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Energy dissipation characteristics of semi-rigid connections

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ABSTRACT

This study presents the energy dissipation characteristics of semi-rigid connections obtained from 48 full-scale cyclic tests. The energy calculation was performed on the hysteresis loops obtained from the experimental data in which the area under the outer loop of each hysteresis was calculated. This study reports that the energy dissipation characteristics of the welded/bolted connections are not directly dependent on the individual parameters defining the hysteresis loops, while the shape of the hysteresis is dependent upon the nonlinear interaction of its parameters coupled with connection plasticity and pinching which governs the energy dissipation capability of the connections.

1. Introduction

Semi-rigid connections have been examined as alternatives to welded connections for seismic regions due to their adequate ductility to dissipate large amounts of energy with a minimal loss of strength or stiffness. Fully restrained welded steel moment frames, even though desirable due to their prescribed prequalified connection detail and not having building height limitations, are known to be vulnerable to seismic loading. This was confirmed in the Northridge earthquake of January 17, 1994, in which more than 150 buildings experienced brittle fractures in their welded moment connections. This premature brittle failure of the welded connections was also noticed in the 1995 Hyogo-ken Nanbu (Kobe) earthquake.

To identify the measure of energy dissipation for a given connection, cyclic and shake table tests are conducted to obtain the moment-rotation hysteresis loops. The area under the outer loop of the hysteresis loops, at failure, is an indicator of the energy dissipation capability of a connection. Typical connection failure is categorized by bolt failure, angle/plate rupture, weld failure, plate bearing failure, and the limit state defined by excessive deformation (rotation) of the connection.

Thus, this study presents the energy dissipation characteristics of a wide range of semi-rigid connections. The cyclic testing program included 48 test specimens of bolted/bolted double web angle, welded/bolted double web angle, top and seat angle; flush end-plate, and extended end-plate connections. The slip-critical A-325 bolts were used in which the bolts were pre-tensioned to 70% of their ultimate strength (proof-load). In obtaining the moment-rotation hysteresis data, it was ensured that the test column deformation did not contribute to overall connection rotation. The energy dissipated from each test was calculated and related to the parameters defining the hysteresis loops, which are: connection initial stiffness; ultimate moment; and ultimate rotation. In calculating the connection initial stiffness the concept of characteristic moment and characteristic rotation is introduced to represent connection yield moment and yield rotation, respectively.

The ductility and energy dissipative characteristics of semi-rigid connections were identified by several researchers. Popov and Pinkey [13] reported hysteresis behavior of steel connections by conducting cyclic tests on beam-to-column connection and subjecting them to inelastic strain which was indicative of their ductility. Ghobarah et al. [8] conducted inelastic cyclic tests on end-plate beam-to-column connection. This study concluded that properly designed and detailed extended end-plate connection can provide excellent ductility as moment-resisting components in the seismic design of frames. Grigorian et al. [5] studied the energy dissipation of the modified slotted bolted connections by slotting or elongating the holes in one of the plates of the connection. It was concluded that connections dissipated energy by means of friction between sliding surfaces, and stable hysteresis was achieved by providing dissimilar coupling frictional surfaces.

Astaneh et al. [4], conducted cyclic tests on the connection assembly in which limited cyclic tests were conducted on the welded-bolted double web angle connections in which significant amount of moment was transferred from the beam end to the column. The experimentally obtained hysteresis loops exhibited energy dissipation capabilities. Liu and Astaneh [12] studied

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the behavior of simple connections subjected to cyclic loads considering the effect of floor slab. Specimens without slab were also tested to establish the basic characteristics and behavior of each shear tab connection on its own. It was shown that the addition of the floor slab to the test specimen resulted in roughly twice the maximum lateral load resistance and at approximately 0.04 radians drift.

The cyclic performance of two-bolt flush end-plate connections was studied by Schuab [15] for which the selection of the test cases was based on the standards and specifications of prefabricated steel buildings. The test results showed non-dissipative hysteresis loops, and the failure modes in general, were governed by bolt rupture rather than end-plate yielding. Hartman [9] modified the geometric variables of the connections tested by Schuab [15] such that dissipative hysteresis loops were obtained.

Kukreti and Abolmaali [10] conducted full-scale experimental investigation of the hysteresis behavior of the top and seat angle semi-rigid connections which showed considerable moment transfer from the beam end to the column. Prediction equations were presented for parameters defining hysteresis loops as function of geometric variables of top and seat angle connection. Also bilinear isotropic hardening and kinematic hardening mathematical hysteresis models were introduced to predict connection hysteresis behavior. Kukreti and Abolmaali [11] investigated the effect of connection hysteresis on the dynamic analysis of semi-rigid frames by modeling the joints as rotational spring. The experimentally obtained hysteresis models were implemented as joint nonlinearity in the developed coupled nonlinear dynamic computer program.

The application of semi-rigid flush end-plate connections to the seismic design of steel frames was investigated by Thomson and Broderick [16] in which flush end-plate moment connections were tested under cyclic loading conditions to simulate the seismic response of frames. This study suggested the use of partial-strength semi-rigid connections to be extended to earthquake resistance design procedures due to their ductile response observed in some test specimens. The study also suggested the inclusion of these connections in dissipative zones of moment-resisting frames.

Forcier et al. [7] studied the cyclic behavior of modern and old *t*-stub riveted connections. The results indicated the failure of the riveted connections was governed by the shear deformations of the rivets accompanied by highly pinched hysteresis. However, the tested connections exhibited ductility as high as 0.03 radians.

Abolmaali et al. [1] investigated the hysteresis behavior of double web angle semi-rigid connections. Two series of tests with angles being bolted or welded to the beam web and bolted to the column flange was considered. Different failure modes were observed and the energy dissipation of the connections tested was noticeable. Abolmaali et al. [2] investigated cyclic behavior of *t*-sub connections with shape memory alloy (SMA) threaded rods. The bolt failure at early load level was reported, however, increased energy dissipation of the connection with SMA bolts was observed when compared to the same connections with steel threaded rods.

Cheol-Ho and Kim [6] performed tests on the reduced beam section steel moment connections which showed that specimens with a bolted web connection tend to perform poorly due to premature brittle fracture of the beam flange at the weld access hole.

2. Selection of test cases

Five types of semi-rigid with *slip critical* connections were selected in which bolts were pre-tensioned to 70% of their ultimate tensile strength (proof load) as suggested in AISC (1994). These connections include: bolted/bolted double web angle; welded/bolted double web angle; top and seat angle; flush end-plate, and extended end-plate. The geometric variables of the all the test cases were selected and varied based on design and fabrication practices of the AISC LRFD Manual of Steel Construction [3].

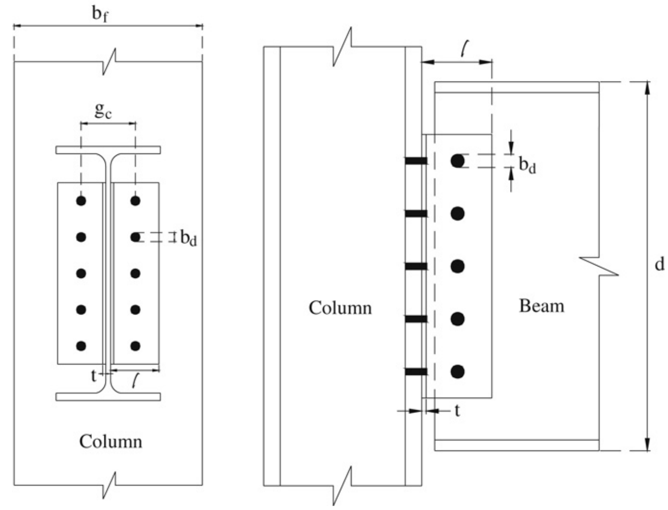

Fig. 1. Typical configuration of bolted/bolted test specimen.

2.1. Bolted/bolted double web angle

This test specimen was bolted to both the beam web and the column flange, as shown in Fig. 1. The adopted test designation was **DW-BB-ℓ-t-$\mathbf{b_d}$-$\mathbf{g_c}$-N-d**, where: **DW** represents the **D**ouble **W**eb angle connection, **BB** represents the bolted-/bolted connection, ℓ is the angle length, t is the angle thickness, $\mathbf{b_d}$, is the bolt diameter, $\mathbf{g_c}$ is the bolt gauge **N**, is the number of bolts, and **d** is the depth of the test beam connected to the column flange (Table 1). Thus, a test designated by DW-BB-102-6-19-114-76-406 (DW-BB-4-$\frac{1}{4}$-$\frac{3}{4}$-4$\frac{1}{2}$-3-16) represents a double angle test specimen that is bolted to both the beam web and the column flange, and consists of 102 mm × 102 mm × 6 mm (4 in. × 4 in. × $\frac{1}{4}$ in.) angles with three 19 mm ($\frac{3}{4}$ in.) diameter bolts at a column gauge of 114 mm (4$\frac{1}{2}$ in.). The depth of the beam used in this test specimen is 406 mm (16 in.), which corresponded to a depth of a W16 × 42 section.

2.2. Welded/bolted double web angle connections

The test specimens of welded/bolted double web angle connections were welded to the beam web and bolted to the column flange as shown in Fig. 2. The test designation (Table 2) was **DW-WB-ℓ-t-$\mathbf{b_d}$-$\mathbf{g_c}$-N-d**, where **WB** represents welded/bolted. All other variables have previously been defined. Therefore, a designated by DW-WB-76-6-13-64-76-610 (DW-WB-3-$\frac{1}{4}$-$\frac{1}{2}$-2$\frac{1}{2}$-3-24), denotes a double angle test specimen welded to the beam web and bolted to the column flange which consists of 76 mm × 76 mm × 6 mm (3 in. × 3 in. × $\frac{1}{4}$ in.) angles with three 13 mm ($\frac{1}{2}$ in.) diameter bolts with a bolt gauge of 64 mm (2$\frac{1}{2}$ in.). The depth of the beam connected to the column flange was 610 mm (24 in.).

2.3. Top and seat angle connections

The geometric variables describing the configuration of a typical top and seat angle are shown in Fig. 3. As shown, one leg of angle is connected to the beam flange by two rows of bolts, and the other leg is connected to the column flange by one row of bolts. The test designation (Table 3) of **TS-ℓ_h-ℓ_v-t-$\mathbf{b_d}$-G-$\mathbf{g_c}$-d** was adopted in which: $\ell_v =$ length of vertical angle leg (the shorter dimension in this study); $\ell_h =$ length of horizontal angle legs (the longer dimension in this study); $\mathbf{d_b} =$ bolt diameter; G $=$ distance from the heel of the angle to the column bolt row; and $\mathbf{g_c} =$ column bolt gauge, and $\mathbf{d} =$ beam depth.