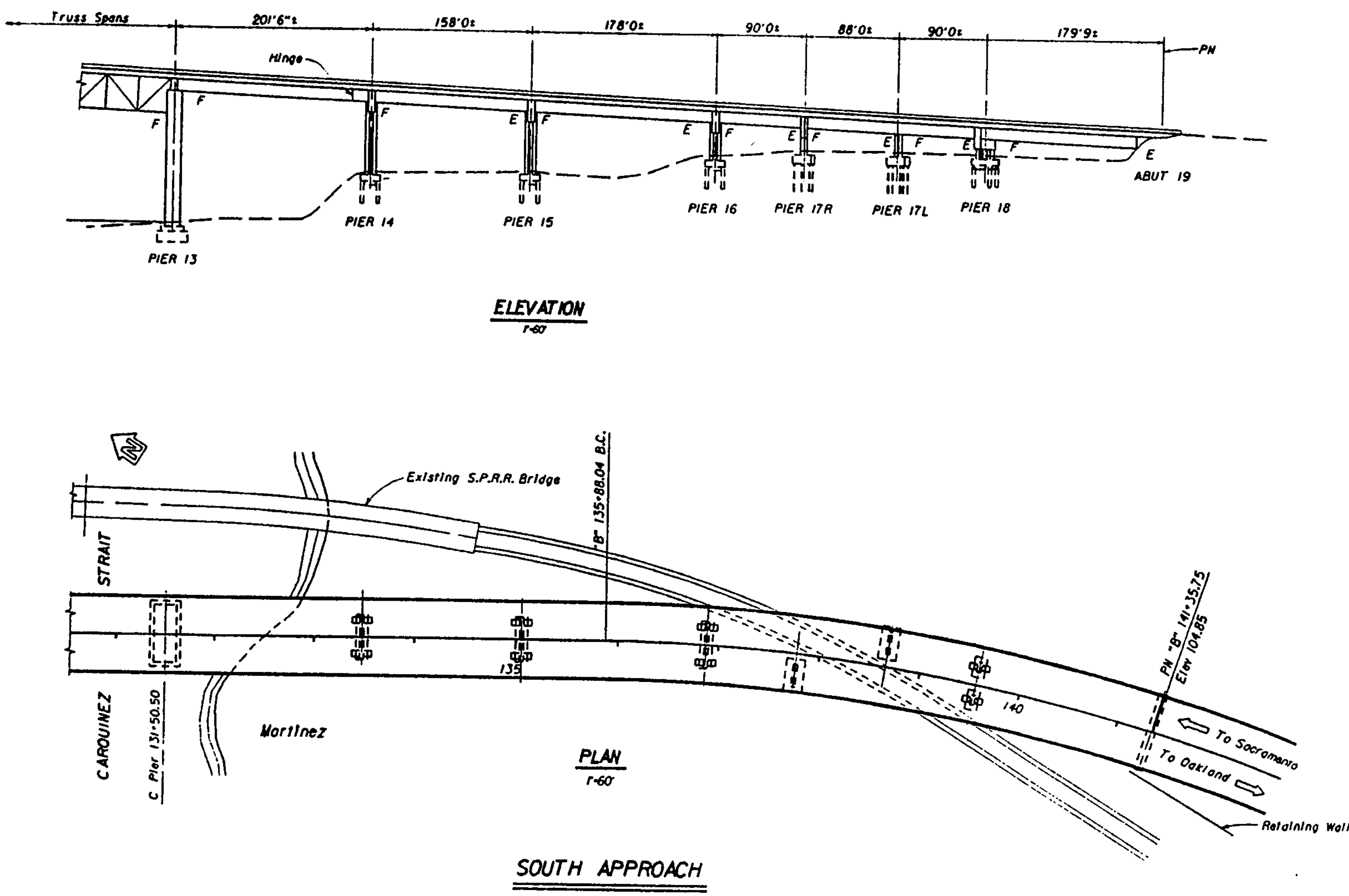
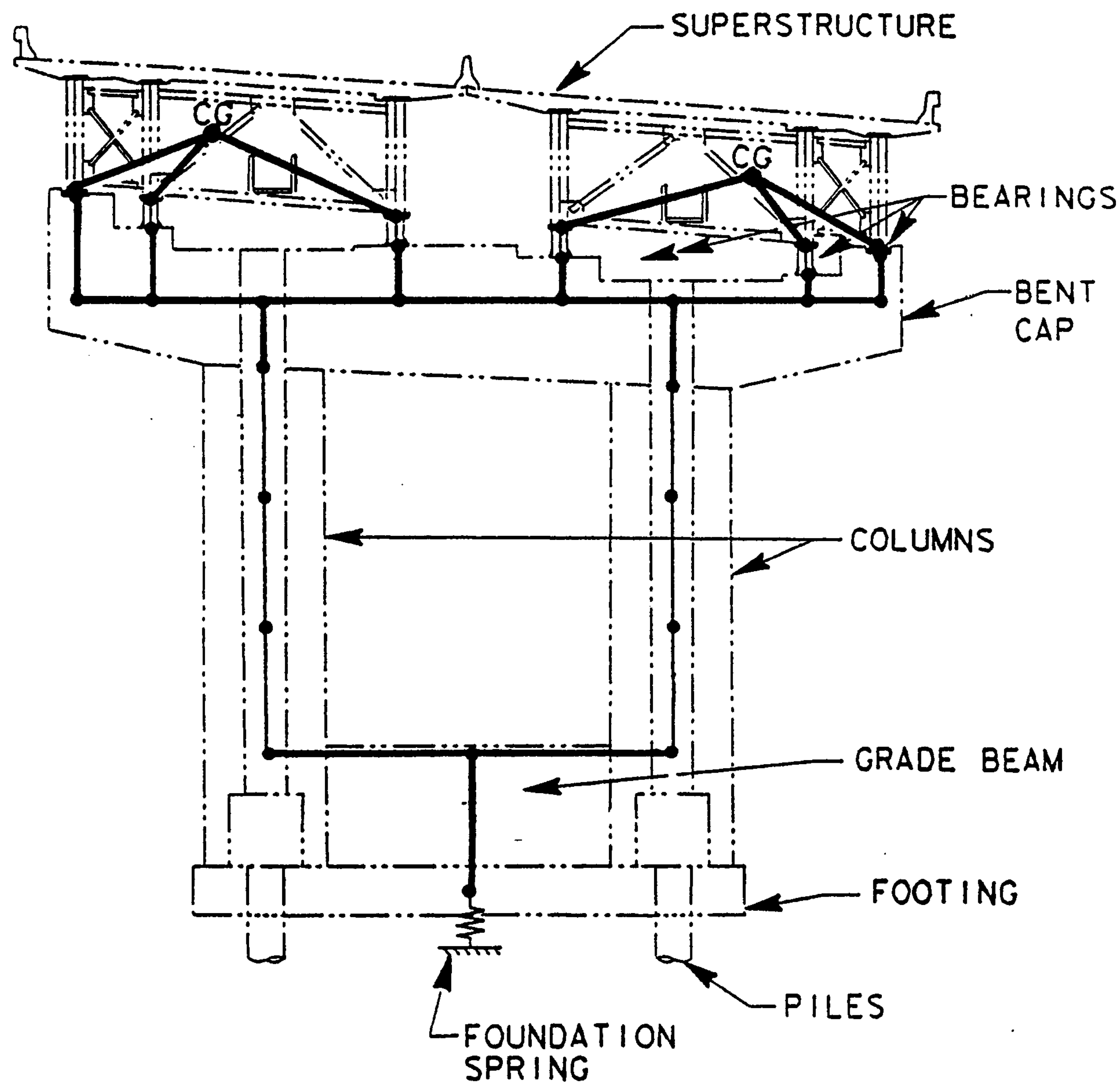


FIGURE A-4: Piers of Benicia-Martinez South Approach used in Study for the Effect of Pier Flexibility on the Isolation System



Pier	Column Height (ft)
16	33
14	61.5
13	123

FIGURE A-5: Model and Bent Configuration of Piers 16, 14 and 13

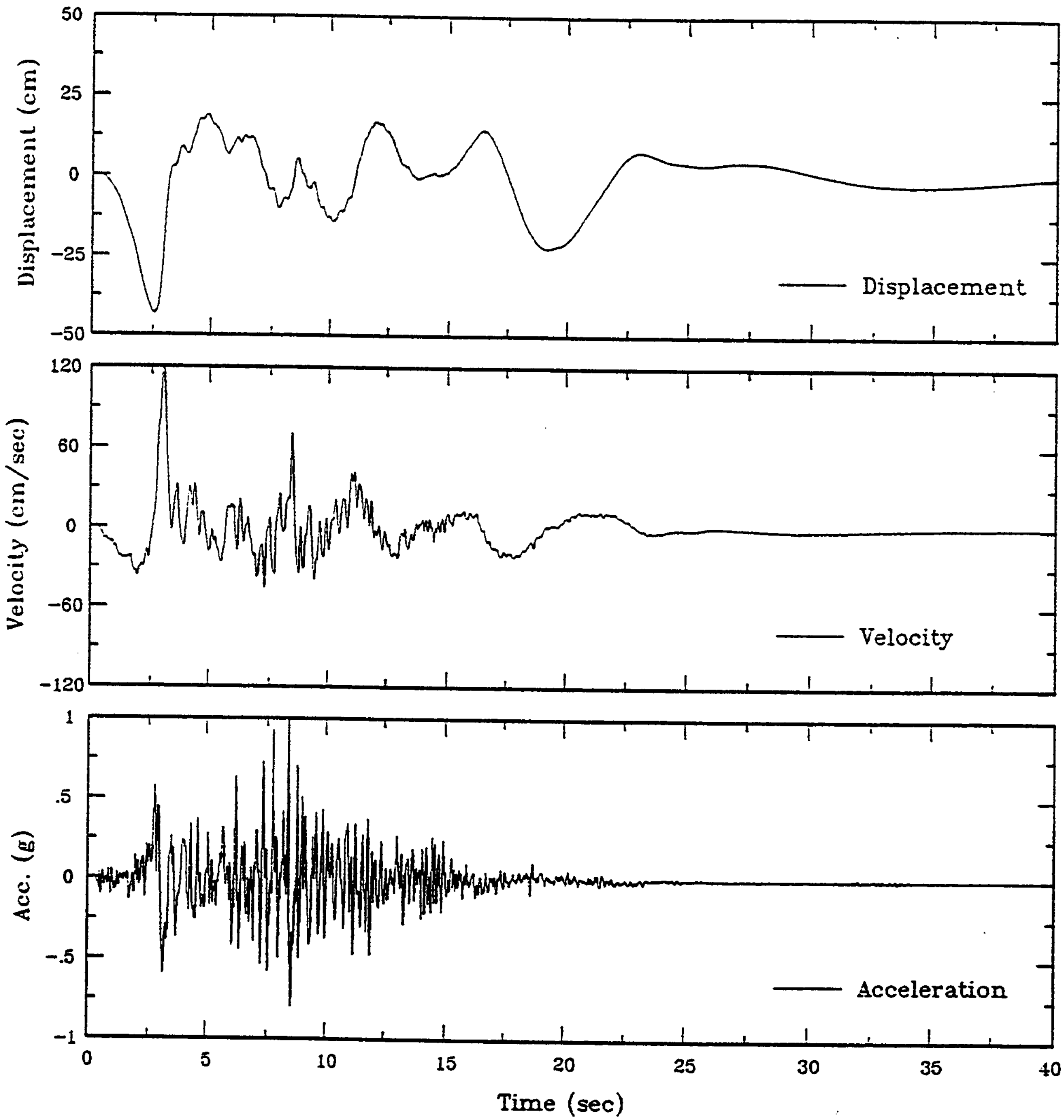


FIGURE A-6: Revised Spectrum-Compatible Rock Time Histories Associated with the Green Valley Event (Transverse Component)

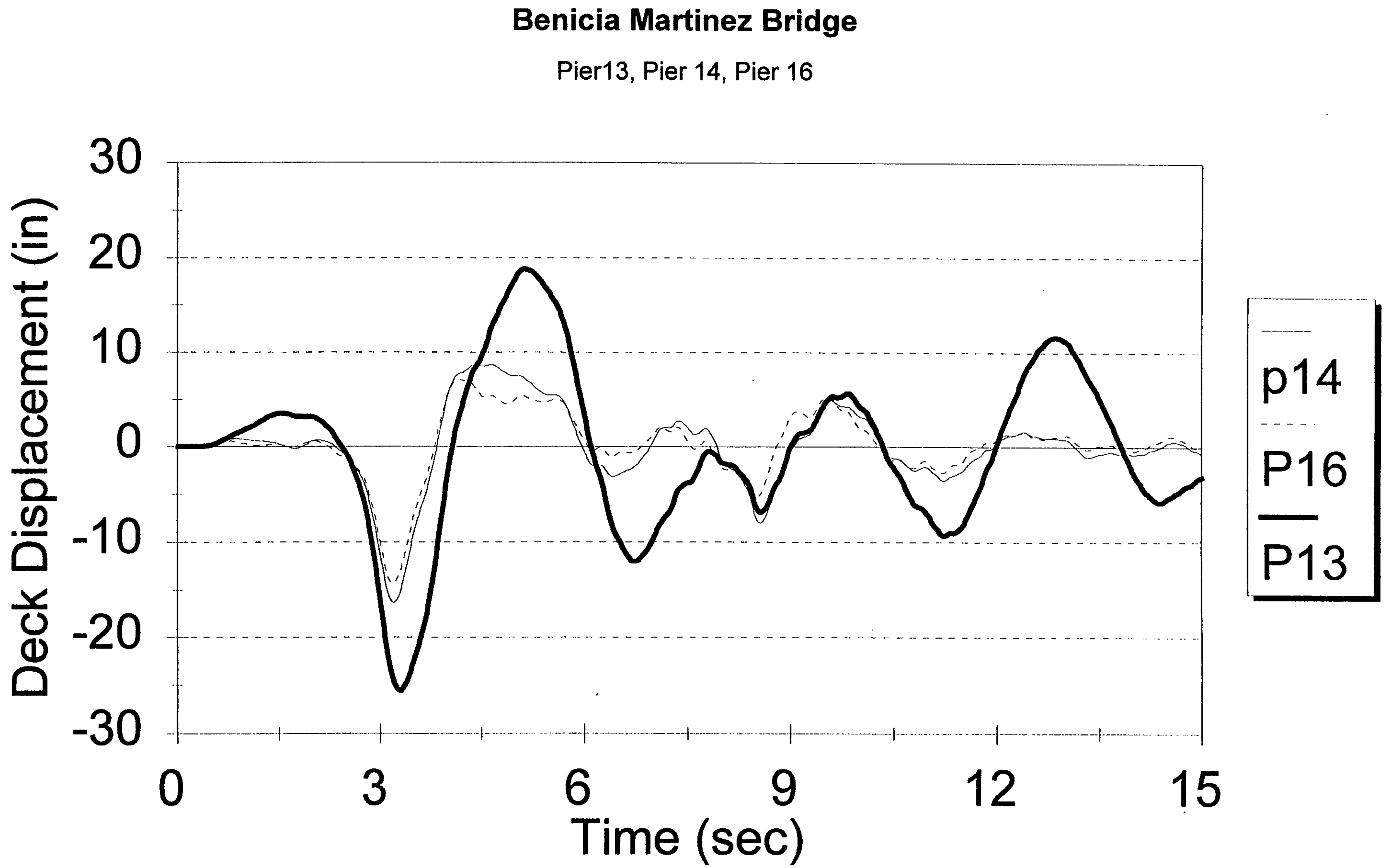


FIGURE A-7: Deck Displacement Time History Simplified Model

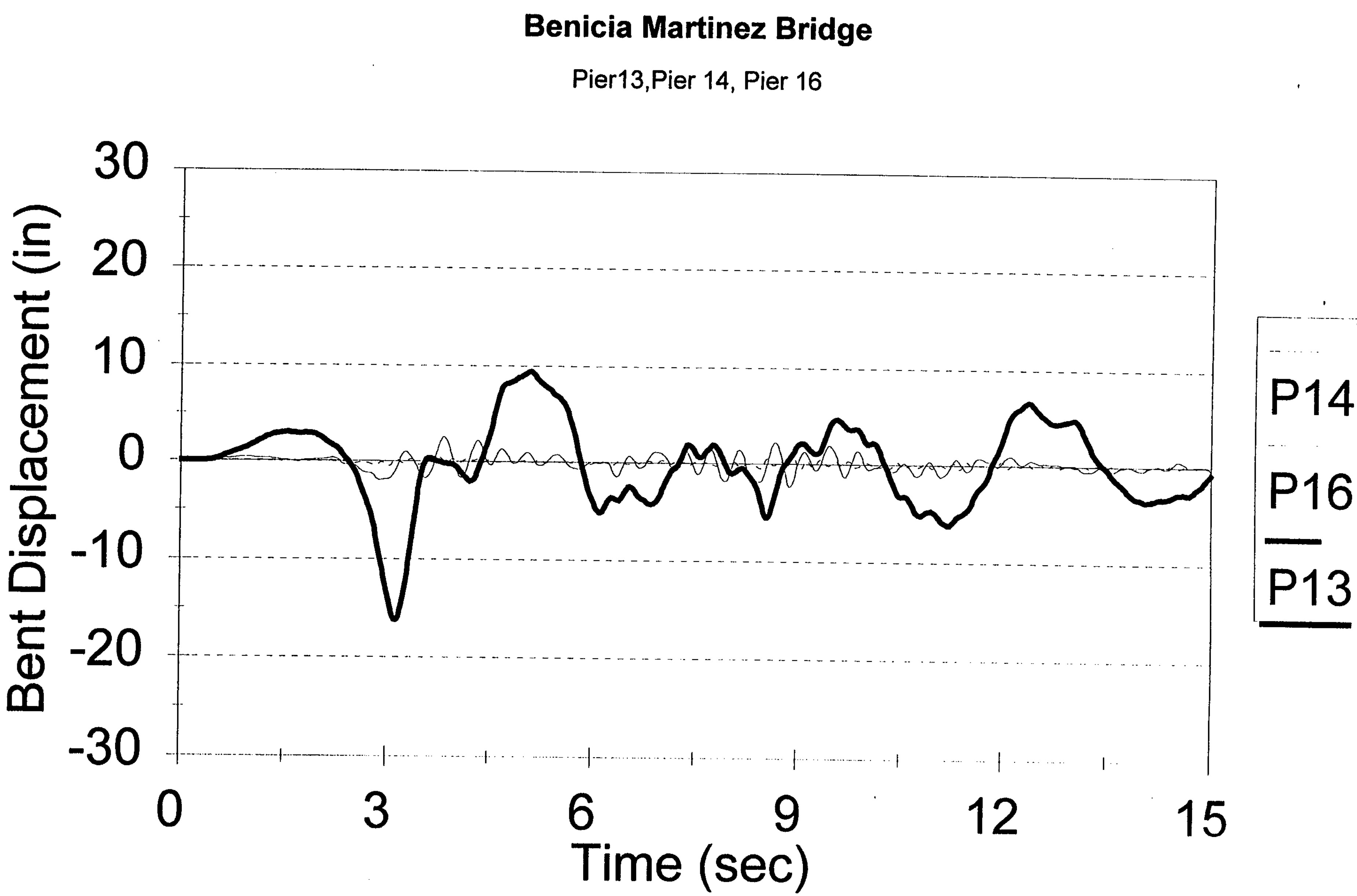


FIGURE A-8: Bent Displacement Time History Simplified Model

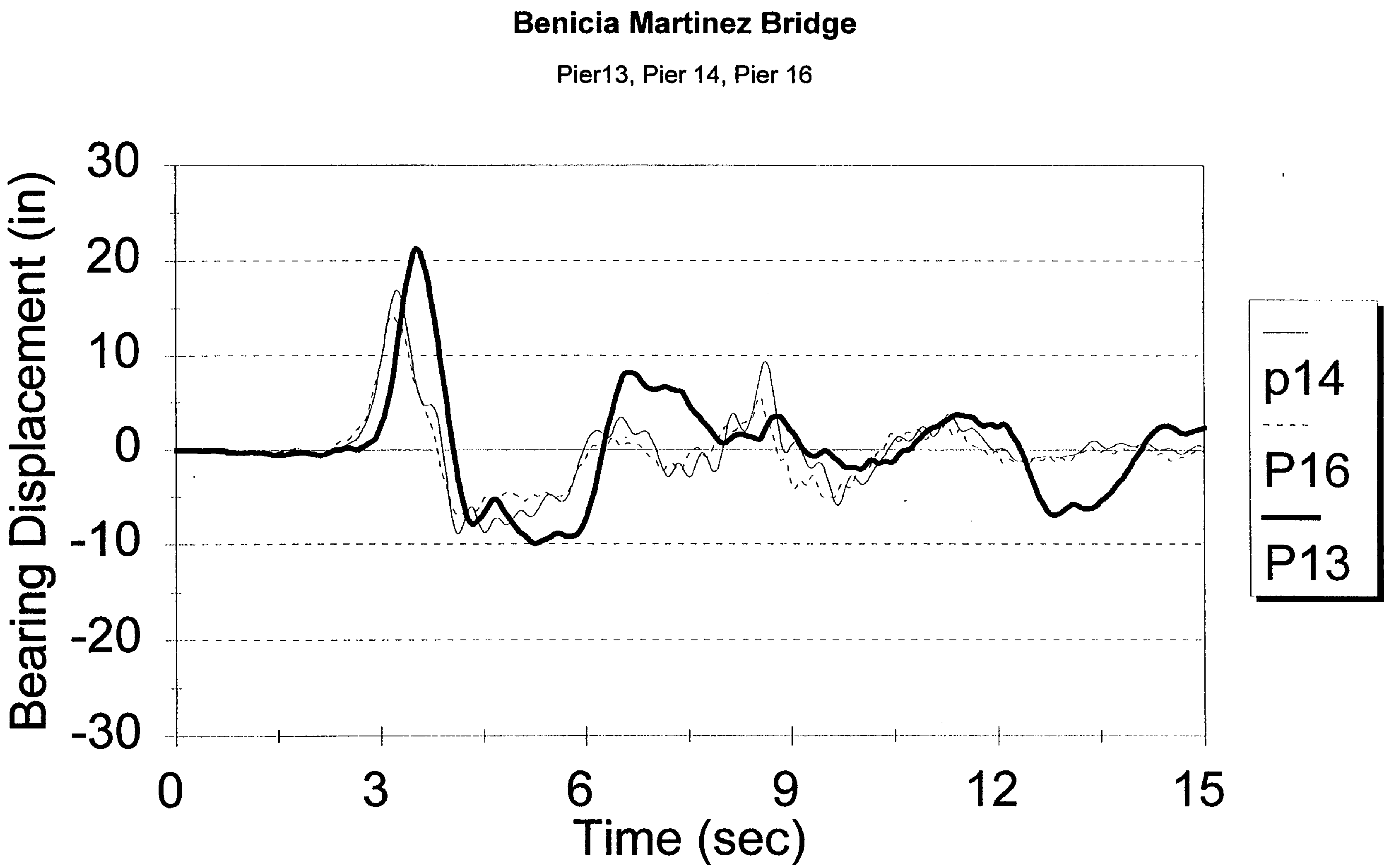


FIGURE A-9: Bearing Relative Displacement Time History Simplified Model

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