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Enhancing the Structural Longevity of the Bridges with Insufficient Seismic Capacity by Retrofitting

Y.C. Sung[1], D.W. Chang[2], M.Y. Cheng[2], T.L. Chang[2] and K.Y. Liu[3]

Abstract: The seismic assessment and retrofit of existing bridges has become important in Taiwan, since the Chi-Chi earthquake in 1999. In order to ensure the safety of the traffic network, a project has been launched to assess the seismic resistance capacity of over 2,200 bridges on the Taiwan Roadway System. A detailed evaluation procedure, using the modified ATC-40 capacity spectrum method was first applied to over 140 regular bridges. The state-of-the-art nonlinear static analysis (pushover analysis) was adopted to calculate the seismic resistance capacity in terms of PGA, in order to compare the seismic demand according to the code requirement. A case study of the Li-Kun bridge in Taiwan will introduce the detailed seismic evaluation procedure. Based on the assessment results, the retrofit strategy might be established to prolong the lifetime, and also enhance the seismic resistance of the bridge. A feasible approach of using a steel jacket can extend the anticipated service life longer than 100 years. Therefore, the goal of structural longevity is achievable at modest cost.

Keywords: Bridge, Seismic assessment, Pushover analysis

1 Introduction

Taiwan is situated in a high seismic hazard zone, and has experienced several earthquakes. Since the Chi-Chi earthquake in 1999, the seismic assessment for the existing bridges has become a major issue for bridge engineering. In the past few years, all the bridge management authorities have focused on the retrofitting of those vulnerable bridges to avoid bridge collapses, if there is another big earthquake in Taiwan. In the Chi-Chi earthquake[Chang (1999); CECI (2000)], the most severly damaged bridges, as shown in Figure 1 and 2, were located close to the Che-Lung-Pu Fault and were under the supervision of the Directorate Gen-

[1] Department of Civil Engineering, National Taipei University of Technology
[2] CECI Engineering Consultants, Inc., Taiwan
[3] National Center for Research on Earthquake Engineering, Taipei, Taiwan

eral of Highways, MOTC. Therefore, a project of seismic retrofitting feasibility study for Taiwan Provincial Highway bridges was issued by Directorate General of Highways, MOTC. A total of 2,213 bridges were evaluated in the project. In order to perform the seismic assessment effectively, it is important to establish a quick and accurate assessment procedure. The seismic assessment procedure is divided into two stages, a preliminary evaluation and a detailed analysis. In the first stage, two screening sheets, one for bridge falling evaluation, and another for ductility/strength evaluation are used to assess all the bridges. In the second stage, approximately 5% of the bridges, about 140 bridges, are selected to conduct a detailed pushover analysis. Based on the evaluation results, adequate retrofitting schemes will be proposed and the approximate retrofitting cost will be estimated. With all this information included, a national construction plan will be submitted to the client, for their annual budget planning of the Taiwan Highway Bridges seismic retrofitting.

Figure 1: The Tong-Tou Bridge which collapsed in the Chi-Chi earthquake

2 Modified ATC-40 Method

In 1996, ATC-40[ATC (1996)] established a procedure to assess the structural capacity based on the capacity spectrum method. The capacity spectrum can be transformed from the pushover analysis. In addition, the demand spectrum obtained from the elastic spectrum, modified by a procedure of effective viscous damping, equivalent to the nonlinear response, was used to present the inelastic structure behavior under a specific ground motion. The intersection of capacity spectrum and

Figure 2: Shear failure of pier wall of the Wu-Shi Bridge in the Chi-Chi earthquake

inelastic spectrum, named as performance point, can be located through an iterative calculation.

A well-defined plastic hinge should be the key point to have an accurate pushover curve. The commercial software package SAP-2000[CSI (2002)] is used to perform the pushover analysis. Provided with some convenient default hinges for the characteristic of plastic hinge of RC member, a study using SAP-2000 found that the analytical results in the force-displacement curve sometimes did not quite match with the time history analysis [Sung *et.* al (2005)]. Five points A∼E, as shown in Fig 3, are needed to be input in order to define a plastic hinge While the segment AB represents the linear behavior, segments BC, CD, and DE are the nonlinear parts. In order to capture the actual behavior of RC columns, and obtain a better simulation of the nonlinear behavior, a modification of the defaulted M3 model in SAP-2000 has been made. The corresponding three different failure modes, namely shear failure, bending to shear failure and bending failure are redefined in Figure 4. The characteristic of the modified plastic hinge is to replace the defaulted M3 model in SAP-2000. With this modification of plastic hinges, SAP-2000 can complete the pushover analysis with better efficiency as well as accuracy. Hence, both a pre-processor and a post-processor were needed to be developed, for the linking of a pushover analysis. The pre-processor determines the modified plastic hinge properties, while the post-processor could convert the capacity curve from the pushover analysis to a graphic output of the structural capacity spectrum.

The basic objective of the ATC-40 is to evaluate the structural performance under a given seismic demand. The structural capacity, the relationship between base

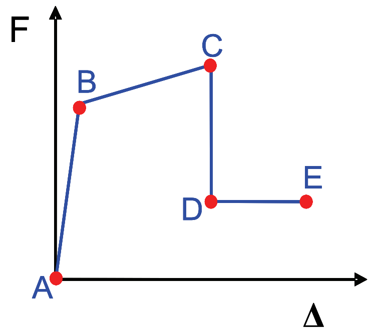
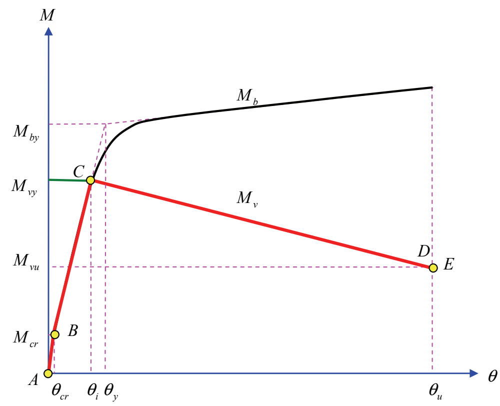
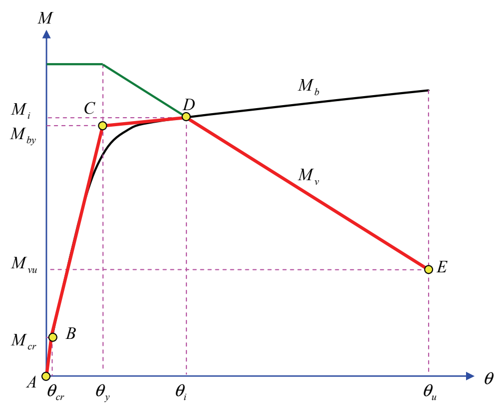


Figure 3: The plastic hinge model used in SAP2000

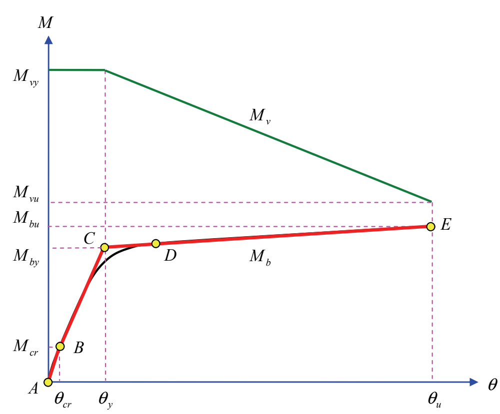
shear force and top displacement, are recorded during the pushover analysis to present the structure behavior from elastic to nonlinear range. On the other hand, the seismic demand is obtained from the elastic response spectrum of acceleration directly. Plotted on the same graph with ADRS format (Acceleration and Displacement Response Spectrum), the capacity spectrum and demand spectrum are transferred from the pushover curve and the elastic response spectrum of acceleration, respectively, under the following assumptions: (1) a multiple degree of freedom system can be simplified as a of single degree of freedom system, (2) the observed dissipated energy through inelastic deformation of the structure is simulated by the equivalent hysteresis damping and is used to reduce the elastic response spectrum of acceleration, in order to generate a nonlinear response spectrum. Therefore, the structure performance or performance point, which is in terms of the spectral acceleration and spectral displacement, can be determined at the intersection point (d_p, a_p) from the capacity spectrum curve and demand spectrum curve by using Procedure A in the ATC-40. Generally speaking, the ACT-40 scheme seems more suitable for the design task rather than the evaluation task. Besides, it is due to the iteration process in Procedure A that the performance point cannot be found in many cases. A new methodology has been proposed by Prof. Sung [Sung (2003)]. Since the bridge does no fail, the structural performance point should be always locating right on the curve of the capacity spectrum. Every single performance point along the capacity curve can be determined as long as the pushover analysis has completed. Therefore, it can be used as "input" to calculate the corresponding seismic demand as "output", in such way that the complicated iterations can be eliminated. The performance point is located at the interaction of the capacity spectrum and the inelastic demand spectrum, as shown in Figure 5, and thus meets the mutual property of both spectrums. Such that spectral acceleration a_{pi}



(a) The plastic hinge characteristic of an RC column for shear failure



(b) The plastic hinge characteristic of an RC column for bending to shear failure



(c) The plastic hinge characteristic of an RC column for bending failure

Figure 4: Characteristics of the plastic hinge for an RC member

and displacement d_{pi} for the capacity spectrum would be the same as (S_a) inelastic and (S_d) inelastic for the inelastic demand spectrum. The effective damping β_{eff} includes the inherent damping basicβ_{basic} in the structure and equivalent viscous damping β_o taking into account for the energy dissipation of the hysteretic loop.

According to ATC-40, β_{eff} is calculated as

$$\beta_{eff} = \beta_{basic} + \beta_o = \beta_{basic} + \frac{0.637\kappa(a_y d_{pi} - d_y a_{pi})}{a_{pi} d_{pi}} \tag{1}$$

Where κ is the damping modification factor to reflect the actual hysteretic behavior of the structure, and a_y and d_y is the spectral yield acceleration and displacement, respectively. The relationship between PGA and the spectral acceleration a_{pi} then can be expressed as

$$PGA = \frac{a_{pi}}{(S_a)_{inelastic} \times C_D}, \tag{2}$$

where

$$C_D = \frac{(S_a)_{inelastic}}{(S_a)_{elastic}} = \frac{1.5}{40\beta_{eff} + 1} + 0.5.$$

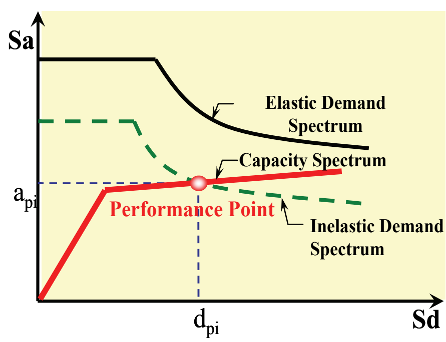


Figure 5: Capacity and the demand spectrum

3 Detailed seismic evaluation procedure

The detailed seismic analysis procedure for a bridge structure is described as follows:

1. Establish the bridge model.

2. Calculate the moment-curvature relationship of each pier column, determine the failure mode, i.e. shear failure, bending to shear failure or bending failure. Define the M3 plastic hinge for SAP-2000.

3. Perform the pushover analysis, establish the capacity curve, and then convert the capacity curve to the capacity spectrum.

4. Prepare the normalized design acceleration spectrum $C(T)$[MOTC (1995)] with chosen soil type.

5. Calculate the yielding ground acceleration PGA_y corresponding to the yielding state of the pier on the capacity spectrum curve. Damping modification factor $C_D = 1.0$.

$$PGA_y = \frac{a_{py}}{C(T)} \tag{3}$$

6. Calculate the ultimate ground acceleration PGA_ucorresponding to the final state of the pier on the capacity spectrum curve, effective damping β_{eff}, the damping modification factor C_D.

$$PGA_u = \frac{a_{pu}}{C(T) \times C_D\left(\beta_{eff}\right)} \tag{4}$$

7. Output the envelope of pier top and bottom reaction forces from pushover analysis.

8. Check the bearing strength, foundation strength and stability.

9. Determine the ground acceleration of first damage case among bearing, pier column and foundation.

4 Case study

4.1 Basic information

The Li-Kun bridge, located in Taiwan No.3 Provincial Highway, is a P.C.I bridge with 43 units (Figure. 6). The span arrangement is 3@35+16@40+(20+6@40+20) +15@40+25m, and the total length of the bridge is 1620m. The Li-Kun bridge was widened in 1992. The old part of bridge was built in 1977, while the new part was built in 1992, so that the bridge was designed according to two different seismic design codes. The deck width of old bridge and new bridge is 8.1m and 17.9m, respectively. The total width of the deck is 26m. The pier for old bridge is column type with average height 8.5m and wall type pier for widened bridge in average

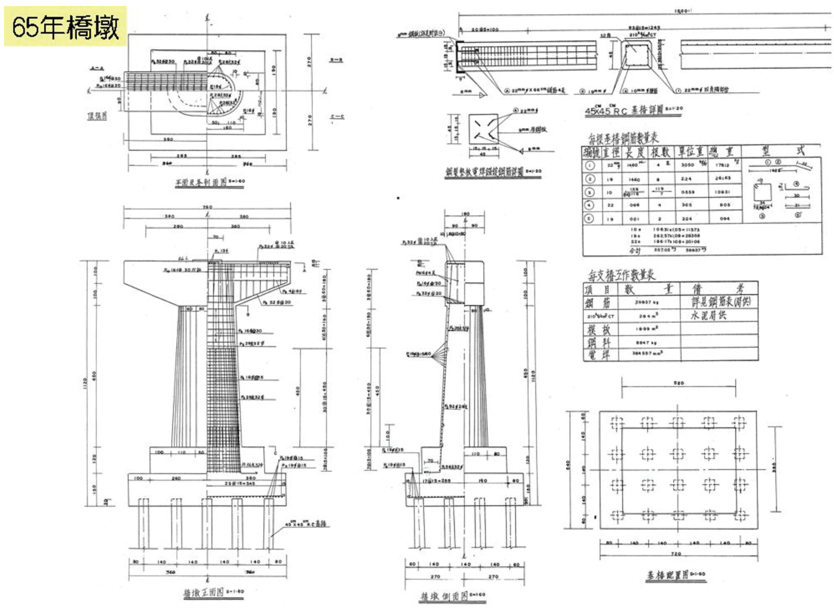
10m high, the reinforcement details of pier columns are shown in Figure 7. The foundation of old bridge is 45 $\times$ 45cm RC piles, and the widen bridge is caisson foundation. The structure is simply supported with hinge on one pier and roller on the other pier.

(a) Side view (b) Front view

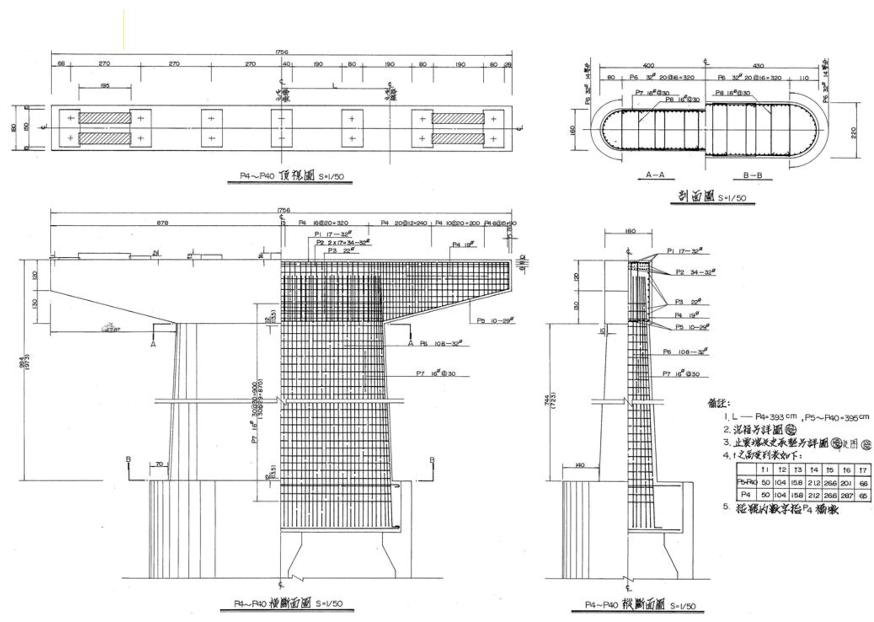
Figure 6: The Li-Kun Bridge

4.2 Analytical model and pushover analysis

The analytical model for a typical evaluation unit is shown in Figure 8 with a typical span length of 40m. Both old and new parts of the bridge are connected. The model includes 10 P.C.I. girders, deck slabs, bearing, intermediate diaphragms and piers. The foundation is simulated as a fix-end support on the ground. The pre-determined moment plastic hinges shown in Figure 9.were obtained by NARC2004 [Sung et. al (2005)]. In this model, only four hinges are needed since the evaluation unit consists of four singe columns, either in longitudinal or transversal direction. Obviously, the moment capacity of the new part in both longitudinal and transversal directions is larger than the old one by satisfying the new seismic design requirement. On the other hand, it is due to the low-rise pier wall that the rotation capacity for new part in the transversal direction is less than the old one. Given those four hinge properties with inelastic behavior, the capacity curves can be determined by pushover analysis. Figure 10 presents the base shear and top displacement of each pier. By combining the shear force from both old and new part of the bridge, the total base shear can be found in longitudinal and transversal direction, respectively. Conservatively, the ultimate point on the combination curve stops at the same displacement on the pushover curve of the old pier. In Figure10, the widened bridge has higher capacity than the old bridge. The base shear capacity is 4.071MN(415



(a) Pier in 1977



(b) Pier in 1992

Figure 7: Detailed drawings of the pier of the Li-Kun bridge

tonf) and 2.197MN(224tonf) for new bridge and old bridge, respectively. The top curve represents the capacity of entire bridge, which is 6.269MN(639 tonf) equal to the sum of widen bridge and old bridge. Same combination rule applies to the transversal direction finds out its ultimate force of the entire bridge

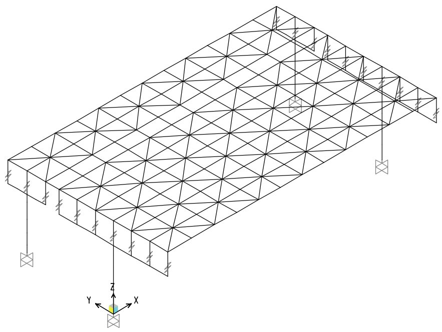
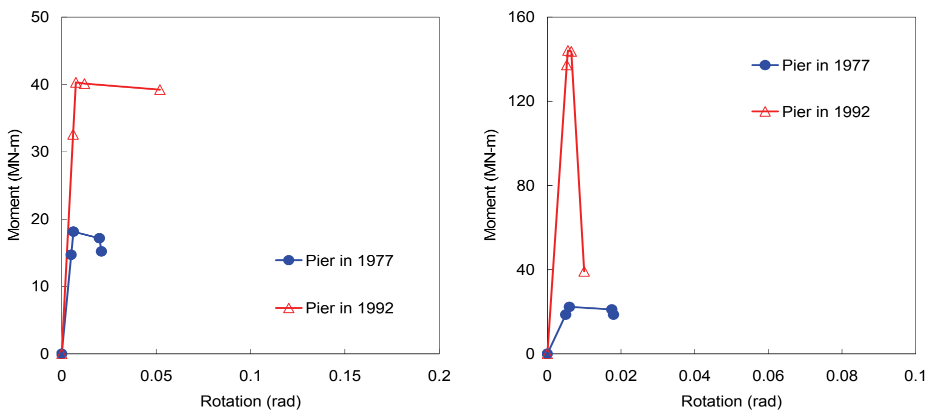


Figure 8: Analytical model used in SAP-2000



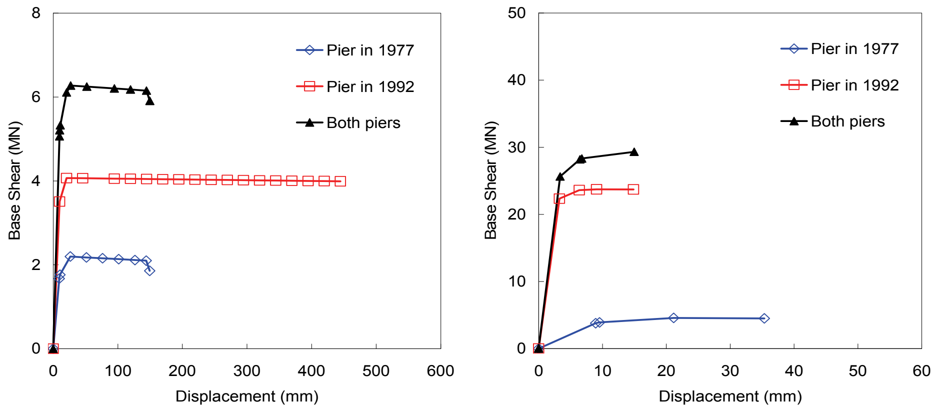
(a) Longitudinal direction (b) Transversal direction

Figure 9: Moment plastic hinge of the Li-Kun bridge (before retrofitting)

4.3 Seismic evaluation result

4.3.1 Original bridge

The capacity spectrum of the bridge is shown in Figure 11 with three different damping modification κ-factors. The κ-factor represents the energy dissipation



(a) Longitudinal direction (b) Transversal direction

Figure 10: Pushover Curve of the Li-Kun bridge (before retrofitting)

ability or a reduction ratio of a full hysteresis loop, depending on the structural behavior: good (1.0), moderate (2/3), and severely pinched (1/3). The yielding and ultimate PGA is 0.138g and 0.354g, respectively, from the analysis results when taking κ=1/3. Considering the uncertainties for the reinforcement detail of column, particularly the inadequate lap splice[Chang et.al (1999)], the plastic hinge of pier won't be fully developed as a flexural failure. Therefore, the seismic capacity of the bridge is conservatively reduced by following the rule: $PGA_y + 1/5\,(PGA_u - PGA_y)$, which gives a value of 0.181g and 0.331 in the longitudinal and transversal direction, respectively (Figure 12). In general, the smaller value controls the evaluation result, resulting in the final PGA capacity of 0.181g. According to the seismic design specification of the bridge [MOTC (1995)], the zone factor Z, associated with the ground acceleration of 0.33g, and an important factor I = 1.2 are multiplied to represent the required ground acceleration demand of 0.396g. Since the PGA capacity is smaller than demand one, the retrofitting measure is recommended to apply to this bridge.

4.3.2 Retrofitted bridge

A standard approach using steel jacket is chosen and the thickness of 9mm is determined based on the design procedure of the flexural failure in FHWA seismic retrofit manual [FHWA (1995)]. Figure 13 presents the moment plastic hinge of the retrofitted bridge. In comparison with Figure 9 and Figure 13, utilizing the flexural retrofitting can dramatically enlarge the rotation capacity but the moment strengths

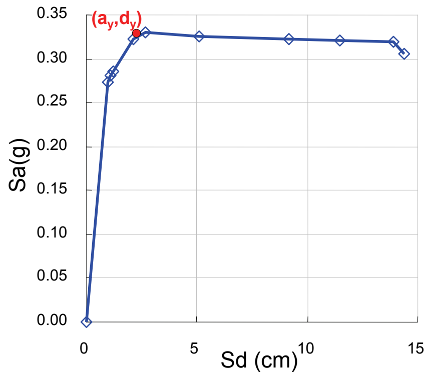
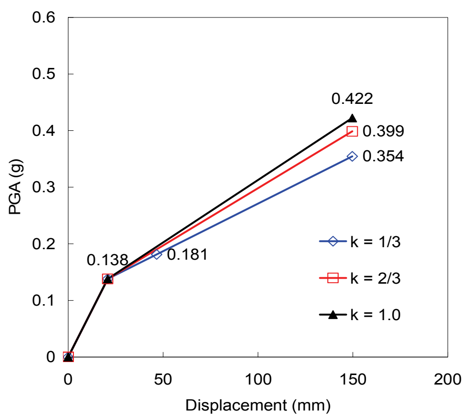
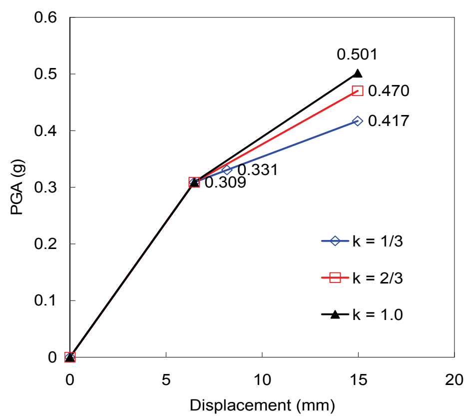


Figure 11: Capacity Spectrum of the Li-Kun bridge in the longitudinal direction (before retrofitting)



(a) Longitudinal direction



(b) Transversal direction

Figure 12: The result of the PGA evaluation of the Li-Kun bridge (before retrofitting)

are almost the same. With those new properties of plastic hinges, the pushover curve of retrofitted the Li-Kun bridge shown in the Figure 14 demonstrates similar trend of the plastic hinge. For a retrofitted bridge, even though the nonlinear deformation ability is ideally improved, indicating κ of 1.0 can be used, a conservative way by adapting the rule $PGA_y + 2/3\,(PGA_u - PGA_y)$ is recommended to reduce resistance capacity of PGA. As a result, the smaller value of PGA, between the results in Figure 15(a) and (b), give a PGA capacity of 0.488g, which is larger than the demand PGA of 0.396g, illustrating that the proposed approach satisfies the .retrofitting target to improve the seismic behavior.

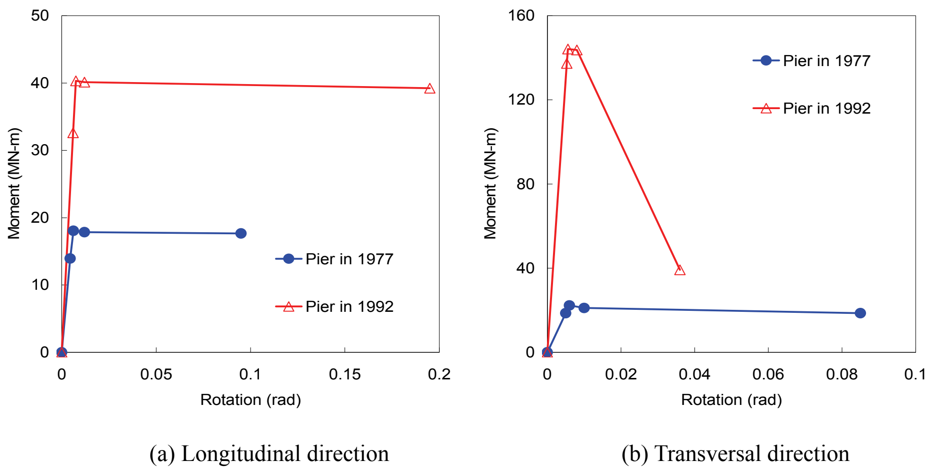


Figure 13: Moment plastic hinge of the Li-Kun bridge (after retrofitting)

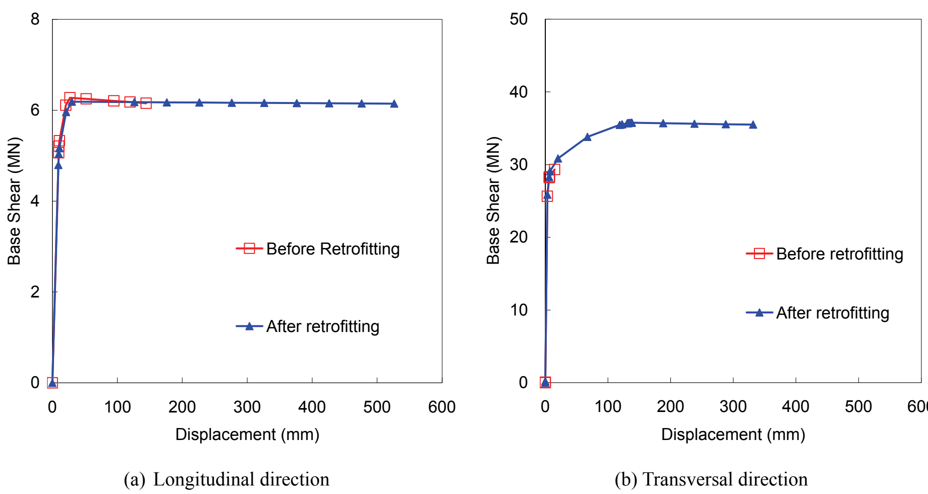
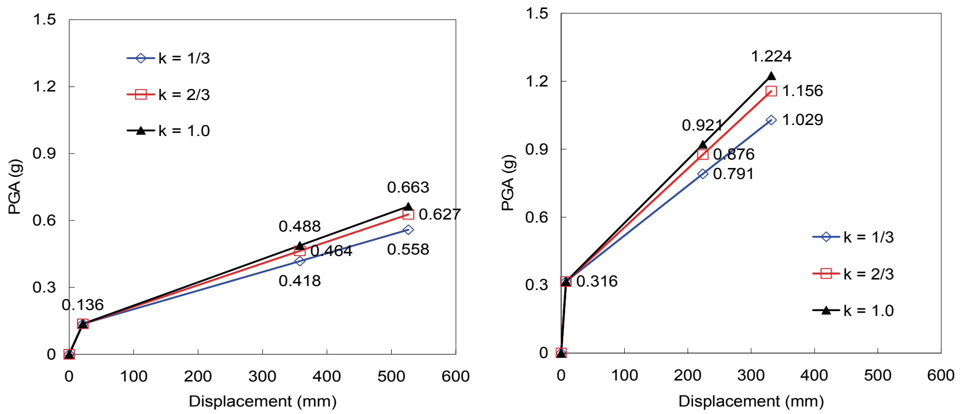


Figure 14: Pushover Curve of the Li-Kun Bridge (after retrofitting)



(a) Longitudinal direction (b) Transversal direction

Figure 15: The result of the PGA evaluation of the Li-Kun Bridge (after retrofitting)

4.3.3 Return period

In general, to judge the seismic evaluation result, the most used parameter is peak ground acceleration, PGA. In the view point of structural longevity, another parameter by using return period of earthquake is a more specified approach. For a structure to be evaluated, the relationship between its target return period and design return period, which is based on the code requirement, can be calculated through seismic hazard analysis [Eurocode 8 (1998)] and Poisson model [Kramer (1996)]as follows:

$$PGA_i = \left(\frac{T_i}{T_g}\right)^k a_g \tag{5}$$

$$1 - e^{-\lambda T} = p \tag{6}$$

$$\lambda = 1/T_i \tag{7}$$

Where, PGA_i is the acceleration, usually, which refers to the acceleration at which the structure is yielding or totally collapsed. a_g is the design acceleration, in this case study, it is 0.33g according to the seismic design specification of the Highway bridge in Taiwan [MOTC (1995)]. T_i and T_g mean the return period corresponding to PGA_i and a_g, respectively. Power k is in the range from 0.3 to 0.45. In addition, $\lambda$is the average rate of occurrences of the event ($\lambda = 1/T_i$), T is the time period of interest or called anticipated service life of the structure, and p is the probability of at least one exceedance in a period of T years. Therefore, if $a_g = 0.33g$,

$PGA_i = 0.488g$, $T_g = 475$year, and $k = 0.45$ are substituted in the equation (4), the outcome we can have is that the return period $T_i = 1131$ year. This result means the retrofitted Li-Kun bridge has the ability to resist an earthquake whose return period is increasing from the current code basis of 475 year ($T_g = 475$) to a longer period of 1131year ($T_i = 1131$). Base on the Poisson model, a level of hazard that has a 0.0884 % probability of exceedance ($p = 0.000884$) in a 50-year exposure period ($T = 50$) can be expected. Instead, given the PGA of 0.663g ($PGA_i = 0.663$) with 10 percent probability of exceedance ($p = 0.1$), one can have an anticipated service life of 236 year ($T = 236$) for this retrofitted bridge. Therefore, given the same conditions of exposure period and probability of exceedance, the anticipated service life of the Li-Kun bridge, either before or after retrofitting, can be calculated by using data from Figure 12(a) and 15(a). As shown in Figure 16, it is expected to prolong the service life from 13 year to 120 year. The goal of extending the structural longevity is achieved. So, engineers can upgrade the structural performance object to a higher level when setting a realistic target in the performance-based design approach.

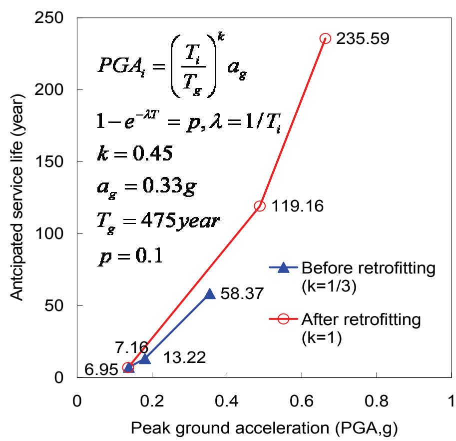


Figure 16: Anticipated service life of the Li-Kun bridge (before and after retrofitting)

5 Conclusions

A simplified and novel seismic assessment procedure has been introduced. A state-of-the-art pushover analysis method is adopted to calculate the seismic resistance capacity of the bridge. A few modifications have been made in the pushover analysis to obtain more reasonable results. The case study of the Li-Kun bridge shows the efficiency of applying the retrofitting measure to improve the seismic behavior in terms of the return period of the earthquake or anticipated service life. Analytical result shows that the Li-Kun bridge can sustain an earthquake with PGA of 0.488g and can function effectively for more than 107 years after performing the seismic retrofitting work. Therefore, the goal of extending the structural longevity is achieved. This paper is useful for engineers to define a higher performance objective when setting a realistic target in the performance-based design approach.

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