# IS DIRECT DISPLACEMENT BASED DESIGN VALID FOR LONG SPAN BRIDGES?

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**ABSTRACT:**

The paper investigates the applicability of current direct displacement based seismic design (DDBD) procedure, developed by Priestley and his coworkers, for straight long span bridges under transverse seismic excitation synchronous to all supports. This category of bridges often possess some additional features such as massive tall piers, highly irregular distribution of mass and stiffness due to unequal superstructure spans and pier heights, large deformation capacity etc. that are absent in short-to-moderate span bridges for which DDBD has extensively been verified. It is shown that DDBD in its current form is unable to capture both displacement and base shear demand when compared with nonlinear dynamic analysis results. Accordingly, a simple mechanics based extension of the current procedure that takes into account the effect of pier mass while computing base shear demand as well as a modal combination rule for estimating displacement demand is proposed and validated using a series of parametric studies. The new procedure also allows engineer to allocate strength at the potential plastic hinge location in more general terms.

**KEYWORDS:** Bridge analysis, direct displacement based design, long span bridges.

## 1. INTRODUCTION

Over the last decade or so, displacement controlled design procedures have emerged as a result of the recognition of the primary role of displacement and deformation as more reliable and direct indices of structural (and non-structural) damage during significant earthquakes than strength, as considered earlier. One of such promising procedure is Direct Displacement Based Design (DDBD) developed by Priestley and his coworkers (Priestley *et al.*, 2003): utilizing the idea of 'Substitute Structure' conceptualized by Shibata and Sozen (1976), a structure is designed to achieve a certain performance limit defined in terms of displacement or deformation quantity under a prescribed seismic hazard described in terms of design/response spectrum. Among the various displacement controlled design procedures developed recently, Sullivan *et al.* (2003) noted that this procedure is simpler to apply, better equipped to address the deficiencies of conventional force based design and developed in a more complete form than others. However, this procedure is validated, till now, only for short-to-moderate span bridges (Alvarez Botero, 2004; Ortiz Restrepo, 2006) which differ significantly in terms of seismic response from long span bridges due to the presence, in the latter, of massive tall piers, uneven distribution of mass and stiffness along the bridge length and higher deformation capacity of the tall piers. These additional features call for further investigations whether DDBD is valid for long span bridges and if not, then what are the possible improvements that can be made.

Following a brief summary of current DDBD procedure for short-to-moderate span bridges, a set of modifications is proposed to overcome the above mentioned issues related to the long span bridges. The accuracy of the proposed modifications as well as the inaccuracy of the traditional strength based design procedures is explored through a set of parametric study. Due to space limitations, only two bridge geometries have been presented here.

## 2. DIRECT DISPLACEMENT BASED DESIGN : SUMMARY

Summarized below are the steps used to design a straight short-to-moderate span bridge under synchronous ground motion represented by a design spectrum, as documented in Priestley *et al.* (2007).

i) Assume a target displacement profile such that at least one pier reaches its critical limit defined either from strain based criterion to limit structural damage or from drift based criterion to protect non-structural damage or to restrict residual displacement.

ii) Estimate the properties of equivalent SDOF system, namely, equivalent system displacement $\Delta_{sys}$ and system mass $M_{sys}$, using the work equivalence equations (Eqn 2.1) developed by Calvi and Kingsley (1995).

$$\Delta_{sys} = \frac{\sum_{i=1}^{n} m_i \Delta_i^2}{\sum_{i=1}^{n} m_i \Delta_i} \quad \text{and} \quad M_{sys} = \frac{\sum_{i=1}^{n} m_i \Delta_i}{\Delta_{sys}} \tag{2.1}$$

where, $m_i$ and $\Delta_i$ are the mass and target displacement associated with the $i^{th}$ location respectively and *n* is the total number of lumped mass positions.

iii) Compute the ductility level ($\mu_\Delta$) for each pier utilizing the chosen target displacement and estimated yield displacement from yield curvature ($\varphi_y$), which depends only on the sectional depth ($h_c$) and yield strain ($\varepsilon_y$) of the reinforcement, as given by Eqn 2.3, applicable only for low axial load ratio $\left(P/f_c A_g \le 0.05\right)$.

$$\mu_\Delta = \frac{\Delta_{target}}{\Delta_y} \quad \text{where} \quad \Delta_y = \varphi_y \frac{h_c^2}{3} \tag{2.2}$$

$$\varphi_y = 1.80 \frac{\varepsilon_y}{h_c} \tag{2.3}$$

iv) Estimate the equivalent viscous damping ($\xi_p$) for individual piers using the empirical relation given by Eqn 2.4 developed for Takeda thin hysteretic rule which is in general valid for pier sections. The additional 5.0% of critical damping is added to the hysteretic counterpart to account for energy dissipation from nonlinearity in the elastic response, soil-structure interaction and other similar mechanisms (Grant *et al*., 2005).

$$\xi_p = 5.0 + 44.4\left(\frac{\mu_\Delta - 1}{\mu_\Delta \pi}\right) \quad (in\ \%) \tag{2.4}$$

v) Compute the system damping $\left(\xi_{sys}\right)$ by combining the individual damping part in some weighted average sense. A number of options are available in the literature (e.g., Kowalsky, 2002). In this study, it was used the most general form of system damping where weighting factor is estimated based on the work done by the component under consideration. Work done by any member can be computed multiplying the target displacement by shear force carried by the member. Since at this stage shear force distribution is unknown to the designer, some realistic assumption should be made. The most general assumption is equal strength for all piers, which is feasible only when superstructure spans are equal or not varying significantly. When strength is equal, the shear carried by the pier is inversely proportional to its height, provided all the piers yielded. Otherwise, ductility factor is to be multiplied. For abutment, the shear distribution proportional to their corresponding displacement seems more logical. Combining all components, the final relation is as below.

$$\xi_{sys} = \frac{x\left(\Delta_{sys} - \Delta_a\right)\xi_{SS} + x\Delta_a\xi_a + (1-x)\left(\sum_{piers} Q_{P,i}\xi_{P,i}\right)}{x\left(\Delta_{sys} - \Delta_a\right) + x\Delta_a + (1-x)\left(\sum_{piers} Q_{P,i}\right)} \tag{2.5}$$

$$with\ Q_{p,i} = \frac{1/H_{P,i}}{\sum_{piers} 1/H_{P,i}}\Delta_{P,i} \tag{2.6}$$

where, $\Delta_a$ is the average abutment displacement, $\xi_{SS}$ is the damping associated with elastic transverse bending of superstructure, taken as 5.00% of critical value, $x$ represents the percentage of base shear $V_B$ carried by the superstructure. This relation is valid only for 'all yield' pier condition. When one or two piers remain elastic, ductility factor, which is obvious less than 1.00, is used at the numerator of the base shear distribution relation among the piers.

vi) Compute the effective period $\left(T_{sys}\right)$ of the structure with the known system displacement $\left(\Delta_{sys}\right)$ and system damping $\left(\xi_{sys}\right)$ for a given hazard level usually defined in terms of elastic spectrum as shown in Figure 1 (left). The EC8 type I spectrum for soil type C (CEN, 2004) and peak ground acceleration of 0.50g with a modified corner period $T_C = 4.0$ s instead of 2.0s (see Priestley et al., 2007 for more details) is used in this work. The EC8 relation is used for scaling elastic spectrum for various viscous damping values.

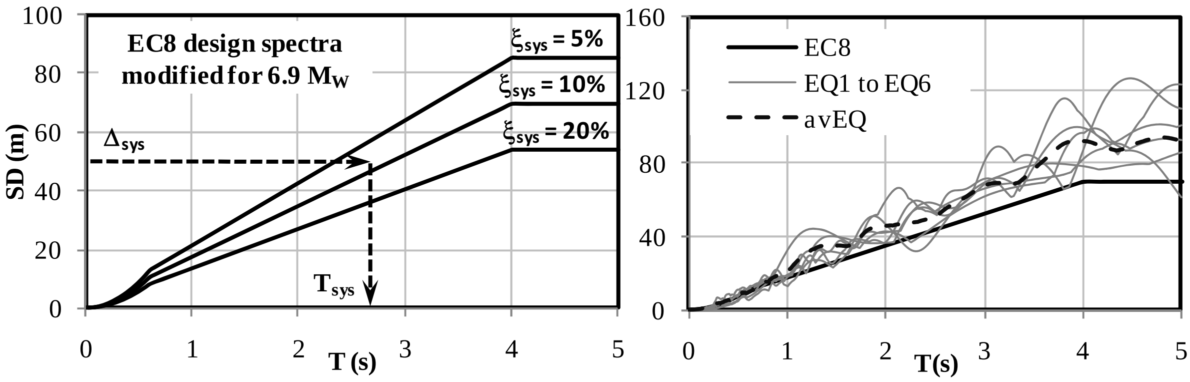



**Figure 1. Left – Schematic representation of effective period calculation, Right – Real records matched with 5% displacement spectrum**

vii) Compute the effective stiffness $\left(K_{sys}\right)$ as well as the design base shear $\left(V_B\right)$ using the relations given by Eqn 2.7. Hence, distribute the base shear among all the lumped mass locations as lateral inertia loading $F_i$ by Eqn 2.8.

$$K_{sys} = M_{sys}\left(\frac{2\pi}{T_{sys}}\right)^2 \quad \text{and} \quad V_B = K_{sys}\Delta_{sys} \tag{2.7}$$

$$F_i = V_B\frac{m_i\Delta_i}{\sum_{i=1}^{n} m_i\Delta_i} \tag{2.8}$$

viii) Now compute the design strength at the potential plastic hinge locations according to the iterative strategy proposed by Priestley and Calvi (2003), where a linear static analysis is performed on the numerical model of the structure, using the lateral inertia force vector calculated by Eqn. 2.8. In the model each element is defined using the effective stiffness which is computed from the ratio of base shear to corresponding displacement Iterative strategy is required because at the beginning of the design process two assumptions were made. First the final (or target) displacement pattern of the structure and second, the percentage of base shear carried by the elastic bending of superstructure. Once these two assumptions converged, the analysis part of DDBD is finished.

ix) Finally, design and detail the structure following capacity design principles.

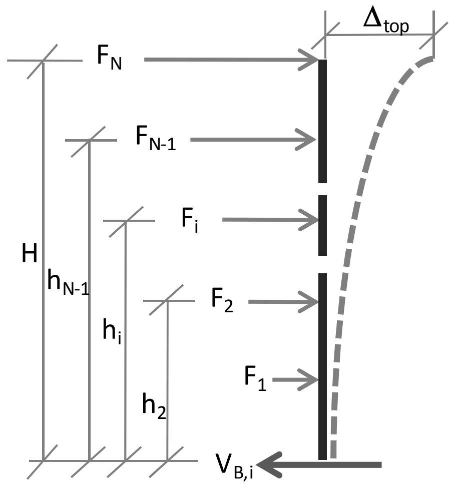
## 3. ISSUES RELATED TO THE LONG SPAN BRIDGES AND PROPOSED MODIFICATIONS

The assumption of equal flexural strength of all pier sections at base, while estimating system damping, is, in general, not valid for long span bridges, where very often span lengths and/or pier heights vary significantly. It results in varying axial load at the pier base and thus varying flexural strength even with same reinforcement content. However, this hypothesis can be easily relaxed by introducing in the Eqn 2.6 a flexural ratio term $(\alpha_P)$, which represents the ratio of flexural strength of the pier under consideration to the maximum flexural strength capacity among all the pier sections, considering appropriate axial load. This factor can be estimated either from load combinations other than seismic condition or from engineering judgment. The modified form of Eqn 2.6 is given by Eqn 3.1.

$$Q_{p,i} = \frac{\alpha_{P,i} / H_{P,i}}{\sum_{piers} \alpha_{P,i} / H_{P,i}} \Delta_{P,i} \tag{3.1}$$

A second issue is related to the numerical modeling of pier column both in terms of stiffness and inertia mass because of tall and massive piers compared to short-to-moderate span bridges. This is illustrated using the following example.

Consider a simple bridge system having a span arrangement of 60-100-60m with pier height of 50m. The total weight of one pier column amounts to 31 MN while tributary deck weight on that pier is 29.7 MN. When 1/3rd of pier weight is lumped at the top of pier, as per conventional practice (Priestley *et al.*, 1996), the total weight for seismic base shear calculation is ~ 40 MN which is ~ 35% less than the actual seismic weight (60.7 MN). This underestimation of base shear will increase further for bridges with taller and massive piers. To avoid this underestimation, a proposal is made based on the principles of statics, whereby the pier element is discretized into a number of sub-elements and tributary masses are lumped at each intermediate node. Since displacement at pier top and lateral force distribution at any stage of DDBD are known, the secant stiffness corresponding to displacement at pier top can easily be calculated from Eqn 3.2, derivation of which is presented graphically in Figure 2.




$$\left.\begin{aligned}
F_N &= V_B - \sum_{i=1}^{N-1} F_i \\
\Delta_{top} &= \sum_{i=1}^{N} \Delta_i = \sum_{i=1}^{N} \frac{\frac{1}{2} F_i h_i \left(H - \frac{h_i}{3}\right)}{E_c I_{sec}} \\
&= \frac{\left(V_B - \sum_{i=1}^{N-1} F_i\right)\frac{H^3}{3} + \sum_{i=1}^{N-1} \frac{1}{2} F_i h_i \left(H - \frac{h_i}{3}\right)}{E_c I_{sec}} \\
E_c I_{sec} \Delta_{top} &= \left(V_B - \sum_{i=1}^{N-1} F_i\right)\frac{H^3}{3} + \sum_{i=1}^{N-1} \frac{F_i h_i^2}{2}\left(H - \frac{h_i}{3}\right)
\end{aligned}\right\} \quad (3.2)$$

**Figure 2 Determination of effective moment of inertia of pier section at base under a set of lateral force vector whose distribution as well as top displacement is known**

Third improvement is proposed to account for higher mode effects at the potential plastic hinge locations using a modified version of Effective Modal Superposition (EMS) (Ortiz Restrepo, 2006; Priestley *et al*. 2007). As defined in its original formulation, the EMS procedure, which is fully compatible with the substitute structure philosophy, uses a Response Spectrum Analysis (RSA) after completion of the DDBD procedure, whereby the stiffness of members having plastic hinges (e.g., piers) is represented by the secant stiffness to the peak displacement response, while elastic members (e.g., superstructure and abutment) are modeled by initial-stiffness values; and seismic hazard is defined by a 5% damped elastic design spectrum. This procedure (in the following mentioned as 'original DDBD - DDBD') was used for determining the design elastic responses e.g., transverse moment at deck, abutment shear force etc, whilst inelastic responses, such as flexural strengths at plastic hinge locations, were taken directly from the first inelastic mode, i.e., from the results of DDBD. This was based on the assumption that only first inelastic mode contributes significantly to the inelastic responses (Ortiz Restrepo, 2007). However, this assumption is not applicable to long span bridges because of inherent flexibility. In fact, for some bridges used in the current study, the estimated mass participation factor for the first inelastic mode is as low as 32% only. Accordingly, in this study, two different design spectra are used instead to determine the design responses at critical locations: 5% damped design spectrum is used for determining the design elastic responses, whereas damped design spectrum with damping value equals to the system damping obtained from the DDBD procedure is used for combining the inelastic responses. With this procedure (in the following mentioned as 'modified DDBD – mDDBD') it is expected to obtain more realistic design values for both elastic and inelastic responses when higher mode contributes significantly to the final design responses. A more detailed description can be found elsewhere (Adhikari, 2007).

## 4. BRIDGE GEOMETRIES AND GROUND MOTIONS

Accuracy of the proposed modifications is then explored through a series of parametric studies using a significant number of long span bridges reproducing real existing bridge geometry: only two case studies are presented here due to space restriction. The methodology followed for verification is: a bridge is first designed following DDBD procedure, considering only seismic load and associated gravity load. Then nonlinear dynamic analyses are performed on the designed structure using a suite of six spectrum compatible ground records as shown in Figure 1. Two response parameters – transverse displacement shape (related to the inelastic response) and deck moment profile (related to the elastic response) – predicted by DDBD procedure are then compared with the average of nonlinear dynamic analyses results. Similar procedure is also followed for current force based design procedure as mentioned in Eurocode 8 (CEN, 2006) to explore its accuracy or inaccuracy in predicting seismic response of current category of bridges. A comparison of reinforcement ratio is also made to understand the impact on the economy of the proposed procedure. A more detail description of the bridges as well as more case studies can be found elsewhere (Adhikari, 2007).

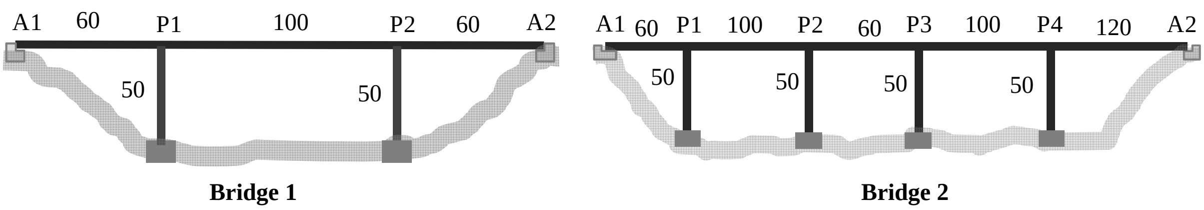



**Figure 3 Schematic diagram of the two case studies showing salient dimensions of the bridges in meter (drawn not to scale)**

## 5. DISCUSSION ON RESULTS

The results in terms of reinforcement content, deck transverse displacements and moments are shown from Figure 4 to Figure 8 with self-explanatory captions: bridge 1 and 2 are reported on the left and on the right, respectively. These plots led to the conclusion that higher modes have, indeed, influence on both elastic and inelastic responses of a bridge, in contrast with the earlier studies (Alvarez Botero, 2004; Ortiz Restrepo, 2006) where it was assumed that higher modes influenced only the elastic responses. This influence is distinctly visible from Figure 5, in which DDBD procedure significantly underestimate the deck displacement, which is also an indicator of inelastic response, at the locations P1 and P2 of bridge 2.

Inclusion of higher modes by mDDBD improvises the results. This also influences the reinforcement content as shown in Figure 4, where DDBD results in a lesser reinforcement content. It is worth noting that these results are applicable for bridges that undergo limited inelastic deformations under design seismic hazard.

Force based design procedure, on the other hand, results in an uneconomical design by increasing reinforcement content significantly, as shown in Figure 4. Moreover, the prediction of both elastic and inelastic response is far from that obtained from nonlinear dynamic analysis (e.g., bridge 1). However, the results improve for the case of bridge 2.

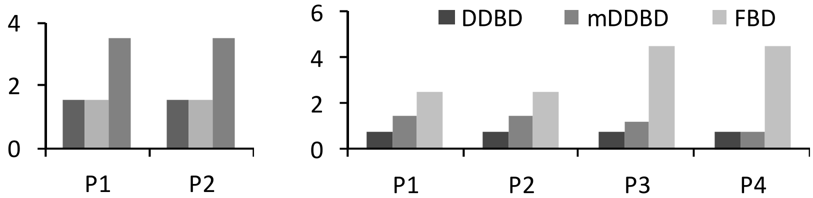



**Figure 4 Reinforcement content (in % of gross sectional area) at pier base obtained from three different design philosophies: original DDBD, modified DDBD and FBD**

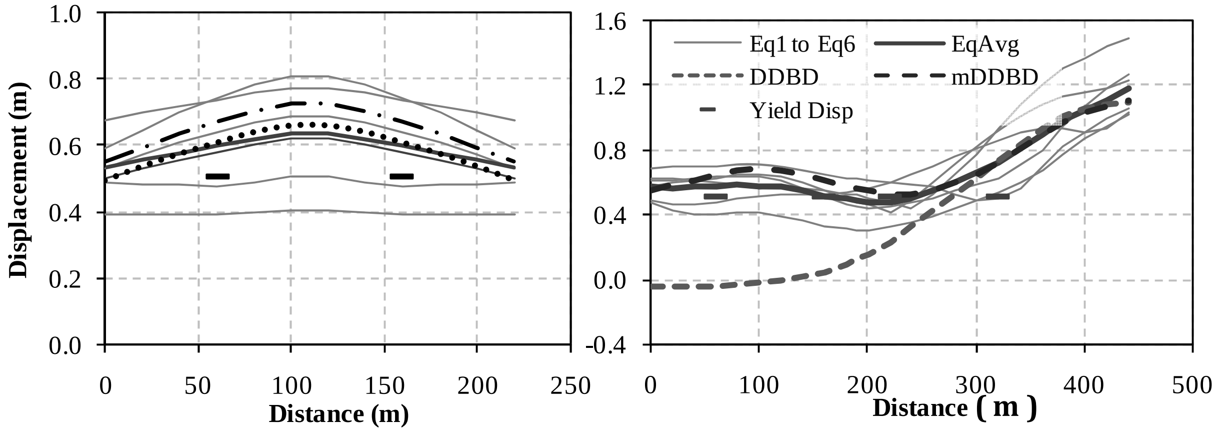



**Figure 5 Comparison of deck transverse displacements (representative of inelastic response) obtained from original DDBD, modified DDBD and nonlinear dynamic analyses**

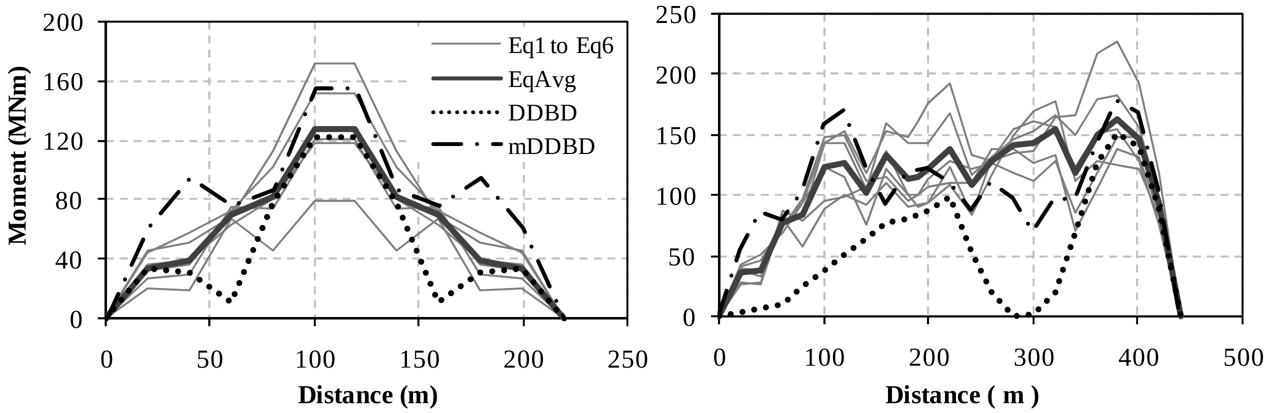



**Figure 6 Comparison of deck transverse moment (representative of elastic response) obtained from original DDBD, modified DDBD and nonlinear dynamic analyses**

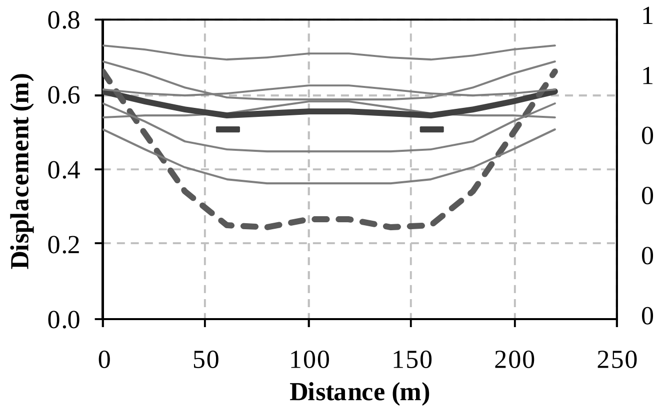



**Figure 7 Comparison of deck transverse displacements (representative of inelastic response) obtained from FBD and nonlinear dynamic analyses**

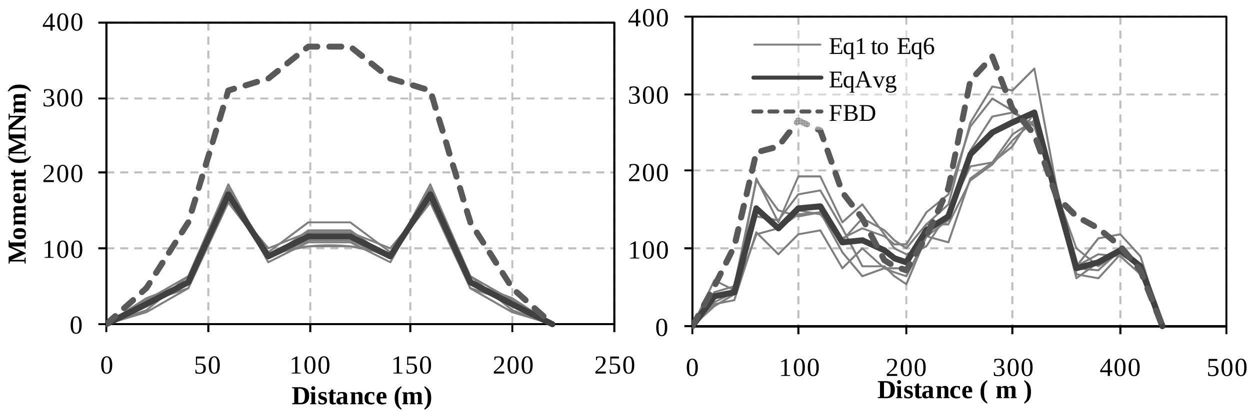



**Figure 8 Comparison of deck transverse moment (representative of elastic response) obtained from FBD and nonlinear dynamic analyses**

## 6. DISCUSSION ON RESULTS

Described in this paper is a study aimed at identifying the potential limitations and subsequent modifications, if required, of the current DDBD procedure for long span concrete bridges under transverse excitation. Three fundamental modifications are proposed here:, the distribution of base shear across the piers and abutments when piers have different moment carrying capacity instead of considering equal strength, consideration of lumped masses along the pier height which is useful for tall massive piers and modal superposition for determining the design inelastic responses also. With these modifications, designers now have more freedom for assigning flexural strengths among the piers and corresponding distribution of shear forces apart from the more economical solution than the current FBD procedure mentioned in the Eurocode 8 (CEN, 2006).

## 7. ACKNOWLEDGEMENT

Part of this work has been carried out under the financial auspices of “Reluis (Rete dei Laboratori Universitari di Ingegneria Sismica) through the project, Progetto esecutivo 2005 – 2008 (Attuazione Accordo di Programma Quadro DPC-Reluis del 15-03-2005) - Linea 4, “Sviluppo di approcci agli spostamenti per il progetto e la valutazione della vulnerabilità”. Such support is gratefully acknowledged by the authors.

The authors would like to acknowledge Prof. Nigel Priestley for the useful discussions on the subject.

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