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Evaluation of Iterative DBD procedures for bridges

LESSLOSS Project / Sub-Project 8: Deliverable 72 – Displacement-Based Design Methodologies

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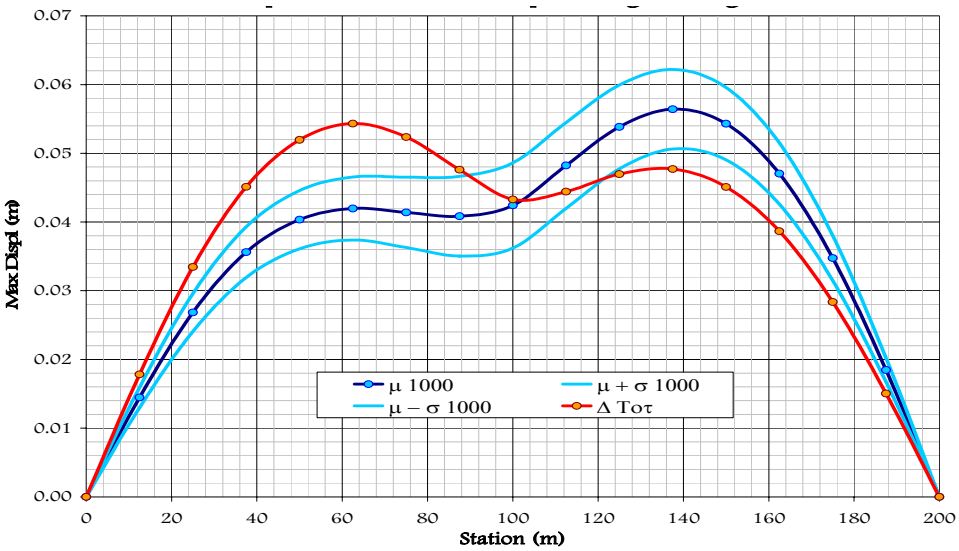

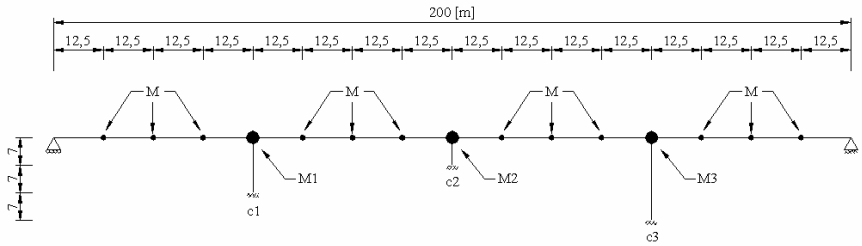

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EXECUTIVE SUMMARY

This report addresses the problem of the Displacement Based Evaluation/Design (DBE/D) of reinforced concrete bridges. Throughout the report, the aims and limitations of current seismic evaluation and design practice and the tendencies of the displacement-based seismic evaluation/design are discussed.

It presents a state-of-the-art review on the most important results and lessons derived from previous works, and based on them, two evaluation/design methods consistent with the performance-based seismic design philosophy are presented.

The first proposed method closely follows current linear displacement based procedures with improvements in the way the equivalent viscous damping and stiffness of the pier sections of a bridge are evaluated. This method takes into consideration the contribution of higher modes of vibration by using, for the calculation of performance, the complete substitute structure instead of an "equivalent" Single Degree Of Freedom (SDOF) system.

The other method, evolution of a previously developed one based on the capacity curve, considers in a direct way the non-linear behaviour of the piers by calculating the non-linear capacity curve of the structure and deriving from it the response curve of a reference SDOF from which the overall performance of the structure is determined.

For both methods the performance indices are assumed to be closely related to pier displacements; however due to the difficulties found in proving this assumption, this topic is still under investigation under the umbrella of this project. It is shown that in both methods proposed, the design approach develops as an inverse operation of the evaluation approach.

To illustrate the application of the DBE/D methods, an extensive numerical activity was carried out by applying the evaluation approach to six typical reinforced concrete multi-span bridge structures designed in accordance with Eurocode 8. For comparison purposes, results of non-linear step by step analyses of the chosen examples are also presented. The theoretical consistency and potential of the methods proposed to produce correct performances and design objectives are discussed. It is shown that both give good approximations for the so called "regular" bridge cases and not so good, and even incorrect results, for "irregular" bridges. It is concluded that the condition defining the boundary between the regularity and irregularity is a function not only of the geometric and design characteristics of a bridge but also of the characteristics of the seismic demand under which the structure is evaluated/designed.

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LIST OF SYMBOLS AND ABBREVIATIONS

Δ_{MAX}	= Maximum Lateral Displacement
W_i^j	= Weighting Factor
$\xi_{eq,i}$	= Equivalent Viscous Damping Ratio for Element i-th
ξ_j	= Viscous Damping Ratio for mode j-th
ξ_0	= Initial Viscous Damping
ξ_{eq}	= Equivalent Viscous Damping
T_{eff}	= Effective Stiffness
r	= Post Yielding to Initial Stiffness Ratio
μ	= Ductility Ratio
T_i	= Initial Period
α	= Unloading Stiffness Factor
β	= Reloading Stiffness Factor
a	= Behaviour Model Parameter
b	= Behaviour Model Parameter
R	= Strength Ratio
Sf	= Scale Factor
M_{res}	= Residual Moment
M_{int}	= Internal Moment
M_y	= Yield Moment
M_{ac}	= Accumulated Internal Moment
Sa	= Pseudo-Spectral Acceleration
ΔSa	= Scaled Pseudo-Spectral Acceleration
Sd	= Elastic Spectral Displacement
ΔSd	= Scaled Spectral Displacement
M_{GL}	= Internal Moment Due to Gravity Loads
Sd^*	= Performance Spectral Displacement
T_c	= Characteristic Soil Period
S_d	= Inelastic Spectral Displacement

R = Ductile Capacity

q_u = Load Reduction Factor

F_y = Strength

m = Mass of the SDOF System

r_{ij} = Performance Index for i-th Element and j-th Analysis

r_i = Performance Index for i-th Element

1. INTRODUCTION

The occurrence of destructive earthquakes in recent years all around the world, *e.g.,* Loma Prieta (1989), Northridge (1994), Kobe (1995), Turkey (1999) and Taiwan (1999), has made evident that seismic design methods proposed and used by current codes do not always provide the safety levels and performances expected when structures are subjected to design demands. The fact that these earthquakes have been particularly damaging to bridges has caused structural design professionals to question the validity of the procedures of seismic design recommended by current codes. Current codes (*e.g.* [AASHTO, 1992, 1998], [ATC, 1996a, 1996b, 2001a], [ATC/MCEER, 2002], [Caltrans, 1994], [Mast *et al.*, 1996 a and b] base their recommendations on a design philosophy which accepts that that the seismic design of bridges may be done with design forces derived from design spectra reduced from the real elastic to consider, among other aspects, the over-strength implicit in the design equations and factors which take into account the inelastic behaviour of the structure implicitly allowed to develop when different levels of damage are accepted. Unfortunately, with the designs produced with these codes it is in general not possible to guarantee that a structure has a performance that fulfils the expected design objectives. This situation makes evident the necessity of using alternative procedures of seismic design which guarantee structures with performances in agreement with those expected when designed.

During the last decade there has been an increasing pressure from owners, insurance companies, politicians and engineers to re-evaluate and improve the state of practice of seismic design to meet the challenge of reducing life losses and the huge economic impact caused by recent earthquakes, which by no means could be considered as unusual or rare. As a result of this pressure, different research groups have reinitiated the investigations on the concepts and procedures for the performance based seismic evaluation and design of structures [Hamburger, 1996].

The idea that gave origin to the present work is based on the fact that there is a number of apparently different evaluation and design methods with which it is possible, in a simplified manner, to obtain the performance of a structure in the evaluation of existent bridges, or to guarantee the design objectives when a new one is designed, and that these methods should be further investigated and, if necessary, improved to guarantee their validity for a successful application in practice. Based on previous work developed within the group responsible of this report, the premise of this investigation is that, regardless of the approximations involved in the different methods considered, the approach used for the evaluation and the design of structures may be considered as only one which, for the evaluation process considers as known the design of the structure and the seismic demand for which it needs to be evaluated, and as unknown the performance of the structure under design actions, while for the design process considers as known the target performance levels and the seismic demands and, as unknown the design parameters which guarantee such performance levels. Within this framework, it is the purpose of this work to carry out a critical review of a particular class of performance based evaluation/design methods based on displacements and to propose new alternatives which correct some of the deficiencies of existent.

All methods considered in this investigation have as theoretical foundations the concepts of structural dynamics approximated to systems with non-linear behaviour [Chopra, 2000], which

allow, in a simple and direct way, the calculation of performances in the case of evaluation and of the correct design forces which guarantee the seismic performance objectives.

In the first method considered, the original structure is substituted by a reference linear elastic structure with elements with reduced stiffness and energy dissipation characteristics consequent with the obtained expected performance levels. This method, iterative in nature, involves the reduction of the substitute structure to an, incorrectly termed, "equivalent" SDOF system from which performance, the evaluation or the design conditions for the complete structure may be found.

The second method, also iterative, is non-linear in nature and considers as basic assumption that the performance of the complete structure, generally expressed in terms of a modal index, *e.g.*, modal ductility, may be approximately transformed to a local ductility, and that the non-linear performance of the structure may be approximately related to that of a reference non-linear SDOF system with a response curve directly derived from the non-linear capacity of the structure.

Regarding the participation to performance of higher modes, most existent methods either neglect this participation or include it in a heuristic and somehow arbitrary way.

Recognizing, based on published results, that under certain circumstances all existent methods fail to give acceptable results, the authors of this report suggest, for both methods considered, that there exists a regularity condition, not only related to geometric considerations but to more general structural and seismic demand characteristics, beyond which the obtained evaluation or design results become incorrect. The investigation involved in the formulation of the two methods proposed has tried with limited success to eliminate this drawback; nevertheless this problem is currently under investigation. To illustrate the application and potential of both methods, six different bridges, considering regular and irregular cases, are evaluated.

2. DIFFERENCES BETWEEN BRIDGES AND BUILDINGS

Most research work on the performance based evaluation and design of structures has been concentrated in building structures with only few authors interested in bridges. Unfortunately, the research results of all these efforts cannot be directly extrapolated to bridge structures, as some important differences exist between the structural characteristics of both types of structures which affect their response when subjected to earthquake actions. The main differences between buildings and bridges which influence the applicability of seismic performance based evaluation/design methods are:

i) Buildings have rigid floor diaphragms, whereas bridges have a flexible superstructure normally assumed as elastic and which controls the deformation of the piers.

ii) Buildings have generally columns of equal height uniformly distributed in each floor, whereas bridges have piers of different heights located in the same plane, differing from the spatial distribution of columns in buildings.

iii) The performance of buildings may be approximately inferred from the displacement of a characteristic point, whereas the performance of a bridge may not.

3. PREVIOUS WORK

Since the beginning of the last decade it has been recognized by authors like Moehle [1992] and Priestley [1993, 2003] that current methodologies of earthquake resistant design of structures based on forces and strengths do not agree with the seismic behaviour observed in real reinforced concrete structures, and that it would be much better to use design methodologies based directly on displacements and deformations and/or other valid seismic performance indices. In accordance with this position, in recent years there have been significant advances in the development of design procedures based on performance having as main objective their incorporation in future design codes. In this context, Moehle [1992, 1996] proposed a general framework for earthquake resistant design of structures based on drift displacements with the seismic demand given by displacement response spectra. This methodology, however, begins by obtaining the stiffness, elastic periods and the strength of the structure, from a direct check of the displacements, instead of an indirect one through the conventional ductility-based factors.

The procedure for the displacement based design of SDOF systems or systems which may be reduced to "equivalent" linear SDOF systems, proposed by Priestley [1993, 2003], Kowalsky *et al* [1995, 1997], Priestley [2000] and Kowalsky [2002], starts from a target design displacement, based on a deformation capacity guaranteed by an appropriate detailing of the structure. Assuming a reasonable value for the yielding displacement, the peak displacement becomes a displacement ductility demand, and starting with this demand and with a set of response displacement spectra, for an equivalent damping ratio which includes the inherent viscous damping characteristics of the structure and that required to consider the energy dissipated by the system through non linear hysteretic behaviour, the effective period of an equivalent linear viscoelastic SDOF system corresponding to the peak displacement is determined. The final result of this process is the required yielding strength determined from the peak displacement and the secant stiffness corresponding to the effective period. Calvi and Kingsley [1995] extended this methodology to Multiple Degree Of Freedom (MDOF) structures which may be transformed to an equivalent SDOF system using an assumed deformed configuration of the structure. For buildings it is proposed that the assumed deformed configuration is that corresponding to a predefined plastic mechanism. The final result of this alternative is the required strength that should be given to the structure to attain the objective performance. An alternative evaluation method described in documents such as FEMA-273/274 [FEMA, 1997a and b], is based on an elastic approximation of maximum displacement multiplied by a set of modification factors to arrive at an estimate of the target inelastic displacement.

An alternative approach to the performance based evaluation/design of structures is based on the use of non-linear static analysis procedures (pushover like analyses) to include, in a simple method, the most important features which influence performance [Freeman, 1978 and 1994], [Fajfar, 1999]. Examples of the methods which use a single mode approximation are described in documents such as the ATC-40 [ATC, 1996a] and Eurocode 8 [CEN, 2003a].

An improvement of the single mode approximation is to include the contribution of higher modes into the forces used for pushover. Relevant formulations of this multimode approximation are in De Rue [1998], Requena and Ayala [2000] and Gupta and Kunnath [2000]. Even though the application of modal spectrum analysis in the inelastic domain to define the distribution of lateral forces used to determine the capacity curve of the structure is theoretically inconsistent, the reported results from this approach show an acceptable approximation.

Two more recent approximate methods based on the combination of modal responses are the modal pushover analysis and Incremental Response Spectrum Analysis (IRSA).

The modal pushover analysis method was originally presented by Paret *et al.* [1996] and Sasaki *et al.* [1998] and later on improved and used by Chopra and Goel [2002]. In this method, a pushover analysis is carried out separately for each of the participating modes and the modal contributions to the performance of the structure are added together using a combination rule. All the important modes, identified in the initial - elastic state, are used separately to determine the distribution of forces for the pushover analyses, *i.e.,* the number of analyses is equal to the number of important modes in the elastic state. When all the analyses are completed, the results are combined using a modal combination rule. The method assumes that the mode shapes are not changing during the response.

The IRSA method was originally developed as an approximated step-by-step piecewise dynamic modal analysis for non-linear structures and then conveniently simplified for practical applications using smooth response spectrum [Aydinoğlu, 2003, 2004]. The method takes into account the influence of all important modes and the changes in the dynamic properties of the structure every time a plastic hinge occurs. Modal capacity diagrams for each important mode are constructed through modal analyses. To calculate the performance of the structure the procedure uses a modal combination rule with previously scaled modal responses according to some "inter-modal scale factors". In the practical version of the method these factors are simplified as constant each time a sequential modal spectrum analysis is carried out. In the method, once the modal capacity curves are defined, the modal performance of the structure is obtained by using, for each mode, the equal displacement rule with the consideration of the short period correction. To obtain the global performance of the structure an accepted modal combination rule is used. This method is different to pushover-based procedures as equivalent static forces are never applied to construct the modal capacity diagrams. Instead, the method uses displacements derived from consecutive modal analysis to obtain the different segments of the modal capacity diagrams corresponding to different performance stages.

More recently, the ATC-55 Project, which has been published as FEMA-440 [ATC, 2001b and 2005], has a detailed description of practical methods for the seismic evaluation of buildings.

From the detailed study of the existing procedures based on the non-linear capacity of the structures it has been found that, in general, all involve the following two tasks:

1. Determination of the deformation capacity of the structure and its corresponding strength for the sequential formation of the events associated to predefined limits states (*e.g.,* distribution of plastic hinges, maximum displacements, etc.) and the corresponding redistribution of the seismic forces which act on the structure.

2. Determination of the seismic performance using displacement/acceleration design spectra; considering SDOF systems (one or several systems, depending on the method) whose non-linear force-displacement relationships are the result of step 1. The use of smooth spectrum produces, for evaluation purposes, the maximum displacement, *i.e.,* the displacement demand for a given design, and for design purposes, the strength demand for a required displacement.

Based on the same concepts which support the two previous tasks, a performance evaluation/design method is proposed. This method is based on the procedure used by Alba *et al.* [2005] and on the approximation of non-linear analysis for the evaluation of buildings proposed by Requena and Ayala [2000]. This method has its foundation in the fact that the response curve of a reference SDOF system derived from the capacity curve of a structure, obtained from sequential modal spectral analyses, may be used to approximate the performance. The maximum

displacement of this reference system is obtained from one of the different variations of the equal displacement rule, *e.g.*, Fajfar and Gaspersic [1996 and 1998] and Ruiz-Garcia and Miranda [2004], and directly transformed to the maximum displacement of the structure by an *ad hoc* modal spectral analysis. The proposed method is similar to the IRSA method, however, they differ in the way in which higher modes are considered. The details of this method are further discussed later on in this report when one variant of it is proposed as a displacement based evaluation/design method for bridges.

4. METHOD BASED ON THE SUBSTITUTE STRUCTURE

4.1 THE SUBSTITUTE STRUCTURE

One of the most popular methods used for the displacement based evaluation/design of bridges is one in which the original structure is substituted by a linear viscoelastic counterpart, *e.g*, Kowalsky [2002]. This "substitute" structure has the same configuration as the original, with equivalent stiffness and damping properties for the elements where damage is assumed to occur under design conditions or where it actually occurs in design evaluation applications.

The concept of introducing viscous damping to represent energy dissipation characteristics of a system was first presented by Jacobsen [1960]. However, the first known earthquake engineering application of this idea to approximately substitute a hysteretic SDOF system subjected to earthquake action by a viscoelastic one was investigated by Rosenblueth and Herrera [1964], Jennings [1968], Iwan and Gates [1979] and Iwan [1980].

For the assessment of real structures, Gulkan and Sozen [1974] introduced formally the concept of substitute structure for a SDOF structure comparing the resulting analytical results with the corresponding experimental. Later on, Shibata and Sozen [1976] extended this formulation to MDOF systems by proposing an approximation to define the modal damping ratio of the whole structure as a weighted average of the element damping ratios. In this approximation, once the equivalent linear stiffness of the elements and the modal damping ratio of the structure are determined, modal spectral analysis may be used to approximately evaluate its seismic performance.

Recent papers by Blandon and Priestley [2005] and Dwairi [2004], among other authors, present a thorough list of different definitions of equivalent viscous damping, ξ_{eq}, and where applicable, effective periods, T_{eff}. In what follows, for the sake of completeness, these and some other definitions available in the specialized literature are presented, considering that for the cases where only the definition of equivalent viscous damping ratio is given, the corresponding effective period is that calculated from the secant stiffness to maximum displacement. These definitions are:

For the bilinear idealization of the response curve, [Blandon and Priestley, 2005]:

$$\xi_{eq} = \xi_0 + \frac{2}{\pi}\left[\frac{(1-r)(\mu-1)}{\mu - r\mu + r\mu^2}\right] \tag{3.1}$$

where ξ_o is the initial viscous damping, r is the post yielding to initial stiffness ratio, μ is the ductility ratio.

Based on a simple Takeda model, Gulkan and Sozen [1974] proposed:

$$\xi_{eq} = \xi_0 + 0.2\left(1 - \frac{1}{\sqrt{\mu}}\right) \tag{3.2}$$

Iwan [1980], using an error minimization procedure in a hysteretic model derived from a combination of elastic and slip elements, proposed:

$$\xi_{eq} = \xi_0 + 0.0587(\mu - 1)^{0.371} \tag{3.3}$$

and an effective stiffness derived from:

$$\frac{T_{eff}}{T_i} = 1 + 0.121(\mu - 1)^{0.939} \tag{3.4}$$

where T_i is the initial period.

For a general Takeda model, Loading et al. [1998] found that:

$$\begin{aligned}\xi_{eq} = \xi_0 + \frac{2}{\pi}\Bigg\{ & 1 - \frac{3}{4}\mu^{\alpha-1} - \frac{1}{4}\left[\frac{r\beta\mu}{\gamma}\left(1-\frac{1}{\mu}\right)+1\right]\left[2-\beta\cdot\left(1-\frac{1}{\mu}\right)\right] \\ & -\mu^{\alpha-1}\gamma - \frac{1}{4}\left[\frac{r\beta^2\mu}{\gamma}\left(1-\frac{1}{\mu}\right)^2\right]\Bigg\}\end{aligned} \tag{3.5}$$

where $\gamma = r\mu - r + 1$, α is the unloading stiffness factor and β is the reloading stiffness factor. The effective period used corresponds to the secant to maximum displacement.

For the Takeda model with model parameters α=0.5 and β=0.0, the above definition reduces to:

$$\xi_{eq} = \xi_0 + \frac{1}{\pi}\left(1 - \frac{1-r}{\sqrt{\mu}} - r\sqrt{\mu}\right) \tag{3.6}$$

and for zero post yielding stiffness:

$$\xi_{eq} = \xi_0 + \frac{1}{\pi}\left(1 - \frac{1}{\sqrt{\mu}}\right) \tag{3.7}$$

Priestley [1993, 2003], using and effective stiffness directly derived from the secant to maximum displacement, proposed for steel members the following expression:

$$\xi_{eq} = \xi_0 + \frac{1.50}{\pi}\left(1 - \frac{1}{\mu}\right) \tag{3.8}$$

for concrete frames:

$$\xi_{eq} = \xi_0 + \frac{1.20}{\pi}\left(1 - \frac{1}{\sqrt{\mu}}\right) \tag{3.9}$$

for concrete columns and walls:

$$\xi_{eq} = \xi_0 + \frac{0.95}{\pi}\left(1 - \frac{1}{\sqrt{\mu}}\right) \tag{3.10}$$

and, for precast walls or frames with unbounded presstressing:

$$\xi_{eq} = \xi_0 + \frac{0.25}{\pi}\left(1 - \frac{1}{\sqrt{\mu}}\right) \tag{3.11}$$

In general, for all models proposed by Priestley [1993, 2003], the equivalent viscous damping equation has the form:

$$\xi_{eq} = \xi_0 + a\left(1 - \frac{1}{\mu^b}\right) \tag{3.12}$$

where *a* and *b* are parameters defined by the behaviour model of the structural element.

Following the same approach as Iwan [1980], Kwan and Billington [2003] calculated an equivalent damping ratio and an effective period using, for six different hysteretic models, 20 records and a range of periods between 0.1 and 1.5 seconds:

$$\xi_{eq} = 0.352\mu\xi_0 + \frac{0.717}{\pi}\frac{\mu - 1}{\mu} \tag{3.13}$$

$$\frac{T_{eff}}{T_i} = 0.8\sqrt{\mu} \tag{3.14}$$

Considering that the original Jacobsen [1960] approach is strictly applicable to harmonic excitation, Dwairi [2004] presented additional empirical equations for the equivalent damping, which reflect the type of assumed hysteretic model and the characteristics of the earthquakes defining the seismic hazard at a particular site. The proposed equations are for precast unbounded columns, post tensioned masonry walls and isolation devices.

$$\begin{aligned} &\xi_{eq} = \xi_0 + C_{RS}\left(\frac{\mu - 1}{\pi\mu}\right) \\ &C_{RS} = 0.30 + 0.35\left(1 - T_{eff}\right) \qquad T_{eff} < 1\text{s} \\ &C_{RS} = 0.30 \qquad T_{eff} \geq 1\text{s} \end{aligned} \tag{3.15}$$

For reinforced concrete beams (Large Takeda):

$$\begin{aligned} &\xi_{eq} = \xi_0 + C_{LT}\left(\frac{\mu - 1}{\pi\mu}\right) \\ &C_{LT} = 0.65 + 0.50\left(1 - T_{eff}\right) \qquad T_{eff} < 1\text{s} \\ &C_{LT} = 0.65 \qquad T_{eff} \geq 1\text{s} \end{aligned} \tag{3.16}$$

For reinforced concrete columns and walls (small Takeda):

$$\begin{aligned} &\xi_{eq} = \xi_0 + C_{ST}\left(\frac{\mu - 1}{\pi\mu}\right) \\ &C_{ST} = 0.50 + 0.40\left(1 - T_{eff}\right) \qquad T_{eff} < 1\text{s} \\ &C_{ST} = 0.50 \qquad T_{eff} \geq 1\text{s} \end{aligned} \tag{3.17}$$

And for steel members (Elasto-Plastic):

$$\xi_{eq} = \xi_0 + C_{EP}\left(\frac{\mu - 1}{\pi\mu}\right)$$

$$C_{EP} = 0.85 + 0.60\left(1 - T_{eff}\right) \qquad T_{eff} < 1\text{s} \qquad (3.18)$$

$$C_{EP} = 0.85 \qquad T_{eff} \geq 1\text{s}$$

Based on 72 records, Miranda and Lin [2004] developed approximate equations for the equivalent damping ratio and for the effective period as a function of the strength ratio, R:

$$\xi_{eq} = \xi_0 + \left(\mathrm{R}^{1.8} - 1\right)\left(0.02 + \frac{0.002}{T_i^{2.4}}\right) \qquad (3.19)$$

$$\frac{T_{eff}}{T_i} = 1 + \left(\mathrm{R}^{1.8} - 1\right)\left(0.027 + \frac{0.01}{T_i^{1.6}}\right) \qquad (3.20)$$

$$\mathrm{R} = m\frac{S_a}{F_y} \qquad (3.21)$$

where m is the mass of the SDOF system, S_a is the pseudo spectral acceleration and F_y is the yield strength of the system.

By minimizing the error between approximate and "exact" responses, Guyader and Iwan [2006] proposed:

$$\xi_{eff} = \xi_0 + A(\mu - 1)^2 + B(\mu - 1)^3 \qquad \text{for } \mu < 4.0 \qquad (3.22)$$

$$\xi_{eff} = \xi_0 + C + D(\mu - 1) \qquad \text{for } 4.0 \leq \mu \leq 6.5 \qquad (3.23)$$

$$\frac{T_{eff}}{T_i} - 1 = G(\mu - 1)^2 + H(\mu - 1)^3 \qquad \text{for } \mu < 4.0 \qquad (3.24)$$

$$\frac{T_{eff}}{T_i} - 1 = I + J(\mu - 1) \qquad \text{for } 4.0 \leq \mu \leq 6.5 \qquad (3.25)$$

$$\xi_{eff} = \xi_0 + E\frac{F(\mu - 1) - 1}{\left[F(\mu - 1)\right]^2}\left(\frac{T_{eff}}{T_i}\right)^2 \qquad \text{for } \mu > 6.5 \qquad (3.26)$$

$$\frac{T_{eff}}{T_i} - 1 = K\left(\sqrt{\frac{(\mu - 1)}{1 + r\left[(\mu - 1) - 1\right]^2}} - 1\right) \qquad \text{for } \mu > 6.5 \qquad (3.27)$$

where A, B, C, D, E, F, H, J and K are the coefficients involved in the above equations for computing equivalent damping and effective period with r=0.05 for the bilinear (BLH), stiffness degrading (STDG), pinching 1 (PIN1) and pinching 2 (PIN2) models (Table 4.1).

Table 4.1. Coefficients for Effective Linear Parameters

Model	A	B	C	D	E	F	G	H	I	J	K
BLH	4.1504	-0.8260	10.1243	1.6428	22.35	0.40	0.1145	-0.0178	0.1777	0.1240	0.768
STDG	5.6420	-1.2962	10.1820	1.8661	19.51	0.38	0.1809	-0.0366	0.1472	0.1640	0.92
PIN1	3.3888	-0.7083	5.6711	1.9015	13.58	0.38	0.2034	-0.0417	0.1367	0.1898	1.05
PIN2	4.9926	-1.1225	9.3702	1.7518	18.16	0.40	0.1820	-0.0365	0.1704	0.1604	0.94

Comartin *et al.* [2004] proposed for FEMA440 [ATC, 2005] empirical expressions relating the equivalent damping ratio and effective period to the maximum ductility, μ, as:

$$\left.\begin{aligned} \xi_{eq} &= 4.85(\mu-1)^2 - 1.08(\mu-1)^3 + 5 \\ \frac{T_{eff}}{T_i} - 1 &= 0.167(\mu-1)^2 - 0.0310(\mu-1)^3 \end{aligned}\right\} \quad \text{for } \mu < 4.0 \qquad (3.28)$$

$$\left.\begin{aligned} \xi_{eq} &= 13.6 + 0.318(\mu-1) + 5 \\ \frac{T_{eff}}{T_i} - 1 &= 0.283 + 0.129(\mu-1) \end{aligned}\right\} \quad \text{for } 4.0 \le \mu \le 6.5 \qquad (3.29)$$

$$\left.\begin{aligned} \xi_{eq} &= 19.01\left[\frac{0.64(\mu-1)-1}{0.64(\mu-1)^2}\right]\left(\frac{T_{eff}}{T_i}\right)^2 + 5 \\ \frac{T_{eff}}{T_i} - 1 &= 0.89\left[\left(\frac{\mu-1}{1+0.5(\mu-1)-1}\right)^{0.5} - 1\right] \end{aligned}\right\} \quad \text{for } \mu > 6.5 \qquad (3.30)$$

4.2 THE SUBSTITUTE STRUCTURE APPLIED TO THE DISPLACEMENT BASED EVALUATION OF BRIDGES

In this section it is assumed that for evaluation purposes the bridge structure under consideration is already designed and that the substitute structure method is used to assess its seismic performance when subjected to a seismic demand given by a design spectrum.

The steps involved in the evaluation version of the method are:

Step-1: Determination of the inelastic behaviour of the pier sections, as moment *vs.* curvature, within the potential damaged region when subjected to increasing cyclic curvature. A procedure to find this behaviour is proposed and exemplified in Deliverable 69 of the LESSLOSS project.

Step-2: Determination of the load-displacement characteristics at the top of the piers. Based on the moment *vs.* curvature curves determined in step 1 and on an assumption for the length of the plastic hinge, load-displacement curves for the top of the piers, considering the different maximum lateral displacement (ductility) levels, are constructed.

Step-3: Determination of equivalent linear viscoelastic properties of the piers. Based on the nonlinear force *vs.* displacement curves of the piers determined in step 2, the equivalent linear viscoelastic properties of the piers, *e.g.,* secant stiffness, K_{eff}, and equivalent viscous damping ratio, ξ_{eq}, at maximum displacement, are calculated.

Step-4: Construction of the curves for each pier depicting the variation of the equivalent stiffness and damping ratio in terms of displacement ductility. To consider the transient nature of the earthquake action in the equivalent properties curves, it is necessary to include a modification factor that takes into account the fact that the maximum displacement during an earthquake occurs only a very limited number of times, *e.g.*, for narrow band records, equivalent properties associated to the maximum displacement multiplied by a factor equal to 0.67 have shown to be a good approximation. These curves are schematically illustrated in Figure 1a and b.

Step-5: Initiation of the iterative procedure for performance determination. Since the equivalent viscoelastic properties of the piers are functions of the associated maximum displacement, it is required to initially assume a distribution of maximum displacements under design conditions. A simple way to obtain this distribution of maximum displacements is to carry out a modal spectral analysis considering for the piers the initial stiffness and the inherent modal viscous damping for this type of structures.

Step-6: Determination of the nonlinear performance of the bridge using an iterative procedure. Once the initial performance is assumed, the viscoelastic properties of the piers are defined using the considered maximum displacements and a modal damping ratio defined as the sum of the inherent modal damping, ξ_o, and that corresponding to the weighted average of the hysteretic damping ratio for all the structural elements, as suggested by Shibata and Sozen [1976] and used in the Direct Displacement Based Design (DDBD) of bridges by Kowalsky [2002]. According to the original approach of Shibata and Sozen [1976], the equivalent modal viscous damping ratio ξ_j at the bridge level can be evaluated for mode *j* as:

$$\xi_j = \frac{\sum_i W_i^j \cdot \xi_{eq,i}}{\sum_i W_i^j} \qquad (3.31)$$

where W_i^j is the weighting factor for each pier, defined as the elastic energy stored at element *i* for mode *j*.

When energy dissipation devices are included at the top of the piers, the additional damping introduced should be added to the modal damping previously defined in Equation (4.31).

Step-7: Comparison of the updated and previous performances. When, during the iteration process, the updated and the previous performances are close enough, *i.e.*, the maximum differences between the displacement configuration of the bridge do not exceed a given value, the process is stopped, otherwise steps 6 and 7 are repeated, updating the last performance obtained to be the initial.

The steps involved in the above described procedure are illustrated in Figure 2.

For the application of the concept of the substitute structure to the displacement based evaluation/design of bridges it is necessary to know, for a variety of pier designs, the equations or graphs defining the corresponding equivalent viscous damping ratios and the reduction factors of the initial stiffness to be considered in the equations defining the effective periods. For these two related equivalent properties of the SDOF system used by most authors to evaluate or design a structure based on displacements, there still exists a strong tendency to show that the existing methods described in Section 3 are different, although all of them, in principle, are similar, differing only in the way in which they define the effective period of the structure and the corresponding equivalent viscous damping ratio.

4.3 SUBSTITUTE STRUCTURE APPLIED TO DISPLACEMENT BASED DESIGN OF BRIDGES

A similar procedure to that described above for evaluation may be used for the DDBD of bridge structures. The design procedure proposed in this report is derived following similar steps to those presented in the above section and it is different to that presented by Kowalsky [2002] inasmuch as it includes information about a target damaged distribution under design conditions, includes participation of all contributing modes and use relations between the inelastic deformation at the top of the piers *vs.* local curvature demands at the hinges at the base of damage piers as basic design information.

To apply this procedure it is necessary to have, for different pier geometries and acceptable design configurations, the corresponding design curves illustrated in Figure 1a and b. This procedure is schematically described in Figure 3.

5. METHOD BASED ON THE NON-LINEAR CAPACITY OF THE STRUCTURE

The proposed evaluation/design method is based on the same hypotheses and considerations as the method developed by Ayala [2001], which explicitly considers the non-linear behaviour of the structure on the derivation/postulation of a target response curve of a reference SDOF system considering the participation of all modes to determine the performance of the otherwise MDOF of the structure under evaluation/design. The characteristics of the response curve of the reference SDOF system are obtained from the calculated/desired distributions of damage for the considered design objective. In this method, the design seismic demands associated to each of the design objectives are concurrently determined using the characteristics of the calculated/assumed response curve of the reference SDOF. When applied to buildings, the design version of this method produces design forces which have led, with good approximation, to the considered distribution of damage and the level of predefined seismic performance.

A key question in the application of displacement-based evaluation/design methods to MDOF structures is how to transform the global performance into demands of local inelastic deformation in the individual structural members. In this respect, detailed procedures intended to achieve this purpose are, for example, those proposed by Krawinkler and Seneviratna [1995 and 1998] and Seneviratna and Krawinkler [1997], however a definite solution to this problem has not been established and it is still the topic of current investigations.

5.1 NON-LINEAR CAPACITY CONCEPT APPLIED TO THE DISPLACEMENT BASED EVALUATION OF BRIDGES

The application of the proposed method involves the following steps:

Step 1. The seismic demand is defined by a smooth response spectrum corresponding to a chosen seismic demand level. This definition has some advantages as this is the normal way in which seismic design is presented in most current design codes. This spectrum represents the envelope of the response spectra from many seismic records of various intensities, generated at different sources.

Step 2. The response curve of the reference SDOF system is obtained through a series of Modal Spectral Analyses (MSA), considering as many damage stages as necessary, developed by a structure until its maximum capacity is reached. The contribution of higher modes of vibration in the response curve is taken into account using a modal combination rule (*e.g.,* SRSS or CQC). In this work, a damage stage is defined every time a plastic hinge is formed at the end section of a pier. A flowchart for the construction of the response curve is schematically presented in Figure 4.

Step 3. Once the MSA "*j*" is performed, the corresponding scale factor, $Sf_{(j)}$, is calculated at each end element using Equation (4.1) and the relationship between the internal moments, obtained from the MSA, and the residual moment M_{res} as defined in Equation (4.2).

$$Sf_{(j)} = \frac{M_{res(j)}}{M_{\text{int}}} \tag{4.1}$$

$$M_{res(j)} = M_y - M_{ac(j)} \tag{4.2}$$

where, for each end element, M_y is the yield moment of the section defined in the elastoplastic idealization of the moment curvature diagram and M_{ac} is the accumulated internal moment, as defined by Equations (4.3) and (4.4).

For the first point of the response curve:

$$M_{ac(1)} = M_{GL} \tag{4.3}$$

where M_{GL} is the internal moment due to gravity loads.

For the remaining points of the response curve:

$$M_{ac(j)} = M_{ac(j-1)} + M_{\text{int}(j-1)} \cdot Sf_{(j-1)} \tag{4.4}$$

The lowest scale factor corresponds to the end element requiring the lowest seismic demand to yield.

Step 4. The scaled pseudo-acceleration, ΔSa, and the scaled spectral displacement, ΔSd, corresponding to the period of the dominant mode of the structure in the damage stage j, are defined from the scaled spectrum, using the acceleration *vs.* displacement format, ADRS, which is the same format in which the response curve is defined. In this representation the coordinates of point "j" of the response curve are defined by Equations (4.5) and (4.6).

$$Sa_{(j)} = Sa_{(j-1)} + \Delta Sa_j \tag{4.5}$$

$$Sd_{(j)} = Sd_{(j-1)} + \Delta Sd_j \tag{4.6}$$

Step 5. The capacity of the structure is reached when a local or global instability occurs, indicating that the construction of the response curve is finished and that the methodology for the evaluation of the target spectral displacement may be continued. Otherwise, a new damage stage has to be considered and a new MSA performed for the determination of the next point on the response curve.

Step 6. The inelastic displacement demand, or performance spectral displacement, Sd^*, may be calculated using the equal displacement rule [Veletsos and Newmak, 1966], with proper consideration of its correction for short periods [Fajfar and Gaspersic, 1996 and 1998], [Ruiz-Garcia and Miranda, 2004] . This rule has the following characteristics:

- It has been thoroughly evaluated for elastoplastic SDOF systems.
- For elastoplastic systems with periods larger than the characteristic soil period T_c, the maximum relative displacement approximately equals that of their corresponding elastic system.
- For the approximation proposed by [Fajfar and Gaspersic, 1996 and 1998] the short period correction is only valid for firm soil conditions.
- As this rule and its correction are used in the method, the following considerations must be stipulated: a) when smooth response spectra are used, the equal displacement rule is the only available solution to the problem of the determination of inelastic displacements and b) to apply this rule, the response curve of a reference SDOF system needs to be constructed.

For the majority of large bridges, the initial period of the most relevant mode, T, will be, in general, larger than the characteristic soil period, T_c. Thus, the above mentioned short period

correction is not necessary, and the target spectral displacement is approximately equal to the elastic spectral displacement, *Sd*.

Furthermore, if the fundamental period of the structure is smaller than the characteristic soil period, the target spectral displacement has to be calculated using the short period correction as the specified by Equations (4.7) to (4.9), obtained from Annex B of EC 8, [CEN, 2003]:

$$q_u = \frac{Sa \cdot m}{F_y} \tag{4.7}$$

$$\mathrm{R} = \left(q_u - 1\right)\frac{T_c}{T} + 1 \tag{4.8}$$

$$Sd^* = \frac{Sd}{q_u}\mathrm{R} \tag{4.9}$$

where R is the ductile capacity, q_u the load reduction factor, *Sa* the elastic pseudo-acceleration and F_y the strength.

Step 7. When the available capacity of the structure exceeds the demand, a new scale factor, Sf_N, needs to be calculated for the first point ("point *j*") of the response curve where the displacement is larger than the target displacement. This is done with Equations (4.10) and (4.11).

$$\Delta Sd_N = Sd^* - Sd_{(j-1)} \tag{4.10}$$

$$Sf_N = \Delta Sd_N \frac{Sf_j}{\Delta Sd_j} \tag{4.11}$$

The seismic performance of the bridge for the selected performance parameter, in this case the maximum lateral pier displacement, is finally calculated as the sum of the corresponding parameters for the *N* modal spectral analyses performed until the target performance displacement is reached, each multiplied by its scale factor, as expressed in Equation (4.12).

$$r_i = \sum_{j=1}^{N}\left(r_{ij} \cdot Sf_j\right) \tag{4.12}$$

Every step involved in this method is schematically illustrated in Figure 5.

5.2 NON-LINEAR CAPACITY CONCEPT APPLIED TO DISPLACEMENT BASED DESIGN OF BRIDGES

The overall design process for a performance level defined by a target design ductility consists of the following steps:

Step 1. The response curve of a reference system corresponding to the mode of the structure with the highest contribution to the response is constructed by considering two structures with different dynamic properties: one with properties derived from the bridge without damage corresponding to a pre-designed structure; the other, the same bridge with reduced properties to incorporate a proposed damage distribution expected to occur under design demands.

Step 2. The distribution of the global lateral strength of the bridge is carried out by means of MSAs corresponding to the two performance stages considered, with a design elastic spectrum reduced by factors defined from the strengths of the elastic system.

Step 3. The design forces corresponding to the last design stage are obtained by combining the element forces of the MSA with the reduced elastic design spectrum, differentiating the element forces from the analysis for gravitational and vehicular loads and for modal spectral analysis in accordance with the EC8 code or any other valid bridge design code.

The steps involved in the application of this method are illustrated in Figure 6, where it may be observed that the way in which global performance quantities, such as modal ductility, are computed, is not presented, as this problem is still under investigation; consideration to this will be given in Deliverables 112 and 113 of the LESSLOSS project.

6. CONDITION OF REGULARITY

From the performance evaluation/design stand point, in this report it is assumed that a bridge becomes irregular when changes occur in the order of the modes that contribute the most to the performance of the structure throughout its evolution from one damage stage to another with increasing seismic demand.

This assumption of regularity is applicable to both groups of methods of evaluation/design considered in this report, in as much, for both, the conditions that define irregularity may lead to wrong results. This problem is later on illustrated for the two different methods considered in the section of application examples in this report.

Due to the inherent differences between the two methods proposed and inconclusive results regarding their application, further research work is under way to prove this assumption and give further explanation of how this irregularity condition appears.

7. SYNTHETIC ACCELEROGRAMS FOR METHOD EVALUATION

To validate the results obtained with the evaluation/design methods, for a family of 1000 synthetic accelerograms was simulated, compatible with the target design spectrum simulated by Isaković *et al.* [2005] was used. A sample synthetic accelerogram is presented in Figure 7, where the shape of the accelerogram and the peak ground acceleration may be observed. To establish the validity of the accelerograms simulated, Figure 8 shows the comparison of the target design spectrum and the mean value of the response spectra generated for a set of 50 accelerograms.

8. APPLICATION EXAMPLES

To illustrate the application and validate the accuracy and potentiality of the proposed methods, six sample bridge structures are evaluated. The first structure is the scaled four span single supported concrete bridge tested at ELSA with a variety of pseudo-static and pseudo-dynamic tests, [Pinto *et al.*, 1996], while the other five structures have the same configuration as the first, but with different dimensions and characteristics of the piers and superstructure, as designed by Isaković and Fischinger [2006]. The bridges are all reinforced concrete structures designed in accordance with current seismic codes.

The scaled bridge model tested at ELSA was used for the evaluation of the method based on the substitute structure. For this bridge, the considered seismic demands were derived from a synthetic record simulated to match the design spectrum given by EC8 [CEN, 1993] corresponding to soil type B, 5% damping ratio, and two peak ground accelerations a_g equal to 0.35g and 0.70g (where g is the acceleration due to gravity). The scaled spectrum is illustrated in Figure 9, [Pinto *et al.*, 1996]. The layout of the bridge is shown in Figure 10 and the corresponding geometric and structural characteristics are shown in Table 8.1.

Figure 11a and b show the calculated performances for the bridge tested at ELSA under the two considered scaled earthquake intensities. In this particular example, to show the approximation of the method based on the substitute structure alone, the results presented were obtained using linear modal time history analyses instead of MSA, in order to avoid the effect of a particular modal combination rule. The results are compared with those obtained from non-linear time history analyses on the same structure subjected to the two scaled synthetic records.

The results for maximum displacements along the bridge axis *x* depicted in Figure 11a, show that, for the lowest intensity, the approximation of the proposed method is not good enough, whereas the results for the highest intensity record, shown in Figure 11b, are in better agreement with those obtained from non-linear time history analysis.

Table 8.1. Pier Properties of bridge model tested at ELSA

M_1 =832 kN*m	M_2 =1048 kN*m	M_3 =555 kN*m
I_{1cr}=0.0553 m^4	I_{2cr}=0.0392 m^4	I_{3cr}=0.0553 m^4

Table 8.2 and Table 8.3 show some dynamic characteristics of this example calculated for the two considered earthquake intensities for the substitute structure at the end of the iteration process and for the structure modelled with a plastic hinge at the base of the pier where inelastic incursions occur, here referred as damaged structure. From these tables it may be observed that the product of the Modal Participation Factors (MPF) and the corresponding spectral acceleration of the substitute structure for the first intensity are different to those of the damaged structure and that this relative difference is not as pronounced for the example with the highest intensity. It may be also observed that, for the lower intensity demand, the effective period of the substitute structure is not as close to the fundamental period of the damaged structure as it is for the larger intensity case. In this report these differences have been attributed to a irregularity condition of the example with the lower intensity, however it might also be said that the found differences affect only the approximation level and not the correctness of the method, it is due to these uncertain conclusions that further research is underway.

Table 8.2. Comparison of modal properties for ELSA bridge with a_g=0.35g

	Substitute structure		Damaged structure	
	T	**MPF*Sa**	**T**	**MPF*Sa**
1st mode	0.261	250.2	0.482	145.5
2nd mode	0.193	74.6	0.195	29.5
3rd mode	0.093	63.3	0.098	104.9

Table 8.3. Comparison of modal properties for ELSA bridge with a_g =0.70g

	Substitute structure		Damaged structure	
	T	**MPF*Sa**	**T**	**MPF*Sa**
1st mode	0.330	427.6	0.482	291.0
2nd mode	0.195	88.9	0.195	59.0
3rd mode	0.096	178.5	0.098	209.9

These differences in modal properties are not reflected in the corresponding modal shapes, Figure 12a and b, where the shapes of comparable modes are approximately the same.

The remaining five bridges considered in the example were analyzed and designed in accordance with the current Eurocodes [CEN, 2003a and 2003b]. The seismic demand considered was the design spectrum of EC8 [CEN, 2003a], corresponding to a type B soil, 5% damping ratio, soil amplification factor S=1.2 and different peak ground accelerations, ranging from 0.20g to 0.70g. The details of the spectrum for a_g equal to 0.30g are illustrated in Figure 13. The nomenclature used for these bridges is V###L where V is for viaduct, # is for pier height (1: short, 2: medium and 3: tall) and the last letter stands for the type of support (P: laterally fixed and R: laterally free).

The geometric characteristics and mass distributions of the five bridge models are shown in Figure 14, Figure 21, Figure 28 and Figure 33 with the corresponding properties of the piers presented from Table 8.4 to Table 8.8.

The results of the evaluation method based on the non-linear capacity concept for all bridge models and earthquake intensities considered are shown from Figure 15 to Figure 20, from Figure 22 to Figure 27, from Figure 29 to Figure 32 and from Figure 34 to Figure 40. In all cases, the results calculated with the approximate evaluation method are compared with the mean value of the results of 50 non-linear time history analyses and those obtained when the bridge was subjected to three real earthquake records scaled to match the smooth design spectrum used in the evaluation, *i.e.,* Peega (Petrovac E-W 1979, Montenegro), Tonga (Tolmezzo N-S 1976, Italy) and Llnga (Llolleo 1985, Chile).

The results of this example show for all earthquake intensities, a distribution of maximum displacements corresponding to a modal shape dominance, different to that obtained through non-linear time history analyses. This lack of approximation is more evident as the intensity increases, fact that will discussed later on in this chapter.

For the rest of the models considered, the approximation obtained is satisfactory: the maximum displacement distribution is represented correctly, but as with the V213 model, the accuracy of the approximation decreases as the seismic demand intensity increases.

Table 8.4. Pier properties of bridge V213P

M_1 =30034 kN*m	M_2 =50508 kN*m	M_3 =331215 kN*m
I_{1cr} =2.22 m^4	I_{2cr} =2.4 m^4	I_{3cr} =2.26 m^4

Table 8.5. Pier Properties of bridge V123P

M_1 =101100 kN*m	M_2 =38100 kN*m	M_3 =31200 kN*m
I_{1cr}= 3.6312 m^4	I_{2cr}=2.1671m^4	I_{3cr}=2.2589 m^4

Table 8.6. Pier Properties of bridge V232P

M_1 =75315 kN*m	M_2 =39191 kN*m	M_3 =375315 kN*m
I_{1cr}=2.97 m^4	I_{2cr}=2.21 m^4	I_{3cr}=2.97 m^4

Table 8.7. Pier Properties of bridge V213R

M_1 =46,750 kN*m	M_2 =46,697 kN*m	M_3 =47,811 kN*m
I_{1cr}=2.33 m^4	I_{2cr}=2.3 m^4	I_{3cr}=2.33 m^4

Table 8.8. Pier Properties of of bridge V213P with increased strength

M_1 =45600 kN*m	M_2 =50508 kN*m	M_3 =331215 kN*m
I_{1cr}=2.22 m^4	I_{2cr}=2.4 m^4	I_{3cr}=2.26 m^4

Figure 40 shows the results obtained with this method with an a_g equal to 0.35g, together with the statistics of 1000 non-linear time history simulations. It may be observed that there is a lack of approximation which may not be attributed to the uncertainties of the synthetic records, but rather to the change of order of the most important participating modes, as shown in Figure 41, where the modes associated to the elastic and the damaged structure corresponding to the performance level are compared. This particular characteristic has been detected as the evidence of the condition of irregularity.

9. CONCLUSIONS

This report presents two different methods of displacement based evaluation/design of bridges which improve previously developed approximations. The results presented may be directly used to construct a "substitute" structure or a response curve of a reference SDOF system which leads to a sought performance or to a bridge design for a specified target performance given by a maximum pier displacement.

This deliverable is compatible with the simplified model presented by the same research group in Deliverable 69 for the determination of the stiffness and energy dissipation characteristics of hollow RC bridge piers, which is used as basis of the methods of displacement based evaluation/design of bridges here developed.

Ongoing research within the framework of the LESSLOSS project on the application of the "substitute" structure approach or of the non linear capacity curve for the seismic performance evaluation of existing bridges and for the DDBD of new bridges, described in this report, has shown that the evaluation and the design options of the proposed methods, may give acceptable results with a limited computational effort as long as the structure is "regular".

The comparison of the results obtained with the simplified methods for the different examples here considered show that the methods based on the "substitute" structure or on the non-linear capacity curve may be successfully applied to "regular" bridge structures. This conclusion unfortunately may not be extended to the case of "irregular" bridges, as particularly shown in the first example of the "substitute" structure approach, where the relative relevance to performance of the modes, considered in the method, changes as a function of earthquake intensity.

Both proposed methods may be considered enhanced versions of others currently under use or investigation by other research groups, as they take into consideration the contribution of higher modes of vibration and the displacement reversal nature of earthquake action through evolving modal spectral analyses, rather than from evolving force or displacement based pushover analyses.

It is demonstrated that the use of smooth response spectrum as seismic demand is an advantage for most currently used methods, as its use does not consider a single record but a complete ensemble of them, however, this does not guarantees sufficient accuracy, or even correct results, for all situations. For instance, the results presented in this report for the V213P bridge are not satisfactory when compared with those of the statistical study of 1000 non-linear time history analyses. The observed lack of approximation may be due to the fact that, for the considered design level, the bridge, due to the occurrence of new damage, changed its fundamental mode shape from a rotational to a translational type.

For a given bridge, the seismic demand for which the change in dynamic characteristics of a bridge occurs, may be considered as a regularity boundary beyond which the method, as presented, stops given correct results. Regarding this point, more research efforts are under way to fully understand why this lack of approximation suddenly occurs to generalise the proposed methods for their reliable application to all bridge configurations and seismic design levels.

Preliminary results show that for bridges with a significant contribution of higher modes and with large non-linearities, the methods proposed, in particularly the one based on the non linear capacity of the structure leads to better results than alternative simplified procedures based on a "substitute structure" and on an "equivalent" SDOF system, which do not explicitly consider the contribution of higher modes. For both methods proposed, in the case of design, the deformation capacity of the structure is obtained by means of a assumed damage distribution, explicitly defined in the design process.

It is shown that the application of these methods is simpler than that of other methods, as all calculations involved may be carried out with commercial analysis software [CSI, 1997a and b]. In particular, for the method based on the non-linear capacity of the structure, the construction of the response curve of the reference system is carried out using partial results of evolving modal spectral analyses This approach is simpler and superior to others currently used, as it does not depend on results of non-linear pushover analyses with evolving lateral force or displacement distributions. Furthermore, the application of modal spectral analysis with accepted mode combination rules for the evaluation/design of bridges gives evaluations and designs corresponding to maximum expected performances. It may be assumed that, by using modal combination rules, an upper bound of performances is guaranteed.

Validation of performance under a seismic demand given by a smooth spectrum obtained though time history analyses may lead to erroneous conclusions when the performance of the structure, against which the approximate results are compared, is calculated only for one or even a limited number of records matching the smooth design spectrum representing the seismic demand.

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FIGURES

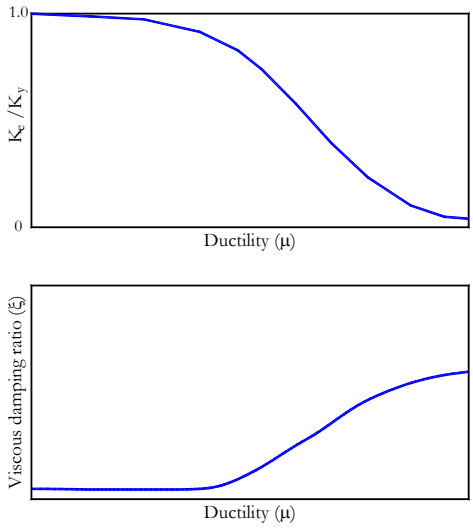

Figure 1. Equivalent a) secant to peak stiffness and b) viscous damping ratio

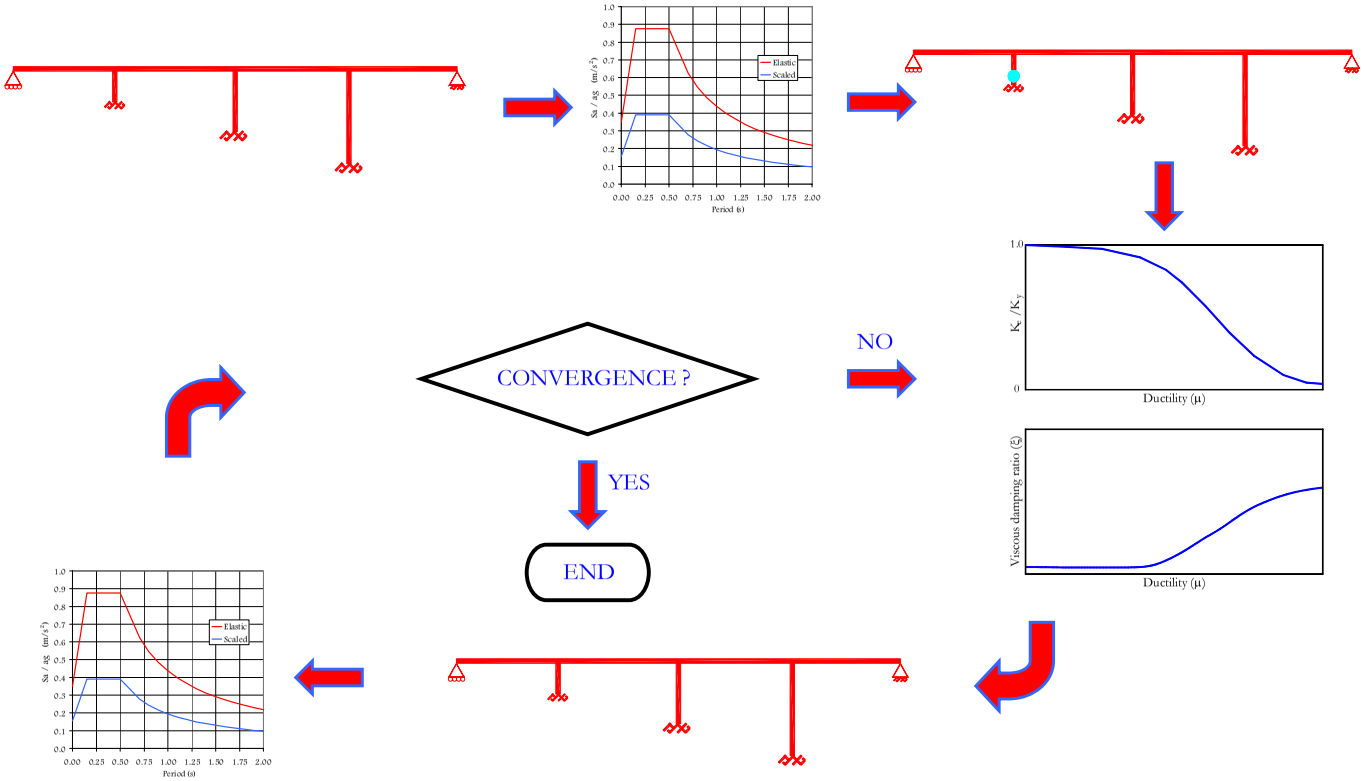

Figure 2. Evaluation procedure for the method based on the substitute structure

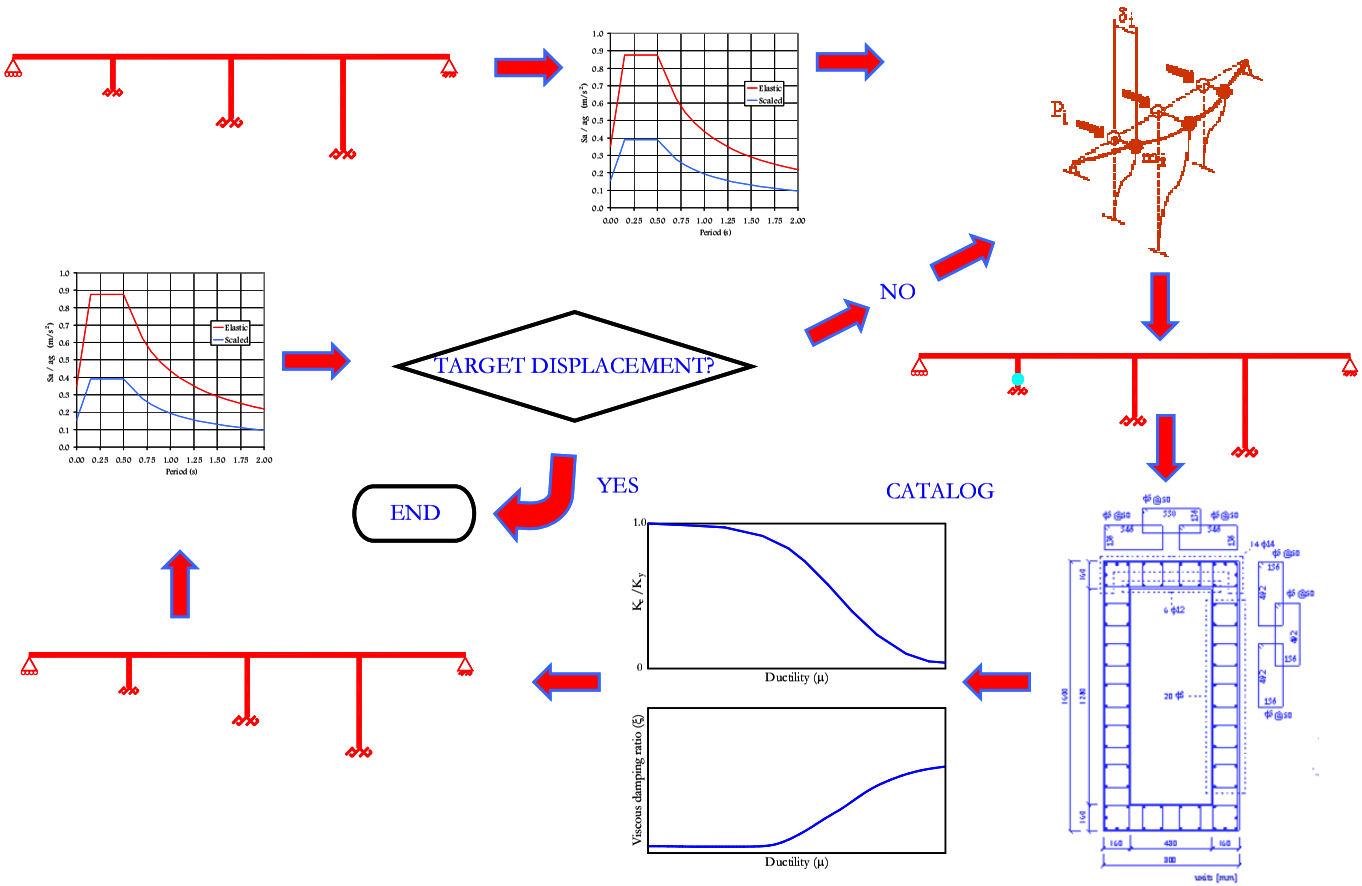

Figure 3. Design procedure for the method based on the substitute structure

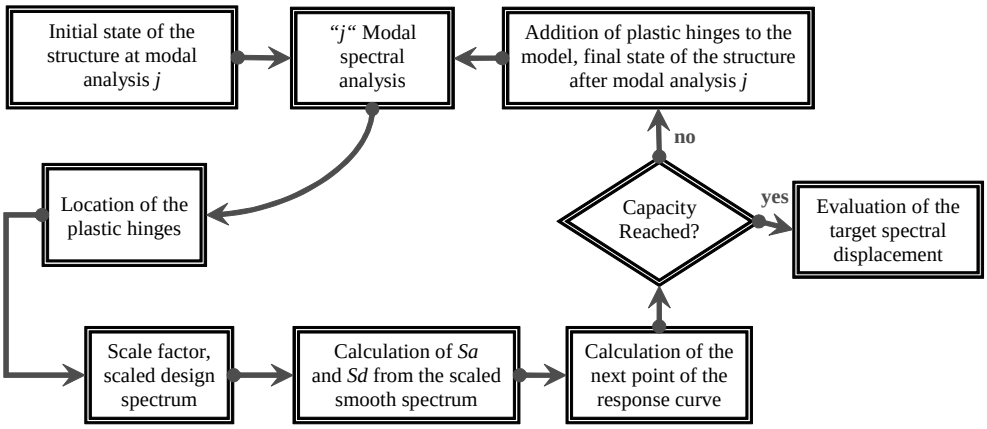

Figure 4. Determination of the response curve of the reference SDOF system

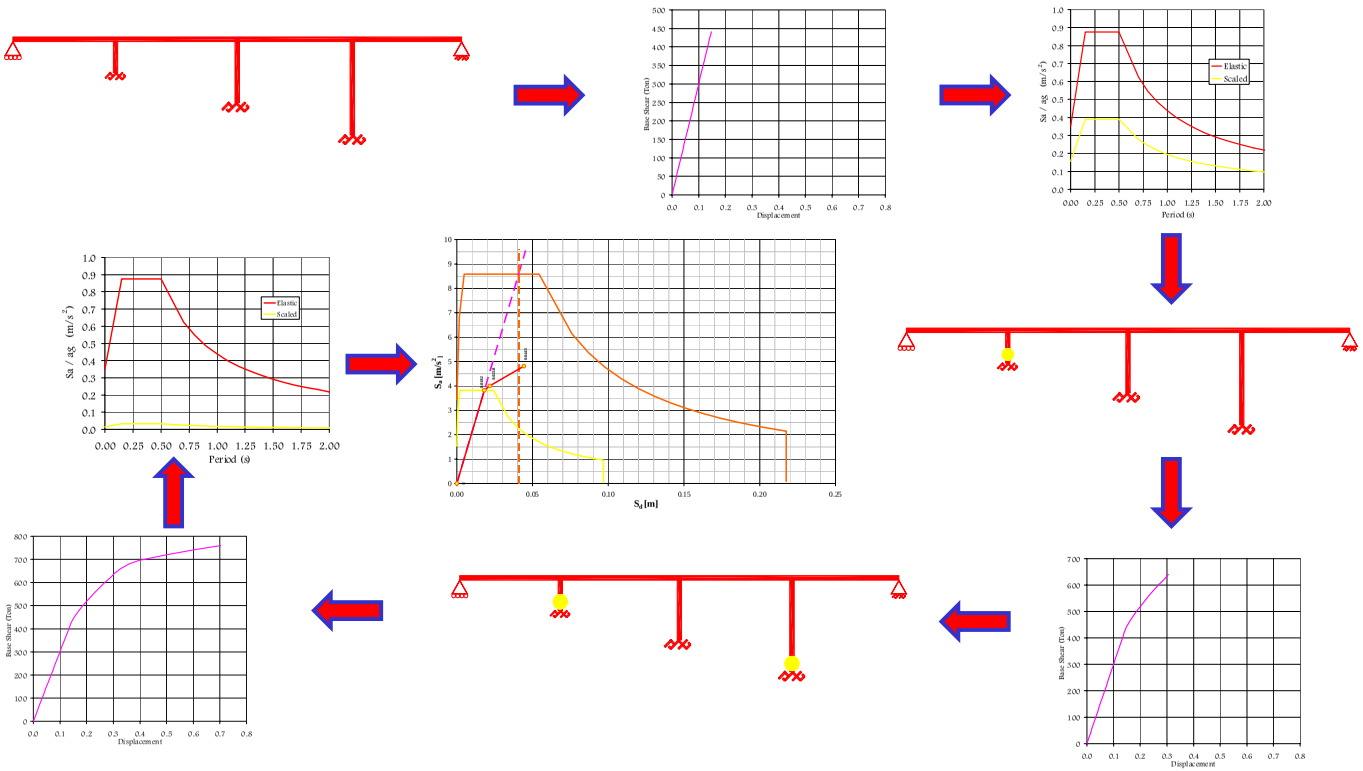

Figure 5. Assessment procedure for the proposed method based on the reference structure

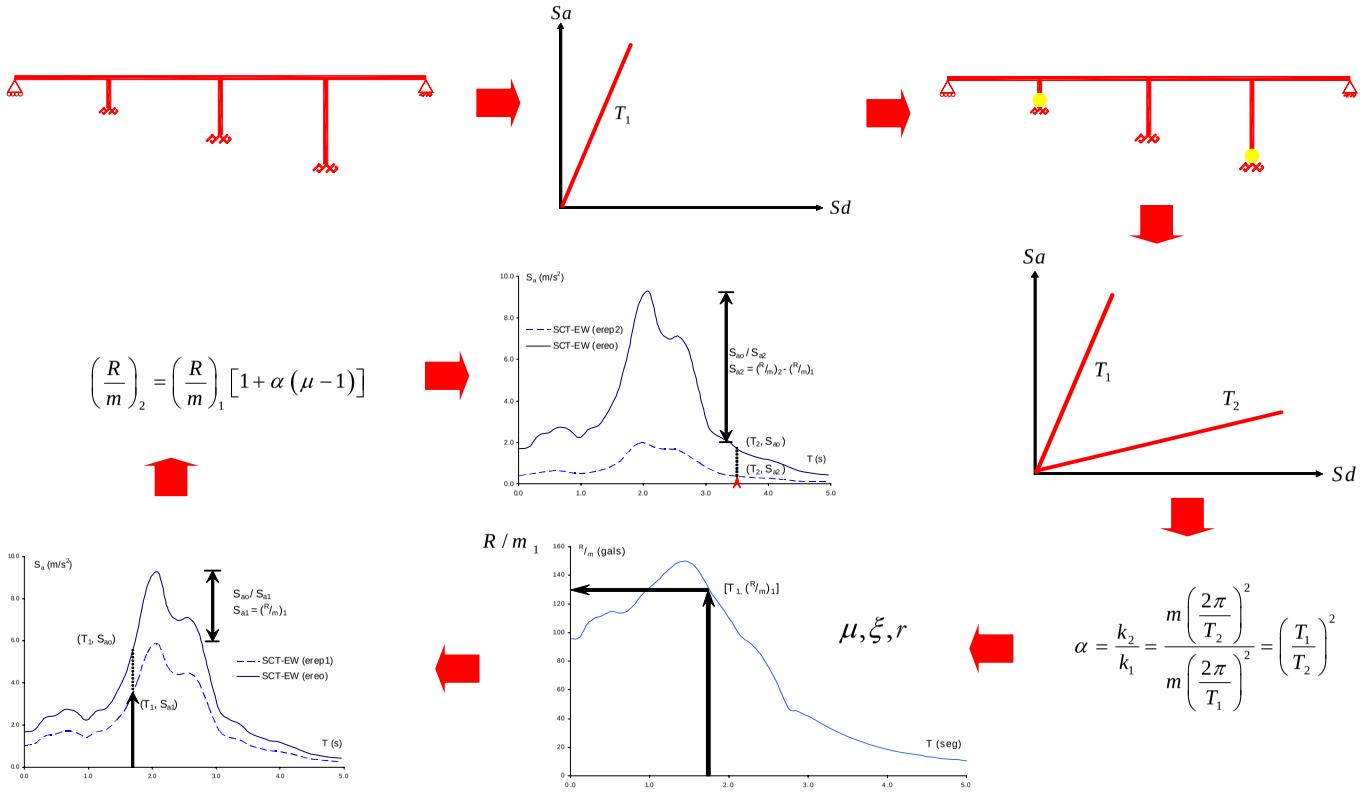

Figure 6. Design procedure for the method based on the reference structure

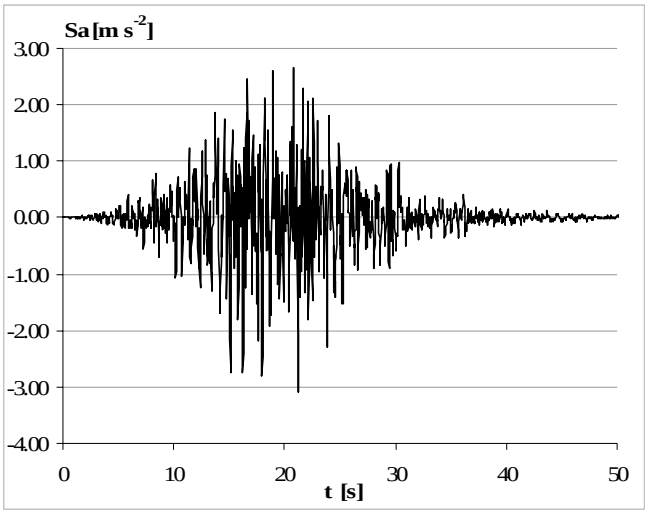

Figure 7. Sample of synthetic accelerogram

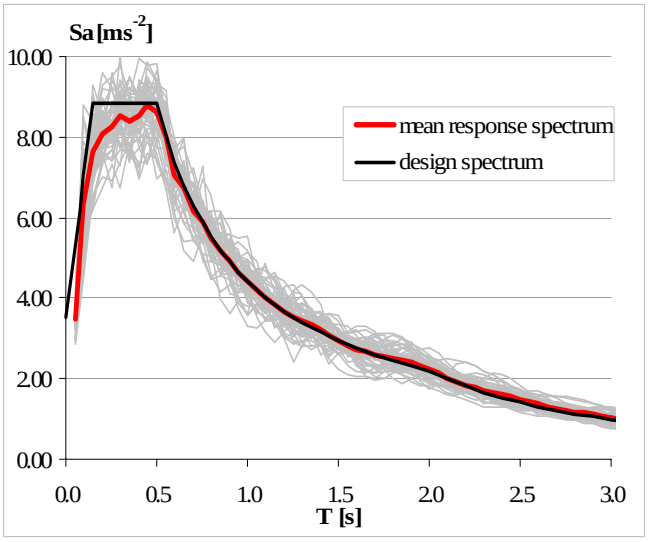

Figure 8. Comparison of spectra for an ensemble of synthetic accelerograms

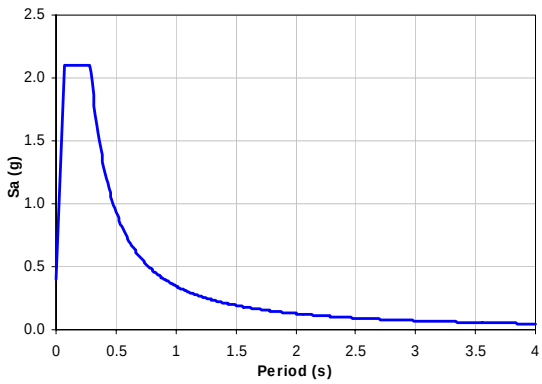

Figure 9. Scaled design spectrum used for the evaluation of the bridge model tested at ELSA

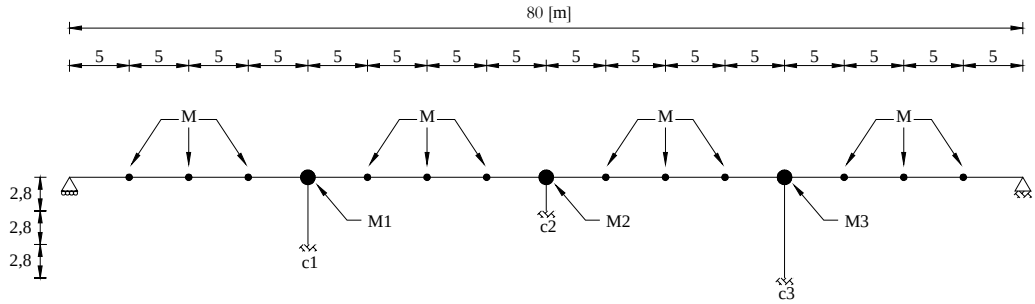

Figure 10. Geometry and location of masses of the bridge model tested at ELSA

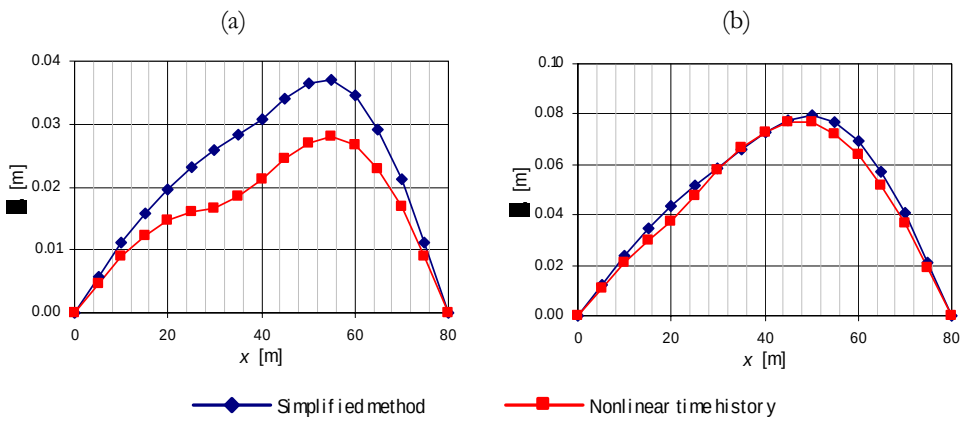

Figure 11. Maximum displacement distribution for the bridge model tested at ELSA

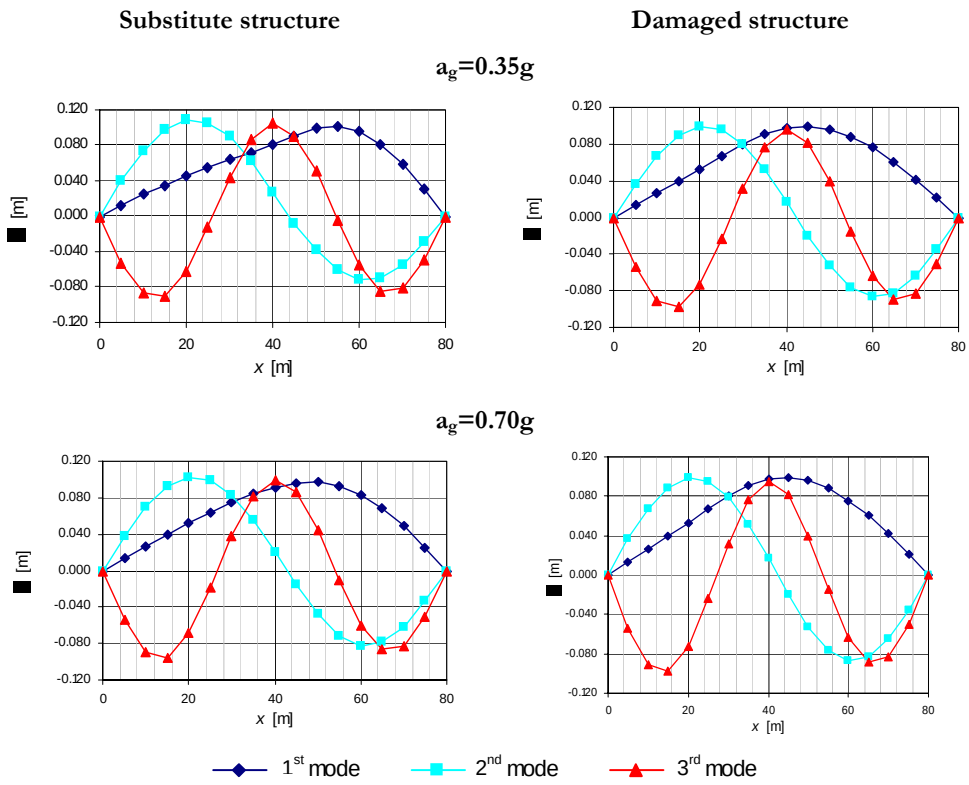

Figure 12. Maximum displacement distribution for the bridge model of example 1

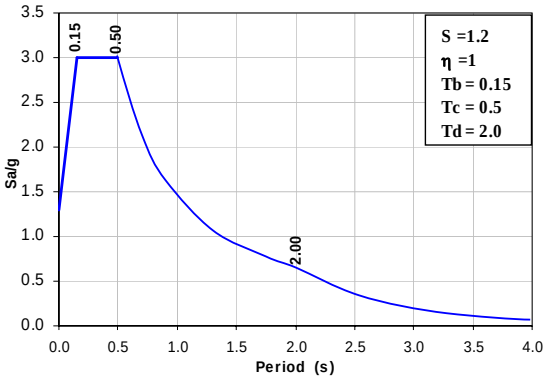

Figure 13. Reference design spectrum used for the evaluation of the bridges V###L

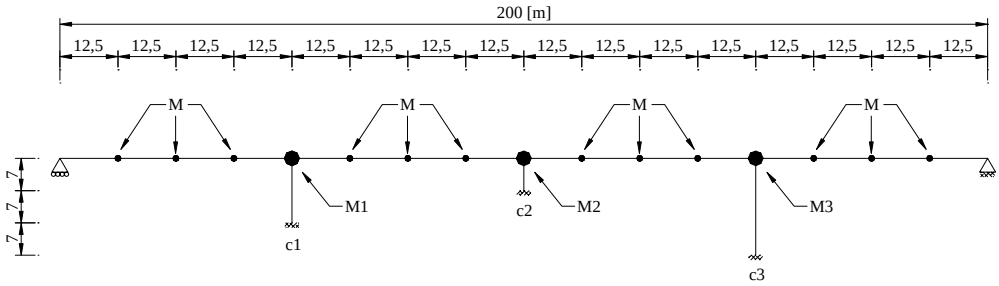

Figure 14. Geometry and location masses of the of bridge V213P

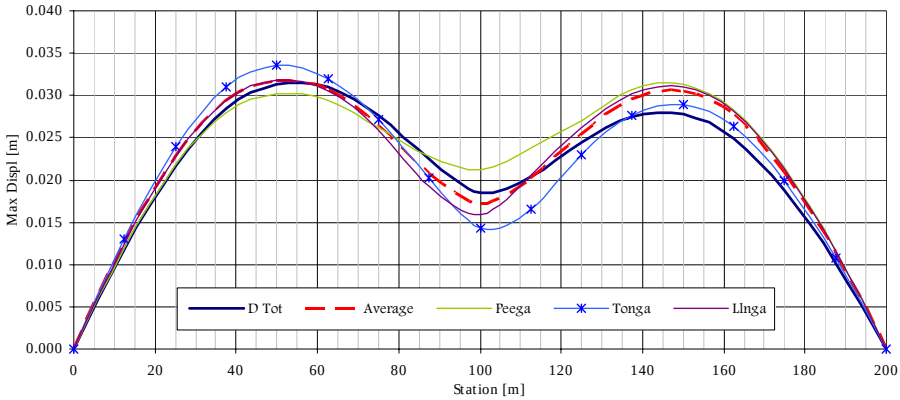

Figure 15. Maximum displacement distribution for bridge V213P, a_g = 0.20g

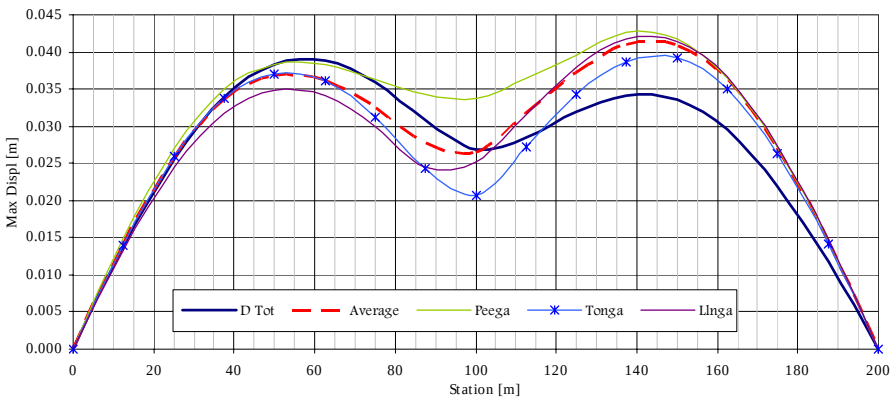

Figure 16. Maximum displacement distribution for bridge V213P, a_g = 0.25g

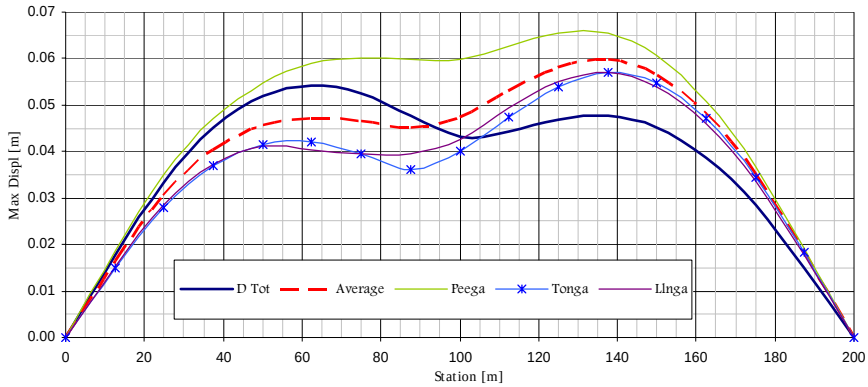

Figure 17. Maximum displacement distribution for bridge V213P, a_g = 0.35g

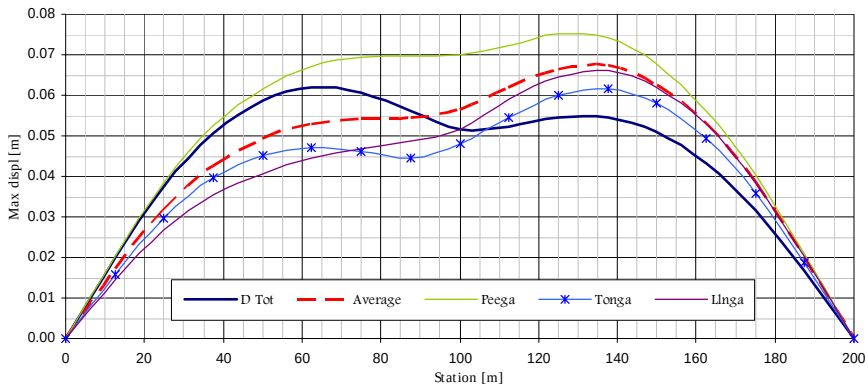

Figure 18. Maximum displacement distribution for bridge V213P, a_g = 0.40g

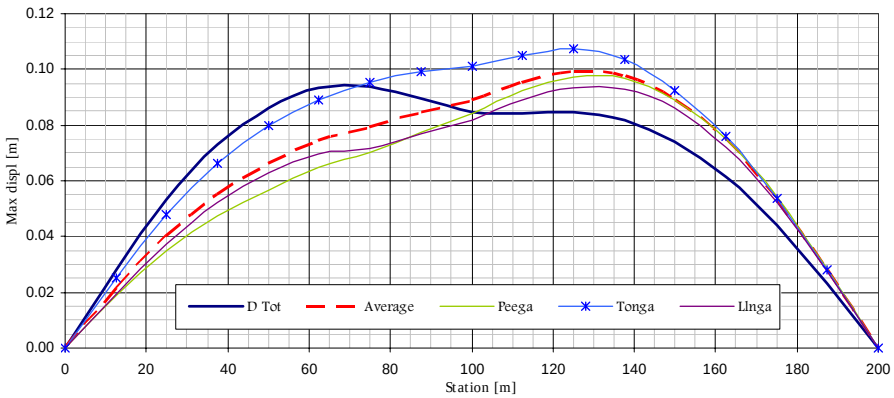

Figure 19. Maximum displacement distribution for bridge V213P, a_g = 0.60g

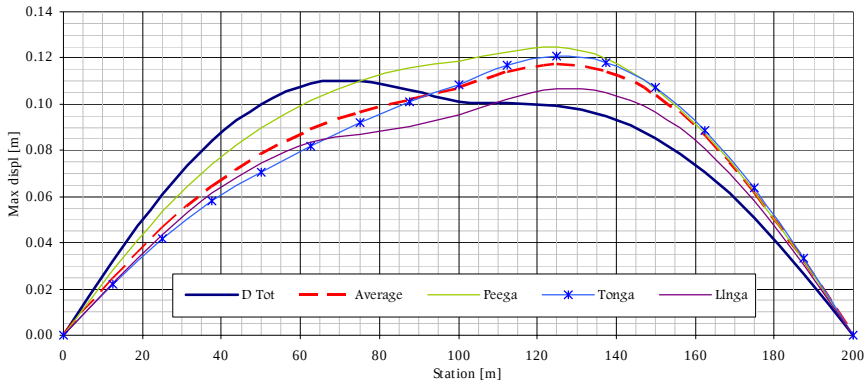

Figure 20. Maximum displacement distribution for bridge V213P, a_g = 0.70g

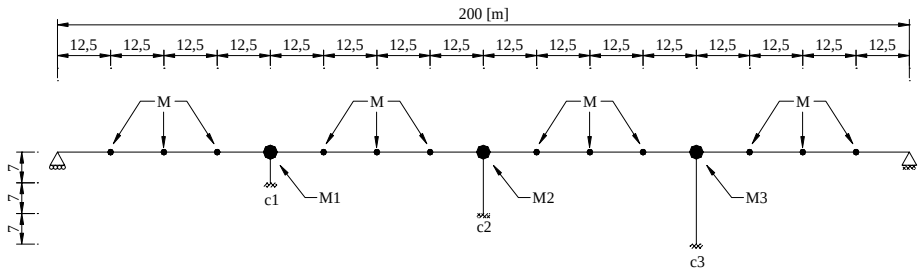

Figure 21. Geometry and location masses of the of bridge V123P

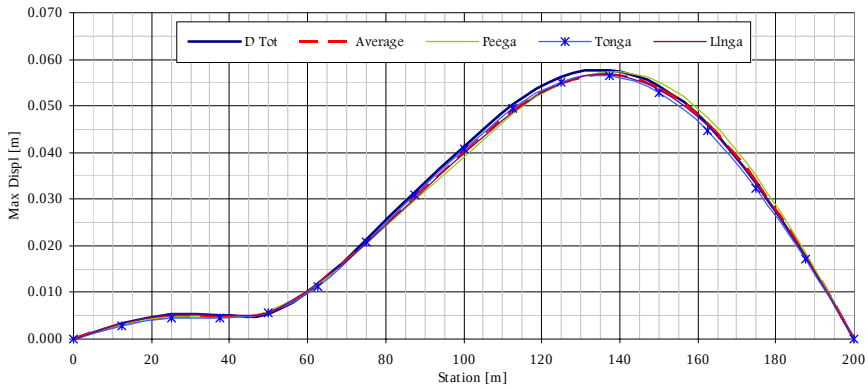

Figure 22. Maximum displacement distribution for bridge, a_g = V123P 0.20g

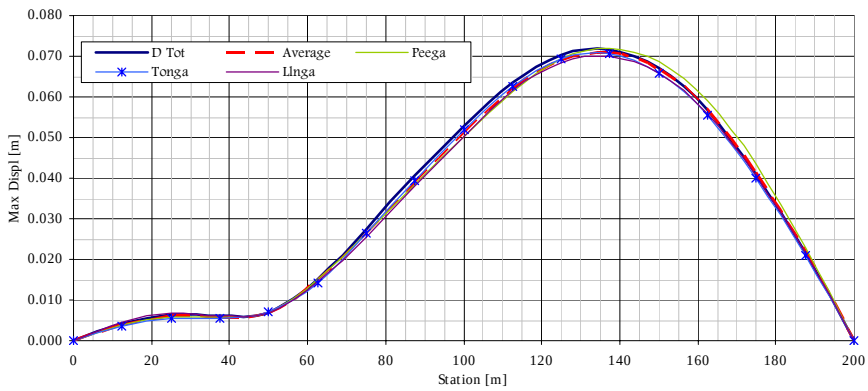

Figure 23. Maximum displacement distribution for bridge V123P, a_g = 0.25g

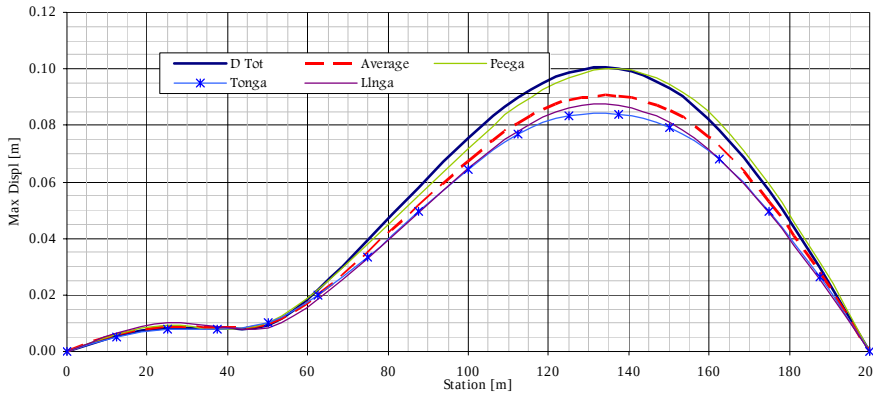

Figure 24. Maximum displacement distribution for bridge V123P, a_g = 0.35g

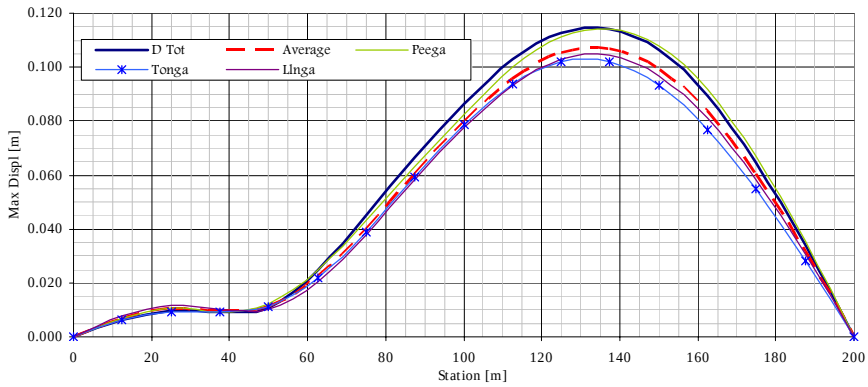

Figure 25. Maximum displacement distribution for bridge V123P, a_g = 0.40g

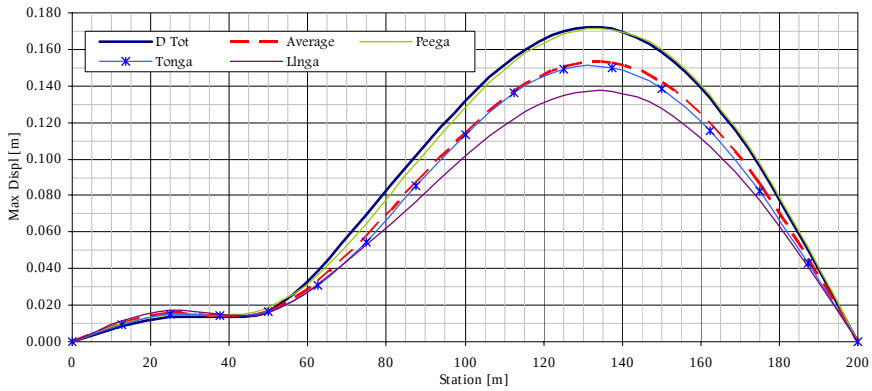

Figure 26. Maximum displacement distribution for bridge V123P, a_g = 0.60g

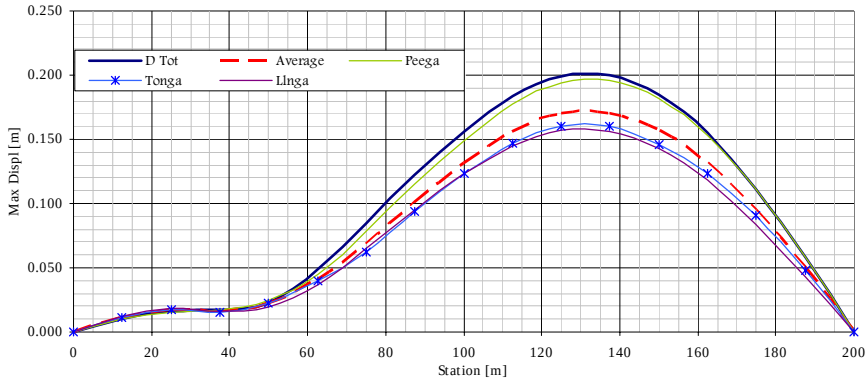

Figure 27. Maximum displacement distribution for bridge V123P, a_g = 0.70g

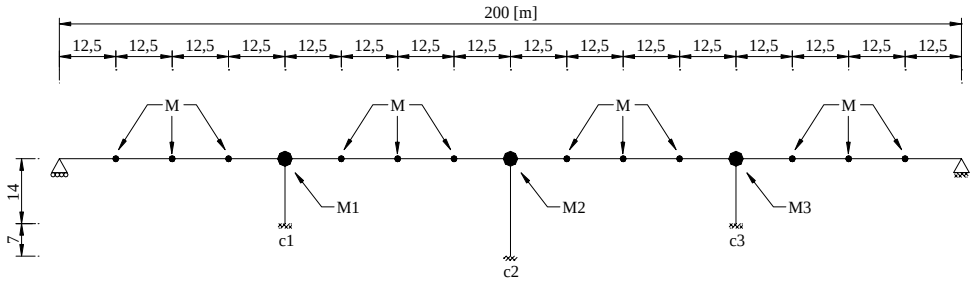

Figure 28. Geometry and location masses of the of bridge V232P

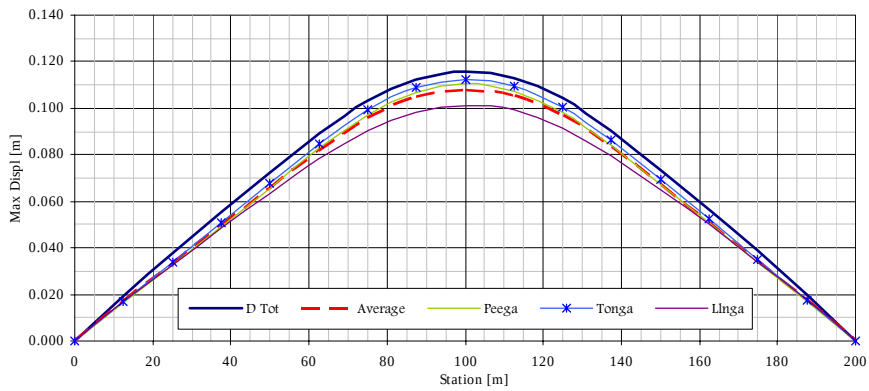

Figure 29. Maximum displacement distribution for bridge V232P, a_g = 0.35g

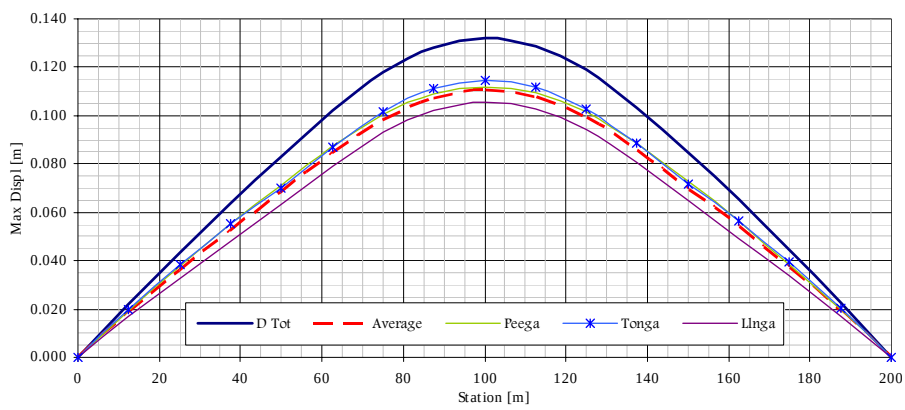

Figure 30. Maximum displacement distribution for bridge V232P, a_g = 0.40g

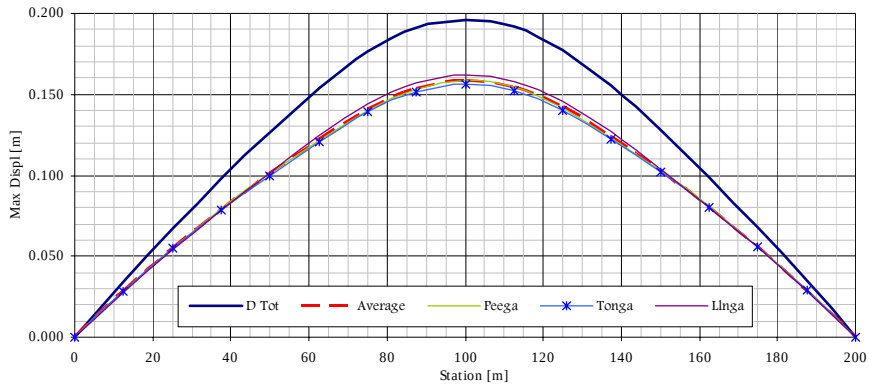

Figure 31. Maximum displacement distribution for bridge V232P, a_g = 0.60g

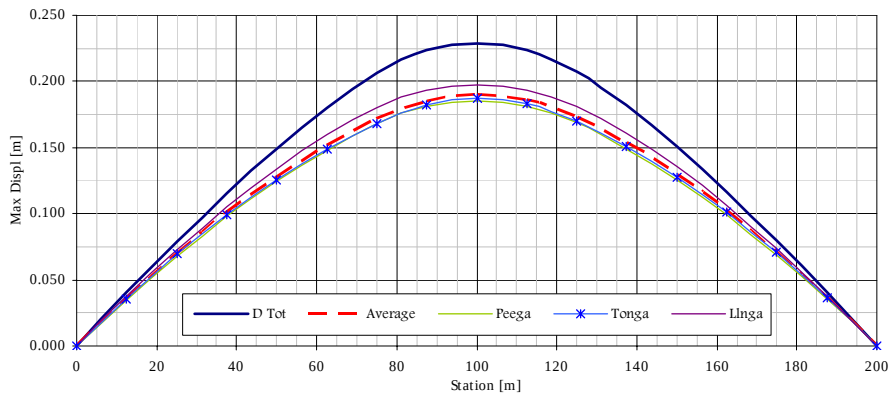

Figure 32. Maximum displacement distribution for bridge V232P, a_g = 0.70g

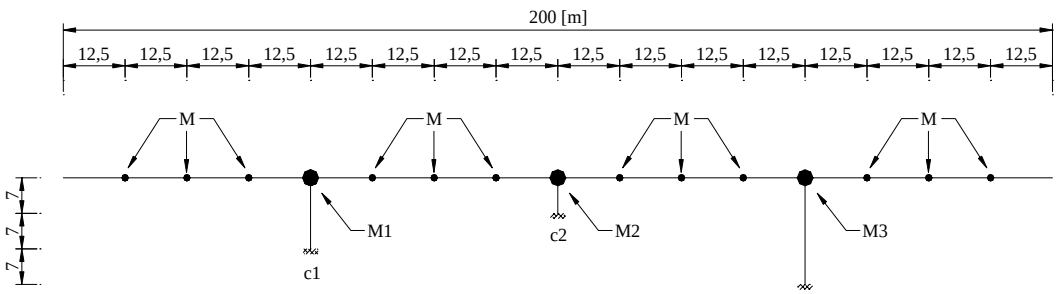

Figure 33. Geometry and location masses of the of bridge V213R

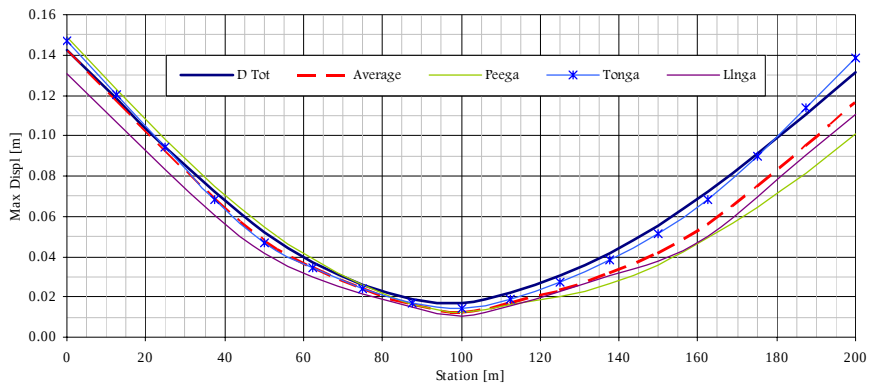

Figure 34. Maximum displacement distribution for bridge V213R 0.20g

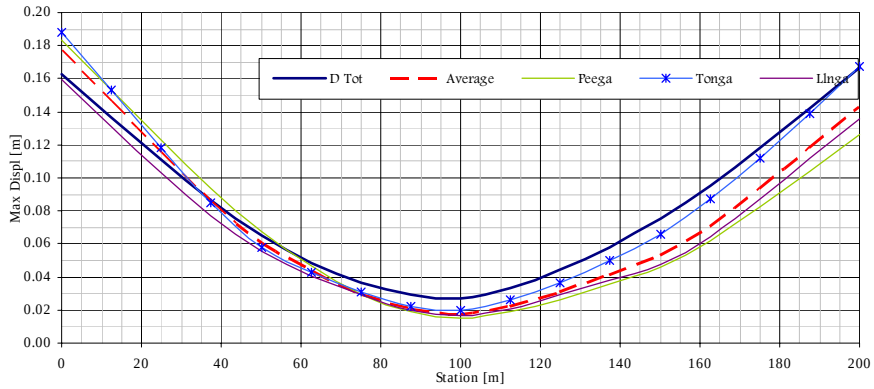

Figure 35. Maximum displacement distribution for bridge V213R, a_g = 0.25g

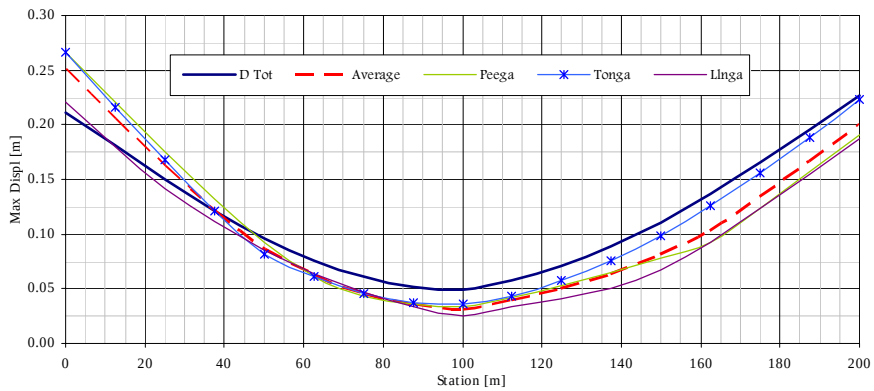

Figure 36. Maximum displacement distribution for bridge V213R, a_g = 0.35g

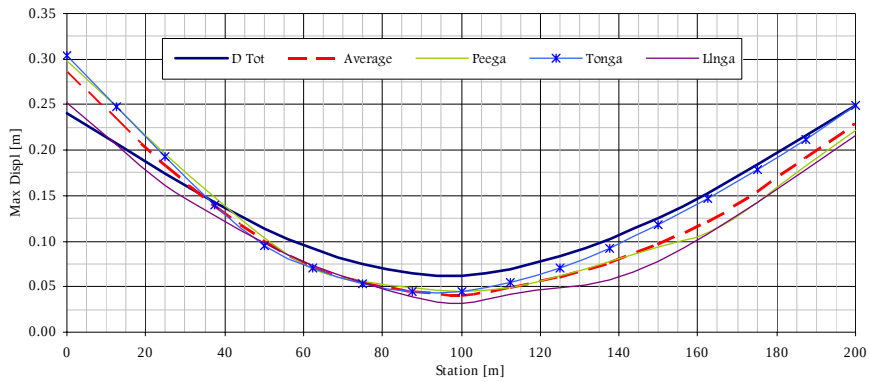

Figure 37. Maximum displacement distribution for bridge V213R, a_g = 0.40g

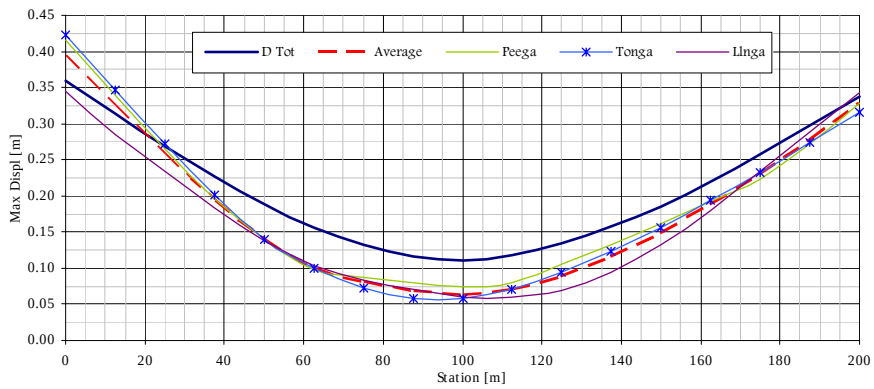

Figure 38. Maximum displacement distribution for bridge V213R, a_g = 0.60g

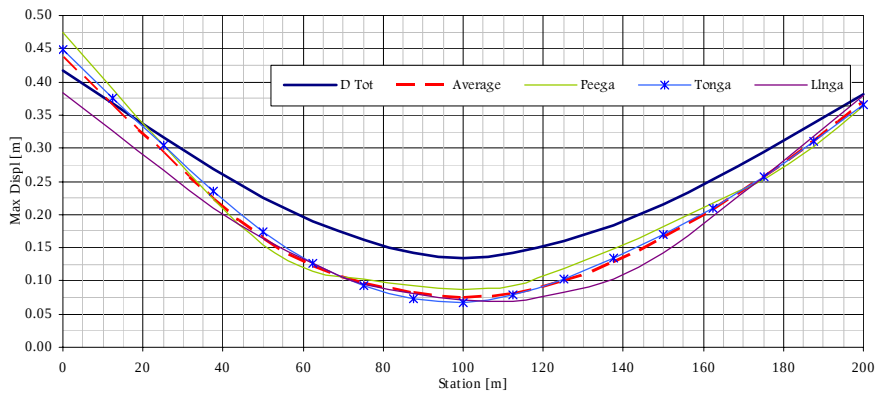

Figure 39. Maximum displacement distribution for bridge V213R, a_g = 0.70g

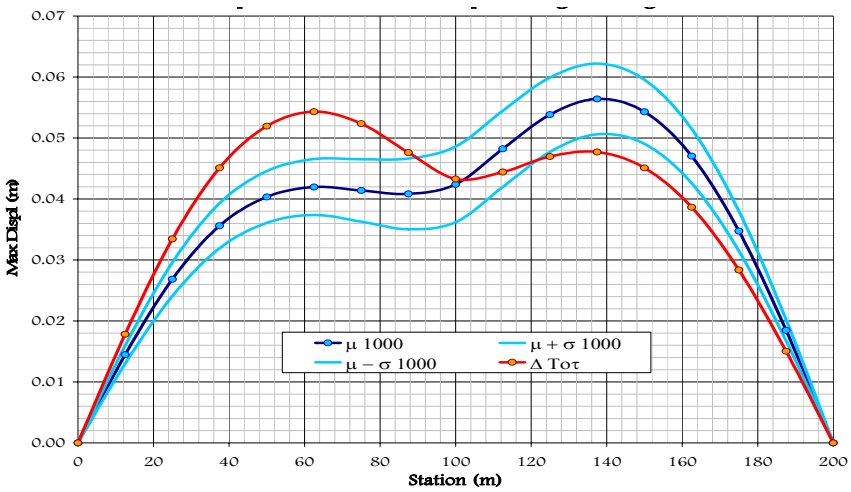

Figure 40. Maximum displacement distribution for bridge V213P with increased strength of pier 1, a_g = 0.35g

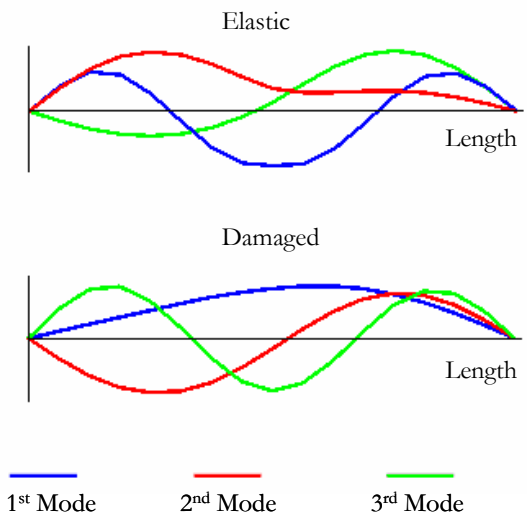

Figure 41. Modal shapes corresponding to the elastic and inelastic stages of bridge V213P

European Commission

EUR 22888 EN – Joint Research Centre – Institute for the Protection and Security of the Citizen
Title: Simplified models/procedures for estimation of secant-to-yielding stiffness, equivalent damping, ultimate deformations and shear capacity of bridge piers on the basis of numerical analysis
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Abstract
This report addresses the problem of the Displacement Based Evaluation/Design (DBE/D) of reinforced concrete bridges. Throughout the report, the aims and limitations of current seismic evaluation and design practice and the tendencies of the displacement-based seismic evaluation/design are discussed.
It presents a state-of-the-art review on the most important results and lessons derived from previous works, and based on them, two evaluation/design methods consistent with the performance-based seismic design philosophy are presented.

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