A procedure of displacement-based seismic design Applied to urban bridges of Mexico City

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SUMMARY:
A displacement-based approach for seismic design of reinforced concrete urban bridges is proposed, in particular for the design of its columns, in which the fulfillment of two levels of performance is sought: serviceability and survival. Displacement capacity of rectangular and circular columns is computed through empirical equations derived in terms of column section, amount of longitudinal reinforcement, level of axial load, ratio of confinement and slenderness of the column. For purpose of evaluating the demands of inelastic displacement, this procedure offers the advantage of being able to choose between three approximate methods: equivalent linearization, displacement modification factors and strength reduction factors. This procedure has been compared with other displacement-based methods in order to assess their feasibility to be incorporated into the design practice in Mexico, taking into account the following aspects: simplicity, versatility and clarity.

Keywords: Bridges, Concrete, Displacement-based design

1. INTRODUCTION

In Mexico the bridge structures have had a reasonable behavior without registering partial or total collapses, however, in the main cities of the country such as Mexico city where the majority of these bridges are relatively new (built after 1985), have not yet been subjected to earthquakes of great intensity, such as occurred in 1985 (M_w = 8.1), and that combined with a lack of clear criteria of design through a official code for bridges (like there is for buildings), there is uncertainty about its structural safety.

In other countries, it has been noted, that as the devastating earthquakes that occurred in Loma Prieta (1989), Northridge (1994), Kobe (1995), Chi Chi Taiwan (1999), Chile (2010), among others, the bridges have suffered severe damage, in particular the columns have experienced shear failure (Fig. 1.1), evidencing a poor ductility capacity, this situation is cause for concern, if it is taken into consideration, that to ensure a good seismic behavior of the bridges must dissipate energy through the potential plastic hinge regions of the columns. Therefore, columns must have a good design to absorb large seismic demands of inelastic deformation.

To ensure that the bridges have a seismic behavior satisfactory, in the last years the displacement-based seismic design (DBSD) has been implemented inside the seismic design of bridges. The reason of adopting this procedure is due to that the damage limits states can be related appropriately with the deformation limits, that in turn are converted in equivalent displacements, with which the structural damage can be controlled efficiently better than with resistance limits. Therefore, in this paper, a displacement-based approach for seismic design of reinforced concrete urban bridges of Mexico City is proposed, in particular for the design of its columns, in which the fulfillment of two levels of performance is sought: serviceability and survival; the comparison of this design procedure with other DBSD methods is presented.

1999 Chi Chi Taiwan earthquake
(NCREE, 2006)

2010 Chile earthquake
(Kawashima, 2010)

Figure 1.1. Shear failure of columns

2. DISPACEMENT-BASED SEISMIC DESIGN (DBSD)

In the last years the displacement-based design has been implemented inside the bridges seismic design. The reason for adopting this procedure is because the damage limits states can be related appropriately with the deformation limits that in turn are converted in equivalent displacements, with which the structural damage can be controlled efficiently better than with resistance limits.

According to Sullivan *et al.* (2003), in the literature have proposed various DBSD methods with different approaches: direct displacement-based design (Priestley *et al*., 2005), yield point spectra (Ascheim and Black, 2000), capacity spectrum (Freeman, 1998), inelastic spectrum (Chopra and Goel, 2001), among others. Table 2.1 presents an array of these methods with their different approaches according to the FIB (2003), in which is indicated on the one hand the procedures for estimating the deformation of the structure (DCB, and DDSB IDSB) and on the other the various criteria that use these methods to assess the seismic demand (response spectra and direct integration).

Table 2.1. DBSD Methods

	Deformation Calculation Based (DCB)	Iterative Deformation Specification Based (IDSB)	Direct Deformation Specification Based (DDSB)
Response spectra: Initial stiffness based	Moehle (1992) FEMA (1997) UBC (1997) Panagiotakos and Fardis (1999) Albanesi *et al.* (2000) Fajfar (2000)	Browning (2001)	SEAOC (1999) Aschheim and Black (2000) Chopra and Goel (2001)
Response spectra: Secant stiffness based	Freeman (1998) ATC (1996) Paret *et al.* (1996) Chopra and Goel (1999)	Gulkan and Sozen (1974)	Kowalsky (1995) SEAOC (1999) Priestley and Kowalsky (2000)
Direct integration: Time history analysis based	Kappos and Manafpour (2000)	Does not apply	Does not apply

3. PROCEDURE OF SEISMIC DESIGN OF URBAN BRIDGES COLUMNS

This design procedure has its antecedents in the Rivera’s research (2005), however in this paper focuses on the design of urban bridges columns type from Mexico City, whose structure is characterized by a superstructure dashed by the presence of fixed or mobile bearing, in addition to that each of these panels is supported on a single column in cantilever or frame, which allows to examine as a single-degree-of-freedom (SDOF) oscillator.

In Figs. 3.1 and 3.2 show the design procedure, in which it's possible to appreciate the steps to follow to the fulfillment of two performance levels: serviceability limit state and survival limit state. This procedure seeks to establish a relationship between the level of desired performance of the structure and the section size and reinforcement, adequate to meet these levels of performance.

In the serviceability limit state seeks to ensure the immediate operation of the bridge after an earthquake, without requiring repairs, therefore it is desirable that the columns have no perceptible residual cracking. In the survival limit state it is accepted that the columns presented severe damage, but no collapse to care for the integrity of the users; this involves providing the columns with sufficient deformation capacity to withstand the deformation demands before an extraordinary earthquake.

For the implementation of this methodology (Figs. 3.1 and 3.2) equations to estimate the capacity of lateral displacement of RC bridges columns were deducted: yielding drift (γ_y) and ultimate drift (γ_u). Equations to calculate the cracked-section moment of inertia (I_{cr}), also were deducted (Rivera, 2005).

Capacity of yielding drift for columns in cantilever, γ_y

$$\gamma_y = \frac{1}{3}\phi_y H \tag{3.1}$$

rectangular section

$$\phi_y = 3.75\frac{\varepsilon_y}{h_c}(0.30+10.50\rho_l - 125\rho_l^{\,2}) \tag{3.2}$$

Circular section

$$\phi_y = 3.75\frac{\varepsilon_y}{D}(0.30+11.20\rho_l - 146\rho_l^{\,2}) \tag{3.3}$$

where, H is the height of bridge column, ρ_l is ratio of longitudinal reinforcement, h_c and D is depth and diameter of cross section, respectively, ε_y is yielding deformation of the reinforcement steel and φ_y is yielding curvature.

From equations 3.2 and 3.3 it is possible to establish a relationship between ρ_l and γ_y for columns in cantilever as

rectangular section

$$\rho_l = 0.042 - \left[0.0042 - 0.0064\frac{h_c\gamma_y}{\varepsilon_y H}\right]^{1/2} \tag{3.4}$$

circular section

$$\rho_l = 0.038 - \left[0.0034 - 0.0054\frac{D\gamma_y}{\varepsilon_y H}\right]^{1/2} \tag{3.5}$$

Equations 3.4 and 3.5 are applicable for ratios of longitudinal reinforcement that are found in the interval between the minimum (0.0048) and the maximum (0.04).

Cracked-section moment of inertia, I_{cr}

rectangular section:

$$\frac{I_{cr}}{I_g} = 0.19 + 11.60\rho_l + 0.012\frac{P}{A_g f'c} - 0.17\rho_l \frac{P}{A_g f'c} \qquad (3.6)$$

circular section:

$$\frac{I_{cr}}{I_g} = 0.22 + 13.44\rho_l + 0.011\frac{P}{A_g f_c'} - 0.16\rho_l \frac{P}{A_g f_c'} \qquad (3.7)$$

where, I_g represents gross section moment of inertia; $P/A_g f'_c$ is vertical load ratio, as a percentage of compressive strength of concrete core (the ratio multiplies by 100).

Capacity of ultimate drift, γ_u

$$\gamma_u(\%) = \beta_0 + \lambda_e \frac{f_{yt}}{14 f'_c}\left(\beta_1 + \beta_2 \frac{P}{A_g f'_c}\right) + \beta_3\left(\frac{P}{A_g f'_c}\right) \qquad (3.8)$$

rectangular section

$$\lambda_e = k_e \rho_{st} \qquad (3.9)$$

circular section

$$\lambda_e = \rho_{st} \qquad (3.10)$$

where, γ_u (%) is ultimate drift capacity (in percentage), k_e is a confinement effectiveness coefficient, f_{yt} yielding strength of traverse reinforcement, f'_c is compressive strength of concrete, ρ_{st} is ratio of transversal reinforcement, and β_o, β_1, β_2 and β_3 are constants which are evaluated in terms of axial load ($P/A_g f'_c$) and its aspect ratio (H/L or H/D) ,according to the Rivera's work (2005).

This procedure take in consideration three alternatives for assessing demands of inelastic displacement in the bridge structure: equivalent linearization, strength reduction factors (R_μ) and displacement modification factors (C_μ). In the revision of survival limit state (fig. 3.2) shows the option of equivalent linearization method, which, in the case of the characteristics of seismic activity in city of Mexico adopted the method of Rosenblueth and Herrera (1964), considering a elasto-plastic behavior of the column, so the expressions to evaluate equivalent period (T_{eq}) and equivalent damping (ξ_{eq}), is given by:

$$T_{eq} = T\sqrt{\mu_\Delta} \qquad (3.11)$$

$$\xi_{eq} = \xi_o + \frac{2}{\pi}\left[\frac{\mu_\Delta - 1}{\mu_\Delta}\right] \qquad (3.12)$$

Where, T is the period of structure vibration and μ_Δ is the displacement ductility factor. For a better estimation of the inelastic displacement demand with this method, the following correction is made:

$$\Delta_d = \frac{\Delta_i(T_{eq}, \xi_{eq})}{\psi} \qquad (3.13)$$

$$\psi = 1.19 - 0.23T \qquad (3.14)$$

Where, Δ_d represents the inelastic displacement demand corrected, Δ_i displacement demand obtained with the method of Rosenblueth and Herrera, and ψ is the corrective factor.

4. COMPARISON WITH OTHER DBSD METHODS

In order to analyze the virtues of the design procedure proposed in this paper, a comparison with other DBSD methods was made, as shown in table 4.1. This table presents the main characteristics of some methods. From this comparison it can be seen that in the methods proposed by Browning (2001), Chopra and Goel (2001) and Priestley and Kowalsky (2000) reviewed only a single level of performance (survival); unlike those proposed by Panagiotakos and Fardis (1999) and Rivera (2005), which reviews two levels of performance, serviceability and survival.

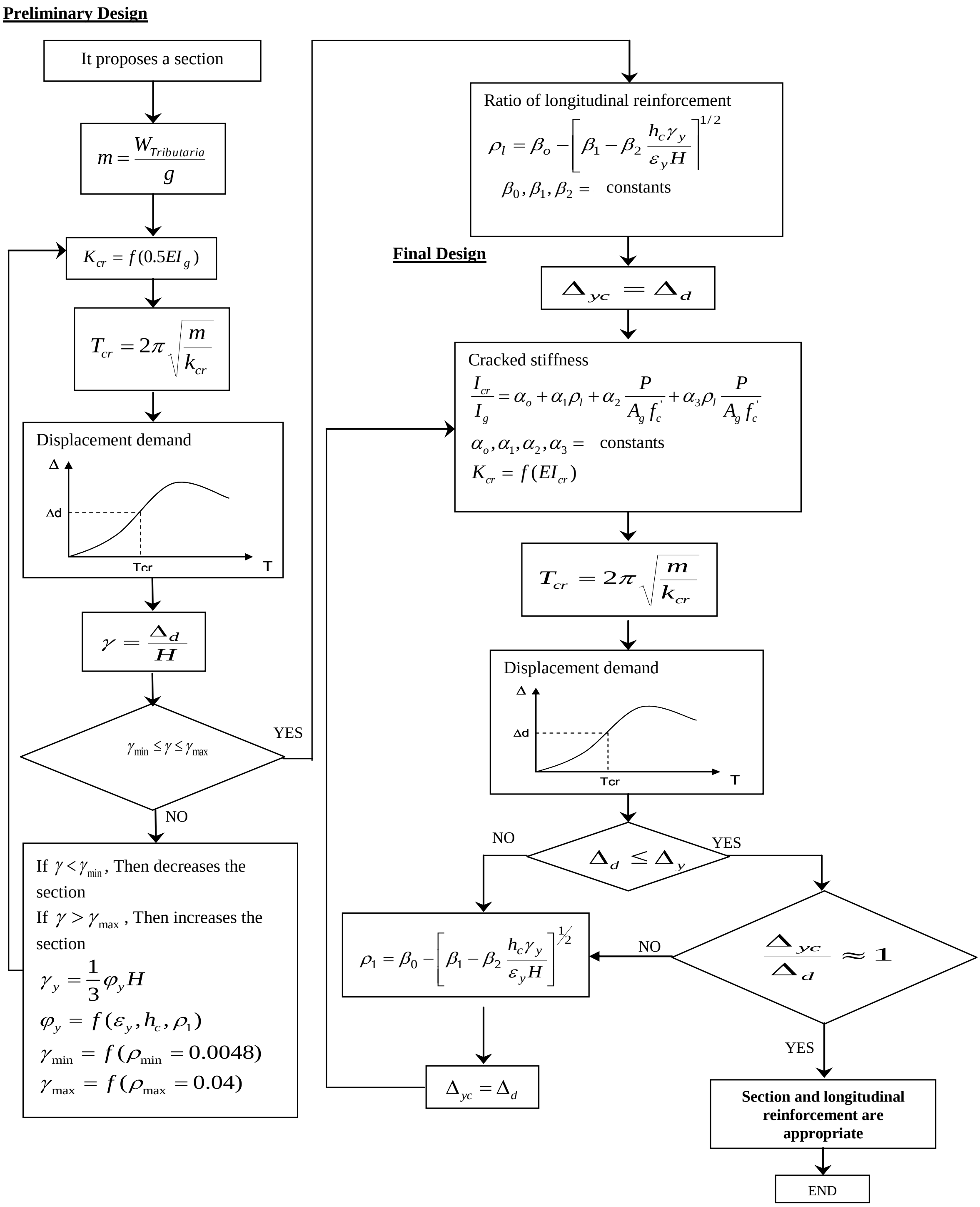


Figure 3.1. Seismic design of bridges columns by serviceability limit state

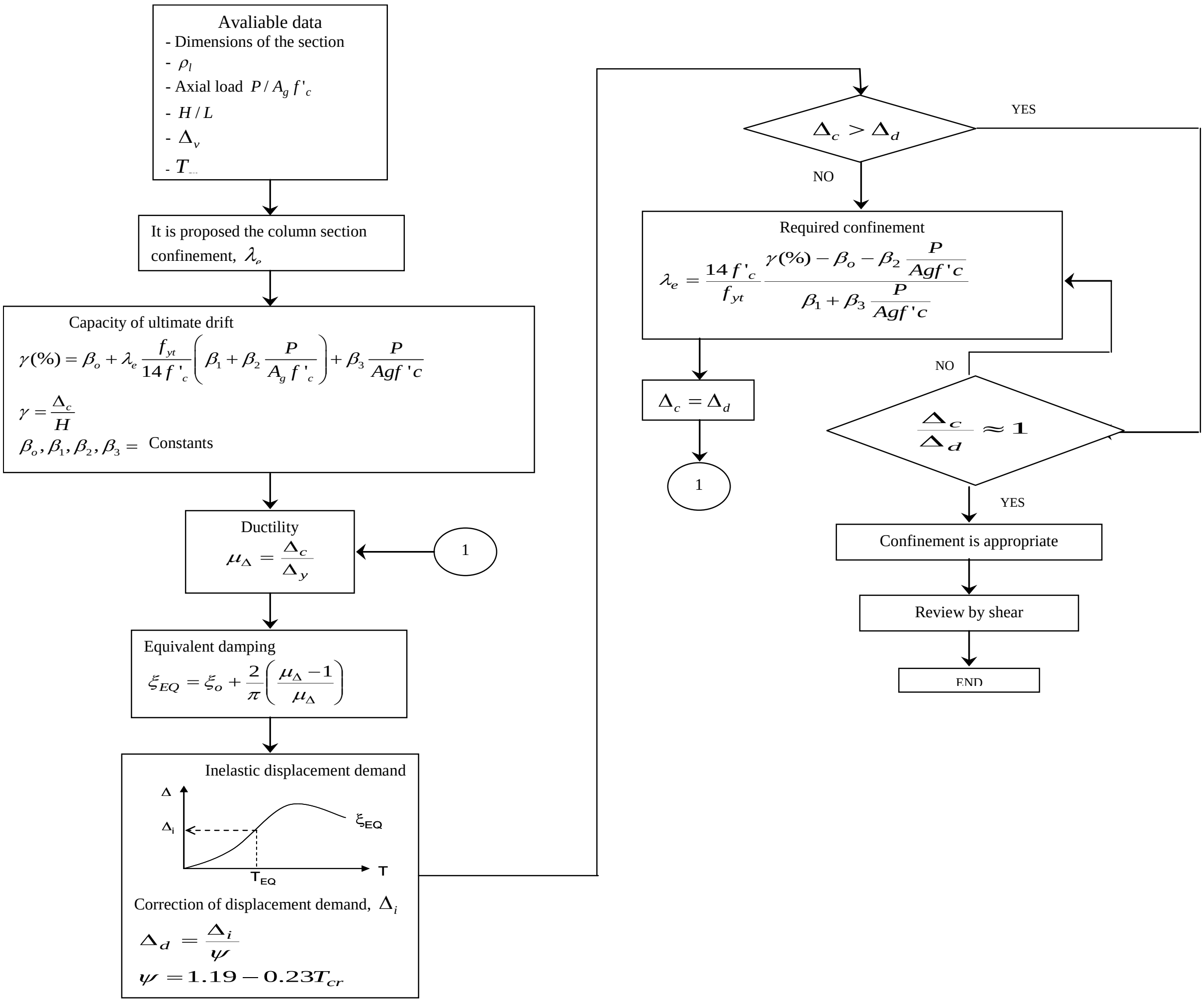


Figure 3.2. Seismic design of bridges columns by survival limit state

On the other hand, to assess the capacity of inelastic deformation of the columns, the methods of Panagiotakos and Fardis (1999) and Priestley and Kowalsky (2000) offer equations that to some extent are complicated to be used in the design practice, requires specific analysis of column section, while in the proposed procedure (Rivera, 2005) and in the methods of Browning (2001) and Chopra and Goel (2001) has more explicit expressions relate directly to the desired performance with the detailed of required reinforcement steel.

Regarding the criteria to estimate the inelastic deformation demand, the majority of these methods apply a single criterion, being usually based on equivalent linearization, while the proposed procedure is structured to use any of the criteria available to predict the inelastic displacement demands, as are: equivalent linearization, strength reduction factors (R_μ) and displacement modification factors (C_μ).

Taking as a model the study of Sullivan *et al.* (2003) with regard to the analysis of the DSBD methods to tending to the simplicity, versatility and clarity, it is proceeded to evaluate them with values of 1 to 5, with the following meanings: 1 very poor, 2 poor, 3 acceptable, 4 Good and 5 excellent, the results of this assessment is presented in fig. 4.1. With the foregoing analysis was able to appreciate that the proposed procedure (Rivera, 2005) may be viable for the practice of bridges design to offer simplicity, versatility and clarity as the methods of Browning (2001), Chopra and Goel (2001), and Priestley and Kowalsky (2000).

Table 4.1. Comparison with other DBSD Methods

Design method	Performance levels	Evaluation of the elastic deformation capacity	Evaluation of the inelastic deformation capacity	Evaluation of the inelastic deformation demand
Panagiotakos and Fardis (1999)	Serviceability and survival	$\theta_y = \phi_y \frac{L_s}{3} + 0.0025 + a_{sl} \frac{0.25\varepsilon_y d_b f_y}{(d-d')\sqrt{f'_c}}$	$\theta_u(\%) = \alpha_{st}\alpha_{cyc}\left(1+\frac{a_{sl}}{2.3}\right)\left(1-\frac{a_{wall}}{2.3}\right)\left[\frac{\max\left(0,01,\frac{\rho' f'_y}{f'_c}\right)}{\max\left(0,01,\frac{\rho f_y}{f'_c}\right)} f'_c\right]^{0.275}\left(\frac{L_s}{h}\right)^{0.45} 1.1^{\left(100\alpha\rho_{sx}\frac{f_{yh}}{f'_c}\right)}\left(1.3^{100\rho d}\right)$	Equivalent linearization
Browning (2001)	Survival	$\phi_y = \frac{\varepsilon_y}{1-k_{ry}} \frac{1}{d}$	$\theta_u = \frac{1}{6}\left(1+3\frac{L_u}{L_c}\right)\frac{\varepsilon_y}{1-k_{ry}}\frac{L_c}{d}$	Displacement modification factors (C_μ)
Chopra and Goel (2001)	Survival	$f_y = ku_y$	$u_m = u_y + h\theta_p$	Strength reduction factors (R_μ)
Priestley and Kowalsky (2000)	Survival	Rectangular section $h_c\phi_y = 2.12\varepsilon_y$ Circular section $D\phi_y = 2.45\varepsilon_y$	$\varepsilon_{cu} = 0.004 + \frac{1.4\rho_s f_{yh}\varepsilon_{sm}}{f'_{cc}}$	Equivalent linearization
Rivera (2005)	Serviceability and survival	Rectangular section $\phi_y = 3.75\frac{\varepsilon_y}{h_c}(0.30 + 10.52\rho_l - 125\rho_l^2)$ Circular section $\phi_y = 3.75\frac{\varepsilon_y}{D}(0.34 + 11.22\rho_l - 146\rho_l^2)$	$\gamma(\%) = \beta_0 + \beta_1\lambda_e + \beta_2\left(\frac{P}{A_g f'_c}\right) + \beta_3\left(\lambda_e\frac{P}{A_g f'_c}\right)$	Equivalent linearization Strength reduction factors (R_μ) Displacement modification factors (C_μ)

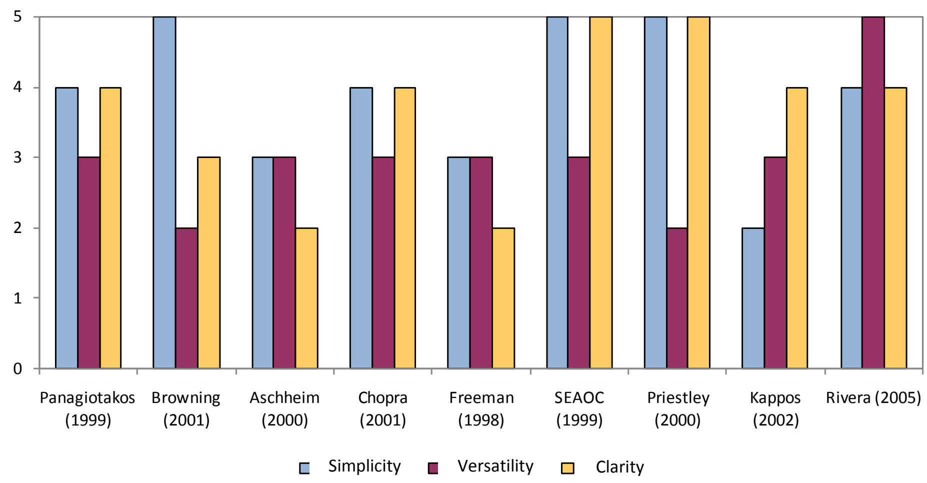
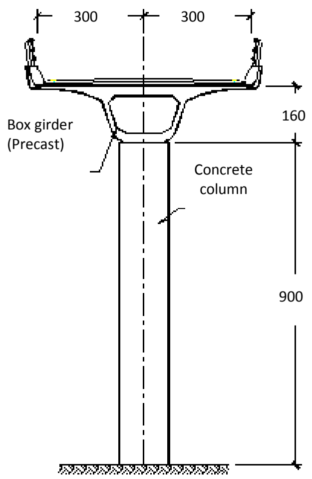


Figure 4.1. Evaluation of DBSD methods

5. APPLICATION AND ASSESSMENT OF THE BRIDGES DESIGN PROCEDURE

To carry out the application and assessment of the proposed design procedure, a prototype of bridge column as a cantilever was designed, whose dimensions shown in Fig. 5.1. Two types of column section were used: circular (ϕ = 160 cm) and rectangular (100 x 160 cm), for each type of section is considered a relation of axial load ($P/A_gf'_c$) and aspect ratio (H/b) of 15 and 5.6, respectively. The concrete compressive strength and the yielding nominal strength of steel reinforcement are f'_c = 300 kg/cm^2 and f_y = 4200 kg/cm^2, respectively; these values are representative of urban bridges from Mexico City. It was considered the station SCT (1985, M=8.1) accelerogram, their spectra ordinates were increased in 50% (Fig. 5.1), by considering to the bridges as critical structures according to Federal District Code (RCDF, 2004).



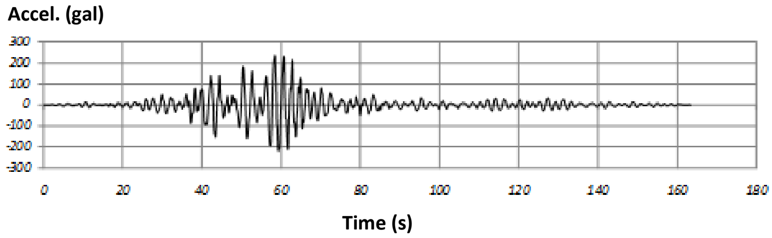


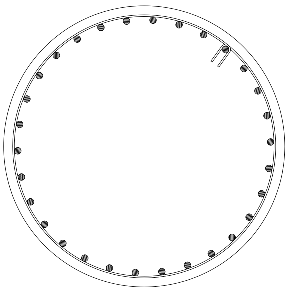
Figure 5.1. Prototype of bridge, structuring in cantilever (dimensions in cm), and ground acceleration history (station SCT, 1985-sep-19, M=8.1)

Fig. 5.2 shows the design obtained with the proposed procedure, as well as the capacity of ultimate deformation and the inelastic displacement demand obtained from a nonlinear time-history analysis with the support of the program Seismostruct (2010). In table 5.1 compares the proposed design procedure with other DBSD methods (the same bridge prototype was used to apply the DBSD methods), in such a way that is reported the expected displacement in accordance with the design procedure (Δ_u calculated) and the demanded displacement by the SCT earthquake (Δ_u Seismostruct).

To analyze the goodness of the proposed method can be seen that gives a good estimate on the bridge response, with similar degree of precision that the methods of Priestley and Kowalsky (2000) and Chopra and Goel (2001).

It should be noted that only a single earthquake record was used in the analysis, so that using a single record to identify the accuracy of a design method is not enough, as each record scaled to a certain intensity will have a different response. Therefore, in the future it will be necessary to review the design method with more seismic records.

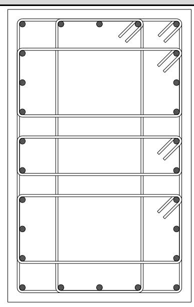
Method	Data for the modeling of columns
Rivera (2005)	



Data	
D =	160 cm
H =	900 cm
bar	30#12
E #	3@4
ρ_l =	1.69%
ρ_{st} =	0.44%

Calculated	
Δ_u =	28.00 cm

Seismostruct	
Δ_u =	29.77 cm



Data	
b =	100 cm
h =	160 cm
H =	900 cm
Bar	26#10
E #	4@15
ρ_l =	1.29%
ρ_{st} =	0.87%

Calculated	
Δ_u =	34.97 cm

Seismostruct	
Δ_u =	37.23 cm

Figure 5.2. Design obtained in accordance with the proposed design procedure

Table 5.1. Comparison of the proposed design procedure with other DBSD methods

Method	Section	Ratio of longitudinal reinforcement ρ_l (%)	Ratio of transversal reinforcement ρ_{st} (%)	Δ_u Calculated (cm)	Δ_u Seismostruct (cm)	$\frac{\Delta_u\ calculated}{\Delta_u\ seismostruct}$
Panagiotakos and Fardis (1999)	Circular 160 cm	2.04	0.64	50.25	43.97	1.14
	Rectangular 100X160 cm	1.28	0.69	47.63	39.74	1.20
Browning (2001)	Circular 160 cm	1.36	0.32	29.94	36.95	0.81
	Rectangular 100X160 cm	1.20	0.32	29.77	38.78	0.77
Chopra and Goel (2001)	Circular 160 cm	2.72	0.70	37.15	31.21	1.19
	Rectangular 100X160 cm	1.40	0.67	35.98	38.41	0.94
Priestley and Kowalsky (2000)	Circular 160 cm	2.44	0.23	27.00	27.34	0.99
	Rectangular 100X160 cm	1.05	0.49	27.00	33.92	0.80
Rivera (2005)	Circular 160 cm	1.69	0.44	28.00	29.77	0.94
	Rectangular 100X160 cm	1.29	0.87	34.97	37.23	0.94

6. CONCLUSIONS

The objective of this paper was to propose a procedure for the seismic design of urban bridges columns displacement-based, under the fulfillment of two performance levels: serviceability and survival, in addition to establishing a comparison with other DBSD methods.

Contrary to other procedures based on displacements, in this procedure necessary tools were implemented to revise in a more rational way the execution of the states previous limits. This way, expressions to evaluate in an approximate and simple way, the yielding lateral displacement and ultimate capacities that can experience the columns in function of the section size and reinforcement were developed. Also, an approximate method to calculate lateral displacement demands was incorporated, which is a modification of the Rosenblueth and Herrera's equivalent linearization method, to have a better estimate of the displacement demands in soft soils of the Mexico city.

Based on the seismic design of bridge prototype, subjected to the action of SCT earthquake, it was noted that the proposed design procedure makes it a good estimate of the non-linear response, which implies an acceptable accuracy for designing the column in accordance with the desired performance, in addition the procedure offers simplicity, versatility and clarity as the methods of Priestley and Kowalsky (2000), and Chopra and Goel (2001). Therefore, it may be feasible to use the proposed design procedure in this work in the practice of bridges design, however in the future must refine the criteria for estimating the displacement capacity and demand in order to achieve a balance between accuracy and simplicity of the design procedure.

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