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Reinforced concrete jacketing of existing structures

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ABSTRACT: One of the techniques widely used for the repair and retrofit of structures in Mexico City, after the 1985 earthquakes, was the jacketing of several of their elements with reinforced concrete. In this paper, quantitative and detailing recommendations as well as general criteria for the use of concrete jacketing are discussed. Emphasis is made on the connection between the existing elements and the new materials added to the structure.

1 INTRODUCTION

The jacketing of a structure main objective is to increase the seismic capacity of the structure. Depending on the type of jacketing used, an increase in strength and stiffness, ductility, or a combination of them can be obtained.

There are several options for the jacketing of concrete members. Usually, the existing member is wrapped with a jacket of concrete reinforced with longitudinal steel and ties, or with welded wire fabric. In Mexico City, concrete is usually cast in place by means of special formwork and windows bored through the slab. In this sense, the design process not only needs to be technically sound, but has to take into account the cost of the formwork and the feasibility of the casting process. In Mexican practice, shotcrete has been seldomly used.

In Mexico City, jacketing was used mainly in moment resisting framed structures. In almost every case, the columns as well as the beams of the existing structure were jacketed. In structures with waffle slab, the increase in stiffness obtained by jacketing the columns and some of the ribs was usually insufficient, and thus, shear walls were used in this type of structures. In some cases, foundation grids had been strengthened and stiffened by jacketing their beams.

2 GENERAL GUIDELINES

Reinforced concrete jacketing can be used as a repair or strenghtening scheme. If there is damage in some of the existing members, they should be repaired prior to their jacketing. Techinques for the repair of concrete members have been reported by several authors, including Iglesias (1985) and UNDP/UNIDO proyect RER/79/015 (1983).

For Mexican practice, the design lateral forces for any structure can be computed from inelastic smoothed pseudoacceleration spectra obtained by reducing the corresponding elastic spectra by an appropiate factor Q (which is equivalent to a global displacement ductility ratio). For ductile moment resisting frames, Q has a value of 4. Because of the uncertainties related to the behavior of jacketed elements under earthquake loading, as well as the impossibility, in the majority of the cases, to provide an adequate detailing to the jacket reinforcement (specially in the joint region); the use of a lower value of Q, than that corresponding to new construction, is recommended for the design of the jacket scheme. Values of Q ranging from 2 to 3 have been used in mexican practice.

As with any other strengthening scheme, the design of the jackets should include the probable redistribution of loads in the structure; possible changes in its failure mechanism; and a change in the dynamic properties of the structure that could lead to changes in the lateral forces induced by the earthquake.

The properties of the jackets concrete should match those established for the concrete in the existing structure. The compressive strength of the new concrete (f'_c) should be greater than that of the existing structure by 50 kg/cm², as recommended by UNDP/UNIDO (1983) and at least equal to that of the existing structure, as recommended by Bass, et. al. (1985).

3. STRUCTURAL ANALYSIS

To determine the design forces in the members of the structure, an elastic analysis of the structures can be performed. Uncertainty exists on how loads distribute between existing members and the jackets. An analysis assuming a monolitic behavior of the jacket and existing member (if the shear transfer mechanism between them is carefully assessed) provides reasonable results for design. For the computation of the geometric properties of the members, the method of equivalent transformed section can be used. If the existing element had previous damage, it is recommended to ignore its contribution to the jacketed element stifness. Because the jacketing of some elements reduce considerably their span/depth ratio, it is important to consider shear effects on the deformations of the concrete jacketed members, to assess correctly the distribution of lateral forces between them.

4. SHEAR STRESSES IN THE INTERFACE

To guarantee a monolithic behavior of a jacketed element, it is mandatory to have an adequate shear transfer mechanism in the contact area (interface) between the jacket and the existing element, so that a relative movement between both concretes is prevented. For this purpose, several approaches can be used. For elements with a four side jacket, as shown in figure 1a, 1b and 1c, the ties used to confine and provide shear reinforcement to the composite element are usually enough to provide an adequate shear transfer at the interface. But, in many real cases in Mexico City, due to the limitations found in the existing structure (e.g. lack of space), structural engineers have used 1, 2 and 3 sided jackets as well, as shown in figures 1d, 1e and 1f. In the latter cases, special reinforcement should be provided to enhance a monolitic behavior. In some cases, ties have been used, as shown in figure 1d and 1e. In other, steel connectors (usually rebar #3 and #4 anchored with epoxy) have been used, as shown in figure 1f. For proper shear stress transfer on composite beams, the new version of the Mexico code (Reglamento de Construcciones para el D.F., 1987) specifies the following (similar approach than ACI 318-83, section 17.5.1 for composite concrete flexural members design):

The value of the resiting shear stress, v_r, can be obtained as:

a. v_r equal 3 kg/cm² if no connectors are used.

b. v_r equal to 25 kg/cm² if steel connectors consisting on bars or ties perpendicular to the plane of contact are used. The minimum area of this reinforcement its equal to $3A/f_y$, where A is the area of contact in cm² and f_y is in kg/cm². Their spacing should not exceed six times the width of the new element (the jacket in this case) nor 60 cm. The connectors should be anchored in both concretes in such a way that they can develop at least 80% of their yielding stress.

c. If v_r exceeds 25 kg/cm², a shear friction approach can be used.

The values of v_r described above are only applicable if the the contact surface between both concretes has been intentionally roughened to a full amplitude of approximately 5 mm and cleaned from dirt and grease, and freed of laitance. It is not recommendable to use a smooth surface at the area of contact between both concretes. The Mexican code uses the values of v_r on a simplified way (ignoring the location of the interface within the composite member depth) to estimate the maximum shear force that a composite concrete beam can carry:

$$V_u \leq \phi\, V_r = \phi\, v_r\, b_v d \qquad (1)$$

where V_u is the factored shear force at the section, ϕ the strength reduction factor, b_v the width of the cross section at the contact plane, and d the distance from extreme compression fiber to centroid of tension reinforcement for entire composite section.

In Mexico City, engineers used all type of geometries for the jacketed elements, and in some cases the interface was located at or near the plane of maximum shear stresses (unlike normal composite elements), as shown in figure 1d. To account for this fact, besides satisfying equation (1), it is recommended that the ultimate shear stresses acting at the contact plane are computed assuming an elastic approach:

$$v_u = \frac{V_u\, Q}{I_x\, b_v} \qquad (2)$$

and that the value of v_u obtained with equation (2) satisfies the following requirement:

$$v_u \leq \phi\, v_r \qquad (3)$$

In equation (2), Q is the static moment of area and I_x the moment of inertia of the section.

It is recommended, if possible, that V_u is computed as the sum of the maximum resiting moments at the member's ends divided by its span and multiplied by 1.25, to make sure the composite member con reach its ultimate flexural capacity.

Altough an elastic approach can't be considered correct, because of the nonlinear behavior of reinforced concrete, it is very important to consider the variation of shear stresses along the depth of the composite element, and the location of the interface within this depth. Any other rational procedure can be used to estimate and accomodate the total shear acting in the interface.

The steel connectors should be distributed uniformely around the interface, avoiding concentrations in specific locations. Mexican engineers usually used rebars anchored with epoxy resins or grouts. A better behavior in tension has been observed in this type of connectors as compared with that of wedge-type anchors. Luke, et. al. (1985) discuss several options for the preparation and anchoring of rebar connectors using epoxy compounds and assess them experimentally. It is recommendable that once a hole has been bored in the existing concrete element (to introduce the rebar), the hole be cleaned from dirt and dust. The hole diameter should be about 6 mm larger than that of the rebar, in such a way as to avoid creep potential on the epoxy resin or grout, as well as to allow a good distribution of this material around the rebar. It is very important to have a strict field inspection program to confirm the capacity of the connectors in the structure.

In Mexican practice, this type of connectors usually didn't satisfy the anchor length required by the code. In cases where due to space limitations, not enough anchorage can be provided, it seems reasonable to increase the area of connectors, A_p, as follows:

$$A_p = A_R \cdot l_R / l_p \qquad (4)$$

where A_R is the area of connectors required by the code, l_R the embedment length required by the code, and l_p the provided embedment length. It is extremely important to provide a minimum embedment length to have a predictable behavior of the connectors. Warner (1980), mentions that epoxy set dowels properly installed will retain their full yield capacity when embedded approximately 10 times their diameter; altough he recommends to provide at least 15 bar diameter embedment. Bass, et. al (1985) assessed experimentally the interface shear capacity of concrete surfaces using epoxy bonded rebars. They conclude that in the tested specimens, embedments of 1/3 to 1/6 of the required development length produced interface shear strengths in excess of the ACI 318-83 shear friction design strength values. Thus, it is not necesary to provide large embedments to obtain satisfactory behavior, moderate embedment lengths of epoxy bonded rebars provide adequate shear transfer across the interface.

5.- COLUMN JACKETING

If possible, a four sided jacket should be used. For its design, a monolitic behavior of the composite columns can be assumed. The minimum width of the jacket should be 10 cm for concrete cast in place and 4 cm for shotcrete, as recommended by UNDP/UNIDO (1983). Two main types of column jacketing have been used, as shown in figure 2. The type illustrated in figure 2a is used to increase the shear capacity of the column, and thus, try to accomplish a strong column-weak beam design. Sugano (1980) recommends the use of a narrow gap to prevent any possible increase in flexural capacity. The second type, where the longitudinal steel of the jacket is made continuous through the slab system, as shown in figure 2b, and carefully anchored to the foundation, was widely used in Mexico, simultaneously with the jacketing of the beams. Because of the existence of the beams, the longitudinal reinforcement usually was concentrated in the column corners, where bar bundles were used as shown in figure 1a. It is recommended that no more than 3 bars are bundled together. Windows are usually bored through the slab to allow the steel to go through, as well as to enable the concrete casting process. Figure 1b and 1c show options for the detailing of the longitudinal reinforcement to avoid the excesive use of bundles.

The percentage of steel of the jacket with respect to the jacket area should be limited between 0.015 and 0.04, and at least, a #5 bar should be used at every corner for a four sided jacket, as recommended by UNDP/UNIDO (1983).

Shear reinforcement should be designed and spaced according to earthquake design practice, altough it is suggested the minimum bar diameter used for ties is no less than 9.5 mm (#3 bar) or 1/3 the diameter of the biggest longitudinal bar. The ties should have 135° hooks with 10 bar diameters of anchorage. In Mexico City, due to the difficulty of manufacturing 135° hooks on the field, 90° hooks were provided to the ties of several structures. To avoid this problem, ties made of multiple pieces, as shown in figure 1, can be used.

Another aspect that has been observed in Mexico City is the significative change of shear span/depth (a/d) ratio of existing columns once they are jacketed. In several jacketing schemes used in framed buildings in Mexico City, where typical story height is 3 m, columns' a/d ratio (computed assuming an inflection point exists along the height of the member) would be typically reduced to values less than two, and in some cases, to ratios less than 1.5. A column with this a/d ratio is very likely to change from a flexural behavior to a shear dominated one. This type of column, subjected to a double curvature deformation, would behave very similar to a deep beam coupling two shear walls: shearing forces and consequent diagonal cracking is likely to cause radical redistribution of tensile forces along the flexural reinforcement. Due to the effects of diagonal tension, members with a/d ratios less than about two have tensile stresses acting along the entire length of their longitudinal reinforcement, even at locations where conventional flexural theory would predict compressive stresses. This have been observed by Paulay (1971) in deep beams, and by Bett, et.al. (1985) in jacketed short columns. For a column with continuous longitudinal steel, figure 2b, the conventional design guidelines for flexural concrete elements could be invalidated, because both tension and "compression" steel could be in tension at a critical section. Thus, the interaction of flexure and shear in this type of elements causes a reduction in the flexural capacity. The inelastic behavior of such members is likely to be strongly affected by shear effects, and thus, their energy dissipation capacity will be diminished.

The lack of space in the structure makes it very difficult to provide midface longitudinal bars, as shown in figure 3, or supplementary crossties to confine the concrete on this portion of the column. Bett, et.al.(1985) found that although this supplementary longitudinal and transverse reinforcement did not have a significant effect on the monotonic stiffness and strength of jacketed short columns for small drifts, they were benefitial in controlling the strength and stiffness degradation under repeated cycles of reversed displacements exceeding 2% drift, where the column worked within its inelastic range of behavior.

Although it can not be asserted that a jacketed column would have a double curvature deformation during an earthquake, the above discussion has shown that its design is not an easy task, and its behavior uncertain. The one aspect that should be outlined is that deep concrete elements are not likely to behave adequately in the inelastic range when not detailed properly (as in the case for the majority of jacketed columns in real structures). This points to the fact that when a framed structure is jacketed, energy dissipation should be concentrated at the beams, while the columns should remain elastic or have limited inelastic demands. Even for cases where the a/d ratio are not reduced to values observed in Mexican practice, the impossibility to provide adequate detailing for inelastic behavior of the jacketed columns leads to a strong column-weak beam design.

6. BEAM JACKETING

In some countries, one side jackets had been used to increase the positive moment capacity of beams, as shown in figure 4. Mexican practice used 3 and 4 side jackets as shown in figure 5. In several occasions, the slab was perforated to allow the ties to go through and to enable the casting of the concrete. The beam should be jacketed throughout its whole length. A minimum width for the jacket of 8 cm must be used if concrete is cast in place or 4 cm for shotcrete, as recommended by UNDP/UNIDO (1983).

It is recommendable to jacket the beams for several purposes: give continuity to the columns jacket, increase the strength of the structure, increase the stiffness of the structure, and try to induce a double curvature behavior of the columns (which, in the majority of the cases, would reduce the ultimate moments of the columns in the bottom stories as compared with those on a structure with non rigid beams). The longitudinal reinforcement should be continuous throughout the different spans. In Mexico City, engineers used mainly two options to attain such continuity, as shown in figure 6. If the longitudinal steel is opened to avoid the existing column, as shown in figure 6a, ties should be provided to avoid tensile failure of the concrete, as shown in the same figure. Also, a reduction in the jacketed beam stiffness should be considered when using this option. A slope of 6 to 1 seems an upper bound for the opening of the longitudinal steel. When jacketing a beam, its flexural resistance must be carefully computed to avoid the creation of a strong beam-weak column system. When the longitudinal reinforcement of the existing element is not known, the percentage of steel on the jacket should be limited to 50% of the total area of the composite section. For shear, the contribution of the existing ties can be ignored if its amount and distribution is ignored. The ties used for the jacket should be closed and have 135° hooks. In each corner of the tie there must be at least a longitudinal bar. The bars used for the tie should have at least 7.9 mm diamater (# 2.5). Multiple piece ties can be used, as discussed before for columns.

Mexican typical spans for beams range from 6 to 7 m. In many cases, the depth of the jacketed beams is limited by the story height, typically 3 m. This two facts limit the reduction of their a/d ratio, and thus, jacketed beams can be designed for ductile behavior.

7. JOINT JACKETING

The joint is defined as that part of the column located through the depth of the beams that intersect that column. It is very important to provide this critical region with enough confinement and shear capacity. This is one of the most difficult tasks in the jacketing of a framed structure. Due to a lack of space in the joint region, it is almost impossible to provide an adequate confinement. If possible, ties should be provided in this region, with a separation equal to that at the columns ends. Alcocer, et. al., (1990) assessed experimentally the behavior of several beam-columns subassemblages, where the joint was confined with a steel cage as shown in figure 7. They found that the above described joint reinforcement provides enough confinement and shear capacity to the joint. The sizes of the steel elements that constitute the cage are indicated in figure 7. In these specimens, the dissipation of energy was mainly concentrated at the beams' ends. Again, it is very import to point out the need to have a very strong column as compared to the beam to avoid driving the column or the joint into significant inelastic behavior.

8. CONCLUSIONS

After an extensive bibliographical study and the revision of several mexican design projects for the retrofit of structures, it can be concluded that there are some outlines that help, not only in a qualitative but in a quantitative manner, the design of a retrofit scheme of a structure by means of concrete jacketing.

Altough these guidelines can give a rational basis for a practical design, research still need to address critical aspects in the behavior of jacketed elements. The change in behavior in jacketed elements whose shear span/depth ratios are significantly reduced, due to their jacketing, needs to be clarified. Conventional design guidelines for flexural concrete elements can give unconservative results when applied to the design of such members. Experimental research needs to address this issue, including the study of biaxial bending efects on the behavior of wide columns.

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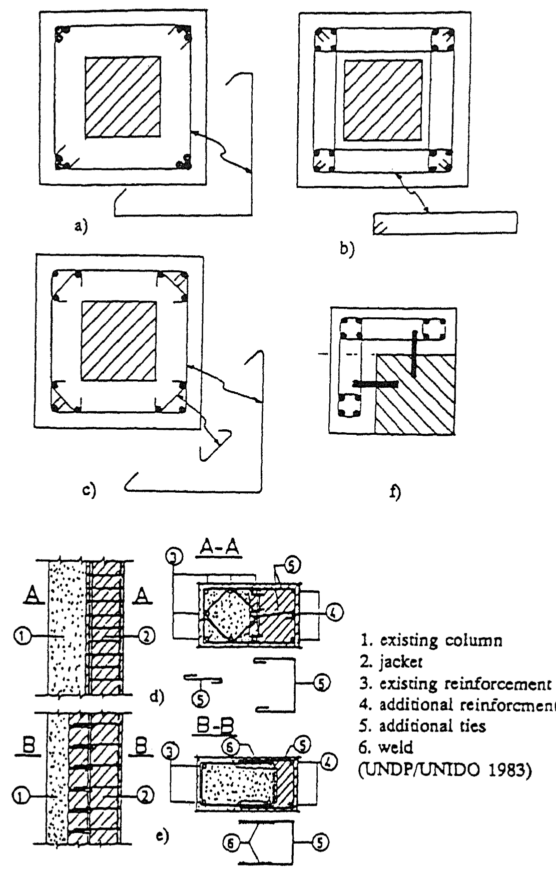

Figure 1. Column jackets.

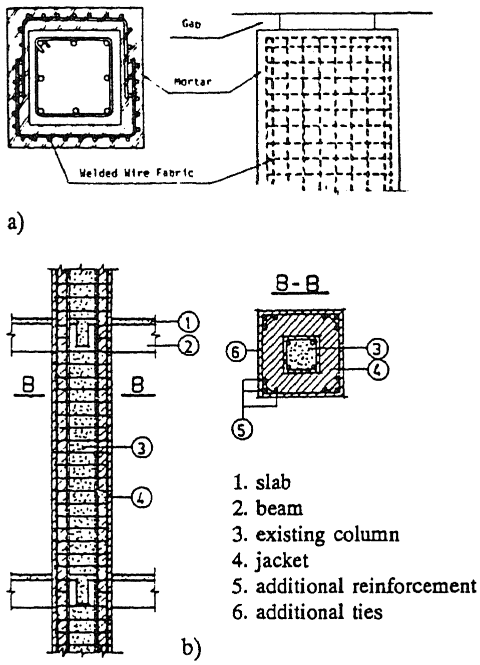

Figure 2. Types of column jacket
(Sugano 1980

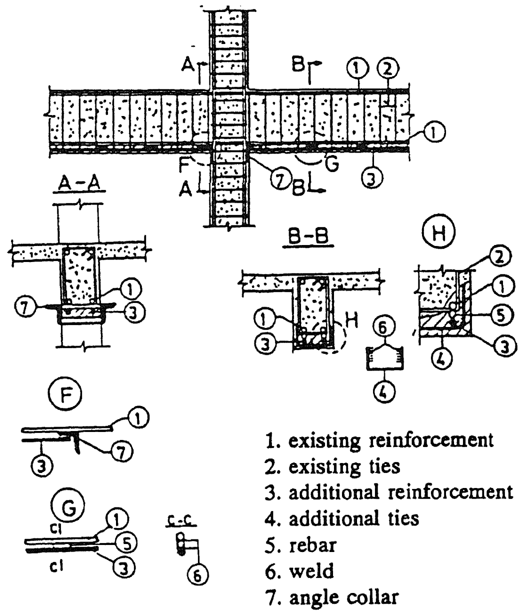

Figure 4. One side jacketing of beams
(UNDP/UNIDO 1983)

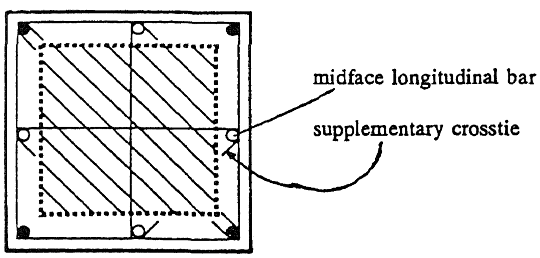

Figure 3. Midface longitudinal bars and supplementary cross ties in columns

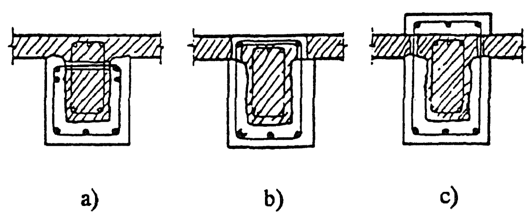

Figure 5. Three and four sides jacketing of beams
(Iglesias 1985)

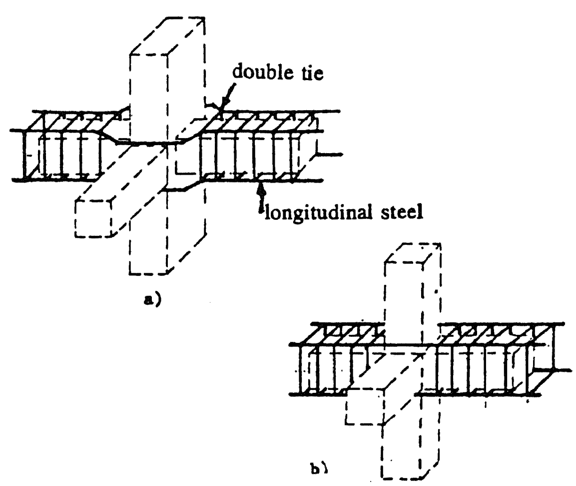

Figure 6. Continuity of longitudinal steel in jacketed beams

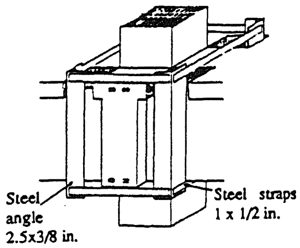

Figure 7. Structural steel cage assembled in the joint (Alcocer 1990)