Evaluation of Analysis Methods in Predicting Limit States for Performance-Based Seismic Design

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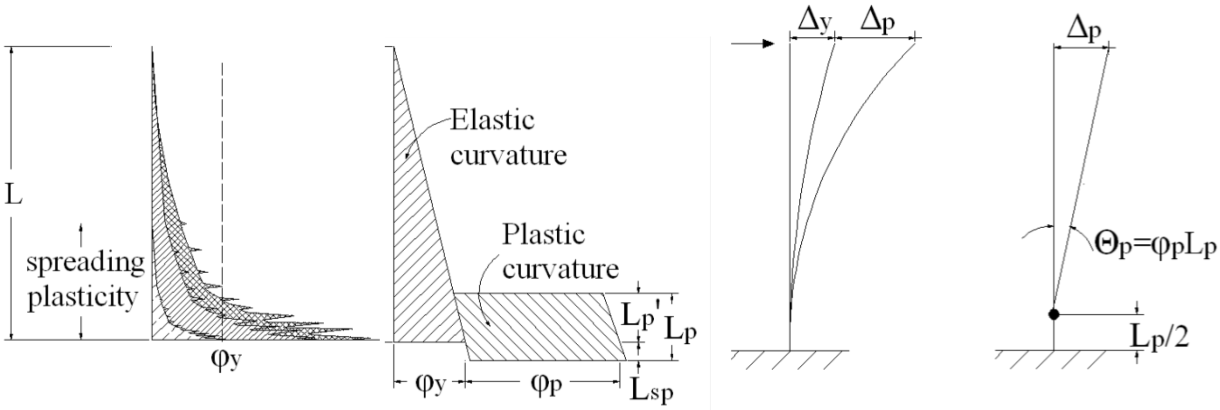
SUMMARY:
As seismic design moves towards a performance-based methodology, the ability to predict limit states by analytical solution is required. Many models exist for predicting inelastic behavior, and while they are reliable for global limit states such as ultimate displacement, their reliability for intermediate limit states, such as strain and curvature, is uncertain. In order to evaluate the performance of existing models, 34 reinforced concrete columns from the PEER database were analyzed using lumped plasticity and distributed plasticity finite elements. The use of different plastic hinge length models showed drastic and random differences for varying limit states, and integration schemes in distributed plasticity models were found to have a significant effect on intermediate response parameters. Preliminary results show sensitivity to aspect ratio, axial load, and reinforcement ratio. The study has highlighted the need for a unified definition of limit states, as well as improved models that may accurately and efficiently predict inelastic behavior at varying limit states.

Keywords: Plastic hinge length, Columns, Bridges, Performance-based-seismic-design

1. INTRODUCTION

Seismic design of bridges is typically detailed according to capacity design principles in which the columns are designed as ductile members that will experience considerable deformations under the seismic event without losing strength. Assessment of the nonlinear force-displacement response of a particular column element typically involves assumptions regarding the spread of plasticity in the member to calculate plastic rotations and displacements based on plastic curvatures. A widespread method for calculating the tip deformation on cantilever columns (using an effective length concept) is based on the approach proposed by Park and Paulay (1975). In this approach ultimate inelastic displacements (Δ_u) are obtained by the addition of elastic and plastic components using a simplified curvature distribution along the length of the column. The final inelastic deformations are the result of inelastic rotations assumed to occur within a region of the column where the inelastic curvatures are thought to be concentrated (see Figure 1). As such, Δ_u can be calculated on the basis of moment-curvature analyses to assess the nonlinear force-displacement response of a column. The simplicity of this approach has resulted in widespread use of the plastic hinge concept in seismic design. However, the accuracy and definition of the plastic design parameters $\phi_p = (\phi_u-\phi_y)$ and L_p as separate components has generated most of the research discussion (Hines et al. 2004) and is again the source of inquiry in this research.

The spread of plasticity in reinforced concrete (RC) members and its simplification into the notion of plastic hinging has been long discussed by researchers all over the world for over 40 years, covering fundamental and empirical approaches and addressing what seems to be all of its fundamental components. The role of the plastic hinge length as a key inelastic deformation parameter continues to this day and among the most recent enhancements are the contributions by Hines et al. (2004), Berry et al. (2008), and Bae and Bayrak (2008). This latest contributions continue to show that in spite of its long development, approaches and the calibration of this parameter against experimental data is still debated. A fundamental difference in current models is the way in which they give credit to the physical phenomena that affects spread of plasticity. The notion of the plastic hinge length L_p assumes a given plastic curvature to be lumped in the center of the equivalent plastic hinge. The physical phenomenon of inelastic actions is perhaps best defined by the length of the plastic hinge region L_{pr}, which is the physical length over which plasticity actually spreads along the element. While a direct relation between L_p and L_{pr} would be expected, this is not necessarily the case if inaccurate values for the plastic curvatures ϕ_p and L_p are used to obtain an accurate value of $\theta_p = \phi_p L_p$ that can be calibrated with experiments. The accuracy of L_p and ϕ_p and their separate determination is the basis for most of the contention on this topic. Nonetheless, it is well identified that the spread of plasticity in reinforced concrete elements is dependent on three distinct phenomena: moment gradient, tension shift and strain penetration (Park and Paulay 1975). The moment gradient reflects the transition between yield moment and ultimate moment in a member and is proportional to a member's shear span. The tension shift effect refers to the tendency of flexural forces to decrease only minimally over a certain distance over a critical section until these forces are transmitted to the compression zone by inclined struts. This effect invalidates the assumption that plane sections remain plane. The strain penetration effect refers to the fact that longitudinal bars can reach significant inelastic levels some distance into the footing or bent-cap. In spite of the recognition of these effects there is still no resolution in their integrated treatment for assessing the spread of plasticity.



a) Actual curvature distribution | b) Idealized curvature distribution | c) Displacement components | d) Plastic rotation and displacement

Figure 1. Schematic of the plastic hinge concept

Debate over the determination and use of the plastic hinge length concept and other plastic design parameters, such as plastic rotation and plastic curvature, has recently gained attention due to the need to assess the nonlinear force-deformation response of reinforced concrete members for an associated damage limit states for use in performance-based design (PBD). Relevant research that has contributed significantly towards improving the role of the plastic hinge in the determination of inelastic performance levels is that of Hines et al. (2004) and Barry et al. (2008). Hines et al. (2004) addressed the determination of L_p from first principles and proposed two models that physically account for the effect of spread of plasticity. A "bond stress model" suitable for wide members (e.g., walls) and a "shear crack" model that addresses moment gradient (and thus slender members) where proposed. The models are elegant in their relation to the physical plastic deformations and they are inherently linked to curvature demands on the element. However, the model's performance was essentially assessed only with respect to ultimate displacement calculation and the "shear crack" model (of most relevance to slender columns) was only compared to two experiments. In addition the models are relatively more complex than commonly used phenomenological expressions. Barry et al (2008) developed enhanced phenomenological plastic hinge models and calibrated them against diverse limit states (Lehman et al. 2004). The use of the models in lumped plasticity analyses is shown to have good results but only against peak lateral displacements. Further, the model is not rigorous enough to be able to account for the effects of tension shift, and its calibration against local damage parameters (i.e., curvature or strains) was not done since data was not available (Lehman et al. 2004). In spite of the noted research advancement, questions still remain about the adequacy of available plastic hinge formulations and the analytical and numerical methods that incorporate them. One is: is it over conservative or under-conservative to predict the correct length of the plastic hinge? One other is: how much does this error actually matter in evaluating the response of bridges? Finally: for which applications does the error become significant either in terms of design or assessment? This paper summarizes the first phase in an effort to find answers to these questions by evaluating the performance of well-known plastic hinge models against archived experimental data and in assessing the performance of finite element formulations for evaluating nonlinear response.

2. NUMERICAL MODELING

Several models have been proposed to predict the inelastic behavior of reinforced concrete columns under seismic demand. Such models can generally be placed into two categories, lumped plasticity and distributed plasticity approaches. The lumped plasticity approach is advantageous due to its simplicity for hand calculations and its computational efficiency when used in the finite element method. The distributed plasticity approach, however, is convenient for defining limit states for PBD since it has the ability to capture local behavior at intermediate element lengths and takes the spread of plasticity along an element length into account.

The performance evaluation of plastic hinge models on cantilevered columns was done by comparing experimental deformation data at different limit states with the predictions from numerical models created in the OpenSees finite element platform (OpenSees 2011). Columns were modeled using fiber-based lumped plasticity and distributed plasticity beam-column elements. Lumped plasticity analyses are completed in OpenSees by using the "beamWithHinges" element, which uses a modified Gauss-Radau integration scheme. This element uses a flexibility formulation and assumes all inelastic behavior is concentrated over a user-defined length at the element ends. Distributed plasticity analyses are possible in OpenSees using a "nonlinearBeamColumn" element. This element uses a flexibility formulation and various integration schemes are available in order to capture the changing nonlinear distribution of element section deformations. The control of sub-elements and integration points allows capturing of the spread of plasticity along the element length.

Fiber sections consisted of a confined concrete core defined by a Chang and Mander (1994) concrete model (OpenSees material Concrete07), as well as an unconfined concrete cover defined by a zero tensile strength concrete model (OpenSees material Concrete01). The longitudinal reinforcing steel was modeled using the OpenSees material model ReinforcingSteel. The parameters given in Table 1 were used to define the stress-strain curve for the steel model. Yield strength was the only parameter given in the PEER database; therefore other inputs were selected based on commonly observed values.

Table 1. Assumed reinforcing steel material properties in finite element modeling

Input Parameter	Assumed Value
Yield Strength (fy)	Given in documentation (varies)
Ultimate Strength (f_u)	$1.5f_y$
Elastic Modulus of Steel (E_s)	29,000 ksi
Elastic Modulus after Strain Hardening (E_{sh})	1633 ksi
Strain at Hardening (ε_{sh})	0.0036
Ultimate Strain (ε_{su})	0.1

All monotonic analyses were performed using a displacement control integrator, with yield and ultimate values as target displacements. Yield and ultimate displacements were calculated from parameters found in the sectional analysis for each column, using the approach proposed by Priestley et. al. (1996). Experimental yield displacement was not given in the PEER database and was therefore calculated. While experimental values for ultimate displacement are available in the PEER database, these values were not used in the analysis as the target displacement. Rather, ultimate displacement was calculated based on a failure criteria.

3. EVALUATION OF PLASTIC HINGE DESIGN MODELS

A statistical analysis was performed on several columns from the PEER Structural Performance Database (PEER 2011) to evaluate the performance of four common plastic hinge expressions in predicting various limit states for implementation in performance-based seismic design. Evaluation was done by conducting monotonic analyses with lumped plasticity finite element models.

3.1. Column Selection

Limit state prediction was compared for various plastic hinge models by analyzing 34 circular columns from the PEER Database (PEER 2011). The database contains a total of 163 circular columns, therefore in order to be considered relevant to this study, columns were chosen according to specific criteria. The criteria for choosing columns for this work are as follows:

- Column failure was of the flexure type.
- Columns must have an aspect ratio (length/diameter) greater than 3.
- Columns must have well-confined flexural hinges.
- Lastly, columns without recorded limit state data were eliminated.

The resulting 34 columns used in this study are listed in Table 2. The table presents information on geometry, material properties, and experimental displacements at different limit states.

Table 2. PEER Database columns used in limit state analyses (PEER 2011)

Reference	Unit	L (in)	L/D	P/f'_cA_g (%)	ρ_l	ρ_{eff}	s/d_b
Davey 1975	No. 2	68.9	3.5	12.07	0.0271	0.0461	3.53
Munro et. al. 1976	No. 1	107.5	5.5	0.34	0.0271	0.0948	1.85
Ng et. al. 1978	No. 3	36.6	3.7	33.95	0.0230	0.2173	0.83
Ang et. al. 1981	No. 1	63.0	4.0	20.81	0.0256	0.0881	2.50
Stone et. al. 1986	Model N6	59.1	6.0	10.49	0.0196	0.1283	2.07
Stone et. al. 1989	Full Scale Flex.	359.8	6.0	6.85	0.0200	0.0826	2.07
Watson & Park 1989	No. 10	63.0	4.0	52.76	0.0192	0.0743	5.25
Kowalsky et. al. 1996	FL1	143.9	8.0	29.65	0.0362	0.1176	4.79
Kowalsky et. al. 1996	FL2	143.9	8.0	27.13	0.0362	0.0724	3.21
Kowalsky et. al. 1996	FL3	143.9	8.0	28.11	0.0362	0.1115	4.79
Kunnath et. al. 1997	A2	54.0	4.5	9.44	0.0204	0.1439	2.00
Kunnath et. al. 1997	A3	54.0	4.5	9.44	0.0204	0.1439	2.00
Kunnath et. al. 1997	A4	54.0	4.5	8.56	0.0204	0.1176	2.00
Kunnath et. al. 1997	A5	54.0	4.5	8.56	0.0204	0.1176	2.00
Kunnath et. al. 1997	A7	54.0	4.5	9.26	0.0204	0.1272	2.00
Kunnath et. al. 1997	A8	54.0	4.5	9.26	0.0204	0.1272	2.00
Kunnath et. al. 1997	A9	54.0	4.5	9.35	0.0204	0.1284	2.00
Kunnath et. al. 1997	A10	54.0	4.5	10.14	0.0204	0.1546	2.00
Kunnath et. al. 1997	A11	54.0	4.5	10.14	0.0204	0.1546	2.00
Kunnath et. al. 1997	A12	54.0	4.5	10.14	0.0204	0.1546	2.00
Hose et. al. 1997	SRPH1	144.1	6.0	14.82	0.0266	0.0965	2.56
Henry 1998	415p	96.0	4.0	12.04	0.0149	0.0857	2.00
Henry 1998	415s	96.0	4.0	6.02	0.0149	0.0428	4.00
Lehman et. al. 1998	407	96.0	4.0	7.22	0.0075	0.1028	2.00
Lehman et. al. 1998	415	96.0	4.0	7.22	0.0149	0.1028	2.00
Lehman et. al. 1998	430	96.0	4.0	7.22	0.0302	0.1028	2.00
Lehman et. al. 1998	815	192.0	8.0	7.22	0.0149	0.1028	2.00
Lehman et. al. 1998	1015	240.0	10.0	7.22	0.0149	0.1028	2.00
Calderone et. al. 2000	828	192.0	8.0	9.06	0.0273	0.113	1.33
Calderone et. al. 2000	1028	240.0	10.0	9.06	0.0273	0.113	1.33
Kowalsky & Moyer 2001	1	96.0	5.3	4.31	0.0207	0.1427	4.01
Kowalsky & Moyer 2001	2	96.0	5.3	4.12	0.0207	0.1365	4.01
Kowalsky & Moyer 2001	3	96.0	5.3	4.44	0.0208	0.1478	4.00
Kowalsky & Moyer 2001	4	96.0	5.3	4.16	0.0208	0.1385	4.00

3.2. Plastic Hinge Design Models

Four plastic hinge design models were chosen to compare limit state prediction for performance-based design (PBD). The models selected represent those most commonly used in practice, as well as recently developed models created to be better suited for PBD or under extreme design parameters. The four plastic hinge expressions are summarized in Table 3.

Table 3. Plastic hinge expressions used in finite element analyses

Reference	Expression
Priestley et. al. (2006)	$L_p = kL + 0.15 d_b f_y \geq 0.3 d_b f_y \ (ksi) where\ k = 0.2\left({f_u}/{f_y} - 1 \right) \leq 0.08$
Berry et. al. (2008)	$L_p = 0.0375L + 0.01 f_y \, {d_b}/{\sqrt{f'_c}} \ (psi)$
Bae and Bayrak (2008)	$L_p = L\left(0.3\frac{P}{P_0} + 3\frac{A_s}{A_g} - 0.1\right) + 0.25h + L_{sp} \ \geq 0.25h$
Corley (1966)	$L_p = 0.5D$

The expression by Priestley is the most well-known and widely used equation consisting of a bending component and strain penetration component (Priestley et al. 2006). The expression by Berry et al. (2008) is one more recently developed after similar studies of the PEER database. The expression was calibrated to reduce the error in predicting the displacement at ultimate, the onset of spalling, and the onset of bar buckling (Berry 2008). Error was calculated based on analyses performed using plastic hinge expressions by Priestley and by Corley. The plastic hinge expression proposed in the study by Berry performed well compared to monotonic force-displacement envelopes. It is therefore the intention to test this expression for other intermediate limit states as well as performance under cyclic loading. Similar to the Priestley equation, the one proposed by Berry et al. includes both a bending and strain penetration component. The expression proposed by Bae and Bayrak (2008) was formulated to solve discrepancies between previous plastic hinge expressions, specifically to solve sensitivity to large axial loads. The expression by Bae and Bayrak does not include a strain penetration component. Therefore one was added to ensure that the models were comparable. The expression by Corley (1966) is one of the oldest plastic hinge length expressions, yet due to its simplicity is of interest to the study. Lumped plasticity approaches are used due to their simplicity, therefore if more complex plastic hinge expressions do not necessarily provide more accuracy, it is possible that the Corley equation is sufficient for seismic analysis. The Corley equation does not contain a strain penetration component; however one was not added to the expression for analysis since it is assumed the simplified expression is meant to include all components of inelastic behavior as proposed.

3.3. Limit State Definitions

Limit states predicted in the OpenSees analysis and compared to the experimental data in the PEER database include: the displacement at concrete core crushing, significant cover concrete spalling, longitudinal bar buckling, longitudinal bar fracture and ultimate displacement/failure. Experimental data for each of these limit states is recorded in the PEER database and is based on observation. In order to predict the onset of each of these limit states in the analyses, strains were defined for each limit state at which these important damage states are thought to occur. The displacement corresponding to each was then compared to the experimental data. A summary of how the strain at various limit strains was predicted is given below:

3.3.1 Crushing

In this study, crushing is defined as the instant when the extreme compression fiber of the concrete core reaches the maximum compressive strain as defined by the Chang and Mander (1994) Concrete Model. The crushing strain is computed with Equation (3.4). This model takes cyclic behavior of concrete into account. In experimental studies, crushing typically refers to the point when the inside of the steel spiral is fully exposed (Lehman 2004), yet this may vary between researchers.

$$\varepsilon'_{cc} = 0.002\left[1 + 5\left({f'_{cc}}/{f'_c} - 1\right)\right] \tag{3.4}$$

3.3.2 Spalling

In this study, significant spalling is defined at the instant when the unconfined concrete strain is 0.005. It was assumed that unconfined concrete typically begins to spall at a strain of about 0.002 therefore a strain of 0.005 would indicate significant spalling. To obtain this point in the OpenSees analysis, the equivalent concrete core strain was calculated according to Equation (3.5); where D is the column diameter and cc is the distance from the concrete cover to the longitudinal reinforcement. Experimentally, significant spalling is completely based on observation and varies with individual perception. This limit state is therefore very ill defined it is expected that error in analytical comparison will be high.

$$\varepsilon_{core} = \frac{0.005(D - cc)}{D} \tag{3.5}$$

3.3.3 Bar Buckling

Longitudinal bar buckling is defined according to the equation proposed by Berry and Lehman (2008). They proposed that the onset of bar buckling is best predicted as a function of the effective confinement ratio, ρ_{eff}. The resulting equation is shown in Equation (3.6). The constants, X_1 and X_2 were calibrated using experimental data from several PEER test columns and determined to be equal to 0.05 and 0.224, respectively. These columns were all tested under cyclic loading. The values of the constants in the expression by Berry and Lehman change depending on the plastic hinge length of the column. For the purposes of this study however, the optimal constants recommended for the optimal plastic hinge by Berry et al. were used for all columns, regardless of plastic hinge expression used for the analysis. The reason is that the different constants are used to resolve error, which defeats the purpose of this study. Since the Berry model is proposed as the most accurate, these constants are used for each analysis.

$$\varepsilon_{bb} = X_1 + X_2\rho_{eff} \tag{3.6}$$

3.3.4 Fracture

Longitudinal bar fracture is defined as the point when the tensile strain in the extreme tensile steel fiber reaches 0.1. It is well confirmed that steel fractures at a strain between 0.1-0.15, therefore the lower limit was used. Since fracture would also typically indicate failure of a column, a steel strain of 0.1 is also used as one of the three failure criteria in the analyses.

3.3.5 Ultimate/Failure

Failure or ultimate conditions defined in the OpenSees analysis are based on the first occurrence of:

i. Concrete Failure: The column is said to have failed by concrete failure if the strain in the extreme fiber of the concrete core reaches ultimate strain (ε_{cu}) as defined by the Chang and Mander concrete model. This expression is given in Equation (3.7).
ii. Steel Failure: The column is said to have failed by steel failure if the strain in the extreme fiber of the reinforcing steel reaches ultimate strain (ε_{su}) as defined in the previous section as 0.1.
iii. 20% Capacity Loss: The column is said to have failed by capacity loss if the load carrying capacity of the structure falls below 80% of the maximum force as determined from the element analysis before reaching failure of concrete of steel as described above.

$$\varepsilon_{cu} = 0.004 + 1.4\rho_s\varepsilon_{su}\,{f_y}/{f'_{cc}} \tag{3.7}$$

3.4. Results

The predicted displacement at several limit states was compared to the recorded experimental results to obtain several statistics. First, the mean error and coefficient of variance was calculated for each plastic hinge model at different limit states by comparing analytical to experimental displacement. Values are given in Table 4. Statistics are further sorted by aspect ratio to determine if high aspect ratios (L/D>8) show higher error. As seen in the table, any such trend is random among limit states.

Table 4. Mean error and coefficient of variance for prediction of limit states using different plastic hinge models

Lp Model	Mean			Coef. Var.		
	4 ≤ L/D < 8	L/D ≥ 8	**Total**	4 ≤ L/D < 8	L/D ≥ 8	**Total**
CRUSHING L.S.						
Priestley	0.4315	0.3204	**0.4037**	0.8123	0.6368	**0.7921**
Berry	0.2936	0.1629	**0.2566**	0.6783	0.3622	**0.7043**
Bayrak	0.2996	0.2474	**0.2697**	0.6787	0.4466	**0.7054**
Corley	0.3453	0.1491	**0.3008**	0.6740	0.5179	**0.7422**
SIGNIGICANT SPALLING L.S.						
Priestley	0.3782	0.1140	**0.3182**	0.6465	1.0860	**0.7783**
Berry	0.4220	0.1641	**0.3634**	0.5832	0.5529	**0.6738**
Bayrak	0.4993	0.2592	**0.4447**	0.4841	0.6139	**0.5505**
Corley	0.4153	0.1789	**0.3616**	0.5951	0.6098	**0.6723**
BAR BUCKLING L.S.						
Priestley	0.4684	0.1935	**0.4206**	0.7069	0.5708	**0.7620**
Berry	0.2330	0.2008	**0.2202**	0.7268	0.3877	**0.7356**
Bayrak	0.3356	0.5252	**0.3722**	0.4295	0.0378	**0.3956**
Corley	0.2559	0.3076	**0.2701**	0.7957	0.2844	**0.6837**
BAR FRACTURE L.S.						
Priestley	0.6794	0.5655	**0.6587**	0.5133	0.0653	**0.4791**
Berry	0.2915	0.0282	**0.2162**	0.5496	0.2255	**0.8034**
Bayrak	0.2720	0.4471	**0.3016**	0.4749	0.0295	**0.4425**
Corley	0.3324	0.0964	**0.2648**	0.4782	0.5668	**0.6977**
ULTIMATE L.S.						
Priestley	0.8124	0.5654	**0.7484**	0.6063	0.3872	**0.5984**
Berry	0.3676	0.1687	**0.3161**	0.8646	0.6857	**0.9214**
Bayrak	0.2699	0.5541	**0.3436**	0.8672	0.2173	**0.7099**
Corley	0.4666	0.1674	**0.3890**	0.5999	0.6450	**0.7169**

The predicted and experimental drift ratios for all limit states were compared graphically by plotting the response from all the analyses and finding a least-squares regression line for the data. Plots for the bar buckling limit state are shown in Figure 2. The crushing and spalling limit states had poor predictions for nearly all models. Spalling prediction was especially poor, most likely due to the fact that concrete strains are difficult to measure and thus this limit state is typically empirically assessed. Buckling, fracture and ultimate are limit states were expected to give more accurate results since there is less ambiguity on their definition. While the models performed better for these limit states, performance was still poor, indicating little reliability for PBD implementation. None of the plastic hinge models were able to lead to accurate results throughout all limit states. However, the Berry et al. model generally had the best performance.

4. EVALUATION OF NUMERICAL METHODS

The performance of common numerical methods for the prediction of intermediate limit states and the effect of aspect ratio was studied by conducting detailed analyses on four of the test columns in the database. The chosen columns had aspect ratios between 4 and 10 and conform to modern seismic design standards for reinforced concrete structures. Their geometric and reinforcement properties are given in Table 5. The columns were analyzed with OpenSees using the lumped and distributed plasticity elements. Monotonic and cyclic analyses were performed.

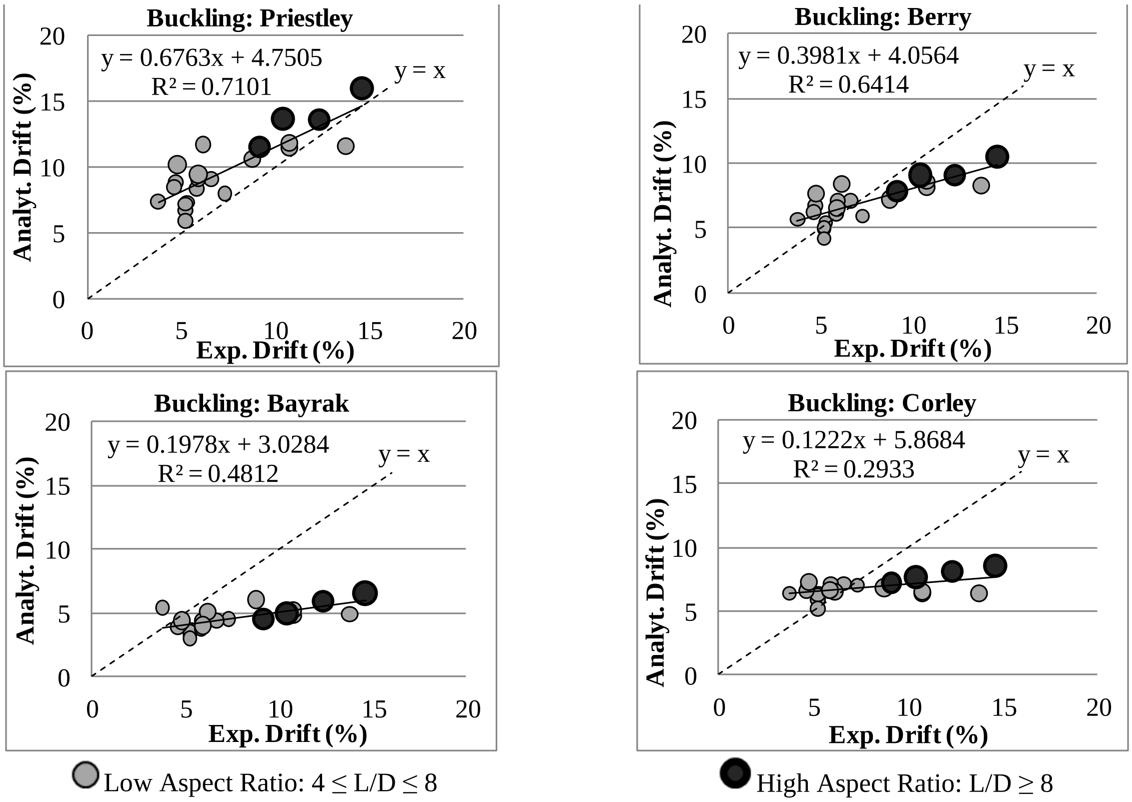


Figure 2. Drift predictions for longitudinal bar buckling limit state

Table 5. Column properties for evaluation of numerical analysis and slenderness effects

Column Reference	Diameter	L/D	Longitudinal Reinf.	Cover	Transverse Reinf.	$P/f'_c A_g$
415 (Lehman 1998)	24"	4	22 # 5, ρ_l=1.5%	0.75"	#2 spiral @ 1.25", ρ_s=0.70%	7.22%
SRPH1 (Hose 1997)	24"	6	20 # 7, ρ_l=2.7%	1"	#3 spiral @ 2.25", ρ_s=0.86%	14.82%
815 (Lehman 1998)	24"	8	22 #5 , ρ_l=1.5%	0.75"	#2 spiral @ 1.25", ρ_s=0.70%	7.22%
1015 (Lehman 1998)	24"	10	22 #5, ρ_l=1.5%	0.75"	#2 spiral @ 1.25", ρ_s=0.70%	7.22%

4.1. Monotonic Analysis Results

Monotonic analyses were performed on each column using the lumped plasticity elements with different plastic hinge definitions. The monotonic envelope from each model compared to the experimental hysteretic response is shown in Figure 3. Also plotted on each graph is the displacement at which each of the limit states occurred. It is seen that the Priestley model generally overestimates limit states, while the Bayrak model typically underestimates them. The Berry and Corley models are somewhere in between, with the Berry model typically showing the best comparison to experimental data. It is also seen that the prediction of global response is best for the aspect ratio 6 column, with decreasing accuracy with aspect ratios above and below this. The most slender column does in fact show the highest error in prediction, as expected.

4.2. Cyclic Analysis Results

A cyclic analysis was also performed on each of the above columns using both lumped plasticity and distributed plasticity elements. Since the Berry model gave the best results of the plastic hinge models in the monotonic analyses, it was chosen as the representative model for comparing lumped plasticity elements to the distributed plasticity analyses. A comparison of both models to the experimental results of each column is given in Figure 4(a) for the SRPH1 column. It is seen that both models give a very similar prediction to each other.

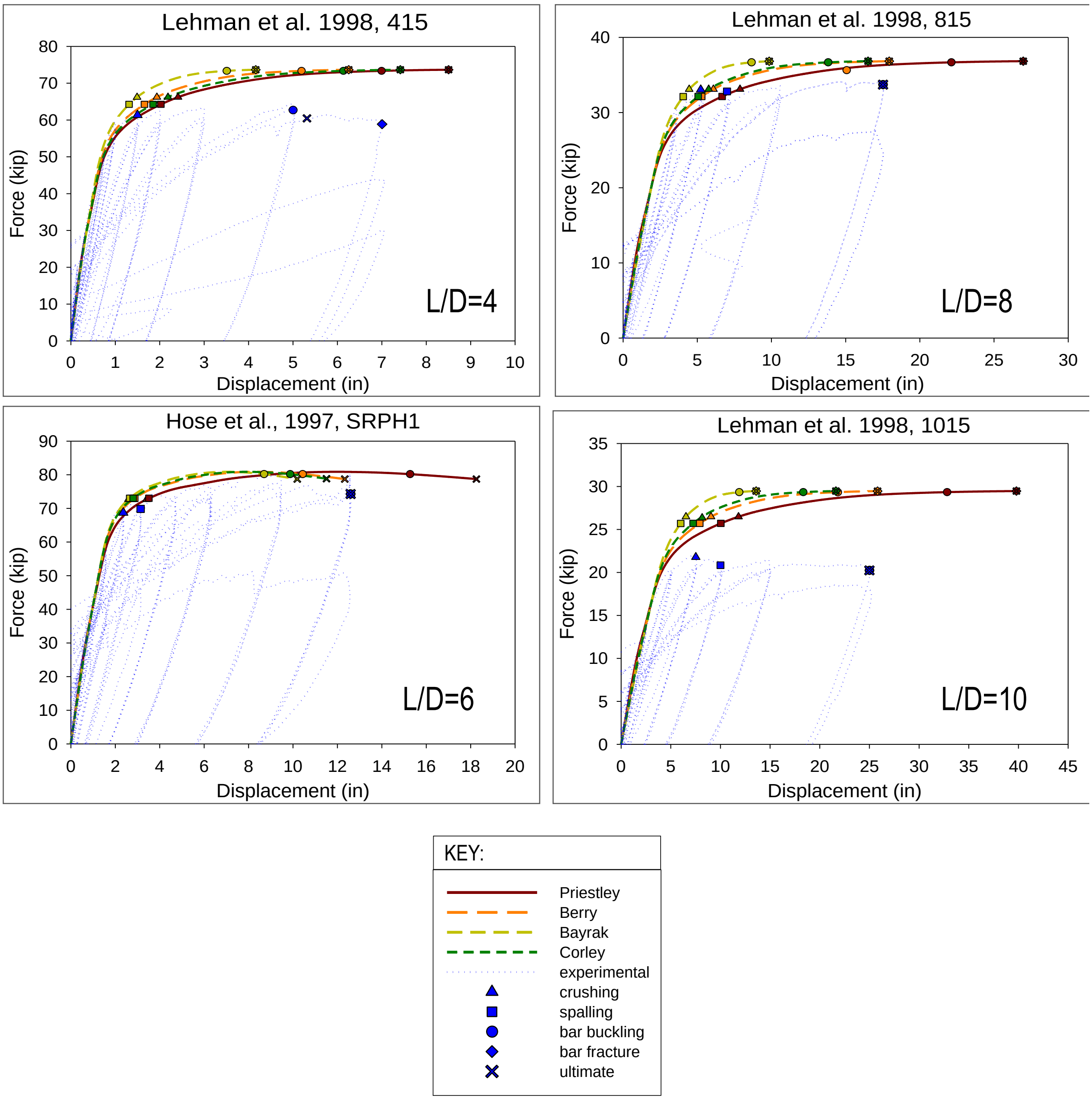
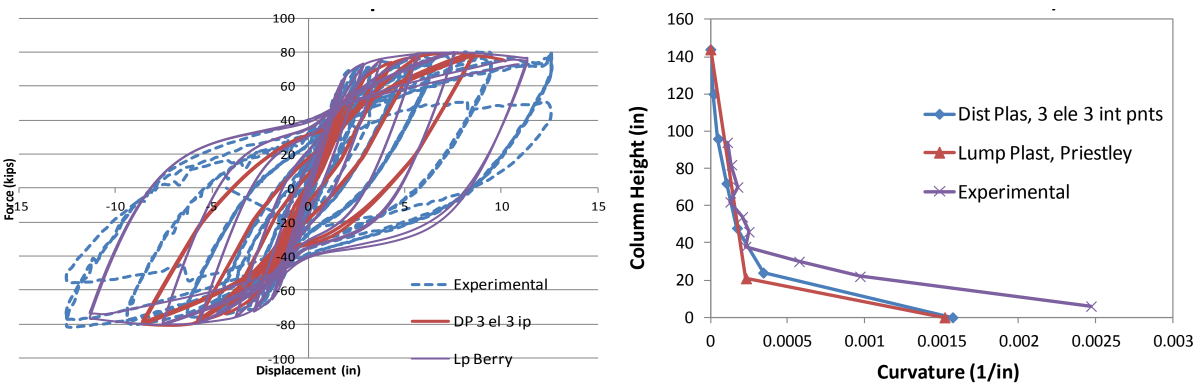


Figure 3. Performance of a lumped plasticity finite element model in assessing intermediate limit states



a) Hysteretic force-deformation response

b) Curvature profiles at $\mu_\Delta = 4$

Figure 4. Cyclic analysis results for SRPH1 column using lumped and distributed plasticity models

Accuracy in prediction to the global experimental behavior, however, is not improved upon by using the distributed plasticity element. The Berry plastic hinge model seems to better predict ultimate displacement, however comparing local behavior such as curvatures and strains is of interest to determine which, if any, model is most accurate. Such comparison was completed for the Hose SRPH1 column. Figure 4(b) shows the curvature profile for the SRPH1 column at a displacement ductility level of 4. Compared in this figure are results from a lumped plasticity element with the Priestley plastic hinge length model and a distributed plasticity model with 3 elements and 3 integration points per element. The data shows that none of the models is very accurate at predicting the local response of this column.

5. CONCLUSIONS

Careful evaluation of commonly used plastic-hinge models shows a wide scatter against experimental data and assessment parameters computed using these models do not comply well with the intended damage level. This is more evident when applied to limit states other than ultimate – the state at which most models were calibrated. The error in predicting inelastic behavior of reinforced concrete columns seems to increases with increasing aspect ratio. This trend is apparent for all numerical techniques studied, including lumped plasticity and distributed plasticity models. However, more research is required to make a final statement on the issue of slenderness.

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