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# FOUNDATION CONSIDERATIONS FOR SEISMIC RETROFIT OF BRIDGES

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## ABSTRACT

Seismic retrofit of bridges requires the development of strategies that address structural, geotechnical and construction issues, particularly with regard to foundation remediation. Tactics available to the engineer include ground improvement to mitigate geotechnical seismic hazards such as liquefaction, lateral spreading, seismic settlement etc., and foundation rehabilitation to improve seismic performance of the foundation. This paper provides background on the selection of appropriate foundation types, often governed by site constraints such as potential for damage to existing structures from construction vibrations, limited overhead clearance or limited right-of-way. To illustrate some of the design and construction issues that are involved, two case histories of short to medium span bridges in Southern California which were retrofitted under the California Bridge Seismic Retrofit Program are presented. This paper serves to highlight how innovation in foundation construction has helped develop economical and more constructible solutions.

## Introduction

In the late 1980's, the California Department of Transportation (Caltrans) undertook an extensive bridge seismic retrofit program with approximately one-third of the total 12,325 bridges state wide identified as potential candidates for retrofit. Subsequent screening analyses led to selection of approximately 2,400 bridges for retrofit. In the 1990's, the author was involved in the retrofit design or construction of several such bridges in northern Los Angeles County in the vicinity of Northridge.

This paper begins with a brief review of the development of seismic bridge design codes in California. The seismic bridge design philosophy along with the foundation design practices are then presented, followed by a description of the impact of higher design earthquakes on foundation design and related geotechnical hazards. Strategies and tactics to mitigate the seismic hazard and improve/retrofit the structures are then presented. This is followed by an examination of two case histories which provide some insight into the inter-related role of structural, geotechnical and construction considerations in foundation remediation.

## History and Background

In 1940, the California Department of Highways, now Caltrans, developed the first criteria within the US for the design of bridges to resist seismic forces, and included them into their design guidelines. Early code requirements for seismic design employed a horizontal force equal to some fraction of the weight of the

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structure. The origins of this early seismic bridge design code are probably the 1930 Uniform Building Code, which required that building structures be designed to resist a static force of 0.075W or 0.10W at every elevation, where W is the dead load above that elevation. These force requirements were small enough that most bridges would have satisfied them, i.e., a bridge designed to carry its own weight plus traffic loads would have more lateral resistance than required by these criteria, even more so since the transverse and longitudinal forces were not applied together. A comparison of the minimum design forces obtained from the Universal Building Code (UBC, 1940 through 1965) with the Caltrans design forces used during the same period, indicates that the forces required by the UBC were 1.5 to 3.9 times as much (Housner, 1990). These Caltrans criteria remained virtually unchanged until they were modified in 1965. These were followed by major revisions after the 1971 San Fernando earthquake. In 1971, after several freeway structures had collapsed during the San Fernando earthquake, Caltrans initiated the development of new design criteria to incorporate technical improvements in site response analyses, including ground motion attenuation and soil effects, and dynamic response of bridge structures. It also introduced ductile detailing for concrete structures. This effort resulted in the Caltrans ARS Spectra, where A, R and S relate to the maximum expected bedrock acceleration (A), the normalized rock response (R), and the soil amplification spectral ratio (S). Between 1978 and 1982, the Applied Technology Council (ATC), in a series of publications presented Seismic Design Guidelines for Bridges which included ground motion spectra. The 5% damped Caltrans ARS Elastic Response Spectra, the ATC-6 Soil Type III Ground Motion Spectra and the equivalent allowable stress design spectra used by Caltrans between 1943 and 1965 for multi-pier bents are shown in Fig. 1. Note that the ARS spectra and the pre-1971 design spectra are not strictly comparable since the ARS values have to be divided by an adjustment factor Z to determine design values. The adjustment factor is based on ductility, redundancy and over-strength provided by different systems.

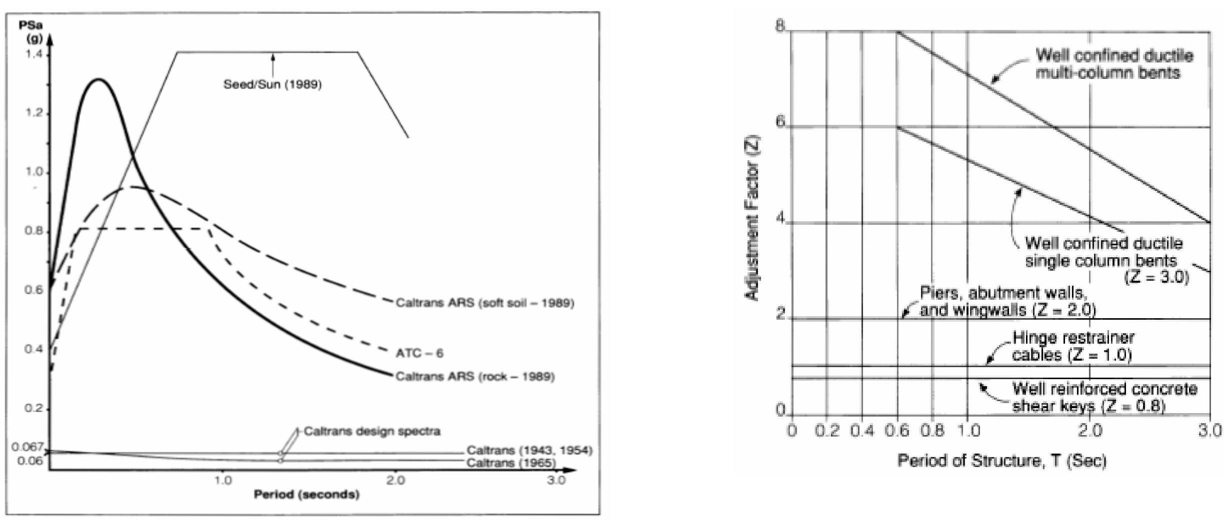



Figure 1. Acceleration Response Spectra Curves for Caltrans Bridges (Housner, 1990).

## Seismic Design Philosophy

The seismic vulnerability of the “important” Caltrans bridges is evaluated for two seismic hazard levels, a Functional-Evaluation Level ground motion that has a 60% chance of not being exceeded in 50 years (which is equivalent to a 500-year return period event) and a Safety-Evaluation Level with a 1000 to 2500-year return period event or a deterministically assessed “maximum credible earthquake.” Under the functional level these bridges are designed to be serviceable, i.e., have minimal damage that is repairable, where such repairs could be performed without road closure and full access could be provided to traffic immediately after the earthquake. This means that these structures are designed to have an essentially elastic performance under the functional level earthquake loading. Under the Safety-Evaluation Level these bridges are designed to prevent collapse, i.e., have some damage that could be repaired, where such repairs could be performed without road closure and nearly full access could be provided to traffic almost immediately after the earthquake. This means that the foundations for these structures are designed to have an essentially elastic performance under both levels of earthquake loading.

The remaining bridges, which are in the vast majority, are classified as Ordinary Bridges, are designed or retrofit to meet the criterion where collapse is to be avoided during a "maximum credible earthquake," but the resulting damage may be significant either requiring closure for major repairs or not be repairable and lead to possible replacement of the structure. It is assumed that the Ordinary Bridges will automatically meet the requirements for the Functional level earthquake if they are designed to meet the performance criteria of the safety-evaluation ground motion. This approach focuses on life safety, or collapse prevention, and the performance (in terms of displacement) that could be expected from such a structure was not defined. Consequently, while structures are designed to prevent collapse, substantial damage contributing to loss of serviceability and related externalities are not explicitly addressed. This paper deals primarily with such bridges in relatively simple soil conditions.

Caltrans seismic bridge design methods consist primarily of comparing the earthquake load demand with corresponding capacity of the structure. These methods assume that the structures respond elastically to earthquakes. The elastic lateral load demand obtained from an elastic response spectral analysis is empirically reduced to reflect energy dissipation from the inelastic behavior of the structural elements. These reduced load demands are then compared with the load capacity of the bridge structure members. The primary focus of the Caltrans' seismic design philosophy is to avoid inelastic behavior in the foundation elements, wherever possible, because of practical difficulties in identifying, assessing and repairing such damage after an earthquake. Therefore, in Caltrans' approach, the foundation elements are either treated as rigid or elastic and it is ensured that they have adequate capacity to meet the structural load demand. The substructure (columns and piers) is designed to perform as pinned connection or to form a plastic hinge above the foundation so as to limit the load demand on the foundation which is designed to carry these reduced loads elastically. Seismic resistance of a bridge structure can be improved by improving strength, ductility, energy dissipation characteristics of the bridge, and also by reducing the seismic loads by seismic isolation.

## Impact on Substructure Design

When the existing bridges, mostly built between 1950's and 1970's were analyzed per the revised seismic criteria, it was found that it resulted in significantly higher design lateral loads and consequently higher overturning moments associated with inertial loading. Analyses indicated a critical structural deficiency in the bridge substructure which directly affected the continued ability of the bridge structure to carry gravity loads. It involved insufficient strength and ductility of the columns caused by insufficient transverse column reinforcement (hoops and stirrups). This lack of reinforcement could result in premature, catastrophic and often brittle column failures during seismic activities. Therefore, the retrofit strategy had to address the bridge columns and their frequently inadequate design for flexure, shear and overall ductility. Long and slender columns could fail by developing a local plastic hinge at the column end which, without proper transverse reinforcement, could result in bar buckling or debonding once the cover concrete spalls off. In these cases, confinement of the concrete column with an external jacket, could prevent the spalling of concrete, resulting in very ductile response with hysteretic energy absorption. Short columns had a potential for failure in diagonal tension, which required added shear strength rather than confinement from the external jackets. Therefore, the external jackets when installed on the entire length of the columns could provide the added shear capacity.

The increased structure stiffness of the retrofitted bridge structure also resulted in additional forces on the foundations. When the foundation systems of these bridges were evaluated for these higher loads, they were generally found to be inadequate. In these bridges, most connections between piles and footings were structurally inadequate for any significant uplift. Moreover, many foundations were supported on concrete piles with insufficient reinforcement continuity for tension loads, making them unreliable under the revised load criteria. Often, pile caps were found to be structurally inadequate because of insufficient reinforcement such as the absence of an upper mat of reinforcement steel for two-way bending and also the absence of vertical steel reinforcement for shear loads. The absence of such reinforcement made the footings inadequate in flexure, shear and moment rotation, requiring substantial foundation rehabilitation. Therefore, new piles that were added as part of a retrofit often were designed to carry all seismic loads. The pile caps derive substantial stiffness from the passive resistance of the soil cover. Often, however, the ground cover surrounding the pile caps was not sufficiently compacted.

The abutment stiffness dominates the seismic response of short span bridges (< 300 ft) with no hinges. Since abutments are somewhat restricted from movement by the bridge structure, they face higher earthquake induced lateral earth pressures. In addition, inertial forces from the bridge push the abutment wall into the backfill, resulting in a passive earth pressure condition, which is generally 10 to 30 times higher than the active earth pressure. Therefore, abutment walls designed for just the active earth pressure case are highly vulnerable to damage under the seismic passive pressure condition. Analyses also indicated that for many bridges that were retrofit with joint restrainers, the abutments lacked sufficient reinforcement or possessed insufficient longitudinal and transverse stiffness, thereby requiring strengthening.

## Geotechnical Seismic Hazards

Higher expected ground accelerations not only impacted the structure in terms of greater lateral forces but also led to a greater potential for geotechnical seismic hazards, including liquefaction, lateral spreading, approach embankment instability and seismic settlement which can potentially impact a bridge structure and the approach embankment.

***Soil liquefaction***: defined as a significant reduction in soil strength and stiffness as a result of increases in pore pressure during dynamic loading, is a major cause of damage during earthquakes. Typically, the hazard from liquefaction occurs in four ways, including: a) bearing failure, b) settlement, c) localized differential lateral movements, and d) ground loss or highly localized subsidence associated with expulsion of material such as “sand boils.” Usually, for soil liquefaction to occur, three conditions must exist: including a) presence of loose, sandy soils or silty soils of low plasticity, b) saturation of the soil with grounwater, and c) a source of sudden or rapid loading, typically associated with earthquakes.

***Seismic Settlement:*** Volumetric strain resulting from earthquake related vibrations will cause some ground settlement. Seismically induced settlements can result in a surficial depression and consequent differential settlement between the pavement and bridge structure. The differential settlement between the pavement and bridge structure can be mitigated by using a structural approach transition slab at the abutment. These volumetric strains also result in downdrag loads on the pile foundations. Downdrag load due to liquefaction and potential seismic settlement should be included in deep foundation design.

***Seismic Stability Analyses:*** For routine work, seismic stability of highway slopes is analyzed by using a pseudo-static earthquake coefficient which accounts for the inertial effect due to shaking. Typically a factor of safety of global stability greater than 1.1 for both the pseudostatic and the post-earthquake conditions is sought. Where the pseudostatic factor of safety is less than 1.0, permanent displacements are calculated.

***Permanent Displacement of Slopes:*** Newmark analyses are conducted to compute permanent ground displacements. First, the yield acceleration of the embankment, which is the horizontal ground acceleration at which the factor of safety of slope stability is 1.0, is computed by trial and error using pseudostatic slope stability analyses. Then, using procedures by Makdisi and Seed (1978), permanent ground displacements during the design earthquake are estimated for the approach embankment. When permanent displacements are more than the typically allowable displacement of 2 inch to 6 inch, some form of mitigation measure is employed. Mitigation measures could include ground improvement as discussed in the next section.

***Potential for Lateral Spreading:*** Lateral spreading occurs primarily by horizontal displacement of surficial soil layer due to liquefaction of underlying granular deposits. The degradation in the undrained shear resistance arising from liquefaction may lead to limited lateral spreads induced by earthquake inertial loading. Such spreads can occur on gently sloping ground or where nearby drainage or stream channels can lead to static shear biases on essentially horizontal ground. The determination of lateral spread potential and an assessment of its likely magnitude ought to be addressed as a part of the hazard assessment process. Available procedures are mostly empirical and based on observations from past earthquakes. Using regression analyses and a large database of lateral spread case histories from past earthquakes, Bartlett and Youd (1995) developed empirical equations relating lateral spread displacements to a number of site and source parameters. Unfortunately, this prediction approach is least reliable in the small displacement range. Therefore, the impact of potential lateral spreading has to be evaluated and addressed for each bridge and its foundations.

## Retrofit Strategies

Strategies available to the engineer include some form of ground improvement to mitigate geotechnical seismic hazards such as liquefaction, lateral spreading, seismic settlement etc., and foundation rehabilitation to improve performance of the foundation.

### Ground Improvement

Numerous ground improvement measures are available such as 1) removal and recompaction, 2) compaction grouting, 3) stone columns, 4) jet grouting, and, 5) deep soil mixing. Other measures such as permeation grouting also are available to the engineer. The primary focus of this paper is on bridges in relatively simple soil conditions not requiring ground improvement. Therefore, these measures are only briefly covered here. The reader is referred to Munfakh (1997, 1999) for a comprehensive treatment on ground improvement methods.

***Removal and Replacement:*** Complete removal of the backfill surrounding pile caps and replacement with cement slurry. The main drawback of this method is that it requires excavation support and may not be feasible in limited spaces where traffic has to be maintained.

***Compaction Grouting:*** can be used to densify granular soils, mitigate liquefaction potential and improve foundation performance. It involves the staged injection of low slump (< 3 inch) mortar grout into soils at high pressures (600 psi) forming grout bulbs which displace and densify the surrounding soils. At each grout location a casing is drilled to the bottom of the zone of loose soil. Compaction grout is then pumped into the casing at increments of one lineal foot. When previously determined criteria of volume, pressure and heave are met, the pumping is terminated and the casing withdrawn and the hole filled. Compaction grouting works best in soils which drain quickly, such as loose sands. One major disadvantage of compaction grouting in improving the bridge foundation soils is that it is quite ineffective at shallow depths and in areas with limited overburden stress. Near the surface, or where overburden stress is limited, the grout bulbs will simply heave the ground, and hence, not compact the soils. Above a depth of about 10 feet, the pressures under which the grout is placed have to be greatly reduced, whereby a lesser degree of soil compaction can be achieved. In addition, compaction grouting can have an impact on existing buried structures and utilities. Therefore, the presence of such constraints is a factor in the selection of this method.

***Vibro-compaction and Stone Columns:*** Vibro-compaction is used primarily in granular soils where excess pore pressures may drain rapidly. It is effective when the relative density is less than 70 percent. This method is not effective in partly saturated soils with 20 percent or more passing the No. 200 sieve. Stone columns are suitable for use in fine grained soil, but are rarely used in coarse-grained soils. The open graded nature of the stone column allows quick dissipation of the excess pore water pressure generated by the earthquake, thus reducing the liquefaction potential. When used in sandy soils, the soils between adjacent columns are displaced and densified by the operation, consequently improving the soil strength, and increasing its resistance to liquefaction. The improved soil in turn provides greater lateral foundation support.

***Jet Grouting:*** Jet Grouting is a soil replacement process in which a high-pressure water jet is used to erode the native soil and mix it or replace it with a stabilizer such as cement resulting in a dense mixture of soil and cement. This method can be used for improving soils of any type. The main limitation of jet grouting is that the very high pressure used in grouting may fracture the surrounding soil or result in heave and excess deformations that affect structures and utilities. Therefore, it is an operator sensitive system and requires a skilled and experienced contractor.

***Deep Soil Mixing:*** Deep soil mixing is the mechanical blending of the in-situ soil with cement at depth using an overlapping auger and mixing paddle arrangement. The soil fabric is disturbed by the penetration of the auger, then mixed with cement grout as the auger is withdrawn resulting in compacted soil-cement columns. The overlapping soil mix columns are sometimes arranged in a lattice pattern to provide resistance in the case when the original soil liquefies under seismic loading.

**Foundation Rehabilitation**

Several foundation retrofit strategies are available to the engineer, including: 1) installing tiedown anchors to provide uplift capacity, 2) increasing footing size, 3) underpinning with higher capacity perimeter piles, and 4) complete replacement of the footing.

***Tiedown Anchors:*** Tiedown anchors are an economical solution where the existing piles supporting the structure have adequate capacity in compression. The tiedown anchors are generally not prestressed so as to avoid overloading the existing foundations in compression. Details of a typical footing retrofit with tiedown anchors are shown in Fig. 2.

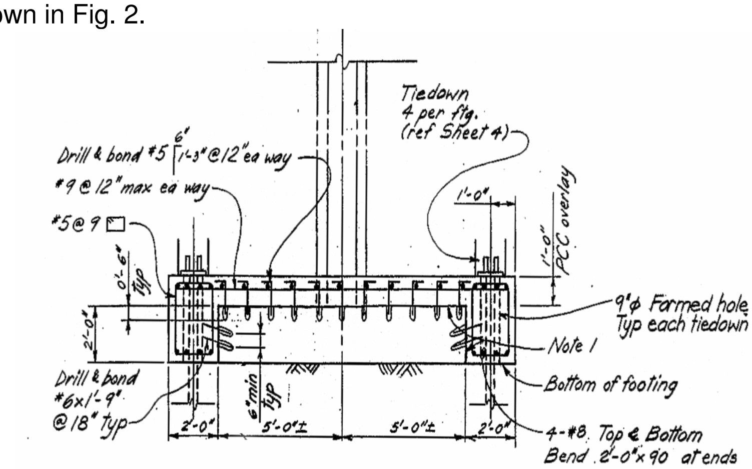



Figure 2. Typical Footing with Tiedown Anchors (PB, 1993).

This, in turn, makes the tiedown anchors prone to deformation softening, i.e., with increasing relative soil-anchor displacement, the shear stress transferred from the anchor to the soil decreases. This deformation softening depends on the relative anchor-soil compressibility and is pronounced for slender unstressed anchors. The relative soil-anchor compressibility is a function of the diameter, length, elastic modulus of the anchor and stiffness of the soil. In view of the above, the use of tiedown anchors is limited to foundations supporting relatively short columns, so as to limit the lateral displacement at the top of the column.

***Increased Footing Size and Underpinning:*** The most common foundation retrofit strategy is to increase footing size, typically in conjunction with underpinning. In this strategy the existing bottom reinforcement mat is extended and tied to the underpinning perimeter piles. High strength dowels are also installed to facilitate shear transfer between the existing and new concrete. Reinforcement overlay is commonly installed on the top of the footing along with vertical tie reinforcement that ties the upper reinforcement with the lower reinforcement. The designer must determine the overlay thicknesses and the number of reinforcement mats to satisfy load demand. The purpose of the overlay is to act as a two-way beam. Details for a typical footing retrofit is shown in Fig. 3.

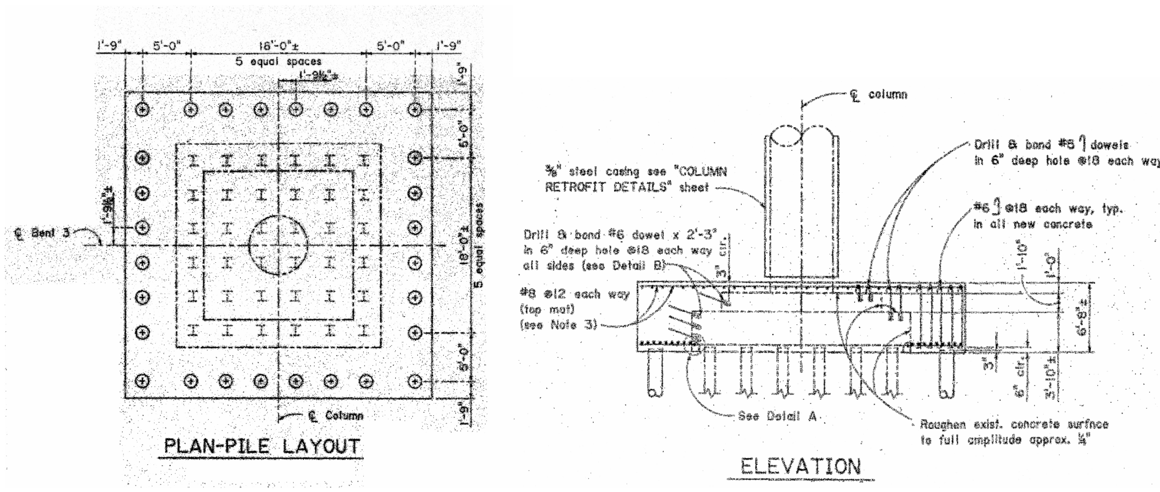



Figure 3. Typical Foundation Underpinning Detail (PB, 1993).

***Foundation Replacement:*** Footing replacement is usually considered where other physical space constraints such as adjacent traffic lanes that cannot be closed and excavation depth prevent retrofit solutions. At other times, where post-earthquake serviceability requirements govern the design causing modifications to be extensive, footing replacement is preferable.

Strategies for abutment retrofit included: 1) installing large diameter drilled shafts behind abutment to improve longitudinal stiffness, 2) installing large diameter drilled shafts on each side of the abutment to improve transverse stiffness, 3) structural reinforcement by adding pilasters, 4) installing a pile supported anchor slab and tying it to the abutment, 5) removal of abutment backfill and replacement with soil cement mix, and 6) complete replacement of the abutment.

Drilled shafts with diameters as large as 4.5 ft have been installed immediately behind the abutment walls along with post-tensioned high strength steel anchors tying the drilled shafts to the abutment wall, and sometimes, extending to the adjacent bent cap. Details for such a abutment retrofit are shown in Fig. 4.

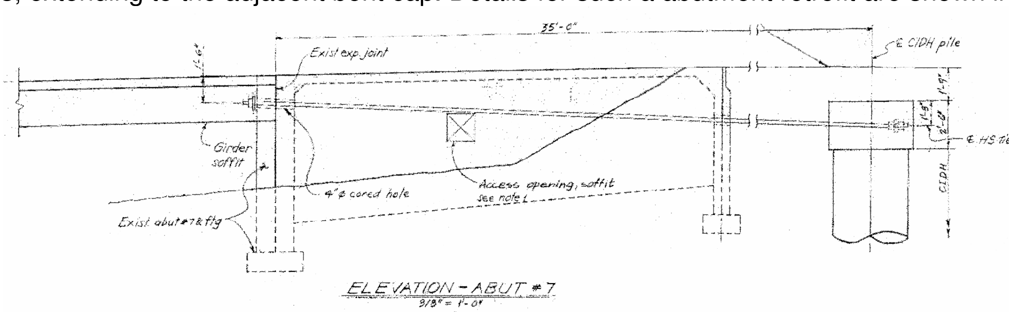



Figure 4. Typical Abutment Retrofit Detail (PB, 1993).

Transverse stiffness requirements are addressed by installing single large diameter drilled shafts on either side. These shafts also serve as shear keys and restrainers for bridge girders on a seat-type abutment. Details for a typical abutment retrofit are shown in Fig. 5.

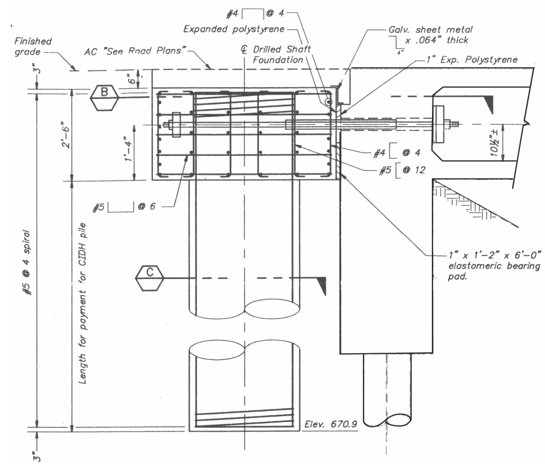



Figure 5.  Typical Abutment Retrofit Detail (PB, 1993).

Where traffic maintenance considerations allow it, pile-supported approach slabs have been installed, effectively anchoring the abutment.  At locations where traffic maintenance considerations allow it, the structure backfill behind the abutment wall is removed and replaced with a 5 percent soil-cement mix to serve as a relatively monolithic gravity mass with ample stiffness.  Only rarely would abutment walls be replaced.

## Foundation Selection and Construction Issues

Constructibility considerations also play a major role in foundation rehabilitation.  Site constraints such as limited access, limited overhead clearance, other obstructions, and proximity to vibration sensitive structures and utilities often have a bearing on the selection of an appropriate deep foundation.  A foundation selection study has become essential to a successful retrofit program.  Various types of piles including steel H-piles, closed and open-ended steel pipe piles, CIDH piles, and small diameter pressure grouted drilled minipiles are evaluated with respect to: 1) lateral and vertical load and moment carrying capacity, 2) construction considerations including the effect on adjacent existing structures, and the presence of obstructions, 3) the potential corrosive soil environment, and 4) economic considerations. The following paragraphs discuss available pile types and the advantages of each.

***Steel H-Piles*** are low displacement piles which, when installed with a hardened driving tip, can be driven through fill materials containing debris or through gravel or cobble deposits, with low risk of damage. When compared to full displacement piles such as closed-end pipe piles, driving steel H-piles cause less disturbance to the surrounding ground and adjacent structures.  However, steel H-piles when installed in loose to medium density sands result in greater driving lengths due to typically lower friction and end bearing resistances.  The disadvantage of the steel H-piles is that they offer limited lateral capacity which will be reduced even further because of pre-drilling.  Nevertheless, uncertainty regarding disturbance and potential vibration related damage to adjacent structures are factors often considered in the selection of steel H-piles.

***Closed-end pipe piles*** offer the advantages of high axial load and moment capacity and ease of availability.  Moreover, driving the piles closed-ended enables inspection of the installed piles for verticality and damage (if any).  Following inspection, the piles are filled with concrete.  However, installing full displacement piles adjacent to existing structures has a great potential for causing vibration-induced permanent ground settlement, ground displacement and related disturbance and distress to nearby structures.  Ground vibrations and disturbance to adjacent structures can be somewhat mitigated by predrilling an undersized hole prior to driving the pile. However, ground vibrations and displacements can still be considerable.  Therefore, these piles are usually not recommended for retrofitting existing foundations.

***Open-end pipe piles,*** like closed-end piles, offer excellent vertical load and moment capacity. To prevent hitting obstructions within the fill open ended piles also have to be installed through predrilled holes. Like steel H piles, open-end pipe piles are also low displacement piles. However, these piles have a tendency to become plugged during driving. Plugged piles tend to behave like closed-end, full displacement piles and can cause ground vibrations and vibration-related disturbance to adjacent structures. Though the soil plug can be augured or jetted out and the pile continued to be driven, resulting in increased pile embedment and a somewhat uncertain capacity which needs verification by load tests.

***Cast in drilled hole (CIDH) Piles*** offer excellent vertical load and moment carrying capacity. Probably most commonly used pile type for California bridges, these piles are drilled using a continuous flight auger or a bucket auger or a reverse circulation drill. If large obstructions are encountered during drilling, a down-hole hammer can be employed to advance through them. CIDH piles are most suitable for ground conditions where the drilled holes will retain their shape and not cave in during drilling and concrete placement. Steel casing can be installed to prevent cave-ins at marginally increased costs. The process of installing the casing and then extracting it from the drilled hole with the reinforcement cage in place and while the concrete is poured is cumbersome and has potential to cause ground vibrations. Therefore, the selection of an appropriately experienced foundation contractor is essential to the successful installation of CIDH piles, which have been increasingly used in California. The potential downside to these piles is the potential for settlement of the existing foundation caused by loss of ground while drilling for a new pile. Therefore, a minimum distance between the existing piles and the new pile is usually required. However, increasing the new pile-to-old pile distance creates the need for a larger footing. Therefore, experience and judgment are required to establish the optimum pile spacing.

***MicroPiles:*** The increased seismic load demands call for using higher capacity piles that are governed by uplift. This need for higher capacity piles which can be installed in limited space with little disturbance to the existing foundations was fulfilled by the introduction of Micropiles. Micropiles are small diameter, drilled and grouted reinforced piles, a subset of cast-in-place piles. With conventional cast-in-place bored piles, small cross-sectional area is synonymous with low structural capacity. This is not the case with micropiles, however. Innovative drilling and grouting methods permit high grout/ground bond values to be generated along the micropile periphery. To exploit this benefit, high capacity steel elements can be used as the principal load bearing element with the surrounding grout serving only to transfer, by friction, the applied load between the soil and steel. End bearing is typically not relied upon, and is insignificant given the large length to diameter ratio for these piles. Early micropile diameters were around 4 inches, but with the development of more powerful drilling equipment, diameters up to 12 inches are now considered practical. Micropiles are capable of sustaining ultimate loads as high as 500 tons. Micropiles can be installed in areas of particularly difficult, variable, or unpredictable geologic conditions such as ground with cobbles, boulders, fill with buried miscellaneous debris, and irregular lenses of competent and weak materials. Soft clays, running sands, and high ground water not conducive to conventional drilled shaft or bored pile construction cause minimal impact to drilled minipile installation. The method of installation of micropiles causes less noise and vibration than conventional pile driving techniques. They are being frequently used for underpinning existing structures and can be installed in environments with space constraints. Moreover, they can be installed very close to an adjacent structure without causing damage. Special admixtures can be included in the grout mix design to reduce and avoid deterioration from acidic and corrosive environment. In view of the abovementioned reasons, micropiles are often being used for bridge foundation retrofitting.

***Jacked Piles:*** Pile jacking provides a noise and vibration free technique for pile installation in urban areas with limited space constraints. The piles are pressed into the ground by means of hydraulic rams that obtain reaction from the existing structure. A range of machines have been developed to install steel tubes up to 5 ft in diameter with a maximum force of 400 tons. Since a continuous measurement of jacking force is provided during pile jacking, the bearing capacity of each pile can be verified. This method offers tremendous advantage in terms of limiting vibration induced ground settlement and its impact on adjacent structures and is increasingly being used. However, this method requires a suitable footing or structure to develop jacking resistance, particularly for higher loads.

## Case Histories from Southern California

***Case No. 1:*** Browns Canyon Wash Bridge (off-ramp) (Br. No. 53-2182S) is located on State Route 118 in Chatsworth, Los Angeles. Completed in 1971, it crosses over Browns Canyon Wash and serves as an off-ramp to Desoto Avenue. The bridge is 551 ft long with three-spans of a three-celled cast-in-place prestressed concrete continuous box girder superstructure, each supported on a single column with a height a 65 ft for Bent 2 and 56 ft for Bent 3. There are no hinges in the structure and the columns are fixed both at the top and bottom. The West abutment and Bent 2 are supported on spread footings, while Bent 3 and the East Abutment are supported on driven H piles.

The major active or potentially active faults within the vicinity of the bridge include the Northridge, Chatsworth, Mission Hills, and Santa Susana Faults (Smith, 1977, Diblee 1992). The Northridge Hills Fault zone is about 2000 ft wide and extends approximately 13 miles southeast across the San Fernando Valley. The Browns Canyon Wash Bridge lies within the northwestern limit of this zone. Therefore, the Maximum Credible Earthquake design event was identified as a Magnitude 7.5 rupture in the Northridge Fault Zone. Iso-seismal maps of bedrock acceleration contours, developed by Mualchin and Jones (1992) from regional seismic hazard analyses, showed the bridge to lie above the 0.6g isoseismal contour, indicating that the next higher value of 0.7g be used for design.

Geotechnical data at the site were obtained from Caltrans Logs of Test Borings. These data indicate that Abutment 1 is underlain by Pleistocene age older surficial sediments that typically consist of angular pebble sized fragments of Miocene shale and sandstone in light gray to tan silty sand matrix mixed with some calcareous caliche. Bents 2 and 3 are underlain by relatively dense alluvial sediments including alluvial gravels, sands, boulders, and clays over the Cretaceous age sandstone of the Chatsworth formation. Abutment 4 rests on the thickly bedded sandstone of the Chatsworth Formation. Near the canyon bottom, groundwater was considered relatively shallow within 10 ft from the ground surface. Liquefaction potential was considered to be minimal because of high Standard Penetration Test (SPT) N-values exhibited by the relatively dense alluvium encountered at the site.

The design loads for evaluation of the structure were developed using the Caltrans standard ARS response spectrum approach which provides estimates of median spectral response for a given design event. Selection of the design ARS was based on 1) distance to a controlling fault, 2) the credible magnitude of the controlling fault, and, 3) the depth of alluvium.

The load demands on the structural members were estimated using the response spectrum method. The rotation and displacement capacity of the yielding columns was also estimated and compared with the displacement demand from the dynamic analyses. The analyses indicated that the columns were not ductile enough and required improvements. Therefore, 65 ft tall column at Bent 2 was jacketed with a 3/8-inch thick steel casing for its lower 10 ft while the 56 ft tall column at Bent 3 was encased for its entire length.

To provide better drift capacity at Bent 3, the existing 21 ft by 21 ft footing was enlarged to 31.5 ft by 31.5 ft with the addition of twenty eight 16-inch diameter CIDH piles with an allowable capacity of 70 tons. This selection of pile type was guided by cost considerations and the possibility of hard driving conditions associated with dense layers containing gravels and cobbles. Pile lengths were 35 ft and were governed by tension requirements.

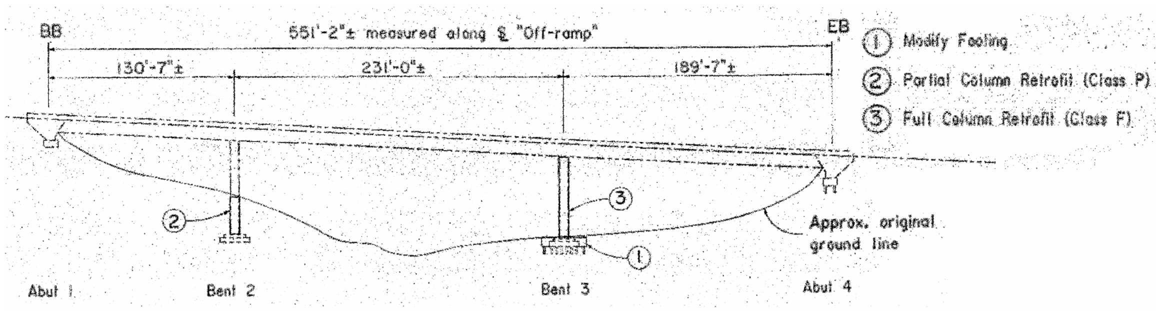
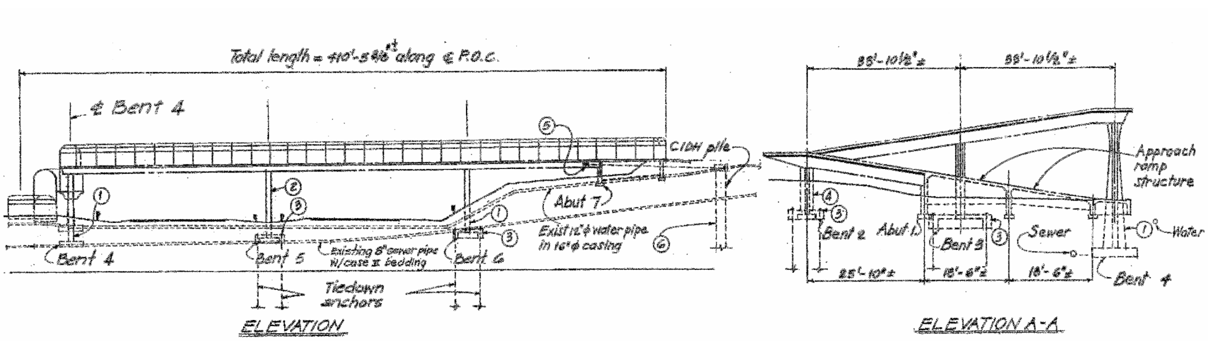



Figure 6. Retrofit measures taken at Brown's Canyon Wash Bridge (off-ramp) (PB, 1993).

High strength dowels were installed to assure shear transfer between the existing and new concrete. Reinforcement overlay was installed on the top of the footing along with vertical tie reinforcement that ties the upper reinforcement with the lower reinforcement. Generally, no significant construction problems were encountered and the repairs and retrofits were completed in mid 1992.

On January 17, 1994, nearly 1.5 years after the completion of the Browns Canyon Wash Bridge retrofit, a Magnitude 6.7 earthquake occurred with its epicenter in Northridge. The epicenter was located approximately 5 miles southeast of the bridge and the resulting peak ground accelerations at the bridge site were estimated to be between 0.6 to 0.8g. These resulting ground motions did not cause any damage to the bridge, indicating that the retrofit measures were serving their purpose.

***Case No. 2:*** Built in 1969, Harding Street Pedestrian Overcrossing (Br. No. 53-1897) is located on I-210 in Los Angeles. The bridge is 410 ft long with six-spans of a single-celled cast-in-place concrete continuous box girder superstructure, each supported on a single flared column with a height of 26 ft for Bent 5 and 30 ft for Bent 6. There are no hinges in the structure and the columns are fixed both at the top and bottom. The abutments and bents are supported on spread footings. The approach consists of a concrete slab supported on spread footings.




TYPES OF RETROFIT

1. Column Retrofit, Type F
2. Column Retrofit, Type F
3. Expand and Anchor Footing
4. Add Pilasters
5. Add Abutment Restrainers
6. Add CIDH Pile

Figure 7. Retrofit measures taken at Harding Street Pedestrian Overcrossing (PB, 1993).

The major active or potentially active faults within the vicinity of the bridge include the Northridge, Chatsworth, Mission Hills, and Santa Susana Faults (Smith, 1977, Diblee 1992). The Santa Susana Thrust fault lies closest to the site. Therefore, the Maximum Credible Earthquake design event was identified as a Magnitude 7.0 rupture in the Santa Susana Thrust fault. Isoseismal maps of bedrock acceleration contours, developed by Mualchin and Jones (1992) from regional seismic hazard analyses, showed the bridge to lie above the 0.6g isoseismal contour, indicating that the next higher value of 0.7g be used for design.

The load demands on the structural members were estimated using the response spectrum method. The rotation and displacement capacity of the yielding columns was also estimated and compared with the displacement demand from the dynamic analyses. Bent 2 was provided lateral restraint by adding pilasters. To improve column ductility, the columns at Bents 4 and 6 were partially jacketed with a 3/8-inch thick steel casing, while the column at Bent 5 was encased for its entire length.

To provide better uplift capacity at Bents 2, 3, 5 and 6, each of the existing spread footings were enlarged and tied down with four anchors with allowable uplift capacity of 120 tons. The anchors were not prestressed so as not overload the existing footings. The selection of anchors was guided by constraints such as the presence of a 12-inch diameter water line and an 8-inch diameter sewer line located on either side of the bridge, and the possibility of hard driving conditions associated with dense layers containing gravels. The tiedown anchors were 9 inches in diameter and 90 ft long. They had an unbonded length of 15 ft and a bond length of 75 ft. Again, high strength dowels were installed to assure shear transfer between the existing and new concrete. Reinforcement overlay was installed on the top of the footing along with vertical tie reinforcement tying the upper reinforcement mat, with the bottom reinforcement.

The absence of any restraint at Bent 4 was addressed by providing an abutment restrainer at Abutment 7. This involved installing a 54 inch diameter CIDH pile behind the abutment and tying it to the abutment with high strength steel tie rods.

Generally, no significant construction problems were encountered and the repairs and retrofits were completed in mid 1992. The epicenter of the Magnitude 6.7 Northridge earthquake was located approximately 8 miles southwest of the bridge and the resulting peak ground accelerations at the bridge site were estimated to be between 0.6 to 0.8g. Ground motions recorded at California Strong Motion Instrumentation Program (CSMIP) stations in the area varied between 0.44g and 0.91g for horizontal ground accelerations. Vertical accelerations measured in the vicinity ranged between 0.6g to 0.2g. These resulting ground motions did not cause any damage to the bridge indicating that the retrofit measures were serving their purpose.

## Conclusions

Most innovation is usually driven by a need. The seismic retrofit program in California created a need for ground improvement technologies to mitigate geotechnical seismic hazards and foundation rehabilitation technologies to improve seismic foundation performance. The need for high capacity foundations that could be installed in restricted right-of-way with limited overhead clearance and with little disturbance to the existing foundations and adjacent structures was fulfilled by technologies such as tiedown anchors, micropiles, and post-grouted piles. The need to mitigate geotechnical seismic hazards such as liquefaction, lateral spreading, seismic settlement etc., was fulfilled by technologies such as compaction grouting, stone columns, jet grouting and deep soil mixing. To illustrate some of the design and construction issues that are involved in foundation retrofitting, two case histories are presented of short to medium span bridges in Southern California which were retrofitted under the California Bridge Seismic Retrofit Program. Many of the foundation retrofit measures shown in these two case histories have been routinely used since and are now standard.

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