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DISPLACEMENT-BASED SEISMIC DESIGN OF CONCRETE CONTINUE BRIDGES

Hamid Assarzadeh

Entrepreneur Mahan Gostar Dehkadeh Corporation
e-mail: Hamid.Assarzadeh@yahoo.com

ABSTRACT: Several efforts have been made in the last decades to address the importance of changing the focus of current seismic design codes from merely preventing collapse in major earthquakes and controlling the damage in minor earthquakes to a more general design philosophy, which takes into account multiple performance objectives based on quantifiable performance criteria; this design philosophy is referred to as Performance Based Design (PBD).

A displacement-based seismic design procedure is proposed and elaborated for concrete bridges with continuous deck integral with the piers. It includes a simple estimation of inelastic deformation demands (chord or plastic hinge rotations in piers, curvatures for the deck) via elastic 5%-damped modal response spectrum analysis [4]. The applicability of the equal displacement rule at the level of member deformations is checked through nonlinear static analyses of one representative regular four Spans Bridge, steel box girders and piers of various cross-sections and about equal and very different heights.

A four-span bridge of 100 meters in total length was analyzed using both the Nonlinear Static Procedure and Displacement-based design method.

The main aim and product of the research line has been the formation of a model code for displacement-based seismic design.

KEYWORDS: Concrete bridges, Displacement-based design, Nonlinear Static Procedure, Eurocode 8, Seismic design of bridge, higher modes

1 INTRODUCTION

Seismic design is currently going through a transitional period. Most of the seismic codes to date utilize force-based seismic design, or what can also be called strength-based design procedures. However, it is now widely recognized that force and damage are poorly correlated and that strength is of minor importance when designing for earthquake loading than for other actions.

These, together with other problems and inconsistencies with force-based design, [Priestley, 2003], have led to the development of more reliable seismic design methodologies under the framework of what has been termed Performance-Based Seismic Design (PBSD). PBSD represents basically the philosophy of designing a structure to perform within a predefined level of damage under a predefined level of earthquake intensity.

2 PROPOSED DISPLACEMENT-BASED SEISMIC DESIGN PROCEDURE

The displacement-based design of multi-degree-of-freedom Bridge structures is based on the concepts the displacement-based design of single-degree-of-freedom. However, some specific issues must be considered carefully during the process. The design displacement shape is a function of the relative stiffness between columns, abutments and the deck. Resistance to transverse seismic excitation is mainly provided by bending of the bridge piers, which are designed to respond inelastically and, if the abutments provide some restraint to transverse displacements, superstructure bending will also develop.

In normal seismic design practice the bridge deck is required to remain elastic under the design level earthquake. As a consequence the seismic inertia forces developed in the deck are taken by two different load paths, one portion is transmitted to the piers foundations by column inelastic bending and the remainder transmitted to the abutments by superstructure elastic bending.

The portion of load carried by each of the two different load paths is unknown at the start of the design process and depends strongly on the relative effective-column and deck stiffness's as well as on the degree of lateral restrain provided by the abutments. Since column stiffness's are also unknown at the start of the design process, an iterative procedure is required.

The design procedure presented here considers the discretization of the deck mass as lumped masses at the top of the piers and at the abutments. A portion of the column masses and the cap beam masses can also be lumped at the top, following the recommendations given in [Priestley, et al., 1996].

The *Direct Displacement-Based Design* procedure for multi-degree-of-freedom bridge structures can be summarized according to bottom flowchart:

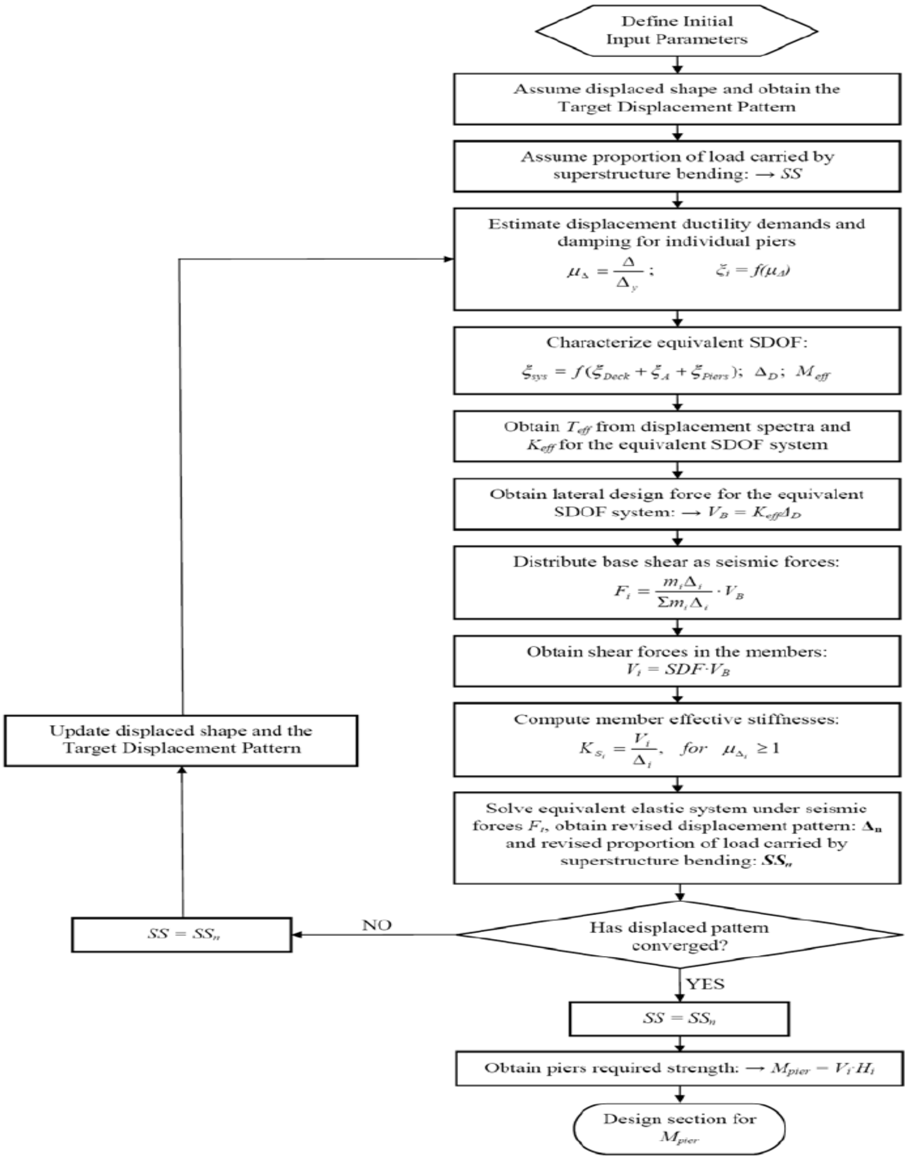

Figure 1. Flowchart for direct displacement - Based design of MDOF-bridges

3 REGULAR AND IRREGULAR BRIDGE CONFIGURATIONS

As previous studies were done in bridges with regular configurations [Alvarez Botero, 2004], in this paper a Regular Bridge will be defined as a bridge in which the structure center of mass, CM, coincides with the structure center of strength, CV. In this case the translational modes of vibration rule the seismic response and the rotational ones are not excited and consequently do not participate in the seismic response of the structure.

An Irregular Bridge will be defined as a bridge in which the structure center of mass, CM, does not coincide with the structure center of strength, CV. In this case, the seismic response is a combination of the translational and rotational modes of vibration.

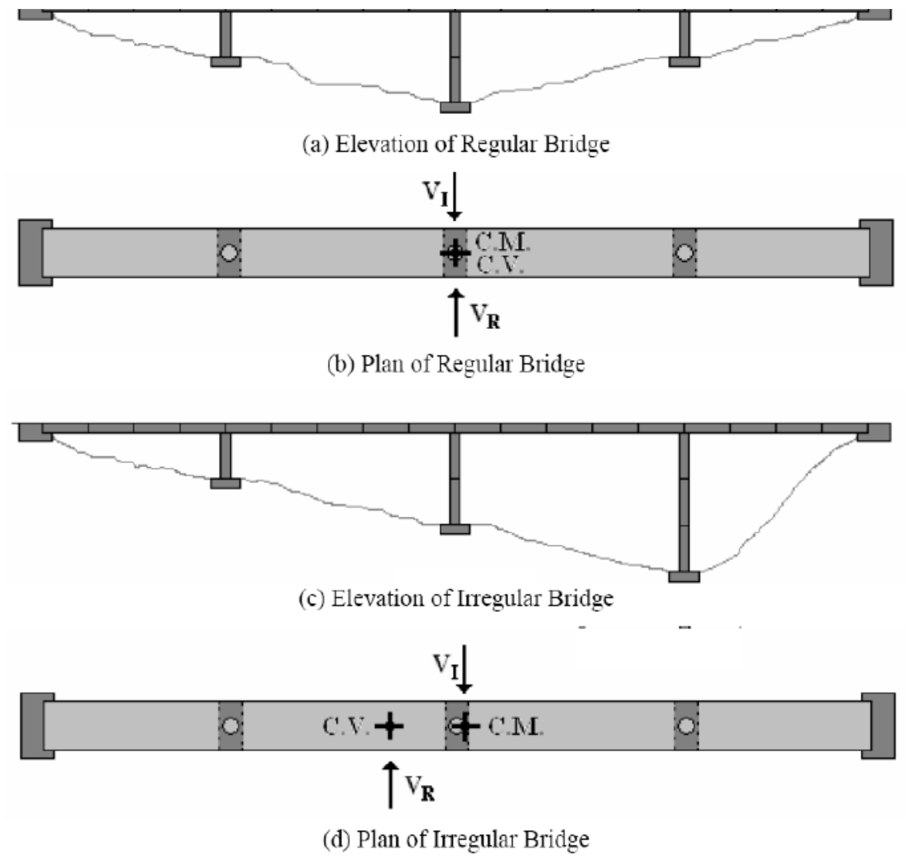

Figure 2. Regular and irregular bridges [3]

3.1 Design displaced shape

Bridge structure composed by several columns connected to a superstructure of defined flexibility will deform in a manner that is influenced by variations in strength, stiffness and mass distribution. The transverse displaced shape will depend strongly on the relative column stiffness, and more considerably, on the degree of lateral restrain provided at the abutments. Figure 3 depicts two different bridge configurations and the possible transverse displaced shapes indicated for the different abutment conditions.

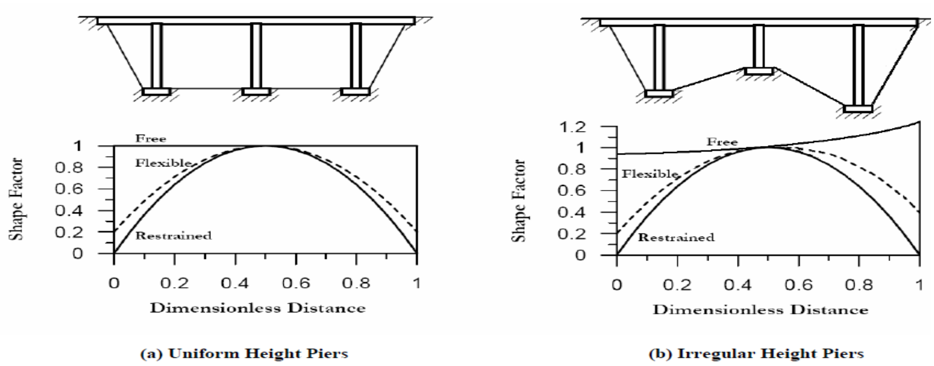

Figure 3. Possible transverse displacement shapes for continuous bridges [3]

Generally a parabolic displaced shape between abutments and piers can be initially assumed for design purposes.

Deck first-mode deformed shape can be obtained either by solving the Eigen-problem for the deck or by using an approximate first mode shape function as the one shown in Eq.(1) based on a parabolic loading shape.

4 MODELING ISSUES

A simplified plan model of the structure, as depicted in Figure 4, was constructed for each of the bridges. The bridge deck was modeled by means of elastic frame elements. Piers were characterized by inelastic springs, while the abutments by linear elastic springs and dashpots that represented the additional elastic energy dissipation associated to them.

4.1 Bridge design

Described in this section, is one design example to demonstrate the direct displacement-based design approach for continuous bridges. The example utilizes the results given in table 2 to identify the displacement pattern and design the bridge for a rigid translational deflected shape and a result of nonlinear static analyses for the bridge were shown in Figures 3, 4.

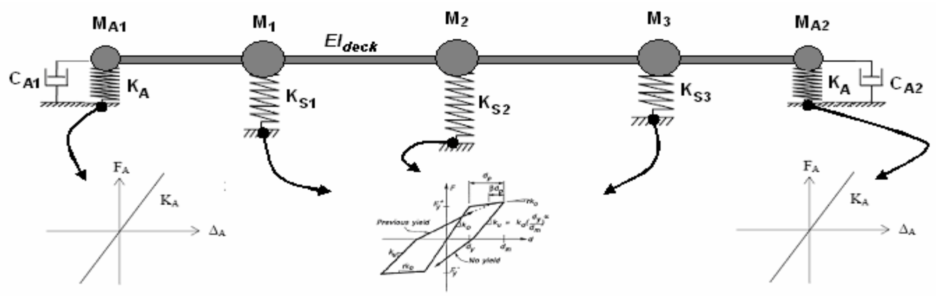

Figure 4. Typical simplified plan model of bridge used in time-history analysis [3]

Step 1: Define initial input parameters

Bridge information and assumption material

Concrete and reinforcing steel properties used for design purposes are presented in table:

Table 1. Material properties for design

Concrete				
f'c=28	MPa	compression strength		
Ec=26457.51	MPa	Elastic modulus		
Wc=24	kN/m^3	unit weight		
Reinforcing steel				
fy=420	MPa	yield stress		
Es=200000	MPa	Elastic modulus		
dbl=0.036	m	Longitudinal bar diameter		
Abutment				
K_A=K_B	3.75E+04	kN/m	m_{A1}=3570	kN
ζ=(%)	0.08		m_{A2}=3570	kN
$\Delta_{(Abutment)1,2}$=0.1m				

Bridge Deck

I_{yy} = 1.47m^4	Elastic modulus = 26457.51MPa	
W_{deck} = 175KN/m	L = 100m	ζ=5%

Column

$H_{1,3}$ = 11M	H_2 = 22m	ζ =0.0021
$W_{1,3}$ (Column+ Cap beam) = 1343KN	W_2 (Column+ Cap beam) = 870KN	
$X_{1,4}$ = 20M	$X_{2,3,}$ = 30m	
D1 = 2M	D2 = 2.2m	
$m_{1,3}$ = 10470KN	m_2 = 15534KN	

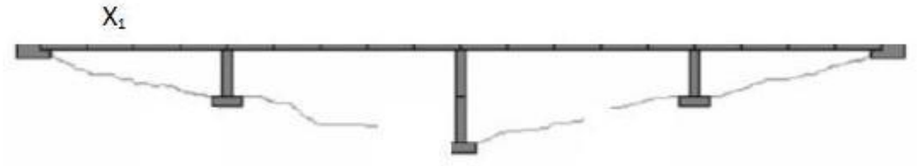

Figure 5. Elevation and position of regular bridge

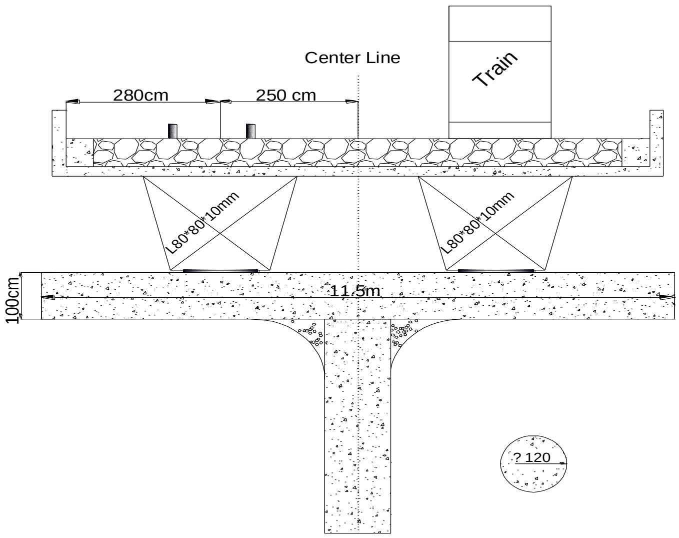

Figure 6. Cross section of the bridge

It is worth mentioning that for this work abutment have been assumed to behave elastically, and the procedure presented based on this assumption.

Step 2: Assume displaced shape and obtain the target displacement pattern based on an approximate first mode shape function

$$\delta_i = (16/5L^4)*(x^4-2Lx^3+L^3*x) \quad (1)$$

(approximate first mode shape function)

Evaluation of the limiting displacements for each of the piers is also required in order to determine the target displacement pattern. Normally the shortest column governs the selection of displacement pattern. For more details about the definition of each of the previous parameters and how to estimate it, see Priestley et al., 1996[1].

Table 2. Approximate first mode shape function

$\delta(A_1)$=	0.074084 m
$\delta(x_1)$=	0.44 m
$\delta(x_2)$=	0.740841 m
$\delta(x_3)$=	0.44 m
$\delta(A_2)$=	0.074084 m
$\delta(x)=<\Delta=0.04*h$	

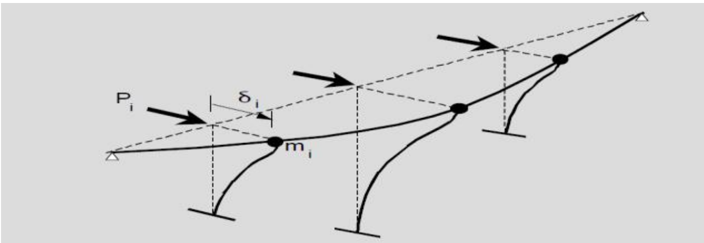

Figure 7. Approximate first mode shape

Step 3: Assume proportion of load carried by superstructure bending SS= -0.01

Step 4: Estimate displacement ductility demands and damping for individual piers

Δy, for a cantilever pier is given by Eq. (2), where ky is the yield curvature and He is the effective pier height, that considers yield penetration, [Priestley et al., 1996].

$\Delta_{y1}=k_y*H_e^2/3=$	0.20227525m	(2)
$He_1=H+Lsp=$	11.33264m	(3)
$Lsp_1=0.022fy*dbl=$	0.33264m	(4)
$ky_1=2.25*\Delta y/D=$	0.004725	(5)
$\Delta_1=$	0.44m	
$\mu_{\Delta 1}=\Delta/\Delta y_{)1}$	2.17525378	(6)
$\zeta_1=0.05+0.444*(\mu-1)/(\pi\mu)=0.126358$		(7)
$\Delta_{y2}=k_y*H_e^2/3=$	0.65460519m	(2′)
$He_2=H+Lsp=$	22.33264m	(3′)
$Lsp_2=0.022fy*dbl=$	0.33264m	(4′)

ky_2=2.25*Δy/D=	0.0039375	(5′)
Δ_2=	0.74084052m	
$\mu_{\Delta 2}$=Δ/$\Delta y_{)2}$	1.1317364	(6′)
ζ_2=0.05+0.444*(μ-1)/($\pi\mu$)=0.066451		(7′)
Δ_{y3}=Δ_y*H_e^2/3=	0.20227525m	(2′′)
He_3=H+Lsp=	11.33264m	(3′′)
Lsp_3=0.022fy*dbl=	0.33264m	(4′′)
ky_3=2.25*Δy/D=	0.004725	(5′′)
Δ_3=	0.44m	
$\mu_{\Delta 3}$=Δ/Δy_3	2.17525378	(6′′)
ζ_3=0.05+0.444*(μ-1)/($\pi\mu$)=0.126358		(7′′)

Step 5: Characterize equivalent SDOF

ζ_{sys}= Δ(vi*ζi)/Σvi=0.114	(8)
Δ_{sys}=Δd=Δ(T,5)=Δmi*Δi^2/Σmi*Δi= 0.59381m	(9)
M_{eff}=Σ(mi*Δi))/Δd=Δ(mi*Δi)^2)/Σ(mi*Δi^2) =13869.97KN	(10)
$\Delta_{(T,5)}$=Sa*g/W^2=sd=0.195686m	(11)
$\Delta_{(T,\zeta)}$=$\Delta_{(T,5)}$*(10/(5+ζ))$^{0.5}$=0.15263m	(12)
Te= T*(Δd/Δ(T,ζ))=3.891sec	(13)

Step 6: Obtain T_{eff} from displacement spectra and K_{eff} for the equivalent SDOF system

$$K_{eff}=4\pi^2 * M_{eff} / T_{eff}^2 = 36176 \text{ KN/m} \quad (14)$$

Step 7: Obtain lateral design force for the equivalent SDOF system

$$V_B = K_{eff} * \Delta d = 21481.6383 \text{ KN} \quad (15)$$

Step 8: Distribute base shear as seismic forces

F_1 = 4656.84679KN
F_2 = 11633.2381KN
F_3 = 4656.84679KN
F_{A1} = 267.353291KN
F_{A2} = 267.353291KN

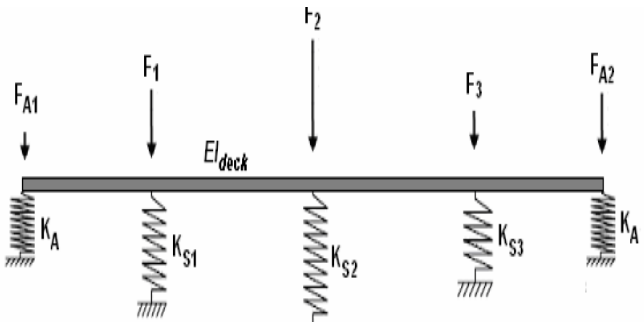

Figure 8. Distribute base shear as seismic forces

Step 9: Obtain shear forces in the members

$$V_i = SDF_i * V_B, \quad SDF = (1/hi)/\Sigma(1/hi)*F_C \tag{16}$$

$V_1 = 8652.80553KN$
$V_2 = 4390.84363KN$
$V_3 = 8652.80553KN$

Seismic force carried by the abutment of the following Eq. (17):

$$SDF_{A1} = \frac{\Delta_{A1}}{\Delta_{A1} + \Delta_{A2}} SS$$

$$SDF_{A2} = \frac{\Delta_{A2}}{\Delta_{A1} + \Delta_{A2}} SS \tag{17}$$

SDF_{A1}= -0.005 V_{A1}= -107.41KN
SDF_{A1}= -0.005 V_{A2}= -107.41 KN

Step 10: Compute member effective stiffness's

$K_{A1} = 37500.00KN/m$

$K_{S1} = 19665.47KN/m$ $\quad$ $Ksi = Vi/\Delta i$

$K_{S2} = 5926.84KN/m$ $\quad$ $Ksi = Vi/\Delta I$ $\quad$ (18)

$K_{S3} = 196665.47KN/m$ $\quad$ $Ksi = Vi/\Delta i$

$K_{A2} = 37500.00KN/m$

Step 11: Solve equivalent elastic system under seismic forces Fi, obtain revised

displacement pattern: Δn and revised proportion of load carried by superstructure

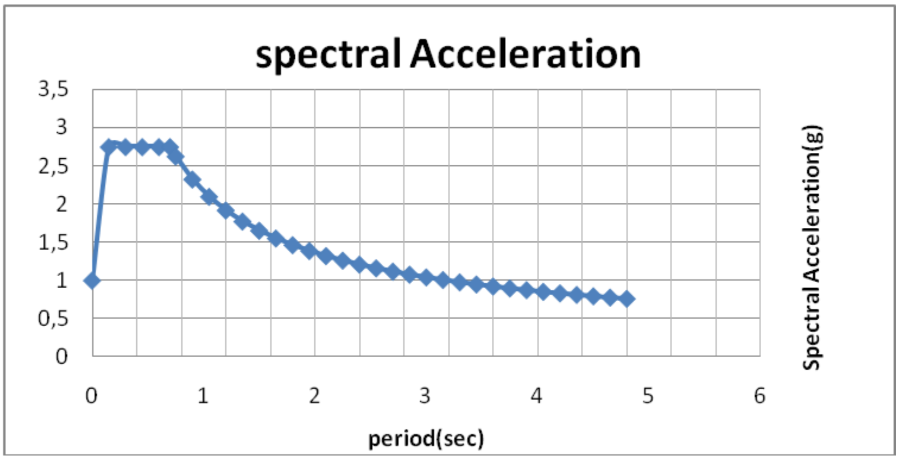

Figure 9. Acceleration spectrum

Table 3. Summary of design for transverse section of bridge

ζ_{sys}(%)	Δd(m)	M_{eff}(kN)	T_{eff}(s)	K_{eff}(kN/m)	V_B(kN)	SS(%)
0.114	0.153	13869.97	3.890	36175.94	21481.638	-1.0

Table 4. Summary of design for longitudinal section of bridge

ζ_{sys}(%)	Δd(m)	M_{eff}(kN)	T_{eff}(s)	K_{eff}(kN/m)	V_B(kN)	V1	V2	V3
15.04	0.594	13869.9	3.514	44336.2	26327.3	12389.3	1548.6	12389.3

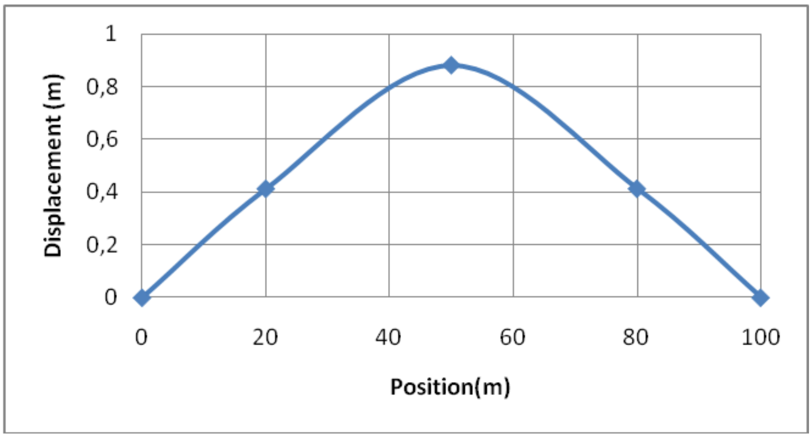

Figure 10. Design displacement profile for transverse section of bridge

4.2 Finite element model

The structural analysis program, SAP2000 Version 14.0.0 Advanced [6], was used to perform analyses. Geometric nonlinearity through considering P-Delta effect was applied to this bridge in addition to material nonlinearity.

4.3 Seismic loading

To perform analysis of structure, the next step after modeling is applying loads. Design response spectrum should be available in order to perform NSP. This bridge is to be built in the Tehran city in a seismic zone with an acceleration coefficient of (PGA = 0.35g). This estimation is based on the AASHTO (2007) specification for an earthquake of 10% probability of occurrence in 50 years, which is equivalent to a recurrence period of 475 years [7].

5 RESULTS AND PARAMETRIC STUDY

Analyses were performed for live Safety level of seismic load intensity. Comparison is performed for the maximum displacement, total base shear resulting from the NSP and the corresponding results from the DDBD. The lateral load behavior of the bridge in longitudinal and Transverse Direction is depicted in Fig. 11.

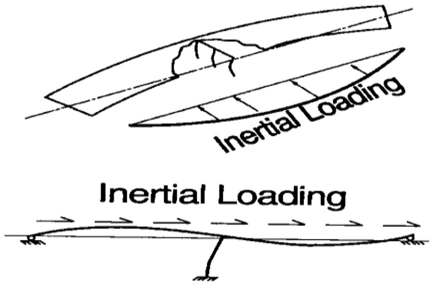

Figure 11. Transverse and longitudinal seismic response of bridge with basic support conditions

5.1 Longitudinal direction

Period of the first mode in this direction is 1 seconds and the modal participation mass ratio is shown in Table 5. Pushover curve for this direction is shown in Fig. 11. The formula shown below (FEMA-356) is used to estimate the target displacement:

$$\delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_e^2}{4\pi^2} \qquad (19)$$

where C_0, C_1, C_2, and C_3 are modification factors to consider different parameters affect the control node displacement, and the rest of the formulae is the equal displacement rule. For more details about the definition of each of the previous parameters and how to estimate it, see FEMA-356. The estimated target displacement is 10.42cm for the Design Level.

5.2 Transverse direction

Period of the first mode in this direction is 1 seconds and the modal participation mass ratio for this mode is shown in Table 5. Pushover curve for this direction is shown in Fig 11. The same formula shown in Eq. 19 is implemented to estimate the target displacement. The estimated target displacement is 9.33cm for the Design Level.

5.3 Evaluation of performance level

Using acceptance criteria provided by FEMA-356 to evaluate performance level of this bridge, rotation of plastic hinges should not exceed the following values for the corresponding performance levels: 0.005 for immediate occupancy, 0.01 for life safety, and 0.017 for collapse prevention. In the longitudinal direction, the bridge satisfies life safety performance level for Design Level .In the transverse direction; this bridge satisfies the life safety performance level for Design Level.

Table 5. Modal participating mass ratios

OutputCase	**StepType**	**StepNum**	**Period**	**SumUX**	**SumUY**	**SumRZ**
Text	Text	Unitless	Sec	Unitless	Unitless	Unitless
MODAL	Mode	1	1.000749	4.70238E-20	0.927621	0.707766
MODAL	Mode	2	0.599413	1.52612E-19	0.927621	0.931045
MODAL	Mode	3	0.531914	0.006790788	0.927621	0.931045
MODAL	Mode	4	0.431656	0.006790788	0.927621	0.934327
MODAL	Mode	5	0.383805	0.006790788	0.927621	0.934327
MODAL	Mode	6	0.358060	0.006790788	0.92841	0.934937
MODAL	Mode	7	0.347756	0.006790788	0.968418	0.965456
MODAL	Mode	8	0.290404	0.371218493	0.968418	0.965456
MODAL	Mode	9	0.242127	0.371218493	0.97320	0.969104
MODAL	Mode	10	0.215351	0.371218493	0.97320	0.969104
MODAL	Mode	11	0.197230	0.371218493	0.97320	0.97060
MODAL	Mode	12	0.190209	0.371218493	0.978894	0.974953
MODAL	Mode	13	0.165004	0.415947943	0.978894	0.974953
MODAL	Mode	14	0.163762	0.415947943	0.978894	0.976625
MODAL	Mode	15	0.159176	0.415947943	0.979065	0.976756
MODAL	Mode	16	0.156756	0.415947943	0.979065	0.976781
MODAL	Mode	17	0.149517	0.904035723	0.979065	0.976781

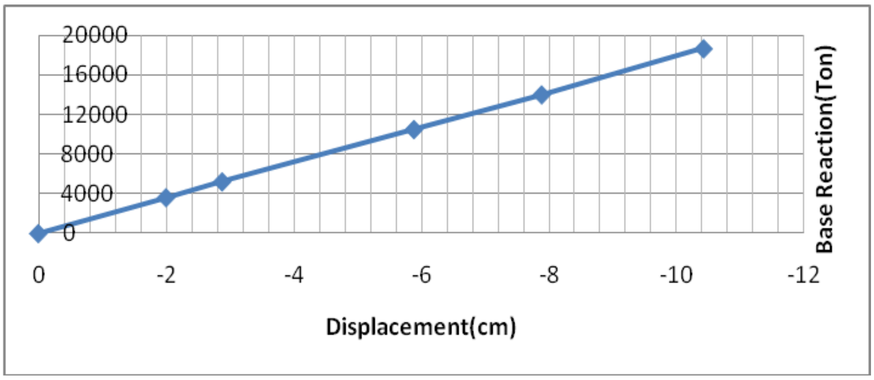

Figure 12. Pushover curve for longitudinal direction

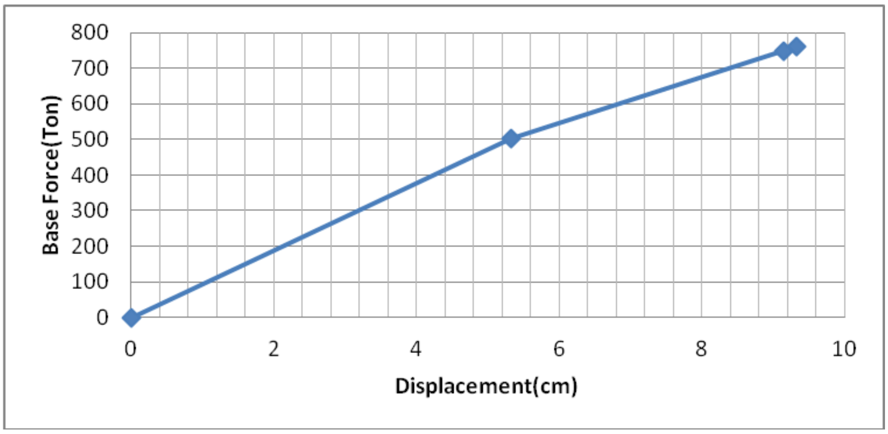

Figure 13. Pushover curve for transverse direction

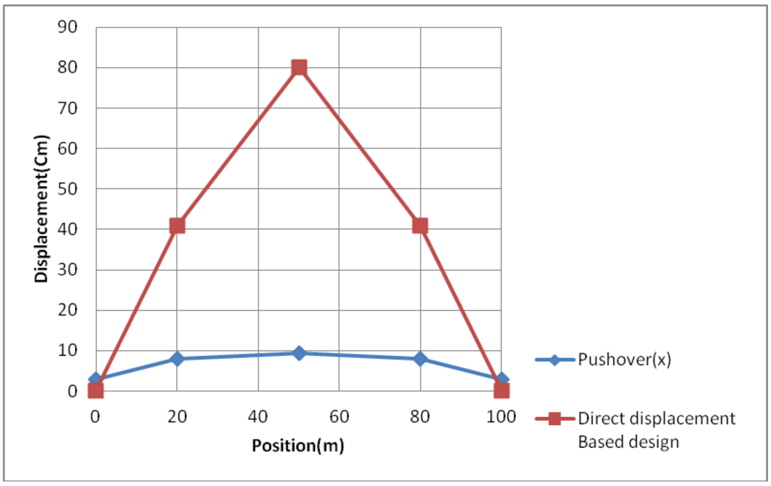

Figure 14. Displaced shape of the bridge by using different methods of analysis

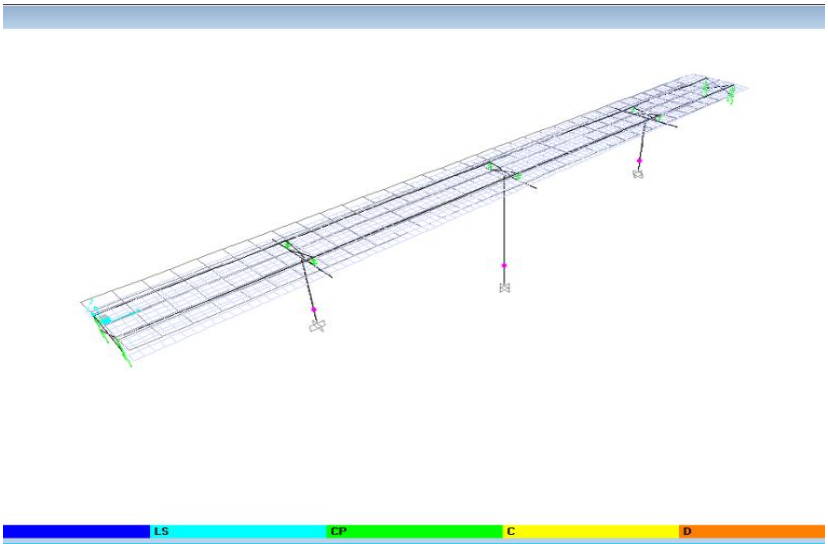

Figure 15. Rotational demands on the columns of the bridge [6]

6 CONCLUSIONS

In this work, a displacement-based design procedure for the transversal seismic design of continuous multi-span reinforced concrete bridge, first proposed by Priestley, has been presented. The method is an iterative procedure which, based on initial assumptions of the bridge displacement pattern and the proportion of the total lateral force carried by the superstructure trough the abutments, aims to design a bridge that will reach the design limit state of deflection. As result, comparison of results obtained from displacement-based design with the Nonlinear Static Procedure.

The method utilizes the Substitute Structure approach, [Gulkan and Sozen, 1974], to model the inelastic structure as an equivalent single-degree-of-freedom system. The equivalent SDOF is characterized by the secant stiffness, K_{eff}, at maximum displacement and an equivalent viscous damping, ξ_{sys}, appropriate for the level of hysteretic energy absorption associated with the inelastic response.

The iterative design procedure was found to be efficient and easy to implement. Very little iteration is required, even if initial assumptions are poor. However, some suggestions are made to provide the method with good initial estimates of the inelastic displaced shape and the proportion of load taken by the abutments. The procedure is then very easy to implement in any programming software.

Finally, it is believed than more investigation is required on the topic, especially for the cases in which the fundamental elastic and inelastic mode shapes differ. Even though all the bridges in this work considered flexible lateral support at the abutments, the procedure can be successfully applied for fixed abutments condition. Application of the method to free abutments condition was not considered and can be also investigated. Moreover consideration of nonlinear inelastic behavior of the abutments could also be implemented in the design procedure.

Additional analysis will be carried out in the future for more bridge configurations, abutment conditions and variable span lengths. The goal will be to identify the inelastic displacement patterns and to estimate the amount of shear force carried by the abutments in the case of a bridge with restrained abutments.

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