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# Reversed cyclic loading tests of precast concrete columns

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## ABSTRACT

Eight full-scale precast concrete columns were tested to determine their reversed cyclic loading responses. The specimens represent typical one-storey precast concrete columns including the connections to the foundation pedestals. This experimental study illustrates the performance of columns designed using the current approach recommended by PCI and CPCI. Methods of improving the performance by increasing the strength and stiffness of the connections and improving the levels of ductility are presented. The resulting performance of the columns are assessed in terms of the expected R factors of the NBCC (1990).


## INTRODUCTION

The National Building Code of Canada (NBCC 1990) gives force modification factors, R, for different structural systems exhibiting different levels of ductility and energy absorption. Precast concrete construction is not covered specifically in the NBCC and by default the designer might use the category for "other lateral-force-resisting" reinforced concrete systems and use an R value of 1.5. In addition the CSA Standard A23.3 (CSA 1984) does not provide special provisions for the seismic design of precast buildings. A little guidance for the seismic design of precast buildings is given in the design handbooks of the Prestressed Concrete Institute (PCI 1985) and the Canadian Prestressed Concrete Institute (CPCI 1987). There is a surprisingly small amount of experimental evidence (Dolan et al. 1987, PCI 1986) in the literature for the reversed cyclic loading response of typical precast columns. This paper summarizes the results of a series of tests of precast columns tested at McGill University as part of a larger research programme (Pilette and Mitchell 1991) investigating the seismic behaviour and design of precast concrete structures.

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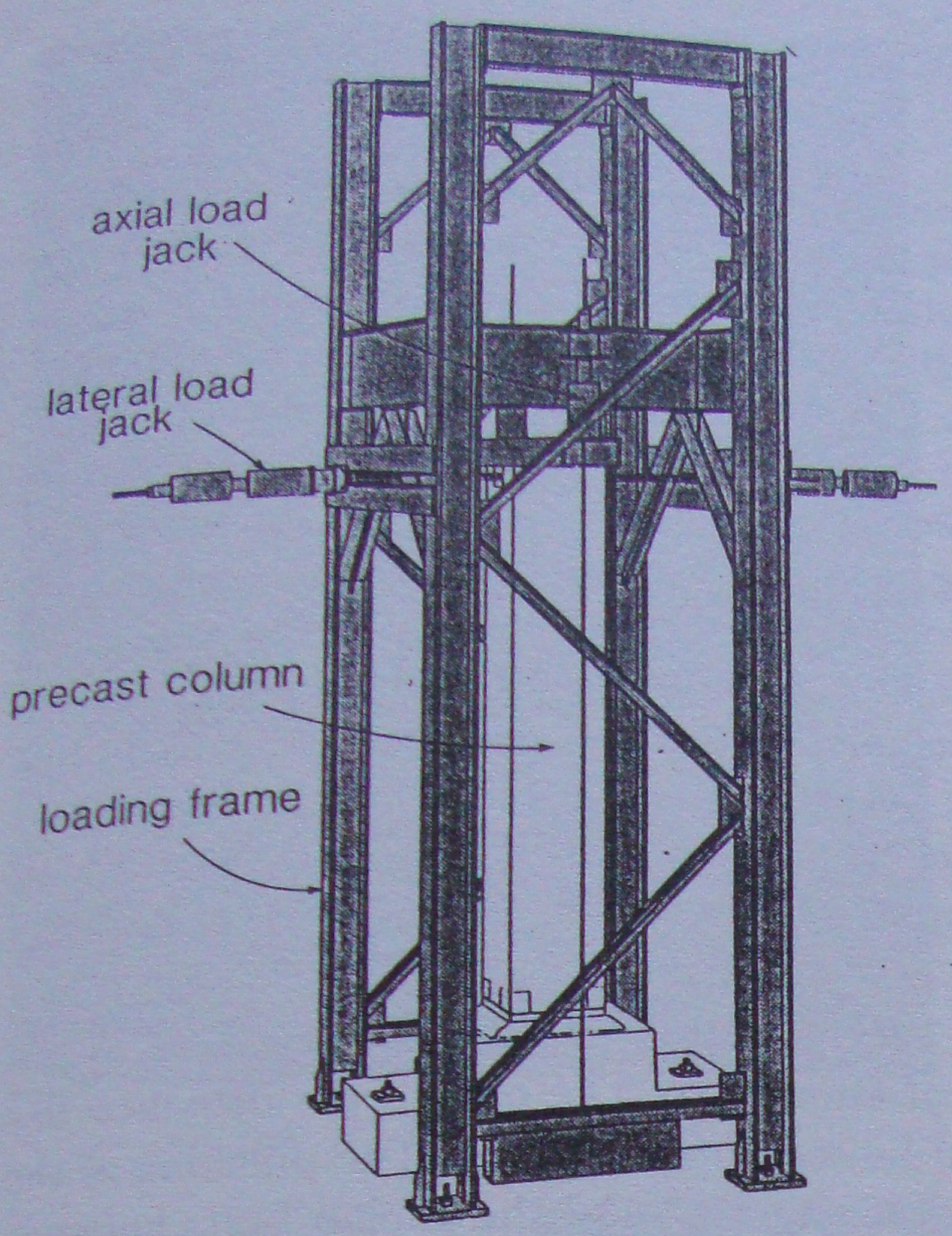
Table 1: Details of test specimens with dimensions in mm

| Specimen | Column details | | | Connection details | |
|---|---|---|---|---|---|
| | Size | Longitudinal bars | Transverse steel | Plate | Bolts |
| R1.5-1 | 475 × 475 | 8-No. 20 | No. 10 @ 300 | 725 × 725 × 25.4 | 4-25.4 |
| R2-1 | 400 × 400 | 8-No. 20 | No. 10 @ 150 | 550 × 550 × 25.4 | 4-25.4 |
| R4-1 | 300 × 300 | 8-No. 15 | No. 10 @ 60 | 450 × 450 × 15.9 | 4-15.9 |
| R4P-1 | 300 × 300 | 8-No. 15 | No. 10 @ 60 | 550 × 550 × 19.0 | 4-19.0 |
| R1.5-2 | 475 × 475 | 8-No. 20 | No. 10 @ 300 | 675 × 475 × 31.7 | 4-28.6 |
| R4-2 | 300 × 300 | 8-No. 15 | No. 10 @ 60 | 500 × 300 × 44.5 | 4-25.4 |
| R1.5-3 | 475 × 475 | 8-No. 20 dowels 4-No. 20 cage | No. 10 @ 300 | 475 × 475 × 25.4 | 4-34.9 |
| R4-3 | 300 × 300 | 8-No. 15 dowels 4-No. 15 cage | No. 10 @ 60 | 300 × 300 × 25.4 | 4-34.9 |

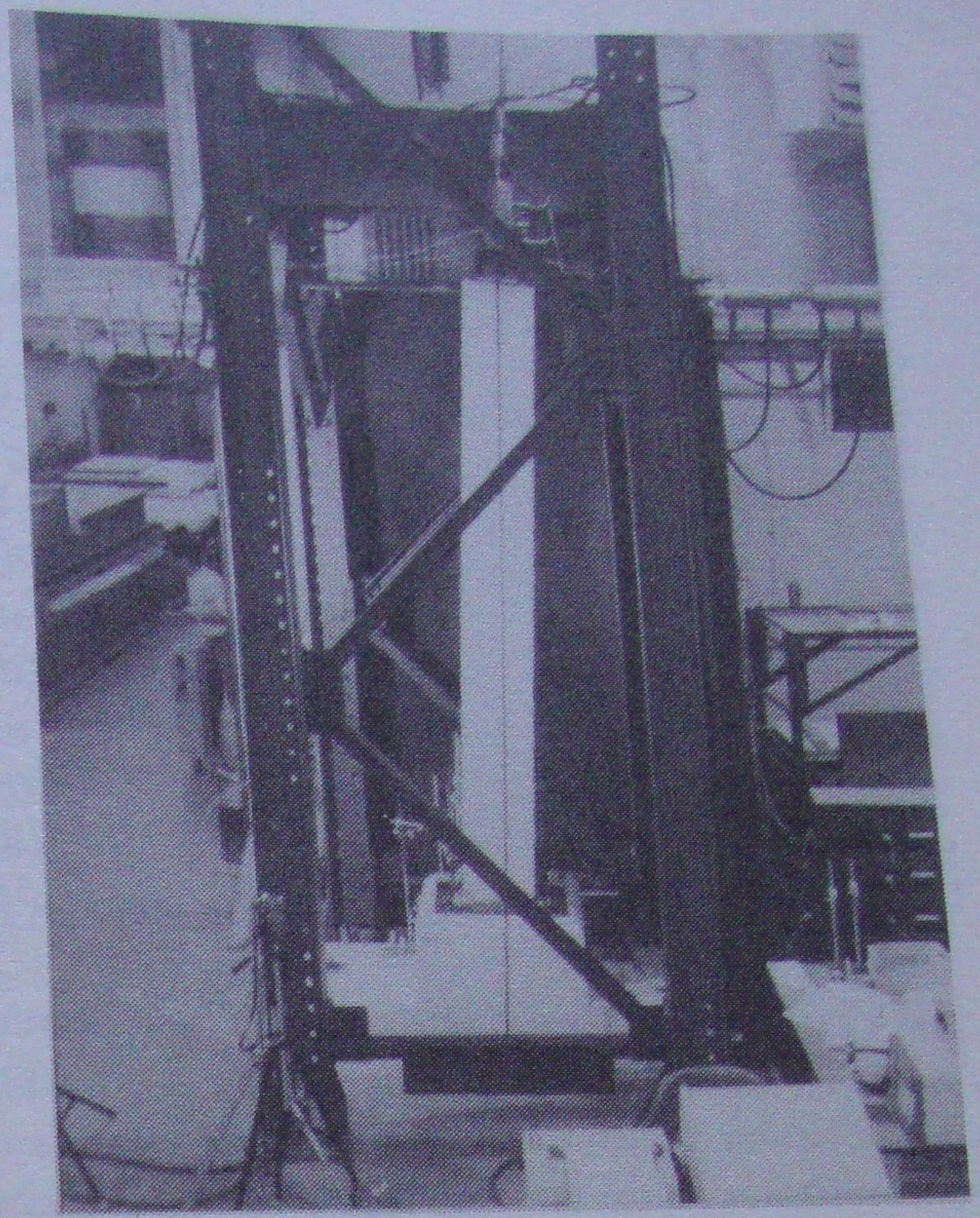
## TESTING PROGRAMME

Eight precast column-connection-foundation specimens were tested under reversed cyclic loading in the testing frame shown in Fig. 1. The lateral load was applied at a height of 2.75 m measured from the bottom of the base plate. Each column was subjected to a constant axial load of 184 kN to simulate the dead load acting on the column. Figure 2 illustrates three standard types of column connection details typically used (CPCI 1987). Type 1 has a base plate which is oversized on all sides of the column, Type 2 is oversized only in the direction of loading and Type 3 uses recessed anchorage pockets. For Types 1 and 2 the longitudinal column reinforcing bars pass through holes in the base plate and are welded to the underside of the base plate connection. Connection Type 3 uses dowel bars, welded to the steel anchor bolt pockets. The column reinforcing cage is slipped over these dowel bars and adequate lap slice length is provided between the dowels and the longitudinal column bars. For all specimens a nut was used above and below the base plate on all anchor bolts to facilitate erection. Grout, having a minimum compressive strength of 60 MPa was packed between the base plate and the top of the pedestal.

Table 1 gives the details of the specimens. The designation for each specimen contains the R value followed by a number indicating the connection type (e.g., Specimen R1.5-1 was designed with an R of 1.5 and has a connection Type 1). The compressive strengths of the concrete at the time of testing were 37, 33 and 42 MPa for connection Types 1, 2 and 3, respectively. The No. 10, No. 15 and No. 20 reinforcing bars had yield stresses varying from 423 to 516, 443 to 456, and 443 to 494 MPa, respectively. The anchor bolts had yield stresses varying from 263 to 315 MPa and the yield stress of the base plates varied from 252 to 365 MPa.

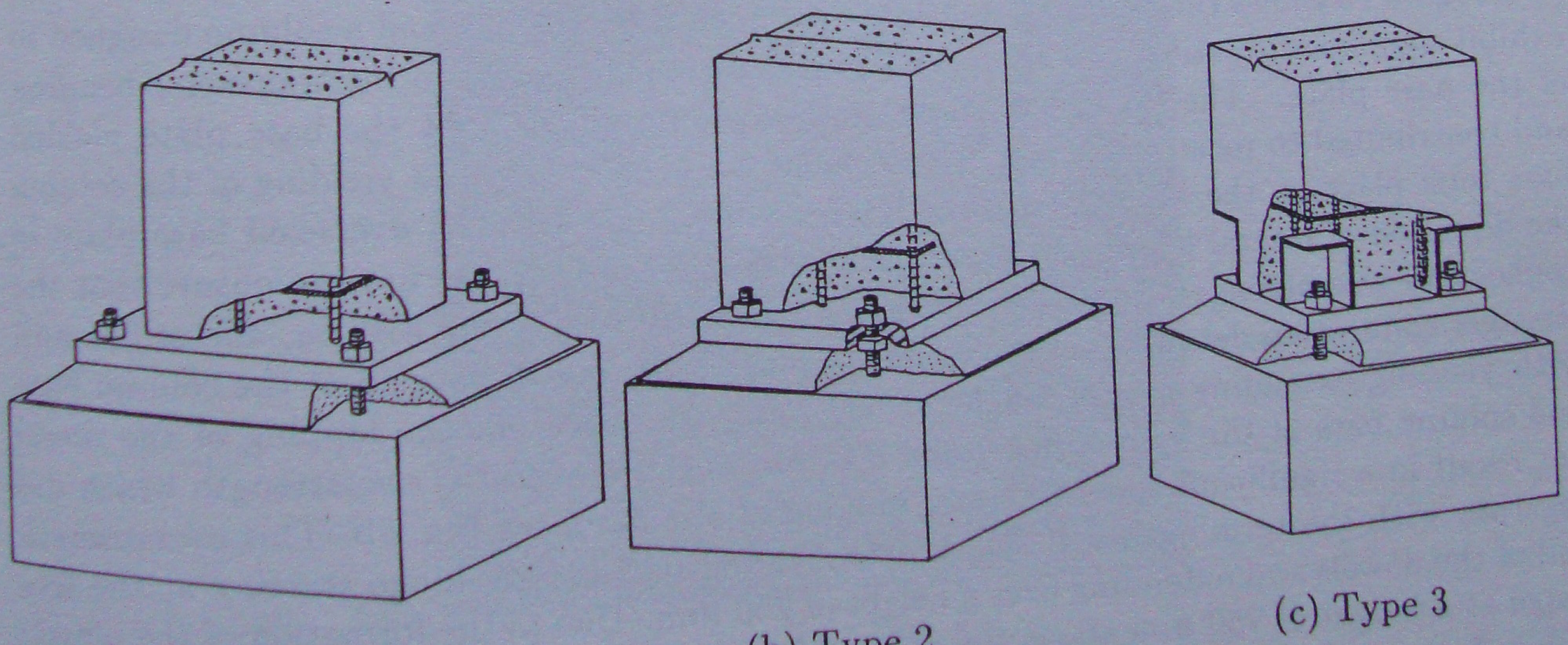



(a) Loading frame



(b) Specimen R4P-1 under load

Figure 1: Test setup for reversed cyclic loading of precast column-connection-foundation specimens



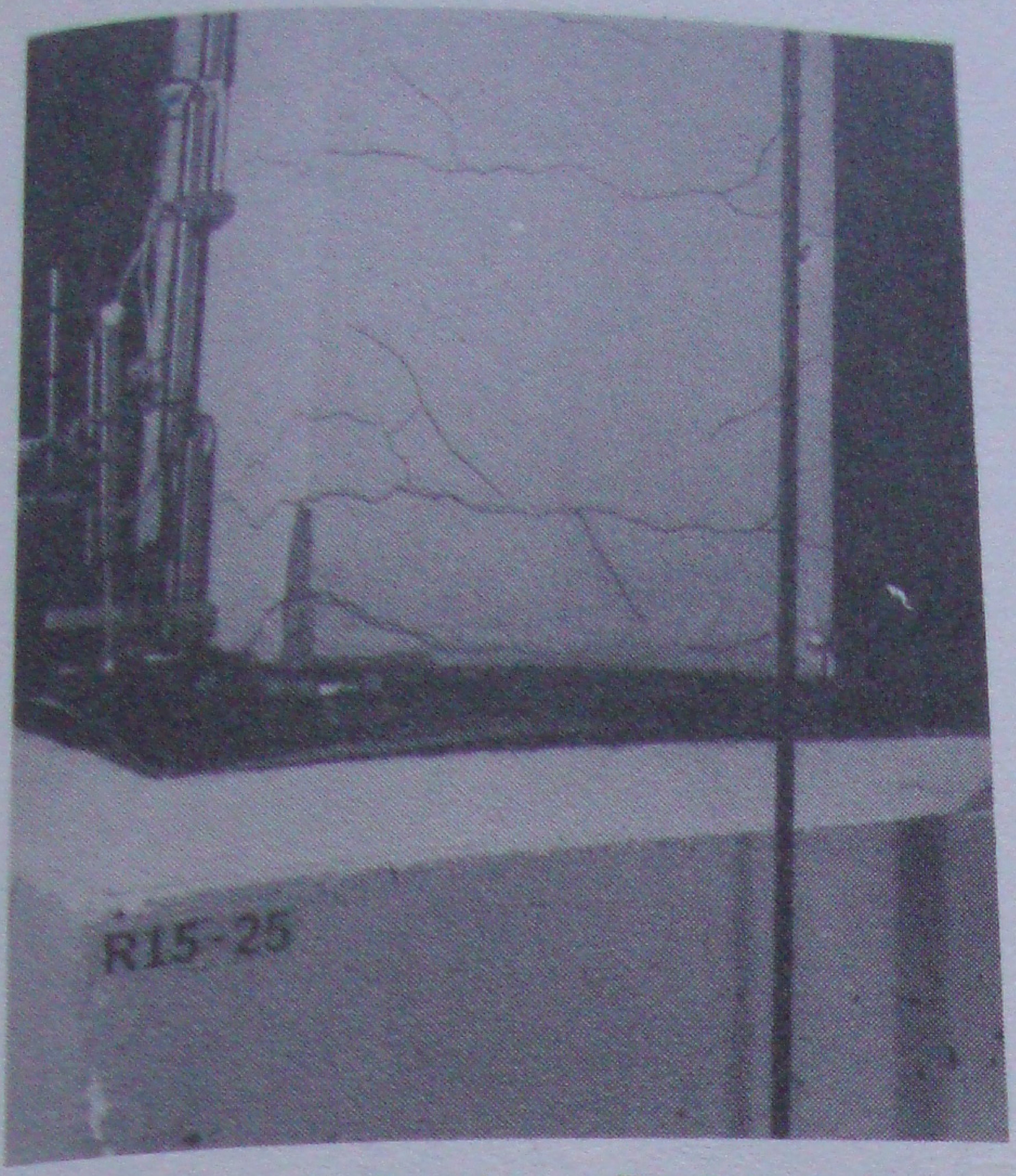
(a) Type 1 (b) Type 2 (c) Type 3

Figure 2: Types of connection details investigated

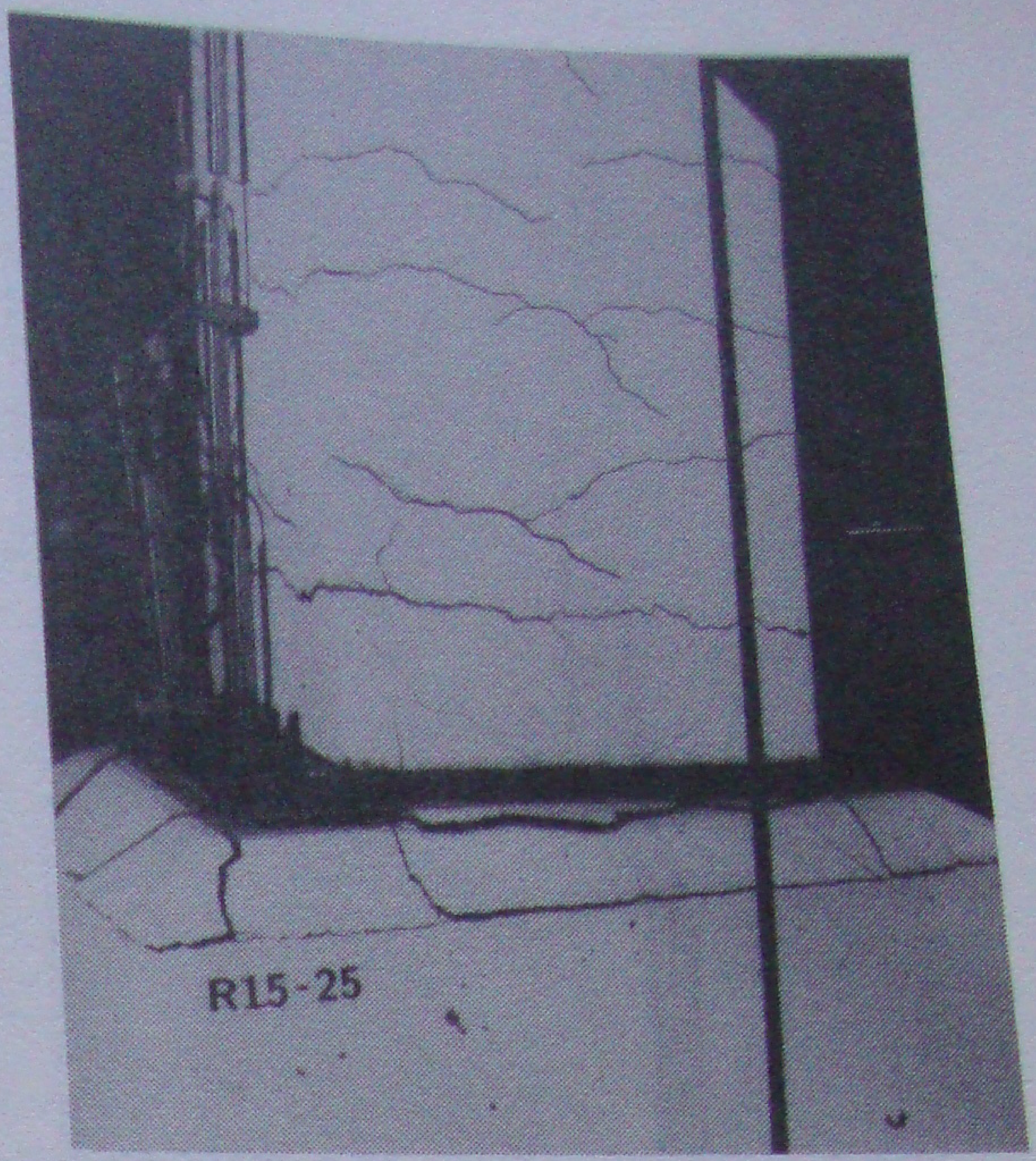
Figure 3 compares the connection deformations of connection Types 1, 2 and 3 for specimens designed with R = 1.5. Although the standard design method (PCI 1985, CPCI 1987) was used for Specimen R1.5-1 the base plate exhibited extremely large deformations due to bending in the direction of loading as well as in the transverse direction (see Fig. 3a). Excessive base plate yielding took place in the transverse direction while the column just reached yielding at its base. It is interesting to note that the standard design methods do not consider the transverse bending of the base plates. Specimen R1.5-2, having a base plate oversized only in the direction of loading, exhibited a much improved performance over Specimen R1.5-1 because of the elimination of transverse bending of the base plate (see Fig. 3b). The improved strength and stiffness of the connection enabled inelasticity to spread into the column. The base plate of Specimen R1.5-3 was designed using yield line analysis to account for bending across the corner between the plates forming the anchor bolt pocket. The connection stiffness and strength was similar to that of Specimen R1.5-2 with localized yielding taking place in the column just above the top of the anchor bolt pockets.

Figure 4a shows the base moment versus the measured top displacement of Specimen R1.5-2, indicating a displacement ductility of about 2.0. The overall performance was controlled by the anchor bolt deformations as can be seen from Fig. 4b. The pinching of the hysteretic response is due mainly to the irrecoverable elongation of the anchor bolts between the nuts located above and below the base plate. Figure 4c shows the contribution of the base plate bending together with the longitudinal column bar elongation over the thickness of the base plate. This displacement contribution is about equal to the anchor bolt displacement contribution. Figure 4d indicates that the bottom portion of the column exhibited some yielding consistent with the design level of ductility.

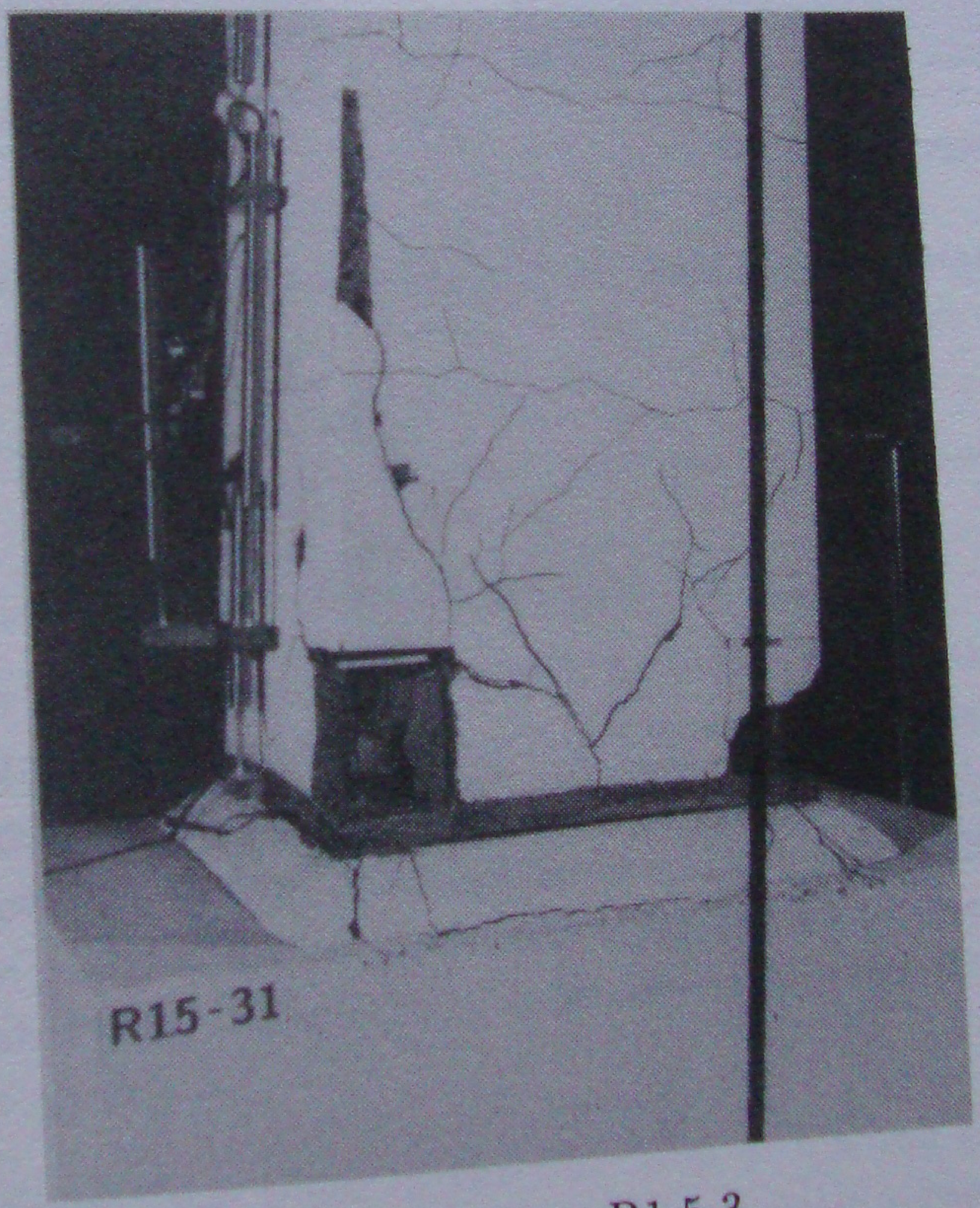
Figure 1b shows the deformed shape of Specimen R4P-1 which had a column designed to exhibit significant ductility but no special consideration was given to the transverse bending of the base plate. The deformed shape of the column shows that the base plate yielded and contributed to most of the top displacement while only localized yielding of the column bars took place at the base. Specimen R4-2 was designed with an oversized base plate in the direction of lateral loading and a capacity design approach was used to ensure that the connection was stronger than the column. As can be seen from Fig. 5a, the stiffer and stronger connection details allowed significant plastic hinging to develop at the column base with yielding extending over a height of 500 mm. As expected, the lapping of the dowel and column bars at the base of Specimen R4-3 produced a region of overstrength which did not result in a significant zone of plastic hinging at the base, see Fig. 5b. This overstrength together with the termination of the dowels resulted in a plastic hinge starting at the free end of the dowels and extending over a height of 250 mm. Due to the formation of the plastic hinge at a height of 750 mm above the column base, the resulting ductility was significantly reduced compared to that observed in Specimen R4-2.




(a) Specimen R1.5-1

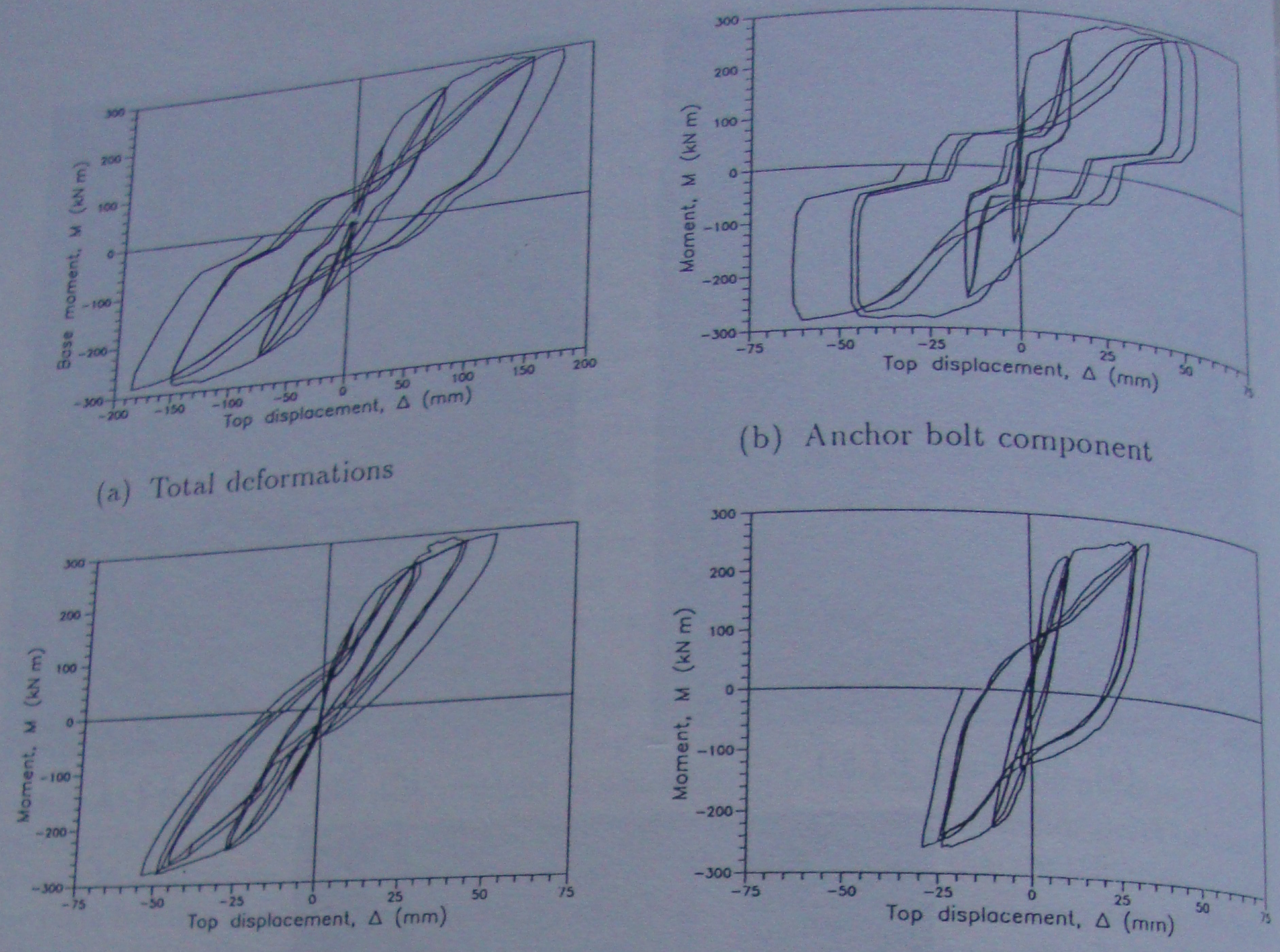



(b) Specimen R1.5-2




(c) Specimen R1.5-3

Figure 3: Deformations for connection Types 1, 2 and 3




(a) Total deformations

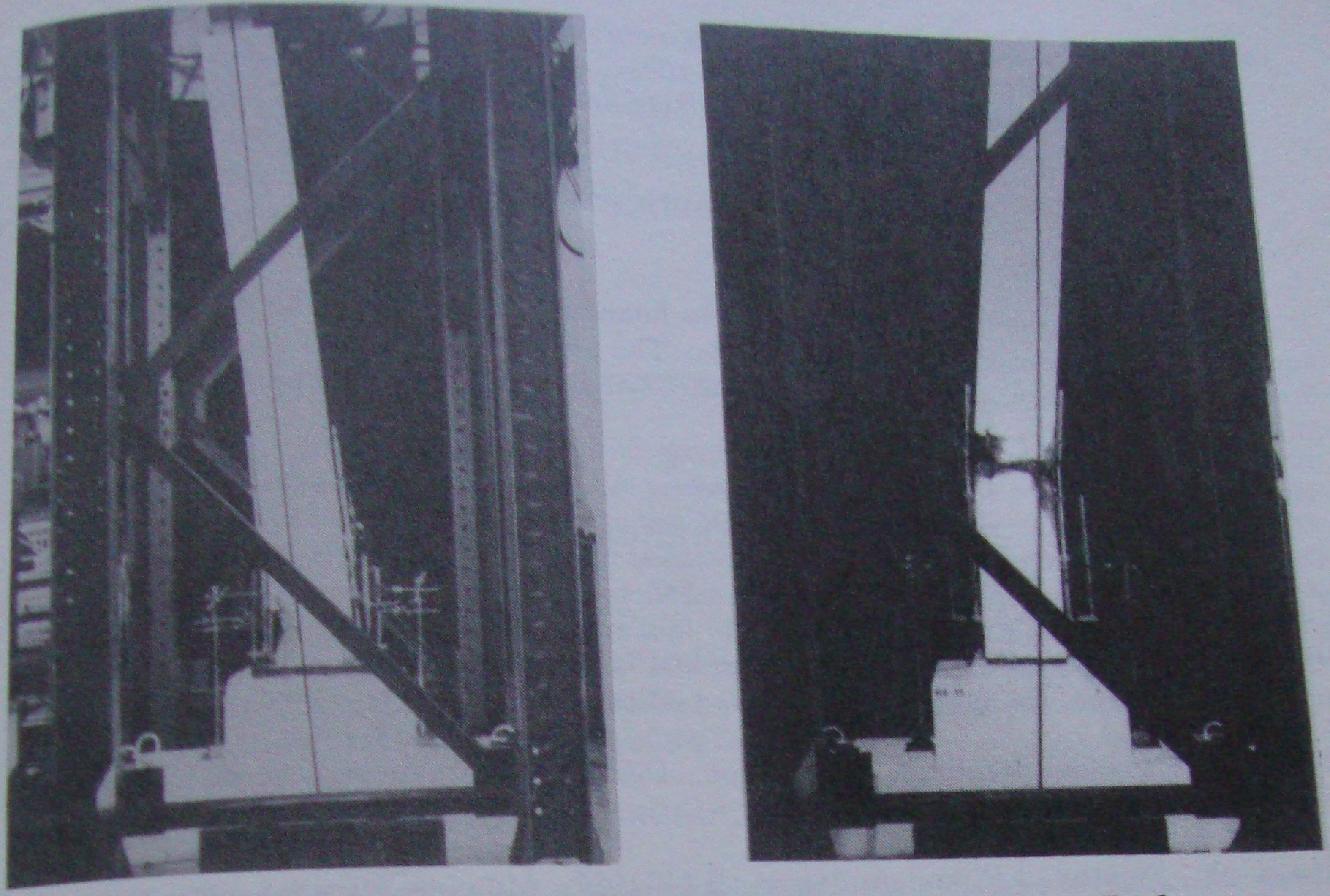
(b) Anchor bolt component

(c) Deformations in base plate and joint

(d) Deformations in bottom 415 mm of column

Figure 4: Moment versus top displacement components for Specimen R1.5-2

Specimens R4-1 and R4-3 exhibited displacement ductilities of about 4.0 with Specimen R4-1 showing poor hysteretic response. Specimen R4-2 had excellent hysteretic response and due to limitations of the loading system the testing had to be stopped at a displacement ductility of 4.0, however larger ductilities would have been possible.



(a) Specimen R4-2 (b) Specimen R4-3

Figure 5: Influence of connections on the overall responses

## CONCLUSIONS

The following conclusions can be drawn from this test series:

1. The current design procedure for precast column connections using oversized base plates (Type 1) results in poor seismic performance due to excessive base plate bending and yielding, without significant yielding developing in the column. The transverse bending of the base plate was a major factor contributing to the reduced stiffness and strength of the connection. This type of connection should not be used for situations requiring R greater than 1.5.

2. The use of an oversized base plate in the direction of loading (Type 2) improved the reversed cyclic loading response. If this is combined with a capacity design approach for the connection, excellent hysteretic response and significant displacement ductilities are attainable.

3. The use of recessed pockets (Type 3) gave acceptable reversed cyclic loading response for $R = 1.5$. However, if significant ductilities are to be attained then lap splices should not be used in regions where plastic hinges are expected.

## ACKNOWLEDGEMENTS

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