20-6 SEISMIC STRENGTH OF CONCRETE BRIDGE SUPERSTRUCTURES

General Purpose and Problem Statement

Caltrans seismic design practice requires essentially elastic behavior in the superstructure (girder and bent cap member), as defined in Caltrans *Memos to Designers*[1] (MTD) 20-1. Concrete structures with integral column or pier connections can transmit seismic forces of considerable magnitude into the superstructure. The designer must account for these forces and load paths to ensure that the expected nominal moment capacity of the superstructure is not exceeded. Maintaining essentially elastic member behavior does not guarantee serviceability after a seismic event, but it is consistent with Caltrans performance criteria outlined in Caltrans *Seismic Design Criteria*[2] (SDC).

This memo will provide the designer with the necessary guidelines to comply with this fundamental practice, and applies to structure types as defined in the SDC[2]. Other structure types may require project specific criteria as specified in MTD[1] 20-1.

Philosophy

A capacity design approach[3] shall be adopted to ensure that any significant plastic action and energy dissipation is directed to the column or pier members. These plastic hinge regions of the bridge framing system must be properly designed and detailed for energy dissipation under the severe deformations expected during a large seismic event. For practicality, the column member can be detailed with the necessary amount of confinement reinforcement to sustain high curvature deformations and is accessible for damage inspection and repair.

The superstructure shall be capacity protected from the potentially large forces transmitted by the ground motions by the fusing action of the column plastic hinge. To ensure essentially elastic performance, the superstructure must have an appropriate strength reserve above the demands generated from the probable column plastic hinge. This procedure is designed to assure that unexpected hinge formation will not occur in the superstructure, thus protecting against a potential failure mechanism.

Loads due to vertical accelerations are assumed not to act simultaneously with peak lateral seismic demands (SDC[2] 7.2.2). Therefore, these load cases should be analyzed independently. The structure designed in accordance with this memo is assumed to have capacity to resist a non-peak vertical and peak lateral seismic load.

General Assumptions

- Computed load demands are subject to *Bridge Design Specifications*[12] (BDS) 3.22 Load Factor Design Group VII combinations.
- Computations of extreme seismic limit state superstructure demands are based on complete plastic hinge formation in all columns or piers within the frame.
- Effective section properties shall be used for modeling the columns or piers, while gross section properties may be used for superstructure elements. Sensitivity studies using both gross and effective superstructure section properties showed no significant difference in superstructure strength demands.
- For multi-column bents, additional column axial force due to overturning effects shall be considered when calculating both effective section properties and the idealized plastic moment capacity of columns and piers. Overturning effects may be disregarded in longitudinal frame analysis since the magnitude of cyclic axial forces is not significant in most cases.
- Superstructure dead load and secondary prestress demands (from each column) are assumed to be uniformly distributed equally to each girder, except in the case of high horizontal curvature, supports with large skews, or bents with unequal column spacing.
- An effective width, as defined in SDC[2] 7.2.1, should be used in assessing superstructure member flexural strength. Girder and slab reinforcing within a 45 degree lateral spread from the bent cap face are considered to be fully effective in resisting longitudinal seismic overload demands.

  If additional contributory width is considered, the designer must ensure the integrity of the torsion/flexure mechanism as suggested from research[6]. Under longitudinal seismic overloads, significant loss of stiffness due to torsional cracking of the bent cap may occur once the cracking extends beyond the effective width. Since the cracked torsional stiffness of the bent cap is small relative to the girders, it is unlikely that the additional capacity beyond the effective width would exceed the torsional cracking moment. Therefore, provisions to increase capacity beyond the effective width would require design and detailing modifications to incorporate mechanisms such as bent cap prestressing, specially designated cap torsional reinforcement and flaring of the slabs and girders.
- Caltrans current design practice utilizes bonded tendons as the prestress force transfer mechanism. Unbonded tendons are not considered because of corrosion issues, as well as reductions in ultimate moment resistance and the large curvatures associated with the unbonded length of the tendons.
- For a prestressed concrete member, load resistance in the elastic and plastic range is carried by mechanisms that are fundamentally different[5].

At the service load level, the precompressed concrete section is idealized as an elastic material. Internal stresses from the uniform prestress compression force are combined with the effects of tendon eccentricity ($P \times e$ primary moments) to counteract the tensile stresses generated by the external load. By this action, cracking of the concrete member is thus prevented and normal elastic design relationships can be used.

When the seismic overload demands are applied, concrete, prestressing and mild steel are stressed into the elasto-plastic range where elastic theory no longer applies. Moment overload demands are then counteracted by an internal resisting couple (see Figure 5) supplied by the prestress and mild steel in tension and the ultimate concrete force in compression. Nominal flexural capacity can then be calculated using equilibrium and compatibility relationships similar to conventional reinforced concrete members where the bonded tendon is treated as high strength steel reinforcement.

- Secondary prestress moments and shears are induced by support restraints in a statically indeterminate prestressed frame[5], and are grouped with the externally applied loads. These moments and shears are a function of both frame shortening and $P \times e$ effects in the superstructure. Column plastic hinging, combined with limited inelastic action of the superstructure will reduce the magnitude of these secondary prestress force effects by a redistribution of forces after the first initial cycles. Due to the uncertainty of the magnitude and redistribution of secondary prestress moments and shears at the extreme seismic limit-state, it is prudent to include these forces in the superstructure when the overall demand is increased, and disregard when the overall demand is decreased.

Design Criteria

The superstructure demand shall be based on the overstrength moment capacity of the column plastic hinge combined with gravity, secondary prestress moments, and other seismic influences when applicable. Both the seismic demand distributed to the superstructure and the capacity to resist those demands are limited to the effective width as defined in SDC[2] 7.2.1. The column overstrength moment is defined as the idealized plastic moment capacity of the column determined from moment curvature analysis, based on expected material properties (SDC[2] 3.2 & 3.3), and the overstrength factor (SDC[2] 4.3). The column/superstructure joints must be properly proportioned and reinforced to transmit the column forces into the superstructure (SDC[2] 7.4).

The designer must ensure that the expected nominal strength (SDC[2] 3.4) and curvature capacity of the superstructure exceeds the column overload demands. A section capacity check shall be based on a stress-strain compatibility analysis[4,5], which assumes a linear strain distribution over the depth of the girder and satisfies force equilibrium on the cross section (see Figure 5). The only difference in the method for a prestressed versus reinforced concrete section is the accounting of the initial or decompression strains in the bonded tendons due to the effective prestress force (on the section prior to seismic load application). This check is important when prestressing steel is present because of the significantly lower ultimate allowable strain value of 3% as compared to 6 to 9% for conventional mild steel.

Moment and shear envelopes should be developed for both member capacity and distributed demand loading.

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