

Structure-Dependent Displacement-Based Intensity Measure for Bridges

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ABSTRACT:

Bridges can be considered as nodes of a more complex system (a network). The state of the bridges as nodes of a network influences the state of the system itself and the effect of the ground motion on the response of the structure is a fundamental aspect in network risk assessment.

The intensity of ground shaking can be measured in many ways. Some of these are structural-response independent (e.g. peak ground acceleration, velocity and displacement, Housner Intensity and Arias Intensity) and others are correlated to the response of the structure (e.g. spectral acceleration, spectral displacement).

In this work, a Displacement-Based intensity measure, that is dependent on the structural features of the specific bridge under consideration, is proposed to describe the characteristics of earthquake ground motions and their effects on bridges. The intensity measure requires a static pushover analysis of the bridge to be produced, ideally with an adaptive pushover algorithm, and the MDOF response of the bridge is then transformed to the response of a SDOF system, again using an adaptive algorithm. The displacement demand to the bridge for a given accelerogram is estimated using the static pushover curve of the SDOF system and an overdamped displacement response spectrum through an iterative procedure that does not suffer from the convergence problems found with other similar methodologies.

The predicted intensity measure has been calculated using over 40 accelerograms and has been compared with the displacement response obtained from nonlinear time-history analyses. These analyses allow the efficiency of the proposed intensity measure to be ascertained by studying the variability in the response, given the intensity measure level. The efficiency of other commonly used intensity measures is also presented in order to highlight the improved performance of the proposed methodology.

Keywords: intensity measure, displacement-based, reinforced concrete bridges

1. INTRODUCTION

Civil infrastructures (e.g. highways, railways, pipelines) are the backbone of modern society but they remain vulnerable to a wide variety of natural and human made hazards. In seismically hazardous regions, assessment of the seismic vulnerability of bridges is of great importance since such information is needed for reliable estimation of the losses that a possible future earthquake is able to induce. The first step in vulnerability assessment is to ascertain the demand to which a given structure might be subjected, and which will depend on the intensity of the ground shaking at the site of the structure.

The size or the intensity of ground shaking (the so-called intensity measure) can be measured in different ways. Instrumental measurements can be used to define a snapshot description of the ground motion. Peak quantities in terms of acceleration, velocity and displacement are used for this purpose. Spectral measures at given periods give information about the maximum response of the structure. Other parameters taking into account the energy content of the ground shaking can be used (e.g. Arias Intensity, Arias (1970)).

In order to predict the damaging potential of a given accelerogram, the intensity measure (IM) needs to be well correlated with the response of the structure. For the buildings this response quantity is in

general defined as the maximum lateral top displacement. For bridges it is necessary define a suitable quantity able to describe the system behavior during a dynamical excitation and that could be easily compared with the structure dependent displacement based intensity measure proposed. These quantities are called Engineering Demand Parameters (EDPs).

Different IM are evaluated herein by comparing them with EDPs for a number of case study bridges in order to compare them in terms of *efficiency*, *sufficiency*, and *proficiency* (Padgett *et al.*,2008).

2. NON-LINEAR MODELS

Four bridges are considered in this study-case. They are characterized by a 200 m deck and a different configuration of piers (Casarotti *et al.*,2007).

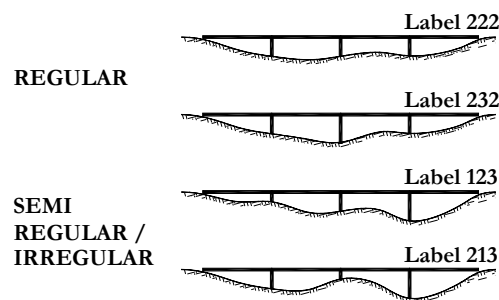


Figure 2.1. Pier Layout

In this study bridges are labelled with different codes composed of a 1, 2 or 3. These numbers represent the order of the length of the piers (e.g. 123 bridge is characterized by 7, 14 and 21 m piers in this order) (Casarotti *et al.*, 2007).

2.1. Description of Models

The models involved in this study are implemented in SeismoStruct (SeismoSoft, 2010), a fibre modelling Finite Elements program for seismic analysis of structures, which can be freely downloaded from the Internet.

The analyses performed in this study are Non-Linear Pseudo Static Analyses (i.e. pushover analyses) with a displacement-based adaptive loading algorithm (DAP) (Antoniou and Pinho, 2004), in order to define the yielding point of the system (bridge), and Time History Analyses to estimate the demand on the structure in terms of different EDPs.

2.1.1. Materials

The expected material strength and stress strain relation are used for unconfined and confined concrete as well as reinforcing steel to more accurately capture the average bridge capacity and behaviour. The constitutive models used in this study are the Constant Confined Mander Concrete model (1988), for both confined and cover concrete, and the Menegotto-Pinto Steel model (1973) for the reinforcement.

2.1.2. Piers and Superstructure

The piers are modelled using a 3D inelastic beam-column element with rectangular hollow section of dimensions 2.0m x 4.0m, a longitudinal steel ratio of 0.76% and a wall thickness of 0.4 m. They are supposed to be fully restrained at one extremity assuming that the foundations are defined at the level of the base fixity. The deck is a 3D elastic non-linear beam-column element.

2.1.3. Piers and Deck Connections

The connection between the pier and the deck is modeled as a linear link and an elastic rigid element

that is introduced to take into account the distance between the top of the column and the centroid of the superstructure.

2.1.4. Abutments

The bridges considered in this study are characterized by abutments with linear symmetric behaviour and rotation-free (Casarotti *et al*, 2005) with the purpose to simulate the effect of linear pot bearings that might be insert at the deck edges; these are modeled as four springs in parallel with defined properties.

3. DEFINITION OF THE ENGINEERING DEMAND PARAMETERS (EDP)

In order to consider the bridge (multi-degree of freedom) as a single degree of freedom and compare the demand with the IM, a displacement of a single point of the system needs to be selected.

For buildings, this control point often corresponds to a top storey point. For standard structures, this point is able to give information about the maximum response of the system (e.g. maximum lateral displacement). For bridges the definition of this point is not easy. The deck displacements are strongly influenced by the disposition of piers, the length of the deck itself and the distribution of the masses. Bridges are in general characterized by considerable length and the foundations can be built on different types of soil; then the excitation can be modified as a function of the site conditions. Sometimes multiple support excitation is to be taken into account. For these reasons it is possible that two (or more) points of the deck respond out of phase and then it is difficult describe the entire system behaviour.

The bridges considered in this study are characterized by well correlated top displacements (as they have a short deck) and the proposed EDPs in this study give information about the mean maximum displacement of the structure.

The EDPs are computed for each time step of the dynamic analyses and then the maximum value is taken into account as representative of the structural response,

The first EDP (EDP01) is formulated as in Eqn.3.1:

$$EDP01 = \frac{\sum_{i=1}^{npiers} d_i^2}{|\sum_{i=1}^{npiers} d_i|} \quad (3.1)$$

This quantity can be considered as a modified expression for the equivalent single degree of freedom displacement, as defined by Casarotti *et al*. (2005). In the proposed formulation, masses are not considered because the top pier displacements (d_i) are the result of Time History Analyses and they already take into account the displacement amplification due to the associated masses.

EDP02 is defined as the maximum mean top pier displacement.

EDP03 is the maximum displacement of the central pier. Selecting the central pier as representative of the system, the displacement shape is easier to predict. If the central pier is taller than the lateral ones the deck deformation shape will be close to the one proposed at the top of Figure (3.1). If, vice versa the central pier is the shortest the expected shape will be similar to the shape proposed at the bottom of Figure (3.1).

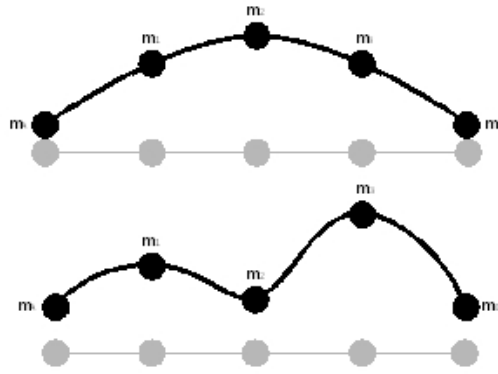


Figure 3.1. Bridge displacement shapes

EDP04 is the maximum displacement of the shortest pier considered as the weakest element in the element layout (Priestley *et al.*, 2007)

4. DEFINITION OF STRUCTURE DEPENDENT DISPLACEMENT BASED INTENSITY MEASURE

The procedure presented for the IM herein is similar to the Capacity Spectrum Method (Freeman, 1985), where the main difference relates to the use of just the displacement spectrum. The structure is modelled as a SDOF and the demand to the system will change during its response to the ground motion, as a function of damage that reduces the stiffness and increases hysteretic damping within the structure.

As shown in Figure 4.1, the first step of the procedure consists in the definition of the yield displacement (Δ_y), the elastic period (T_y) and the ratio between elastic and the post-yielding stiffness (b) of the bridges. These parameters are defined by using the capacity curve obtained by Pushover Adaptive Analysis (Antoniou *et al.*, 2004).

Response displacement spectra are computed for a large number of earthquakes (i.e. 40 accelerograms) from the PEER database. The accelerograms are selected in order to take into account the possible variability in terms of magnitude, distance, duration and energy content.

The spectral displacement at the elastic period is then compared with the yield displacement in order to compare the demand and the capacity of the structure. If the spectral displacement does not exceed the yield displacement, the former is taken into account as the Intensity Measure and the algorithm stops. Otherwise the response spectrum is reduced by the reduction factor (Eqn.4.1) computed as a function of ductility (μ_i) and the equivalent damping (ξ_{eq}) (Priestley *et al.*, 2007).

$$\eta = \sqrt{\frac{0.07}{0.02 + \xi_{eq}}} \quad (4.1)$$

Hence the new spectral displacement at the post yield period (Eqn.4.2) is obtained and compared with the previous displacement.

$$T_i = \sqrt{\frac{\mu_i}{1 + b\mu_i - b}} \quad (4.2)$$

If the difference between the two quantities does not exceed a defined tolerance (i.e. 0.001) the displacement is taken into account as the IM otherwise the *damped simple iteration scheme* (De Vahl

Davis, 1986) is started until the difference in displacement between consecutive steps is within the aforementioned tolerance and thus the converged displacement demand is identified.

This procedure is repeated for each bridge, for each earthquake considered.

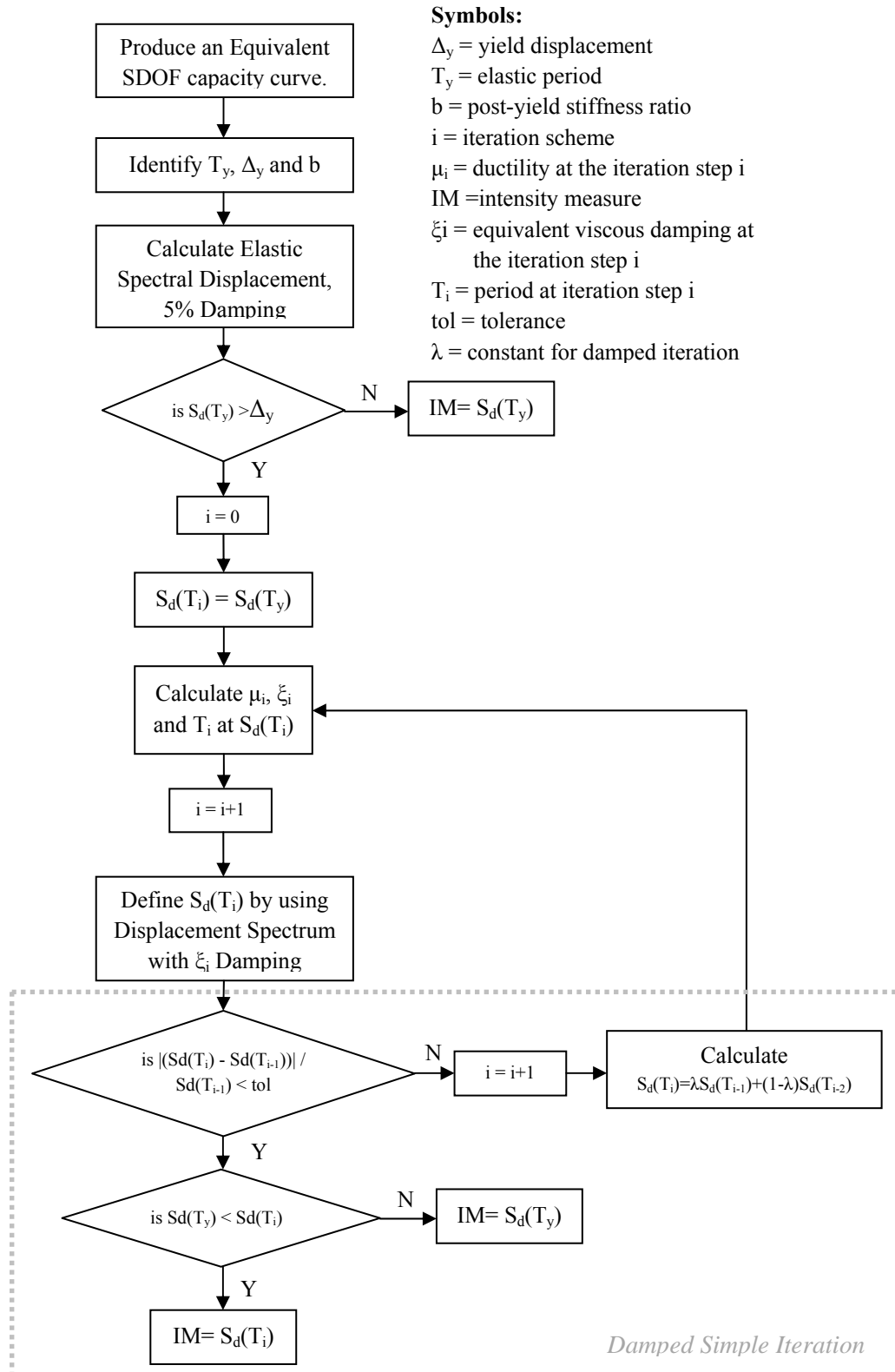


Figure 4.1. Flowchart of the Displacement Spectrum Method

The IMs obtained in this way are then compared with the all EDP values that result from the Time History Analyses; linear regression is carried on in order to obtain predictive equations for the response of the structure given the IM.

The form of the predictive equation is proposed in Eqn.4.3.

$$\log(EDP) = \log(aIM^b) + error \quad (4.3)$$

The error due to the parameters variability is considered as normal distributed with zero mean and standard deviation equal to the standard deviation of the logarithmic residuals. The logarithmic residuals are defined in Eqn.4.4.

$$\log(EDP) - \log(aIM^b) \quad (4.4)$$

5. COMPARISONS AND CONCLUSIONS

First of all a linear regression of data is made in order to evaluate the *efficiency, practicality, and proficiency* of the each IM (Padgett *et al.*, 2008).

An efficient IM is able to reduce the amount of variation in the estimated demand for a given IM value. A more efficient IM yields less dispersion (β) about the estimated median in the results of the non linear time history analysis. Practicality is the measure of the direct correlation between an IM and the demand placed on the structure and it is measured with the regression parameter b . When b approaches zero, the IM term contributes negligibly to the demand estimate. Proficiency measures the combined effect of efficiency and practicality and it is defined in Eqn.5.1. A more proficient IM corresponds to a lower value of ζ that represents the uncertainty introduced into the analyses by use of a particular IM.

$$\zeta = \frac{\beta}{b} \quad (5.1)$$

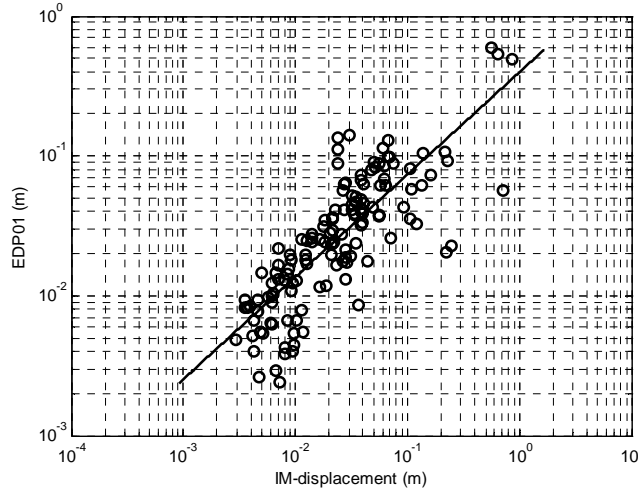


Figure 5.1. Structure dependent IM versus EDP01

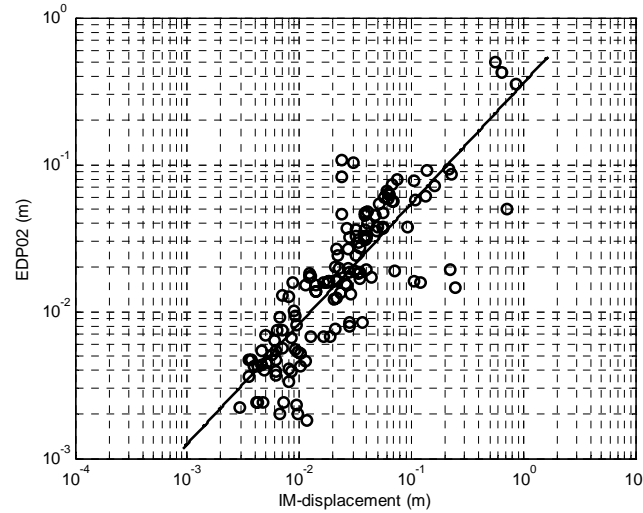


Figure 5.2. Structure dependent IM versus EDP02

Figure 5.1 and Figure 5.2 show the linear regression for the structure dependent IM proposed in this study versus EDP01 and EDP02. The results of the linear regression for the other common intensity measures (i.e. PGA, PGV, PGD, Spectral acceleration at different periods of vibration and Arias Intensity) are proposed in Table 5.1. The structure dependent IM is shown in the table as IM*.

Table 5.1. Linear Regression Parameters

PGA	b	a	β	ζ
EDP01	0.88	-1.85	0.77	0.87
EDP02	0.99	-2.06	0.77	0.76
EDP03	1.16	-1.92	0.80	0.69
EDP04	1.20	-2.17	1.00	0.83
Sa(0.2)	b	a	β	ζ
EDP01	0.71	-2.71	0.82	1.15
EDP02	0.79	-3.03	0.86	1.07
EDP03	0.92	-3.08	0.84	0.91
EDP04	0.94	-3.35	1.08	1.15
Sa(0.6 sec)	b	a	β	ζ
EDP01	0.84	-2.17	0.47	0.56
EDP02	0.92	-2.44	0.45	0.49
EDP03	0.98	-2.54	0.50	0.51
EDP04	0.94	-2.87	0.87	0.92
Sa(1 sec)	b	a	β	ζ
EDP01	0.73	-1.96	0.50	0.69
EDP02	0.80	-2.23	0.50	0.63
EDP03	0.84	-2.33	0.51	0.61
EDP04	0.83	-2.67	0.91	1.10
PGV	b	a	β	ζ
EDP01	0.94	-1.70	0.60	0.64
EDP02	1.03	-1.92	0.60	0.58
EDP03	1.27	-1.94	0.88	0.69
EDP04	1.12	-2.26	0.92	0.82
PGD	b	a	β	ζ
EDP01	0.52	-2.30	0.86	1.67
EDP02	0.56	-2.61	0.91	1.62
EDP03	0.60	-2.73	1.18	1.98
EDP04	0.54	-3.11	1.22	2.24
Arias Intensity	b	a	β	ζ
EDP01	0.48	-2.94	0.73	1.52

EDP02	0.53	-3.29	0.76	1.42
EDP03	0.59	-3.41	1.00	1.68
EDP04	0.60	-3.71	1.03	1.72
IM*	b	a	β	ζ
EDP01	0.86	-0.27	0.59	0.69
EDP02	0.98	-0.33	0.59	0.60
EDP03	1.10	-0.05	0.83	0.75
EDP04	1.09	-0.40	0.88	0.81

As shown in Table 5.1, EDP03 and EDP04 lead to higher values of dispersion (β) for all the Intensity Measures investigated. The IM related to these two EDPs, in most cases, are not giving acceptable results in terms of efficiency and proficiency (i.e. β and ζ are close or larger than 1), but they often lead to good levels of practicality.

Considering EDP01 and EDP02, structure dependent intensity leads to lower values of dispersion (β) if compared with the other intensity measures. The best results in given by spectral acceleration at a period equal to 0.6s and 1.0s (i.e. lower and upper bounds for the range of periods defined in this study).

Looking at the definition of practicality given by Padgett *et al.*(2008) the best measure is given by the PGA and PGV. The purpose of this study was to investigate the potentiality of a structure-based IM. The proposed IM seems to give comparable results compared with the other ones, though the increased complexity of this IM is perhaps not justified.

The quite high dispersion obtained from the linear regression needs to be reduced with the introduction of a better EDP, ensuring always that the EDP is well correlated with damage. Future developments will be carried out in this direction.

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REFERENCES

- Antoniou, S. and Pinho, R. (2004). Development and Verification of a Displacement-Based Adaptive Pushover Procedure, *Journal of Earthquake Engineer* **8**,643-661.
- Bal, E I., Crowley, H. and Pinho, R. (2008). Displacement-Based Earthquake Loss Assessment for an Earthquake Scenario in Instambul. *Journal of Earthquake Engineer* **12:1**,12-22.
- Casarotti, C., Pinho, R.(2007). An Adaptive Capacity Method for Assessment of Bridges Subjected to Earthquake Action. *Bulletin of Earthquake Engineering* **5**: 377-390
- Casarotti, C., Pinho, R. and Calvi, M.J.(2005). Adaptive push over based methods for seismic assessment and design of bridge structures.IUSS Press, Pavia.
- De Vahl Davis, G. (1986). Numerical Methods in Engineering and Science. Allen and Unwind Ltd, London, UK.
- Freeman S.A. (1998). Development and Use of the Capacity Spectrum Method. *Proceeding of the 6th US National Conference on Earthquake Engineering, Seattle*. CD ROM, EERI, Oakland.
- Priestley, M.J.N., Calvi, G.M.and Kowalsky, M.J. (2007). Displacement-Based Design of Structures.IUSS Press, Pavia, Italy.
- Priestley, M.J.N., Calvi, G.M.and Seible, F. (1996). Seismic Design and Retrofit of Bridges. Wiley-Interscience.West Sussex, U.K.