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Experimental Studies of Hinges in Existing Bridge Columns

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ABSTRACT

Hinged column-footing connections are a common detail found in existing bridge structures in moderate seismic regions, such as eastern Canada. The seismic performance of typical hinge details was assessed by a series of half-scale reinforced concrete bridge piers tested under reversed cyclic loading. Test specimens included square columns with both one-way and two-way hinges, and a circular column with a two-way hinge.

In order to assess the behaviour of such hinges it is necessary to consider the details of the reinforcement and the effects of the confinement of the concrete in the hinge region. Response predictions are compared to the measured responses and guidance is given on the assessment of these hinges.

INTRODUCTION

Hinges in columns, typically at column-footing connections, are a common feature of existing bridge structures in moderate seismic regions such as eastern Canada. Typical hinges are formed by reducing the column dimension over a very small portion of the height with dowel bars extending from the footing into the column. For a one-way hinge, moment transfer is minimized in the smaller direction of the hinge and often two-way hinges are used to limit moments in both directions.

Previous research reported by Base (1965) emphasized that the compressive strength of the concrete in the hinge throat was significantly increased by the confinement provided by the adjacent column and footing surfaces. Saiidi, Orie and Douglas (1988), and Saiidi and Straw (1993) performed tests on four 1/8 scale models and four 1/6 scale models. The specimens consisted of one-way hinge details subjected to lateral load in the moment-resisting direction. Variables in these tests included the aspect ratio of the column, the presence of axial load, and the influence of cyclic and monotonic lateral loading. It was concluded that the contribution of the dowels to the shear stiffness of the hinge was very

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small, and that the shear capacity was due primarily to aggregate interlock in the compression zone. In these tests, the peak load was controlled by flexure with the well-confined hinge concrete developing strains of up to 0.03. Lim and McLean (1991) reported on an experimental program that included cyclic lateral loading of nine 1/5 scale models and over fifty 1/20 scale models of well-confined hinge details. They concluded that varying the axial stress levels in the hinges had little effect on specimen response, with flexure controlling the behaviour of all specimens.

TEST PROGRAMME

A series of half-scale reinforced concrete bridge columns were tested under reversed cyclic loading to study the response of hinged column-footing connections. Figure 1 describes the test setup showing the hinge region and the dowel reinforcement through the hinge. Figure 2 shows the dimensions and the reinforcement details for the hinge and the column regions of the five test specimens. Test specimens included a circular column with a two-way hinge (C1) and square columns with both two-way (C2 and C3), and one-way hinges (C4). To form the hinges, the columns were reduced to one third of their dimension in the direction of the hinge and contained reinforcement only in the throat area. The reinforcement details and amount of reinforcement for both the hinges and the columns were chosen to reflect details consistent with older existing columns. The reinforcement ratios in the throat area varied between 3.1 and 3.8% and the dowel bars extended a distance of $1.7\ell_d$ into the column. Anchorage of the dowel bars into the 600 mm thick footing was provided by 90-degree standard hooks. A clear cover of 40 mm was provided for the dowel bars in the throat region while 25 mm cover was provided in the columns. No cross ties were provided for the 510x510 mm square columns and all of the columns contained transverse reinforcement which is much below that required for adequate confinement. The longitudinal reinforcement ratios in the columns were 1.17% for the circular column and 1.23% for the square columns.

Each column was subjected to a reversed cyclic lateral load applied at its inflection point. The point of inflection in the prototype was assumed to be located at a distance equal to 1/3 of the column height from the base. This corresponds to a prototype column which is "fixed" at its top and hinged at its base. In addition to the lateral loading, a constant axial load of 312 kN was applied to simulate the dead load of the superstructure (see Fig. 1). The resulting stresses due to axial loading were 5% and 4% of $A_g f_c'$ for the circular and the square columns, respectively.

The applied loads were measured by load cells while linear voltage differential transducers (LVDTs) were used to measure the top deflection and the sliding at the base. LVDTs were also used to determine rotations and curvatures over the height of the hinge and the lower 500 mm of the column (see Fig. 1(b)). Strain gauges were located on the dowel bars to measure the strains at the throat, as well as the variations of the strains over the length of the dowels. Strain gauges were also located on the two ties or spirals closest to the base of each column to determine the participation of the ties in resisting shear.

SPECIMEN RESPONSES

The specimens were loaded in "load control" until general yielding was observed. The deflection at general yielding, Δ_y, was defined as the deflection beyond which a significant reduction in stiffness occurred. After general yielding, displacements were imposed in increments of $0.5\Delta_y$ up to $4\Delta_y$. After reaching a displacement of $4\Delta_y$, steps of either Δ_y or $2\Delta_y$ were used until closure of the hinge gap occurred. The columns were cycled 3 times at each displacement level.

The moment curvature responses of the hinges of Specimens C1, C2, C3 and C4 are shown in Figure 3. The axis about which the bending occurred is also indicated on this figure. For all of the test specimens, a significant decrease in the stiffness of the hinge was observed as they were cycled at greater deflections. This loss of stiffness was not accompanied by a sudden drop in the strength of the section. Furthermore, the second and third cycles at each displacement level showed a softening of the hinge responses.

Gap closure was defined as the point at which the edge of the full-section column came in contact with the top of the footing. When the hinge gap closed, a rapid increase in both strength and stiffness was observed (see Fig. 3). For each specimen, testing was halted shortly after the gap closed. Gap closure was not reached for Specimen C4, since crushing of the concrete occurred in the throat region at a deflection of about $4\Delta_y$.

Very little evidence of cracking could be seen on the sides of the full-section columns that had two-way hinges (Specimens C1, C2 and C3), even though considerable deterioration of the hinge regions had occurred. The one-way hinge specimen, C4, subjected to bending about its weak axis, developed significant cracks on the full-section column on the faces in contact with the edges of the hinge. No cracks appeared on the faces away from the hinge.

PREDICTING RESPONSES OF SPECIMENS

The computer program RESPONSE (Collins and Mitchell, 1991) was used to predict the moment-curvature responses of the test specimens. This computer program performs a plane-section analysis for flexure assuming a tri-linear stress-strain relationship for the reinforcement and a non-linear stress-strain relationship for the concrete. One feature that is important to model is the significant confinement of the hinge concrete due to the presence of the full-section column and the footing. Figure 4(a) illustrates the manner in which the concrete stress strain relationship was modified to account for confinement. Both the strength of the concrete and the peak strain reached in the concrete were increased by multiplying the cylinder values by $\sqrt{(A_2/A_1)}$. This is the same factor used for confined strength in the CSA Standard (1994), however, the upper limit on this factor of 2.0 was not applied. For example, for Specimen C1 the area of the full-section column is 6.477 times the area of the hinge and hence, the confinement factor is 2.545. This resulted in a confined concrete compressive strength of 78.4 MPa being used in the analysis.

The predicted moment-curvature response of the hinge region of Specimen C1 agreed well with its measured response (see Fig. 4(b)). A comparison between the predicted and the measured responses of all the specimens indicated that this analysis technique can be used to obtain very accurate predictions.

CONCLUSIONS

These tests demonstrated that hinged column-footing joints permit very large column deflections accompanied by very little energy dissipation. A method of retrofitting these types of column hinges is currently being investigated to determine ways of increasing the joint stiffness, to limit the displacements at the top of the column and to improve the energy dissipation.

An increase in the compressive strength of the hinge region concrete was observed during testing. This was due to the presence of unloaded concrete above the hinge region, in the full-section column. A simple method was developed to account for the influence of this confinement on the stress-strain relationship of the hinge concrete. Additional tests will be carried out to investigate further the effects of confinement on the concrete in the hinge throat region.

It must be noted that visual inspection of a two-way hinge in an existing structure is very difficult since no signs of distress are typically visible either on the faces of the full-section column or on the sides of the hinge.

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REFERENCES

Base G.D., 1965, Tests on Four Prototype Reinforced Concrete Hinges, *Research Report No. 17*, Cement and Concrete Association, London.

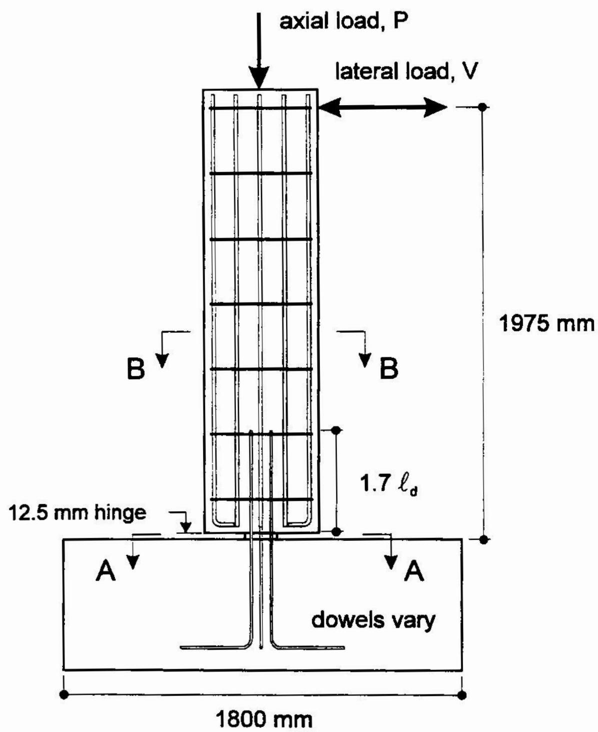
Canadian Standards Association (CSA), 1994, CSA A23.3-94, Design of Concrete Structures, CSA, Rexdale, Ont.

Collins, M. P., and Mitchell, D., 1991, Prestressed Concrete Structures, Prentice Hall, Englewood Cliffs.

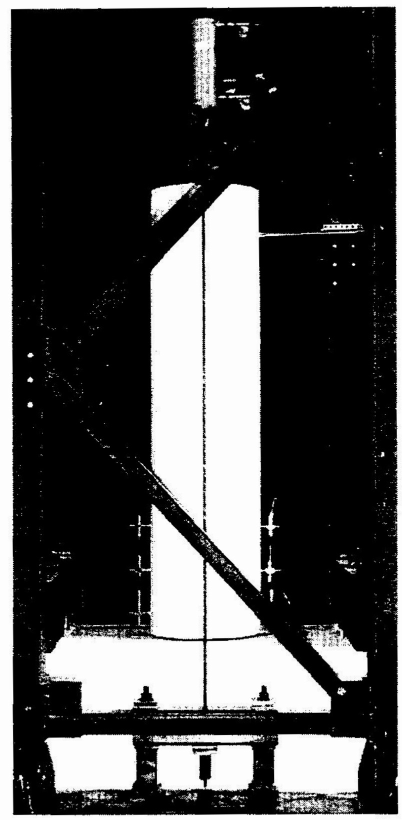
Lim, K.Y. and McLean, D.I., 1991, Scale Model Studies of Moment-Reducing Hinge Details in Bridge Columns, *ACI Structural Journal, Proceedings* V.88, No. 4, July-Aug. 1991, pp. 465-474.

Saiidi, M., Orie, J.L., and Douglas, B., 1988, Lateral Load Response of Reinforced Concrete Bridge Columns with a One-Way Pinned End, *ACI Structural Journal, Proceedings* V.85, No. 6, Nov.-Dec. 1988, pp. 609-616.

Straw, D.L., and Saiidi, M., 1992, Monotonic and Cyclic Response of One-Way Reinforced Concrete Bridge Pier Hinges in the Strong Direction, *ACI Structural Journal, Proceedings* V.90, No. 5, Sept.-Oct. 1993, pp. 568-573.



(a) Test specimen details



(b) Photograph of C1

Figure 1 Test specimen details

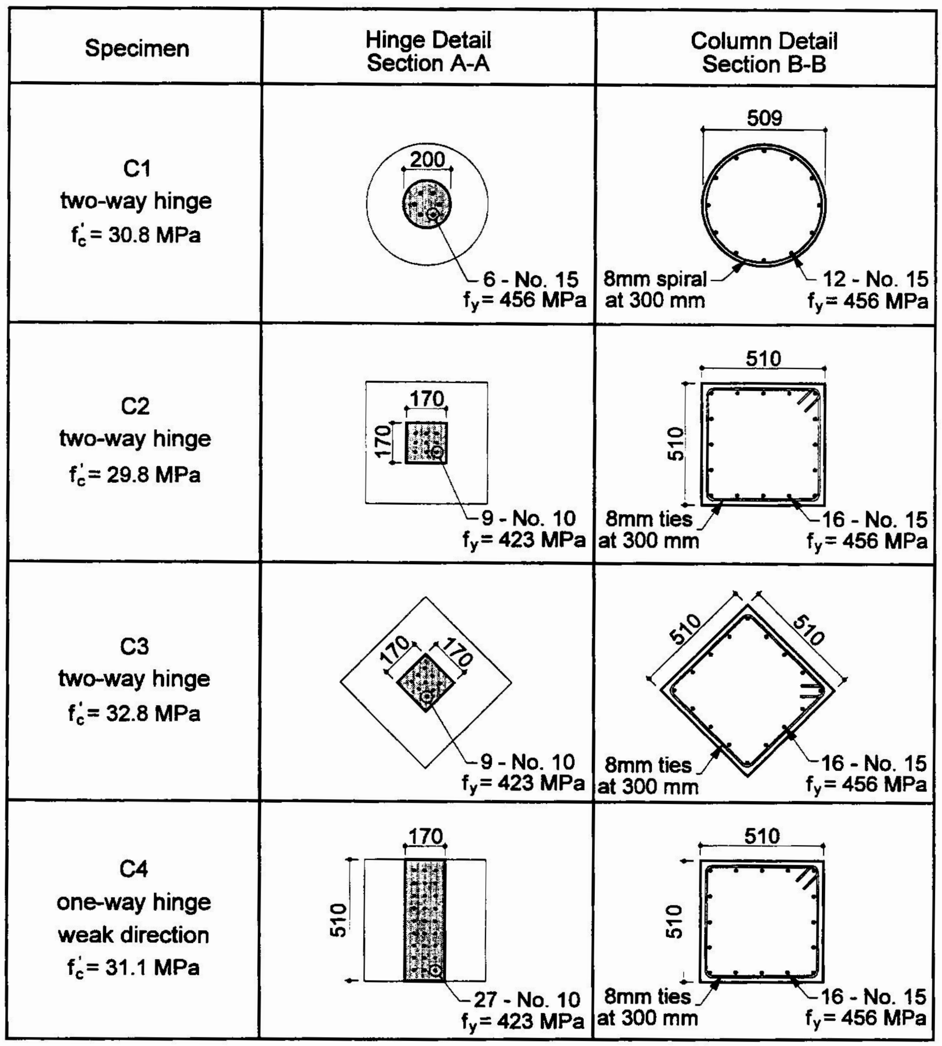


Figure 2 Column and hinge details typical of existing bridges

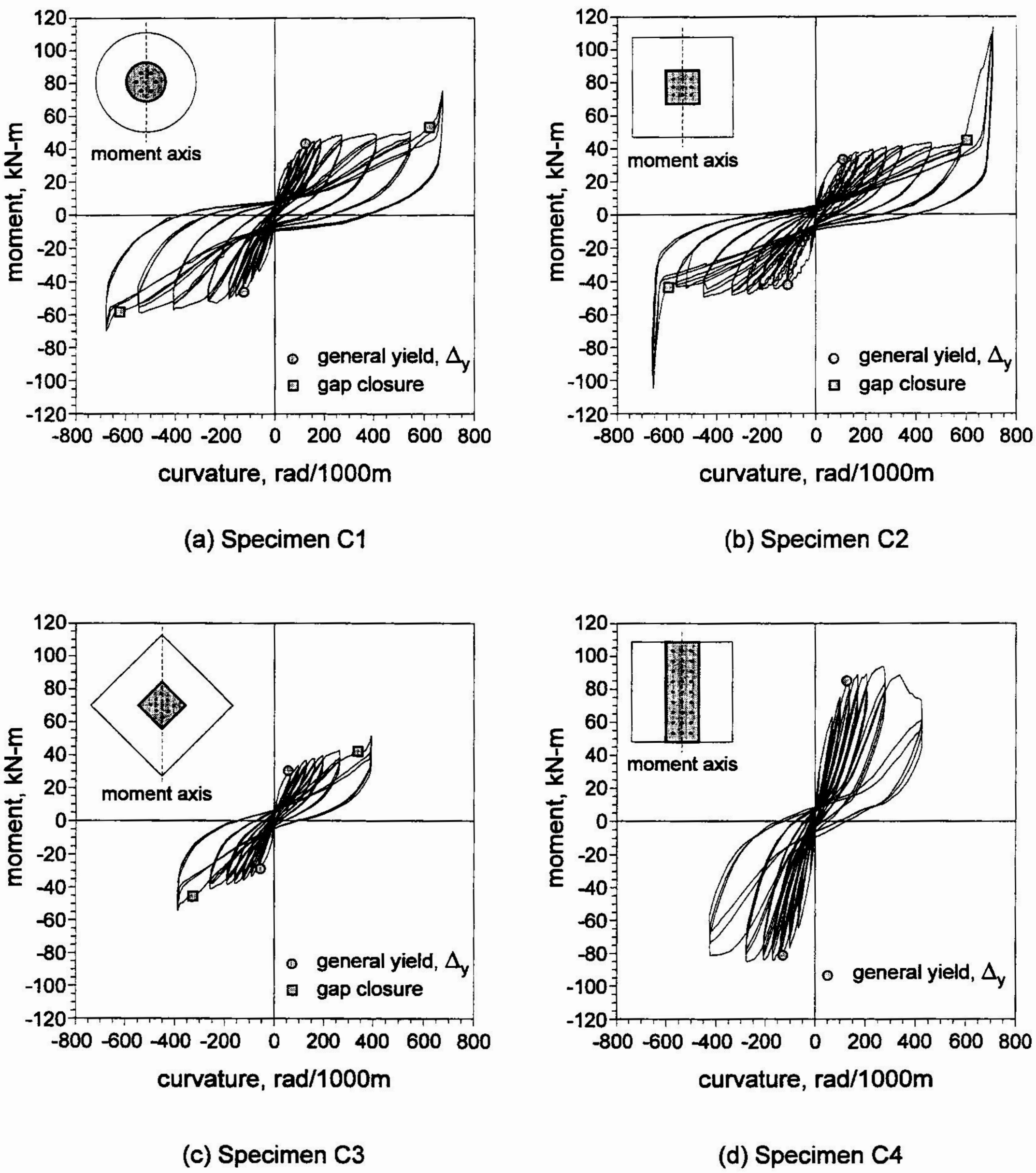
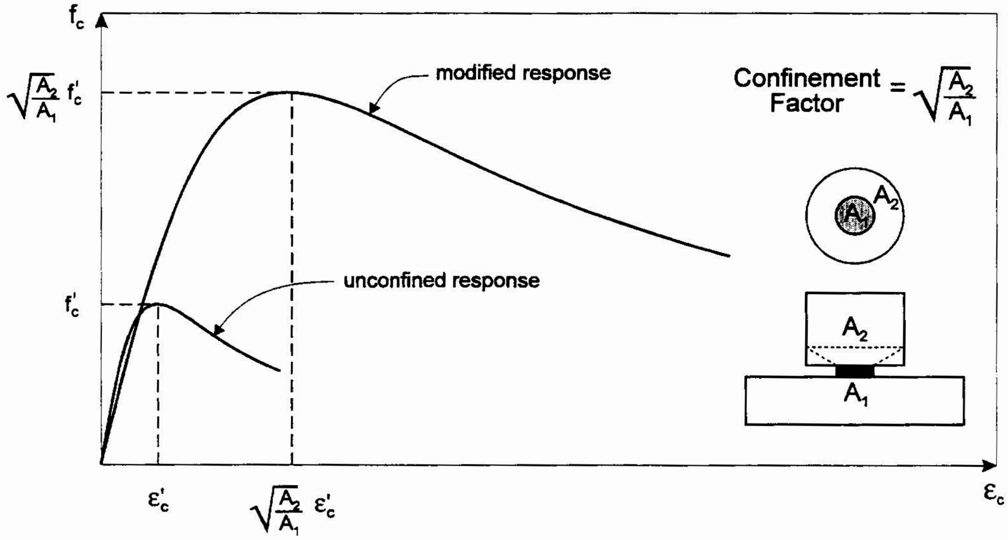
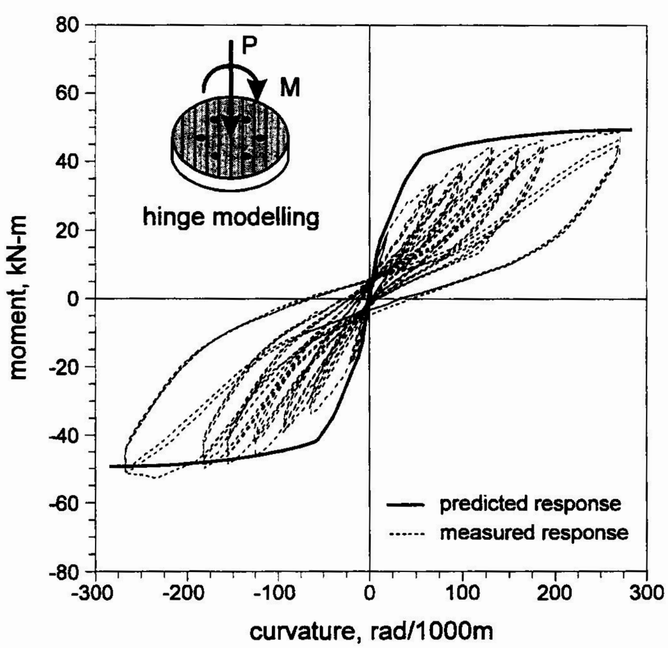


Figure 3 Observed hysteretic responses of test specimens



(a) modified concrete stress-strain relationship



(b) Predicted and observed responses of Specimen C1

Figure 4 Predicted responses and modelling details