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工學院專班營建技術與管理組

碩 士 論 文

既存橋梁結構之耐震能力評估

A Study on Evaluation of Seismic Capacity for Existing Bridges

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摘　　要

九二一集集地震後，國內橋樑設計規範對於耐震力的要求作了相關的修正，於修正規範中新建橋樑要求在地震發生時，需要有較舊設計規範佳的耐震能力。以公眾安全的角度而言，修正後之耐震規範的確可以增加橋樑在地震發生時的可信賴度，然而在經濟性的考量上，新規範所產生的較保守設計，就可能增加額外的建造成本，因此本研究對於新修正的耐震設計規範進行探討。此外，針對依據舊規範所建造之既有橋樑，在面對新的規範地震力要求時，是否有足夠的耐震能力，也是另一個重要的探討議題。本研究採用位移設計法來評估既有橋樑耐震能力，依據墩柱的位移韌性能力，檢視橋樑於地震發生時是否有足夠韌性，以避免發生非預期之橋樑損壞，以保障大眾運輸安全。

A Study on Evaluation of Seismic Capacity for Existing Bridges

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ABSTRACT

The seismic design specification has been revised to meet the increased seismic loads requirement / demand in which all the new bridges are designed to higher seismic capacity after the event of Chi-Chi Earthquake. For ensuring public safety and increasing reliability of the bridge, the revised seismic design specification has adopted a more conservative design approach. Therefore, it would increase additional constructing cost. This study is to assess the related modifications to reflect the revision seismic design specification and another significant matter is to analyze whether the existing bridge employing the previous seismic specification has an adequate seismic capacity to meet the revised seismic demand. In this report, the methodology of displacement-based approach is adopted for the evaluation of existing bridges. The displacement ductility capacity of piers is employed to estimate whether the bridge has an adequate ductility meeting the safety requirement of public transportation during earthquake.

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1 INTRODUCTION

1.1 Motive

Bridges are constructed island-wide in Taiwan for its drastically varied geographies. They have been widely used in various aspects, including pedestrians, vehicles and railway. Indeed, bridges are among the most critical structures in traffic infrastructure.

The design requirement for bridges is very stringent as it directly regards public safety. The development of the design code for bridges has been taken seriously in Taiwan over the past two decades. In 1987, Ministry of Transportation and Communications (MOTC) first issued the Design Specification for Highway Bridges [1]. And in 1995, the seismic design provisions have been revised [2] to comply with the most updated knowledge in earthquake engineering. A few years later in response to the impact of 1999 Chi-Chi Earthquake, the seismic design criteria are further enhanced by MOTC with the current version of Seismic Design Specifications for Highway Bridges [3] in 2000 (referred to as SDSHB 2000). While new bridges are to be designed in compliance with SDSHB 2000, safety concerns for those designed earlier based on the 1995 code and other foreign design codes but constructed after 2000, such as those for the Taiwan High Speed Rail, need to be justified.

1.2 Purpose

To verify if the seismic capacity of the as-built bridge structures meets the current seismic demand, this study first identifies the differences between the previous seismic design specification and SDSHB 2000, and assess the impact of the previously designed bridges based on SDSHB 2000. Next, the design concepts and theories of SDSHB 2000 and other international seismic codes adopted worldwide are explored to justify the adequacy of the original design. Finally, an approach for the re-evaluation of seismic capacity and performance of existing bridges is to be proposed.

2 SEISMIC DESIGN SPECIFICATION FOR HIGHWAY BRIDGES

2.1 The Previous Seismic Design Specification

In the early bridge design specifications of Taiwan, the seismic design guidelines for bridge structures were only presented briefly in conceptual statements. Not until 1995 did the MOTC publish the first Seismic Design Specifications for Highway Bridges after conducting a series of studies in reference to the development of Japanese and USA seismic codes. The basic design philosophy implied by the specification is to make sure the bridge structure kept in the elastic range during moderate earthquakes while allowing them to fail in a ductile pattern without collapse during severe earthquakes.

2.2 Static Analysis Method

For regular bridge structures, a static analysis approach can be used, while for irregular bridges, a dynamic analysis procedure of either spectrum analysis or time history analysis should be adopted. Regular bridges refer to those of six spans or less, no abrupt or unusual changes in mass, stiffness or geometry, and no significant variations in these parameters from span to span or pier to pier. For regular bridges, the equivalent static analysis method can be applied to calculate the design seismic forces for structural analysis.

2.3 Static Seismic Design Forces

To determine member forces due to earthquakes, the minimum total design horizontal force, V, shall be calculated as the following:

$$V = Z_d CW = \frac{ZICW}{1.2\alpha_y F_u} \tag{2.1}$$

where

Z_d : the design horizontal ground acceleration coefficient

C : the normalized acceleration response spectrum coefficient

W : total dead weight of the bridge unit including the weight of the superstructure and pier

Z : the horizontal ground acceleration coefficient

I : the important factor

α_y: the ratio of the ground acceleration expected to initiate yielding in the structure to the design ground acceleration

F_u : the seismic force reduction factor for bridge system

Note that, in equation (2.1), the ratio $\frac{C}{F_u}$ should be regulated by inequality (2.2) as

$$\frac{C}{F_u} \leq \begin{cases} 1.2(R* = 2.0) \\ 1.1(R* = 3.0) \\ 1.0(R* = 5.0) \end{cases} \tag{2.2}$$

where $R*$ is the property factor for structural system associated with types of substructure as shown in **Table 2.3**. For short-period bridge, the seismic force will be magnified due to smaller F_u associated with period. But consideration of soil-structure interaction effect for the short-period bridge, the soil spring has more deformation induces the higher damping ratio. Hence the limitation of inequality is adopted for actual calculation.

As a result, the design earthquake force is modified as

$$V = \frac{ZI}{1.2\alpha_y}\left(\frac{C}{F_u}\right)_m W \tag{2.3}$$

in which $\left(\frac{C}{F_u}\right)_m$ is the modification of acceleration response spectrum coefficient regulated by inequality (2.2).

2.3.1 Horizontal Ground Acceleration Coefficient

The horizontal ground acceleration coefficient, Z, is represented for the ratio of the seismic ground acceleration of 475-year return period of to the gravitational acceleration, g. Taiwan is categorized into 4 seismic zones respectively with coefficients 0.33, 0.28, 0.23 and 0.18. The seismic zoning map is illustrated in **Figure 2-1**.

2.3.2 Important Factor

For critical bridge structures that need to maintain their function immediately after an earthquake event, I of 1.2 should be applied. Otherwise I of 1.0 is suggested.

2.3.3 Normalized Acceleration Response Spectrum Coefficient

The normalized acceleration response spectrum coefficients are expressed in terms of fundamental period and soil profiles as tabulated in **Table 2-1** while the normalized vertical acceleration response spectrum coefficients are tabulated in **Table 2-2**.

Soil profiles may be classified in accordance with the fundamental period of the site, T_G, into three types:

Type I .... $T_G \leq 0.2$ sec

Type II .... 0.2 sec < $T_G \leq 0.6$ sec

Type III .... 0.6 sec < T_G

The fundamental period of the site, T_G, can in turn be estimated by the following equation:

$$T_G = 4\sum_{i=1}^{n}\frac{H_i}{V_{si}}$$

where

H_i: the thickness (m) of the i-th subsoil layer

V_{si} : the shear wave velocity (m/s) of the i-th subsoil layer at low strains

n : the number of layers above the base layer

It is recommended that shear wave velocities be directly measured via site investigations. In the absence of measured values, shear wave velocities may be obtained by using empirical formulae based on the Standard Penetration Test N-value as the following, For cohesive soils : $V_{si} = 100\ N_i^{1/3}$ $(1 < N_i < 25)$ (m/s)

For sandy soils : $V_{si} = 80\ N_i^{1/3}$ $(1 < N_i < 50)$ (m/s)

The base layer is defined as the layer under which all lower layers have an N-value greater than 25 for cohesive soils, or 50 for sandy soils.

2.3.4 α_y and Seismic Force Reduction Factor for Bridge System

The amplification factor of the design earthquake load, α_y, takes into account the fact that initial yielding of the bridge structure commences as the actual seismic force reaches α_y times of the design earthquake load. The value of α_y is dependent on the types of bridge structures as well as the design methods adopted. For steel bridges, α_y of 1.7 is considered. For reinforced concrete bridges, α_y of 1.65 is used as the USD approach adopted and 1.9 as the WSD approach adopted.

The seismic force reduction factor for a bridge system, F_u is related to the ductility capacity, R, of the structural system, the period of the structure, T, and the soil profiles. The relationships between the ductility capacity, R, the property factor, R* and the allowable ductility capacity, R_a, are as following:

$$R = \frac{R^*}{1.2} \tag{2.4}$$

$$R_a = 1 + \frac{(R-1)}{1.5} \tag{2.5}$$

where R^* is dependent on the type of substructure as tabulated in **Table 2-3**.

The values of F_u are dependent on the type of soil profile as shown in the following:

(a) Soil Type I

$$F_u = R_a$$

$$F_u = \sqrt{2R_a - 1} + \left(R_a - \sqrt{2R_a - 1}\right)\frac{(T - 0.242)}{0.091} \qquad 0.242\text{ sec} \le T \le 0.333\text{sec}$$

$$F_u = \sqrt{2R_a - 1}$$

$$F_u = \sqrt{2R_a - 1} + \left(\sqrt{2R_a - 1} - 1\right)\frac{(T - 0.15)}{0.12} \qquad 0.03\text{sec} \le T \le 0.15\text{ sec}$$

$$F_u = 1.0$$

(b) Soil Type II

$$F_u = R_a$$

$$F_u = \sqrt{2R_a - 1} + \left(R_a - \sqrt{2R_a - 1}\right)\frac{(T - 0.308)}{0.157} \qquad 0.308\text{sec} \le T \le 0.465\text{sec}$$

$$F_u = \sqrt{2R_a - 1}$$

$$F_u = \sqrt{2R_a - 1} + \left(\sqrt{2R_a - 1} - 1\right)\frac{(T - 0.15)}{0.12} \qquad 0.03\text{sec} \le T \le 0.15\text{ sec}$$

$$F_u = 1.0$$

(c) Soil Type III

$$F_u = R_a$$

$$F_u = \sqrt{2R_a - 1} + \left(R_a - \sqrt{2R_a - 1}\right)\frac{(T - 0.406)}{0.205} \qquad 0.406\text{sec} \le T \le 0.611\text{sec}$$

$$F_u = \sqrt{2R_a - 1}$$

$$F_u = \sqrt{2R_a - 1} + \left(\sqrt{2R_a - 1} - 1\right)\frac{(T - 0.2)}{0.17}$$

F_u =1.0

(d) Taipei Basin

$$F_u = R_a$$

$$F_u = \sqrt{2R_a - 1} + \left(R_a - \sqrt{2R_a - 1}\right)\frac{(T - 0.8)}{0.6} \qquad 0.8\text{sec} \le T \le 1.4\text{ sec}$$

$$F_u = \sqrt{2R_a - 1}$$

$$F_u = \sqrt{2R_a - 1} + \left(\sqrt{2R_a - 1} - 1\right)\frac{(T - 0.2)}{0.17} \qquad 0.03\text{sec} \le T \le 0.2\text{sec}$$

F_u =1.0

2.3.5 Distribution of Seismic Forces of the Bridges

The seismic force per unit length, $p_e(x)$, applied along the bridge is calculated as:

$$p_e(x) = Fw(x)U(x) \tag{2.6}$$

where

$$F = \frac{V}{\int w(x)U(x)dx} = \frac{V}{\beta} \tag{2.7}$$

$w(x)$: weight per unit length

$U(x)$:   displacement

Furthermore, it is to define a minimum horizontal seismic design force, V*, that is required to avoid early yielding of the bridge under moderate earthquakes. It is calculated as the following:

$$V^* = \frac{ZIF_u}{3.0\alpha_y}\left(\frac{C}{F_u}\right)_m W \qquad (2.8)$$

2.3.6 Vertical Seismic Forces

No vertical seismic force is considered in the bridge design.

2.4 Modification of Seismic Design Specifications after Chi-Chi Earthquake

In response to the disaster of Chi-Chi Earthquake, MOTC revised the Seismic Design Specifications for Highway Bridges in April 2000. Major changes in the specifications are modifications of horizontal ground acceleration coefficients and the consideration of vertical seismic force.

2.4.1 Horizontal Ground Acceleration Coefficient

In the previous design code, the horizontal ground horizontal acceleration coefficients are classified into 4 grades corresponding to the 4 seismic zones of Taiwan, as described in Section 2.3.1. Nevertheless, after Chi-Chi Earthquake it has been reduced to only two design levels with Z=0.23 and 0.33 respectively. See **Figure 2-2** for the seismic zoning map. Furthermore, the maximum amplification factor of the normalized acceleration response spectrum for Taipei Basin is increased from 2 to 2.5.

2.4.2 Vertical Seismic Force

In the previous seismic design code, no vertical seismic force is considered for the bridge structure but only the bearing facility. In the revised version, the effect of vertical ground acceleration is taken into account [4]. The vertical seismic force, V_v, should be calculated as the following:

$$V_v = \frac{Z_v I C_v W}{1.2\alpha_y F_{uv}} \tag{2.9}$$

where

Z_v : the ground vertical acceleration coefficient which is $\frac{2}{3}$Z in Zone A and $\frac{1}{3}$Z in Zone B.

C_v : the normalized vertical acceleration response spectrum coefficient

F_{uv} : the vertical seismic force reduction factor for bridge system. The values of F_{uv} are shown in the following:

(a) Soil Type I

$F_{uv} = R_a$

$F_{uv} = \sqrt{2R_a - 1} + \left(R_a - \sqrt{2R_a - 1}\right)\frac{(T - 0.194)}{0.094}$ 0.194 sec≤T≤0.288sec

$F_{uv} = \sqrt{2R_a - 1}$

$F_{uv} = \sqrt{2R_a - 1} + \left(\sqrt{2R_a - 1} - 1\right)\frac{(T - 0.1)}{0.07}$ 0.03sec≤T≤0.1 sec

$F_{uv} = 1.0$

(b) Soil Type II

$F_{uv} = R_a$

$F_{uv} = \sqrt{2R_a - 1} + \left(R_a - \sqrt{2R_a - 1}\right)\frac{(T - 0.252)}{0.151}$ 0.252sec≤T≤0.403sec

$F_{uv} = \sqrt{2R_a - 1}$

$F_{uv} = \sqrt{2R_a - 1} + \left(\sqrt{2R_a - 1} - 1\right)\frac{(T - 0.1)}{0.07}$ 0.03sec≤T≤0.1 sec

$F_{uv} = 1.0$

(c) Soil Type III

$$F_{uv} = R_a$$

$$F_{uv} = \sqrt{2R_a - 1} + \left(R_a - \sqrt{2R_a - 1}\right)\frac{(T - 0.315)}{0.215} \qquad 0.315\text{sec} \le T \le 0.53\text{sec}$$

$$F_{uv} = \sqrt{2R_a - 1}$$

$$F_{uv} = \sqrt{2R_a - 1} + \left(\sqrt{2R_a - 1} - 1\right)\frac{(T - 0.1)}{0.07} \qquad 0.03\text{sec} \le T \le 0.1\text{sec}$$

$$F_{uv} = 1.0$$

(d) Taipei Basin

$$F_{uv} = R_a$$

$$F_{uv} = \sqrt{2R_a - 1} + \left(R_a - \sqrt{2R_a - 1}\right)\frac{(T - 0.71)}{0.61} \qquad 0.71\text{sec} \le T \le 1.32\text{ sec}$$

$$F_{uv} = \sqrt{2R_a - 1}$$

$$F_{uv} = \sqrt{2R_a - 1} + \left(\sqrt{2R_a - 1} - 1\right)\frac{(T - 0.1)}{0.07} \qquad 0.03\text{sec} \le T \le 0.1\text{sec}$$

$$F_{uv} = 1.0 \qquad T \le 0.03\text{sec}$$

Definitions of other parameters are the same as in Section 2.3.

Note that, in equation (2.9), the ratio $\frac{C_v}{F_{uv}}$ should be regulated by inequality (2.10) as：

$$\frac{C_v}{F_{uv}} \le \begin{cases} 1.2(R^* = 2.0) \\ 1.1(R^* = 3.0) \end{cases} \qquad (2.10)$$

As a result, the design earthquake force is modified as

$$V_v = \frac{Z_v I}{1.2\alpha_y}\left(\frac{C_v}{F_{uv}}\right)_m W \tag{2.11}$$

in which $\left(\frac{C_v}{F_{uv}}\right)_m$ is the modification of acceleration response spectrum coefficient regulated by inequality (2.10).

2.4.3 Prevention of Loss of Span

To prevent loss-of-span, both seismic design specifications suggest applying anti-fall-off devices. In the previous seismic code, the device is identified as the second level protection after the bearings is damaged by horizontal shear force so the yielding design strength of the anti-fall-off devices is two times of the design capacity of bearing, where the horizontal design shear force associated with plastic moment is applied to design bearings. In SDSHB 2000, the yielding design strength of anti-fall-off devices is equal to the dead load applied to the bearing. It supposes that the superstructure can be hold by this device when loss of span occurring.

2.5 Influences of the Modified Specifications on Bridge Structure Design

In accordance with the Design Specifications of Highway Bridges, the design loads of various combinations are to be considered. Among which, those related to the seismic force, EQ, are usually most dominant, especially for the design of substructure. As previously discussed, by the modified seismic specification, the design demands of bridges are increased.

2.5.1 The Demand on Pier Section

The difference on the design horizontal seismic forces between the original code and the revised one is mainly attributed to the change of the horizontal ground acceleration coefficient, *Z*. It could be increased by as much as 83%. Take the sites in the Jiu-Ru, Ping-Dong County for example, the horizontal seismic force by Eq. (2.1) of the previous specification is:

$$V_{previous} = \frac{ICW}{1.2\alpha_y F_u} \times 0.18$$

While by the revised version, it is:

$$V_{revised} = \frac{ICW}{1.2\alpha_y F_u} \times 0.33$$

The design horizontal seismic force is drastically increased by the revised design specification.

Besides, in the previous version, the overall design seismic loads for the bridge structure are to be determined from the combinations of the two orthogonal horizontal seismic forces as:

$$|S_x| + 0.3|S_y|$$

$$|S_y| + 0.3|S_x|$$

where S_x, S_y are the horizontal seismic forces in the longitudinal and transverse directions of the bridge. Symbol | | denotes the absolute value or magnitude of the force or moment.

In the revised specification, an additional vertical seismic force is considered in the loading combination as the following:

$$|S_x| + 0.3|S_y| + 0.3|S_Z|$$

$$|S_y| + 0.3|S_x| + 0.3|S_Z| \qquad (2.12)$$

$$|S_z| + 0.3|S_y| + 0.3|S_x|$$

where S_z denotes the vertical seismic force. It is evident that an additional seismic force of $0.3|S_z|$ is considered.

For the design of pier section, various loading combinations were employed as demands to check the pier's section nominal capacity that is described as a axial force-moment interaction curve, as shown in **Figure 2-3.** The curve area shall cover the points of demands of axial forces and moments induced by various loading combinations. The loading combination with earthquake force experientially is to be the main demand on the pier' section design. When the increment of horizontal seismic force and additional axial force induced by he vertical seismic force are considered, the revised demands will affect the design of the pier section.

2.5.2 The Design of Superstructure

The various loading combinations are also employed to design the superstructure's section. In accordance with the pervious design experiences, the loading combination with earthquake force is not the main controlling demand for the superstructure design. However, when a new seismic combination, $|S_z| + 0.3|S_y| + 0.3|S_x|$, is considered, it would impose a significant effect on the design of the superstructure, in particular the increase of the depth of bridge girders.

2.5.3 Plastic Moments

In accordance with the concept of ductility design, the bridge piers during severe earthquakes are to resist the seismic forces inelastically via a yielding process without collapse. To ensure forming of the plastic hinges in the piers prior to damage of other structural members or bearings, the design capacity of other bridge members should be larger than a certain values determined by the plastic moment strength, M_p, of the piers at the plastic hinge locations. In the seismic design specifications, plastic moment is associated with the cross sectional nominal moment strength of the pier with consideration of a certain safety factor, say 1.3, for reinforced concrete bridges.[3] When calculating the nominal moment strength associated with M_p, the seismic design specification further demands that the designed axial load of the pier includes not only the dead load but also the axial load induced by earthquake. For piers in a single-column form, the earthquake-induced axial force is minor and can be neglected by the previous specification as it considers only the horizontal seismic force. However, with the design vertical seismic force required by the revised specification, additional axial load to the piers is introduced, which in turn affects the axial force–moment interactive behaviour of the reinforced concrete columns. As a result, the design capacity of the substructure may have to be enhanced upon increase of the axial forces.

2.6 Summary

The design of bridges is more demanding by SDSHB 2000, which emphasizes the ductility design with enhanced horizontal seismic force for most seismic zones as well as additional consideration of vertical seismic force, as compared with those by the earlier version. The increment of plastic moment demand due to consideration of vertical seismic load is acknowledged. To avoid an economically irrational design with the ductility-based design concept adopted, the following practical concerns need to be addressed:

(a) The requirement of vertical seismic force in bridge design regardless of seismic area is controversial and debatable. Why and specifically where consideration of vertical seismic force is necessary needs to be reasonably defined.

(b) The allowable ductility capacity is critical to seismic design of bridges. The ductility capacity of bridge piers suggested by SDSHB 2000 is associated only with the types of substructures. However, consideration of types of substructure alone is too simple to reflect the actual ductile behavior of the bridge columns. As a result, either reliability or economy of the design is not warranted.

(c) The bearings, based on the ductility design concept, should be strong enough to transfer as much seismic forces as required to form plastic hinges in the piers. This, however, might not be always achievable and could lead to other unexpected failure modes.

3 DIFFERENT CONCEPTS IN SEISMIC BRIDGE DESIGN

The conventional seismic design approach of bridges is based on a capacity design concept with the capacity defined in terms of strength to ensure integrity of the bridges under design earthquakes, while reserving sufficient ductile capability of the piers to avoid collapse of the bridges in severe earthquakes. The SDSHD2000 of Taiwan adopts such a force-based design approach that determines the seismic force levels from the acceleration spectra. This method is initially developed for building structures. The structural characteristics of bridges, however, are intrinsically different from bridges in a sense that buildings extend vertically in space while bridges extend horizontally. Whether or not the force-based approach for seismic design of buildings is adequate for bridges is questionable. Recently, a displacement-based seismic design approach of Caltrans Seismic Design Criteria (SDC) [5] is developed where the displacement levels are determined from the ductility capacity of the bridge piers. SDC is the currently-in-practice code of seismic design and analysis methodologies for the design of new bridges in California, USA. It adopts a performance-based approach specifying minimum levels of performance for structural system as well as components.

The background in the development of SDC in California is similar to that of SDSHD in Taiwan. It has shown from the past earthquakes of California that structures designed in accordance with non-ductile design standards are seismically vulnerable. As a result, Caltrans has embarked on an extensive seismic retrofit program to strengthen the existing bridges to ensure satisfactory performance of the bridges during anticipated future earthquakes. The concept and methodology of the displacement-based approach of SDC will be reviewed and discussed herein.

3.1 Design Philosophy

The seismic design philosophy based on the ductility of flexible structural members has been accepted worldwide. Ductility is defined as the ratio of the ultimate deformation to the deformation at yield of the primary structural member. Ductile response of structural components is characterized by the hysteretic loops of forces with respect to inelastic deformations in cycles without significant degradation of strength or stiffness. The area enclosed by the hysteretic loops represents the energy dissipated during the inelastic deformation process of the member. Structures with sufficient ductility are more earthquake-resistant and economic than non-ductile structures based on an elastic design approach.

Despite both SDSHD2000 and SDC follow the same design philosophy of ductility, the ductile components to be considered in the design are somewhat different. In SDSHD2000, the pier column is the only component allowed for ductile behaviour via formation of plastic hinges during extreme earthquakes. But in SDC, the ductile behavior can be contributed internally within the structural members by the formation of plastic hinges in piers and/or externally by supplemental protective devices such as isolation bearings or seismic dampers. In this way, the deformation or displacement of the protective components is limited to prevent the bridge structure from exceeding its ductility capacity.

3.2 Design Methodology

As mentioned earlier, two alternative design methodologies are developed for bridge seismic design: (1) the force-based approach, where the design seismic force levels are the elastic forces deducted from the ultimate forces of otherwise non-ductile structures determined from the acceleration response spectra based on a ductility-related reduction factor, with additional detailing of the members to ensure that adequate displacement/deformation capacity of the earthquake-resisting members is preserved; and (2) the displacement-based approach, where the ultimate design displacement based on a specified performance level is first determined with the corresponding seismic force calculated accordingly.

By the displacement-based approach of Caltrans SDC [8], the designer needs to ensure sufficient displacement ductility capacity and strength of the primary structural components to withstand the demand displacements imposed by the design earthquake of a desired performance level while maintain a minimum level of inelastic capacity at all potential plastic hinge locations. The displacement capacity of the bridges can be assessed with an inelastic static pushover analysis that incorporates non-linear inelastic load/deformation behavior of selected components.

The demand of displacement is described in terms of the displacement ductility, μ_D defined as:

$$\mu_D = \frac{\Delta_D}{\Delta_Y} \qquad (3.1)$$

Where Δ_D is the estimated global frame displacement demand；

Δ_Y is the yield displacement of the subsystem from its initial position to the formation of plastic hinge

The global displacement, Δ_D, includes components attributed to foundation flexibility, Δ_f .(i.e. foundation rotation or translation), flexibility of capacity protecting components such as bent caps, Δ_b, and the flexibility attributed to elastic and inelastic response of ductile members, Δ_Y, and , Δ_P respectively, as shown in **Figure 3-1a** and **3-1b**. In SDC, it is recognized that the global displacement can mostly be attributed to the flexibility of pier columns, therefore substituting column's lateral displacement for global displacement is acceptable. The displacement ductility capacities in analysis for various types of substructures have been calibrated to laboratory test result of fix-based pier columns. The design limits for displacement ductility demand are suggested as below:

For single column bents supported on fixed foundation, $\mu_D \le 4$;

For multi-column bents supported on fixed or pinned footings, $\mu_D \le 5$;

For pier walls (weak direction) supported on fixed or pinned footings, $\mu_D \le 5$;

For pier walls (strong direction) supported on fixed or pinned footings, $\mu_D \le 1$.

The elastic displacement demand of the primary structural members is determined by dividing the global displacement with μ_D. The design force can in turn be determined from the elastic displacement demand using an iterative procedure until convergence of the strength and stiffness of the members has been achieved.

3.3 Material

SDSHD2000 adopts in design the 28-day compressive strength for concrete and 4200 kg/cm^2 for the yield strength of reinforcement bar.

In SDC, the capacity of concrete components to resist seismic demands, except shear demand, are based on the most probable (expected) material properties to provide a more realistic estimate of design strength [7]. The shear capacity shall be considered in a more conservative way by discounting the nominal material strengths to prevent shear failure prior to formation of plastic hinge as $V_D \leq \phi V_n$ with ϕ = 0.85.

For reinforcing bars, SDC adopts the actual test data to design. If unavailable, the Park complex strain hardening model [7] that considers the phenomenon of strain hardening is adopted. Per the model, the yield point should be defined by the expected yield stress of the steel, f_{ye}, where the expected yield stress is 1.1 times of the normal yield stress recommended by Caltrans. The length of the yield plateau shall be a function of the steel strength and bar size. The strain-hardening curve can be modeled as a parabola or other non-linear relationship and should be terminated at the ultimate tensile strain, ε_{su}. The ultimate strain should be set at the point where the stress begins to drop with strain increased as the bar approaches fracture, as shown in **Figure 3-2**. It is Caltrans' practice to reduce the ultimate strain by up to 33% to avoid fracture of the reinforcement. For the property of concrete, Caltrans refers to Mander's concrete model [8] in which the reinforced concrete shows larger strength than the design value due to the previous statistical data. It suggests to use either 1.3 times of the specified concrete strength as the expected concrete compressive strength, f'_{ce}, or the actual value by the compressive test if available. The concrete stress-strain model is shown in **Figure 3-3**. Hence, the consideration of material property by SDC is more practical and economical than SDSHD2000.

3.4 Plastic Moment

The success of ductility design approach relies on bridge's capability to endure dependable deformation in plastic hinge regions without experiencing brittle failure during earthquakes. The plastic moment in association with the formation of the plastic hinge is the basis for the design of other components that are to remain essentially elastic under seismic load.

In SDSHD2000, the plastic moment of the pier is used to design for other protected members. The procedure to determine the plastic moment capacity is shown as the following:

(a) Determine the axial force of the pier due to dead load and seismic load if consideration of the vertical seismic force.

(b) Determine the section property of the pier column.

(c) Conduct the section axial force-moment interaction analysis to find the nominal moment strength of the pier, M_n .

(d) Calculate the plastic moment, M_p , which shall be equal to the product of M_n and a magnification factor, 1.3.

On the other hand, by SDC, the plastic moments of all ductile concrete members are calculated by moment-curvature analysis based on the expected material properties. Moment-curvature analysis derives the curvatures in association with a range of moments for a cross section based on the principles of strain compatibility and static equilibrium, using the expected concrete and steel strengths when either the concrete strain reaches ε_{cu} or the reinforcing steel strain reaches ε_{su}^{R}. The moment –curvature curve, as shown in **Figure 3-4,** can be idealized with an elastic perfectly plastic response to estimate the plastic moment capacity of a member's cross section. The elastic portion of the idealized curve should pass through the point marking yielding of the first reinforcing bar. The idealized plastic moment capacity is obtained by balancing the areas between the actual and the idealized beyond the yield point of the first reinforcing bar. The procedure of calculating the plastic moment is shown below:

(a) Idealize the material property.

(b) Conduct the moment – curvature analysis with an indicated limitation of material strength of inelastic components to determine the plastic moment.

Briefly speaking, SDSHD2000 and SDC are different in their ways to find the plastic moments of the piers. The design capacity of plastic moment by SDSHD2000 seems to be more conservative with a larger safety margin for the other protected components. On the other hand, SDC tries to bring the full material strength rather than nominal strength into account to estimate the plastic moment capacity. The design model for estimating plastic moment by SDC is more realistically reflecting the actual ductile performance. Nevertheless, because of considering idealized material properties without imposing a safety factor in the design, the safety margin of designing the bridge by SDC might be lower than that by SDSHD2000.

3.5 Vertical Seismic Force

In consideration of the effects of vertical seismic force, SDSHD2000 adopts a force-based formula similar to horizontal seismic force as discussed in Section 2.4.2. For sites in the near-fault area, consideration of the vertical seismic force for bridge design is required.

In SDC, consideration of vertical seismic force is required for sites with peak rock acceleration of 0.6g or greater, and the vertical seismic load is applied only to the superstructure in the analysis. It also suggests that if vertical acceleration is to be considered, a separate analysis of the superstructure's nominal capacity shall be performed based on a uniformly distributed vertical force equal to 25% of the dead load applying both upwards and downwards, as shown in **Figure3-5**. Moreover, regarding the loading combination cases with seismic force, SDC excludes simultaneous consideration of horizontal and vertical seismic loads in contrast to SDSHD2000 depicted in Section 2.5.1.

3.6 Bearing Assembly

The bearing assembly includes not just the pot bearing but also the shear key, hold-down device, lock-up device and supplemental dampers, if any, as a whole. The dampers can be used to prevent loss-of-span in both longitudinal and transverse directions via dissipating seismic energy.

The design concepts on bearing assemblies in SDSHD2000 and SDC are entirely different. In SDSHD2000, the bearing assemblies are considered as the protective components, therefore the strength of the bearing should be designed to resist the horizontal shear force associated with the plastic moment of the pier at the ultimate state. Other protective devices used for preventing loss-of-span are also required to deform in the elastic range at the ultimate state of the plastic hinges in the pier.

On the contrary, the bridge bearings are considered as sacrificial elements by SDC. Typically, bearings are designed and detailed for the state of service loads. The bearing strength shall be designed to ensure that their capacity and failure mode are consistent with the assumptions made in the seismic analysis.

Therefore, the bearings should be designed in a way that they can be easily inspected and replaced or repaired after an earthquake if damaged [9].

3.7 Summary

How bridges respond inelastically during earthquakes is indeed a complex problem. Investigations on seismic structural behavior of bridges and methods for improving their performance have been constantly explored worldwide. The design concepts and methodologies are different between SDSHD2000 and SDC, yet it is difficult to decisively tell which one is favorable over the other. Nevertheless, it is worthwhile developing an appropriate approach to evaluate the existing bridge structures designed by SDSHD2000 using the concept and methodology by SDC. The tasks may include the following:

(a) To re-estimate the component capacity with the expected material properties. SDC adopts the expected material properties for estimating the capacity of flexural structural components. It has been considered more economical from a construction company's point of view.

(b) To evaluate the actual ductility of existing structures. According to the force-based approach of SDSHD2000, the seismic design force is determined as a function of the spectral acceleration reduced by a reduction factor related to the ductility. The values of ductility considered in design are specified in **Table 2-3** simply based on the bridge’s foundation type, regardless of the member size and detailing. Despite it is easy for the designer to use, this does not reflect the actual ductility and its effects on structural response at all. While the SDC applies the displacement ductility for design that would more realistically reflect the actual ductile performance of the structure.

(c) To re-estimate the plastic moments of the pier via moment-curvature analysis.

(d) To re-define the design concept and desired function of the bearing assembly. In SDC, not only the bridge columns but also the bearing assemblies are contributed to the overall inelastic behavior, and by which the design allows for easy inspection and repair after earthquake if damaged. These devices can be considered for providing supplemental damping to the structure or seismic isolation of the structure.

4 EVALUATION PROCEDURE

For many existing bridges, construction was initiated before the seismic performance was adequately understood. As a result, existing bridges that were designed to a lower seismic standard than the requirement today are potentially at a higher risk of failure during earthquakes unless a proper retrofit plan is implemented. Before a retrofit measure of existing bridges can be attempted, a suitable evaluation process based on the most recent requirement is required. Referring to the proposal of Priestley [10], the method for the seismic assessment of existing bridges is considered separately from the design measures of new bridges. An evaluation approach that employs a displacement-based method is adopted herein to evaluate existing bridges that were designed using a force-based method.

4.1 Material Property

As suggested by SDC, the material properties considered in design are not the nominal strengths as generally adopted. In order to represent the actual behavior of existing bridges, actual material properties from test data should be used instead of the nominal design values. In the absence of actual material test data, the expected material properties based on SDC and other available technical research can be used for the evaluations.

4.1.1 Reinforcing Steels

The properties of reinforcing steels are modeled based on a stress-strain relationship that exhibits an initial linear elastic portion, a yield plateau, and a strain-hardening range in which the stress increases with strain. The yield point is defined by the expected yield stress of the steel, f_{ye}. The strain-hardening curve can be modeled as a parabola or some other non-linear relationship that terminates at the ultimate tensile strain ε_{ue}. The ultimate strain should be set at the point where the stress begins to drop with increased strain as the bar approaches fracture. The properties of reinforcing steels proposed by SDC are listed below:

Modulus of elasticity	Es = 200000 MPa
Specified minimum yield strength	f_y = 420 MPa
Expected yield strength (typical)	f_{ye} = 475 MPa

Specified minimum tensile strength f_u = 550 MPa

Expected tensile strength (typical) f_{ue} = 655 MPa

Nominal yield strain ε = 0.0021

Expected yield strain (typical) ε_{ye} = 0.0023

Ultimate tensile strain (reduced by 33%)

$$\varepsilon_{su}^{R} = \begin{cases} 0.09, \#9 \ \ and \ \ smaller \\ 0.06, \#10 \ \ and \ \ l\arg er \end{cases}$$

Onset of strain hardening

$$\varepsilon_{sh} = \begin{cases} 0.015 \ \ \#8 \\ 0.0125 \ \ \#9 \\ 0.0115 \ \ \#10 \,\&\, 11 \\ 0.0075 \ \ \#14 \\ 0.005 \ \ \#18 \end{cases}$$

It is possible that the expected yield stress of the steels in the ductile components may be less than 475 MPa (recommended by SDC); this will result in a reduced ratio of the actual plastic moment strength to the design strength, which will result in an underestimation of the strength requirement of the protected components. Therefore, a magnification factor of 1.1 has been proposed by NCHRP 12–49 [11] to define the value of f_{ye}; this definition is more conservative than that by SDC:

$$f_{ye} = 1.1 \times f_y \qquad (4.1)$$

This value will be adopted in the assessment herein. The stress-strain relationship of reinforcement is provided below:

$$
\begin{cases}
\varepsilon \le \varepsilon_Y & f_s = E \times \varepsilon \\
\varepsilon_Y < \varepsilon \ge \varepsilon_{sh} & f_s = f_{Ye} \\
\varepsilon_{sh} < \varepsilon \le \varepsilon_{su} & f_s = f_u - (f_u - f_{ye})(\dfrac{\varepsilon_{su} - \varepsilon}{\varepsilon_{su} - \varepsilon_{sh}})^2
\end{cases}
$$

4.1.2 Concrete

A stress-strain relationship model for confined and unconfined concrete is required in order to determine the capacity of the ductile concrete members. The initial ascending curve may be represented by the same equation for both the confined and unconfined concrete since the confining steel has no effect in this range of deformation. As the curve approaches the nominal compressive strength f'_c, the stress of the unconfined concrete begins to fall as the strain increases and rapidly reduces to zero at the spalling strain ε_{sp}. Typically, the value of spalling strain is 0.005. In the case of confined concrete, the curve continues to ascend until the confined compressive strength f'_{ce} is reached. This segment is followed by a descending curve that is dependent on the parameters of the confining steel. The ultimate strain ε_{cu} should be the point where strain energy equilibrium is reached between the concrete and confinement steel. Mander's stress-strain model is a commonly used model for confined concrete [8]. The properties of concrete proposed by SDC are listed below:

28-day concrete strength (design/tested strength): $f'c$

Expected concrete compressive strength: $f'ce = 1.3 f'c$

Modulus of Elasticity: $E_c = 0.043 \times w^{1.5} \times \sqrt{f_c'}$ MPa

Unconfined concrete compressive strain
at maximum compressive stress: $\varepsilon_{c0} = 0.002$

Ultimate unconfined compression (spalling strain): $\varepsilon_{sp} = 0.005$

The expected concrete compressive strength is recommended to be 1.3 times the design concrete strength according to SDC. However, in consideration of the actual engineering environment in Taiwan, an overstrength factor of 1.1 is adopted in this study. That is,

$$f'_{ce} = 1.1 \times f'_c \qquad (4.2)$$

According to ACI, the specified compressive strain of concrete is considered to be 0.003, as is the case with SDSHD2000. However, in accordance with a study by Blume [12], the unconfined compressive strain of concrete of 0.003 is too conservative and the value of 0.004 is considered adequate. Hence, an unconfined compressive strain ε_{cc} of 0.004 is adopted in this study.

The ultimate concrete strain follows the model of Mander [8]. The value of ε_{cu} is calculated using the equation given below

$$\varepsilon_{cu} = 0.004 + \frac{1.4\,\rho_s f_{yh} \varepsilon_{hu}}{f'_{ce}} \qquad (4.3)$$

where

ρ s: steel ratio of confining reinforcement

fyh: yield strength of confining reinforcing steels

ε hu: ultimate tensile strain of confining reinforcing steels; the value of 0.09 is recommended by Caltrans

4.1.3 Allowable Material Ultimate Strain

The displacement ductility capacity of a flexural component is calculated using a displacement-based approach based on a moment-curvature curve in which the ultimate curvature corresponds to the extreme structural response at the ultimate strain of steel or concrete, whichever is reached first. However, if the ductility capacity is considered at the ultimate strain, it is equivalent to allowing for structural collapse at the seismic intensity of design earthquakes. In the absence of a safety margin, this would impose a high risk to life safety. Therefore, it is favorable to design bridge structures to withstand small repairable cracks when the displacement ductile capacity is reached. To achieve this goal, a reduction of the ultimate material strain is considered in the estimation of the displacement ductility capacity [13]. The performances at various stages of seismic levels with their corresponding allowable displacement ductility ranges, as proposed by UCSD [14], are shown in **Table 4-1**. The suggested ductility range refers to the definition of repairable bridges proposed for the transit system in the San Francisco Bay Area, BART [15]. For BART, a reduction factor of 0.67 is suggested for the upper bound of the ultimate strain addressing the structural displacement ductility, and a reduction factor of 0.5 is suggested as the lower bound to maintain an economical design. Therefore, in this study, the ultimate strains of steel and concrete are respectively considered to be

$$\varepsilon_{sA} = 50\% \times \varepsilon_{su}^{R} \qquad (4.4\text{-a})$$

$$\varepsilon_{cA} = 50\% \times \varepsilon_{cu} \qquad (4.4\text{-b})$$

4.2 Conditions of Analysis

The coefficient of the normalized acceleration response spectrum issued in SDSHD2000 is adopted herein for analysis.

4.2.1 Site Soil Condition

The seismic design is based on site-specific data. The original geotechnical design data are considered valid in the evaluation. If the "liquefaction potential" increases as a result of changes in the seismic load, the conditions should be re-evaluated. Otherwise, the original design calculations remain valid.

4.2.2 Allowable Ground Horizontal Acceleration

Referring to SDSHD2000, the design horizontal ground acceleration is based on a return period of 475 years. An important premise in the assessment of existing bridges is that every existing bridge has a limited designing service life. Taking into account the service life of bridges, it is recommended to consider a reasonable reduction of the design horizontal ground acceleration in accordance with their remaining service years. If the remaining service life of an existing bridge structure is less than 20 years, a minimum of 20 remainging service years is required [16]. If the remaining life is T years, then the horizontal ground acceleration can be calculated. In this study, the coefficient of the allowable horizontal ground acceleration is Z_a .

The return period T_r associated with the remaining life T is given by

$$T_r = \frac{1}{1-0.9^{1/T}} \qquad (4.5)$$

The allowable ground horizontal acceleration associated with the return period is

$$Z_a = \left(\frac{T_r}{475} \right)^K \times Z \qquad (4.6)$$

Here, the value of the factor K is between 0.3 and 0.45. The value of 0.3 is generally adopted for conservativeness.

4.2.3 Vertical Seismic Force

According to SDC, the vertical seismic force is usually not required to be considered except for sites with a peak rock acceleration of 0.6g or greater. In comparison, according to SDSHD2000, a consideration of the vertical seismic force is required for a near-fault effect. Hence, the consideration of the vertical seismic force depends on the site location.

According to a study conducted by the Central Geological Survey, MOEA, there are three types of faults in Taiwan [17]. The type I fault is defined as an active fault that has a higher potential of causing an earthquake and affects the safety of structures, as shown in **Figure 4-1**. Referring to the study of Lin [18], the near-fault effect is considered effective within 10 km from the epicenter. Therefore, it is suggested that a consideration of the vertical seismic force is required for sites located within 10 km of a type I fault.

4.3 Analysis of Pier

In this evaluation, an assessment of the displacement ductile capacity, shear capacity, and P-Δ effect of the pier is required. The pier can be considered safe if the capacities meet the demands.

4.3.1 Displacement Ductility Capacity

According to the principle of ductility design, the pier is the major structural component for dissipating earthquake energy via an inelastic response. The assessment herein evaluates the displacement ductility capacity to verify if it is sufficient for a design earthquake. If the displacement ductility capacity exceeds the requirement, the pier is considered to be seismically safe.

The displacement ductile demand is a measure of the post-elastic deformation imposed on a member and is mathematically defined by

$$\mu_D = \frac{\Delta_D}{\Delta_Y} \qquad (4.7)$$

where

Δ_D : estimated global frame displacement

Δ_y : yield displacement of subsystem from its initial position to the formation of plastic hinge.

The global displacement includes the displacement of the foundation flexibility Δ_f and the displacement of the ductile piers, Δ_y and Δ_p, as shown in **Figures 3-1a** and **3-1b**. The foundation is designed to resist deformation; hence, the foundation is assumed to be fixed, and its displacement is considered to be minimum. Herein, the displacement of the foundation can be ignored, and the global displacement is attributed to the displacement ductility of the piers. According to SDC, the displacement demand is estimated by an equivalent static analysis. That is supposed that the seismic horizontal force will provide an elastic displacement with the section rigidity $E_c I_{eff}$ and the stiffness I_{eff} can be obtained by Figure 4-2. After $E_c I_{eff}$ is obtained, the displacement demand can be calculated by the following steps:

$$K = \frac{3E_c I_{eff}}{L^3}$$

$$\Delta_D = \frac{F_D}{K}$$

where

L : distance from the bottom of the pier to the center of gravity of the superstructure

F_D : seismic demand force, equal to the product of the spectral acceleration and tributary weight

For an accurate estimation of the displacement demand, an alternative method of the Substitute Structure Analysis is adopted in the evaluation [6]. This method involves the modeling of the seismic displacement of an inelastic bridge structure with an equivalent elastic system of the stiffness of various effective systems. The analytical procedure is as follows:

(a) Establish the force-displacement relationship of the bridge and determine K_o at the first yield of the reinforced bars.

(b) The seismic force is divided by K_o to yield the displacement.

(c) Input the displacement into the equation for the force-displacement relationship and determine a new value of K_1.

(d) The period of the bridge is modified according to the change in K_1, and a new seismic demand force is calculated.

(e) Repeat steps (b) to (d) until the seismic demand force converges. The final displacement demand associated with the convergent force can then be obtained

The displacement ductility capacity of the pier is curvature dependent. The curvature capacity of the pier is determined by a moment-curvature analysis for which the geometric representation of the inelastic deformation of the pier is provided in **Figure 4-3**.

According to SDC, the idealized yield displacement of the pier is calculated using the equation given below:

$$\Delta_y' = \frac{L'^2}{3} \times \phi_y$$

where

L': distance from the bottom of the pier to the center of gravity of the superstructure in the transverse direction or the bearing center in the longitudinal direction.

ϕ_y: idealized yield curvature

The yield displacement equation adopted in this study is similar to that provided by SDC. However, considering the moment provided by the superstructure, the equation will be

$$\Delta_y = \frac{FL^3}{3EI} + \frac{M_s L^2}{2EI} = \frac{FL^3}{3EI} + \frac{FaL^2}{2EI} = \frac{FL^2}{EI} \times \left(\frac{L}{3} + \frac{a}{2}\right)$$

Because $\phi = \frac{M}{EI} = \frac{F(L+a)}{EI}$, the equation can be changed to

$$\Delta_y = \frac{FL^2}{EI} \times \left(\frac{L}{3} + \frac{a}{2}\right) = \frac{\phi L^2}{L+a} \times \left(\frac{L}{3} + \frac{a}{2}\right) \qquad (4.8)$$

where

F: horizontal seismic force

M_s: moment provided by the superstructure in an earthquake

L: distance from the top of the pier cap to the bottom of the pier

a: distance from the top of the pier cap to the center of gravity of the superstructure in the transverse direction or the bearing center in the longitudinal direction

The elevation of the bridge in the transverse direction is shown as **Figure 4-4**.

The displacement ductility capacity is calculated by the following steps:

(a) Build up the moment-curvature curve with the axial load of the total dead load.

(b) Based on the moment-curvature curve, calculate the effective rigidity of the pier by $Ec \times I_{eff} = \dfrac{M_y}{\phi_y}$ (4.9)

(c) Determine M_{yi} at ε_c = 0.004, and then calculate the idealized yield curvature of the pier by $\phi_{Yi} = \dfrac{M_{yi}}{EI_{eff}}$.

(d) Determine ϕ_A from ε_{sA} and ε_{cA}, whichever is reached first.

(e) Calculate the displacement Δ_C as follows:

$$\Delta_C = \Delta_{yi} + \Delta_P \quad (4.10)$$

where

$$\Delta_{yi} = \frac{\phi_{yi} L^2}{L+a} \times \left(\frac{L}{3} + \frac{a}{2} \right)$$

$$\phi_P = \phi_A - \phi_{Yi}$$

$$L_P = 0.08L + 0.022 f_{ye} d_{bl} \geq 0.044 f_{ye} d_{bl}$$ [19]

$$\theta_p = L_P \times \phi_P$$

$$\Delta_P = \theta_P \times (L - 0.5 L_P) \quad (4.11)$$

The displacement ductility capacity is then determined by

$$\mu_C = \frac{\Delta_C}{\Delta_{yi}} \qquad (4.12)$$

where

M_y : moment capacity of the section at first yield of the reinforcing steel

ϕ_y : curvature of the section at first yield of the reinforcing steel

L : distance from the bottom of the pier cap to the bottom of the pier

a : distance from the bottom of the pier cap to the contraflexure, e.g., girder C.G. in the transverse direction and bearing center in the longitudinal direction

L_P : equivalent length of the plastic hinge

Δ_P : idealized plastic displacement capacity due to the rotation of the plastic hinge

Δ_{yi} : idealized yield displacement of the column as the plastic hinge initiates

ϕ_{yi} : idealized yield curvature defined by an elastic-perfectly plastic representation of the M-φ curve of the cross section

ϕ_P : idealized plastic curvature capacity (assumed constant over Lp)

ϕ_A : curvature capacity at the failure limit state, defined as the concrete strain reaching ε_{cA} or the confinement reinforcing steel reaching the reduced ultimate strain ε_{sA}

θ_P : plastic rotation capacity

d_{bl} : diameter of the longitudinal bars of the main column

If the displacement ductility capacity of the existing bridge pier is higher than the displacement ductility demand, the pier is considered safe, otherwise additional retrofit strengthening measures are required.

4.3.2 Seismic Shear Capacity

The pier is required to have a sufficient shear capacity to avoid shear failure prior to the full development of the plastic hinge. The seismic shear capacity of the pier is evaluated conservatively based on the nominal design material strength. The consideration of seismic shear is shown below:

$$V_D \le \varphi \times V_n \qquad \varphi = 0.85 \tag{4.13}$$

where

V_D: column shear demand associated with the column plastic moment at the fixed column ends

V_n: nominal shear strength including the concrete shear strength V_C and reinforcement shear strength V_S

V_C and V_S are calculated as suggested by SDC.

$$V_C = v_C \times A_e, \quad A_e = 0.8 \times A_g \tag{4.14}$$

For columns with tension,

$v_c = 0$

Inside the plastic hinge zone,

$$v_c = \gamma_1 \times \gamma_2 \times \sqrt{f_c'} \le 0.33\sqrt{f_c'} \quad \text{(MPa)}$$

Outside the plastic hinge zone,

$$v_c = 0.25 \times \gamma_2 \times \sqrt{f_c'} \le 0.33\sqrt{f_c'} \quad \text{(MPa)}$$

The γ factors are defined as

$$\gamma_1 : 0.025 \le \gamma_1 = \frac{\rho_s \times f_{yh}}{12.5} + 0.305 - 0.083\mu_D < 0.25$$

$$\gamma_2 = 1 + \frac{P_C}{13.8 A_g} < 1.5$$

For confined circular or interlocking core sections,

$$V_S = \frac{A_V f_{yh} D}{s}$$

For rectangular sections with hoops and cross ties,

$$V_S = \frac{A_V f_{yh} d}{s}$$

where

P_C : column axial compression

f'_c : design concrete strength

ρ_s : reinforcement ratio

A_g : gross column area

A_e : effective column area

D: hoop diameter

μ_D : displacement ductility demand

A_V : total hoop/shear reinforcement area

f_{yh} : shear reinforcement yield strength

4.3.3 P-Δ Effect

Per SDC, when adopting the displacement ductility design, $P-\Delta$ effect will be another important issue. An equation is suggested as below for establishing a conservative limit for lateral displacements associated with axial load.

$$P_{dl} \times \Delta_D \leq 0.2 \times M_P \quad (4.15)$$

Where:

P_{dl} : axial load attributed to deal load

If this equation is satisfied, the P-Δ effect can typically be ignored [20]. If it is not satisfied, Δ_D is judged to be higher than the displacement capacity even if the displacement ductility capacity meets the demand. A retrofitting approach is needed to reduce the displacement demand.

4.4 Analysis of Foundation

Foundation components, including the column-cap joint connection, are required to have a sufficient strength capacity to resist column base forces and moments in the event of a design earthquake. In ductility design, the foundation design capacity is used depending on the plastic moment and shear transferred from the pier, as shown in **Figure 4-5**. However, a simplified measure is adopted when the original design moment of the existing foundation structure, M_O, corresponding to the original design plastic moment is higher than the expected column plastic moment proposed in this study. In this case, no further foundation evaluation is required.

4.4.1 Expected Plastic Moment

In ductility design, the plastic moment capacity of the pier is employed to determine the design capacity of the protected components. SDSHD2000 recommends that the value of the plastic moment be calculated using 1.3 times the value of the nominal moment of the pier with an indicating axial load. Referring to the discussion in Chapter 3, the evaluation approach herein adopts the moment -curvature analysis to calculate the plastic moment.

An elastic-plastic bilinear response is used with an idealized nonlinear moment-curvature curve, as shown in **Figure 4-6**. The plastic moment capacity of the pier is calculated by a moment-curvature analysis under the assumption that the curvature capacity is higher than the allowable curvature capacity. This suggests that the plastic curvature capacity ϕ_P is determined by the allowable material ultimate strain multiplied by a factor of 1.2. The plastic moment capacity can then be identified at the curvature equal to the plastic curvature capacity. The shear demand for estimating the shear capacity of the foundation can be calculated using the plastic moment:

$$V_P = \frac{M_P}{L}$$

Thus, the plastic moment and plastic shear can be employed for evaluating the performance of the foundation. If the capacity of the foundation does not satisfy the requirements, additional retrofit measures are required.

4.5 Analysis of Superstructure

Similar to the foundation components, the superstructure components are also defined as those that have an elastic performance in a design earthquake. For the existing bridge structure, the revised horizontal seismic force will have a minor influence on its structural reliability if the column can undergo relative displacement without shear failure [21]. However, vertical seismic loading is a major concern in the design of superstructures because the variation in the vertical seismic force affects the moment capacity of the superstructures and the deflection. If the original seismic design of the superstructure components considers the vertical seismic loading, the following equation can be adopted for the internal stress consideration.

$$\mathrm{M_O} \times a_O \geq \mathrm{M_R} \times a_R \qquad (4.16)$$

where

Mo: original design mass of the superstructure

a_O: original design vertical seismic acceleration

$\mathrm{M_R}$: actual mass of the superstructure

a_R: revised vertical seismic acceleration

If the equation is not satisfied or the original superstructure is designed without considering the vertical seismic load, a retrofit design of the superstructure is required that considers the new vertical seismic load.

4.6 Analysis of Bearing Assembly

In accordance with SDSHD2000, the bearing and the relevant devices are required as protected components in order to demonstrate an elastic response to an earthquake. Moreover, with reference to the actual damage situation of bridges during earthquakes [22], some bearing failures occur before the plastic hinge can carry out its protection function. As a result, it cannot fulfill the design target. Contrarily, the failure of the bearing assembly is accepted by SDC, and the bearing can thus be considered as a sacrificed component. The failure of the bearing assemblies is defined as a type of repairable damage, and it also induces a fuse behavior at the interface of the superstructure and substructure. The fuse behavior of the bearings is similar to the friction force performance, as shown in **Figure 4-7**. When the lateral force is higher than the ultimate force capacity of the bearings, the upper bound force that is transferred to the substructure as a demand force is equal to the ultimate force capacity of the bearings, and the seismic energy will be dissipated by the frictional sliding performance of the bearings. The substructures subjected to this upper bound force are required to respond in an elastic manner, otherwise the pier will collapse and a retrofit design is required. **Table 4-2** shows an actual result for the manner in which the column moment demand will reduce, relative to the permitted design by AASHTO [23].

Although it is valid to assume a fused behavior of the bearings in the evaluation of the existing bridge, the resulting larger displacements at the superstructure-substructure interface are checked against the available sliding seat width after the earthquake.

4.7 Moment –Curvature Analysis

In the displacement ductility analysis, the moment-curvature relationship is an important reference to estimate the plastic moment and displacement ductility. In accordance with a study by Priestley [6], the moment-curvature curve for a circular column may be generated for specified values of the extreme fiber compression strain ε_c by considerations of the axial and moment equilibrium. From a consideration of the axial equilibrium,

$$P = \int_{x=(D/2)-c}^{D/2} \left[b_{c(x)} f_c(\varepsilon_x) + \left(b_{(x)} - b_{c(x)} \right) f_{cu}(\varepsilon_x) \right] dx + \sum_{i=1}^{n} A_{si} f_s(\varepsilon_{xi}) \qquad (4.17)$$

where

$$\varepsilon_x = \frac{\varepsilon_c}{c}(x - 0.5D + c)$$

From a consideration of the moment equilibrium,

$$M = \int_{x=(D/2)-c}^{D/2} \left[b_{c(x)} f_c(\varepsilon_x) + (b_{(x)} - b_{c(x)} f_{cu}(\varepsilon_x) \right] x dx + \sum_{i=1}^{n} A_{si} f_s(\varepsilon_{xi}) x_i \qquad (4.18)$$

and the curvature is

$$\phi = \frac{\varepsilon_c}{c} \qquad (4.19)$$

In Eqs. (4.17) and (4.18), $f_c(\varepsilon)$, $f_{cu}(\varepsilon)$, and $f_s(\varepsilon)$ are the stress-strain relationships for confined concrete, unconfined concrete, and reinforcing steel, respectively, and A_{si} is the area of a reinforcing bar at a distance x_i from the centroidal axis. The remaining nomenclature is defined in **Figure 4-8**.

Equation (4.17) is solved for c by trial and error using the known axial load P and the specified extreme fiber compression strain. This enables the moment M and curvature ϕ to be calculated directly from Eqs. (4.18) and (4.19), respectively. The entire moment-curvature curve is generated by specifying a sequence of ε_c values up to the ultimate compression strain. Substituting $b_{(x)} = b$ and $b_{c(x)} = b_c$, Eqs. (4.17) to (4.19) can also be made to apply to a rectangular section, as shown in **Figure 4-8.**

According to the moment-curvature analysis of a pier section with various axial loads, as shown in **Figure 4-9**, it is found the axial force-moment relationship is similar to the nominal P-M curve, in which the moment will increase with the increment of axial force under the balanced steel ratio. Besides, the allowable ultimate curvature, associated with $\varepsilon_{CA} / \varepsilon_{SA}$, will be reduced with the increasing trend of axial force, and subtracting the values of yield curvature from the allowable ultimate curvature will also reduce then the plastic curvatures. Hence the displacement ductility capacity of piers will be diminished with the increment of axial force. As the result, it is necessary conducting the moment-curvature analysis with the adequate axial force to accurately estimate the displacement ductility capacity.

Herein, a commercial program, XTRACT, produced by Imbsen & Associates, Inc. is employed conducting the moment–curvature analysis.

4.8 Evaluation Process

An evaluation procedure that proceeds by the following steps is suggested:

(a) Material properties are required for test data or idealization.

(b) M_D for every revised seismic force is compared with M_{DO} for every original seismic design. If M_D is greater than M_{DO}, further evaluation is required. Otherwise, the bridge structure is considered safe and assessment is discontinued.

(c) M_D and M_{yi} are compared when a further step is required. If M_D is greater than M_{yi}, the displacement ductility capacity of the pier is checked, and the associated plastic moment is used to estimate the other protected components. If M_D is less than M_{yi}, it indicates that the elastic response of the pier is below the seismic demand, and M_{yi} is then used to assess the other components.

5 EVALUATION OF EXISTING BRIDGE STRUCTURE

As an example, a bridge is evaluated using the approach discussed in this study. It is designed using Seismic Design Specifications for Highway Bridges, 1995.

5.1 Structure Description

The bridge consists of a single span concrete box girder, supported on single column piers, founded on pilecap and piles. The span length of girder is 30m and the girder connects the pier by pot bearings. The substructure is a round column, which is 9.642m high and 3m wide. The pilecap is supported on 4 piles. The original design axial load is 1515.22 ton.

5.2 Material Property

<u>Concrete:</u>

Unit weight: 2.5 t/m^3

Concrete strength: $f_c' = 27.5$ MPa

Expected concrete strength: $f_{ce}' = 1.1 \times f_c' = 30.2$ MPa

Modulus of elasticity: $E_c = 29538$ MPa

<u>Reinforcing Steel:</u>

Yield strength: $f_y = 420$ MPa

Expected yield strength: $f_{ye} = 1.1 \times f_y = 462$ MPa

Modulus of elasticity: $E_s = 200000$ MPa

The stress-strain relationships for unconfined concrete, confined concrete, and reinforcing steel are shown in **Figures 5-1~5-3**.

5.3 Pier

<u>Pier Section Properties:</u>

Diameter: 3 m

Area: 7.07 m^2

Moment of inertia: I_g = 3.98 m^4

Pier height: 9.642 m

Depth of bearing: 0.5 m

Depth of girder: 3.38 m

Number of longitudinal bars: 122

Longitudinal bar size: 36 mm

Transverse reinforced bar size: 19 mm (spacing = 0.115 m)

The cross section of the pier is provided in **Figure 5-4**

5.3.1 Moment Check

The relationship between M_D and M_{yi} is examined to determine whether the pier shows an inelastic response.

M_{yi}:

The curve obtained from the moment-curvature analysis is shown in **Figure 5-5**; the data is tabulated in **Table 5-1.**

The effective rigidity at the steel strain that reaches ε_{ye} = 0.0023 is determined.

$$EI_{eff} = \frac{M_{ye}}{\phi_{ye}} = \frac{6.043E+4}{1.265E-3} = 4.78\text{E} + 7\ kN - m^2$$

From **Table 5-1**, at $\varepsilon_c = 0.004$, the idealized yield moment capacity is M_{yi} = 82000 $kN - m$. The calculation of the idealized yield curvature is then given as $\phi_{yi} = \dfrac{M_{yi}}{EI_{eff}}$ = 1.72E-3

M_D:

Effective stiffness: $K = \dfrac{3EI_{eff}}{(H+a)^3}$ = 141254 kN/m

Mass: W = 1515220 kg

Period: T = $2\pi\sqrt{\frac{W}{K}}$ = 0.65 s

Locate at Soil Type II, C = 1.998

Seismic force: V = ZCW = 0.33 × 1.99 × 14859 = 9795 kN

$M_D = V \times (H + a)$ = 9795 × (9.642 + 0.5/2) = 96872 $kN - m$

Because M_D is higher than M_{yi}, the pier shows an inelastic response.

5.3.2 Displacement Ductility Check

According to the moment-curvature analysis shown in **Table 5-1**, the allowable curvature at ε_{CA} or ε_{SA} is ϕ_A = 1.07E-2. The allowable moment at the allowable curvature is then calculated as M_A = 86405 $kN - m$.

According to Eq. (4-8), $\Delta_{yi} = \frac{\phi_{yi} L^2}{L+a}\left(\frac{L}{3}+\frac{a}{2}\right) = 55 \quad mm$

$\phi_p = \phi_A - \phi_{yi}$ = 8.98E-3

$L_P = 0.08L + 0.022 f_{ye} d_{bl} \geq 0.044 f_{ye} d_{bl}$ = 1.137 m > 0.732 m, L_P = 1.137 m

$\Delta_P = \theta_P \times (L - 0.5L_P) = (L_P \times \phi_P) \times (L - 0.5L_P)$ = 93 mm

$\mu_A = \frac{\Delta_{yi} + \Delta_P}{\Delta_{yi}}$ = 2.69

Next, the displacement demand is calculated by the Substitute Structure Analysis. The results are shown in **Figure 5-6** and **Table 5-2**; the displacement is $\Delta_D = 90 \quad mm$.

$\mu_D = \frac{\Delta_D}{\Delta_{yi}}$ = 1.6 < μ_A, Check OK.

5.3.3 Shear Force Check

From **Table 5-2**, the curvature for the plastic moment associated with 120% ε_{CA} or ε_{SA} is ϕ_P = 1.18E-2. The plastic moment at ϕ_P is M_P = 87320 $kN - m$

$$V_D = V_P = \frac{M_P}{L} = 8829 \text{ kN}$$

$$\gamma_1 = \frac{\rho_s \times f_{yh}}{12.5} + 0.305 - 0.083\mu_D = \frac{0.00681 \times 420}{12.5} + 0.305 - 0.083 \times 1.6$$

$= 0.4 > 0.25$, $\gamma_1 = 0.25$

$$\gamma_2 = 1 + \frac{P_C}{13.8A_g} = 1 + \frac{14.85}{13.8 \times 7.07} = 1.2 < 1.5, \ \gamma_2 = 1.2$$

$$v_c = \gamma_1 \times \gamma_2 \times \sqrt{f_c'} = 0.25 \times 1.2 \times \sqrt{27.5} = 1.57$$

$$V_C = v_C \times 0.8A_g = 8.88 \text{ mN} = 8880 \text{ kN}$$

$$V_S = \frac{A_V f_{yh} D}{s} = 9478 \text{ kN}$$

$$V_n = \phi(V_C + V_S) = 0.85 \times (8880 + 9478) = 15604 \text{ kN} > V_D = 8829 \text{ kN},$$

Check OK.

5.3.4 P-Δ Effect Check

$P_{dl} \times \Delta_D = 14859 \times 0.09 = 1337 \ kN - m < 0.2 \ M_P = 17464 \ kN - m$, Check OK.

In accordance with the evaluation of the pier is found adequate by above procedures, the pier is verified as safe.

5.4 Foundation

The M_P and V_P are used to check the foundation base shown in **Figure 4-3**. Herein because M_{PO}, equal to 1.3M_n (1.3×76025=98833 $kN - m$), is more than M_P (87320 $kN - m$), the further evaluation for foundation is not required.

6 CONCLUSION

Seismic effect is a very complex and specific activity for design of bridge structures and other architecture structures. In Taiwan, the study of seismic design code and technology are not as early as in the United States and Japan but the study has been advanced recently due to the special geographic location of Taiwan. Especially after the Chi-Chi earthquake, the bridge design requirements become more rigid to cater for seismic scenario. On the other hand, for the safety of existing bridges, a pragmatic economical evaluation approach is mandatory. This study tries to explore the SDSHB 2000 with viewpoint from the displacement-based ductility to establish an economical procedure to assess the existing bridges at Taiwan.

(a) SDSHB 2000 requires designer to consider the effect of vertical seismic force, which is one of the major design parameters for the calculation of plastic moment. However, the definition is ambiguous on the component associated with the vertical seismic force and the situation that requires to be undertaken. As the result, all design will have to consider the vertical seismic force. Furthermore, it is an uneconomical construction so a clear definition about the requirement for vertical seismic force demand is necessary.

(b) The material properties used for the nominal design strength to evaluate the existing bridge are conservative. They will result in the actual strength capacity of existing pier stronger than the design capacity. The plastic hinge can not serve its protection function when the earthquake occurs. Therefore, an expected material property is employed to evaluate the component capacity instead of material nominal property. On the other hand, for the evaluation of existing bridges, it is strongly recommended to adopt the actual material test data of bridge on site. Thus the evaluation result can be more accurate.

(c) In ductility design, the moment capacity of plastic hinge is a major element. It is suggested to calculate the plastic moment via the moment-curvature analysis associated with the definition of material strain. Thus the capacity of plastic moment can be accurately estimated under the ductility requirement.

(d) Although the expected material properties are recommended for evaluating the moment capacity, it is suggested to calculate the shear capacity conservatively with the nominal design material strength to prevent the shear damage during the performance of plastic hinge.

(e) The bearing assembly is allowable as a fuse in the study. Nevertheless the displacement of bearing must be limited to prevent loss of span. Despite the fusing activity can effectively reduce the force demand for the substructure, some technical conditions need to verify before to launch the actual evaluation.

i. The actual ultimate force capacity of bearing.

ii. The friction coefficient for sliding

(f) The purpose of study is to evaluate the existing bridge at Taiwan to make sure it is safe for operation. The evaluation result reveals that under the new seismic demand, the existing bridge structures probably can be considered as safety with the displacement ductility analysis. Hence the effect of the displacement ductility should be considered when the seismic design code in Taiwan is modified in future.

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Appendix A: Figure

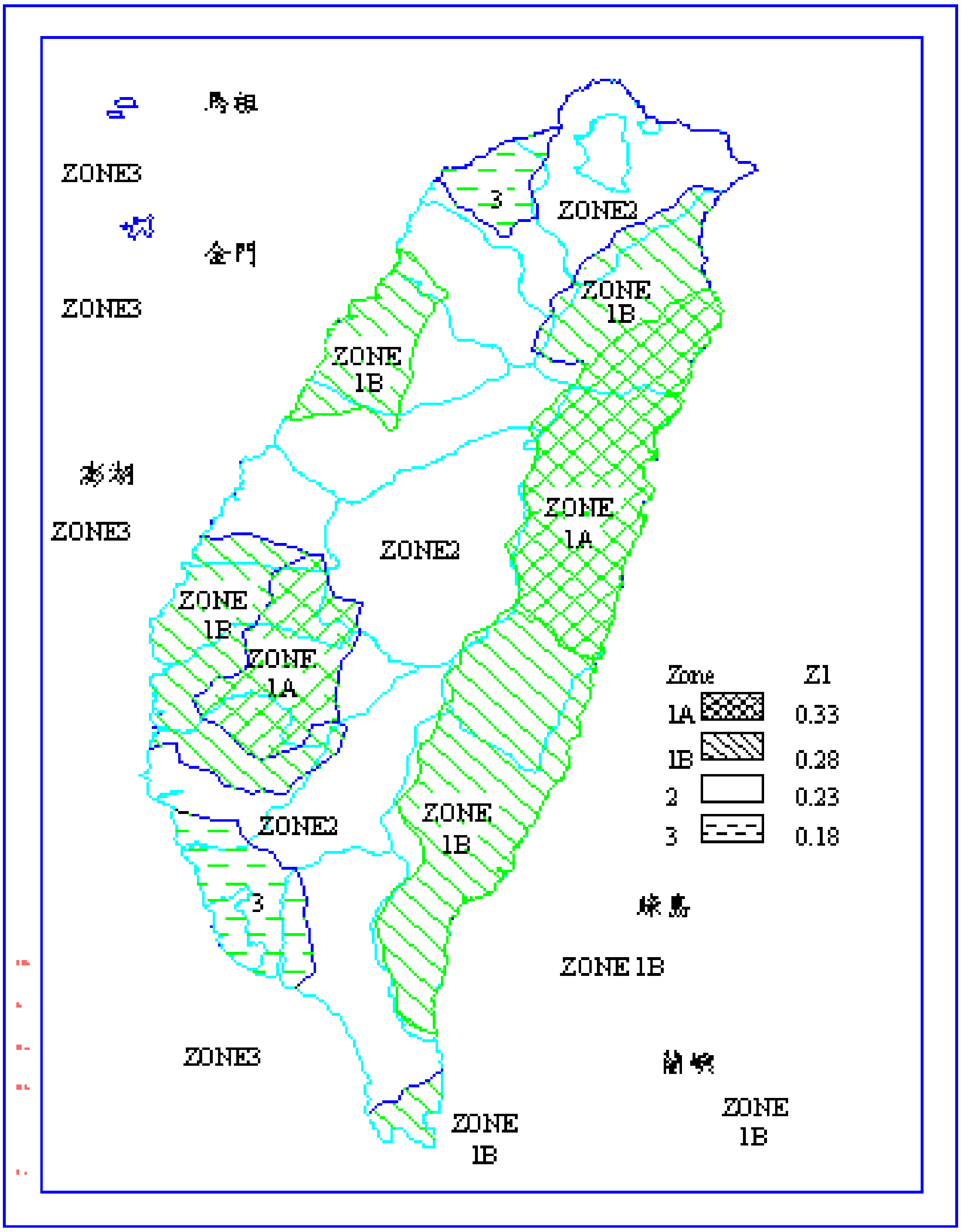


Figure 2-1 : Design Ground Acceleration Coefficient Zone - 1995

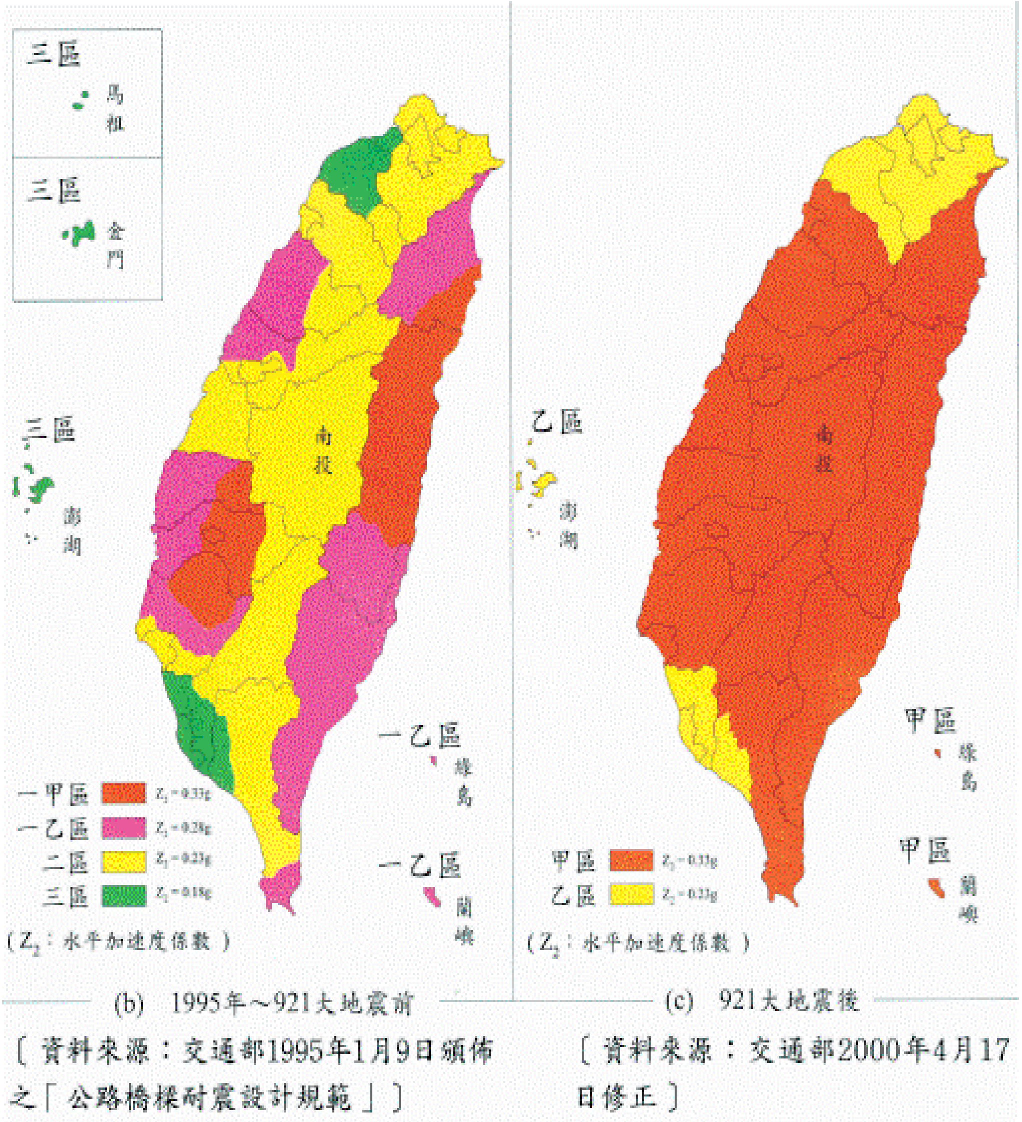


Figure 2-2 : Difference on Seismic Zone

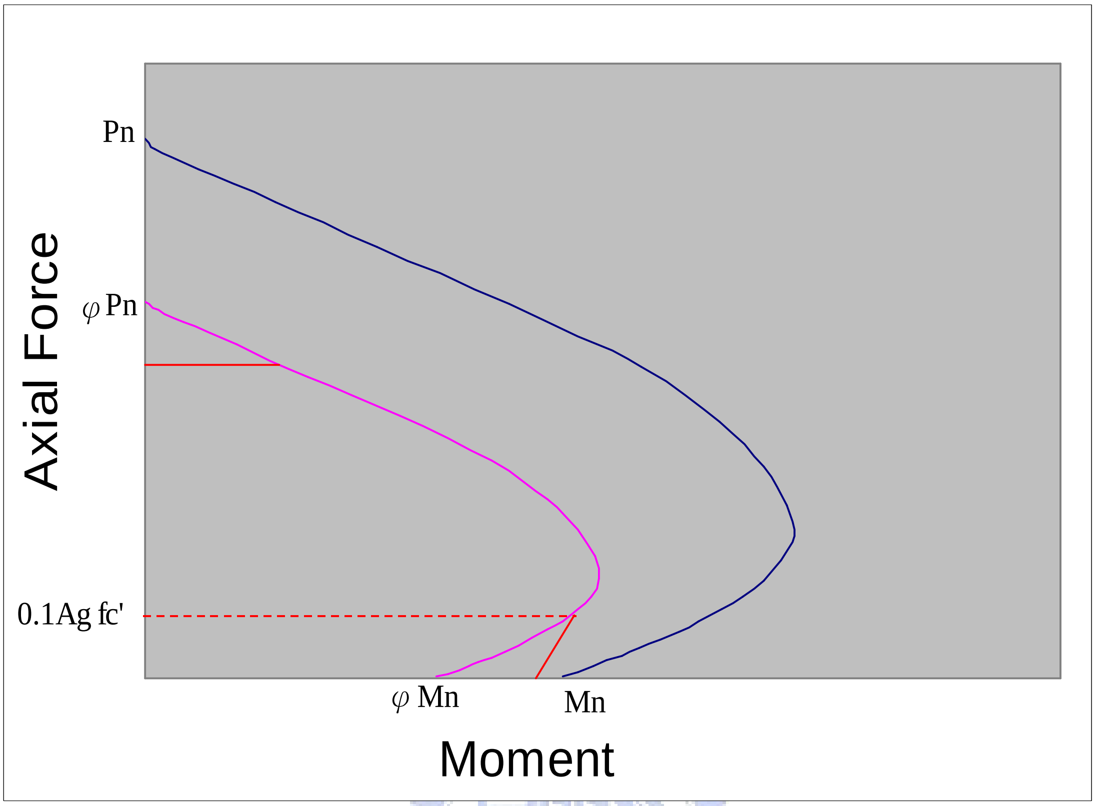


Figure 2-3 : Axial Force-Moment Curve

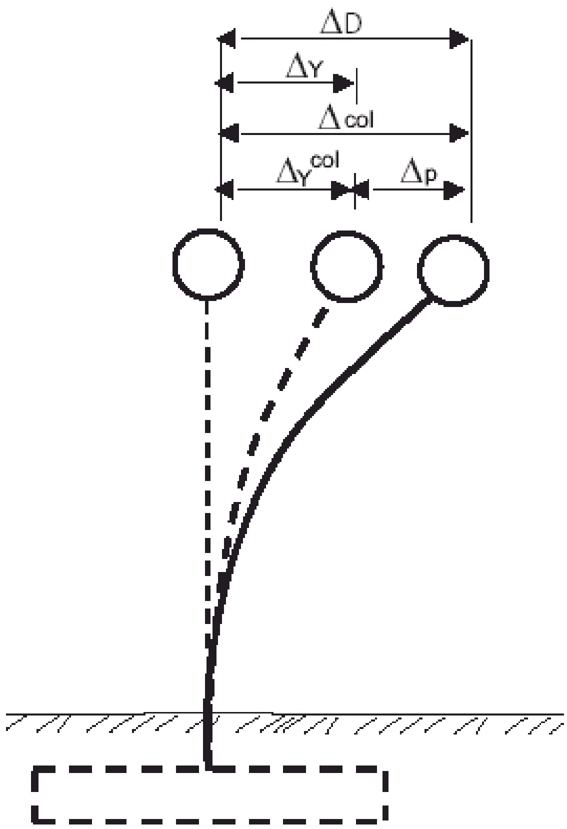


Figure 3-1a : Displacement Demand of Fixed footing

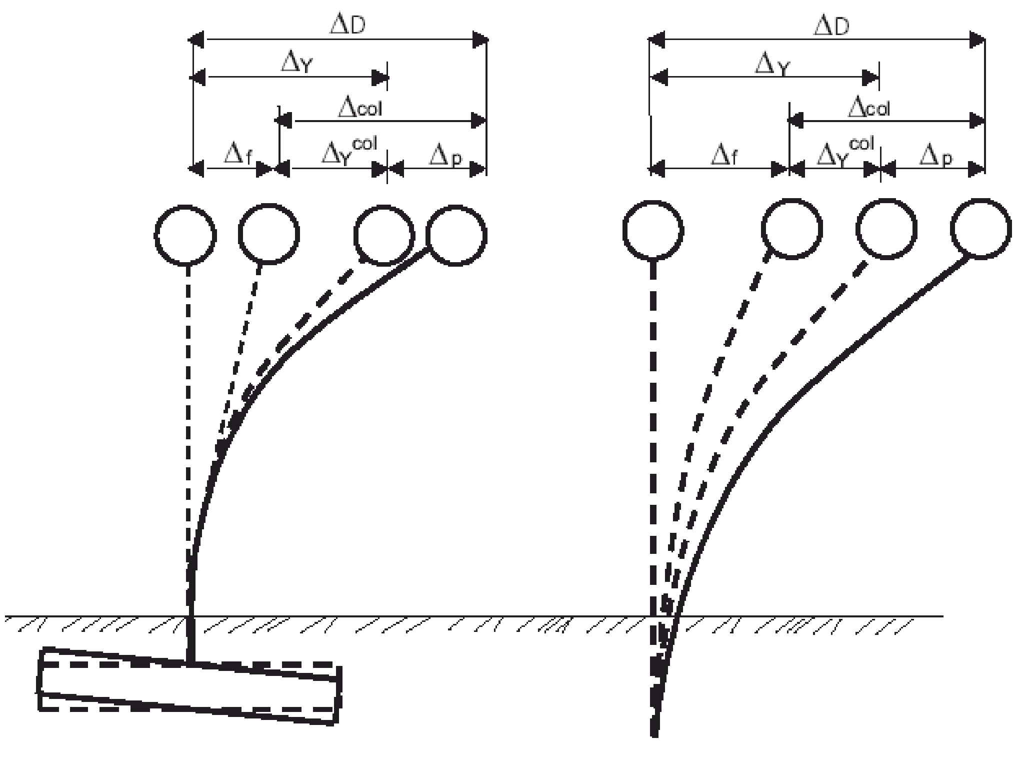


Figure 3-1b : Displacement Demand of Foundation Flexibility

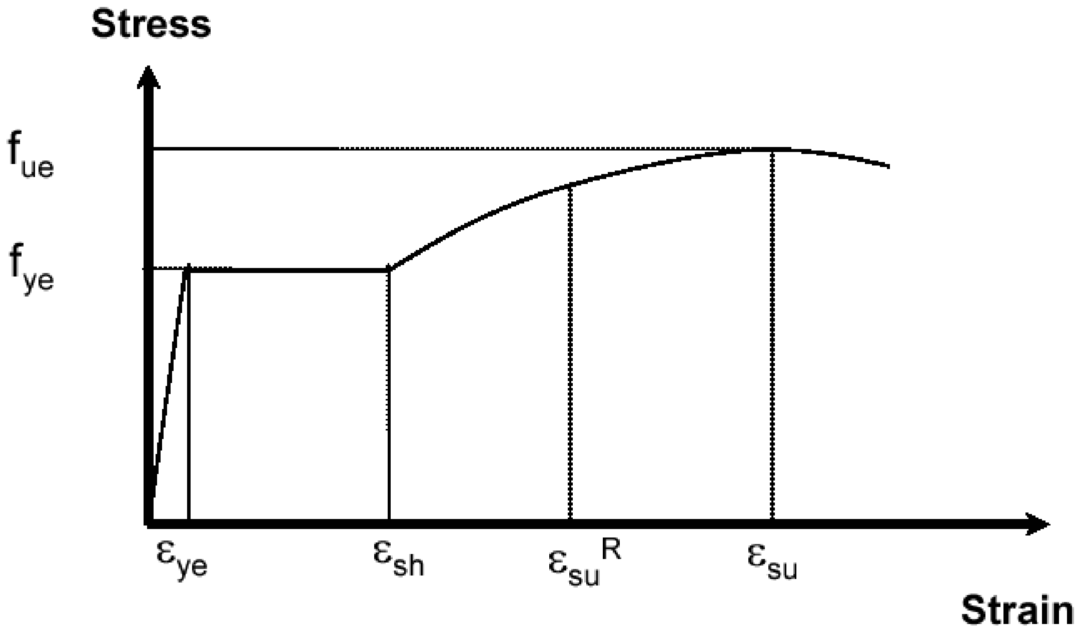


Figure 3-2 : Steel Stress-Strain Model

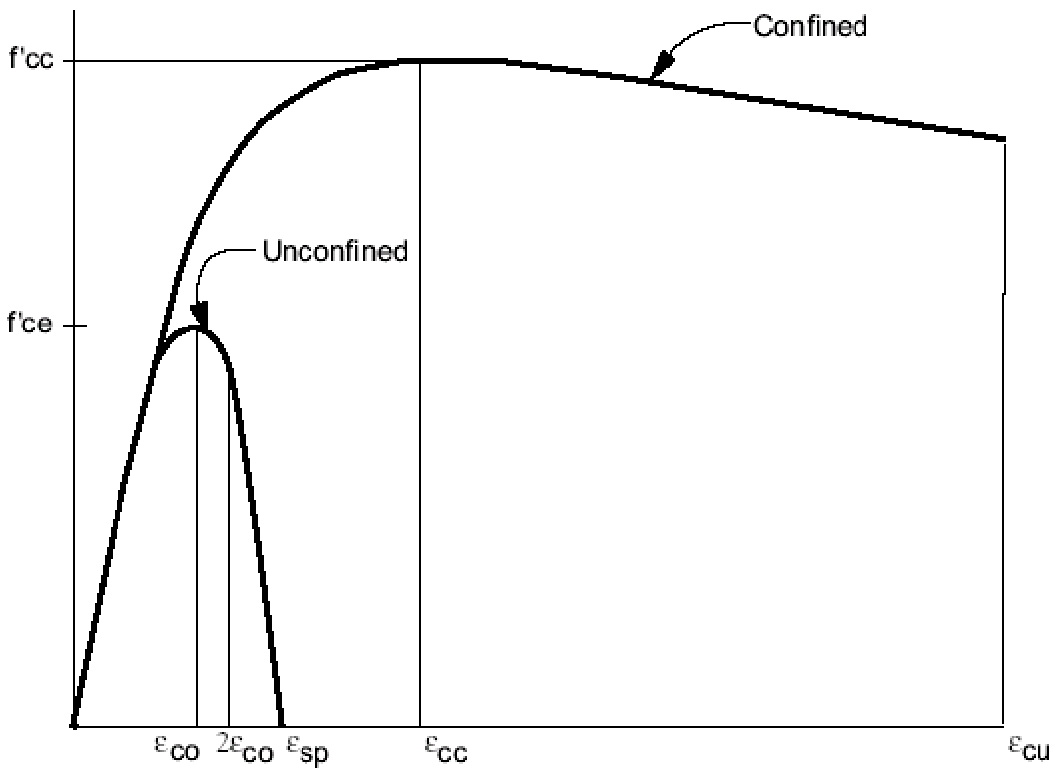


Figure 3-3 : Concrete Stress-Strain Model

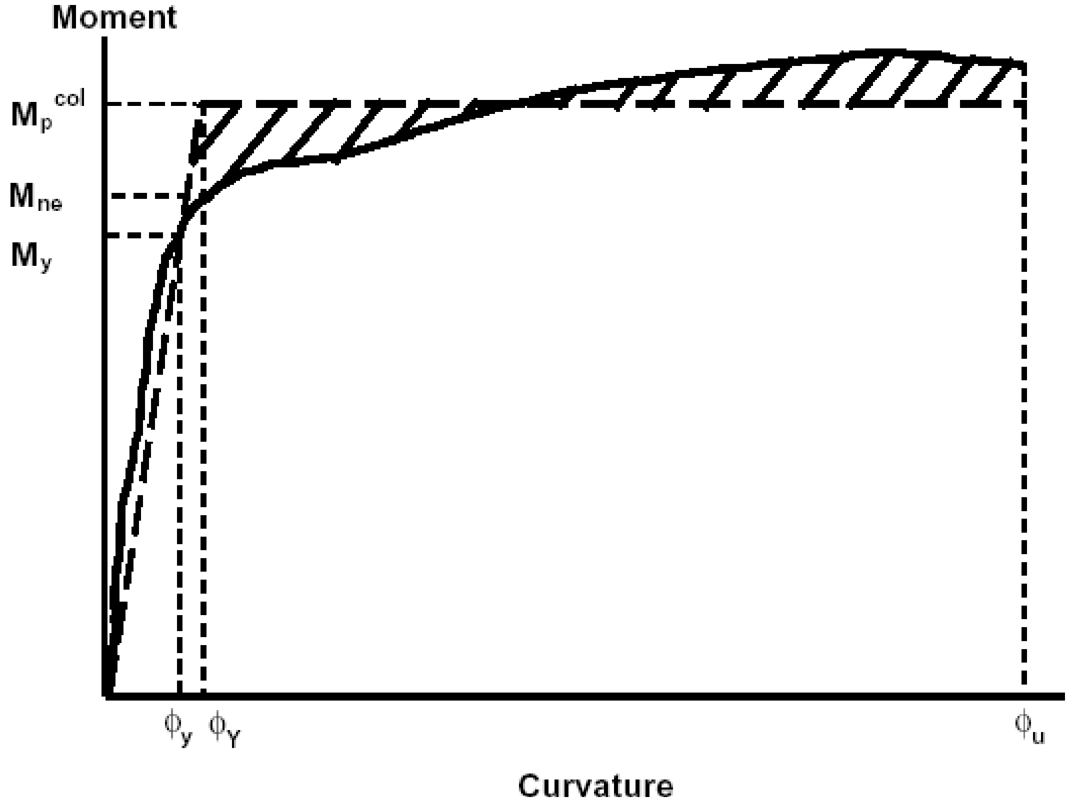


Figure 3-4 : Moment Curvature Curve

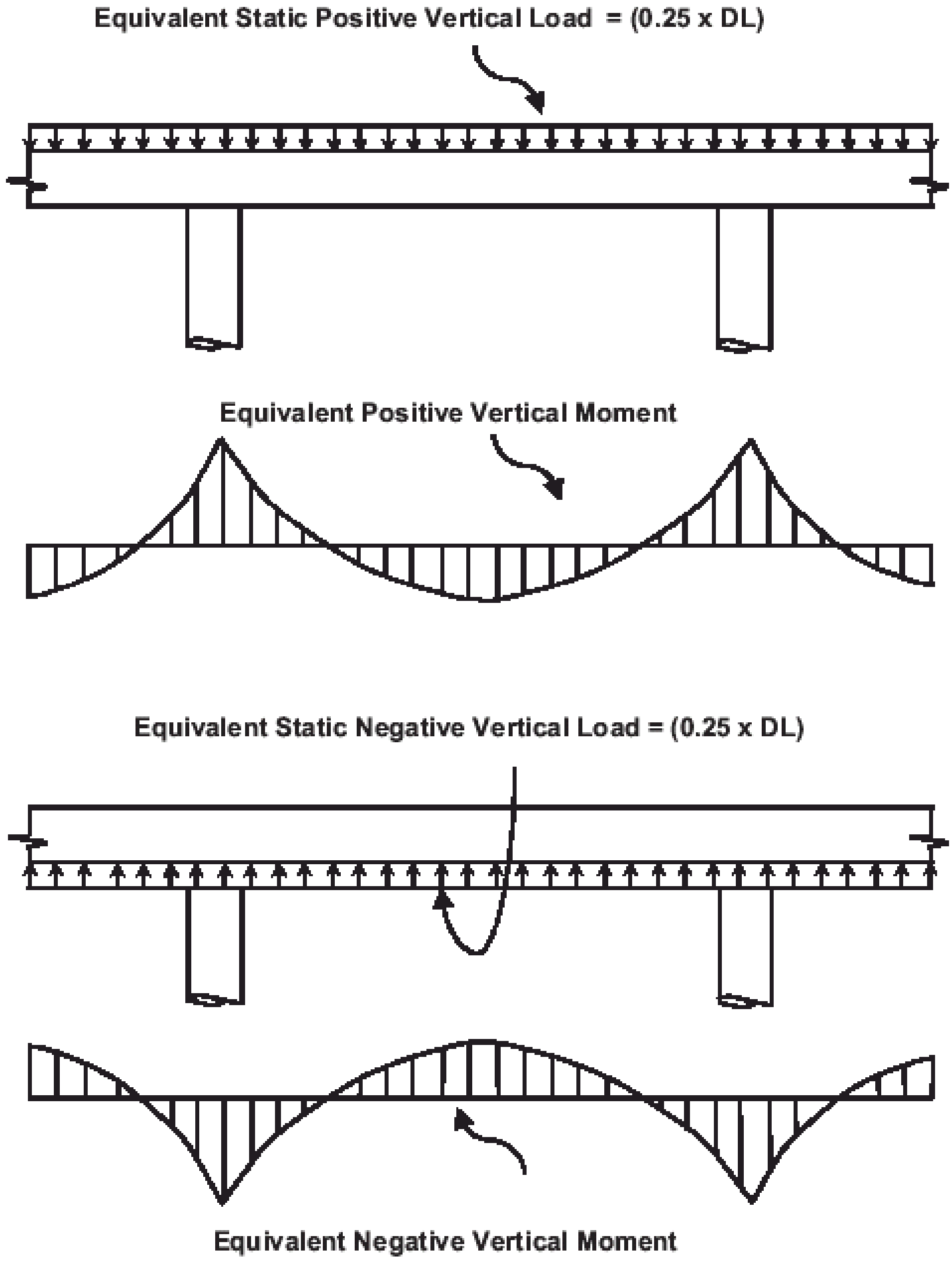


Figure 3-5 : Equivalent Static Vertical Loads & Moments

Figure 4-1 : Distribution of Taiwan Faults

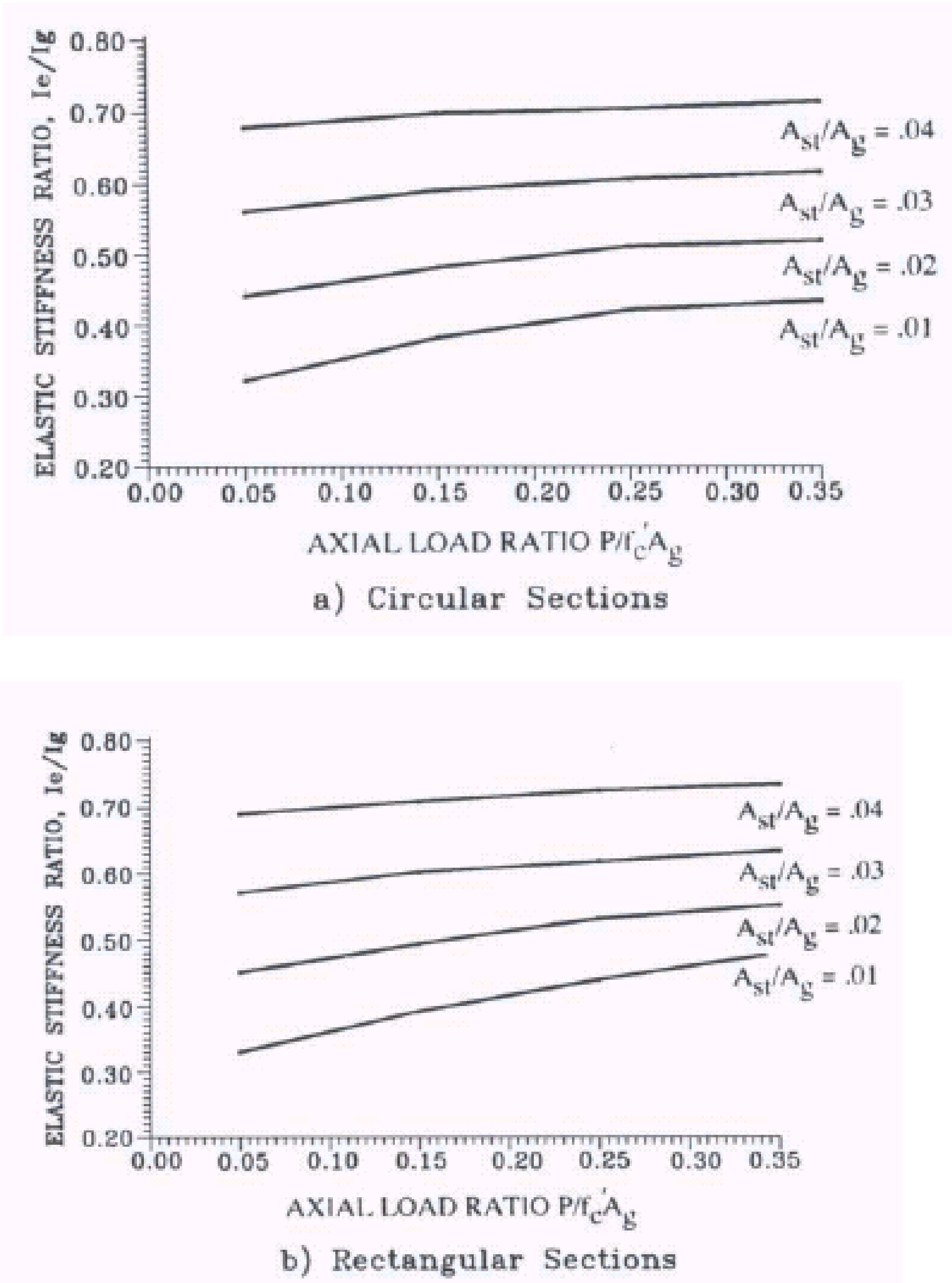


Figure 4-2 : Effective Stiffness of Reinforced Concrete Section

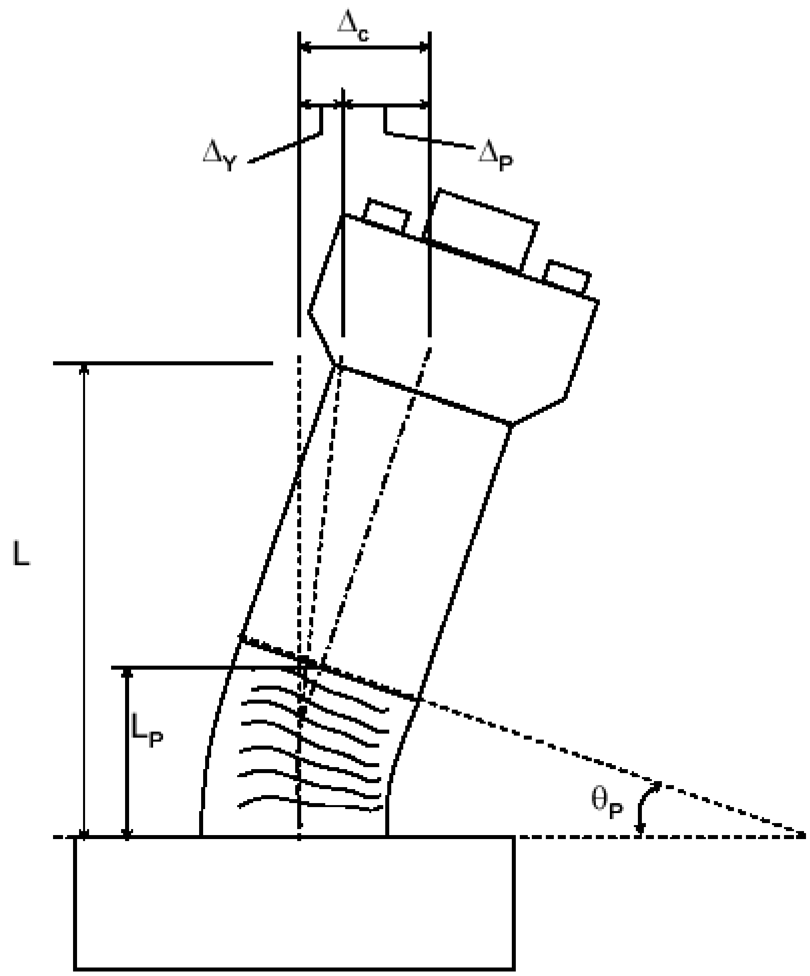


Figure 4-3 : Displacement Performance for Pier

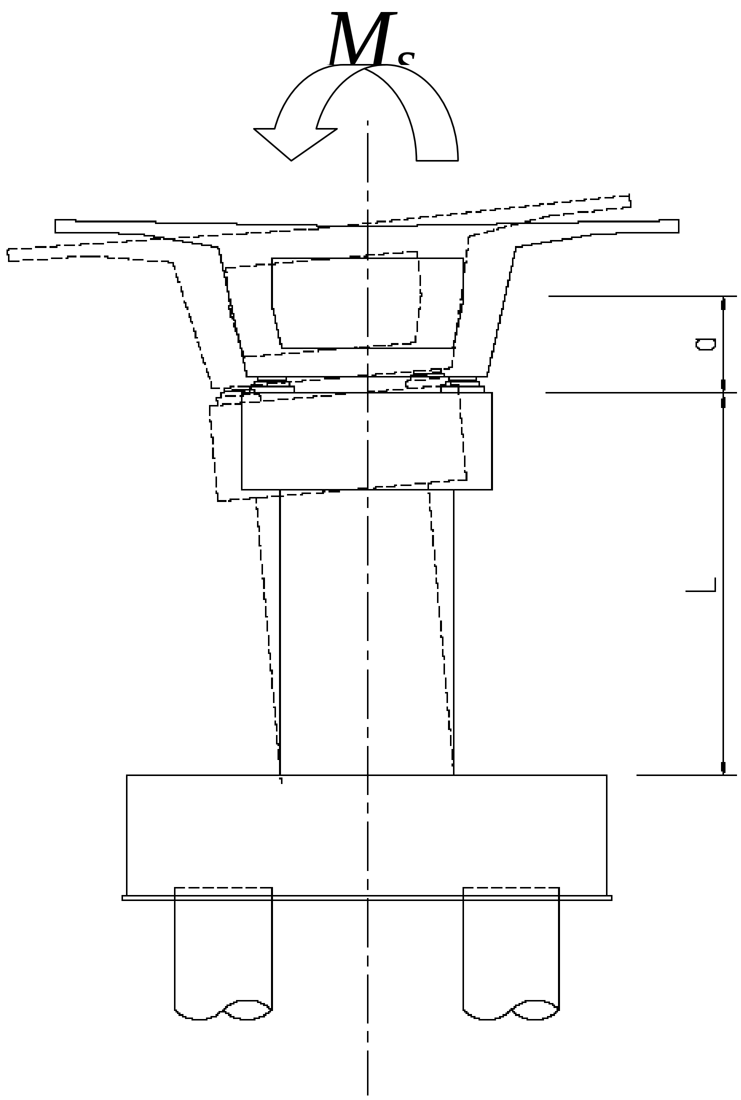


Figure 4-4 : Elevation of Bridge in the Transverse Direction

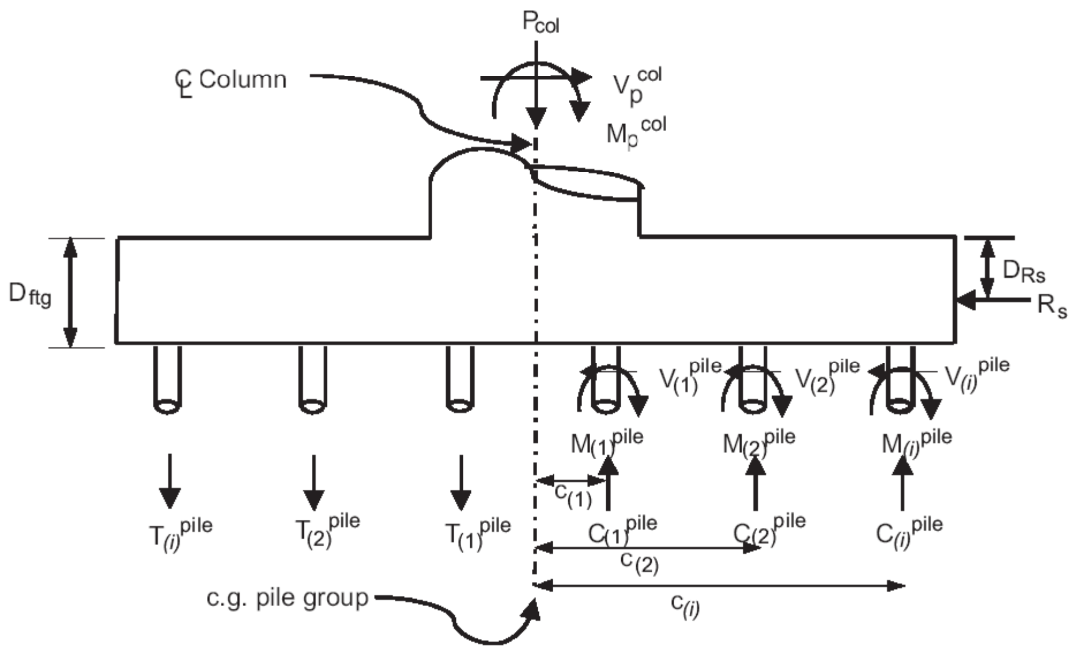


Figure 4-5 : Foundation Force Equilibrium

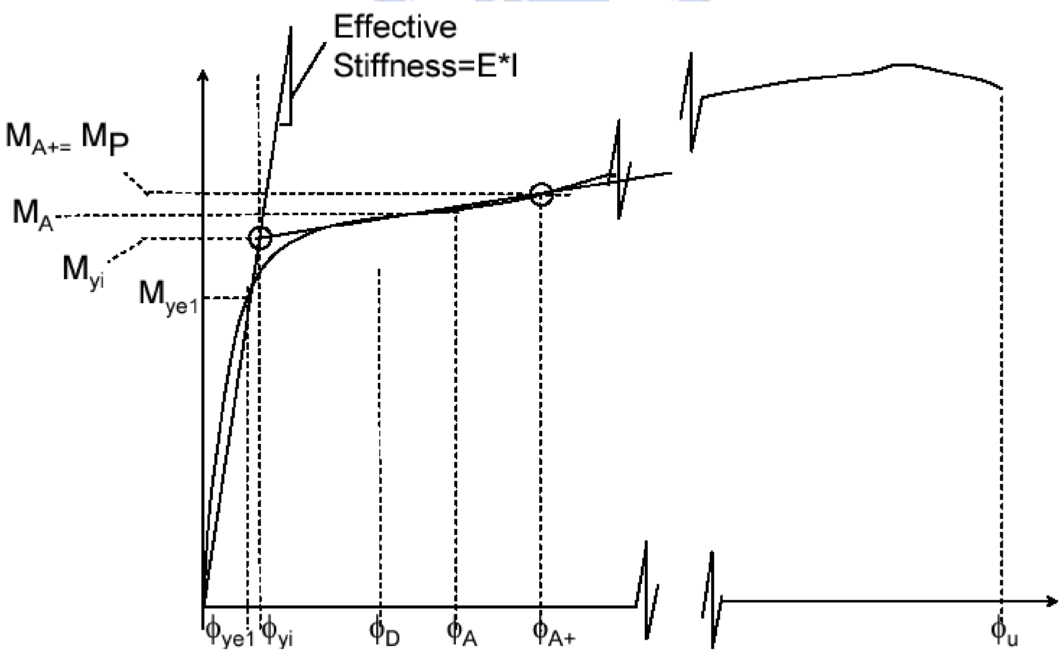
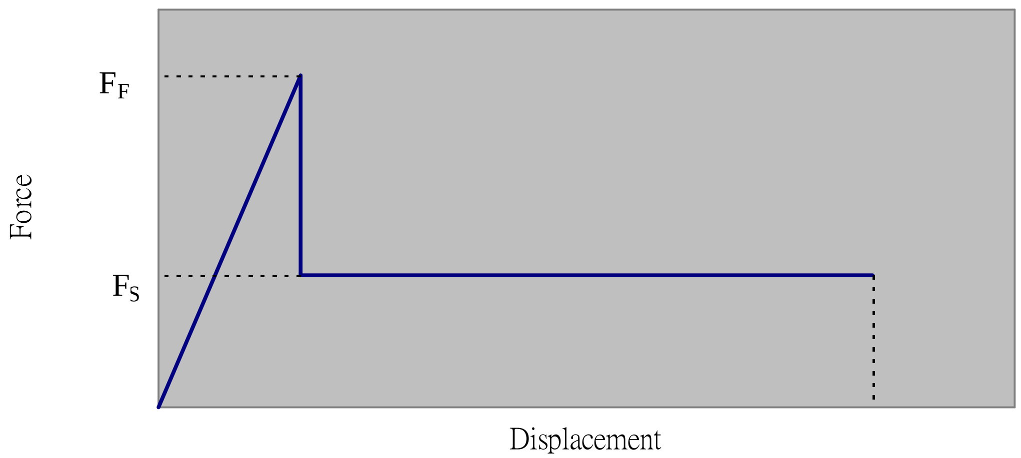


Figure 4-6 : Moment Curvature Curve for Bilinear Response



F_F = Ultimate capacity of bearing

F_S = Frictional sliding force of bearing (interface friction force)

Figure 4-7 : Fuse Performance of Bearing

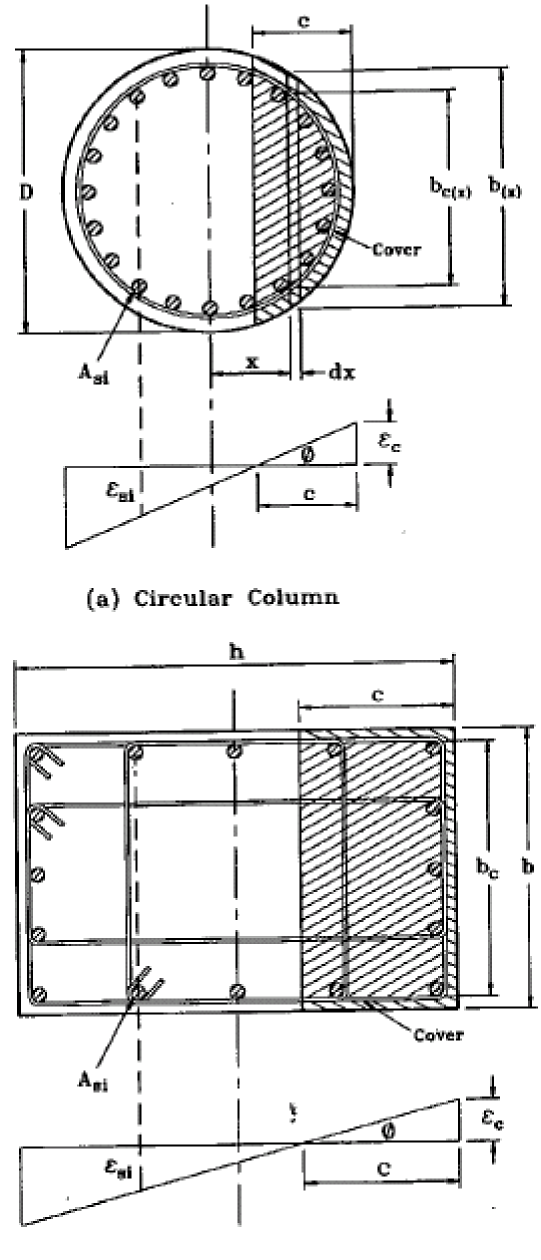


Figure 4-8 : Moment-Curvature Analysis of Column Section

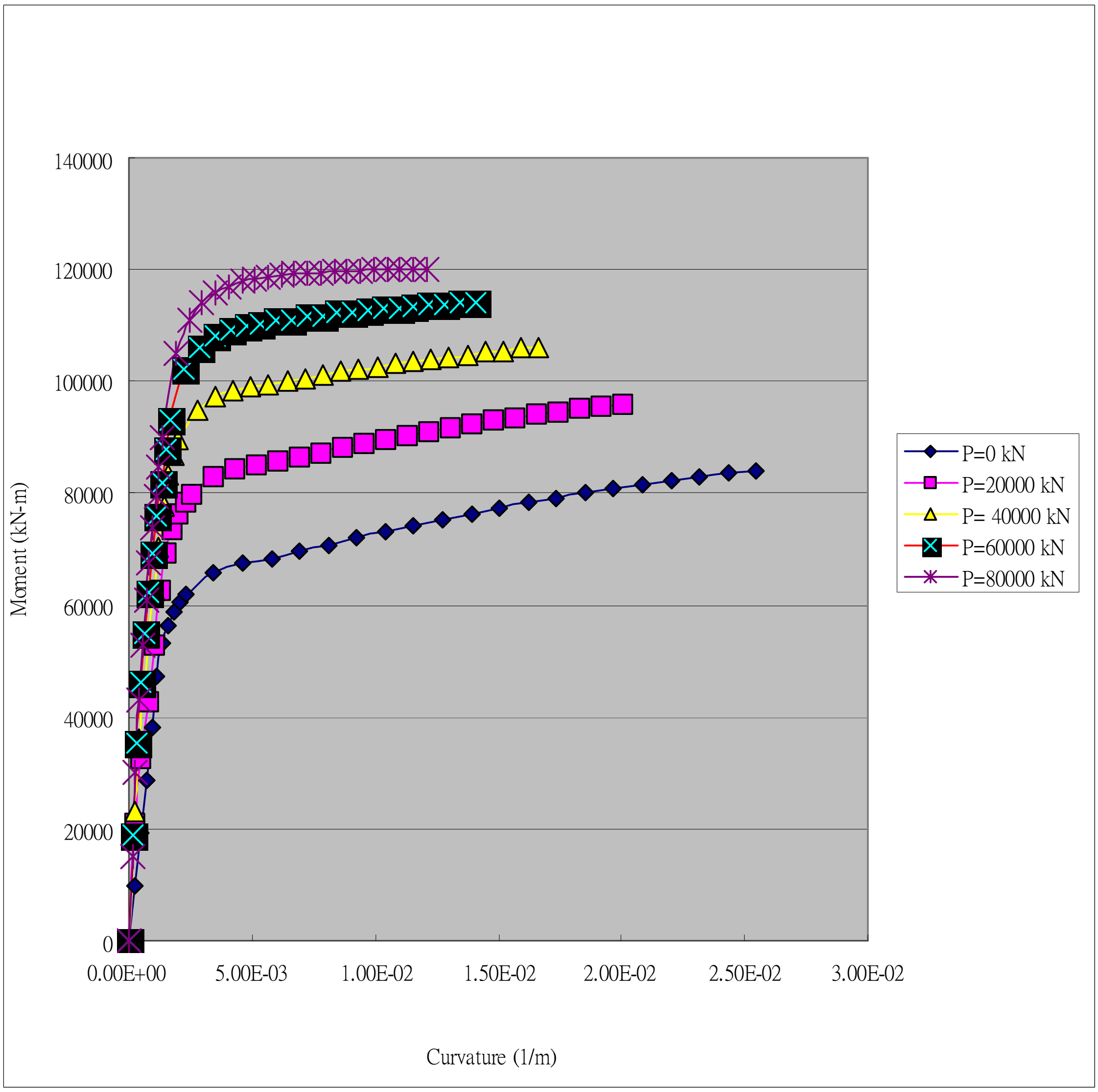


Figure 4-9 : Moment-Curvature Analysis with Various Axial Forces

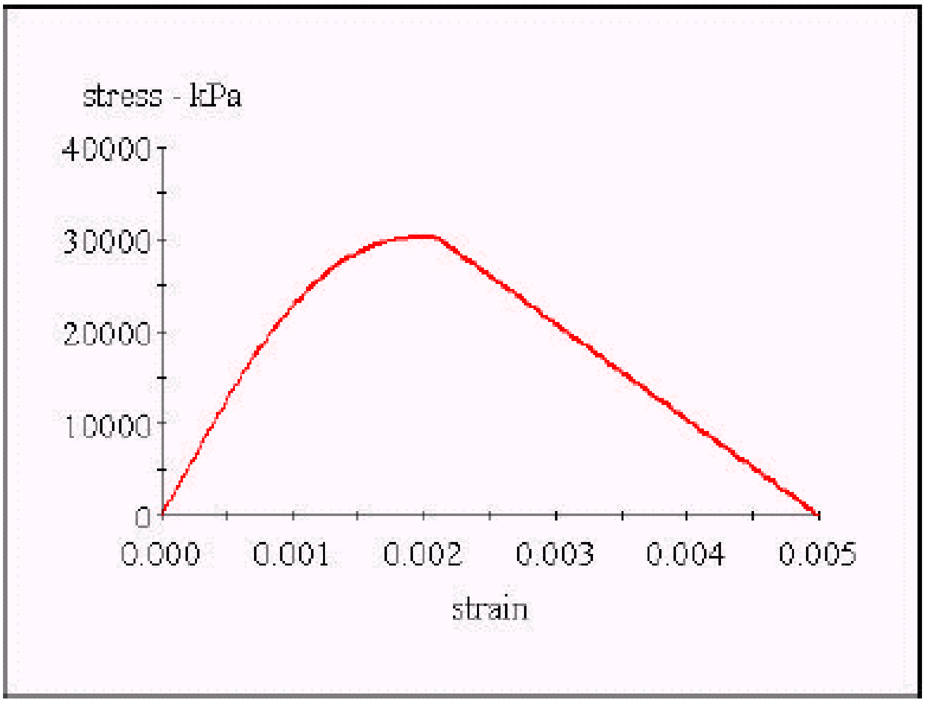


Figure 5-1 : Stress-Strain Diagram of Unconfined Concrete

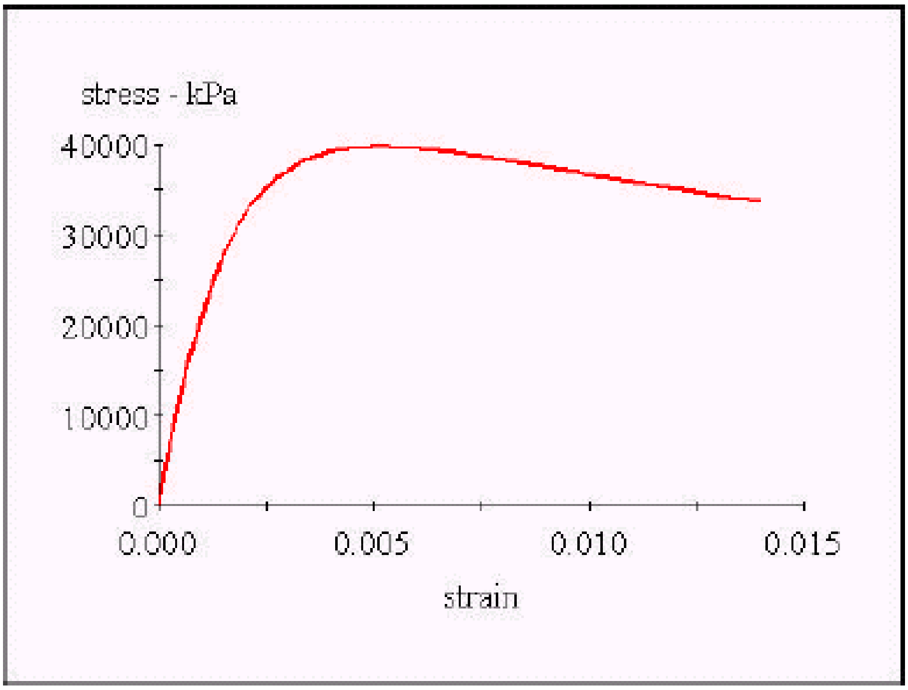


Figure 5-2 : Stress-Strain Diagram of Confined Concrete

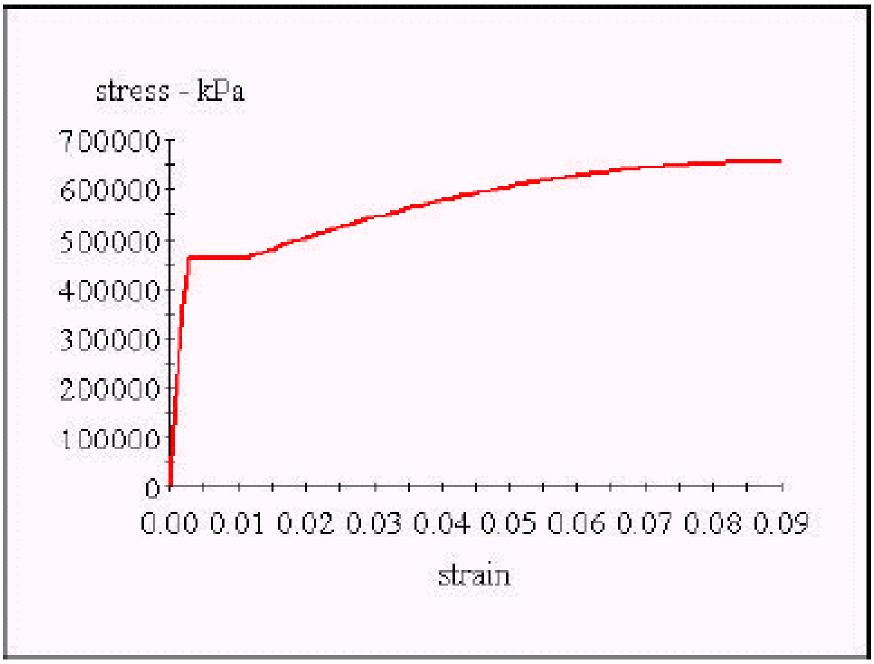


Figure 5-3 : Stress-Strain Diagram of Reinforce Steel

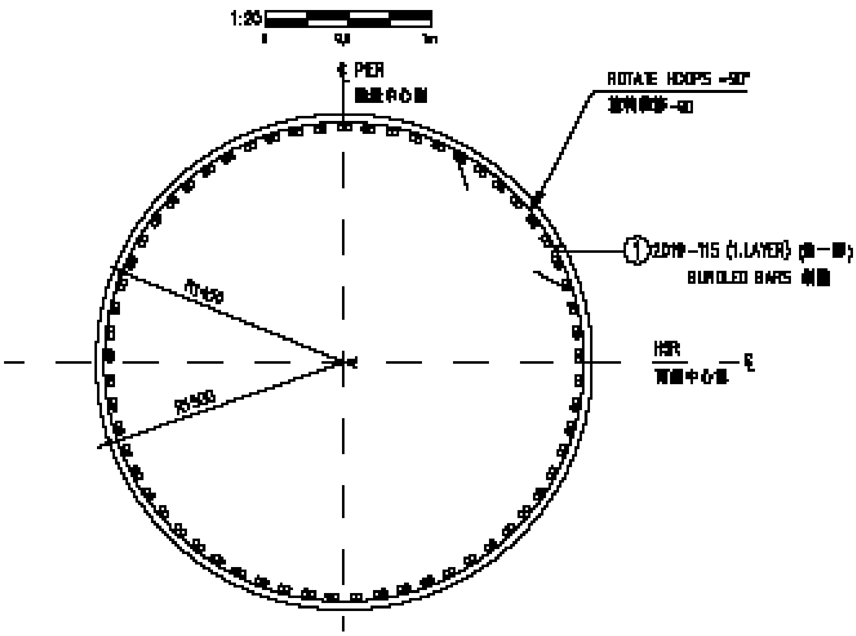


Figure 5-4 : Cross Section of Pier

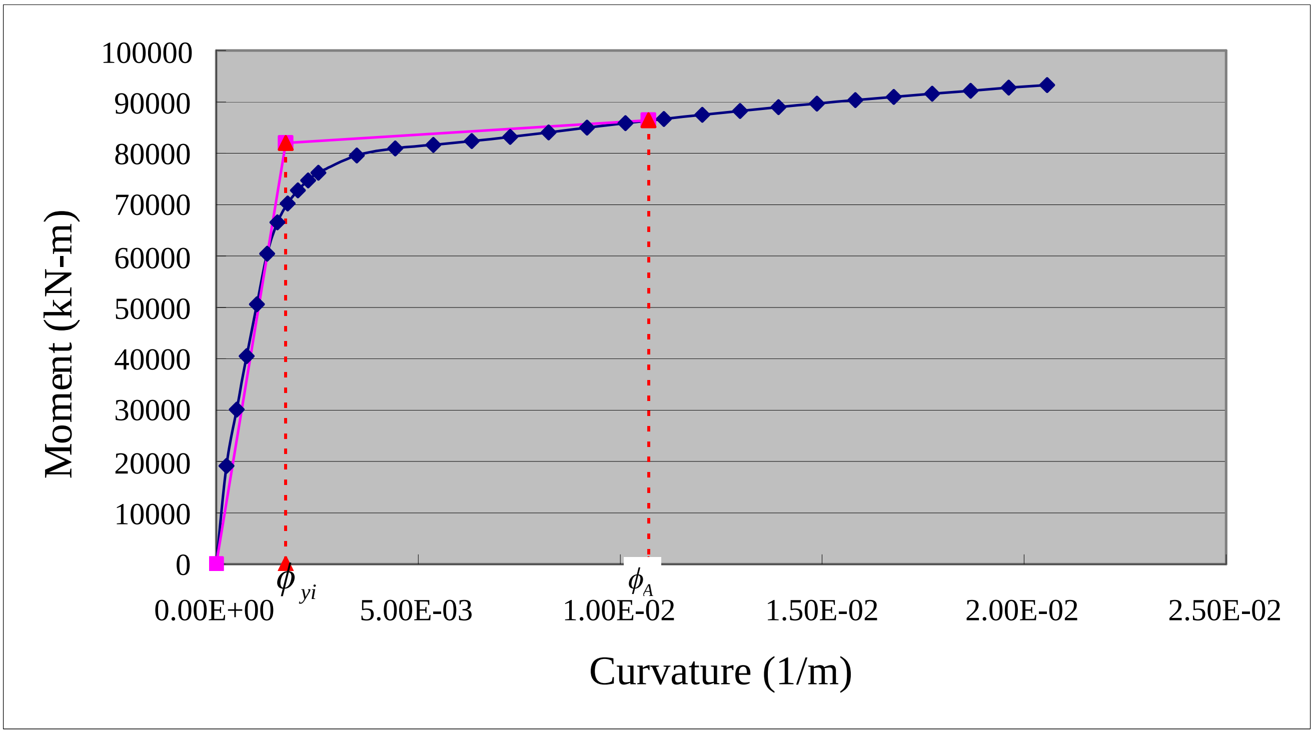


Figure 5-5 : Moment-Curvature Curve

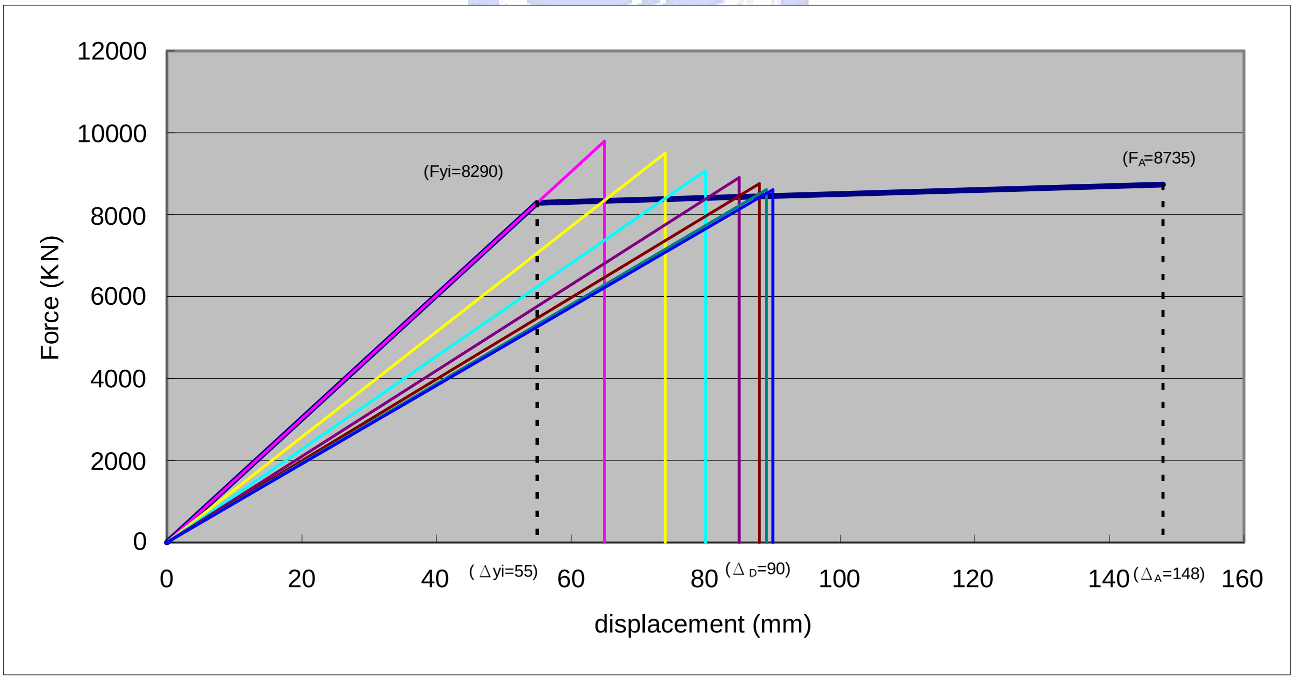


Figure 5-6 : Force Displacement by Substitute Structure Analysis Diagram

Appendix B: Table

Table 2-1 : Horizontal Normalised Acceleration Response Spectrum Coefficient for Different Soil and period

C Soil Profile Type	Extremely Short Period	Very Short Period	Short Period	Moderate Period	Long Period
Type I	T<0.03sec 1.0	0.03sec<T≤0.15sec 12.5T+0.625	0.15sec<T≤0.333sec 2.5	0.333sec<T≤0.941sec $1.2/T^{2/3}$	T>0.941sec 1.25
Type II	T<0.03sec 1.0	0.03sec<T≤0.15sec 12.5T+0.625	0.15sec<T≤0.465sec 2.5	0.65sec<T≤1.315sec $1.5/T^{2/3}$	T>1.315sec 1.25
Type III	T<0.03sec 1.0	0.03sec<T≤0.2sec 8.824T+0.7352	0.2sec<T≤0.611sec 2.5	0.611sec<T≤1.728sec $1.8/T^{2/3}$	T>1.728sec 1.25
Taipei Basin	T<0.03sec 1.0	0.03sec<T≤0.2sec 5.882T+0.824	0.2sec<T≤1.65sec 2	1.65sec<T≤3.3sec 3.3/T	T>3.3sec 1.0

Table 2-2 : Vertical Normalised Acceleration Response Spectrum Coefficient for Different Soil and period

C Soil Profile Type	Extremely Short Period	Very Short Period	Short Period	Moderate Period	Long Period
Type I	T<0.03sec 1.0	0.03sec<T≤0.1sec 25T+0.25	0.1sec<T≤0.288sec 2.75	0.288sec<T≤1.139sec $1.2/T^{2/3}$	T>1.139sec 1.1
Type II	T<0.03sec 1.0	0.03sec<T≤0.1sec 25T+0.25	0.1sec<T≤0.403sec 2.75	0.403sec<T≤1.592sec $1.5/T^{2/3}$	T>1.592sec 1.1
Type III	T<0.03sec 1.0	0.03sec<T≤0.1sec 25T+0.25	0.1sec<T≤0.530sec 2.75	0.530sec<T≤2.093sec $1.8/T^{2/3}$	T>2.093sec 1.1
Taipei Basin	T<0.03sec 1.0	0.03sec<T≤0.1sec 21.43T+0.357	0.1sec<T≤1.32sec 2.5	1.32sec<T≤3.3sec 3.3/T	T>3.3sec 1.0

Table 2-3 : Property Factor of Structure System, R*

Type	Substructure	R*
1	Wall-Type Pier	2
2	Single Pier	3
3	Multiple Pier Frame	5
4	RC Pile-Type Pier : Vertical Pile Inclined Pile	 3 2
5	Steel Pile/ Steel-Concrete Pile : Vertical Pile Inclined Pile	 5 3

Table 4-1 : Seismic Performance Criteria of Bridge Issued by SSRP-99/08,UCSD.

Level	Damage Classification	Damage Description	Repair Description	Socio-economic Description	Qualitative Performance Description	Ductility
I	No	Barely visible Cracking	No Repair	Fully Operation	Onset of hairline cracks	Elastic, <1
II	Minor	Cracking	Possible Repair	Operation	Theoretical first yield of Longitudinal reinforcement	1 to 2
III	Moderate	Open Cracking Onset of Spalling	Minimum Repair	Life safety	Initiation of inelastic deformation, Onset of concrete spalling Development of Diagonal cracks	2 to 4
IV	Major	Very wide cracks extended concrete spalling	Repair	Near Collapse	Wide Crack widths/spalling over Full location mechanism region	4 to 8
V	Local Failure /Collapse	Visible permanent deformation Bucking/rupture of reinforcement	Replacement	Collapse	Bucking of main reinforcement Rupture of transverse reinforcement	>8

Table 4-2 : Comparison of column response of fused vs. non-isolated strategies for a typical bent of a multi-span, multi-column structure on the Legacy Parkway Project.

Bearing properties	Non-Fused	Fused Bearings
	Pinned	**Fuse at 40%g**
Governing Column Moment demands (kips-ft)	38,000	8400
Column Plastic Moment Capacity (1.5% rebar, Kips-ft)	13249	13249
Foundation Size (footing, no. of piles) Seismic Load Design	21.3 ‘ x 21.3 ‘ 25-150 Ton Piles	20’ x 20’ 12-150 Ton Piles
Foundation Size (footing, no. of 150 ton piles) Service Load Design	18’ x 18’ 8-150 Ton Piles	18’ x 18’ 8-150 Ton Piles

Table 5-1 : Data of Moment Curvature

No.	Confined Concrete Strain	Steel Strain	Mxx	Kxx
	strain	strain	kN-m	1/m
1	-7.59E-05	0	0	-8.98E-21
2	-3.67E-04	3.44E-04	1.92E+04	2.53E-04
3	-5.93E-04	8.30E-04	3.01E+04	5.06E-04
4	-8.10E-04	1.33E-03	4.05E+04	7.59E-04
5	-1.03E-03	1.82E-03	5.06E+04	1.01E-03
6	-1.25E-03	2.31E-03	6.04E+04	1.27E-03
7	-1.45E-03	2.82E-03	6.65E+04	1.52E-03
8	-1.62E-03	3.36E-03	7.02E+04	1.77E-03
9	-1.78E-03	3.91E-03	7.28E+04	2.02E-03
10	-1.94E-03	4.47E-03	7.47E+04	2.28E-03
11	-2.09E-03	5.03E-03	7.62E+04	2.53E-03
12	-2.65E-03	7.14E-03	7.96E+04	3.48E-03
13	-3.20E-03	9.26E-03	8.10E+04	4.43E-03
14	-3.75E-03	1.14E-02	8.17E+04	5.38E-03
15	-4.30E-03	1.35E-02	8.24E+04	6.33E-03
16	-4.86E-03	1.56E-02	8.32E+04	7.28E-03
17	-5.42E-03	1.77E-02	8.41E+04	8.23E-03
18	-5.99E-03	1.98E-02	8.50E+04	9.18E-03
19	-6.55E-03	2.19E-02	8.59E+04	1.01E-02
20	-7.13E-03	2.40E-02	8.67E+04	1.11E-02
21	-7.72E-03	2.61E-02	8.75E+04	1.20E-02
22	-8.31E-03	2.82E-02	8.83E+04	1.30E-02
23	-8.91E-03	3.03E-02	8.90E+04	1.39E-02
24	-9.52E-03	3.23E-02	8.97E+04	1.49E-02
25	-1.01E-02	3.44E-02	9.04E+04	1.58E-02
26	-1.08E-02	3.64E-02	9.10E+04	1.68E-02
27	-1.14E-02	3.85E-02	9.16E+04	1.77E-02
28	-1.20E-02	4.05E-02	9.22E+04	1.87E-02
29	-1.27E-02	4.25E-02	9.28E+04	1.96E-02
30	-1.33E-02	4.46E-02	9.33E+04	2.06E-02
31	-1.40E-02	4.66E-02	9.38E+04	2.15E-02

Table 5-2 : Data of Substitute Structure Analysis

No.	1	2	3	4	5	6	7	8
Force(kN)	9795	9511	9072	8910	8755	8607	8607	8607
Δ(mm)	65	74	80	85	88	89	90	90
K(kN/m)	150730	128270	113260	105120	99220	96000	94970	93970
T(S)	0.68	0.73	0.75	0.77	0.79	0.79	0.79	0.79