CYCLIC LOADING PROTOCOL FOR BRIDGE COLUMNS SUBJECTED TO SUBDUCTION MEGA EARTHQUAKES

Ramiro Bazaez[1] and Peter Dusicka[2]

Abstract

Current structural design philosophies rely on the inelastic capacity of structures for resisting seismic excitations. In order to assess such capacity, cyclic loading protocols have been used as a common practice. However, analytical and experimental results have shown that the rotation capacity of columns is highly influenced by the loading. For that reason, quasi-static loading protocols that reflect the increase in inelastic demands on reinforced concrete bridge columns subjected to subduction mega earthquakes are developed and their influence on bridge columns is examined.

Introduction

All structural components have limited capacity. For that reason, understanding their behavior under strong ground motion excitations has always been a major objective of earthquake engineering. One method to assess the performance of structural components is via experimental evaluations utilizing quasi-static cyclic loading. The relatively slow application of the load in quasi-static tests allows experimentalists to relate structural metrics such as top displacement, chord rotation, drift, strains, etc. to visual damage of specimens (e.g. first cracking, spalling of the concrete, buckling of longitudinal reinforcement). Current earthquake design procedures for structural components have been established based on experimental results utilizing quasi-static cyclic tests. Moreover, design codes are trending to a relatively new design methodology called "Performance-based seismic design" (PBSD). In this methodology, a number of performance levels, which are frequently defined in terms of acceptable levels of damage, need to be satisfied under different levels of seismic hazards.

Under this design methodology the assessment of different structural components plays a fundamental role. Numerous experimental and analytical studies have been conducted in order to assess structural components, define limit states and acceptance criteria to be used in performance-based seismic design (Hose & Seible, 1999) (FEMA 356, 2000) (ASCE/SEI 41-06, 2007). However, recent occurrence of highly devastating subduction mega earthquakes of long duration (2010, Chile and 2011, Japan) have increased researchers' interest in how earthquake duration and number of cycles affect structural response and collapse assessment. Studies have

[1] Graduate Student Researcher, Department of Civil & Environmental Engineering, Portland State University, USA, rbazaez@pdx.edu.
[2] Associate Professor, Department of Civil & Environmental Engineering, Portland State University, USA, dusicka@pdx.edu.

indicated that ground motion duration and number of cycles have a major role on ductility demands and structural collapse when compared to ground motions of similar peak ground acceleration but less duration, e.g. Dusicka & Knoles (2012), Raghunandan & Liel (2013), Chandramohan et al. (2013). This effect is mostly attributed to the rate of structural strength and stiffness deterioration due to an increase in load reversals imposed for large magnitude and long duration ground motions. Others have revealed that the response of a structure depends significantly not only on the amplitude of the ground motion, but also on its duration (van de Lindt & Goh, 2004) (Chandramohan, et al., 2013). Earthquake ground motion duration has shown to have significant effects on the level of damage sustained by structures during strong earthquakes. This aspect is particularly relevant in subduction zones due to the fact that larger magnitude earthquakes are associated with strong motions of long duration. The main objective of the research summarized in this paper was to develop appropriate loading protocols in order to assess the capacity of reinforced concrete bridge columns subjected to subduction zone earthquakes. Furthermore, the influence of the proposed protocol on a bridge column capacity is briefly examined.

Limited experimental data can be found on columns subjected to long duration protocols that try to simulate subduction zone earthquakes since most of the seismic assessment of bridge columns have been carried out using a standard cyclic loading protocol, as that shown in Figure 1 (Cheung, et al., 1991), (Priestley, et al., 2002), which does not necessarily represent the demands imposed by subduction zone mega earthquakes. Experimental studies have shown that the displacement capacity of structural components is influenced by the loading history applied. A relevant research was carried out by Takemura and Kawashima (1997) to study the influence that different loading histories have on the ductility capacity of reinforced concrete bridge piers. In Takemura's research six nominally identical specimens were tested under different loading protocols resulting in six different responses. Another relevant research was carried out by Kunnath, et al. (1997) to investigate the cumulative seismic damage on circular reinforced concrete bridge columns, which were mostly controlled by flexural behavior. Using the concept of low-cycle fatigue and the cumulative damage model employed in the research carried out by Kunnath, experimental tests were performed at the Washington State University in order to investigate the performance of pre-1975 concrete bridges subjected to subduction earthquakes (McDaniel, et al., 2006). In this research, eight circular lightly confined reinforced concrete columns were tested using different displacement history. The results, as well as those obtained by Kunnath (1997), showed that the failure mode of the columns depends on the displacement history applied to them. A similar study was recently performed at MCEER, University at Buffalo in conjunction with the National Taiwan University of Science and Technology (Ou, et al., 2013). In this case, reinforced concrete bridge columns were tested applying two different loading protocols to investigate the influence of the number of cycles on bridge columns. Test results showed that columns under a long duration protocol behave significantly different in terms of strength and stiffness degradation than those columns under conventional (standard) protocols, showing that in high levels of damage the strength and stiffness degradation of the specimen subjected to long duration earthquakes would increase markedly.

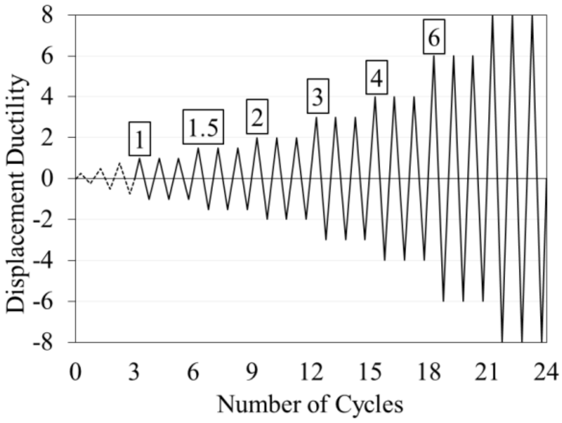

FIGURE 1 STANDARD PROTOCOL.

Cyclic Protocol Development

With the aim of developing representative loading protocols for components of the lateral resisting system of bridges under subduction zone earthquakes, a selection of earthquakes has to be done in order to determine the inelastic demands imposed by subduction earthquakes. The subduction zone earthquake sets used in this study were chosen from the 1985 Valparaiso (COSMOS), 2007 Sumatra (COSMOS), 2010 Maule (U. Chile), and 2011 Tohoku (K-Net) earthquakes with distances to the epicenter greater than 100 km to avoid near-fault pulse characteristics. It can be observed (Table 1) the vast amount of subduction ground motions used in the study, which pretends increase the applicability of the results. Vertical components were not considered due to the complexity to implement this variable in actual tests. A set of crustal earthquakes, on the other hand, was employed to allow demand comparisons. Crustal earthquakes, referred to herein as "Crustal" set, were chosen from the FEMA P695 far-field record (FEMA P695, 2009).

TABLE 1 GROUND MOTION SETS.

Set	M_w[3]	Site Class	PGA Range (g)	Number of Records	Average Bracketed Duration (sec)
Crustal	6.5-7.6	C/D	0.15-0.56	37	15
Valparaiso	7.8[4]	B/D	0.11-0.71	36	39
Sumatra	7.9	-	0.13	2	48
Maule	8.8	B/D	0.09-0.69	31	53
Tohoku1	9.0	B/C/D	0.50-2.01	27	153
Tohoku2	9.0	D/E	0.16-0.81	166	110

In order to predict the damage that a structure undergoes during severe earthquakes, it is important to represent in a realistic way the behavior of structural components during loading reversals. The peak oriented Ibarra-Krawinkler hysteretic model (Ibarra, et al., 2005) as is illustrated in Figure 2, which includes strength

[3] M_w: Moment magnitude
[4] M_s: Surface wave magnitude

capping, residual strength, and strength and stiffness deterioration due to load reversals, was employed. This model was calibrated using test results of bridge columns dominated by flexural behavior (PEER, 2003). This process allowed finding appropriate parameters to closely simulate load-deformation behavior of the components in study. Numerous nonlinear time-history analyses of single degree of freedom systems (SDOF), which were performed in a previous study (Dusicka & Knoles, 2012), were utilized to obtain bridge columns response under the selected subduction zone earthquakes. In that study, the constant ductility inelastic response approach (Ridell & Newmark, 1979) was utilized. Nonlinear analyses were performed to reach determined ductility ratios of 2, 4 and 8 with the aim of being representative of a wide range of structural ductilities in period ranges from 0.2 to 4.0 seconds.

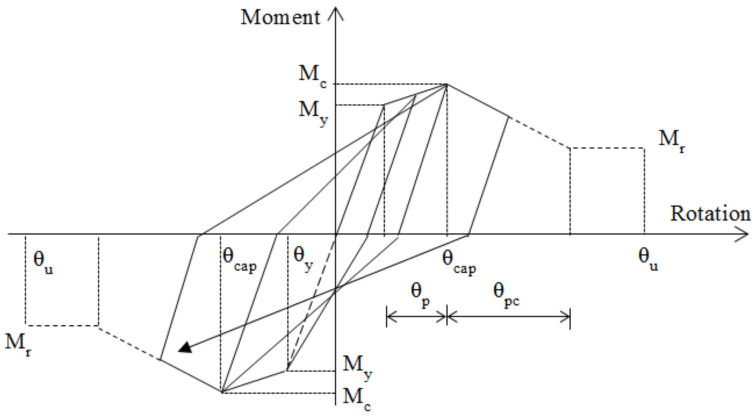

FIGURE 2 STRENGTH AND STIFFNESS DETERIORATION MODEL (OPENSEES, 2011)

Current testing protocol developments and experimental works have been done based on a general cumulative damage concept using the Coffin-Mason model and the Miner's rule of linear damage accumulation as a baseline (Krawinkler, et al., 1983). Another extensively damage index used in reinforced concrete structures it is that formulated by Park and Ang (1985). This damage index considers that damage is caused by structure's maximum deformation and cumulative dissipated energy. However, in order to calculate the damage indices, in a meaningful way, some parameters have to be experimentally obtained and validated, which can lead to undesirable uncertainties and arbitrariness. For that reason, in this study another damage index was employed based on cumulative damage called "Normalized Cumulative Plastic Displacement", which is a metric of structural plastic demand. This index is calculated by adding the ratio of plastic displacement range under an excursion ($\Delta\delta_{pi}$) to the yield displacement (δ_y) as is shown in Eq 1. In this damage index, the number of damaging cycles (N) and the sum of damaging cycle ranges ($\Sigma\Delta\delta_{pi}$) are important parameters in the development of testing protocols. A cycle is considered damaging when its amplitude is greater than the yield displacement.

$$\mathrm{NCPD} = \sum_{i=1}^{N} \frac{\Delta\delta_{pi}}{\delta_y} = \sum_{i=1}^{N} \frac{\delta_i - \delta_y}{\delta_y} \tag{1}$$

The response shown by a structural component contains excursions that are not symmetric and do not follow a consistent pattern under different ground motions. To rationalize the development of the testing protocol and compare the demands imposed by different sets of ground motion, the time history responses were converted into a series of cycles using the simplified rainflow counting (ASTM E1049-85, 2005). This procedure allows obtaining symmetrical cycles ordered in either decreasing or increasing amplitudes. The rainflow counting procedure was applied to non-linear time history response of structures with periods of 0.2 through 4.0 seconds in order to count the effective number of cycles and their amplitude. Statistical measures become necessary in order to achieve data reduction in a rational way. For that reason, the number of inelastic cycles and NCPD were represented employing the 84th percentile as target value. Statistical analyses of the rainflow counting results show a high dependence of the parameters in the type of earthquake and fundamental period of the bridge, as is illustrated in Figure 3. For that reason, 0.5, 1.0 and 2.0 seconds were selected as a benchmark to be representative of expected bridge fundamental periods. The argument to select different periods is that the use of only one period as a benchmark may lead to overestimate of the amount of inelastic cycles that the structure undergoes and distort the assessment of the behavior through physical testing.

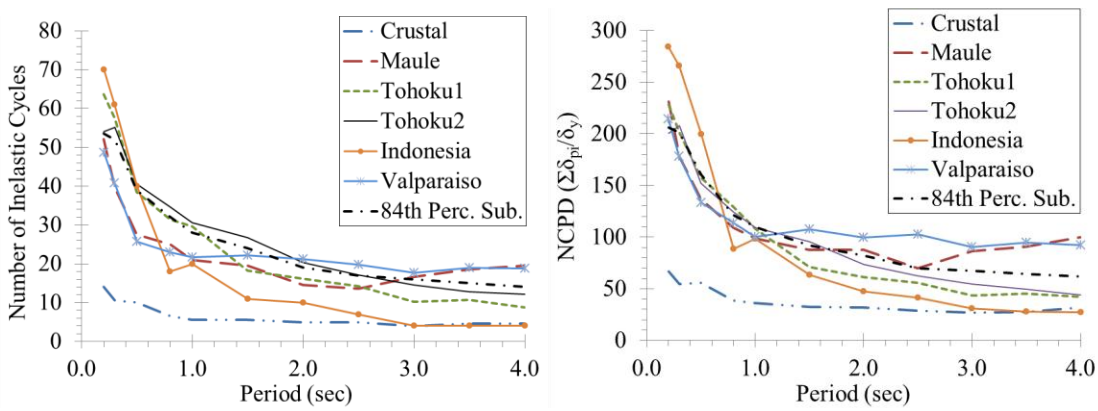

FIGURE 3 INFLUENCE OF PERIOD ON NUMBER OF INELASTIC CYCLES AND NCPD FOR STRUCTURES OF DUCTILITY 8.

For the benchmark periods, results have shown a nearly linear relation in the NCPD for different ductilities as is illustrated in Figure 4. This implies that for structures with other ductilities, the cumulative ductility may be found by linear interpolation of the values presented in Table 2. On the other hand, the number of inelastic cycles does not show a linear relation (Figure 4). Therefore, analyses with other ductilities are necessitated in order to determine a more accurate relationship. Thus, results led to differentiating the testing protocol in terms of ductility and period of the structure. For that reason, in order to closely reflect the subduction zone demands the loading protocols were developed using the target values of the parameters shown in Figure 4 and summarized in Table 2.

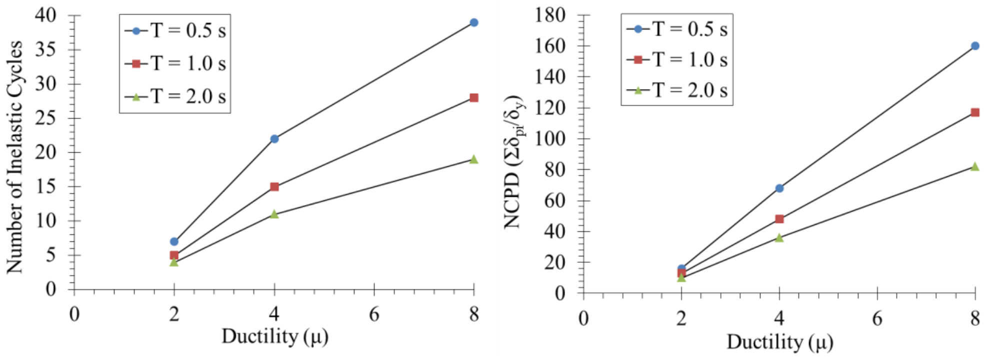

FIGURE 4 NUMBER OF INELASTIC CYCLES AND NCPD FOR DIFFERENT DUCTILITIES.

TABLE 2 TARGET VALUES AND PROPOSED PARAMETERS.

Period T	**Max μ**	**$N_{cycle} > \delta_y$**		**$\Sigma\Delta\delta_{pi} / \delta_y$**	
		Target Value	**Proposed**	**Target Value**	**Proposed**
0.5	2	7	7	16	18
	4	22	22	68	71
	8	39	40	160	177
1.0	2	5	7	13	18
	4	15	15	48	51
	8	28	28	117	119
2.0	2	4	5	10	14
	4	11	11	36	38
	8	19	19	82	88

Proposed Protocols

The proposed loading protocols consider two stages. The first stage consists of three cycles, in each of the following displacements (or loads), $0.25\delta_i$ (V_i), $0.5\delta_i$ (V_i), $0.75\delta_i$ (V_i) and one cycle at $1.0\delta_i$ (V_i) in order to visualize low damage states (e.g. first cracking). Where, δ_i is the theoretical yield displacement and V_i is the theoretical strength at first yield. The second stage of inelastic cycles aims to replicate the demands imposed on concrete bridge columns by subduction zone earthquakes of long duration. The loading histories are illustrated in Figure 5, in which the dotted lines represent the first stage and the solid lines the second stage. It is worth mentioning that the proposed protocols for structures of ductility two ($\mu = 2$) are not presented since they are unlikely to be applicable to typical bridge columns failing in flexure.

Since the proposed protocols are based on increments of ductility it is essential to determine the yield displacement of the specimen. A first estimate of the yield displacement can be found by performing a moment-curvature analysis of the bridge column section based on measured material properties. The moment-curvature analysis also allows the experimentalist to determine the target ductility of the specimen, although it is known that the specimen ductility might decrease during cyclic tests due to the stiffness and strength degradation that the component undergoes under load reversals. In order to determine the ideal yield displacement (δ_y) researchers have employed two approaches. The first approach consists of performing a monotonic test before cyclic loading tests. The second approach consists of a first stage based on load control. The load control is based on percentages of the theoretical component strength (V_i), usually $0.25V_i$, $0.5V_i$, $0.75V_i$, and V_i. The theoretical strength is determined dividing the first yield moment, which is obtained from a moment-curvature analysis following conventional flexural theory, by the column cantilever length. Then the experimental yield displacement (δ_y) is established by using the ratio of the theoretical force at which the concrete cover reaches a strain of 0.004 to the experimental elastic stiffness (K_e) which is calculated as the ratio of the theoretical first yield force (V_i) to the displacement measured experimentally (δ_y').

Sequence effects have not been fully established in the development of testing protocols (FEMA 356, 2000). In Figure 5 is shown the proposed protocols using the concept of pre-peak excursions cycles. This approach was used since cycles that occur after the maximum displacement will cause less cumulative damage and should be considered separately from pre-peak excursions (Krawinkler, et al., 2000). For that reason, in cases when the specimen does not reach the failure under the applied stepwise loading protocol, the test may continue under lower amplitude cycles (trailing cycles) instead of displacement ductility increments.

Illustrative Numerical Case Study

This study is part of a project which goal is to assess the behavior of pre-1970 bridge columns located in Oregon, USA. The State of Oregon lies near the Cascadia subduction zone, where a mega thrust earthquake of long duration forms a major component of the seismic risk. The case study contemplates the numerical study of a representative pre-1970 bridge column subjected to the standard protocol and the proposed subduction protocol. These columns usually are lightly reinforced and lap-spliced in places where plastic hinge formation is expected. Typical column properties and dimensions are summarized in Table 3 and the cross section is illustrated in Figure 6.

In order to model the inelastic behavior of the column the concentrated plasticity approach was utilized. The plastic hinge was modeled using the hysteretic model developed by Ibarra et al. (2005), as was illustrated in Figure 2, and implemented in the software OpenSees (2011). Model parameters for column hinges, such as moment capacity and rotation capacity, have been obtained from empirical equations based on a vast amount of column tests (Haselton, et al., 2008) (Biskinis & Fardis, 2009).

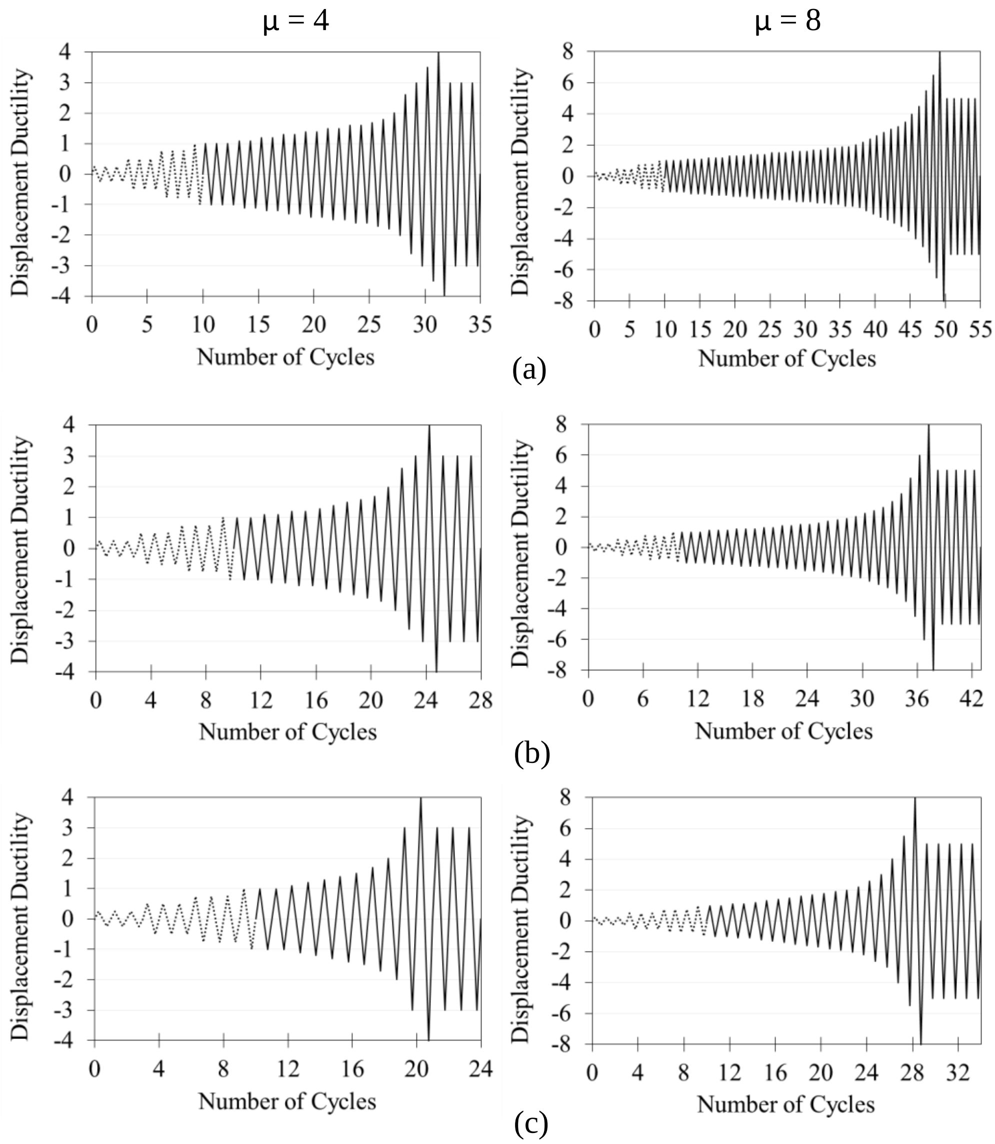

FIGURE 5 PROPOSED LOADING PROTOCOLS FOR DUCTILITIES (μ) = 4 AND 8. (a) T = 0.5 SEC, (b) T = 1.0 SEC, (c) T = 2.0 SEC.

TABLE 3 COLUMN PROPERTIES AND DIMENSIONS.

f'_c (MPa)	f'_{ce} (MPa)	f_y (MPa)	f_{ye} (MPa)	Length[5] (m)	Width (mm)	Depth (mm)	Axial Load (kN)	Axial Load Ratio (%)[6]	ρ_{sh} (%)	ρ_L (%)
22.8	29.6	413.7	468.8	2.82	609.6	609.6	712	6.5	0.094	0.88

[5] Cantilever Length
[6] Axial load ratio = $P/(A_g f'_{ce})$

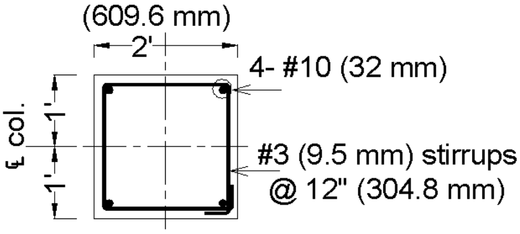

FIGURE 6 CROSS SECTION OF A TYPICAL PRE-1970 RECTANGULAR REINFORCED CONCRETE COLUMN IN OREGON, USA.

The hysteretic energy dissipation capacity plays a fundamental role in the assessment of bridge columns subjected to subduction zone ground motion. Haselton et al. (2008) has proposed equations to calculate this capacity (λ), which according his equation depends on the amount of transverse reinforcement, shear capacity and axial load ratio. Another equation also proposed by Haselton is included in the PEER/ATC 72-1 (2010) report, in which the value of λ only depends on the axial load ratio. The PEER/ATC report stated that for a typical column with seismic detailing, typical values of the parameter λ are on the order of 10 to 20. On the other hand, in the study carry out by Haselton (2008) values from 2 to 5 were employed for highly deteriorated components. This means that a lower λ indicates that the element has a high rate of strength and stiffness deterioration and therefore less capacity to dissipate energy. Since pre-1970 columns were built without seismic detailing the behavior of these columns is expected to be represented by λ values near 2.

The model parameters using equations proposed by Haselton (2008), Biskinis (2009), and moment-curvature analysis are summarized in Table 4. The moment – curvature analysis was based on conventional reinforced concrete flexure theory following AASHTO Specifications (2009). It is worth mentioning that all the analyses utilized the expected material properties, where $f'_{ce} = 1.3f'_c$ and $f_{ye} \approx 1.1f_y$.

TABLE 4 MODEL PARAMETERS.

Reference	My (kN-m)	M_c/M_y	EI_{eff}/EI_c	M_r/M_y	θ_y (rad)	θ_p (rad)	θ_{pc} (rad)	θ_u (rad)	λ
Theory (AASHTO, 2009)	544	1.07	0.29	0.8	0.006	0.043	-	0.049	-
Haselton (2008)	544	1.13	0.20	-	0.009	0.019	0.033	0.062	42
Biskinis (2009)	542	-	0.19	-	0.010	0.022	-	0.032	-
PEER/ATC 72-1 (2010)	544	1.13	0.20	0.0	0.009	0.019	0.033	0.062	24
This study	544	1.13	0.20	0.2	0.009	0.019	0.033	0.062	42 24 2

Some of the shortcomings of the equations proposed by Haselton (2008) and Biskinis & Fardis (2009) is that they do not include the effect of number of cycles on the column rotation capacity. Moreover, Haselton's equations do not account for the effect of lap-spliced rebars in expected plastic hinge locations. Despite this fact, Haselton's and Biskinis's equation lead to similar plastic rotation capacity (θ_p).

Figure 7 shows the results using the model parameters summarized in Table 4. These plots show the effect of the standard protocol and the subduction protocol for structures of ductility 8. Protocols with that target ductility were used because the ductility obtained from moment-curvature analysis was equal to 7. Comparing the results from the two protocols it can be observed that for structures with high values of λ, i.e. low rate of strength and stiffness deterioration, the behavior of the column under both protocols is quite similar in terms of rotation capacity, which is considered as the rotation when a reduction in moment capacity of 20% occurs.

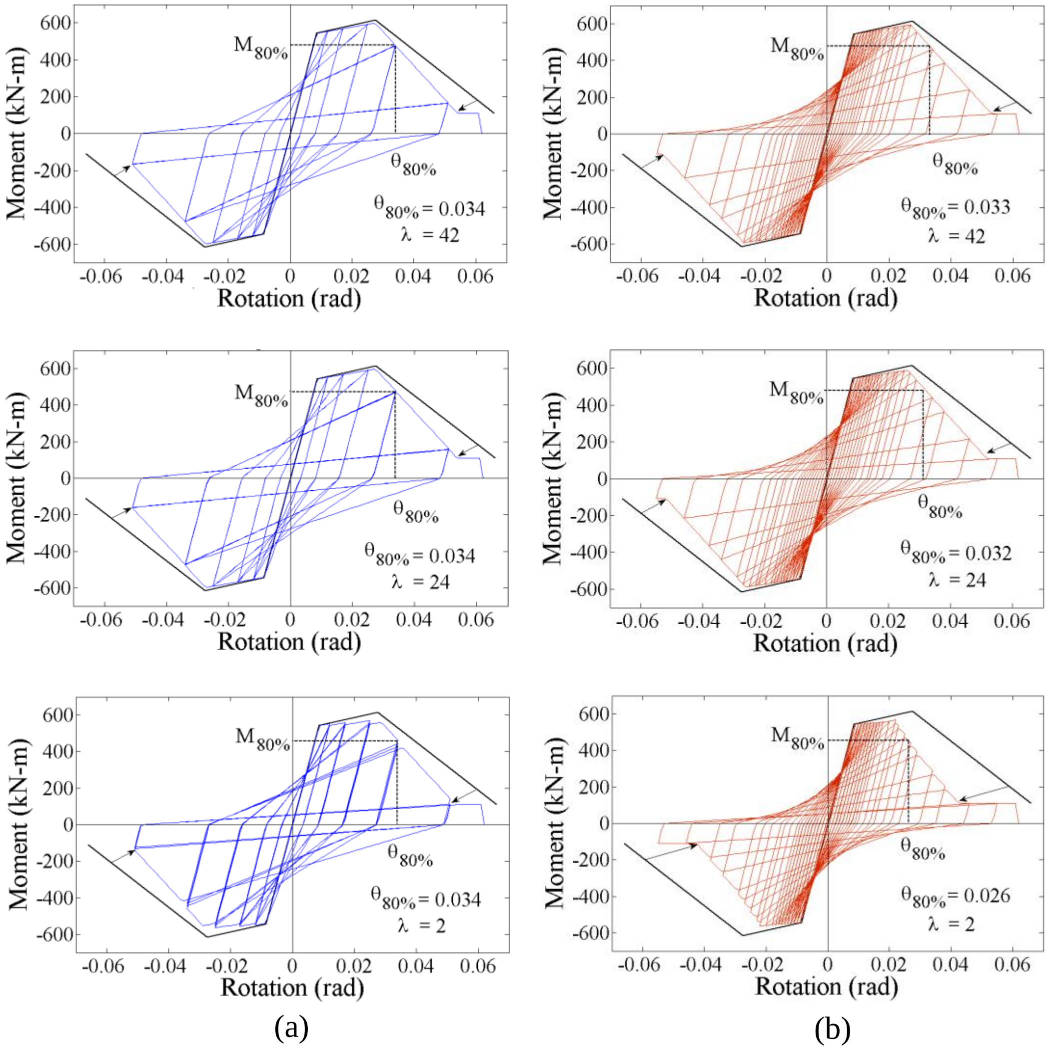

FIGURE 7 EFFECT OF LOADING PROTOCOL AND MODEL PARAMETERS ON COLUMN RESPONSE. (a) STANDARD PROTOCOL. (b) SUBDUCTION PROTOCOL

On the other hand, if a high rate of deterioration (low λ) is considered the column under the subduction protocol shows less rotation capacity as compared to the column under the standard protocol. This implies that the faster the rate of deterioration, the more significant the expected effect of number of inelastic cycles.

A high rate of deterioration is expected on pre-1970 columns due to the fact that they were built with lap splices in plastic hinge regions and insufficient transverse reinforcement. Therefore, the behavior of these columns would be highly influenced by subduction mega earthquakes. This result is consistent with experimental and numerical studies, e.g. Ibarra & Krawinkler (2005), Borg, et al. (2012), Ou, et al. (2013), Chandramohan, et al. (2013). In those studies were concluded that structural components' capacity and collapse are influenced by the duration of ground motion and the number of inelastic cycles. Thus, the proposed cyclic deformation histories capture more closely the inelastic demands and therefore their application would improve the seismic assessment of bridge columns through testing.

Summary and Conclusions

The simplified rainflow procedure was employed to convert the inelastic response obtained from non-linear time history analyses utilizing recorded strong motion data into symmetric cycles. This procedure also allowed computing required parameters such as number of inelastic cycles and the normalized cumulative plastic displacement metric. Statistical values of those parameters were used in order to develop quasi-static loading protocols. Different loading protocols were proposed for three different column ductilities (2, 4 and 8) and for three different periods of the component (0.5, 1.0 and 2.0 sec). The proposed loading protocols show an increasing number of low amplitude inelastic cycles as compared to the standard protocol, revealing that the standard loading protocol commonly used in experimental testing tends to replicate unrealistic drift demands because numerous large inelastic reversals are imposed in the component.

A representative pre-1970 lightly reinforced and lap-spliced bridge column was studied to observe the effect of the proposed protocol on the behavior of reinforced concrete bridge columns. Despite the fact that the standard protocol contains a higher number of large inelastic excursion, results showed that the use of the subduction protocol can highly influence the response of deteriorating components. Even though, more extensive analytical and experimental studies are needed to reach broader conclusions, the assessment of bridge columns through representative testing load protocols would play a key role in the future establishment of limit states and acceptance criteria to be applied in performance-based seismic design of bridge columns.

Acknowledgments

This paper is based upon research funded by the Oregon Department of Transportation, whose support is gratefully acknowledged. Any opinions, findings, and conclusions or recommendations expressed in this material are those of the authors and do not necessarily reflect the views of the sponsors.

References

AASHTO, 2009. *Guide Specifications for LRFD Seismic Bridge Design*, s.l.: American Association of State Highway and Transportation Officials.

ASCE/SEI 41-06, A. S. o. C. E., 2007. *ASCE/SEI 41-06: Seismic Rehabilitation of Existing Buildings,* Reston, Virginia: American Society of Civil Engineers.

ASTM E1049-85, 2005. *Standard Practices for Cycle Counting in Fatigue Analysis.* West Conshohocken, PA: s.n.

Biskinis, D. & Fardis, M., 2009. *Deformations of Concrete Members at Yielding and Ultimate under Monotonic or Cyclic Loading (Including Repaired and Retrofitted Members),* Patras, Greece: Report No. SEE 2009-0, Department of Civil Engineering, University of Patras.

Borg, R., Rossetto, T. & Varum, H., 2012. *The Effect of the Number of Response Cycles on the Behaviour of Reinforced Concrete Elements Subject to Cyclic Loading.* Lisboa, 15th World Conference on Earthquake Engineering.

Chandramohan, R., Lin, T., Baker, J. & Deierlein, G., 2013. *Influence of ground motion spectral shape and duration on seismic collapse risk.* Tokyo, Japan, 10CUEE CONFERENCE PROCEEDINGS, 10th International Conference on Urban Earthquake Engineering.

Cheung, P., Pauley, T. & Park, R., 1991. *New Zealand tests on full-scale reinforced concrete beam-column-slab sub-assemblages designed for earthquake resistance,* Detroit, MI: ACI Special Publication SP 123-1, American Concrete Institute.

COSMOS, 2012. *Consortium of Organizations for Strong Motion Observation Systems.* [Online] Available at: http://www.cosmos-eq.org/

Dusicka, P. & Knoles, S., 2012. *Subduction Megathrust Earthquake Demands on Ductile Bridge Columns.* Oakland, CA, (7NSC), Seventh National Seismic Conference on Bridges and Highways.

FEMA 356, 2000. *Prestandard and commentary for the seismic rehabilitation of buildings,* Washington, DC: Report No. FEMA 356. Prepared by American Society of Civil Engineers. Prepared for Federal Emergency Management Agency.

FEMA P695, 2009. *Quantification of Building Seismic Performance Factors,* Washington, D.C.: Federal Emergency Managment Agency.

Haselton, C., Liel, A., Taylor, S. & Deierlein, G., 2008. *Beam-Column Element Model Calibrated for Predicting Flexural Response Leading to Global Collapse of RC Frame Buildings,* Berkeley, California: PEER Report 2007/03, Pacific Earthquake Engineering Research Center.

Hose, Y. & Seible, F., 1999. *Performance Evaluation Database for Concrete Bridge Components and Systems under Simulated Seismic Loads,* Berkeley, California: PEER Report 1999/11, Pacific Earthquake Engineering Research Center.

Ibarra, L. & Krawinkler, H., 2005. *Global Collapse of Frame Structures under Seismic Excitations (PEER 2005/06),* Berkeley, CA: Pacific Earthquake Engineering Research Center.

Ibarra, L., Medina, R. & Krawinkler, H., 2005. Hysteretic models that incorporate strenght and stiffness deterioration. *Earthquake Engineering and Structural Dynamics,* Volume 34, pp. 1489-1511.

K-Net, n.d. *National Research Institute for Earth Science and Disaster Prevention. Kyoshin Network Database.* [Online]
Available at: http://www.k-net.bosai.go.jp/

Krawinkler, H. et al., 2000. *Development of a Testing Protocol for Woodframe Structures,* Richmond, CA: CUREE Publication No. W-02.

Krawinkler, H. et al., 1983. *Recommendations for Experimental Studies on the Seismic Behavior of Steel Components and Materials,* s.l.: John A. Blumel Center, Department of Civil Engineering, Stanford University.

Kunnath, S., El-Bahy, A., Taylor, A. & Stone, W., 1997. *Cumulative Seismic Damage of Reinforced Concrete Damage Piers,* Gaithersburg, MD: Building and Fire Research Laboratory, National Institute of Standards and Technology.

Lignos, D. & Krawinkler, H., 2012. *Sidesway collapse of deteriorating structural systems under seismic excitations,* Stanford, CA: Report No. 177, The John A. Blume Earthquake Engineering Center.

McDaniel, C., Cofer, W., D., M. & Rodriguez-Merek, A., 2006. *Performance of pre-1975 Concrete Bridges in Cascadia Subduction-Zone Earthquakes,* Pullman, WA: s.n.

OpenSees, 2011. *Open System for Earthquake Engineering Simulation, Pacific Earthquake Engineering Research Center, University of California, Berkeley.* [Online]
Available at: http://opensees.berkeley.edu

Ou, Y. et al., 2013. Ground Motion Duration Effects on Hysteretic Behavior of Reinforced Concrete Bridge Columns. *Journal of Structural Engineering,* p. Accepted Manuscript.

Park, Y. & Ang, H., 1985. "A Mechanistic Seismic Damage Model for Reinforced Concrete". *Journal of Structural Engineering, ASCE,* 111(4), pp. 722-739.

PEER/ATC-72-1, 2010. *Modeling and acceptance criteria for seismic design and analysis of tall buildings,* Redwood City, California: Prepared by Applied Technology Council. Prepared for Pacific Earthquake Engineering Research Center (PEER).

PEER, 2003. *Structural Performance Database (SPD), Pacific Earthquake Engineering Research Center, University of California, Berkeley.* [Online]
Available at: http://nisee.berkeley.edu/spd/

Priestley, M., Seible, F. & Hines, E., 2002. *Seismic performance of hollow rectangular reinforced concrete piers with highly-confined boundary elements,* La Jolla, California: Department of Structural Engineering, University of California, San Diego. Report No. SSRP-99/15..

Pujol, S., Sozen, M. & Ramirez, J., 2006. Displacement History Effects on Drift Capacity of Reinforced Concrete Columns. *ACI Structural Journal,* 103(2), pp. 253-262.

Raghunandan, M. & Liel, B., 2013. Effect of ground motion duration on earthquake-induced structural collapse. *Structural Safety,* Volume 41, pp. 119-133.

Ridell, R. & Newmark, N., 1979. *Statistical analysis of the response of nonlinear systems subjected to earthquakes,* Urbana: Struct. Res. Series No. 468, University of Illinois.

Takemura, H. & Kawashima, K., 1997. Effect of loading hysteresis on ductility capacity of reinforced concrete bridge piers. *Journal of Structural Engineering,* Volume 43A, pp. 849-858 (in Japanese).

U. Chile, 2010. *,University of Chile. Earthquakes of Chile.* [Online]
Available at: http://terremotos.ing.uchile.cl/registros/164

van de Lindt, J. & Goh, G., 2004. Effect of earthquake duration on structural reliability. *Engineering Structures,* Volume 26, p. 1585–1597.