6th Canadian Conf. Earthquake Engineering / Toronto / 1991 / 6ème Conf. Canadienne Sismique

Bolted beam-to-column subassemblages under repeated loading

A. Osman[I], R.M. Korol[II], and A. Ghobarah[II]

ABSTRACT

Four beam-to-column subassemblages, representing parts of a typical moment resisting frame, were built and tested under a controlled cyclic displacement program. In these subassemblages, the beams were connected to the column flanges using extended end-plate connections. The tests were conducted to investigate the stiffness, strength, ductility, and the energy dissipation capacity of such a joint type and that of its individual components. Special emphasis was placed on the behaviour of the joint's elements, i.e the connection and the panel zone, and on their contribution to the overall response of the subassemblage.

INTRODUCTION

Current design specifications for steel structures in seismically active areas (UBC.1988 ;CAN3-S16.1-M89.1989) recommended that, the joints' panel zones in ductile moment resisting frames (MRFs) participate efficiently with the beams in dissipating the earthquake input energy. As a result, a design criterion that allows the columns' panels to yield and undergo sufficient inelastic deformation (about 2 to 4 γ_y , where γ_y is the panel average shear strain at yield) prior to the yielding of the beams was developed. In establishing such a criterion, the experimental results obtained by Krawinkler and Popov (1982) on beam-to-column subassemblages utilizing fully welded connections or connections with beam flange welded and beam web bolted to column flanges, were taken as the basis. The implication of adopting the same approach when other connecting media such as extended end-plate connections, employed to join beams to columns, was not investigated. The seriousness of such a problem can be understood from examining Fig.1. This figure shows the deformed shapes of the panel zones when both fully welded connections and extended end-plate connections are employed. As can be observed, the end-plate must deform in order to allow for the panel deformation, i.e the end-plate flexural stiffness contributes to the panel shear resistance. This contribution may be significant in the case of thick end-plates. Detailing the panel zones according to current design criteria without taking the end-plate contribution into account would generally result in

[I]Graduate Research Assistant, Department of Civil Engineering and Engineering Mechanics, McMaster University, Hamilton, Ontario L8S 4L7.

[II] Professors, Department of Civil Engineering and Engineering Mechanics, McMaster University, Hamilton, Ontario L8S 4L7.

relatively strong panels. As such, limited inelastic action would take place in the panel and dedicated most of the energy absorbtion to the beams. This contradicts the basis of earthquake design philosophy of requiring the input energy to be dissipated extensively throughout the structure.

Another important aspect that should be considered as a result of adopting such a joint type is the fact that the assumption of infinitely rigid connections can no longer be maintained. Recent research on end-plated connections showed that, relative rotation between the beam and the column can be observed at high loading levels (Ghobarah et al.1990). Consequently, the connection itself can contribute significantly to the overall drift of frames.

As a result, this study was conducted to investigate the cyclic behaviour of beam-to-column subassemblages utilizing extended end-plate connections. The main objectives are to gain information that can be helpful in detailing such a joint type in order to achieve good seismic performance.

EXPERIMENTAL PROGRAM

Description of subassemblages:

Four exterior joint subassemblages denoted as,CB-1, CC-1, CC-2, and CC-3, were tested. Each subassemblage consisted of a 2800 mm. long column connected to a beam at its mid height by an extended end-plate attachment (Fig.2). The connections were designed according to the design criteria proposed by Korol et al. (1991). The details of the joints are shown in Fig.3.

In designing each specimen, the strengths of the panel zones and the beams were deliberately changed relative to each other by altering the panel zone thicknesses. As a result, the effect of changing the panel zone strength on the overall behaviour of the subassembalge could be investigated. Table 1 shows the theoretical relative yield strength of the subassemblage components in terms of the beam tip load. It should be noted that the columns were designed to remain essentially elastic throughout the tests.

Test setup and procedure:

A special setup which allows application of the axial load to the columns while subjecting the beams to a cyclically controlled displacement was constructed. Fig.4 shows an illustration of the test setup. During the tests a constant axial load was initially applied to the columns, after which the beams subjected to a displacement program as in Fig.5.

The applied loads, the beam-tip deflection, the panel zone deformation, the connection rotation, and the internal stresses at various locations in the specimens were recorded. A more detailed description of the instrumentation used is reported elsewhere (Ghobarah et al.).

EXPERIMENTAL RESULTS

A summary of experimental results for the four tests is given in Table 2. The table shows for each test the maximum beam load recorded, P_{max}, the corresponding beam tip displacement , Δ_{max}, the maximum panel zone average shear strain, γ_{max}, and the maximum plastic beam rotation,θ_p. Also, the overall hysteretic behaviour for two of the tested specimens is presented in Fig.6.

Examining Table 2 and Fig.6 shows that with the exception of specimen CC-2, all the specimens exhibited stable hysteretic behaviour. The strength deterioration observed in specimen CC-2 was attributed to severe local buckling of its beam. This buckling was triggered by the high demand imposed on the beam as a result of stiffening the panel zone. The latter responded elastically with only localized plastification. In specimen CB-1, the panel underwent several inelastic strain reversals and suffered shear buckling. However, this did not result in significant strength deterioration as there evolved the formation of an alternative system for transmitting the forces through the panel (diagonal tension field). In specimens CC-1 and CC-3, the panel participated jointly with the beam in dissipating the input energy. This imposed relatively low demands on each component resulting in good subassemblage performance.

Behaviour of connections:

Fig.7 shows a typical moment-rotation hysteretic behaviour of a connection in one of the tested specimens (CC-3). As can be seen, the connection suffered degradation in its stiffness with the progression of loading. Similar response was observed previously in cyclic tests conducted on extended end-plate connections. (Osman et al. 1990).

Behaviour of panel zones:

Figs.8 and 9 show the responses of the panels in specimens CB-1 and CC-3. In these figures, the theoretical applied moments required to yield the panels and the theoretical moments corresponding to the panel zones shear strengths are shown by a solid and dotted lines, respectively. It should be noted that, in calculating these moments the effect of axial force was neglected and the equation recommended by the codes to calculate the panel shear strength without taking the resistance provided by the end-plate into account was used. This equation is given by:

$$V_u = 0.55 d_c t_{cw} F_y [1 + \frac{3 b_c t_{cf}^2}{d_c d_b t_{cw}}] \qquad (1)$$

where

t_{cw} = the total thickness of the joint panel including doubler plates.
d_b = the depth of the beam.
d_c = the column depth.
b_c = the width of the column flange.
t_{cf} = the thickness of the column flange.

It should be noted that Eq.1 gives the shear resistance corresponding to a distortion of approximately 4 γ_y in the joint. Comparing these theoretical results with the experimental results revealed the following:
1) The predicted moments required to yield the panels in specimens CB-1 and CC-3 were 1.26 and 1.10 that of the actual recorded moments, respectively.
2) The predicted moments corresponding to the shear strengths of the panels in specimens CB-1 and CC-3 were 0.79 and 0.71 that of the actual recorded strengths at distortion of 4 γ_y, respectively.

Early yielding of the panels was attributed to the presence of the axial forces. However, the recorded high shear resistance of the panels in the post- elastic range was attributed to the stiffening effect of the end-plate. As can be seen, this effect counted for 21 to 29% of the total resistance.

Analysis of the panels' responses in other specimens (CC-1 and CC-2) supports the previous conclusions. Also, it was observed that the doubler plate welded to the panel participated efficiently in resisting the panel shear deformation.

Contribution of joints to overall behaviour:

The contribution of the individual components in the subassemblage to its overall behaviour can be expressed either in terms of the cumulative energy dissipated by each component separately, or in terms of the components' participation to subassemblage deflection. The first approach is considered important when assessing the ductility and the capacities of the components to undergo several strain reversals without failure. The second approach is desirable when drift is the issue. Both approaches were applied to the tested specimens. The results for specimen CC-3 are shown in Figs.10 and 11. As can be seen, the panel, the connection and the beam dissipated 28% ,13% ,and 59% of the total input energy ,respectively. In terms of the displacement, the panel, the connection, the column, and the beam contributed 21.5%, 16%, 17.5% and 45% to the total deflection. Similar results were obtained for other specimens. This shows the importance of incorporating joint behaviour in the analysis of MRFs.

CONCLUSIONS

Based on the previous investigation, the following conclusions may be drawn:

1. The panel zone is a very ductile element that can undergo several inelastic reversals without signs of distress.

2. The extended end-plate joints contributes significantly to the frame's interstorey drift and neglecting such an effect in the analysis will lead to serious errors.

3. Excellent seismic performance can be achieved by allowing both the panel zone and the beam to participate jointly in dissipating the input energy.

4. The end-plate as an adjoining element to the panel zone contributes significantly to the post-elastic panel shear strength.

5. Adopting current design criteria for detailing the panel zones in the case of extended end-plate joints will lead to strong panels that can impose higher demands than expected on the beams.

ACKNOWLEDGEMENTS

The authors wish to acknowledge the support of the Natural Science and Engineering Research Council of Canada. This work is carried out under NSERC grants to McMaster University.

REFERENCES

Canadian Standards Association. 1989. Limit state design of steel structures. CSA-Standard, CAN3-S16.1-M89, Canada.

Ghobarah, A., Osman, A.,and Korol, R.M.1990. Behaviour of extended end-plate connections under cyclic loading. Engineering structures,Vol.12, No.1,15-27.

Ghobarah, A., Korol, R.M.,and Osman, A. Cyclic behaviour of extended end-plate joints. J. Struct. Div., ASCE, to be published.

Korol, R.M., Ghobarah, A.,and Osman, A.1991. Extended end-plate connections under cyclic loading: behaviour and design. J. Const.steel Research, in print.

Krawinkler, H.,and Popov, E.P. 1982. Seismic behaviour of moment connections and joints. J. Struct. Div.,

ASCE, 108, ST2.
Osman, A., Korol, R.M.,and Ghobarah, A. 1990. Seismic performance of extended end-plate connections.Proceeding of 4th U.S National Conference on Earthquake Engineering, Palm Springs, CA, Vol.2.
Uniform Building Code. 1988. International Conference of Building Officials, Whittier, CA.

Table 1. Yield strength of specimen components

specimen	P_{ybeam}	P_{ypanel}	P_{yconn}
CB-1	159	105	103
CC-1	172	163	218
CC-2	172	275	218
CC-3	181	163	414

All loads are in kN.

Table 2. Summary of experimental results

specimen	P_{max} (kN.)	Δ_{max} (mm.)	γ_{max} (rad.)	θ_{max} (rad.)
CB-1	175	145	0.041	-
CC-1	250	135	0.015	-
CC-2	250	115	0.004	0.057
CC-3	260	140	0.012	0.032

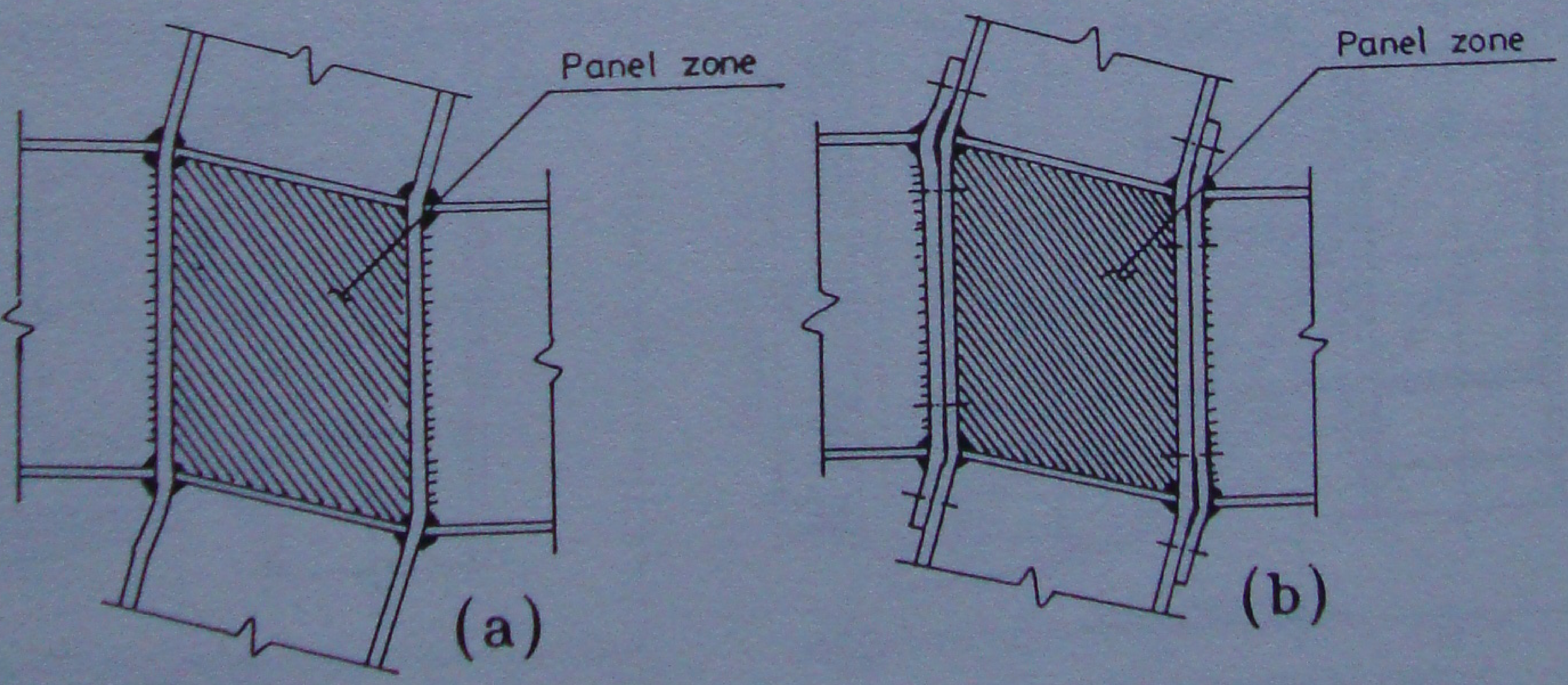

Fig.1 Deformed panel zones

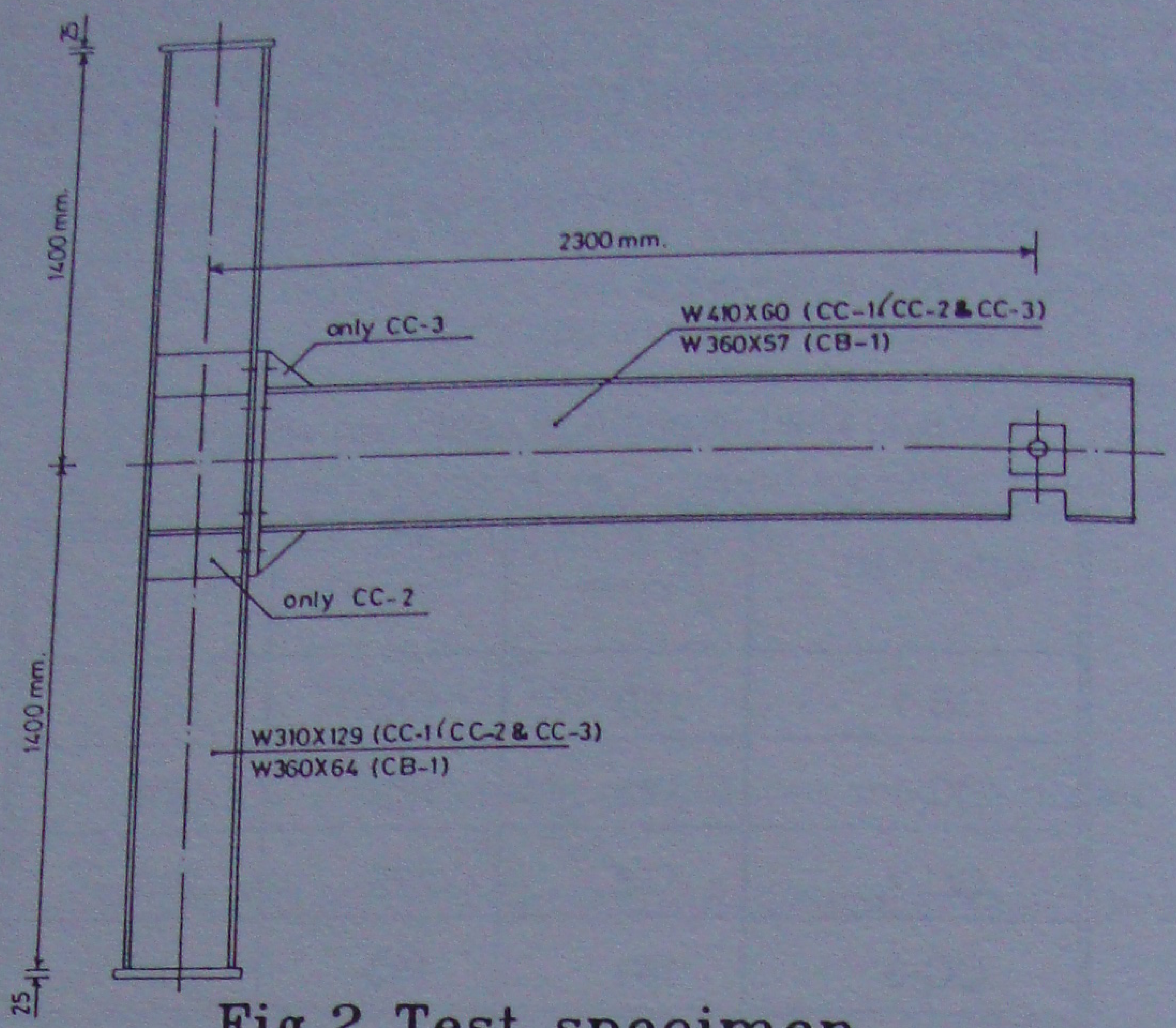

Fig.2 Test specimen

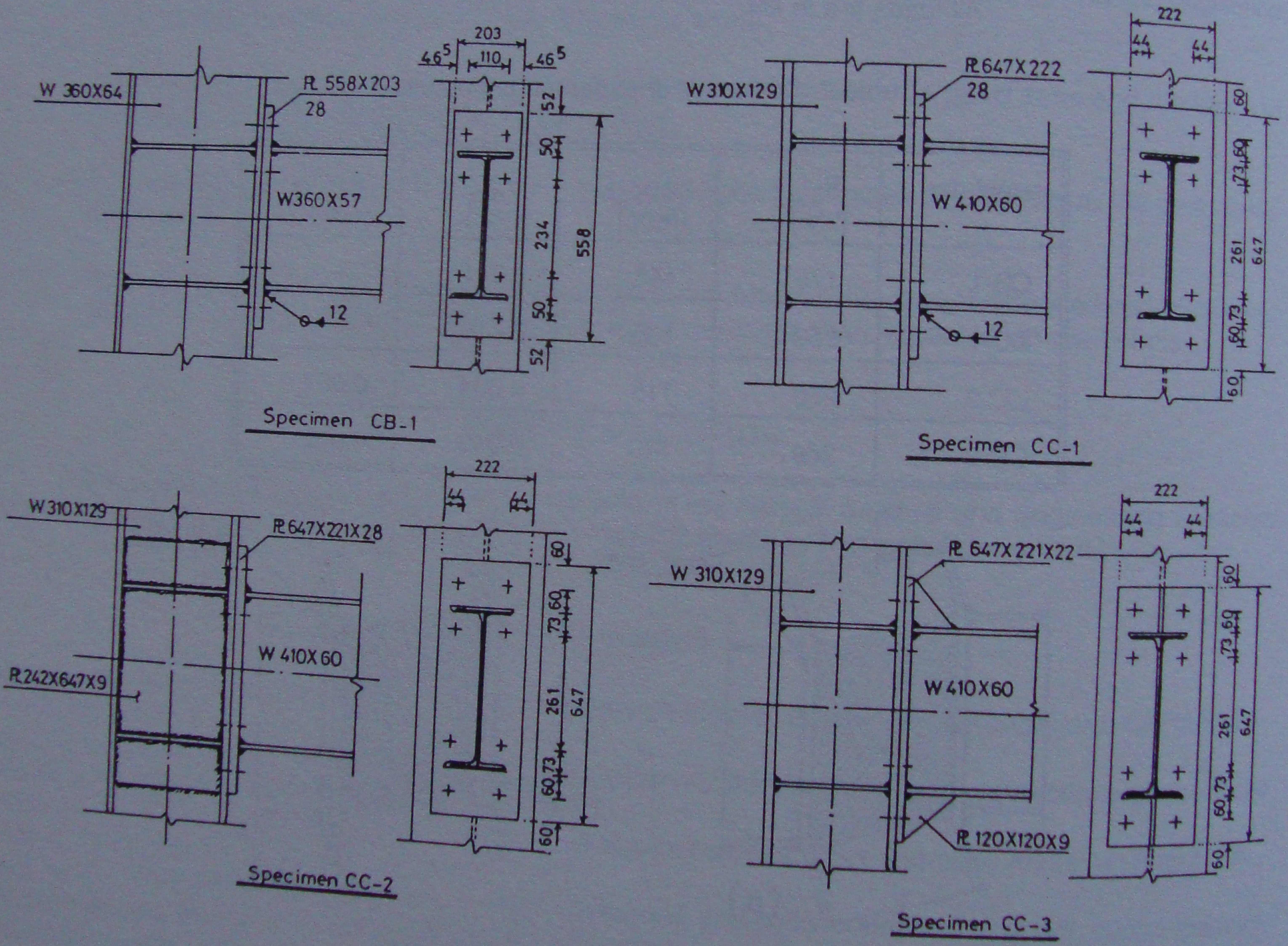

Fig.3 Joints details.

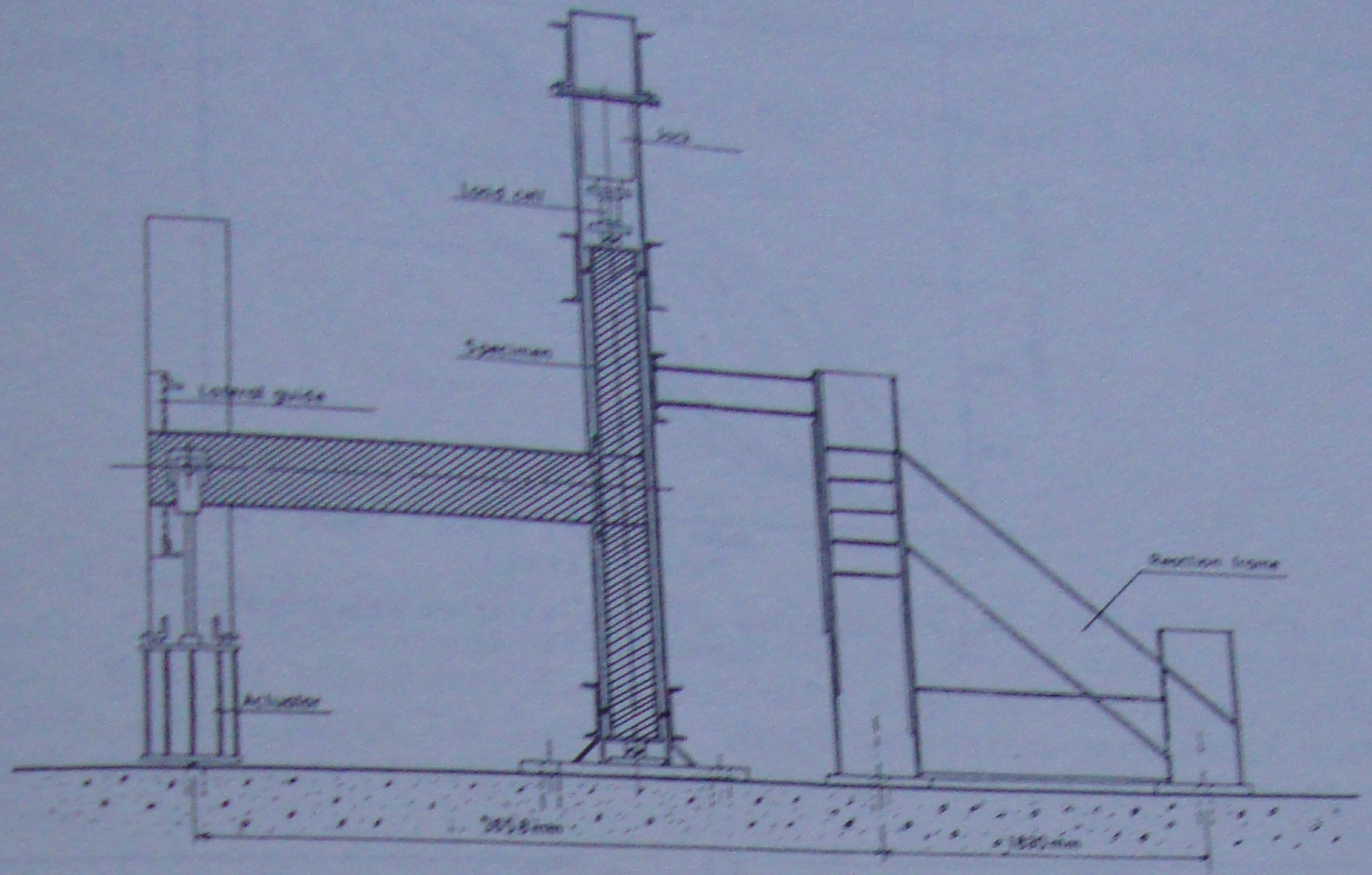

Fig.4 Test setup.

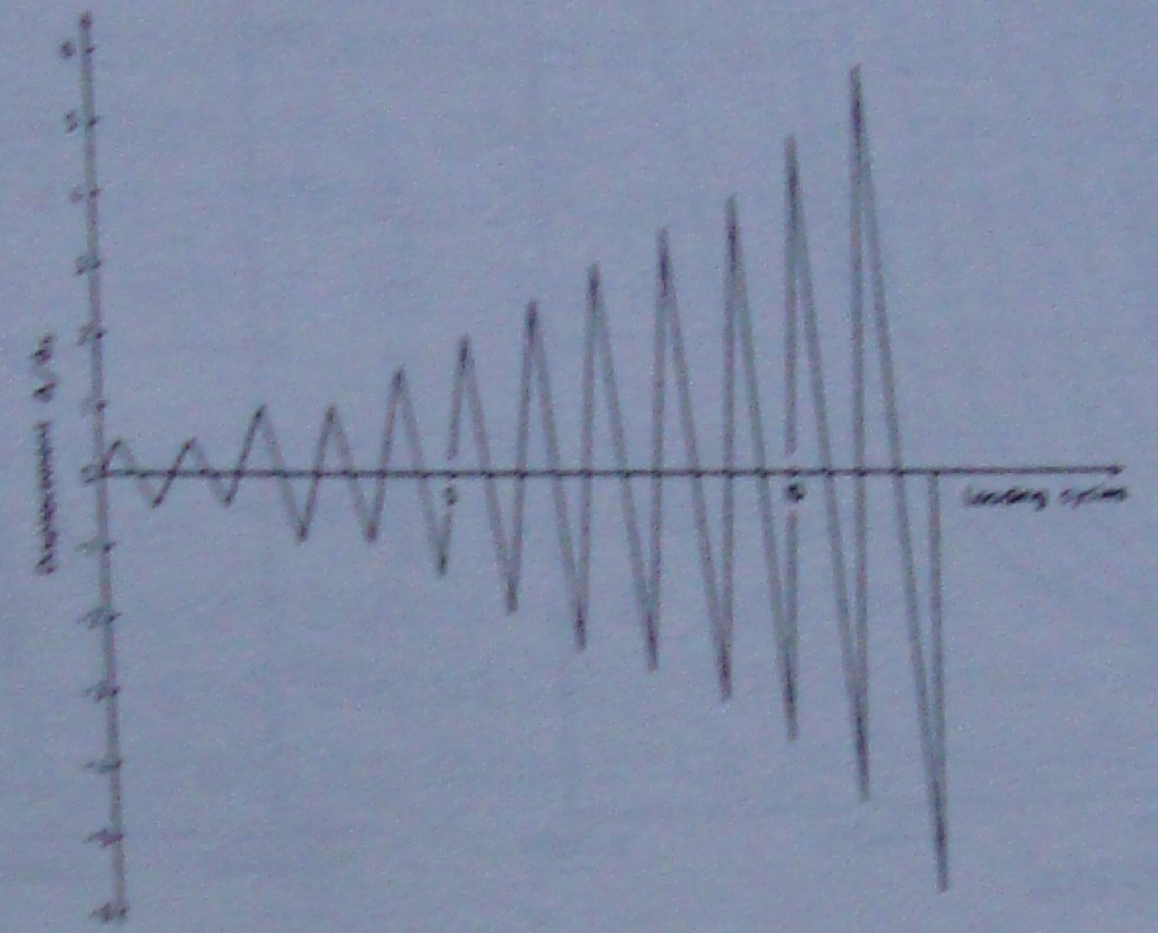

Fig.5 Loading routine.

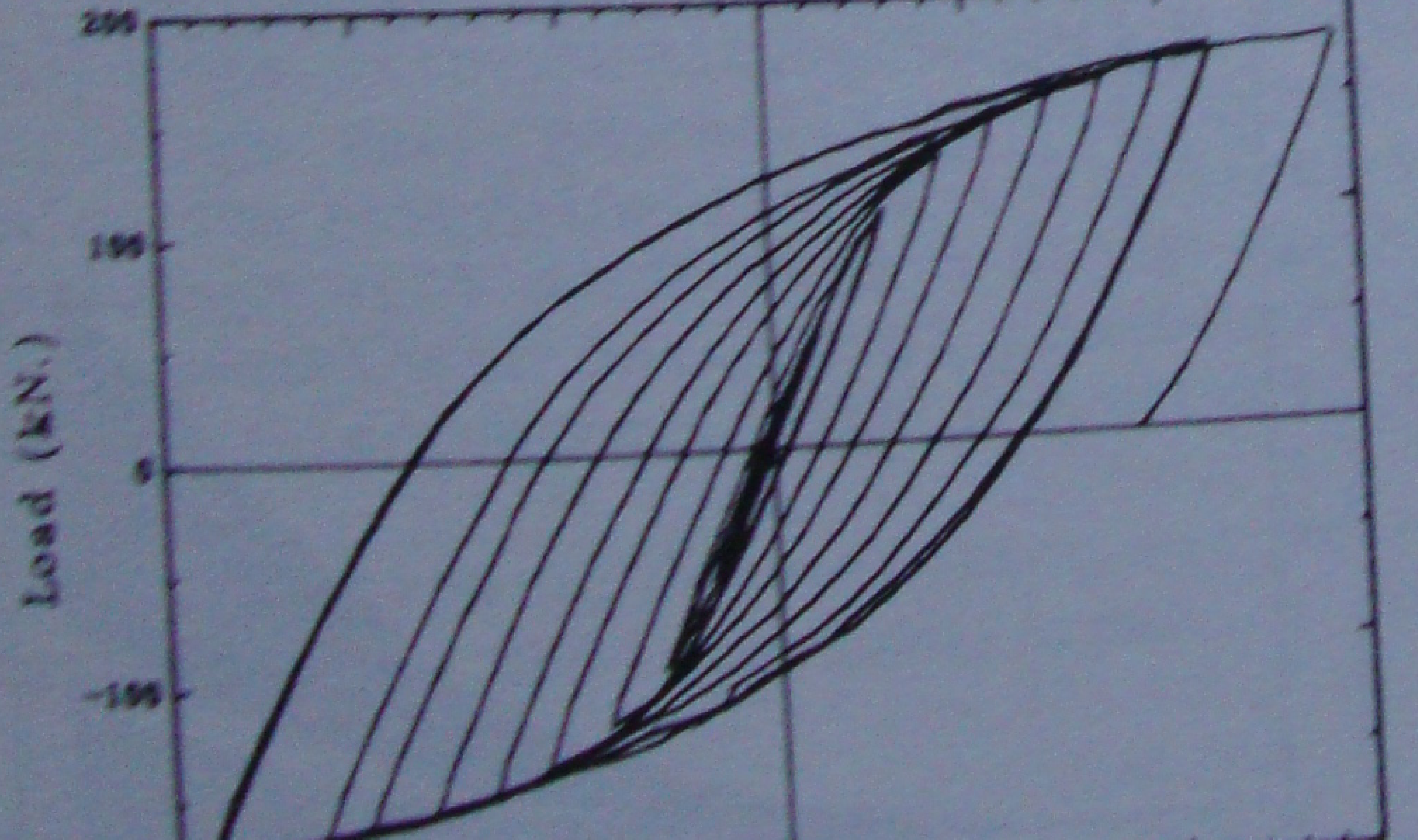

Fig.6(a) Beam tip load-deflection relationship for specimen CB-1.

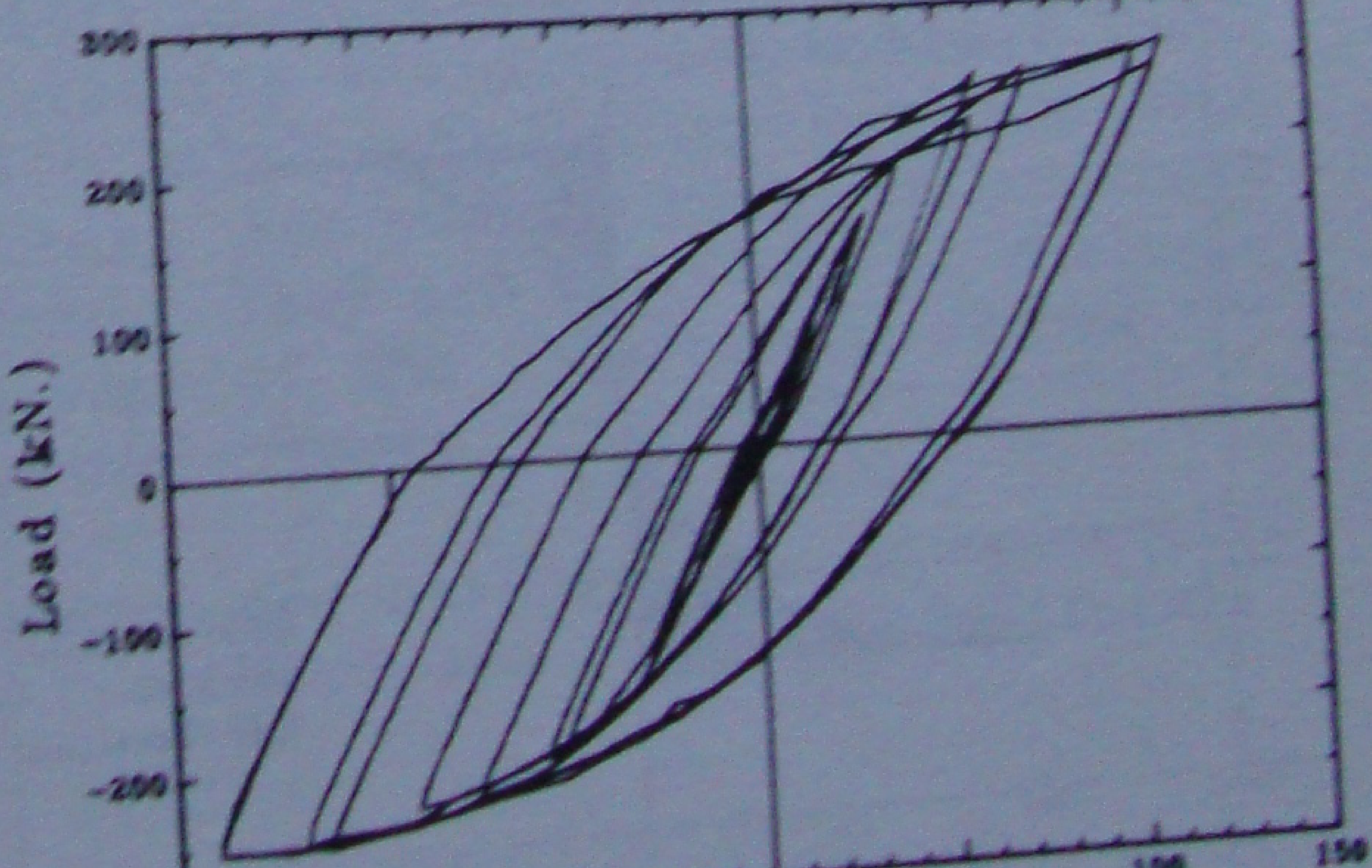

Fig.6(b) Beam tip load-deflection relationship for specimen CC-3.

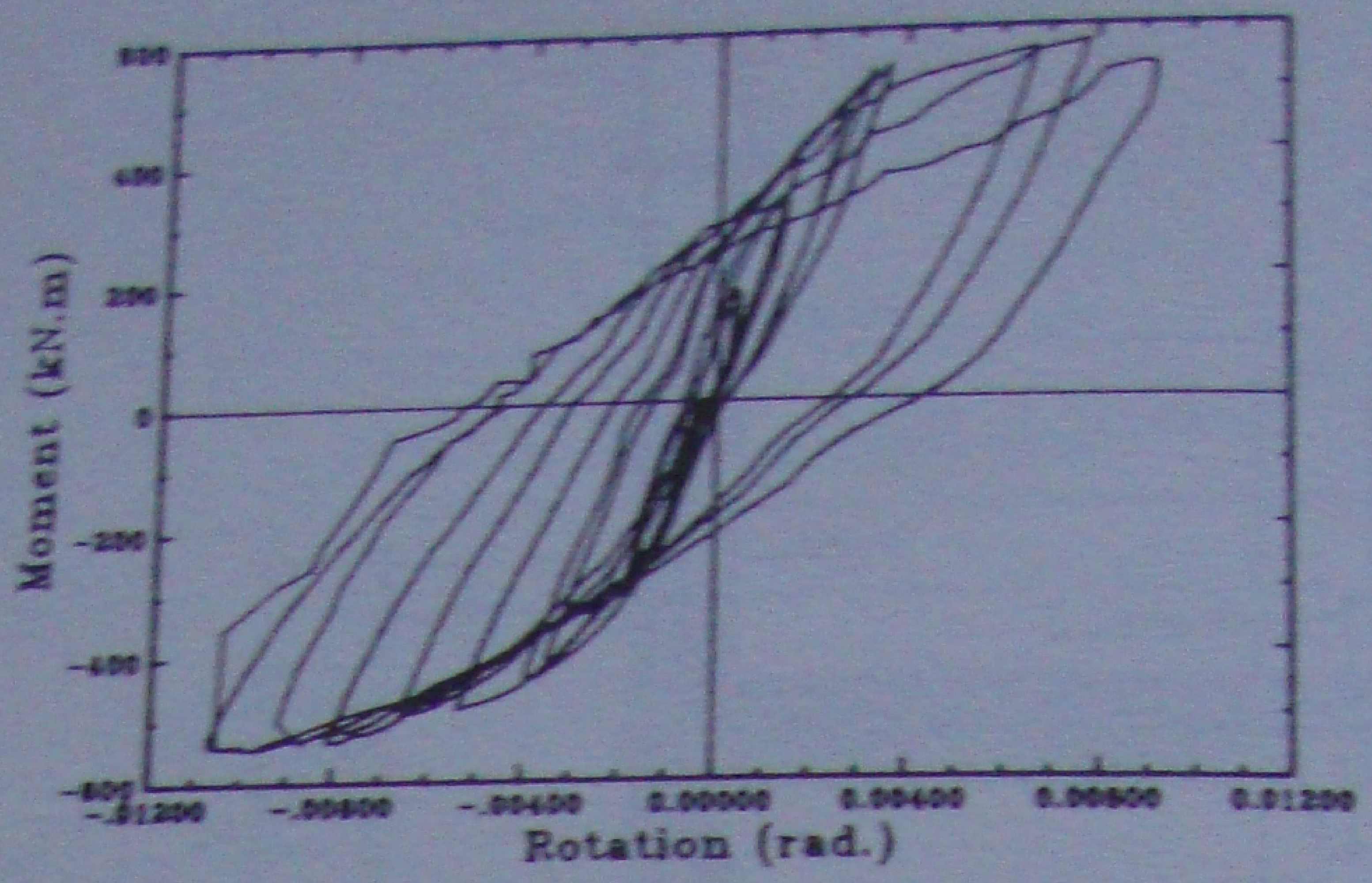

Fig.7 Connection moment-rotation relationship (specimen CC-3).

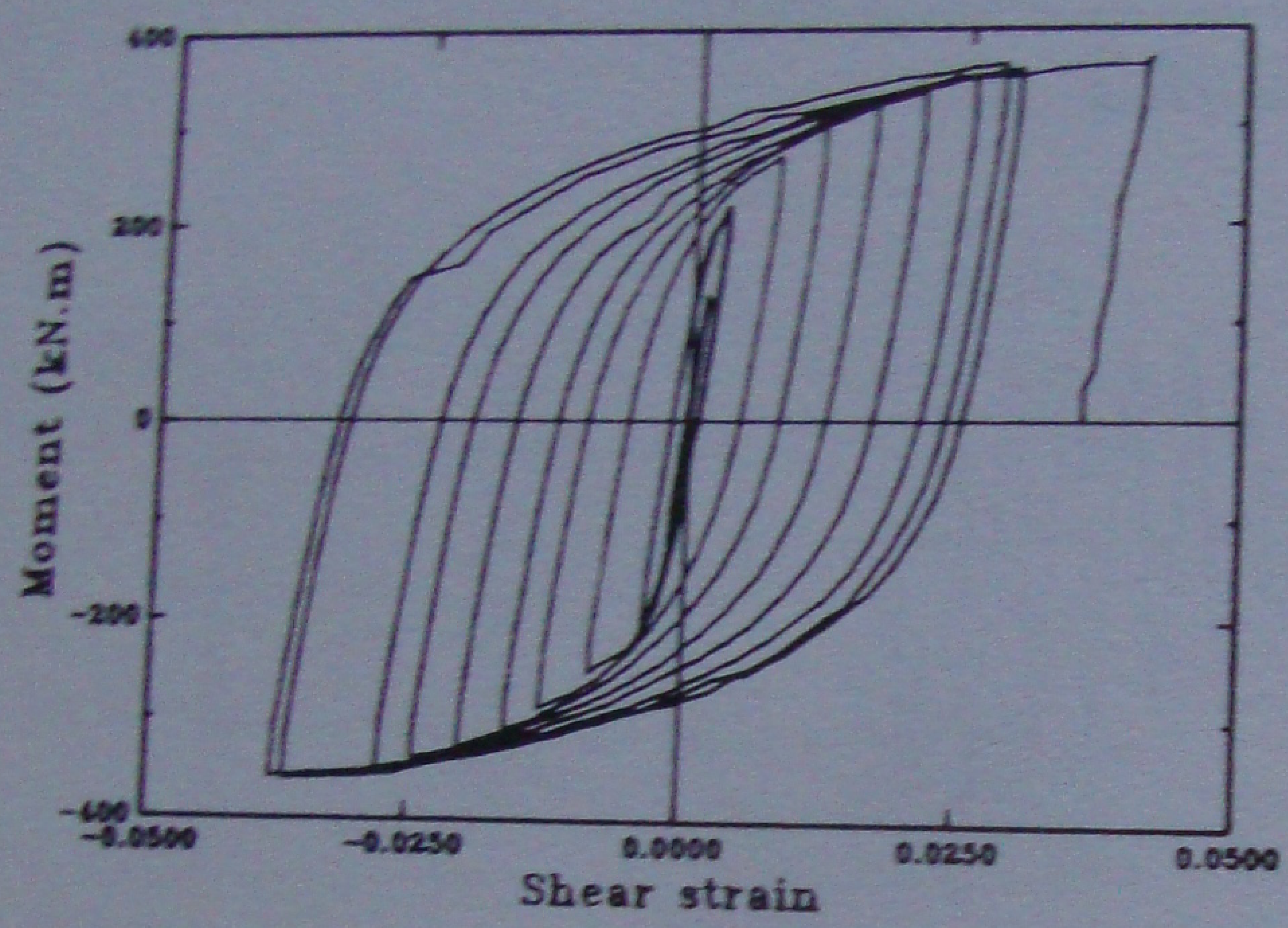

Fig.8 Applied moment-shear strain relationship for the panel zone (specimen CB-1).

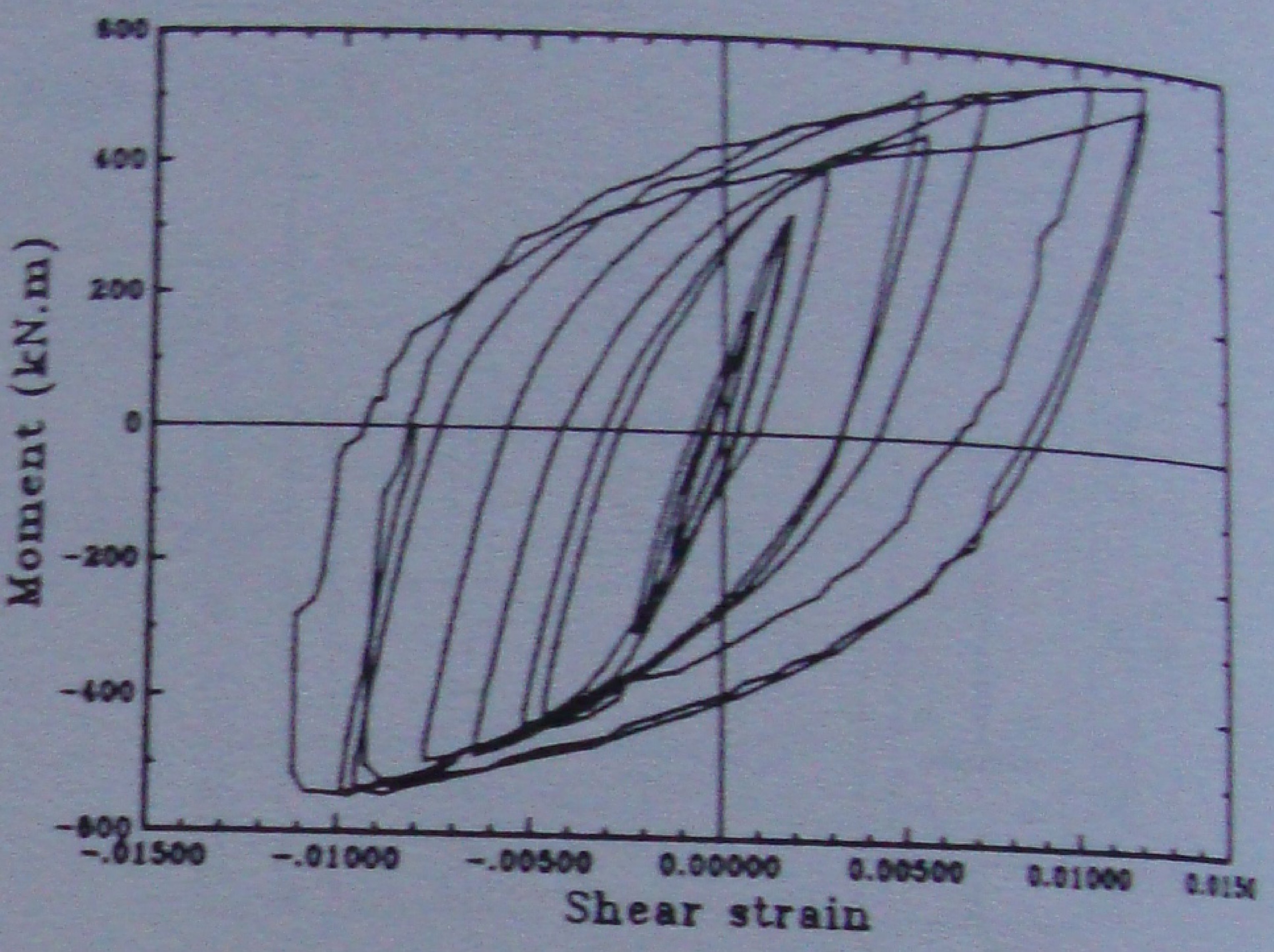

Fig.9 Applied moment-shear strain relationship for the panel zone (specimen CC-3).

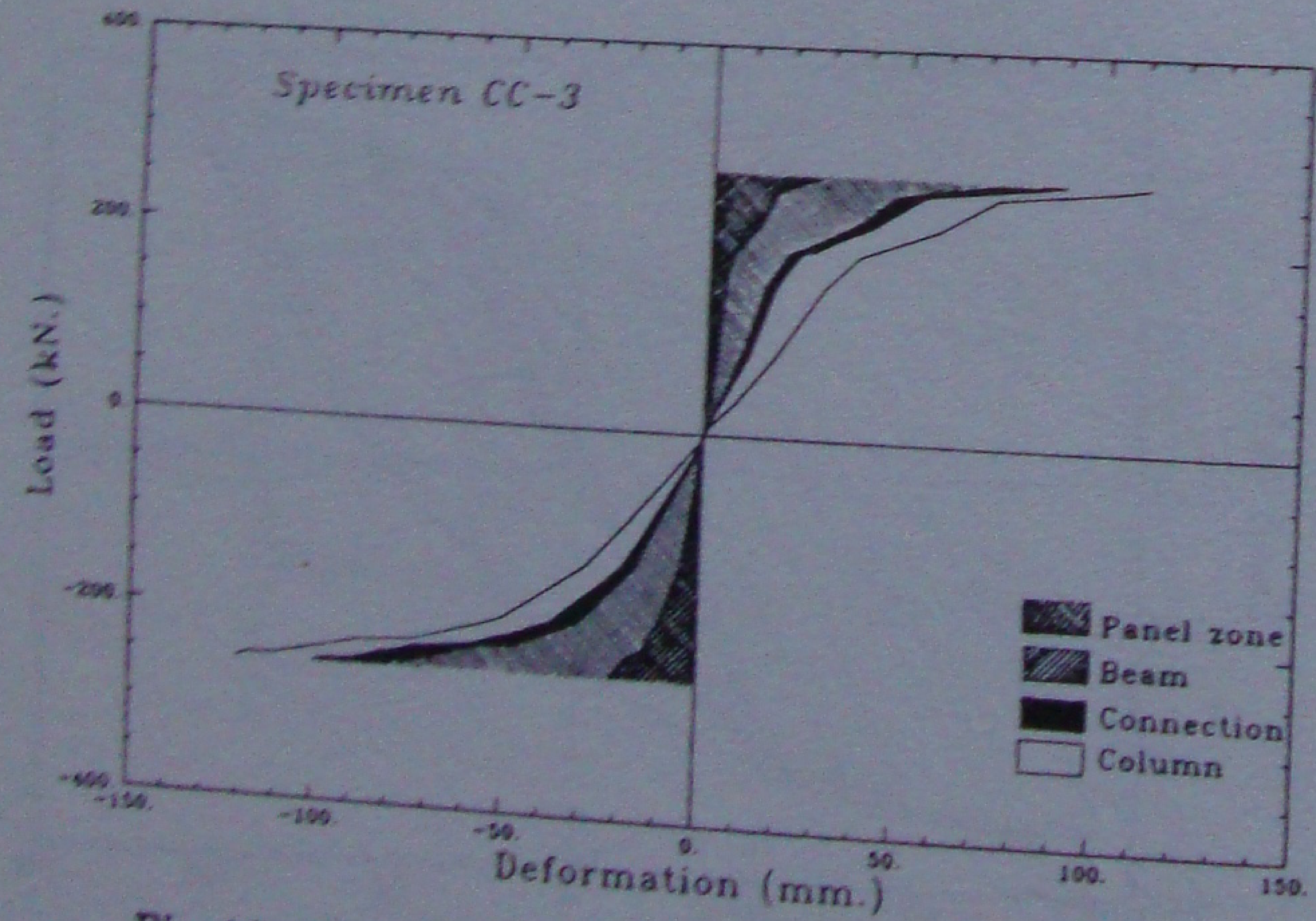

Fig.10 Contribution of specimen components to beam-tip deflection (specimen CC-3).

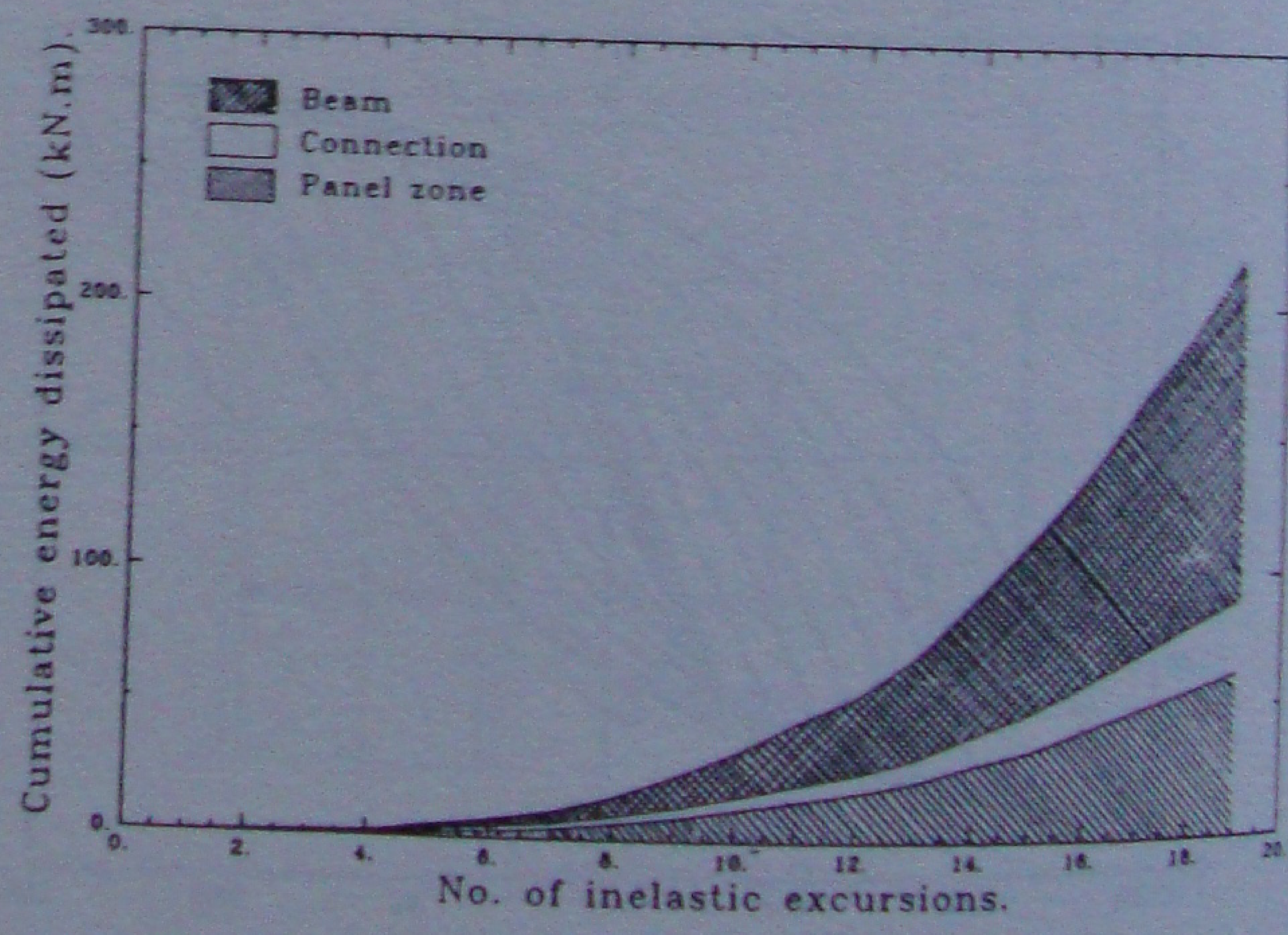

Fig.11 Cumulative energy dissipated by each component in specimen CC-3.