17th World Conference on Earthquake Engineering, 17WCEE

Sendai, Japan - September 27th to October 2nd, 2021

# MODELLING CYCLIC DEGRADATION OF BRIDGE R. C. COLUMNS SUBJECTED TO CONCURRENTLY SEISMIC EVENTS

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## *Abstract*

Lower magnitude events typically are concomitant with seismic events, thus cumulative damage in structures has become an increasing concern in the structural engineering community. Currently, structural non linear models are capable of considering cyclic degradation, however there are several difficulties to perform the calibration of those parameters. In this research, the application of Approximate Bayesian Computation algorithms is explored to perform the inference of cyclic cumulative degradation parameters of a model constructed in OpenSees software, using laboratory tests results as a dataset. The data has been taken from a scaled reinforced concrete column subjected to a dynamic load, performed in PUCP laboratory. Two models are presented in this study: (1) a blind prediction using nominal model parameters for concrete columns and (2) a simplified model that accounts for possible non-measured lateral slip; both models do not include degradation to latter demonstrate the importance of this matter. Results show that the simplified model, calibrated using an Approximate Bayesian Computation (ABC) technique, better predicts the test deformation records; also, the strengths of the method for the inference of those parameters are displayed, as they give information to allow a probabilistic seismic assessment of a reinforced concrete column that includes the modelling uncertainties. Later on, this model will be used to evaluate the effects of cumulative damage on structural design criteria.

*Keywords: bridge columns; shaking table tests; seismic damage accumulation; modelling cyclic degradation; Bayesian algorithms*

## 1. Introduction

When an earthquake of a significant magnitude takes place, it is usually followed (and sometimes, preceded) by a series of earthquakes, together known as a seismic sequence. The magnitudes of these events can be almost as high as the main event. In the seismic sequence of the 2016 Pedernales earthquake, the main event was 7.8 Mw magnitude, eight aftershocks with magnitude greater than 6.0 Mw were observed, in addition to hundreds of smaller quakes. The largest aftershock reported 6.9 Mw magnitude.

Structural design codes present methodologies for designing for the so-called "design earthquake" as an isolated event, however, history has shown us that the main event is linked to a seismic sequence and this set also affects structures. This reality tells us that when structures are affected by the so-called main earthquake, they have already suffered a previous earthquake of less magnitude and that, after the main earthquake, the structure will be affected by other earthquakes, which will affect the already degraded structure.

For this reason, it has been necessary to investigate the process of degradation of strength and stiffness of structural elements. For this purpose, specimens of elements have been constructed to test them and obtain real responses of the elements and the behavior of the materials that form it. This experimentation provides real answers, but because of its complexity and cost to implement massively, the results obtained from few trials are used to construct numerical models that replicate the observed behavior. These models undergo a calibration process, so they are able to be used for predictions on the behavior of similar structural elements.

Usually, the calibration process of numerical models is performed on the basis of test and error methods, in which the response of the proposed model is visually compared with the response measured in an experiment. Examples of these procedures can be found in recent research for different structural components (LeBorgne & Ghannoum [1]; Lee & Han [2]; Sattar & Liel [3]); in some cases, stricter procedures (i.e. Haselton, Liel, Lange & Deierlein [4]) are adopted, but in general "manual" methodologies are used.

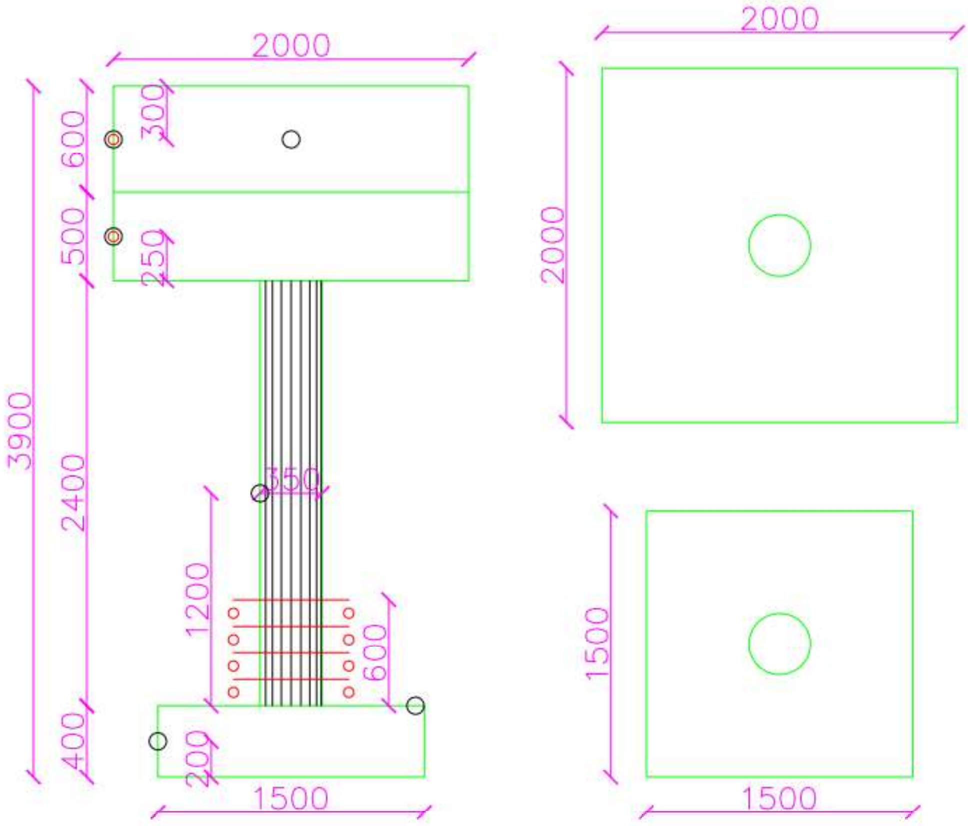
Recently, Chiachío, Barros & Chiachío [5], have proposed the use of inference methods based on Approximate Bayesian Computation (ABC) to calibrate non-linear numerical models of reinforced concrete columns subject to cyclic static type loads. The methodology allows to infer the damage parameters, which are usually very difficult to estimate using a "manual" procedure.

In this work, the ABC-SubSim algorithm (Chiachío, Beck, Chiachío & Rus [6]) is used to infer the parameters of a simplified model of the column subject to dynamic loads, in order to infer some possible slip movements of the specimen. The importance of considering degradation and the effects of earthquake sequences for structural evaluation will be demonstrated.

## 2. Specimen and Dynamic Test

### 2.1 Specimen characteristics

The specimen consists of 4 parts: a square footing of 1.5 m on the side, a circular column 35 cm in diameter and free height of 2.40 m, a square head and a square upper mass of 2.0 m on the side and joint height of 1.10 m. The footing, column and head form a monolithic element while the upper mass is a separate element that will be placed and fixed for testing and removed when finished; the independence of this mass allows it to be used for several tests. The total weight of the specimen is 149.5 kN, with the weight of the upper mass being 110 kN. Fig. 1 presents a cross-section and plant view of the specimen.



(a) Specimen cross section (b) Plan view of the upper mass and the footing.

Fig 1. Dimensions of the specimen. Red circles indicate the location of LVDT's, whilst black circles indicate the location of the accelerometers.

The concrete compressive strength (f'c) was obtained following ASTM C39 [7] requirements, resulting equal to 25 MPa. The longitudinal reinforcement of the column was conformed with 12 bars of 12mm in diameter, which corresponds to an amount of steel 1.41%. The tensile strength tests of the longitudinal bars indicate that the yield stress of the steel (fy) is 460 MPa with a relative strain of.0.23% and the ultimate stress (fu) is 737 MPa with 15% of relative strain.

To obtain displacement records, displacement sensors (LDVT) were placed in 15 positions: 7 for longitudinal readings and 8 for transverses. The shake table also has a motion sensor. The arrangement is presented in Fig. 1. To obtain the acceleration records, 5 accelerometers were placed in the specimen: 4 to obtain longitudinal accelerations placed in the center of the west face and one for transverse accelerations in the center of the south face, to capture some twisting that could take place. The shake table has an accelerometer that also recorded accelerations at that level. Fig. 1 shows the location of the accelerometers.

## 2.2 Dynamic Tests

The event analyzed in this study is the first event in the seismic sequence to which the column was submitted and corresponds to Maule's Earthquake in Chile of 27/02/10 recorded in the Constitucion station. The record has a maximum acceleration of 0.77 g. Figure 2 shows the acceleration record for this event.

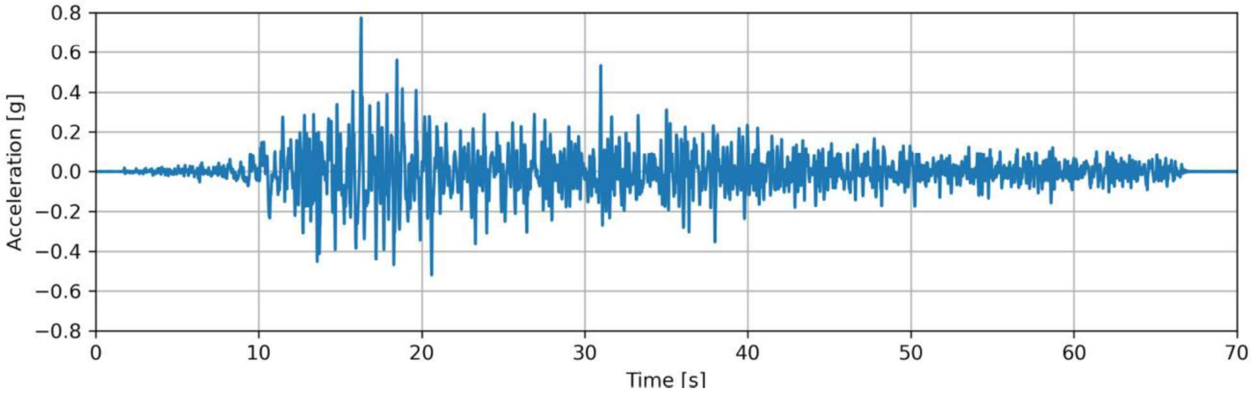


Fig 2. Maule-Chile 2010 seismic record (Constitucion station).

With the instrumentation installed, horizontal and vertical displacements, accelerations and unit deformations were recorded according to the installed instrumentation. From Fig. 3 it can be seen that the earthquake generated maximum deformations of 120 mm in the upper mass and that the degradation of the column was such that there were permanent deformations, this is evident as the baseline of the record is displaced approximately -10 mm.

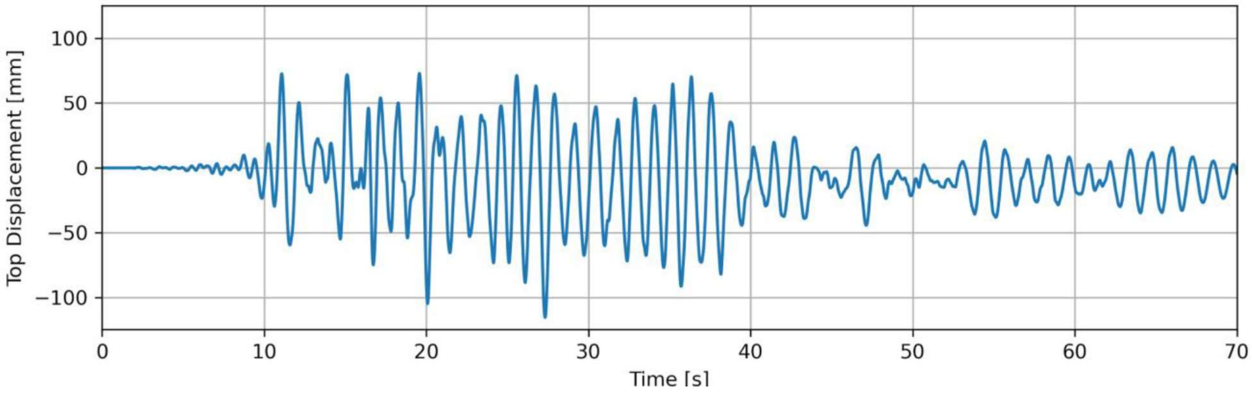


Fig 3. Maule-Chile 2010 upper mass displacement record.

## 2.3 Bayesian Inference Method

Bayesian inference aims to find the parameter values of a proposed model, which explain the results observed in an experiment, using any previous information about the event. The concerned reader may refer to Beck [8].

Bayesian approach algorithms can be used for this purpose; in particular, the ABC-SubSim algorithm has been used in this work, because of the advantages it presents in terms of its efficiency and ability to work with a large number of parameters; for details on this algorithm, refer to Chiachío, Beck, Chiachío & Rus [6]. Chiachío, Barros & Chiachío [5] demonstrated that this algorithm allows to develop the inference of nonlinear models based on static tests; in this work, the method will be used to infer the dynamic behavior of the specimen structure in the next section.

The method consists in adopting an equation called "metric" that compares the results of the model with those of the test. Some distribution is adopted for each parameter based on some prior knowledge, known as "prior distribution". The algorithm starts its search based on the "prior" to find the values of the parameters that tend to minimize the metric. As a result, a distribution of values is obtained for each parameter, known as "posterior distribution".

## 3. Numerical Models

This section presents the development of two numerical models to predict the behavior recorded in the test presented in the previous section. First a complex model is presented as a "blind prediction". Then a simplified model without degradation is presented that is calibrated using the ABC-SubSim.

### 3.1 Model 1 - Column DB

Based on the test conditions, dimensions and characteristics of the materials, the specimen was modeled using ASDEA Software's STKO [9] pre and post processor for OPENSEES [10]. The model has 11 nodes from the joint of the column and the cap to the top edge of the specimen. This joint is considered fixed. Fig. 4 shows the model geometry.

The nodes that make up the column are joined by frame elements with displacement-based formulation, fed by a nonlinear fiber section. The elements have 5 integration points, following the formulation of Newton-Cotes. Concrete fibers were modeled with the constitutive "concrete02", using the parameters suggested by Mander, Priestley & Park [11]. Reinforcement steel was represented by the constitutive "steel02".

The nodes that make up the upper mass are joined with elastic "frame" elements, and in the central nodes of the two divisions of the upper mass were assigned the respective participatory masses.

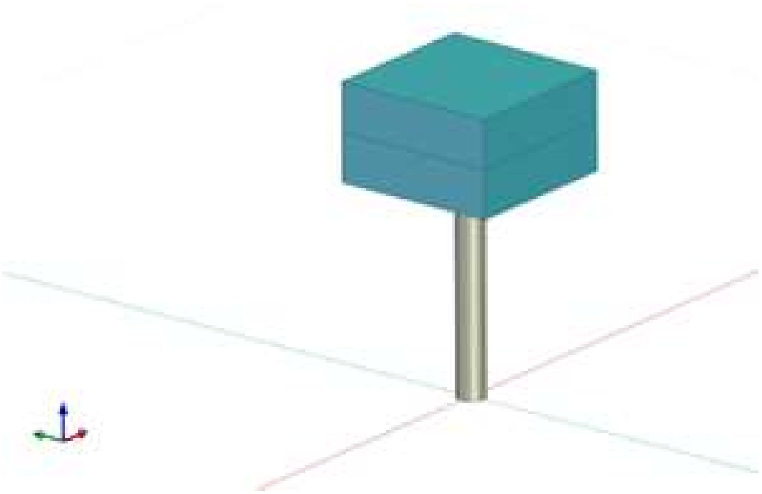


Fig 4. Model 1.

This model performed a dynamic analysis with Maule's earthquake record including the effects of gravitational loads, as well as P-Δ effects. The results of the analysis differ with what is recorded in the laboratory as can be seen in Fig. 5 showing the deformation obtained from the test and the deformation in the analysis at the same location (the upper mass of the column).

Among the causes of this difference, two main ones can be mentioned. First, classic models of material behavior such as steel and concrete are based on displacement-controlled or force-controlled tests where there is no interaction between them, so any interrelationship between them should be considered additionally. Second, the parameters of the elements must be adjusted as the deterioration progresses in order to represent the actual damage level. This becomes more noticeable in this type of analysis where the overall dynamic response of the structure is conditioned by the state of the elements at every step of time during the earthquake.

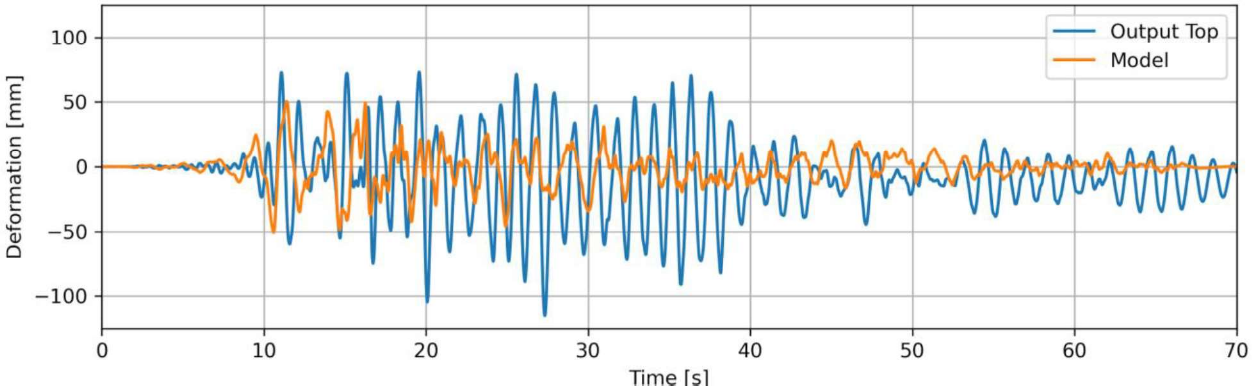


Fig 5. Comparison between displacement test record and model 1 displacement

### 3.2 Model 2 – Bilineal Column

Based on the test conditions and a previous study with different simplified models, it was concluded that the structural system can be represented with a three degrees of freedom serial spring system, which are: (1) some possible slippage at the base, (2) column deformation and/or (3) the sliding of the upper mass. Fig. 7 shows the simplified model adopted, where the values $\theta_i$ are the parameters to be inferred; the model consists of three bilinear behavior springs with elastic stiffness "E", yield force "$F_y$" and post-yield stiffness "$b_E$". Each spring is joined by specific masses. In addition to the 12 parameters shown in Figure 6, factors α and β are included in the inference with modifiers $\theta_{13}$ and $\theta_{14}$; these factors correspond to the damping factor proportional to the mass and current stiffness, respectively, for the Rayleigh damping model.

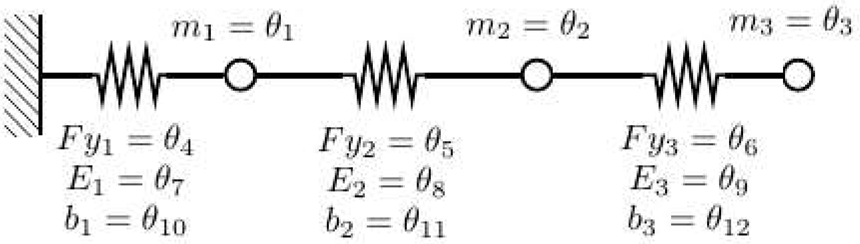


Fig. 6. Model 2 and parameters for inference.

Table 1. Prior distribution of the parameters.

| ***θ*** | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| L | 1e3 | 1e3 | 1e3 | 1e5 | 1e5 | 1e5 | 1e6 | 1e6 | 1e6 | 1e-4 | 1e-4 | 1e-4 | 0 | 0 |
| U | 1e4 | 1e4 | 1e4 | 1e6 | 1e6 | 1e6 | 1e7 | 1e7 | 1e7 | 1 | 1 | 1 | 1 | 1e3 |

Units: kg, kN and m.
"L": lower limit of the uniform prior distribution.
"U": upper limit of the uniform prior distribution..

Table 2. Posterior distribution of the parameters..

| $\boldsymbol{\theta}$ | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| $\boldsymbol{\mu}$ | 3.85 e3 | 2.46 e3 | 9.33 e3 | 2.56 e5 | 1.43 e5 | 7.78 e5 | 3.85 e6 | 6.57 e6 | 8.07 e6 | 5.17 e-1 | 6.97 e-2 | 6.21 e-1 | 7.56 e-1 | 2.76 e2 |
| $\boldsymbol{\sigma}$ | 1.04 e2 | 2.60 e2 | 7.21 e1 | 9.43 e3 | 8.21 e3 | 4.01 e4 | 2.71 e4 | 9.44 e4 | 1.48 e5 | 1.68 e-2 | 9.51 e-4 | 4.96 e-2 | 2.42 e-3 | 1.86 |

Units: kg, kN and m.
"$\boldsymbol{\mu}$": mean value of the posterior distribution.
"$\boldsymbol{\sigma}$": standard deviation of the posterior distribution.

Table 1 presents a summary of the prior information adopted (prior) to apply the methodology described in section 2.3; an uniform distribution of values was chosen, so that, in that range, there is always the same probability of choice. The θ-parameters of the second spring shown in Fig. 6 were selected to be in range of the column behaviour. The metric was defined in terms of frequencies, using Eq. (1).

$$\rho = \frac{\sum |y_1 - y_2|}{\sum y_1} \tag{1}$$

Where $y_1$ is the fourier transform of the measured deformation signal, $y_2$ the fourier transform of the displacement response of node 3 of the model (see Fig. 7). Table 2 shows the results (posterior) of each parameter in terms of mean and standard deviation, and Fig. 7 shows the result of the inference, where it can be noticed that the model response is very close to the test response; however, the model presented is not able to explain the residual deformation observed at the end of the trial. This occurs because the model, although non-linear, has no degradation and is not able to represent this effect.

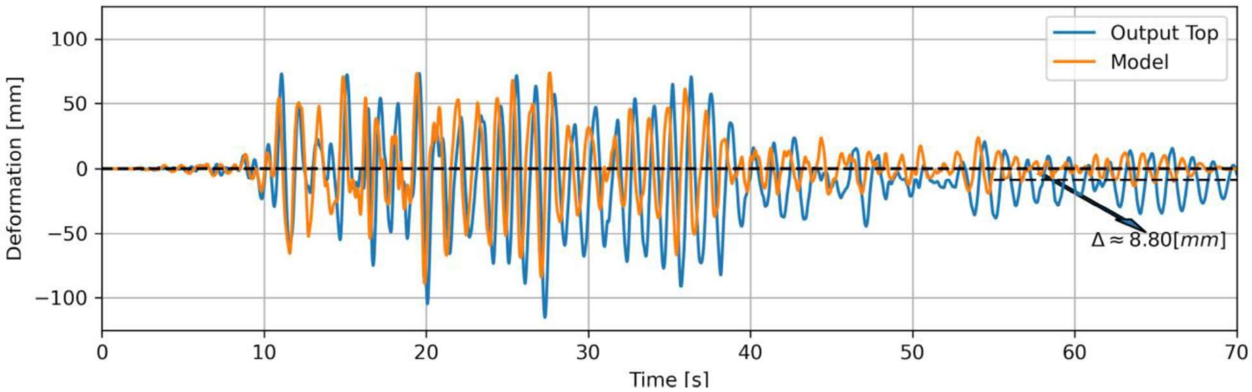


Fig. 7. Comparison between displacement test record and model 2 displacement

## 4. Conclusions

From the results obtained in section 3, we can conclude the following:

Model 1, using nominal values of material behavior, usually obtained from static tests, can provide predictions that result differently from the dynamic behavior of a structure.

Model 2 corresponds to a simplification to establish the possibility of having characteristic factors of the test, which could significantly affect the interpretation of the performed measurement. A detailed study of the presented results of model 1, it can be concluded that the recorded movement is not synchronized with what is estimated in the model. Bayesian inference developed for model 2 demonstrates that there are other test components involved in the measurement, such as base and top mass slippage. In addition, it also demonstrates that there are model components that could affect prediction, such as damping model parameters. With manual calibration, it would have been virtually impossible to reach these conclusions.

Future work in this matter includes a more detailed investigation on damping model parameters and the inference of degradation parameters on a non-simplified concrete column model.

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