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Predictive equations for shear link modeling toward collapse

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ABSTRACT

In this paper the predictive equations for collapse assessment of shear links used in eccentric braced frames are developed. An extensive database including results of over 70 cyclic tests on steel wide flange shear links is collected and the structural parameters governing the hysteresis behavior are calibrated using a simplified numerical model. The methodology of calibration is to minimize the discrepancy between the experimental hysteresis loops and the corresponding numerical results using Particle Swarm Optimization (PSO) algorithm. The objective function of PSO algorithm is minimized by iterating parameters that govern the hysteresis behavior of the numerical model. Stepwise multivariable regression is used to present equations for modelling shear link behavior parameters. The coefficient of determination for the derived empirical equations shows that the proposed equations can accurately capture the pre-capping, post-capping and cyclic deterioration behavior of the links for collapse assessments.

1. Introduction

Reliable collapse assessment of buildings needs accurate hysteresis behavior for the structural components. Various researchers have developed mathematical hysteresis models for different components of the building. The nonlinear behavior of steel and concrete components for various structural systems is also presented in different codes, such as ASCE41-13 [1,2]. The stiffness and strength detrioration is implicitly defined in this code by utilizing the backbone of the cyclic hysteresis curve without incorporation of deterioration models. Moreover, for some structural components the post-capping behavior is not well defined that makes it impossible to capture the collapse of the building. For instance, for shear links in eccentric brace frames (EBF) the residual strength is defined to be 80% of the nominal strength of the section. More sophisticated models are developed within the last ten years to capture the post-capping behavior and also stiffness and strength deteriorations. For example, Lignos and Krawinkler [3] developed empirical equations for evaluation of cyclic behavior of plastic hinge in steel moment resisting frames. The same was developed by Haselton et. al. [4] for the prediction of the flexure cyclic behavior of plastic hinge in concrete columns leading to global collapse of reinforced concrete structures. Lignos and Karamanci [5] developed equations for modelling the cyclic buckling and fracture of steel braces. All of these models are developed using results of quasi-static cyclic tests on the components. For instance, Lignos and Krawinkler [3] and Haselton et al. [4] used a set of experimental data including over 300 and 255 tests, respectively, to calibrate their numerical model with the test data.

All the above-mentioned hysteresis models are developed based on a specific numerical hysteresis model. Several hysteresis models are developed in the past forty years to capture cyclic behavior of structural components including stiffness and strength deterioration. Clough and Johnston model proposed in 1966 considers stiffness softening based on the maximum displacement in the loading history [6]. In 1970 Takada et al. [7] developed a trilinear model to capture the cyclic behavior of reinforced concrete components. In this model stiffness deterioration initiates after the flexural cracking and yielding of rebar. Bouc in 1967 [8] presented a smooth hysteresis model for the behavior of single degree of freedom systems. A series of modification has been conducted on this model over the years [8–11]. The updated model is called Bouc-Wen-Baber-Noori model that considers stiffness and strength deterioration as a function of cyclic dissipated energy. This model also incorporates pinching in the behavior of structural components. Rainhorn and Sivaselavan in 2000 [12] proposed a smooth hysteresis model that incorporates stiffness and strength deterioration for models with or without pinching. This model is also based on the firstly presented Bouc [8] model. Song and Pincheira model [13] presented in 2000 benefits post-capping behavior, ultimate deformation and residual path. Stiffness deterioration and pinching has been incorporated in this model. strength deterioration in this model, can be incorporated using the approximate methodology introduced by Pincheira et al. [14]. A series of papers has been published by Krawinkler from 90th where the basis of all

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models was on the energy concept of plastic hinges. The initial step was put forward by Ranama and Krawinkler in 1993 [15]. This model was modified for several times in the last twenty years by other researchers [15–20]. In the most recent modifications Lignos [19] implemented residual strength, asymmetrical deterioration and ultimate deformation to the model, named Modified IMK model. Modified IMK model is capable of capturing response of different types of structural components using a bilinear behavior. This model is also capable of capturing deterioration and collapse of structural components. In order to present predictive equations for model parameters, linear regression analysis has been used in the previous studies [3–5]. Castaldo et al. [21] used multivariable non-linear regression analysis to present predictive equations for ultimate flexural resistance and rotation capacity of RHS temper T6 aluminum alloy beams. Güneyisi et al. [22] presented a new formulation for steel beams flexural overtrength factor using artificial neural network. Güneyisi et al. [23] used neural network and genetic expression programming to predict rotation capacity of cold formed steel beams. Güneyisi et al. [24] also used the same approach to develop a new formulation for flexural overstrength factor of thin-walled circular hollow sections. In the most recent investigation on modelling steel links, Rossi et al. [25] used an experimental database to calibrate a simple refined model. This model which is based on Ramadan and Ghobarah [26] approach, includes five elements connected in series that can capture pre-capping part of cyclic response of the link. Rossi et al. [25] model does not consider strength, stiffness deterioration and effects of axial load; this model is only suitable for seismic assessment of eccentric braced frames before any important deterioration happens [25].

As mentioned, various models are available in the literature for modeling different structural components. By the knowledge of the authors no model is so far developed for modeling shear links that is capable of capturing post-capping behavior. In this paper empirical equations are developed based on the results of all available tests conducted on links with wide flange rolled section. In the following research objective, methodology, test database, and regression equations are presented.

2. Research objective

The purpose of this study is to develop empirical equations to predict the pre- and post-capping behavior of structural links. This is achieved by matching the experimental results of a set of test data with the corresponding numerical model. The governing parameters of the numerical model are calibrated such that the difference between each cycle of the test is minimized compared to that of the numerical model. The process is automated with an optimization procedure to find the best set of numerical parameters. Regression is finally used to propose the empirical equations for the dominant parameters of the links. In the following the methodology, deterioration model and the optimization procedure is described.

3. Methodology

Fig. 1(a) shows the experimental cyclic curve of a typical shear link. The numerical result of the calibrated corresponding zero-length link is also presented in this figure under the same loading protocol. The governing parameters of the modified IMK model in Opensees are calibrated such that the discrepancy between the numerical and experimental results is minimized. In a similar study on steel moment resisting frames Lignos [3] used engineering judgments to match the experimental and numerical results. Engineering judgment could result in different set of model parameters based on the standpoint of various users; thus an automated approach is generally more accurate. Palomino in 2011 [27] used genetic optimization algorithm for finding the suitable set of governing parameters. Herein, Particle Swarm Optimization (PSO) algorithm is used to calibrate modified IMK model parameters. Genetic algorithm (GA) could also be used to calibrate model parameters that leads to the same set of parameters for each specified case. However, Particle Swarm Optimization (PSO) algorithm is used due to the inherent ability to select input parameters from a continuous range which is desirable is in this investigation. To simulate the behavior of structures toward collapse it is necessary to capture the full response of components including, pre-capping, post-capping and deterioration. Fig. 1(b) shows the similar data of Fig. 1(a) with an alternative x axis replaced with the cumulative link rotation. In this figure the initiation and evolution of the post-capping behavior of the link can be observed clearer. The normalized difference in each cycle of the numerical and the experimental cycles is presented in Fig. 1(c). According to Eq. (1) the objective function is to minimize the summation of the square of the area difference in each cycle between the numerical and experimental results.

$$\varepsilon = \sum_{i=1}^{n} \alpha_i (Y_{exp} - Y_{num})^2 \tag{1}$$

where Y_{exp} and Y_{num} are the experimental and numerical area of each cycle, respectively. α_i also shows the normalized weight of each cycle. For example, if the post-capping behavior is of particular importance, α_i can be set higher in the optimization procedure in the cycles of post-capping. In this paper α_i is set to unity for all cycles for simplicity. MATLAB software is used to minimize the objective function using PSO algorithm by changing values of input parameters in their respective domains. In the following a brief review of PSO algorithm and the parameters domain are presented.

3.1. Deterioration model

Backbone curve defines the non-deteriorative behavior of components by means of bounds of strength and deflection, see Fig. 2. Strength parameters include V_{eq} or M_y (equivalent strength of section), V_c or M_c (capping strength) and post yield strength ratio (V_c/V_{eq}) or (M_c/M_y). Deflection parameters include θ_p (pre-capping rotation capacity), θ_{pc} (post-capping rotation capacity), and θ_u (ultimate rotation capacity). Using these parameters the skeleton of the response of the components is presented regardless of deterioration. In this model, three different modes of deterioration have been implemented, namely, basic strength, post-capping strength, unloading and reloading stiffness deterioration. Considering Rahnama and Krawinkler [15] rules for defining deterioration of structural components each component has an inherent dissipating energy capacity which is shown by E_t in equation (2). This inherent capacity is independent of loading history. The parameter β_i presented in equation (3) demonstrates deterioration of each mode in excursion i.

$$E_t = \lambda \theta_p V_{eq} \tag{2}$$

$$\beta_i = \left(\frac{E_i}{E_t - \sum_{j=1}^{i-1} E_j} \right)^c \tag{3}$$

where $\lambda\theta_p$: reference cumulative rotation capacity, θ_p: pre-capping rotation capacity, V_{eq}: equivalent strength of cross section, β_i: deterioration parameter in extrusion i, E_i: dissipated energy in extrusion i, E_t: reference dissipation energy capacity, $\sum_j^E$: dissipated energy in pervious extrusions, c: rate of cyclic deterioration.