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# DAMAGE INVESTIGATION AND SEISMIC RETROFIT OF BRIDGES IN TAIWAN AFTER 921 CHI-CHI EARTHQUAKE

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**ABSTRACT**: The devastating Chi-Chi earthquake on September 21, 1999 brought catastrophic disaster to the central part of Taiwan. Some of the bridges located in this area suffered severe damage. Damage to these bridges includes fault rupturing, collapsed spans, landslides, soil settlement, slope failures, flexural and/or shear failures, and liquefaction. These damages made it necessary for part of these bridges to be retrofitted or rebuilt and also lead special attention to the upgrade of seismic design and retrofit in Taiwan. The Chi-Chi earthquake brought an intense impact on the seismic design and technical development of bridge engineering. Based on the investigations of existing bridges damaged in Chi-Chi earthquake and the significant amount of retrofit research and actual implementations of seismic evaluation and retrofitting on existing bridges, the seismic design code in Taiwan was revised and a manual related to seismic evaluation and retrofit for highway bridges was proposed. This paper begins with a brief description on the damage investigation of bridges located in the catastrophic region after Chi-Chi earthquake, followed by a general depiction of the urgent recovery, repair and reconstruction of the damaged bridges. In the end, the Seismic Retrofitting Manual for Highway Bridges proposed by NCREE (National Center for Research on Earthquake Engineering) was also introduced. This manual includes current advances in earthquake engineering, the performance-based seismic evaluation technique, and retrofitting design and measures.

**Key Words**: Chi-Chi earthquake, highway bridge, seismic assessment, seismic retrofit

## INTRODUCTION

Chi-chi earthquake with the magnitude of ML = 7.3 struck the central region of Taiwan in the early morning on September 21, 1999. Approximately 1,000 highway bridges, on the provincial or county routes in Taichung, Nantou, Changhua, and Yunlin counties, escaped from serious damages, while approximately 20% of them suffered minor-to-major damage due to fault rupturing, collapsed spans, landslides, soil settlement, slope failures, flexural and/or shear failures, and liquefaction. This paper begins with a general depiction of the performance of highway bridges located in the four counties during the earthquake. Several major damaged bridges with typical damage modes are illustrated and explored. Lessons learned from the field observations are also discussed.

Most of the severely damaged bridges serve as the vital route in these areas and the traffic flow need to be restored in a short time, so the immediate decision for the urgent recovery plan and the strategies for seismic retrofit/reconstruction are crucial. At the time shortly after Chi-Chi earthquake, several adverse conditions were against the reconstruction design project. For instance, the design schedule was tight, causes of failures were not well documented, local design code related to new devices, such as isolation devices, were not available, and a need to revise "Seismic Resistant Code for Highway Bridges" according to the newly found characteristics of Chi-Chi earthquake was not yet fulfilled, too. In order to achieve the goal of recovering the traffic flow as soon as possible, the top priority for the reconstruction design was the upgrade of seismic safety and the reduction of construction time. In this paper, a brief description on the seismic retrofit strategies, which include urgent recovery plan, repair and reconstruction strategies is provided. The reconstruction experiences, including several main points of reconstruction design, were also briefly presented.

The extensive damages of bridges due to the Chi Chi earthquake lead special attention to the upgrade of seismic design and retrofit of bridges in Taiwan. Based on the investigations of existing bridges damaged in Chi-Chi earthquake, a significant amount of retrofit and seismic evaluation research was performed during the last decade. According to these researches, the seismic design code in Taiwan was revised and a manual related to seismic evaluation and retrofit for highway bridges was proposed. In addition, several actual applications of seismic evaluation and retrofitting on existing bridges were completed. In this paper, the manual related to seismic evaluation and retrofit for highway bridges will also be briefly introduced and the retrofitting project performed by Taiwan's National Freeway Bureau and Directorate General of Highway based on this manual will also be summarized.

## DAMAGE INVESTIGATION

The primary disaster area in the 921 Chi-Chi earthquake is composed of four counties, Taichung, Nantou, Changhua and Yunlin. There are approximately 1,000 highway bridges spread on the main provincial and county routes in the disaster area. The construction completion dates of those bridges range from 1960 through 1999, and hence are involved in the evolutionary history of seismic design codes. Table 1 summarizes the locations and damage levels of the inspected highway bridges. In this table, 80% of the bridges performed well without damage, 17.3% of them suffered minor to moderate damage, and 2.7% of them received major damage and even collapse.

Table 1 Summary of damaged highway bridge in 1999 Chi-Chi earthquake.

| County | Number | | | |
|---|---|---|---|---|
| | bridges | Non damaged | Minor-to-moderate damaged | Major damaged |
| Taichung | 196 | 131 | 52 (26.5%) | 13 (6.6%) |
| Changhua | 199 | 182 | 17 (8.5%) | 0 |
| Nantou | 410 | 315 | 82 (20.0) | 13 (6.6%) |
| Tyunlin | 176 | 158 | 18 (10.2) | 0 |
| Summary | 981 (100%) | 786 (80.1%) | 169 (17.2%) | 26 (2.7%) |

* the percentage is based on the total inspected bridges in each county.

Most of the highway bridges escaped from severe damage and experienced only minor distress such as the settlement of approach fills behind abutment back-walls. Approximately 20% of the bridge inventory suffered minor-to-major damage. Damage to these bridges include collapse of superstructures, displaced bearings, unseated girders from bearing supports, shear failures in columns, pier walls, and caissons, abutment back-wall failure, settlement of approach slab, foundation failures due to slope instabilities, joint failures in column-to-girder connections, cable fracture, fault rupture, and liquefaction. However, only Lyu-mei and Wan-lun bridges located in Yuen-lin town were observed to have liquefied appearance since it was not easy to distinguish when it occurred in a flowing river.

Table 2 shows a summary of comparison among 11 different damage modes. It is seen that in these 195 bridges, the ratios of damage to auxiliary facility, abutment, deck, and approach slab are 35.6%, 32.5%, 26.3%, and 21.1%, respectively, significantly larger than those of other damage modes. From the field observation, it was found that pounding on abutments due to excessive longitudinal vibration of the superstructure usually occurred with damage to the approach slabs and/or bridge decks. The deck damage defined in this paper includes the expansion-joint failure.

Regarding to the classification of damaged bridges based on their completed years, extent of damage, and structural types, it is found that over half of those damaged bridges were constructed before 1989, generally before the issue of “Design Code for Highway Bridges” in 1987. Moreover, over 95% of those injured bridges before 1989 were simply supported. The decreasing percentage of injured, simply-supported bridges after 1989 may be attributed to the decreasing of simply-supported bridges and better seismic design methods as well as construction technologies.

Table 2 Summary of comparison among 11 different damage modes

| Mode | Item | Number | Percentage (%) |
|---|---|---|---|
| 1 | collapse | 9 | 4.6 |
| 2 | deck | 51 | 26.2 |
| 3 | girder | 16 | 8.2 |
| 4 | bearing | 18 | 9.2 |
| 5 | pier cap | 12 | 6.2 |
| 6 | column / pier | 21 | 10.8 |
| 7 | foundation | 14 | 7.2 |
| 8 | abutment | 63 | 32.3 |
| 9 | approaching slab | 41 | 21.0 |
| 10 | land slide | 19 | 9.7 |
| 11 | Ancillary facilities | 69 | 35.4 |

Table 3 lists all the major damaged highway bridges in the inventory, the associated routes, and their primary damage modes. The collapse of the seven bridges was due to fault ruptures crossing the bridges directly. These collapsed bridges have attracted more attention from the researchers and government agencies, while a large number of bridges which experienced minor-to-moderate damage have not been well discussed and documented. For example, the Wu-Shi bridge with a recorded peak ground acceleration (PGA) of 388 gal is located across the Chelungpu fault, as shown in Fig. 1. The damage includes the span falling and pier damage at the third span. The first two spans from the northern abutment were unseated due to the ground movement. However, the right-side piers did not experience massive shear failure, but the bearings and concrete shear keys were severely damaged. The piers on the left side suffered major damage due to the large transverse forces transmitted from the superstructure without damaging the bearings and concrete shear keys. A similar failure mode is also found in the Don-Fong bridge (492 gal PGA). Figure 2 shows the relative movement of the spans and the sliding mechanism. The movement of the superstructure and the dislocation of the bearings have resulted in settlement between adjacent deck slabs. Similar examples of the sliding mechanism of the bearings were also observed for the Yen-Feng bridge (500 gal PGA) and Shin-Yi bridge (545 gal PGA). Under the strike of the Chi-Chi earthquake with high PGAs (greater than 500 gal), the superstructures were moved and some bearings were displaced, but the piers suffered only minor-to-moderate damage. The same damage to bearings was also observed in the Kuan-Lung bridge (439 gal PGA). Although abutment fill settlement occurred and the expansion joints were largely pulled out, the bridge column performed well during the earthquake.

Table 3 Major damaged highway bridges

| Name | Route | Year | Span (m) | Length (m) | Damaged mode |
|---|---|---|---|---|---|
| Shi-wei | Provincial 3 | 1994 | 25 | 75 | collapse |
| Chang-geng | local | 1987 | 25 | 300 | collapse |
| Dong-feng | Provincial 3 | 1962/1988 | 26 | 572 | girder/column |
| Pi-feng | local | 1991 | 25 | 300 | collapse |
| E-jian | County 129 | 1972 | 11 | 264 | collapse |
| Wu-shi | Provincial 3 | 1981/1983 | 34.7 | 624 | collapse/column |
| Mao-loh-shi | Provincial 3 | 1999 | 40~70 | 500 | column |
| Ming-tsu | Provincial 3 | 1990 | 25 | 700 | collapse |
| Ji-lu | local | 1999 | 150 | 300 | pylon/bearing |
| Tong-tou | County 149 | 1980 | 40 | 160 | collapse |
| Guang-long | local | 1986 | 28 | 56 | deck/abutment |
| Guan-de | local | 1977 | 20 | 60 | collapse |
| Bei-keng | County 129 | 1959 | 5.7 | 5.7 | deck/abutment |
| Long-an | County 129 | 1986 | 35 | 280 | column |
| Cheng-feng | County 136 | 1986 | 25.6 | 184 | column/abutment |
| Yan-feng | Provincial 14 | 1984 | 35 | 455 | column |
| Pu-ji | Provincial 16 | 1979 | 35 | 105 | pier cap |
| Hsing-shi-nan | County 127 | 1994 | 50 | 500 | column/bearing |
| Yan-ping | Provincial 3 | 1986 | 13 | 78 | abutment |
| Hsin-yi | Provincial 21 | 1981 | 29 | 180 | column |
| Long-men | Tou 53 | 1982 | 40 | 480 | collapse |
| Li-yu | Tou 53 | 1988 | 39 | 546 | bearing |
| Ping-lin | Tou 6 | 1969 | 25 | 500 | collapse/column |
| Mo-keng No.1 | Provincial 16 | 1996 | 14.6 | 14.6 | abutment |
| Mo-keng No.2 | Provincial 16 | 1996 | 40 | 40 | abutment |
| Da-feng | Chung 105 | 1992 | - | - | deck |


Fig. 1 (Left) Collapsed spans and damaged piers of the Wu-Shi bridge at the third span
Fig. 2 (Right) Sliding/friction mechanism of bearing of the Don-Fong bridge

The aforementioned six bridges with sliding mechanisms in the bearing systems suffered only minor to medium damage to the bridge column during the severe shaking. The damage pattern is significantly different from the major damage modes to bridges observed in the Loma Prieta and Northridge earthquakes. Although it is easy to understand that damage to a bridge results resulted from the high PGA and large ground distortion, the underlying causes of these phenomena may be understood by discussing the structure system and comparing the damaged components of bridges. Multiple-span simply supported bridges with PCI girders are the most commonly seen bridge type in the existing bridges of Taiwan constructed before 1990. The PCI girders are supported by the rubber bearings on the cap beam. According to the service purpose, rubber bearings can be considered as a hinge or a roller depending on the details of the bearing systems. For the hinge situation, one low-strength steel rod is installed in the center of the rubber bearing pad to limit the movement of the superstructure due to thermal effects, traffic load, or seismic force in the longitudinal direction. Besides, two concrete shear keys are also installed on each cap beam to prevent the girders from unseating in the transverse direction due to a large earthquake. Therefore, because of the difference of strength capacity and rigidity of the bearing system in the longitudinal and transverse directions, it is expected that the shear forces transferred from the bearing to the column will result in more damage in the transverse direction than that in the longitudinal direction. Also, it is concluded that the bearing damage can limit the damage to the columns in most circumstances. More importantly, from the point of view regarding the post-earthquake functionality, these bridges with slight damage to the columns can be quickly repaired in a very short time and can be used as emergency routes for rescue work.

## SEISMIC RETROFIT/RECONSTRUCTION STRATEGIES

Chi Chi earthquake caused severe damage on several bridges located in central Taiwan. The traffic break down due to the damage of bridges resulted from earthquake also interfered with the ongoing of rescue activity to a large extend, so how to retrofit the damaged bridges and recover the traffic flow in a quickest way became an urgent issue. Under the prerequisite that the traffic must be opened up within a limit time and the bridge can provide a higher degree of earthquake protection in the future, the recovery strategies include three levels: (1) immediately recovery strategies; (2) retrofit strategies and (3) reconstruction strategies.

### Immediately recovery strategies

After 921 Chi-Chi earthquake, damaged bridges are immediately recovered depend on the damaged levels, location and emergency conditions.

For the severe damaged bridge with nearby substitutive roads which still remain function, bridge was closed and emergent repairs or reconstructions were performed during closing period, such as Wu-shi bridge, Mao-luo-shi bridge, and Hsing-shi-nan bridge.

For the severe damaged bridge without substitutive roads around, temporary bypass or sidewalk was constructed by embedding RC or steel culverts, filling gravel and sand into cargo boxes, and building temporary steel bridge. Embed RC culvert is useful for a bridge located at a short span and middle-depth river. Short time recovery is the major advantage. Engineers can buy pre-cast RC culverts easily and embed them in the river, and then pave AC on the upper level to open to the traffic. However, massive sands and rocks accumulated in the river course are not beneficial for the runoff in the period of flooding. Shi-wei bridge (Fig. 3) is the typical example. With the same reasons of RC culverts, embed steel culverts are also used a lot for the immediate recovery. It should be noticed in the period of flooding. Tonng-tou (Fig. 4) is the typical example. Heap cargo boxes is adopted for bridges with high columns or bridges located at a short span and deep river. Compared to embed RC/steel culverts at the same place, saving time and space are its advantage. Engineers can fill the cargo boxes with gravel and sand first, and then heap them to adjust tilted bridge to the original height. Ming-tsu bridge (Fig. 5) is the typical example. Temporary steel bridge is adopted for bridges located at middle span and middle depth river. The benefit is to allow discharge, but the construction price is higher. E-jian bridge (Fig. 6) is the typical example.

As for the moderate damaged bridge, bridge was supported by steel frame, and truck weight and speed were limited. It is suggested to support the moderate damaged bridge with steel frames, if no substitutive roads can share its daily traffic. To prevent further damages, Engineers should investigate and repair the bridges, and limit the weight and speed of traveling cars in the emergent supported period. Tong-fong bridge (Fig. 7) and Hsin-yi bridgeare typical examples.

## Repair strategies

For minor damaged bridges, some necessary measures are applied to request sufficient time for the further repairing works. For example, CFRP jacket in Hsing-shi-nan bridge (Fig. 8), steel jacket in Mao-lou-shi bridge, and RC jacket in Yan-ping bridge and Cheng-feng bridge (Fig. 9) are ultilized. One special example of expanding cap beam width and adding columns was used in Yan-feng bridge. It is because longitudinal reinforcements were cutoff on the top surface of the columns. In addition, in Yan-Feng bridge (Fig. 10), because large dislocations occurred at pot bearings, box girder were removed to original place and bearings were replaced.

Fig. 3 Shi-wei Bridge

Fig. 4 Tong-tou Bridge

Fig. 5 Ming-tsu Bridge

Fig. 6 E-Jian Bridge

Fig. 7 Tong-Fong Bridge

Fig. 8 Shi-shi-nan Bridge

Fig. 9 Cheng-feng Bridge

Fig. 10 Yan-feng Bridge

**Reconstruction Strategies**

For a severe damaged or near collapse bridge reconstruction could be an economical choice than retrofitting or repairing. The primary concern following the earthquake was the need to reestablish these transportation routes that are vital to the local and global economy as soon as possible. Therefore, the reconstruction of all the damaged bridges had to be completed in a short time. At that moment after the earthquake occurred, the collection of earthquake data and revise of new design code were still proceeding. Reconstruction design was mostly in accordance with the experience of the other advanced countries. In addition, since the priority of this reconstruction work was the speed of construction and the upgrade of safety, but not the reduction of production cost, the basic consideration and principle of reconstruction design were slightly different from average cases. The basic consideration of reconstruction includes the following points:

(1) The severe damaged or near collapse bridges were reconstructed as soon as possible. Remaining parts of the damaged bridge with no obvious damages observed should be preserved temporary. However, in order to strengthen their seismic resistance and ensure an equal seismic capacity with the nearby bridges, they are suggested to be retrofitted in the future.

(2) Due to the limited time, the extent and level of bridge damages and the reconstruction method were determined based on visual inspection. During the construction, if new damages which may jeopardize the safety of bridges were found, other necessary retrofit or protective measures should be taken.

(3) In consideration of the feeling of the people living in disaster area, the major consideration of this reconstruction design was the upgrade of seismic resistance, but not the reduction of production cost.

The principle of reconstruction design includes the following points

(1). A peak ground acceleration of 0.33g was used to determine the minimum design horizontal seismic forces of these bridges. The value 0.33g is the maximum design ground acceleration regulated in “Seismic Resistant Design Code for Highway Bridges” of Taiwan at that time.

(2). In order to decrease dead loads of superstructure, thereby to reduce the inertial forces induced by earthquake, the preferred construction type for replacement bridges was the use of composite decks supported by continuous steel plate girders. By doing so, the construction time can be shorten as well.

(3). To reduce the potential of collapse span, continuous bridges with multi-span were adopted.

(4). To regularize seismic force distribution on different piers and provide an effective way to dissipate seismic energy conducted on bridges, internationally recognized seismic isolation devices were adopted.

(5). To avoid the original foundations and reduce the blocking of water flow, bridges with long span and fewer fundations were adopted.

(6). The design details should conform to the requirement of ductility design, so as to increase the inelastic deformation capacity of bridge system.

(7). Sufficient unseating prevention devices that can mitigate the risk of unseating of bridge span and absorb the shock induced by seismic forces should be installed.

**The main points of reconstruction design**

1. Selection of bridge site

(1) Due to the fact that most of the bridges which need to be reconstructed serve as important routes in central Taiwan, the need for recovering the flow of transportation was urgent and the time left for design was limit, to reselect bridge site became impractical. As a consequence, all these bridges were rebuilt on the original site, which is the quickest way to connect the original route.

(2) Chi-Chi earthquake was a result of the rupture of the Chelungpu Fault, a major reverse fault. Its magnitude and the damage caused was so tremendous that it's rarely seen in hundreds year. Therefore, the probability to induce a major fault rupture (displacement due to the movement of tectonic plates) at the same location in a short time is not high, even though its probability to suffer from the ground shaking of earthquake is still high. As such, the bridges were reconstructed at the original site.

2. Use of seismic isolation devices

In view of the reconstruction experience of United States and Japan in the aftermath of Northbridge earthquake and Kobe earthquake, respectively, seismic isolation devices were adopted to reduce the transmission of the earthquake motion to the structure. Among the various types of isolation system, Lead Rubber Bearing (LRB) and High Damping Rubber Bearing (HDR) were most commonly and had been matured enough for wide-ranging use in the earthquake protection of bridges. Consequently, these two types bearing were adopted in this retrofit construction.

3. Sufficient support length and unseating prevention device

The most economical and effective way to prevent the unseating of bridge span is to provide sufficient support length and install appropriate unseating prevention device. In order ensure that the unseating of bridge span will not happen, in addition to the support length that must be provided, at least two types of unseating prevention devices, such as restrainer and stopper, must be installed at the ends of a superstructure. Restrainer is to connects the adjoining superstructure together to prevent large relative displacement between two adjoining superstructure. Stopper is installed on the top of pier cap to prevent large relative displacement between superstructure and substructure.

## DEVELOPMENT OF THE SEISMIC RETROFIT MANUAL FOR HIGHWAY BRIDGE

The "Seismic Retrofitting Manual for Highway Bridges" was firstly published by the Ministry of Transportation and Communications in 1999. The 2009 edition expands upon the previous publication by including procedures for performance-based seismic evaluation and retrofitting of bridges, as well as basic principles for evaluating and retrofitting scoured bridges with an exposed foundation structure. The revised manual maintains the basic format of the retrofitting process described in the 1999 manual. However, major changes were made to include current advances in earthquake engineering, field experience with retrofitting highway bridges, and the performance of bridges in recent earthquakes as well as the 1999 Chi-Chi earthquake. The revised manual is comprised of seven chapters as follows:

Chapter1: Seismic retrofitting of Highway bridges
Chapter2: Seismic ground motion hazard
Chapter3: Seismic simplified assessment methods and prioritization
Chapter4: Seismic detailed assessment methods for existing bridges
Chapter5: Seismic retrofitting by using system approach
Chapter6: Seismic retrofitting by using component approach
Chapter7: Special issue on souring and exposed foundation

Chapter 1 provides a complete overview of the retrofitting process including the philosophy of performance-based retrofitting, a characterization of the seismic and geotechnical hazards, and summaries of recommended simplified and detailed assessment procedures, retrofit strategies, and approaches and measures based on a system or component -by-component approach perspective. Performance levels PL0, PL1, PL2 and PL3, (Table 4) are recommended according to bridge importance (Table 5), safety, serviceability, and reparability (both short and long period), with a more rigorous performance being required for important, relatively new bridges, and a lower level for standard bridges (Table 6). A higher level of performance is required for event of design earthquake than for the frequent earthquake. Table 7 shows the assessment methods according to the performance level and regularity.

Chapter 2 characterizes the seismic and geotechnical hazards. Basically, the seismic demand shown in Fig. 11 is the same as the current "Seismic design specification of bridge structures" published in 2009, except for disregarding the maximum considerable earthquake level with a 2500 return period. Structural performance, especially nonlinear deformation capability, is mainly verified according to a design earthquake level with 475 return period, which is a design earthquake level corresponding to a peak ground acceleration of $0.4S_{DS}$; while the frequent earthquake level linearly reduces the force demand from the previous level by dividing it by 3.25. Retrofitted bridge should display good ductility under the design earthquake level and simultaneously remain elastic under the frequent earthquake level.

Table 4 Performance matrix of the retrofitting bridge

| Performance level | Safety | Serviceability | Reparability | |
|---|---|---|---|---|
| | | | Short period | Long period |
| PL3 | Structure remains elastic and no unseating. | As same as prior to the earthquake | Simply repair | Regular repair |
| PL2 | Limited damages and no unseating. | Repair in short time | Repair in short time by conventional approach | Repair in long time by conventional approach |
| PL1 | Limited residual inelastic deformation and no unseating | Repair in short time with limited vehicle weight and speed | Replace or retrofit the damaged member | Close and partially rebuild the bridge |
| PL0 | Collapse prevention and no unseating | Close and use alternative route | Tore down or partially rebuild | Tore down or partially rebuild |

Table 5 Important factor of the bridge

| Bridge type | Important factor |
|---|---|
| Highway bridge | 1.2 |
| Bridge is on the main route, or bridge is above important infrastructure, or bridge is the only one connection between two towns | 1.2 |
| others | 1.0 |

Table 6 Performance level for standard/regular or important bridge

| Seismic hazard level | Published year of the Design Specification of highway bridge structures | | |
|---|---|---|---|
| | 1995 and 2000 | 1960 and 1987 | Before 1960 |
| Frequent earthquake | PL3 | PL3 | PL3 |
| Design earthquake | PL2 | PL1 | PL0 (PL1*) |

*Note: for important bridge

Table 7 Seismic assessment methods

| Performance level | Regular bridge | Irregular bridge |
|---|---|---|
| PL3 | Linear elastic/dynamic, or Nonlinear elastic/dynamic | Linear / Nonlinear dynamic |
| PL0, PL1, and PL2 | Nonlinear elastic/dynamic | Nonlinear dynamic |

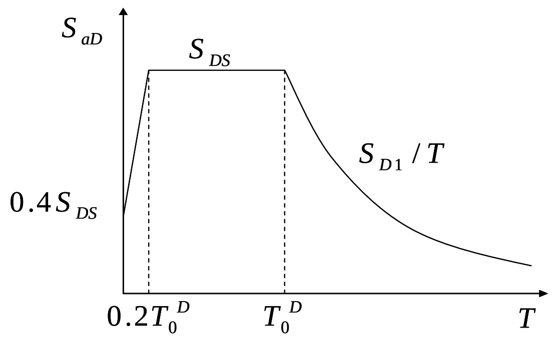


Fig. 11 Elastic acceleration response spectrum as the seismic demand for bridge to be retrofitted

Chapter 3 provides two simplified seismic assessment methods. Engineers are encouraged to carry out field investigations using the tables provided in Method A to examine both the unseating probability and strength capacity. Method B is an in-house task that using fragility curve to determine the damage probability of a bridge in terms of collapsed, severe, moderate, slight and none for 12 kinds of bridges. For method A, there is no need to perform a detailed assessment if the score is smaller than 30 but it is mandatory to carry out a detailed assessment as described in Chapter 4 when the score is larger than 60. For the case whose score is between 30 and 60, engineers are also encouraged to do the detailed assessment so that to check the structure safety, followed by the seismic assessment and retrofitting strategy, as shown in Fig. 12. The evaluation results from either method is applied to calculate the strength-ductility and falling index for prioritizing bridges that need to be retrofitted.

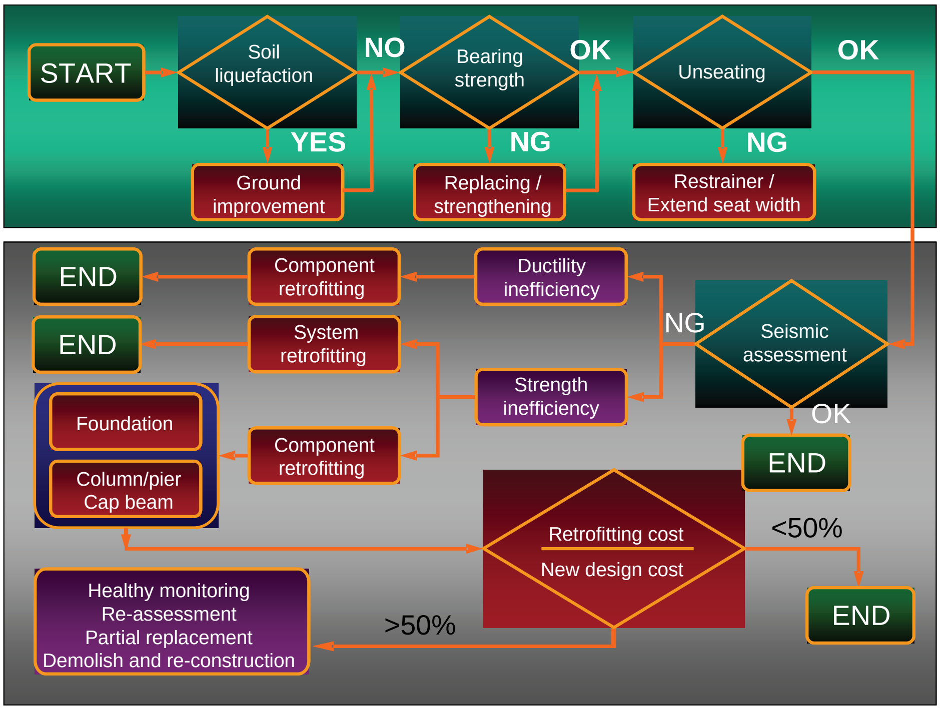


Fig. 12 Seismic assessment and retrofitting strategy

Chapter 4 discusses the modeling of the bridge in detail, for the superstructure, bearing system, substructure, and the foundation. The soil spring model for spread footings, pile foundations and caissons are also introduced, since the soil profile is both required and important for any seismic evaluation. If the bridge foundations are located in the river, it is necessary to consider the scouring effect and exposed foundations. In order to estimate a reasonable behavior of the structure, the revised manual allows engineers to use either a linear static, nonlinear static, nonlinear static or nonlinear dynamic method, according to the seismic hazard level and the regularity of the bridge. Among those four methods, the pushover analysis by the definition of plastic hinges and the modified capacity spectrum method are proposed as the standard procedure to obtain the yielding and anticipated ultimate peak ground acceleration. It is recommended to apply the Kawashima model and the Mander model for bridges before and after retrofitting, respectively. The pushover curve is transformed to capacity spectrum as recommend in the ATC-40, however, in stead of finding a performance point, in this manual, a revised ATC-40 procedure (Fig. 13), without dealing with iteration and converge problem, is applied to obtain the peak ground acceleration corresponding to the top displacement of the column or girder. Since performance level, PL1 for example, has been selected as defined in Chapter1, if the PGA with respect to PL3 is smaller than $S_{DS}/3.25$, the bridge need a strength retrofitting. In addition, if the seismic demand, 0.4 $S_{DS}$, is larger than the PGA of PL1, it is required to make ductility retrofitting.

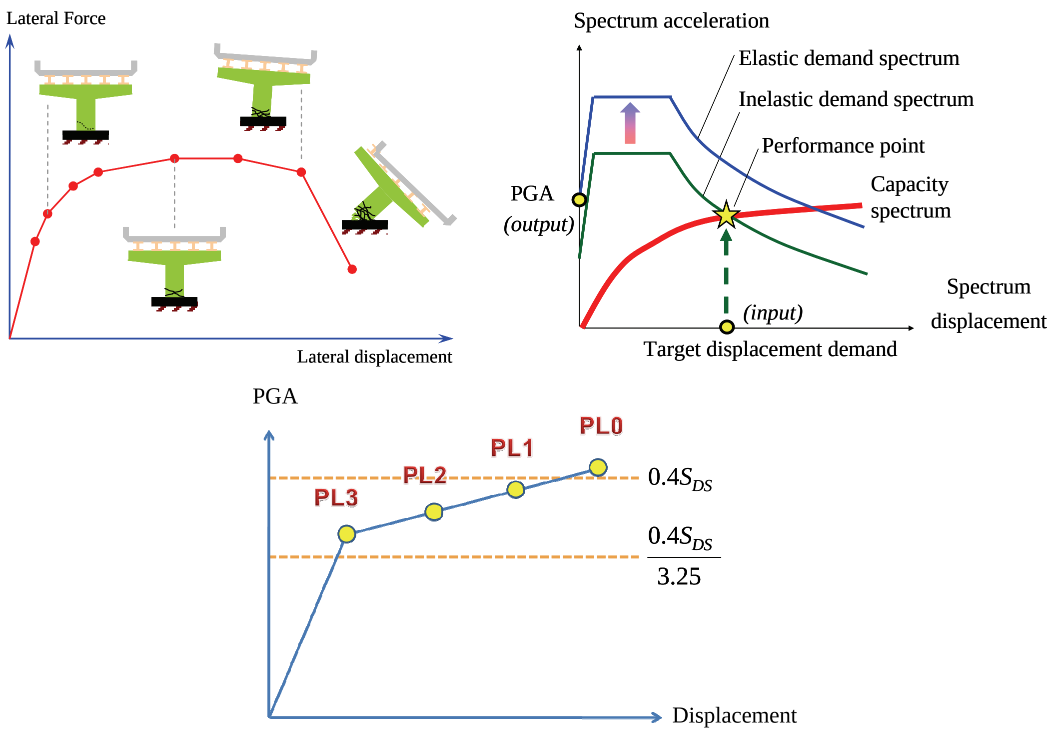


Fig. 13 Seismic assessment procedure and qualification requirement

Chapter 5 describes the approach to retrofit a complete bridge by following four methods: (1) equalize the inertial force distribution; (2) optimize the existing bearing system; (3) use isolation bearings; or (4) add dampers. In addition, it is important to avoid girders from falling. To prevent this, the unseating retrofitting method is introduced by extending the bearing-seat length on the cap beam if there is sufficient space, or by adding devices to prevent unseating, such as for example, restrainers connecting adjacent girders. Figure 14 shows an example using viscous damper in a highway bridge.

Chapter 6 presents the component-by-component retrofitting approach. A brief summary of the retrofitting measures is shown in Fig. 15. For the bridge columns, based on the ultimate strain theory, engineers can apply specific formulas as proposed in the notes of this manual to calculate the thickness for steel, FRP, and concrete jacketing. As a result, after retrofitting the shape of a rectangular column becomes an elliptical section to ensure that the confining stress can be developed. In addition, the design procedure for strengthening the footing can be found similar to that the design procedure referenced in the “Seismic Retrofitting Manual for Highway Structures” of the 2006, Federal Highway Administration publication.

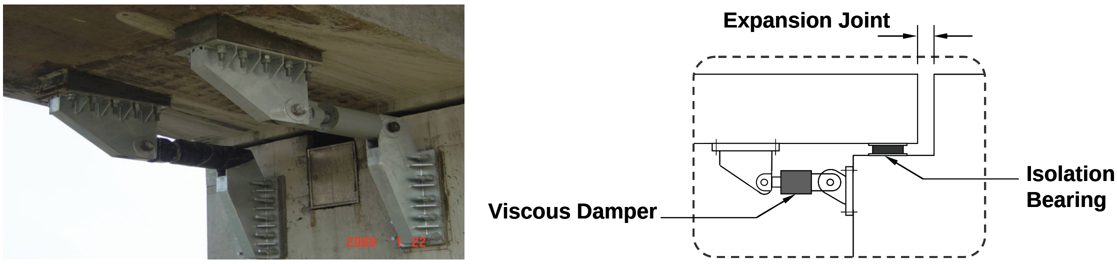


Fig. 14 Seismic retrofitting by using system approach: isolation bearing and external damper

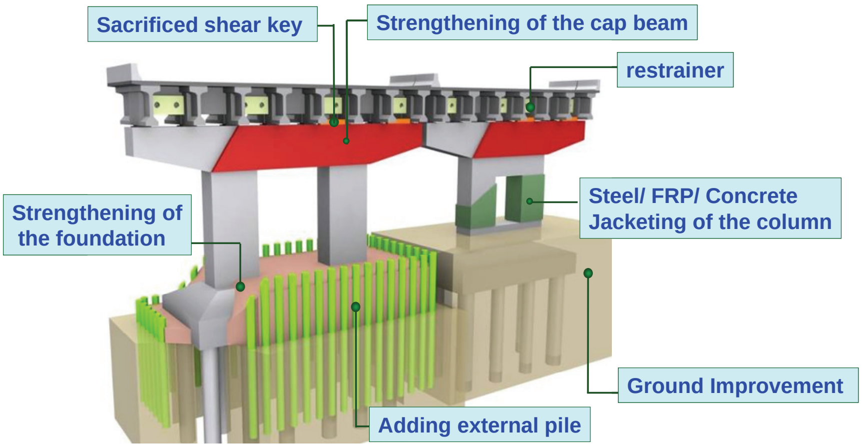


Fig. 15 Seismic retrofitting by using component approach

Chapter 7 deals with scouring-induced seismic safety evaluation and retrofitting measures. Considering the difficulties due to the variation in river channel and soil profiles, a simplified procedure using a reduction factor as a nonlinear function of the remaining length of the pile or caisson is proposed for a quick assessment of the structural performance.

## CONCLUSIONS

The Chi-Chi earthquake brought an intense impact on the seismic resistant design and technical development of bridge engineering. In this paper, the performance of highway bridges located in the catastrophic region and the extent of damage to them was summarized, followed by a general depiction of the seismic retrofitting/reconstruction strategies after the earthquake. The revised “Seismic Retrofitting Manual for Highway Bridges” proposed recently was also briefly described. This manual refers to the damage investigation and reconstruction experiences after Chi Chi earthquake and also includes current advances in earthquake engineering, field experience with retrofitting highway bridges.

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