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Monolithic reinforced concrete bridge joints under cyclic excitation

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ABSTRACT

Joint regions in monolithic bridge connections often present geometric complexities owing to the structural detailing of the converging members, such as the occasional presence of passages through the body of the connections, the use of relatively high reinforcing ratios and the type of column section (circular or rectangular). An open issue in such circumstances is the definition of the effective joint area mobilized in shear transfer, and the effectiveness of joint reinforcement placed outside the joint panel in the adjacent members. In order to explore the effects of these geometric complexities with particular reference to the adequacy of the Eurocode 8-II (2005) design guidelines for bridge joints an experimental program was carried out on scaled specimens representing bridge monolithic connections under reversed cyclic loading and combined gravity loads. A total of ten specimens were tested. Of those, six represented pier column-superstructure joints, loaded either in the direction transverse to the bridge axis (four specimens) or along the bridge axis (two specimens), at a scale of 1/5. The remaining four specimens represented column-footing connections at a scale of 1/10. Parameters of study were the presence of openings longitudinally or transversally through the joint, the option of decongesting the joint by placing some of the required reinforcement in the adjacent converging members, the shape of the column cross section and the depth of the connection body since these parameters both define the joint volume engaged in shear transfer. The paper summarizes the experimental programme, the primary findings and the implications on established design practice for improved redundancy of force transfer.

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1. Introduction

Design procedures for bridge monolithic connections as a separate class of elements with special performance requirements was introduced for the first time in Eurocode 8-II (EC8-II, 2005) [1]. After the 1989 Loma Prietta earthquake many examples of bridge infrastructure damages were reported, some failures concerning inadequately confined monolithic connections (joints) between pier and superstructure or between pier and caissons [2]. Several experimental programs on monolithic bridge joints under simulated earthquake loading followed, on a variety of specimen forms, specimen scales and loading patterns.

Based on these experimental programs a total of 56 experiments are available in the literature that document performance of bridge monolithic connections under seismic action. Although the number of conducted experiments is low as compared to the sheer variety of existing geometric characteristics and configurations of the joints, a wide range of design parameter combinations have been explored that are known to affect the seismic behavior of joints. These include the geometry of the joint (Sritharan et al. [11], Naito et al. [12], Gibson et al. [14]), the scale of the specimens (Priestley et al. [6]), the design philosophy used in reinforcement detailing (Pantelides et al. [13], Mazzoni and Moehle [10]), the condition of the specimen (prototype, repaired or retrofitted) (Thewalt and Stojadinovic [3], Xiao et al. [4], Sexsmith et al. [5], Lowes and Moehle [8]) and the magnitude of shear stress demand with reference to the capacity. The morphology of the connection has also been a subject of investigation: many specimens represented joints of outrigger beams with columns (Ingham et al. [7]), whereas very few specimens model deck to pier connections or connections between bridge pier and footing (McLean and Marsh [9]). Due to the large variety in specimen form, the number of experiments that are directly comparable among the available tests so as to illustrate the influence of any single variable is still very low. Thus, statistical evaluation of test results and generalization of design methods cannot be drawn.

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Nevertheless, due to the pressing need for design procedures for this problem, the primary outcomes of this research activity served as the basis in drafting the seismic provisions for monolithic connections in the Caltrans [15] and EC8-II [1] design codes. Requirements concern cap-beam to pier connections, beam to pier connections as well as footing to pier connections. The salient points of these design approaches are as follows: (a) A bridge monolithic joint is capacity-designed in shear so as to resist the forces from the adjacent plastic hinge in the pier in the plane of seismic action. (b) The acceptance criteria refer to average joint shear stress, where the peak value of total shear stresses developed at the midpoint of the joint panel is compared against a semi-empirical design value obtained from first principles and calibrated against experimental data [15]. (c) Concrete and reinforcement contributions are separately taken into account in force equilibrium. (d) If the joint shear stress exceeds the limit value associated with concrete cracking, the required joint reinforcement is estimated from force equilibrium; otherwise, minimum joint reinforcement is placed in the joint. (e) Both codes allow the placement of a part of the vertical joint reinforcement in the beam outside of the joint. Although a primary outcome of the experimental research was that anchorage conditions of pier longitudinal reinforcement within the joint is a critical parameter in the assessment of existing joints, Caltrans [15] allows the configuration of straight column anchorages inside joint areas, whereas EC8-II [1] requires formation of hooks at the bar ends near the free joint face.

In particular the EC8-II [1] design provisions were based on an extension of a mechanistic model originally developed from building joints. To-date, these provisions have not been tested against experiments that represent the special morphology of bridge connections. To assess several of the open issues in the adopted design requirements and recommendations (such as the bond conditions of the main pier reinforcement anchorages, the effective joint area, the presence of openings in the body of the joint panel) with particular reference to the effectiveness of the Eurocode 8-II (2005) [1] design guidelines for bridge monolithic connections an experimental program was carried out on bridge joint specimens under reversed cyclic loading that simulates earthquake effects, along with combined axial loads. The following section provides a brief review of the current seismic design provisions in the European practice which serves as a background to the problem studied; the experimental program is presented in detail in the subsequent chapters of the paper.

2. Design background on bridge monolithic connections

For the benefit of better appreciation of the open issues in detailing and dimensioning bridge joints the following sections present a brief review of the relevant seismic design requirements currently in force in the European practice. Forces are associated to the formation of a plastic hinge in the bridge pier adjacent to its connection either at the top (to the cap-beam) or at the bottom (to the footing or pile-cap). Calculations refer to the center of the joint where moment transfer is evaluated as depicted in Fig. 1.

2.1. Summary of EC8-II (2005) requirements for bridge joint verification

In the EC8-II [1] requirements and guidelines for the design of bridge column-to-cap beam or superstructure joints reference is made to a solid joint panel, therefore certain bridge-specific details (openings, members with box-cross sections) are not actually addressed. A column with a solid rectangular cross section of section height h_c and width b_c (perpendicular to the plane in which the joint is studied) is considered. For the purpose of stress reduction from stress resultants the effective width of the joint $b_{j,eff}$ of the joint is defined as follows:

(a) When the column frames into a slab or into a transverse rib of a hollow slab:

$$b_{j,eff} = b_c + 0.5 \cdot h_c \tag{1}$$

(b) When the column frames directly on a longitudinal web of width b_w (where b_w is parallel to b_c):

$$b_{j,eff} = \min\{b_w, b_c + 0.5 \cdot h_c\} \tag{2}$$

(c) For circular column of diameter d_c, the above definitions are still applicable taking:

$$b_c = h_c = 0.9 \cdot d_c \tag{3}$$

(thus the design provision do not distinguish between a rectangular and a circular column section shape in determining the effective joint dimensions). Joint demand in terms of forces transferred through the joint and the corresponding joint stress are obtained by considering the free body diagram of the left (or right) part of the joint panel after a vertical section through the joint. With reference to Fig. 1, the design vertical shear force, V_{jz}, transferred through the joint is equal to:

$$V_{jz} = T_c - V_{br} = \alpha \cdot f_y \cdot \frac{A_{s,col}}{2} - V_{br} \tag{4}$$

where $T_c = 0.5 \cdot A_{s,col} \cdot f_y$ is the resultant force of the tensile reinforcement of the column that is associated with the design flexural resistance M_{Rd} of the plastic hinge, α is the over-strength factor and V_{br} is the shear force of the beam adjacent to the tensile face of the column, resulting from capacity-design considerations when a plastic hinge occurs in the column (Fig. 1).

The design horizontal shear of the joint V_{jx} is evaluated from the following equation:

$$V_{jx} \cdot z_b = V_{jz} \cdot z_c \tag{5}$$

where $z_b = 0.9 \cdot h_b$ is the internal force lever arm of the beam's section at the face of the joint and $z_c = 0.9 \cdot h_c$ is the internal force lever arm of the column's section within the plastic hinge location at the face of the joint.

Joint shear strength verification is carried out at the joint center, where in addition to V_{jz} and V_{jx}, the simultaneous influence of the following axial forces is taken into account:

(a) The vertical axial joint force N_{jz} that arises if it is considered that: (i) the gravity related axial stress is uniformly distributed over a horizontal column cross section in the joint region. (ii) Vertical loads are transferred gradually from the deck to the column over the height of a joint. It is assumed that the axial force at the joint-midheight is equal to half of the axial force of the column for T-shaped or Γ-shaped joints when $b_{j,eff} = b_c$. (iii) When $b_{j,eff} > b_c$ the axial force of the joint is reduced following the pattern of stresses which are assumed to be distributed over an effective area that is greater than the area of the column's section. Therefore:

$$N_{jz} = 0.5 \cdot N_{c,G} \cdot \frac{A_c}{A_{j,eff}} = 0.5 \cdot \frac{b_c}{b_{j,eff}} \cdot N_{c,G} \tag{6}$$

where $N_{c,G}$ is the axial force of the column under permanent actions (serviceability limit state).

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