A case study on pre 1970s constructed concrete exterior beam-column joints

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Abstract

The exterior beam-column (EBC) joints in the second generation concrete structures (built prior to 1970s) were observed to underperform during earthquakes mainly due to the lack of shear reinforcement and improper construction practices. In this paper, results of a detailed parametric study conducted using 3D finite element analysis are presented to illustrate the performance of three types of substandard EBC joints that are typical to pre 1970s construction practices designed according to: (1) ACI 318-1971, (2) old Japanese practice and (3) old Indian practice. In addition, the influence of concrete strength, beam and column flexural reinforcements on the load carrying capacity of substandard EBC joints are investigated. The substandard joints analyzed in this study failed in a brittle fashion due to the shear failure of joint core concrete and anchorage slippage.

Introduction

The reinforced concrete frame structures constructed before the introduction of seismic design codes (prior to 1970s) were found to be incompetent to withstand the loads generated during seismic events as they are primarily designed for gravity loads. Lack of transverse beam-column joint [1] reinforcement, use of plain bars for longitudinal reinforcement, poor anchorage detailing, and low concrete strength are the most common deficiencies of pre 1970s reinforced concrete frame structures [2]. The reinforced concrete frame structures located in seismically active zones that are still in their serviceable life but incompetent to the anticipated seismic forces are to be retrofitted. It is important to assess the existing strength of these seismically deficient structures to arrive at a retrofitting strategy. Strength assessment of existing structures requires a detailed understanding of structural behavior under critical loading scenarios. With the latest advances in solid mechanics and availability of abundant computational resources, it is possible to simulate and analyze critical components of seismically deficient structures to better understand their behavior and thereby assess their capacity prior to retrofitting.

The current manuscript is concerned with 3D finite element analysis of exterior beam-column (EBC) joints typically found in pre 1970s constructed reinforced concrete structures [2–5]. The two main objectives of this article are: (1) to compare the performances of substandard EBC joints that were designed according to different pre 1970s design and construction practices (ACI 318:1971 [1], old Japanese practice and old Indian practice) and (2) to qualitatively establish the effect of various parameters (cylindrical concrete strength ($f_c$), percentage beam ($\rho_b$) and column flexural reinforcement ($\rho_c$) on...
the performance of EBC joints. The objectives of this article have implications on arriving at a plausible shear strength assessment model for EBC joints which is the first step for retrofitting strategy.

Finite element analysis

Micro-Plane Analysis Program (MASA®), an in-house finite element solver developed at University of Stuttgart (Germany) [6] that is capable of simulating fracture in concrete [7] and bond between reinforced steel and concrete [8,9] is employed in this study. This finite element program was successfully used to study the behavior of reinforced concrete structures in a previous study [10].

Validation of finite element model

The substandard EBC joints tested by Genesio [10] are used in the calibration of finite element models. Genesio [10] tested EBC joints that were designed according to (a) ACI 318:1971 [1], (b) old Japanese practice and (c) old Indian practice. The detailing of subassemblies and joint reinforcement is provided in Fig. 1(a–f). Detailed design details of all the specimens can be found in [10].

In this study, the experimentally obtained cyclic behavior of EBC joints designed according to ACI 318:1971 [1] and old Japanese practice are matched with finite element results for the validation of finite element models. Three dimensional hexahedral elements with side length of approximately 20 mm were adopted for the mesh in the Joint (Fig. 1g). The load is applied on the beam and hinge boundary conditions are used at the column ends as shown in the Fig. 1(g). The mechanical properties of steel and concrete are obtained from [10] and [11] respectively. A cylindrical concrete strength ($f_{c0}$) of 26 MPa is used for the analysis purpose. The bond slip properties of longitudinal deformed bars and plain round bars corresponding to this concrete strength are obtained from Lettow [8] and Fabbrocino et al. [12] and can be found in [10]. Finally, the cyclic load displacement behavior predicted by the finite element model is compared with the experimental results for EBC joints designed according to ACI 318:1971 and old Japanese practice in the Fig. 2(a and b). The cracking pattern obtained from the finite element analysis is compared to the experimental results in the Fig. 3(a). From the Figs. 2 and 3(a) it is clear that the calibrated finite element model is able to fairly predict the load displacement behavior and ultimate cracking patterns of ACI 318:71 [1] and old Japanese specimens. In the further sections, the finite element models are employed to investigate the influence of various parameters on the load carrying capacity of substandard EBC joints.

Parametric study

Finite element models are used to conduct a detailed parametric study on factors that have a major influence on the load carrying capacity of substandard EBC joints. The critical factors chosen for the parametric study are: (1) anchorage...
configuration, (2) cylindrical concrete compressive strength \(f_c\), (3) percentage beam flexural reinforcement \(\rho_b\) and (4) percentage column flexural reinforcement \(\rho_c\). As a part of the parametric study, monotonic load displacement behavior of the EBC joints predicted by the finite element analysis obtained for different parameters are compared and the influence of individual parameters are highlighted in the following sections.

Anchorage configuration

The anchorage configuration (Fig. 1d–f) encompasses the following details in the design of substandard EBC joints: (1) horizontal anchorage \(l_{dh}\), (2) hook (90° hook or 180° hook) and (3) extension of the hook \(l_v\) (7.5 \(\phi\) for ACI 318:1971 [1] and old Japanese specimen, 4 \(\phi\) for old Indian practice). The minimum horizontal anchorage \(l_{dh}\) is necessary to transfer the tensile stress in the flexural reinforcement in to the joint. The hook is responsible for the formation of the compression strut which in the case of substandard EBC joints is one of the major load transfer mechanisms [4,5,13] from beam to column through the joint core. Sufficient extension \(l_v\) of the hook is necessary to prevent the pullout of flexural reinforcement. All the EBC joints used in the parametric study do not have any joint reinforcement and these anchorage configurations typically represent poor design and construction practices that are often observed in reinforced concrete structures built prior to 1970s. The three anchorage configurations used in the parametric study do not have any joint reinforcement and these anchorage configurations typically represent poor design and construction practices that are often observed in reinforced concrete structures built prior to 1970s.

The three anchorage configurations used in the parametric study are: (1) ACI 318:1971 design: The main deficiencies of this joint design are (1) lack of joint core shear reinforcement and (2) insufficient vertical extension \(l_v\) of the 90° hook (7.5 \(\phi\) tail extension according to ACI 318:1971 [1] vs. 12 \(\phi\) tail extension according to ACI 318:1995 [14]; (2) Old Japanese practice: This EBC joint design is performed according to ACI 318:1971 [1] but with an additional deficiency of bending the bottom tail extension away from the EBC joint core (Fig. 1e). This construction practice was widely adopted in the old Japanese practice.
constructions for the ease of compaction of joint core concrete; and (3) Old Indian practice: This EBC joint detailing was a common practice in India prior to the introduction of IS 13920:1993 [15]. In these joints, 180° hooks with a 4 Φ anchorage extension made of plain round bars are used as the anchorage configuration (Fig. 1f). Lack of joint shear reinforcement, deficient anchorage and poor bonding properties of plain round bars are the major deficiencies of EBC joints designed according to old Indian practice.

The monotonic load displacement behavior of these anchorage configurations are compared in Fig. 3(b). Shear failure of joint is observed in all the EBC joint configurations (Fig. 3a). This joint shear failure observed in all the EBC joints is a direct consequence of the absence of shear reinforcement in the joint core. Among the considered anchorage configurations, the EBC joint designed according to old Indian practice exhibited most brittle behavior when compared to other joints due to the combined shear failure of the joint core and slippage of longitudinal bars due to the use of plain round bars (Fig. 3b). Although the percentage flexural reinforcement ($q_b$) and vertical extension ($l_v$) of the hooks are same for EBC joints designed according to ACI 318:1971 [1] and old Japanese practice, the EBC joints designed according to old Japanese practice exhibited brittle behavior due to the lack of proper confinement of the core concrete resulted from not anchoring the hook in to the joint core. From these numerical studies, it is observed that pre 1970s constructed EBC joints have brittle modes of failure as a result of lack of shear reinforcement in the joint core, improper confinement of core concrete and insufficient anchorage.

**Influence of cylindrical concrete strength**

A major portion of the entire shear force has to be transferred by concrete alone due to the development of compressive strut mechanism in the EBC joints with no joint shear reinforcement [4]. Hence it is important to examine the effect of cylindrical concrete strength ($f_c^t$) on the performance of substandard EBC joints. For this purpose the monotonic load displacement behavior of EBC joints designed according to ACI 318: 1971 [1] and old Japanese practice are obtained for cylindrical concrete strength ($f_c^t = 23,58$ N/mm$^2$). The bond properties of the reinforced steel for different concrete strength are obtained from [8]. The results of the parametric study are presented in the Fig. 4. From the Fig. 4(a and b), it can be concluded that the substandard EBC joints constructed using concrete with high cylindrical concrete strength ($f_c^t$) perform better than their counterparts. This can be attributed to the increased load carrying capacity of the strut mechanism and enhanced bond properties of the longitudinal bars. In addition, it is found that the increase in the concrete strength does not ensure increase in the load carrying capacity after yielding of the longitudinal beam bars as shown in Fig. 4(a). The numerical studies conclude that cylindrical concrete strength ($f_c^t$) plays an important role in determining the load carrying capacity of substandard EBC joints.

**Influence of percentage beam reinforcement**

An increase in the percentage beam reinforcement ($q_b$) increases the flexural stiffness of the hook. The increased flexural stiffness of the hook provides better confinement to the joint core concrete and also accommodates the formation of compression strut mechanism which in turn increases the load carrying capacity of the EBC joint. In this study, the percentage beam reinforcement ($q_b$) is varied between 0.5% and 3.0% to quantify its influence on load carrying capacity of substandard EBC joints. The results shown in the Fig. 5(a and b), indicate that the increase in the percentage beam reinforcement has a positive effect on the load carrying capacity of the EBC joints. It can also be observed that the strength enhancement is more evident in the case of EBC joints designed according to ACI 318:1971 [1] (Fig. 5a) when compared to joints designed according to old Japanese practice (Fig. 5b). This can be attributed to the better confinement of joint core concrete in the case of EBC joint designed according to ACI 318:1971 [1] when compared to EBC joint designed according to old Japanese practice. The
numerical analyses results highlight the role of percentage beam reinforcement \( (\rho_b) \) in enhancing the load carrying capacity of substandard EBC joints.

Influence of percentage column reinforcement

With the increase in the column flexural reinforcement the stiffness of the column increases and hence more loads are transferred to the column. This increases the compression zone of the column in the joint core which helps in better confinement of core concrete. But the load transfer to the column is possible only when beam and column are connected through a properly designed joint. In cases where the joint is incompetent to transfer the load from beam to column, the increase in the column reinforcement may not have much influence on the load carrying capacity of the joint. In this study, the percentage column reinforcement \( (\rho_c) \) is varied between 1.8% and 5.2% to quantify its influence on load carrying capacity of substandard EBC joints. The results obtained from the numerical analysis are presented in the Fig. 6(a and b). The EBC joint designed according to ACI 318:1971 \([1]\) (Fig. 6a) is superior to the joint designed according to old Japanese practice (Fig. 6b) in terms of transferring the load to the column. Hence increase in the percentage column reinforcement \( (\rho_c) \) increased the load carrying capacity of the EBC joint designed according to ACI 318:1971 \([1]\) but this enhancement in the load carrying capacity is not observed in the case of EBC joint designed according to old Japanese practice. This is due to the deficiency of the EBC joint designed according to old Japanese practice to transfer the load from beam to column. However, it should be noted that the enhancement in load carrying capacity in EBC joints designed according to ACI 318:1971 \([1]\) is not very significant. The numerical analysis highlights the fact that the EBC joints designed prior to 1970s do not have the capability to facilitate complete load transfer from beam to column due to the lack of shear reinforcement and required anchorage and hence the enhancement in the load carrying capacity due to increase in percent column reinforcement \( (\rho_c) \) is not always warranted.

Conclusions

The following are the important conclusions of the study
The anchorage configuration has a significant influence on load carrying capacity and failure mode of EBC joints. Shear failure of joint core and slippage of flexural reinforcement are found to be the modes of failure in substandard EBC joints. While the EBC joints designed according to ACI 318:1971 [1] and old Japanese practice failed due to the lack of shear reinforcement, the EBC joints designed according to old Indian practice failed due to slippage of flexural reinforcement in addition to joint shear failure. Among the substandard EBC joints considered in the study, the joint designed according to ACI 318:1971 [1] is observed to have better performance when compared to the other anchorage configurations. This can be attributed to the better confinement to the joint core concrete provided by the anchorage configuration specified by ACI 318:1971 [1].

In the case of substandard EBC joints with no shear reinforcement, cylindrical compressive strength ($f_{c}^0$) of the concrete is found to be the most influential parameter that governs the load carrying capacity of the joint.

Increase in the percentage of beam reinforcement ($q_b$) is found to increase the load carrying capacity of the joint. This can be attributed to the increased flexural resistance of the anchorage bars which in turn prevents opening of the hook thereby preventing spalling of concrete cover and providing confinement to the joint core concrete.

Increase in the percentage of column reinforcement ($q_c$) increases the load carrying capacity only when the EBC joints are designed properly. Especially in the case of substandard EBC joints, this effect can be neglected as the applied loads are not completely transferred to the column due to poor joint design.

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