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Seismic Response of Multiple Span Prestressed Concrete Girder Bridges in the New Madrid Seismic Zone

New Madrid 지진대의 다경간 PSC 교량의 지진거동

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국문 요약 >> 본 연구는 미중부지역의 New Madrid 지역에서 일반적으로 존재하는 다경간 PSC 교량의 지진거동을 평가하였다. 지진해석은 비선형 교량모델과 인공지진파를 사용하여 수행하였으며, 인공지진파는 50년 동안 발생확률이 10%와 2%의 두 가지 수준을 사용하였다. 10%의 지진파에 대해서는 해석교량은 양호한 응답을 보였으나, 2%의 지진파에 대해서는 비선형 거동을 보이면 응답이 좋지 않았다. 바닥판 사이의 충돌로 인하여 기둥의 요구량이 증가하였으며 교량받침의 파손이 발생하였다. 또한 PSC 거더를 연속화하면 매우 만족한 응답의 개선효과가 있었으며, 이러한 연속화는 유지보수의 절감 및 사하중에 의한 모멘트의 감소를 위해서 일반적으로 행해지는 것이다.

주요어 교량, 지진해석, New Madrid, PSC 거더, 다웰, 중진대

ABSTRACT >> This paper evaluates the seismic response of multi-span prestressed concrete girder bridges typically found in the New Madrid Seismic Zone region of the central United States. Using detailed nonlinear analytical models and synthetic ground motion records for Memphis, TN, nonlinear response history analyses are performed for two levels of ground motion: 10% probability of exceedance (PE) in 50 years, and 2% probability of exceedance (PE) in 50 years. The results show that the bridge performance is very good for the 10% PE in 50 years ground motion level. However, the performance for the 2% PE in 50 years ground motion is not so good because it results in highly inelastic behavior of the bridge. Impact between decks results in large ductility demands on the columns, and failure of the bearings that support the girders. It is found that making the superstructure continuous, which is commonly performed for reducing dead load moments and maintenance requirements, results in significant improvement in the seismic response of prestressed concrete girder bridges.

Key words bridges, seismic, mid-america, prestressed concrete girder, dowel, moderate

1. Introduction

The 1999 Chi-Chi (Taiwan) and the 1999 Izmit (Turkey) earthquakes illustrated that the multi-span simply supported prestressed concrete (PSC) girder bridge is highly vulnerable to damage from moderate-to-strong ground motion.[1,2] The vulnerabilities include unseating of the superstructure, damage due to impact between decks, and failure of columns due to inadequate lateral reinforcement, end anchorage, and splice length. Additional vulnerabilities include non-uniform distribution of column stiffnesses along the bridge and inadequate flexural and shear strengths in the columns. This bridge type is also commonly found in regions of moderate seismicity in the central United States, in particular in the area known as the New Madrid Seismic Zone (NMSZ), which covers the region of northeastern Arkansas, southeastern Missouri, western Tennessee, and southern Illinois. Detailed analysis of the bridge inventory in this region has found that

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approximately 20% of the bridges are made of prestressed concrete girder superstructure.[5] The majority of these bridges are slab-on-girder type bridges with a reinforced concrete substructure. The vulnerability of these bridges in the central United States is amplified by the fact that these bridges were likely designed without consideration of adequate seismic forces.

Previous studies have shown that the superstructure -to-substructure connection is a primary factor controlling the seismic response of steel-girder bridges.[7,8] However, unlike steel-girder bridges, there are few studies on the seismic behavior of prestressed concrete girder bridges. One of primary differences between the prestressed concrete girder bridge and the steel girder bridge is the support condition under the bridge girders. Steel girder bridges in the New Madrid Seismic Zone typically use steel fixed and expansion bearings, whereas the prestressed concrete girder bridges typically use dowels and rubber pads for the substructure to superstructure connection. Hence, it is expected that the seismic behavior of PSC girder bridges would be different from that of steel girder bridges.

In this paper, the seismic performance of multi-span simply supported and multi-span continuous prestressed concrete girder bridges typically found in the NMSZ region is evaluated. Detailed nonlinear analytical models of the prestressed concrete girder bridge are developed, including the nonlinear behavior at the support bearings. The bridge models are subjected to synthetic ground motion for three cities in the New Madrid Seismic Zone. Suites of ground motion representing both the 10% probability of exceedance (PE) in 50 years and 2% probability of exceedance (PE) in 50 years hazard levels are evaluated.

2. Research Significance

Little is known about the seismic vulnerability of prestressed concrete girder bridges, which is a typical type of construction found in the New Madrid Seismic Zone region of the central United States. This study provides guidelines on assessing the seismic performance of these bridge types using nonlinear analytical models and nonlinear time history analysis. The results of this study are important for understanding the parameters that affect the seismic performance of these bridges, and can serve as an important tool for assisting local state bridge engineers in prioritizing the retrofit of bridges that are at risk of significant damage during an earthquake. In addition, the effect of making the bridge continuous by casting a parapet between the spans is evaluated.

3. Previous Research

Trochalakis et al. (1996) developed a 2D bridge model for a four-span, simply supported, prestressed concrete girder bridge to assess the effect of restrainer cables in reducing the maximum relative displacement at the piers. In this study, the girders in the bridge were supported only by elastomeric pads that work primarily through friction. The results of the study found that pad friction significantly affects the maximum relative displacement of the bridge. Hwang and Huo (1998) and Hwang et al. (2000) used a four-span simply supported bridge with prestressed concrete girders to assess the seismic fragility of the bridge. Fragility curves, which represent the conditional probability that the structural demand caused by various levels of ground shaking exceeds the structural capacity, is becoming a very popular tool in assessing the seismic vulnerability of structures. This study found that the peak ground acceleration for a 50% probability of exceeding minor damage was 0.12 g. However, the study did not account for impact between the decks, which can significantly affect the response of these types of bridges.

Prestressed concrete girders are often supported at the ends by elastomeric bearing pads without dowels. In that case, the slippage of the bearings under the girder is the major problem. McDonald et al. (2000) found that the slippage occurs between the pad and girder for tapered bearings when the girder undergoes thermal movement. Therefore, they recommended roughening the concrete surface to increase the coefficient of friction to reduce slippage of the bearings.

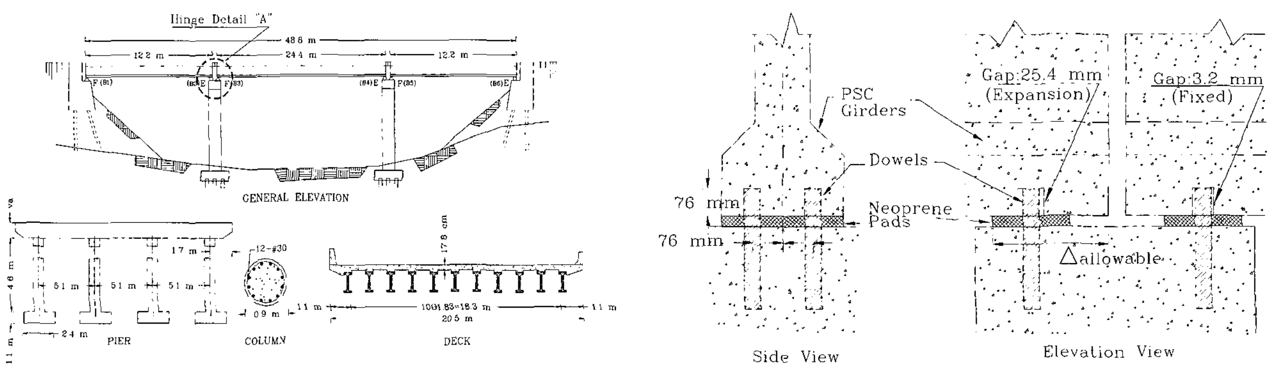
In Korea, many researchers concentrated on the seismic study of reinforced concrete bridge piers.[28-30] Juhn and Lee (1998) has investigated the structural characteristics of RC piers in Korea and used them to estimate the seismic performance of the piers. They indicated in the study that the bridge piers in Korea do not have enough strips and possess the lap-slice at the bottom of the piers which are usually considered serious problems to weaken the seismic performance of piers. Lee and his colleagues (2000) have conducted an experimental test of a RC small scaled pier that is not designed following a seismic code. They also indicated that the lap-slice of longitudinal reinforcements is critical on the bridge seismic performance. A similar test was performed by Kim and his coworkers (2001). They assessed the seismic performance of piers without seismic detailing. They also showed that even piers without seismic detailing can reveal limited ductility comparing the piers with lad-splice.

4. Characteristics Of Multi-Span Prestressed Concrete Girder Bridges

As previously mentioned, the prestressed concrete girder bridge is a common bridge type in the NMSZ region of the central United States, where potential for moderate-to-strong ground shaking exists. The prestressed concrete girder bridge typically consists of 2-5 spans, with each span ranging from 20 m-30 m in length and widths ranging from 12 m-30 m, as shown in Figure 1. Each girder is typically supported by neoprene bearing pads and dowels. At the end of a girder, there are slots in the longitudinal direction to allow for movement due to thermal expansion, typically 27-35 mm (2-10 mm gap) for a fixed bearing and 70-80 mm (45-55 mm gap) for an expansion bearing. The 25.4 mm diameter dowels are typically used to prevent excessive movement in the longitudinal and transverse directions due to wind and braking loads. However, they are not designed with consideration of seismic loads. When the dowels fracture, the movement of the superstructure is resisted by the friction of the bearing pads and/or via contact with other girders or the abutment.

The substructure of the prestressed concrete girder bridges typically consists of multi-column bents, which are about 1m in diameter, with 1% vertical reinforcement and #10 or #13 transverse bars spaced at 305 mm. The pile caps do not have top steel reinforcement or shear reinforcement. Most of the bridges are founded on driven pile foundations with no positive connection between the piles and pile cap. The column vertical reinforcing is lap spliced at the top of the pile cap at a potential plastic hinge zone. The column vertical reinforcement extends into the bent cap without hooks or bends.

Multi-span simply supported prestressed concrete girder bridges are often made continuous by casting a parapet between decks. This is done to reduce maintenance and the dead load moment.[22,27] In many cases, new prestressed concrete girder bridges are now constructed without hinges in the deck to reduce the maintenance requirements. While the seismic performance is not considered when the decks are made continuous, this is expected to have



Hinge Detail "A"

〈Figure 1〉 Typical Multispan Simply Supported Prestressed Concrete Girder Bridge Used in the Central and Southeastern United States.

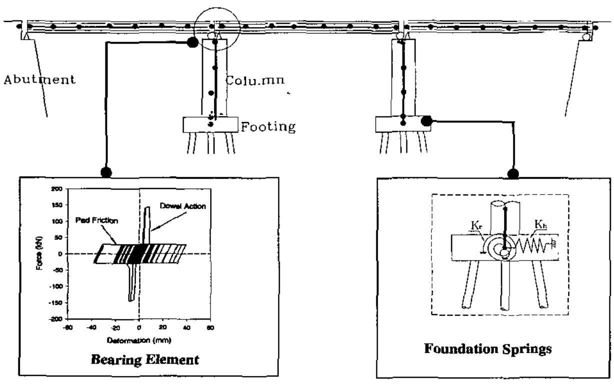
a significant effect on the seismic response of typical bridges. As part of this study, the effect of making the deck and/or girders continuous on the seismic response of the bridge will be investigated.

5. Analytical Modeling of Multi-Span Prestressed Concrete Girder Bridge

Since the multi-span simply supported (MSSS) and multi-span continuous (MSC) prestressed concrete girder bridge consist of elements that may exhibit highly nonlinear behavior (elastomeric pads/dowels, columns, abutments, impact between decks), a two-dimensional nonlinear analytical model of the bridges is developed using DRAIN-2DX[20], as shown in Figure 2. Since the deck is expected to remain linear under longitudinal earthquake motion, it is modeled using linear elements that represent the stiffness and mass properties of the composite prestressed concrete girder - reinforced concrete deck. Preliminary studies show that the response of the bridge is controlled primarily by the piers and the bearings used to connect the superstructure to the substructure.

5.1 Modeling of Concrete Columns

Inelastic two-dimensional beam-column elements are used to model the column for each of the piers in the bridge. Typically, 5-7 elements are used to model the column from the footing to the bent cap, as shown in Figure 2. The beam-column element used in this study is



〈Figure 2〉 Nonlinear Analytical Model of Prestressed Concrete Girder Bridge. Shown in boxes are details on (left) abutments, (center) bearings, and (right) Column

Type 15 in DRAIN-2DX, which uses a fiber model of the cross section. Each fiber has a stress-strain relationship, which can be specified to represent unconfined concrete, confined concrete, and longitudinal steel reinforcement. The distribution of inelastic deformation is sampled by specifying cross section slices along the length of the element. The loss of stiffness and strength caused by concrete cracking, and yielding of steel reinforcement can be easily represented using a fiber model.

In DRAIN-2DX, a piecewise linear approximation is used to represent the Park model for confined and unconfined concrete. The model by Park et al. (1982) defines a stress-strain relationship specified by:

$$f_c(\varepsilon_c)=\begin{cases} Kf'_c\left[\dfrac{2\varepsilon_c}{\varepsilon_0 K}-\left(\dfrac{\varepsilon_c}{\varepsilon_0 K}\right)^2\right] & \text{for} \quad \varepsilon_c \le \varepsilon_0 K \\ Kf'_c[1-Z_m(\varepsilon_c-\varepsilon_0 K)] \ge 0.2Kf'_c & \text{for} \quad \varepsilon_c > \varepsilon_0 K \end{cases} \tag{1}$$

where

f_c = concrete stress in MPa

ε_c = concrete strain

ε_0 = strain at which peak concrete stress is attained for unconfined concrete

$$K=1+\frac{\rho_s f_{yh}}{f'_c}$$

ρ_s = reinforcement ratio of transverse reinforcement

f_{yh} = yield stress of transverse reinforcement (MPa)

f'_c = specified compressive strength in concrete

$$Z_m=\frac{0.50}{\dfrac{3+0.29\varepsilon_0 f'_c}{145f'_c-1000}+0.75\rho_s\sqrt{\dfrac{h''}{s_h}}-\varepsilon_0 K}$$

h'' = width of concrete core measured to outside of hoops (mm)

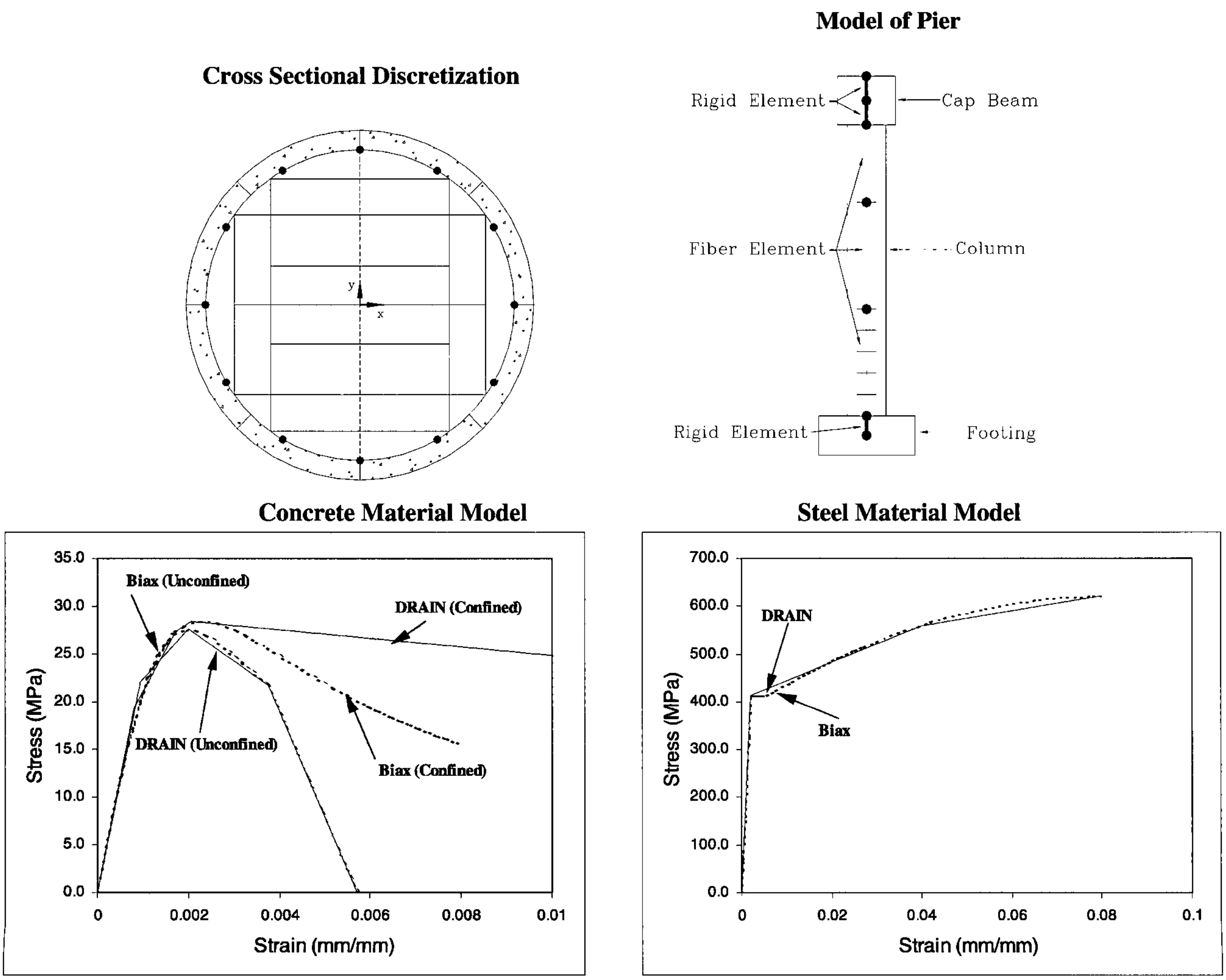
s_h = center-to-center spacing of transverse reinforcement (mm)

Figure 3 shows a comparison of the stress-strain for confined and unconfined concrete using the cross-section analysis program, BIAX[25], with the multi-linear approximation used in DRAIN-2DX. The rapidly descending stress-strain branch that is used in the BIAX concrete model is difficult to implement in DRAIN-2DX and often results in problems converging to a solution. Therefore, a more gradual descending stress-strain branch is used in the DRAIN-2DX type 15 element.

Pinching and bond slip are not included in the present model, and shear and torsion are represented elastically. The steel material model used for the analysis is based on the BIAX steel model which assumes an initial elastic behavior up to yield, followed by a yield plateau, and a strain hardening region. The cross sectional discretization of the column for the DRAIN-2DX model, shown in Figure 3, is represented by 24 concrete and 12 steel fibers. The concrete fibers are placed at the geometric centroids of the concrete areas of the concrete areas shown in the figure.

5.2 Bearings and Dowels

The multi-span prestressed concrete girder bridge has dowels and rubber pads to support spans and to resist transverse and longitudinal movements. Previous research has been performed to evaluate the nonlinear force-displacement relationship of the dowel bars by testing dowels retrieved from an existing bridge.[16] Using these results and a finite element model of the dowel bar embedded in concrete, an analytical model of the bearing /dowel bar is created. The dowels have an ultimate force of 57.8 kN at a deformation of 5.3 mm. Once this deformation has been exceeded, the dowel bars fracture, and are no longer represented in the model. Since the height of rubber pads (25 mm) is small, the deformation of the pads can be ignored and the sliding of the pads resists the movement of spans. Hence, the analytical



〈Figure 3〉 Analytical Fiber Model Used for Concrete Columns (a) Stress-strain for unconfined & confined concrete, (b) stress-strain for steel reinforcement, (c) Fiber cross-section for column (d) Location of integration slices and nodes along column.

model of the pads is elasto-perfectly plastic to represent Coulomb friction between the pads and concrete surfaces. An equation from Scharge's study (1981) is used for the frictional coefficient of the pads, which is the function of normal stress acting on the pads. The combined behavior of the two dowels and rubber pads is shown in Figure 2.

5.3 Abutments, Pile Foundations, and Impact

The abutment properties used in this model are based on recommendations by Caltrans (1990), and results from previous experimental studies.[10,17] The model represents the multi-linear inelastic behavior of the abutments in both active action (tension) and passive action (compression), as shown in Figure 4. The stiffnesses in active and passive action, shown in the figure, are based on the properties of the abutments in this bridge.

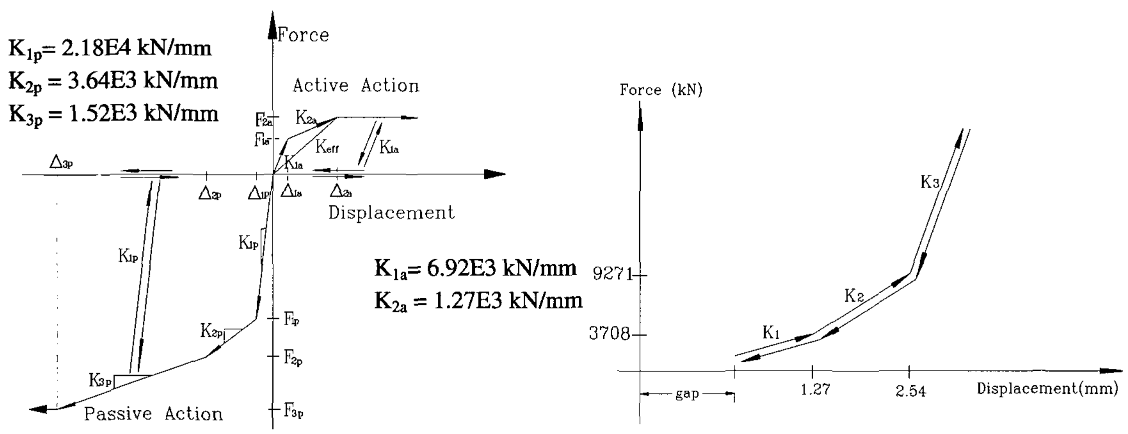
The contact element approach is utilized to model impact between decks and at the abutments.[15] Previous studies have shown that when a linear element with very high stiffness is used for impact, it can produce unrealistically high impact forces and accelerations.[6] Therefore, a trilinear element with elastic loading/unloading with a gap, shown in Figure 4, is used to represent impact between decks and deck and abutments. The stiffness of the impact elements, K3=8.73e3 kN/mm, K2=0.50 K3, and K1=0.33 K3, are selected such that the penetration due to pounding is limited to less than 2.5 mm, and approximately represents the axial stiffness of the composite deck.

The pile foundation is modeled using a combination of linear springs in the horizontal and rotational direction at the center of the footing. The pile foundation stiffnesses are based on the type and number of piles, as well as the soil properties. The translational and rotational spring stiffnesses for the foundation were taken as 1.27E3 kN/mm, and 4.56E9 kN-mm/rad, respectively. Detailed information on the properties of the soil springs used in this paper can be found in the study by Choi (2002).

6. Representative Ground Motion in the Central and Southeastern Unites States

The seismic hazard in the central United States is primarily due to the New Madrid Seismic Zone. The New Madrid Seismic Zone covers the region of northeast Arkansas, southeast Missouri, western Tennessee, western Kentucky and southern Illinois. In 1811 and 1812, a series of 3 large earthquakes struck the New Madrid region.[24] This sequence of earthquakes is believed to have had moment magnitudes in the range of 7.0-8.0, and are generally considered the largest earthquakes to have occurred in the continental United States. However, there still have not been any recordings of strong ground motion in the CSUS. Therefore, to assess the performance of structures to strong ground motion, simulated ground motion records must be developed.

A recent study by Wen and Wu (2001) resulted in the development of simulated earthquake records for three



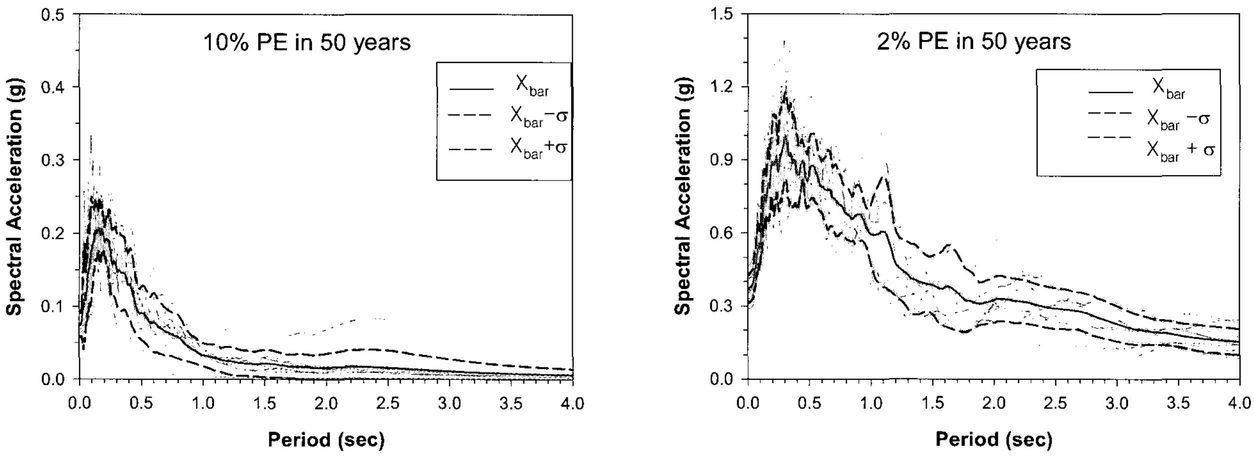
〈Figure 4〉 Analytical model used for abutments (left) and nonlinear impact model (right).

cities in the CSUS: Memphis, TN, Carbondale, IL, and St. Louis, MO, based on a New Madrid seismic event. These cities were selected for study because they represent a cross-section of the CSUS cities at risk. The ground motion simulation method used follows the procedure proposed by Hermann and Akinci (1999), which is based on Boore's point-source simulation method (1996). Soil amplification due to local site soils is considered, based on soil profile data in Memphis, Carbondale, and St. Louis. The ground motion is developed for two hazard levels: 10% and 2% probability of exceedance in 50 years. For each city and hazard level, a suite of 10 ground motion records is developed, resulting in a total of 60 ground motion records. The ground motion records will henceforth be referred to as 10% PE or 2% PE ground motion records. In this study, the ground motion from Memphis, TN, will be used to evaluate the performance of prestressed concrete girder bridges in the CSUS. The results from Carbondale, IL & St. Louis, MO., led to similar conclusions.[5]

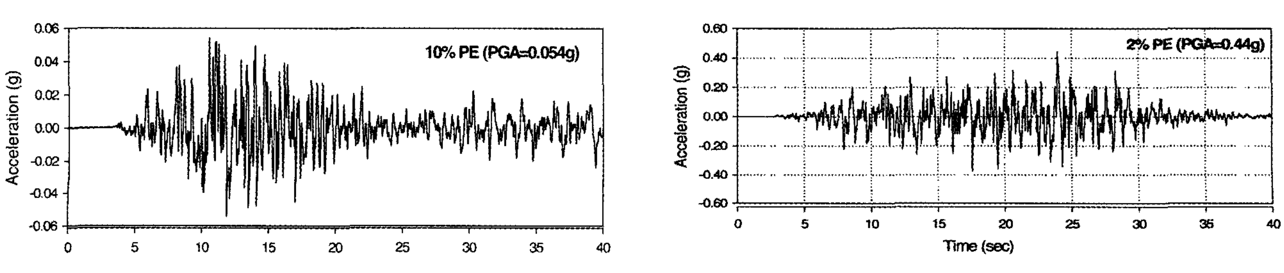
The mean, mean-minus-one and mean-plus-one standard deviation of the response spectral accelerations (MRS) for 5% damping for the 10% and 2% PE ground motion suites for the Memphis, TN is shown in Figure 5. As shown in the figure, the mean peak ground acceleration for the 10% and 2% PE ground motion suites are 0.08 g, and 0.38 g, respectively. The spectral accelerations at the fundamental period of the MSSS prestressed concrete girder bridge (T1=0.52 sec) are 0.09 g and 0.85 g for the 10% and 2% PE ground motion suites, respectively.

7. Seismic Response of Multi-Span Concrete Grider Bridge

To assess the seismic performance of the multi-span prestressed concrete girder bridge, the analytical model described above is used with a representative ground motion record from the ground motion suite from Memphis, TN for both the 10% and 2% probability of exceedance hazard level. The ground motion records, shown in Figure 6, have a peak ground acceleration of 0.054 g and 0.44 g for the 10% and 2% PE in 50 years



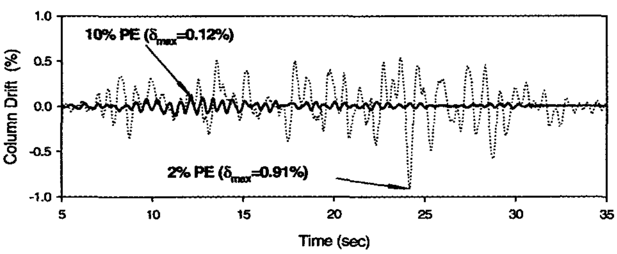
<Figure 5> Acceleration Response Spectra for Synthetic Ground Motion Suite for Memphis, TN, Used in the Seismic Response Analysis of the MSSS Prestressed Concrete Girder Bridge.



<Figure 6> Representative Ground Motion Records for 10% and 2% PE in 50 Years for Memphis, TN.

earthquakes, respectively. Preliminary studies show that the critical parameters in the response of the bridge are the column ductility (or drift), and the deformation of the elastomeric pad/dowel bar. Previous studies of typical non-seismically designed columns, such as that represented in this study, show that lap splice failure occurs at a drift of approximately 1.0%[14], and moderate damage can occur for curvature ductility demands greater than μ_ϕ=2.0.[9] In addition, failure or fracture of the dowel bar can lead to large relative hinge displacements, which may result in unseating of the deck.

Figure 7 shows the response history of the column 1 drifts for a representative ground motion record for the 10% and 2% PE ground motion records. As shown in the figure, the response to the 10% PE ground motion is considerably less than that for the 2% ground motion record. The maximum column drift of the bridge subjected to the 10% PE ground motion is 87% less than that for the 2% PE ground motion. The maximum drift

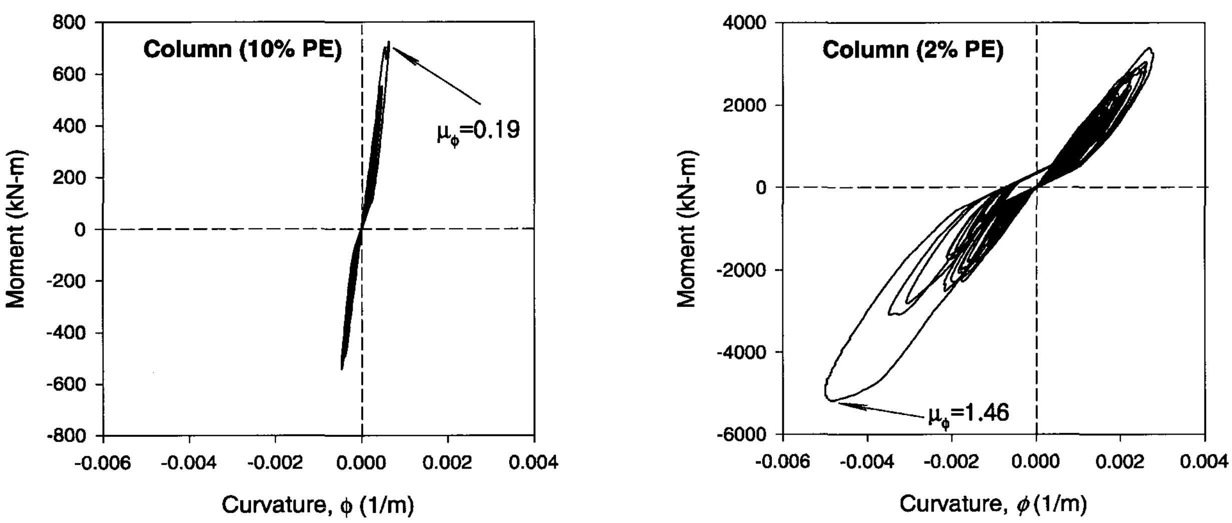


〈Figure 7〉 Column 1 Drift Response History for MSSS Prestressed Concrete Girder Bridge for Representative Ground Motion in the Central and Southeastern United States for the 10% PE and 2% PE in 50 Years Hazard Levels.

during the 2% PE ground motion, δ=0.91%, approaches the level where lap splice failure may occur, based on previous studies.[14]

Figure 8 shows the moment-curvature response of column 1 for the 10% and 2% PE representative ground motion records. As shown in the figure, column 1 experiences significant nonlinear behavior for the 2% PE ground motion. However, for the 10% PE ground motion, the column response is essentially linear. In fact, the response of all of the critical components of the bridge during the 10% PE ground motion are essentially elastic.

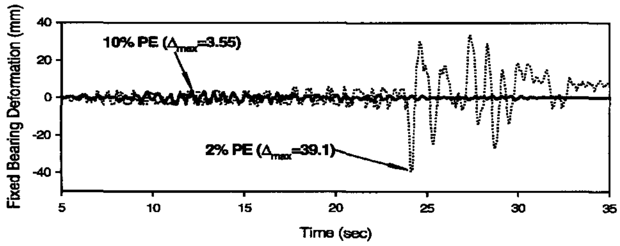
Figure 9 shows the time history response of the fixed dowel deformation at the left abutment, for a representative ground motion record from the 10% and 2% PE ground motion suites. Once again, the response from the 10% PE record is significantly less than that from the 2% PE record. For the 10% PE record, the dowel bars have engaged, but have not reached the deformation level corresponding to failure. However, for the 2% PE ground motion record, the maximum deformation of the dowels, $\triangle_{max}$=39.1 mm, exceeds the deformation at fracture. It should be noted that the large deformation in the dowel occurs at approximately 24 seconds in the response and is due to impact between deck 1 and deck 2. Following fracture, a number of cycles at large displacements are observed in the response history plot. The maximum relative hinge displacement (which corre-



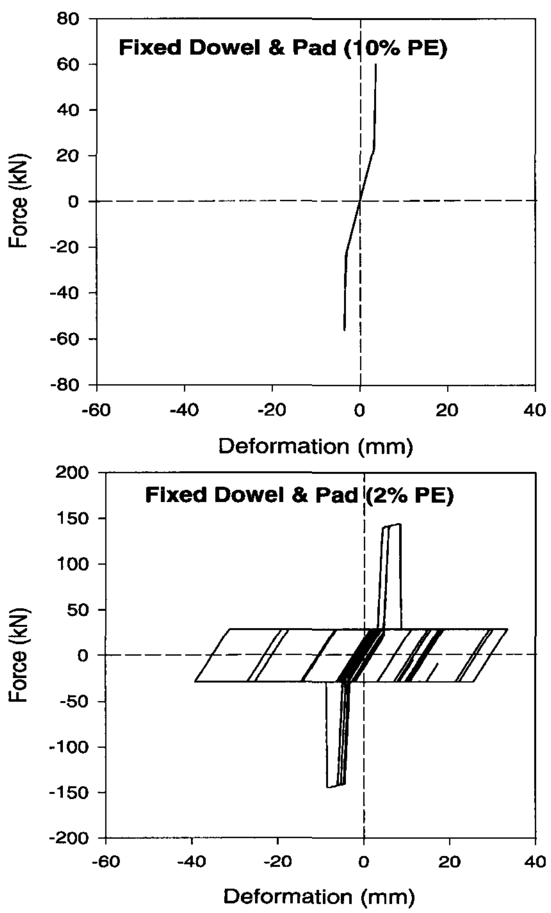
〈Figure 8〉 Column 1 Moment-Curvature Response for MSSS Prestressed Concrete Girder Bridge.

sponds to the pad deformation) is less than the typical allowable support, indicating that unseating of the deck is not likely. Figure 10 shows the force-deformation for the dowel bar/bearing pad element corresponding to the response history in Figure 9. As shown in the response, significant deformations occur in the 2% PE ground motion response, compared with the 10% PE ground motion response.

The analysis of the multi-span simply supported prestressed concrete girder bridge showed that the primary vulnerabilities include the response of non-ductile columns and failure of bearing supports due to excessive longitudinal displacement. In the next section, the effect of making the deck continuous on the seismic response is evaluated.



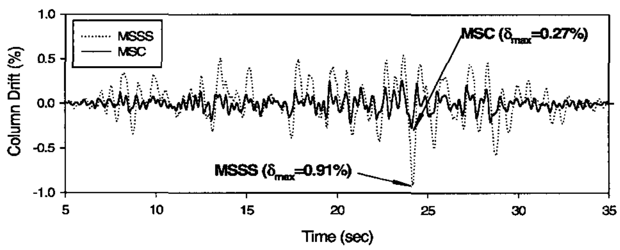
〈Figure 9〉 Fixed Bearing Response History for the MSSS Prestressed Concrete Girder Bridge for Repre-sentative Ground Motion in the Central and Southeastern United States for the 10% PE and 2% PE in 50 Years Hazard Levels.



〈Figure 10〉 Fixed Dowel Force Deformation Response for the MSSS Prestressed Concrete Girder Bridge.

8. Seismic Response of the Multi-Span Continuous Prestressed Concrete Girder Bridge

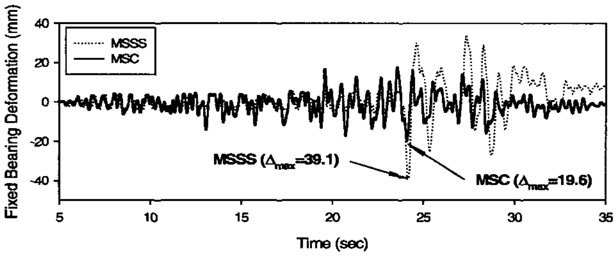
Multi-span simply supported prestressed concrete bridges are often made continuous by casting a parapet between the girders, or for many new bridges, the intermediate hinges are eliminated. This is done to reduce dead load moments and maintenance requirements. The effects that providing continuity to the deck has on the seismic response of the MSSS prestressed concrete girder bridge is evaluated by performing seismic response analyses, similar to that in the previous section. The nonlinear analytical model used in the previous section is modified to account for the continuity of the superstructure. The substructure properties and the bearing support properties remain the same. Figure 11 shows a comparison of the column drift response history for the multi-span simply supported (MSSS) prestressed concrete girder bridge and the multi-span continuous (MSC) prestressed concrete girder bridge for the 2% PE ground motion record shown in Figure 7. The response shows that the drift is significantly reduced in the MSC prestressed concrete girder bridge compared with the MSSS prestressed concrete girder bridge. The maximum drift in the



〈Figure 11〉 Column 1 Drift Response History for the MSSS and MSC Prestressed Concrete Girder Bridge for Repre-sentative Ground Motion in the Central and Southeastern United States for the 2% PE in 50 Years Hazard Levels.

MSC prestressed concrete girder bridge, δ=0.27%, is 70% less than the drift in the MSSS prestressed concrete girder bridge. The reduction in column response is primarily due to the reduced range of motion of the superstructure resulting from the continuity of the deck and the elimination of impact between the decks.

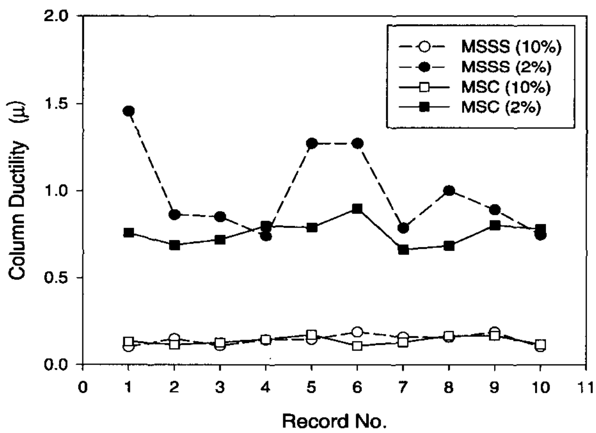
Figure 12 shows the fixed bearing deformation response history for the MSSS bridge and the MSC bridge for the same ground motion record. As observed in the column drift response, the MSC bridge has significantly reduced maximum deformations in the bearings, as compared with the MSSS bridge. The maximum deformation of the fixed bearing in the MSSS prestressed concrete girder bridge, $\triangle_{max}$=39.1 mm, is reduced to $\triangle_{max}$=19.6 mm when the bridge is made continuous.

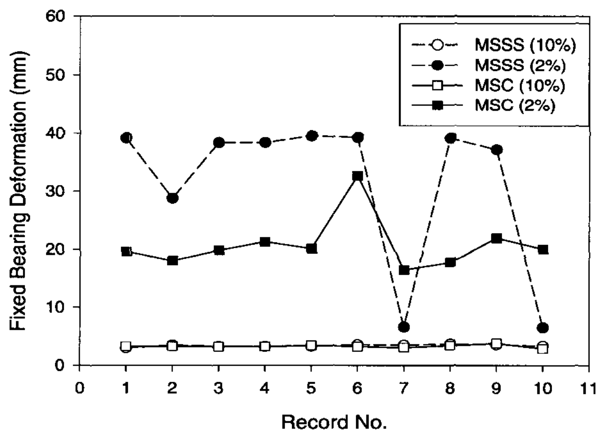


〈Figure 12〉 Fixed Bearing Deformation Response for the MSSS and MSC Prestressed Concrete Girder Bridge for Representative Ground Motion in the Central and Southeastern United States for the 2% PE in 50 Years Hazard Level.

9. Mean Seismic Response of Msss and MSC Prestressed Concrete Girder

To obtain a statistically meaningful representation of the response of the multi-span simply supported and multi-span continuous prestressed concrete girder bridge, the analytical models are evaluated for the suite of 20 ground motion records for Memphis, TN, representing the 10% and 2% probability of exceedance in 50 years hazard levels. The two response parameters that are evaluated are the column curvature ductility (μ_ϕ), and fixed bearing deformation ($\triangle$). Figure 13 (a) shows the mean column ductility demands for the MSSS and MSC prestressed concrete girder bridges for the 10% and 2% PE ground motion suites. As previously mentioned, the response of the bridges is considerably less in the 10% PE ground motion suite compared to the 2% PE suite. The mean column ductility for the 10% PE ground motion suite is approximately μ_ϕ=0.14 for both the multi-span simply supported and multi-span continuous bridges. For the 2% PE suite, the mean column ductility demand for the MSSS bridge is μ_ϕ=1.0, compared with μ_ϕ=0.76 for the MSC bridge. As shown in the figure, the range of column ductility demands for the MSSS bridge, μ_ϕ=0.74-1.46, is much larger than the range for the MSC bridge, where μ_ϕ=0.66-0.90. The increased scatter in the response of the MSSS is due primarily to the uncertainty in response resulting from impact between decks. As discussed in the previous section, much of the column displacement is associated with





〈Figure 13〉 Mean Response of MSSS and MSC Bridges for Suite of Ground Motion for Memphis, TN, for 2% PE Hazard Levels (a) Column Ductility, and (b) Fixed Bearing Deformation.

impact between the decks. When impact occurs, the column displacements (and thereby ductility demands) are very large. However, when impact does not occur, the column demands are similar to that found in the continuous bridge. This explains the large variability in the column ductility demands for the 2% PE ground motion suite.

Figure 13 (b) shows the mean deformations for the fixed bearing for the 10% and 2% PE ground motion suites for the MSSS and MSC prestressed concrete girder bridges. The mean bearing deformation for the MSSS bridge for the 2% PE ground motion suite, $\triangle$=31.3 mm, is approximately 50% greater than the mean bearing deformation for the MSC bridge. Although the response in the MSC bridge is considerably less than that of the MSSS bridge, the deformation is still greater than that which would cause fracture of the bars. Note the variability in the bearing deformation in the MSSS bridge. In particular, for records no. 7 and 10, the fixed dowel response in the MSSS bridge is considerably less than the other responses and is less than the response in the MSC bridge. This occurs because these ground motion records induced less pounding between the bridge decks compared with the other ground motion records.

10. Conclusions and Observations

This paper presents the evaluation of the seismic response of a typical multi-span simply supported prestressed concrete girder bridge commonly found in the New Madrid Seismic Zone (NMSZ) region of the central United States. These bridges were typically not designed with consideration of seismic loads. However, recent studies have shown that the seismic hazard and expected ground motion in the NMSZ region of the central United States may be sufficient to induce damage, particularly for an earthquake with a 2% probability of exceedance in 50 years.

Using detailed nonlinear analytical models and synthetic ground motion for Memphis, TN, the seismic performance of a typical MSSS prestressed concrete girder bridge was evaluated. The results show that for a 475-year characteristic earthquake (10% probability of exceedance in 50 years), the seismic performance of the MSSS prestressed concrete girder bridge is very good. The demands on the columns as well as the bearings are in the elastic range. However, the seismic performance of the bridge during a 2475-year earthquake (2% probability of exceedance in 50 years) shows several vulnerabilities for the bridge. First, the column curvature ductility demands range from μ_ϕ=0.74 to μ_ϕ=1.46. For the non-seismically designed columns considered in this study, this would result in minor-to-moderate damage. Furthermore, the results of the analysis show that the elastomeric pads used to transfer the load from the PSC girder to the substructure are likely to fail during moderate-to-strong ground motion. The demands on the bearings far exceed their capacity, resulting in fracture of the dowel bars in the bearings.

The effect of making the deck continuous in a prestressed concrete girder bridge is also evaluated. The results of the analysis show that making the bridge continuous significantly improves the seismic performance of the bridge, primarily because it eliminates pounding, which often results in excessive forces in the bearings and columns. Results of the analysis show that the continuous bridge has, on average, 24% less drift in the piers and 50% less demand on the bearings compared with the multi-span simply supported bridge. This is an important result because many departments of transportation are moving towards the design of the superstructure in prestressed concrete girder as continuous.

In Korea, there are many PSC girder bridges. Most of them have bridge bearings, however, others were constructed with dowels to restrict the movement of PSC girders. Therefore, this study results can contribute to improve the understanding of the seismic behavior of PSC girder bridges in Korea. Also, the continuity of PSC girders can by applied on any type of bearings in Korea.

Notation

f_c = concrete stress

f'_c = concrete compressive cylinder strength

f_{yh} = yield strength of steel hoops
h" = width of concrete core measured to outside of hoops
K = confining factor
s_h = center-to-center spacing of transverse reinforcement
Z_m = factor used for the calculating the stress-strain relationship for the confined concrete
ε_c = concrete strain
ε_0 = strain at which peak concrete stress is attained for unconfined concrete
ρ_s = Reinforcement ratio of transverse reinforcement
δ = drift ratio of columns
δ_{max} = maximum drift ratio of columns
$\triangle$ = deformation of dowels
$\triangle_{max}$ = maximum deformation of dowels
μ_ϕ = ductility of columns
X_{bar} = Mean Spectral Acceleration
σ = Standard Deviation of the Spectral Acceleration

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