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CAPACITY DESIGN OF INFILLED FRAME STRUCTURES

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SUMMARY

Reinforced concrete frames with masonry infills are still used as structural system to provide earthquake resistance in seismic regions where masonry is a convenient material due to economical and traditional reasons. Even though infilled frames can exhibit an adequate response when properly designed, severe damage and loss of life have occurred in past earthquakes. Some of the problems associated with the damage or collapse of infilled frame buildings are: irregular distribution of the masonry infills, inappropriate detailing of the reinforced concrete frame, partial infill, and deficiencies in materials and workmanship. However, inadequate design criteria and the lack of comprehension of the structural behaviour are also critical issues. It is very important, therefore, to develop simple and rational design procedures in order to obtain a safe and economical solution.

Using the principles of capacity design, a new design approach is proposed for cantilever infilled frames, in which the ductile behaviour is achieved by controlled yielding of the longitudinal reinforcement at the base of the columns. A pre-cracked connection is induced between the infilled frame and the foundation, where plain round dowels can be placed to control shear sliding. The proposed procedure also assures a simple evaluation of the lateral resistance, avoiding the uncertainties associated with the complexity of the panel-frame interaction. The use of tapered beam-column joints with diagonal reinforcement is recommended in order to reduce the opening of the joints and to improve the transfer of the lateral forces from the frame to the masonry panel.

INTRODUCTION

It has been generally recognized that infilled frame structures exhibit poor seismic performance, since numerous buildings have failed in past earthquakes. One of the most important problems is the degradation of stiffness, strength and energy dissipation capacity observed under cyclic loading, which results from the progressive damage of the masonry wall and the deterioration of the panel-frame interfaces. Consequently, only low to medium displacement ductilities can be achieved. The lack of comprehension of the structural behaviour has also contributed to bad performance of infilled frames. It must be recognized that these composite structures exhibit a complex and markedly nonlinear response, which results from the brittle behaviour of the unreinforced masonry, the ductile nonlinear characteristics of the frame, the different deformational properties and strengths of both components, and the variable conditions at the panel-frame interfaces.

Infilled frames are commonly used for low and medium-height buildings all over the world in regions of high seismicity, especially in developing countries where the labour costs are not very high or where masonry structures are used for traditional reasons. It is believed that the development of rational design procedures is a critical issue not only to reduce the loss of life and property damage, but also to obtain a safe end economical solution.

The term "infilled frame" designates a composite structure formed by one or more infill panels surrounded by a frame. Different terminology is normally used depending on the constructive techniques. In some cases, the term infilled frame refers to the situation in which the frame is built first and then infilled with one or more masonry panels. Several researchers introduced the term "confined masonry" to describe the case in which the

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reinforced concrete frame is cast after the construction of the masonry panel. In the authors' opinion, this is an improper denomination because the masonry panel is not confined by the frame, especially under lateral forces which produce cracking at the panel-frame interfaces. Therefore, the denomination "framed masonry" is used here to specifically refer to this type of structure, whereas "infilled frame" is used in a general sense.

DESIGN CRITERIA

Background

Experimental data reported by different researchers [3] indicate that a reasonably ductile response can be obtained if the infilled frame is properly designed and constructed. In the inelastic range, the surrounding frame is able to resist large deformations, whereas the masonry panel cracks and fails under relatively low distortions. Consequently, they can be considered as structure of limited ductility (global ductility factor between 2.0 and 3.0), provided that brittle modes of failure are avoided or adequately controlled to delay their effect on the response.

Infilled frames are usually designed according to simplified procedures, in which the lateral resistance of the structure is evaluated taking into account the shear bond strength of masonry and the applied vertical load. The surrounding reinforced concrete frame is usually designed to resist the axial forces resulting from the truss mechanisms, without taking into account the particular aspects resulting from the interaction between the panel and the frame. Other aspects, such as adequate design of the floor beams and the beam-columns joints, and control of potential sliding shear failure of the columns, should be also considered.

In the last years, significant efforts have been made to improve the response of infilled frames by providing horizontal reinforcement in the mortar joints of the panel. This reinforcement is properly anchored in the surrounding columns, and design recommendation based on this criterion has been proposed (Aguilar et al. [1], Alvarez [2], and Zarnic and Tomazevic [10]).

Paulay and Priestley [6] and San Bartolomé et al. [9] suggested that infilled frames can be designed to fail in a "flexural mode" by yielding of the tension column in order to obtain a reasonably ductile behaviour, avoiding other brittle types of failure. However, yielding of the tension column could produce lack of restraint and stability problems. When the columns of the infilled frame are subjected to tensile forces, numerous near-horizontal cracks cross the width of the column. Increasing forces will produce yielding of the longitudinal reinforcement, if the masonry panel is able to resist the diagonal compressive force resulting from the equivalent truss mechanism. Since the tensile force, and consequently the strain, is approximately constant along the column, yielding of the reinforcement results in a significant elongation of the member, which is usually incompatible with the brittle characteristics of the masonry panel. The elongation of the columns reduces or eliminates the beneficial effect of the frame in restraining the masonry panel and jeopardizes the stability of the panel against out-of-plane actions.

Proposed Procedure

The solution for a proper design can be achieved in a different way. Following principles of capacity design, undesirable modes of failure in the surrounding frame or in the masonry panels can be avoided, while plastic deformations are deliberately induced in special parts of the structure, which are adequately detailed to this purpose. In this case, it has to be assured that the foundation system is able to resist elastically the actions transmitted by the superstructure. As a first step, it is very important to identify the modes of failure or other detrimental effects which need to be controlled or avoided. The most important are:

- **Shear cracking of the masonry**. Cracking in the masonry panel due to shear stresses is a very common type of failure observed in infilled frame buildings affected by earthquakes. This type of failure is mainly controlled by the shear strength of the mortar joints, the tensile strength of the masonry units and the relative values of the shear and normal stress. Depending on these parameters, the combination of shear stresses with vertical axial stresses can produce either cracks crossing the masonry units or debonding along the mortar joints (also termed as shear friction failure). In the latter case, the cracks usually develop following a stepped pattern along the diagonal direction. Shear cracking does not necessarily represent a failure condition provided that the cracked panel is restrained by the surrounding frame and the shear distortion is controlled. The formation of diagonal cracks is regarded only as a serviceability limit state. For hollow masonry, however, cracking of the masonry units can produce the failure of the structure

- **Elongation of the reinforced concrete members**. The longitudinal bars of reinforced concrete members can yield in tension with significant ductility. However, this is not convenient in infilled frame structures because the excessive elongation of the frame members reduces the beneficial effect of the frame, which restrains the shear distortion of the masonry wall. Consequently, the columns and beams of the surrounding frame should be designed to resist the tensile axial forces resulting from seismic actions without yielding of the reinforcement.

- **Beam-column joint failure**. High normal and tangential stresses develop along the contact lengths in the zones near to the loaded corners, resulting in large shear forces and bending moments. The stress state induced in these beam-column joints may cause the formation of wide diagonal cracks running across the joint from the interior to the exterior corner. Minor attention has been given to this mode of failure, even though it has been observed in different investigations. The failure of the beam-column joint causes unfavourable effects in the behaviour of infilled frames, because the lateral forces cannot be transferred from the floor beam to the columns and the masonry panel. Furthermore, the formation of diagonal cracks causes the opening of the joint. Therefore, the contact length at the loaded corners and the width of the equivalent strut decrease, resulting in an increase of the stresses in the masonry panel.

- **Shear failure of the columns.** The columns can fail due to the shear forces resulting from the interaction with the infill panel. The maximum shear forces occur along the contact length, near the loaded corners. Sliding shear failure is a particular case which can occur at the top of the columns, close to the beam face, as a result of the unfavourable combination of shear and tensile axial forces.

In the following sections, a new approach is proposed for the ductile design of cantilever infilled frames. The term "cantilever" is used here to refer to infilled frames in which flexural effects are significant and yielding of the longitudinal reinforcement of the columns can occur. Design recommendations for squat infilled frames have also been developed by Crisafulli [3], although they are not included here due to limitations in space.

PROPOSED DESIGN OF CANTILEVER INFILLED FRAMES WITH FLEXURAL YIELDING

General Description of the Procedure

The seismic response of cantilever infilled frames can be improved by a rational design, in which ductile behaviour is achieved by controlled yielding of the columns subjected to tensile axial forces, and the masonry panels are prevented from suffering severe damage. In order to preserve the geometry of the masonry panel, the elongation of the columns is controlled with additional longitudinal reinforcement, as Fig. 1 illustrates. These additional bars, which are not anchored to the foundation, assure the formation of a weak region at the base of the columns, where most of the plastic deformations will develop. According to this criterion, the capacity of the column to resist tensile axial forces is determined by the amount of anchored reinforcement. Therefore, the lateral strength of the infilled frame can be easily evaluated, considering a simple flexural mechanism and avoiding some of the uncertainties related to the panel-frame interaction. Based on this mechanism, the amount of longitudinal reinforcement of the columns anchored in the foundations, Asa, can be determined to resist the earthquake design forces.

According to the principles of capacity design, it is required to estimate the maximum shear force that could be sustained by the infilled frame in order to assure that the selected plastic mechanism is achieved and brittle modes of failure are avoided. The design shear force, Vu, can be determined from the shear demand specified by the code, V, considering the flexural overstrength of the infilled frame and the influence of higher modes of response [6]:

$$V_u = \omega_v \phi_o V$$

where No is the flexural overstrength factor defined as the ratio of the flexural overstrength to the overturning moment resulting from code forces, and Tv is the dynamic shear magnification factor defined as a function of the number of storeys of the building [6].

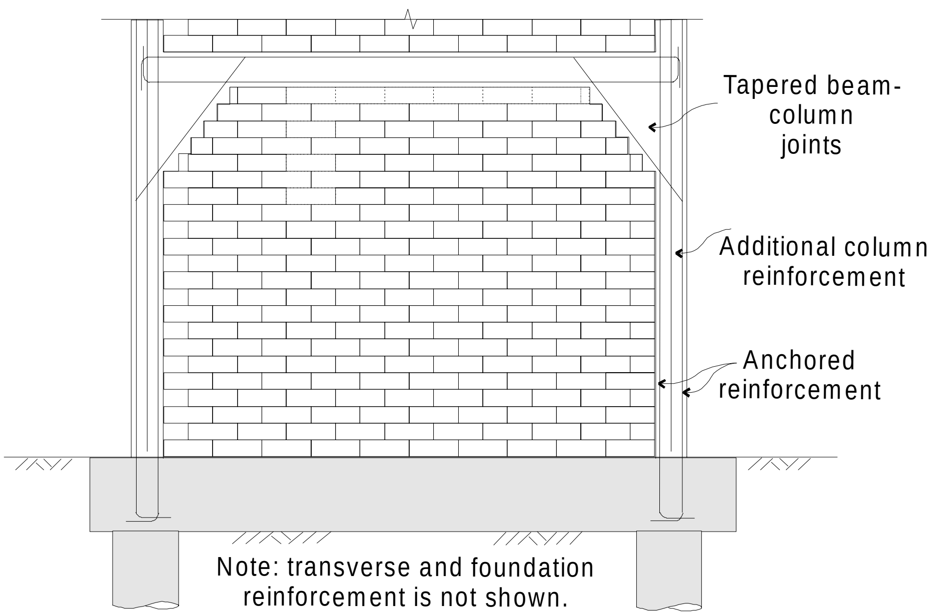

Figure 1. Reinforcing details for infilled frames designed according to the proposed approach.

Sliding shear can occur at the bottom of the compression columns, due to the main crack previously formed when the column was subjected to tension. The designer should be aware of this problem and the cracked section should be verified to assure that the strength is equal or greater than the applied shear force. The shear strength of a cracked section results from a combined mechanism of friction between the concrete surfaces and dowel action in the reinforcing bars crossing the crack. Since the development of dowel action requires large relative displacements, it is usually assumed in design that a conservative estimation of the shear capacity of the cracked section is given by the shear force resisted only by the friction mechanism, Vf, which can be calculated according to the following expression:

$$V_f = \mu\left(\Sigma A_{si} f_{si} - P\right)$$

where μ is the coefficient of friction of concrete (normally equal to 1.4 for concrete cast monolithically [5]), P is the gravity load acting on the section and Asi fsi represents the axial force in the longitudinal bars which provide a clamping action (the forces considered in this equation are positive when tensile).

The design approach proposed in the previous section is also applicable for multibay infilled frames, provided that tensile yielding of the longitudinal reinforcement of the columns occurs. The evaluation of the actions in the frame members and in the masonry panels can be conducted using a simple model, such as the equivalent truss mechanism. It is important, however, to consider a realistic distribution of the seismic forces along the beams. An inadequate representation, such as assuming that the total lateral force corresponding to a certain floor level is applied at one end of the beam, can lead to significant errors in the axial forces of the structural members. The flexural strength of the connection can be calculated applying a similar procedure to that considered for reinforced concrete columns with multiple reinforcing steel layers. Equilibrium and compatibility equations allow the calculation of the stresses and strains of the longitudinal reinforcement crossing the connection.

Enhancement of the Behaviour

Additional measures can be taken to improve the behaviour of infilled frames subjected to seismic actions. The use of tapered beam-columns joints with diagonal reinforcement contributes to a reduction of the distortion of the masonry panel by limiting the opening of the joints (see Fig. 2). This improves the transfer of the lateral forces from the frame to the panel and increases the width of the compressive strut. Furthermore, the formation of a sliding shear crack at the top of the column is improbable in this case, because most of the lateral force is transmitted to the panel by the diagonal surface of the joint. The beneficial effect of the proposed detail has been experimentally verified [3, 4]. For practical reasons, tapered beam-column joints are easy to build in framed masonry structures, that is when the reinforced concrete frame is cast after the construction of the masonry wall.

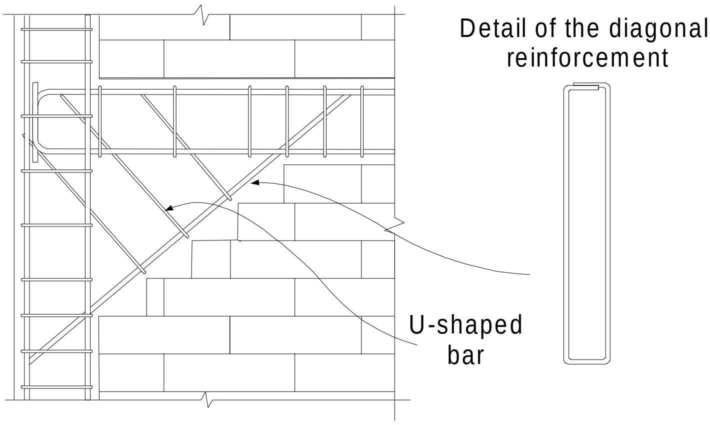

Figure 2. Detail of the tapered beam-column joints.

Fig. 3 illustrates a further development in the design of infilled frames, in which a reinforced concrete beam is constructed between the masonry panel and the foundation. In this way, the frame completely encloses the lower masonry panel, thus preserving the geometry of the panel. When large displacements are imposed on the structure, yielding of the longitudinal reinforcement concentrates at the base of the column and the bottom beam separates from the foundation, allowing the masonry panel to accompany the deformation of the frame with a markedly reduced distortion. In order to achieve this behaviour, it must be assured that a crack forms along the beam-foundation interface. A pre-cracked connection can be obtained by applying an adequate sealant over the foundation to break the adhesion between hardened and fresh concrete, avoiding the development of tensile stresses at the interface. The top surface of the foundation should be properly roughened to allow the transfer of shear stresses by friction.

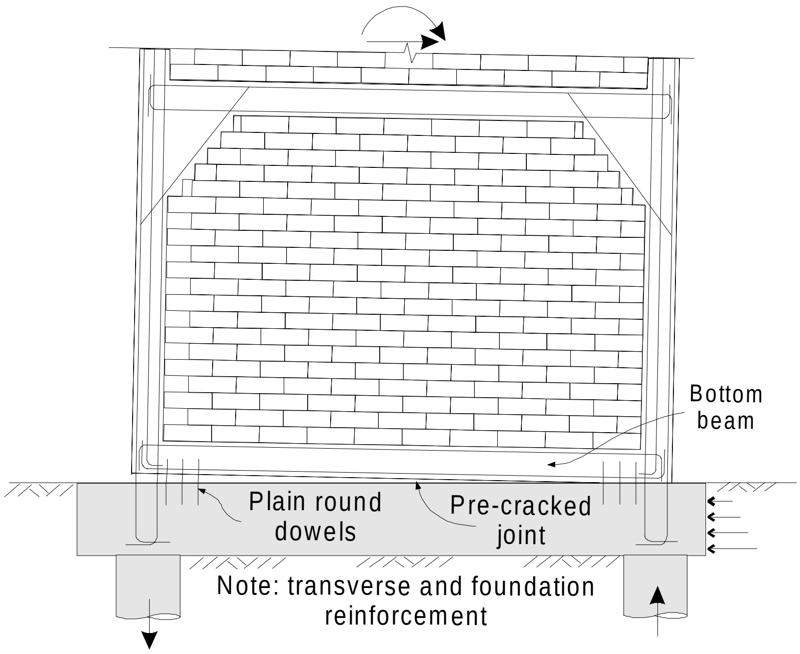

Figure 3. Recommended design of infilled frames with a pre-cracked connection.

Plain round dowels can be embedded into the foundation and into the bottom beam to control sliding shear of the infilled frame, in those cases where the frictional strength of the pre-cracked connection is exceeded. The plain bars can resist part of the total shear force by a combination of shear and flexure mechanisms, whereas they do not contribute to the flexural strength of the infilled frame. For design purposes, it can be assumed that the shear forces resisted by the plain round dowels is:

$$V_d = 0.42 A_{sd} f_{yd}$$

where Asd is the total area of the plain round dowels and fyd is the yield strength. This equation was proposed by Restrepo et al. [7] under the assumption that the shear force in the dowel is transferred by a flexure mechanism [5], with a distance between plastic hinges equal to the diameter of the dowel. The total area of the dowels, Asd, can be obtained from the previous equation to resist a shear force Vd = Vu – 0.75 Vf (the factor 0.75 is introduced to consider the detrimental effect of cyclic loading in the frictional strength). In order to calculate Vf the coefficient of friction can be adopted as $\mu = 1.0$, considering that the pre-cracked connection is not monolithic concrete [5].

The plain round dowels should be located in a symmetrical arrangement, preferably along the centre line of the bottom beam, with a separation larger than 15 times the diameter of the dowel. This value is conservatively assumed, in the lack of experimental information, in order to avoid local cracking of the surrounding concrete due to the splitting effect induced by the dowels. The use of several dowels of small diameter could be preferred. The bottom beam has to transfer part of the shear force from the joints of the frame to the dowels. In some cases, depending on the configuration of the infilled frame and on the location of the dowels, tensile axial forces may develop in the bottom beam. This situation should be also considered in the design.

Dowel action represents a very flexible mechanism and significant pinching of the hysteresis loops can be expected under cyclic loading. When the lateral forces reverse after significant plastic deformations have developed in the longitudinal reinforcement, the shear has to be entirely transferred by dowel action until contact between the surfaces of the pre-cracked connection is restored. In order to reduce this effect, additional dowels can be placed in the connection to increase the stiffness.

Ductility Requirements

According to the design procedure outlined in the previous section, most of the plastic deformations are concentrated in a small region of the structure, where large strains can be induced in the longitudinal reinforcement of the columns. In order to assure that fracture of the steel does not occur, the deformation capacity of those bars should be compared with the demand imposed by severe seismic actions. It is useful, therefore, to find a relationship between the maximum strain in the reinforcing bars and the global ductility of the structure.

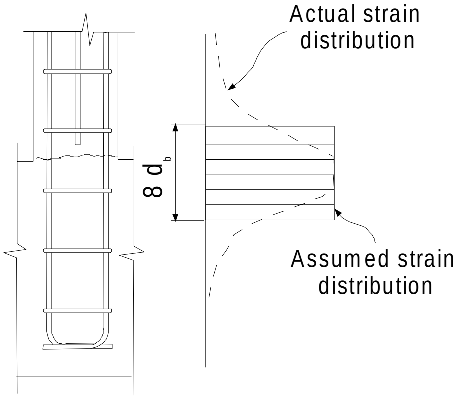

Figure 4. Variation of the strains in the reinforcing bars of the columns.

The lateral displacement of the infilled frame can be evaluated taking into account the contribution of two components, one due to shear and flexural deformation of the structure and the other due to strain penetration occurring at the base of the columns. The latter component depends on the elongation of the anchored reinforcement which can be approximately evaluated assuming an equivalent uniform distribution of the strains along a length equal to 12 times the bar diameter, db, as indicated in Fig. 4. This simplified criterion is adopted in the lack of more precise experimental data, considering typical values of strain penetration in flexural plastic hinges [6] and results reported by Restrepo et al. [8]. Based on these considerations it can be found that the relationship between the maximum strain, γmax, expected in the anchored reinforcing bars of the columns and the global displacement ductility, :), is given by:

$$\varepsilon_{\max} = \varepsilon_y + \frac{(\mu_\Delta - 1)\Delta_{fy} L}{12 d_b h^*}$$

where γy is the yield strain of the reinforcing steel,)fy is the lateral displacement of the infilled frame due to shear and flexure deformation at first yield, L is the distance between the centre lines of the end columns and h* is the height of the resultant of the seismic lateral forces. The maximum strain permissible in the reinforcing steel when subjected to tension depends on several parameters, such as characteristics of the steel, strain history and temperature. Based on the experimental data available in the literature and considering that sliding shear can bend the longitudinal reinforcement in double curvature at the base of the columns, inducing compressive strains in the concave side of the bar which can significantly reduce the fracture strain, it is recommended that the maximum strain in the longitudinal reinforcement should not exceed 50% of the ultimate (uniform) strain, γsu.

SUMMARY OF THE PROCEDURE

The design procedure outlined in the previous sections can be summarized as follows:

- Evaluate the actions in the infilled frame based on a simple model, such as the equivalent truss mechanism, assuming an adequate distribution of the lateral forces along the beams.

- Calculate the longitudinal reinforcement of the columns of the lower storey to provide adequate flexural strength to the pre-cracked connection, taking into account the gravity loads and the strength reduction factor prescribed by the design code. This reinforcement, with total area A_{sa} should be properly anchored in the foundations. Provide additional longitudinal reinforcement in the lower columns to induce a "weak region" at the base. The area of the additional reinforcement should be at least equal to 0.5 A_{sa}.

- Evaluate the design shear force V_u, taking into account the flexural overstrength of the pre-cracked connection and the influence of the higher modes of response.

- Verify that the masonry panel is able to resist the diagonal compressive forces induced by the design shear V_u without failure. When solid masonry units are used, cracking of the panel is regarded as a serviceability limit state.

- Design the floor beams and the columns of the upper storeys to resist the axial tensile forces (considering flexural overstrength) without excessive elongation. Yielding of the longitudinal reinforcement should be avoided.

- Provide tapered beam-columns joints, if possible, with diagonal reinforcement, assuring that the inclined surface of the joint is perpendicular to the diagonal of the masonry panel. The vertical an horizontal dimensions of the tapered joint should be greater than 1.5 times the depth of the beam and the column, respectively. The total area of the diagonal reinforcement can be taken as 0.5 A_{sa}.

- Design the transverse reinforcement of the frame members to resist the shear forces, to provide confinement to the concrete and to avoid premature buckling of the longitudinal reinforcement.

- Produce a pre-cracked connection between the infilled frame and the foundation, assuring that adhesion is broken and that the top surface of the foundation is properly roughened. The connection should be able to transfer the total shear force by a friction mechanism. Otherwise, plain round dowels should be placed to connect the foundation and the infilled frame to control sliding shear.

- Check the maximum tensile deformation expected in the longitudinal reinforcement of the columns does not exceed 0.5 γ_{su}.

- Design the foundations of the infilled frame to resist elastically the gravity loads and the actions resulting from the plastic mechanism of the superstructure.

CONCLUSIONS

A new design approach for cantilever infilled frames is proposed, in which ductile behaviour is achieved by yielding of the longitudinal reinforcement of the columns. Yielding is limited to the base of the columns, avoiding large elongations of these members. A pre-cracked connection is induced between the infilled frame and the foundation, where plain round dowels can be placed to control shear sliding. The use of tapered beam-columns joints with diagonal reinforcement is recommended to reduce the opening of the joints and to improve the transfer of the lateral forces from the frame to the masonry panel.

Cantilever infilled frames can be designed to obtain a reasonable ductile response. With this aim, a design framework has been proposed, based on rational considerations and the experimental results. It must be recognized, however, that more research is needed to clarify some aspects of the design, such as convenient dimensions of the tapered joints, area of the additional longitudinal reinforcement and response of the pre-cracked connection, and to verify the behaviour of multibay and multistorey structures. Considering that this type of structures are still used in many developing countries for residential buildings, further investigation is encouraged. If the proposed design procedure is corroborated by subsequent research, efforts should be made to incorporate these recommendation in seismic codes.

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