ENGINEERS AUSTRALIA

Australian Earthquake Engineering Society
2024 National Conference, 21-23 November 2024
Adelaide, South Australia

Analytical Evaluation of Wide-Flange Steel Columns with Predominant Local Buckling Failure Mode

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Abstract

Understanding the seismic behaviour of structural elements is fundamentally guided by their force-displacement characteristics. These characteristics are typically derived from moment-curvature analyses coupled with the application of equivalent plastic hinge lengths. This study utilized the plastic hinge analysis method to establish the force-displacement relationships for wide flange steel columns of moment resisting frames. To assess the accuracy of existing equivalent plastic hinge length relations, analytical results were compared with experimental data from a database of 33 columns. The analysis revealed that the current relations do not reliably predict the displacement capacities of these columns. In response, a new formula to calculate the equivalent plastic hinge length for wide flange steel columns was developed. This formula was informed by a detailed numerical parametric study, conducted using validated finite element models in Abaqus software. The results underscore that factors such as the slenderness ratios of the web and flange, as well as the intensity of axial loads, significantly affect the columns' failure modes, ultimate strength, and drift capacity. The proposed equation offers improved accuracy in estimating the drift capacities of steel columns, particularly those prone to failure by local buckling.

Keywords: Steel column, wide flange section, plastic hinge analysis, finite element modelling.

1 Introduction

Steel moment-resisting frames (MRFs) play a pivotal role in contemporary construction, offering essential structural stability for buildings across different heights and purposes. The primary elements of steel MRFs (beams, columns, and beam-to-column connections) are vital, with columns being particularly crucial for maintaining overall structural stability. Design and assessment standards underscore the importance of column strength to prevent premature instability. Experimental and numerical research has been conducted to understand the seismic performance of steel columns, investigating a range of section types. Notable contributions to this research field include studies by Newell and Uang (2006), Fogarty and El-Tawil (2016), Ozkula et al. (2017), Elkady and Lignos (2018b, 2018a), Cravero et al. (2020), Suzuki and Lignos (2021), and Ozkula and Uang (2023).

In the past twenty years, multiple guidelines and codes have been developed to evaluate the seismic performance of existing structures. These include ATC40 (1996), FEMA 356 (2000), FEMA 400 (2005), AISC 342 (2022), and ASCE 41 (2023). Unlike the design codes for new constructions, which primarily use a force-based design philosophy, these assessment codes incorporate modern methodologies like Performance-Based Design (PBD) and Displacement-Based Design (DBD).

2 Performance-Based and Displacement-Based Design

PBD and DBD aim to understand the nonlinear behaviour of structural elements, with an emphasis on deformation-controlled components to assess damage. Various approaches estimate the deformation capacity of structural members under different internal forces. The Direct Rotation Method (ASCE41-23 2023) utilizes empirical formulas derived from experimental and numerical studies to predict force-displacement relationships and the ultimate deformation capacity of structural elements.

In addition to empirical methods, analytical approaches largely based on first principals are employed to estimate the force-displacement relationships of structural elements. The Plastic Hinge Analysis Method is a recognized technique for translating moment-curvature into force-displacement curves for flexurally governed members. Foundational works by Blume et al. (1961) and Park and Paulay (1975) have extensively developed this method for reinforced concrete (RC) elements. However, its application to steel elements, especially those susceptible to local buckling failure modes, has been less explored.

This study investigates the Plastic Hinge Analysis Method as an analytical tool for estimating the force-displacement curve of steel columns that primarily fail due to local buckling. A finite element (FE) parametric study examines the seismic behaviour of wide flange steel columns. The study introduces an equivalent plastic hinge length specifically for these columns, determining their displacement capacity for the first time. The accuracy of the Plastic Hinge Analysis Method, combined with the proposed plastic hinge length approach, is then evaluated against test results from existing literature.

3 Relationship Between Curvature and Displacement

In the Plastic Hinge Analysis Method, the flexural plastic behaviour of an element is represented by an equivalent segment where plasticity is concentrated along an equivalent plastic hinge length (L_p), as depicted in Figure 1. To derive the force-displacement curve of a member, the first step involves calculating the moment-curvature relationship of the section. This requires a section analysis procedure, which includes selecting appropriate uniaxial stress-strain models for the different materials in the section. Next, the total displacement of the element linked to the ultimate section curvature (ϕ_u) is determined by adding the yield displacement (Δ_y) to the plastic displacement (Δ_p). The equations for a double bending element are shown below.

$$\Delta_y = \frac{\phi_y l^2}{6} + \frac{2M_y}{GA_g} \tag{1}$$

$$\Delta_t = \Delta_y + (\phi_u - \phi_y)L_p(l - L_p) \tag{2}$$

Where ϕ_y and M_y are the yield curvature and moment, respectively. l is the column height and A_g is its gross section area. Moreover, $G = \frac{E}{2(1+\mathcal{V})}$ is the shear modulus of material related to the elastic modulus (E) and Poisson ratio (v). In Equation (2), the equivalent plastic hinge length (L_p) is a key parameter necessary for converting the section curvature to the element's displacement.

The lateral load at different loading stages (F_i) corresponding to internal bending moment (M_i), and yield force (F_y) are calculated for a double bending element as follows:

$$F_y = \frac{2M_y}{l} \; (until\ yield) \tag{3}$$

$$F_i = \frac{2M_i}{l-L_p}\ (after\ yield) \quad (4)$$

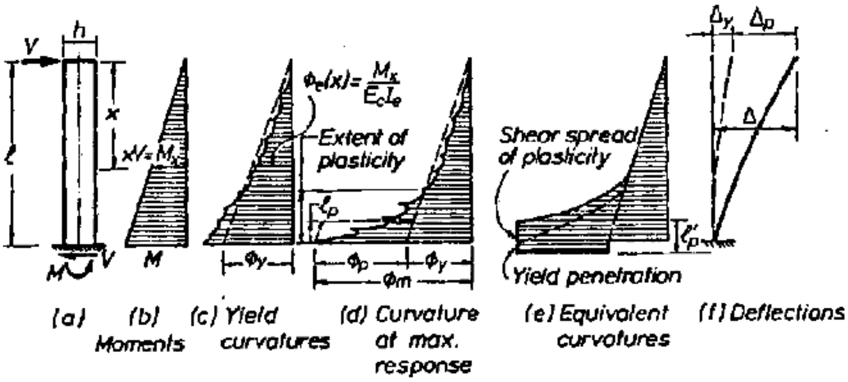

Figure 1. Moment, curvature, and deflection relationships for a RC cantilever element subjected to a point load (Paulay and Priestley 1992)

4 Plastic Hinge Analysis of Steel Columns

In contrast to RC members, for which well-established nonlinear material models exist for both concrete (Kent and Park 1971, Mander et al. 1988, Saatcioglu and Razvi 1992) and steel reinforcement (Menegotto and Pinto 1973), the development of equivalent plastic hinge lengths and appropriate material models for steel members is still limited. This limitation is especially notable for steel members that demonstrate local buckling behaviour. Effective plastic hinge analysis depends on these elements to accurately understand and predict the performance of steel structures under various loading conditions.

4.1 Stress-strain material model for steel

In this research, a recently developed stress-strain model by Kolwankar et al. (2020) is utilized for the plastic hinge analysis of steel columns. This model, derived from an extensive numerical study of wide flange steel columns subjected to cyclic loads, is depicted in Figure 2. A notable aspect of this model is its inclusion of local buckling effects in steel columns with wide flange sections. The stress and strain at the onset of buckling, denoted as $\sigma_{y_{cap}}$ and ε_{ycap}, respectively, are defined as follows:

$$\sigma_{y_{cap}} = 1.1 \times\ \sigma_u - 2.17 \times \left(b_f/2t_f\right) \quad (5)$$

$$\varepsilon_{ycap} = \frac{\sigma_y}{E} + \frac{\sigma_{ycap} - \sigma_y}{h \times E} \quad (6)$$

Where σ_y and σ_u are the steel yield and ultimate strength, $b_f/2t_f$ is the flange slenderness, and h is the strain hardening parameter, assumed as 0.05 in this model suggested by Elkady and Lignos (2015).

The slopes of the softening branch of the stress-strain model in compression, $E_{d1.m}$ and $E_{d2.m}$, respectively, are determined as follows:

$$E_{d1.m} = (\sigma_{d\_cap} - \sigma_{y\_cap})/\varepsilon_{res} \quad (7)$$

$$E_{d2.m} = 0.2 * E_{d1.m} \quad (8)$$

In these equations, the residual stress and strain, σ_{d_cap} and ε_{res} can be determined by the following equations.

$$\sigma_{d\_cap} = \sigma_y - 1.44\frac{b_f}{2t_f} \geq 0 \quad (9)$$

$$\varepsilon_{res} = 0.15 - 0.014\frac{b_f}{2t_f} \geq \frac{\sigma_y}{E} + \frac{\sigma_{cr}-\sigma_y}{h\times E} \tag{10}$$

To define the stress-strain behaviour of steel in tension, certain parameters related to kinematic and isotropic hardening, including *Q*, *b*, *c*, and *γ* are required. These parameters are typically determined through axial tests for the specific type of steel.

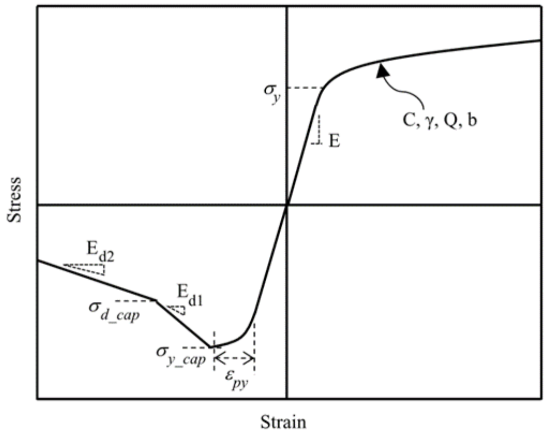

Figure 2. Stress-strain model for steel proposed by Kolwankar et al. (2020)

4.2 Available equivalent plastic hinge length models for steel column

While the application of plastic hinge analysis in PBD and DBD for steel structures is not yet widely adopted in practice, three distinct formulations for calculating the plastic hinge length of steel columns have been proposed in the literature. Bruneau et al. (2011) and MacRae (1989) suggest that the plastic hinge length is directly related to the height of the column (l) and the section depth (d), respectively. In contrast, Elkady and Lignos (2018b) define the plastic hinge length as a function of the section web slenderness, the global slenderness of the column, and the axial load intensity.

$$LP_1 = 0.067L_c \tag{11}$$

$$LP_2 = 1.837d(\frac{h}{t_w})^{-0.443}(\frac{L_b}{r_y})^{0.287}(1-\frac{P}{P_y})^{-0.259} \tag{12}$$

$$LP_3 = d \tag{13}$$

Where, L_c is the length from the critical section to the point of contraflexure in the member, h/t_w is the web slenderness, L_b is the laterally unbraced length of the column, r_y is the weak-axis radius of gyration, P is the axial load, and P_y is axial yield strength.

4.3 Database of tests on wide flange steel columns

To assess the effectiveness of the plastic hinge analysis method in determining the force-displacement curve and drift capacity of steel columns, a database comprising 33 wide flange steel section specimens was assembled. These specimens, previously tested under cyclic lateral loads in experimental studies, primarily failed due to local buckling. The characteristics of these test specimens are provided in Table 1. Using the plastic hinge analysis method, the lateral load-displacement curves for each column were analysed, and their ultimate drift capacity was determined.

The mean, standard deviation, and coefficient of variation (COV) for the estimated versus experimental drift capacities of the columns, based on different equivalent plastic hinge length equations, were calculated and are presented in Table 2. A mean value greater than one indicates that the equation overestimates the column's drift capacity. The results show that the existing plastic hinge length equations (Equations 11-13), when applied through the plastic hinge analysis method, do not produce sufficiently accurate results. Therefore, to improve the method's accuracy in estimating the drift capacity of columns, it is crucial to develop a more suitable equivalent plastic hinge length.

Table 1. A summary of the test specimens in the experimental database

Number	Tested by	Section	$b_f/2t_f$	h/t_w	L_b/r_y	P/P_y	Boundary conditions
1	Ozkula.et al. (2017)	W30x173	7.04	40.8	63.16	0.4	Fixed-fixed
2		W30x173					Fixed-flexible
3		W30x90	8.52	57.5	75	0.2	Fixed-fixed
4		W24x176	4.81	28.7	52	0.2	Fixed-fixed
5		W24x176				0.4	Fixed-fixed
6		W24x176				0.6	Fixed-fixed
7		W24x176				0.6	Fixed-flexible
8		W24x131	6.7	35.6	53	0.2	Fixed-fixed
9		W24x131				0.4	Fixed-fixed
10		W24x131				0.6	Fixed-fixed
11		W24x104	8.5	43.1	54	0.2	Fixed-fixed
12		W24x104				0.4	Fixed-fixed
13		W24x104				0.6	Fixed-fixed
14		W24x84	5.86	45.9	81	0.2	Fixed-fixed
15		W24x84				0.4	Fixed-fixed
16		W18x130	4.56	23.9	58	0.4	Fixed-fixed
17		W18x130					Fixed-flexible
18		W18x76	8.11	37.8	60	0.2	Fixed-fixed
19	Elkady and Lignos (2018a)	W24x146	5.92	33.2	52	0.2	Fixed-fixed
20		W24x146					Fixed-flexible
21		W24x84	5.86	45.9	81	0.2	Fixed-flexible
22	Cravero.et al. (2020)	W14x82	5.92	22.2	64	0.5	Fixed-flexible
23		W14x82				0.75	Fixed-flexible
24		W16x89	5.92	27	63	0.5	Fixed-flexible
25	Suzuki et al. (2021)	W14x53	6.11	30.9	82	0.3	Fixed-flexible
26		W14x61	7.75	30.4	64	0.3	Fixed-flexible
27		W14x82	5.92	22.4	64	0.3	Fixed-flexible
28	MacRae (1989)	C3	8.94	37.6	78.57	0.3	Fixed-flexible
29		C4				0.4	Fixed-flexible
30		C5				0.5	Fixed-flexible
31		C6				0.6	Fixed-flexible
32		C7				0.7	Fixed-flexible
33		C8				0.8	Fixed-flexible

Table 2. Evaluation of the analytical drift vs experimental results

Method	Analytical/Experimental drift estimation		
	Mean	**Standard Deviation**	**COV**
LP_1 (Bruneau et al. 2011)	0.72	0.33	0.46
LP_2 (Elkady and Lignos 2018b)	1.79	0.79	0.44
LP_3 (MacRae 1989)	1.43	0.59	0.41

5 Numerical Parametric Study of Columns

To determine a suitable plastic hinge length for steel columns with wide flange sections, a numerical study was performed using the FE software Abaqus (Smith 2009). The primary reason for relying on numerical results was the lack of sufficient experimental data to develop an empirical equation. Initially, the FE model was validated against test results from previous studies on steel columns. After this validation, a comprehensive parametric study was conducted, leading to the proposal of a new expression for the equivalent plastic hinge length of steel columns. The FE model was validated against several columns that had been experimentally tested by other researchers. For more details, refer to Niroomandi et al. (2024).

To evaluate the impact of various factors such as the geometric properties of the columns and the intensity of axial loads on the seismic behaviour of steel columns with wide flange sections, a thorough parametric study was carried out using the validated FE model. This study involved analysing columns with 20 different wide flange sections under five levels of constant axial loads (10%, 20%, 35%, 50%, and 60%), covering a broad range from low to high axial load intensities typically observed in steel frames. As a result, a total of 100 columns, all with fixed-restrained boundary conditions at both ends, were simulated and analysed. The selected wide flange sections featured flange slenderness ratios ranging from $3.1 < b_f/2t_f < 8.52$ and web slenderness ratios of $6.9 < h/t_w < 57.5$. The global slenderness ratios of these columns varied between $36.9 < L_b/r_y < 117.5$. All columns in the parametric study were modeled with a uniform height of 4.0 meters.

6 Developing an Equivalent Plastic Hinge for Steel Columns

An expression for the equivalent plastic hinge length of steel columns with wide flange sections is proposed. Utilizing a database from our numerical study, the required plastic hinge length to accurately obtain the drift capacity was determined for each column. The drift capacity criterion was defined as a 20% reduction in strength on the force-displacement curve indicative of local buckling failure mode. Since plastic hinge length is associated with local behaviour of columns, the database was confined to models exhibiting local buckling failure modes encompassing 60 models. Columns exhibiting lateral torsional buckling failure were consequently excluded. The efficacy of the plastic hinge analysis method in conjunction with our proposed plastic hinge length was validated by comparing our analytical results with experimental test data. The final expression for the plastic hinge length emerged from a nonlinear regression analysis achieving a correlation coefficient $R^2 = 0.77$ as follows:

$$L_{p,proposed} = 13.192d\left(\frac{P}{P_y}\right)^{-0.653}\left(\frac{b_f}{2t_f}\right)^{1.617}\left(\frac{h}{t_w}\right)^{-1.276}\left(\frac{l}{r_y}\right)^{-0.481} \tag{14}$$

The estimated drift capacities of the columns were compared with the results from FE analyses, as shown in Figure 3. The mean and standard deviation of the ratio of estimated drift to FE-analysed drift for the columns are 1.04 and 0.18, respectively, resulting in a COV of 0.17. This indicates that the method not only accurately estimates the drift capacity of columns but also produces results with relatively low variability.

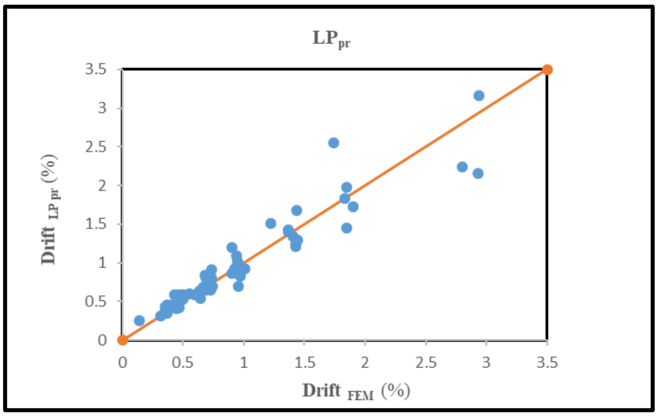

Figure 3. The estimated vs. actual lateral drift capacity of the columns investigated in numerical study

7 Conclusions

This study offers a significant advancement in the understanding and prediction of seismic behaviour in steel columns with wide flange sections, particularly those prone to local buckling failure. By employing a validated FE model and conducting an extensive numerical parametric study, we developed a new, more accurate expression for the equivalent plastic hinge length of steel columns. The proposed formula, derived from comprehensive analysis, addresses the limitations of existing models by incorporating key factors such as flange and web slenderness, global slenderness, and axial load intensity.

Our findings demonstrate that the proposed plastic hinge length significantly enhances the accuracy of the plastic hinge analysis method in estimating the drift capacity of steel columns. The close agreement between the analytical results and experimental data validates the robustness of the new expression.

This research not only fills a critical gap in the application of plastic hinge analysis for steel columns but also provides a practical tool for engineers in the seismic design and assessment of steel structures. Future work should focus on further validation with additional experimental data and exploring the applicability of the proposed model to other structural elements and materials.

8 Limitations and Future Work

This paper represents one of the initial steps of a larger effort to develop displacement-based design methodologies for steel columns. The presented study has certain limitations, particularly in the scope of the parametric study and the range of cases analyzed. The number of cases analyzed was limited, specifically in terms of variations in column sections and geometric properties. Notably, columns susceptible to lateral torsional buckling were excluded from the development of the proposed model.

Ongoing studies by the research team and other international efforts are addressing these limitations, expanding the scope to encompass more complex failure modes and further enhance the robustness of the proposed model.

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