Seismic Analysis of Concrete Bridge Piers:
Practical Considerations

P. Gauvreau[1]

ABSTRACT

The nonlinear static load-deflection response of concrete bridge piers under the combined action of dead load and monotonically increasing horizontal load provides a rational basis for identifying structural deficiencies and establishing priorities for retrofit. The reliability of such an analysis depends to a large extent on the reliability of models for local load-deformation behavior (moment-curvature diagrams) and of the analytical link between local and global deformations (usually characterized in terms of plastic hinge length). This article describes how moment-curvature behavior and plastic hinge length are affected by material properties and structural details. The focus is on common features of bridge piers built in Canada in the 1950s and 1960s. The concepts discussed are illustrated using examples taken from experiences gained on the seismic upgrading of three major bridges in the Vancouver area.

INTRODUCTION

Nonlinear static "pushover" analysis is a useful application of the theory of plasticity to seismic retrofit of reinforced concrete bridge piers. Procedures for its use are well established (Priestley 1991, 137-44) and usually consist of the following steps:

1. For a given pier, establish an idealized nonlinear load-deflection diagram due to the combined action of dead load and a monotonically increasing horizontal static load Q applied at the center of seismic force
2. For each plastic hinge, calculate the global displacement of the pier at which local deformation capacity is reached
3. For all other components of the pier, calculate the global displacement of the pier at which force demand exceeds the corresponding force capacity
4. Calculate global displacement demand Δ_{demand} of the pier due to the design earthquake using dynamic analysis

The behavior of piers under the action of a maximum design earthquake can normally be considered adequate provided the global displacements associated with the events described in Steps 2 and 3 are greater than Δ_{demand}. These concepts are illustrated in the following figure.

The validity of nonlinear static analysis is directly related to the nature of seismic action on structures. Seismic loads result from the response of structures to dynamic displacements

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[1]Senior Design Engineer, Buckland & Taylor Ltd., 1591 Bowser Avenue, North Vancouver, B.C. V7P 2Y4
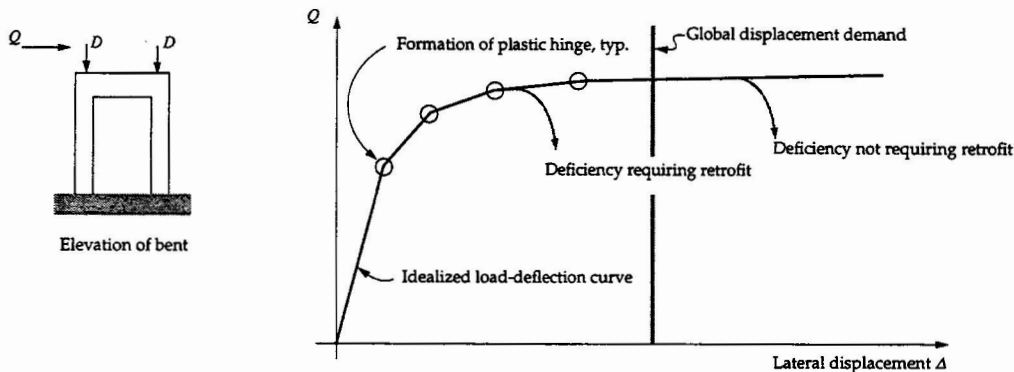
Nonlinear static analysis of concrete bridge piers

applied at their supports and are affected by changes to the dynamic characteristics of structures. Ductile behavior in a given structural component limits the force it can transmit. Regardless of the forces that would have been induced by an earthquake in an equivalent structure with elastic response (i.e., that sustained no damage), the imposed dynamic displacements of the supports cannot induce forces in an ideally ductile system that exceed the capacities of ductile components. The relation between Q and Δ for monotonically applied horizontal static load is thus an upper bound envelope for the load-deflection hysteresis curves of the pier subjected to cyclic seismic loading. (It does not account directly for degradation of response due to phenomena related to reversal of load.) Equilibrium is maintained provided:

A. The deformation capacity of ductile components is greater than the deformations imposed on them by the earthquake, and

B. The force capacity of other structural components is greater than the forces corresponding to ductile behavior elsewhere

(In this context, "structural components" are understood to mean any independent sectional force required for equilibrium. For example, shear capacity at a flexural plastic hinge must be sufficient to maintain equilibrium with the associated plastic moment.) Criteria A and B lead directly to the limits on global displacement defined previously for static pushover analysis.

Bridges are normally required to satisfy additional requirements governing their condition after maximum design earthquake, namely, that earthquake-induced damage can be repaired with an expenditure of time and money that is acceptable to the owner, and that residual capacity to carry traffic immediately after the seismic event is in accordance with the owner's functional requirements. These criteria vary from bridge to bridge and take into account the importance of a given bridge in the overall transportation infrastructure. Although no definitive statement can be made in this regard, the absence of brittle failure as demonstrated by a pushover analysis greatly increases the likelihood that damage can be repaired with acceptable means and that at least limited use can be made of bridges relatively soon after the earthquake.

The value of pushover analysis in identifying deficiencies and establishing priorities for retrofit arises from associating structural components of insufficient deformation or force capacity with a global displacement of the pier. Vulnerabilities corresponding to global dis-

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placements greater than Δ_{demand} need not be addressed. If funding constraints require seismic upgrading to be carried out over several phases, retrofits can be scheduled in order of increasing global displacement to ensure that, at each phase, a definite improvement in seismic performance has been achieved.

BASIC ANALYTICAL TOOLS

The checks prescribed by Criteria A (governing displacement capacity of plastic hinges) and B (governing force capacity of all other components) require reliable models of local load-deformation behavior at plastic hinges and of the relation between global deformation of the pier and local deformation at plastic hinges. Local load-deformation behavior can be defined using moment-curvature $(M-\phi)$ relationships. The link between local and global deformations can be approximated analytically as $\theta = L_h(\phi + \phi_y)/2$, where θ is rotation of the hinge (a global deformation), ϕ is curvature at the section of maximum moment (a local deformation), ϕ_y is curvature at onset of yielding, and L_h is length of plastic hinge.

The deformation criterion at plastic hinges (Criterion A) can be expressed as $\phi_{demand}(\Delta) \leq \phi_u$, where ultimate curvature ϕ_u is obtained directly from the moment-curvature diagram of the critical section. Curvature demand $\phi_{demand}(\Delta)$ corresponding to a given global displacement Δ is obtained through the relation $\theta(\Delta) = L_h(\Delta)(\phi_{demand}(\Delta) + \phi_y)/2$. (It is assumed that the relation between global displacement and global rotation at the hinge under consideration is known.) Solving for $\phi_{demand}(\Delta)$ and substituting into the previously given inequality yields the maximum global displacement for which local deformation capacity is greater than local deformation demand.

In a similar manner, the force criterion for sectional forces other than plastic moments (Criterion B) can be expressed as $F_{demand}(\Delta) \leq F_{capacity}(\Delta)$, where both demand and capacity have been considered as functions of global displacement of the pier. The plastic branch of moment-curvature diagrams, although often idealized as horizontal, actually increases due to plastic behavior of concrete in the flexural compression zone and due to strain hardening of reinforcing steel. Plastic moments thus generally increase with increasing plastic rotation. In the idealized free-body diagram shown in the following figure, for example, shear demand at either end
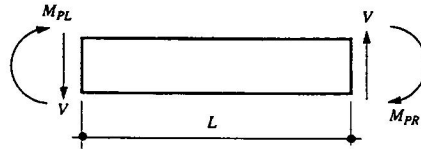
Shear forces developed by plastic hinging

is given by the equation $V_{demand}(\Delta) = (M_{PL}(\Delta) + M_{PR}(\Delta))/L$. Plastic moment M_P is obtained for a given Δ by calculating $\phi_{demand}(\Delta)$ using the expression $\theta(\Delta) = L_h(\Delta)(\phi_{demand}(\Delta) + \phi_y)/2$ derived previously, and reading the corresponding moment M_P directly from the moment-curvature diagram. (It is assumed in this article that the relation between sectional force capacity and global displacement is known. Relations between plastic deformation and sectional force capacity, in particular for shear, are still under development. Empirical models are given by Priestley (1991, 115). Gauvreau (1993) has proposed calculating shear capacity in plastic hinges

using truss models based on the moment-curvature relations of the sections under consideration.)

Simplified expressions have been proposed for both moment-curvature diagrams and length of hinge. These should only be used, however, within the limits of the assumptions underlying the simplifications. Decisions regarding how to model moment-curvature relations and length of hinge can be made with greater confidence when the effect of material properties and structural details on moment-curvature behavior and hinge length are understood.

The remainder of this article presents of brief discussion of these issues. Concepts are illustrated using examples typical of bridge design practice in Canada in the 1950s and 1960s. Bridges from this era are particularly relevant to the discussion because they make up a large proportion of the bridges requiring seismic upgrading, and because the materials and details characteristic of these structures clearly illustrate the effect of these properties on moment-curvature behavior and length of plastic hinge.

MOMENT-CURVATURE RELATIONS

The ductility of cross-sections subjected to constant axial compression N and increasing moment is related to the ratio c/d at maximum moment, where c is distance from the extreme compression fiber to the neutral axis and d is distance from the extreme compression fiber to the outer layer of reinforcement in tension. Small values of c/d imply small increments of compressive strain for a given increment of curvature, and hence high ductility. For a cross-section of given dimensions, c/d decreases with increasing axial tension and decreasing ratios of reinforcement. For Canadian bridge piers and bents built in the 1950s and 1960s, c/d is generally low. Values of less than 0.1 are not uncommon.

Cross-sections with low c/d can be said to be governed by reinforcing steel, in the sense that the shape of the moment-curvature diagram closely resembles that of the stress-strain curve of reinforcing steel. The following figure shows the substantial increase in moment that is possible after yielding when c/d is low and strain hardening of steel is pronounced. Valuable insights regarding moment-curvature behavior can therefore be gained from an understanding of the stress-strain behavior of reinforcing steel. (The stress-strain diagram shown is a bilinear
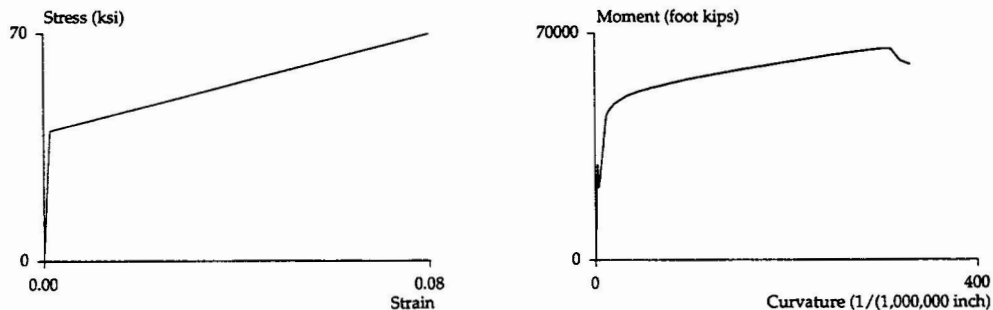

Steel stress-strain and moment-curvature diagrams (Port Mann Bridge, Bent 5N, top of column, N=-2000 kips)

approximation based on specified material properties.)

Reinforcing steel used in Canada in the 1950s and 1960s is characterized by a pronounced

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strain-hardening branch. The stress-strain diagram shown in the preceding figure shows that the commonly made assumption of linear-perfectly plastic behavior is only valid over a small portion of the overall range of strains. For "intermediate grade" reinforcing steel, a commonly used grade in Canada in the 1950s and early 1960s, specified tensile strength is 1.5 times specified yield strength. At plastic hinges where large curvatures are expected, therefore, the assumption of a constant plastic moment may lead to an underestimate of force demands in other components.

The mechanical properties of reinforcing steel are subject to considerable statistical variability. The following figure shows the distribution of yield strength obtained from tensile tests of reinforcing steel performed at the time of construction. Mean and maximum measured val-
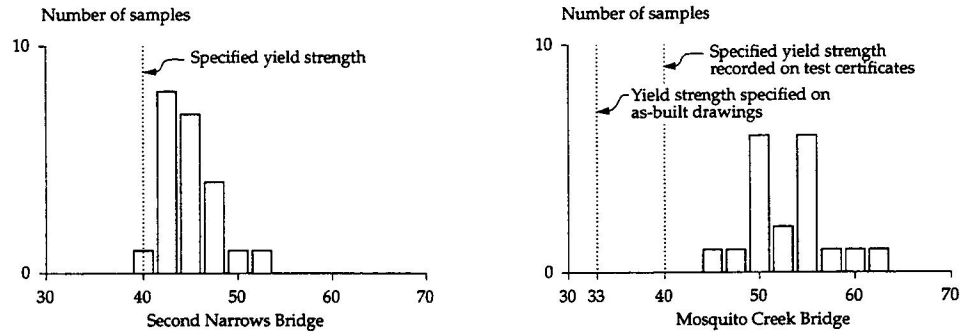
Histograms of measured yield strength (ksi)

ues for Second Narrows Bridge are 1.12 and 1.28 times specified yield strength, respectively. The distribution of yield strength shown for Second Narrows Bridge is typical of variation that can be expected within a random sample from bars of a given grade of steel. Variation in measured mechanical properties can also arise, however, from the presence of steels of more than one grade. Several grades of steel were in available in Canada in the 1950s and 1960s. The most common of these were "structural" (specified yield strength 33 ksi), "intermediate" (specified yield strength 40 ksi), and "hard" (specified yield strength 50 ksi). There is anecdotal evidence that steel of grades higher than specified was occasionally substituted during this period. This may have been the case, for example, at Mosquito Creek Bridge. Mean and maximum measured yield strengths are 1.32 and 1.56 times the specified yield strength indicated on test certificates. Further complicating the issue is a discrepancy between the grade of steel specified on the as-built drawings (structural) and the grade shown on the test certificates (intermediate). Measured mean and maximum are 1.61 and 1.89 times specified yield strength for structural grade.

For members with low c/d ratios, variability in mechanical properties will be reflected in variability of moment-curvature behavior, and hence in variability of force demands due to plastic hinging. To guard against underestimating these forces, lower and upper bound stress-strain diagrams can be developed based on statistical data, from which lower and upper bound moment curvature diagrams can be obtained. For a given state of deformation in the pier, force demand of structural components due to plastic hinging can then be calculated using the upper-bound moment-curvature diagrams.

Although the use of archival tensile test data to compute design values of yield strength

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has been proposed by Priestley (1991, 96), it is common practice to lump the combined effects of statistical variability and strain hardening into a single "overstrength" factor, often taken as 1.5 for existing structures. This value may underestimate the actual stress in reinforcing steel. At Mosquito Creek Bridge, for example, a factor of close to 1.5 would be required to account for the statistical variability of yield strength alone. Priestley and Seible (1991, 229) have proposed overstrength factors that increase with global ductility demand. Their model does not account, however, for the effect of structural details on length of plastic hinge. As shown in the following section, this effect can be significant. Problems associated with overstrength factors can be avoided by working with the upper bound moment-curvature diagrams discussed previously.

LENGTH OF PLASTIC HINGE

For the purposes of this article, length of plastic hinge can be defined as the distance along the member axis from the section of maximum moment to the section at which bending moment is equal to M_y, the moment for which the outer layer of tensile reinforcement is at yield strain. (As discussed by Gauvreau (1993), this definition actually underestimates the total length over which reinforcement is yielding, since diagonal compression field action is not accounted for. For sections with low ratios c/d, neglecting this action still enables reasonably accurate relations between curvature and rotation to be established.)

For constant maximum moment demand, length of plastic hinge depends on the relative characteristics of the diagrams of bending demand and capacity. As shown in the following figure, similarly shaped diagrams of demand and capacity correspond to significantly longer plastic hinges than for diagrams that differ in shape. In developing diagrams of bending capacity,
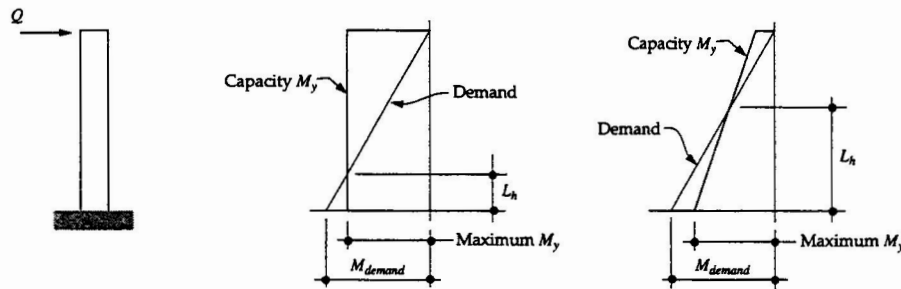

Effect of shape of bending demand and capacity diagrams on length of plastic hinge

careful consideration must be given to development of reinforcement, in particular to bars where anchorage provided may be inadequate. (Questionable reinforcing steel details are common in Canadian bridges from the 1950s and 1960s.) Diagrams of bending demand must include three-dimensional effects. Length of hinge is greater, for example, in members with hollow cross-section than in otherwise equal solid members, since diagonal shear forces are required in the compression and tension flanges to bring flexural tension and compression into equilibrium (Menn 1990, 125).

For given shapes for bending demand and capacity diagrams, L_h increases with increasing maximum moment demand. For moment-curvature diagrams with relatively flat plastic

branch, increase in moment beyond first yield and hence growth of L_h are limited. Increase in length of hinge can, however, be considerable for moment-curvature diagrams with pronounced strain-hardening behavior.

The following example, taken from Port Mann Bridge near Vancouver, illustrates in a particularly striking way how length of plastic hinge can be affected by deficiencies in detailing of reinforcing steel. The detail considered is commonly found in bridges of the 1950s and 1960s in Canada. Bottom steel at the ends of the bent cap consists of fourteen #11 bars. (Cross-sectional area of one #11 bar is 1.56 square inches.) Twelve of these are embedded only 2 feet into the column. This embedment length is roughly 60% of the development length prescribed by CSA Standard S6-88. It must therefore be assumed that these twelve bars will pull out of the column before they can be brought to yield. When this occurs, a reduced bending capacity is maintained by the remaining two bars only.
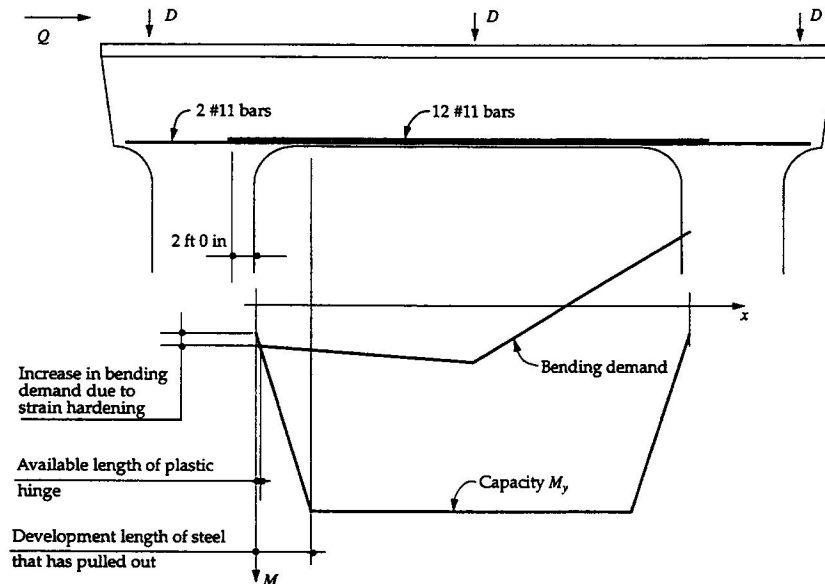
Elevation of bent cap of Bent 5N, Port Mann Bridge (other reinforcement not shown)

The diagram of bending capacity after pull-out must account for the rapid increase in resistance due to development of the twelve bars away from the section of maximum moment. (This increase has been assumed here to be linear over the specified development length, beginning at the face of the column.) The steep gradient in the capacity diagram that results severely limits growth of the plastic hinge. As shown in the preceding figure, after steel at the face of the column has strain hardened to a stress of 55 ksi (an increase in moment of roughly 1.38 times M_y), L_h is roughly 3 inches (3% of depth of section or 1% of clear span).

Although the cross-sections in the bent cap have good ductility, the short hinge length is likely to use up rotational capacity at the critical section after only a small increment of global displacement. In this case, any of the commonly used empirical expressions for length of hinge (typically taken as the depth of the member or some constant times the length from the critical

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section to the nearest point of inflection in the diagram of bending demand) would have led to a substantial overestimate of L_h, and hence an unsafe underestimate of curvature demand.

CONCLUSIONS

This article has described the significance of moment-curvature relations and length of hinge in static pushover analysis. The effect of material properties and structural details on moment-curvature behavior and hinge length has been outlined and illustrated with examples taken from three bridges in the Vancouver area.

Simplified models for moment-curvature diagrams and length of hinge can be of great help to engineers in assessing the seismic performance of existing bridges. Care must be taken, however, to ensure that the assumptions behind the simplifications are consistent with actual conditions in the structure under consideration. Although more general models based on engineering principles and on a realistic assessment of the variability of material properties may be slightly more tedious to use, they can be applied with confidence to any situation.

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APPENDIX: CONVERSION OF UNITS

U. S. Customary Unit	Metric Equivalent
1 inch	2.54 cm
1 foot	0.305 m
1 kip	4.45 kN
1 foot kip	1.36 kN m
1 ksi	6.90 MPa