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SEISMIC DESIGN STRATIGIES AND DETAILS APPROPRIATE TO MODERATE SEISMICITY REGIONS

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SUMMARY

Korea is located in moderate seismicity region. It is realized that the design codes and underlying design concept of high seismicity region may not be appropriate to low and moderate seismicity regions. The aim of this paper is to search seismic design concept and seismic details that are deemed to be appropriate to low and moderate seismicity regions. To this end, the characteristics of seismic hazard in moderate seismicity regions are reviewed. The seismic responses of structures subjected to the ground shaking of moderate intensity are examined. Analysis examples and experimental results are presented to understand the behavior of structures subjected to moderate level of ground shaking. The present code on seismic design of bridges is briefly reviewed. Based on these observations, the performance objectives appropriate to the moderate seismicity regions are proposed along with the recommendations on the ground motion and site effects. A design approach called limited ductility design is recommended as design concept appropriate to the moderate seismicity regions. Specific recommendations on the seismic design and detailing of bridges are proposed.

INTRODUCTION

Korea belongs to a low to moderate seismicity region. Historic documents such as the Annals of the Choson Dynasty (1392 AD – 1910 AD) contain many entries on the earthquake events that claimed many human casualties and serious property damages in the past 2000 years [1]. Recordings on earthquake events in Korea consist of historic and instrumental data. The historic records on earthquakes encompass the period from 2 AD to 1904 AD [1]. Since 1905, earthquake events have been recorded using instruments. Those historic records are considered to be very reliable. The Annals of the Choson Dynasty (1392 AD –1910 AD) are very famous for its accuracy. It is estimated that there may have occurred 389 events greater than or equal to V on MMI scale [1]. The number of events greater than or equal to VII on MMI scale appears to be over 45 [1]. The maximum intensity seems to reach IX on MMI scale. Due to heavy reliance on the historic records the seismic hazard assessment in Korea is characterized by its large

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uncertainty. Interpretation of the historic records tends to be very dependent on the judgment of the individual researcher. In 1997, eight leading seismologists of Korea got together for the first time and produced seismic hazard maps for the development of new seismic design codes [1]. Figure 1 and 2 are seismic hazard maps of 10% probability of exceedance in 50 years, and 250 years, respectively.

Earthquake resistant design has been introduced to Korea since 1986 for high-rise buildings, since 1992 for highway bridges. Seismic design codes currently in use in Korea for the seismic design of buildings and bridges are based on UBC and AASHTO Specifications, respectively. Implicit in these codes is that buildings and bridges will experience large inelastic deformation during strong earthquakes. However, the simple fact that the seismic response of structures is a function of the intensity of ground motion has been largely unrecognized. Structures without good seismic detailing may have fair amount of inherent lateral load carrying capacity. Many of them may survive earthquakes of moderate intensity without collapse. The ductility demand for maximum credible earthquake in moderate seismicity regions will not be as high as in the high seismicity regions. In this case, if the structures have seismic detailing for the moderate degree of ductility potential, then they may survive the maximum credible earthquake without collapse.

This article concerns with a new seismic design concepts appropriate to low or moderate seismicity regions. The seismicity of Korea will be briefly introduced first. Important aspects of seismic design in low or moderate seismicity region will be discussed. Examples of structural behavior will be presented. Based on these observations a conceptual framework of the seismic design in low or moderate seismicity region will be discussed. It is concluded that limited but adequate amount of ductility needs to be provided in the seismic design in low to moderate seismicity regions.

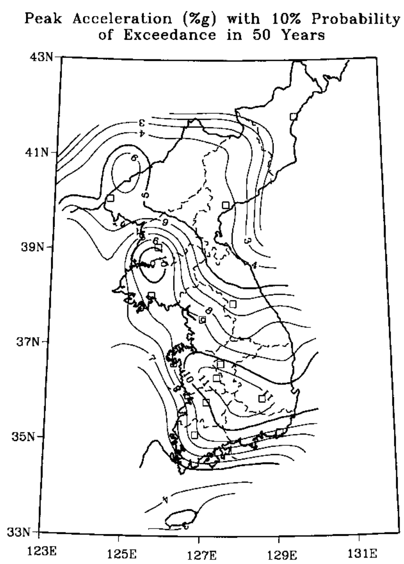


Figure 1. Seismic hazard map of 10% probability of exceedance in 50 years

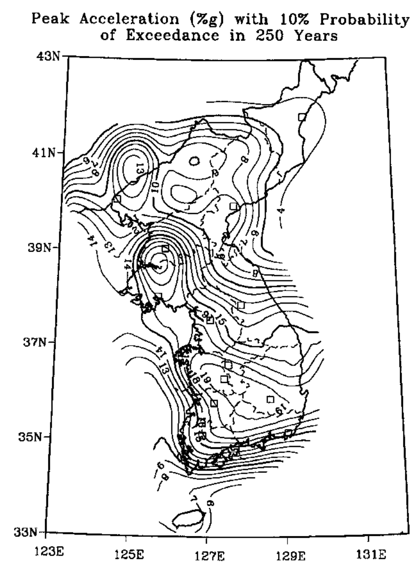


Figure 2. Seismic hazard map of 10% probability of exceedance in 250 years

CHRACTERISTICS OF SEISMIC HAZARD IN MODERATE SEISMICITY REGIONS

Korea is believed to belong to a moderate seismicity region. It shares many common characteristics in seismic hazard with stable continental regions. In stable continental regions, the seismic hazard and ground motion show characteristics significantly different from those in active tectonic regions where the majority of high seismicity countries are located.

In the stable continental regions the seismic sources show the following characteristics [2]:

- Seismic sources are not well understood and defined
- Maximum earthquake cannot be readily estimated on the basis of fault dimensions
- Recurrence rates are estimated based on the seismicity data
- Rates of attenuation may be slower than in active tectonic regions
- The presence of hard rocks may increase high frequency contents of ground motion

The above observations may have important implications to design as follows [2,3,4]:

- Probabilistic approach may be more appropriate for the estimation of maximum earthquake
- Large uncertainty and high risk factor may justify the use of design earthquake of longer return period
- The standard spectral shape in current codes may considerably overestimate long-period ground motion
- Local site effects will be more pronounced than in the region of high intensity ground motion because the degree of soil non-linearity is expected less severe and soil damping lower in low intensity regions

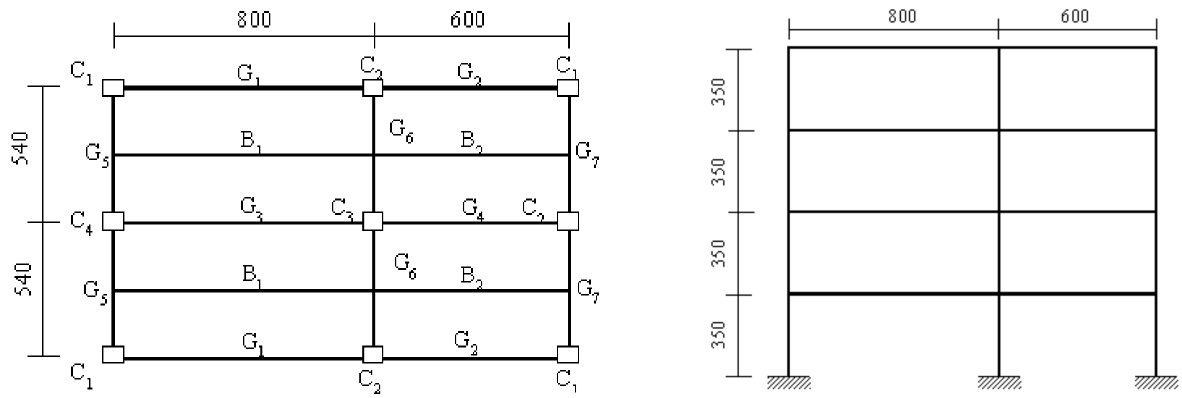
The above observations have not been considered in the design codes for the low and moderate seismicity regions. It is because the codes adopted the design principles developed for the used in the high seismicity regions.

CHARATERISTICS OF STRUCTURAL RESPONSE AND DUCTILITY DEMAND IN MODERATE REGIONS

Structural response in moderate seismicity regions

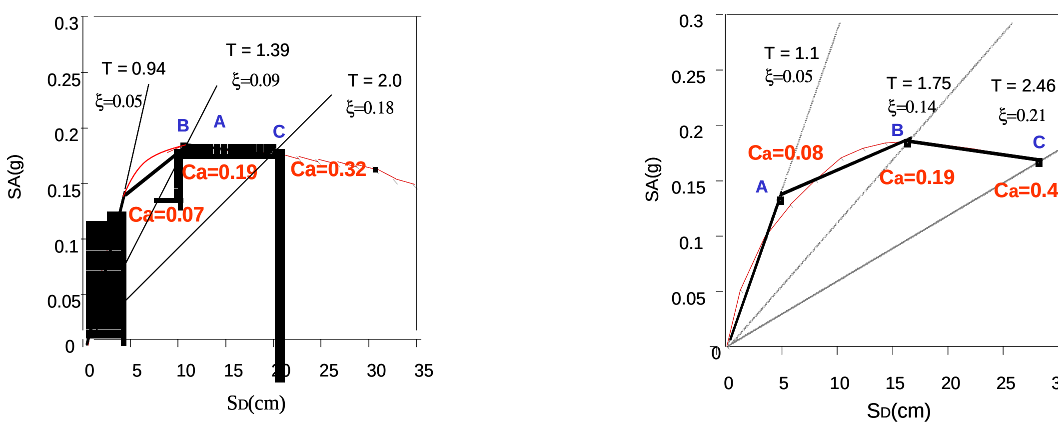
It is natural that the seismic design must be based on the dynamic response characteristics of structures under the earthquake loading. The dynamic response of structures is a function of the ground motion and mechanical properties of structures. The earthquake ground motion is local. Therefore seismic design must take into account characteristics of ground motions expected at the site. The seismic codes currently in use are developed in the region of high seismicity and high intensity region [4]. The basic concept is utilizing inelastic deformation under the design earthquake [4,5]. If high ductility and energy absorbing capacity are provided, the structures may withstand the earthquake ground motion many times more strong than the design earthquake without collapse. In the United States, this concept is implemented by introducing response modification factor. But as pointed out in Power [2], Beavers and Hunt [3], and ATC [5], structural responses under low intensity ground motion will be very different from those expected under high intensity ground motion. It is highly probable that the structures may not experience inelastic deformation at all if they are adequately designed for the conventional loads other than earthquake. In such cases, it is very awkward to design structures under the assumption that they will experience inelastic deformation. To meet detailing requirements for the ductility, the constructions may be unnecessarily complicated.

It has been observed in many tests that structures without seismic detailing may have significant amount of lateral load resisting capacity. A 4-story reinforced concrete building is designed according to the Korean Building Code[6]. The plane view and elevation are shown in Figure 3. The model is not detailed for the lateral load such as wind and earthquake. But it has adequate amount of resistance to the design wind load. It is assumed that the walls are separated from the frames and do not contribute to the building stiffness. A capacity curves are obtained using 3-D nonlinear analysis method. They are converted into ADRS spectra and compared with demand spectra in Figure 4. At several points on the curve PGA thresholds and maximum drift ratios are calculated. It appears that this structure may withstand the design earthquake without collapse but may fail in the event of MCE. The maximum ductility ratio is estimated to be 2.7. But it has to be pointed out that these capacity curves are obtained under the assumption that premature local failure will not develop at the lap spliced regions. Shaking table test results of 1/3 scale model of an unreinforced brick building model demonstrated that properly constructed masonry building could have considerable amount of laterally load resisting capacity. Serious cracks were initiated at PGA=0. 25g and it collapsed at the PGA=0.35g or higher [7].



(a) Plane view **(b) Elevation**

Figure 3. 4-story building



(a) Longitudinal direction **(b) Transverse direction**

Figure 4. Capacity spectrum

The bridges designed for the loads other than earthquake may have considerable amount of lateral load resisting capacity as demonstrated in the tests with full scale prototype and 1/3 scale model of a conventionally bridge pier [8]. An analytical study has been performed to understand the seismic performance requirement of typical bridges constructed in Korea. The superstructure is assumed being supported by RC piers of solid circular section. The diameter of the section is 3.5m and height 17.5m. Reinforcement ratio of longitudinal bars is 0.858 % and the axial load level is 10 % of $f'_c A_g$. The aspect ratio is 5. The weight of one span is 23MN. The seismic capacity of the bridge in longitudinal direction is evaluated using 3-D nonlinear finite element method for the varying span length. The schematic profile is given in Figure 5. The performance of the bridge is demonstrated in Figure 6(a), (b), (c) and (d) for 2-span, 5-span and 7-span bridges, respectively, in ADRS format. The results are summarized in Table 1. It appears that the ductility requirement is quite limited even for the 3-span continuous bridge.

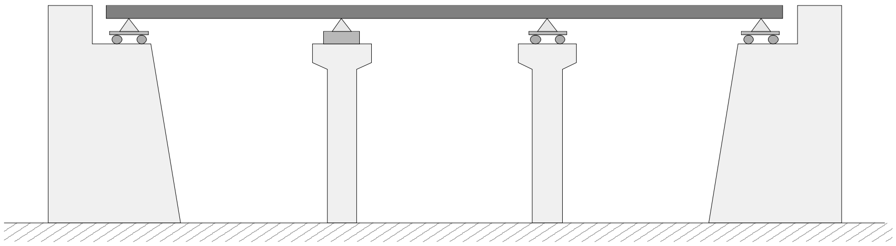
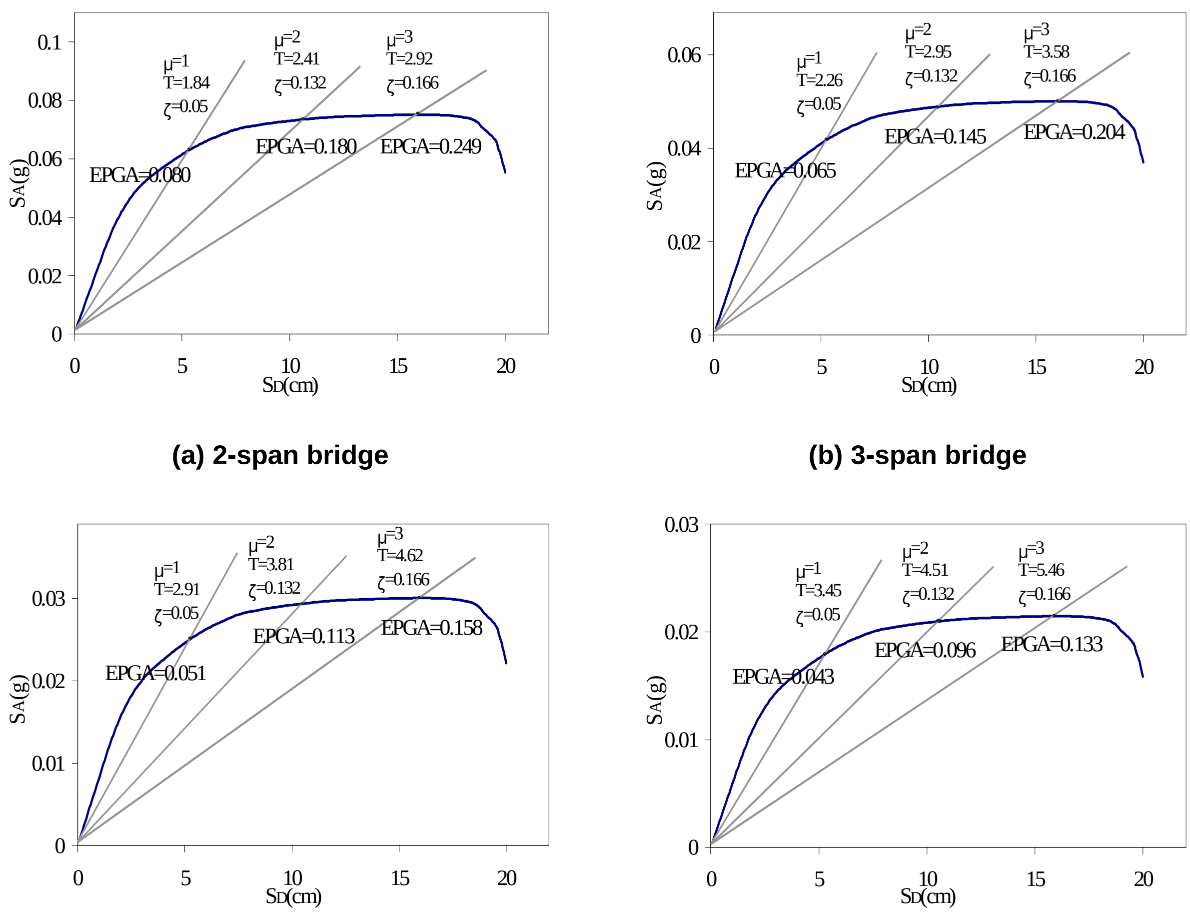


Figure 5. Schematic profile of bridge



(a) 2-span bridge

(b) 3-span bridge

(c) 5-span bridge

(d) 7-span bridge

Figure 6. ADRS of model bridges

Table 1. Seismic performance of model bridges

Ductility	2-span bridge	3-span bridge	5-span bridge	7-span bridge
$\mu=1$	0.080g	0.065g	0.051g	0.043g

μ =2	0.180g	0.145g	0.113g	0.096g
μ =3	0.249g	0.204g	0.158g	0.133g

These results indicate very clearly that conventionally designed frame structures possess considerable amount of inherent lateral strength though they may not show ductile failure mode. By improving joint details, the ductility may increase to prevent premature brittle shear failure.

Ductility demand of bridges in moderate regions

Nonlinear pushover analyses were performed to obtain data on the ductility demand of multi-span continuous bridges constructed in moderate seismicity regions. Analysis cases were chosen based on the parameters provided in Table 2. Widely used values were chosen for geometric proportions, mechanical and material properties.

Table 2. Parameters for analyses

Description	T-type pier	Variable/Constant
Diameter of pier(m)	3.0	Constant
Height of pier(m)	12, 15, 18, 21, 24	Variable
Reinforcement ratio of longitudinal bars(%)	1	Constant
Axial force ratio(%)	8	Constant
Number of continuous span	1, 2, 3, 4, 5, 6, 7	Variable
Soil condition	S_B, S_C, S_D	Variable
Con'c strength(MPa)	24	Constant
Yield strength(MPa)	300	Constant

The load-displacement relations were obtained from the nonlinear analysis of pier models. Based on them, the ductility demand of various types of the bridge systems were calculated for the different site conditions. To this end, first the reduction factor (R-factor) was defined as the ratio elastic seismic force (P_E) to the strength (P_n). Secondly, the ductility demand was estimated from the ADRS (Acceleration Displacement Response Spectrum). Alternatively, for elasto-perfectly-plastic systems the ductility demand can be evaluated from the R-factor. For the convenience of calculation, the Takeda model[9] was assumed for the inelastic behavior of piers subjected to reverse cyclic loading. The EPGA of design earthquake was assumed as 0.15g at rock site. The calculated R-factors and ductility demands at S_D soil profile are plotted in Figure 7 and Figure 8, respectively. It can be recognized in Figure 7 that R-factor increases as the pier becomes taller and the number of continuous span becomes larger. The data in Figures show that the demand in R-factor is quite limited except those multi-span continuous bridges with short piers. Similar tendency can be identified in ductility demand given in Figure 8. The maximum demand is 4.5 for 7-span continuous bridges with short piers. In general the ductility demand is less than 3.

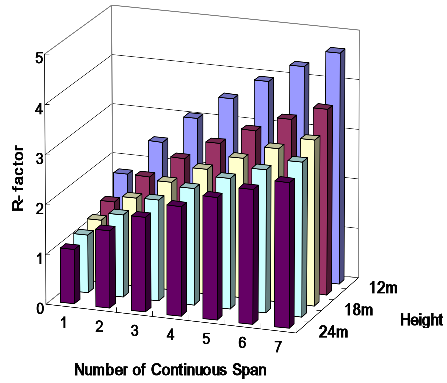


Figure 7. R-factor

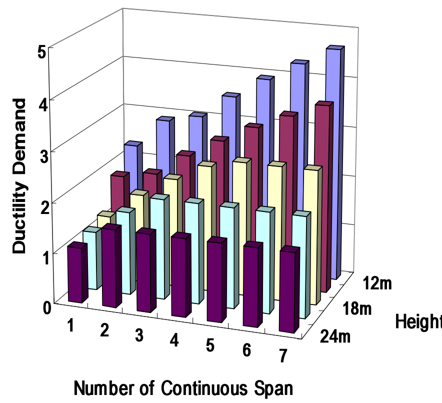


Figure 8. Ductility demand

SEISMIC DETAILS

Provisions for seismic details in current code

As mentioned in Introduction, the seismic provisions in the current Korea Highway Bridge Design Standards (KHBDS) are in line with the AASHTO Specifications. The seismic design concept of AASHTO specifications is based on the ductility design. AASHTO Specifications defines 4 seismic categories. But Highway Bridge Design Standards defines only two Seismic Zones. The comparisons between two codes are given in Table 3. Similar to AASHTO requirements applicable for seismic zone II, lap splice in plastic hinge zone is allowed implicitly in Korea Highway Bridge Design Standards. A large amount of transverse reinforcement bars with end hooks is, however, required to confine core concrete effectively so that sufficient ductility can be achieved. This large amount and complex assembling of transverse reinforcement cause high cost in construction of bridges. But the fact that the confinement requirement is due to the allowance of lap splice in the plastic hinge region has not been explicitly appreciated. A much simpler detailing such as using continuous longitudinal bars with less amount of transverse reinforcement could be introduced as an alternative detail still satisfying the seismic performance objective.

Table 3. Comparisons between AASHTO specifications and KHBDS

<table>
<tr><td>Description</td><td colspan="4">AASHTO</td><td colspan="2">KHBDS</td></tr>
<tr><td>Seismic Zone</td><td>I</td><td>II</td><td>III</td><td>IV</td><td>II</td><td>I</td></tr>
<tr><td>Seismic Coefficient(A)</td><td>A≤0.09</td><td>0.09<A≤0.19</td><td>0.19<A≤0.29</td><td>0.29<A</td><td>A=0.07</td><td>A=0.11</td></tr>
<tr><td>Lap Splice</td><td rowspan="2">No Consideration of seismic forces : Conventional design</td><td>Not specified</td><td colspan="2">Used only within the center half of column height (>400mm or 60d_b)</td><td colspan="2">Not specified</td></tr>
<tr><td>Transverse reinforcement</td><td>Amount & Spacing are specified</td><td colspan="2">Same as the Category II</td><td colspan="2">Amount & spacing are specified</td></tr>
</table>

Seismic details appropriate to moderate seismic region

Seismic behaviors of columns with different seismic details of longitudinal bars

Recently, several experimental studies have been done in Korea. The first one is the study done by Kim [10, 11]. They constructed concrete columns of solid circular section. The scale was 3.5 but still the diameter of the model was 1.0m and the height 5.0m. Two kinds of connections details of longitudinal bars were considered as shown in Figure 9: one has lap spliced longitudinal bars in plastic hinge regions and the other continuous. The load-displacement hysteresis loops are provided in Figure 10(a), (b) for the lap spliced model and continuous bar model respectively. The displacement ductility is 1.5 and 4.5 in the lap-spliced model and continuous one respectively. These experimental results indicate that the avoidance of lap splice in the plastic hinge zone can significantly enhance the seismic performance of a RC pier. Other researchers confirmed this fact in the tests conducted independently recently.

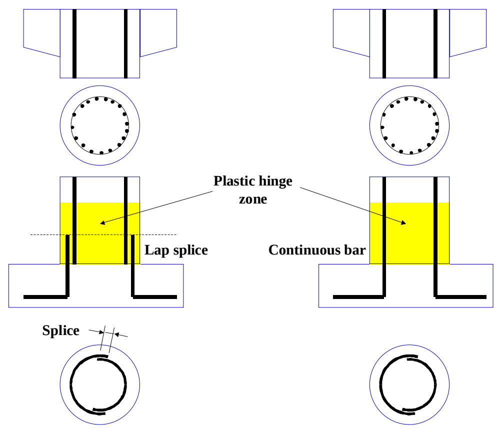
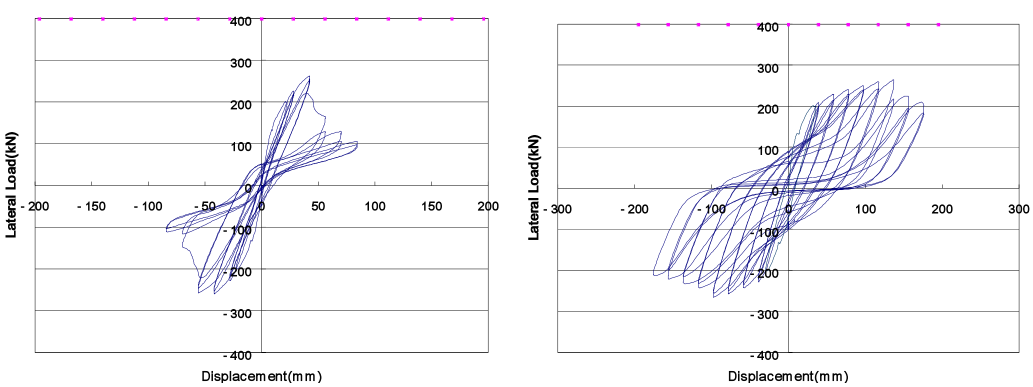


Figure 9. Details of longitudinal bars



(a) Lap splice model **(b) Continuous model**

Figure 10. Load-displacement curves

Transverse reinforcements for limit ductility

As observed in Figures 7 and 8, the ductility demand of multi-span continuous bridges in moderate seismicity regions appears to be limited within the range 3-4 in most cases. The governing reinforcement details that determine ductile behavior of piers are lap splice of longitudinal bars and lateral confinement steel in plastic hinge region. The lateral confinement steel depends on the fact whether the lap splice is allowed or not.

The displacement ductility may be defined by Eq. (1), the plastic displacement, Δ_p, can be estimated by Eq. (2) [12].

$$\mu=\Delta_u/\Delta_y=1+\Delta_p/\Delta_y \tag{1}$$

$$\Delta_p=\theta_p(H-L_p/2) \tag{2}$$

where, H is the height of pier, L_p denotes the equivalent plastic hinge length and θ_p means the plastic rotation.

The plastic curvature required to provide plastic displacement can be estimated from the stress-strain relation such as given in Figure 11 if the longitudinal bars are not lap spliced in the plastic hinge regions [13]. From these data and relations the requirement of lateral confinement steel can be estimated and is as shown in Figure 12.

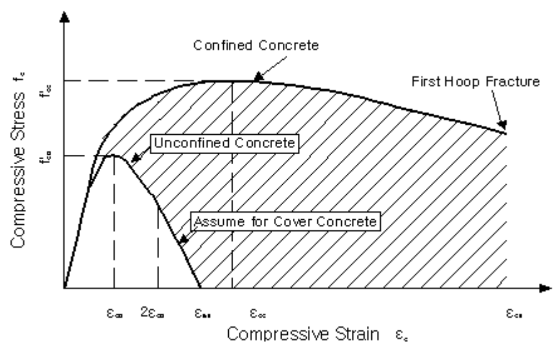


Figure 11. Stress-strain of confined concrete

If lap splice of longitudinal steel is allowed within the plastic hinge region, the bond failure due to tension shows a very brittle failure mode if a sufficient amount of confinement steel is not supplied. In this case a minimum amount of confinement steel should be provided to prevent the bond failure of the lap spliced steel bars. Priestley [12] proposed Eq. (3) for the estimation of this minimum amount of confinement steel:

$$f_l(=0.5\rho\times 0.0015E_s)=1.21A_bf_y/pl_s \tag{3}$$

where, A_b and f_y are the area and yield strength of the longitudinal steel, respectively, p is the the perimeter of the cracked surface and l_s denotes the lap splice length of the deformed bars in tension. Eq. (3) implies that the tension strain of the confinement steel should be less than the yield strain to prevent bond failure. The lateral confinement steel estimated from Eq. (3) is about 0.5 % in volumetric ratio.

Even though the bond failure is prevented by the minimum confinement steel, the compression region should be confined sufficiently in order to secure the required displacement ductility. The confinement of compressed concrete by the lateral steel will be effective until the fracture of the confinement steel. However if cyclic loading is applied to the pier, the same region will experience tension and compression alternatively. Even though the concrete is confined effectively in compression, if the confinement steel deformed beyond yield displacement, the confinement steel may not be able to confine effectively in tension phase. Hence, the strain of the confinement steel needs to be less than the yield strain. As can be seen in Figure 11, the strain of the confined concrete should be less than allowable strain ε_{cc}, in order to confine the lap spliced region effectively and secure the required displacement ductility. The strain ε_{cc} of Figure 11 is given by Eq. (4) and f'_{cc} can be expressed by Eq. (5) as a function of confinement stress that is expressed in terms of volumetric ratio of the confinement steel.

$$\varepsilon_{cc}= 0.002[1+5(f'_{cc}/f'_{c}-1)] \quad (4)$$

$$f'_{cc} =fn(f_l),\ \ f_l= f_y\rho_{sh}/2 \quad (5)$$

The confinement steel can be estimated using Eq. (4) and Eq. (5). The results calculated for the model in Table 2 are plotted in Figure 12.

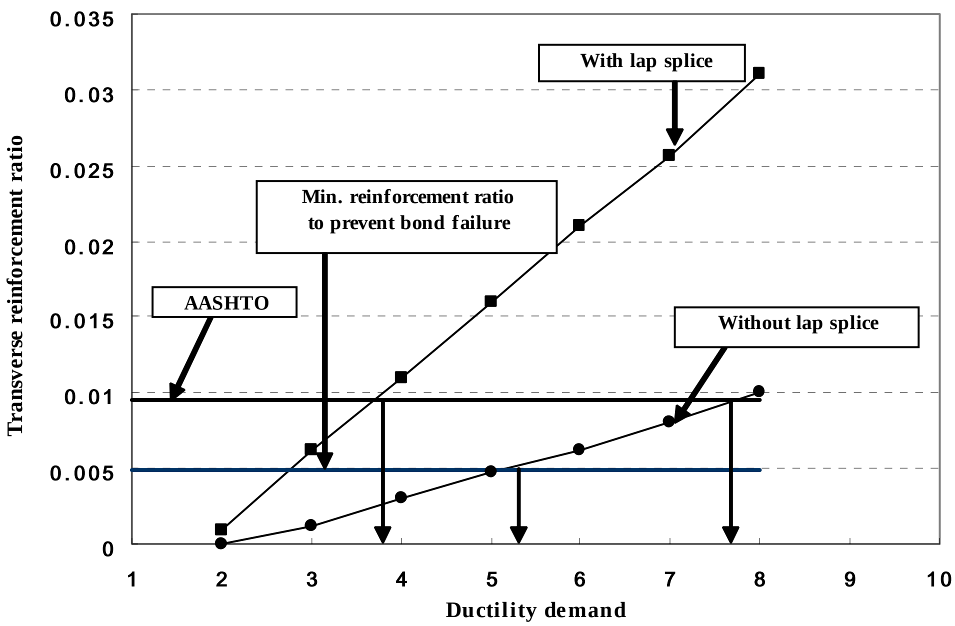


Figure 12. Relation between ratio of transverse steel and ductility demand

First of all, the Figure 12 shows very clearly that the requirement of lateral confinement steel is approximately 3 times lateral confinement steel is required if lap splice is allowed model than the not allowed case to provide the same level of ductility. It means that the lap splice of longitudinal bars has adverse effects more detrimental than expected. Alternatively if lap splice is not allowed, then the displacement ductility 5 and above can be achieve with only 0.5% lateral confinement steel. If 1.0 % of confinement steel is provided, then the displacement ductility reached 7 easily. This fact has a very important meaning to the seismic design in moderate seismicity regions. According to AASHTO lap splice is allowed implicitly if the design intensity is of moderate level. Instead, rather large amount of confinement steel is specified. But the results in Figure 12 suggest an alternative design approach for the moderate seismicity regions. If the longitudinal bars are not lap spliced in the plastic hinge regions, then with minimum amount of confinement steel the ductility required in moderate seismicity regions can be achieved. It may be a seismic detail that is more appropriate and economical in the regions like Korea.

SEISMIC DESIGN STRATIGIES AND DETAILS

Design Earthquakes

The seismic risk factor is defined as the ratio of the peak ground acceleration of a given return period to that of a reference return period (475-year return period) are calculated. In active tectonic regions, this factor is close to unity. But in stable continental regions this factor turned out to be very large and in New Madrid region it can reach as high as 5. In Korea the risk factor for 2375-year return period was estimated to be 2 [1]. The design earthquake is usually defined as an earthquake of 475-year return period. The assumption is that if the structure is designed for this earthquake then, the structure will survive the maximum credible earthquake without collapse. This reasoning makes sense in active tectonic regions where the risk factor for maximum credible earthquake is close to unit value. But in stable continental regions, the risk factor will be much higher than unit. It means that the structures designed for the earthquake of 475-year return period may have high possibility of collapse during maximum credible earthquake. Therefore the return period of design earthquake in stable continental regions need to be longer than current 475 years.

Design ground motion

The ground motion is known to be rich in short period components. Hence the standard spectrum developed in active tectonic regions will not be appropriate to use in moderate seismicity regions that belongs to a stable continental region. Moreover the site effects may be more pronounced in moderate seismicity regions. Therefor soil amplification factors will be higher in moderate seismicity regions than high seismicity regions

Performance objectives

In moderate seismicity regions, the gravity load may govern the design. It is very likely that the members proportioned for the gravity load may have significant amount of lateral load carrying capacity. If premature local failure can be prevented, the demand for inelastic deformation will not be very large even under the maximum credible earthquake. In performance-based design, multiple performance levels are being under consideration. As pointed out by Bertero [14], the present design approach requires the structures undergo large amount of inelastic deformation before developing their full capacity. Even though the structure may not collapse, but the damage level will be too severe for the structure to have of any economic value after strong earthquake. Hence the present design concept based on large amount ductility may not be adequate even in the high seismicity regions. From the above observations, seismic performance levels may have to be defined based on limited damage instead of large ductility.

Limited ductility design concept

In moderate seismicity regions, the ductility demand will not be very large as seen above. In order to limit the damage during the earthquake, the deformation should not be excessive. Therefore the design concept to design structures to have limited ductility will make a lot of sense in moderate seismicity regions. Since minimum amount of confinement steel will be sufficient to prevent premature local failure and insure the ductility required in moderate level of ground shaking, limited ductility concept would be very economical approach in moderate seismicity regions. The current design approach does not take into account explicitly the effects of biaxial horizontal ground motion. According to the test results by Kim et al. [15], the ductility capacity can be reduced by one unit under biaxial loading condition. It may be important in the case of limited ductility design.

Seismic detailing
In the case of RC structures, reinforcement details are very important to provide required ductile behavior. The seismic details currently in use have been developed in high seismicity regions. The seismic design concept based on limited or restricted ductility has been studied already [16, 17, 18]. There are research results on the seismic detailing for the any required ductility [17, 18]. But still most research efforts have been directed to the seismic design for high intensity design earthquake. Recently Kim et al. suggested alternative seismic detailing that could provide limited ductility with minimum efforts [15]. In the case of bridge columns, by using continuous bars in the plastic hinge regions, the limited ductility behavior can be achieved with small amount of lateral confinement steel. However, in conventional approach, heavy amount of lateral confinement steel has been required while lap splices of longitudinal bars are allowed. In certain circumstances, the alternative approach will be easier to implement in construction site. As mentioned above the effects of biaxial loading on the capacity of structures need be studied.

Seismic design of bridges in moderate seismicity regions
The components that are vulnerable to earthquakes are known to be the bearings, piers, foundations and abutments. The deck also can experience falling down during earthquake. The bridges of multi-span continuous type seem to be particularly vulnerable to the longitudinal direction. In general, the bearings appear to be most vulnerable to ground shaking and prone to damage. Hence the first priority will be preventing damage of bearings in order to make load transfer between deck and pier. It is also important to prevent falling down of deck structure. In the case of multi-span continuous bridges the piers can experience damage if the longitudinal movement is restrained at the supports of single pier. But if several piers can share the seismic load, the damage to the piers can be minimized. This objective can be achieved by alternative earthquake resistant design method.
One alternative is using seismic isolation bearing instead of conventional bearings. Friction bearings, friction pendulum bearings and various laminated rubber bearings can be used to this end. The advantage of using these bearings is that the seismic load transmitted to the piers will be minimal. Consequently, the damage to piers can be prevented with minimum amount of lateral load resisting capacity. Another alternative approach is to employ seismic load distribution system such as shock transmitters and mechanical seismic load transmitting device.

CONCLUSIONS

Korea is located in a moderate seismicity region. In order to search a new seismic design method appropriate to the moderate seismicity regions, the characteristics of seismic hazard in moderate seismicity regions are reviewed and the seismic responses of structures subjected to the ground shaking of moderate intensity are examined. It is realized that the design codes and underlying design concept of high seismicity region may not be appropriate to low and moderate seismicity regions. The present code on seismic design of bridges is briefly reviewed.

Based on these observations, the performance objectives appropriate to the moderate seismicity regions are proposed along with the recommendations on the ground motion and site effects. A design approach called limited ductility design is proposed as the design concept appropriate to the moderate seismicity regions. And specific recommendations are made for the design bridges in moderate seismicity regions.

The proposal in the article is quite general and describes only the outline of new concept. But the concept has been developed in several years. And they are based on the observations, experiments and analyses. The concept needs further refinement. At the same time, the well-coordinated research efforts are crucial to be adopted in a new seismic code.

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