Zeitschrift: IABSE reports = Rapports AIPC = IVBH Berichte
Band: 999 (1997)

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DOI: https://doi.org/10.5169/seals-1095

Behaviour of the Composite Beam-to-Steel H Column Connection

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Summary

The main purposes of this paper are to know the behavior of the composite beam-to-steel H column connection and to give the shear yielding and maximum strengths and panel moment $_pM$- shear deformation γ relation. The maximum strength is formulated through limit analysis. The panel moment - shear deformation relation is formulated considering the Bauschinger effect and the isotropic and kinematic hardening rule.

1. Introduction

It is well known that a concrete slab connected to a steel beam increases the stiffness and strength of the beam. It is also presumable that the concrete slab increases the shear stiffness and strength of the beam-to-column connection. The shear strength and the panel moment $_pM$- shear deformation γ relation of bare steel beam-to-column connections are presented(Matsuo,1995). However, the yield strength of the panel with a concrete slab is 20 to 40% larger than the bare steel connection(Nakao,1984). This paper presents the experimental results and the formulation of the shear strengths and $_pM$- γ relation considering the effects of the concrete slab.

2. Experimental Plan and Results

The experimental parameters are the shapes(X-type, T-type of frames), member strength(weak beam, weak column), the ratio of the panel yield strength to other members($R_{py} \fallingdotseq 0,0.5,0.7$), aspect ratio($H_b/H_c$=1.0,1.5) and displacement ratio(δ^-/δ^+=1.0,1.2). δ^- and δ^+ are the displacements of the loading points of the negative and positive bending beams. For example, the specimen X10B45-10 indicates X-type, H_b/H_c=1.0, weak beam, R_{py}=0.45 and δ^-/δ^+=1.0. Experimental results of the 13 specimens are listed in Tab.1, where $_pM_{yc}$ and $_pM_{ue}$ are the experimental yield and maximum panel strengths respectively. An experimental $_pM$- γ relation is shown in Fig.1

3. Analytical Strength and $_pM$- γ Model of the Panel

Analytical yield strength $_pM_{yc}$ is obtrained by Eq.1 following to Nakao(1987). M_{b1},M_{b2},Q_c and V_p are negative and positive face moment in the beam, column shear force and an effective volume of the panel. $_pM_{yc}$ predicts the experimental $_pM_{yc}$ fairly well in Tab.1. It is well known that the load carrying capacity of the panel increases after yielding, which is caused by strain hardening effect of the steel plate and direct transmission $_FM$ of the bending moment from the beam to column through 4 corners of the panel and concrete slab. Changing each parameter in Eqs.2 the ultimate strength of $_FM$ is given as the minimum value of $_FM^+$+$_FM^-$ (Eqs.2) which is derived from the plastic deformations illustrated in Fig.2. In Eqs.2 L_c, L_b , σ_{yw} and t_{wb} are the lengths of the column and beam, the yield stress and thickness of the beam web. As an approximate value $_FM^+$=$_FM^-$+C_ud' is also given by neglecting the axial deformations of the beam flanges(n=0). The total strength of the connection is given by Eq.3. The panel strength when the concrete slab is crashed is given by $(\tau_y+\tau_u)V_p/2$, as the crash of the concrete slab started at an early stage. The compressive strength C_u of the concrete at the face of the column flange is obtained as follows.

Eqs.2 and 3 are first applied to the specimens which have no panel plate and $1.4F_cA_c$ is found as the most appropriate value of C_u, where F_c and A_c are the compressive strength of concrete and the area of the flange in contact with the slab. This C_u is also used for the other cases where the connection has the panel plate. In Tab.1 ${}_pM_{uc1}$ and ${}_pM_{uc2}$ correspond to the maximum strengths calculated, considering and neglecting the axial deformation e of the beam flanges. Both ${}_pM_{uc1}$ and ${}_pM_{uc2}$ predict the experimental strength well. Judging from Eq.3 in case of n=0 the resisting panel moment corresponding to the current shear deformation γ consists of τV_p, ${}_FM_s$ and Cd', where τ, ${}_FM_s$ and C are the resisting shear stress of the panel, the bending moment transmitted through the 4 corners of the panel and the compressive force of the slab corresponding to γ. ${}_pM$-γ relations are formulated as follows. τV_p -γ and ${}_FM_s$-γ relations are seperately formulated as two springs connected in series. Each spring is given as a bi-linear type and satisfies the isotropic and kinematic hardening rule (Tsuji,1988). The compressive force C to the contraction Δ (=γd') relation of the slab is followed to Shiga(1988). Predicted ${}_pM$-γ relation is shown in Fig.1.

$$\tau = [\, M_{b1} + M_{b2}\, H_b/(H_b+d') - Q_cH_b] / V_p \tag{1}$$

$$ {}_FM^- = 2\,S\,a\,(a+b-c)^2/b + 2\,(1+a/b)(M_{ubf}+M_{ucf}) + 2\,a\,(M_{pbf}+M_{pcf})/b \tag{2a}$$

$$ {}_FM^+ = [\,(S\,a\,(a+b-c)^2\,(2+k_2(2-H_b/2a))/b + M_{ubf}k_2Z(2+(nN_{ubf}/2M_{ubf})^2\,(1+((H_b+Z)/Z)^2))/n + 2M_{pbf}\,a(1+k_2)/b + M_{ucf}(2k_1+(2k_1-H_b/b)k_2) + M_{pcf}\,a(2+k_2(2-H_b/a))/b + C_u((-Z+d')k_2+d')] \,/\, (1+k_2(1-H_c/L_b)/k_3) \tag{2b}$$

where $k_1 = 1+a/b$, $k_2 = k_1n/(Z-k_1n)$, $k_3 = 1-H_b/L_c-H_c/L_b$, n= $eZ/(Hb+Z)\theta$ and $S=\sigma_{yw}t_{wb}$

$$ {}_pM_u = (\tau_y+\tau_u)\,V_p/2 + {}_FM^+ + {}_FM^- \tag{3}$$

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Tab.1 Experimental and analytical results (unit:tmm)

Specimens	pMye	pMyc	pMyc/pMye	pMue	pMuc1	pMuc1/pMue	pMuc2	pMuc1/pMue
X10B45-10	7167	7116	0.993	12021	13221	1.100	12170	1.012
X10B45-12	6333	6944	1.096	12839	13465	1.048	12293	0.957
X10B67-12	10850	10748	0.991	17257	17608	1.020	16436	0.952
X10B00-10	-	-	-	5822	6965	1.196	5914	1.016
X10B00-12M	-	-	-	5662	7209	1.273	6037	1.066
X15C33-10	9800	9724	0.992	18872	17725	0.939	16522	0.875
X15C33-12	9200	9338	1.015	18710	17725	0.947	16522	0.883
X15C66-10	18667	20244	1.084	31030	28762	0.927	27559	0.888
X15C00-10	-	-	-	6600	8292	1.256	7089	1.074
X15C00-10M	-	-	-	6642	8292	1.248	7089	1.067
T15B59	8850	8781	0.992	15602	17092	1.096	15658	1.004
T15B77	11700	11905	1.018	20618	21030	1.020	19596	0.950
T15B00	-	-	-	6259	7659	1.224	6225	0.996

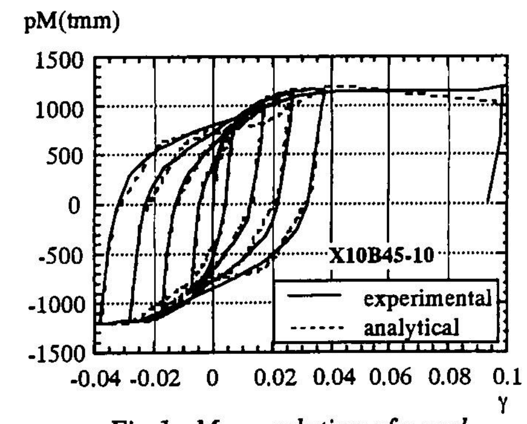

Fig.1 pM- γ relation of panel

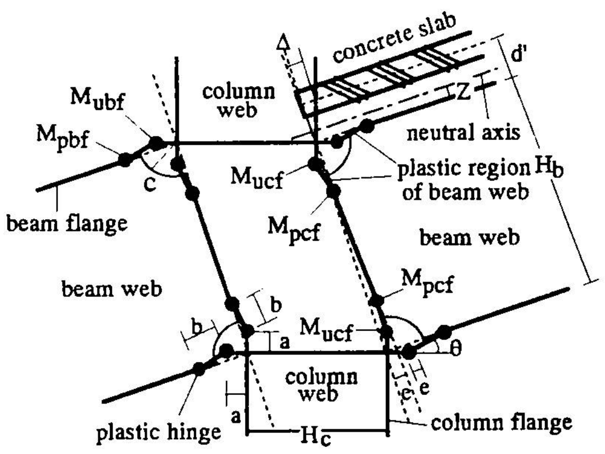

Fig.2 Plastic deformation of the connection