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DISPLACEMENT-BASED SEISMIC DESIGN OF MDOF CONTINUOUS GIRDER BRIDGES

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SUMMARY

The proposed procedure is applied to the transverse seismic response design of MDOF continuous girder bridges with flexible or rigid girders and reinforced concrete piers on the transverse direction of the bridges. The predetermined target displacement and displacement ductility of each bridge pier to sustain an assigned damage level are defined as the design parameters.

The effects of transverse modes of irregular bridges and the modal mass distribution due to the relative stiffness between superstructures and columns are investigated. The direct displacement-based design method (DDBD) for MDOF system including the influence of higher order modes is presented and the formula of equivalent damping of the MDOF equivalent substitute system is also proposed.

In order to directly reflect the nonlinear behavior of the inelastic system, the spectral displacement of each mode responding to modal period of elastic original structural system is determined from the inelastic displacement design spectra related to the effective system damping and ductility of each mode. The overall displaced shape is then obtained by SRSS method. After determining the target-displaced shape of inelastic system, the proposed procedure characterizes the MDOF system as an equivalent SDOF system. The system hysteretic damping and effective system mass considering the effects of higher modes are determined. The effective period of the equivalent SDOF system is obtained by entering the inelastic displacement response spectra with the system target displacement. Then the effective stiffness and lateral resisting force of the equivalent SDOF system can be obtained.

The complete design procedure is outlined for the proposed method and there is no iteration in this procedure. The elastic displacement design spectra compatible with the spectrum of dynamic amplification factor in the UBC97 is presented. The inelastic displacement design spectra based on structural damage performance including ductility factor is also constructed in this paper. This is followed with some design examples and the validation study was performed with dynamic inelastic time-history analysis using the near-source earthquake records to confirm the proposed method is reasonable.

INTRODUCTION

The direct displacement-based seismic design method has attracted considerable research in the seismic design and retrofit of bridges[1] due to its direct relationship with structural deformation demand and damage performance. Using the substitute structure model shown in Fig.1, Kowalsky, Calvi et.al[2,3,4] characterized RC bridge columns and MDOF continuous concrete bridges to the equivalent SDOF substitute structure, and proposed the displacement-based seismic design philosophy and comprehensive

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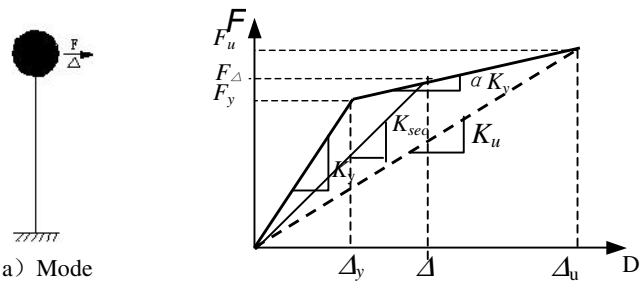
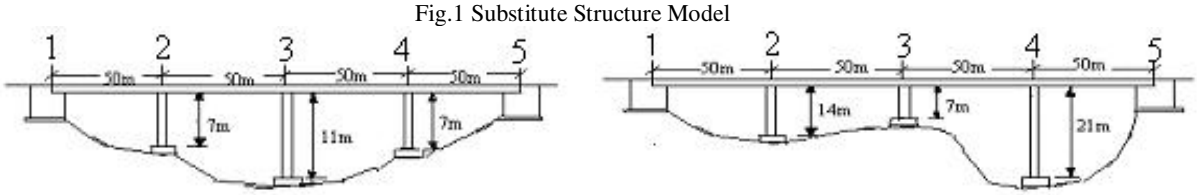
design procedure. Junqing Gong et.al[5] presented an improved non-iterative displacement-based seismic design method for RC bridge columns using inelastic constant-ductility displacement response spectra. This method defined the displacement demand and equivalent ductility of structure as the design targets, and used the elastic stiffness of original structure as the effective stiffness of the substitute structure, and then estimated the effective period of inelastic SDOF system consistent with the inelastic displacement response spectra. On the basis of elastic response spectrum-based approach including the effects of higher modes, this paper proposed an improved displacement-based seismic design method based on inelastic constant-ductility displacement response spectra for MDOF continuous girder bridges, which is applied to the transverse response of bridge structures. The detailed design procedure, sample bridge designs for symmetrical and asymmetrical bridges shown in Fig.2, and validation studies using dynamic inelastic time-history analysis are illustrated as following.



（a）Mode （b）Secant stiffness characteristics

Fig.1 Substitute Structure Model



（a）Symmetrical Bridge （b）Asymmetrical Bridge

Fig.2 MDOF Continuous Girder Bridge

DISPLACEMENT RESPONSE SPECTRA

Elastic Displacement Response Spectra

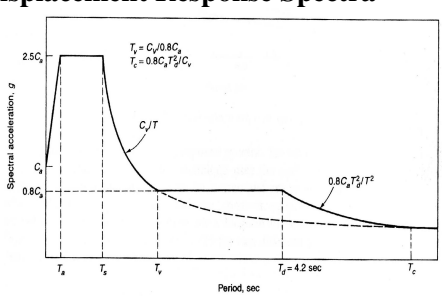


Fig.3 Acceleration response spectra of UBC97

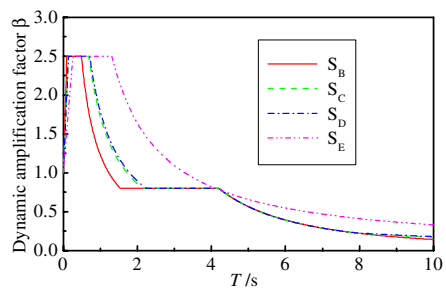


Fig.4 Dynamical amplification factor β spectra

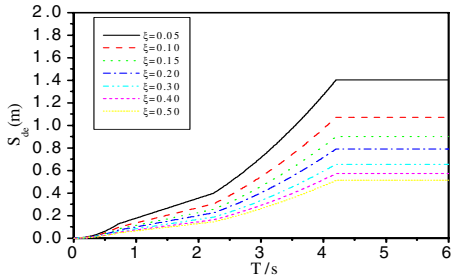


Fig.5 Elastic displacement response spectra

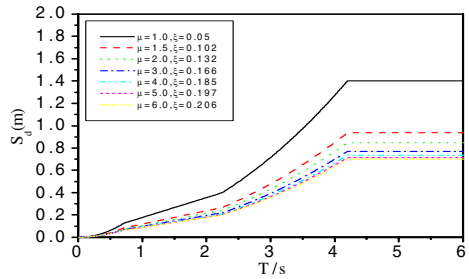


Fig.6 Inelastic displacement response spectra

The appropriate ground response coefficients C_a and C_v are defined and the near-source amplification factors N_a and N_v are introduced in the acceleration response spectra shown in Fig.3 of UBC97[6] in order to reflect the near-source effects on the structural response. The spectrum of dynamical amplification factor responding to four different soil profile types of Zone 4 in the UBC97 is presented in Fig.4, and the elastic displacement design spectra under the different a_g of design peak accelerations compatible with the spectrum of dynamical amplification factor is constructed using equation (1):

$$S_{de} = \eta a_g \beta(T)\left(\frac{T}{2\pi}\right)^2 \qquad (1)$$

where T is the natural period of the SDOF structure; η is the modified coefficient of damping, which is expressed as $\eta = \sqrt{0.07/(0.02+\xi)}$ in order to consider the effects of damping property on the structural displacement response. In this study, the elastic displacement response spectrum related to S_D soil type under the design peak acceleration 0.4g for various levels of viscous damping is generated shown in Fig.5.

Inelastic Displacement Response Spectra based on Damage Performance

In this study, the Park-Ang[7] model was employed for the assessment of seismic damage which is expressed as a linear combination of the damage caused by maximum inelastic deformation and the cumulative damage resulted from repeated cyclic response, and the dual-damage criterion matched the experiment result well. Park and Ang suggested the total damage could be represented in terms of a damage index DI as following:

$$DI = \frac{\delta}{\delta_u} + \beta\frac{E_H}{P_y\delta_u} = \frac{\mu}{\mu_u} + \beta\frac{E_H}{P_y\delta_y\mu_u} \qquad (2)$$

Ang et al.(1993)[8] suggested the following detailed classification to describe the structural damage performance:

DI<0.1	No damage or localized minor cracking
0.1≤DI<0.25	Minor damage—light cracking throughout
0.25≤DI<0.40	Moderate damage—severe cracking, localized spalling
0.40≤DI<1.0	Severe damage—crushing of concrete, reinforcement exposed
DI>1.0	Collapsed

A formula of equivalent ductility coefficient was presented by Farjfar et.al[9] under Park-Ang dual-damage criterion as:

$$\mu = \frac{\sqrt{1+4DI\beta\gamma^2\mu_u}-1}{2\beta\gamma^2} \qquad (3)$$

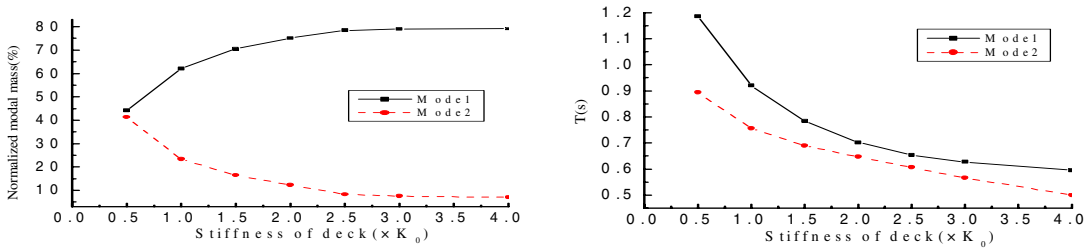
where $\gamma = \frac{\sqrt{E_H/m}}{\omega\delta}$, γ value is designated to 0.8, β value is fixed to 0.15, and the ultimate ductility coefficient μ_u resulted from the monotonic loading experiment is assigned to be 10 in this paper.

Equation (3) indicates that the equivalent ductility coefficient μ is directly related to the structural damage index DI.

The strength reduction factors R_μ [10], directly related to the equivalent ductility coefficient resulted from the damage index DI under the well-known Park-Ang dual-damage criterion, had been studied. By using the obtained R_μ, the constant-ductility inelastic displacement response spectra shown in Fig.6 were constructed based on the elastic displacement spectra shown in Fig.5. The relationship between those spectra was expressed as:

$$S_d = \frac{\mu}{R_\mu} S_{de} \tag{4}$$

THE SIMPLIFIED DISPLACEMENT-BASED DESIGN PROCEDURE



(a) (b)

Fig7 Effects of variety of deck stiffness on normalized modal mass and natural modal periods

For general regular bridge structures, the transverse moment-resisting stiffness of deck is very large so that the fundamental modal effects resulting from transverse vibration of the superstructure plays the predominant role during the total response of structures. With the decrease of the transverse stiffness of the superstructure, the transverse vibration response of bridge structures is not only controlled by the first modal effect predominantly resulted from superstructure response, but also by the multiple modes effects contributed from the transverse vibration of the system. Especially for the asymmetrical bridge shown in Fig.2(b), the increase of pier stiffness and the great difference of the stiffness between adjacent piers have salient effect on the transverse vibration of structures. The effects of variety of deck stiffness on the first two normalized modal masses and natural modal periods of transverse vibration for the asymmetrical bridge are seen in Fig.7(a)(b), in which K_0 is the original transverse moment-resisting stiffness of deck. With the increase of the deck stiffness, the first normalized modal mass of transverse vibration increases gradually, and the effects of the higher modes decrease accordingly. If the transverse moment-resisting stiffness of deck is large enough, consideration of the basic modal effect is suitable in the displacement-based design procedure, while it must consider multiple mode effects for the bridge structures of flexible superstructure.

The proposed displacement-based seismic design procedure for MDOF bridge structures is summarily described as following: firstly to perform modal analysis for the elastic MDOF system, then to calculate the modal parameters such as the modal shape, modal participation factors and corresponding modal periods of the original elastic MDOF system. The key step in the complete design procedure is to determine the spectral displacement of each mode for piers by entering the inelastic displacement response spectra with reference to the equivalent ductility, equivalent viscous damping and modal periods of each mode. The next important step is to obtain the displacement at top of each pier using SRSS method to consider the multiple mode effects, and then to determine the target displacement pattern of MDOF system in the transverse direction according to the calculated displacement at top of each pier and the prescribed damage limit states for piers. Having established the approach for determining the target-displaced shape, the next step is to characterize the MDOF system of multi-span bridge as an equivalent SDOF system. The various system properties of the equivalent SDOF system such as system target displacement, system hysteretic damping and effective system mass must be defined. After the equivalent SDOF system has been identified, the effective period of the substitute structure is obtained by entering the inelastic displacement response spectra shown in Fig.6 with the system target displacement with reference to the equivalent ductility and system hysteretic damping, and then the effective stiffness at maximum response and base shear force of the equivalent SDOF system can be obtained. The last step is to perform the lateral force distribution, the cross section design and the capacity check for piers.

Using the same design procedure for the symmetrical and asymmetrical bridges shown in Fig.2, the important difference between them is in that the proposed design procedure can obtain the rational design results by considering effect of the first mode for symmetrical bridge, while it should consider multiple mode effects for asymmetrical bridge. The detailed design process is described as following:

At first, the target displacement criteria of each bent can be specified as[4]:

$$\Delta_{i0} = drift \cdot h_i \qquad (5)$$

where, *drift* means the design drift ratio determined by damage control limit states, in this paper it's value is fixed to 1%; h_i means the height of the *ith* pier.

According to the linear elastic system of MDOF bridge structures under the consistent oscillation of seismic ground motion, the equation of motion is written in the form:

$$[M]\{\ddot{X}(t)\}+[C]\{\dot{X}(t)\}+[K]\{X(t)\}=-[M]\{1\}\ddot{u}_g \qquad (6)$$

Equation（6）can be decoupled using modal analysis procedure:

$$\ddot{q}_j + 2\xi_j\omega_j\dot{q}_j + \omega_j^2 q_j = -\gamma_j\ddot{u}_g \qquad (j=1,2\cdots n) \qquad (7)$$

where, u_g is the displacement of the input ground motion; ω_j and ξ_j are the natural frequency and damping ratio of the *jth* mode respectively; γ_j represents the *jth* modal participation factor as:

$$\gamma_j = \frac{\{\phi\}_j^T[M]\{1\}}{\{\phi\}_j^T[M]\{\phi\}_j} = \frac{\sum_{i=1}^{n} m_i\phi_{ij}}{\sum_{i=1}^{n} m_i\phi_{ij}^2} \qquad (8)$$

where the index *i* represents the bent number and the index *j* represents the mode number, m_i is the equivalent mass of *ith* pier, and ϕ_{ij} is the modal function for bent *i* and mode *j*. Δ_{ij} is the displacement quantity at the top of pier associated with bent *i* and mode *j*, and then can be expressed as:

$$\Delta_{ij} = \phi_{ij}\gamma_j Sd_j\left(T_j, \xi_{effj}\right) \qquad (9)$$

in equation (9), the inelastic spectral displacement $Sd_j(T_j, \xi_{effj})$ for mode *j* is obtained from the inelastic displacement response spectra in Fig.6 by considering the *jth* modal period, equivalent ductility coefficient μ_{effj} and equivalent system damping value ξ_{effj} for mode *j*. It's noted that T_j is corresponding to the *jth* modal period of the original elastic MDOF system taken as the substitute structure in the design procedure, which can be easily obtained from the previously performed modal analysis. The equivalent ductility coefficient μ_{effj} and equivalent system damping value ξ_{effj} for mode *j* can be expressed as:

$$\xi_{effj} = \sum_{i=1}^{n} w_{ij}\xi_i \quad \mu_{effj} = \sum_{i=1}^{n} w_{ij}\mu_i \quad w_{ij} = \frac{Q_i\phi_{ij}}{\sum_{k=1}^{n} Q_k\phi_{kj}} \quad Q_i = \frac{h_i}{6(EI)_i}\left(M_{ai}^2 + M_{bi}^2 - M_{ai}M_{bi}\right) \qquad (10)$$

where w_{ij} is the weighting factor for bent *i* and mode *j*, M_{ai} and M_{bi} are the member ending moments of *ith* pier respectively determined by the initial structural analysis, ξ_i is the equivalent viscous damping ratio of the *ith* pier[2]:

$$\xi_i = 0.05 + \frac{1-(1-\alpha)/\sqrt{\mu_i} - \alpha\sqrt{\mu_i}}{\pi} \qquad (11)$$

where μ_i is the equivalent ductility coefficient of the *ith* pier, α is the post-yielding stiffness ratio assumed to be 0.05.

Through response spectrum analysis considering higher mode effects, the overall displaced shape in the transverse direction of the MDOF bridge structure is then obtained by an appropriate combination of modes such as SRSS or CQC method and considering the target displacement criteria of each bent. The displacement pattern of each bent is determined by equation(12):

$$\Delta_i = \sqrt{\sum_{j=1}^{m} \Delta_{ij}^2} \qquad (12)$$

After determining the overall target displacement shape of MDOF bridge structures, the MDOF system can be characterized as an equivalent SDOF system by defining the responding system target displacement, system hysteretic damping and system mass.

（1）System target displacement

According to the requirement that the work done by the equivalent SDOF system must be the same as that done by the MDOF system, the system target displacement Δ_{eff} can be defined as[4]:

$$\Delta_{eff} = \frac{\sum_{i=1}^{n} m_i \Delta_i^2}{\sum_{i=1}^{n} m_i \Delta_i} \tag{13}$$

（2）System hysteretic damping and equivalent system ductility

Shibata and Sozen[11] suggested that system hysteretic damping can be weighted according to flexural strain energy for the substitute approach. The system hysteretic damping ξ_{eff} and equivalent ductility μ_{eff} of the equivalent SDOF system can be obtained by equation (14).

$$\xi_{eff} = \sum_{i=1}^{n} w_i \xi_i \quad \mu_{eff} = \sum_{i=1}^{n} w_i \mu_i \quad w_i = \frac{Q_i}{\sum_{k=1}^{n} Q_k} \quad Q_i = \frac{h_i}{6(EI)_i}\left(M_{ai}^2 + M_{bi}^2 - M_{ai} M_{bi}\right) \tag{14}$$

In equation (14), w_i is the weighting factor of the *ith* pier which is assumed to be not variable during the seismic response of the MDOF system in this paper, so the weighting factor of each bent and the system hysteretic damping are then determined through the initial structural analysis. The equivalent ductility coefficient μ_{eff} can also be calculated through the same procedure by considering the ductility capacity of each bent and the yielding mechanism of overall bridge structures according to Eurocode 8.

（3）Effective system mass

The effective system mass is defined by the requirement of work equivalence between the MDOF system and responding equivalent SDOF system[4], and then it can be obtained by equation (15) where Δ_i is the target displacement for *ith* pier and m_i is the equivalent mass of *ith* pier.

$$M_{eff} = \sum_{i=1}^{n} \left(\frac{\Delta_i}{\Delta_{eff}} m_i \right) \tag{15}$$

Since the overall displaced shape of the bridge structures in the transverse direction, which is consisted of the target displacement of each bent, was established with reference to multiple modes of response, the effective system mass of equivalent SDOF system considers the effects of multiple modes of response.

（4）Effective system stiffness and force distribution

After determining the equivalent SDOF system, the effective period of the substitute structure is obtained by entering the constant-ductility inelastic displacement response spectra in Fig.6 according to the system target displacement, equivalent system ductility and system hysteretic damping. The effective stiffness and base shear force for the equivalent SDOF system are then obtained as following:

$$K_{eff} = \frac{4\pi^2 M_{eff}}{T_{eff}^2} \qquad F_{eff} = K_{eff} \Delta_{eff} \tag{16}$$

After establishing the total design force of the equivalent SDOF system, the column design forces of the responding MDOF system are then determined with equation (17) suggested by Kowalsky[3] where h_i is the height of *ith* column.

$$F_i = \frac{1/h_i}{\sum_{i=1}^{n} 1/h_i} F_{eff} \tag{17}$$

Once the column lateral forces are obtained with equation (17), the column moments are easily calculated, the column sections are then designed according to the moment capacity, and then the

longitudinal and transverse reinforcement are designed to resist the responding design moment and target displacement.

EXAMPLE BRIDGE DESIGNS

The proposed displacement-based seismic design procedure was evaluated by applying it to the symmetrical and asymmetrical MDOF continuous bridges shown in Fig.2 used previously in reference [3]. Each bridge had four 50m spans and identical pier cross-sections, differing only in the heights of the three columns. The yield stress value of steel was assigned to be 455Mpa, and concrete compressive strength was 35Mpa, and the elastic modulus of all the concrete members was 29.6Gpa. The bridge deck was assumed to be pinned at the abutments, and all the piers were assumed to be fully fixed at the base and pinned at the deck. The seismic excitation has been applied simultaneously to the bases of all piers, neglecting any effect of non-synchronism excitation. Smoothed elastic and inelastic displacement response spectra are shown in Fig.5 and Fig.6 responding to a maximum ground acceleration of 0.4g on subsoil class D defined in the UBC97. The section properties of two bridges are listed in Table 1 and the detailed designs of both bridges are described below.

Table 1. Section properties of symmetrical and asymmetrical bridges

Section Properties	Symmetrical Bridges（7+11+7）		Asymmetrical Bridges（14+7+21）	
	Deck	Pier	Deck	Pier
Area（m^2）	4.0	1.766	4.81	3.14
I_{xx} （m^4）	25.0	0.248	16.46	0.784
I_{yy} （m^4）	4.0	0.248	1.91	0.784
Weight（t/m）	12.44	4.5	12.44	4.5

Symmetrical Bridge Design

In the design of the symmetrical bridge, the diameter of each circular pier was assumed to be 1.5m, and the initial target displaced shape was determined by specifying a desired drift ratio of 1% in each of the piers, or displacements of 0.07, 0.11 and 0.07m respectively at the top of the piers. According to the symmetrical bridge, the ductility capacity of each pier was assumed to be performed well during the seismic response. In this paper the damage index DI of each pier was assigned to be 0.55 with reference to the previously presented design limit states under the specified design ground motion. The equivalent ductility coefficient of each pier is 4.0 calculated with equation (3) according to the responding damage index DI, and the equivalent viscous damping of each pier is 0.185 calculated with equation (11). Since the symmetrical bridge responds predominantly in the first mode, the simplified design procedure can obtain the rational design results by considering effect of the first mode in the transverse direction. The detailed design results are summarized in Table 2.

Table 2. Summary of Design Results for Symmetrical Bridge

Bent No.	Pier 2	Pier 3	Pier 4
h_i(m)	7.0	11.0	7.0
M_{ai},M_{bi}	M_{a2}=1.73, M_{b2} =9.67	M_{a3}=15.6, M_{b3} =4.28	M_{a4}=1.73, M_{b4} =9.67
w_i	0.152	0.696	0.152
m_i(t)	630.0	634.0	630.0
μ_i	4.0	4.0	4.0
ξ_i	0.185	0.185	0.185
Modal analysis	T_1=0.83s, γ_1=1.33, ξ_{eff1}=0.185, μ_{eff1}=4.0,sd_1=0.077m		
Δ_i(m)	0.051	0.102	0.051
Equivalent SDOF System	ξ_{eff}=0.185, μ_{eff}=4.0, Δ_{eff}=0.077m,M_{eff}=1688*10^3kg,T_{eff}=0.83s, K_{eff}=96733KN/m, F_{eff}=7448KN		
F_i(KN)	2825	1798	2825
M_i(KN.m)	19775	19778	19775
ρ_i(%)	4.15	4.15	4.15

Asymmetrical Bridge Design

In the design of the asymmetrical bridge, the diameter of each circular pier was assumed to be 2.0m, and the initial target displaced shape was determined by specifying a desired drift ratio of 1% in each of the piers, or displacements of 0.14, 0.07 and 0.21m respectively at the top of the piers. According to the asymmetrical bridge, the ductility capacity of each pier couldn't be simultaneously performed well during the seismic response, so it's impossible to form the plastic hinge at the bottom of each pier at the same time. Generally the first plastic hinge was generated at the bottom of the shortest one of pier3, while the pier4 maybe sustained elastic until the collapse of the whole bridge structure. In this paper the damage index DI was assigned to be 0.24, 0.55 and 0.11 respectively for pier2, pier3 and pier4 with reference to the previously presented design limit states under the specified design ground motion. The equivalent ductility coefficients are 2.0, 4.0 and 1.0 for pier2, pier3 and pier4 calculated with equation (3) according to the responding damage index DI, and the equivalent viscous damping are 0.143, 0.185 and 0.05 for pier2, pier3 and pier4 calculated with equation (11). Unlike the design of the symmetrical bridge, the simplified design procedure can obtain the rational design results by considering effect of the first two modes in the transverse direction. The detailed design results are summarized in Table 3.

Table 3. Summary of Design Results for Asymmetrical Bridge

Bent No.	Pier2	Pier 3	Pier 4
h_i(m)	14.0	7.0	21.0
M_{ai},M_{bi}	M_{a2}=5.49, M_{b2} =9.22	M_{a3}=13.6, M_{b3} =11.9	M_{a4}=3.85, M_{b4} =14.4
w_i	0.199	0.296	0.505
m_i(t)	638.0	630.0	646.0
μ_i	2.0	4.0	1.0
ξ_i	0.132	0.185	0.05
Modal analysis	First modal: T_1=0.89s, γ_1=0.57, ξ_{eff1}=0.043, μ_{eff1}=1.01,sd_1=0.196m Second modal: T_2=0.68s, γ_2=1.36, ξ_{eff1}=0.107, μ_{eff2}=1.93,sd_2=0.076m		
Δ_i(m)	0.113	0.031	0.119
Equivalent SDOF System	ξ_{eff}=0.106, μ_{eff}=2.09, Δ_{eff}=0.106m,M_{eff}=1589*10^3kg,T_{eff}=0.90s, K_{eff}=77442KN/m, F_{eff}=8255KN		
F_i(KN)	2251	4503	1501
M_i(KN.m)	31514	31549	32521
ρ_i(%)	2.76	2.76	2.76

Verification of Design Approach with Time-history Analysis

In order to verify the accuracy of the simplified displacement-based design procedure in terms of meeting the design displacement and the specified damage levels, the equivalent SDOF systems responding to the symmetrical and asymmetrical bridges were subjected to dynamic inelastic time-history analysis. The analysis was performed using eight near-field acceleration time histories on subsoil class D defined in the UBC97 of which the peak values were all adjusted to 0.4g in order to compare the non-linear time-history analysis results with the desired design displacements resulting from the presented displacement-based design procedure in this paper.

Table 4. Time-history Analysis Results of Equivalent SDOF System

Earthquakes	PGA(g)	DAF	D_{sym} (m)	D_{asym}(m)
ImperialValleyElcentro#5230	0.38	1.0526	0.082	0.123
ImperialValleyElcentro#6230	0.44	0.9091	0.073	0.095
ImperialValleyElcentro#7230	0.46	0.8696	0.094	0.141
ImperialValleyHotvillePostOffice#225	0.25	1.6	0.079	0.119
ImperialValleyMelolandOFF#000	0.31	1.2903	0.076	0.112
CapmendicinoPetrolia#000	0.59	0.6779	0.069	0.103
Kobetakarazuka#090	0.69	0.5797	0.089	0.135
LomaprietaGilroyHistoricBLDG#090	0.28	1.4286	0.068	0.086
Average Value	0.425	/	0.0787	0.114

As shown in Table 4, the calculated average maximum displacements from the non-linear analysis of the equivalent SDOF systems for symmetrical and asymmetrical bridges are 0.0787m and 0.114m respectively. Compared to the respectively desired design displacements of the equivalent SDOF systems, which are 0.077m shown in Table 2 and 0.106m shown in Table 3, the calculated average maximum displacements from the non-linear analysis agree well with the desired design displacements respectively.

CONCLUDING REMARKS

The displacement-based seismic design offers the ability to control explicitly the displacement demand in each member and to consider structural damage directly related to displacement demands responding to different displacement limit states. In this study, the elastic displacement design spectra considering near-field effects were presented, and the inelastic displacement design spectra based on structural damage performance are also constructed. This paper introduced the response spectrum-based approach to consider the effects of higher modes on the structural response of MDOF continuous bridges on the transverse direction under earthquake excitation. An improved displacement-based seismic design method for MDOF bridge structures based on constant-ductility inelastic displacement design spectra is presented. In the design procedure, there is no iteration and the specified design objective ductility can be directly realized. In this complete design process for two example bridges, the simplified design procedure can obtain the rational design results by considering effect of the first mode for symmetrical bridge, while it must consider effects of the first two modes for asymmetrical bridge to obtain the rational design results in the transverse direction. Since the non-linear dynamic time-history analysis results for the equivalent SDOF system agree well with the desired design results respectively, it indicates the simplified displacement-based seismic design method for MDOF bridge structures is sound.

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