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Behaviour of panel zones in extended end-plate joints

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ABSTRACT: The results of a series of experiments conducted on beam-to-column subassemblages utilizing extended end-plate joints are reported. The subassemblages were tested under controlled cyclic displacement. Emphasis was placed on the cyclic behaviour of panel zones in the tested specimens. Based on the experimental results, a simple analytical model capable of simulating both the monotonic and the cyclic behaviour of the panel zones was proposed.

1 INTRODUCTION

Experimental research on the reversed cyclic loading response of building components, together with the availability of accurate modelling for their behaviour, provides the bases for predicting the performance of structures during severe earthquakes. In a moment resisting steel frame (MRF), the beam-to-column joint panel zone is considered one of the frame's components that can affect substantially its response. Attention to the importance of studying the cyclic response of panel zones was first drawn by Krawinkler et al (1971) and Bertero et al (1973). Based on their investigations, they concluded that by properly designing and detailing the joint panel zone, the MRF performance can be improved. Furthermore, motivated by these findings, new design concepts which allow dissipation of earthquake input energy in the panel zones were proposed (Krawinkler 1978 and Kawano 1984). However, all these previous investigations focused mainly on examining the response of panel zones in fully welded joints. No work was done to examine the behaviour when other connecting media were employed. Also, most of the earlier proposed models were either semi-empirical in nature (Krawinkler 1978) or a complicated analytical model based on finite element (Lui and Chen 1986).

On the basis of this review, there is a need to study the cyclic behaviour of panel zones when other connecting media rather than full welding joints is employed. Also, there is still a need for developing a simple analytical model to simulate the panel zone behaviour.

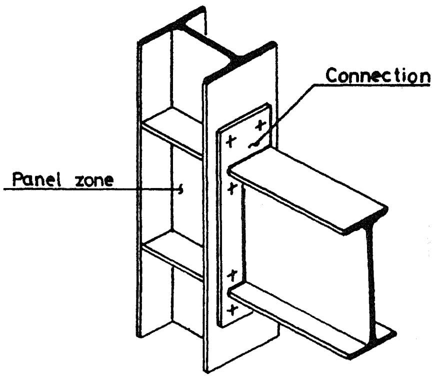

Figure 1. Typical extended end-plate joint

In this study an experimental-analytical program was initiated. The main objectives of this program is to investigate the cyclic behaviour of panel zones in beam-to-column joints utilizing extended end-plate connections (Figure 1). Then, based on these experimental investigation a simple analytical model that is capable of simulating the panel zone cyclic response can be formulated.

2 EXPERIMENTAL PROGRAM

2.1 Description of subassemblages

Four subassemblages designated as CB-1, CC-1, CC-2 and CC-3 were tested. Each specimen consisted of a single column 2800 mm long connected at its mid-height to a cantilever beam of 2300 mm length by an extended end-plate connection (Figure 2). The

Table 1. Details of tested specimens

Specimen No.	Beam size[a]	Column size[a]	End-plate thickness (mm)	Panel zone thickness (mm)
CB-1	W360x57	W360x640	28	08
CC-1	W410x60	W310x129	28	13
CC-2	W410x60	W310x129	28	22*
CC-3	W410x60	W310x129	22	13

[a]Depth in mm, mass in Kg/m.
*Web was reinforced with 9 mm thick doubler plate.

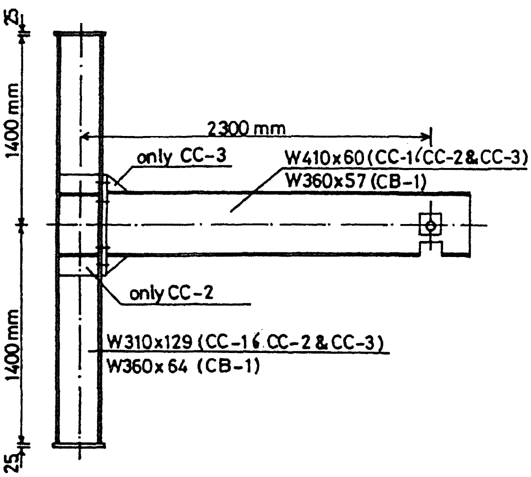

Figure 2. Beam-to-column subassemblage

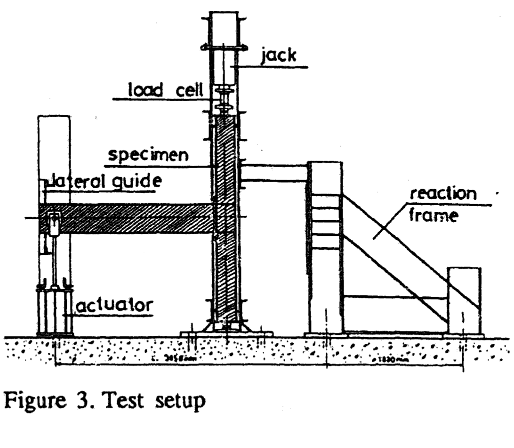

Figure 3. Test setup

connections were designed in accordance to the design criteria proposed by Korol et al (1991).

Table 1 shows the beam and column sections used for the tested specimens. In addition, it provides information on the Joints configuration. A more detailed description for the tested specimens and the mechanical properties of the material used can be found in Ghobarah et al (1992).

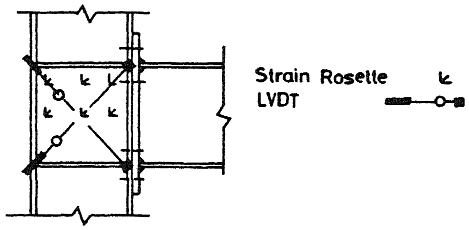

Figure 4. Panel zone instrumentation

In the design of the test specimens, the panel zone shear resistance was deliberately changed by altering the panel zones thicknesses. As a result, the panels shear strength varies from weak panels such as in the case of specimen CB-1 to strong panels as in specimen CC-2. Specimens CC-1 and CC-3 posses panels, with moderate thicknesses.

2.2 Test setup and procedure

During each experiment, a constant axial load was applied to the column, while the beam was subjected to a cyclic controlled displacement at its tip. The setup used for such purpose is shown in Figure 3.

Throughout the tests the panel zones behaviour were continuously monitored. LVDTs positioned diagonally over the panel as shown in Figure 4 and strain rosettes glued to the panel web were used to record both the panel overall response and the progress of plastification throughout the panel.

3 EXPERIMENTAL RESULTS

For each test, the maximum attained moment during the test, M_{max}, the maximum panel zone shear deformation, γ_{max}, the panel ductility, μ, and the amount of energy dissipated by the panel, E_d, were recorded. These results as measured from the tests are summarized in Table 2.

Examining this table reveals that, the panel zone in specimen CB-1 which is recognized as a weak panel experienced large shear deformation and participated efficiently in dissipating the input energy (80% of the total input energy). On the other hands, panel zones in specimens CC-1 and CC-3 designated as moderate panels experienced only a limited shear deformation and dissipated only 33% of the total input energy. In the case of specimen CC-2, the panel was strong and responded elastically.

During the experiments, the panels overall

Table 2. Summary of experimental results

Specimen	M_{max} (kN.m)	γ_{max} (rad)	μ^*	E_d (kN.m)
CB-1	383	.041	17.98	150.
CC-1	565	.015	07.21	69.1
CC-2	565	.004	01.34	11.4
CC-3	588	.012	05.64	62.4

*Ductility is calculated by dividing the absolute maximum panel zone shear deformation, γ_{max}, by the deformation at panel yield, γ_y.

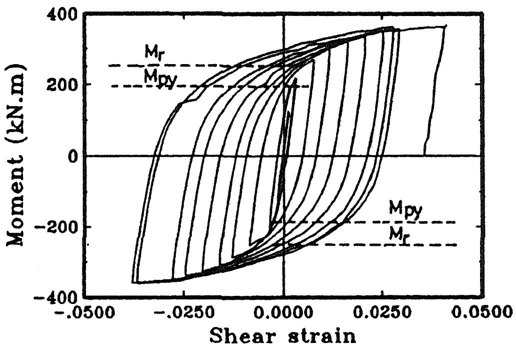

Figure 5. Applied moment-shear strain relationship for the panel zone (Specimen CB-1)

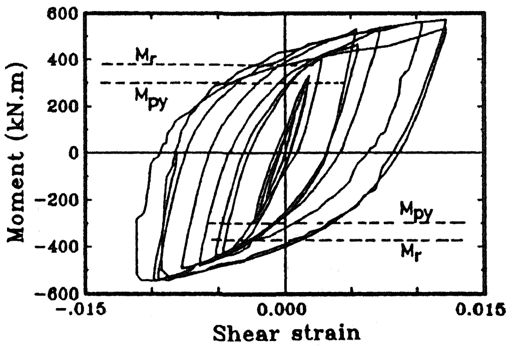

Figure 6. Applied moment-shear strain relationship for the panel zone (Specimen CC-3)

behaviour represented as the relationship between the panel applied moment, M_p, and the panel shear deformation, γ were recorded. Figures 5 and 6 show the panels behaviour for specimens CB-1 and CC-3.

As can be noted, the panel zones in these specimens exhibit stable, repetitive hysteretic behaviour without any signs of distress or stiffness degradation. Such behaviour was typical for all the tested panels even when the panel zone buckled such as in the case of specimen CB-1. Also, the figures show the panels excellent capacities to dissipate energy.

Indicated on the same figures by dotted lines the theoretical moments required to yield the panels, M_{py}, as determined from the following equation

$$M_{py}=0.55 d_c t_{cw} \sigma_y d_b \tag{1}$$

Where

d_c = The column depth
d_b = The beam depth
t_{cw} = The panel zone thickness
σ_y = Yield stress of the column material

Also, noted on the figure, the ultimate panel resistance, M_r, as calculated from the following formula provided by the codes (CAN3-S16.1-M89 and UBC 1988)

$$M_r=0.55\sigma_y d_c t_{cw} d_b[1+\frac{3b_{cf}t_{cf}^2}{d_b d_c t_{cw}}] \tag{2}$$

Where

t_{cf} = The column flange thickness
b_{cf} = The column flange width

As can be noted, all panels sustained loads higher than their nominal yield load. Also, the code formula severely underestimated the panel zone resistance. The reason is that the formula neglects the stiffening effect of the end-plate. This effect counted for 21% to 29% of the resistance.

A more rigorous analysis to the panels behaviour shows that the panel response is characterized by elastic behaviour followed by a post-elastic response

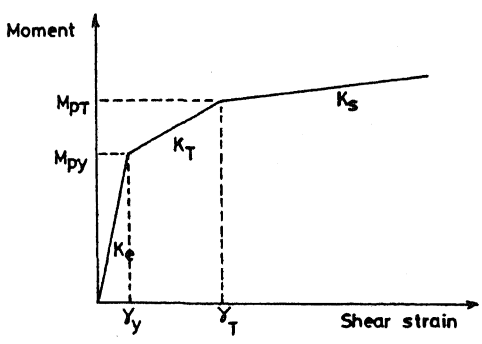

Figure 7. Proposed tri-linear model

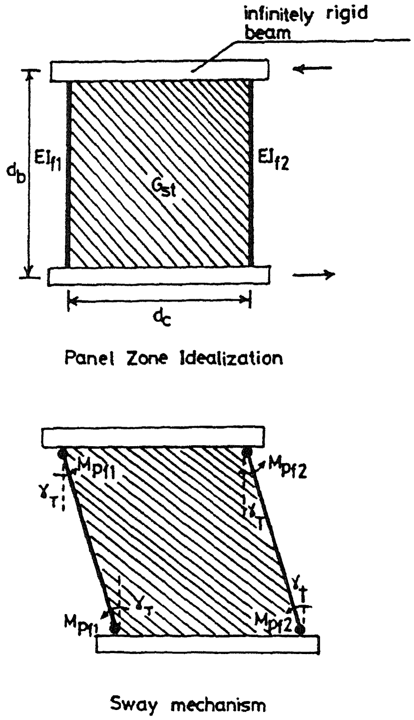

Figure 8. Idealization of the panel zone in the plastic range

in which the stiffness decreases until it finally reaches a stage of constant strain hardening rate.

4 ANALYTICAL MODEL

Based on the experimental investigation, it can be concluded that the applied moment-shear strain relationship for the panel can be best approximated by a tri-linear piece line curve as shown in Figure 7. This curve can be divided into three ranges.

4.1 Elastic range

In this range, it is assumed that the shear deformation is resisted by both the elastic shear stiffness of the panel and the bending stiffness of the column in the panel area. Based on that, the applied moment, M_p, can be related to the panel distortion, γ, by the following formula

$$K_e = \frac{d_b}{\dfrac{1}{Gt_{cw}(d_c - t_{cf})} + \dfrac{d_b^2}{12EI}} \frac{1}{1-\alpha} \tag{3}$$

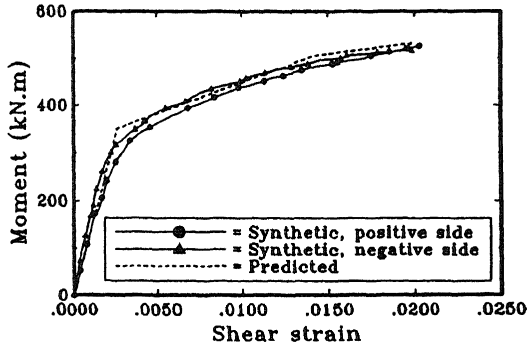

Figure 9. Predicted vs. experimental M-γ relationship for specimen CC-1

Where, K_e represents the stiffness of panel zone in the elastic range, t_{cf}, is the column flange thickness, G, the shear modulus and I is the column's inertia. The factor α is added to the equation to account for the effect of column shear and is equal to d_b/L_c where L_c is the column height.

This elastic range normally extends up to the point where the panel is yielded. The corresponding panel moment ,M_{py}, at that point can be determined by Eq.1

4.2 Plastic range

Following the full plastification of the panel web, the panel stiffness starts to deteriorate. During this stage, the column flanges and the end-plates are the main sources for resisting the panel shear deformation. To evaluate the model during this stage, the panel zone was idealized as a closed rigid frame consisting of two columns connected by two infinitely rigid beams, as shown in Figure 8. The infinitely rigid beams represent the elastic column section above and below the yielded panel. The flexible columns represent the column flanges and the adjoining end-plates. In calculating the columns inertia, two assumption can be made, either the column flange and the adjoining end-plate are in complete or partial contact. Model verification proved that the assumption of complete contact provides better results.

Using the simple plastic theory the deflection, γ_T, at the formation of the last plastic hinge in the frame sway mechanism can be calculated as follow

$$\gamma_T = \frac{M_{pfmin} d_b}{6EI_{fmin}} \tag{4}$$

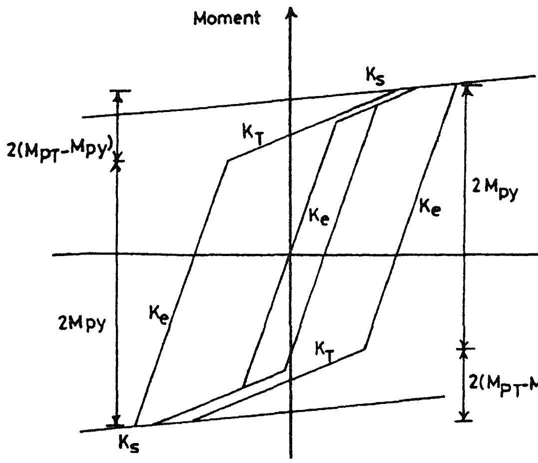

Figure 10. Proposed hysteretic model

Where, M_{pfmin} is the plastic moment of the thinner flexible column and I_{fmin} is the corresponding inertia. M_{pfi} for any column is given by

$$M_{pfi}=\sigma_y Z_{fe} \tag{5}$$

Where, Z_{fe} is the plastic modulus of the column flange and the adjoining end-plate as one unit.

Once γ_T is determined, limiting moment at which the plastic zone ends can be determined as

$$M_{pT}=M_{py}+2(M_{pf1}+M_{pf2})+Q^{*}d_b \tag{6}$$

where

$$Q^{*}=(\gamma_T-\gamma_y)G_{st}t_{cw}(d_c-t_{cf}) \tag{7}$$

G_{st} is defined as the plastic shear modulus and is equal in accordance to Prandl-Reuss theory (Malvern, 1969) to $\rho E/3$, where ρ is the uniaxial strain hardening ratio.

Once M_{pT} is determined, K_T can be calculated as follows

$$K_T=\frac{M_{pT}-M_{py}}{(\gamma_T-\gamma_y)} \tag{8}$$

4.3 Strain hardening range

Following the formation of the sway mechanism, the strength of the panel increases slightly due to material strain hardening. The increase is steady with constant stiffness, K_s, which is equal to

$$K_s=\frac{E_{st}}{E}K_e \tag{9}$$

5 VERIFICATION OF THE MODEL

The accuracy of the proposed model was verified by comparing the model predictions with the experimental results. Since the available results were mainly for panels tested under cyclic loads, the approach suggested by Kato et al (1981) was used to construct the monotonic synthetic curves from the cyclic ones. Figure 9 shows the comparison between the model prediction and the experimental results for specimen CC-1. As can be noted, a good agreement exists between the model predictions and the experimental measurements.

This simple proposed model was extended to predict the hysteretic behaviour of the panels. The proposed hysteretic model is shown in Figure 10. It should be noted that this model can only simulate the kinematic hardening which was evident from the tests. No attempt was made to model the isotropic hardening which is considered insignificant in such case.

The accuracy of the hysteretic model was verified by comparing it with the cyclic panel zone behaviour for specimen CB-1. The comparison is shown in Figure 11. The model satisfactorily predicted the hysteretic behaviour of the panel as evident from the figure.

6 CONCLUSIONS

The paper summarizes the results of an experimental-analytical research conducted to investigate the cyclic behaviour of panel zones in extended end-plate joints. A series of tests were conducted on full scale beam-to-column subassemblages. information regarding the behaviour of panel zones cyclic response were recorded. The tests highlightened the fact that the connection media can affect the panel zone response. Caution should be used when the formula provided by the code is used to estimate the panel zone shear resistance in cases rather than fully welded joints. Also, a general analytical model capable of describing the hysteretic behaviour of the panel zone was proposed. Predictions made by the model show good agreement with test results. The model is simple and can be incorporated in nonlinear dynamic analysis of steel buildings to asses the effect of panel zone flexibility on frame overall behaviour.

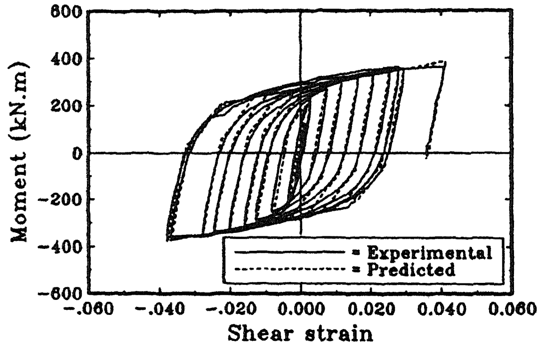

Figure 11. Comparison between experimental and predicted panel response for specimen CB-1.

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