Testing of Beam-to-RHS Column Connections Without Weld-access Holes

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ABSTRACT

This paper presents 3 full-scale tests of beam-to-RHS column connections without weld-access holes. The aim of the tests was to investigate the plastic deformation capacity and ultimate strength of these types of beam-to-column connections. The ultimate moment capacities of the welded beam-to-column connections were predicted by simple formulas based on elementary plastic analysis. In all the specimens, the connections had sufficient overstrength to allow formation of plastic hinges at the beam ends, although one specimen failed through brittle fracture starting from the toe of the weld between the beam flange and the horizontal haunch.

NOMENCLATURE

\( A_w \) : cross-sectional area of beam web
\( b_b \) : width of beam flange
\( b_c \) : width of column
\( d_j \) : inner distance between top and bottom diaphragm
\( E \) : elastic modulus
\( E.L. \) : elongation
\( G \) : shear modulus of elasticity
\( H_b \) : height of beam
\( H_d \) : outer distance between top and bottom diaphragm
\( I \) : moment of inertia
\( j_b \) : distance between centroids of beam flange
\( L \) : distance between loading point and column face
\( l_e \) : effective weld length
\( L_{haunch} \) : length of haunch
\( L_{fracture} \) : length of fracture path
\( M_{f,u} \) : ultimate moment carried by beam flange
\( M_{m} \) : moment at column face
\( M_{max} \) : maximum moment
\( M_p \) : full plastic moment of beam
\( M_{w,u} \) : ultimate moment of beam-to-column connection
\( M_{w,u} \) : ultimate moment carried by beam web connection
\( S_h \) : space between edge of diaphragm and beam flange
\( S_r \) : vertical space of beam cope
\( t_c \) : thickness of column
\( t_d \) : thickness of diaphragm
\( t_f \) : thickness of beam flange
\( t_{fl} \) : thickness of flange plate
\( t_s \) : thickness of shear tab
\( u_{1}, u_{2} \) : horizontal displacements
\( v_{1}, v_{2} \) : vertical displacements
\( Y.R. \) : yield ratio
\( Z_{w,pe} \) : plastic modulus of effective cross-section of beam web
\( \alpha \) : ratio of \( M_{max} \) and \( M_p \)
\( \eta \) : cumulative plastic deformation factor
\( \sigma_{c,y} \) : yield strength of column material
\( \sigma_{f,u} \) : ultimate tensile strength of beam flange material
\( \sigma_{s,y} \) : yield strength of shear tab material
\( \sigma_{s,u} \) : ultimate tensile strength of shear tab material
\( \theta_n \) : rotation of connection between beam and column
\( \theta_p \) : elastic beam rotation at \( M_p \)

INTRODUCTION

Both the 1994 Northridge and 1995 Hyogoken Nanbu (Kobe) earthquakes took structural engineering professionals by surprise in that many of the welded connections in modern steel-building frames showed brittle fractures. Before these earthquakes, engineers commonly assumed that the connections between the beam flanges and columns using complete joint penetration (CJP) groove welds satisfied overstrength criteria to allow formation of plastic hinges in beams (CEN 1994, ICBO 1994). These fractures most frequently occurred in regions around the beam bottom flange groove welds. In Northridge, brittle fractures initiated at a very low level of plastic demand and, in some cases, while structures remained nearly elastic. In Kobe, however, the majority of...

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