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JACKETING OF REINFORCED CONCRETE MEMBERS

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ABSTRACT

Tests were carried out to study the behavior of jacketed reinforced concrete beams, similar to those used for upgrading buildings damaged by earthquakes. Four different conditions of the contact surface of the beams and the jackets were used. Shear stresses in the interface were computed from the tests results with methods presented in the building codes and compared with those recommended in the same codes. Some differences were found and recommendations are presented to review specified values of those shear stresses.

KEYWORDS

Strengthtening; jacketing; reinforced concrete; damaged buildings; shear stresses; dowels.

INTRODUCTION

After the 1985 Mexico City earthquake, many buildings are being upgraded by jacketing the structural members with reinforced concrete in one or more sides. A common hypothesis is that the strength of a jacketed member is equal to the strength of a monolithic member, but for this hypothesis to be true, the interface of the original member and the jacket must be able to transmit the shear stress developed under the loads acting on the structure. A common practice is to design a jacketed member as a composite beam and check the shear stresses in the interface according to the recomendations of the Mexico City Building Code, which are in this respect similar to the ACI Building Code recommendations. The horizontal shear stresses are computed with the equation $v = V/bd$, and these stresses are compared with allowable stresses depending of the roughness and cleareness of the interface and the existence of shear connectors or dowels (Section 17.5.2 of ACI 318-89). The ACI Building Code includes another method to calculate the horizontal shear stresseses in a composite beam: compute these stresses by dividing the actual change in compressive or tensile force in any segment between the area of the contact surface A_c .

Since numerous buildings have been reinforced with jackets, most of them in difficult construction conditions, it was considered important to check the strength of members obtained according to the mentioned hypothesis against the actual strength of test specimens.

EXPERIMENTAL PROGRAM

Three series of tests have been carried out. The first one included simple supported beams with a concentrated load at midspan. One test is a monolithic beam and the others are beams with a jacket cast in the upper side, so that the total cross section is the same in all the beams, Fig. 1. Four different conditions of the interface were tested:

a) clean and intentionally roughened,
b) clean, intentionally roughened and with steel dowels,
c) clean, not roughened and with steel dowels, and
d) clean and not roughened (for comparison purposes).

Five specimens for each condition were tested. A description of these tests is presented by Guerrero *et al*,1993.

The second series consists of beams similar to those of the first series, but with a greater span, 170 cm for series No. 1 and 200 cm for series No. 2. One more bar was placed in the tension side of these beams in order to obtain a greater flexural strength corresponding to a greater flexural moment due to the longer span. Three specimens for each condition were tested (Guerrero *et al*, 1994*a*).

In the third series, specimens consisting of three concrete blocks were tested as shown in figure 2. The contact surface between the central block and the lateral blocks had the same four conditions of the contact surface between the beams and the jackets as in series 1 and 2 (Guerrero *et al* al, 1994*b*).

In general, the contact surfaces were prepared according to the recommendations of the building codes, but an important difference existed in the embedment length of the dowels into the concrete. The building codes specify that the development length must be provided at both sides of the contact surface. However, in practical cases it is sometimes impossible to provide this length because the jacket depth is frequently shorter than the development length. In a previous work (Terán, 1989), it was concluded that a shorter embedment length can be used if the provided area of the dowels is larger than the required area.

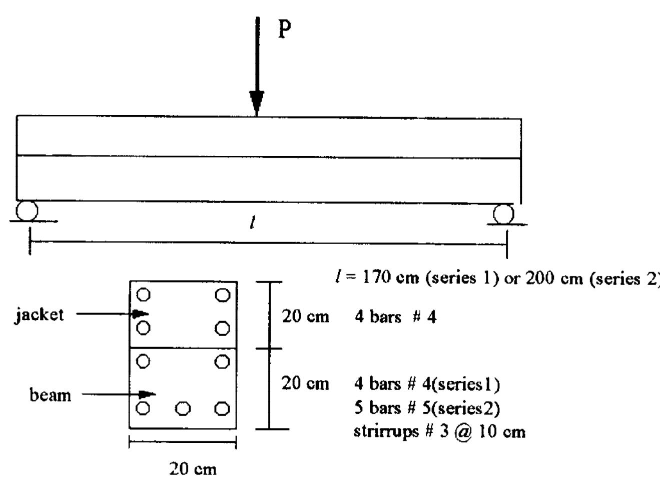

Fig.1 Test specimen for series 1 and 2

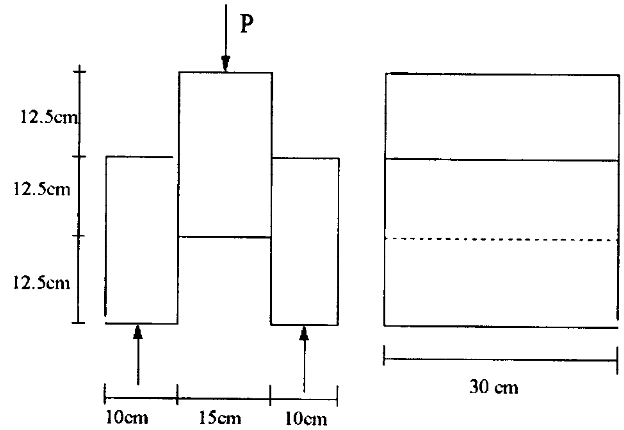

Fig.2 Test specimen for series 3

The following equation was obtained to calculate the provided area of dowels:

$$A_{vf} = A_{vfn} \frac{l_d}{l_a} \qquad (1)$$

where:
A_{vf} = area of dowels provided in the contact surface
A_{vfn} = area of dowels required by building codes specifications
l_d = required development length
l_a = provided length

The dowels in the three series were made of #3 bars which require a development length l_d = 30 cm. The building codes permit to use an embedment length of $0.8l_d$ in composite members, so the #3 bars require 24 cm. In the beams of series 1 and 2 the embedment length was 12 cm and the provided area was twice the required area. In the specimens of series 3, two different patterns of dowels were used. One pattern consisted of 10 cm long dowels and the other of 8 cm long dowels. The provided area was adjusted in each case according to equation 1.

EXPERIMENTAL RESULTS

The behavior of beams of series 1 corresponding to the monolithic specimens and composite specimens with contact surface conditions a), b) and c) were similar. In a load-deformation graph, a linear stage was defined for small loads, then an inelastic stage up to the yield of the reinforcement and finally an almost horizontal stage with large deformations. The crack pattern was similar to that observed in the tests of deep beams, with vertical cracks in the center of the beams and inclined cracks from near the supports to near the point load. Failure ocurred by crushing of the concrete in the compression zone after yielding of the reinforcement. The failures were typical flexural failures, very similar in the monolithic and the composite beams. The behavior of the beams with surface condition d), clean and not roughened, was different. When the flexural and diagonal cracks were still very thin, a sudden separation of the beam and the jacket ocurred along the contact surface. The deformations of the beam were small at this moment, so failure was of the brittle type. It must be remembered that this condition of the contact surface is not permitted in the building codes, and it was included in these tests for comparison of the results with those of the other surface conditions.

In the beams of series 2, a different behavior was observed in beams with dowels from that of beams without them. In the monolithic beams and in the beams with surface conditions b) and c), a ductile behavior was observed with large flexural cracks and large deformations at failure. Since the span of the beams in this series was longer than the span in series 1, the diagonal cracks typical of deep beams were not developed. However, in the specimens with surface condition a), the specimens split into the beam and the jacket at small deformations, before yielding of the flexural steel. It seems that the deep beam behavior avoided this type of failure in the specimens of series 1 with this type of surface condition. Of course, in the specimens with surface condition d), the jacket was also separated from the beam at small deformations.

In the specimens of series 3, the central block sheared from the lateral blocks at small deformations. The failures were more brittle in specimens without dowels. The 10 cm long dowels supported bending deformations but they were not pulled out the concrete. In the specimens with 8 cm long dowels, the failure ocurred at higher loads than in the specimens with 10 cm long dowels. It appears that more dowels are better than few dowels although their length is shorter. However, some limits must be determined for equation 1 since l_a can not be too small even if A_{vf} is increased.

COMPARISON OF ANALYTICAL AND EXPERIMENTAL RESULTS

Moment-curvature relationships were computed for the two cross sections using the strees-strain curve for local reinforcing bars determined by Rodríguez, 1994, the stress-strain curve for concrete recommended by Kent, 1971, to take into account the effect of concrete confinement, and the actual position of the reinforcing bars as shown in figure 3. From these moment - curvature relationships, the moment at yielding of the reinforcement bars near the bottom of the beams, and the maximum moment were determined. The moment at yielding was compared with the moment corresponding to the beggining of large deformations in the experimental load - deformation graphs, and the maximum moment, with the experimental flexural strength. These comparisons are shown in table 1. Each test value is the average of 5 specimens for series 1 and 3 specimens for series 3.

For series 1, the experimental yield moment values and the experimental maximum moments resulted significantly greater than the computed values. This ocurred even for contact surface condition d) that failed by splitting in the interface, although the maximum moment for this condition was lower than for the other conditions. It is possible that a deep beam effect was the cause of these differences, although the beams are not classified as deep beams according to the definitions of the ACI or Mexico City building codes.

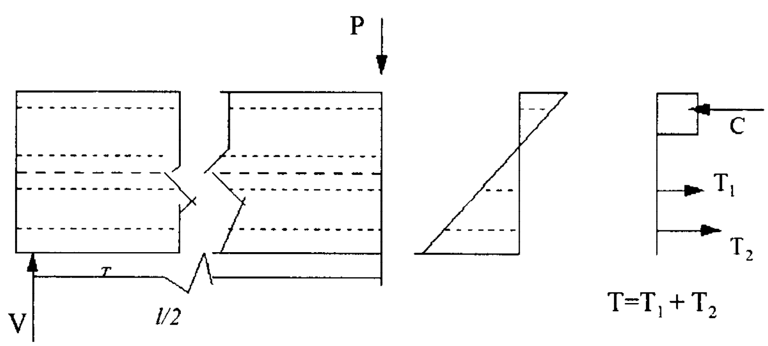

Fig. 3 Computation of tensile force to be transmitted

Table 1. Computed and tests moments for series 1 and 2

Series	Surface condition	$M_{yielding}$ (computed) ton.m (kN.m)	$M_{yielding}$ (test) ton.m (kN.m)	M_{max} (computed) ton.m (kN.m)	M_{max} (test) ton.m (kN.m)
1	monolithic	6.80 (66.68)	9.77 (95.81)	9.80 (96.10)	13.38 (131.2)
1	a) clean and roughened	6.80 (66.68)	8.50 (83.36)	9.80 (96.10)	12.15 (119.1)
1	b) clean, roughened and dowels	6.80 (66.68)	8.92 (87.47)	9.80 (96.10)	14.02 (137.5)
1	c) clean, not roughned and dowels	6.80 (66.68)	8.07 (79.14)	9.80 (96.10)	12.92 (126.7)
1	d) clean and not roughened	6.80 (66.68)	7.65 (75.02)	9.80 (96.10)	10.62 (104.1)
2	a) clean and roughened	8.50 (83.36)	8.50 (83.36)	11.70 (114.7)	10.00 (98.1)
2	b) clean, roughened and dowels	8.50 (83.36)	9.00 (88.26)	11.70 (114.7)	13.50 (137.4)
2	c) clean, not roughned and dowels	8.50 (83.36)	9.00 (88.26)	11.70 (114.7)	13.50 (137.4)
2	d) clean and not roughened	8.50 (83.36)	9.25 (90.71)	11.70 (114.7)	10.00 (98.1)

For series 2, there is a good correlation between computed and experimental yielding moments. The experimental maximum moments were greater than the computed for surface conditions b) and c), and lower for surface conditions a) and c). The specimens with these last surface condition failed by splitting in the interface before failing by flexure.

The beams were designed to have a vertical shear strength well in excess of the flexural strength. So, although some shear craks appeared in the tests, the shear force at failure was much lower than the theoretical shear strength.

As it was said in the Introduction, the ACI Building Code presents two methods to calculate the horizontal shear stresses. In the first one, the stresses are calculated with the equation (without the strength reduction factor):

$$v_1 = \frac{V}{bd} \tag{2}$$

where:
v_1 = horizontal shear stress according to first method
V = shear force
b = beam width
d = beam depth

In the second method, the "horizontal shear may be investigated by computing the actual change in compressive or tensile force in any segment, and provisions made to transfer that force as horizontal shear to the supporting element". Applying this concept to the tests beams, the change in force to be transfered from the beam to the jacket is equal to the tensile force in the reinforcement bars at failure of the beams in the center of the span, figure 3, minus the tensile force at the supports, that is equal to zero. So, the shear stresses can be computed from the equation:

$$v_2 = \frac{T}{b\left(\frac{l}{2}\right)} \tag{3}$$

where:
v_2 = horizontal shear stress according to second method
T = tensile force to be transfered
l = center to center span

Table 2. Shear stresses for series 1 and 2

Series	Surface condition	v_1 kg/cm^2 (MPa)	v_2 kg/cm^2 (MPa)	v building codes kg/cm^2 (MPa)
1	a) clean and roughened	21.66 (2.12)	18.82 (1.85)	3 (0.29)
1	b) clean, roughened and dowels	25.00 (2.45)	18.82 (1.85)	25 (2.45)
1	c) clean, not roughned and dowels	25.00 (2.45)	18.82 (1.85)	6 (0.59)
1	d) clean and not roughened	20.00 (1.96)	18.82 (1.85)	-
2	a) clean and roughened	18.33 (1.80)	18.67 (1.83)	3 (0.29)
2	b) clean, roughened and dowels	23.33 (2.29)	18.67 (1.83)	25 (2.45)
2	c) clean, not roughned and dowels	21.66 (2.12)	18.67 (1.83)	6 (0.59)
2	d) clean and not roughened	16.66 (1.63)	18.67 (1.83)	-

The denominator of equation 3 represents the area of the contact surface. from the center of the span to the support.

The shear stresses v_1 and v_2 corresponding to these two methods are presented in Table 2. The first one was computed from the measured shear force V=P/2 using equation 2. The second one was obtained from equation 3, computing the tensile force T from the equilibrium of the cross section of the beam, as shown in figure 3, for the strain distribution corresponding to flexural failure, using the same hypothesis above mentioned for the moment - curvature relationships. It must be observed that v_1 is a value obtained from the experimental value of the force P, while v_2 is a computed value. Since experimental maximum moments are greater than computed moments, except for contact surface conditions b) an c) of series 2, the actual value of the tensile force T must be greater than the computed value, and the actual values of v_2 must be greater than those of Table 2.

It can be seen in Table 2 that important differences exist between shear stresses computed with the two methods presented in the ACI Building Code. Those computed with the first method are larger in most cases than those computed with the second one. It seems that the latter represents better the behavior of jacketed beams as those tested in this research.

Stresses v_1 and v_2 are also compared in Table 2 with stresses specified in the Mexico City and in the ACI building codes for different contact surface conditions. It can be seen that building codes values are much lower than computed values from these tests, except for contact surface condition b). Even considering the value of v_2 as a minimum value, the difference between the experimental and the computed values of maximum moments for this contact surface condition is lower than the difference between computed and building codes values. It is also important to point out that the difference between building codes values for surface conditions b) and c), from 25 to 6 kg/cm^2 (2.45 to 0.59 MPa), appears too large to be justified only on the basis of a roughened surface.

The experimental values obtained in series 3 are presented in Table 3. In general, they are lower than the values obtained in the beam tests. It seems that the test specimen used in this series, although very simple, does not represent adequately the behavior of the interface in composite beams. The vertical stresses that must exist in the interface are favourable to the behavior of beams and they are not present in test specimens as those of series 3.

Table 3. Shear stresses for series 3

Surface condition	V_{test} kg/cm^2 (MPa)	$V_{building\ codes}$ kg/cm^2
a) clean and roughened	5.7 (0.56)	3 (0.29)
b) clean, roughened and 10 cm dowels	17.8 (1.75)	25 (2.45)
c) clean, not roughened and 10 cm dowels	9.0 (0.88)	6 (0.59)
c') clean, not roughened and 8 cm dowels	12.7 (1.25)	6 (0.59)
d)clean and not roughened	3.2 (0.31)	-

CONCLUSIONS

Reinforced concrete beams jacketed with reinforced concrete can attain the same flexural strength and ductility than monolithic beams with the same cross section and reinforcement when the jacket is designed according to the specifications of ACI or Mexico City building codes.

When the interface of the original beam and the jacket is clean and roughened, the shear stresses obtained from tests results were much greater than those specified in the building codes. However, it is recommended to use dowels in the interface whenever possible, since a more ductile behavior is attained.

When the interface is clean, not roughened and with dowels, shear stresses obtained from tests results were also much greater than specified shear stresses. It seems that the specified value of 6 kg/cm^2 (0.59 MPa)for this contact surface condition is too conservative. The difference between this value and the corresponding to interfaces clean, roughened and with dowels, 25 kg/cm^2 (2.45 MPa), is too large according to the results of these tests.

The shear stresses obtained from beam tests results for interfaces clean, roughened and with dowels were always lower than specified shear stresses. As the beams failed by flexure, it is not possible to assure from the testing of these beams that the specified shear stresses are too high. However, the specimens of series 3 with this interface condition also failed at shear stresses lower than those specified. It is recommended to check the corresponding value.

Shear stresses obtained from direct shear tests, as those of series 3 in this research, do not correspond to stresses obtained in beam tests. It seems that the state of stresses is more favourable in beams. So, it is recommended to specify shear stresses for jacketing purposes on the base of beam tests.

Mexico City Building Code does not include a method similar to the second method of the ACI Building Code for computing horizontal shear stresses in composite members. Since this method seems more appropiate for jacketed beams than the first method, it is recommended to include it in the Mexico City Building Code.

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