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A hysteretic model for the rotational response of embedded column base connections

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ABSTRACT

Embedded Column Base (ECB) connections are commonly used in mid- and high-rise steel moment frames, to connect the steel column to the concrete footing. Although recent research has shown these connections to be highly ductile, they are typically designed to be stronger than the adjoining column, resulting in significant cost. To enable assessment of strong-column-weak-base systems that leverage the inherent ductility of these connections, an approach is presented to simulate their hysteretic and dissipative response. The proposed approach simulates ECB connections as an arrangement of two springs in parallel, to reflect moment contributions due to horizontal and vertical bearing stresses. This is informed by recent work that provides physical insight into the internal force transfer within these connections. The springs' response is defined by the pinched Ibarra-Medina Krawinkler (IMK) hysteretic model, which is able to capture both in-cycle and cyclic degradation in strength and stiffness. The model is shown to reproduce the response of ECB connections with reasonable accuracy. Guidelines to calibrate model parameters are presented; these include physics-based estimation of selected parameters such as strength and stiffness, accompanied by empirical calibration of ancillary parameters associated with cyclic deterioration. Limitations are discussed.

1. Introduction

Column base connections in steel moment frames may be classified as of the exposed or embedded type. Exposed base plate connections (such as the one shown in Fig. 1a) are common in low-rise (1–3 story) moment frame buildings, where the base moment, shear, and axial force demands are relatively modest. These are less preferable for mid- or high-rise moment frames, since the higher moment demands necessitate a large number of deeply embedded anchor rods and/or thick base plates. In these cases, Embedded Column Base (ECB) connections, such as the one shown in Fig. 1b are more preferable. These connections resist base moments and forces through a combination of bearing stresses on the column flanges and the embedded base plate. Besides, exposed base plate connections may be shallowly embedded under a slab-on-grade cast on top of the base plate. This shallow embedment (typically less than 300 mm) increases the strength and stiffness of the connection.

Exposed base plate connections are well-researched, with validated models for strength (Drake, and Elkin [1]), stiffness (Kanvinde et al. [2], Trautner et al. [3]), component hysteretic response (Torres-Rodas et al. [4]), and methods for design (Fisher and Kloiber [5], Gomez et al. [6]). In contrast, ECB connections (constructed as per US practice) have attracted research attention only recently; this work includes some of the first experiments on deeply embedded column bases (Grilli et al. [7]), and shallowly embedded column bases (Barnwell [8]). These experiments have led to validated strength models and design methods (Grilli and Kanvinde [9] for deeply embedded, and Barnwell [8] for shallowly embedded), as well as stiffness characterization approaches (Torres-Rodas et al. [10] for deeply embedded, and Tyron [11] for shallowly embedded). A secondary finding of these studies is that ECB connections are ductile (rotation capacity in the range of 0.03–0.08 rad) for the specimens tested by Grilli and Kanvinde [9], and Barnwell [8], even when not explicitly detailed for ductility. In seismic regions, where ECB connections are generally designed to remain elastic (AISC [12]), making this finding to be important. More specifically, ECB connections (and more generally, column base connections in seismic moment frames) are designed to resist a moment equal to $1.1R_yM_p$ of the connected column (i.e., a "strong-base-weak-column design"). This is based on the presumption that a plastic hinge within the column section (usually a wide-flange cross-section) possesses greater rotation capacity compared to the base connection. This is problematic for two reasons:

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Notation

α	Fraction of the moment applied and resisted by vertical bearing mechanism
a_{pinch},Λ_{K_I},$\Lambda_{M_{peak}}$,F_{pr}	ECB hysteretic parameters
β_i	Cyclic deterioration parameter
B	Base plate width perpendicular to plane of lateral loading
C	Constant defining interaction of column with concrete
c,c_{K_I},$c_{M_{peak}}$	Rate of deterioration parameters (equal to one in all cases)
d_{embed}	Embedment depth
d_{ref}	Depth at which horizontal bearing stresses attenuate to zero
$d\theta_p$,θ_p	Plastic rotation
$\in$ $\in_u$ $\in_c$	Error function, Error function for unconstrained and constrained calibration
$E_{concrete}$,E_{steel}	Modulus of Elasticity of Concrete, Steel
E_i, E^T	Energy dissipated at cycle "i", Reference Energy
Γ_i,Γ_{i-1}	Values of generic quantity during cycle i and $i-1$
$K_I^{initial}$ $K_{I,VB}^{initial}$ $K_{I,HB}^{initial}$	Initial Elastic Stiffness, Vertical spring, Horizontal spring
I_{column}	Moment of inertia of embedded column
M_{base},M_p	Base moment, Nominal plastic flexural strength
M_{HB},M_{VB}	Moment resisted through horizontal bearing, and vertical bearing
M_y,M_y^+,M_y^-	Moment at first yield of connection, Moment at first yield in the forward direction, Moment at first yield in the reverse direction
M_{peak},$M_{peak,HB}$,$M_{peak,VB}$	Peak moment of the connection, Peak moment of horizontal bearing spring, Peak moment of vertical bearing spring
M_{test},M_{MODEL}	Moment obtained from the test, and model
N	Base plate length in the direction of loading
θ_{peak}	Rotation at peak strength of the connection
R, Ω_o	Seismic response modification factor, System overstrength factor
Ry	Ratio of the expected yield stress to the specified minimum yield stress
s_p, $M_{\max}$	Rotation at which elastic unloading hits horizontal axis, Maximum Moment from previous excursion
t_p	Thickness of Base plate at bottom of the column

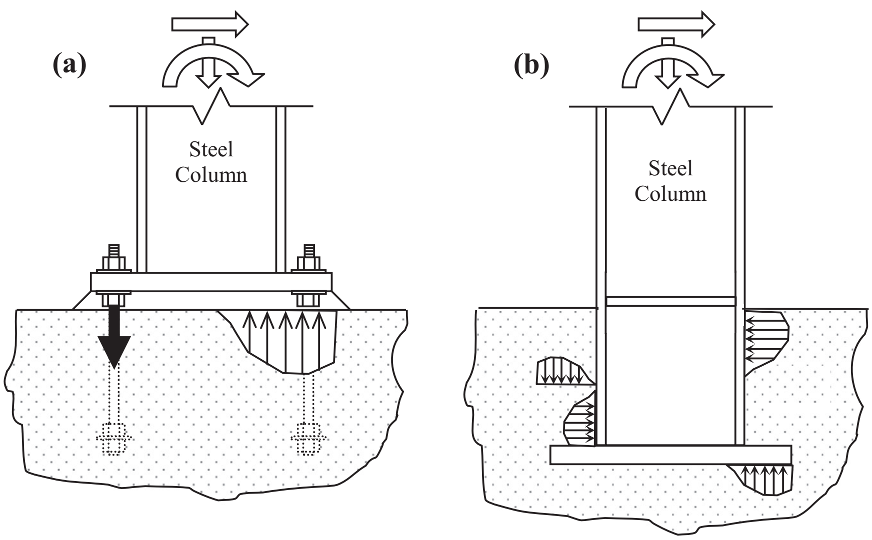

Fig. 1. Types of column base connections: (a) exposed base plate connection with forces resisted by vertical bearing and anchor rod tension, (b) embedded column base, with forces resisted by horizontal bearing stress on column flanges, and vertical bearing stresses on embedded plate.

1. From a mechanistic standpoint, the strong-base-weak-column design may not provide superior performance, since the column plastic hinges themselves may have lower rotation capacity (influenced by local and lateral torsional buckling) than the base connections, as determined from experimental data curated by Lignos and Krawinkler [13]. In contrast, the rotational capacity of the base connections referenced above is comparable to that of beam-column moment connections (FEMA 350 [14]), which are the designated "fuse" element in steel moment frames.
2. From a constructional standpoint, requiring the base connection to be stronger than the column is expensive, requiring deep embedment, thick embedded base plates, and logistical overhead in terms of multi-stage concrete installation.

In summary, the current design methodology may well be counterproductive, disregarding the deformation capacity of the base connections to promote inelastic action in the columns, resulting in inferior performance at increased costs. Retrospectively, the prevalence of the strong-base-weak-column paradigm may be attributed to the notion that column hinges are likely to be more ductile than base connections, in the absence of test data to indicate the contrary. A collateral outcome of the strong-base-weak-column paradigm is that the post-yield or hysteretic response of column bases has remained virtually unexamined, since they are designed to remain elastic. Consequently, an approach to simulate hysteretic response of ECB connections is not available within prospective weak-base-strong-column systems that leverage the ductility and dissipative characteristics of base connections. More specifically, hysteretic models for ECB connections are not available for use within nonlinear time history simulations that establish interrelationships between base connection strength, ductility, and system performance. Such simulations (e.g., as outlined in FEMA-P695 [15] and NEHRP [16]) may be used to quantify frame performance metrics such as acceptable response modification factors (i.e., R & Ω_0), deformation demands, and probabilities of collapse and their relationship to base connection design.

Within this context, the main objective of this paper is to present a validated method to represent the hysteretic response of ECB connections. The method integrates physical behavior with previously developed models for strength and stiffness, to provide generalized modeling guidelines that effectively represent various aspects of ECB response.