DESIGNING FOR DAMAGE ACCUMULATION AND LOW CYCLIC FATIGUE IN REINFORCED CONCRETE BRIDGE COLUMNS

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Abstract

Two columns tested in shake table under several records suffered damage for the design earthquake and the damage increased after the following records. To predict the test results, a damage accumulation (DA) and a low cyclic fatigue (LCF) models, both based on strain responses, were incorporated to a fiber finite element model (FFEM2) demonstrating 1) that FFEM2 results were close to test measures, 2) that to meet performance based seismic design it is convenient to accumulate the damage induced by each record and remove the fibers that reached 100% DA by means of the LCF model. When DA and LCF models are not included in FFEM2, the major damages to be prevented, observed during tests, are not predicted. A practical solution to prevent failures by LCF, based on increasing the confinement of the concrete, is proposed.

Keywords: damage accumulation, low cyclic fatigue, bridge columns

1. Introduction

Materials have memory and a fatigue life, and reversal strains, capable to induce vertical cracks in the cover or initial yielding in the steel, start a damage accumulation (DA) process which increases during the strong duration of the motion reducing the fatigue life of the materials. If more earthquakes excite the column, DA will increase more and the fatigue life of the materials may be exhausted causing failure by Low Cyclic Fatigue (LCF), a physical phenomenon that occurs when DA reaches 100%.

The use of one earthquake per performance level for retrofit design is recommended in [1] and [2, 3] to meet Performance Based Seismic Design (PBSD) however, there is no prescription to accumulate the effects of each earthquake, a necessary procedure for design of long life-time structures, as bridges.

The study focuses on the calculation of DA and LCF and its effects in bridge columns using finite fiber element models (FFEM) [4] and FFEM2, developed in this study. Both are based on the OpenSees framework (Open System for earthquake engineering simulation) that is an Object-oriented Finite Element Program [5, 6].

Two columns, tested in shake table under several records, are studied using FFEM2 to simulate the responses of the materials. Lara [4], using FFEM, predicted the fracture of bars due to LCF in one of the columns and the response of the other using FFEM2, deserving a prize of excellence [7].

2. Objectives and Scope

Objectives. 1) To study strain response when previous records effects (PRE) are captured and when such effects are not captured (NPRE), 2) If both responses are different, verify if the damages are due to DA and LCF using FFEM2 to predict the damages observed during the tests after each record, and 3) If DA and LCF caused the damage, to suggest simple solutions for design of bridge columns.

Scope. The columns, shake table tested, one in UC San Diego (UCSD) [8] and the other, in UC Berkeley (UCB) [9], are studied only for flexural response because columns were designed preventing shear failure. Responses are captured in only in one direction, as they were tested.
3. The Fiber Finite Element Model 2 (FFEM2)

3.1 Description of the model

Fig. 1(a) shows the 3-D Finite Fiber Element Model (FFEM2) [4] which contains three beam-column elements, [10], and Fig. 1(b) the section of the column, located at the ends of each element. The initial model, FFEM, required calibration of several parameters [4]: lengths of plastic hinge ($L_p$) and strain penetration ($L_{sp}$) which are similar to equations in [13], inelastic parameters for steel bars [12]: $R_0 = 20$, $R_1 = 0.925$, and $R_2 = 0.15$, and damping ratio $\xi = 0.03$. The constitutive relations for the concrete [11] and the steel [12], are explained in [4].

4. DA and LCF models for UC San Diego (UCSD) and UC Berkeley (UCB) columns

4.1 DA and LCF models included in FFEM2.

DA model [14] calculates the increasing damage induced by strain reversals in the materials, and LCF model [15, 16] controls the number of cycles at different strain amplitudes $\varepsilon_i$ inducing failure Eq. (1), and removes the fibers reaching 100% DA. In this study, materials damages are cracking, spalling, and yielding, and associated failures, as specified in the code, are: crushing of the core, which starts close to the bars and grows inside the column, fracture of bars, and instability.

\[ \varepsilon_i = \varepsilon_0 \left[ N_{\beta i} \right]^{-m} \]  

$\varepsilon_i = \text{strain amplitude response captured by FFEM2}$, $\varepsilon_0 = \text{strain at which the material fails in one cycle}$, $N_{\beta i} = \text{the number of accumulated cycles with amplitude } \varepsilon_i$ inducing failure by LCF, $m = \text{slope of the line of failure}$

The equation for the steel in UCSD column bars after calibration of $\varepsilon_0$ and $m$ [4], for $\varepsilon_s \geq 0.0026$ is:

\[ \varepsilon_i = 0.07 \left[ N_{\beta i} \right]^{-0.37} \]  

(2)

For the concrete, calibrations of $\varepsilon_0$ are: for spalling $\varepsilon_0 = 0.006$ (UCSD (2010)), and for crushing of the core, from [11], $\varepsilon_0 = 0.02$. Calibration of $m$ is explained below.

The damage induced by each strain is calculated using Eq. (3) [14]:

\[ D_i = n_i / N_{\beta i} \]  

(3)

$n_i$ is the number of strains with amplitude $\varepsilon_i$ captured in each strain history response, and $DA$ ([14] is:

\[ DA = \Sigma D_i \]  

(4)
To calculate $D_i$, one unknown: $m$, still needs definition and there are not fatigue tests on reinforced concrete columns that could support calibrations therefore, to calibrate an iterative procedure was performed:

From the history response assume $m$ and for each strain reversal $\varepsilon_i$, larger than 0.0015 [20], calculate $N_{fi}$ Eq. (1). $D_i$ induced by each strain is calculated with Eq. (5) and $DA$ with Eq. (4). $DA$ should be equal to unity at the time of failure, if not, start iterations changing $m$ until $DA = 1.0$ at failure. The procedure permits to calculate the number of cycles with amplitude $\varepsilon_i$ to induce LCF and to build the line of failure.

$$D_i = \frac{I}{N_{fi}}$$

(5)

After several iterations, the fatigue equations for LCF in the concrete, in the UCSD column, are:

In the unconfined concrete:  
$$\varepsilon_i = 0.006 \left[ \frac{N_{fi}}{N_{fr}} \right]^{-0.63}$$

(6)

In the confined concrete, West side: 
$$\varepsilon_i = 0.02 \left[ \frac{N_{fi}}{N_{fr}} \right]^{-0.76}$$

(7)

In the confined concrete, East Side:  
$$\varepsilon_i = 0.02 \left[ \frac{N_{fi}}{N_{fr}} \right]^{-0.89}$$

(8)

The procedure used for UCSD column is also used for UCB column. The parameters for the cover are similar in both columns, but $\varepsilon_0$ for the core vary due to [11] and there are slight differences in $m$, $\varepsilon_0$ and $m$ also vary for the steel in UCB column, Eq. (12), due to the differences in the diameter of the bars, [4],[17].

In the unconfined concrete:  
$$\varepsilon_i = 0.006 \left[ \frac{N_{fi}}{N_{fr}} \right]^{-0.63}$$

(9)

In the confined concrete, West side: 
$$\varepsilon_i = 0.015 \left[ \frac{N_{fi}}{N_{fr}} \right]^{-0.84}$$

(10)

In the confined concrete, East side:  
$$\varepsilon_i = 0.015 \left[ \frac{N_{fi}}{N_{fr}} \right]^{-0.77}$$

(11)

In the steel bars:  
$$\varepsilon_i = 0.16 \left[ \frac{N_{fi}}{N_{fr}} \right]^{-0.57}$$

(12)

These equations deliver an equivalent $N_{fi}$ for a constant strain but, fatigue means a number of strains of different amplitudes associated to the same $N_{fi}$ causing DA or LCF therefore, the procedure to calibrate $m$ secures the participation of the strains captured during responses.

5. The UCSD column and records used for the test and simulation

The reinforced concrete bridge column was tested under several earthquakes: frequent (EQ1), moderate (EQ2), occasional or design (EQ3), aftershock (EQ4), strong (EQ5), and aftershock (EQ6). Then, EQ7 was applied four times named EQ7 to EQ10 to induce the collapse, [18]. EQ7 is considered here as the rare earthquake. Table 1 shows the sequence of the records filtered by the shake table, and the Scale Factor (SF) for each one, and Fig. 2 shows the materials characteristics and design parameters of the column.

6. Limits for seismic design preventing failures: UCSD column

The code [19] specified limits are: 1) Ultimate confined concrete strain: $\varepsilon_{cu} \leq 0.02$, 2) Ultimate steel strain: $\varepsilon_{su} \leq 0.09$ for the 35mm bars, 3) Instability: $P-A \leq 0.25M_p$, $P = 2380kN$ and $M_p = 5800kN$-m Fig. 3(b), $A = u_m = 60cm$. 4) $\mu \leq 5$, $u_{max} = \mu u_y$. $u_y = 10cm$, $u_{max} \leq 50cm$. A larger $\mu$ is not a failure then, $u_m = 60cm$, controls the design, Fig. 3.

Table 1 - Structure period, T, Scale factor (SF) x record, peak ground acceleration (PGA) / period (T_s) of the record.

<table>
<thead>
<tr>
<th>EQ</th>
<th>SF</th>
<th>Record</th>
<th>PGA / T record (s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>EQ1: 1.0Agnews</td>
<td>0.20</td>
<td>0.10</td>
<td>EQ6: 1.0EQ3</td>
</tr>
<tr>
<td>EQ2: 1.0Corralitos</td>
<td>0.40</td>
<td>0.10</td>
<td>EQ7: 1.0Takatori</td>
</tr>
<tr>
<td>EQ3: 1.0Los Gatos</td>
<td>0.50</td>
<td>0.65</td>
<td>EQ8: 1.0EQ7</td>
</tr>
<tr>
<td>EQ9: 1.0EQ7</td>
<td>0.85 / 1.31</td>
<td>0.85 / 1.31</td>
<td></td>
</tr>
<tr>
<td>EQ10: 1.0EQ7</td>
<td>0.85 / 1.31</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Fig. 2 - Geometry and material characteristics
7. Inelastic and fatigue study of the UCSD column to the ten records

7.1 Comparison of hysteretic responses: PRE and NPRE

Inelastic dynamic responses (IDR) including DA and LCF analysis are calculated simultaneously. This is the reason why DA is captured during IDR, called previous records effects (PRE) responses, while NPRE means no DA. PRE responses show, since EQ3, stiffness degradation due to both: spalling, which starts the DA process, and crushing and, since EQ8, reductions of strength and displacement capacities due to fracture of bars and crushing, while NPRE responses show only the effect of IDR to each record and there is no material failure.

PRE and NPRE responses to the less damaging earthquakes: EQ1 and EQ2, are similar since both responses are close to elastic. Fig. 4 shows PRE and NPRE responses for EQ7, when crushing starts close to the bars, and to EQ9, when bars fractured inducing loss of strength and stiffness.

7.2 Force-Displacement responses

Fig. 5 shows the force-displacement (F-D) responses of the column to a long pulse applied at the end of each earthquake, and Table 2 the stiffness for PRE and NPRE responses: KPRE and KNPRE, measured at a force of 100kN. K0 is the initial stiffness measured in the F-D curve for EQ1 at the same force. Since EQ3 and on, KPRE is lower than KNPRE, also energy dissipation capacities, and drifts and ductility demands, Table 2. It is observed the reduction in strength at maximum displacement once bars fractured and crushing continued.

7.3 Concrete strain histories for PRE and NPRE responses

In this study, strains reversals equal or larger than 0.0015 [20] cause vertical cracks and DA leading to spalling of the cover and crushing of the confined concrete. In the bars DA starts at the yielding strain 0.0026 as it was tested in UCSD [8]. The history responses are in [21].

In Fig. 6, PRE responses show, from EQ2 to EQ7, 16 peaks in the West and 12 in the East larger than 0.0015 inducing DA which reached 100% at EQ7 and the core crushed. Since there is no DA for NPRE responses, there are only 2 peaks in the West and 2 in the East for EQ7, larger than the peaks recorded for PRE responses, but not enough in number to induce crushing.

DA depends on the number and amplitude of strains reversals captured during each record, therefore individual responses to earthquakes do not provide the appropriate information for seismic design and even worse if several records are used to meet PBSD.

In the following sections, results are related to crushing of the confined concrete since this failure, as captured by FFEM2 and coinciding with [8], occurred due to DA and LCF, before than instability and fracture of bars changing the code limits to prevent failures. Effects of DA and LCF on the steel bars is in [21]

7.4 Statistics of strain responses, PRE captured

The histograms and the percentage of damage calculated with Eq. (6), Eq. (7), and Eq. (8) per interval of strains, is shown in Fig. 7 for the West and East sides.
Fig. 4 - Hysteretic responses to records EQ7 and EQ9 PRE and NPRE

![Hysteretic responses to records EQ7 and EQ9 PRE and NPRE](image)

Fig. 5: Force-displacement curves. a) PRE, b) NPRE

![Force-displacement curves](image)

Table 2 – PRE and NPRE responses

<table>
<thead>
<tr>
<th>EQ</th>
<th>Max. Displacement demand, $\Delta_{MAX}$ (m)</th>
<th>Max. Strength demand, $V_{MAX}$ (kN) at $\Delta_{MAX}$</th>
<th>$K_{PRE}/K_{NPRE}$</th>
<th>Drifts</th>
<th>$\mu_{PRE}$</th>
<th>Max. Displacement demand, $\Delta_{MAX}$ (m)</th>
<th>Max. Strength demand, $V_{MAX}$ (kN) at $\Delta_{MAX}$</th>
<th>$K_{PRE}/K_{NPRE}$</th>
<th>Drifts</th>
<th>$\mu_{NPRE}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>EQ1</td>
<td>7</td>
<td>2513</td>
<td>10.90</td>
<td>0.010</td>
<td>0.70</td>
<td>7</td>
<td>2513</td>
<td>10.90</td>
<td>0.010</td>
<td>0.70</td>
</tr>
<tr>
<td>EQ2</td>
<td>12</td>
<td>4519</td>
<td>216.60</td>
<td>0.016</td>
<td>1.20</td>
<td>12</td>
<td>4519</td>
<td>216.60</td>
<td>0.016</td>
<td>1.20</td>
</tr>
<tr>
<td>EQ3</td>
<td>36</td>
<td>5941</td>
<td>2716.90</td>
<td>0.37</td>
<td>3.60</td>
<td>36</td>
<td>5041</td>
<td>2812.00</td>
<td>0.42</td>
<td>3.60</td>
</tr>
<tr>
<td>EQ4</td>
<td>16</td>
<td>2655</td>
<td>198.76</td>
<td>0.34</td>
<td>2.02</td>
<td>16</td>
<td>2451</td>
<td>216.60</td>
<td>0.016</td>
<td>1.20</td>
</tr>
<tr>
<td>EQ5</td>
<td>48</td>
<td>5144</td>
<td>4931.60</td>
<td>0.33</td>
<td>4.00</td>
<td>48</td>
<td>5282</td>
<td>5398.27</td>
<td>0.41</td>
<td>5.70</td>
</tr>
<tr>
<td>EQ6</td>
<td>36</td>
<td>4617</td>
<td>2722.40</td>
<td>0.33</td>
<td>3.60</td>
<td>36</td>
<td>5041</td>
<td>2812.00</td>
<td>0.42</td>
<td>3.60</td>
</tr>
<tr>
<td>EQ7</td>
<td>57</td>
<td>5199</td>
<td>6503.00</td>
<td>0.29</td>
<td>5.70</td>
<td>59</td>
<td>5330</td>
<td>7261.60</td>
<td>0.28</td>
<td>6.90</td>
</tr>
<tr>
<td>EQ8</td>
<td>58</td>
<td>5118</td>
<td>8144.20</td>
<td>0.26</td>
<td>5.80</td>
<td>75</td>
<td>5462</td>
<td>9138.30</td>
<td>0.34</td>
<td>7.50</td>
</tr>
<tr>
<td>EQ9</td>
<td>56</td>
<td>3764</td>
<td>5565.60</td>
<td>0.30</td>
<td>5.60</td>
<td>77</td>
<td>5462</td>
<td>9221.70</td>
<td>0.34</td>
<td>7.50</td>
</tr>
</tbody>
</table>

Table: PRE and NPRE responses

Fig. 6 - Strain histories for PRE and NPRE responses.

The displacement to prevent instability in this column is 60cm [19] however for PRE responses, crushing occurs during EQ7 at 57cm at $\varepsilon_c = 0.0094$ in the West after 16 strain reversals, and at $\varepsilon_c = 0.0088$ in the East after 12 reversals, while the code strain to prevent crushing is $\varepsilon_{cu} = 0.02$, no reversals included. These maximum strains captured at both sides, at failure, induced only 19% and 20% DA respectively Fig. (7). Therefore, such strains were not the direct cause of crushing. The maximum NPRE strain captured during EQ7 is 0.0128, less than 0.02, larger than 0.0094, but there is no crushing since 4 reversals are not enough, as seen later in Fig. 9.

Therefore, crushing is not due to just one large strain but due to the large one and also due to several previous strains damaging the materials.
8. Dynamic Fatigue Analysis (DFA) and design proposal to delay LCF

8.1 DA vs. time in the concrete and in the steel of the UCSD column: PRE and NPRE responses

Fig. 9 shows the relation DA vs. time, calculated using Eq. (6), Eq. (7), and Eq. (8) for the unconfined and confined concrete at the West and East sides of the UCSD column. It is observed in Fig. 9, that there is a progressive damage in the concrete: spalling and later crushing, both captured by FFEM2 in fibers 000 at the cover, and in fibers 0085, located 8.5cm inside the hoops, close to the bars.

In Fig. 9 the level of DA in the core, after full spalling during EQ5, is high, about 60% since the core cracked during EQ5 [8], therefore its potential to crush during EQ7 is also high. Fig. 12 shows that for NPRE responses, maximum DA in the core during EQ7 is 30% in the West because there is no DA captured.

During EQ8, 2 bars fractured and during EQ9, 4 more bars fractured. The bars started buckling during EQ7, but due to the large number of reversals strains, about 95, bars failed by LCF and not due to buckling [4], [22], Fig. 11. The fracture of bars due to DA requires a larger number of inelastic strains than peak compression strains to induce failures in the concrete.
8.3 Designing to delay DA and LCF effects, UCSD column

DA and LCF effects demonstrated that failures in the concrete occur at reversals strain amplitudes lower than code limit strains therefore, following design philosophy proposed in [19], a possible solution to delay spalling and crushing due to LCF might be to increase the volumetric ratio of transversal reinforcement, improving the confined concrete constitutive relation [11].

The reduction in hoop spacing increases $\varepsilon_{cu}$ [11] = $\varepsilon_0$ in Eq. (1), and also the number of peak compression strains required to reach crushing. After calibrations, if spacing is reduced from 15 to 9cm, using the same bar diameter, DA vs. time PRE results in Fig. 12 show that failure does not occur during the rare earthquake EQ7, but during EQ9 at both sides, therefore there is a delay in crushing. Also, the number of peak strains to crush the core, with the new spacing of the hoops, is 24 in the West and 18 in the East.

9. Column B1 tested at UC Berkeley (UCB)

9.1 Characteristics of the record and the column

In this study, UCB column [9] tested for one component of the Llolleo record, repeated several times with different scale factors (SF), is chosen to study the effects of DA and LCF. Table 3 shows the testing sequence.

Fig.13 shows the geometry of the prototype and the specimen, the longitudinal and transverse reinforcement, the properties of the materials, and the axial load. The Llolleo record applied to UCB column has a long duration close to two minutes and maximum PGA is 0.7g. The duration was reduced by 2.12 to shake the model and the record scaled up 1.29 times to reach the code spectrum for the design earthquake. The length scale prototype to model was 4.5 and the design of the column was performed using a strength reduction factor of 4.

Table 3 shows the characteristics of the record, the number of runs the record was applied to UCB column, the peak acceleration of the filtered records exciting the shake table during each run, and the damage observed after a run, [9].
9.2 Strain and displacement code limits

In Fig. 14: 1) Ultimate confined concrete strain: $\varepsilon_{cu} \leq 0.015$, 2) Ultimate steel strain: $\varepsilon_{su} \leq 0.12$ for the 12.7 mm bars, 3) Instability: $P-\Delta \leq 0.25M_p$, since $P = 290kN$ and $M_p = 188kN-m$ Fig. 15, $\Delta = u_m = 16cm$. 4) $\mu \leq 5$, $u_{max} = \mu u_y$. Since $u_y = 3cm$, $u_{max} \leq 15cm$. A $\mu$ slightly larger than 5 is not a failure mechanism then, the limit displacement for the $P-\Delta$ effect, $u_m = 16cm$ controls the design of the column.

Fig. 14 - Lateral monotonic capacity of the column, (a) Force-Displacement and; (b) Moment-Curvature

10. Inelastic and fatigue study of the UCB column to the nine records

10.1 Comparison of results  hysteretic responses: PRE and NPRE

Runs 1 and 2 are less than 27% Llolleo therefore, the hysteretic responses show slight changes in stiffness for PRE responses. However, DA causes stiffness degradation after Run 3, 39% Llolleo, and such reduction
increases for Runs 4 and 5, 52% and 130% Lolleo, respectively. Runs 6, 8 and 9 contain the maximum design level: 2.6 Lolleo and Fig. 15 shows the hysteretic PRE and NPRE responses to Run 6, when crushing of the core occurred, and to Run 9 when 1 bar fractured, after fracture of 2 bars during Run 8. NPRE responses show less reductions in stiffness and strength than PRE responses.

![Fig. 15 - Hysteretic PRE and NPRE responses of UCB column](image)

10.2 Force-Displacement responses

Fig. 16 shows the force displacement (F-D) responses of the column for the 9 Runs and for PRE and NPRE responses. As in UCSD column analysis, NPRE responses for UCB column exhibit larger stiffness, strength, drifts, and ductility ratio capacity because there is no DA captured, Table 4.

![Fig. 16 – Force-Displacement responses of UCB column](image)

<table>
<thead>
<tr>
<th>RUN</th>
<th>Max. Displacement demand, $u_{\text{max}}$ (mm)</th>
<th>Max. Strength demand, $V_{\text{max}}$ (kN) at $u_{\text{max}}$</th>
<th>Total Energy dissipation capacity (kJ/m)</th>
<th>$K_{\text{PRE}}/K_{\text{UC}}$</th>
<th>Drifts</th>
<th>$\mu_{\text{PRE}}$</th>
<th>Max. Displacement demand, $u_{\text{QR}}$ (mm)</th>
<th>Max. Strength demand, $V_{\text{QR}}$ (kN) at $u_{\text{QR}}$</th>
<th>Total Energy dissipation capacity (kJ/m)</th>
<th>$K_{\text{NPRE}}/K_{\text{UC}}$</th>
<th>Drifts</th>
<th>$\mu_{\text{NPRE}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>RUN #1</td>
<td>1</td>
<td>27.0</td>
<td>1.65</td>
<td>1.00</td>
<td>0.004</td>
<td>0.84</td>
<td>1</td>
<td>27.8</td>
<td>1.55</td>
<td>1.00</td>
<td>0.004</td>
<td>0.33</td>
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<tr>
<td>RUN #2</td>
<td>2.1</td>
<td>48.4</td>
<td>1.96</td>
<td>0.85</td>
<td>0.009</td>
<td>0.70</td>
<td>2.1</td>
<td>48.4</td>
<td>1.96</td>
<td>0.95</td>
<td>0.009</td>
<td>0.70</td>
</tr>
<tr>
<td>RUN #3</td>
<td>2.9</td>
<td>58.9</td>
<td>2.36</td>
<td>0.45</td>
<td>0.012</td>
<td>0.97</td>
<td>2.9</td>
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<td>2.36</td>
<td>0.55</td>
<td>0.012</td>
<td>0.97</td>
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<tr>
<td>RUN #4</td>
<td>4.1</td>
<td>63.2</td>
<td>2.75</td>
<td>0.43</td>
<td>0.017</td>
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<td>4.1</td>
<td>63.2</td>
<td>2.75</td>
<td>0.60</td>
<td>0.017</td>
<td>1.87</td>
</tr>
<tr>
<td>RUN #5</td>
<td>7.1</td>
<td>66.3</td>
<td>29.70</td>
<td>0.38</td>
<td>0.029</td>
<td>2.37</td>
<td>8</td>
<td>67.0</td>
<td>34.85</td>
<td>0.45</td>
<td>0.033</td>
<td>2.66</td>
</tr>
<tr>
<td>RUN #6</td>
<td>11.7</td>
<td>70.1</td>
<td>198.10</td>
<td>0.36</td>
<td>0.048</td>
<td>3.90</td>
<td>12</td>
<td>68.8</td>
<td>108.20</td>
<td>0.39</td>
<td>0.049</td>
<td>4.00</td>
</tr>
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<td>RUN #7</td>
<td>7.1</td>
<td>60.4</td>
<td>26.20</td>
<td>0.34</td>
<td>0.029</td>
<td>2.37</td>
<td>8</td>
<td>67.0</td>
<td>34.85</td>
<td>0.45</td>
<td>0.033</td>
<td>2.66</td>
</tr>
<tr>
<td>RUN #8</td>
<td>12.1</td>
<td>68.8</td>
<td>117.82</td>
<td>0.91</td>
<td>0.050</td>
<td>4.09</td>
<td>12</td>
<td>68.8</td>
<td>108.20</td>
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<td>0.049</td>
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<td>66.0</td>
<td>191.10</td>
<td>0.76</td>
<td>0.052</td>
<td>4.20</td>
<td>12</td>
<td>68.8</td>
<td>108.20</td>
<td>0.39</td>
<td>0.049</td>
<td>4.00</td>
</tr>
</tbody>
</table>

10.3 Concrete strain histories for PRE and NPRE responses

In Fig. 17, PRE responses show in the concrete, from Run 4 to Run 6, 15 peaks in the West and 16 in the East larger than 0.0015 inducing DA which reached 100% at Run 6 when it is assumed here that the core crushed since 1 bar buckle and 2 others started to buckle, Table 3. NPRE response for Run 6 shows 7 peaks in the West and 8 in the East, however the number of reversals are not enough to induce crushing as it will be seen later.

As it was established for the UCSD column, PRE responses show that failures are related to the number and amplitude of the strains reversals (LCF), while NPRE results do not provide the appropriate information for seismic design, particularly if several records are used to meet PBSD.
10.4 Statistics of strain responses, PRE captured
Statistics for the UCB column resemble those of the UCSD column. The strain at crushing [11] is $\varepsilon_{cu} = 0.015$ however, 1) the maximum strains and number of peaks in the West and East sides, PRE captured, are $\varepsilon_c = 0.0064$ and 15 peaks, and $\varepsilon_c = 0.0058$ and 16 peaks, causing 16% and 18% DA respectively at crushing at each side, 2) The reversals captured up to Run 6 are less than 0.015, but induced 100% DA and the core crushed close to the bars at both sides, 3) the maximum NPRE reversal captured during Run 6 is 0.006, at both sides, less than 0.015, but there was not crushing as it will be seen in Fig. 18.

Therefore, once again, crushing, a failure mechanism [19], is due to DA induced by several previous strains and not due to just the maximum captured at failure.

11. Dynamic fatigue analysis (DFA) and design proposal to delay LCF
11.1 Variation of DA vs. time in the confined concrete and in the steel bars, UCB column
Fig. 18 shows that there is no damage in the core for the first 3 runs, but for Run 4, when yielding occurred, DA in the core reached 4% at both sides. For run 5, the cover spalled and DA in the core is 32% in the West and 40% in the East. Crushing occurred at both sides, Run 6, and maximum displacement was 11.7cm. During NPRE response to EQ6, Fig. 18(c), DA reaches 24% in the West and 43% in the East therefore, there is no crushing because there is no DA measured for the previous records.

In the study about DA in the bars [4] Fig. 19, during Run 8 2 bars fractured and during Run 9, 1 more bar fractured, due to LCF in both cases. The bars started buckling during Run 6, but due to the large number of reversals strains, about 95, bars failed by LCF and not due to buckling [4], [22]. The fracture of bars requires a larger number of inelastic strains than peak compression strains required to induce failures in the concrete.

11.3 Delay DA and LCF effects, UCB column
Using the same procedure to delay effects of DA and LCF in the UCSD column, calibrations to increase the volumetric ratio of transversal reinforcement, to improve the confined concrete constitutive relation [11], show that for the UCB column if the spacing is reduced from 3.1cm to 1.8cm, same bar diameter, crushing is delayed to Run 9, Fig. 20. The number of cycles increases considerably to 48 in the West and to 46 in the East side.
12. Conclusions

Testing of two columns, under several earthquakes, studied here, demonstrated that the code provide excellent methods for seismic design of reinforced concrete bridge columns for the specified life-safety earthquake. However, in areas of high seismic hazard, several earthquakes of different intensities may excite the column and the responses will contain a number of reversal strains inducing DA in the materials therefore, it becomes important to study seismic response including DA and LCF.

To meet PBSD several records should be used and every one induces an amount of damage that can be captured by DA and LCF models however, at present, codes do not require to accumulate damage on a structure during every earthquake. DA required calibrations of $m$, Eq. (1), for each column, and calibrated values are similar thus, if more columns are studied a reliable value of $m$ will permit to predict, during design, the occurrence of the first failure that for the columns was crushing of the concrete, with not previous test.

The comparison of responses including previous records effects, PRE responses, which capture DA for every record, with NPRE responses which do not capture DA, show that during PRE responses failures occurred when DA reached 100% in the material fibers which are then removed by LCF model and the predictions agreed with test results. When DA is not captured, NPRE responses exhibit larger stiffness, strength, and ductility ratios capacities than PRE responses, and even none of the observed failures.

According to the code, instability at 60cm displacement controls the design of the UCSD column, and at 16cm the design of the UCB column however, according to this study crushing of the confined concrete occurred previously at 57cm and 12cm respectively. DA and LCF changed the mechanism controlling the design [19]: from instability to crushing of the confined concrete.

Regarding strains, maximum ones captured did not reach even half the maximum code specified strains [19], however failures occurred, no after those maximum but due to a number of strains captured during previous records in addition to the maximum strains captured at failures.

To prevent crushing, one proposal is presented: Reduce the code calculated spacing of the hoops or increase the corresponding bar diameter. In this study, reducing 40% the code hoops spacing, same bar diameter, will increase the maximum confined strength and the ultimate strain of the confined section [11] delaying the occurrence of crushing since the number of peak strains required to induce such failure, increases.

Based on the results for the two columns, which characteristics are similar, subduction earthquakes require more reversals strains but with lower amplitudes than earthquakes containing long pulses to cause failure. Another fact is that fracture of bars requires a larger number of inelastic strains, at least 95 for first fracture, than peak compression strains, about 12, required to induce crushing of the confined concrete.
Stiffness degradation in the concrete induced by DA, does not cause strength reduction at maximum displacement demand. [23]. However, strength reduces at maximum response demand when, in addition to stiffness degradation, bars fracture due to LCF.

13. References