중약진지역에 위치한 콘크리트교량의 수정내진설계

Modified Earthquake Resistant Design for a Concrete Bridge in the Low to Moderate Seismic Region

국 승 규*
Kook, Seung Kyu

국문요약

구조물에 내진설계를 적용하는 목적은 지진에 노출되는 구조물에 안전성과 경제성을 고려한 파괴메카니즘을 부여하는 것이다. 내진설계에 보편적으로 적용하고 있는 응답스펙트럼해석법은 선형해석법으로 구조물의 비선형 동적거동에 의한 영향은 특정한 계수로 반영한다. 그러나 기존의 내진설계시방서들이 강진지역에 있는 나라들에 의해 제정 및 개정되어 왔기 때문에 응답스펙트럼해석법 뿐만 아니라 기타의 적용규정이 강진지역에 위치한 구조물의 상황만을 고려하여 제시되었다. 따라서 중약진지역에 위치한 구조물의 내진설계에 대한 별도의 연구가 요구되고 있다. 이 연구에서는 중약진지역에 위치한 콘크리트 교량을 선정하여 비선형 동적거동을 반영하는 계수를 결정하고 응답스펙트럼해석법을 적용하였다. 연구결과를 바탕으로 중약진지역의 교량에 대해 내진설계의 목적을 만족하는 개선된 내진설계절차를 제시하였다.

주요어 : 내진설계, 파괴메카니즘, 응답스펙트럼해석법, 비선형 동적거동, 중약진지역

ABSTRACT

The objective of the earthquake resistant design of structures is to provide failure mechanisms to structures under the seismic events in view of safety and economy. The response spectrum analysis method is a widely introduced linear analysis method for the earthquake resistant design, where the influence of the non-linear dynamic behaviour of structures is taken into account by introducing particular factors. However, because the earthquake resistant design codes are developed in strong seismic countries, not only the response spectrum analysis method but also the other provisions are prepared considering the situation of structures located in those countries. Therefore it is required to perform separate studies for the earthquake resistant design of structures in the low to moderate seismic regions. In this study a concrete bridge located in the low to moderate seismic regions is selected. The determination of a factor representing the non-linear dynamic behaviour of the bridge is carried out and the response spectrum analysis method is applied. From the study results, modified procedures satisfying the objective of the earthquake resistant design for bridges in low to moderate seismic regions are summarised.

Key words : *earthquake resistant design, failure mechanism, response spectrum analysis method, non-linear dynamic behaviour, low to moderate seismic regions*

1. Introduction

For the earthquake resistant design of bridges the response spectrum analysis method can be applied as provided in Eurocode 8, Part 2[1] or in AASHTO.[2] The response spectrum analysis method is a simplified linear analysis method, because the non-linear dynamic behaviour is considered by introducing either the beha-

* 김회원 · 부경대학교 토목공학과 조교수
본 논문에 대한 토의를 2000년 12월 30일까지 학회로 보내 주시면 그 결과를 게재하겠습니다.

viour factor as provided in Eurocode 8 or the response modification factor as provided in AASHTO. Different values according to the bridge types are provided for the behaviour factor as well as for the response modification factor. In Eurocode 8 the elastic response spectrum is modified first with the behaviour factor to get the design response spectrum, from which the basic design forces are obtained. Then the basic design forces are adjusted to achieve the intended failure mechanism(capacity design). In AASHTO the section forces are calculated first with the elastic design response spectrum and these forces are modified into the basic design forces with different response modification factors for connection, substructure, foundation such that the capacity design effects are satisfied. Although the two methods are different in their applications, they have the same objective to provide a safe and economic failure mechanism.

For the determination of the response spectrum the regional seismicity is considered as the design seismic event, which is generally defined as an earthquake having a 500-year return period. In the strong seismic region the section forces obtained with the seismic design load combination are beyond the section forces determined with the non-seismic design load combinations, e.g. including wind. The seismic design load combination governs for the decision of the basic design forces. In the low to moderate seismic regions, however, the section forces obtained with the seismic design load combination are in many cases less than the section forces from the non-seismic design load combinations. The seismic design load combination does not govern for the decision of the basic design forces. Also, for bridges in the low to moderate seismic regions, simplified criteria such as the accessibility of potential plastic hinges, the design of bearings/links and the required seating lengths are provided in the codes. Nevertheless, because these criteria are also based on the results of the response spectrum analysis method, they can not satisfy the objective of the earthquake resistant design. For bridges in the low to moderate seismic regions, therefore, modifications of the earthquake resistant design procedures provided in the present codes are required.

In this study a concrete bridge with a single pier located in the low to moderate seismic regions is selected for the earthquake resistant design. The dynamic behaviour of the bridge is analysed through non-linear time step calculations and evaluated as a behaviour factor. Then the response spectrum analysis method is applied according to the DIN 4149[3], NAD (National Application Document) of Eurocode 8, and the section forces are determined. Based on the calculation results the modified earthquake resistant design procedures are set forth for bridges in the low to moderate seismic regions.

2. Determination of the behaviour factor

2.1 Analysis model

The concrete bridge with a single pier is linked to a cable stayed composite bridge with a concrete pylon(Fig. 1).

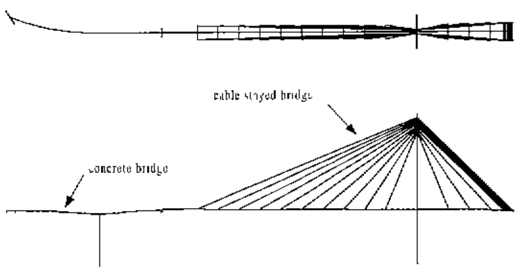

Fig. 1 Plan and side view of the model for the two linked bridges

The following definition of the global directions is used:

- x and y direction : horizontal, parallel and transverse to the bridge axis respectively
- z direction : vertical, positive direction according to the gravity vector

For the demonstration of the earthquake resistant design the behaviour factor for the global y direction is to be determined in this study. As the dynamic behaviour of the concrete bridge is interrelated with the linked cable stayed bridge, the concrete bridge is adjusted in such a way that the same dynamic behaviour is to be obtained when the adjusted model is analyzed alone. The adjustment is carried out as follows:

- Using the model of the two linked bridges the modal analysis is performed, the results of which are eigenmodes, eigenfrequencies and corresponding participation factors. Table 1 shows the significant modes in the global y direction from the 40 modes, where ε_x, ε_y and ε_z represent the modal mass participation factor for each direction respectively.
- The original model of the concrete bridge is adjusted by applying an additional mass at the right end of the bridge deck(Fig. 2). Table 2 shows the significant modes in the global y direction from the 10 modes.
- The first mode in the global y direction of the two linked bridges(Fig. 3) is compared to the first mode in the global y direction of the adjusted model(Fig. 4).

The comparison of the first horizontal modes shows that the adjusted model can be taken as an analysis model representing the dynamic behaviour of the concrete bridge as a part of the two linked bridges. The analysis model has 58 frame type elements and 59 nodes. The

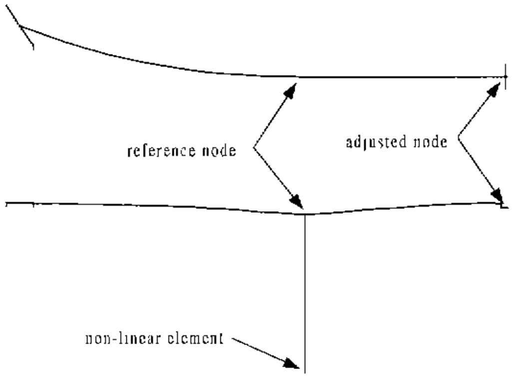

Fig. 2 Plan and side view of the model for the concrete bridge

Table 1 Eigenfrequencies and participation factors of the two linked bridges

MODE	ω[rad/s]	ε_x	$\sum\varepsilon_x$	ε_y	$\sum\varepsilon_y$	ε_z	$\sum\varepsilon_z$
1	1.81	0.000	0.000	0.370	0.370	0.000	0.000
28	6.42	0.000	0.572	0.367	0.741	0.000	0.191
31	6.52	0.000	0.572	0.021	0.763	0.000	0.191
36	6.68	0.000	0.580	0.007	0.770	0.000	0.192

Table 2 Eigenfrequencies and participation factors of the analysis model

MODE	ω[rad/s]	ε_x	$\sum\varepsilon_x$	ε_y	$\sum\varepsilon_y$	ε_z	$\sum\varepsilon_z$
1	1.81	0.001	0.001	0.594	0.594	0.000	0.000
4	7.86	0.000	0.930	0.201	0.796	0.000	0.087
6	24.32	0.000	0.937	0.020	0.816	0.000	0.399

bridge deck has trapezoidal type sections with variable width and depth(Fig. 5). The pier is composed of two box type sections and the lower section is given in Fig. 6. The designed elements satisfy the non-seismic design load combinations.

Fig 3 First horizontal mode in the global y direction of the model for the two linked bridges (plan view)

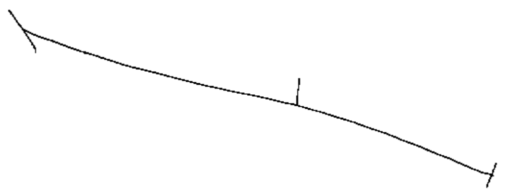

Fig 4 First horizontal mode in the global y direction of the adjusted model for the concrete bridge (plan view)

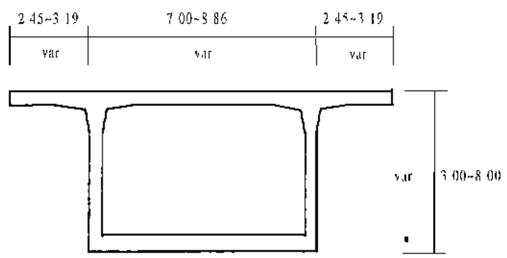

Fig. 5 Bridge deck section with variable width and depth (unit in m)

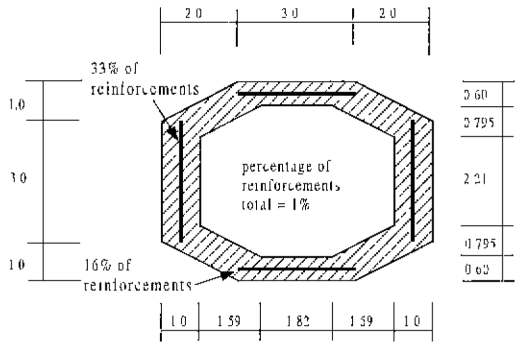

Fig. 6 Lower section of the pier with the assumed reinforcement distribution (unit in m)

2.2 Non-linear behaviour

The behaviour factor determination requires the estimation of the elastic limit of the analysis model. As the dissipative zone is to be located at the pier base(Fig. 2), the plastic moment at the lower section of the pier is used for the estimation of the elastic limit. For bending about the global x axis the plastic moment of M_{pl}=204000kNm is calculated with the reinforcement ratio of 1%. The reinforcement distribution in the lower section is assumed as shown in Fig. 6. In order to approximate the real behaviour of the dissipative zone leading to a plastic hinge formation the following material properties are applied without any safety factor:

- concrete
  modulus of elasticity : 3700KN/cm^2
  compressive strength : 4.50KN/cm^2
  tensile strength : 0.45KN/cm^2
  maximum allowable compressive strain : 3.5‰
- reinforcement
  modulus of elasticity : 21000KN/cm^2
  yield strength : 42KN/cm^2
  tensile strain : 2%

For the non-linear time step calculations the non-linear moment rotation characteristic of the dissipative zone is determined considering the effects of crack formation in concrete, tension stiffening and 1% strain hardening of reinforcement(Fig. 7). The tension stiffening effect is considered according to DIN 188005[4] with an ideal reinforcements area $A_{s,id}$.

$$A_{s,id} = \frac{A_s}{1 - \frac{0.5 \cdot f_{ctm}}{\rho_s \cdot f_{sk}}} \tag{1}$$

In the equation (1) A_s is the reinforcements

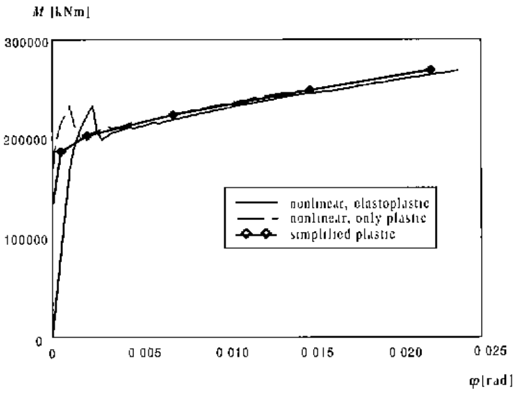

Fig. 7 Moment rotation characteristic of the non-linear element

area, ρ_s is the reinforcements ratio, f_{ctm} is the tensile strength of concrete and f_{sk} is the yield strength of reinforcement. The initial curve showing the crack formation is simplified with the assumption of concrete already cracked and represented as a simplified plastic curve. This non-linear moment rotation characteristic is used for the calculation of the hysteresis of the plastic hinge at the pier base.

2.3 Synthetic motions

With the program SIMQKE[5] 10 synthetic motions (acceleration time history) are simulated as seismic inputs. The elastic response spectrum provided in DIN 4149 is used for the simulation.

$$T \leq T_B: S_e(T) = a_g \cdot S \cdot \left\{1 + \frac{T}{T_B}(\eta \cdot \beta_0 - 1)\right\} \quad (2a)$$

$$T_B \leq T \leq T_C: S_e(T) = a_g \cdot S \cdot \eta \cdot \beta_0 \quad (2b)$$

$$T_c \leq T \leq T_D: S_e(T) = a_g \cdot S \cdot \eta \cdot \beta_0 \cdot \left(\frac{T_C}{T}\right)^{k_1} \quad (2c)$$

$$T_D \leq T: S_e(T) = a_g \cdot S \cdot \eta \cdot \beta_0 \cdot \left(\frac{T_C}{T_D}\right)^{k_1} \cdot \left(\frac{T_D}{T}\right)^{k_2} \quad (2d)$$

In the equation (2) T is the vibration period of a linear SDOF system, T_B & T_C are the limits of the constant spectral acceleration branch and T_D is the value defining the beginning of the constant displacement range. S_e is the ordinate of the elastic response spectrum, a_g is the design ground acceleration for the reference period and β_0 is the spectral acceleration amplification factor for 5% viscous damping. S is the soil parameter, η is the damping correction factor (η=1 for 5% viscous damping) and k_1 & k_2 are exponents which influence the shape of the spectrum for a vibration period greater than T_C & T_D respectively. a_g of 0.6m/s^2 is taken as EPA (effective peak ground acceleration of the design seismic event) and the parameters for the soil type B3 provided in DIN 4149 are given in Table 3. The synthetic motions have a total duration of 16 seconds (Fig. 8; 2 sec. rise time, 5 sec. level time).

Both the target elastic response spectrum and the response spectrum with 5% damping obtained with the synthetic motion 01 are presented in Fig. 9.

Table 3 Parameters for the elastic response spectrum (soil type B3)

S	η	β_0	T_B [s]	T_C [s]	T_D [s]	k_1	k_2
1.0	1.0	2.5	0.1	0.6	2.0	1.0	2.0

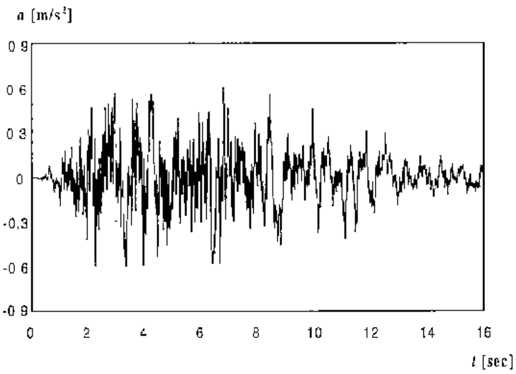

Fig. 8 Synthetic motion 01 (seismic input 01, a_g of 0.6m/s^2)

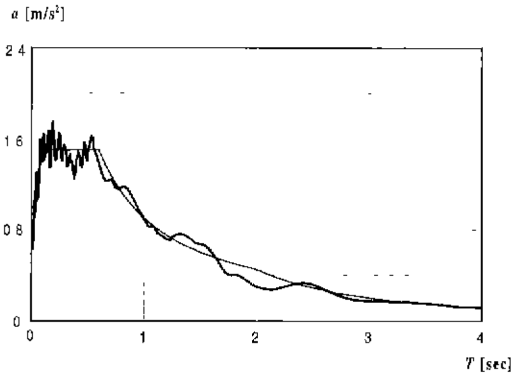

Fig. 9 Response spectrum (5% damping) obtained with the synthetic motion 01

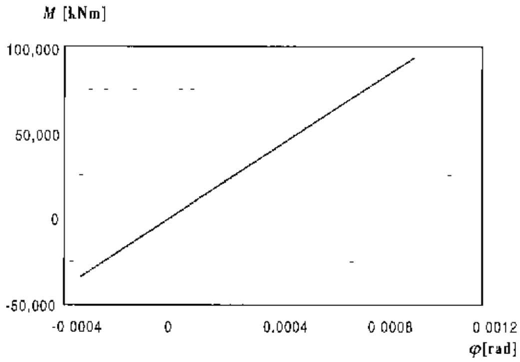

Fig. 10 Hysteresis of the non-linear element obtained with the seismic input 01, a_g of 0.6 m/s^2 (M_{max}=93500kNm, φ_{max}=0.000938)

2.4 Non-linear time step calculations

The non-linear time step calculations are carried out with the program DYNACS.[6] For all seismic inputs acceleration values leading to the elastic limit at the plastic hinge are determined for the first step and considered as factor 1 values(Table 4 & Fig. 10).

The second step is to calculate the displacement time histories of the reference node (pier top as shown in Fig. 2) with the seismic inputs of factor 1, from which the maximum and minimum displacements are obtained(Fig. 11). Further calculations are carried out with the multiples of factor 1 values(Table 4). From the calculation results the hysteresis of the plastic hinge are checked with regard to the assumed allowable compressive strain of 3.5‰(Fig. 12) and the

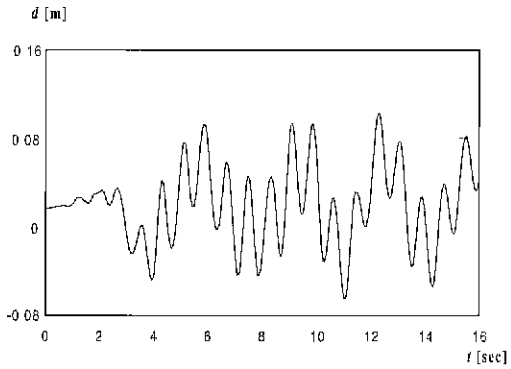

Fig. 11 Displacement history of the reference node obtained with the seismic input 01, factor 1 (d_{max}=0.1026m, d_{min}=-0.0657m)

Table 4 Factors for the non-linear time step calculations (normalized by a_g of 0.6m/sec^2)

Seismic input	factor 1 (1×M_{pl})	factor 2 (2×M_{pl})	factor 3 (3×M_{pl})	factor 4 (4×M_{pl})	factor 5 (5×M_{pl})
01	2.182	4.364	6.545	8.727	10.909
02	2.698	5.397	8.095	10.794	13.492
03	2.776	5.551	8.327	11.102	13.878
04	2.316	4.631	6.947	9.262	11.578
05	2.356	4.711	7.067	9.423	11.778
06	2.331	4.663	6.994	9.326	11.657
07	2.464	4.928	7.391	9.855	12.319
08	2.724	5.447	8.171	10.895	13.618
09	2.297	4.595	6.892	9.189	11.486
10	2.249	4.498	6.748	8.997	11.246

maximum and minimum displacements of the reference node are obtained(Fig. 13).

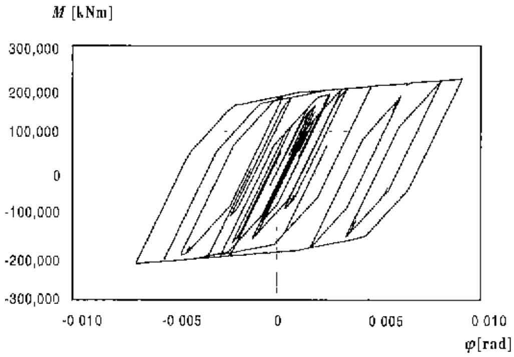

Fig. 12 Hysteresis of the non-linear element obtained with the seismic input 01, factor 5 (M_{min}= −215000kNm, φ_{min}=−0.0069, M_{max}=223000kNm, φ_{max}=0.0090)

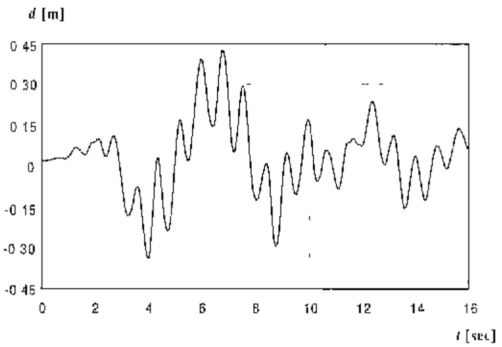

Fig. 13 Displacement history of the reference node obtained with the seismic input 01, factor 5 (d_{max}=0.4245m, d_{min}=−0.3428m)

2.5 Behaviour factor

The behaviour factor is to be determined according to the definition of Ballio/Perotti[7] with the extension according to the research works.[(8)-(10)] The displacements of the reference node (absolute sum of the maximum and minimum values) obtained with different factors constitute a non-linear curve, which is compared with the extrapolated linear elastic behaviou(Fig. 14).

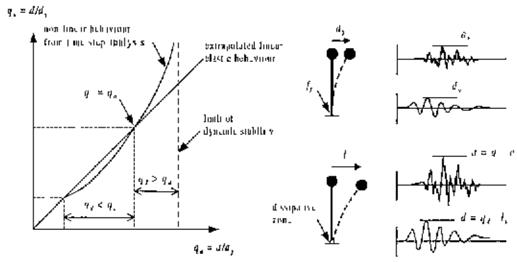

Fig. 14 Definition of the behavior factor with q_a and q_d

According to the definition of Ballio/Perotti the behaviour factor, q factor as defined in Eurocode 8, Part 1-1[(11)], is obtained at the intersection point and this point is also regarded as the dynamic stability limit. In this case the value q_a is equal to the value q_d with the consequence, that the displacement values obtained through a reduction of the elastic response spectrum by q_a should be increased by q_d. The results of the research works[(8)-(10)] have shown, however, that the dynamic stability limit does not always correspond to the intersection point depending on the P-Δ effect or the possible strength degradation. In case the dynamic stability limit is achieved later, it is more economical to take the dynamic stability limit as the q factor. This factor is introduced in Eurocode 8, Part 1-2[(12)] as the displacement behaviour factor q_d. Conservatively q_a factor can be taken for the reduction of the elastic response spectrum and q_d factor can be taken for the adjustment of the displacement values.

q_a and q_d factors determined with the non-linear time step calculation results are listed in Table 5. Three representative cases are given in Fig. 15. For the analysis model the most conservative factor q_a of 2.5 is obtained by applying the seismic input 03. The conservative decision is based on the consideration that the behaviour factors are evaluated with an assumption of the stable hysteresis. The

Table 5 Behavior factors obtained with 10 seismic inputs

Seismic input	01	02	03	04	05	06	07	08	09	10
q_a	> 5.0	4.0	2.5	> 5.0	> 5.0	> 5.0	4.0	4.5	> 5.0	> 5.0
q_d	> 5.0	> 5.0	4.5	> 5.0	> 5.0	> 5.0	> 5.0	> 5.0	> 5.0	> 5.0

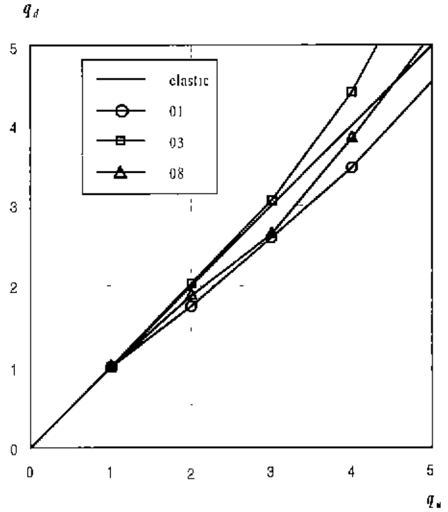

Fig. 15 Results of the non-linear time step calculations (seismic input 01, 03 & 08)

stable hysteresis without any premature damage in the dissipative zones requires the following measures.

- The cross section of the dissipative zone has to be designed satisfying the required ductile behaviour. Ductile materials should be used to prevent a premature rupture of the reinforcement in tension. The concrete cracks should be well distributed. Also sufficient confinements should be provided to prevent a local buckling of the reinforcements in compression.
- All other structural parts(foundation, connection of the pier to the foundation, connection of the bearings and superstructure) have to be designed according to the capacity design considering the material overstrength in order to prevent unintended failure mechanisms(overturning of the foundation, collapse of the connections).

The behaviour factor of 2.5 is within the range provided in Eurocode 8, Part 2(1.5 for limited ductile bridges and 3.5 for ductile bridges). Also it is comparable to the response modification factor(R_w) of 3.0 for the single pier provided in AASHTO. The results of the nonlinear time step calculations obtained with the seismic inputs 01 and 03(Table 6) provide that.

- the elastic limit is reached with the seismic input 01 at a_g=1.3m/s^2
- the dynamic stability limit is reached with the seismic input 03 at a_g=4.2m/s^2

Table 6 Section forces of the non-linear element from the non-linear time step calculations

factor	seismic input 01			seismic input 03		
	a_g [m/s^2]	M_x [kNm]	V_y [kN]	a_g [m/s^2]	M_x [kNm]	V_y [kN]
–	0.600	93500	4520	0.600	73500	3850
1	1.309	155000	7550	1.666	141000	7680
2	2.618	197000	11800	3.331	193000	12500
3	3.927	207000	15000	4.996	206000	16900
4	5.236	214000	16300	6.661	215000	20600
5	6.545	223000	18000	8.327	225000	24900

The two limits are to be considered as limit values for the basic requirements defined by the codes.

- minimization of damage (serviceability limit state) under seismic events with high probability of occurrence during the design life of bridge
- no-collapse requirement (ultimate limit state) under the design seismic event

3. Response spectrum analysis method

3.1 Design spectrum

For the application of the response spectrum analysis method the design response spectrum provided in DIN 4149 is used.

$$T \leq T_\beta:\ S_d(T) = \alpha \cdot S \cdot \left\{1 + \frac{T}{T_B}\left(\frac{\beta_0}{q} - 1\right)\right\} \tag{3a}$$

$$T_B \leq T \leq T_C:\ S_d(T) = \alpha \cdot S \cdot \frac{\beta_0}{q} \tag{3b}$$

$$T_C \leq T \leq T_D:\ S_d(T) = \alpha \cdot S \cdot \frac{\beta_0}{q} \cdot \left(\frac{T_C}{T}\right)^{k_1} \tag{3c}$$

$$T_D \leq T:\ S_d(T) = \alpha \cdot S \cdot \frac{\beta_0}{q} \cdot \left(\frac{T_C}{T_D}\right)^{k_1} \cdot \left(\frac{T_D}{T}\right)^{k_2} \tag{3d}$$

In the equation (3) S_d is the ordinate of the design response spectrum normalized by g, and α is the ratio of the design ground acceleration a_g to g. The parameters are same as those given in Table 3. Both the elastic response spectrum and the design response spectrum are presented in Fig. 16, where q factor of 2.5 is applied.

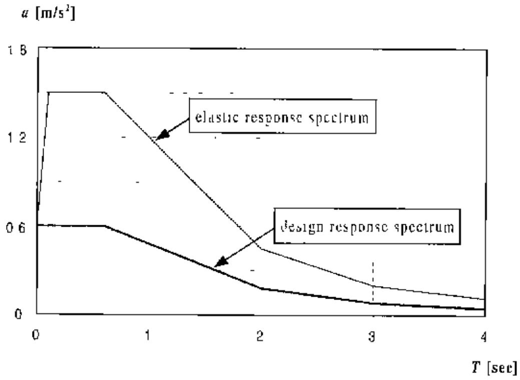

Fig. 16 Elastic response spectrum and design response spectrum with q=2.5

3.2 Section forces

Section forces M_x and V_y in the global direction obtained at the pier base are given in Table 7. Through a seismological study a_g of 0.6m/s^2 and 1.0m/s^2 are given for the design seismic events of 500-year and 5000-year return period respectively. The section forces due to earthquakes show the linearity of the response spectrum analysis method, the ratio of a_g between the two design seismic events and the ratio of q=2.5 between the two spectra.

Table 7 Section forces of the non-linear element (response spectrum analysis method)

Response spectrum	a_g [m/s^2]	Self-weight		Earthquake		Total	
		M_x [kNm]	V_y [kN]	M_x [kNm]	V_y [kN]	M_x [kNm]	V_y [kN]
Design	0.6	28400	1570	18100	515	46500	2085
	1.0			30100	857	58500	2427
Elastic	0.6			45000	1280	73400	2850
	1.0			74100	2110	102500	3680

Even the section forces determined with the elastic response spectrum with a 5000-year return period are below than those determined at the estimated elastic limit of factor 1(Table 6).

4. Earthquake resistant design

According to the design concept of Eurocode 8 the section forces calculated with the design response spectrum (q=2.5) are taken as the basic design forces. In case of applying AASHTO the design force of the pier base moment is to be determined by dividing the section force calculated with the elastic response spectrum by R_w factor of 3(single pier). Both cases mean that the existing elastic limit of the analysis model should be lowered through a reduction of the moment capacity at the pier base. However these values can not be taken as the basic design forces of the analysis model, because they are far below the elastic limit predetermined by the non-seismic design load combinations. Therefore any revision of the pier is not necessary for the analysis model. Nevertheless the capacity design procedure is to be carried out in order to provide the intended failure mechanism, where material overstrength distributions as well as adequate confinements satisfying the ductility requirement should be considered. In this regard the earthquake resistant design procedures for bridges in the low to moderate seismic regions are modified as given in the flow chart(Fig. 17).

4. Conclusions

In this study a concrete bridge with a single pier located in the low to moderate seismic region is selected for the earthquake resistant design. The non-linear dynamic behaviour of the bridge is represented as the behaviour factor determined through the non-linear time step calculations. From the calculation results a conservative decision provides a behaviour factor of 2.5. Also the response spectrum analysis method is applied. Both the section forces with the design response spectrum using the behaviour factor and the section forces with the elastic response spectrum are obtained. The conclusion from this study is to be summarized as follows.

- For bridges in the low to moderate seismic regions in which the section forces obtained with the seismic design load combination are less than the section forces from the non- seismic design load combinations, any redesign of the intended plastic hinge section is not necessary(The seismic design load combination does not govern for the decision of the basic design forces).

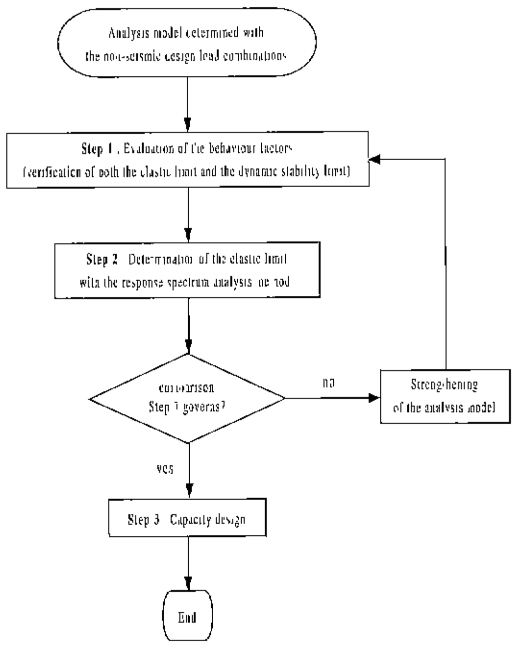

Fig. 17 Modified earthquake resistant design procedures for bridges in the low to moderate seismic regions

- The capacity design procedure is to be carried out in order to provide the intended failure mechanism and to avoid any brittle failure mode, where material overstrength distributions should be considered. Also adequate confinements for the plastic hinge zone satisfying the ductility requirement should be provided.

Acknowledgement

This study is supported by the Pukyong National University, the Steel Construction Institute of RWTH Aachen and the Korea Earthquake Engineering Research Center.

References

1. CEN, *Eurocode 8 - Design Provisions for Earthquake Resistance of Structures, Part 2: Bridges*, Brussels, 1994.
2. AASHTO, *Standard Specification for Highway Bridges, Division I-A: Seismic Design*, Washington, 1996.
3. NABau, *DIN 4149 - Bauten in Deutschen Erdbebengebieten: Auslegung von Hochbauten Gegen Erdbeben*, Berlin, 1998.
4. NABau, *DIN 18800-5 - Stahlbauten, Teil 5: Verbundtragwerke aus Stahl und Beton, Bemessung und Konstruktion*, Berlin, 1999.
5. Vanmarcke, E. H., Cornell, C. A., Gasparini, D. A., and Hou, S. N., "SIMQKE- Simulation of earthquake ground motions," Massachusetts, 1988.
6. Hoffmeister, B. and Kuck, J., "DYNACS: dynamic non-linear analysis of composite and steel structures," Institute of Steel Construction Aachen, 1997.
7. Ballio, G., Perotti, F., Rampazzo, L., and Setti, P., "Determinazione del coefficiente di struttura percostru zioni metalliche sogette a caichi assiali," Convegno Nazionale L'Ingegneria Sismica in Italia, Vol. 2, 1984.
8. Kuck, J, "Anwendung der dynamischen fliegelenktheorie zur Untersuchung der verhaltensbeiwerte fr die erdbebenbemessung von stahlbauten," Institute of Steel Construction Aachen, 1994.
9. Nguyen, B. T., "Beitrag zur bestimmung der maximalen bauwerksverformung bei der erdbebenbelastung von stahlbauten mit bercksichtigung der duktilitt," Institute of Steel Construction Aachen, 1993.
10. Kook, S. K., "Beitrag zur definition der bauwerksregularitt und zur bestimmung der verhaltensbeiwerte fr die erdbebenbemessung von stahlbauten," Institute of Steel Construction Aachen, 1994.
11. CEN, *Eurocode 8 - Design Provisions for Earthquake Resistance of Structures, Part 1-1: General Rules Seismic Actions and General Requirements for Structures*, Brussels, 1994.
12. CEN, *Eurocode 8 - Design Provisions for Earthquake Resistance of Structures, Part 1-2: General Rules for buildings*, Brussels, 1994.