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ASSESSMENT OF LOW-CYCLE FATIGUE DAMAGE IN R.C. FRAME BUILDINGS UNDER LONG-DURATION EARTHQUAKES

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Abstract:
Long-duration ground motions were recorded recently during the 2010 Muale earthquake in Chile and the 2011 Tohoku earthquake in Japan. Both earthquakes caused significant structural damage that was reported by different research teams who inspected numerous buildings after each earthquake. One of the most observed failure modes as reported by researchers and professional engineers was the rupture of reinforcing bars in reinforced concrete (R.C.) members due to earthquake-induced low-cycle fatigue. The long duration of both earthquakes induced a large number of loading cycles on the affected buildings, which led to significant damage especially at locations where high strains were expected to occur.

This paper presents preliminary results of a study to estimate the low-cycle fatigue damage in R.C. frame buildings when they are subjected to long-duration earthquakes. A 20-story and a 6-story case study buildings representing structural configurations with different dynamic characteristics were selected for investigation through nonlinear time history analysis. Both buildings are existing instrumented buildings that experienced several earthquakes during their service life and have recorded responses that were used to verify the analysis models. The models for both buildings were analysed under long-duration ground motions to assess the low-cycle fatigue damage within the R.C. members.

Based on the results from the nonlinear models, detailed fatigue analysis was conducted using the strain time histories obtained at each critical location within the buildings. Due to the irregularity of the strain histories, the rain-flow counting method was used to convert the histories into equivalent number of cycles of certain strain amplitudes. The fatigue life relationships for reinforcing bars were found in the literature based on experiments conducted by several researchers on different grades and sizes of reinforcing steel. The Palmgren-Miner damage rule was used to estimate the reduction in the fatigue life due to the applied ground motions in both buildings using those fatigue life relationships.

The preliminary findings of this study showed significant reduction in the fatigue life of reinforcing bars due to long-duration earthquakes that could increase the damage potential in R.C. frame buildings. It was also found that the dynamic characteristics of the building affect the number of cycles, which controls the low-cycle fatigue damage.

Introduction

Earthquakes induce loading cycles on structures based on the dynamic properties of their structural systems and the ground motion parameters. Due to the cyclic nature of the earthquake loading, deterioration of both strength and stiffness is expected to happen for different structural members resulting in a cumulative reduction in the service life. The occurrence of long-duration earthquakes increases the total number of loading cycles which the building might experience during its lifetime. The accumulation of seismic damage within the elements of a building due to long-duration earthquakes can lead to increased vulnerability to failure. The fatigue effect on the structural response is one of the major contributors to the damage accumulation phenomenon which depends on the amplitude and the number of loading cycles. Various construction materials have different fatigue characteristics which affect the remaining life of the structure subjected to a loading history of earthquakes. Since the number of cycles induced by earthquake loading is relatively low, the fatigue behavior of materials in the low-cycle range (less than 1000 cycles) is controlling. Also, the deformations

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and straining actions due to earthquake loadings are expected to be in the inelastic range which leads to high stress and strain amplitude for the loading cycles.

The current seismic design practice relies on the inelastic flexural response at the plastic hinges as the primary source of energy dissipation. This requires that proper care be taken in detailing the locations where plastic hinges are expected to occur. For reinforced concrete (RC) structures, potential plastic hinge locations are designed to experience large rotations in order to dissipate energy which requires the longitudinal reinforcing steel bars to undergo high strain amplitudes during earthquake loading cycles. Based on numerous experimental studies, it has been observed that failure in RC members is influenced by the number of loading cycles during the history of events that affects the structure and can arise in the following modes (Dutta & Mander, 1998):

a. Rupture of longitudinal reinforcing steel bars due to low-cycle fatigue.
b. Fracture of transverse steel which leads to failure of the confined concrete core.
c. Compression buckling of the longitudinal reinforcing bars.

A better evaluation of existing RC buildings is needed in order to assess the different types of damage especially those that may not be readily apparent to visual inspection (such as the reduction in the fatigue life of reinforcing bars) after a certain event. By estimating the reduction in the fatigue life of steel reinforcement at each of the critical locations within an RC building, the building could be classified according to its vulnerability to low-cycle fatigue failure.

The goal of this paper is to estimate the damage potential due to the low-cycle fatigue behavior of reinforcing steel bars used in RC moment frame buildings during long-duration earthquakes.

Damage Observations from the 2010 Chile Earthquake

The most recent evidence on damage accumulation in RC buildings (especially those due to low-cycle fatigue effect) is the severe damage that happened in numerous buildings during the Maule earthquake in Chile. After the earthquake on February 27, 2010, certain types of damage were observed repeatedly in existing RC buildings that were inspected by an investigation team of the Los Angeles Tall Buildings Structural Design Council (LATBSDC). One of the most observed failure modes was the rupture of main bars at the extreme ends of RC walls at the location of failure, particularly at the bottom levels of the affected buildings as shown in figure 1. The investigation team reported that they believe the tearing of these bars is related to low-cycle fatigue caused by the numerous cycles of loading and unloading due to the long duration of the earthquake and the large inelastic deformations due to the magnitude of the earthquake as well (Naeim, et al., 2011), (Youssef, et al., 2011).

Figure 1. Low-cycle fatigue failure due to rupture of the main bars at the end of RC walls, Chile (Naeim, et al., 2011), (Youssef, et al., 2011).

Based on the previous observations, research efforts are needed to study the damage accumulation due to a relatively small number of loading cycles (low-cycle fatigue) induced by long-duration earthquakes which can lead to higher damage potential and a possibility of collapse. Hence, the evaluation of the current state of an existing building should estimate the effects of low-cycle fatigue.

Case Study Buildings

This investigation was performed using “Nonlinear Time History Analyses” of two existing RC moment frame buildings that were instrumented with accelerometers at different locations and have recorded time histories of earthquake response. The recorded base motions served as input to the numerical models and the calculated responses will be verified versus the recorded responses. The analyses will be extended by applying different long-duration earthquakes to the building models in order to examine the low-cycle fatigue damage potential of reinforcing bars within the RC members. These buildings represent the most common case for RC moment frame buildings which are medium- and high-rise buildings as shown in figure 2. The design and detailing of these buildings were checked and were found to comply with most of the code requirements for ductile RC frame systems such as strong column-weak beam concept, over-strength shear design, as well as proper confinement for beams and columns (ACI318R-11, 2011), (ACI352R-02, 2002). Also, the hoop spacing at the beam ends doesn’t exceed six time the diameter of the longitudinal reinforcing bars which prevents the buckling of longitudinal bars (Dutta & Mander, 1998).

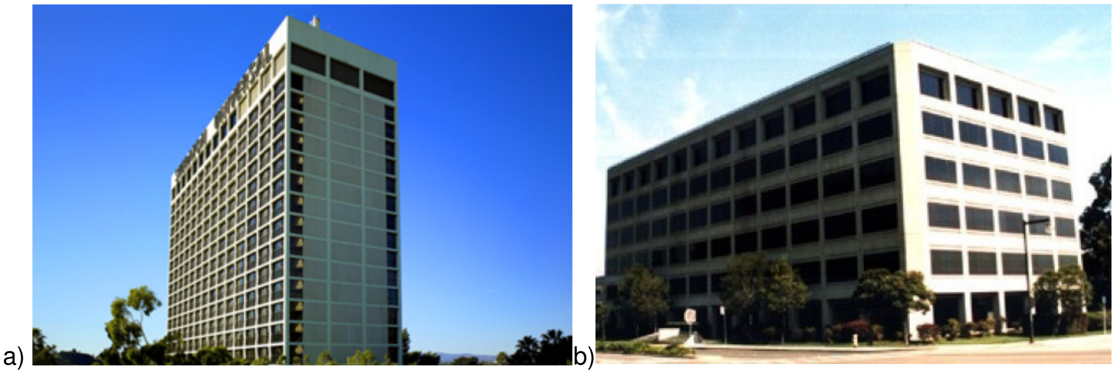


Figure 2. a) A Hotel Building in North Hollywood, California (20-story) (NOAA, 1973), b) An Office Building in San Bruno, California (6-story) (Anderson & Bertero, 1993)

a. **A Hotel Building in North Hollywood, California (20-story):**

An existing twenty-story building in North Hollywood, California has been selected as an example of a high-rise building that has moment resisting RC frames as a lateral force resisting system. The building was constructed in 1968 (NOAA, 1973) and was instrumented with nine sensors then the instrumentation was updated to sixteen sensors at five different floor levels in 1981. It has been subjected to three moderate to strong earthquakes during its lifetime and has recorded responses during 1971 San Fernando, 1987 Whittier, and 1994 Northridge earthquakes of magnitudes 6.6, 5.9 and 6.7 respectively.

b. **An Office Building in San Bruno, California (6-story):**

An existing six-story office building in San Bruno, California is an instrumented building that has been selected as an example of a medium-rise building. The building was built in 1978 with lateral resistance is provided by five moment resisting RC frames, four of which are located on the perimeter and one which is located on the interior in the transverse direction (Anderson & Bertero, 1993). The building was instrumented with 13 sensors at different levels as part of the California Strong Motion Instrumentation Program (CSMIP). The building has a recorded response from the Loma-Prieta earthquake in 1989 of magnitude 6.9.

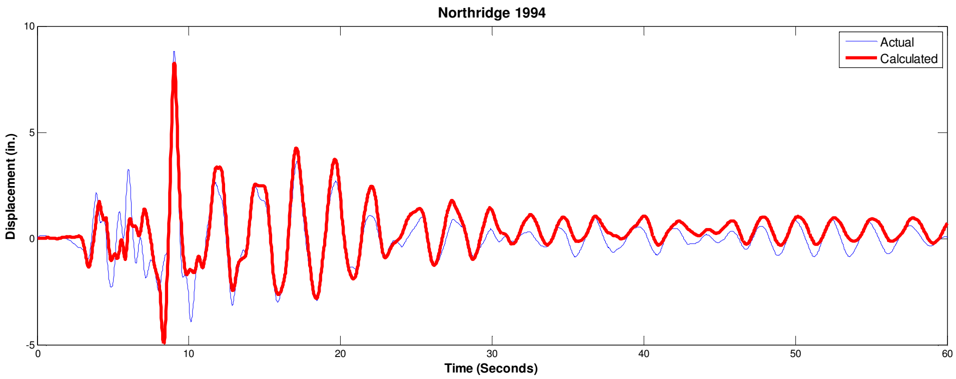
Modeling of the Selected Buildings

Nonlinear time history analyses were performed using a 2-D finite element model for a typical RC frame in a certain direction for each of the case study buildings using the finite element program PERFORM-3D from Computers & Structures Inc. (CSI, 2007). In order have a good representation of the building behavior under earthquake loading; attention should be paid toward several modeling parameters. The model includes a variety of elements with different types of behavior, such as elastic frame members, rotational plastic hinges and joint panel zones that requires identifying many parameters to control their behavior.

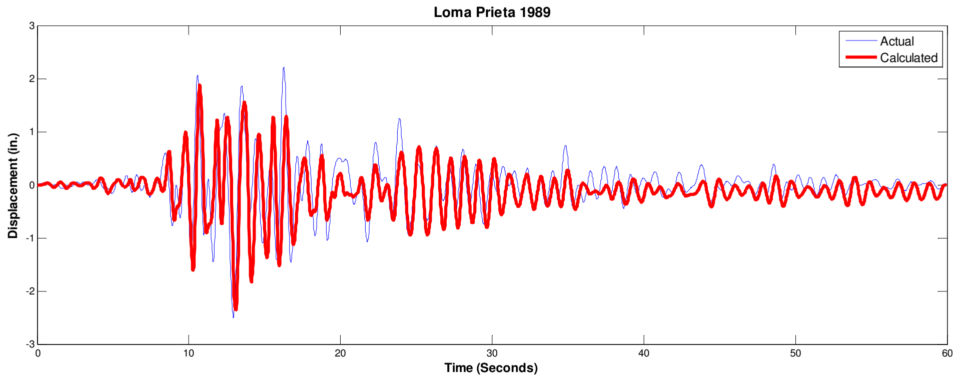
A typical lumped plasticity element was used to model both beams and columns in the RC frame buildings where plastic hinges are defined at the ends of the frame component. This element consists of, an elastic beam segment, rotational plastic hinges at the ends, and rigid end zones at both ends just before and after the plastic hinges inside the joint region. The frame elements have a uniform elastic cross-section for which cracked stiffness is considered based on the monotonic moment-curvature relationship for each RC section. The rigid end zone has the length of one half of the column width or beam depth (depending on the orientation of the frame element), and the stiffness of this zone is assumed to be 10 times larger than the stiffness of the elastic frame element. Rotational plastic hinges are the basic elements that represent the inelastic behavior of the frame component taking place throughout the hinges by defining the backbone curve.

Model Verification

The results of the models of the case study buildings were used to evaluate the accuracy of the nonlinear modeling parameters in predicting the responses under earthquake loading in the light of the recorded response during past earthquakes. This verification process is necessary for further extension of the analysis using the same models in order to achieve the goals of this study. The results of the analysis are presented in terms of building periods, maximum roof displacement, and displacement histories. The calculated periods as well as the maximum roof displacement values were compared to the values obtained from analyzing the recorded data in a certain direction as shown in table (1). Also, the calculated displacement histories were compared with the actual histories from the recorded response at the roof level as shown in figure 3.



a) North Hollywood Building



b) San Bruno Building

Figure 3. Comparison between actual and calculated displacement histories at the roof for the case study buildings.

Table 1. Comparison between actual and calculated values for the case study buildings.

Building	Direction	Earthquake	Actual Period (sec.)	Calculated Period (sec.)	Actual Disp. (in)	Calculated Disp. (in)
North Hollywood	Transverse	Northridge, 1994	2.560	2.589	8.790	8.270
San Bruno	Longitudinal	Loma Prieta, 1979	0.931	0.951	2.500	2.370

According to the previous comparisons, the three models for the selected case study buildings showed good simulated response in the light of the recorded responses for each building. Hence, the level of agreement between the recorded and the calculated values is considered satisfactory to represent the behavior of the RC frame buildings if they are subjected to long-duration earthquakes. Based on this verification, the developed models for the case study buildings in PERFORM-3D can be used for further study using additional records to investigate the effect of long-duration earthquakes on the RC frame buildings in order to estimate the low-cycle fatigue damage which is the primary goal of this study.

Long-Duration Ground Motions

Since the objective of this study is to investigate the effects of long-duration earthquakes on the low-cycle fatigue damage in RC frame buildings, it was important to select earthquake records that provide a realistic representation of ground motion magnitudes, frequency contents, and durations which creates a variety of real scenarios of long-duration earthquakes. The earthquake records were downloaded from the available large strong-motion databases via the internet such as "Pacific Earthquake Engineering Research Center (PEER)", "COSMOS Virtual Data Center", "Center of Engineering Strong Motion Data" (CESMD), and "GeoNet". The selection criteria for the records that define the long-duration ground motion from the available acceleration time histories follows the screening method adopted by Chandramohan et al. (Chandramohan et al., 2013). The selected ground motions should have a minimum value for the PGA of 0.10g and a minimum significant duration "D_s" of 40 seconds. The significant duration accounts for seismic energy and is considered as an explicit definition of duration. It was defined according to Trifunac and Brady (Trifunac & Brady, 1975) as the time interval over which a 5%-95% of the Arias Intensity is attained (Arias, 1970).

Fve long-duration earthquake records were selected based on the previous criteria to be applied to the numerical models of the case study buildings after they were verified using the recorded response for each building as shown in the previous section. The selected long-duration earthquake records are shown in table (2) that presents the ground motion characteristics of each record.

Table 2. Selected long-duration earthquake records.

Earthquake	Magnitude	Station	Direction	PGA (g)	Duration (sec.)	D_s (sec.)
Tohoku, 2011	9.0	FKS018	EW	-1.090	300	83.61
Muale, 2010	8.8	Constitución	NS	-0.537	140	59.80
Wenchuan, 2008	8.0	051SFB	EW	0.567	160	90.10
Tokachi, 2003	8.0	HDK1000	NS	0.825	300	40.11
Imperial Valley, 1979	6.4	6605	NS	0.351	100	50.33

Analysis Results

The results of the nonlinear time history analysis showed that all plastic hinges occurred at the ends of the floor beams of the case study buildings with several additional plastic hinges occurred within the bottom floor columns at the foundation level in San Bruno building.

The curvature histories were obtained from the analysis of the numerical models of the case study buildings for each of the selected earthquakes using PERFORM-3D. Then, the strain values were calculated from the curvature values using the results obtained from the section analysis of each RC frame component where strain vs. curvature relationships were developed especially for beams where high inelastic defamations were developed at plastic hinges. Two equations are developed for each beam section in both positive and negative curvature. An example of the strain-curvature relationships for the typical beam section in North Hollywood building is presented below:

For positive curvature: $\varepsilon = 86895\varphi^3 + 1776\varphi^2 + 14.756\varphi + 0.001$ (1, a)

For negative curvature: $\varepsilon = 5665761\varphi^3 - 9113\varphi^2 + 6\varphi$ (1, b)

where "φ" refers to the curvature of the beam section in flexure. Similar relationships were developed for the beam sections in San Bruno building, then were used to obtain the strain time histories from the curvature time history in each floor beam.

According to the analysis results, the case study buildings experienced significant inelastic deformation during the selected earthquakes. Sample strain histories for the Muale earthquake at plastic hinge locations in sample floor beams in the three building models are shown in figure 4. It is clear that beams experienced significant plastic strains at the plastic hinge locations. These plastic strains are considered as a non-zero mean strain cyclic loading affecting the low-cycle fatigue behavior of the main reinforcing bars at plastic hinges.

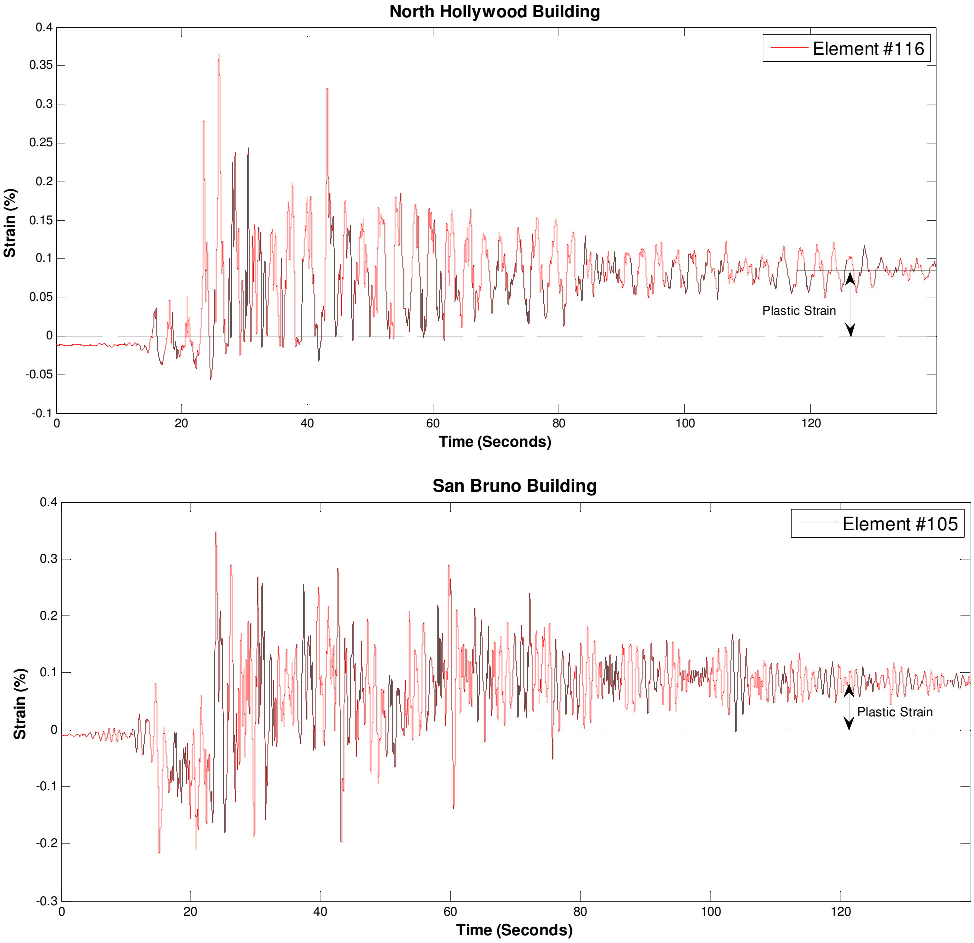


Figure 4. Sample strain histories for floor beams in the case study buildings during the Muale earthquake (Constitución).

Rain-flow Counting

The obtained strain histories at each plastic hinge location are highly irregular containing large number of random cycles of varying amplitudes so, the rain-flow counting method was used to extract the strain cycles of high amplitudes. The rain-flow method is very common in fatigue analysis which defines the corresponding number of cycles at each stress or strain amplitude. It was developed in 1968 by T. Endo and since then it has been widely in structural engineering applications (Matsuishi & Endo, 1968).

Fatigue Life Relationships

Typical strain amplitudes in RC members during moderate to severe seismic events can easily exceed the yield strain which is almost 0.2% for (Grade 60) reinforcing steel, and the number of cycles to failure at such large amplitudes is usually less than 1000 cycles which could result from long-duration earthquakes. Low-cycle fatigue is defined as failure in a material due to a relatively small number of load or deformation cycles (< 1000) and typically involves large deformations that exceed the elastic limit.

Based on the available literature, several fatigue life relationships were developed through cyclic testing of reinforcing bars under constant amplitude loading. The existing relationships predict the number of cycles to failure at a given strain amplitude. These relationships are in an exponential form though when it is plotted in a log-scale, it become linear. The constants of the fatigue formulae are obtained from test results performed by several researchers including various types of steel. The equation developed by Koh and Stephens (Koh & Stephens, 1991) has two major fatigue parameters (constants). The equation is presented in terms of the total strain amplitude “ε_a” and the number of cycles to failure “N_f”. The Koh Stephens equation is presented below where parameters “ε_f” and “m” are defining linear relationship in log-scale.

$$\varepsilon_a = \frac{\Delta\varepsilon}{2} = \varepsilon_f (2N_f)^m \tag{2}$$

The research done by Mander (Mander et al., 1994) was one of the first investigations of the low-cycle fatigue behavior of reinforcing bars where both reinforcing steel and pre-stressing steel were tested under constant strain amplitudes. Different fatigue life relationships were provided out of this research for both types of steel to define a single line which could be used in estimating the approximate number of cycles to failure under constant amplitude strain as follows:

Reinforcing steel:

$$\varepsilon_a = \frac{\Delta\varepsilon}{2} = 0.0795(2N_f)^{-0.448} \tag{3, a}$$

Pre-stressing steel:

$$\varepsilon_a = \frac{\Delta\varepsilon}{2} = 0.0791(2N_f)^{-0.381} \tag{3, b}$$

Based on the previous review of the research on low-cycle fatigue behavior of reinforcing steel bars, the fatigue life relationships (S-N curves) could be used to estimate the damage due to seismic loading cycles.

Palmgren-Miner Damage Rule

After determining the number of cycles for each strain level, the results were used to estimate the accumulated damage due to reduction in the fatigue life. This reduction is calculated using the Palmgren-Miner Rule to assess the condition of the reinforcing in the members. It was assumed that the damage due to fatigue type loading (stress or strain cycles) is accumulating linearly according to the number of loading cycles. It simply states that the fatigue life of a structure or an element is exhausted when the sum of all the fatigue fractions at different loading amplitudes from a random fatigue loading is equal to unity (Miner, 1945).

$$\frac{N_1}{N_{f1}} + \frac{N_2}{N_{f2}} + \frac{N_3}{N_{f3}} + \ldots\ldots = \sum \frac{N_i}{N_{fi}} = 1 \tag{4}$$

For the strain histories obtained from the analysis of the case study building, the number of strain cycles at each strain increment “N” was determined using the rain-flow counting method. The available fatigue life relationships were then used to calculate the number of cycles to failure at each strain level “N_f”. Hence, the Palmgren-Miner damage rule was used to calculate the cumulative fatigue damage in the longitudinal reinforcing bars. The increase in the cumulative fatigue damage is corresponding to the reduction in the fatigue life of the structural member subjected to fatigue type loading.

Expected Fatigue Damage

The available expressions for the fatigue life of reinforcing steel were used to calculate the number of cycles to failure at each strain level as determined through the rain-flow counting.

Since, low-cycle fatigue is associated with high strain amplitudes due to large inelastic deformations at plastic hinges during each earthquake scenario, only strain amplitudes beyond the 50% of the yield strain of each type of steel were considered. This threshold was also adopted to be consistent with the fatigue life relationships from the literature that were developed using high strain amplitude testing. It was clear from the strain histories that large plastic deformations occur during early seconds of each earthquake which influences the fatigue damage during the later loading cycles. In order to account for the effect of the non-zero mean strain loading due to permanent plastic strains, a modification suggested by Collins (Collins, 1993) was applied to the previous fatigue life relationships as follows:

$$\varepsilon_a = \frac{\Delta\varepsilon}{2} = \frac{(1-R)\varepsilon'_f}{\left[(4N_f - 1)(1-R)^a + (2)^a\right]^{1/a}} \tag{5}$$

where “R” is the strain ration, “ε'_f” and “a” are determined based on the used fatigue life relationship. The fatigue damage due to a non-zero mean strain loading is usually more than the cyclic loading with zero mean strain which leads shorter fatigue life. After calculating the number of cycles at each strain level using the rain-flow method and the number of cycles to failure using the previous equation, the Palmgren-Miner rule is used to calculate the cumulative fatigue damage within the main reinforcement at critical locations.

The main purpose of selecting these case study buildings of different heights (medium- and high-rise) is to study the effect of the change in the fundamental period of vibration of RC moment frame buildings on the low-cycle fatigue damage. The period of vibration of the building characterizes the structural response to earthquake loading. It affects the number of loading cycles that the building undergoes during a certain earthquake which is a major factor in determining the reduction in fatigue life at the critical sections within the building.

The selected case study buildings have six and twenty stories as described earlier which lead to the fundamental periods shown in table (1) which are 0.931 and 2.560 seconds respectively. This variation in the fundamental period affected the response during the selected long-duration earthquakes as shown in the resulted strain histories presented in figure 4. It is clear from those strain histories that for the same earthquake, San Bruno building which has shorter fundamental period has experienced significantly larger number of vibrating cycles than the North Hollywood building with the longer fundamental period. The density of strain histories plotted in figure 4 increases as the fundamental period decreases.

In order to evaluate the effect of the fundamental period on the low-cycle fatigue damage, the fatigue life relationship proposed by Mander (Mander et al., 1994) for reinforcing bars was used in estimating the fatigue damage within the floor beams in the three case study buildings. The analysis results showed that the largest values of plastic deformation occurred in the floor beams with a few exceptions for San Bruno building at the base level. The fatigue damage was estimated at the ends of each beam then the maximum value of the fatigue damage within all the beams at each floor was determined. Figure 5 presents the distribution of the maximum estimated fatigue damage at each level over the height of each building for the selected long-duration earthquake records. The main observation from the presented plots is the significant difference in the expected fatigue damage between San Bruno building and North Hollywood building. San Bruno building is expected to experience fatigue damage up to 35% due to Tohoku earthquake, 15% due to Muale earthquake and 40% due to Tokachi earthquake. On the other hand the maximum expected fatigue damage for the North Hollywood building didn’t exceed 10% due to any of the selected earthquake records. It is clear that the buildings with shorter fundamental periods were expected to experience more fatigue damage significantly higher than the buildings with longer fundamental period.

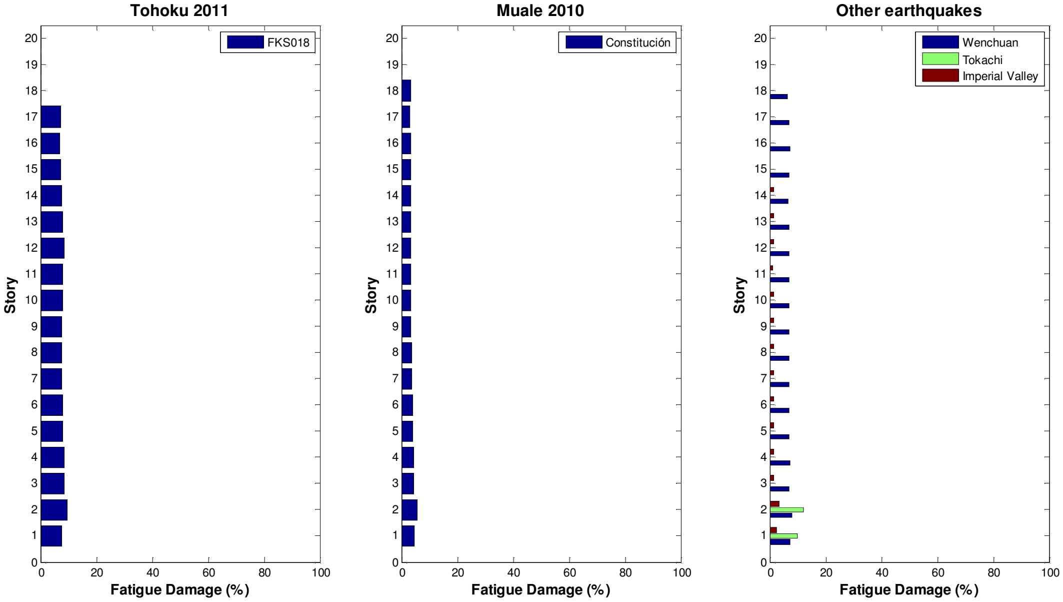
Conclusions

This paper presents a study of the accumulation of structural damage in RC members due to low-cycle fatigue behavior of the main reinforcing bars in order to estimate the extent of damage suffered by buildings due to long-duration earthquakes. Long-duration earthquakes were defined in this study as those ground motions that have significant duration of a minimum 40 seconds. The following conclusions can be made:

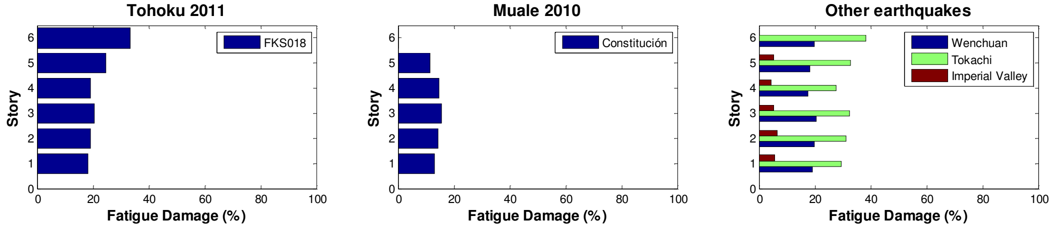
- Several cases of rupture of reinforcing bars in RC buildings have been reported after the 2010 Muale earthquake in Chile which were believed to be due to low-cycle fatigue.

- The fatigue analysis performed in this paper using the prescribed methodology showed that long-duration earthquake could lead to significant fatigue damage in reinforcing bars up to 40% which can be translated to a significant reduction in the fatigue life.
- Low- and medium-rise buildings are expected to have higher fatigue damage than high-rise buildings due to their short fundamental period which leads to more cycles of vibration that affect the fatigue life.
- Measurements from instrumented buildings can be an effective tool for investigation of structural behavior during earthquakes by using the recorded responses in verifying the accuracy of the numerical models and their response parameters.

Generally, the results showed that significant fatigue damage may occur due to the cyclic nature of the earthquake loading which is usually associated with inelastic strains in the case of large earthquakes. Also, the increased number of cycles during long-duration earthquakes may lead to higher potential of low-cycle fatigue damage. Current code provisions for seismic design of RC members don't consider the damage potential due to the low-cycle fatigue of reinforcing bars as there are no limits on the steel strains due to loading reversals. The strain limits provided in the ACI provisions are based monotonic tests which doesn't account for low-cycle fatigue (ACI318R-11, 2011).



a) North Hollywood Building



b) San Bruno Building

Figure 5. Maximum expected fatigue damage within the floor beams at each level of the case study buildings.

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