A Model for Seismic Performance Assessment Of Bridge Piers

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SUMMARY:
Bridges design codes are in a transitional state toward performance-based design and it is still not clear how actual design criteria can meet different performance criteria for different class of bridges. The paper presents a modeling technique that can predict the local performance of piers subjected to earthquake loading with adequate accuracy. The concrete model accounts for confinement, tension stiffening and cyclic behaviour of concrete with the Laborderie Model, a model based on damage mechanics. The cyclic behaviour of steel is predicted with a model based on Menegoto Pinto Model. The model includes also the effects of reinforcement buckling. The models have been included in the open source software Opensees. The modeling technique results are compared to experimental results in the literature. The predictions are very close to the experimental values on a global level as well as on a local performance level.

Keywords: Bridge pier, local performance, reinforcement buckling, model

1. INTRODUCTION

Major seismic events during the past few decades have continued to demonstrate the destructive power of earthquakes, with failures to structures such as bridges, as well as giving rise to great economic losses. Economic losses for bridges very often surpass the cost of damage and should therefore be taken into account in selecting seismic design performance objectives. The structural engineering community in its transition to performance-based seismic design codes has proposed several methodologies for performance-based seismic design or upgrading.

Design codes have adopted different approaches to achieve required performance objectives. However, the performance objectives in the design codes are defined qualitatively in terms of design principles called the "seismic design philosophy". It is not clear how design requirements are related to design principles and economic considerations. To assess code requirements, a vast amount of experimental evidences would be necessary. However, such experiments are normally very expensive. Numerical techniques could be an alternative but the behavior of reinforced concrete columns is complex and the testing strategy should be adapted. The objective of the article is to present a modeling strategy that provides a high level of accuracy and is able to capture complex behavior such as buckling of longitudinal reinforcement bars, confinement, crack opening, and localization of strain. After defining levels of performance, this paper presents the predictions of three large scale pier column tested by Lehman and Moehle 2000 to evaluate the adequacy of the modeling strategy.

2. PERFORMANCE OF BRIDGE PIERS

The term performance can be defined as the association of a certain seismic demand to a performance objective, which can be associated with a local limit state or a damage level. Fig. 2.1 presents the different limit states that a bridge pier can undergo during an earthquake. Table 2.1 provides actual performance level that might be related to code based performance principle and are in line with recent development of performance-based seismic assessment (Hose et al. 2000 Lehman et al. 2004). Both qualitative and quantitative performance levels are described in Table 2.1 and are associated with engineering parameters. Up to limit state 1A, there are only barely visible cracks and no repair is necessary. The onset of yielding defines the limit state 1B. The limit state 2 is represented by the onset of concrete cover spalling which correspond to an extreme fiber compression strain of 0.004 as proposed by Sheikh and Légeron 2010. This limit state corresponds to moderate damage where the bridge circulation may only be allowed for emergency vehicles. For the limit state 3, extensive damage occur and repair may not be possible, reconstruction may be needed. This limit state is defined by the onset of core crushing, bar buckling or fracture of transverse hoops. Based on recommendation (Sheikh and Légeron 2010), onset of initial core crushing has been taken as the point where $\epsilon_c=\epsilon_{cc50}$.

Table 2.1. Performance level description

Limit states (LS)	Operational performance level	Post earthquake serviceability	Qualitative performance description	Quantitative performance description	Repair
1A	Fully Operational	Full service	Onset of hairline cracks	Cracks barely visible	No repair
1B			Yielding of longitudinal reinforcement	Crack width <1 mm	Limited epoxy injection
2	Delayed Operational	Limited service	Initiation of inelastic deformation; onset of concrete spalling; development of longitudinal cracks	Crack width: 1-2 mm ϵ_c = -0.004	Epoxy injection; concrete patching
3	Stability	Closed	Wide crack width/ spalling over full local mechanism regions; buckling of main reinforcement; fracture of transverse hoops; crushing of core concrete; strength degradation	Crack width>2 mm $\epsilon_c=\epsilon_{cc50}$ (initial core crushing) $\epsilon_c=\epsilon_{cu}$ (fracture of hoops) $\epsilon_s>0.06$ (longitudinal reinforcement fracture)	Extensive repair / reconstruction
ε_c=axial strain of concrete; ε_{cc50}=post peak axial strain in concrete when capacity drops to 50% of confined strength; ε_{cu}= ultimate strain of concrete; ε_s=tensile strain at fracture					

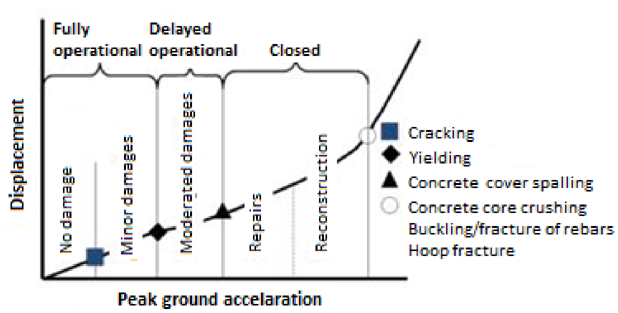

Figure 2.1. Limit states of RC bridge piers

3. MODELING TECHNIQUES FOR BRIDGE PIERS

In order to predict seismic performance of reinforced concrete bridge piers, it is important that the model be able to predict the global response as well as the local response of the structure. As seen in Table 2.1, local performance is associated with damage level in the structure which is related to structure reparability. Local performance is expressed by the level of cracking, reinforcement yielding, buckling and fracture, as well as the irreversible damage in compression of concrete that is observed following an earthquake. The modeling of bridge pier local limit states presented in this paper is done using the open source software Opensees. This section presents the models used to be able to predict the local performance of bridge piers.

3.1 Concrete Cracking

As seen in Table 2.1, crack width, ω_{max}, may be used as a quantitative performance level. Crack opening is also related to the level of repair needed. There are many equations in different design code that are used to predict crack width in concrete structures subjected to flexure. In this section, the method used in the EC2 and the CSA-S6 design codes as well as the Gergely-Lutz and the Chowdury and Loo 1998 equations are compared. As demonstrated in Fig. 3.1 and 3.2, the Gergely-Lutz equation (Eqn. 3.1) is the one that gives the best results, therefore it will be used in the model. This model was included in Opensees.

$$\omega_{max} = 2.2\beta\varepsilon_{scr}\sqrt[3]{d_c A} \qquad (3.1.)$$

where β may be taken as h_2/h_1, where h_1 is the distance from the neutral axis to the extreme fiber in tension and h_2 is the distance from the neutral axis to the centre of gravity of the reinforcement in tension. ε_{scr} is the steel deformation at the crack, d_c is the distance from the extreme fiber in tension to the center of the closest reinforcement and A is the effective concrete area in tension.

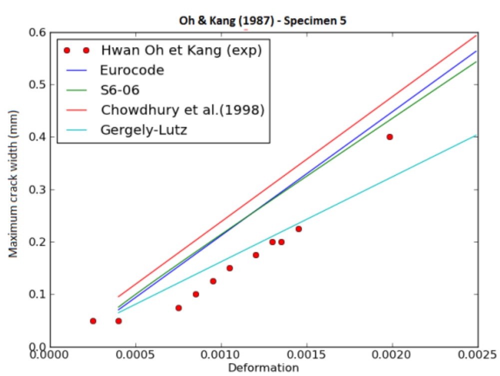

Figure 3.1. Crack width comparison

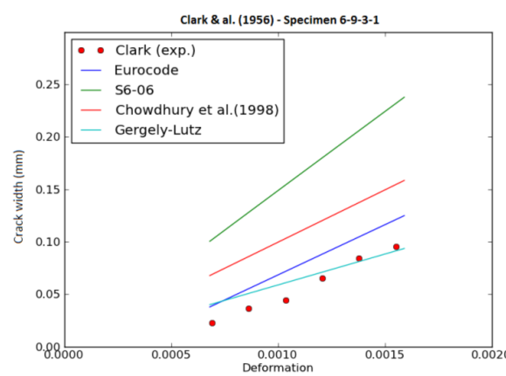

Figure 3.2. Crack width comparison

3.2 Cyclic behaviour of concrete

The cyclic behaviour of concrete is modeled with the uniaxial damage mechanics model proposed by LaBorderie 1991 (Fig. 3.3). This model uses damage variables to quantify the damage states of concrete and takes into account the stiffness loss, the residual deformations and the stiffness restitution. It considers that it is the damage variables that control the stiffness loss and the residual deformations. The model was shown to be adequate for modeling of concrete structures by Legeron et al. 2005. The model was added to Opensees.

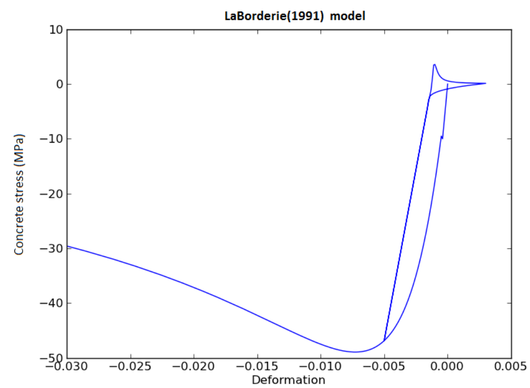

Figure 3.3. Cyclic concrete model (LaBorderie 1991)

All the parameters describing the cyclic behaviour of concrete are identified independently. The monotonic stress-strain law in compression is modeled after the Légeron and Paultre 2003 approach to account for confinement effect. The Légeron and Paultre 2003 approach was demonstrated by Sharma et al. 2005 to be one of the best models to account for confinement. The parameters of the model adjusting the residual strain in compression are based on the Dodd and Cooke 1994 model. The tensile behaviour is modeled with a method based on experimental results, and this approach is described in Legeron et al. 2005

3.3 Concrete cover loss

The behaviour of the concrete cover must be able to represent cover loss. It has been found that concrete loss occurs at deformations around 0.004 as stated in Sheikh and Légeron 2010, therefore the concrete model is implemented in order to limit the concrete contribution up to a deformation of 0.004 after which the concrete resistance is near zero. The parameters of the model are adjusted in order for the concrete loss model to follow the unconfined concrete curve as shown in the Fig. 3.4.

3.4 Modeling of reinforcing steel

The cyclic steel behaviour is modeled using the STEEL02 material included in Opensees which is based on the Menegotto-Pinto model and includes the bauschinger effect and the isotropic hardening.

3.5 Buckling of longitudinal reinforcement

It has been shown that a good model for buckling of rebar must include the L/D ratio as a parameter where L is the tie spacing and D the bar diameter (Russo 1989, Monti and Nutti 1992 and Mander et al. 1994). Dhakal and Maekawa 2002 and Gomes and Appleton 1999 have proposed constitutive models that include L/D ratio as the main parameter. However, L/D ratio may not be sufficient to accurately model longitudinal bar buckling in reinforced concrete piers as the buckling may occur over more than one tie spacing as seen in Lehman and Moehle 2000. Based on early work performed within this research project, it was observed that none of the existing models were able to predict the results of buckling on several ties. As this kind of buckling is very difficult to model with a simplified approach, it was decided to describe the bars in the Opensees model as well as the ties. The approach is complex but provides good results. Buckling is obtained by introducing an initial eccentricity to the longitudinal bar. Comparison with experimental results of Monti and Nuti 1992 as shown in Fig. 3.5, 3.6, and 3.7 for different L/D ratios shows that this method gives good results. The Opensees model obtained is shown at Fig. 3.9 for a bridge column.

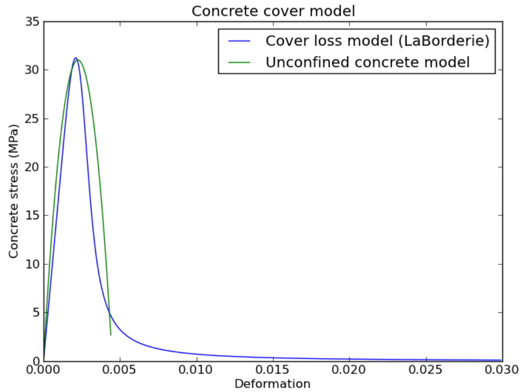

Figure 3.4. Model for concrete cover

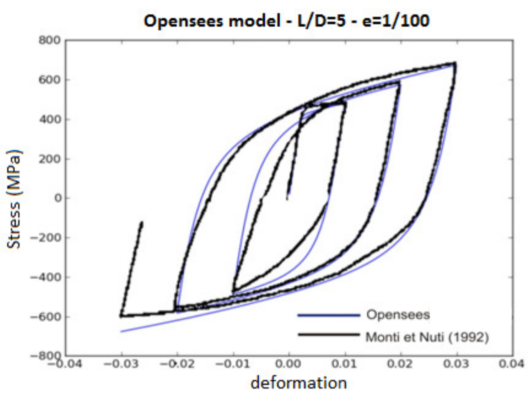

Figure 3.5. Buckling modeling (L/D=5)

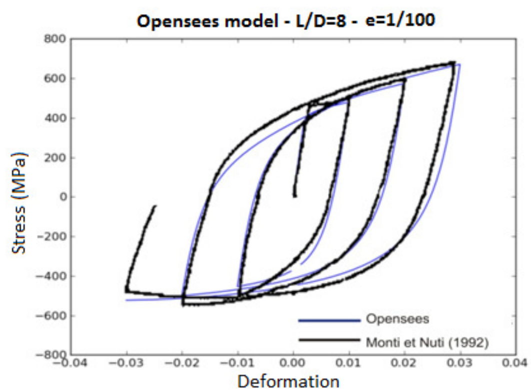

Figure 3.6. Buckling modeling (L/D=8)

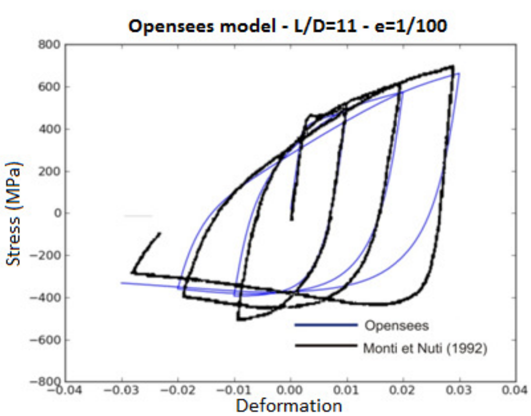

Figure 3.7. Buckling modeling (L/D=11)

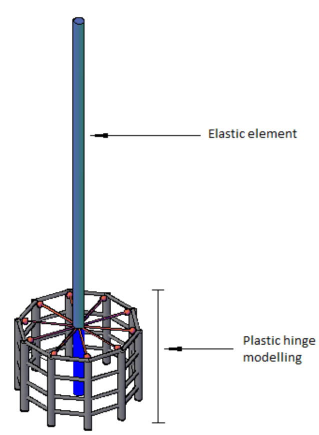

Figure 3.8. Bridge pier model considering bar buckling

3.6 Modeling of reinforcement slip

Reinforcement slip in the plastic hinge region creates additional rotation in the plastic hinge region which will lead to additional tip displacement. It also has a great effect on strength of the column. Lap splice slip and bond slip (strain penetration at the pier foundation interface) are two phenomena that produce reinforcement slip. For the lap splice slip phenomena, the models proposed by Harajli 2006 and Xiao et al. 1998 are compared (Fig. 3.9) and it is found that both models predict well experimental results. In Opensees, lap splice split phenomena is implemented with Pinching4 material. Because it is more easily implemented, model proposed by Harajli 2006 is used. The bond slip behaviour is included in the model with Bondslip material in Opensees (Fig. 3.10). Both behaviours can be added in the model by introducing spring elements at the base of the pier.

3.7 Localization of deformations

Localization of deformation is a modeling problem that occurs when the sectional response of an element is elastic-perfectly plastic or when there is strain softening. In those cases, the local deformation will be localized at the base element as the force is limited to the elastic limit and cannot spread to the next elements (see Légeron et al. 2005). When the loading is in displacement control, this means that the deformation in the base element will be greater for smaller elements and vice versa. For sectional response with soft hardening, the force-displacement response is also affected by the phenomena (Fig. 3.11). In order to take into account the phenomena, the constant fracture energy concept can be used as proposed by Bazant and Oh 1983 and modified for compression by Coleman and Spacone 2001. This method assumes that the fracture energy is a material constant. Hence the post peak stress-strain law of concrete is modified to account for the size of the element in order to provide the same fracture energy. When the stress strain behaviour is modified to take into account the element length, the force-displacement response is adjusted as seen in Fig. 3.12 and the prediction become independent on the size of elements.

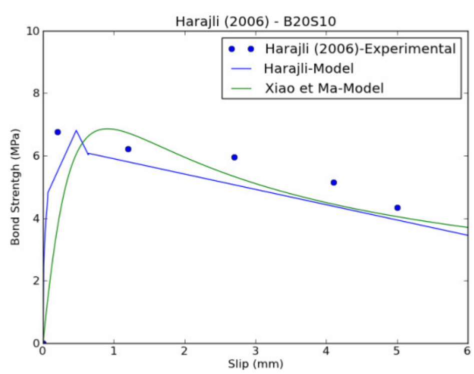

Figure 3.9. Lap splice slip model comparison

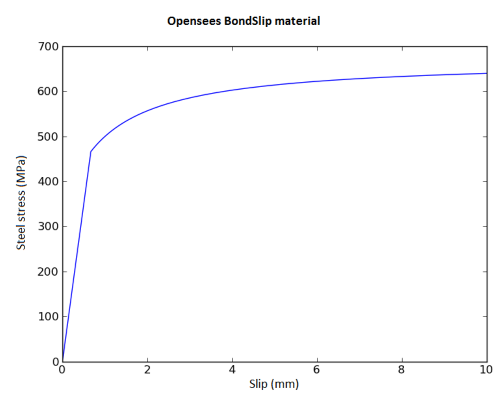

Figure 3.10. Bond slip model (Opensees)

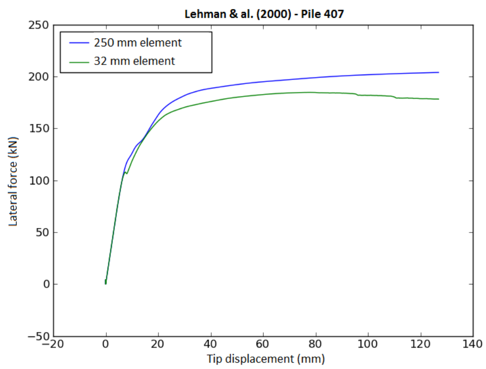

Figure 3.11. Localization effect on global response

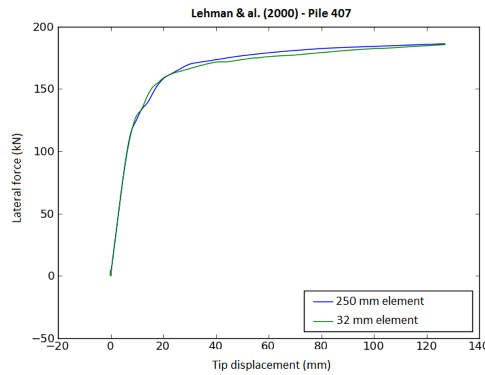

Figure 3.12. Global response after modifications

4. MODELING LARGE SCALE COLUMNS

The modeling technique described in the previous section is used to predict the results of Lehman and Moehle 2000. The results are compared with the experimental results and are reported on Fig 4.1 to 4.3. The global behaviour of the piers is well predicted. The model predicts well the initiation of cracking, yielding and spalling of concrete cover, and initiation of reinforcement buckling for pier 407. Buckling of bars in Pier 415 and 430 are predicted slightly before it actually happens. This might be due to the initial eccentricities that may affect whether they are toward the inside of the column or the outward. In our model, it is assumed in the outward direction which is the most conservative, but it may not be exactly the same in the real column.
Local predictions are compared to the local measures at Table 4.1. It can be seen that overall, the model predict very well the experimental behaviour of the columns tested by Lehman and Moehle 2000.

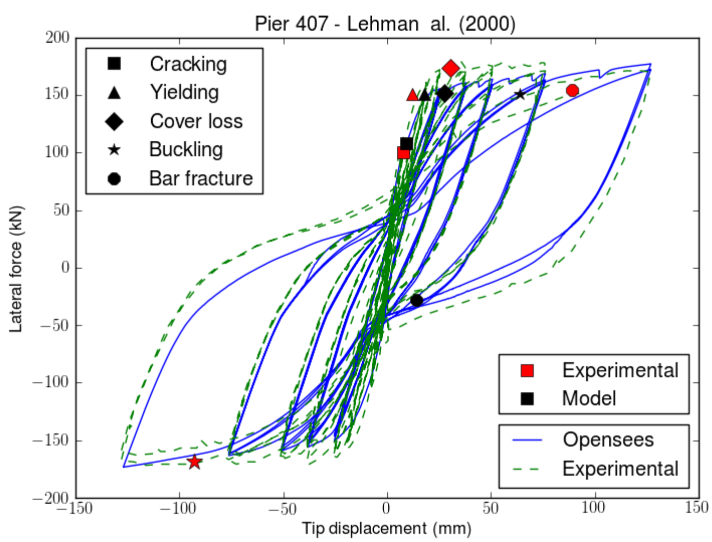

Figure 4.1. Modeling of Pier 407 (Lehman and Moehle 2000)

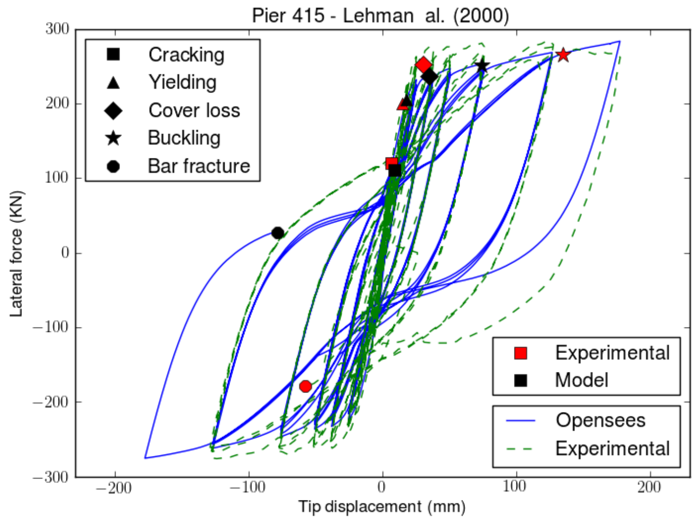

Figure 4.2. Modeling of Pier 415 (Lehman and Moehle 2000)

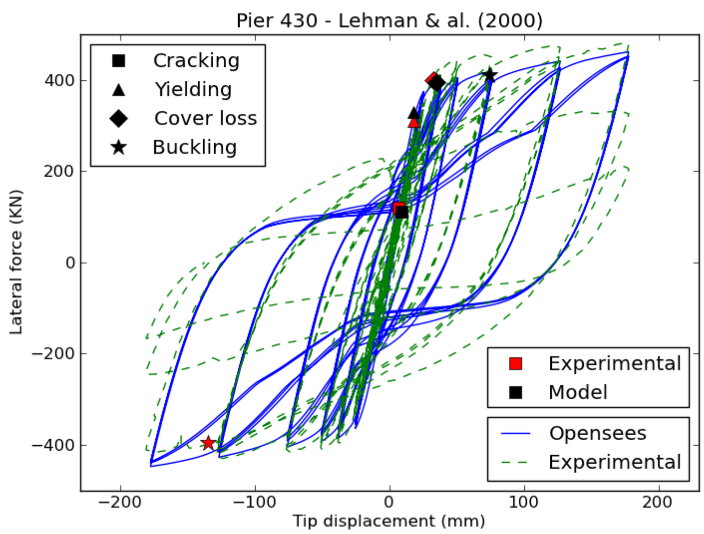

Figure 4.3. Modeling of Pier 430 (Lehman and Moehle 2000)

Table 4.1. Local measurements

		51mm		76mm		127mm		178mm	
Pier		Exp.	Model	Exp.	Model	Exp.	Model	Exp.	Model
407	Crack width (mm)	3.3	2.5	4.8	5.5	6.4	10.9	-	-
	Tensile strain	0.027	0.04	-	0.06	-	0.07	-	-
	Compression Strain	-0.01	-0.008	-0016	-0.011	-0.31	-0.018	-	-
415	Crack width (mm)	3.2	2.95	3.2	4.4	4.6	8.04	-	12.1
	Tensile strain	0.01	0.022	0.028	0.033	-	0.060	-	0.091
	Compression Strain	-0.01	-0.005	-0.018	-0.009	-0.033	-0.018	-0.053	-0.025
430	Crack width (mm)	3.2	2.13	4.83	4.4	-	9.01	-	12.9
	Tensile strain	0.016	0.016	0.024	0.033	-	0.065	-	0.093

5. CONCLUSION

This paper presents a modeling technique that takes into account most of the important phenomena in the behaviour of a concrete column under severe loading comparable to earthquake loading. Each of the models is compared to experimental results available in the literature, and it is concluded that they are well predicted. The models are included in the open source software Opensees and large scale columns tested in the past were used to evaluate the performance of the model. It is observed that the modeling technique is adequate for the evaluation of the performance of bridge piers.
The modeling technique should be compared to more experimental result in order to define better its limit and provide a complete modeling technique for numerical studies that could be used to develop performance based analysis on bridges.

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