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20-4 Seismic Retrofit Guidelines For Bridges In California

Introduction

Caltrans *Memo to Designers* (MTD) 20-4 describes policies and procedures for the seismic retrofit of California's bridges.[1] Caltrans, *Bridge Design Aids* (BDA) 14-5 and *Bridge Standard XS Detail Sheets* Section 7 include common retrofits that can be used by designers. The Federal Highway Administration has published a bridge retrofitting manual (Buckle, 2006), with examples of common retrofits. This manual is a useful reference, however the specifications and details are not approved by Caltrans.

While MTD 20-4 is intended to provide guidelines for retrofitting existing structures, it is not possible to anticipate every situation that may be encountered. It is the designer's responsibility to accurately assess the performance of the existing structure, to show a collapse mechanism if it exists, and to develop retrofit strategies that ensure the structure meets the no collapse performance standard.

Expected Performance

The primary performance standard for retrofitting bridges is to prevent the structure from reaching the collapse limit state[2] for the Design Earthquake[3] . The goal of this "No Collapse" performance standard is to protect human life and there are no serviceability expectations for retrofitted bridges.

An acceptable determination of collapse is captured through an analysis of the bridge model subject to the Design Earthquake. However, determining collapse is different than simply determining that demand exceeds capacity. First of all, capacity is more conservatively

1. This memo is intended to apply to Ordinary Standard state and local bridges. In cases where this memo does not apply, the designer is referred to MTD 20-1 and 20-11.
2. The collapse limit state is defined as the condition where any additional deformation will potentially render a bridge incapable of resisting the loads generated by its self-weight. The "No Collapse" performance standard prevents failure of this type while allowing for the possible localized failure of some individual components (typically redundant or secondary components that are not necessary for structural stability).
3. In this memo the 'Design Earthquake' is substituted for the term 'Design Seismic Hazards' used by Caltrans Geotechnical Services to refer to the collection of seismic hazards at the bridge site used for the design of bridges.

defined for new than for existing bridges. Secondly, exceeding a single member capacity may not lead to system collapse. Collapse means that the demand is so large that the bridge will become unseated, that it will break a significant load bearing element, or it will cause some other collapse mechanism that will positively bring down the bridge. An equally valid solution is to demonstrate through analysis that a collapse will not occur. This would be the preferred alternative since construction (with its costs and risks) would not be required.

There are several reasons why seismic performance requirements are higher for new bridges. Designers can provide additional seismic resiliency on new bridges whereas they are often constrained by geometry or structural configuration with an existing bridge. Moreover, the seismic demands for existing bridges (with a shorter remaining life) can be less conservative than for new bridges and still provide an acceptable level of risk. MTD 20-4 only requires the minimum seismic retrofit to prevent collapse while Caltrans *Seismic Design Criteria* (SDC) for new bridges has additional requirements that provide a larger safety factor against collapse. Therefore existing bridges are allowed to have behavior that is discouraged for new bridges. For instance, rocking of existing bridge foundations is acceptable for ground shaking hazards and more drift is allowed on piles and shafts of existing bridges in laterally spreading soil.

Currently the Structure Replacement and Improvement Needs (STRAIN) Report identifies bridges with many needs including seismic retrofit and each district chooses projects from the report. When post event structural serviceability is a design requirement, this memo will not apply, and a more conservative approach based on project specific performance standards must be followed. MTD 20-11 (Caltrans, 1999) must be used to establish this criterion.

The Design Earthquake

Ground shaking is the one seismic hazard that can occur at every bridge site. The designer must carefully read Caltrans MTD 20-17 "*Understanding Directionality Concepts in Seismic Analysis*" (Caltrans, 2014) to understand how ground shaking demands are obtained for different methods of analysis. All of these methods originate from the Design Spectrum described in Caltrans SDC Section 2.1 and in Appendix B (Caltrans, 2013). Amplification of the ground shaking hazard for near fault and basin effects is accomplished by increasing the long period response. Caltrans Design Spectrum is also used to produce time histories of ground motion that include these effects.

In rare cases a bridges may need to be analyzed for two or even three seismic hazards. Designers are notified of all seismic hazards at the bridge site in the *Preliminary Foundation Report*. Caltrans MTD 20-8 and MTD 20-10 provide methods for determining the surface faulting hazard and the resulting demands on bridges. MTD 20-13 provides a method for determining the hydrodynamic forces due to the tsunami hazard which are used to determine the demands on bridges. MTD 20-14 discusses how to proceed when liquefiable soil may be an issue. MTD 20-15 provides a method for analyzing bridges for lateral spreading. However, these memos were written for the design of new bridges. It is overly conservative to design existing bridges for several simultaneously occurring seismic hazards with a 5% in 50 year probability of occurrence. The chances that the largest ground shaking, liquefaction, and lateral spreading hazards in 975 years will occur simultaneously is extremely unlikely and even more unlikely for a bridge with a small remaining service life. For existing bridges the designer must analyze for each hazard separately, determine if any of them can cause the bridge to collapse, and achieve a retrofit design (if needed) that will accommodate the effects of each hazard on the bridge[4].

Caltrans uses the larger of the deterministic and probabilistic (for a 1000 year return period) derived seismic hazards for both new and existing bridges. However, bridges that will remain in service for less than five years only need to be analyzed for the hazards that are likely to occur during a 100 year return period (10% in 10 years). For instance, if there is a delay in the replacement of a vulnerable existing bridge, an interim retrofit for the smaller return period of 100 years may be performed to reduce the risk to the public at a reasonable cost. MTD 20-2 "*Site Seismicity for Temporary Bridges and Stage Construction*" provides the rules for the seismic design of new and existing temporary bridges. Of course, an acceptable alternative for 'interim' retrofits is to do nothing if a collapse mechanism does not form for this smaller hazard.

Our understanding of seismic hazards and bridge earthquake response has increased since MTD 20-4 was first published in 1990. Larger ground motions as well as previously unconsidered seismic hazards means that retrofits done in the 1990s may need to be revisited. However, because Caltrans has to prioritize the many life safety concerns on state highways and locally owned bridges, undue conservatism is not appropriate for the seismic retrofit of ordinary bridges.

Background Work And Review

As a preliminary step in determining if a structure requires a retrofit, the designer must verify the existing conditions. This would include a review of all the as-built plans including any previous work done on the structure, checking *Structure Maintenance and Investigations* (SM&I) records, obtaining site seismicity and geological conditions, and visiting the site (if possible) to compare as-built and current site (including traffic and utility) constraints. When evaluating a state highway bridge, the designer must also review the STRAIN to

4. Long term scour is combined with seismic loads for existing bridges. See the appropriate memo for rules on combining hazards.

assess the need to combine retrofit work with other work such as deck rehabilitation, barrier replacement, etc. wherever possible. This must be done as early in the project development phase as possible in order to properly scope the project. The designer should contact the SM&I bridge program coordinator to discuss the STRAIN recommendations.

Initial Assessment of Structure

Careful consideration must be given to assess the structural response of the entire system for the Design Earthquake (as provided in the *Geotechnical Services Foundation Report*) in order to develop an effective seismic retrofit strategy. Prescribed procedures may not apply to every situation. For example, yielding of a single element may not be sufficient to create a collapse mechanism. The redistribution of additional load in a structural system after incremental yielding will be different for each structure. Table 1 provides the maximum ductility demand values that are allowed for poorly reinforced substructures that were built before the 1971 San Fernando earthquake. These values are based on tests of older columns and piles (Priestley, 1991) and of pier walls (Haroun, 1993) that were done during the legislative-mandated retrofit program in the 1990s and can be used for an initial assessment of older bridges. The table represents the tested performance of columns with continuous reinforcement, and also notes the maximum ductility capacity observed when a member contains poorly confined lap splices in main reinforcement[5]. When analyzing older columns, after the substructure elements have reached their maximum ductility, a pinned connection can be substituted for the fixed connection and the push-over analysis can be continued.

Table 1. Maximum allowable displacement ductility capacity, $\mu_{c,max}$ for poorly confined members

	Poorly Confined/No Retrofit		Steel/Fiber Casing Retrofit	
Substructure Member Type	**lapped main bars**	**cont. main bars**	**lapped main bars**	**cont. main bars**
Round Columns	1.5	3.0	5.0	8.0
Rectangular Columns	1.0	3.0	6.0	8.0
Pile/Shaft Extensions	1.5	3.0	5.0	8.0
Pier Walls in weak direction	1.0	4.0	5.0	8.0

5. If the starter bars have an effective lap beyond the plastic hinge, approximately equal to the wall thickness, then it will act as a continuous main bar.

The designer must evaluate the global bridge model for collapse rather than the failure of individual elements. This 'diagnostic model' is created to analyze the structure in the as-built condition and identify the different collapse scenarios that can occur. Then an incremental approach is used to determine the level of retrofit necessary to develop a retrofit strategy that achieves the most economical retrofit design while meeting the "No Collapse" performance standard. For modeling and analysis guidelines, the designer can refer to the Caltrans SDC:

- Section 2.1 for determining the maximum demands due to the Design Spectrum
- Section 2.1.5 for damping factors
- Section 5.2 – 5.5 for analytical methods.
- Section 5.3 for global analysis modeling including bridges with irregular geometry
- Section 5.6.1 for effective section properties
- Section 6.1 for site seismicity and analyzing for different seismic hazards
- Section 7.8 for abutment response (existing bridges can take greater advantage of abutment stiffness to protect weak columns and piers)

Note that acceptable limit states for assessment may be different from those in the SDC. For instance new columns have a target displacement ductility demand of 4 to 5, well short of their actual capacity, while retrofitted columns are allowed a target ductility demand of up to 8 (based on Priestley, 1991). Similarly, the shear strength of new columns is based on nominal properties but it is based on expected properties for existing columns. The shear model used in SDC is relatively conservative compared to results of experimental testing of existing and new columns. In certain situations the UC San Diego shear model can be utilized to compute higher capacities on existing columns (Priestley, 1991). Similarly, pier wall shear capacity in the weak direction may be overly conservative using the SDC column shear degradation model at moderate levels of ductility (Haroun, 1994). The use of alternative shear models must be approved at the strategy meeting.

The designer must estimate various modeling parameters, such as abutment stiffness, cracked section properties, etc., and run the diagnostic model assuming structural integrity is maintained. The resulting displacement demands are then compared with member capacities. Some of the Demand/Capacity ratios the designer must check include (but are not limited to) ultimate displacement, shear, pile capacities, and seat length. For some pile types such as 'Raymond' step tapered or timber piles, the capacities are usually assumed to be zero. The initial modeling assumptions, such as abutment stiffness, etc., used in the diagnostic model are then verified. If necessary, the model is rerun with revised assumptions, and then checked again. This process is repeated until the results converge with the assumed modeling parameters.

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Material Properties For Existing Bridges

Stresses and strains for structural steel, concrete, and steel reinforcement have changed over time. The Concrete Reinforcing Steel Institute published a report (CRSI, 2001) with rebar specifications from 1900 to 2001. The expected compressive strength of portland cement concrete in good condition can be taken as 5000 psi. The properties of bar reinforcement that are not in the table (or in the references) must be established on a project specific basis.

Table 2. Properties for Moment Curvature Analysis

Property	Symbol	A706	A615 Gr 60	A615 or older Gr 40
Specified Minimum Yield Stress	F_y min	60 ksi	60 ksi	40 ksi
Specified Maximum Yield Stress	F_y max	78 ksi	NA	NA
Expected Yield Stress	F_{ye}	68 ksi	68 ksi	48 ksi
Specified Minimum Tensile Stress	F_u	80 ksi	90 ksi	60 ksi
Expected Tensile Stress	F_{ue}	95 ksi	95 ksi	68 ksi
Nominal Yield Strain	ε_y	0.0021	0.0021	0.00138
Expected Yield Strain	ε_{ye}	0.0023	0.0023	0.00166
Ultimate Tensile Strain #4 to #10 #11 to #18	ε_{su}	 0.120 0.090	 0.090 0.060	 0.120 0.090
Reduced Ultimate Tensile Strain #4 to #10 #11 to #18	ε_{su}^R	 0.090 0.060	 0.060 0.040	 0.090 0.060
Onset of Strain Hardening #8 and smaller #9 #10 and #11 #14 #18	ε_{sh}	 0.0150 0.0125 0.0115 0.0075 0.0050	 0.0150 0.0125 0.0115 0.0075 0.0050	$14\ \varepsilon_y =$ 0.0193

Development Of Retrofit Strategy

If the diagnostic model indicates that a collapse mechanism exists then the designer must estimate the minimum amount of retrofit required[6] to meet the "No Collapse" performance standard. The diagnostic model with the proposed retrofit is then run. If a collapse mechanism for the structural system still exists, additional retrofit measures are required. If the retrofit model indicates there is no collapse mechanism and that the associated member demands are significantly less than their capacities, the designer must consider reducing the amount of retrofit and re-running the model. This procedure is repeated until an optimal, or "preferred" retrofit strategy is obtained.

The designer must consider costs when developing a retrofit bridge model. For instance, the abutment and superstructure can sometimes be modified to reduce demands to the columns at considerable savings over a column and foundation retrofit. To obtain the cost codes for contract items the designer can go to: http://www.dot.ca.gov/des/oe/construction-contract-standards.html. The codes are input at http://sv08data.dot.ca.gov/contractcost/, which provides costs for retrofit and other construction items. This can be useful for estimating costs (although final costs will be supplied by Caltrans Structure Office Engineer).

The designer must also consider the hierarchy of different retrofit strategies. Large seats have the most direct effect on preventing collapse. Increasing column ductility with casings is a common strategy. Increasing column shear strength is also very effective. Strengthening foundations may have little effect unless liquefaction with lateral spreading is a threat. Even when poor soil is a problem, it is usually more effective (and less expensive) to turn the superstructure into a strut that uses the abutments to restrain movement. Single column bents are more vulnerable to collapse and may benefit from foundation work (see Section 8). On a shorter bridge, putting timber blocking between the abutment backwall and the superstructure (if there is a gallery) is sometimes sufficient to reduce displacements and protect vulnerable elements.

Seismic design is a balance between strength and ductility. Increasing the ductility of existing bridges is usually the most straightforward retrofit. When strength is added to existing bridges, other members in the load path must be rechecked to ensure the reliability of the retrofit scheme. It is better not to add strength as it usually just makes the seismic demands larger.

The designer must try to use standard retrofit details as much as possible. *Bridge Standard Detail Sheets* (XS Sheets) Section 7 (Caltrans, 2014) provides the most common retrofit details that have been tested and known to be effective. Other seismic retrofit details can be

6. The minimum amount of retrofit is typically the retrofit alternative that satisfies the project report and environmental document and can be constructed for the lowest cost. Future maintenance costs must also be considered.

found in Caltrans BDA 14-5 *Example Seismic Retrofit Details* (Caltrans, 2008) and in the different parts of MTD Section 20. Any deviation from these standard retrofits requires a design exception as described in Caltrans MTD 20-11.

For any alternative retrofit strategy, the designer must clearly demonstrate that the strategy is the minimum that meets the "No Collapse" performance standard. The designer must also develop sufficient conceptual details for the strategy in order to show that the strategy is feasible. Each strategy must address geotechnical, hydraulic, aesthetic, highway, environmental, constructability, utility, and other relevant issues. During the strategy development phase, the Lead Office must consult with the Office of Earthquake Engineering (OEE) for complex strategies.

Following the development of the retrofit strategy, the respective Lead Office must schedule a Retrofit Strategy Meeting. Other relevant Functional Offices must be present at the meeting.

Lead Offices

- Offices of Structure Design
- Office of Special Funded Projects/Structures Local Assistance (SFP/SLA)

Functional Offices

- Earthquake Engineering
- Geotechnical Design Offices within Geotechnical Services
- Structure Design (for in-house designs and SFP/SLA projects)
- Structure Maintenance and Investigations
- Structure Office Engineer (as needed)
- Structure Construction
- Bridge Architecture and Aesthetics (as needed)
- Structure Hydraulics (as needed)

The Lead Office must provide a Strategy Report to the meeting attendees at least one week prior to the Strategy Meeting for simple projects and at least two weeks prior to the Strategy Meeting for complicated bridges with multiple frames and/or with multiple hazards. As a minimum, the report must include:

- A General Plan indicating the retrofit work for each alternative
- All pertinent as-built plans for the existing bridge

- A summary of demand/capacity ratios (μ_D/μ_C), structural vulnerabilities, potential collapse mechanisms, and modeling assumptions for the diagnostic model and each retrofit alternative. If special retrofit requirements are a result of the findings of the Project Report or Environmental Document, they should be shown on the Strategy Report
- Preliminary Foundation Report for bridges including geotechnical seismic recommendations with ground shaking plus liquefaction or for other multiple hazards
- Conceptual details that show the retrofit alternatives are feasible
- A cost estimate for each alternative

In addition, the designer must be prepared to discuss the analysis methods used to evaluate the existing structure as well as all retrofit alternatives.

Caltrans OEE provides a key role before the strategy meeting and must approve the earthquake retrofit strategy. The use of pre-strategy consultations with the Office of Earthquake Engineering is essential for projects with multiple hazards, as seismic criteria and engineering practice are still evolving.

While it is the responsibility of the designer to accurately assess the seismic performance of the existing structure, and to develop the retrofit strategy, a successful Strategy Meeting achieves consensus among all attendees and confirms that the retrofitted structure meets the required performance standard[7]. Unusual retrofit strategies or performance standards require a design exception.

The Lead Office Chief will give final approval of the retrofit strategy and grant exceptions to retrofit requirements when necessary. When disagreements occur between OEE and the Lead Office, they will be resolved by the OEE Chief. After approval the Seismic Retrofit Assessment Form (MTD 20-4 Attachment A)[8] must be completed by the designer and included in the Final Strategy Report. The Lead Office must also submit a copy to the OEE Chief, for incorporation into the permanent bridge records.

Structures may require seismic evaluation and retrofit when modified (widening, rehabilitation, etc.) as discussed in MTD 20-12 (Caltrans, 2013) and MTD 9-3 (Caltrans, 2010). In these cases, the Strategy Meeting may be combined with the Type Selection Meeting (See MTD 1-29). The designer is required to demonstrate that the new or widened portion

7. The minimum required performance standard is "No Collapse" unless directed otherwise by the Lead Office Chief with concurrence from the Chief of OEE.

8. The purpose of the Seismic Retrofit Assessment form is to keep a record of previous seismic evaluations for future reference. Sometimes an APS or Strategy Meeting concludes that no retrofit is required. This conclusion should be documented on a Seismic Retrofit Assessment form.

of the structure meets the SDC requirements while the combined structure meets the "No Collapse" performance standard. (See MTD 9-3 for additional guidelines and information). For complex strategies, the Lead Office may consider meeting with OEE prior to the Type Selection/Strategy Meeting in order to gain consensus on the recommended seismic retrofit strategies. In cases where there is an adjacent structure with potential seismic vulnerabilities similar to the bridge being modified (for example left and right bridges), it is important to ensure the adjacent structure is either retrofitted or programmed for future retrofit assessment. This must be accomplished by submitting a Seismic Retrofit Assessment Form (Attachment A) to the Office of Earthquake Engineering.

Retrofit Design Considerations

In order to meet the goal of the "No Collapse" performance standard, the designer must consider the most common vulnerabilities that may lead to collapse mechanisms and are described below.

Single Column Bents

Prior to 1971, single column bents were constructed with dowels protruding from the top of the footing (called 'starter bars'). The column cage was then connected to the dowels by lap splices. These lap splices usually had insufficient length and confinement to maintain enough fixity to develop the plastic capacity of the column.

Slippage of the lap splice at the bottom of the column may compromise the fixity and affect the overall stability of the structure. When retrofitting a column to maintain flexural capacity, the column's overstrength moment ($M_o^{col} = 1.2 \times M_p^{col}$) will be transferred to the footing and consideration must be given to strengthening the footing in order to resist the resulting moment. However, rotation of a footing is not necessarily a collapse mechanism. Axial displacement of a pile through the competent soil will dissipate energy during the earthquake. Therefore, it may not be necessary to ensure fixity at every column/footing connection. Slipping of the lap splices may be permitted provided the vertical load carrying capacity of the column is not compromised. Retrofit design allows some lap splices to release provided there is sufficient strength in the frame to prevent collapse.

When instability of a single column bent could result in a bridge collapse, the designer should consider using a Class F column casing to protect the column and the connection to the foundation.

Multi-Column Bents

In multi-column bents, the columns are typically pinned at the base. In these cases, the designer must check that the footing can resist moments and forces based on the shear capacity of the pin. If the column/footing connection is fixed, the designer must consider the consequences if the fixed condition is lost during the earthquake. If the fixed condition is necessary for structural stability, the designer must take appropriate measures (such as Class F casings and footing retrofits) to prevent collapse.

Foundations

Damage to abutments and footing piles is acceptable provided this does not lead to a collapse of the structure. In a pile type foundation, if a fixed column condition is not required, foundation damage that could result in a substantial loss of fixity of the column may be acceptable. However, there must be a sufficient number of piles in the resulting effective foundation region to maintain the vertical capacity of the structure. The effective foundation region is assumed to be an area bounded by the column and one half of the footing depth on either side of the column. Similarly for spread footings, the effective area under the column must be sufficient to maintain vertical load carrying capacity.

Pile Extensions

In the case of relatively short slab bridges (typically 4 spans or less) on pile extensions, the diaphragm-type abutments typically provide most of the lateral resistance. The pile extensions may exceed their ultimate displacement capacities provided they maintain their vertical load carrying capacity.

Transverse Reinforcement

Shear failures are brittle, and therefore the shear demand/capacity ratio must remain below 1.0. For structures with minimal and poorly detailed (#4 ties at 12 inches) transverse lapped reinforcement, the designer must assume that only the concrete provides shear resistance. In this case the bridge should be modeled as unconfined concrete.

For bridges that have improved transverse column reinforcement details, it may be assumed that both concrete and steel provide shear resistance. The designer may refer to "*Seismic Assessment and Retrofit of Bridges*" (Priestley, 1991) for help evaluating the shear capacity of older columns. The shear capacity of existing columns may be determined with the methods described in SDC Section 3.6 using expected properties instead of the nominal properties that are required for new bridges. Refer to Section 5 of this memo for more information on evaluating the shear capacity of columns.

Abutments

On shorter bridges (typically 4 spans or less), the abutments may provide significant resistance to longitudinal movement. Using methods discussed in SDC Section 7.8.1, the designer may apply longitudinal abutment springs to structural models. Typically on seat type abutments, the shear keys and backwalls will fail at the Design Earthquake. It may also be worthwhile to increase the damping of shorter bridges by following the procedure in Caltrans SDC Section 2.1.5.

Bent Caps

In bridges with multi-column bents, hinging could occur in the bent cap. While this is not desirable, it may not necessarily lead to a collapse of the structure. For box girder bridges, the bent cap remains effective as long as its displacement ductility capacity (measured in rotation, curvature, or displacement) is greater than the displacement demand from the Design Earthquake. For other types of bridges, as long as the transverse displacement of the bent is less than two times the displacement that causes the bent cap to yield, and there is sufficient shear reinforcement (V_S) in the cap to resist the shear due to the plastic moment of the bent cap and dead load (V_P+V_{DL}), they remain effective in preventing collapse. When there are tightly spaced stirrups in the cap (to prevent excessive cracking), V_C may also be considered when determining the shear capacity of the bent cap. The effective width of the bent cap for considering its flexural and shear capacity is the cap width plus 12 times the top or bottom slab thickness (as illustrated in SDC Section 7.3.1.1).

At displacement ductility ratios above 2.0, the designer must demonstrate that even if the bent cap hinge degrades to a natural hinge (pin), adjacent elements like columns and abutments will continue to support the superstructure and prevent collapse.

P-Δ Effects

The P-Δ check is intended to ensure adequate results when using the equal displacement principle (between linear and nonlinear systems). The SDC treats P-Δ at the local level and the limit of 0.2 in SDC Section 4.2 was adopted to be on the conservative side for new bridges. MTD 20-4 treats P-Δ as a system parameter that is often addressed by ensuring continuity of the superstructure. Therefore the P-Δ limit for existing bridges can vary from 0.2 to 0.3. For movements in the longitudinal direction, the soil mass behind the abutment may be sufficient to prevent additional movement caused by P-Δ (the soil mass acts as a restoring force).

For cases where the P-Δ effect is a concern, the designer may evaluate the marginal increase in the displacement demand due to second order effects using time history methods of analyses that include geometric nonlinearity. The designer should consult with the Office of Earthquake Engineering (or their liaison engineer) for more information.

Pier Walls

Pier walls must be analyzed as columns in the weak direction, and as a shear element in the strong direction. For bending in the weak direction (given continuous main reinforcement in plastic hinges) the calculated displacement ductility demand is capped at 4.0. For lapped starter bars, the ductility of all structural members must be limited to $1.5\mu_C$ (but see footnote 5). More information on the behavior of pier walls is available from a series a tests that were done at UC Irvine (Haroun, 1993). For existing bridges, the shear demand of pier walls in the strong direction can be calculated as the peak of the Design Spectra while the capacity can be determined from the less conservative UCSD shear equation. Damage to piles is acceptable in the strong direction provided the stability of the pier wall is not compromised in the weak direction.

Unbalanced Bents and Frames

Previous earthquakes have demonstrated the vulnerability of unbalanced columns in a bent and unbalanced bents in a frame. In these systems there is unequal sharing of the seismic demand. The stiffer elements will carry more of the inertial load and be unable to displace as much as the other members and they can break. It is difficult to modify an unbalanced system. The best solution is to provide isolation casings in the soil around stiffer elements to give them a greater displacement capacity. Column casings and isolation bearings have also been used to increase the displacement capacity of stiffer elements.

Unbalanced frames have out-of-phase motion that can result in the frames moving away from each other and dropping a span at the hinge. The solution for these situations is provide pipe seat extenders or other devices to prevent unseating.

Expansion Joints

On longer bridges with continuous superstructures, expansion joints are used to allow for thermal expansion. The designer must ensure that the hinge has sufficient seat length to accommodate differential movements between adjacent frames for the Design Earthquake. Caltrans SDC Section 7.2.5.4 provides guidance for determining adequate seat length, however, the 24-inch minimum seat length required by Caltrans SDC does not apply to retrofits. When in-span hinge seats are less than twelve inches, the seat must be retrofitted with pipe seat extenders. Use of cable restrainers instead of pipe seat extenders to prevent unseating requires a design exception.

When it is necessary to core through hinge diaphragms or bent caps in order to place pipe seat extenders or hinge restrainers, the designer is cautioned to avoid structurally critical elements such as pre-stressing steel or shear reinforcement.

On some existing cable restrainer systems, the cables were grouted into the openings, essentially reducing the effective length of the cables to a few inches. The designer must refer to the as-built plans to determine if the existing cables were grouted. The designer must consider that in a seismic event, grouted cable restrainers could fail at small movements thus leaving the hinge unrestrained, and therefore take appropriate measures such as pipe seat extenders.

Simple Spans

On bridges with simple span superstructures, the designer must ensure that the spans remain seated on the abutments and bents for the Design Earthquake. Often, it is not practical to place pipe seat extenders in these situations. Catcher blocks and shear keys are an effective means of retrofit for these situations and typical details may be found in BDA 14-5. Use of cable restrainers to prevent unseating of bridges with simple spans requires an approved design exception.

Rocker Bearings

On some structures, tall rocker bearings were used at the abutments and at the bent caps on simple span configurations. For the Design Earthquake these bearings could fail and result in a drop of the superstructure. While a drop of six inches or less is not typically catastrophic, a potential drop greater than this must be investigated in order to ensure that the structure is not vulnerable. When the height of the rocker bearing is greater than 2/3 of the seat length, the superstructure could become unseated and the designer must consider appropriate retrofit measures.

Flared Columns on Multi-Column Bents

Flares on columns are an architectural feature on some bridges in California. It is desirable for plastic hinges to form at the top and bottom of the column as this minimizes its plastic shear and rotational demands. However, flares on multi-column bents typically cause a hinge to form at the base of the flare rather than at the top of the column thus increasing the column's plastic shear demand in the prismatic portion and potentially exceeding its rotational capacity.

Liquefaction

Liquefaction (the loss of strength of saturated cohesionless soils during earthquakes) can damage a bridge due to reduced lateral resistance, excessive foundation settlement, or due to increased axial loading (as a result of downdrag forces). The designer must determine if liquefaction will result in collapse. If a potential collapse mechanism exists, either footing modification or soil improvement is usually required to meet the "No Collapse" performance standard. In these situations, the designer is referred to MTD 20-14 and 20-15 for guidance.

Lateral Spreading

A bridge can be damaged by lateral soil movement caused by a combination of sloping ground, horizontal shaking, and reduced soil strength. If foundations are not sufficiently stiff or strong enough to resist these lateral displacement demands, damage may occur in the form of superstructure unseating and/or excess deformation of columns and foundation elements. MTD 20-15 addresses lateral spreading for new bridges and requires that the lateral spreading demand should be combined with the demand due to ground shaking. Following the policy previously stated in this memo, lateral spreading and the ground shaking are considered separately for existing bridges and the designer must ensure that the bridge will not collapse for either of these demands.

Scour

Scour is the transportation of the soil supporting bridge foundations in streams and rivers. Although most hazards are considered separately for retrofit design, scour must be considered in combination with seismic hazards. Caltrans SDC Section 2.2.5 provides the rules for considering scour in combination with different seismic hazards on new bridges. For existing bridges the seismic evaluation must be based on long term scour plus each seismic hazard (considered separately) where long term scour considers the remaining life of the bridge and the hydraulics report.

Joint Shear

Since the early 1990's, greater emphasis has been placed on joint shear considerations in the seismic design of bridges. Previously, joints were modeled as either fixed or pinned if demands exceeded the elastic joint shear capacity. As a joint is cycled at high ductilities during a seismic event, it may lose some of its ability to carry moment and degrade to a rotational spring or pin. Degradation models for modeling column/beam joints as a spring are available. A procedure and example for determining the effects of joint shear may be found in the BDA 14-4.

While joint shear is not typically a collapse mechanism and retrofit is not usually required, on long viaducts a large number of adjacent joints that form pins could potentially lead to instability of the structure. In these situations, with the concurrence of the Lead Office Chief, the designer must demonstrate that a potential collapse mechanism exists and retrofit the minimum number of joints to ensure structural stability.

The procedure for determining joint shear on pre-1994 structures was developed from research (Mazzoni, 2004). The procedure may require modification as the knowledge base increases. The proof test for the joint shear retrofit strategy on existing bridges is still pending. Therefore, the Lead Office must obtain approval for the design and details for joint retrofit from OEE.

Common Retrofit Measures For Existing Bridges

Steel Column Casing

The most common column retrofit is to encase the column with a steel casing to increase the confinement and to improve the flexural ductility and shear capacity of the column. There are two classes of steel column casing retrofit currently in use, Class F and Class P/F. These types of casings must be circular for square and round columns, and elliptical for rectangular columns (refer to BDA 14-2 for casing and radius requirements). However, when retrofitting for shear only, it is not necessary to maintain a circular or elliptical shape. Flat plates may be used when required due to limited horizontal clearance.

In the Class F retrofit, no gap is provided in the space between the column and the steel casing resulting in full-length confinement of the column. This limits the dilation of the concrete and prevents lap splices from slipping thus ensuring the fixed condition of the column/footing connection remains intact. The supporting footing must be stronger than 1.2 M_p of the column if the bridge system requires successful plastic hinging at this location.

In the Class P/F retrofit, a gap between the column and steel casing is provided around the plastic hinge region near the bottom of the column. This allows the concrete to dilate and the lap splices to slip and ensures that a pin will form at the bottom of the column. The Class P/F column casing just allows the column's nominal moment capacity (M_n) to be transferred to the footing, often eliminating the need for a footing retrofit. However, the column shear capacity in the lap splice region is limited to the capacity of the steel casing. Details for column casings (Both Class F and Class P/F) can be found in BDA 14-2 and the *Bridge Standard Detail Sheets* XS7-010.

Footings

When Class F column shells are used in single column bents, it is assumed that the footing (including pile caps) resists the column's overstrength moment or the controlling foundation moment capacity. For structures designed prior to 1971, the following vulnerabilities may exist in the footings:

- No top mat of reinforcing steel.
- Inadequate tension ties connecting the pile and the footing.
- Inadequate pile capacity for the column's plastic moment[9].
- Insufficient shear strength in the piles to resist the column's plastic shear.

Composite Column Casings

Occasionally, space or clearance considerations do not allow steel column casings to be used for retrofit. In some of these cases, Fiber Reinforced Polymer (FRP) composite casings may be used instead. The primary column retrofit for flexural and shear issues is steel casings. However, FRP has been proven to be effective under certain conditions. Caltrans has limited test data for the shear capacity of FRP wrapped columns, but these results show it to be an effective retrofit strategy (Pulido, 2002). See BDA 14-3 for procedures and specifications when using this alternative.

Infill Walls

In multi-column bents, the infill wall is an inexpensive and effective retrofit for addressing transverse vulnerabilities both in the columns and in the bent cap. Research has shown that infill walls perform the same whether the concrete is poured up to the soffit or a six inch gap is left between the top of the wall and the soffit (Haroun, 2001). Doweling into the soffit of the bent cap does not provide any additional capacity and thus is not recommended. Typical details for the in-fill wall may be found in BDA 14-5. In the longitudinal direction infill walls act as a catcher to prevent collapse. Because infill walls are shear-critical elements their use is discouraged when a flexural system with larger displacement capacity is feasible.

9. Typical details for a footing retrofit may be found in BDA 14-5.

Abutment Strengthening

On short bridges, mobilizing the soil behind the abutments may be sufficient to reduce displacement demands below the structure's displacement capacity. This may be accomplished by strengthening the abutment diaphragm, or in the case of seat type abutments, connecting the superstructure end diaphragm to the seat. When a large gap exists between the end diaphragm of the superstructure and the abutment backwall, the soil behind the backwall can be mobilized by eliminating the gap with concrete or timber blocking. The designer is cautioned to leave a small gap that still allows for service load and temperature movements of the structure.

Catcher Blocks

Abutment bearings frequently fail during seismic events. However, such localized failure is not generally catastrophic unless the drop exceeds six inches. Seat catchers are an effective and inexpensive method of limiting superstructure drop and providing additional seat length as well. Catchers may also be used on bent caps for simply supported spans.

Cable Restrainers

Use of restrainers are at the discretion of the designer. They are effective in limiting the displacements for small to moderate seismic events. Restrainers are not considered to be effective at preventing unseating, and so their use to reduce displacement demand in a seismic retrofit requires an approved design exception. If existing restrainers are retained, anchorages must be checked for proper gapping and anchor nuts secured with a thread locking system.

Pipe Seat Extenders

Pipe seat extenders are effective in preventing collapse of a hinge span; however, the bridge may not be serviceable when the hinge opens sufficiently to engage the extenders. Therefore when pipe seat extenders are used for retrofit, consideration must be given to placing cable restrainers through the pipe and anchoring them to the adjacent bent cap. Restrainers may limit the differential movement in the hinge during moderate events and reduce damage to the bearing pads and expansion joints.

The typical detail, found in BDA 14-5 and *Bridge Standard Detail Sheets* (XS Sheets) Section 7, for a pipe seat extender makes use of Pipe XX-Strong (ASTM A-53 Grade B). The allowable load that can be carried by pipe seat extenders depends on the anticipated displacement demand, Δ_D and the seat length (See Figure 1).

For an 8-inch seat (or less), each pipe can carry 300 kips at unseating, 200 kips at Δ_D = 12 inches, and 135 kips at Δ_D = 18 inches.

For a 12-inch seat each pipe can only carry 200 kips at unseating and 135 kips at Δ_D = 18 inches.

For an 18-inch seat each pipe can only carry 135 kips at unseating.

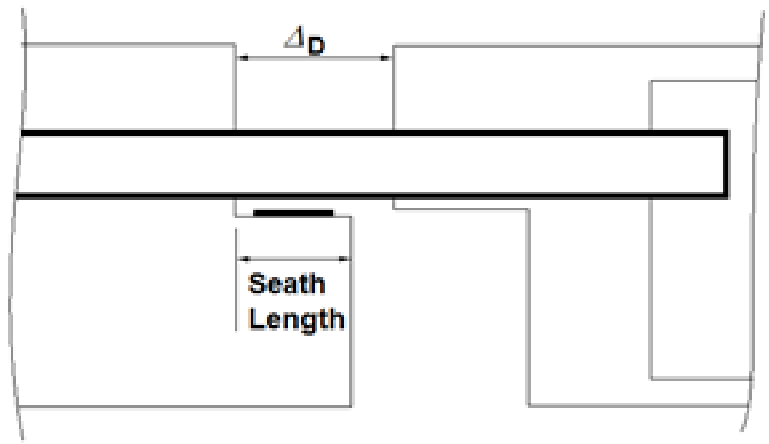

Figure 1. Unseating at pipe seat extender

Pipe seat extenders must be installed so that movement of the bridge under service conditions is not restricted (typically the extenders must be placed parallel to the girders). In addition, the designer must evaluate the capacity of the supporting hinge diaphragm. Pipe seat extenders are also effective as shear keys for existing bridges.

Foundation Retrofit

Typically, footings are strengthened by the addition of a top mat of reinforcing steel and additional piles. A foundation retrofit is usually costly and careful consideration must be given to retrofitting only the minimum number required to meet the "No Collapse" performance standard. Past foundation retrofits have included tie-downs (as an alternative to piles in tension) and micropiles (handy when working under the superstructure). Typical details for footing retrofits may be found in BDA 14-5.

Flare Isolation

Isolating a column flare is an inexpensive and effective method of eliminating the potential hinge formation at the base of the flare. Flares may be isolated by cutting the flare steel. However, the designer must ensure that the steel being cut is not necessary for structural integrity, and in any case, the main column reinforcement must not be cut or damaged. If the flare steel is main column reinforcement, other retrofit measures must be used. In addition to cutting the steel, the top four inches of concrete is removed in order to allow the top of the column to rotate freely (Sanchez, 1997). The removal of the concrete will increase the span length of the bent cap and the designer must ensure that the modified bent cap meets service load requirements.

Seismic Isolation

Occasionally, a situation is encountered where physical constraints prevent the use of more conventional measures for retrofitting the substructure of a bridge. In these cases, isolation may be used as an alternate method by reducing the seismic forces transmitted to the substructure from the superstructure and reducing the need for substructure retrofit. However, the force transfer through the isolation device may overload an existing column with poor confinement in which case a substructure retrofit will still be required. Seismic isolation may also be used to improve the mass/stiffness ratio of adjacent frames. However, when using seismic isolators, there must be sufficient clearance between the soffit of the superstructure and the top of the bent cap in order to place the isolators. In addition, the superstructure must be free to move a sufficient amount for the isolators to be effective. The designer is referred to the AASHTO *Guide Specifications for Seismic Isolation Design* for more information (AASHTO, 2014).

Other Retrofit Measures

While these retrofit measures are the most commonly used by Caltrans, there are many other methods available to the designer for retrofitting highway structures. In developing alternative retrofit measures, the designer must ensure that these measures address the vulnerabilities identified in the diagnostic model, and that the retrofitted structure meets the "No Collapse" performance standard. See BDA 14-5 for common seismic vulnerabilities and typical details for common seismic retrofits. Alternative retrofit measures require an exception from Caltrans OEE.

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Original signed by Mark Mahan

Mark Mahan, Chief

Office of Earthquake Engineering, Analysis, and Research

Structure Policy and Innovation

Division of Engineering Services