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Span length effect on alternate load path capacity of welded unreinforced flange-bolted web connections

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ABSTRACT

It is intuitive that for a building of set dimensions, a shorter span length will increase the ultimate load carrying capacity, subjected to an arbitrary column loss. However, the capability of beam-to-column connections to develop and maintain the catenary action of the beams in this situation is not considered directly in the design stage. Therefore, to determine the optimum span length, which provides the required load carrying capacity and cost-effectiveness, sensitivity analyses are needed. In order to save the computational efforts, a component-based model for improved Welded Unreinforced Flange-Bolted web (WUF-B) connections is developed in this study. The capability of this model for predicting the failure and ultimate load carrying capacities was validated. Using this model, analyses results of three case study structures of set dimensions showed that by decreasing the span length by 25%, and 40% the ultimate load carrying capacity increased by 33%, and 72%, respectively. However, this increase was not as much as what was expected by using concentrated plasticity model recommended by UFC. Therefore, the concentrated plasticity model does not always conservatively predict the ultimate load carrying capacity of the studied connection. Moreover, although the distributed plasticity model could predict ultimate load capacity of the connections reliably in the longest span it overestimated the load carrying capacity for the shortest span by 13%. This explains the significance of component-based modelling approach in order to simulate the structural behaviour subjected to a column loss.

1. Introduction

Structures may encounter extreme loading events throughout their service life in which some of their structural members may suffer significant damages or fail to serve their design purpose. Such failure if affecting other structural members, can progress to the collapse of some parts of the structures; hence it has been termed disproportionate or progressive collapse. Since the partial collapse of the Ronan Point apartment in London in 1968 due to a gas explosion, the progressive collapse has been a controversial issue among structural engineers [1]. However, progressive collapse attracted highest attention after the terrorist attack on the WTC towers in 2001. In response to the possibility of progressive collapse, some design guidelines have been developed such as General Service Administration (GSA) [2] and Unified Facility Criteria (UFC) [3] in the United States. According to these guidelines, if a member, especially a column, is suddenly eliminated, an Alternate Load Path (ALP) has to be provided. This path enables the structure to bridge over the affected parts and redistribute the loads carried by the eliminated member to the intact parts of the structure, thereby guarding against progressive collapse. Many researchers have studied the effect of column loss on the structural response of structures [4–12]. However, the discussion about the robustness of steel structures is still ongoing considering the effect of ductility, energy absorption, and redundancy. In the case of sudden column elimination, beam-to-column connections in steel moment resisting frames, can contribute to the redistribution of gravitational loads. This contribution involves experiencing a magnified bending moment and axial load which ultimately produces catenary action in the beam [13–17]. Research has shown that this mechanism can eventually replace the flexural action in carrying the gravitational loading [18–20]. To do so, the beam and the connection must be able to develop and maintain a sufficiently large axial tension when subjected to large deformation which explains the significance of the role of connections in mitigating progressive collapse [21,22]. Li et al. [23] experimentally studied beam-to-tubular column connections subjected to column removals and showed that welded flange-bolted web exhibits more ductility and strength due to its confined fracture in the tensile flange at the flexural stage and hence developing an effective catenary action beyond the flexural failure.

It is intuitive that, for a building of set dimensions, shorter span lengths will give improved performance from a redundancy standpoint, just based on the fact that the load that must be bridged will be smaller.

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List of symbols

Δ	Vertical displacement correspondent to static load
Δ_0	Vertical displacement correspondent to dynamic load
$\varepsilon_{nominal}$	Nominal strain
ε_{true}	True strain
θ	Angle between the diagonal truss element and the horizontal or angle of rotation
θ_y	Yield rotation of the beam
$\sigma_{nominal}$	Nominal stress
σ_{true}	True stress
b_{cf}	Column flange width
d_b	Beam depth
d_c	Column depth
f_{pz}	Yield strength of the panel zone element
F_y	Yield strength of steel
G	Shear modulus of steel
k_{pz}	Stiffness of the panel zone element
N	Axial force of beam
P	Applied load
S_1, S_s	Site response factors
t_{bf}	Beam flange thickness
t_{cf}	Column flange thickness
t_{pz}	Panel zone thickness
V	Shear force of beam
$w_{dynamic}$	Approximate dynamic load
w_{static}	Computed static load in the static pushdown analysis

However, in the seismic design of buildings, beam-to-column connections are designed based on the flexural capacity of the connected members so that the beams fail prior to the connections; and the catenary action of the connection is not considered directly in the designing stage. So, span length and the connection detailing affect the resistance, ductility and energy absorption of beam-to-column connections subjected to an arbitrary column loss.

Performing detailed finite element analyses to determine the optimum span length for a building of set dimensions to provide the best structural performance as well as cost-effectiveness is computationally expensive. Therefore, using a component-based (macro-element) model is beneficial for this purpose especially for the preliminary robustness assessment of buildings. Accordingly, this research aims to propose a component-based model to investigate the structural behaviour of improved Welded Unreinforced Flange-Bolted web (WUF-B) connections which are widely used in ordinary moment-resisting frames. By using this model, the effect of span length on structural performance of three case study structures is then investigated. Eventually, by comparing the results obtained from the proposed component-based model with other widely used models, the differences in determining the ultimate load carrying capacity are discussed.

2. Component-based modelling of improved WUF-B connection

Improved WUF-B connections utilize Complete Joint Penetration (CJP) groove welds to connect the beam flanges to the column flanges and a shear tab which is welded to the column using groove or fillet welding. To transfer the shear forces, this element is also bolted to the beam web. To resist against the large forces transferred to the column by beam flanges, continuity plates are added so that the column flange and web are strengthened. Fig. 1(a) illustrates the schematic view of this type of connection.

The component-based model was developed by using OpenSEES [24]. The active joint components for the improved WUF-B connections include the shear tab and the beam web in bearing, the bolts in a single shear, the friction between the beam web and the shear tab, the beam flanges in tension or compression, the panel zone in shear, and the weld in tension or compression. In the proposed model, it has been assumed that the weld component is rigid which according to the requirements of FEMA 350 [25] is deemed reasonable. Therefore, the possible failure mechanisms for the improved WUF-B connection model consist of the bearing failure of the plates, the shear failure of the bolts, local buckling or fracture/tearing of the beam flanges and the yielding of the panel zone [26–30].

Fig. 1(b) and (c) illustrate the proposed component-based model of the improved WUF-B connection. To model the structural members' behaviour, Steel02 material, and hysteresis material together with min-max material for limiting the normal strains or deformations were employed. In this model, each spring is modelled using zero-length elements (shown with finite length for clarity) introduced in OpenSEES library with a nonlinear relationship between the force and the displacement of the component. To model the rigid links, the elastic beam-column elements with sufficiently high elastic modulus and cross section were implemented. Fiber sections together with the corotational transformation of the geometric stiffness matrix which accounts for the large displacements were used to model the behaviour of these elements. The springs representing the bolted lap joints shown in Fig. 1(b), are composed of four active components, namely, the beam web and the shear tab in bearing, the friction between these two plates and the bolt in a single shear as shown in Fig. 1(c). Material properties, as provided in Table 1, are assumed to be the same as those reported by Sadek et al. [26] in order to validate the modelling approach and also to use the real stress or strain capacity of materials rather than the minimum standardised values.

2.1. Panel zone

A diagonal truss element with an elastic-perfectly plastic force-displacement curve representing the shear behaviour of the panel zone is assigned to each joint. The stiffness and strength of this element are computed based on the model proposed in [6]. In the model, stiffness of the panel zone element, k_{pz}, is computed by equating the panel zone response with that of the column web subjected to pure shear and given in Eqs. (1) and (2). In these equations, d_c is the column depth, d_b is the beam depth, t_{cf} is the column flange thickness, t_{bf} is the beam flange thickness, t_{pz} is the panel zone thickness, θ is the angle between the diagonal truss element and the horizontal, and G is the shear modulus of steel. The yield strength of the panel zone element (f_{pz}) is calculated using similar equilibrium relationships and the panel zone strength equation as provided in AISC 2006 Seismic Provisions [31]. The strength formula is given in Eq. (3) in which b_{cf} is the column flange width, and F_y is the yield strength of steel. Since the panel zone region is very ductile, it is unlikely that it fails prior to the failure of other components. Thus, it is assumed that the panel zone element can deform indefinitely.

$$k_{pz} = \frac{G(d_c - t_{cf})t_{pz}}{(d_b - t_{bf})\cos^2\theta} \tag{1}$$

$$\cos^2\theta = \frac{(d_c - t_{cf})^2}{(d_c - t_{cf})^2 + (d_b - t_{bf})^2} \tag{2}$$

$$f_{pz} = \frac{0.6F_y d_c t_{pz}}{\cos\theta}\left[1 + \frac{3b_{cf}t_{cf}^2}{d_b d_c t_{pz}}\right] \tag{3}$$

2.2. Bolted lap Joint

The shear tab and the beam web in bearing, the bolt in a single shear and the friction between the connected components are accounted for