Seismic Evaluation of Prestressed Concrete Bridges by Using DCM, CSM and Spectrum Analysis

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Abstract: Bridges are one of the most important connecting elements in lifeline system. Most existing bridges are designed in accordance with outdated standards and therefore their performance should be reevaluated for earthquake conditions. In earthquakes, induced structural displacements are the main case of damage to structures. This is one of the main noted points in performance design and evaluation of structures. In this paper, two models of prestressed concrete bridges constructed by cantilever method (Frame & continuity) have been selected. Subsequently, the Capacity Spectrum Method (as a nonlinear static procedure), Spectrum Analysis and Displacement Coefficient Method (DCM) have been used to evaluate the seismic performance of these models based on displacement control. Also, this study helps to evaluate the applicability of DCM (which has been used essentially for buildings) to bridges.

Key words: Prestressed Concrete Bridges · Seismic Evaluation · Damage · Cantilever Method · Capacity Spectrum Method · Displacement Coefficient Method

INTRODUCTION

Considering the prominence of the bearing elements deformation in structural performance, new methods devised for seismic behavior of structures are mainly based on deformation. The design methods based on capacity showed that the distribution of resistance through out the structures is much more important than the amount of base shear. Also seismic design based on performance approach includes the definition of earthquake risk, performance level and the intended performance in each level, design control, the quality of actual installation and strengthening of the structure during its useful life. Therefore direct design and evaluation based on displacement was paid more attention than before.

Research Done on Design and Evaluation Based on Displacement Has Arrived at the Following Conclusions: In 1980, Newmark and Riddell showed that the equivalent displacement law (the equivalency of maximum displacement in both elastic and plastic system) is not correct especially in the short period ranges [1]. Also Otani obtained the same results in 1981 [2].

In 1982, Newmark and Hall divided the elastic response spectrum into 3 sections and reached the conclusion that in long periods (constant velocity and displacement) there will be no difference between elastic and plastic displacements, but in short time ranges the plastic displacements will be greater than the elastic one [3].

In 1985, Sozen and Shimazaki assigned the period of the common point of constant velocity and acceleration curves to the maximum exerted energy spectra values. Also regarding the proportionality of displacement and damage, Sozen proposed the use of displacement information for the selection of the economically optimum structural system [4].

In 1992, Moehel proposed the displacement to be used as the main factor in selection of structural systems and stated that by the help of target displacement, definition of structural detailing is possible [5].

In 1995, Calvi and Kingsley reached the conclusion that the displacement-based design can be useful for the symmetrical bridges and should be studied more for the irregular structures. In this method, study of the demand displacement shows the direct effect of energy dissipation, hysteretic characteristics and the acceptable

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damage of a defined level on the design of structures. It shows also that the correct selection of viscous damping, as a function of the hysteretic response, clearly plays a fundamental role in the design process [6].

In 1995, Kowalsky, Priestley and Macrae explained the desirability of using the design based on displacement over the force method. They also explained some of the difficulties of using force method such as use of force reduction factor and definition of service and ultimate limit states. They emphasized the advantages of the displacement method by employment of the rational seismic design criteria and in compatibility with the philosophy of structural design in plastic deformation range and under large-scale earthquakes while satisfying the service limits criteria under short scale earthquakes.

Considering the above recommendations, the initial design parameters in displacement method are the column height and target displacement. Strength and stiffness values and reinforcement details are obtained by applying this method, which depend on the chosen target displacement [7].

In the year 2000, studying two elastic and inelastic spectrum designs, Chopra and Goel reached the conclusion that the deformation and ductility factors in structural design (based on elastic design spectra, equivalent linear systems and the secant stiffness method) are much smaller than the equivalent values obtained in nonlinear analysis by using the inelastic design spectra [8].

In 2002, Fu and Alayed compared the results of Displacement Coefficient Method (DCM) with nonlinear dynamic analyses (Time History), by analyzing a three-span concrete bridge [9].

Following the above mentioned researches using the DCM and Capacity Spectrum approaches, the seismic performance of prestressed concrete bridges are analyzed by SAP2000 and their results are presented in this paper.

MODELS OF STUDY

In this paper, two models of existing prestressed concrete bridges constructed by cantilever method (Frame & Continuity) have been selected and studied.

The First Model: The first model is a three-span bridge 77.3 m in total length and 11m width for which the skew piers are fixed to the deck by prestressed cables. Two prestressed precast concrete box segments 2.4 m in length and 5.5 m in width, adjacent to each other have been used in the deck sections. The depth of the deck sections is 1.9 m in mid-span and 2.3 m over the piers.
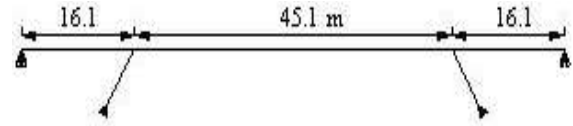

Fig. 1: View of the first model

The top of the piers have been rigidly connected to the deck and the piers are under bending in longitudinal and transversal direction, therefore the probable bending-axial (PMM) Hinge is produced at the top of the piers, but to evaluate the exact nonlinear structural behavior, the PMM, axial (P) and shear (V) hinges are assumed to be at the top and bottom of the piers.

The Second Model: The second model is a seven-span bridge, 304(m) in total length and 13.8(m) wide for which the continuous deck sits on piers via bearing plates. The prestressed precast concrete box segments with a length of 2 m have been used in the deck sections, for which the height is 2.5 m in the mid-span and 2.85m for the piers. Six columns and two concrete abutments support this bridge.

The piers are under bending condition and probable PMM hinge produced will be more at the bottom. As the first model the PMM, P and V hinges are assumed at the top and bottom of the piers.

PERFORMANCE EVALUATIONS BY BASE DESIGN SPECTRUM

Considering the bridges characteristics and behavior under different loading conditions, linear static analysis is generally unreliable. The nonlinear time history analysis is employed for accurate seismic performance evaluation. However, considering complexity of this method, the nonlinear static analysis (NSP) such as Capacity Spectrum Method (CSM) and Displacement Coefficient Method (DCM) are recommended for this evaluation.

Pushover Analysis (Capacity Spectrum Method): For nonlinear static analysis, the Applied Technology Council (ATC40) spectrum has been used.

$$C_A = 0.4 \ S_{XS} *A = 0.4*2*0.35 = 0.28$$
$$C_V = 2 \ T_0 *A = 0.5*2*0.35 = 0.35$$

After defining the design spectrum, the models are analyzed under gravitational forces, considering the effects of P_Δ simultaneously. Following this and with the initial conditions obtained due to gravitational forces, the pushover analysis is done on the models in both
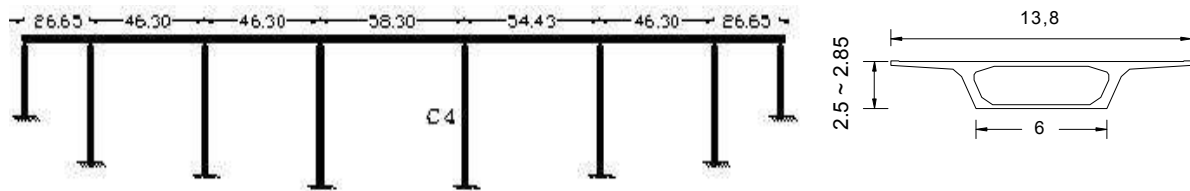
Fig. 2: View of the second model

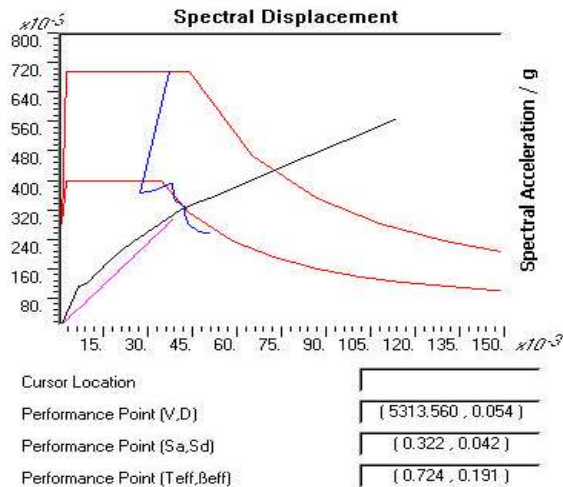| Cursor Location	
Performance Point (V,D)	(5313.560 , 0.054)
Performance Point (Sa,Sd)	(0.322 , 0.042)
Performance Point (Teff,ßeff)	(0.724 , 0.191)

Fig. 3: Pushover curve and performance point in longitudinal direction for the first model

longitudinal and transversal directions considering appropriate modes and P_Δ effects. Considering the aforementioned points, the performance level of structure is highly important [10, 11].

Performance Condition of the First Model: Figure 3 shows the performance point specification of the first model in longitudinal direction, which will be obtained at the point of the capacity spectrum curve and reduced demand spectrum in Acceleration-Displacement Response Spectrum (ADRS) format (based on A type which demonstrate a good structural behavior [10]).

According to this, the performance point of structure has happened at a displacement of 54 mm and at effective damping of 19.1% in longitudinal direction. Also PMM hinge dominated at the top of the piers and the rotation of the structures at the performance point was 0.0042radian. By comparing this rotation to accepted values, performance level of this model in longitudinal direction will be obtained in IO-LS (between Immediate Occupancy and Life Safety Levels). Based on the result, the structure is not considerably weak in longitudinal direction and therefore element performance level of structure and global behavior curve is being considered.
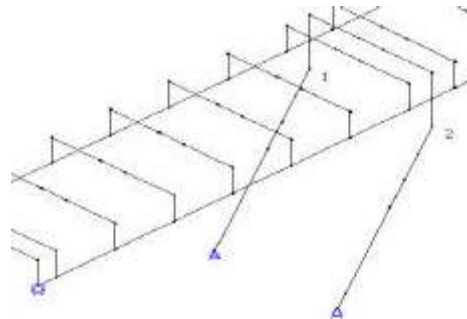

Fig. 4: Production of the plastic hinges at piers in longitudinal direction of bridge

At first, the structure is affected by the initial conditions caused by the gravity and prestress loads. Then lateral force is increased step by step, to the 5th step when two initial hinges are produced at the top of the column, as shown in Figure 4.

By increasing the lateral force, it is seen that the hinges number 1 and 2 reach the same level as IO-LS in the 9th step. In the 10th step and in the next columns, hinges number 3 and 4 are produced between B-IO levels. At the 14th step, these hinges reach the IO-LS levels. Increasing the load and in the 16th step, hinges number 1 and 2 reach to level between Safety and Collapse Prevention (LS-CP) of performance. These processes
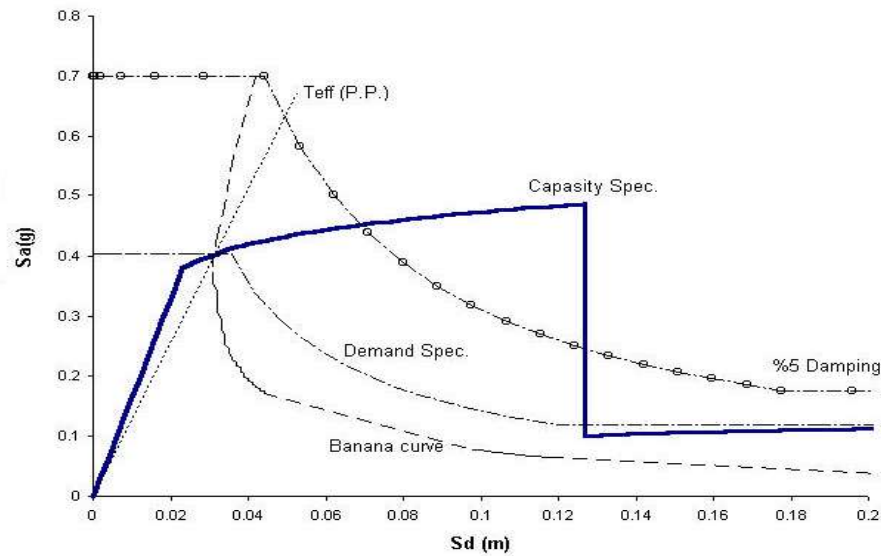
Fig. 5: Pushover curve and determination of performance point of C4 column in longitudinal direction

will be continued until hinges number 1 and 2 reach to Level after CP, that is C-D level and at this point, the structure will get unstable and finally will collapse.

Performance Condition of the Second Model: In this model since the continuous deck sits on piers via bearing plates, the piers have a critical performance separately. Here the performance condition of one separate pier, which has the maximum height and longest span, will be considered.

According to the Figure 5, the performance point of C4 column (by A type assumption) has happened at a displacement of 31mm and effective damping of 18.6% in longitudinal direction. Also PMM hinge dominated at the bottom of the column and based on plastic rotation of structure, its performance level will be obtained in B-IO (before the Immediate Occupancy performance level).

Variation of effective damping of C4 column as a function of the increase of lateral displacement (in X direction) is studied. According to the results, when the structure is in elastic range, the effective damping will stay at 0.05 but when the behavior of structure passes the elastic limit, the effective damping will increase by increasing the lateral displacement.

Also the damping at performance point (type A) has been obtained to be 18.6% that is about 2.72 times the inherent damping of the system. The reason for the above increase is the yielding of the structure and so a decrease of stiffness and a nonlinear range of behavior in its performance. Also by the same reason, period of the performance point (0.561) increases compared to the initial elastic period 0.49.

Displacement Coefficient Method (DCM): In this section, the applicability of the Displacement Coefficient Method to fixed multi-degree-of-freedom (MDOF) bridge structures such as the first case study will be considered.

If C_0 is defined as the correcting factor to relate spectrum displacement of single-degree-of-freedom (SDOF) system to MDOF system and the bridge is assumed as a one-story building, C_0 will then be defined as: $C_0 = 1$ C_1 as the correction coefficient for inelastic displacements of the system is obtained from the structural behavior curve (type A) as follows:

$$V_y = 3000 \text{ KN}$$
$$T_e = 0.571 \text{sec} > T_0 = 0.5 \ , \ S_a = 0.62 \ , \ C_1 = 1$$

By considering the inelastic behavior of the structure and also effects of the reduced stiffness and strength on displacement and with the following assumptions, (Frame type one, T=0.571 and performance level at life safety), coefficient of C_2 is set equal to 1.10 from the appropriate table.

C_3 is set equal to 1, because α is greater than zero (α is the stiffness after yielding divided by stiffness before it).

Now with these coefficients and by assuming the life safety performance under base design spectrum, target displacement of structure is calculated as follows:

$$\delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_e^2}{4\pi^2} g = 56.4 (mm)$$

At the end, comparing δ_t (obtained from DCM) and ultimate displacement of bridge structure (obtained from pushover analysis) and also by assuming a full hysteresis loops (type A), it is concluded that the bridge models are not weak in longitudinal direction.

$$\delta_t = 56.4mm < \delta_{ult} = 128mm$$

Now by comparison between the performance levels of target displacement obtained from DCM, using the assumption of the life safety level and the structural behavior curve (obtained from pushover analysis in the same direction), it is seen that the performance of DCM target displacement has occurred before the life safety level. And this proves the conservativeness and applicability of the DCM for the bridges as for buildings. This method was applied to the second model as was to the first model and the same results were obtained.

EVALUATIONS AND
COMPARISON OF THE RESULTS

Effect of the Hysteresis Loops Type on the Performance of the First Model: According to Table 1, when the hysteretic behavior of the structure fades, type A, B and C shows good, average and poor ductility respectively, i.e., the effective damping and the earthquake force absorption of the structure is dropping Therefore shear force, displacement and plastic rotation values at the performance point will increase, also performance level will be closer to L.S. level.

Comparison Between the Results of the CSM and the DCM in the First Model Analysis: The analytical results in longitudinal direction of the bridge are shown in Table 2 and 3.

Table 1: Effect of the hysteresis loops type on the performance of the first model under DBE in longitudinal direction

Hysteresis loops type	A	B	C
To	0.68	0.68	0.68
Teff	0.724	0.745	0.777
β_eff	0.191	0.154	0.106
Δ_u	0.128	0.128	0.128
Δ_y	0.027	0.027	0.027
Vy	3000	3000	3000
Vu	9695	9695	9695
μ	4.8	4.8	4.8
Vp	5418.1	5701.5	6199.7
Δ_p	0.054	0.059	0.068
θ_p	0.0036	0.0046	0.0057
Performance level	IO-LS	IO-LS	IO-LS
Dominated hinge	PMM	PMM	PMM

According to the comparison above, the target displacement in DCM will be approximately equal to the displacement in CSM. Also by considering the structural behavior curve, the performance level has not reached the life safety level. (Note: For DBE condition the judgment is based upon the life safety level and for MCE condition the judgment is based on the collapse prevention level, to determine the target displacement in DCM). And this proves the conservativeness of the coefficient method (DCM).

Effect of the Piers Prestressing on the Behavior of the First Model: To consider this effect, two models one of prestressed and the other with no prestress of skew piers were made and the CSM was applied to them.

The analysis of both cases above shows that the existence of prestress force in piers reduces their ductility and increases their base shear capacity. That proves the conformity of shear capacity increase theory due to compressive force.

Table 2: Comparison between the results of the DCM and the CSM under DBE spectrum

TYPE	Disp. at performance point in CSM		Target disp. in DCM		DIF (%)
A	0.054	IO-LS	0.0564	IO-LS	4.4
B	0.059	IO-LS	0.0564	IO-LS	-4.4
C	0.068	IO-LS	0.0564	IO-LS	-17

Table 3: Comparison between the results of the DCM and the CSM under MCE spectrum

TYPE	Disp. at performance point in CSM		Target disp. in DCM		DIF (%)
A	0.068	IO-LS	0.0783	IO-LS	15
B	0.076	IO-LS	0.0783	IO-LS	3
C	0.089	LS-CP	0.0783	IO-LS	-12

Table 4: Comparison between the results of the CSM and the DCM for C4 column of the second model under DBE spectrum

TYPE	Disp. at performance point in CSM		Target disp. in DCM		DIF (%)
A	0.031	B-IO	0.0473	IO-LS	34
B	0.034	B-IO	0.0473	IO-LS	28
C	0.041	IO-LS	0.0473	IO-LS	13

Table 5: Comparison between the results of the CSM and the DCM for C4 column of the second model under MCE spectrum

TYPE	Disp. at performance point in CSM		Target disp. in DCM		DIF (%)
A	0.039	IO-LS	0.066	IO-LS	41
B	0.044	IO-LS	0.066	IO-LS	33
C	0.055	IO-LS	0.066	IO-LS	17

Table 6: Comparison between the results of the CSM and spectrum analysis for C4 column of the second model under DBE spectrum

TYPE	Disp. at performance point in CSM		Disp. due to spectrum analysis	DIF (%)
A	0.031	B-IO	0.0428	27.5
B	0.034	B-IO		20.5
C	0.041	IO-LS		4.2

Table 7: Comparison between the results of the CSM and spectrum analysis for C4 column of the second model under MCE spectrum

TYPE	Disp. at performance point in CSM		Disp. due to spectrum analysis	DIF (%)
A	0.039	IO-LS	0.055	29
B	0.044	IO-LS		20
C	0.055	IO-LS		0

The next important point is the conversion of the P hinge into the PMM hinge at piers due to the existence of prestress force. This occurs because prestress force counters the earthquake tension force.

Comparison Between the Results of the CSM and the DCM in the Second Model Analysis: As it was noted, because of the critical performance of the single pier in the second model, the analyses were done on one separate pier with maximum height and longest span.

The analytical results for longitudinal direction of the bridge pier are shown in Table 4 and 5.

According to the comparison above, it can be noted that the DCM results are more conservative than the CSM results. And also by considering the structural behavior curve, its performance level has not reached the life safety level.

Considering the poor hysteretic behavior (type C), the displacement parameter in DCM has smaller difference with the same parameter in CSM. And this proves the conservativeness of this method in estimation of displacement parameter in this model.

Comparison of the Results of the Spectrum Analysis and Capacity Spectrum Method in the Second Model: In spectrum analysis, the Iranian standard design spectrum (2800 code) was used. Also coefficient of 2 has replaced the coefficient of 2.5 in the section of constant acceleration for bridges. In this model, deck sits on piers by Neoprene, so the behavior coefficient (R) has been selected to be 4.

The analytical results of the comparison of these methods in longitudinal direction of the bridge pier are as shown in Table 6 and 7.

According to Table 6 and 7, the values of displacements corrected as a result of the spectrum analysis, are greater relative to CSM Results. It is worth mentioning that this difference gets to the minimum in poor hysteretic behavior (type of C) and this proves the conservativeness of this method in estimation of displacement.

CONCLUSION

Application of Capacity Spectrum Method and Displacement Coefficient Method to the Prestressed Concrete bridges and evaluations of its results give the following conclusions:

- The comparison between DCM and CSM proves that the DCM results for the bridge under study are acceptable and at the same time conservative, which is recommended for buildings.
- The results of pushover analysis indicate that, due to a nonlinear behavior, the effect of hysteretic behavior type becomes prominent. But the absence of nonlinear behavior in special levels of earthquake causes the used damping in the structure to be the same as viscous damping and the type of hysteretic behavior to have no effect on the performance of structure.
- In most cases and shown by the comparison between PMM hinge (bending- axial) and shear hinge (V), the produced hinges are mostly from the PMM type, which is due to the difficult and limiting conditions on shear design and transverse and confinement bars forced by the existence design codes.

- When the structural behavior remains linear, the maximum displacement due to the spectrum analysis will be equal to the performance point displacement of pushover analysis. But when the structure enters the nonlinear behavior, the displacement parameter in spectrum method gets bigger relative to that of pushover (by considering the full and moderate types of hysteresis loops). It is worth mentioning that the above difference gets much lower in poor hysteretic behavior (type C), which proves the conservative ness of this method.

- The results of pushover analysis show that the existence of prestress in skew piers increases the piers base shear capacity.

- The P hinge converts to the PMM hinge at skews piers due to the existence of prestress force in the model under studying, which can be explained by considering the neutralization of earthquake tension force by prestressing force.

- According to the results of pushover analysis on two models, the structure performance under earthquake will be in the L.S. level, which will be acceptable according to existing standards.

REFERENCES

1. Newmark, N.M. and R. Riddell, 1980. Inelastic spectra for seismic design, Proc. 7th world Conf. on earthquake engng., Istanbul, Turkey, 4: 129-136.

2. Otani, S., 1981. Hysteresis model of reinforced concrete for earthquake response analysis, J. the Faculty of Engineering (B).,University of Tokyo, Tokyo, Japan, 36(20: 125-159.

3. Newmark, N.M. and W.J. Hall, Earthquake spectra and design, Earthquake Engineering Research Institute, Okland, pp: 103.

4. Shimazaki, K. and M.A. Sozen, 1985. Seismic drift of reinforced concrete structures, Special research paper, Hazama-Gumi, Ltd., Tokyo, Japan.

5. Moehel, J.P., 1992. Shear strength and deformability of RC bridge columns subjected to inelastic cyclic displacements, *Report No.UCB/EERC-92/04*, Earthquake engineering research center, University of Colifornia at Berkeley, March.

6. Calvi, G.M. and G.R. Kingsley, 1995. Displacement-based seismic design of multi-degree-of-freedom bridge structures, *Earthquake Engng. Struct. Dyn.*, April 1995, 24: 1247-1266.

7. Kowalsky, M.J., M.J.N. Priestley and G.A. Macrae, 1995. Displacement- based design of RC bridge columns in seismic regions, *Earthquake Engng. Struct. Dyn.*, 24: 1623-1643.

8. Chopra, A.K. and R.K. Goel, 2000. Direct displacement-based design: Use of Inelastic Design Spectra Versus Elastic Spectra.

9. Fu, Ch.C. and H. Alayed, 2002. Seismic analysis of bridges using displacement – based approach, A2C08 Seismic Design of Bridge.

10. Applied Technology Concil (ATC), 1996. Seismic evaluation and retrofit concrete buildings, ATC-40, Report No. SSC96-01, *Seismic safety commission*, November 1996, Redwood City, California, 1(2).

11. Federal Emergency Management Agency (FEMA), 1997. NEHRP Guidelines for the seismic rehabilitation of buildings, *FEMA 273 and 356*, Washington, D.C.

12. Building and Housing Research Center, 1999. Iranian code of practice for seismic resistant design of buildings, standard No. 2800, BHRC-PN S253, 2nd edition, Autumn Iran.