7th Canadian Conference on Earthquake Engineering / Montreal / 1995
7ième Conférence canadienne sur le génie paraséismique / Montréal / 1995

Grouted Steel Tubes as Seismic Retrofit for Beam to Column Joints

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ABSTRACT

The paper describes experimental and analytical studies to determine the strength of reinforced concrete beam to column joints retrofitted with grouted steel tubes.

Cyclic testing of reinforced concrete beam and column sub-assemblies have proven that a very substantial increase in bending and shear strength can be achieved in the joint area by encasing the region with a steel tube and filling the cavity with cement grout. Failures were deflected from the joint area to adjacent members, which were intentionally weakened to form plastic hinges. Subsequent tests on the remaining joint specimens, which forced the failure mechanism into the joint region, provided strength and ductility data for the joint itself.

A non-linear finite element program was written for the analysis of the joint area, utilizing non-associated plasticity for the constitutive description of the concrete and steel material models. Presently the program is able to simulate monotonic loading which is adequate for comparing strength envelopes. Refinements are still underway.

INTRODUCTION

The 1971 San Fernando, California, earthquake led to significant changes in the practice of seismic design, particularly in the high risk seismic zones of North America. Many reinforced concrete structures that conformed to design and construction standards at that time behaved poorly, prompting many modifications to the reinforced concrete design codes. These modifications specifically addressed the ductility requirements of reinforced concrete members and, in particular, the joint region between beams and columns. This region is subject to large shear forces during lateral seismic loading, particularly when beam moments on opposite faces of a column have the same orientation. Under severe loading, plastic hinges are expected to form at the ends of the beams adjacent to the joint, and transverse reinforcement in both the beam and the column are required to provide confinement to the concrete in the core region, thereby safeguarding the ductility of the joint.

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While updated design codes address the construction of new structures, older structures that were built according to earlier design codes may not meet today's seismic standards. Many are inadequate, and pose a severe risk to society. What can be done about them? One available option is to retrofit such structures; that is, to modify them to assure compliance with current design provisions. One such method is to encase deficient beam-to-column joints with a steel jacket and fill the void with concrete grout (Fig. 1). This is an elegant and simple solution; the steel provides both lateral confinement and shear reinforcement, thereby adding strength and ductility to the joint. This approach can be applied to undamaged deficient structures and, when appropriate, to frames that have been damaged by an earthquake.

A considerable amount of research has been done to increase the strength and ductility of reinforced concrete compression members by enhancing the effectiveness of confinement by such methods as: increasing the transverse steel ratio, increasing the yield strength of the steel, adjusting the longitudinal reinforcement geometry, using steel fibre reinforced concrete, welded-wire fabric, prestressed bolts and, with limited success, increasing the concrete strength.

Confinement for concrete columns can also be provided by encasing the concrete core within a steel shell. In new construction this is achieved by filling hollow steel sections with concrete, while externally applied steel jackets, filled with grout, are typically used in the case of retrofits or repairs. In both cases, the external steel completely encloses the entire concrete core, and effectively confines all the concrete, inclusive of the cover concrete, thus assisting in reducing bond failures.

The beam-to-column connection, which is considered part of the column, is expecially susceptible to debonding of the reinforcement and subsequent shear failure when insufficient confinement is provided. Ideally, this region should remain elastic, thus deflecting plastic hinge formation to adjacent beams in compliance with the rules of standard practice: strong column/weak beam, strong joint/weak element, strong shear/weak moment. Modern design codes address these concepts by specifying a ratio of strengths between beam and column.

Based on previous research, the use of grouted steel casings in the joint area could provide the ideal solution when existing structures need to be retrofitted. Very little research has been done in this area and it was considered worthwhile to investigate this approach.

EXPERIMENTAL STUDY

To gain insight into the behaviour of encesed beam-to-column joints, an experimental study was undertaken on a partial frame assembly. As a basis for the experimental work, a two-storey frame for an office building, situated in Vancouver, B.C. was designed in accordance with all the requirements of the 1970 National Building Code of Canada, and the CAN3-A23.3-M66 code (1) for reinforced concrete structures. A beam-to-column joint sub-assembly of this structure was subsequently considered for the detailed retrofit study, and its design modified slightly - all transverse reinforcement was omitted from the joint region, which was common practice at the time due to ambiguities in the interpretation of the code. The dimensions of the test specimens were primarily dictated by the availability of formwork and laboratory testing capacities, resulting in a beam-to-column joint model of approximately half-scale. Four specimens were fabricated, two with circular steel tubes and two with square tube retrofits (Fig.2).

In the joint region the column reinforcement consisted of 6 - 10mm diameter bars, three on each face, while the beam had 4 - 10mm diameter bars at the top and 2 - 10mm diameter bars at the bottom.

Steel jackets were used as a substitute for the lateral reinforcement steel that was missing in the original design. They were designed to provide confinement and additional ductility to the joints, without increasing the moment capacity of the specimens. Based on modern design codes, which require a spacing of 35mm for 10mm ties in both the beams and the columns, the necessary thickness of the jackets in the joint region was 2.86 mm (0.110"). The lengths of the retrofit were made equal to the member depth (d) along the columns and twice the member depth (2*d) along the beam, measured from the beam-to-column interface. A gap of 25mm was left halfway along the length of the beam jacket, to create a flexural hinge at that point (Fig. 2). The size of the steel jackets was kept to a minimum to reduce the amount of disruption to the structure during the retrofit process, and also to limit the increase in strength of the frame members, which would deflect the failure elsewhere in the structure.

Testing proceeded in two stages: Firstly, two bare reinforced concrete frames were tested. After these and the two undamaged joints had been retrofitted, they were again tested in the same way. Secondly, the beam-to-column joint regions, which had not failed in the second stage, were tested to failure, again under cyclic loading.

The first series of six tests served to observe the behaviour of a frame sub-assembly when subjected to quasi-static cyclic loading, similar to that experienced by a two-storey portal frame in an earthquake. First, two unretrofitted reinforced concrete frames were tested until shear failure occurred in the joint region. The joints were then repaired with a circular and a square grouted steel tube. Four tests were conducted in a similar way on the retrofitted specimens, two of which were the previously damaged ones with subsequent repairs. Neither of the four specimens failed in the joint region. Plastic hinges formed either in the beam or the column with various degrees of ductility. Full details of this stage of the program are given in (2).

Subsequent to the first series of tests, the beam portions of the specimens were cropped and minor repairs were done to the concrete in preparation for the next series. The column part of each specimen was bolted down to the test floor and a cyclic load with increasing amplitude was applied to the beam stubs (Fig. 3).

Various measuring instruments, controlled by a computer data acquisition system, were used to record the applied load, joint rotation and steel jacket strains at selected locations.

TEST RESULTS

Material Properties

All the reinforcing steel used in the testing program originated from the same batch and tests on four tensile coupons produced an average yield strength of 566 MPa; the ultimate strength was approximately 800 MPa. The stress-strain curves did not feature a marked yield plateau and the stress at 0.2% offset strain was used to define the tensile yield stress (Fig.4). This value thus only provided a nominal yield value for use in design; the actual yield point of the specimen varied, depending on the

CONCLUSIONS

An effective retrofit method has been studied for deficient or damaged beam-to-column joints in reinforced concrete frames. Grouted steel tubes provided the necessary strength and confinement to avoid distress of the joint region. Circular tubes, proved to be more effective than rectangular tubes, although the latter also met the general expectations.

The increase in moment capacity in the joint and adjacent members that were encased, resulted in significant frame stiffness redistributions. As a result, failures were deflected to adjacent regions that were not designed for the loads they encountered, and unexpected failures occurred. It is recommended that strength reducing elements such as gaps be included in the steel casing, close to the beam-to-column joint area. A number of gaps may be advisable to avoid concentrated yielding and reduced ductility of the reinforcement bars within the gap areas.

A jacket thickness designed to replace the missing transverse steel was found to provide sufficient strength and assure ductile behaviour. For practical reasons during construction (e.g. welding), a minimum thickness is advisable.

Analytical modelling to determine the moment and shear resistance of the beam-to-column joints shows promising results. This will provide a relatively simple means for designers to determine casing requirements.

ACKNOWLEDGEMENTS

The direct and indirect support of the Natural Sciences and Engineering Research Council of Canada and the Department of Civil Engineering at the University of British Columbia is gratefully acknowledged.

REFERENCES

(1) **Canadian Standards Association (CSA)**, (1966), "Code for the Design of Plain or Reinforced Concrete Structures.", CSA Standard A23.3-1966, Canadian Standards Association, Ottawa ON

(2) **Hoffschild, T.E.**, (1992), "Retrofitting Beam-to-Column Joints for Improved Seismic Performance", M.A.Sc. Thesis, Dept. of Civil Engineering, University of British Columbia.

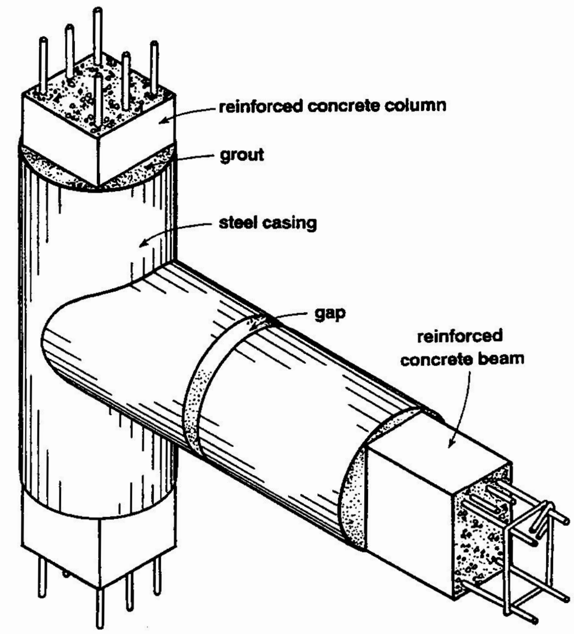


Figure 1: Reinforced concrete joint with grouted steel tube retrofit

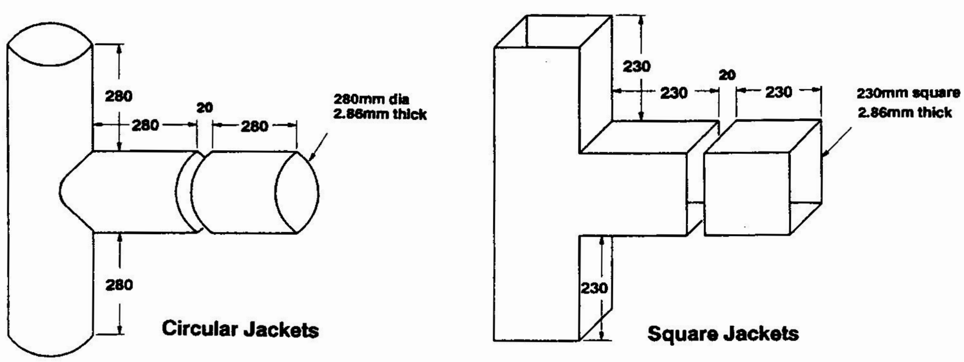


Figure 2: Steel Casing geometry

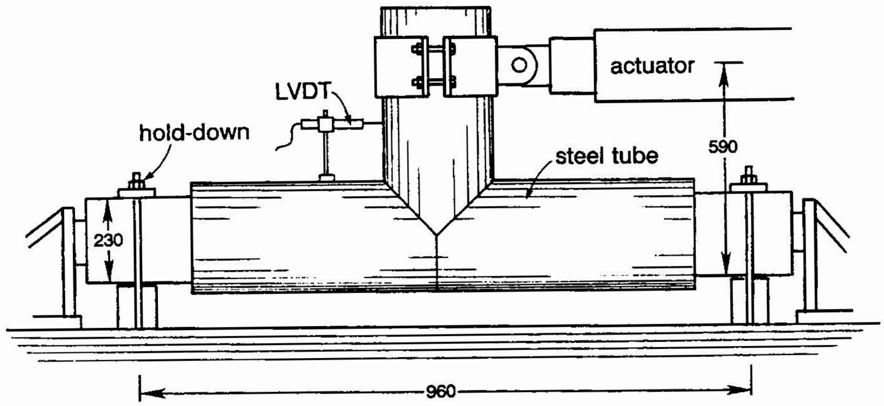


Figure 3: Test setup for joint test

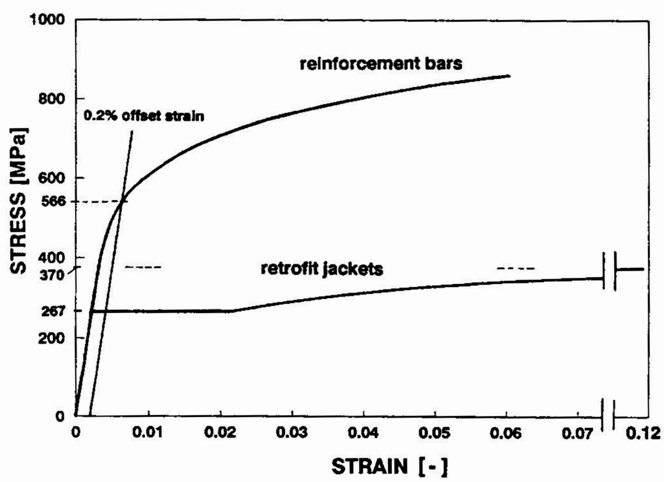


Figure 4: Steel material properties

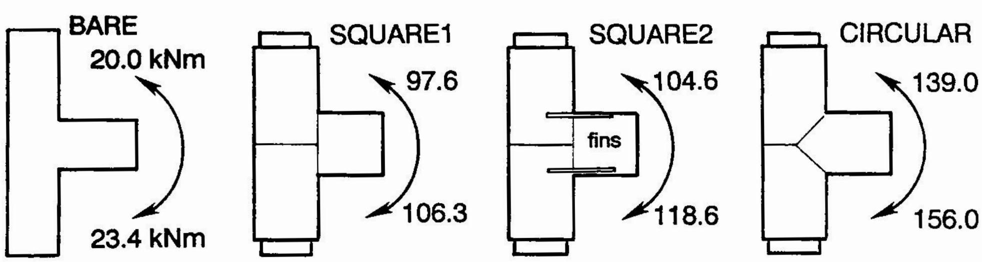
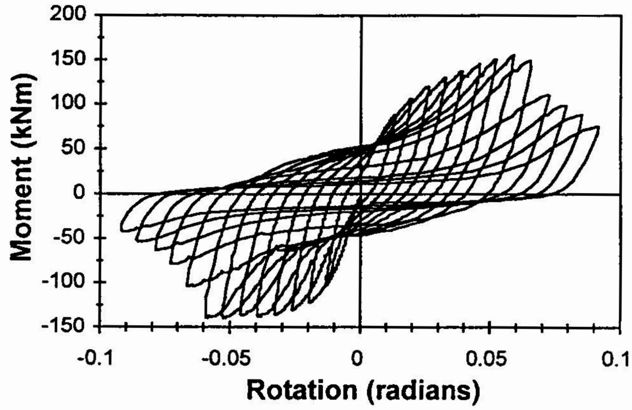
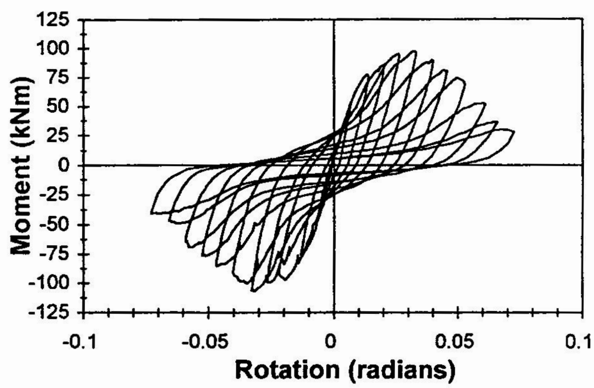


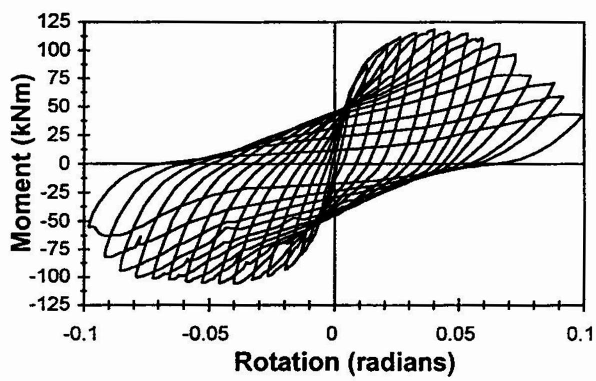
Figure 5: Test results



(a) Circular retrofit

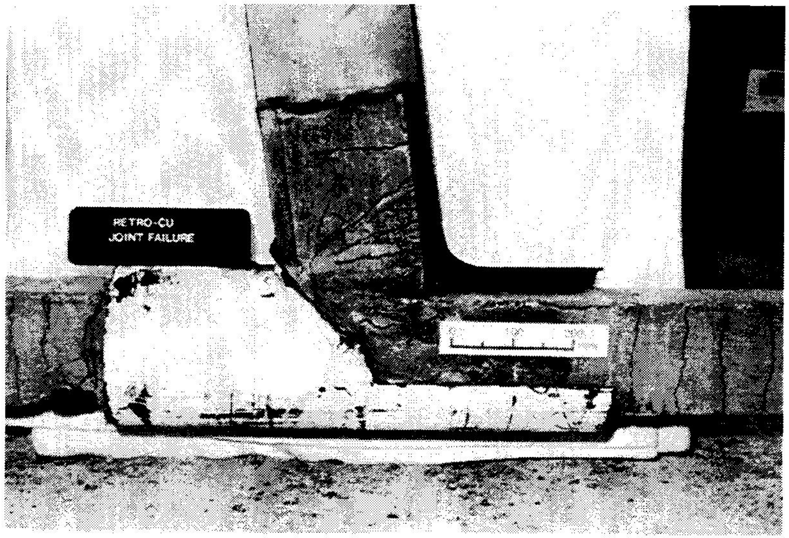


(b) Square retrofit



(c) Square retrofit with fin reinforcement

Figure 6: Hysteresis curves of joint tests



Circular retrofit

Square retrofit with fin reinforcement

Figure 7: Photos of failed joints