# **DisplacementBasedDesign for transverse response of rc bridges: evaluation of iterative and direct procedures**

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#### SUMMARY

Transverse response prediction for continuous rc bridges in a displacement-based framework presents some critical issues related to representativeness of the equivalent SDOF structure in respect to the original MDOF system, particularly the estimation of the design displacement profile for irregular bridges.

This paper investigates the accuracy of the current iterative Direct Displacement-Based procedure: parametric analyses were carried out considering multiple configurations of regular and irregular continuous girder bridges with 3 to 5 spans, designed with target drift limits of 1% to 4%, and subsequently checked with non linear time history analyses. It can be seen that for regular bridges the DDBD method can be applied with satisfactory reliability for both low and high ductility design cases, while for irregular bridges the method leads to high overstimations in some cases.

At the same time a non-iterative method based on the effective stiffness and ResponseSpectrumAnalysis is proposed, with the aim of simplifying the current iterative procedure for everyday design use, while maintaining the required accuracy.

Keywords: direct displacement-based design, continuous rc bridges, substitute structure

# **1. INTRODUCTION**

In its current formulation (Priestley et al., 2007) the Direct Displacement-Based Design method in its uses a substitute linear equivalent structure (ESDOF), characterized by a secant stiffness  $K_{eff}$ , and an appropriate level of equivalent viscous damping  $\xi_{eq}$ , in order to represent the seismic behavior of a MDOF system. The equivalent damping value  $\xi_{eq}$ , is used to scale the elastic displacement-spectrum through the correction factor  $R_{\xi}$ , and consequently to calculate the effective period  $T_{sys}$  and the effective stiffness  $K_{sys}$  of the ESDOF system. The calibration of the equivalent damping value  $\xi_{eq}$ , which has to be related to the hysteretic energy dissipated by the structure in the non linear field, introduces a first approximation of the method (Tecchio et al, 2011), which has to be added to an other error component related to the representation of the real system (MDOF) with an equivalent SDOF, through the definition of the target displacement profile.

In the transverse response of a continuous bridge the relative stiffness between deck and piers affects the ultimate displacement profile, depending on the deck transverse stiffness and the type of bearings at the abutments. If the superstructure is effectively rigid and the bearings are very deformable transversally to the bridge axis, the deck reacts like a rigid body, and the design displacement profile is simplified, being a combination of rigid translation and rotation (Dwairi and Kowalsky, 2006). Conversely, when the abutment bearings are fixed transversally, the superstructure is subjected to a transverse global deformation on the entire length of the bridge (with fixed points at the abutments), restraining pier top displacements proportionally to its transversal stiffness.

The inelastic displacement profile is also conditioned by the pier transverse stiffness relative ratios, depending on the pier strengths and ductilities, that are not initially known (Priestley et al., 2007). For this reason, in the case of continuous bridges the current procedure is iterative, being the ultimate displacement shape an input value of the DDBD method.

In this paper a parametric study is performed on the transverse response of multi-span continuous bridges with the abutment bearings transversally fixed; the aim is to evaluate the accuracy of the current DDBD method (called DDBD-IT in this work), quantify the errors for a wide range of bridge configurations in respect to non-linear Time-History analysis, and try to evidence the error components related to equivalent viscous damping calibration (ESDOF system) and the inelastic displacement shape estimation.

At the same time a non-iterative method, herein called DirectEffectiveMethod (DEM), is proposed in order to simplify the DDBD procedure for everyday design use. The approach derives from the Effective Modal Superposition (EMS) initially proposed by Ortiz Restrepo (2006) and subsequently supported by Priestley et. al (2007). The EMS method uses a spectral response analysis (SRA) after completion of the DDBD (iterative) procedure, whereby stiffness of members with plastic hinges (e.g. piers) are represented by secant stiffness to the peak displacement response, while elastic members (e.g. superstructure) are modelled by initial stiffness value, and seismic hazard is defined by a 5% damped elastic design spectrum (Adhikari et al., 2010). In the EMS procedure the final results are obtained combining the higher mode-elastic forces from SRA with the DDBD inelastic first mode design forces using SRSS or CQC combination rule.

The DDBD-DEM method herein proposed is applied for bridges in a non-iterative fashion: the DDBD procedure based on a substitute equivalent SDOF structure is applied in one direct step, assuming an initial displacement profile  $\Delta_{1i}$ , to obtain the effective pier stiffness  $K_{eff, j}$  and the damped function  $\xi_{eq}$ . Subsequently the values  $K_{eff, j}$  are assigned to calibrate the piers' stiffness in a spectral response analysis, where the design spectrum is overdamped according to the  $\xi_{eq}$  value. In this way a better estimation  $\Delta_{2i}$  of the inelastic displacement profile is obtained by normalizing the displacement shape  $\delta_{2i}$  calculated by SRA, to the critical displacement  $\Delta_c$ . The shear forces and moments are calculated directly, assuming the displacement shape estimation  $\Delta_{2i}$  and the values  $K_{eff,j}$  of the first step as sufficient approximation, and the design process is completed.

Using SRSS superposition, the effects of higher modes can be included when significant; it is assumed that ductility substantially influences only the first-mode response (Priestley et al. 2007), and the higher mode effects are the same in the inelastic range as in the elastic range.

In this paper it is shown that the proposed method has the adavantage of being a direct procedure, and maintains the required reliability if compared with the accuracy of the DDBD-IT current approach on the same set of case-studies.

# **2. SEISMIC INPUT**

The reference design spectrum used for the parametrical analysis was derived from the smoothed elastic spectrum "Type 1" presented in EN 1998-1:2004, with the following assumptions: type C soil (S=1.15, TB=0.20s TC=0.6s, TD=2.0s), peak ground acceleration PGA=0.35g, return period  $T_R$ =475years (reference occurrence probability  $P_{LR}$ =10% in a reference period  $T_L$  =50 years).



Figure 1. Acceleration and displacement smoothed design response spectrum sumperimposed with spectra generated by synthetic compatible ground motions

According to the modifications proposed by Faccioli et al.(2004) and supported by Calvi et al. (2009), the corner period value  $T_c$  was modified in order to correlate it to the effective magnitude value acting in situ (a magnitude Mw = 6,9 was assumed). The reference spectrum was subsequently scaled in

order to fit the seismic design intensity levels required for a Class of Importance III (Calvi et al., 2009), through the use of the coefficient of importance  $\gamma_1 \approx (P_L/P_{LR})^{-1/3}$  given in EN 1998-1:2004. The following PGA values were obtained for the three seismic intensity levels considered (T<sub>L</sub>=50 years, Calvi et al., 2009): PGA<sub>1</sub>=0,28g for L<sub>1</sub> level (P<sub>L</sub>=20%), PGA<sub>2</sub>=0,49g for L<sub>2</sub> (P<sub>L</sub>=4%) and PGA<sub>3</sub>=0,77g for L<sub>3</sub> (P<sub>L</sub>=1%).

In the present study the displacement elastic response spectra, are reduced by a scaling factor  $R_{\xi}$ . according to an equivalent viscous damping model  $\xi_{eq}$  calibrated with reference to the Takeda Thin hysteretic law (well-representative of structural elements with significant axial loads, such as bridge piers). The following expression are used ( $\mu_{\Delta}=\Delta_u/\Delta_v$  represents the displacement ductility):

$$R_{\xi} = (0.10 / 0.05 + \zeta))^{0.5} \tag{2.1}$$

$$\zeta_{eq} = 0.05 + 0.444(\mu_{\Delta} - 1) / \mu_{\Delta}\pi$$
(2.2)

Seven synthetic acceleration records, compatible with the proposed design spectra were generated with SIMQKE program (Gasparini and Vanmarke, 1976), and used as input ground motions in nonlinear Time History analyses for the verification study. The seismic input to all piers is assumed coherent and in phase: possible effects due to spatial variability of ground motion are not considered.

### **3. CASE-STUDIES SET**

In the parametric study, a set composed by 36 different bridge configurations was analyzed; 8 different four-spans bridge geometries and other 10 with six-spans were considered. Two different deck types were adopted, a PrestressReinforcedConcrete (PRC) box girder deck, and a composite SteelConcrete (SC) deck. The PRC deck is characterized by a transverse bending stiffness about three times higher than the SC deck (for simplicity the SC deck was replaced in the F.E. model with an equivalent box steel section). Deck properties are reported in Table 3.1, and all bridge geometrical configurations are presented in Table 3.2. Concrete C40/45 and reinforcement steel B450C were used for piers, while concrete C75/85 for PRC deck and structural steel S355were used for deck materials.

Each bridge is identified by the deck code and the specific sequence of piers height values (e.g. PRC132), where H=1 is the reference height equal to 4.0m. All piers are single cantilevers, with circular section of variable diameter D (specified in Table 3.2); in transverse direction the superstructure is assumed to be connected to the piers with fixed bearings, and lateral restraints are provided at the abutments. A relative stiffness index RS can be introduced to relate superstructure and piers' transversal stiffness (Priestley et al., 2007):

$$RS = K_{S} / \sum_{i=1}^{n} K_{P_{i}}$$
(3.1)

where  $K_s$  is the transversal stiffness of the deck, derived from the static scheme of a simply-supported beam between the abutments undergoing a uniform load,  $K_s = 384/5 \cdot (EI)_s/L_s^3$ , and  $K_{Pi} = 1/(1/K_f + 1/K_v)$  is the transverse pier stiffness  $(K_{fi} = 3(EI)_{Pi}/H_{Pi}^3)$  and  $K_{vi} = GA_{Pi}/H_{Pi}$ , negligible for slender piers). In this paper the RS index is calculated considering the effective pier stiffness  $K_{Pi,e}$ , taking as yield secant stiffness the initial value reduced to 60% and then scaling it by the ductility factor  $\mu_{\Delta}^{j}$  for a drift level  $\theta = 3\%$  ( $\mu_{\Delta}^{j}$  is calculated considering  $\Delta_{u}^{j}$  obtained directly from the design drift, and estimating the yield displacement  $\Delta_{v}^{j}$ ).

<b>Fable 3.1.</b> Properties of the PRC deck and	d the equivalent steel box	, substitutive of the com	posite SC deck.
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PRC - A= $7.3m^2$  J<sub>v</sub>= $81m^4$ , E=40,82GPa W=220 SC - A= $0.75m^2$  J<sub>v</sub>= $5.4m^4$ , E=206GPa W=150 kN/m

Table 3.2. Bridge configurations and RS values	
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Geometric configuration				Bridge Code	D <sub>pier</sub> (m)	RS		
	× 4	10 m 50	) m 50	0 m <u>* 40 m</u>				
	A1	Н 2Н	- SH	- 2H	A2	PRC/SC 222	1.95/1.85	4.2/1.6
		P1	P2	p3		PRC/SC 131	1.65/1.50	5.0/2.2
	A1	P1		P3	A2	PRC/SC 323	2.00/1.95	5.0/1.8
	A1	Зн	2H	Зн	A2			
			P2			PRC/SC 132	1.95/1.80	3.5/1.5
	A1	H P1	ЗН	P3	A2			
	Δ1		P2 - 3H	НЗН	A2	PRC/SC 133	2.00/1.85	3.9/1.7
		P1	p2	P3				
	A1	<u>II H</u> P1	2н	ЗН	A2	PRC/SC 123	2.00/1.90	3.2/1.3
	A1	2н	- 2н	4H	A2			
		P1	P2	- Ep3		PRC/SC 224	2.30/2.20	3.8/1.5
	A1	ЗН	ен	- 4H	A2		2 20/2 20	1 1/1 6
		P1	P2	P3		PKC/SC 324	2.30/2.20	4.1/1.0
40 مر م	m <u>y 50 m</u>		n <u>*</u> 50 m		40 m	PRC/SC 22222	2.00/1.80	1.7/0.8
A1	Нен	<u>Н</u> 2н	2H	<u>H</u> 2H		2		
A1	Н гн	2н	- 3н	2н		PRC/SC 22322	2.05/1.85	2.2/0.9
	P1	P2	 P3	P4	P5			
A1	3н	Егн	ен	ен	ЗН	PRC/SC 32223	2.00/1.80	2.1/1.0
	P1	P2	P3	P4	P5	1		
A1	- 4H	ЗН	2н	ЗН	4H A:	PRC/SC 43234	2.30/2.10	2.2/1.0
	P1	P2		P4	<u></u>		//	
A1	ЗН	Егн	4H	Ен	ЗН АЗ	<b>PRC/SC 32423</b>	2.20/1.95	2.2/1.0
	P1	P2	 P3	P4	P5	DDC/SC 11124	2 50/2 20	12/06
A1	Егн	2н	2H	ЗН	4H A:	<b>PRC/SC 22234</b>	2.30/2.30	1.5/0.0
	P1	P2	P3	<u>II</u> P4	 P5	PRC/SC 11313	2.00/2.00	1.1/0.4
A1			ЗН		ЗН	2		
	PI	P2	P3	P4	P5			
A1			ЗН	2H		<b>PRC/SC 11321</b>	1.90/1.80	1.6/0.6
	P1	P2 ~	~_ <u>_</u>	P4		1		
A1	Нен	ен	E 2H	6H	EH A	<sup>2</sup> PRC/SC 22262	2.65/2.55	1.7/0.7
	P1	P2	P3		r 45			
				P4		]		
A1	<u>Н</u> 2н	Нен	<u> </u>	4H	<u>Н 2н</u> аз	<sup>2</sup> PRC/SC 26242	2.70/2.55	1.1/0.4
		L/		P4				
		P2						

# 4. PERFORMANCE CRITERIA AND DESIGN PROCEDURES

Two values of maximum drift  $\theta$  were considered for of each sample in the case-study set as performance criteriafor high ductility design, according to the reference values proposed by Calvi and Sullivan (2009): drift limit  $\theta$ =3% was defined for Level 2 (damage-control) of earthquake design intensity, while value  $\theta$ =4% was chosen for L3 level (collapse prevention), though representing probably an upper limit for usual design. In addition a very low drift  $\theta$ =1% was considered for serviceability limit state (Level1) in order to obtain low ductility design cases, with piers' mean ductility values close to 1.

Two different displacement based-design procedures were evaluated in this work: the first one is the

current DDBD (DDBD-IT) iterative design procedure for the transverse response of continuous bridges, as revised by Priestley et al. (2007).



Figure 2. Flowchart of the current DDBD-IT procedure for transverse design of continuous bridges.



Figure 3. Flowchart of the proposed non iterative DDBD-DEM procedure.

In Fig. 2 the flowchart of the iterative procedure is presented; more details of the DDBD-IT method and analysis parameters can be found elsewhere (Priestley et al. 2007). In this work a sine-based mode shape was assumed as initial displacement estimate (step a), but a different choice (e.g. the first modal shape, calculated with cracked stiffness for piers) does not substantially influence the design displacement profile finally achieved, the procedure being iterative; it affects only the number of iterations required for convergence.

In the DirectEffectiveMethod (DEM) herein proposed, the DDBD procedure based on a substitute equivalent SDOF structure is applied only in one direct step. The procedure needs the support of an elastic F.E. model of the structure, because linear static analyses (LSA) and a spectral response analysis have to be carried out.

The design process can be summarized as follows:

a) Initial displacement shape estimate. The initial displacement vector  $\delta_{ii}$  is assumed as the first modal shape. It is suggested to perform a modal analysis with a cracked stiffness for piers, reducing it uniformly for all piers, or better (as in the examples presented in this paper) taking as yield secant stiffness the initial value reduced to 60%, and then scaling it for each pier trough the displacement ductility factor  $\mu_{\Delta}^{j}$  (that can be obtained directly from the design drift). This displacement shape is then normalized to the critical displacement  $\Delta_c$ , to obtain the initial displacement profile:

$$\Delta_{1i} = \delta_{1i} \left( \frac{\Delta_c}{\delta_c} \right) \tag{4.1}$$

b) Estimate of the lateral force fraction carried by superstructure. The value of the lateral force fraction x, carried by superstructure, can be calculated trough a static analysis (LSA) of the structure with imposed transverse displacements  $\Delta_{Ii}$  and pier stiffness calculated before.

$$V_{a1} + V_{a2} = xV_{base} \tag{4.2}$$

c) Determination the ESDOF system properties and displacement. Effective displacement  $\Delta_{eff}^d$ , mass  $M_{eff}$ , height  $H_{eff}$  and damping  $\xi_{eff}$  of ESDOF system have to be evaluated as in the typical DDBD design process, by using the following expressions:

$$\Delta_{eff}^{d} = \sum_{i=1}^{n^{\circ} mass} \left( m_{i} \Delta_{i}^{2} \right) / \sum_{i=1}^{n^{\circ} mass} \left( m_{i} \Delta_{i} \right)$$

$$(4.3)$$

$$M_{eff} = \sum_{i=1}^{n^{\circ} mass} (m_i \Delta_i) / \Delta_{eff}^d$$

$$H_{eff} = \sum_{i=1}^{n} (m_i \Delta_i h_i) / \sum_{i=1}^{n} (m_i \Delta_i)$$

$$(4.4)$$

$$(4.5)$$

where  $m_i$ ,  $\Delta_i$ ,  $h_i$  are respectively the i-th mass, its displacement and height ( the offset due to the deck height is accounted in the calculation of  $h_i$  in respect to  $H_i$  of the pier).

$$\xi_{eff} = \frac{x\Delta_{eff}^{d} \cdot 0.05 + (1-x)\left(\sum_{j=1}^{n^{\circ} piers} \frac{C}{H_{j}} \cdot \Delta_{j}\xi_{j}\right) / \sum_{j=1}^{n^{\circ} piers} \frac{C}{H_{j}}}{x\Delta_{eff}^{d} + (1-x)\left(\sum_{j=1}^{n^{\circ} piers} \frac{C}{H_{j}} \cdot \Delta_{j}\right) / \sum_{j=1}^{n^{\circ} piers} \frac{C}{H_{j}}}$$
(4.6)

where H<sub>j</sub>,  $\Delta j$ ,  $\xi j$ , are the height, top displacement and damping, calculated with Eq.(2.2), of the j-th pier . Elastic damping (5%) is adopted for the superstructure, and its displacement is assumed as equal to the system displacement  $\Delta_d^{eff}$ .

The coefficient C=1 is taken for yielded piers, while the modifying factor  $C = \mu_{\Delta}$  has to be assumed for piers remaining elastic under seismic excitation.

d) Determination of the design base shear of the ESDOF system. Determination of the effective period  $T_{eff}$ , entering the displacement spectra (damped trough the  $R_{\xi}$  factor, Eq. 2.1) with  $\Delta_{eff}^d$ . The effective stiffness  $K_{eff}$  and the total base shear  $V_{base}$  (accounting fo the P-D effects) are then calculated as follows:

$$K_{eff} \cong 4\pi^2 \frac{M_{eff}}{T_{eff}^2} \tag{4.7}$$

$$V_{base} = K_{eff} \,\Delta^d_{eff} + 0.5 \,P \,\Delta^d_{eff} \,/ \,H_{eff} \tag{4.8}$$

e) Estimate of the effective stiffness of piers. Distributing total base shear  $V_{base}$  for each pier in a simplified way (proportional to " $1/H_j$ " for yielded piers, and to " $\mu_{\Delta}/H_j$ " for elastic piers), the i-th pier effective stiffness estimate  $K_{eff,j}$ , is obtained as follows:

$$V_j = (1-x) V_{base} \left(\frac{c}{H_j} / \sum_{j=1}^{n^\circ piers} \frac{c}{H_j}\right)$$
(4.9)

$$K_{eff j} = V_j / \Delta_{1j} \tag{4.10}$$

f) *Estimate of the modal effective shape*. A spectral response analysis (SRA) is performed to obtain a better estimate of the inelastic effective shape  $\delta_{2i}$ .  $K_{eff j}$  values for piers and a displacement spectrum

damped by the factor  $R_{\xi}(\xi_{eff})$  are used.

g) *Estimate of the inelastic design profile.* The modal effective shape  $\delta_2$  determined at the previous step is normalized with the Eq. 4.1 to the critical displacement  $\Delta_c$ , to obtain the inelastic design profile estimate  $\Delta_{2i}$ .

**h**)*Estimate of the design strength required.* Shear force  $F_j$  carried by each pier is calculated considering the obtained displacement profile  $\Delta_{2i}$ , and the previous estimate of piers' effective stiffness  $K_{eff j}$  (Eq. 4.10). The design moment  $M_j$  is finally calculated.

$$F_{j} = K_{eff j} \cdot \Delta_{2j}$$

$$M_{basej} = F_{j} \cdot H_{j}$$

$$(4.11)$$

$$(4.12)$$

g)*Reinforcement design.* Reinforcement in critical sections is designed for forces estimated in the previous step.

### **5. VERIFICATION STUDY**

The accuracy of the DDBD-IT and DDBD-DEM procedures were evaluated trough rigorous nonlinear TimeHistoryAnalyses (THA) using the free available software Opensees (2006); numerical models reproduce the 3D real bridges' geometries, incorporating the realistic distribution of mass and stiffness, and using BeamColumn elastic elements for the superstructure and fiber-section representation for piers (see Fig.4). The material properties are summarized in Table 5.1.

**Table 5.1.** Concrete and steel models used for the verification study with THA: Opensees Concrete 02 model for confined and unconfined concrete, and Menegotto-Pinto model for steel reinforcement





Figure 4. Conceptual model considered for the TH analyses.

In the verification process, each of the 36 bridge samples (18 geometrical configurations for pier section and heights and 2 deck types being used, as described before) was designed according to the DDBD-IT and DDBD-DEM procedure, for the 3 adopted different performance levels ( $\theta$ =1,3,4%), and detailed with the reinforcement required, 216 structural designs in all being executed. Then each designed bridge was subjected to a suite of 7 ground motions (3 accelerograms series for the 3 different design spectra adopted), for a total of 1512 non-linear Time-History analyses.

A typical example of the complete output obtained for the DDBD-IT and DDBD-DEM verification study is reported in Tab.5.2, in the case of one symmetric bridge (PRC22322). The main properties of the ESDOF system are reported ( $K_{eff}$ ,  $\xi_{eq}$ ,), as well as the piers' required ductility and piers' design shear and moment. The design displacement profiles superimposed with the displacement profiles

obtained by the THA are then reported in Fig. 5, for the DDBD-IT and DDBD-DEM methods relating a sample of PRC six-spans symmetric and non symmetric bridges.

**Table 5.2.** Example of typical output obtained for DDBD-IT procedure: a) superposition of the design displacement profiles with THA results, and b) design properties. Bridge PRC22322, design drift  $\theta$ =3%.



**Figure 5.** Six-spans bridges with PRC deck. Output obtained for DDBD-IT and DDBD-DEM procedures: relative errors and superposition of the design displacement profiles with THA results. Design drift  $\theta$ =3%.

The mean error, indicating whether the displacement design shape is on the whole a reliable representation of the real inelastic displacement profile, is calculated as follows:

$$E_m = \sum_{i=1}^{N_p} \left| \Delta_i^D - \Delta_i^{TH} \right| / \sum_{i=1}^{N_p} \Delta_i^{TH}$$

$$(5.1)$$

where N<sub>p</sub> is piers' number,  $\Delta_i^{\nu}$  is the i-th pier design displacement,  $\Delta_i^{IH}$  is the i-th pier top displacement obtained by THA. The minimum and maximum error for each pier were also calculated (E<sub>min</sub> is reported in Fig.6 to highlight underestimations), as E<sub>max,min</sub>=max,min( $\Delta_i^{D}-\Delta_i^{TH}$ )/ $\Delta_i^{TH}$ .

 $E_{esdof}$  error is related to the equivalent viscous damping calibration and is obtained by comparing the ultimate design displacement of the equivalent SDOF system (ESDOF) with THA displacement:

$$E_{esdof} = \left(\Delta_{esdof}^{D} - \Delta_{esdof}^{TH}\right) / \left(\Delta_{esdof}^{TH}\right)$$
(5.2)

where  $\Delta^{D}_{esdof}$  is the design displacement of DDBD procedure, while  $\Delta^{IH}_{esdof}$  is the ultimate

displacement obtained by the ESDOF with non-linear THA (an elastic-perfectly plastic Takeda Thin model was assumed for the ESDOF hysteretic law).

# 6. COMPARISON OF RESULTS

The verification study results for the two compared methods DDBD-IT and DDBD-DEM are presented in Fig.6. It can be observed that the DDBD-IT method is almost always conservative, being  $E_{min}(DDBD-IT)>0$  except in single cases, and the overestimation error tending to increase in the inelastic range for high ductility design cases (i.e. for high drift limit design cases).



Figure 6. DDBD-IT and DDBD-DEM methods: relative errors respect to THA medium displacement results

The accuracy of the DDBD-IT method appears to be closely related to structural regularity: when applied to very regular bridges, corresponding to uniform or "v-shaped" symmetric configurations with high values of RS index (approximately RS>2), the method is reliable, with a low error range with respect to TH analyses. For low-ductility design cases ( $\theta$ =1%), the mean error range is  $E_M(DDBD-IT)<20\%$ , and remains less than 35% for high ductility design cases corresponding to  $\theta$ =3%, ( $E_M < 45\%$  for drift  $\theta$ =4%, but this represents a drift upper limit for common design).

Considering all symmetric bridges (on the left of the graphs in Fig.6) the mean error range is  $E_M < 25\%$  for  $\theta=1\%$  and  $E_M < 50\%$  for high ductility design cases; this overestimation could be considered still acceptable on the basis of the significative approximations introduced by the simplified method. The same error range is valid also for non symmetric bridges with RS>2; this means that the ESDOF system is quite representative also for non-symmetric bridges with a very rigid superstructure dominating the response. For other cases a verification with non linear THA is required; in particular for non-symmetric bridges with RS<1 the error range is unacceptable, reaching more than 80%. Results show also that  $E_{esdof}$  is a small component of the total error, rarely exceeding the value of 10%. As regards the DDBD-DEM method herein proposed, the results show that, though it's a direct method, it enhances the accuracy of the current procedure, especially for high-ductility design cases. As can be seen from the general error trend, DDBD-DEM generally leads to better results, not only for symmetric bridges (with an enhancement of the 20-25% of the mean error  $E_M$ ), for which the iterative current method is already accurate enough, but in particular for irregular cases, being the medium error  $E_M$ (DDBD-DEM) always within the range 55% in respect to THA results.

## 7. CONCLUSIONS

The parametric study carried out in this work shows that the DDBD-IT leads to design overestimation for the transverse response of RC continuous bridges. It can be observed that the mean error  $E_{M}$ (DDBD-IT), with respect to THA, increases significantly with the ductility demand, and although it is relevant, in most cases it can be considered acceptable if compared with the significative simplifications introduced by the design method. The best results were obtained for very regular bridges (uniform or "v-shaped" symmetric pier configurations with high values of RS index), but in general quite reliable results could be obtained for all the symmetric bridges and also for the irregular bridges with high values of deck-pier transversal stiffness ratio RS. In all these cases the substitute ESDOF system is still representative of the MDOF original structure, and the mean error value E<sub>M</sub> is lower than 25% for low-ductility design cases ( $\theta$ =1%), and never exceeding the range 50% for a design drift upper limit  $\theta$ =4%. As regards the non-iterative procedure (DDBD-DEM) proposed, it offers the advantages of a direct design, generally leading also to better estimates too: the results show that the method suggested enhances the accuracy of the current DDBD-IT procedure not only for symmetric bridges (with a decrease of the 20-25% of the mean error in respect to  $E_M(DDBD-IT)$ ), but in particular for irregular cases, where the iterative procedure DDBD-IT leads to very high overestimates, and a verification with non linear THA is consequently required.

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