11 WCEE

Copyright © 1996 Elsevier Science Ltd
Paper No. 1736. (quote when citing this article)
Eleventh World Conference on Earthquake Engineering
ISBN: 0 08 042822 3

BOND REQUIREMENTS IN INTERIOR R/C BEAM-COLUMN JOINTS WITH RESTRICTED DUCTILITY

A. P. GIULIANO, J. A. AMADO, M. A. LEIVA, R. D. GARCÍA and E. A. BARROS

National Institute of Seismic Prevention
Roger Balet 47 Norte, 5400, San Juan, Argentina

ABSTRACT

Six large scale specimens were tested representing interior joints of a medium rise building. The specimens were grouped into two series, one corresponding to the upper part of the building, and the other to the lower one. The beam steel bar diameters were progressively increased, keeping constant the moment capacity in each series, to represent more adverse situations in bond requirements. The global behavior of the specimens was studied by means of the hysteresis loops, strength and stiffness degradation, cumulative hysteretic energy and equivalent damping. The results obtained for each specimen were compared with that corresponding to one reference specimen in each series designed according to the earthquake-resistant regulations presently enforced in Argentina. It can be concluded that in structures with restricted ductility, bond requirements of the longitudinal beam bars passing through an interior joint, according to the earthquake-resistant regulations presently enforced in Argentina, may be relaxed.

KEYWORDS

Joints; restricted ductility; bond requirements

INTRODUCTION

The requirements to control bond failures in reinforced concrete beam-column joints according to the earthquake-resistant regulations presently enforced in Argentina (INPRES, 1991), present undue construction problems due to steel congestion. These requirements limit the diameters of beam and column bars passing through the joint, and are based on results from tests of subassemblages expected to develop full ductility. In many situations, however, earthquakes are unlikely to impose significant ductility demands on certain types of structures, as is the case of low to medium-rise reinforced concrete framed buildings commonly employed in the most seismically active regions of Argentina. In view of this fact, it was decided to carry out an experimental research project in the Structure Laboratory of the National Institute of Seismic Prevention to evaluate the effect of bond deterioration in the hysteretic response of beam-column joints subassemblages with restricted ductility under quasi-static loading. The National Department of Science and Technology supported the project

TEST PROGRAM

Six large scale specimens grouped into two series of three specimens each were tested. The specimens represented interior joints of medium rise framed buildings commonly used in the most hazardous seismic zone of Argentina.

The first series corresponded to the upper level of the building, and the second to the lower one. The three specimens of each series were designed maintaining the same moment capacity under the strong column-weak beam concept. The beam bar diameters were progressively increased to develop more adverse situations in bond requirements.

In each series, a reference specimen was included, designed according to the earthquake-resistant regulations presently enforced (INPRES, 1991) and corresponding to the most favorable bond conditions likely to be obtained in current practice in Argentina.

The specimens in series 1 were labeled N1, N2, N3; N1 being the reference, and those in series 2; N4, N5, N6; N4 being the corresponding reference specimen. Details of the specimens are shown in Fig. 1.

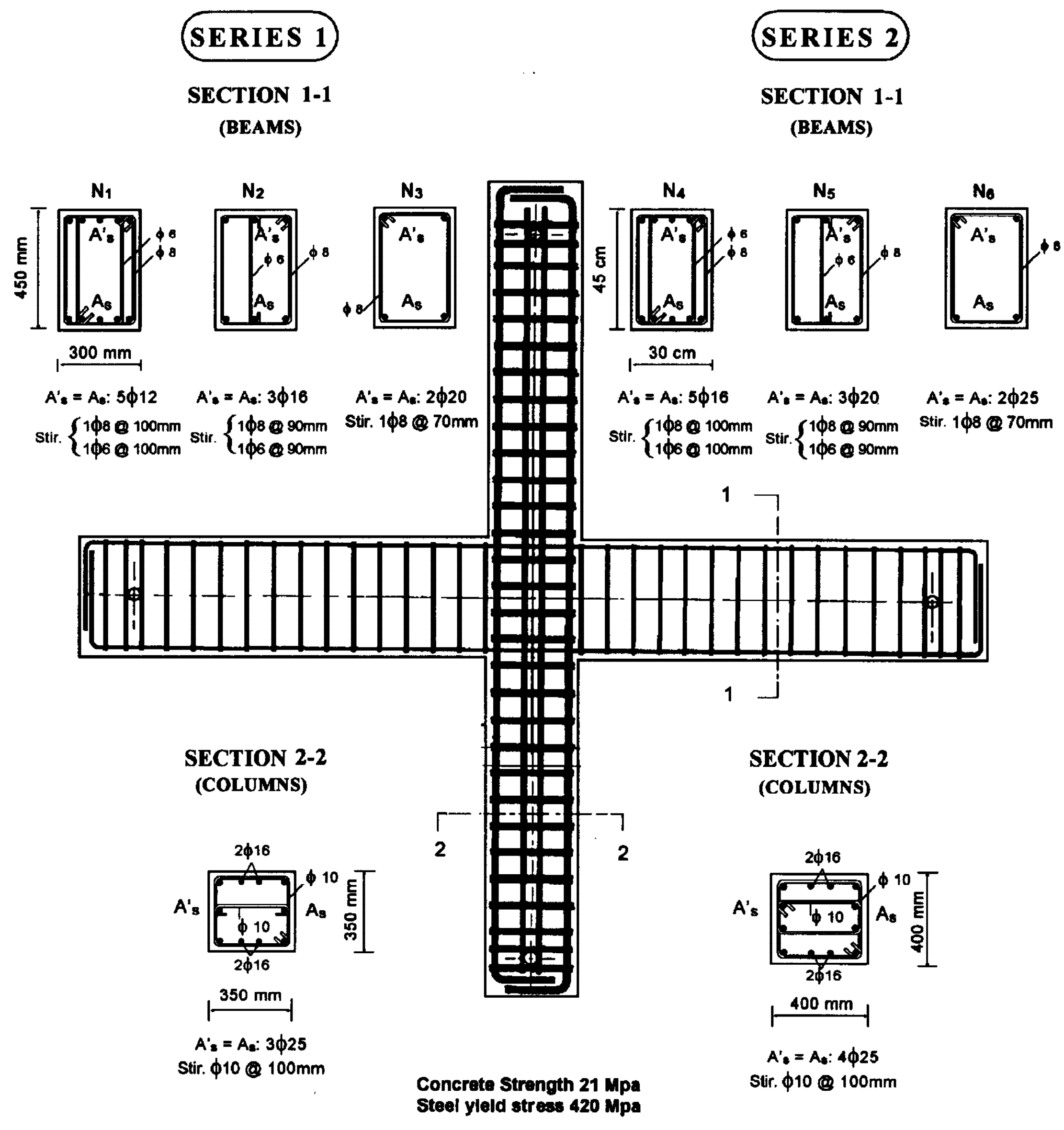

Fig. 1: Details of the specimens in series 1 and 2.

The tests were displacement controlled. The load history consisted in two cycles with a maximum story drift index of 0.5%, two cycles of 1%, four cycles of 2% and finally four cycles of 3.7% Horizontal displacements were applied in the upper part of the column. No axial load was present.

BEHAVIOR OF THE SPECIMENS

The global behavior of the specimens was studied by means of the hysteresis loops, strength and stiffness degradation, displacement ductility, cumulative hysteretic energy and equivalent damping. A definition of each follows.

Response Characstheristics

Displacement Ductility Factor. The traditional definition of displacement ductility factor is, in principle, only applicable to elastoplastic systems. As the behavior of reinforced concrete is not elastoplastic, different approaches have been suggested. In this study, the one depicted in Fig. 2 was adopted, where Si is the ideal or nominal strength, that is, the strength derived from the dimensions, reinforcing content, details of the designed section, and code-specified nominal material strength properties (Paulay and Priestley, 1992).

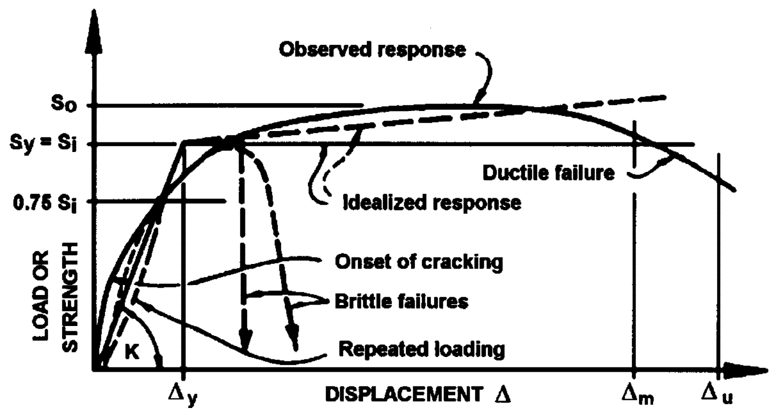

Fig. 2: Typical load-displacement relationship for a reinforced concrete element.

Peak-to-Peak Stiffness. Stiffness degradation in each cycle was evaluated by computing the peak-to.peak stiffness, defined as the slope of the straigh line joining maximum story shears in both directions within a cycle (Alcocer and Jirsa, 1991), as shown in Fig. 3.

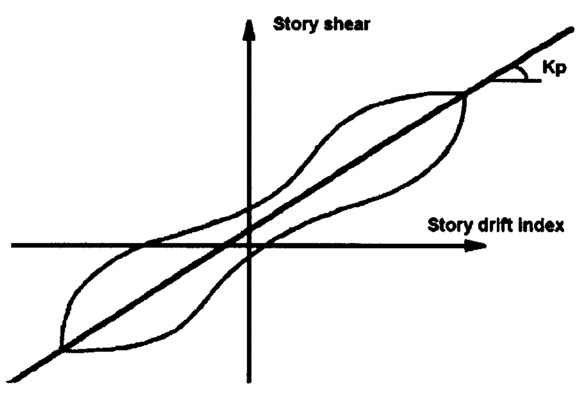

Fig. 3: Peak-to-peak stiffness

Hysteretic Energy. The energy dissipated was evaluated as the area within the hysteresis loops.

Equivalent Damping. The equivalent damping is the damping of a linear system responding with a maximum displacement equal to the displacement of the non-linear system subjected to a given periodic excitation (Alcocer and Jirsa, 1991). Accordingly, it may be computed equating the energy dissipated in one cycle by the linear system to its linear equivalent, as depicted in Fig. 4.

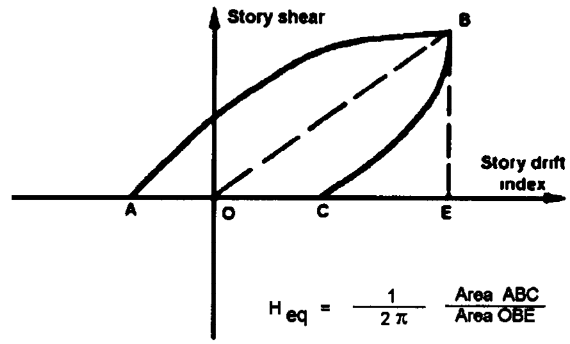

Fig. 4: Equivalent damping.

Series 1

As stated above, series 1 consisted of specimens N1, N2, and N3, N1 being the reference specimen. The column depth for the three specimens was 350 mm. The diameters of the longitudinal beam bars passing through the joint , and the column depth-beam bar diameter ratio are shown in Table 1.

Table 1. Diameters and column depth-beam bar diameter ratio for series 1

Specimen	Diameter (mm.)	Column depth-beam bar diameter ratio
N1	12	29
N2	16	22
N3	20	18

Design and detailing of N1 followed the earthquake resistant regulations presently enforced, corresponding to the most favorable bond conditions likely to be obtained in current practice. The diameters of beam bars for N2 and N3 were progressively increased to obtain less favorable bond conditions.

Stiffness degradation referred to the elastic stiffness of N1, as a function of story drift index and displacement ductility factor, is shown in Fig. 5. For a story drift index of 3.7%, the displacement ductility factor reached 5. Obviously, this level of story drift can not be tolerated in a structure. If the story drift index should be kept within tolerable limits - a maximum of around 2% is usually stipulated in most building codes - the response of the structure should be developed with restricted ductility.

A pronounced degradation is observed, even for N1 and for moderate displacements ductility factors like 2 or 3. This degradation should be considered as inevitable. Moreover, more pronounced degradations, due to bad quality workmanship, are to be expected at construction sites.

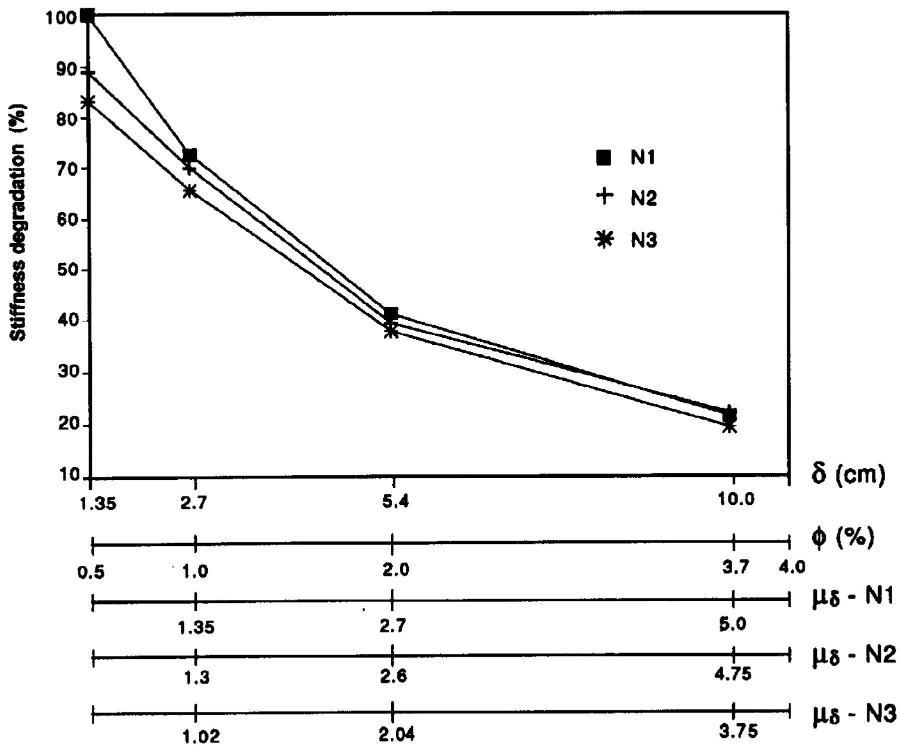

Fig. 5: Stiffness degradation referred to the elastic stiffness of N1

As depicted in Fig. 5, the behavior of the three specimens regarding stiffness degradation is practically identical, even for story drift indexes of around 4% and displacement ductility factors of around 5. Obviously, increasing the diameters of the longitudinal reinforcing bars from 12 mm to 20 mm, has no effect concerning stiffness.

The evolution of the hysteretic energy as a percentage of the total hysteretic energy dissipated by N1 is illustrated in Fig. 6. It is important to point out that the three specimens were designed and detailed so that the diameters of the longitudinal reinforcing bars in beams were the only variable. Accordingly, any additional pinching in the hysteresis loops of N2 and N3 as compared to N1, should be attributed to bond failure. This does not mean that beam bars may pull through, but they should be anchored in the beam on the other side of the column.

As shown in Fig. 6, for small to moderate story drift indexes, ranging from 0.5% to 1%, and up to displacement ductility factors of 1.3, as should be expected, the three curves coincide. For story drift indexes greater than 1%, the curves start to diverge. Thus, at 2%, N1 dissipated 33% of the total energy, while N2 only 27% and N3, 23%. This means that N2 dissipated only 80% of the energy dissipated by N1, and N3, 70%. These differences remained practically constant for story drift indexes around 4%. For 2%, the displacement ductility factor reached values around 3, and close to 5 for 3.7%.

The equivalent damping follows similar trends to the hysteretic energy as shown in Fig. 7

From the results presented above, it can be concluded that the performance of N2 and N3 is slightly inferior to N1. However, this behavior is considered acceptable.

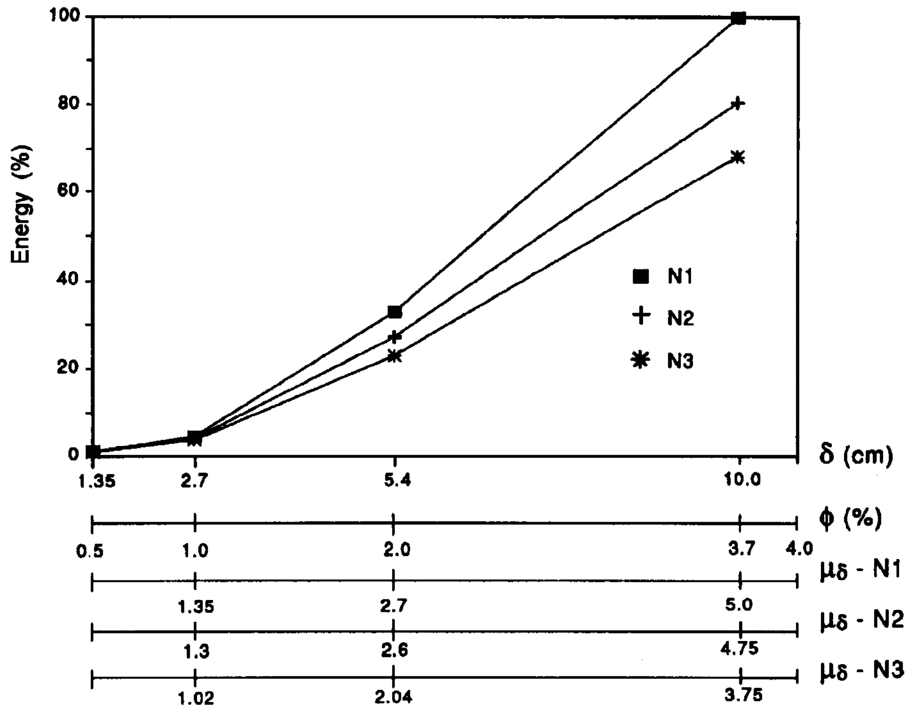

Fig. 6: Evolution of the hysteretic energy.

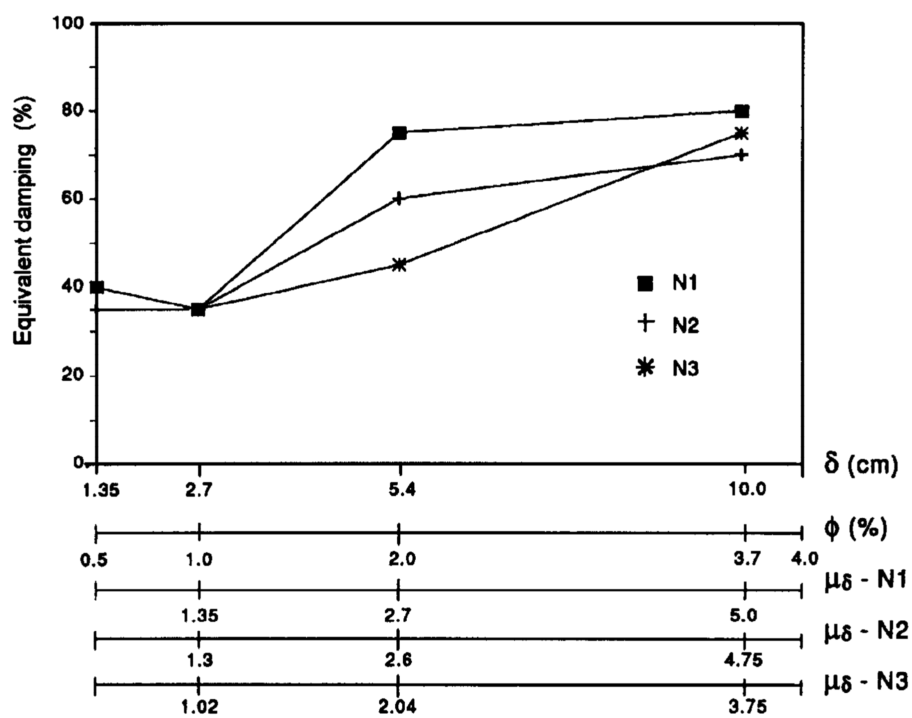

Fig. 7: Equivalent damping.

Series 2.

Series 2 consisted of specimens N4, N5, and N6, N4 being the reference specimen. The column depth for the three specimens was 400 mm. The diameters of the beam longitudinal bars passing through the joint, and the column depth - beam bar diameter ratio are shown in Table 2.

Table 2. Diameters and column depth-beam bar diameter ratio for series 2

Specimen	Diameter (mm.)	Column depth-beam bar diameter ratio
N4	16	25
N5	20	20
N6	25	16

Design and detailing of N1 followed the earthquake resistant regulations presently enforced, corresponding to the most favorable bond conditions likely to be obtained in current practice. Also, in series 2, the diameters of the beam bars for N5 and N6 were progressively increased to obtain less favorable bond conditions.

Plots like those already shown for the specimens in series 1, were developed for the specimens in series 2 and are not reproduced here due to space limitations. Additional information can be found in Giuliano *et al.*, 1993. However, the main results are discussed below. Stiffness degradation is more pronounced for series 2 than for series 1. Thus, N5 exhibited stiffness, in general, 85% of that corresponding to N4, and N6, 80%, except for story drift indexes close to 4, where it reached 66%. However, for restricted displacement ductility factors, 2 to 3, the differences are not so important.

The evolution of the hysteretic energy followed the same trends. The differences encountered are also greater than for series 1. The specimen N5, dissipated, in general, 70% of the energy dissipated by N4, while N6, dissipated 50% to 60%.

The values for the equivalent damping were similar to those in series 1.

It can be stated that the specimens in series 1 behaved better than those in series 2.

The severity of bond conditions is related to the column depth-beam bar diameter ratio. The larger it is the better the bond conditions . In Tables 1 and 2, it can be noticed that series 1 exhibited greater values that series 2, although the differences are not very significant. In general, all codes limit the above ratio to 25 to 30 for structures with full ductility.

Table 3 shows the nominal shear stress for both series.

Table 3. Nominal shear stress of specimens in series 1 and 2

Series	Specimen	Nominal shear stress (Mpa)
	N1	5.2
1	N2	5.6
	N3	5.8
	N4	7.1
2	N5	6.7
	N6	7.0

As can be observed, the nominal shear stresses for the specimens in series 2 are larger than for series 1. This increment in the nominal shear stresses seems to be the major reason for the specimens in series 1 to behave better than those in series 2.

CONCLUSIONS

Based on the results obtained from this study, it can be concluded that bond requirements in interior reinforced concrete beam-column joints, according to the earthquakes-resistant regulations presently enforced in Argentina, may be relaxed in structures designed with restricted ductility. In this respect, the column depth-beam bar diameter ratio may be decreased to 17.

ACKNOWLEDGEMENTS

This research was supported by The National Department of Science and Technology under grant N° 296/91. The support is gratefully acknowledged. Any opinions, discussions, findings, conclusions, and recommendations are those of the authors and do not necessarily reflect the views of the sponsor.

REFERENCES

Alcocer, S. M. and J. O. Jirsa. (1991). Reinforced concrete frame connections rehabilitated by jacketing. *PMFSL Report 91-1.*

Giuliano, A. P. , J. A. Amado, M.A. Leiva, R. D. García and E. A. Barros (1993). Comportamiento de nudos viga columna de hormigón armado con ductilidad limitada ante cargas cíclicas reversibles. *IR 93-5*

INPRES. (1991). *Reglamento INPRES-CIRSOC 103, Normas Argentinas para Construcciones Sismorresistentes, Parte II, Construcciones de Hormigón Armado y Hormigón Pretensado.* INPRES, San Juan, Argentina.

Paulay, T and M.J.N. Priestley (1992). *Seismic Design of Reinforced Concrete and Masonry Buildings.* Mc Graw Hill, New York.