Keady, K.I., Alameddine, F., Sardo, T.E. "Seismic Retrofit Technology."
Bridge Engineering Handbook.
Ed. Wai-Fah Chen and Lian Duan
Boca Raton: CRC Press, 2000

43

Seismic Retrofit Technology

Kevin I. Keady
California Department of Transportation

Fadel Alameddine
California Department of Transportation

Thomas E. Sardo
California Department of Transportation

43.1 Introduction

Upon completion of the seismic analysis and the vulnerability study for existing bridges, the engineer must develop a retrofit strategy to achieve the required design criteria. Depending on the importance of structures, there are two levels of retrofit. For ordinary structures, a lower level of retrofit may be implemented. The purpose of this level of retrofit is to prevent collapse. With this level of retrofit, repairable damage is generally expected after a moderate earthquake. Following a major earthquake, extensive damage is expected and replacement of structures may be necessary. For important structures, a higher level of retrofit could be required at a considerably higher cost. The purpose of this level of retrofit is not only to prevent collapse, but also to provide serviceability after a major earthquake.

There are two basic retrofit philosophies for a concrete bridge. The first is to force plastic hinging into the columns and keep the superstructure elastic. This is desirable because columns can be more easily inspected, retrofitted, and repaired than superstructures. The second is to allow plastic hinging in the superstructure provided that ductility levels are relatively low and the vertical shear load-carrying capacity is maintained across the hinge. This is desirable when preventing hinging in the superstructure is either prohibitively expensive or not possible. In other words, this strategy is permissible provided that the hinge in the superstructure does not lead to collapse. To be conservative, the contribution of concrete should be ignored and the steel stirrups need to be sufficient to carry 1.5 times the dead-load shear reaction if hinging is allowed in the superstructure.

There are two basic retrofit philosophies for steel girder bridges. The first is to let the bearings fail and take retrofit measures to ensure that the spans do not drop off their seats and collapse. In this scenario, the bearings act as a "fuse" by failing at a relatively small seismic force and thus protecting the substructure from being subjected to any potential larger seismic force. This may be

TABLE 43.1 Seismic Performance Criteria

Ground Motion at Site	Minimum performance level	Important bridge performance level
Functional Evaluation	Immediate service level repairable damage	Immediate service level minimal damage
Safety Evaluation	Limited service level significant damage	Limited service level repairable damage

the preferred strategy if the fusing force is low enough such that the substructure can survive with little or no retrofit. The second philosophy is to make sure that the bearings do not fail. It implies that the bearings transfer the full seismic force to the substructure and retrofitting the substructure may be required. The substructure retrofit includes the bent caps, columns or pier walls, and foundations. In both philosophies, a superstructure retrofit is generally required, although the extent is typically greater with the fixed bearing scheme.

The purpose of this chapter is to identify potential vulnerabilities to bridge components and suggest practical retrofit solutions. For each bridge component, the potential vulnerabilities will be introduced and retrofit concepts will be presented along with specific design considerations.

43.2 Analysis Techniques for Bridge Retrofit

For ordinary bridges, a dynamic modal response spectrum analysis is usually performed under the input earthquake loading. The modal responses are combined using the complete quadratic combination (CQC) method. The resulting orthogonal responses are then combined using the 30% rule. Two cases are considered when combining orthogonal seismic forces. Case 1 is the sum of forces due to transverse loading plus 30% of forces due to longitudinal loading. Case 2 is the sum of forces due to longitudinal loading plus 30% of forces due to transverse loading.

A proper analysis should consider abutment springs and trusslike restrainer elements. The soil foundation structure interaction should be considered when deemed important. Effective properties of all members should be used. Typically, two dynamic models are utilized to bound the assumed nonlinear response of the bridge: a "tension model" and a "compression model." As the bridge opens up at its joints, it pulls on the restrainers. In contrast, as the bridge closes up at its joints, its superstructure elements go into compression.

For more important bridges, a nonlinear time history analysis is often required. This analysis can be of uniform support excitation or of multiple supports excitation depending on the length of the bridge and the variability of the subsurface condition.

The input earthquake loading depends on the type of evaluation that is considered for the subject bridge. Table 43.1 shows the seismic performance criteria for the design and evaluation of bridges developed by the California Department of Transportation [1]. The safety evaluation response spectrum is obtained using:

1. Deterministic ground motion assessment using maximum credible earthquake, or
2. Probabilistically assessed ground motion with a long return period.

The functional evaluation response spectrum is derived using probabilistically assessed ground motions which have a 60% probability of not being exceeded during the useful life of the bridge. A separate functional evaluation is usually required only for important bridges.

With the above-prescribed input earthquake loading and using an elastic dynamic multimodal response spectrum analysis, the displacement demand can be computed. The displacement capacity of various bents may be then calculated using two-dimensional or three-dimensional nonlinear static push-over analysis with strain limits associated with expected damage at plastic hinge locations. When performing a push-over analysis, a concrete stress–strain model that considers effects of transverse confinement, such as Mander's model, and a steel stress–strain curve are used for considering material nonlinearity [2]. Limiting the concrete compressive strain to a magnitude smaller than the confined concrete ultimate compressive strain and the steel strain to a magnitude

TABLE 43.2 Strain Limits

	Significant Damage	Repairable Damage	Minimal Damage
Concrete strain limit ε_c	ε_{cu}	2/3 * ε_{cu}	The greater of 1/3 * ε_{cu} or 0.004
Grade 430 bar #29 to #57 steel strain limit ε_s	0.09	0.06	0.03
Grade 280 bar #29 to #57 steel strain limit ε_s	0.12	0.08	0.03
Grade 430 bar #10 to #25 steel strain limit ε_s	0.12	0.08	0.03
Grade 280 bar #10 to #25 steel strain limit ε_s	0.16	0.10	0.03

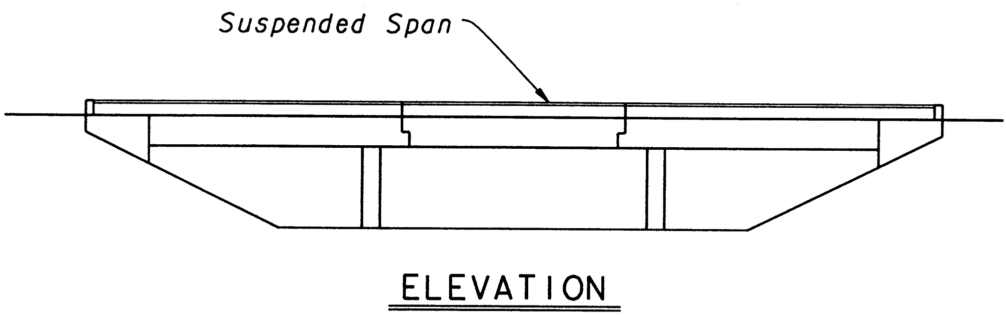


FIGURE 43.1 Suspended span.

smaller than steel rupture strain results in lesser curvature of the cross section under consideration. Smaller curvatures are usually associated with smaller cracks in the plastic hinge region.

Table 43.2 shows the general guidelines on strain limits that can be considered for a target level of damage in a plastic hinge zone. These limits are applied for the ultimate concrete strain ε_{cu} and the ultimate strain in the reinforcing steel ε_{su}. The ultimate concrete strain can be computed using a concrete confinement model such as Mander's model. It can be seen that for a poorly confined section, the difference between minimal damage and significant damage becomes insignificant. With displacement demands and displacement capacities established, the demand-to-capacity ratios can be computed showing adequacy or inadequacy of the subject bridge.

43.3 Superstructure Retrofits

Superstructures can be categorized into two different categories: concrete and steel. After the 1971 San Fernando, California earthquake, the primary failure leading to collapse was identified as unseating of superstructures at the expansion joints and abutments, a problem shared by both types of superstructures. Other potential problems that may exist with steel superstructures are weak cross-bracing and/or diaphragms. Concrete bridge superstructures have the potential to form plastic hinges during a longitudinal seismic response which is largely dependent upon the amount of reinforcement used and the way it is detailed.

43.3.1 Expansion Joints and Hinges

During an earthquake, adjacent bridge frames will often vibrate out of phase, causing two types of displacement problems. The first type is a localized damage caused by the frames pounding together at the hinges. Generally, this localized damage will not cause bridge collapse and is therefore not a major concern. The second type occurs when the hinge joint separates, possibly allowing the adjacent spans to become unseated if the movement is too large. Suspended spans (i.e., two hinges within one span) are especially vulnerable to becoming unseated (Figure 43.1).

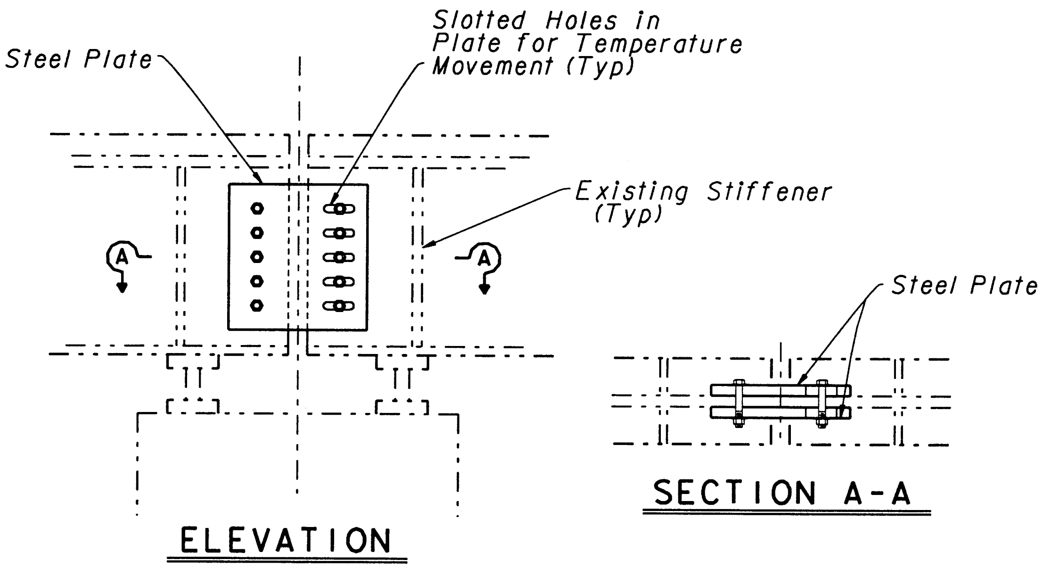


FIGURE 43.2 Steel girder hinge plate retrofit.

43.3.1.1 Simply Supported Girders

The most common problem for simply supported structures is girders falling off their seats due to a longitudinal response. If the seismic force on the structure is large enough to fail the bearings, then the superstructure becomes vulnerable to unseating at the supports.

There are several ways of retrofitting simply supported steel girders and/or precast concrete girders. The most common and traditional way is to use cable restrainers, since the theory is fundamentally the same for both types of girders. For more about cable restrainers, refer to Section 43.3.1.3. Care should be taken when designing the cables to intrude as little as possible on the vertical clearance between the girders and the roadway. Note that the cable retrofit solution for simply supported girders can be combined with a cap seat extension if expected longitudinal displacements are larger than the available seat width.

Another possible solution for steel girders is to make the girders continuous over the bents by tying the webs together with splice plates (Figure 43.2). The splice plate should be designed to support factored dead-load shears assuming the girder becomes unseated. The splice plate is bolted to the girder webs and has slotted or oversized holes to allow for temperature movement. This retrofit solution will usually work for most regular and straight structures, but not for most irregular structures. Any situation where the opposing girders do not line up will not work. For example, bridges that vary in width or are bifurcated may have different numbers of girders on opposite sides of the hinge. Bridges that are curved may have the girders at the same location but are kinked with respect to each other. In addition, many structures may have physical restrictions such as utilities, bracing, diaphragms, stiffeners, etc. which need to be relocated in order for this strategy to work.

43.3.1.2 Continuous Girders with In-Span Hinges

For continuous steel girders, the hinges are typically placed near the point of zero moment which is roughly at 20% of the span length. These hinges can be either seat type as shown in Figure 43.3 or hanger type as shown in Figure 43.4. The hanger-type hinges are designed for vertical dead and live loads. These loads are typically larger than forces that can be imparted onto the hanger bar from a longitudinal earthquake event, and thus retrofitting the hanger bar is generally unnecessary. Hanger-type hinges typically have more seismic resistance than seat-type hinges but may still be subjected to seismic damage. Hanger bars are tension members that are vulnerable to differential

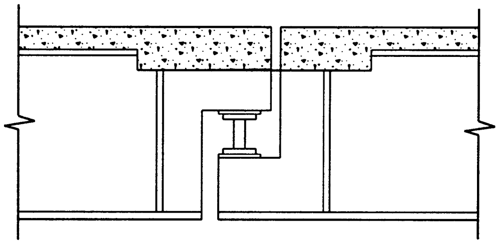


FIGURE 43.3 Seat-type hinge.

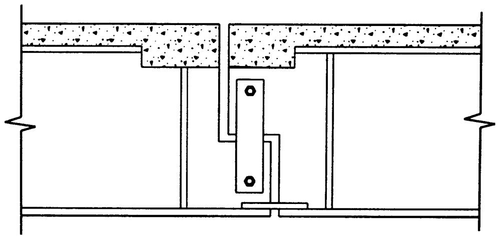


FIGURE 43.4 Hanger-type hinge.

transverse displacement on either side of the hinge. The differential displacement between the girders causes the hanger bars to go into bending plus tension. These hinges often have steel bars or angles that bear against the opposite web, or lugs attached to the flanges, which were designed to keep the girders aligned transversely for wind forces. These devices are usually structurally inadequate and are too short to be effective with even moderate seismic shaking. Consideration should be given to replacing them or adding supplemental transverse restrainers [3]. Cross-bracing or diaphragms on both sides of the hinge may have to be improved in conjunction with the transverse restrainer.

It can generally be assumed that any seat-type hinge used with steel girders will need additional transverse, longitudinal, and vertical restraint in even moderately severe seismic areas [3].

Continuous concrete box girders typically have in-span-type hinges. These hinge seats were typically 150 to 200 mm, while some were even less on many of the older bridges. Because of the localized damage that occurs at hinges (i.e., spalling of the concrete, etc.), the actual length of hinge seat available is much less than the original design. Therefore, a means of providing a larger hinge seat and/or tying the frames together is necessary.

43.3.1.3 Restrainers

Restrainers are used to tie the frames together, limiting the relative displacements from frame to frame and providing a load path across the joint. The main purpose is to prevent the frames from falling off their supports. There are two basic types of restrainers, cables and rods. The choice between cables and rods is rather arbitrary, but some factors to consider may be structure period, flexibility, strength of hinge/bent diaphragm, tensile capacity of the superstructure, and, to some degree, the geometry of the superstructure.

There are various types of longitudinal cable restraining devices as shown in Figures 43.5 to 43.7. Cable restraining units, such as the ones shown in Figures 43.5 and 43.6 generally have an advantage over high-strength rods because of the flexibility with its usage for varying types of superstructures. For simply supported girders, cables may be anchored to a bracket mounted to the underside of the girder flange and wrapped around the bent cap and again anchored as shown in Figure 43.8. This is the preferred method. Another possibility is to have the cables anchored to a bracket mounted to the bottom flange and simply attached to an opposing bracket on the other side of the hinge, as

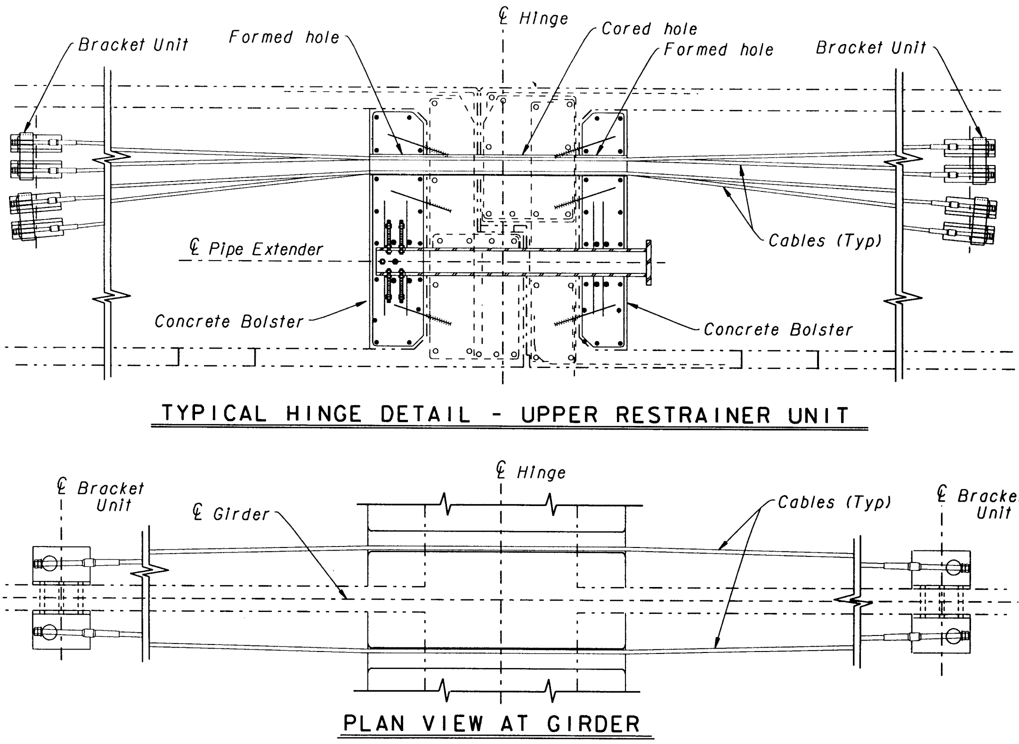


FIGURE 43.5 Type 1 hinge restrainer.

shown in Figure 43.9. The latter example is generally used for shorter bridges with a larger seat area at the bent cap or in situations where vertical clearances may be limited. Moreover, cables have the advantage of using a variety of lengths, since the anchorage devices can be mounted anywhere along the girders, whether it be steel or concrete, in addition to being anchored to the nearest bent cap or opposite side of the hinge diaphragm. For example, if the restrainer is relatively short, this may shorten the period, possibly increasing the demand to the adjacent bridge frames. Therefore, for this example, it may be desirable to lengthen the restrainer keeping the force levels to within the capacities of the adjacent segments [4]. On the other hand, if the restrainer is too long, unseating can thus occur, and an additional means of extending the seat length becomes necessary.

High-strength rods are another option for restricting the longitudinal displacements and can be used with short seats without the need for seat extenders. Unlike cables, when high-strength rods are used, shear keys or pipes are generally used in conjunction since rods can be sheared with transverse movements at hinge joints. Geometry may be a limiting factor when using high-strength rods. For example, if a box-girder bridge is shallow, it may not be possible to install a long rod through a narrow access opening. For both cables and rods, the designer needs to consider symmetry when locating restraining devices.

43.3.1.4 Pipe Seat Extenders

When a longer restraining device is preferred, increased longitudinal displacements will result and may cause unseating. It is therefore necessary to incorporate pipe seat extenders to be used in conjunction with longer restrainers when unseating will result. A 200 mm (8 in.), XX strong pipe is used for the pipe seat extender which is placed in 250 mm (10 in.) cored (and formed) hole (Figure 43.10). A 250-mm cored hole allows vertical jacking if elastomeric pads are present and replacement is required after the pad fails. Longitudinal restraining devices (namely, cables and rods) must be strain compatible with the seismic deflections imposed upon the hinge joint. In other

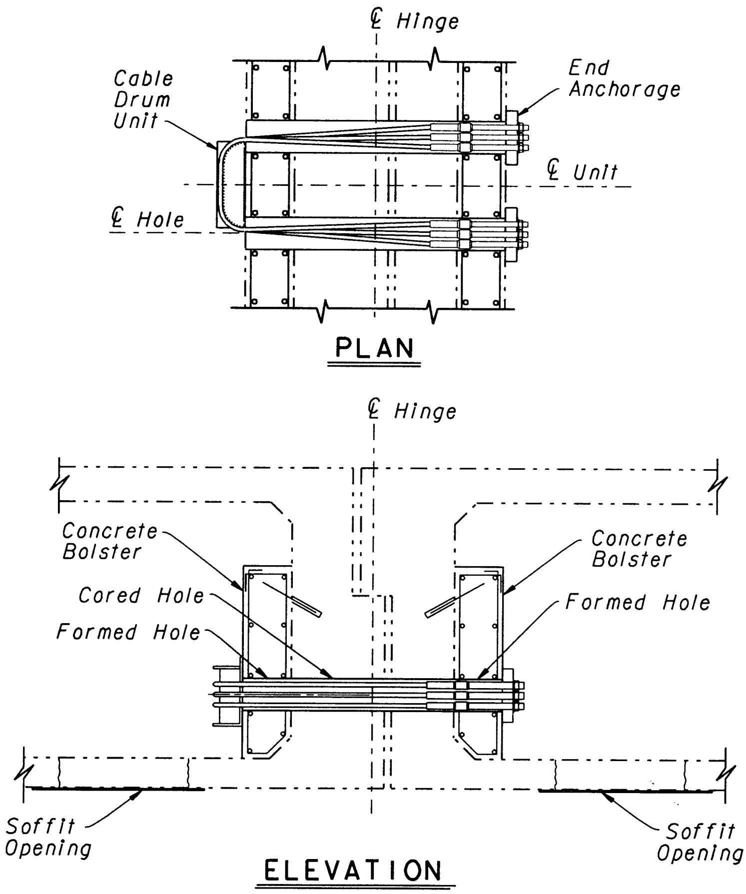


FIGURE 43.6 C-1 hinge restrainer.

words, if the longitudinal restrainers were too short, the device would have yielded long before the pipe seat extender was mobilized, deeming the longitudinal restrainers useless. To limit the number of cored holes in the diaphragm, a detail has been developed to place the restrainer cables through the pipes as shown in Figure 43.11. The pipes are not only used for vertical load-carrying capacity, but can also be used successfully as transverse shear keys.

43.3.2 Steel Bracing and Diaphragms

Lateral stiffening between steel girders typically consists of some type of cross-bracing system or channel diaphragm. These lateral bracing systems are usually designed to resist wind loads, construction loads, centrifugal force from live loads and seismic loads. The seismic loads prescribed by older codes were a fraction of current code seismic loads and, in some cases, may not have controlled bracing design. In fact, in many cases, the lateral bracing system is not able to withstand the "fusing" forces of the bearing capacity and/or shear key capacity. As a result, bracing systems may tend to buckle and, if channel diaphragms are not full depth of the girder, the webs could cripple. In general, the ideal solution is to add additional sets of bracing, stiffeners, and/or full-depth channel diaphragms as close to the bearings as physically possible.

Retrofit solutions chosen will depend on space restrictions. New bracing or diaphragms must be placed to not interfere with existing bracing, utilities, stiffeners, cable restrainers and to leave enough access for maintenance engineers to inspect the bearings. Skewed bents further complicate space

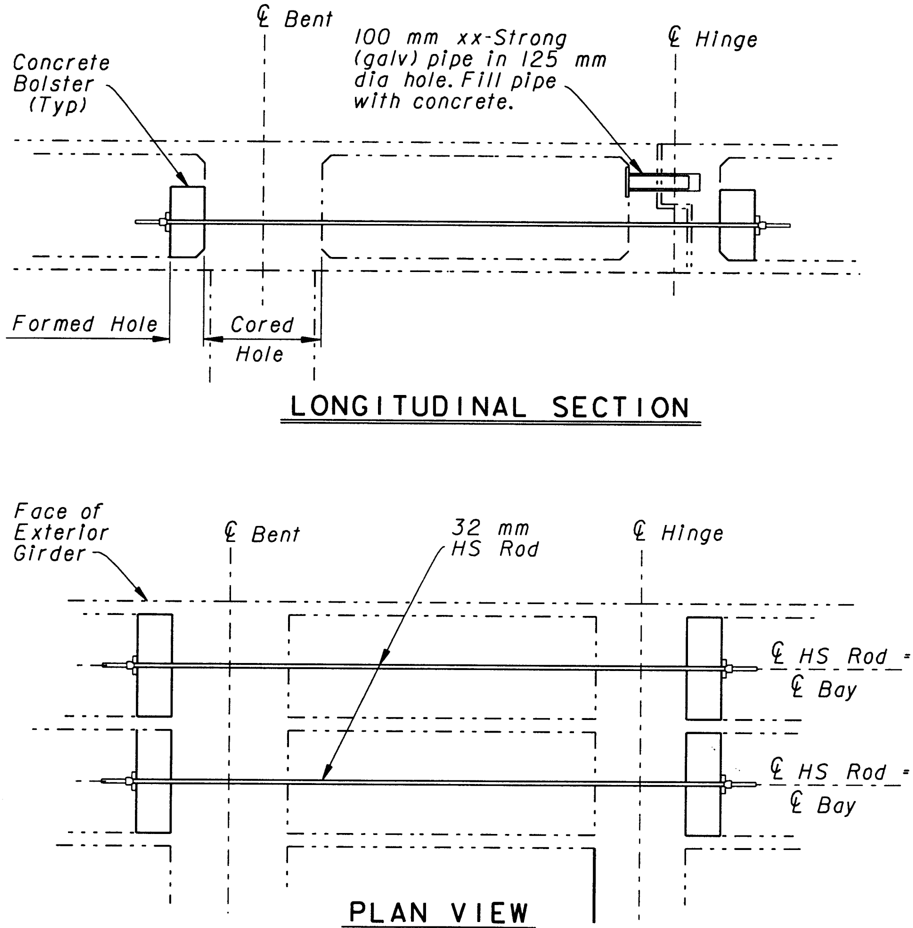


FIGURE 43.7 HS rod restrainer.

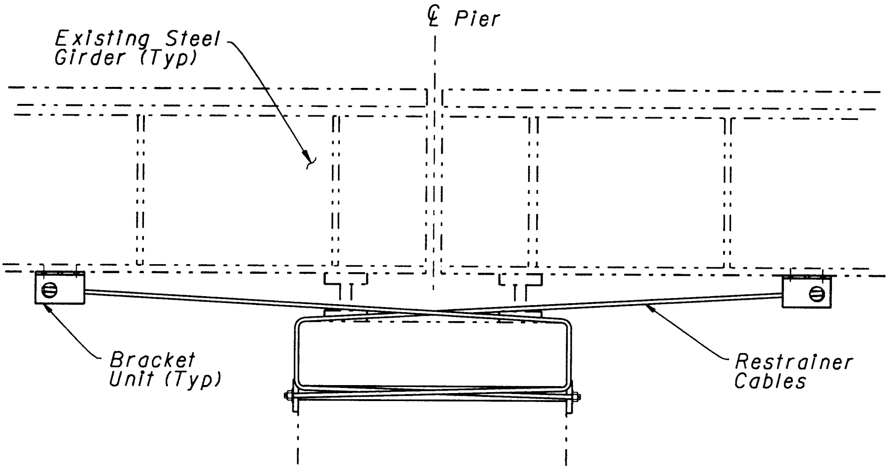


FIGURE 43.8 Cable restrainer through bent cap.

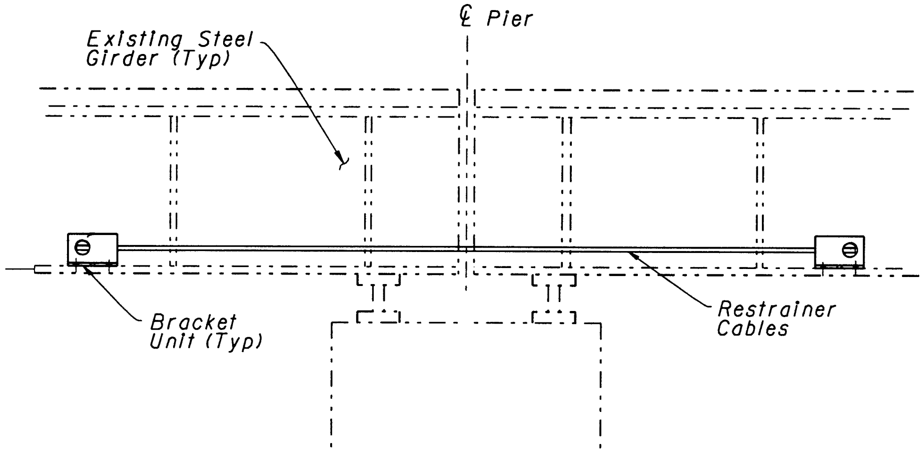


FIGURE 43.9 Girder-to-girder cable restrainer.

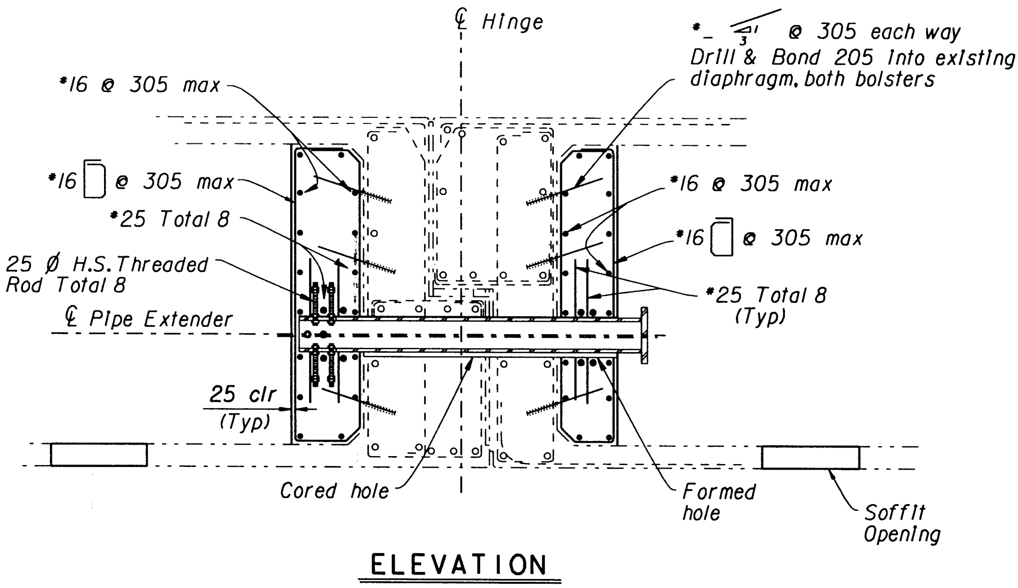


FIGURE 43.10 Pipe seat extender.

restrictions. When choosing a retrofit solution, the engineer must keep in mind that the retrofit will be constructed while the structure is carrying traffic. Stresses in a bracing member tend to cycle under live load which makes it difficult for the engineer to assess actual member stresses. As a result, any retrofit solution that requires removing and replacing existing members is not recommended. In addition, careful consideration should be given to bolted vs. welded connections. As previously mentioned, structures are under live loads during the retrofit operation and thus members are subjected to cyclic stresses. It may be difficult to achieve a good-quality weld when connecting to a constantly moving member. If bolted connections are used, preference should be given to end-bearing connections over friction connections. Friction connections have more stringent surface preparation requirements which are difficult and expensive to achieve in the field, as is the case with lead-based paint.

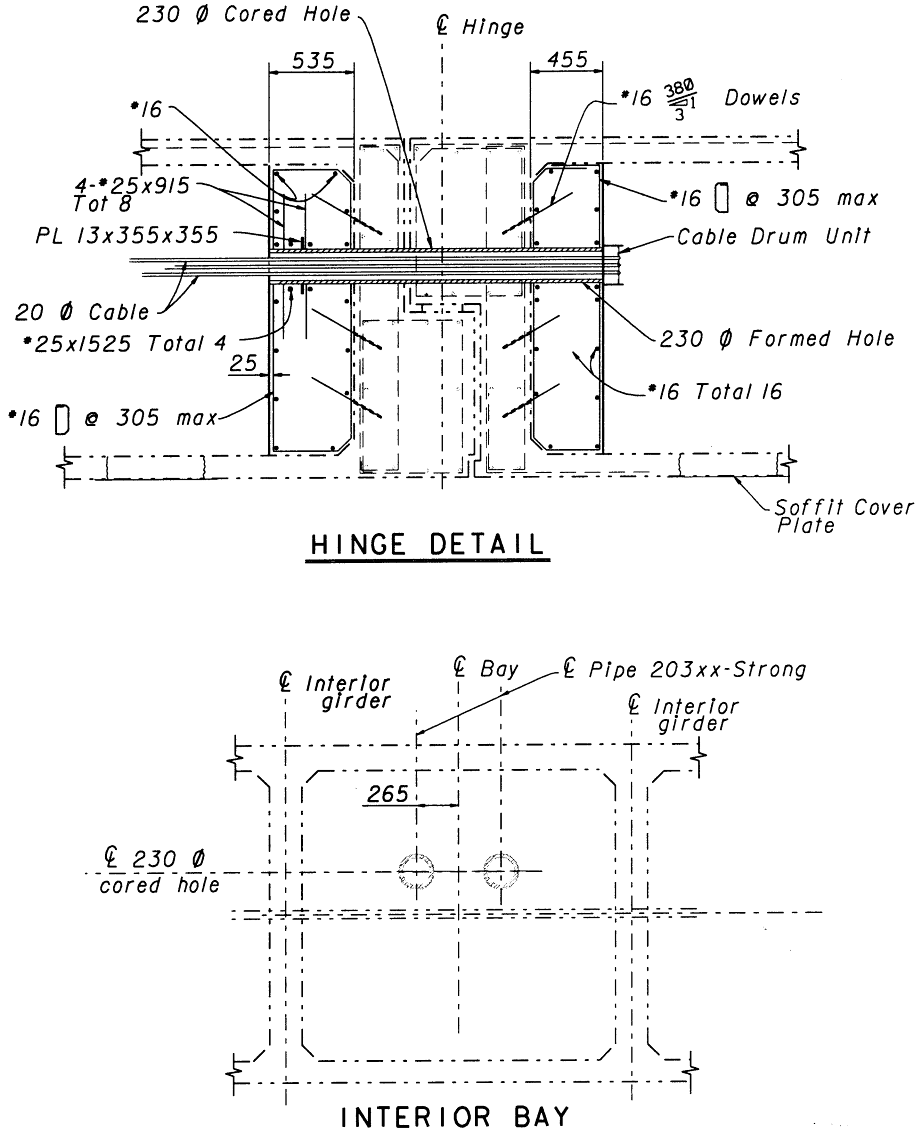


FIGURE 43.11 Restrainers/pipe seat extender.

43.3.3 Concrete Edge Beams

Edge beams are used to enhance the longitudinal capacity of a concrete bridge. These beams link consecutive bents together outside the existing box structure. In the United States, edge beams have been used to retrofit double-deck structures with long outriggers.

In a single-level bridge structure, outriggers are vulnerable in torsion under longitudinal excitation. Two retrofit alternatives are possible. The first alternative is to strengthen the outrigger cap while maintaining torsional and flexural fixity to the top of the column. The second alternative is to pin the top of the column; thus reducing the torsional demand on the vulnerable outrigger cap. Using the second alternative, the column bottom fixity needs to be ensured by means of a full footing retrofit.

In a double-deck structure, pinning the connection between the lower level outrigger cap and the column is not possible since fixity at that location is needed to provide lateral support to the upper deck. In situations where the lower deck is supported on a long outrigger cap, the torsional softening of that outrigger may lead to loss of lateral restraint for the upper-deck column. This weakness can be remedied by using edge beams to provide longitudinal lateral restraint. The edge beams need to be stiff and strong enough to ensure plastic hinging in the column and reduce torsional demand on the lower-deck outrigger cap.

43.4 Substructure Retrofits

Most earthquake damage to bridge structures occurs at the substructure. There are many types of retrofit schemes to increase the seismic capacity of existing bridges and no one scheme is necessarily more correct than another. One type of retrofit scheme may be to encase the columns and add an overlay to footings. Another might be to attract forces into the abutments and out of the columns and footings. Listed below are some different concepts to increase the capacities of individual members of the substructure.

43.4.1 Concrete Columns

Bridge columns constructed in the United States prior to 1971 are generally deficient in shear, flexure, and/or lateral confinement. Stirrups used were typically #13 bars spaced at 300 mm on center (#4 bars at 12 in.) for the entire column length including the regions of potential plastic hinging. Typically, the footings were constructed with footing dowels, or starter bars, with the longitudinal column reinforcement lapped onto the dowels. As the force levels in the column approach yield, the lap splice begins to slip. At the onset of yielding, the lap splice degrades into a pin-type condition and within the first few cycles of inelastic bending, the load-carrying capacity degrades significantly. This condition can be used to allow a "pin" to form and avoid costly footing repairs. Various methods have been successfully used to both enhance the shear capacity and ductility by increasing the lateral confinement of the plastic hinge zone for bridge columns with poor transverse reinforcement details. Following is a list of these different types and their advantages and/or disadvantages.

43.4.1.1 Column Casings

The theory behind any of the column casing types listed below is to enhance the ductility, shear, and/or flexural capacity of an existing reinforced concrete column and, in some cases, to limit the radial dilating strain within the plastic hinge zone. Because of the lap splice detail employed in older columns, one of the issues facing column retrofitting is to maintain fixity at the column base. The lateral confining pressure developed by the casing is capable of limiting the radial dilating strain of the column, enough to "clamp" the splice together, preventing any slippage from occurring. Tests have shown that limiting the radial dilating strains to less than 0.001, the lap splice will remain fixed and is capable of developing the full plastic moment capacity of the section [5]. Contrary to limiting the radial strains is to permit these strains to take place (i.e., radial dilating strains greater than 0.001) allowing a "pin" to form while providing adequate confinement throughout the plastic hinge region.

Steel Casings

There are three types of retrofit schemes that are currently employed to correct the problems of existing columns through the use of steel jacketing. The first type is typically known as a class F type of column casing retrofit, as shown in Figure 43.12. This type of casing is fully grouted and is placed the full height of the column. It is primarily used for a column that is deficient in shear and flexure. It will limit the radial dilating strain to less than 0.001, effectively fixing the lap splice from

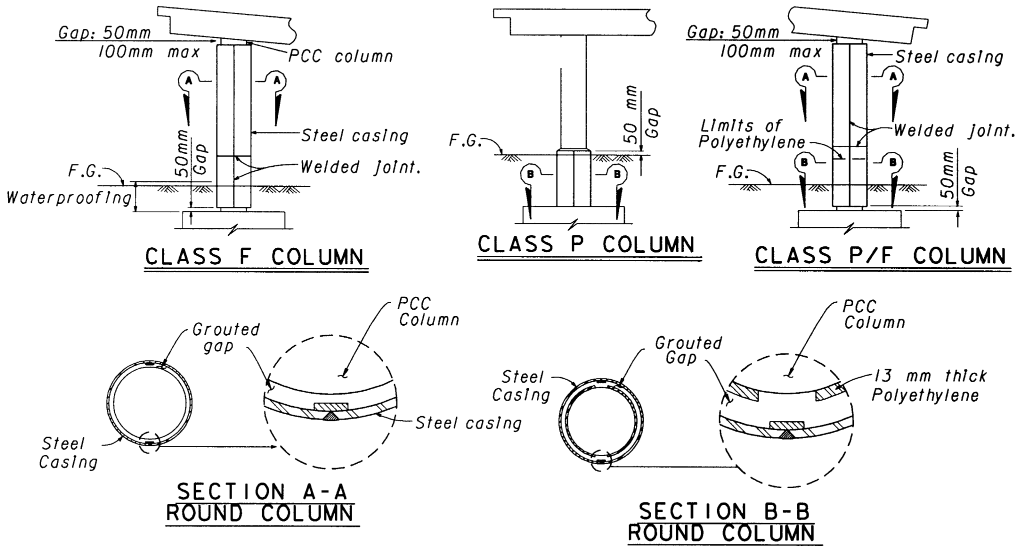


FIGURE 43.12 Steel column casings.

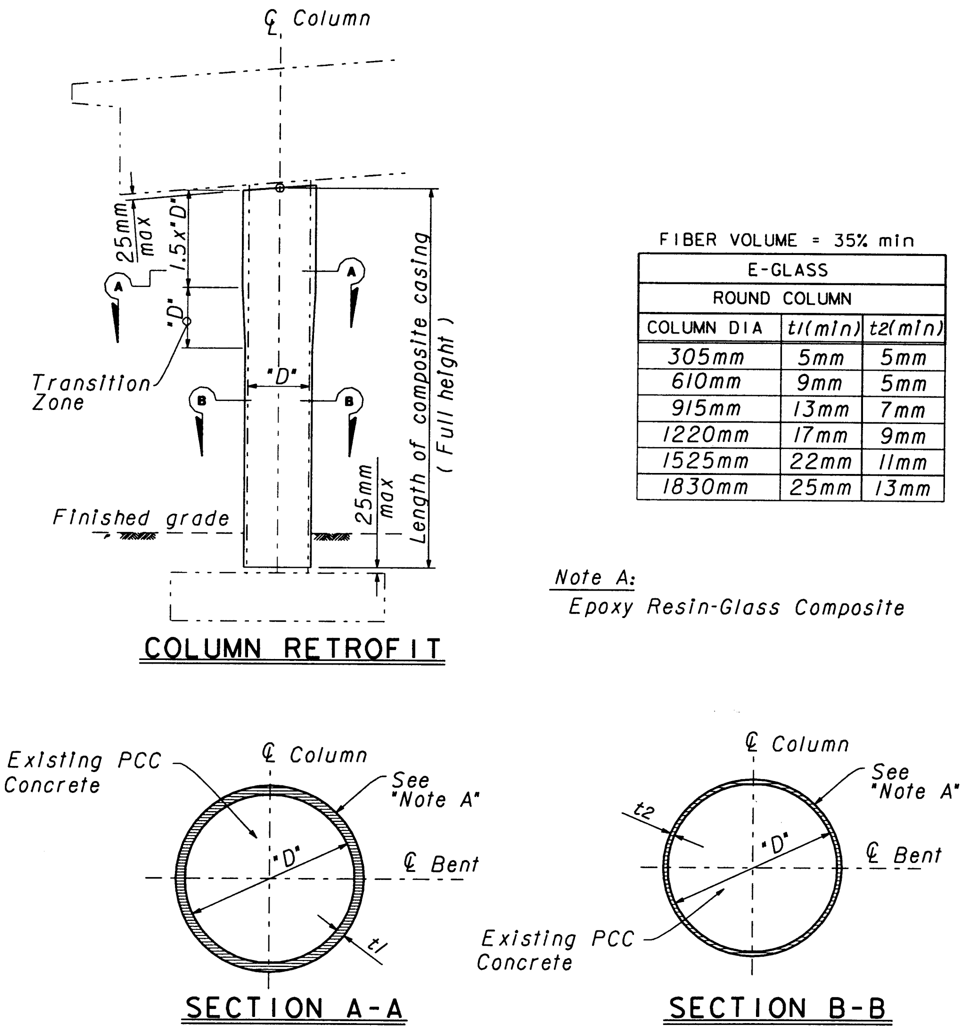
slipping. The lateral confining pressure for design is taken as 2.068 MPa (300 psi) and, when calculated for a 13-mm-thick steel casing with A36 steel, is equivalent to a #25 bar (#8 bar) at a spacing of about 38 mm. It can therefore be seen that the confinement, as well as the shear, is greatly enhanced. The allowable displacement ductility ratio for a class F column casing is typically 6, allowing up to 8 in isolated locations. It has been tested well beyond this range; however, the allowable ductility ratio is reduced to prohibit fracturing of the longitudinal bars and also to limit the level of load to the footing.

The second type of casing is a class P type and is only a partial height casing; therefore, it does not help a column that is deficient in shear. As can be seen from Figure 43.12, the main difference between a class F and a class P type column casing is the layer of polyethylene between the grout and the column. This will permit the column to dilate outward, allowing the strain to exceed 0.001, forming a pin at the base of the column. It should be noted that the casing is still required in this condition to aid in confining the column. The limit of this type of retrofit is typically taken as 1.5 times the columns diameter or to where the maximum moment has decreased to 75%.

The third type of steel column casing is a combination of the first two and, hence, is known as a class P/F, also shown in Figure 43.12. It is used like a class P casing, but for a column with a shear deficiency. All of these casings can be circular (for circular or square columns) or oblong (for rectangular columns). If a rectangular column is deficient in shear only, it is sometimes permitted to use flat steel plates if the horizontal clearances are limited and it is not possible to fit an oblong column in place. For aesthetic purposes, the class P casing may be extended to full height in highly visible areas. It can be unsightly if an oblong casing is only partial height. It is important to mention that the purpose of the 50-mm gap at the ends of the column is to prevent the casing from bearing against the supporting member acting as compression reinforcement, increasing the flexural capacity of the column. This would potentially increase the moments and shears into the footing and/or bent cap under large seismic loads.

Concrete Casings

When retrofitting an unusually shaped column without changing the aesthetic features of the geometry of the column, a concrete casing may be considered as an alternative. Existing columns are retrofitted by placing hoops around the outer portion of the column and then drilling and bonding bars into the column to enclose the hoops. The reinforcement is then encased with concrete,



FIBER VOLUME = 35% min

E-GLASS ROUND COLUMN		
COLUMN DIA	t1(min)	t2(min)
305mm	5mm	5mm
610mm	9mm	5mm
915mm	13mm	7mm
1220mm	17mm	9mm
1525mm	22mm	11mm
1830mm	25mm	13mm

FIGURE 43.13 Advanced composite column casing.

thus maintaining the original shape of the column. The design of a concrete jacket follows the requirements of a new column. Although this method increases the shear and flexural capacities of the column and provides additional confinement without sacrificing aesthetics, it is labor intensive and therefore can be costly.

Advanced Composite Casings

Recently, there has been significant research and development using advanced composites in bridge design and retrofit. Similar to steel casings, advanced composite casings increase the confinement and shear capacity of existing concrete bridge columns. This type of column retrofit has proved to be competitive with steel casings when enhancing column shear capacity and may also provide an economic means of strengthening bridge columns (Figure 43.13). However, currently, composites are not economic when limiting lap splice slippage inside expected plastic hinge zones. The advantage of using some types of composite casings is that the material can be wrapped to the column without changing its geometric shape. This is important when aesthetics are important or lateral clearances at roadways are limited.

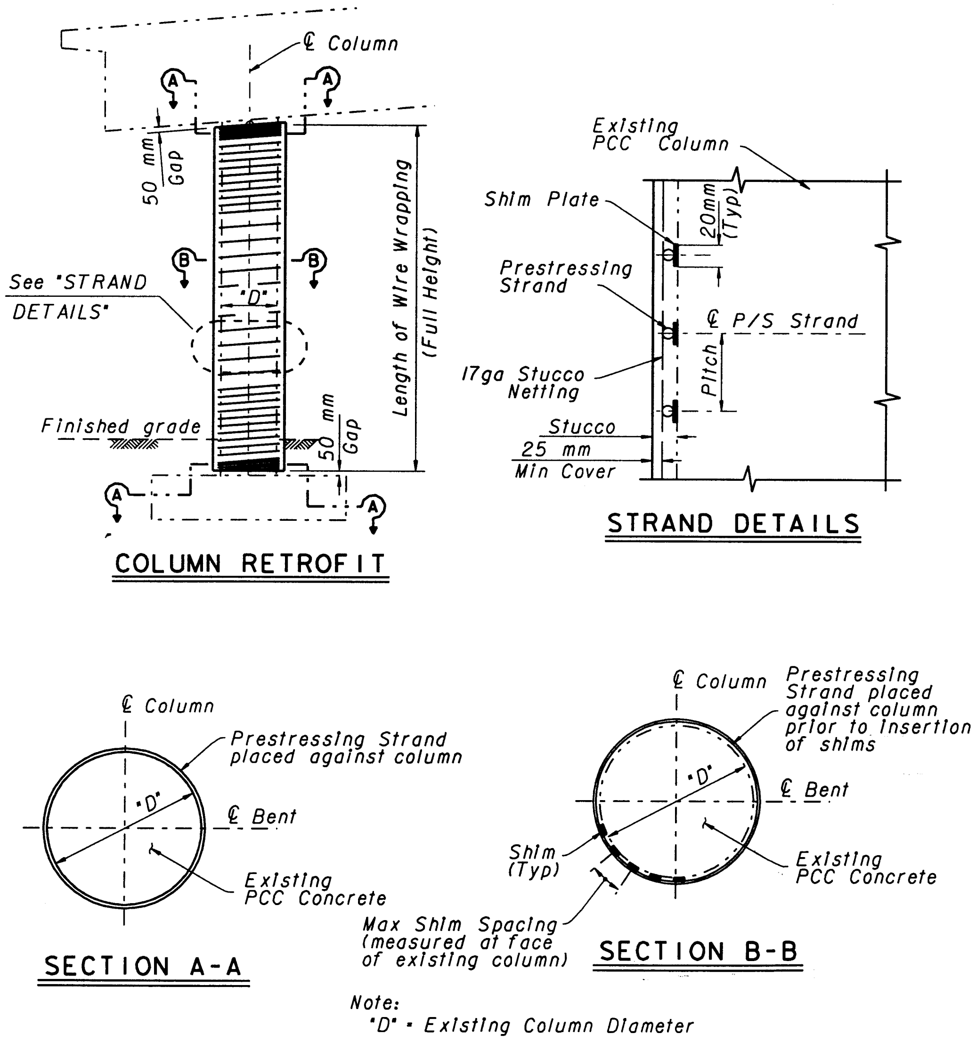


FIGURE 43.14 Column wire wrap.

Wire Wrap Casings

Another type of system that was recently approved by the California Department of Transportation is a "wire wrap" system. It consists of a prestressing strand hand wrapped onto a column; then wedges are placed between the strand and the column, effectively prestressing the strand and actively confining the column, as shown in Figure 43.14. The advantage of this type of system is that, like the advanced composites, it can be wrapped to any column without changing the geometric shape. Its basic disadvantage is that it is labor intensive and currently can only be applied to circular columns.

43.4.1.2 In-Fill Walls

Reinforced concrete in-fill walls may also be used as an alternative for multicolumn bridge bents, as shown in Figure 43.15. This has two distinct advantages: it increases the capacity of the columns in the transverse direction and limits the transverse displacements. By limiting the displacements

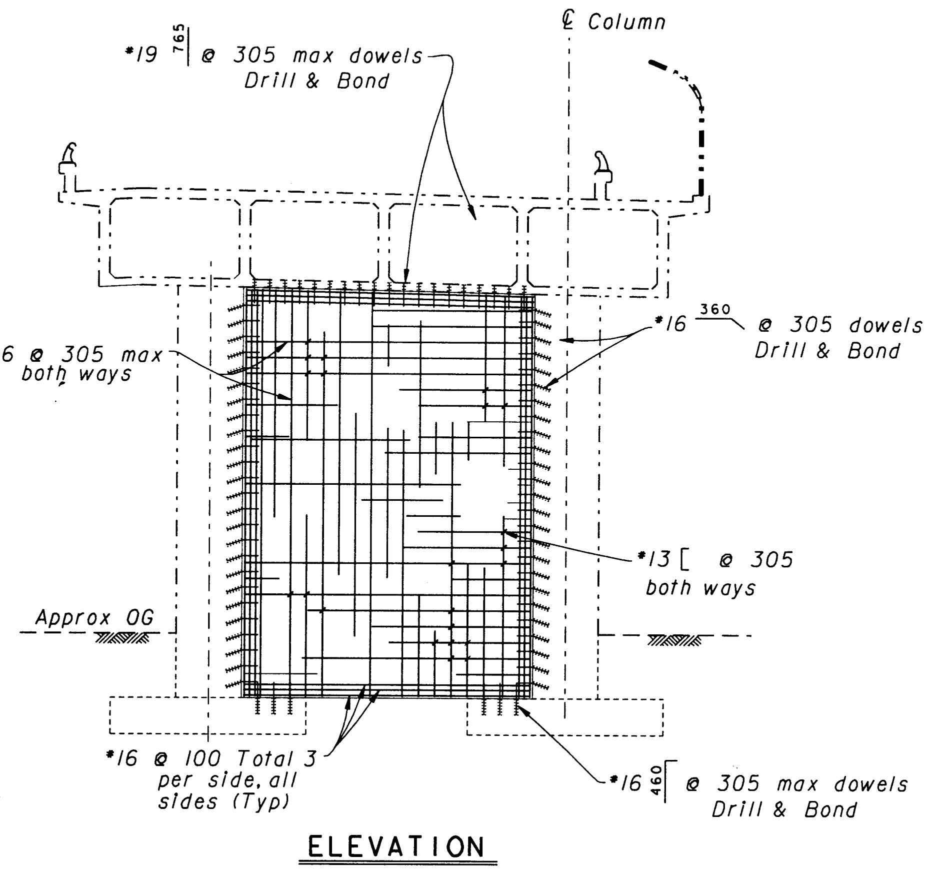


FIGURE 43.15 In-fill wall.

transversely, the potential for plastic hinge formation in the bent cap is eliminated. Therefore, cost may prove to be less than some other retrofit alternatives mentioned earlier. It is important to note that the in-fill wall is not effective in the longitudinal capacity of bridge bents with little or no skew.

43.4.1.3 Link Beams

Link beams are used to enhance the transverse capacity of a concrete bent. The placement of a link beam over a certain height above ground level determines its function.

A link beam can be placed just below the soffit level and acts as a substitute to a deficient existing bent cap. The main function of this kind of a link beam is to protect the existing superstructure and force hinging in the column.

In other cases, a link beam is placed somewhere between the ground level and the soffit level in order to tune the transverse stiffness of a particular bent. This type of retrofit can be encountered in situations where a box superstructure is supported on bents with drastically unequal stiffnesses. In this case, the center of mass and the center of rigidity of the structure are farther apart. This eccentricity causes additional displacement on the outer bents which can lead to severe concentrated ductility demands on just a few bents of the subject bridge. This behavior is not commonly preferred in seismically resistant structures and the use of link beams in this case can reduce the eccentricity between the center of mass and the center of rigidity. This structural tuning is important in equally distributing and reducing the net ductility demands on all the columns of the retrofitted bridge.

43.4.2 Pier Walls

Until recently, pier walls were thought to be more vulnerable to seismic attack than columns. However, extensive research performed at the University of California, Irvine has proved otherwise [6]. Pier walls, nevertheless, are not without their problems. The details encompass poor confinement and lap splices, similar to that of pre-1971 bridge columns. Pier walls are typically designed and analyzed as a shear wall about the strong axis and as a column about the weak axis. The shear strength of pier walls in the strong direction is usually not a concern and one can expect a shear stress of about 0.25 $\sqrt{f_c'}$ (MPa). For the weak direction, the allowable demand displacement ductility ratio in existing pier walls is 4.0. Similar to columns, many older pier walls were also built with a lap splice at the bottom. If the lap splice is long enough, fixity will be maintained and the full plastic moment can be developed. Tests conducted at the University of California, Irvine, have shown that lap splices 28 times the diameter of the longitudinal bar to be adequate [6]. However, a lap splice detail with as little as 16 bar diameters will behave in the same manner as that of a column with an inadequate lap splice and may slip, forming a pin condition. Because of the inherent flexibility of a pier wall about its weak axis, the method of retrofit for this type of lap splice is a plate with a height two times the length of the splice, placed at the bottom of the wall. The plate thickness is not as critical as the bolt spacing. It is generally recommended that the plate be 25 mm thick with a bolt spacing equal to that of the spacing of the main reinforcement, only staggered (not to exceed approximately 355 mm). If additional confinement is required for the longer lap splice, the plate height may be equal to the lap splice length.

43.4.3 Steel Bents

Most steel bents encountered in older typical bridges can be divided in two groups. One group contains trestle bents typically found in bridges spanning canyons, and the second group contains open-section built-up steel columns. The built-up columns are typically I-shaped sections which consist of angles and plates bolted or riveted together. The second group is often found on small bridges or elevated viaducts.

Trestle steel bents are commonly supported on pedestals resting on rock or relatively dense foundation. In general, the truss members in these bents have very large slenderness ratios which lead to very early elastic buckling under low-magnitude earthquake loading. Retrofitting of this type of bent consists basically of balancing between member strengthening and enhancing the tensile capacity of the foundation and keeping connection capacities larger than member capacities. In many situations, foundation retrofit is not needed where bent height is not large and a stable rocking behavior of the bent can be achieved. Strengthening of the members can be obtained by increasing the cross-sectional area of the truss members or reducing the unsupported length of the members.

Figure 43.16 shows the retrofit of Castro Canyon bridge in Monterey County, California. The bent retrofit consists of member strengthening and the addition of a reinforced concrete block around the bent-to-pedestal connection. In this bridge, the pedestals were deeply embedded in the soil which added to the uplift capacity of the foundation.

For very tall trestle bents (i.e., 30 m high), foundation tie-downs, in addition to member strengthening, might be needed in order to sustain large overturning moments. Anchor bolts for base plates supported on top of pedestals are usually deficient. Replacement of these older bolts with high-strength bolts or the addition of new bolts can be done to ensure an adequate connection capable of developing tension and shear strength. The addition of new bolts can be achieved by coring through the existing base plate and pedestal or by enlarging the pedestal with a concrete jacket surrounding the perimeter bolts. The use of sleeved anchor bolts is desirable to induce some flexibility into the base connection.

The second group of steel bents contains open section built-up columns. These members may fail because of yielding, local buckling, or lateral torsional buckling. For members containing a single I-shaped section, lateral torsional buckling typically governs. Retrofit of this type of column

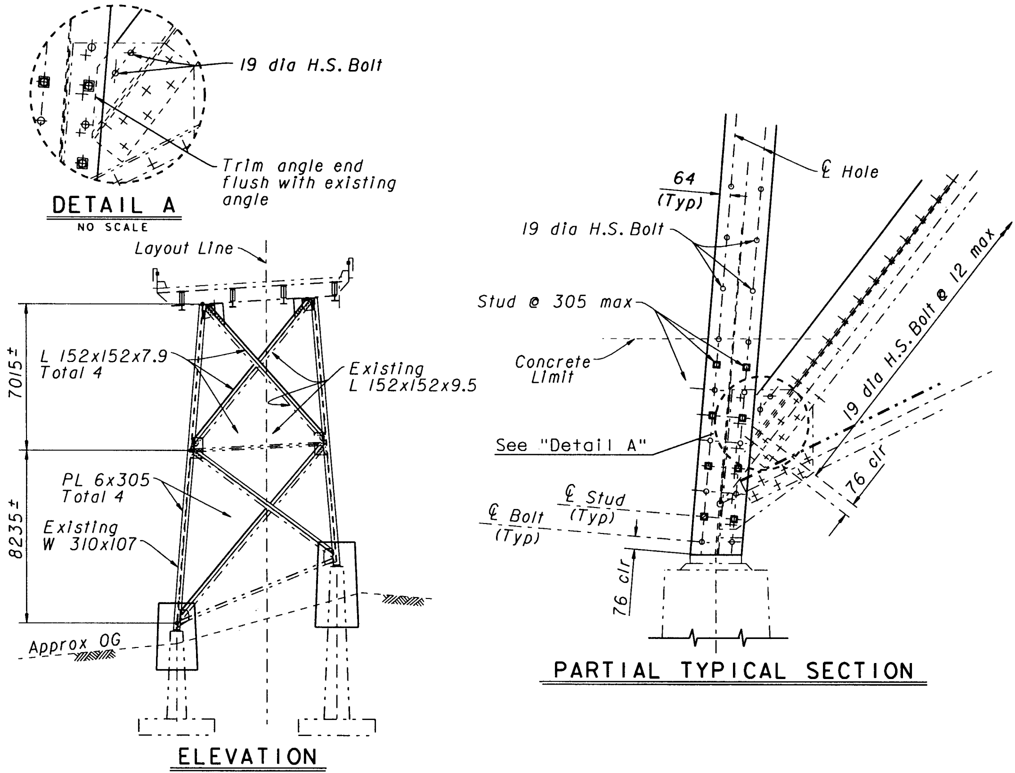


FIGURE 43.16 Trestle bent retrofit.

consists of enclosing the section by bolting channel sections to the flanges. Figure 43.17 shows this type of retrofit. Installation of these channels is made possible by providing access through slotted holes. These holes are later covered by tack welding plates or left open. For larger members with an open section as seen in Figure 43.18, retrofit consists of altering the existing cross section to a multicell box section.

The seismic behavior of the multicell box is quite superior to an open section of a single box. These advantages include better torsional resistance and a more ductile postelastic behavior. In a multicell box, the outside plates sustain the largest deformation. This permits the inside plates to remain elastic in order to carry the gravity load and prevent collapse during an earthquake. To maintain an adequate load path, the column base connection and the supporting foundation should be retrofitted to ensure the development of the plastic hinge just above the base connection. This requires providing a grillage to the column base as seen in Figure 43.18 and a footing retrofit to ensure complete load path.

43.4.4 Bearing Retrofit

Bridge bearings have historically been one of the most vulnerable components in resisting earthquakes. Steel rocker bearings in particular have performed poorly and have been damaged by relatively minor seismic shaking. Replacement of any type of bearing should be considered if failure will result in collapse of the superstructure. Bearing retrofits generally consist of replacing steel rocker-type bearings with elastomeric bearings. In some cases, where a higher level of serviceability is required, base isolation bearings may be used as a replacement for steel bearings. For more information on base isolation, see the detailed discussion in Chapter 41. Elastomeric bearings are preferred over steel rockers because the bridge deck will only settle a small amount when the bearings

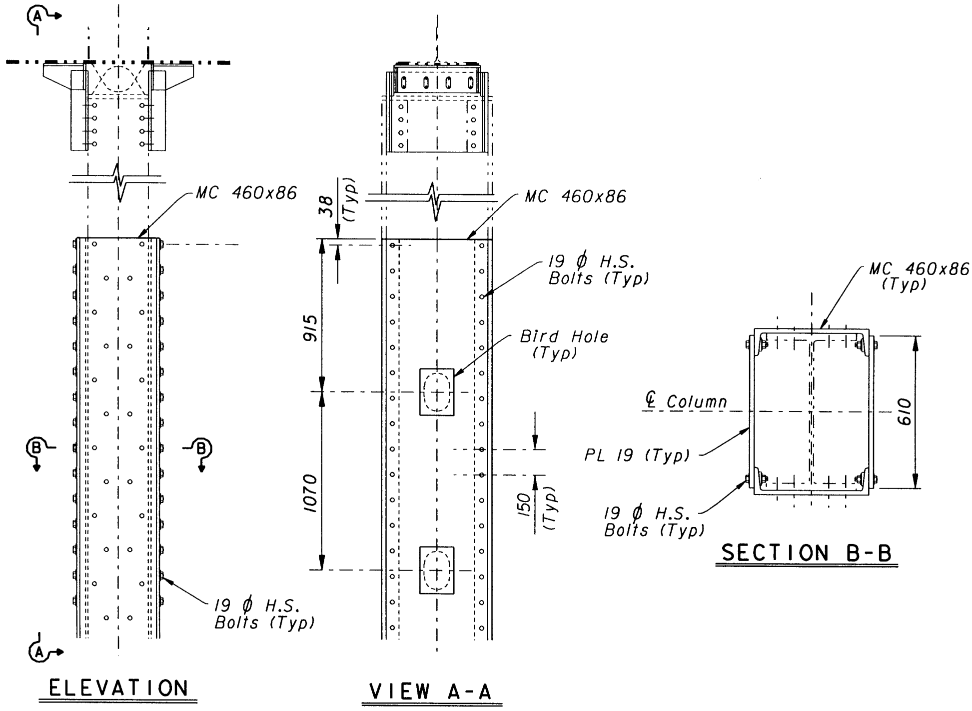


FIGURE 43.17 I-shaped steel column retrofit.

fail, whereas the deck will settle several inches when rockers fail. Elastomeric bearings also have more of a base isolation effect than steel rockers. Both types of bearings may need catchers, seat extenders, or some other means of providing additional support to prevent the loss of a span. Although elastomeric bearings perform better than steel rockers, it is usually acceptable to leave the existing rockers in place since bearings replacement is more expensive than installing catchers to prevent collapse during an earthquake.

43.4.5 Shear Key Retrofit

The engineer needs to consider the ramifications of a shear key retrofit. The as-built shear keys may have been designed to “fuse” at a certain force level. This fusing will limit the amount of force transmitted to the substructure. Thus, if the shear keys are retrofitted and designed to be strong enough to develop the plastic capacity of the substructure, this may require a more expensive substructure retrofit. In many cases, it is rational to let the keys fail to limit forces to the substructure and effectively isolate the superstructure. Also note that superstructure lateral bracing system retrofits will also have to be increased to handle increased forces from a shear key retrofit. In many cases, the fusing force of the existing shear keys is large enough to require a substructure retrofit. In these situations, new or modified shear keys need to be constructed to be compatible with the plastic capacity of the retrofitted substructure. There are other situations that may require a shear key retrofit. Transverse movements may be large enough so that the external girder displaces beyond the edge of the bent cap and loses vertical support. For a multiple-girder bridge, it is likely that the side of the bridge may be severely damaged and the use of a shoulder or lane will be lost, but traffic can be routed over a portion of the bridge with few or no emergency repairs. This is considered an acceptable risk. On the other hand, if the superstructure of a two- or three-girder bridge is displaced transversely so that one line of girders loses its support, the entire bridge may collapse. Adequate transverse restraint, commonly in the form of shear keys, should be provided.

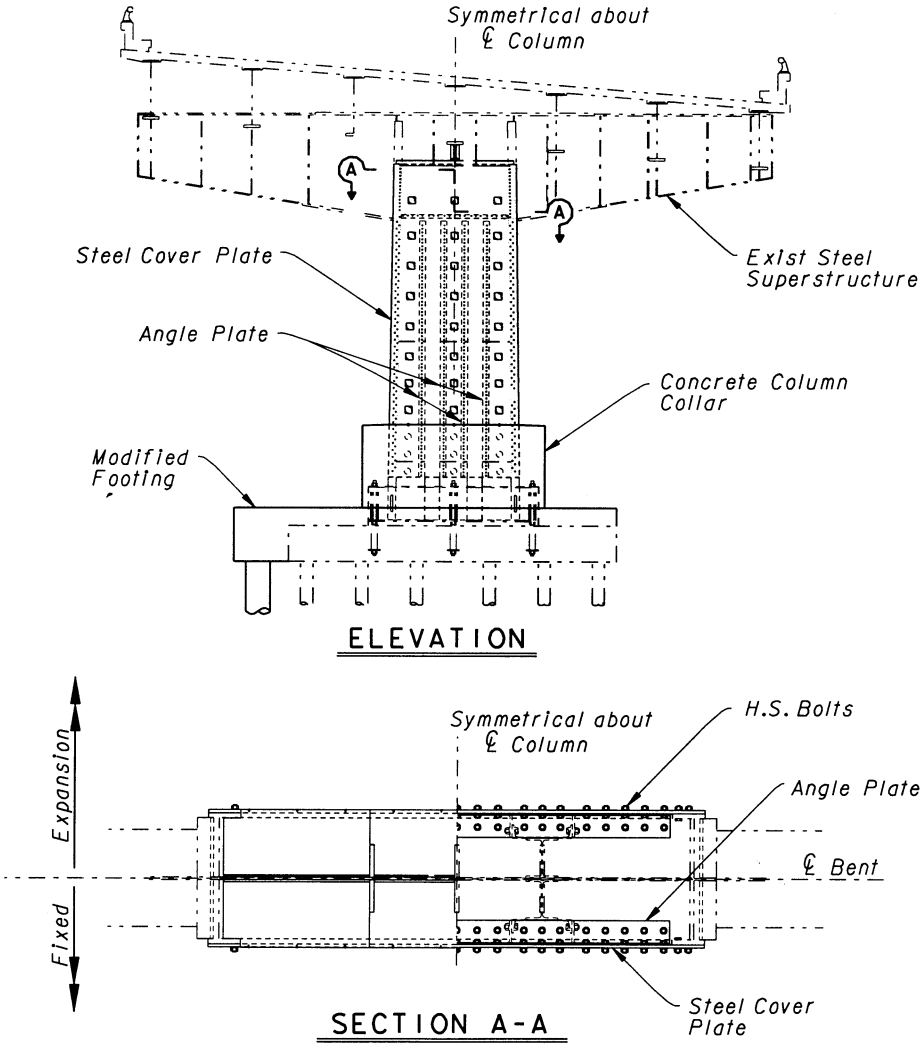


FIGURE 43.18 Open-section steel column retrofit.

43.4.6 Cap Beam Retrofit

There are several potential modes of failure associated with bent caps. Depending on the type of bent cap, these vulnerabilities could include bearing failures, shear key failures, inadequate seat widths, and cap beam failures. Table 43.3 lists several types of bent caps and their associated potential vulnerabilities.

Cap beam modes of failure may include flexure, shear, torsion, and joint shear. Prior to the 1989 Loma Prieta earthquake, California, there was very little emphasis placed on reinforcement detailing of bent cap beams for lateral seismic loads in the vicinity of columns. As a result, cap beams supported on multiple columns were not designed and detailed to handle the increased moment and shear demands that result from lateral transverse framing action. In addition, the beam–column joint is typically not capable of developing the plastic capacity of the column and thus fails in joint shear. For cap beams supported by single columns, although they do not have framing action in the transverse direction and are not subjected to moment and shear demands that are in addition

TABLE 43.3 Potential Bent Cap Vulnerabilities

				Cap Beam				
Cap Type	Bearings	Shear Keys	Seat Width	Moment	Shear	Torsion	Joint Shear	Bolted Cap/Col Connection
Concrete drop cap — single column bent	x	x	x				x	
Concrete drop cap — multicolumn bents	x	x	x	x	x		x	
Integral concrete cap — single column bent							x	
Integral concrete cap — multicolumn bent				x	x	x	x	
Inverted T — simple support for dead load, continuous for live load — single col. bent		x					x	
Inverted T — simple support for dead load, continuous for live load — multi col. bent		x					x	
Inverted T — simple support for both dead load, live load — single column bent	x	x	x				x	
Inverted T — simple support for both dead load and live load — multiple column bent	x	x	x	x	x	x	x	
Steel bent cap — single column bent	x	x	x					x
Steel bent cap — ulticolumn bent	x	x	x					x
Integral outrigger bent						x	x	
Integral C bent						x	x	

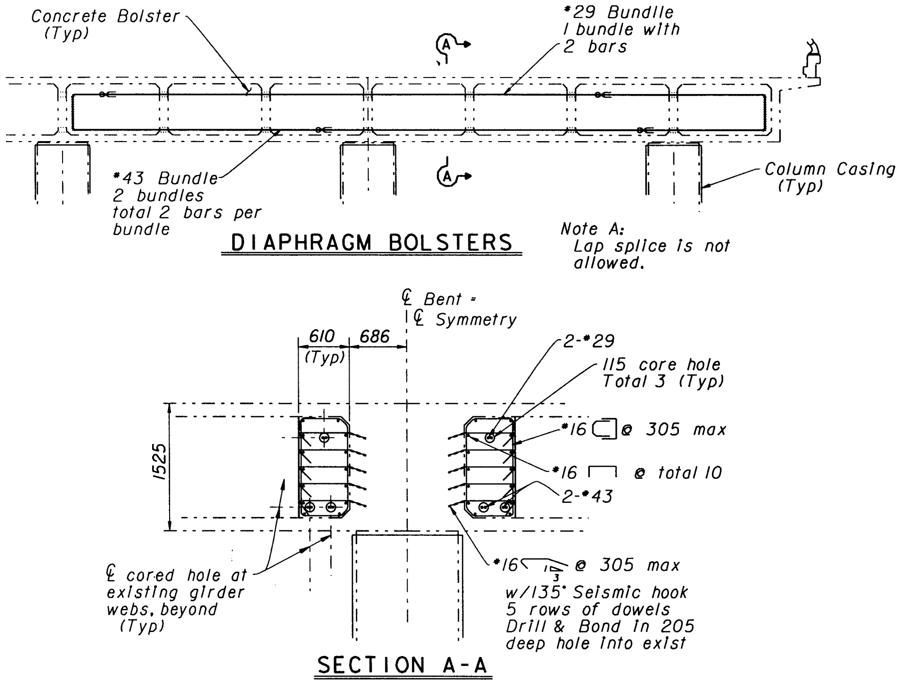


FIGURE 43.19 Bent cap retrofit.

to factored vertical loads, joint shear must still be considered as a result of longitudinal seismic response. In these situations, retrofit of the superstructure is not common since single-column bents are typically fixed to the footing and fixity at top of the column is not necessary.

Retrofit solutions that address moment and shear deficiencies typically include adding a bolster to the existing cap (Figure 43.19). Additional negative and positive moment steel can be placed on the top and bottom faces of the bolsters as required to force plastic hinging into the columns. These bolsters will also contain additional shear stirrups and steel dowels to ensure a good bond with the existing cap for composite action. Prestressing can also be included in the bolsters. In fact, prestressing has proved to be an effective method to enhance an existing cap moment, shear, and joint shear capacity. This is particularly true for bent caps that are integral with the superstructure.

Special consideration should be given to the detailing of bolsters. The engineer needs to consider bar hook lengths and bending radii to make sure that the stirrups will fit into the bolsters. Although, the philosophy is to keep the superstructure elastic and take all inelastic action in the columns, realistically, the cap beams will have some yield penetration. Thus, the new bolster should be detailed to provide adequate confinement for the cap to guarantee ductile behavior. This suggests that bolsters should not be just doweled onto the existing bent cap. There should be a continuous or positive connection between the bolsters through the existing cap. This can be achieved by coring through the existing cap. The hole pattern should be laid out to miss the existing top and bottom steel of the cap. It is generally difficult to miss existing shear stirrups so the engineer should be conservative when designing the new stirrups by not depending on the existing shear steel. The steel running through the existing cap that connects the new bolsters can be continuous stirrups or high-strength rods which may or may not be prestressed.

The cap retrofit is much easier with an exposed cap but can be done with integral caps. In order to add prestressing or new positive and negative steel and to add dowels to make sure that the bolsters in adjacent bays are continuous or monolithic, the existing girders have to be cored. Care must be taken to avoid the main girder steel and/or prestressing steel.

Torsion shall be investigated in situations where the superstructure, cap beam, and column are monolithic. In these situations, longitudinal loads are transferred from the superstructure into the columns through torsion of the cap beam. Superstructures supported on cap beams with bearings are unlikely to cause torsional problems. Torsion is mainly a problem in outriggers connected to columns with top fixed ends. However, torsion can also exist in bent cap beams susceptible to softening due to longitudinal displacements. This softening is initiated when top or especially bottom longitudinal reinforcement in the superstructure is not sufficient to sustain flexural demands due to the applied plastic moment of the column. Retrofit solutions should ensure adequate member strength along the load path from superstructure to column foundation.

In general, the philosophy of seismic design is to force column yielding under earthquake loads. In the case of an outrigger, the cap beam torsional nominal yield capacity should be greater than the column flexural plastic moment capacity. Torsion reinforcement should be provided in addition to reinforcement required to resist shear, flexure, and axial forces. Torsion reinforcement consists of closed stirrups, closed ties, or spirals combined with transverse reinforcement for shear, and longitudinal bars combined with flexural reinforcement. Lap-spliced stirrups are considered ineffective in outriggers, leading to a premature torsional failure. In such cases, closed stirrups should not be made up of pairs of U-stirrups lapping one another. Where necessary, mechanical couplers or welding should be used to develop the full capacity of torsion bars. When plastic hinging cannot be avoided in the superstructure, the concrete should be considered ineffective in carrying any shear or torsion. Regardless where plastic hinging occurs, the outrigger should be proportioned such that the ultimate torsional moment does not exceed four times the cracking torque. Prestressing should not be considered effective in torsion unless bonded in the member. Unbonded reinforcement, however, can be used to supply axial load to satisfy shear friction demands to connect outrigger caps to columns and superstructure. Bonded tendons should not be specified in caps where torsional yielding will occur. Designers must consider effects of the axial load in caps due to transverse column plastic hinging when satisfying shear and torsion demands.

43.4.7 Abutments

Abutments are generally classified into two types: seat type and monolithic. The monolithic type of abutment is commonly used for shorter bridges, whereas longer bridges typically use a seat type. Contrary to the seat-type abutment, the monolithic abutment has the potential for heavy damage. This is largely due to the fact that the designer has more control through the backwall design. The backwall behaves as a fuse to limit any damage to the piles. However, since this damage is not a collapse mechanism, it is therefore considered to be acceptable damage. Additionally, the monolithic abutment has proven itself to perform very well in moderate earthquakes, sustaining little or no damage. Some typical problems encountered in older bridges are

- Insufficient seat length for seat-type abutments;
- Large gallery, or gap, between the backwall and superstructure end diaphragm;
- Insufficient longitudinal and/or transverse shear capacity;
- Weak end diaphragms at monolithic abutments.

Following are some of the more common types of retrofits used to remedy the abutment problems mentioned above.

43.4.7.1 Seat Extenders

Seat extenders at abutments and drop caps generally consist of additional concrete scabbed onto the existing face (Figures 43.20 and 43.21). The design of seat extenders that are attached to existing abutment or bent cap faces should be designed like a corbel. When designing the connecting steel between the new seat extender and the existing concrete, shear friction for vertical loads should be considered. Tensile forces caused by friction should also be considered when the girder moves in

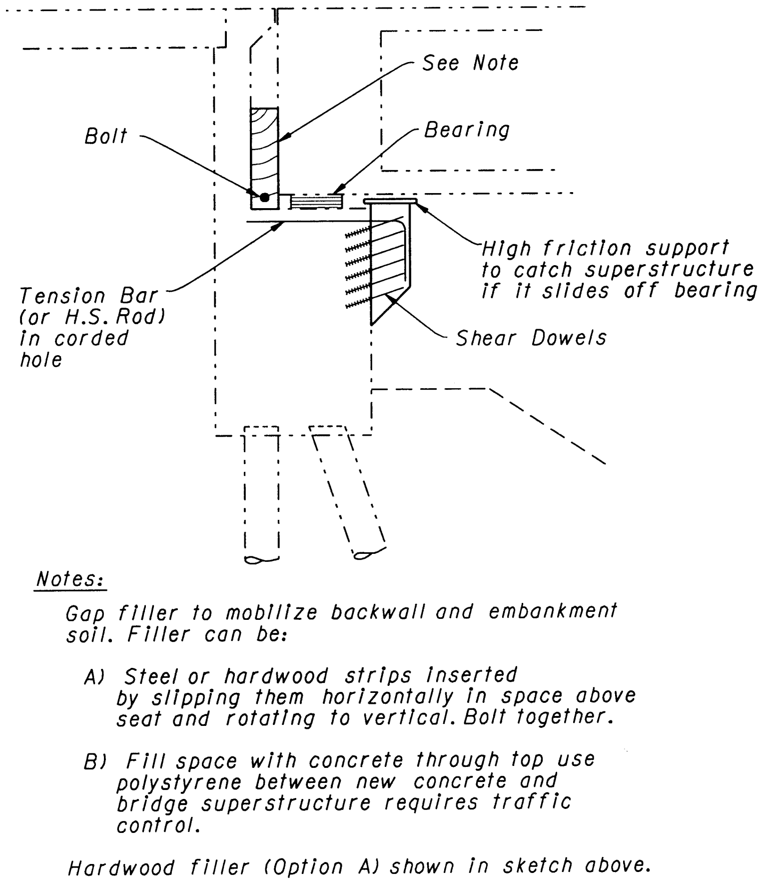


FIGURE 43.20 Seat extender at abutment.

the longitudinal direction and pulls the new concrete away from the existing bent cap or abutment. Compression strut and bearing loads under the girder also need to be considered. Note that the face of the existing concrete should be intentionally roughened before the new concrete is placed to ensure a good bond.

If bearing failure results in the superstructure dropping 150 mm or more, catchers could be added to minimize the drop. Catchers generally are designed to limit the superstructure drop to 50 mm and can provide additional seat width. In other words, catchers are basically seat extenders that are detailed to reduce the amount the superstructure is allowed to drop. The design procedure is similar for both seat extenders and catchers. In some cases, an elastomeric bearing pad is placed on top of the catcher to provide a landing spot for the girder after bearing failure. The friction factor for concrete on an elastomeric pad is less than for concrete on concrete so the tension force in the corbel could possibly be reduced. One special consideration for catchers is to make sure to leave enough room to access the bearing for inspection and replacement.

43.4.7.2 Fill galleries with timber, concrete or steel

Some seat-type abutments typically have a gallery, or a large gap, between the superstructures end diaphragm and the backwall. It is important to realize that the columns must undergo large deformations before the soil can be mobilized behind the abutment if this gap is not filled. Therefore, as a means of retrofit, the gallery is filled with concrete, steel, or timber to engage the backwall and, hence, the soil (Figure 43.22). However, timber is a potential fire hazard and in some parts of the

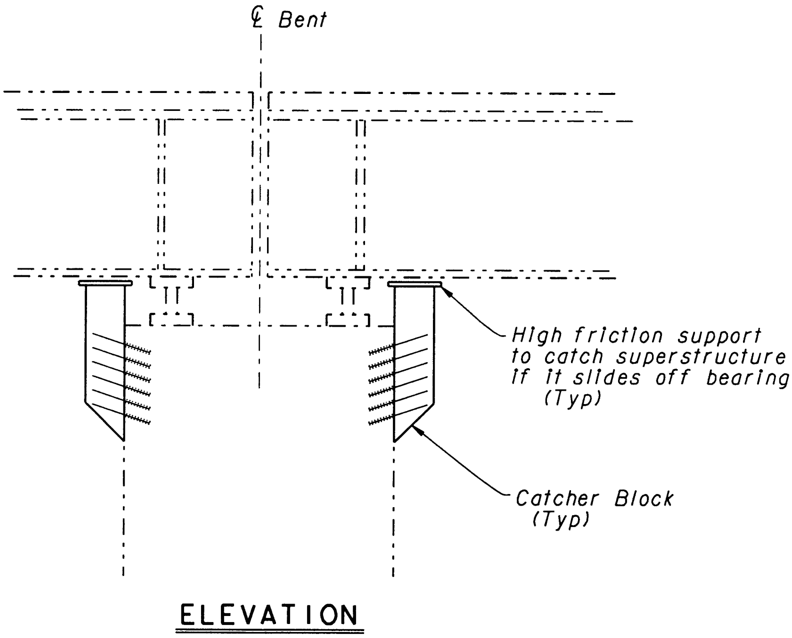


FIGURE 43.21 Seat extender at bent cap.

United States may be susceptible to termite attack. When filling this gap, the designer should specify the expected thermal movements, rather than the required thickness. This prevents any problems that may surface if the backwall is not poured straight.

43.4.7.3 L Brackets on Superstructure Soffit

Similar in theory to filling the gallery behind the backwall is adding steel angles (or brackets) to the flanges of steel I-girders, as seen in Figure 43.23. These brackets act as "bumpers" that transfer the longitudinal reaction from the superstructure into the abutment, and then into the soil.

43.4.7.4 Shear Keys, Large CIDH Piles, Anchor Slabs, and Vertical Pipes

For shorter bridges, an effective retrofit scheme is to attract the forces away from the columns and footings and into the abutments. This usually means modifying and/or strengthening the abutment, thereby "locking up" the abutment, limiting the displacements and, hence, attracting most of the loads. Although this type of retrofit is more effective in taking the load out of columns for shorter structures, the abutments still may require strengthening, in addition to retrofitting the columns, for longer bridges.

Methods that are intended to mobilize the abutment and the soil behind the abutment may consist of vertical pipes, anchor piles, seismic anchor slabs, or shear keys as shown in Figures 43.24 and 43.25. For heavily skewed or curved bridges, anchor piles or vertical pipes are generally the preferred method due to the added complication from geometry. For instance, as the bridge rotates away from the abutment, there is nothing to resist this movement. By adding an anchor pile at the acute corners of the abutment, the rotation is prohibited and the anchor pile then picks up the load.

43.4.7.5 Catchers

When the superstructure is founded on tall bearings at the abutments, the bearings, as mentioned earlier, are susceptible to damage. If tall enough, the amount of drop the superstructure would undergo can significantly increase the demands at the bents. To remedy this, catcher blocks are constructed next to the bearings to "catch" the superstructure and limit the amount of vertical displacement.

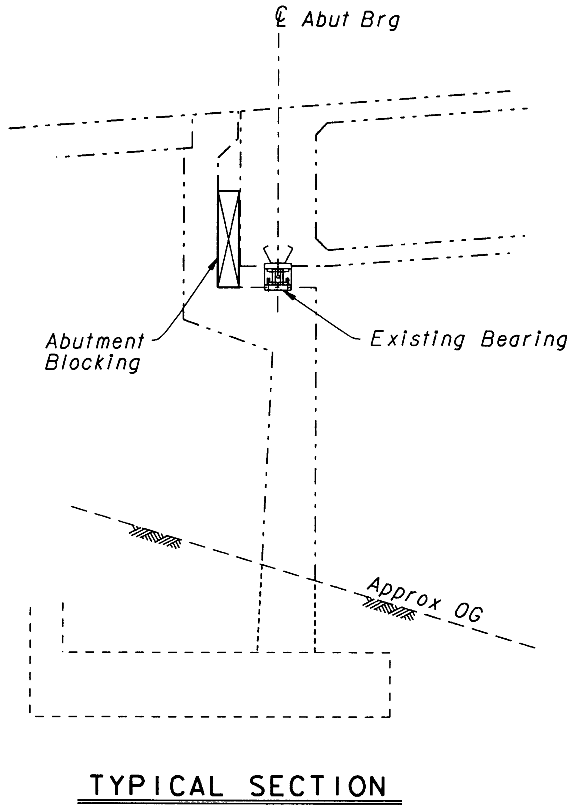


FIGURE 43.22 Abutment blocking.

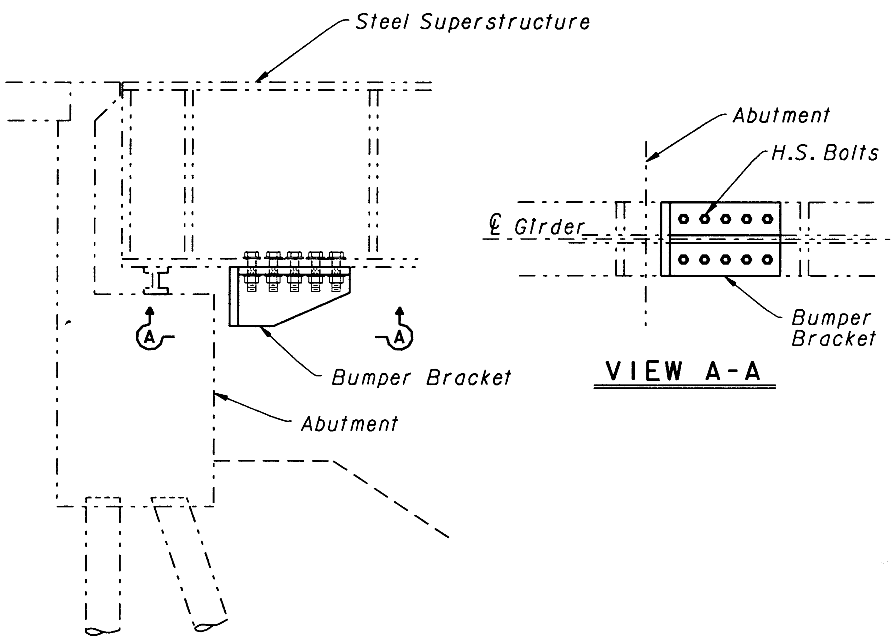


FIGURE 43.23 Bumper bracket at abutment.

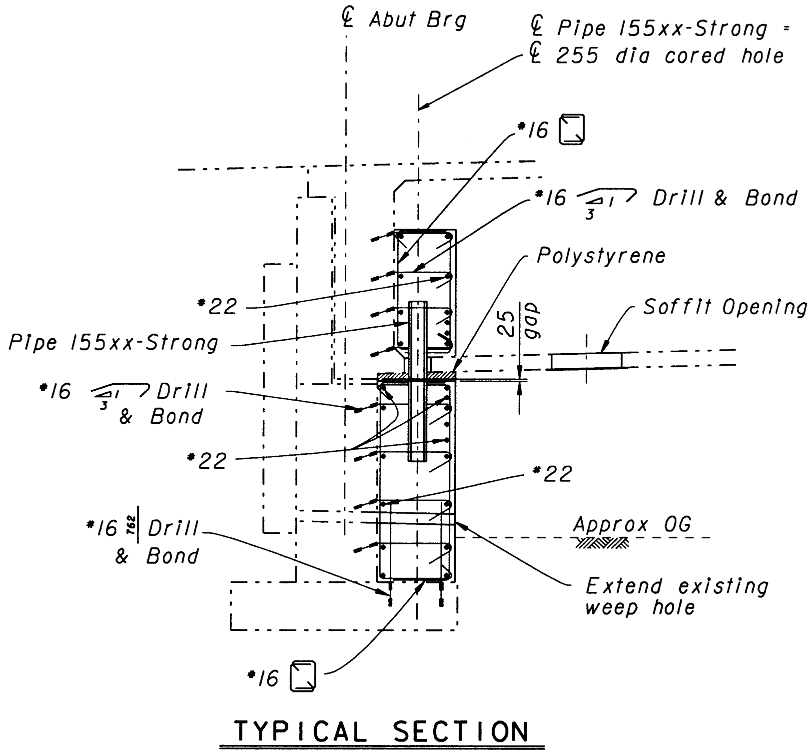


FIGURE 43.24 Vertical pipe at abutment.

43.4.8 Foundations

Older footings have many vulnerabilities that can lead to failure. The following is a list of major weaknesses encountered in older footings:

- Lack of top mat reinforcement and shear reinforcement;
- Inadequate development of tension pile capacity;
- Inadequate size for development of column plastic moment.

Footings can be lumped into two categories:

I. Spread footings resting on relatively dense material or footings resting on piles with weak tension connection to the footing cap. This latter group is treated similarly to spread footings since a strategy can be considered to ignore the supporting piles in tension.
II. Footings with piles that act in tension and compression.

In general, retrofit of footings supporting columns with a class P type casing is not needed; retrofit of footings supporting columns with class F casing is needed to develop the ultimate demand forces from the column. Typically, complete retrofit of one bent per frame including the column and the footing is recommended. However, retrofit in multicolumn bents can often be limited to columns because of common pin connections to footings. Footing retrofit is usually avoided on multicolumn bridges by allowing pins at column bases as often as possible. Pins can be induced by allowing lap splices in main column bars to slip, or by allowing continuous main column bars to cause shear cracking in the footing.

For category I footings supporting low- to medium-height single columns, rocking behavior of the bent should be investigated for stability and the footing capacity can be compared against the

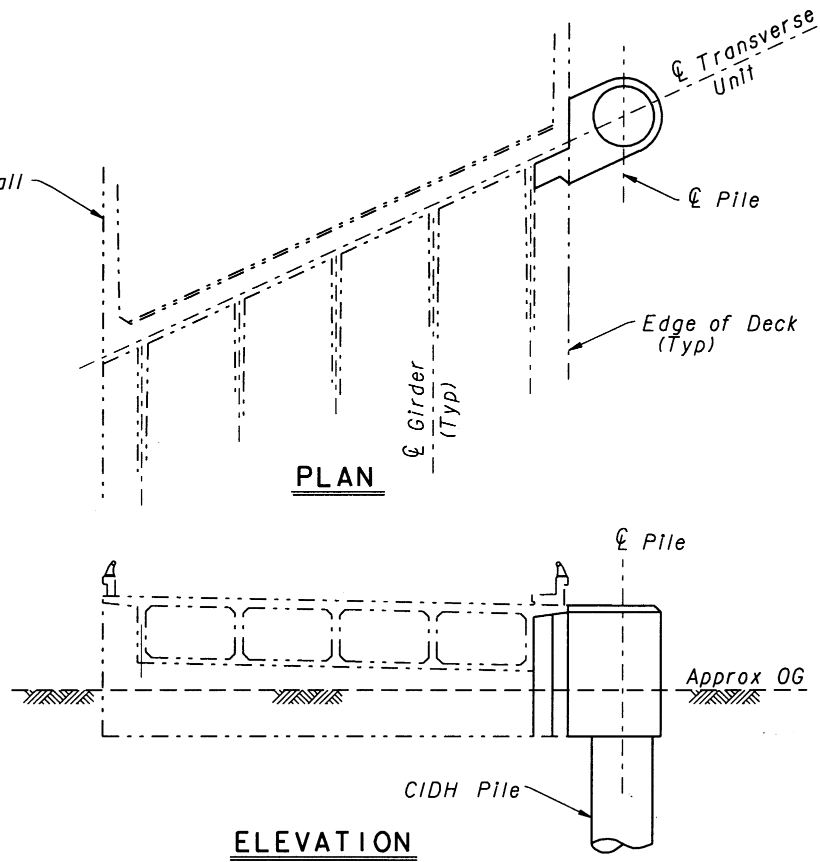


FIGURE 43.25 Anchor pile at abutment.

resulting forces from the rocking analysis. These forces can be of lesser magnitude than forces induced by column plastic hinging. Typically, retrofits for this case consist of adding an overlay to enhance the footing shear capacity or even widening of the footing to gain a larger footprint for stability and increase of the flexural moment capacity. The new concrete is securely attached to the old footing. This is done by chipping away at the concrete around the existing reinforcement and welding or mechanically coupling the new reinforcement to the old one. Holes are then drilled and the dowels are bonded between the faces of the old and new concrete as shown in Figure 43.26.

For category I footings supporting tall, single columns, rocking behavior of the bent can lead to instability, and some additional piles might be needed to provide stability to the tall bent. This type of modification leads to increased shear demand and increased tension demand on the top fiber of the existing footing that requires addition of a top mat reinforcement (Figure 43.26). The top mat is tied to the existing footing with dowels, and concrete is placed over the new piles and reinforcement. Where high compressive capacity piles are added, reinforcement with an extension hook is welded or mechanically coupled to existing bottom reinforcement. The hook acts to confine the concrete in the compression block where the perimeter piles are under compression demand.

When tension capacity is needed, the use of standard tension/compression piles is preferred to the use of tie-downs. In strong seismic events, large movements in footings are associated with tie-downs. Generally, tie-downs cannot be prestressed to reduce movements without overloading existing piles in compression. The tie-down movements are probably not a serious problem with short columns where P–Δ effects are minimal. Also, tie-downs should be avoided where groundwater could affect the quality of installation.

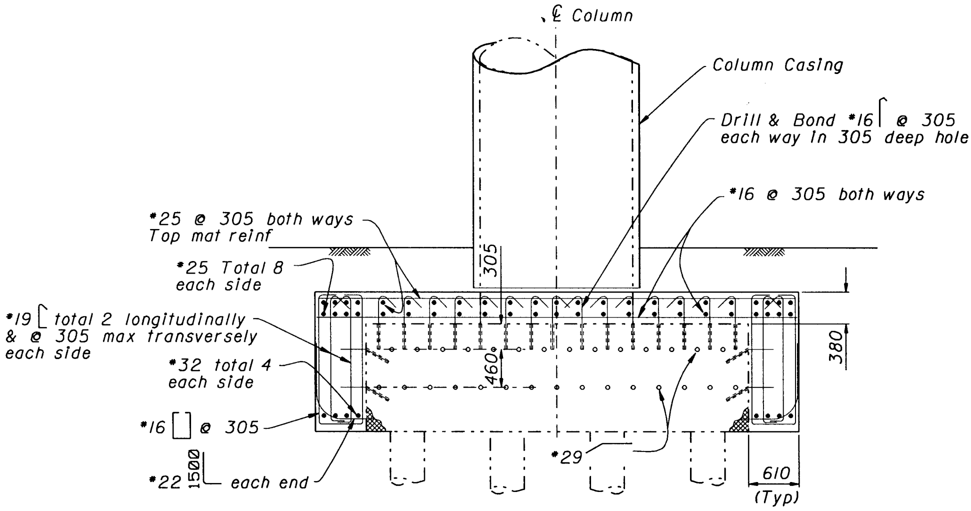


FIGURE 43.26 Widening footing retrofit.

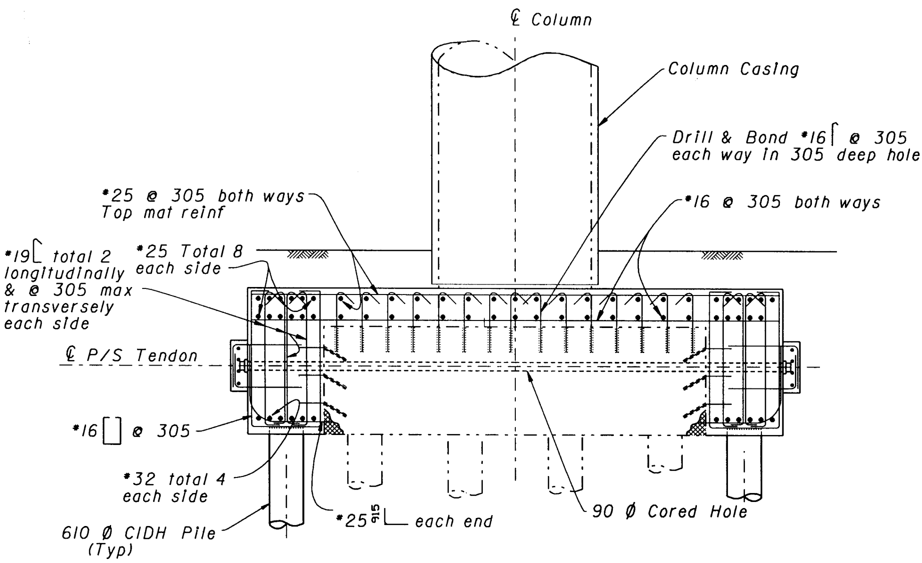


FIGURE 43.27 Footing retrofit using prestressing.

For category II footings, the ability of existing piles to cause tension on the top fiber of the footing where no reinforcement is present can lead to footing failure. Therefore, adding a top overlay in conjunction with footing widening might be necessary.

In sites where soft soil exists, the use of larger piles (600 mm and above) may be deemed necessary. These larger piles may induce high flexural demands requiring additional capacity from the bottom reinforcement. In this situation, prestressing of the footing becomes an alternative solution since it enhances the footing flexural capacity in addition to confining the concrete where perimeter piles act in compression (Figure 43.27). This retrofit is seldom used and is considered a last recourse.

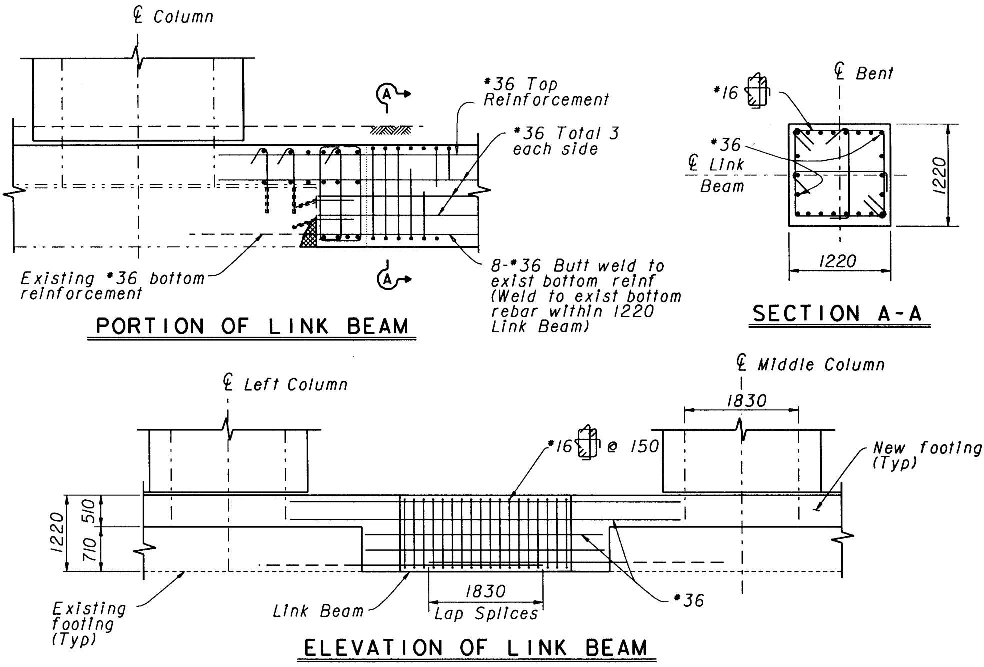


FIGURE 43.28 Link beam footing retrofit.

A rare but interesting situation occurs when a tall multicolumn bent has pinned connections to the superstructure instead of the usual monolithic connections and is resting on relatively small spread footings. This type of bent is quite vulnerable under large overturning moments and the use of link beams, as shown in Figure 43.28, is considered economical and sufficient to provide adequate stability and load transfer mechanism in a seismic event.

43.5 Summary

The seismic-resistant retrofit design of bridges has been evolving dramatically in the last decade. Many of the retrofit concepts and details discussed in this chapter have emerged as a result of research efforts and evaluation of bridge behavior in past earthquakes. This practice has been successfully tested in relatively moderate earthquakes but has not yet seen the severe test of a large-magnitude earthquake. The basic philosophy of current seismic retrofit technology in the U.S. is to prevent collapse by providing sufficient seat for displacement to take place or by allowing ductility in the supporting members. The greatest challenge to this basic philosophy will be the next big earthquake. This will serve as the utmost test to current predictions of earthquake demands on bridge structures.

References

1. Department of Transportation, State of California, Bridge Design Specifications, unpublished revision, Sacramento, 1997.
2. Mander, J. B., Priestley, M. J. N., and Park, R., *Theoretical stress-strain model for confined concrete, J. Struct. Eng. ASCE,* 114(8), 1804–1826,
3. Department of Transportation, State of California, Caltrans Memo to Designers, Sacramento, May 1994.

4. Department of Transportation, State of California, Caltrans Bridge Design Aids, Sacramento, October 1989.
5. Priestley, M. J. N. and Seible, F., *Seismic Assessment and Retrofit of Bridges*, Department of Applied Mechanics and Engineering Sciences, University of California, San Diego, La Jolla, July 1991, Structural Systems Research Project SSRP — 91/03.
6. Haroun, M. A., Pardoen, G. C., Shepherd, R., Haggag, H. A., and Kazanjy, R. P., *Cyclic Behavior of Bridge Pier Walls for Retrofit*, Department of Civil and Environmental Engineering, University of California, Irvine, Irvine, December 1993, Research Technical Agreement RTA No. 59N974.