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Reference Material for Design and Assessment of Bridges in Regions of Low to Moderate Seismicity – The Hong Kong Context

Reference Material for Design and Assessment of Bridges in Regions of Low to Moderate Seismicity – The Hong Kong Context

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FOREWORD

Hong Kong lies within the Eurasian Plate and is fortunate to be relatively remote from the Pacific Ring of Fire where many of the strongest earthquakes have occurred. Although the seismicity of Hong Kong is regarded as low to moderate, as a densely populated area in the region, Hong Kong has been preparing itself for possible earthquake hazards through a series of coordinated investigations. They include studies of seismic hazard in Hong Kong, review of earthquake data for the Hong Kong region and the subsequent studies on the introduction of seismic-resistant building design code for Hong Kong. The release of the 2013 version of *Structures Design Manual for Highways and Railways*, which has incorporated the state-of-the-art seismic design methodology of bridges, is a major step forward.

This design guide entitled "Design and Assessment of Bridges in Regions of Low to Moderate Seismicity – The Hong Kong Context" has been written to assist practitioners to appreciate the seismic provisions adopted in *Structures Design Manual for Highways and Railways 2013* from a holistic perspective, and to understand the fundamentals and up-to-date analytical techniques in the earthquake resistant design and assessment of bridges.

Since the establishment of The University of Hong Kong and the Faculty of Engineering in 1912, the Department of Civil Engineering has nurtured many brilliant leaders in the civil engineering discipline and made significant contributions to the local, Mainland China and overseas community. The Department is constantly looking ahead to enhancing its goals in education, research and community services in order to keep abreast of the ever-changing demands of modern society. In addition to offering a broad range of internationally competitive academic programmes and conducting high-impact research relevant to the needs of industry and society, the Department is also committed to supporting the industry and society through technology transfer and other forms of professional services.

I congratulate Prof. Francis T.K. Au for his leadership and visionary blueprint in developing this design guide to promote technological advancement in sustainable infrastructure development in Hong Kong. It is also a worthwhile effort that will surely benefit the engineering profession.

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PREFACE

This design guide has stemmed from release of the 2013 version of *Structures Design Manual for Highways and Railways*, which has underscored commitments of the Government of Hong Kong Special Administrative Region to enhancement of the civil infrastructure in respect of seismic design. With *Structures Design Manual for Highways and Railways 2013* taking effect, the seismic design of bridges in Hong Kong has been elevated to the international standard as exemplified by the structural Eurocodes. The increase from the previous nominal ground acceleration of $0.05g$ (where g is the acceleration due to gravity) to reference peak ground acceleration of $0.12g$ together with possible amplification due to subsoil conditions implies that the current design seismic action can be many times that of the previous value. Even though the scenarios with design seismic actions may or may not be those controlling the design of a particular structure, the aforementioned enhancement of the design code will call for a complete revamp of design strategies instead of just allowing for a higher design seismic action purely based on strength consideration as in the previous simplistic design approach. Proper seismic design should address not only strength and stiffness, but also ductility and deformability in order to achieve safety under extreme events. To help ensure a smooth transition to the state-of-the-art seismic design methodology, it is desirable to develop a user-friendly set of practical guidance that allows engineers to make informed decisions even from the initial conceptual design stage for compliance with the current requirements.

With the support of the Construction Industry Council of Hong Kong, together with contributions from members of the research team in the past 36 months, this design guide has been prepared taking into account the local conditions and specific design requirements, established engineering practices including those prevalent locally, in Mainland China and overseas, effective measures to ensure seismic resistance, buildability, increased use of prefabrication for better quality assurance and economy, as well as practicality of long-term asset management and maintenance. I sincerely hope that readers find this design guide useful.

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EXECUTIVE SUMMARY

This design guide has been prepared in response to the recent adoption of *Structures Design Manual for Highways and Railway 2013* (SDMHR 2013), aiming to facilitate a smooth transition to the state-of-the-art seismic design methodology as introduced by the structural Eurocodes and to encourage a holistic approach from the initial conceptual design stage for compliance with the current requirements. It compares the performance requirements and the corresponding design seismic hazard levels currently adopted by SDMHR 2013 in relation to those used by other countries and regions. It organizes the seismic bridge design specifications as prescribed in the Eurocodes BS EN 1998-1 and BS EN 1998-2 with modifications by SDMHR 2013, where relevant, in a logical manner and also provides guidance on the choice of earthquake-resisting systems, articulations and intended seismic behaviour based on the established engineering practices in other countries and regions as well as some additional findings of this project. The topics covered in each chapter are as follows:

Chapter 1 introduces the background of this design guide, highlighting the major revisions of SDMHR 2013 on seismic bridge design as compared to the earlier SDMHR 3rd edition, and identifies the typical characteristics of existing bridges in Hong Kong that form the basis of this design guide.

Chapter 2 presents a general introduction to the seismic design of bridges based on a comprehensive review of several major seismic bridge design codes from other countries and/or regions, including their development history, design philosophy, major compliance approaches, methods of seismic demand analysis, and representation of seismic action.

Chapter 3 summarizes the seismic bridge design specifications for Hong Kong, including those for highway and railway bridges, in a sequential order of seismic design procedures followed by a flowchart.

Chapter 4 provides guidance on the seismic design of new bridges at the conceptual design stage in respect of the selection of earthquake-resisting systems, articulations, configurations and intended seismic behaviour, i.e. elastic, limited-ductile or ductile behaviour, by examining the relation between the choices and the design and response of structures under seismic actions.

Chapter 5 describes the preliminary compliance assessments of typical existing highway bridges (previously designed to older codes) with reference to the current requirements of SDMHR 2013 in regard to the design seismic actions and structural details. It then presents the methodologies for performance-based seismic structural assessment of bridges and the development of fragility curves of some typical classes of bridges in Hong Kong for the purpose of rating of existing bridges.

Chapter 6 presents some of the latest trends of bridge engineering in respect of material, construction method and structural system, with special emphasis on the implications of these developments on seismic bridge design and retrofitting.

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CHAPTER 1 BACKGROUND

Hong Kong has a highly developed and sophisticated transportation network with approximately 2,101 km of roads, supporting 732,000 vehicles, and densely packed on its 1,104 sq. km of territory. Any disruption to the transportation network is likely to cause significant social and economic impact. Earthquake damage in recent decades around the world has revealed that bridges are one of the most vulnerable components of the transportation system under seismic actions. As of September 2016, there are 775 footbridges and 1,351 road bridges that form essential components of the highway network of Hong Kong (Highways Department, 2016a, 2016c). Based on the Travel Characteristics Survey of the Hong Kong Government, over 90% of the journeys made are by public transport, in which railway transportation accounts for 41% (Highways Department, 2016b). Although much of the railway system runs in underground tunnels, some viaducts above ground still carry part of the railway traffic. Furthermore, two iconic long-span bridges, i.e. Tsing Ma Bridge and Kap Shui Mun Bridge, carry the Airport Express Line on their lower decks. It is therefore essential that these bridges shall be designed for sufficient earthquake resistance.

The release of Structures Design Manual for Highways and Railways 2013 (SDMHR 2013) (Highways Department 2013) that follows the paradigm shift from the British Standards to the Eurocodes has significant impact on the design of new bridge structures and modification of the existing structures, especially in respect of the seismic design. For new bridges, there are differences in both analysis and design as compared to those in accordance with SDMHR 3rd edition. To reap the full benefits of the contemporary seismic design methodology as exemplified by the structural Eurocodes, practising engineers need to upgrade their mindset and design approach. For the existing bridges designed to earlier codes, the concern is whether they would perform satisfactorily at the level of seismicity as specified in the new code. Strengthening may be necessary in case the necessary level of seismic resistance is not reached.

This chapter attempts to set the basis for practical guidelines for seismic design and assessment of bridges in Hong Kong. Section 1.1 presents an overview of the bridges in Hong Kong, including the development history, prevalent structural types and characteristics. The seismic performance of bridges depends largely on the structural properties of bridge and seismology at the bridge site. Identifying and characterizing typical bridges is critical for derivation of the seismic behaviour of representative bridges and development of regional bridge fragility curves applicable to Hong Kong. Section 1.2 reviews the major revisions of seismic bridge design provisions brought about by SDMHR 2013, focusing on the upgrade of design seismic intensity and design philosophy. Finally, Section 1.3 describes the aims and scope of this design guide.

1.1 Bridges in Hong Kong

1.1.1 Development history

The major phase in the development history of the built environment in Hong Kong dated back to the 1950s, when the rapid economic and urban development in the post-war period necessitated the development of transport infrastructure. Highways and bridges were needed to improve the accessibility to the new towns and new areas developed to accommodate the booming population. Before the 1960s, footbridges were built mainly to cross difficult terrains. However, the rapid urban development then led to the need for grade separation of pedestrian and road traffic for efficiency. The first footbridge for such a purpose was constructed in 1963, crossing Leighton Road near Victoria Park (Highways Department, 2016a). During this period, cast in-situ span-by-span method was commonly used for road bridge construction.

The 1970s saw the first generation of Mass Transit Railway development. The first line connecting Shek Kip Mei to Kwun Tong was opened in 1979. Part of the line is supported on viaducts. Furthermore, the new town development continued to grow in the New Territories with the demand for new roads and associated bridges (Highways Department, 2016c). The cast in-situ balanced cantilever construction method was first used in the 1970s. The Tsing Yi Bridge (now known as Tsing Yi South Bridge), opened in 1974, was the first bridge in Hong Kong built using this method. The Tsing Yi Bridge, connecting Tsing Yi Island with Kwai Chung, contributed greatly to the development of Tsing Yi Island (Highways Department, 2016c). Another example was the Ap Lei Chau Bridge, which was opened in 1980. Both of these bridges cross over navigation channels, making cast in-situ balanced cantilever a more suitable construction method than the span-by-span method. The use of precast components also started in the 1970s. A commonly used structural form since the 1970s involved the use of precast beams with cast in-situ concrete slab. The precast beams may be I-beams or U-beams. The first instance of its use in Hong Kong was Canal Road Flyover, which was opened in 1972. Figure 1.1 shows Canal Road Flyover, where the precast I-beams placed at regular intervals can be seen clearly.

Figure 1.1 Canal Road Flyover

In the late 1980s, precast segmental construction method was first used in Kwun Tong Bypass (Figure 1.2), which was opened in stages from 1989 to 1991. This method is a variation of the cast in-situ balanced cantilever construction method to facilitate faster construction with less disruption to the traffic below, yet it overcomes the limitation on span length that precast span-by-span method will normally have. Moreover, footbridges also evolved from simple crossings to elevated walkway systems (Highways Department, 2016c).

Figure 1.2 **Kwun Tong Bypass**

There were barely any long-span bridges before the 1990s. The 1990s saw the construction of several long-span bridges, including Tsing Ma Bridge (a suspension bridge), Kap Shui Mun Bridge (a cable-stayed bridge) and Ting Kau Bridge (also a cable-stayed bridge). These bridges serve to connect the new Hong Kong International Airport to the rest of Hong Kong and to facilitate the Territorial Development Strategy. In addition to the long-span bridges, viaducts were erected for the same purpose, such as West Kowloon Expressway, North Lantau Expressway and Tsing Yi North Coastal Road.

The major bridges completed in the 2000s include Stonecutters Bridge and Shenzhen Bay Bridge. The Shenzhen Bay Bridge, a 5.5 km long dual three-lane carriageway bridge opened in 2007, serves as the fourth vehicular boundary crossing between Hong Kong and Mainland China. The bridge consists of a series of concrete viaducts and two cable-stayed bridges above the navigation channels. Stonecutters Bridge is a cable-stayed bridge with a main span of 1,018 m and two back spans of 289 m. It was one of the two bridges of its form with a main span in excess of 1,000 m upon its completion in 2009.

In the 2010s, one of the major infrastructure projects involving bridge construction is the Hong Kong–Zhuhai–Macao Bridge.

1.1.2 Structural forms

The design of a bridge is governed by many factors, including but not limited to its intended functions, the site constraints, the materials and labour available, economy and aesthetics. Though each bridge takes on its unique form, all bridges boil down to several basic types, some of which found in Hong Kong are described below.

(1) Arch Bridges

An arch is a curved member supported in such a way that intermediate transverse loads are transmitted to the supports primarily by axial compressive forces in the arch rib. The arch system efficiently utilizes the compressive strength of materials that are strong in compression but possibly weak in tension, such as concrete and masonry. Arch bridges can also be built of steel.

While arch bridges have been built since antiquity, it is not a structural system commonly seen in Hong Kong mainly because of the need for supports capable of providing sufficient horizontal thrust. Nevertheless, tied arch bridges are used in some footbridges (Figure 1.3).

Figure 1.3 A tied arch footbridge at Wong Tai Sin

(2) Truss Bridges

A truss is a triangulated assembly of straight members. The applied loads are resisted primarily by axial forces in the truss members. As the truss members may take tension or compression, most truss bridges are made of steel.

Although very few road bridges in Hong Kong are purely of truss form, trusses are often used as the stiffening girders of suspension bridges for their rigidity and lightness. Moreover, a significant proportion of footbridges in Hong Kong are of truss form due to its efficient use of material and lightness (Figure 1.4).

Figure 1.4 A truss footbridge over Kwun Tong Road

(3) Girder Bridges

Girder bridges, which primarily utilize the bending and shearing actions of the decks, have been the most commonly used structural system for a few decades. Multi-span viaducts are typical examples of this structural form. There may be many possible variations in respect of the articulation. Many of the early girder bridges are either made up of a series of simply supported spans or of Gerber beam configuration comprising numerous half joints. Although their design is relatively simple, maintenance may not be convenient. With the growing awareness of the need for convenience of maintenance and its cost, continuity is often preferred in the modern design of girder bridges. The girder can take on a great variety of forms and materials. Commonly used structural systems include steel plate girders, composite steel and concrete construction, prestressed concrete box girders, etc. Many design approaches are possible in the connections between the bridge deck and substructures, and they may affect the seismic performance of the whole bridge. Figure 1.5 shows an example of a girder bridge.

Figure 1.5 A typical girder bridge

(4) Suspension Bridges

The suspension bridge is the structural form that is capable of spanning the longest distance to date. Cables, with high strength-to-weight ratios, are highly efficient as the primary load-carrying elements. The basic structural components of a suspension bridge system include the main cables, main towers, anchorages, suspenders and stiffening trusses. The stiffening trusses act as the bridge deck that supports and distributes the traffic loads, and also contributes to the aerodynamic stability. It should be noted that steel is typically used for the deck structure of a suspension bridge to minimize the dead weight. High-strength steel wires forming the main cables act in tension to support through the suspenders the weight of stiffening trusses and traffic loads. The main towers as intermediate supports for the main cables help transfer the loads to the foundation. Massive concrete anchorages are normally provided to resist the pull from the cables (Okukawa *et al.*, 2014).

There is only one suspension bridge in Hong Kong, i.e. Tsing Ma Bridge, which is shown in Figure 1.6.

Figure 1.6 Tsing Ma Bridge (Courtesy of Highways Department)

(5) Cable-stayed Bridges

The cable-stayed bridge is the more recent development in bridge systems. Similar to the suspension bridge, it utilizes the high tensile strength of steel cables, but in a different way. The cable-stayed girder bridge consists of a main girder system at deck level, supported on abutments and piers, and in addition by a system of nearly straight cables emanating from the towers and anchored at the main and approach spans. The deck structure of a cable-stayed girder bridge can be steel, concrete or composite, depending on the cost-benefit analysis of the specific project.

A cable-stayed bridge consists of a number of triangles, comprising the tower, the deck and the cables. If relatively closely spaced stay cables are provided, the loads are mainly transferred as axial forces rather than bending, making the structure more efficient. The compressive axial forces in the towers and in the deck are balanced by the axial tension in the stay cables. The dead load of the deck structure would also pre-tension the stay cables, thereby increasing the stiffness of the structural system (Vejrum and Nielsen, 2014). One of the advantages of cable-stayed bridge is that the structure is usually self-anchored and there is no need for large anchorage, as in the case of a typical suspension bridge. This makes the cable-stayed bridge a good solution at locations where the soil conditions are unfavourable and the foundation cost would be excessive (Vejrum and Nielsen, 2014).

The major cable-stayed bridges in Hong Kong include Stonecutters Bridge, Kap Shui Mun Bridge, Ting Kau Bridge and Shenzhen Bay Bridge. Figure 1.7 is a photograph of Kap Shui Mun Bridge. Apart from the long-span road bridges mentioned above, the cable-stayed bridge system is versatile and can also be used for footbridges, as shown in Figure 1.8.

Figure 1.7 Kap Shui Mun Bridge (Courtesy of Highways Department)

Figure 1.8 Pedestrian cable-stayed bridge over West Kowloon Highway at Olympic

1.1.3 Small- to medium-span bridges

Short- to medium-span girder bridges constitute the most prevalent bridge form in Hong Kong. This section mainly reviews the local practices in the past four decades for this category of bridges, including the design process, common structural configurations and design parameters.

(1) Design process

The design of bridges, like any structural design, is iterative in nature. It is essentially a continuous refining process as illustrated in Figure 1.9. One starts by gathering all the relevant information on the project, including but not limited to its intended use, alignment, economy, site conditions and environmental impact. The initial ideas funnel down to the conceptual design, including the choice of structural form and material, deck section, articulation, bearing arrangement, and choice of substructure and foundation. The conceptual design materializes through dimensioning member size, structural analysis and detailed design. It is also important to take into consideration construction, operation and maintenance in the design process. The initial design ideas may need to be reviewed and revised in the design process. A bridge design is the end product of such a continuous refining process. While the general process of bridge design follows the same rationale, the design practice and workflow vary from firm to firm. Hong Kong engineers, through the rapid urban development, have accumulated rich experience in bridge design.

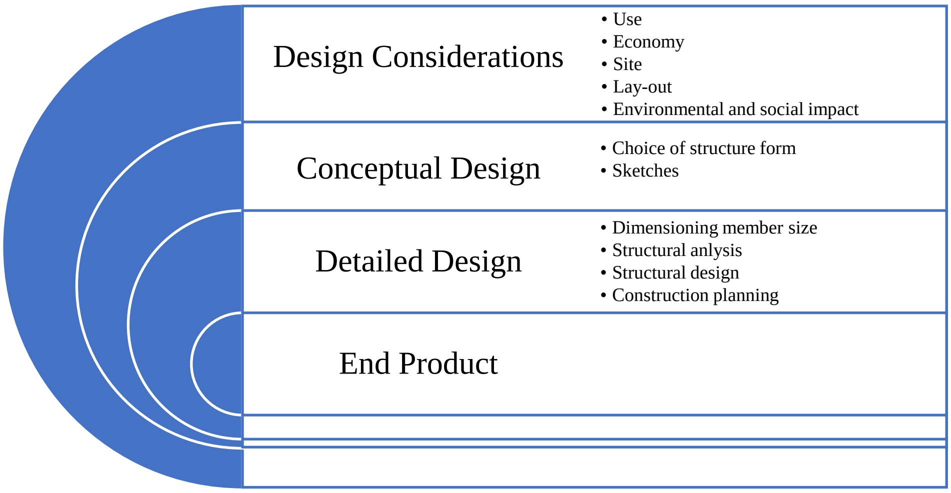


Figure 1.9 Illustration for typical design process

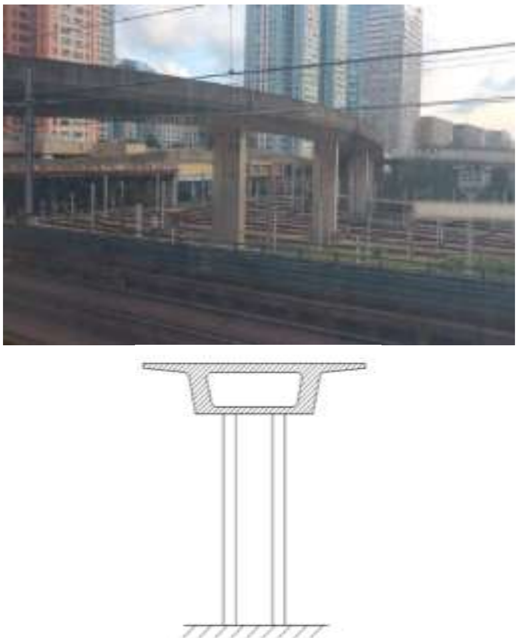
(2) Forms of structural components

The beam-and-slab and concrete box girder are two of the most common forms of superstructure. The beam-and-slab bridges are constructed with precast span-by-span method. Some typical precast beam sections include I-beam, M-beam and U-beam. This structural form was popular during the 1970s and 1980s. The first beam-and-slab bridge in Hong Kong is Canal Road Flyover as shown in Figure 1.1. More examples include some spans in West Kowloon Corridor and Island Eastern Corridor as shown in Figure 1.(a) and Figure 1.(b) respectively. Box girder sections have become popular with its higher torsional stiffness and more efficient distribution of load compared to beam-and-slab sections. Early box girder bridges in Hong Kong consisted of a variety of designs, including single-cell, twin-cell and multi-cell box girders. Some of the examples are shown in Figure 1.(c) to Figure 1.(f). However, in view of the convenience in construction, the single-cell box girders have become more popular in recent years.

The bridge deck is supported along its length on piers at appropriate locations and on abutments at the end, either by bearings or monolithically. Depending on the type and arrangement of the superstructure, the pier also has a variety of representative configurations as shown in Figure 1.. The cross-section of pier may be circular, rectangular or octagonal.

(a) West Kowloon Corridor Span 7/1N-7/27N:
M-beam deck, “Y”-shaped pier

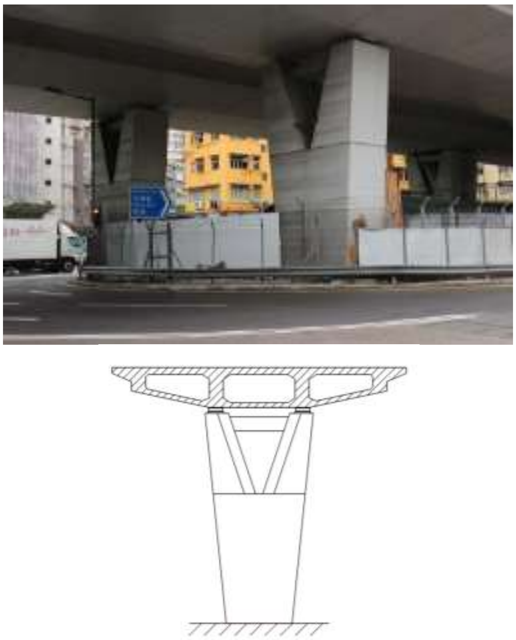
(b) Island Eastern Corridor:
U-beam deck, twin-column bent pier



(c) Kwun Tong Bypass Phase I K70E:
single-cell box girder, single-column pier

(d) Kwun Tong Bypass Phase II K77L:
single-cell box girder, single-column pier

Figure 1.10 Common bridge forms of small- to medium-span bridges in Hong Kong

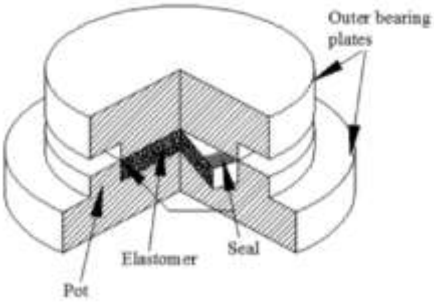


(e) West Kowloon Corridor Span 7/28–7/45:
multi-cell box girder, “Y”-shaped pier

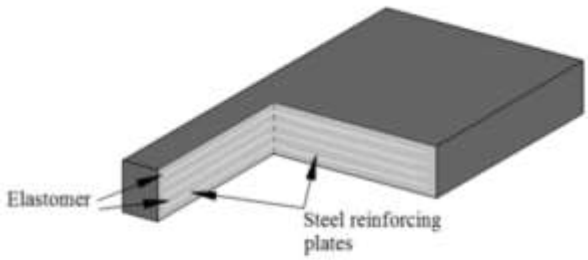
(f) Kwun Tong Bypass Phase III K76:
twin-box girder, “Y”-shaped pier

Figure 1.11 Common bridge forms of small- to medium-span bridges in Hong Kong (continued)

Bearings transmit mainly vertical loads from the bridge deck to the substructure, while restraining movement in some directions and allowing movement in others. Depending on the design of a bearing, it may also be expected to resist horizontal loads resulting from various actions. The most popular bearing type is the pot bearing (a kind of mechanical bearing) which can take relatively high loading, as shown in Figure 1.(a). The second most popular is the elastomeric bearing that is normally used for lighter loading. Most elastomeric bearings used in bridges are laminated and consist of steel reinforcing plates, as shown in Figure 1.(b).



(a) Mechanical bearing



(b) Elastomeric bearing

Figure 1.12 Common types of bearings

Movement joints are normally provided in the longitudinal direction of a bridge as a means of releasing the stresses arising from induced deck deformations due to thermal actions, shrinkage and other actions. To accommodate the relatively large movement between adjoining bridge decks or between the bridge deck and abutment, the bearings at movement joints are usually a combination of the elastomeric bearings and plain sliding bearings consisting of low friction polytetrafluoroethylene (PTFE).

In the earlier bridges in Hong Kong, ease of maintenance was not accorded high importance as today. There were some connection details which may cause problems with maintenance. They include the extensive use of half joints in Gerber beam type of bridges, use of dowel bars for structural fixity, bearing details that do not allow convenient access and replacement, etc. Apparently these details are no longer adopted in modern bridge designs. Nevertheless, the maintenance of the existing bridges constructed several decades ago will still be a major challenge for bridge engineers.

Occasionally, the piers are designed to be monolithic with the bridge deck, thereby eliminating the need for bearings and movement joints there. The bridge structure should have been designed to take the stresses induced by restrained movement. Figure 1. shows one example of such bridges. As bearings and movement joints require a lot of maintenance efforts, there is increasing preference for monolithic construction.

Figure 1.13 Flyovers over Castle Peak Road

Deep foundations are normally used to support bridges in Hong Kong. The selection of pile type depends on the specific project requirements, site conditions, environmental considerations and engineering judgment. Spread footings can be used in some cases to support bridges, e.g. availability of good bearing stratum or bedrock at shallow depth.

(3) Values of design parameters

Based on an initial survey of as-built bridges in Hong Kong, Figure 1. shows the typical ranges of the span lengths and Figure 1. shows those of the carriageway widths for common types and arrangements of superstructure. Furthermore, the number of continuous spans is found to vary between three and six spans.

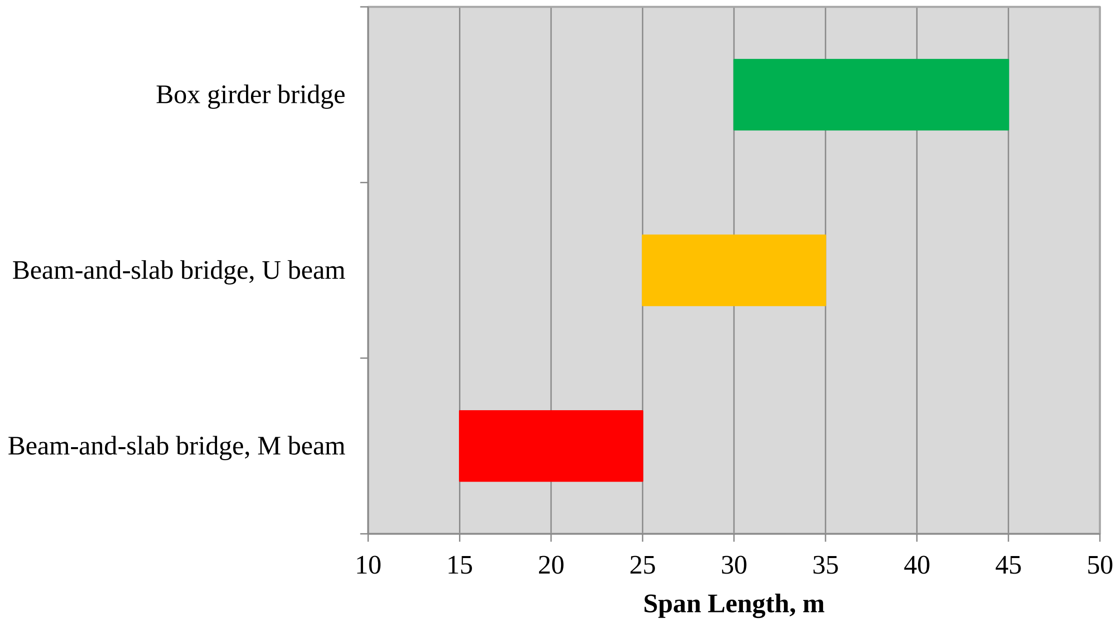


Figure 1.14 Typical span lengths for common structural forms

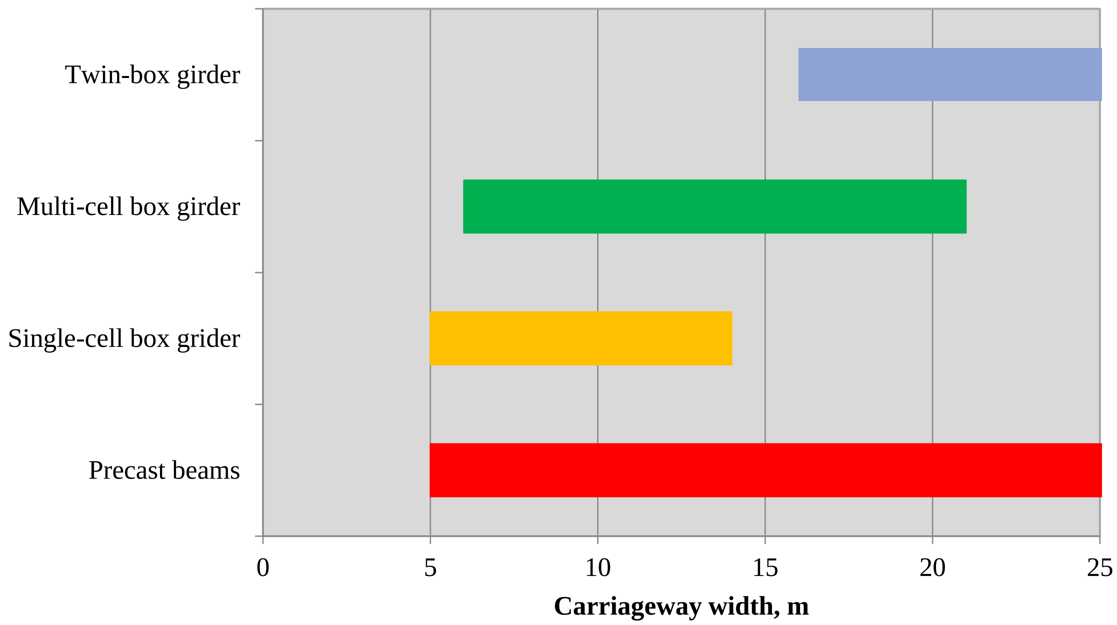


Figure 1.15 Typical carriageway widths for common structural forms

1.2 Revision of Structures Design Manual for Highways and Railways

1.2.1 General

The SDMHR has been providing guidance for the design of highway and railway structures in Hong Kong since its first publication in August 1993, with the second and third editions released in November 1997 and August 2006, respectively. The latest version was published in May 2013 (Highways Department, 2013). In the 2013 edition, the Manual has been revised for migration from British standards to Eurocodes. It is stated in Clause 1.1(4) of SDMHR 2013 that:

"Eurocodes shall be used for the design of new and modification of existing highway structures and railway bridges, but not for the structural assessment of existing structures, unless agreed with the Chief Highway Engineer/Bridges and Structures."

The publication of SDMHR 2013 would have significant impact on the design of new highway and railway bridges and modification to the existing structures. The provisions given in BS EN 1998-1 (BSI, 2004c) and BS EN 1998-2 (BSI, 2005b), in particular, shall be followed for the design for earthquake resistance.

There are differences in both analysis and design for earthquake resistance. The new SDMHR 2013 adopts the dynamic approach given in the Eurocodes using response spectrum analysis as the reference procedure, while the previous SDMHR 3rd has adopted a relatively straightforward equivalent static force approach. In respect of design, the change in code of practice calls for new considerations in conceptual design and more stringent detailing rules. The fundamental modifications are summarized in Table 1.1.

Table 1.1 Key differences for seismic design between SDMHR 3rd and SDMHR 2013

	SDMHR 3rd	SDMHR 2013
Importance class	N/A	Importance Classes I, II and III
Ground type	N/A	Ground Types A, B, C, D and E
Ground acceleration	0.05*g*, where *g* is the acceleration due to gravity	Depending on the structural vibration period (T), Importance Class (I, II or III) and site conditions (A, B, C, D or E), with a reference value of 0.12*g* for infinitely stiff structures (T = 0) of Importance Class I on Ground Type A
Components of seismic action	Transverse and longitudinal	Transverse, longitudinal and vertical (if applicable)
Method of analysis	Equivalent static force method	Response spectrum method (with allowance for alternative methods)
Combination with other loads	• Permanent loads, with a partial factor of 1.2 for dead load, 1.75 for surfacing and 1.5 for other super-imposed dead load; • 1/3 HA traffic load of United Kingdom Highways Agency's Departmental Standard BD 37/01 on one notional lane in each direction, with a partial factor of 1.25; and • Nominal seismic force with a partial factor of 1.4.	• Permanent loads including dead load and super-imposed dead load at their characteristic values; • 20% of Load Model 1 traffic load of BS EN 1991-2 on each of the notional lanes and remaining areas; • Seismic action; and • No partial factors need to be applied for seismic combination.
Combination of components	The design seismic force shall be applied successively longitudinally and transversely at footing level and to the superstructure, making four loading conditions to be considered in all.	The seismic action is applied at top of footings, or relevant surfaces of footings in case the soil stiffness is taken into account, separately in the longitudinal, transverse and vertical (if applicable) directions of the bridge. The probable maximum action effect due to the simultaneous occurrence of the components of seismic action may be estimated through application of the SRSS rule or the 30% rule expressed by Equations (4.18)-(4.22) in BS EN 1998-1.

1.2.2 Revised seismic intensity

The ground acceleration of 0.05*g* combined with a partial factor of 1.4 as prescribed in SDMHR 3rd was based on the seismicity records for Southern Guangdong implying that structures built in Hong Kong to withstand ground accelerations of 0.07*g* would probably have survived all the earthquakes recorded in Guangdong since 288 AD.

SDMHR 2013 has specified a reference peak ground acceleration on Type A ground (i.e. essentially rock) of 0.12*g*, which corresponds to a reference return period of 475 years based on comprehensive probabilistic seismic hazard analyses for Hong Kong (Atkins, 2012, 2013). Aside from the reference return period of 475 years, longer return period (i.e. 1,000 years and 2,500 years) may be considered for critical infrastructures through the introduction of importance factor. The classification of importance classes and values of importance factors as specified in Table 4.1 of SDMHR 2013 are shown in Table 1.2.

Table 1.2 Importance Classes and Importance Factors specified by SDMHR 2013

Importance Class	Importance Factor	Relevant Highway Structures
Class I	1.0	All highway structures not under Importance Class II or III.
Class II	1.4	When any one of the following conditions is met: • on traffic sensitive routes (Red and Pink Routes); • on public transport sensitive routes; or • on expressway.
Class III	2.3	When any one of the following conditions are met: • any span length > 150 m; • on expressway with total length > 1000 m; or • critical for maintaining communications, especially in the immediate post-earthquake period (e.g. on sole access routes to hospital).

Note: The importance factors of 1.0, 1.4 and 2.3 correspond to return periods of 475, 1,000 and 2,500 years, respectively.

For new bridges, depending on the vibration period of the specific structures, their classes of importance and the conditions of the sites where they are located, the seismic design forces based on the SDMHR 2013 in accordance with Type 2 response spectra of BS EN 1998-1 (BSI, 2004c) would, under most circumstances, be significantly larger than those derived based on SDMHR 3rd.

1.2.3 From static approach to dynamic approach

For the seismic design of a bridge, structural analysis should be performed to evaluate the structural behaviour under seismic actions and to provide the information necessary for the design, such as forces, moments and deformations. The seismic analysis can be classified as "static" or "dynamic", depending on whether the earthquake loading is treated as equivalent static forces or time-dependent forces.

A static seismic effect method is specified in SDMHR 3rd. The method has the simplified assumptions that the structure is perfectly rigid and moves at same pace with the ground motion. Thus the seismic force is simply the product of the effective mass and ground acceleration. A nominal earthquake load equivalent to 5% of the total vertical load is specified for design together with partial factors of 1.4 for ultimate limit state and 1.0 for serviceability limit state.

In SMHDR 2013, the standard procedure for seismic analysis as recommended in BS EN 1998-2 is response spectrum analysis. This method consists in mainly an elastic calculation of the peak dynamic responses of all significant modes of the structure using the ordinates of the site-dependent design spectrum. The overall response is then obtained by statistical combination of the maximum modal contributions. The direct dynamic methods, such as nonlinear time-history analysis that explicitly includes the nonlinear properties of the members based on step-by-step integration of the equations of motion in connection with ground motion time-histories, may be required for analysis of irregular structures.

1.2.4 From elastic design to ductile design

The design philosophies of SDMHR 3rd and SDMHR 2013 are essentially different. SDMHR 3rd does not require explicit analysis for inelastic behaviour of the bridge under various kinds of loads, including seismic loads, whereas BS EN 1998-2 that forms an important basis of SDMHR 2013 requires that the bridge shall be designed for either ductile or limited ductile behaviour under the design seismic actions. In contrast to SDMHR 3rd, the seismic design philosophy of SDMHR 2013 consists of three distinctive components:

- The bridge of ductile behaviour shall be provided with reliable means to dissipate a significant amount of the input energy under earthquakes. This can be accomplished by providing for the formation of flexural plastic hinges or by using isolation devices.
- The intended formation of flexural plastic hinges shall be used in conjunction with the capacity design strategy. While the flexural plastic hinges shall be formed at selected locations, members where no plastic hinges are intended to be formed and which resist shear forces shall be protected against all brittle modes of failure using the "capacity design effects".
- The detailing plays an extremely crucial role in the ductility seismic design. The intended plastic hinges shall be provided with adequate curvature/rotation ductility to ensure the required overall ductility of the structure. As a consequence, the concrete compression zone shall be properly confined, and all main longitudinal bars shall be restrained against outward buckling in the potential plastic hinge regions.

1.3 Aims and Scope of this Guide

1.3.1 Aims

With the SDMHR 2013 taking effect, the seismic design of bridges in Hong Kong has been elevated to the international level as exemplified by the structural Eurocodes. The increase from the previous nominal ground acceleration of $0.05g$ to reference peak ground acceleration of $0.12g$ together with possible amplification due to subsoil conditions implies that the current design seismic action can be many times that of the previous value. Even though the scenarios with design seismic actions may or may not be those controlling the design of an individual structure, the aforementioned enhancement of the design code will call for a complete revamp of design strategies instead of just allowing for a higher design seismic action purely based on strength consideration as in the previous simplistic design approach. Proper seismic design should address not only strength and stiffness, but also ductility and deformability in order to achieve safety under extreme events. Examination of the possible failure modes under a severe earthquake is necessary and energy absorption by ductile structural behaviour in such an extreme event is often considered desirable.

To help ensure a smooth transition to the state-of-the-art seismic design methodology and to encourage a holistic approach even from the initial conceptual design stage for compliance with the current requirements, it is desirable to develop a comprehensive set of design guidelines taking into account local conditions and specific design requirements, established engineering practices including those available locally, in Mainland China and overseas, effective measures to ensure seismic resistance, buildability, increased use of prefabrication for better quality assurance and economy, possible standardization of detailing, as well as practicality of long-term asset management and maintenance. This design guide aims to achieve the following objectives:

- To provide a systematic approach for earthquake-resistant design of bridges in compliance with the current requirements;
- To devise design guidelines for optimization of new bridges under local conditions, which assist in decision making in the light of performance in various aspects;

- To develop a framework for assessment of existing bridges under upgraded seismicity; and
- To present a state-of-the-art review of latest trends of bridge engineering for local adoption in the future.

1.3.2 Scope

This design guide will focus primarily on short- to medium-span concrete girder bridges that are most commonly built in Hong Kong, although the design guide may also be applicable to other types of bridges. The topics covered in each chapter are as follows:

Chapter 2 presents a general introduction to the seismic design of bridges based on a comprehensive review of several major seismic bridge design codes from other countries and/or regions, including their development history, design philosophy and major compliance approaches, methods of seismic demand analysis, and representation of seismic action.

Chapter 3 summarizes the seismic bridge design specifications for Hong Kong, including those for highway and railway bridges, in a sequential order of seismic design procedures followed by a flowchart.

Chapter 4 provides guidance on the seismic design of new bridges at the conceptual design stage in respect of the selection of earthquake-resisting systems, articulations, configurations and intended seismic behaviour, i.e. elastic, limited-ductile or ductile behaviour, by examining the relation between the choices and the design and response of structures under seismic actions.

Chapter 5 describes the preliminary compliance assessment of typical existing highway bridges (previously designed to older codes) with reference to the current requirements of SDMHR 2013 in regard to the design seismic actions and structural details. It then presents the methodologies for performance-based seismic structural assessment of bridges and the development of fragility curves of some typical classes of bridges in Hong Kong for the purpose of rating of existing bridges.

Chapter 6 presents some of the latest trends of bridge engineering in respect of material, construction method and structural system, with special emphasis on the implications of these developments on seismic bridge design and retrofitting.

It should be noted that the results presented in this design guide are primarily obtained based on the assumption of short- to medium-span concrete girder bridges with fixed-base foundations. This is also stressed in the text where appropriate.

CHAPTER 2 INTRODUCTION TO SEISMIC DESIGN OF BRIDGES

2.1 Lessons Taught by Past Earthquakes

Earthquakes are one of the most disastrous natural hazards in the world and have caused many of the most terrible catastrophes in human history. Seismic damage to bridges can have severe consequences. The worst scenario is the collapse of a bridge, which places the people on or below the bridge at risk, and the bridge must be replaced after the earthquake unless alternative routes of sufficient capacity are available. The less dramatic damage may still require temporary closure of the bridge. Even a temporary bridge closure can have tremendous consequences. In the aftermath of an earthquake, the closure of a bridge can also impair emergency response operations. The economic impact of a bridge closure increases with the period over which the bridge is closed, the importance of the bridge as a link in the transportation network and the repair cost of the bridge.

It is possible to gain insight into the structural behaviour and to identify potential weakness in the existing and new bridges by examining the typical vulnerabilities that bridges have experienced in past earthquakes. The bridge damage observed from past earthquakes has usually been the impetus for many improvements in seismic engineering codes and practice. For example, California adopted new seismic design criteria including a detailed requirement for site-specific ground motions after the damaging earthquake in 1971, i.e. the M6.6 San Fernando Earthquake in USA. In Japan, bridges had traditionally been designed only for a large offshore earthquake. After the 1995 M6.9 Kobe Earthquake in Japan highlighting the risk of near-fault ground motions, Japanese bridge engineers started to design for two types of earthquakes: a subduction earthquake and a large crustal earthquake (Yashinsky *et al.*, 2014). The seismic design practice has also improved significantly in Mainland China as a result of experience gained from the Wenchuan earthquake. In 2008, the M7.9 Wenchuan Earthquake in Sichuan, China highlighted the vulnerability of roads and bridges to strong shaking in a mountain setting. Landslides caused quake lakes that washed away bridges, isolating the people in the mountains. Just after the 2008 Wenchuan Earthquake, the Ministry of Communications of the People's Republic of China issued a new edition of Guidelines for Seismic Design of Highway Bridges (the MCPRC Guidelines) (MCPRC, 2008). Compared to the previous edition (MCPRC, 1989), the selection criteria for bridge site were greatly refined in the light of seismic safety.

In addition to the necessary attention to design philosophy and design criteria, it is of particular importance to examine miscellaneous structural details. Lots of damage to bridges that earthquakes commonly induce can be attributed to inadequate detailing. For example, the biggest lesson taught by the 1971 San Fernando Earthquake was the problem of poor development of longitudinal reinforcement. During that particular earthquake, some concrete columns were pulled out of pile caps and pile shafts as the bridge moved back and forth, thereby contributing to bridge collapse. As a result, the California Department of Transportation (Caltrans) required all major column reinforcement in new bridges to be fully developed through the foundation and cap beam thereafter (Yashinsky *et al.*, 2014). Other significant damage contributing to poorly behaved columns, such as buckling of longitudinal rebars, fracture of transverse reinforcement, and shear failure as featured by steeply inclined diagonal cracks, have been associated with insufficient confinement to core concrete and longitudinal reinforcement. Concern about this kind of columns prompted changes to the design practice that all new bridges were required to have columns with larger diameter confining reinforcement at a closer spacing after the San Fernando Earthquake (Yashinsky *et al.*, 2014). Moreover, Caltrans initiated its first seismic retrofitting programme following the San

Fernando Earthquake which included installation of cable restrainers at the expansion joints to prevent dropping span in the light of a considerable number of bridge collapses caused by unseating of girders (Yashinsky *et al.*, 2014).

It is true that each subsequent earthquake will cause additional bridge damage and provide additional lessons on bridge seismic behaviour. For instance, the cable restrainers incorporated in highway bridges in California after the 1971 San Fernando Earthquake extended through end diaphragms that had not been designed originally for the forces associated with restraint. Some punching shear damage to the end diaphragms retrofitted with cable restrainers was observed following the 1989 Lorna Prieta Earthquake in USA (Moehle and Eberhard, 2000). Nevertheless, those bridges that were designed after the San Fernando Earthquake generally performed well, which was very encouraging (Yashinsky *et al.*, 2014). Taking advantage of knowledge gained over the years through a substantial amount of laboratory investigations and field data collected from past earthquakes, modern seismic engineering codes and practice have had significant improvements.

2.2 Seismic Performance Requirements for Bridges

In view of the catastrophic consequences of bridge collapses, it is the minimum requirement to design bridges in seismic regions for the performance level of "no collapse". In accordance with BS EN 1998-2 (BSI, 2005b), bridges shall be designed to withstand the design seismic events without local or global collapse, thus retaining the structural integrity and residual load-bearing capacity after the seismic events. Implied in such a statement is that damage is explicitly permitted and expected under the design seismic action. Such responses are unlike that expected for most other load combinations, because seismic actions are so large that elastic design would be prohibitively expensive. Therefore, a bridge may suffer significant damage under the design seismic event, but no collapse is allowed. Use by emergency vehicles should be available after structural inspections and clearance of debris in the aftermath of the earthquake, although partial or even complete replacement of the bridge may be required later.

The design seismic event represents the largest ground motion that can be expected with a reasonable likelihood during the life of a bridge. This likelihood is usually expressed as a return period in years, but it may also be described by an annual probability of occurrence, or a probability of exceedance. When setting the seismic hazard level for design event, it is fair to select a return period with some level of conservatism but not be overly conservative for ordinary bridges; otherwise it would be too costly. The 475-year return period, which corresponds to 10% probability of exceedance in 50 years, is recommended by BS EN 1998-1 (BSI, 2004a) and is also adopted in Structures Design Manual for Highways and Railways 2013 (SDMHR 2013) (Highways Department, 2013). For bridges whose failure is associated with a large number of probable fatalities and/or would possibly create a major economic impact, these bridges should be considered as critical for a local emergency plan and a longer return period can be economically justified for the design seismic event. SDMHR 2013 has specified the 1000-year and 2500-year return periods for important and critically important bridges, respectively, which correspond to 5% and 2% probabilities of exceedance in 50 years.

As discussed above, the choice of design ground motion level cannot be considered separately from decisions regarding the risk versus benefit. The set of design seismic hazard levels as adopted in SDMHR 2013 for the design of various important classes of bridges for the "no collapse" performance objective are consistent with those currently adopted in other regions and countries such as Mainland China (MCPRC, 2008), Australia (Standards Australia, 2017) and New Zealand (NZTA, 2016), as listed in Table 2.1. In the USA, a raise of the hazard level from 500 years to 1000 years for the design of ordinary bridges was approved by the Association of State Highway and Transportation Officials (AASHTO) in 2007 and then subsequently adopted for the revised AASHTO Guide Specifications for LRFD Seismic Bridge Design (referred to as AASHTO Guide Specifications) (AASHTO, 2011). However, owners may choose to deviate from the ground motion criteria if the situation warrants it.

Table 2.1 Performance requirements and corresponding design motion levels for seismic bridge design adopted in different codes

Region/Country	**Hong Kong**[1]	**New Zealand**[2]	**Australia**[3]	**USA**[4]	**Mainland China**[5]	
Design Approach	Single-Level Design				**Dual-Level Design**	
Requirement	No-Collapse				No-Collapse	Minimal Damage
Bridge Class	**Return periods (year) of design motions**					
Critical	2500	2500	2000	—*	2000	475
Important	1000	1000	1000	1000	1000	75 – 100
Minor	475	500	500	1000	475	50

[1] SDMHR 2013 (Highways Department, 2013)
[2] Bridge Manual (NZTA, 2016)
[3] Bridge Design – Part 2: Design loads (Standards Australia, 2017)
[4] AASHTO Guide Specifications for LRFD Seismic Bridge Design (AASHTO, 2011)
[5] JTG/T B02-01-2008: Guidelines for Seismic Design of Highway Bridges (MCPRC, 2008)
* Critical and/or essential bridges are not included in the scope of AASHTO Guide Specifications.

Aside from the "no collapse" requirement, there is usually another performance requirement such as the "minimisation of damage" requirement as in BS EN 1998-2. Bridges shall be designed to withstand earthquake ground motions having a higher probability of occurrence than the design earthquake, without the occurrence of significant damage and the associated limitations of use. In other words, a seismic action with a higher probability of occurrence than the design level may cause only minor damage to secondary components and to those parts of the bridge intended as energy dissipators so that the bridge can remain fully functional once inspected after the event.

Overall, the expected seismic performance of bridges is that they should withstand smaller earthquakes without significant damage and should withstand larger earthquakes without collapse and without posing any threat to life. In practice, however, bridges have usually been explicitly designed for ground motion for a single-level earthquake, i.e. only the upper-level earthquake will be checked during design based on the expectation that bridges so designed would be able to resist smaller earthquakes without significant damage by default. This single-level design approach is employed partially to simplify the seismic design effort and so far it has been widely adopted in the seismic bridge design codes worldwide.

However, the assumption made in the single-level design that satisfactory performance under the upper-level earthquake also implies satisfactory performance at lower-level ground motions may not invariably be true, unless elastic performance is targeted at the upper-level ground motion. For certain important bridges, the owners may require that the bridges will remain fully functional following more frequently occurring earthquakes than those that occur every one or two thousand years. In like manner, the decision on the seismic hazard level for checking functionality can vary from project to project, involving balancing the risk and cost of service disruption versus the cost associated with additional design and construction measures. In the USA, where owners have chosen to check functionality, the return periods for functionality-checking earthquakes have varied from 72 years for toll road projects in Orange County, CA to 500 years for major water-crossing bridges in New York City (FHWA, 2014). Unlike BS EN 1998-1, the MCPRC Guidelines (MCPRC, 2008) adopt a dual-level seismic bridge design approach and requires explicit performance checking under different levels of seismic hazard.

The MCPRC Guidelines use notations “E1” and “E2” to describe the dual-level ground motions. The E1 level motions have a return period of 50 – 475 years for checking functionality and the E2 level motions have a return period of 475 – 2000 years for checking the requirement of no-collapse, depending on the importance of bridge as presented in Table 2.1.

2.3 Seismic Design Strategies

There is an essential requirement of “no collapse” for seismic bridge design, which has been adopted by many regions and countries. This design philosophy implies that a bridge will likely be damaged and behave inelastically during the design earthquake. Technically speaking, it is possible to design a bridge to remain undamaged during a moderate-to-strong earthquake. This bridge must respond elastically to avoid damage and hence substantial forces will be developed in the columns and foundations. However, in all but low seismic regions, these forces will be too large to be resisted economically. Furthermore, there is a small but tangible possibility that an earthquake larger than the design event can occur in the lifetime of the bridge and produce even larger forces. Elastic design is usually unable to manage excess forces. Instead, it is decided to design bridges for ductile behaviour under large seismic actions, which generally involves yielding of various structural members and the corresponding plastic deformation in these members, i.e. formation of plastic hinges. Once yielding occurs, the forces in the bridge cannot exceed those that yield the members, even during very large earthquakes. This is a useful concept since it places a cap on the forces that have to be considered in the design. The use of ductility design has both an economic basis and a technical basis (FHWA, 2014). Economically, it avoids spending undue resources on responding elastically to an extreme event that has a low likelihood of occurrence by permitting some damage. Technically, it limits internal forces, making the structure less vulnerable to earthquake events that are larger than the design event.

Ensuring the ductile response of bridges under seismic action requires design principles different from those for most other load types. Although it is essential that the demand on any component in the structure should be less than the capacity of the component to resist that demand, simply providing more strength or capacity in an element without regard to its impact on the other parts of the structural system is inappropriate for seismic design. Normally, a bridge is composed of both key elements that yielding will be permitted and the other elements that should not be damaged before the yielding elements reach their capacity. The key elements will be determined first and designed to have capacity in excess of demand induced by the action of design ground motion. The maximum feasible strength at designated plastic hinge locations can be obtained at the end of this step taking into account potential overstrength. The remaining elements shall then be designed to provide a load path to accommodate the forces and deformations imposed on the selected key elements without the remaining elements losing their strength, i.e. to capacity-protect the remaining elements in the load path. This is achieved by ensuring that the dependable strength of the capacity-protected elements exceeds the maximum feasible strength of the key elements in the load path. The purpose of the capacity design strategy is to ensure that undesirable modes of inelastic deformation, such as plastic hinging at unintended locations or brittle shear failure, cannot occur. The capacity design is another equally important concept for the seismic bridge design.

Moreover, ductility design must be combined with detailing rules to ensure stable ductile response, i.e. the key components should be detailed so that they will continue to resist the applied loading with little or no degradation under reversal of both loading and deformation. The transverse reinforcement, which may be in the form of either spirals or hoops in circular sections, and links and ties in rectangular sections, is required to provide sufficient confinement when yielding occurs. The confining steel shall be able to help resist the high compressive inelastic strains imposed on the concrete core, as well as to restrain the longitudinal rebars against buckling. The effectiveness of confinement depends heavily on the size and spacing of well anchored transverse steel. The introduction of ductility-based detail design is believed also to ensure minimum levels of inelastic deformation capacity at the plastic hinges by enabling high compression strains to develop within the concrete core even after spalling of the concrete cover.

Therefore, ductility design, capacity design and ductility-based detail design are essentially three indispensable components of modern seismic bridge design. Damage is explicitly permitted for the bridge under design ground motions, provided it is ductile in nature and then only in members specifically designed and detailed for such behaviour. Where possible, the preferable ductile members should be limited to locations within the bridge that can be easily inspected and repaired following an earthquake, and these may include columns, piers, seismic isolation and damping devices, bearings, shear keys, and steel end diaphragms.

There are basically three types of global strategy, depending on the ductile members selected. Typically, the columns and piers are the major components designed for potential plastic hinging to occur, while the bridge decks are not normally expected to behave in an inelastic manner during an earthquake. The provisions in BS EN 1998-2 (BSI, 2005b) also seek to ensure the ductile behaviour of bridges by proper formation of flexural plastic hinges in columns and piers. In fact, the design of a ductile substructure with an essentially elastic superstructure is the most recommended seismic bridge design strategy that is widely adopted in seismic engineering codes. The approach of placing seismic isolation bearings between the superstructure and substructure that increase the lateral flexibility of bridge and provide additional damping is another strategy included in BS EN 1998-2. Isolation physically uncouples a bridge superstructure from the horizontal components of earthquake ground motion, leading to substantially reduced forces during an earthquake. Furthermore, when an isolated bridge is subjected to an earthquake, the deformation occurs largely at the isolation bearings rather than the substructure components. These greatly reduce the seismic forces and displacements transmitted from the superstructures to the substructures, which can achieve the condition of both elastic superstructures and elastic substructures at little cost (AASHTO, 2011). Other devices in either passive or active form have also been proposed (FHWA, 2006) to provide additional damping or energy dissipation. The methods for design of such elements are beyond the scope of this monograph. The AASHTO Guide Specifications (AASHTO, 2011) include another strategy for the design of an essentially elastic substructure with a ductile superstructure, based on observations in some past earthquakes that plastic behaviour has occurred in cross bracing of steel superstructure and some more subsequent research. This type of global strategy is essentially an emerging technology and the design requirements for such systems are not yet fully developed. There is currently no mention of ductile superstructures in BS EN 1998-2.

2.4 Seismic Design Methodologies

Two fundamental methodologies have been developed to conduct seismic design. These are the Force-Based Method and Displacement-Based Method. Fundamentally, both methods seek to ensure that a structure has certain elements designed to yield in a ductile manner and all the rest designed to remain essentially elastic during the design earthquake. It is only the method of achieving this result that differs.

2.4.1 Force-based method

The Force-Based Method is the more common method that is used by most of the seismic codes including those mentioned in Section 2.2. The method develops the design seismic forces for yielding elements F_{yeild} by dividing the elastic forces F_{elastic} obtained from the demand analysis assuming no yielding occurs by an appropriate "Force Reduction Factor" as illustrated in Figure 2.1. Inherent in this method is the expectation that the intended ductile members will yield at a force level equal to the reduced elastic demand when subjected to the design ground motions. The shortfall in strength is to be compensated by deformation ductility capacity.

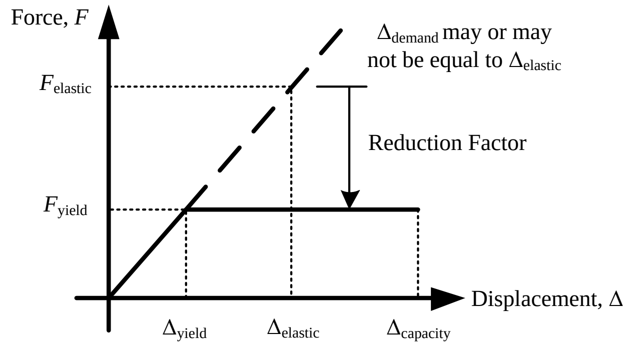


Figure 2.1 Determination of design seismic force in force-based method

The general workflow for the Force-Based Method is summarized as follows:

(a) Calculate the earthquake forces and their distribution based on elastic analysis.

(b) Reduce the seismic elastic forces of intended ductile members by an appropriate reduction factor and combine with other concurrent loads.

(c) Determine the required design strength of ductile members and design the members to meet the strength demands; and provide structural detailing to ensure deformation capacity.

(d) Provide capacity protection for the members that are designed to remain elastic and shear protection for ductile members.

The force-reduction factor may be called "Behaviour Factor (q)" as in BS EN 1998-2, or "Ductility Modification Factor" as used by some other codes. The force reduction factor is often thought of as equivalent to the ductility factor which is defined as the ratio of the displacement demand Δ_{demand} to the yield displacement Δ_{yield}. However, this is valid only when the bridge forms its plastic hinges instantly rather than one at a time and the period of vibration is such that Δ_{demand} is equal to $\Delta_{elastic}$, i.e. the equal displacement approximation should be effective (Priestley *et al.*, 2014). It is well established that the equal displacement approximation is inappropriate for structures of very short or very long periods as manifested in some design codes in the form of "Displacement Modification Factor" where the elastic displacement of a structure under the action of reduced seismic forces Δ_{yield} is multiplied by a factor to provide an estimate of the nonlinear response displacement Δ_{demand}. The displacement modification factor may be equal to the force reduction factor in some cases, which implies the equal displacement approximation. It may be similar to the force reduction factor in other cases, implying subtle variations from the equal displacement approximation. BS EN 1998-2 also specifies a factor called "Displacement Ductility Factor (μ_d)" that considers the short- and long-period modifications when determining the displacement demand.

Nevertheless, the force reduction factor roughly implies the expected ductility demand imposed on inelastic members. An appropriate force reduction factor reduces the elastic response force by a level that is consistent with the implied ductility capacity of the structure under consideration, while the ductility capacity is ensured by prescribed confinement details. The maximum values of the force reduction factors associated with common seismic resisting systems as specified in different regional codes are shown in Table 2.2. It is found that BS EN 1998-2 is relatively conservative in the allowable maximum ductility demands as compared to other codes, especially the New Zealand code.

2.4.2 Displacement-based method

The Displacement-Based Method is currently included in the MCPRC Guidelines (MCPRC, 2008) and AASHTO Guide Specifications (AASHTO, 2011). Unlike the Force-Based Method, the Displacement-Based Method does not require the calculation of specific design forces for the yielding elements. Instead, the Displacement-Based Method focuses on providing the structure with sufficient displacement capacity to meet the displacement demand, i.e. to ensure $\Delta_{capacity} > \Delta_{demand}$. The designer is free to proportion the yielding system as necessary to ensure that the displacement demand is less than the displacement capacity at each pier, provided that a minimum lateral strength threshold is provided for each pier and that all of the non-seismic load cases are also satisfied (FHWA, 2014). The displacement capacity is based on the ductility of yielding columns and may be controlled by both the longitudinal and transverse reinforcement in the columns as well as the configuration of substructure such as shear spans of columns. The displacement capacity of the columns is determined explicitly by conducting section analyses. A direct check of the displacement capacity with respect to the demand is made in the end.

The general workflow for the Displacement-Based Method is summarized as follows:

(a) Propose a lateral load resisting system and the corresponding displacement capacities.

(b) Perform a seismic response analysis to determine the displacement and force demands.

(c) Ensure the proposed displacement capacities are adequate to meet the imposed displacement demands.

(d) Check the load path and apply capacity protection principles to the elements that are designed to remain elastic or essentially elastic.

It is worth mentioning that the maximum ductility demands imposed on the yielding elements are normally restricted, although theoretically any displacement demand can be satisfied by design. However, this explicit control of ductility demand in the Displacement-Based Method is essentially different from that in the Force-Based Method where implicit control is achieved by setting maximum values for the force reduction factors. The allowable maximum member ductility demands for different seismic resisting systems taken from the AASHTO Guide Specifications (AASHTO, 2011) are also listed in Table 2.2, together with the maximum values of force reduction factors from other codes adopting the Force-Based Method.

Table 2.2 Recommended allowable maximum member ductility for ductile reinforced concrete members as specified by different codes

Code and Factor	Configuration detail		Maximum allowable value*
BS EN 1998-2: Behaviour Factor (q) (BSI, 2005b)	Piers with plastic hinges accessible or reasonably accessible for inspection and repair	Vertical	3.5
		Inclined	2.1
	Piles with plastic hinges inaccessible for inspection and repair, e.g. formed at the foot of a pier shaft immersed in deep water, or the heads of piles beneath a large pile cap	Vertical	2.1
		Inclined	1.5
	Prestressed or post-tensioned members		1.0
New Zealand Bridge Manual: Design Displacement Ductility Factor (μ) (NZTA, 2016)	Members with plastic hinges formed in accessible positions, e.g. above ground or normal (or mean tide) water level		6.0
	Members with plastic hinges formed in reasonably accessible positions, e.g. less than 2 m below ground but not below normal (or mean tide) water level		4.0
	Members with inaccessible plastic hinges, e.g. formed more than 2 m below ground or below normal (or mean tide) water level		3.0
	Raked piles in which earthquake load induces large axial force		2.0
Australian Bridge Design: Design Ductility Factor (μ) (Standards Australia, 2017)	Superstructure on fixed pot or spherical bearings or elastomeric bearings with translational movement restraint in the direction considered at piers on:	- stiff foundations (i.e. with negligible translational and rotational movements)	4.0
		- flexible foundations with significant contribution to the displacement at pier top (i.e. piles in 10 m or more of soft soil)	3.0
	Piers integral with superstructure, and superstructure on bearings at abutments		4.0
	Hollow reinforced concrete piers		2.0
	Abutments integral with superstructure		2.0
	Wall-type piers		2.0
	Prestressed concrete piers with:	- bonded strands	2.0
		- external or unbonded strands	1.0
AASHTO Guide Specifications: Ductility Demand (μ_D) (AASHTO, 2011)	Single-column bent		5.0
	Multiple-column bent		6.0
	Pier wall in the weak direction		5.0
	Pier wall in the strong direction		1.0
	Drilled shafts, cast-in-place piles, and prestressed piles subjected to in-ground hinging		4.0

*A value of 1.0 for the allowable maximum member ductility means the members are not intended as ductile elements.

2.5 Methods of Demand Analysis

Despite different end results, both the Force-Based Method and Displacement-Based Method require a seismic demand analysis to be conducted. This process involves building a global model, conducting a response analysis, and determining the relevant forces and displacements from the analytical results for use in the design of individual components. The analysis for dynamic loads in an earthquake is generally not as straightforward as that for static loads, particularly if the bridge response exceeds its elastic limit and becomes nonlinear.

Various analytical methods have been developed for the purpose of estimating the force and displacement demands on a bridge during an earthquake. The most common methods are listed in Table 2.3, ranging from simple approximate methods to complex rigorous methods. For many years, the common method to analyse a bridge with nonlinear behaviour has been to solve an equivalent linear problem using the equivalent linearized properties such that the elastic methods of analysis can still be used. However, the degree of conservatism in the results is hardly known. More rigorous methods are now available with the development of explicit nonlinear static and dynamic methods, i.e. pushover and time-history methods respectively.

The choice of an analytical method is based on many factors including the seismic hazard at the bridge site, and complexity of the bridge and its foundation. Generally speaking, increasing seismic hazard and irregularity of the bridge require increasingly rigorous analysis. Often an equivalent static analysis may suffice for a preliminary design followed by a more detailed elastic dynamic analysis, i.e. response spectrum analysis, or even in special cases nonlinear analysis such as static pushover analysis or dynamic time-history analysis in the final stages of design.

All but the simplest of the methods presented in Table 2.3 require the use of a computer programme for their implementation, and computer software is readily available nowadays. Even though a designer in practice may not be familiar with the details, the designer should still be mindful of the limitations of each method. For this reason, this section will focus on the principles of these methods and their applications.

Table 2.3 Common methods for seismic response analysis

	Static	**Dynamic**
Elastic and linear	Single-mode response spectrum analysis	Multi-mode response spectrum analysis
Inelastic and nonlinear	Pushover analysis	Nonlinear time-history analysis

2.5.1 Single-mode response spectrum analysis

In the single-mode response spectrum analysis, the seismic forces are derived from the inertial forces using the response spectral coefficient corresponding to the fundamental period of the structure independently in both the longitudinal and transverse directions. The single-mode response spectrum analysis is one of the equivalent static methods, which assumes that the dynamic response of the bridge can be sufficiently approximated by its fundamental mode. This assumption holds for conventional bridges that are simple and regular with uniform span lengths and column heights. This method is rarely used in detailed design but is helpful for preliminary design and checking.

2.5.2 Response spectrum analysis

The multi-mode response spectrum analysis, or simply response spectrum analysis, is based on the superposition of modal response of all significant modes of the structure calculated from the response spectrum coefficients. The method of the Complete Quadratic Combination (CQC) is recommended for combining the modal responses. Alternatively, the Square-Root-of-the-Sum-of-the-Squares (SRSS) is used when the modal periods are well separated. Measures are introduced to determine the number of significant modes. The use of “effective modal mass” is deemed to be efficient where it is possible to base the number of modes to be used in an analysis on the participation of a minimum percentage of the total mass of the bridge, e.g. the sum of the effective modal masses for the modes considered amounts to at least 90% of the total mass of the bridge as used in BS EN 1998-2. This percentage is based on past experience with the analysis of conventional bridges. It may be necessary to raise this figure to 95% for non-conventional bridges or large multi-segmental bridges which tend to be governed by the higher mode effects (FHWA, 2014). A detailed description of modal analysis and response spectrum analysis is given by Chopra (2016) and Priestley *et al.* (1996). This method in general does not have constraints other than being restricted to linear elastic behaviour.

2.5.3 Pushover analysis

Pushover analysis is a nonlinear static analysis of the structure subjected to constant gravity loads and monotonically increasing lateral loads representing the action of a horizontal seismic ground motion. Second-order effect is also accounted for. As output, the force may be plotted against the displacement to illustrate the progression of inelastic deformation of bridge until collapse. The method is usually used to determine the displacement capacity of individual bents, frames or the whole bridge.

The evaluation of bridge displacement capacity is typically based on expected capacities of inelastic components (usually piers with formation of plastic hinges), starting from ultimate strain capacity of the material, further proceeding to ultimate curvature capacity of the section, ultimate rotation capacity of the plastic hinge, and ultimate deformation capacity of the member, and finally ending up with global ultimate displacement capacity of the bridge. Apart from the ultimate limit state, intermediate damage limit states may also be defined for those inelastic components based on consideration of the serviceability and repairability of the structure (Kowalsky, 2000; Priestley *et al.*, 2007). Although the analysis is based on monotonically increasing load, the effects of cyclic loading is taken into account by selecting appropriate material models considering strength and stiffness degradation.

This method of analysis provides additional information on the sequence and final pattern of plastic hinge formation, redistribution of forces following the formation of plastic hinges and expected deformation demands of columns and foundations, which provides the designer with a greater understanding of the expected performance of the bridge than that obtained from elastic analysis procedures.

2.5.4 Nonlinear time-history analysis

Nonlinear time-history methods are dynamic methods that calculate the demand on a bridge while explicitly including the nonlinear properties of the members based on step-by-step integration of the equations of motion where the structural stiffness matrix is updated as the inelastic response develops. The seismic input is ground motion accelerogram. Such methods can be very time consuming and computationally demanding for a typical bridge. However, a nonlinear dynamic time-history analysis may be necessary for some important and complex bridges.

2.6 Representation of Seismic Action

This section describes the process for determining the seismic hazard considered in seismic design and basic types of representation of seismic action for use in design practice. In addition, the derivation of the seismic action for Hong Kong prescribed in SDMHR 2013 (Highways Department, 2013) is reviewed as an illustration.

2.6.1 Seismic hazard analysis

The quantitative estimation of ground shaking hazard at a particular area involves seismic hazard analysis. There are two basic philosophies for the seismic hazard analysis, i.e. deterministic and probabilistic seismic hazard assessment. Deterministic seismic hazard assessment is to identify the maximum credible earthquake (MCE) that will affect a site. A deterministic seismic analysis uses geology and seismic history to identify earthquake sources and to interpret the strongest earthquake each source is capable of producing regardless of time.

These are the MCEs. The underlying philosophy is that, as one cannot safely predict when an earthquake will happen, the MCEs are what a critical structure should be designed for if the structure is to avoid surprises. Probabilistic seismic hazard analysis (PSHA), on the other hand, advocates that the likelihood of occurrence should be considered in view of the fact that the life of a structure is very short compared to the recurrence intervals of large events. The PSHA involves integrating the probabilities of experiencing a particular level of ground motion during a specified life period due to the total seismicity expected to occur in the area (about 300 km radius) of a site of interest (Anderson and Trifunac, 1978; Cornell, 1968). It provides the estimate of ground motion with a specified confidence level of the probability of not being exceeded. The PSHA generally follows the steps below:

(a) Identify seismically active regions, structures and faults using earthquake catalogues, geologic evidence and geodetic strains, etc.; the ordered pair (i, j) indicates an earthquake of magnitude M_j occurring at distance R_i from the site of interest.

(b) For each fault or region, estimate the average event rate v_{ij}, using both instrumental and historical seismicity data, and geodetic strains; then the expected number of earthquakes with magnitude M_j occurring at distance R_i from the site during a time interval of Y years is given by $n_{ij}(Y) = v_{ij}Y$.

(c) Attenuate the ground motion from each source to the given site through an empirical ground-motion prediction equation, which is a function of the distance, earthquake magnitude and the geological conditions. A large number of such relations have been developed over the years and a profound review of more than 450 ground-motion prediction equations developed during the period 1964 – 2010 can be found from Douglas (2011).

(d) Forecast the ground motion, parameterized as an engineering variable such as the peak ground acceleration (PGA) and to a much lesser extent the peak velocity and displacement, in terms of the probability of exceedance or the expected return period of a given level of shaking.

(e) Sum the exceedance probabilities of the different sources to account for the fact that a given site is potentially subject to shaking due to a variety of earthquakes sources.

(f) To summarize, letting X be the random ground motion variable and x be a possible level of X, and also letting $q(x|M_j,R_i)$ be the conditional probability that the value x will be exceeded due to an earthquake of type (i, j), the expected number of times for which $X > x$ occurs due to all possible earthquakes is given by $N(x) = \sum_{i=1}^{I} \sum_{j=1}^{J} q(x|M_j,R_i)\, n_{ij}(Y)$; assuming that the occurrence of the event $X > x$ from earthquakes of type (i, j) is a selective Poissonian process and that from all possible earthquakes is also Poissonian with $N(x)$, the probability of $X > x$ at a site may thus be obtained as $P|(X>x) = 1.0 - \exp[-N(x)]$.

Therefore, the PSHA approach has incorporated uncertainties with respect to earthquake location, magnitude and ground motion, producing a weighted average of all possibilities that is a best estimate of the risk associated with seismic activity. A decision still must be made about the appropriate risk level to use in design. The choice of the design ground motion level cannot be considered separately from the level of performance specified for the design event as discussed in Section 2.2. Common performance levels used in the design of transportation facilities include the protection of life safety and maintenance of function after the event. Keeping a bridge functional after the event is a more rigorous requirement than simply maintaining life safety. In general, the bridges shall be able to withstand the design earthquakes without collapse though they may suffer significant disruption in service and significant damage, while they are expected to be able to avoid the occurrence of damage and limitation of use after the more frequent seismic events. The determination of the design ground motion level may also need to take into account the importance level of the facility. The cost of designing facilities for an excessively long return period would generally be prohibitive, but a longer return period may be justified for critical bridges when, for instance, an extended duration in loss of operation of the structure would cause undue cost to the community or the structure is part of a lifeline route for emergency operations. The PSHA as an approach providing better defined risk levels in terms of return periods or the annual rate of exceedance is considered the most appropriate basis for making rational design decisions about risk versus benefit, and it has been widely adopted for use in establishing design ground motions. SDMHR 2013 (Highways Department, 2013) has employed a probabilistic approach and adopted a 475-year reference return period as the basis to establish design ground motions for design of ordinary bridges, which corresponds to 10% probability of exceedance in an exposure period of 50 years. SDMHR 2013 has also specified 1000-year and 2500-year return periods for design of important and critically important bridges, respectively. A different return period may also be used if sufficiently justified.

A probabilistic approach also has the merit of being able to distinguish relative contributions to risk from the more active versus the less active faults. As opposed to PSHA, which carries out integration over the total expected seismicity during a given exposure period to provide the estimate of a strong-motion parameter of interest with a specified confidence level, it has become common to display the relative contribution to the total hazard of each source in terms of magnitude and source-to-site distance. This is known as the de-aggregation of the PSHA. De-aggregation enables identification of controlling magnitude and distance combinations of the seismic hazard. This information is used, for example, when developing acceleration time-histories for structural and geotechnical time-history analyses.

Despite all these merits, it should be noted that a probabilistic hazard approach is much more complex than a deterministic approach and the results of probabilistic hazard solutions can sometimes be questionable, particularly at very long return periods. It is generally suggested to employ deterministic solutions as a sanity check on the results of probabilistic analyses for long return periods.

2.6.2 Site response

The results of a hazard analysis may need to be modified to account for local site effects due to the presence of soil overburden. At most sites, the rock will be covered by some thickness of soil that can markedly influence the nature of the motions transmitted to the structure as well as the loading on the foundation. Generally speaking, the soils act as filters to amplify the response at some frequencies and de-amplify it at others. The greatest degree of amplification occurs at frequencies corresponding to the characteristic site period.

The factors affecting the manner in which a site responds during an earthquake include the near-surface stiffness gradient, the surface topography that can reflect and refract the incoming waves, near-surface material boundaries, and deeper basin geometries. The interaction between seismic waves and near-surface materials can be complex, particularly when the surface topography and/or subsurface stratigraphy are complex. The quantification of site response has usually been accomplished by either empirical or analytical methods.

For ordinary bridges, or for projects in which detailed subsurface information is not available, the empirical approach is generally more prevalent. The empirical methods are built on the database of strong ground motion records developed over the years. Division of the records within this database according to the general site conditions has enabled the development of empirical correlations for different site conditions. Usually, the site effects are expressed in terms of the ratio of ground surface motion parameter to reference motion parameter for a given site condition, i.e. site factors. Reference motions are generally taken as recorded motions for a “rock” site. Classification of ground type is a complex matter with multiple criteria. The available empirical methods have described site conditions in terms of surficial geology, geotechnical classification and near-surface shear wave velocity (Scawthorn and Kramer, 2014). The latter approach, in which the site conditions are characterized by the average shear wave velocity of the upper 30 m of a profile ($v_{s,30}$) has become common. Bridge codes have historically used relatively simple site classification schemes. Most of the codes (e.g. British Standards Institution 2004, Standards New Zealand 2004, Standards Australia 2007, MCPRC 2008, AASHTO 2011) determine the site classes based on $v_{s,30}$ in combination with soil depth, but also allow average standard penetration test resistance (N_{SPT}) and undrained shear strength (c_u) to be used when the shear wave velocity is not available. However, while there is similarity in the criteria, the exact values for classification are quite different in these codes. To illustrate, it is assumed that the ground types specified in different codes by average shear wave velocity are broadly comparable as shown in Figure 2.2.

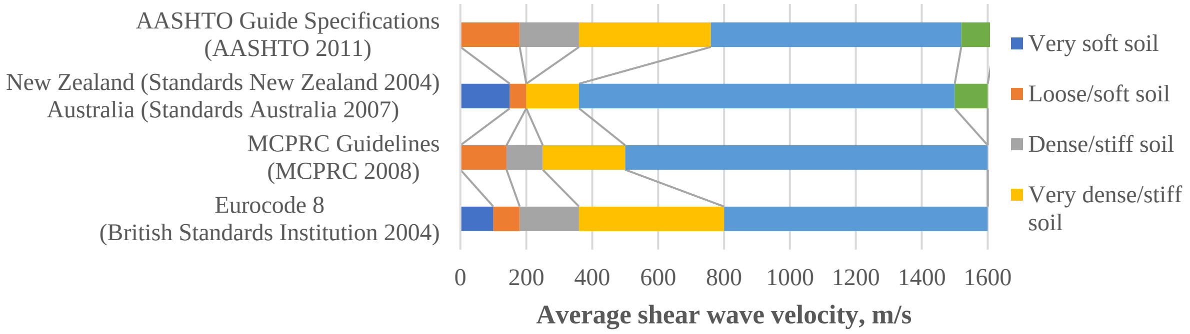


Figure 2.2 Ground type classification by average shear wave velocity

The reasonableness of empirically based methods for estimation of site response effects depends on the extent to which the particular site conditions match the site conditions in the databases from which the empirical relationships were derived. The empirical expressions of site effects are based on regression analyses, and therefore they correspond best to sites with characteristics, such as shear wave velocity profiles, that are similar to the average characteristics of the profiles in the databases upon which the expressions are based. It is important to be aware of the empirical nature of such methods.

For critical bridges and for sites with unusual characteristics, it may be necessary to perform site-specific ground motion response analyses. Site-specific ground motion response analyses involve: (a) characterizing a soil profile down to bedrock, (b) collecting and adjusting appropriate “rock” input earthquake acceleration time-histories (horizontal ground motions), and (c) modelling the propagation of the ground motions from bedrock up to the ground surface. The analyses may be conducted in one, two or three dimensions.

Soils are highly nonlinear, even at very low strain levels. To account for the effects of nonlinear soil behaviour, ground response analyses are generally performed using one of the two basic approaches: equivalent linear approach or nonlinear approach. In practice, owing to the difficulty of characterizing nonlinear constitutive model parameters, the use of equivalent linear analysis is preferred in which the effects of nonlinearity are approximated in a linear analysis with the use of strain-compatible soil properties. In the equivalent linear approach, a linear analysis is performed using shear moduli and damping ratios that are based on an initial estimate of the strain amplitude. The strain level computed using these properties is then compared with the estimated strain amplitude and the properties adjusted until the computed strain levels are very close to those corresponding to the soil properties.

The seismic hazard analysis may also need to be modified to account for near-fault effects. It has been found over the last three decades that when a bridge site is located sufficiently near a fault, and if the earthquake is a forward rupturing event (i.e. the fault ruptures toward the site), the long-period motions at the bridge site can be significantly enhanced with the occurrence of very large velocity pulse. It is necessary to take special precautions in these situations since such motions can be very damaging to certain classes of bridges. The need to consider near-fault effects, however, is generally limited to sites with well-defined shallow active faults, partially because methods used to quantify near-fault effects are still a subject of research. Furthermore, only bridge sites within about 16 km of a rupturing fault need to be considered for fault directivity effects. For these reasons, methods to account for near-fault effects are not elaborated here.

2.6.3 Representation of seismic action

Once the seismic hazard analysis has been conducted, the design ground motions for use in design corresponding to the return periods determined must be characterized, either by response spectra or time-histories.

(1) Response spectra

If a structure is subjected to a time-history of ground motion, the elastic structural response can be readily calculated as a function of time, generating a structural response time-history. However, it is often sufficient to know only the maximum amplitude of the response time-history for design purposes.

The response spectra are curves plotted between the maximum response amplitudes (displacement, velocity or acceleration) of single-degree-of-freedom (SDOF) systems subjected to specified earthquake ground motions and the natural periods. Usually the response of an SDOF system is determined by time domain or frequency domain analysis, and for a given time period of system, the maximum response is picked. This process is continued for all range of possible time periods of the SDOF system. Such plots pertain to specified damping ratio and input ground motion. The response spectra can be interpreted as the locus of maximum response amplitude of an SDOF system for a given damping ratio. The same process can be carried out with different damping ratios to obtain the overall response spectra. The plots can be used to determine the peak structural responses within the linear range, thus facilitating the earthquake-resistant design of structures. The response spectra form the basis of much of the structural analysis and design in modern earthquake engineering.

In general, the design ground motions are expressed in terms of acceleration response spectra, which can be employed by both the force-based method and displacement-based method to calculate the seismic demand. The acceleration response spectra for horizontal elastic earthquake response can generally be defined as a dimensionless spectral shape factor (or normalized horizontal elastic response spectrum) modified by a hazard factor (or reference peak ground acceleration in m/s^2) and a dimensionless return period factor accounting for deviation from the reference return period in a design scenario (also known as importance factor). The spectral shape factor is a piece-wise function of the time period, usually consisting of a linearly ascending branch (short-period zone), a horizontal branch (constant acceleration response-control zone), a descending branch inversely proportional to the time period (constant velocity response-control zone) and possibly another descending branch inversely proportional to the square of time period (constant displacement response-control zone) as shown in Figure 2.3, where the respective corner periods may be defined by T_B, T_C and T_D. Besides, the spectral shape factor is generally derived based on the "rock" site and a default damping ratio of 5%, and it should be adjusted by soil factors for the local site conditions and by damping correction factors for different damping ratios.

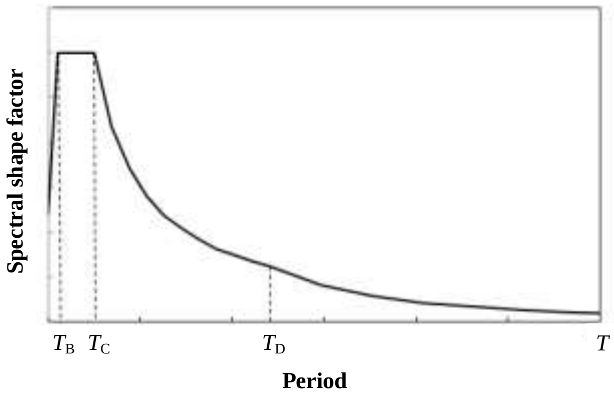


Figure 2.3 Shape of the elastic acceleration response spectrum

Figure 2.4 shows the plots of the spectral shape factors as defined in the bridge codes mentioned in Section 2.2. The Eurocode spectral shape factor shown is based on Type 2 response spectrum of BS EN 1998-1 (BSI, 2004c) since it is considered more relevant to the Hong Kong region in accordance with SDMHR 2013 (Highways Department, 2013) compared to Type 1 response spectrum. The AASHTO Guide Specifications (AASHTO, 2011) are not considered here for two reasons. First, the document adopts a 1000-year reference return period which is quite distinct from the 475- or 500-year return periods used by the other codes. Second, it adopts a unique approximation method to construct the acceleration response spectrum based on two more spectral accelerations in addition to the peak ground acceleration so that a spectral shape factor cannot be separated from the hazard factors. Type 2 response spectrum of BS EN 1998-1 generally gives a narrow plateau at the peak of the spectrum and, as a consequence, the seismic coefficient decays more rapidly with the increase of structural period, especially for loose/soft soil, as shown in Figure 2.4.

The consideration of vertical component of the ground motion is occasionally required in design apart from the horizontal components. When the vertical component of the seismic motion needs to be taken into account, the vertical response spectra may generally be developed by scaling the horizontal response spectra.

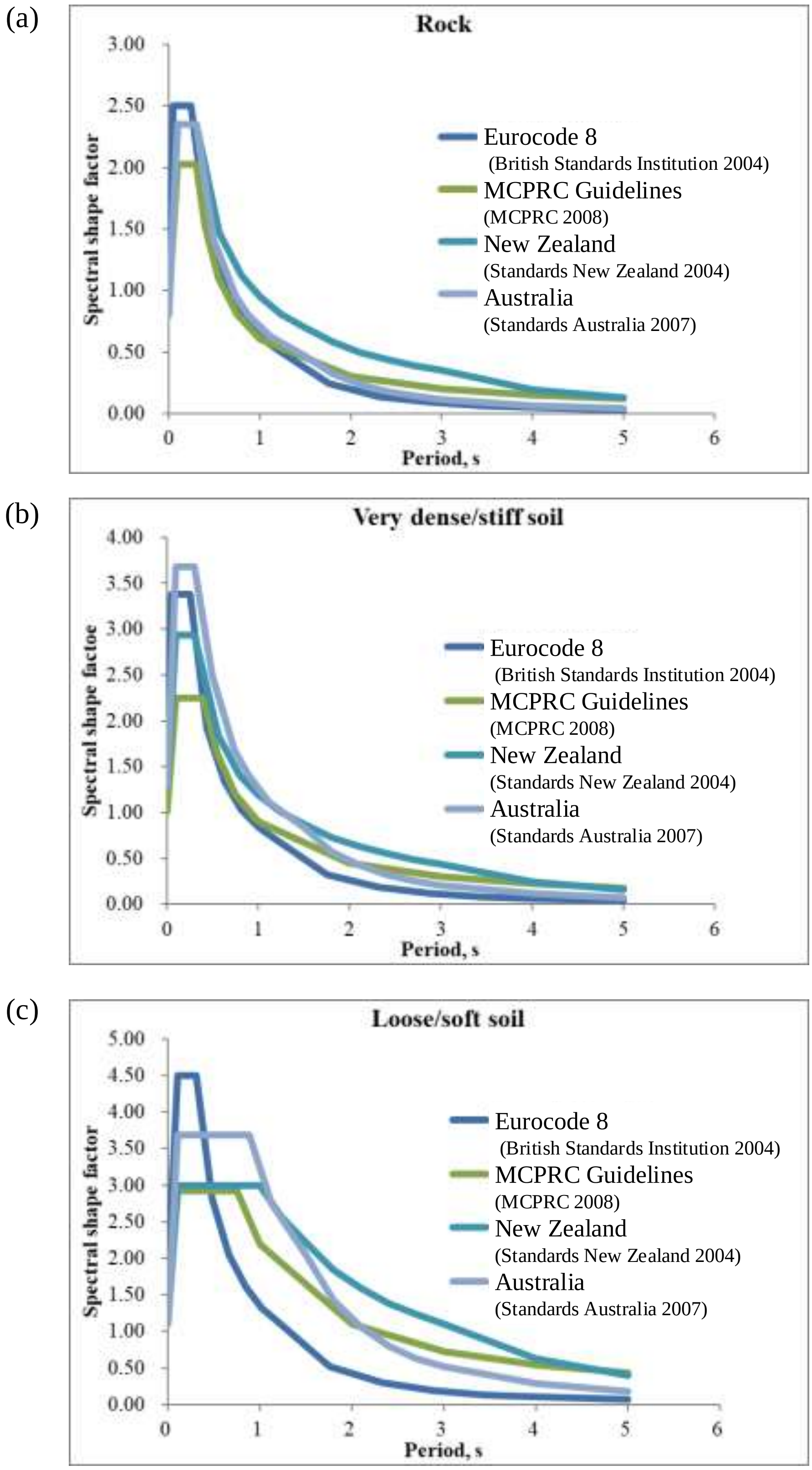


Figure 2.4 Normalized horizontal elastic response spectrum defined in different codes for (a) Rock site; (b) Very dense/stiff soil site; and (c) Loose/soft soil site

(2) Acceleration time-history

While the response spectrum method is the most common method of seismic analysis for conventional bridge design, many geotechnical analyses and some of the more complex structural analyses require the development of a set of acceleration time-histories to represent the design earthquake. The appropriate time-histories should be consistent with the target elastic response spectrum, and so the development of these time-histories generally starts with establishing the elastic response spectrum according to relevant return period and site conditions. The time-histories can be obtained from either recorded events or artificial accelerograms.

Time histories are developed to be consistent with the target spectrum by first selecting a candidate set of appropriate ground motion records with magnitudes, source distances and mechanisms, duration of motions and other relevant seismic source characteristics consistent with those defining the design seismic action. The candidate ground motion records are then scaled so that the resultant mean spectrum developed from the scaled response spectra closely matches the target response spectrum. The simplest form of time-history modification is amplitude scaling by a constant factor, and this is also preferred by practising engineers. The basic problem with such generated time-histories is that a single scaled time history will generally fit the target spectrum only over a very narrow range of period because the acceleration response spectrum generated from a naturally recorded time-history normally has many peaks and valleys. Therefore, a suite of scaled time-histories is required to encompass the entire target spectrum. According to BS EN 1998-2 (BSI, 2005b), a minimum of three scaled time-histories shall be employed if scaled natural records are to be used in an analysis. The candidate acceleration time-histories can be selected from one of the many available databases of recorded earthquake ground motions (e.g. the Pacific Earthquake Engineering Research Center - PEER database) with constraints on earthquake parameters such as earthquake magnitude, source distance and shear wave velocity of site soil. The Next Generation Attenuation (NGA) database developed by PEER (2018) including over 3000 strong ground motions records from shallow crustal earthquakes in active tectonic regimes throughout the world is widely employed. The PEER website also includes a tool for selecting and scaling time-histories to match a target spectrum.

When appropriate ground motion records are not available, appropriate artificial accelerograms may be used. A great number of computer programmes are available for generating spectrum-compatible artificial accelerograms, e.g. SIMQKE_GR (Gelfi, 2012). A spectrum-compatible time-history has been modified also in the time or frequency domain in addition to amplitude to achieve a match with the target spectrum. Even for an analysis using spectrum-compatible time-histories, it may be necessary to use three sets of such motions to achieve a statistically stable result.

2.6.4 Determination of seismic parameters for Hong Kong

SDMHR 2013 (Highways Department, 2013) refers to BS EN 1998-1 (BSI, 2004c) for the equations of spectral shape factor and ground type classification criteria for defining the response spectrum, but with supplementary specifications on the hazard factor and return period factor so as to suit specific conditions in Hong Kong. The determination of these seismic parameters is reviewed below.

The PGA as adopted in SDMHR 2013 is largely based on the seismic hazard analyses carried out by the Geotechnical Engineering Office (GEO) of Civil Engineering and Development Department, Hong Kong in 1995 with the University of Hong Kong. GEO Report No. 65 (Lee *et al.*, 1998) was then issued. The GEO assessment adopted the probabilistic approach by considering 16 closest potential source zones near the Hong Kong region and 119 recorded earthquakes in South China with appropriate modifications for the analysis of seismic hazard in the Hong Kong region. The bedrock PGA corresponding to a return period of 475 years was determined as about 0.08*g* in the northern part of Hong Kong and about 0.11*g* in the southern part based on the GEO assessment, where *g* is the acceleration due to gravity. On the other hand, Hong Kong has been included in the Seismic Ground Motion Parameters Zonation Map of China since the 2001 edition (CSP, 2001). In the latest seismic zonation map (CSP, 2015), the northern part of Hong Kong falls in the region of 0.10*g* and the southern part of Hong Kong falls in the region of 0.15*g*, both of which are considered as belonging to intensity VII. Although the seismic zonation map of China gives a higher value of 0.15*g* for the southern part of Hong Kong, it may be due to the differences in terms of data coverage and data modification in the GEO assessment. In the absence of other comprehensive assessment of seismicity in Hong Kong, the "Final Report on the Study for Seismic Actions" prepared by Atkins (2012) recommended to adopt a PGA value of 0.12*g* corresponding to a return period of 475 years as the reference peak ground acceleration for the design of ordinary highway and railway bridges in Hong Kong, which encompasses the recommended range of 0.077*g* to 0.117*g* across the Hong Kong region from the GEO assessment.

Additionally, based on the recommendation given in GEO Report No. 65, the ratio of the PGA value corresponding to a return period of 2500 years to that corresponding to a return period of 475 years ranges from 2.0 to 2.3. Therefore, on the basis of GEO Report No. 65, the "Final Report on the Study for Seismic Actions" (Atkins, 2012) recommended a multiplicative importance factor of 2.3 for seismic action with a return period of 2500 years. The importance factor for seismic action with a return period of 1000 years was calibrated as 1.4.

The GEO commissioned Arup to carry out another study in 2009, involving an overall seismic hazard assessment of Hong Kong and an area-specific seismic micro-zonation assessment of the north-west New Territories area as well as an evaluation of the potential effects of earthquakes on natural terrain. The findings together with updated seismic hazard contour plots for Hong Kong were published in the "Final Report on Overall Seismic Hazard Assessment" (Arup, 2012). A rock PGA ranging from 0.09*g* to 0.12*g* for a return period of 475 years could be obtained from these updated contour plots. The "Final Supplementary Report on the Study for Seismic Actions" (Atkins, 2013) concluded that the recommended reference PGA of 0.12*g* as in the earlier "Final Report on the Study for Seismic Actions" was still appropriate for Hong Kong (Atkins, 2013). GEO Report No. 311 (Arup, 2015) provided a thorough comparison of the seismicity of the Hong Kong region with those of some other regions including the UK, Eastern USA, Western USA, the Philippines, Japan, Greece and Beijing. It is found that the average recurrence relationship of the Hong Kong region is very similar to that of Eastern America, which is much higher than that of the UK (which is known to have pretty low seismicity) and is about 40 times less than that in highly seismic areas such as California, Japan, Taiwan or the Philippines.

The "Final Report on Overall Seismic Hazard Assessment" (Arup, 2012) also contains de-aggregation plots of all earthquake occurrences that have contributed to the ground-motion hazard values. For a hazard level with a return period of 475 years, it is found that the total contribution of earthquakes with surface-wave magnitudes $M_s = 4.42$ and $M_s = 5.3$ amounts to 51%, which is marginally higher than the total contribution of earthquakes with $M_s = 5.97$ to $M_s = 7.43$ (Atkins, 2013). It is thus considered that the earthquakes that contribute most to the seismic hazard have an M_s not greater than 5.5. BS EN 1998-1 (BSI, 2004c) has specified two types of standard response spectrum, i.e. Type 1 and Type 2, and it explains in Clause 3.2.2.2(2)P NOTE and 3.2.2.3(1)P NOTE that, *"… if the earthquakes that contribute most to the seismic hazard defined for the site for the purpose of probabilistic hazard assessment have a surface-wave magnitude, M_s, not greater than 5.5, it is recommended that the Type 2 spectrum is adopted…"*. In view of the findings of the "Final Report on Overall Seismic Hazard Assessment" (Arup, 2012), the BS EN 1998-1 Type 2 response spectrum shall be used in Hong

Kong as recommended in the “Final Supplementary Report on the Study for Seismic Actions” (Atkins, 2013).

CHAPTER 3 SEISMIC BRIDGE DESIGN SPECIFICATIONS FOR HONG KONG

3.1 Introduction

The Structures Design Manual for Highways and Railways (SDMHR) has been providing guidance and requirements for the design of highway and railway structures in Hong Kong since its first release in August 1993. The second and third editions of SDMHR were published in November 1997 and August 2006, respectively, while the latest edition, i.e. SDMHR 2013, was released in May 2013 (Highways Department, 2013). Unlike the previous editions which referred to the British Standards for design guidance, SDMHR 2013 has been revised for migration to the structural Eurocodes. Specifically, the seismic bridge design shall follow the provisions given in BS EN 1998-1 (BSI, 2004c), BS EN 1998-2 (BSI, 2005b), the UK national annexes to BS EN 1998-1 (BSI, 2004e) and BS EN 1998-2 (BSI, 2005e), and the recommendations in PD 6698 (BSI, 2009) except where modified by the manual to suit Hong Kong conditions.

The key features of the seismic bridge design practices in SDMHR 2013 include:

- SDMHR 2013 has established three importance classes, depending on the consequences and economic losses associated with collapse of a bridge, closure of the bridge in the immediate aftermath of the earthquake, and the cost of repair and/or replacement of the bridge.
- SDMHR 2013 has adopted probabilistic representation of the seismic hazard and adoption of the 10%, 5% and 2% probabilities of exceedance in 50 years (i.e. 475-, 1000- and 2500-year return periods, respectively) for the development of design spectra for bridges of Importance Classes I, II and III, respectively. For a reference return period of 475 years, the peak ground acceleration (PGA) on rock is taken to be 0.12*g*, where *g* is the acceleration due to gravity, as recommended by Atkins (2012). This recommendation aims to improve the seismic design requirements for bridges in Hong Kong. The ratios of the PGAs corresponding to the 1000- and 2500-year return periods to that corresponding to a 475-year return period, i.e. importance factors, are taken to be 1.4 and 2.3, respectively (Atkins, 2012). A single value of PGA for the entire territory of Hong Kong is considered sufficient.
- SDMHR 2013 has adopted the use of response spectrum and site classification system that consider the site factors for determining the design seismic actions in recognition of the dynamic characteristics of both the structures and soils. Two types of response spectrum have been defined in BS EN 1998-1 depending on the surface-wave magnitude of the earthquakes that contribute most to the seismic hazard. It is recommended to adopt the Type 2 response spectrum based on the de-aggregation of the overall seismic hazard assessment of Hong Kong (Atkins, 2013).
- SDMHR 2013 has adopted the response spectrum method as the basic method for seismic demand analysis. Nonlinear time-history analysis is allowed as supplementary analysis.
- SDMHR 2013 has applied the concept of intended ductile and limited ductile behaviour of bridges. In accordance with BS EN 1998-2, the bridge shall be designed so that its behaviour under the design seismic action is either ductile or limited ductile, unless requested otherwise by the owners. A force-based approach with the adoption of behaviour factor, as used in the BS EN 1998-2, aims at providing bridge engineers with

an easy way to conduct ductility design. The other measures essential to ductility design are also introduced, including capacity design and detailed design.

The provisions given in SDMHR 2013 are the minimum requirements developed for the design of new conventional bridges (i.e. slab, beam, girder and box girder superstructures) in Hong Kong to resist the effects of earthquake motions. For special bridges (e.g. suspension bridges, cable-stayed bridges, truss bridges, arch bridges and bridges with span exceeding 150 m) and bridges of critical importance, the economic consequence of collapse or closure of such bridges is comparatively higher than other bridges. Therefore, the owner should specify appropriate project-specific requirements and compliance criteria for such bridges to achieve higher performance for repairable and minimum damage. The owner may also conduct site-specific hazard analysis for establishing the design seismic actions.

3.2 Design Seismic Action

3.2.1 Elastic response spectrum

According to Clause 4.7(1) of SDMHR 2013, "*In the absence of site-specific hazard analysis, Type 2 elastic response spectrum as defined in Clause 3.2.2 of BS EN 1998-1 shall be adopted with ground type determined in accordance with Table 3.1 in Clause 3.1.2 of BS EN 1998-1.*"

The horizontal elastic response spectrum S_e is constructed using the shape factor modified by soil factor S and damping correction factor η, reference peak ground acceleration a_{gR} and return period factor (importance factor) γ_I as shown in Figure 3.1. The values of a_{gR} and γ_I are specified in SDMHR 2013 and the spectral shape factor is obtained from the Type 2 elastic response spectrum of BS EN 1998-1 as discussed above. The soil factor and corner periods T_B, T_C and T_D defining the shape of the Type 2 elastic response spectrum for a ground type as shown in Table 3.1 are taken from Table 3.3 in Clause 3.2.2.2 of BS EN 1998-1. These soil factors generally increase as the soil profiles becomes softer (i.e. in going from ground type A to D) because of the strongly nonlinear behaviour of soil. The nonlinear soil behaviour amplifies the structural response especially at periods from 0.10 s to 1.2 s as shown in Figure 3.2. The horizontal elastic response spectral acceleration for 10% probability of exceedance in 50 years is tabulated in Table 3.2 with the structural period, showing that the seismic coefficients for structures of periods shorter than 1.0 s to 1.6 s will likely exceed 0.07g defined in last version of SDMHR for the ultimate limit state.

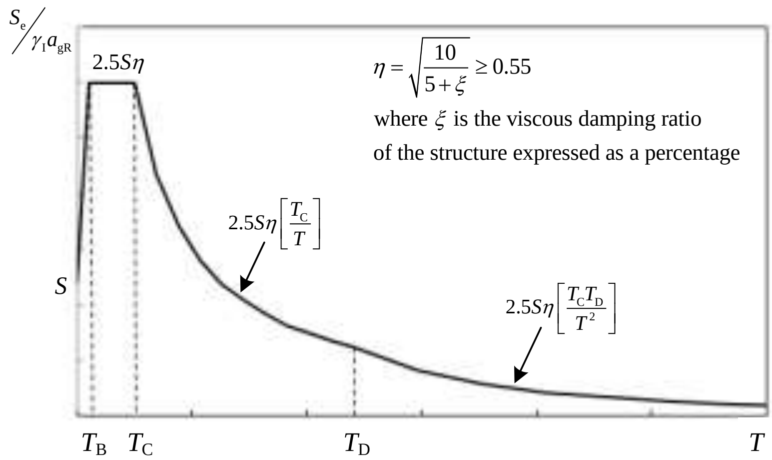


Figure 3.1 Definition of horizontal elastic response spectrum adopted by SDMHR 2013

Table 3.1 Values of parameters describing the Type 2 horizontal elastic response spectrum

Ground Type	S	T_B (s)	T_C (s)	T_D (s)
A	1.00	0.05	0.25	1.20
B	1.35	0.05	0.25	1.20
C	1.50	0.10	0.25	1.20
D	1.80	0.10	0.30	1.20
E	1.60	0.05	0.25	1.20

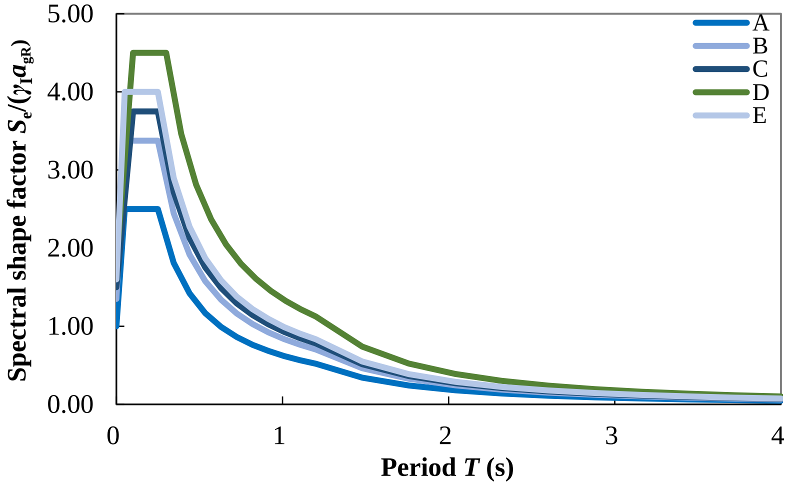


Figure 3.2 Horizontal elastic response spectra with ξ = 5% for the five basic sites

Table 3.2 Elastic response spectral acceleration for 10% probability of exceedance in 50 year (unit: *g*)

Period	Type A	Type B	Type C	Type D	Type E
0.00	0.120	0.162	0.180	0.216	0.192
0.05	0.300	0.405	0.450	0.540	0.480
0.25	0.300	0.405	0.450	0.540	0.480
0.35	0.217	0.293	0.326	0.415	0.348
0.44	0.170	0.230	0.256	0.338	0.273
0.54	0.140	0.189	0.210	0.284	0.224
0.63	0.119	0.161	0.179	0.245	0.190
0.73	0.103	0.140	0.155	0.216	0.166
0.82	0.091	0.123	0.137	0.193	0.146
0.92	0.082	0.111	0.123	0.174	0.131
1.01	0.074	0.100	0.111	0.159	0.119
1.11	0.068	0.092	0.102	0.146	0.109
1.20	0.063	0.084	0.094	0.135	0.100
1.30	0.053	0.072	0.080	0.115	0.085
1.40	0.040	0.062	0.069	0.099	0.073
1.48	0.041	0.055	0.062	0.089	0.066
1.60	0.035	0.047	0.053	0.076	0.056
1.76	0.029	0.039	0.044	0.063	0.046
2.04	0.022	0.029	0.032	0.047	0.035
2.32	0.017	0.023	0.025	0.036	0.027
2.60	0.013	0.018	0.020	0.029	0.021
2.88	0.011	0.015	0.016	0.023	0.017
3.16	0.009	0.012	0.014	0.019	0.014
3.44	0.008	0.010	0.011	0.016	0.012
3.72	0.007	0.009	0.010	0.014	0.010
4.00	0.006	0.008	0.008	0.012	0.009

Note: For elastic response spectral accelerations for 5% and 2% probabilities of exceedance in 50 years, which are adopted for the design of bridges in Importance Classes II and III, the above values shall be multiplied by importance factors of 1.4 and 2.3, respectively.

The vertical elastic response spectrum S_{Ve} is defined as shown in Figure 3.3. The parameters defining the magnitude and shape of the Type 2 vertical elastic response spectrum as shown in Table 3.3 are taken from Table 3.4 in Clause 3.2.2.3 of BS EN 1998-1.

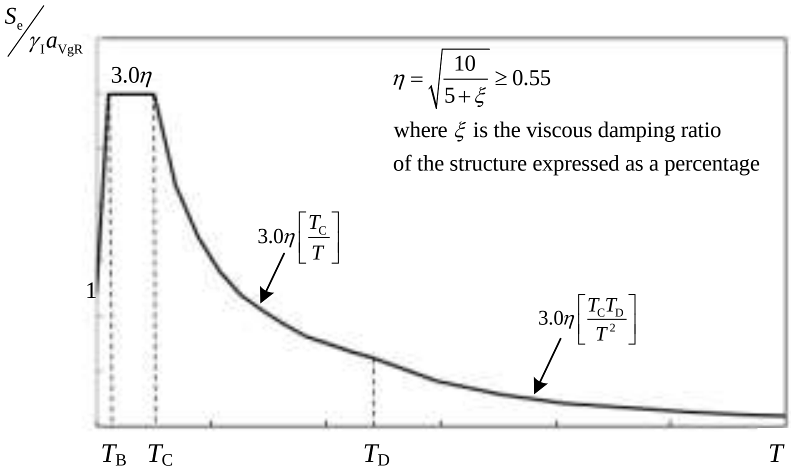


Figure 3.3 Definition of vertical elastic response spectrum adopted by SDMHR 2013

Table 3.3 Values of parameters describing the Type 2 vertical elastic response spectrum

a_{VgR}/a_{gR}	T_B (s)	T_C (s)	T_D (s)
0.45	0.05	0.15	1.00

Site-specific probabilistic hazard analysis shall be conducted to develop spectra that are more accurate for the local seismic and site conditions than that obtained using the general procedure if any of the following applies:

- Soils at the site require site-specific evaluation, i.e. ground types S_1 and S_2, according to Clause 3.1.2(4) of BS EN 1998-1.
- The bridge is considered to be critical or essential, for which a higher degree of confidence of meeting the performance requirement is desirable.

A site-specific procedure shall also be used if the site is located within 10 km horizontally of a known active seismotectonic fault that may produce an event of Moment Magnitude higher than 6.5 according to Clause of 3.2.2.3 of BS EN 1998-2. Based on GEO Report No. 65 (Lee *et al.*, 1998), none of the faults in Hong Kong or in the vicinity of Hong Kong exhibits any evidence of significant recent activity. Therefore, it is recommended in Clause 4.7(3) of SDMHR 2013 that "*all faults within 10 km horizontally of Hong Kong may be considered not active*" and the near source effects in BS EN 1998-2 need not be considered in Hong Kong (Atkins, 2012).

3.2.2 Design spectrum for elastic analysis

It is desirable to design a bridge for ductile behaviour or limited ductile behaviour especially in regions of moderate-to-high seismicity as specified in Clause 2.3.2 of BS EN 1998-2, which means that the structure could go into the inelastic range that permits their design for resistance to seismic forces to be smaller than those corresponding to a linear elastic response. To avoid explicit inelastic analysis, a behaviour factor (q) is allowed for the structure to undertake a fraction of the elastic seismic action necessary for an elastic response, i.e. equivalent linear analysis. The values of the behaviour factor for various materials and structural systems should be taken from Table 4.1 in Clause 4.1.6 of BS EN 1998-2 according to the relevant ductility classes and modified as necessary to account for effects of normalized axial force and accessibility of intended plastic hinges in accordance with Clause 4.1.6(5)-(7) of BS EN 1998-2. The design seismic actions shall be derived by dividing the elastic seismic actions by the behaviour factor, but need not be smaller than 20% of the elastic seismic actions in accordance with Clause 3.2.2.5 of BS EN 1998-1.

3.3 Seismic Demand Analysis

3.3.1 Method of analysis

BS EN 1998-2 has mentioned various methods, including the fundamental mode method (single-mode response spectrum method), response spectrum method and both static (pushover analysis) and dynamic (time-history analysis) nonlinear methods for the purpose of estimating the force and displacement demands on a bridge during an earthquake. The response spectrum method is the primary method and may be replaced by the fundamental mode method to consider only the fundamental mode of the structure for very simple bridges described in Clause 4.2.2.2 of BS EN 1998-2. The nonlinear time-history analysis can be used only in combination with a standard response spectrum analysis to provide insight into the post-elastic response as stipulated in Clause 4.2.4.1(2) of BS EN 1998-2. The nonlinear time-history analysis or static nonlinear analysis may be required for bridges of irregular seismic behaviour as described in Clauses 4.1.8 and 4.1.9 of BS EN 1998-2. Specifically, a force reduction factor r_i for ductile member i is obtained as

$$r_\mathrm{i} = q\frac{M_\mathrm{Ed,i}}{M_\mathrm{Rd,i}} \tag{3.1}$$

where $M_\mathrm{Ed,i}$ is the maximum value of design moment at the intended plastic hinge location of ductile member i as derived from the analysis for the seismic design situation; q is the behaviour factor used in the analysis; and $M_\mathrm{Rd,i}$ is the design flexural resistance of the same section with its actual reinforcement under the concurrent action of non-seismic action effects in the seismic design situation. A bridge is considered to have irregular behaviour in the horizontal direction considered when the following condition is not met:

$$\rho = \frac{r_\mathrm{max}}{r_\mathrm{min}} \le \rho_0 \tag{3.2}$$

where r_max and r_min are the maximum and minimum values of the local force reduction factors among all ductile members, respectively; and $\rho_0 = 2$ is recommended in BS EN 1998-2.

The rationale is associated with the varying overstrength among piers. The sequential yielding of the ductile members may cause concentration of unacceptably high ductility demands and redistribution of stiffnesses after the formation of the first plastic hinges, leading to deviations of the results of the equivalent linear analysis performed with the assumption of a global force reduction factor q (behaviour factor) from those of the nonlinear response of the bridge.

3.3.2 Loading

According to Clause 5.5 of BS EN 1998-2, the load combination in the seismic design situation is:

$$E_\mathrm{d} = G_\mathrm{k} \text{ "+" } P_\mathrm{k} \text{ "+" } A_\mathrm{Ed} \text{ "+" } \Psi_{21}Q_{1\mathrm{k}} \text{ "+" } Q_2 \tag{3.3}$$

where "+" = “to be combined with”; G_k = the permanent actions including self-weight and superimposed dead loads (e.g. road surfacing, parapet, etc.) with their characteristic values; P_k = the characteristic value of prestressing after all losses; A_Ed = the design seismic action; $Q_{1\mathrm{k}}$ = the characteristic value of traffic load; Ψ_{21} = the combination factor for traffic loads; and Q_2 = the quasi-permanent value of actions of long duration (e.g. earth pressure, buoyancy, currents, etc.). In particular, the accompanying traffic and the value of factor Ψ_{21} are taken in accordance with Clause 4.9 of SDMHR 2013 as $\Psi_{21} = 0.2$ for Load Model 1 (BSI, 2003) for road bridges and $\Psi_{21} = 0.2$ for uniformly distributed load q_{fk} (BSI, 2003) for footbridges.

The seismic input is quantified in terms of either a design response spectrum or a spectrum-compatible ground motion time-history generated in accordance with Clause 3.2.3 of BS EN 1998-2 for each component of the earthquake ground motion, depending on the method of analysis. In general, only the horizontal components of the seismic action need to be taken into account for the design of bridges. The vertical earthquake consideration may not be ignored for bridges having unusual framing or geometrical configurations as described in Clause 4.1.7 of BS EN 1998-2. Bridges with components sloping in the vertical direction, prestressed concrete deck, and components such as bearings and links will be affected by vertical ground motions. According to Clause 3.1.2 of BS EN 1998-2, the bridge can be analysed separately for the components of the seismic action in the longitudinal, transverse and vertical directions, and the action effects in each direction under consideration are then combined by applying the SRSS rule (i.e. square root of the sum of squares of the modal responses) or the 30% rule in accordance with Clause 4.2.1.4 of BS EN 1998-2 (which refers to Clause 4.3.3.5.2(4) of BS EN 1998-1) when the response spectrum method is applied, while the bridge should be analysed under the simultaneous action of different components when nonlinear time-history analysis is performed.

For bridges with a continuous bridge deck, the spatial variability of ground motions should be included in the analysis if either the supports are founded on more than one ground types or the length of the continuous bridge deck exceeds a limiting length regardless of the ground types as stipulated in Clause 4.8 of SDMHR 2013. The simplest way to consider the spatial variability of ground motions is by using a single input seismic action (e.g. a single response spectrum or the corresponding accelerogram sets) corresponding to the most severe ground type underneath the bridge supports for the entire structure as described in Clause 3.3(4) of BS EN 1998-2.

3.3.3 Modelling

The models used for dynamic analysis should include the strength, stiffness, mass and energy dissipation characteristics of the structural members and components of the bridge. Depending on the method of dynamic analysis, different approximations may be used for modelling these quantities. Clause 4.1 of BS EN 1998-2 provides general guidance on the distribution of stiffness and mass to capture the response of a bridge subjected to an earthquake.

- In general, the number and location of the displacement degrees of freedom in a bridge model determine the way mass is represented and distributed throughout the structure. The degree of discretization shall represent the distribution of mass so that all significant deformation modes and inertial forces are activated under the design seismic excitation. As most of the mass of a bridge is in the superstructure, the superstructure may generally be divided into four to five elements per span (Marsh *et al.*, 2014).

- The seismic response of a bridge with piers in deep water is affected by the hydrodynamic mass of a volume of water that is forced to move with the piers. This mass should be added to the mass of the pier when modelling the mass of a bridge as stipulated in Clause 4.1.2(5) of BS EN 1998-2. The hydrodynamic added mass is obtained by the mass of a cylinder of water of length equal to the immersed depth. Reasonable estimates of the diameter of the cylinder of water, depending on the shape of the pier cross-section, are given in Annex F of BS EN 1998-2. For piers of circular cross-section, for instance, the diameter of the cylinder of water is equal to the diameter of the pier.

- The model should represent the stiffness of individual structural elements considering the material properties and section dimensions. When an elastic analysis is used to determine the response of an inelastic structure, the stiffness may be based on an equivalent linearized value. A value calculated on the basis of the secant stiffness at the theoretical yield point as shown in Figure 3.4 may be used for reinforced concrete piers in bridges designed for either ductile or limited ductile behaviour, as specified in Clause 2.3.6.1 of BS EN 1998-2. In the absence of sophisticated moment-curvature analysis, the secant stiffness at the theoretical yield point may be determined by one of the two approximate methods presented in Annex C of BS EN 1998-2. For prestressed or reinforced concrete decks, on the other hand, the stiffness of the uncracked gross concrete sections may generally be used. However, the torsional stiffness of concrete decks should be discounted to account for cracking. For open sections or slabs, the torsional stiffness may be ignored. For prestressed concrete box sections, the effective stiffness may be based on 50% of the uncracked gross section stiffness. For reinforced concrete box sections, 30% of the uncracked gross section stiffness may be used in the modelling.

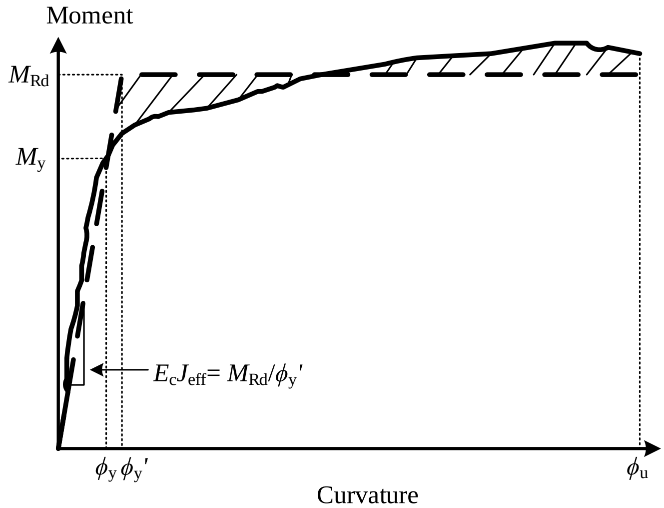


Figure 3.4 Moment-curvature relationship of cross-section for reinforced concrete

- The energy dissipation in a bridge is represented by viscous damping. The selection of an effective viscous damping ratio depends on the type of dynamic analysis. This is usually done by scaling the earthquake response spectrum for the correct amount of damping when response spectrum analysis is used. Suitable damping values may be obtained from field measurements of induced free vibrations or from forced vibration tests. In lieu of measurements, equivalent viscous damping ratios recommended in Clause 4.1.3(1) of BS EN 1998-2 on the basis of the material of the ductile members may be used. Equivalent viscous damping ratios of 5% and 2% are usually assumed for reinforced concrete and prestressed concrete, respectively. In nonlinear analysis, the stress-strain diagrams for both concrete and reinforcement in the regions of potential plastic hinges should reflect the probable post-yield behaviour, taking into account the confinement of concrete and strain hardening effect for steel as discussed in Annex E.2 of BS EN 1998-2. The shape of hysteresis loops should also be properly modelled, taking into account the degradation of strength and stiffness, and hysteretic pinching, if dynamic time-history analysis is performed.

- In accordance with Clause 4.1.4(1) of BS EN 1998-2, soil-structure interaction normally need not be taken into account in the seismic analysis of the entire structure with the supporting members (e.g. piers and abutments) fixed on the foundation soil. However, when the soil flexibility contributes more than 20% of the total displacement at the top of pier, the soil-structure interaction effects should always be considered as stipulated in Clause 4.1.4(2) of BS EN 1998-2, using the appropriate impedances or appropriately defined soil springs in accordance with BS EN 1998-5 (BSI, 2004d). Similar provisions can be found in the AASHTO Guide Specifications for LRFD Seismic Bridge Design (AASHTO Guide Specifications) (AASHTO, 2011) as described in Table 3.4.

Table 3.4 Definition of foundation modelling method (AASHTO, 2011)

Foundation type	Modelling method I	Modelling method II
Spread footing	Rigid	Rigid for rock sites and hard soil sites. For other soil types, foundation springs required if footing flexibility contributes more than 20% to pier displacement.
Pile footing with pile cap	Rigid	Foundation springs required if footing flexibility contributes more than 20% to pier displacement.
Pile bent / drilled shaft	Estimated depth to fixity	Estimated depth to fixity or soil-springs based on *P-y* curves.

3.4 Design and Capacity Verification

In BS EN 1998-2, the earthquake-resisting capacity of a bridge shall be verified according to the method of analysis used. In general, strength verification shall be conducted for bridges designed by the equivalent linear method taking into account ductile or limited ductile behaviour of the structure, whereas the deformation verification shall be applied on the basis of results of nonlinear analysis.

3.4.1 Strength verification

(1) Materials and design strength

The design flexural resistance M_{Rd} and design shear resistance V_{Rd} of a reinforced concrete section shall be determined in accordance with Clauses 6.1 and 6.2 and of BS EN 1992-1 (BSI, 2004a), respectively, and with Clause 5.6.3.4(2)(3)(4) of BS EN 1998-2 where relevant, based on the actual dimensions of the cross-section, final amount of reinforcement, interaction with the axial force and possibly with biaxial bending where relevant. In particular, the ductile concrete members in bridges designed for ductile behaviour shall be reinforced with steel of Grade 500C described in CS2:2012 (SCCT, 2012) in accordance with Clause 5.2.1 of BS EN 1998-2 and Clause 4.10 of SDMHR 2013. Non-ductile concrete members of bridges designed for ductile behaviour and all concrete members of bridges designed for limited ductile behaviour may be reinforced with steel of Grade 500B described in CS2:2012 (SCCT, 2012).

(2) Design seismic action effects and capacity design effects

The design value of moment as derived from the linear analysis for the seismic design situation, after modification to take into account second order effects as specified in Clause 5.4 of BS EN 1998-2, is designated as M_{Ed}. The design value of shear force, i.e. the value as derived from the linear analysis for the seismic design situation multiplied by the behaviour factor q used in the analysis, is designated as V_{Ed}.

For structures designed for ductile behaviour, additional capacity design effects M_C and V_C shall be calculated by analysing the intended plastic mechanism as described in Clause 5.3 of BS EN 1998-2. Specifically, to account for the variability of material strength properties, an overstrength moment M_0 of a section is defined as

$$M_0 = \gamma_0 M_{Rd} \tag{3.4}$$

where γ_0 is the overstrength factor whose value depends on the material and the normalized axial force as specified in Clause 5.3(4) of BS EN 1998-2. A value of 1.35 is recommended for concrete members. The capacity design effects M_C and V_C within the length of members that develop plastic hinge(s) are then calculated under the level of seismic action at which all intended flexural hinges have developed bending moments equal to their overstrength moment as shown in Figure 3.5, but they need not be taken as greater than those resulting from the linear analysis for the seismic design situation multiplied by the behaviour factor q used in the analysis. Besides, the capacity design moment M_C in the vicinity of the hinge need not be taken as greater than the relevant design flexural resistance M_{Rd} of the nearest hinge.

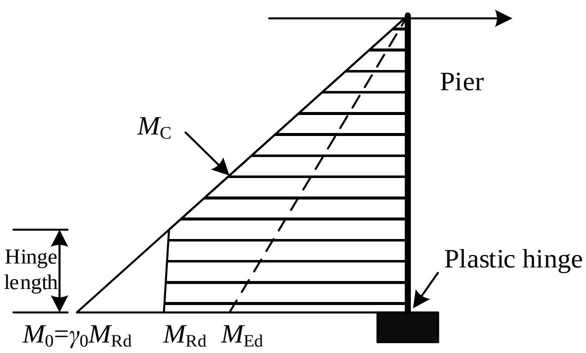


Figure 3.5 Capacity design moments M_C within the member cotaining plastic hinge(s)

(3) Resistance verification

The general criteria for resistance verification are given in Clause 2.3.3 of BS EN 1998-2. In bridges designed for ductile behaviour, the regions of plastic hinges shall be verified to have adequate flexural strength to resist the design seismic action effects, while the shear resistance of the plastic hinges as well as both the shear and flexural resistances of all other regions shall be designed to resist the capacity design effects. The purpose of the capacity design is to ensure that an appropriate hierarchy of resistance exists within various structural components and undesirable modes of inelastic deformation, such as plastic hinging at unintended locations, or shear failure, cannot occur. In bridges designed for limited ductile behaviour, all sections should be verified to have adequate strength to resist the design seismic action effects. These criteria are summarized and compared in Table 3.5.

Table 3.5 Resistance verification criteria of concrete sections

Resistance	Bridges of ductile behaviour		Bridges of limited ductile behaviour
	Plastic hinges	Outside plastic hinges	All sections
Flexural	$M_{Ed} \le M_{Rd}$	$M_C \le M_{Rd}$	$M_{Ed} \le M_{Rd}$
Shear	$V_C \le V_{Rd} / \gamma_{Bd}$	$V_C \le V_{Rd} / \gamma_{Bd}$	$V_{Ed} \le V_{Rd} / \gamma_{Bdl}$

Note: The shear resistance of concrete members shall be divided by an additional safety factor against brittle failure. The safety factor for structures of limited ductile behaviour γ_{Bd1} is recommended to be 1.25 and that for structures of ductile behaviour γ_{Bd} is related to γ_{Bd1} as specified in Clause 5.6.3.3(1)b) of BS EN 1998-2.

It is noted that BS EN 1998-2 does not require capacity design for bridges based on the design for limited ductile behaviour. In fact, the design shear force for limited ductile behaviour V_{Ed} with additional amplification by using a safety factor γ_{Bd1} of about 1.25 does not necessarily cater for the possible material overstrength. As a consequence, shear failure may still occur. The designer may wish to adopt ductile design instead.

(4) Verification of protected components

According to BS EN 1998-2, no significant yielding as described in Clause 5.6.3.6(2) shall occur in the bridge deck. Verification should be carried out under the most adverse design seismic action effects for bridges of limited ductile behaviour, and under the capacity design effects for bridges of ductile behaviour.

All critical structural components of concrete abutments shall be designed to remain essentially elastic under the design seismic action. For those abutments flexibly connected to the deck as defined in Clause 6.7.2 of BS EN 1998-2, the capacity design effects from the bearings should be taken into account for the seismic design of the abutments if a ductile behaviour has been assumed for the bridge.

Any joint between a vertical ductile pier and the deck or a foundation element adjacent to a plastic hinge in the pier shall be designed in shear to resist the capacity design effects of the plastic hinge in the relevant direction. The design and verification of joints shall be conducted in accordance with Clause 5.6.3.5 of BS EN 1998-2.

3.4.2 Deformation verification

A ductile member is intentionally designed to deform inelastically for several cycles without significant degradation of strength or stiffness under the design seismic actions. Besides, the intended plastic hinges shall be provided with adequate ductility to ensure the required overall global ductility of the structure. Note that conformance to accompanying details specified in the code is deemed to ensure the availability of adequate local and global ductility as well as stable ductile behaviour. When nonlinear static or dynamic analysis is performed, however, the chord rotation demands shall be checked against the available rotation capacities of the plastic hinges in accordance with Clause 4.2.4.4 of BS EN 1998-2. Specifically, the plastic hinge rotation demands $\theta_{p,E}$ shall be safely lower than the rotation capacities as follows:

$$\theta_{p,E} \leq \theta_{p,u} / \gamma_{R,p} \tag{3.5}$$

where $\theta_{p,u}$ is the probable rotation capacities derived from relevant test results or calculated from the ultimate curvatures and plastic hinge lengths as shown in Annex E.3 of BS EN 1998-2; and $\gamma_{R,p}$ is a safety factor that reflects local defects of the structure, uncertainties of the model and/or the dispersion of relevant test results.

3.5 Seismic Relevant Detailing

3.5.1 Confinement for concrete and main bars

The reliable performance of ductile members largely depends on the structural details. To be able to sustain the levels of inelastic deformation required under the design-level earthquake without premature failure, the potential plastic hinge regions must be properly detailed. The detailing rules are as follows:

- Ductile behaviour of the compression concrete zone shall be ensured within the potential plastic hinge regions. This is accomplished by providing an adequate amount of confining reinforcement over the design length of potential plastic hinge as described in Clause 6.2.1 of BS EN 1998-2. Confinement of compression zone shall be provided in the potential hinge regions where the normalized axial force exceeds 0.08 as specified in Clause 6.2.1.1(2) of BS EN 1998-2 and in critical sections of bridges designed for limited ductile behaviour as specified in Clause 6.5.1(2) of BS EN 1998-2, unless confinement is not necessary according to 6.2.1.1(3) of BS EN 1998-2. Pursuant to Clause 6.2.4(4) of BS EN 1998-2, there is no need for verification of the confining reinforcement in accordance with

Clause 6.2.1 of BS EN 1998-2 in piers with simple or multiple box section unless the normalized axial force exceeds 0.20, provided that avoidance of buckling of longitudinal compression reinforcement is ensured in accordance with Clause 6.2.2 of BS EN 1998-2.

- Buckling of longitudinal reinforcement shall be avoided along potential hinge areas of bridges designed for ductile behaviour and along critical sections of bridges designed for limited ductile behaviour, even after several cycles into the post-yield region. Transverse reinforcement (hoops or cross-ties) perpendicular to the longitudinal bars in accordance with Clause 6.2.2 of BS EN 1998-2 shall be provided.
- The confining reinforcement shall be properly anchored and/or spliced in accordance with Clauses 6.2.2(3)a), 6.2.3(1) and (2) of BS EN 1998-2. Besides, no splicing by lapping or welding of longitudinal reinforcement is allowed within the plastic hinge region.

3.5.2 Overlap length and clearance

During an earthquake, out-of-phase response between adjacent girders and between girders and abutments may lead to large relative hinge displacements including hinge opening and closing, resulting in unseating of girders and pounding of adjacent elements. These shall be avoided by providing sufficient overlap length and/or clearance at articulations to accommodate the displacements in the seismic design situation.

(1) Design value of displacement

The design seismic displacement d_E shall be derived in accordance with Clause 2.3.6.1(6) of BS EN 1998-2, with the displacement determined from a linear analysis based on the design spectrum multiplying by the displacement ductility factor specified in Clause 2.3.6.1(8) of BS EN 1998-2. Additionally, the displacement in the seismic design situation shall also take into account the long-term displacement due to post-tensioning, shrinkage and creep for concrete decks d_G, displacement due to thermal movements d_T, and the effective displacement of two parts due to the spatial variation of the seismic ground displacement d_{eg} estimated in accordance with Clause 6.6.4(3) of BS EN 1998-2.

(2) Minimum overlap length and clearance

At locations where the relative movement between structural elements is intended under seismic conditions, clearance shall be provided between those elements as specified by Clause 2.3.6.3 of BS EN 1998-2. At supports where the relative movement between the supported and supporting members is intended, a minimum overlap length shall be provided to maintain the function of the support under extreme seismic displacement as specified by Clause 6.6.4 of BS EN 1998-2. The values of minimum overlap length and clearance are slightly different from location to location as summarized below:

- For protection of critical or major structural members, the clearance shall accommodate d_{Ed}, i.e. the total design value of the displacements d_E, d_G and d_T combined in accordance with Clause 2.3.6.3(2) of BS EN 1998-2.
- In the case of ductile / resilient members or special energy absorbing devices provided between major structural members to prevent large shock forces caused by unpredictable impact, a slack at least equal to d_{Ed} shall be designed.
- For non-critical structural components such as deck movement joints and abutment back-walls that are expected to be damaged due to the design seismic action but avoiding any damage under frequent earthquakes, the clearance may accommodate fractions of d_E and of d_T, respectively, after allowing for any d_G as specified in Clause 2.3.6.3(5) of BS EN

1998-2. The appropriate values of such fractions shall be chosen to cater for a predictable mode of damage and provide for the possibility of permanent repair.

- At an end support of an abutment, the basic minimum overlap length l_{ov} in accordance with Clause 6.6.4(3) of BS EN 1998-2 shall be provided, i.e. sum of d_{Ed} and d_{eg} plus a minimum support length for safe transmission of the vertical reaction (no less than 400 mm).
- In the case of an intermediate separation joint between two sections of the deck such as in-span hinge, the minimum overlap length should be estimated by taking the square root of the sum of the squares of l_{ov} calculated for each of the two sections of the deck.
- At an end support of a deck section on an intermediate pier, the minimum overlap length should be taken as l_{ov} plus d_E of the top of the pier.
- For decks connected to piers or to an abutment through seismic links with slack, the minimum overlap lengths defined above shall be increased by the slack.

3.5.3 Bearings and seismic links

The bearings comprise fixed bearing, moveable bearing and elastomeric bearing. The seismic demands on bearings shall be taken in accordance with Clause 6.6.2 of BS EN 1998-2.

Fixed bearings shall generally be designed to resist the design seismic action effects determined through capacity design. Fixed bearings may be designed solely for the effects of seismic design situation from the analysis, provided that they can be replaced without difficulties and that seismic links are provided as a second line of defence. Moveable bearings shall be capable of accommodating without damage the displacement d_{Ed}. Elastomeric bearings to resist only non-seismic horizontal actions shall be designed to resist the maximum shear deformation imposed by vertical compression, total design horizontal displacement and total design angular rotation.

3.6 Foundation Design

According to SDMHR 2013, Eurocodes shall be used for the design of new highway structures, with the exception of foundation design. The provisions for the design of foundations given in BS EN 1997-1 (for normal operation design) (BSI, 2004b) and BS EN 1998-5 (for earthquake resistance design) (BSI, 2004d) are not followed. The foundations of highway structures in Hong Kong shall be designed in accordance with the principles set out in BS 8004:1986 (BSI, 1986) and the pile design shall be carried out by reference to Geotechnical Engineering Office Publication No. 1/2006 (GEO, 2006).

The design philosophy of BS 8004:1986 is fundamentally different from that of Eurocodes. In 2015, however, the second edition of BS 8004 was released and the design philosophy was completely changed reflecting advances in foundation technology over the past 30 years. The design philosophy of BS 8004:2015 (BSI, 2015) is fully compatible with the Eurocodes. It is desirable for practising engineers to be mindful of the latest changes and the difference in foundation design concepts between BS EN 1997-1 and BS 8004:1986.

3.6.1 Limit states

BS EN 1997-1 has been drafted on the basis of limit state design and, more importantly, it makes a clear distinction between the ultimate limit state (ULS) design and serviceability limit state (SLS) design for spread foundations and piled foundations as shown in Table 3.6. For each geotechnical design situation, it shall be verified that no relevant limit state is exceeded.

Unlike BS EN 1997-1, BS 8004:1986 provides much advisory information, i.e. not obligatory, and it often refers readers to datasheets and other publications. Because of the nature of its design philosophy, BS 8004:1986 does not distinguish ULS from SLS explicitly. Instead, it goes through a whole list of design considerations one by one as shown in Table 3.7, which essentially overlap with items in BS EN 1997-1 (Wang and Thusyanthan, 2008).

Table 3.6 General design requirements set out in BS EN 1997-1 (BSI, 2004b)

Limit states	Design requirements	
	Spread foundations	Piled foundations
Ultimate limit state (ULS)	Overall stability Adequate bearing resistance Adequate sliding resistance Adequate structural capability	Overall stability Adequate bearing resistance Adequate uplift resistance Adequate structural capability
Serviceability limit state (SLS)	No excessive settlement Design against heave Design for vibration loads	No excessive settlement No excessive lateral movement Design against heave Design for vibration loads

Table 3.7 Comparison between BS 8004:1986 and BS EN 1997-1 on design requirements

BS 8004:1986	BS EN 1997-1
Ground movement, due to • Application of load • Removal of load • Factors independent of foundation load	Overall stability; settlement; heave
Ground water; flooding	Heave; uplift; downdrag
Allowable bearing pressure	Bearing capacity
Structural considerations	Structural capacity
Ground / Structure interdependence	Soil / Structure interaction

Moreover, BS EN 1997-1 subdivides ULS into 5 broad categories as described below:

- EQU: Loss of equilibrium of the structure or the ground, considered as a rigid body, in which the strengths of structural materials and the ground are insignificant in providing resistance.
- STR: Internal failure or excessive deformation of the structure or structural elements, including footings, piles, basement walls, etc., in which the structural material is significant in providing resistance.
- GEO: Failure or excessive deformation of the ground, in which the strength of soil or rock is significant in providing resistance.
- UPL: Loss of equilibrium of the structure or the ground due to uplift by water pressure (buoyancy) or other vertical actions.
- HYD: Hydraulic heave, internal erosion and piping in the ground caused by hydraulic gradients.

The main focal points in ULS design are the GEO and STR limit states. The UPL and HYD limit states should only be checked if buoyancy and hydraulic gradients are matters of concern, while the EQU limit state is limited to rare cases such as a rigid foundation bearing on rock (BSI, 2004b). Each category has a particular set of partial factors, which should be applied in the corresponding design calculations.

3.6.2 Basis of design

BS EN 1997-1 allows limit states, including both ULS and SLS, to be verified by one or a combination of the following methods:

- Use of calculations
- Adoption of prescriptive measures
- Experimental models and load tests
- An observational method

(1) Design by calculation

Design by calculation is a direct method, which comprises separate analyses for each of the limit states. It involves examination of actions, properties of substrate materials, geometrical data, limiting values of deformations, crack widths, vibrations, etc., and calculation models, as illustrated in Figure 3.6. It shall be verified that the design effects of actions will not exceed the design resistance and any relevant serviceability criterion is within its limiting value. The resistance to an action as well as the settlement may be calculated using an analytical model, a semi-empirical model or a numerical model.

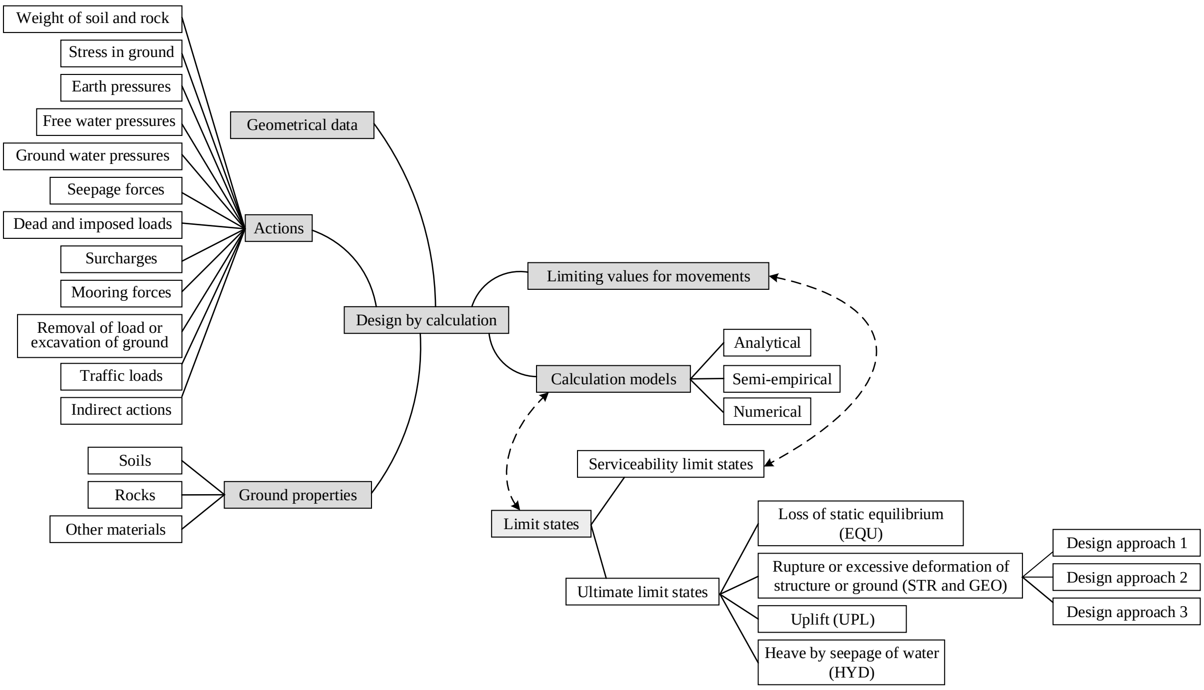


Figure 3.6 Illustration for design by calculation (Bond and Harris, 2008)

In view of the fact that many components can affect the performance of a foundation such as material properties and types of actions, BS EN 1997-1 introduces three distinctive Design Approaches (DAs) for the GEO and STR limit states in the persistent and transient situations. They differ in the way the partial factors are distributed between actions or the effects of actions (denoted by A_i, i = 1, 2), soil properties (denoted by M_i, i = 1, 2) and resistances (denoted by R_i, i = 1, 2, 3, 4) as explained below with the i-th set of values of the partial factors shown in Table 3.8:

- DA-1 Combination 1: A1 “+” M1 “+” R1

  In this approach, partial factors are applied to actions or effects of actions alone.

- DA-1 Combination 2 (except for the design of axially loaded piles and anchors): A2 “+” M2 “+” R1

  In this approach, partial factors are applied mainly to ground strength parameters.

- DA-1 Combination 2 (for axially loaded piles and anchors): A2 “+” (M1 or M2) “+” R4

In this approach, partial factors are applied to actions or effects of actions and to ground resistances for calculating resistances of piles, and sometimes to ground strength parameters for calculating unfavourable actions on piles, for example, owing to negative skin friction or transverse loading.

- DA-2: A1 “+” M1 “+” R2

  In this approach, partial factors are applied to actions or effects of actions and to ground resistances simultaneously.

- DA-3: (A1 or A2) “+” M2 “+” R3

  In this approach, partial factors are applied to actions or effects of actions from the structure and to ground strength parameters simultaneously.

Table 3.8 Partial factor sets for ULS foundation design based on BS EN 1997-1

Partial factors on actions (γ_F) or effects of actions (γ_E), based on Table A-3 of BS EN 1997-1 Annex A

Action		Symbol	Set A1	Set A2
Permanent	Unfavourable	γ_G	1.35	1.0
	Favourable		1.0	1.0
Variable	Unfavourable	γ_Q	1.5	1.3
	Favourable		0	0

Partial factors for soil parameters (γ_M), based on Table A-4 of BS EN 1997-1 Annex A

Material Property	Symbol	Set M1	Set M2
Angle of shearing resistance	γ_{φ}'	1.0	1.25
Effective cohesion	γ_c'	1.0	1.25
Undrained shear strength	γ_{cu}	1.0	1.4
Unconfined compressive strength	γ_{qu}	1.0	1.4
Weight density	γ_{γ}	1.0	1.0

Partial resistance factor (γ_R) for spread foundation, based on Table A-5 of BS EN 1997-1 Annex A

Resistance	Symbol	Set R1	Set R2	Set R3
Bearing	$\gamma_{R;v}$	1.0	1.4	1.0
Sliding	$\gamma_{R,h}$	1.0	1.1	1.0

Partial resistance factor (γ_R) for driven piles, based on Table A-6 of BS EN 1997-1 Annex A

Resistance	Symbol	Set R1	Set R2	Set R3	Set R4
Base	γ_b	1.0	1.1	1.0	1.3
Shaft (compression)	γ_s	1.0	1.1	1.0	1.3
Total/combined (compression)	γ_t	1.0	1.1	1.0	1.3
Shaft in tension	$\gamma_{s,t}$	1.25	1.15	1.1	1.6

Partial resistance factor (γ_R) for bored piles, based on Table A-7 of BS EN 1997-1 Annex A

Resistance	Symbol	Set R1	Set R2	Set R3	Set R4
Base	γ_b	1.25	1.1	1.0	1.6
Shaft (compression)	γ_s	1.0	1.1	1.0	1.3
Total/combined (compression)	γ_t	1.15	1.1	1.0	1.5
Shaft in tension	$\gamma_{s,t}$	1.25	1.15	1.1	1.6

Note: Only the partial factor tables for the STR and GEO limit states are shown. For the EQU, UPL and HYD limit states, one must use the other partial factor tables specified in Annex A of BS EN 1997-1. The values of partial factors for SLS are normally taken to be 1.0.

As opposed to BS EN 1997-1, BS 8004:1986 adopts an overall safety factor γ of 2.0 or 3.0 for the ultimate bearing stress while the loads are unfactored. Since a single factor is applied to cater for uncertainties in all components including the action or effect of action, soil properties and resistance, the global safety factor inevitably adopts a rather conservative value than the partial factors will normally do.

(2) Load test and observation method

When there is little local experience or insufficient information on the ground conditions, it is difficult to predict the structural behaviour with sufficient accuracy. In these cases, reliance is placed on the load tests or the observation method. Tests may be carried out on a sample of the actual construction. When the results of load tests on full scale or reduced scale models are used to justify a design, allowance shall be made for the differences in ground conditions between the test and the actual construction, the time effects especially if the duration of the test is much less than the duration of loading in the actual construction, and the scale effects. In the observation method, the design is reviewed during construction. A range of potential behaviour and the relevant acceptable limits of behaviour must be identified before construction. Depending on the observed behaviour during construction, the planned contingency actions may be put into operation as appropriate, e.g. when the limits of behaviour are exceeded. This approach is normally based on serviceability limits and does not explicitly provide sufficient reserve against ultimate failures. It is therefore important that the limiting design criteria are suitably conservative.

(3) Prescriptive method

In addition to the use of calculation and indirect methods, such as load tests, tests on experimental models and observation method, the prescriptive method based on presumed bearing resistance is also allowed where the ground conditions are well known. Spread foundations on rock may normally be designed using the method of presumed bearing resistance. However, unlike BS 8004:1986 providing presumed allowable bearing values for a variety of sites including rocks, non-cohesive soils, cohesive soils, peat and organic soils, made ground fill, high porosity chalk and Keuper marl, BS EN 1997-1 only includes presumed bearing values for rocks.

The presumed vertical bearing values for granitic and volcanic rocks, meta-sedimentary rock, intermediate soil (e.g. decomposed granite and decomposed volcanic), non-cohesive soil and cohesive soil, the presumed lateral bearing pressure for rock as well as the presumed bond or friction between rock and concrete or grout of piles for use in Hong Kong may be found in the Code of Practice for Foundations (BD, 2017).

3.7 Summary of Procedures for Seismic Design of Highway Bridges

The procedures for seismic design of highway bridges discussed above are summarized in the flowchart in Figure 3.7.

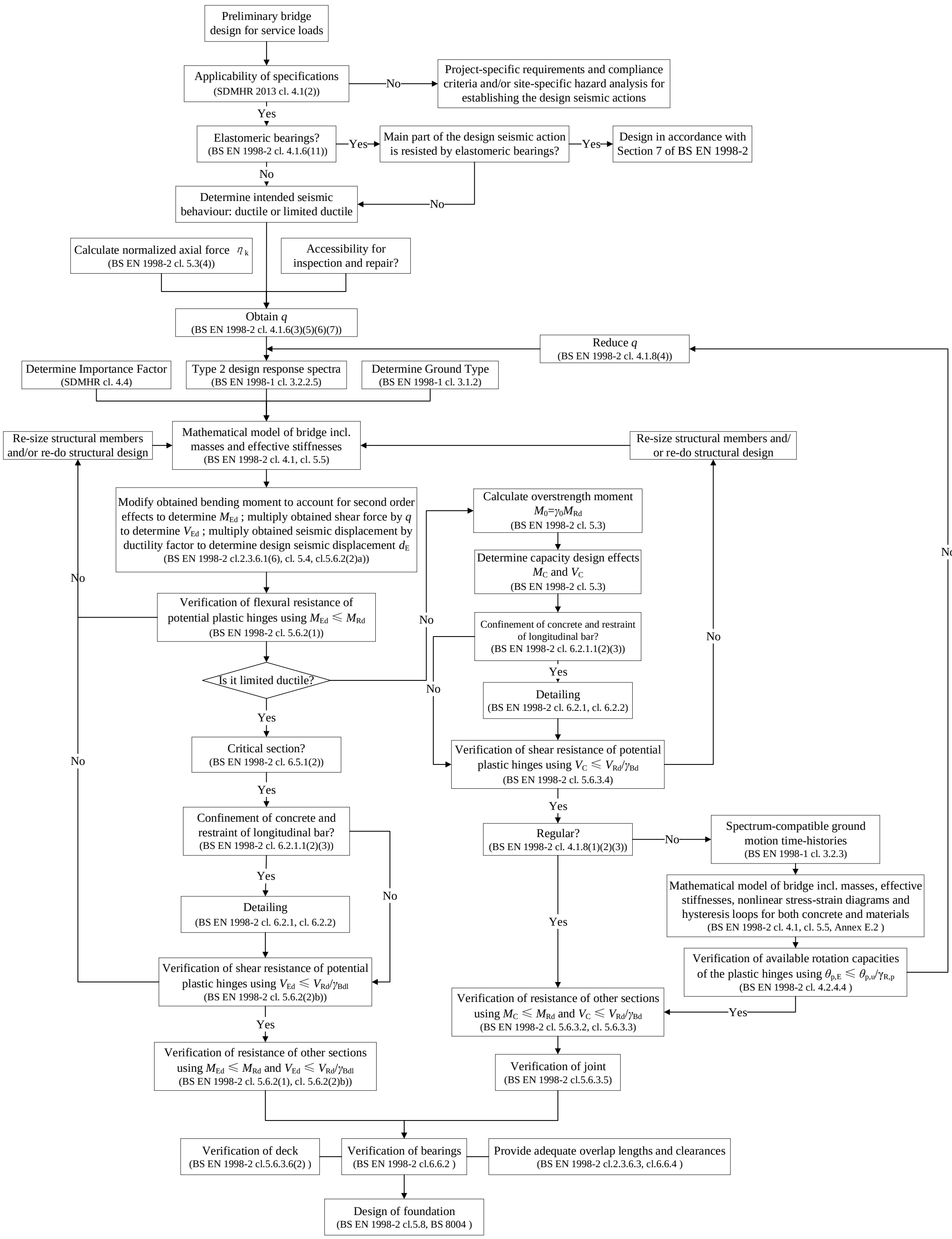


Figure 3.7 Procedures for seismic design of highway bridges based on SDMHR 2013

3.8 Seismic Design of Railway Bridges

The railways of Hong Kong are predominantly run and maintained by the Mass Transit Railway Corporation Limited (MTRCL). The corporation has adopted a separate in-house design manual for railway structures, i.e. New Works Design Standard Manual (NWDSM) (MTRCL, 2013). NWDSM also includes provisions for earthquake loading and earthquake analysis of bridges, which are different from those described in SDMHR 2013.

NWDSM specifies a reference peak ground acceleration of $0.15g$ with a return period of 1000 years. According to SDMHR 2013, the reference peak ground acceleration that corresponds to a reference return period of 475 years is $0.12g$, while the peak ground acceleration is taken as $0.12g \times 1.4 = 0.168g$ for a return period of 1000 years. Aside from this, NWDSM stipulates that bridge design and analysis shall comply with American Association of State Highway Transport Officials - Standard Specifications for Highway Bridges, Seismic Design (AASHTO Standard Specifications) (AASHTO, 2002), while SDMHR 2013 mainly refers to BS EN 1998-1 and BS EN 1998-2 for seismic bridge design.

The determination of seismic action and the method of seismic analysis adopted by NWDSM seem to be compatible with those by SDMHR 2013 as elaborated below:

- For bridges of conventional steel and concrete girder and box girder forms with spans not exceeding 150 m, the AASHTO Standard Specifications classify them as either essential bridges or other bridges on the basis of social / survival and security / defence requirements, which are roughly equivalent to bridges of Importance Classes II and I, respectively, as defined in SDMHR 2013.
- In accordance with the AASHTO Standard Specifications, bridges with a seismic coefficient of 0.15 shall be assigned to Seismic Performance Category B. For Seismic Performance Category B, the design seismic force for the superstructure, supporting substructure including the columns and piers, and the connections between the superstructure and the supporting substructure shall be determined by dividing the elastic seismic force by the appropriate Response Modification Factor (R). For reinforced concrete piers, the recommended R-Factors to be used for single column and multiple-column bent are 3.0 and 5.0, respectively. These are similar to the Behaviour Factors (q) used in BS EN 1998-2. However, BS EN 1998-2 specifies q-values of 1.5 to 3.5 for reinforced concrete piers, depending on the intended ductility. In other words, the piers designed in accordance with the NWDSM are intended for ductile behaviour from the perspective of BS EN 1998-2. For connections such as bearings and shear keys, the AASHTO Standard Specifications have prescribed the R-factor of 1.0 to enforce elastic behaviour. BS EN 1998-2 adopts capacity design effects for the non-ductile components, which are determined from the strength of adjacent ductile members with an overstrength factor of at least 1.3 but it need not be greater than the elastic seismic forces.
- According to the AASHTO Standard Specifications, the design seismic coefficient S_d is defined using the Acceleration Coefficient (A), the Spectral Factor (C) and the Response Modification Factor (R) as

  $$S_\mathrm{d} = AC/R \tag{3.6}$$

  NWDSM specifies that the Spectral Factor C proposed in the paper "Seismic Design of Buildings in Hong Kong" by Scott *et al.* (1994) shall be used to suit Hong Kong conditions, which is related to the Site Coefficient (S) and Period of the bridge (T) as

  $$C = S/T^{1.1} \leq 2.7 \tag{3.7}$$

  where S = 0.47 for rock and hard soil sites, and S = 0.67 for reclaimed sites.

  The values of the design seismic coefficient obtained using the NWDSM model with an R-factor of 3.0 and using the BS EN 1998-1 Type 2 spectrum with a q-factor of 3.5, both for single-column piers intended for ductile behaviour, are compared in Figure 3.8.

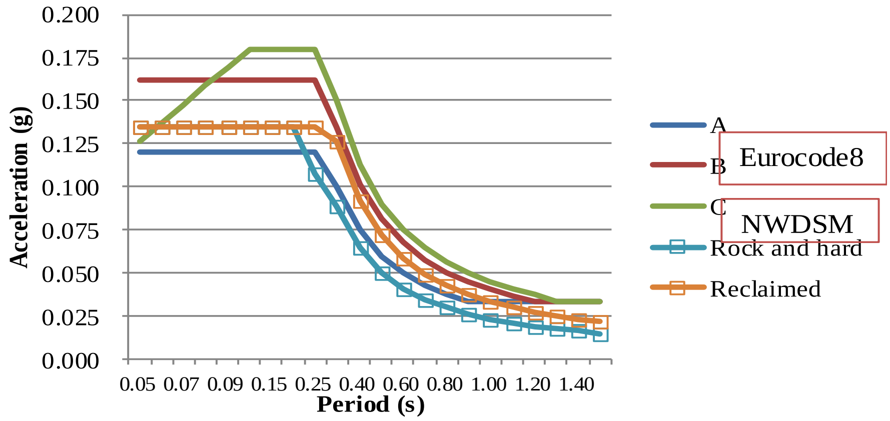


Figure 3.8 Design response spectra for typical ground types

BS EN 1998-1 has defined five site classes in total, i.e. Ground Types A, B, C, D and E, mainly based on the shear wave velocity. However, only Ground Types A, B and C are presented here as they are considered to be the most representative of site conditions in Hong Kong. On the other hand, the NWDSM classification comprises two, i.e. rock and hard sites, and reclaimed sites. Figure 3.7 shows that the design seismic coefficients defined for hard sites and reclaimed sites by NWDSM are very close to those defined for Ground Types A and B, respectively, by BS EN 1998-1 for structural periods of 0.30 s to 0.90 s. Generally speaking, the design seismic forces in accordance with NWDSM are comparable to those based on SDMHR 2013 or even smaller considering that *R*-factors greater than 3.0 can be used, suggesting that the NWDSM seismic design is aimed for more ductile bridge structures.

- The AASHTO Standard Specifications and BS EN 1998-2 have similar requirements for the selection of a suitable method of analysis for a particular bridge. The fundamental mode method, i.e. single-mode spectral method, can be used for the analysis of "Regular" bridges. The multi-mode spectral method or more rigorous, generally accepted procedure such as the time-history method, are recommended for the analysis of bridges that are "Not Regular".

The NWDSM and SDMHR 2013 are slightly different in terms of detailing as shown in Table 3.9. Confining reinforcement shall be provided in the potential plastic hinge regions, which are generally located at the top and/or bottom of the columns. Both the diameter and spacing of the confining reinforcement shall satisfy certain criteria. The NWDSM requires that the detailing of reinforced concrete members shall comply with the latest issue of Code of Practice for Structural Use of Concrete (CoP-SUC), while SDMHR 2013 refers to BS EN 1992-1 (BSI, 2004a) and BS NA EN 1992-2 (BSI, 2005d) for general rules and BS EN 1998-2 for rules for seismic ductility. Table 3.9 shows that CoP-SUC 2013 (BD, 2013) and BS EN 1998-2 have slightly different specifications for these requirements. In Hong Kong, the least dimensions of highway bridge piers are often about 1500 mm to 2000 mm, the majority of longitudinal bars used are 32 mm or 40 mm in diameter, and the transverse reinforcement is mostly of 16 mm. The longitudinal spacing of the confining reinforcement can then often exceed 200 mm in accordance with SDMHR 2013, while this spacing should not exceed 150 mm in accordance with NWDSM. As a result, more confining reinforcement shall be provided for confinement of concrete in accordance with NWDSM, which may result from the intended higher levels of ductility implied in AASHTO Standard Specifications. In addition to the hoops, supplementary cross-ties or links shall also be provided to make sure that the transverse distance between hoop legs or cross-ties is less than 200 mm such that any bar is not far away from a restrained bar based on BS EN 1998-2. CoP-SUC 2013 has specified a similar requirement.

Table 3.9 Requirements for confining reinforcement

Requirement	BS EN 1998-2	CoP-SUC 2013
Minimum diameter	6 mm or 1/4 the diameter of the largest longitudinal bar, whichever is greater	10 mm or 1/4 the diameter of the largest longitudinal bar, whichever is greater
Maximum spacing along the column	1/5 of the minimum dimension of the confined core or 6 times the longitudinal bar diameter, whichever is smaller	8 times the longitudinal bar diameter or 150 mm, whichever is smaller
Maximum distance between hoop legs	1/3 of the minimum dimension of the confined core or 200 mm, whichever is smaller	20 times the link bar diameter or 250 mm, whichever is smaller

To conclude, although NWDSM and SDMHR 2013 refer to different design codes for the seismic design of railway bridges, the seismic design method specified in AASHTO Standard Specifications and that in BS EN 1998-2 are found to be comparable to each other. In relation to the intended seismic behaviour, NWDSM appears to call for design of railway bridges for a higher level of ductility and require more closely spaced confining reinforcement at the potential plastic hinge zones as compared to SDMHR 2013. Actually in a sense, NWDSM had been more advanced in seismic design philosophy than SDMHR prior to its revision in 2013.

CHAPTER 4 GUIDANCE FOR SEISMIC DESIGN OF NEW BRIDGES

The specifications and procedures for seismic design of bridges in Hong Kong as introduced in the Structures Design Manual for Highways and Railways 2013 (SDMHR 2013) (Highways Department, 2013) have already been summarized in Chapter 3. This chapter will focus on the related conceptual design, which covers considerations of bridge location, sizing, geometry and bridge systems. In the seismic design situation, it is clear that choices made early in selecting the bridge type, articulation and configuration can have significant effects on the seismic design and ultimately the seismic response. It is particularly important that implications of the seismic action are considered at the conceptual design stage of the bridges so as to determine the most suitable solution for a bridge to meet the performance specifications for the design-level earthquake, while balancing the life-cycle cost and all other constraints. While various structural options for seismic resistance are subject to a number of non-seismic constraints such as the traffic flow or dominating ground terrain, it is still desirable to identify the preferred structural characteristics for seismic resistance that would apply in the absence of any constraints. In addition, recommendations are provided on the determination of intended seismic behaviour of the bridge, i.e. elastic, limited-ductile or ductile behaviour, based on a series of parametric studies conducted to illustrate the effects of the intended behaviour on the seismic design of bridge pier and foundation.

4.1 Structural Forms

The response of a bridge under seismic loading depends largely on its structural form, including the bridge geometry, structural type, member strengths, and member articulations. These attributes combine to govern how lateral loads will be induced in a bridge by an earthquake and how such loads will be resisted by the bridge. Some good practices and the underlying principles are presented below.

4.1.1 Earthquake resisting system

Identification of a suitable lateral force-resisting scheme and selection of the necessary elements to realise the scheme should be accomplished in the conceptual design phase.

Bridge structures are normally composed of the superstructure, substructures, foundations and their connections. The superstructure of a bridge is the main structural component that carries not only its self-weight but also the traffic and other loads acting on it, thereby contributing most of the mass. Under earthquake excitation, the lateral inertial force on the superstructure will be transferred to the nearest bent through the bearings or joints, through the cap beam of the bent to the columns, from the columns to the foundations, and finally to the soil.

Except for extremely important bridges, it is generally expedient to introduce a certain level of ductility into the bridge in order to bring down the forces in the seismic load path, especially in moderate-to-high seismicity areas. The use of ductile systems, where the maximum element forces developed in the system are capped by the element yield strengths and somewhat less than those calculated for an equivalent fully elastic system, also limits the maximum inertial forces that can be developed in the system based on force equilibrium. Thus, the designer can control the maximum inertial force that will be developed in the structure by controlling the maximum lateral force that the structural system can transmit.

The approach to achieve ductile behaviour for bridges is by either providing for the formation of a dependable plastic mechanism or using seismic isolation and energy dissipation devices. Essentially, the earthquake resisting systems should meet the following requirements:

- The locations selected for energy dissipation should be chosen with easy accessibility for inspection and repair wherever possible.
- The earthquake resisting system must provide a reliable and uninterrupted load path for transmitting seismically induced forces to the surrounding soil without any potential of brittle failures and have sufficient means of energy dissipation and/or restraint to reliably control seismically induced displacements.
- The inelastic action of a structural member cannot jeopardize the gravity load-carrying capability of the structure.

Clause 4.1.6 of BS EN 1998-2 (BSI, 2005b) has provided some guidance on the selection of earthquake resisting elements that contribute to energy dissipation during an earthquake. A more comprehensive list of all kinds of earthquake resisting elements and their appropriateness can be found in the AASHTO Guide Specifications for LRFD Seismic Bridge Design (AASHTO Guide Specifications) (AASHTO, 2011), which consist of three categories, i.e. "Permissible", "Permissible with owner's approval", and "Not recommended for new bridges", for the purpose of encouraging good practice in seismic design as shown in Table 4.1. One guiding principle that distinguishes elements in the three categories is the desire to keep potential damage at locations that can be inspected (Marsh *et al.*, 2014). For instance, plastic hinges that form above ground fall into the "Permissible" category. On the other hand, plastic hinges that form below the ground surface (i.e. in-ground plastic hinges such as plastic hinging in piles below a wall pier or an integral abutment or in piles with the same strength as the extension above ground) are not preferred and thus fall within the "Permissible with owner's approval" category. This can be interpreted as being "Permissible under exceptional circumstances with full justifications". This recognizes the fact that in-ground plastic hinging may not be avoidable in all cases and, once such behaviour is expected, it may be utilized with the approval of the owner. This is important because the owner should be aware that in-ground damage may be difficult or impossible to detect and thus such damage will likely be unrepairable. One exception is that plastic hinges that form just below the ground surface are permissible owing to their accessibility by reasonable excavation. These considerations are also included in BS EN 1998-2 in terms of the magnitude of the behaviour factor. The maximum values of behaviour factor as specified in Table 4.1 of BS EN 1998-2 may be used only if the locations of all the relevant plastic hinges are accessible for inspection; otherwise these values shall be multiplied by 0.6 to enforce a lower extent of ductility as stipulated in Clause 4.1.6(6). Other earthquake resisting elements listed in Table 4.1 that require the owner's approval represent less conservative design methods than the traditional options, although these systems should perform well and meet the no-collapse requirement if the design is done correctly. It should be noted that these systems, particularly the foundation systems, are generally not recommended for use as intentional sources of energy dissipation in accordance with BS EN 1998-2. The "Not recommended" category contains four examples of earthquake resisting elements that have performed poorly in past earthquakes and are therefore strongly discouraged. In particular, plastic hinging in the superstructure is not recommended due to the interaction of gravity loading (Marsh *et al.*, 2014). As opposed to the relatively lightly loaded columns and piers in compression, the horizontal framing system of the superstructure of a bridge is generally optimized around the gravity loading so as to increase the span lengths. It is advisable to assure that the superstructure will not form plastic hinges and jeopardize the performance of the system with potentially poor inelastic behaviour. Note that this is opposite to the intended plastic mechanisms for buildings, where plastic hinging is restricted to beams as columns are usually more heavily loaded. BS EN 1998-2 also restricts the formation of plastic hinges in the superstructure except for secondary deck members such as continuity slab.

Table 4.1 Earthquake resisting elements (Imbsen, 2014)

Categories	Description of earthquake resisting element
Permissible	• Plastic hinges below cap beams including pile bents. • Above ground / near ground plastic hinges. • Seismic isolation bearings or bearings designed to accommodate expected seismic displacements with no damage. • Piles with 'pinned-head' conditions. • Columns with moment reducing or pinned hinge details. • Capacity-protected pile caps, including caps with batter piles, which behave elastically. • Plastic hinges at base of wall piers in weak direction. • Pier walls with or without piles. • Spread footings that satisfy the overturning criteria.* • Passive abutment resistance using a limited passive resistance. • Seat abutments whose back-wall is designed to fuse. • Columns with architectural flares – with or without an isolation gap. • Seat abutments whose back-wall is designed to resist the expected impact force in an essentially elastic manner.
Permissible with owner's approval	• Passive abutment resistance using full passive resistance. • Sliding of spread footing abutment allowed to limit force transferred.* • Foundations permitted to rock.* • More than the outer line of piles in group systems allowed to plunge or uplift under seismic loadings. • Wall piers on pile foundations that are not strong enough to force plastic hinging into the wall and are not designed for the design earthquake elastic forces. • Plumb piles that are not capacity-protected (e.g. integral abutment piles or pile-supported seat abutments that are not fused transversely).* • In-ground hinging in shafts or piles. • Batter pile systems in which the geotechnical capacities and/or in-ground hinging define the plastic mechanisms.
Not recommended	• Plastic hinges in superstructure. • Cap beam plastic hinging (particularly hinging that leads to vertical girder movement) also includes eccentric braced frames with girders supported by cap beams. • Bearing systems that do not provide for the expected displacements and/or forces (e.g. rocker bearings). • Batter-pile systems that are not designed to fuse geotechnically or structurally by elements with adequate ductility capacity.

Note: * Not recommended in BS EN 1998-2.

At the bridge level, the earthquake resisting system with permissible resisting elements forming individual contributors must be rationally designed. There are a couple of general rules. Firstly, conventional plastic hinging is permitted for only intermediate substructure locations. Secondly, isolation bearings are also permitted, but there should be no mixed use of plastic hinging and isolation bearings at the same pier, i.e. isolation bearings are permitted with plastic hinges at other pier locations. Thirdly, abutments required as part of the earthquake resisting system are also permissible, although limited passive resistance shall be mobilized. Finally, the number of supporting members (i.e. piers and abutments) that will be used to resist the seismic forces in the longitudinal direction may have to be less than the total number of supporting members, as it is desirable to provide sliding or flexible mountings between the deck and some piers in the longitudinal direction to reduce the stresses arising from imposed deck deformations due to thermal actions, shrinkage and other non-seismic actions. The permissible earthquake resisting systems based on these general rules are listed in Table 4.2.

Table 4.2 Permissible earthquake resisting systems (Imbsen, 2014)

Component	Description of earthquake resisting system	Application in the axis of bridge
Intermediate structural	• Plastic hinges at inspectable locations	Longitudinal and transverse
	• Isolation bearings accommodate full displacement	Longitudinal

components	•Multiple simply supported spans with adequate seat lengths •Plastic hinges at inspectable locations	Longitudinal
	• Plastic hinges at inspectable locations • Isolation bearings elsewhere with or without energy dissipaters to limit overall displacements	Longitudinal and transverse
Abutment	• Not used as part of earthquake resisting system • Knock-off back-walls permissible	Longitudinal
	• Not used as part of earthquake resisting system • Breakaway wing-walls permissible	Transverse
	• Required to resist design earthquake elastically • Limited passive resistance can be mobilized	Longitudinal and transverse

4.1.2 Articulation

The manner in which the superstructure is articulated in particular will affect where and how the forces are resisted within the structure.

Consider a bridge with the abutments flexibly connected to the deck that permit longitudinal movement of the superstructure relative to the abutments. The articulation provided by sliding or elastomeric bearings means that little longitudinal load will be resisted by the abutments provided that the longitudinal movement does not close the gap between the superstructure and the abutment back-wall. Such unrestrained movement at the abutment places the bulk of the inertial forces on the intermediate substructures. This will likely require that the piers be designed to be stiffer and stronger in the longitudinal direction than if the abutments participate in the seismic resistance in the cases of having rigid connections to the deck through either monolithic joints or fixed bearings.

The influence of articulation of a bridge on the lateral load path is even more prominent in the transverse direction and must be considered at the conceptual design stage. For bridges laterally loaded in the transverse direction, the superstructure may form a diaphragm that acts in parallel with the lateral resistance of the substructure. Take a common concrete box girder bridge for example. The lateral stiffness of such a superstructure is normally much larger than that of the substructure. If the superstructure is continuous over the piers, it would undertake the bulk of the transverse inertial forces such that the piers are unlikely to yield regardless of how weak the pier is made. As a result, it would be difficult to achieve the ductile response of system without having the superstructure yielding, but permitting yielding in the superstructure is not preferred due to the need for preserving the integrity of the gravity support system. Alternatively, fusible shear keys might be used at the locations where the superstructure and the piers are connected to attain the ductile response after the first yield of the keys. Moreover, depending on the in-plan aspect ratio and stiffness, the superstructure may suffer diaphragm bending effect as transverse inertial forces acting on the superstructure and the maximum superstructure moment about the vertical axis will occur near the weakest pier. If the superstructure is changed to prestressed concrete girders with a cast-in-place deck and full integral diaphragm over the piers, careful attention must be paid to the detail used to tie the adjacent spans together. This is important since such integral construction does not necessarily meet the minimum support lengths. Besides, the designer must ensure that the superstructure can sustain the forces that will be induced. Even if the elastic-based design forces are used, additional factors of safety may be used to increase the elastic demands for added conservatism (Marsh *et al.*, 2014). On the other hand, if the superstructure consists of multiple simple spans rather than a continuous one, the pier would take its proportional share of the transverse inertial forces without the associated large moments being induced in the deck. In such a case, however, the potential for unseating of spans and impact of two adjacent spans as they deflect laterally would need to be considered.

4.1.3 Desirable characteristics

Bridges provided with clear and dependable earthquake resisting system, reliable articulation, and balanced stiffness, mass and strength have been proved to perform much better during earthquake than those without these features, and their responses are more accurately predicted with elastic analysis. In recognition of this, it is desirable to incorporate simplicity, integrity and symmetry into the design to the extent possible (Marsh *et al.*, 2014). General guidance that should be followed are summarized below:

- It is desirable to use balanced frame geometry and balanced stiffness so that different piers or frames have similar seismic response during the earthquake. The optimum post-elastic seismic behaviour is achieved if the intended plastic hinges develop approximately simultaneously. An unbalanced stiffness between two columns in a pier will cause more damage to the stiffer one as a result of column torsion generated by rigid body rotation of the superstructure. The unbalanced frame geometry will likely induce out-of-phase response between adjacent frames (i.e. structural units on either side of an articulation joint, such as an expansion joint), leading to large relative displacements between the frames that increase the likelihood of longitudinal unseating and pounding between frames at the expansion joints. The pounding will not only damage the expansion joint but also place additional seismic demand on the frame, which can be detrimental considering the stand-alone capacity of the frame. As a reference, the California Department of Transportation (Caltrans) Seismic Design Criteria (Caltrans, 2013) includes the balanced stiffness provision such that the variation in stiffness may differ by not more than 50% between any two piers within a frame or between any two columns in a pier, and by not more than 25% between adjacent piers within a frame or between adjacent columns within a pier. Caltrans (2013) also includes the balanced frame geometry provision that adjacent frames shall have fundamental vibration periods that are within 30% of one another. It can be challenging to achieve balanced stiffness and balanced frame geometry when restricted by the terrain. Some useful concepts have been developed in an attempt to balance the stiffness and dynamic characteristics of a bridge, including the following (Marsh *et al.*, 2014): (a) adjusting the effective column height by lowering footings; (b) using oversized drilled shafts; (c) modifying end fixities / restraints; (d) varying the column cross section and longitudinal reinforcement; (e) adding or relocating columns or bents; (f) reducing or redistributing the superstructure mass, and (g) incorporating seismic isolators or dampers.
- It is desirable to minimize the skew and curvature of bridge deck to the extent possible, although the skew and curvature are typically a function of roadway alignment. Both the skew and curvature are features that tend to couple lateral response in the transverse and longitudinal directions, making the dynamic response of bridge more complex. Skew and curvature can make the design more difficult, which calls for more advanced analysis. Additionally, skew and curvature complicate the dynamic analysis because of the torsional motions of the bridge about a vertical axis so induced. It is suggested that the skew should be minimized overall to less than 30 degrees if possible (Yashinsky and Karshenas, 2003).
- It is desirable to reduce the number of expansion joints to an acceptable minimum and avoid half-joints and in-span hinges. In general, bridges with continuous deck behave better under seismic conditions than those with many movement joints. However, the expansion joints may not be completely eliminated owing to the built-up of thermal stresses or for construction ease. The strategy to achieve a "continuous" deck in order of preference is then as follows: (a) using continuous or integral construction; (b) providing generous support lengths, and (c) adding restrainers or shock transmission units to

interconnect adjacent frames. Restrainers are the least preferred solution for preventing unseating for new construction because the forces in restrainers under the design earthquake are usually quite difficult to predict and may thus be difficult to design as capacity-protected elements from past experience (Marsh *et al.*, 2014).

4.2 Intended Seismic Behaviour

The seismic design strategies of SDMHR 3rd (Highways Department, 2006) and SDMHR 2013 (Highways Department, 2013) are essentially different. SDMHR 3rd has implicitly assumed the bridge to resist various kinds of loads including seismic action elastically, whereas BS EN 1998-2 that forms the fundamental basis of SDMHR 2013 requires that the bridge shall be designed for either ductile or limited ductile behaviour under the design seismic action. In particular, it is generally preferable to design a bridge for ductile behaviour in cases of moderate-to-high seismicity, whereas a limited ductile behaviour may be justified in cases of low-to-moderate seismicity. The type of intended seismic behaviour should also be decided at the conceptual design stage. Different design approaches may be adopted for different seismic behaviour.

To enable the appropriate seismic design of bridges, it is necessary to identify the effects on the seismic design and seismic performance of bridges, which may be brought about by different intended seismic behaviour first. This will be elaborated through a series of parametric studies on the seismic design of bridge piers and piled foundations.

4.2.1 Structural design of pier under different design

The structural design of reinforced concrete bridge piers intended for elastic, limited ductile and ductile seismic behaviour are presented and compared below.

(1) Description of the piers

A series of rectangular bridge piers under designated axial forces represented by the normalized axial force (η_k) are designed for earthquake resistance. Note that confining reinforcement shall be provided in the potential hinge regions in accordance with Clause 6.2.1.1(2) of BS EN 1998-2 once η_k exceeds 0.08. Thus, two categories of piers with η_k of 0.066 and 0.093 are studied in order to determine the effect of choice between ductile design and limited ductile design on the required amount of transverse reinforcement. Both normalized axial forces are deemed to be reasonable for bridge structures. The piers are assumed to be fixed at the base and are connected to the deck at the top either through fixed bearings that have little hysteresis energy dissipation or monolithically. As the structural connection through fixed bearings is usually provided over one pier only in a multi-span bridge frame, while there can be two consecutive monolithically connected piers, the tributary mass (m) of the deck and quasi-permanent traffic load at the top of the pier is halved when the articulation is changed to integral. The parameters of the piers are listed in Table 4.3, together with the depth (D) and width (W) of the cross section, and the clear height (H). The clear height of pier varies from 5 m to 17 m.

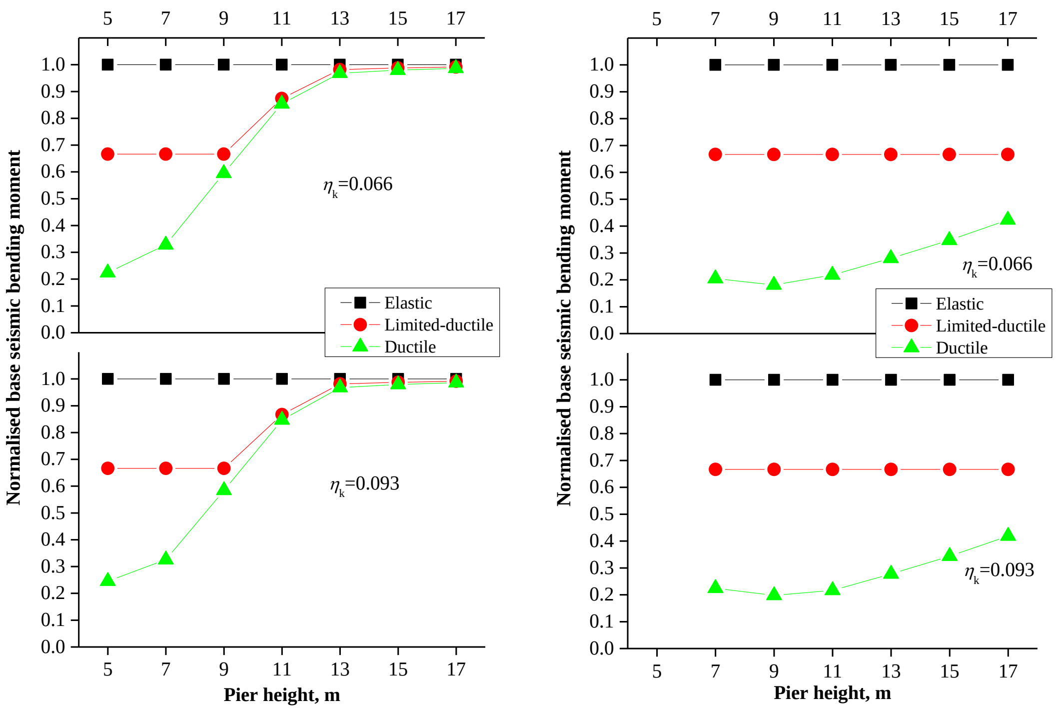
Table 4.3 Summary of parameters of piers

#	η_k	D (m)	W (m)	H (m)	m (kg)	Connection	#	η_k	D (m)	W (m)	H (m)	m (kg)	Connection
1	0.066	1.50	2.50	5	3760	Bearing	14	0.093	1.80	3.00	5	7653	Bearing
2	0.066	1.50	2.50	7	3760	Bearing	15	0.093	1.80	3.00	7	7653	Bearing
3	0.066	1.50	2.50	9	3760	Bearing	16	0.093	1.80	3.00	9	7653	Bearing
4	0.066	1.50	2.50	11	3760	Bearing	17	0.093	1.80	3.00	11	7653	Bearing
5	0.066	1.50	2.50	13	3760	Bearing	18	0.093	1.80	3.00	13	7653	Bearing
6	0.066	1.50	2.50	15	3760	Bearing	19	0.093	1.80	3.00	15	7653	Bearing
7	0.066	1.50	2.50	17	3760	Bearing	20	0.093	1.80	3.00	17	7653	Bearing
8	0.066	1.50	2.50	7	1880	Monolithic	21	0.093	1.80	3.00	7	3826	Monolithic
9	0.066	1.50	2.50	9	1880	Monolithic	22	0.093	1.80	3.00	9	3826	Monolithic
10	0.066	1.50	2.50	11	1880	Monolithic	23	0.093	1.80	3.00	11	3826	Monolithic
11	0.066	1.50	2.50	13	1880	Monolithic	24	0.093	1.80	3.00	13	3826	Monolithic
12	0.066	1.50	2.50	15	1880	Monolithic	25	0.093	1.80	3.00	15	3826	Monolithic
13	0.066	1.50	2.50	17	1880	Monolithic	26	0.093	1.80	3.00	17	3826	Monolithic

(2) Structural design against seismic action

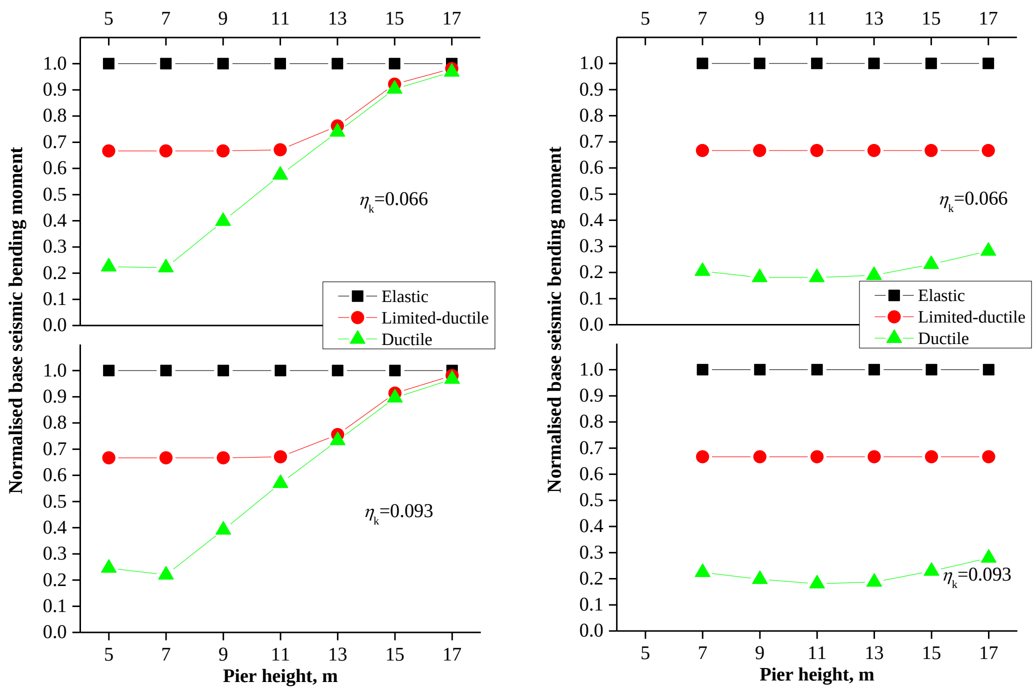
The horizontal design seismic action to which the pier is subjected along the direction of its cross-sectional depth at the centre of mass is determined first. The fundamental mode method is adopted for this preliminary analysis. The elastic seismic action is then derived from the inertial force corresponding to the natural period of the fundamental mode of structure by multiplying the tributary mass by the relevant ordinate of Type 2 elastic response spectrum in BS EN 1998-1 with a reference peak ground acceleration of 0.12g as specified in SDMHR 2013, where g is the acceleration due to gravity. For reinforced concrete piers, the stiffness to be used in linear analysis can be based on the flexural stiffness of uncracked gross section in the elastic and limited ductile design. In ductile design, the effective stiffness of pier to be used for the equivalent linear analysis shall be taken to be the secant stiffness at the theoretical yield point as specified in Clause 2.3.6.1(2) of BS EN 1998-2. As it is almost impossible to define accurately the secant stiffness at the beginning of design process, an estimate of 30% of the flexural stiffness of uncracked gross section is often used to start the design process and an iterative procedure is used to update it. The elastic seismic actions will be modified in limited ductile and ductile design to include the effect of ductility, by dividing them by the relevant behaviour factors determined in accordance with Clause 4.1.6 of BS EN 1998-2.

The inertial force of superstructure is transmitted to the structural connection, which in turn transmits the loading to the pier. The design moments at the base of the pier, corresponding to elastic, limited ductile and ductile design, are then determined. The values are compared in Figure 3.1 with the values normalized by those for the elastic design.



Bearing connection Monolithic connection

(a) Importance Class I and Ground Type A



Bearing connection Monolithic connection

(b) Importance Class II and Ground Type C

Figure 4.1 Design moments intended for different seismic behaviour

Figure 3.1 shows that the design moment can generally be reduced significantly if the pier is designed for ductile behaviour or limited ductile behaviour as compared with that for elastic behaviour, especially for short piers.

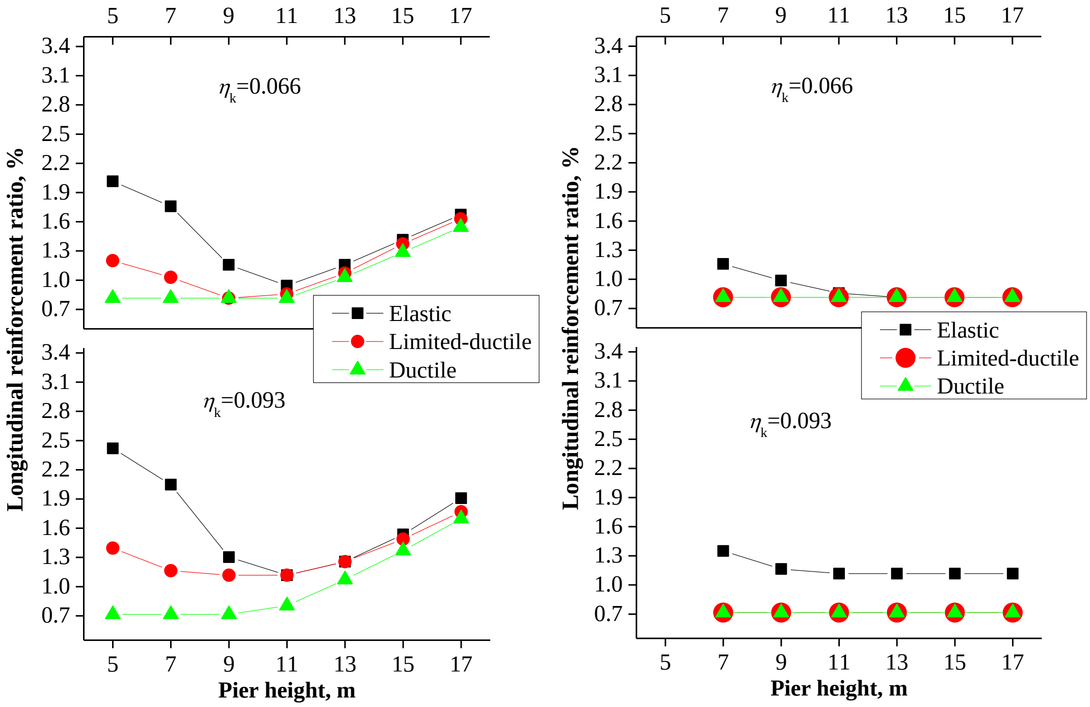
Once the seismic action effects in the pier have been calculated, the structural design can be conducted. In accordance with Clause 5.6 of BS EN 1998-2, if the bridge is intended for ductile behaviour, the regions of plastic hinges, e.g. the bottom junction of a pier with the foundation, are verified to have adequate flexural strength to resist the design moments, whereas the shear resistances of the plastic hinges as well as sections outside the plastic hinges should be designed to resist the "capacity design effects" determined in accordance with Clause 5.3 of BS EN 1998-2. If the bridge is intended for limited ductile behaviour or elastic behaviour, all sections are verified against the design moments and shear forces derived directly from the demand analysis. Moreover, the detailing should conform to the basic requirement set forth in Clauses 6.2 and 6.5 of BS EN 1998-2 for ductile and limited ductile concrete piers, respectively, or comply with those outlined in Clause 9.5 of BS EN 1992-1 for elastically designed concrete columns. In this preliminary study, the assumed pier sizes are maintained with the required flexural and shear resistances of pier achieved by provision of adequate reinforcement.

(3) Results and discussions

The longitudinal reinforcement ratios under different design strategies are compared in Figure 4.2. It is anticipated that a minimum amount of longitudinal reinforcement would be needed if the bridge is designed for ductile behaviour. However, despite the much smaller design moments, the design for ductile behaviour may require as much longitudinal reinforcement as that intended for limited ductile behaviour as shown in Figure 4.2(a) for tall integral piers. The same amount of longitudinal reinforcement may also be adequate for the elastic design, as the reinforcement design is also controlled by detailing rules in such cases. Nevertheless, pier re-sizing to smaller sizes are not considered in this preliminary study for the size may be controlled by load combinations other than those for the seismic design situation. Owing to over-reinforcing, the intended ductile behaviour may not be able to develop, but may end up with limited ductile or even elastic behaviour under the design seismic action, as observed from the design moment (M_{Ed}) in relation to the design flexural resistance of the base section shown in Figure 4.3(a). When bridges are on a strategic route and the site conditions are relatively poor, however, Figure 4.2(b) shows that the amount of longitudinal reinforcement may vary significantly with the design strategy adopted. The pier will be provided with the minimum amount of longitudinal reinforcement if it is designed for ductile behaviour. The intended distinct modes of behaviour are generally achieved through the design, as observed from the design moment in relation to the design flexural resistance of the base section shown in Figure 4.3(b). It is worth mentioning that the piers of heights of 5 m and 7 m with fixed bearing connection to the deck are incapable of developing sufficient flexural strength for achieving elastic behaviour even with the maximum allowable reinforcement, which will require larger sections and will in turn incur greater seismic actions. Should the larger piers still fail to meet the requirements for elastic behaviour, it will be necessary to design the bridge for ductile behaviour or limited ductile behaviour, or alternatively adopt seismic isolation.

Figure 4.4 shows the results of transverse reinforcement ratio. If a bridge is considered to belong to the category of average importance and the site condition is not poor so that it does not incur significant seismic actions, it may not be essential to design the bridge pier for ductile behaviour. Figure 4.4(a) shows that much more transverse reinforcement is required for relatively heavier loaded piers intended for ductile behaviour due to the provision of confinement, although the intended ductile behaviour may not develop under the design seismic actions as discussed above. These bridges may better be designed for limited ductile behaviour instead. If the bridge is of critical importance and/or if the site condition is poor, the choice of ductile behaviour might be more economical and safer especially for short piers, since ductile piers can dissipate the input energy more efficiently through the rotation associated with plastic hinges, thereby likely requiring much less reinforcement as shown in Figure 4.2(b) and

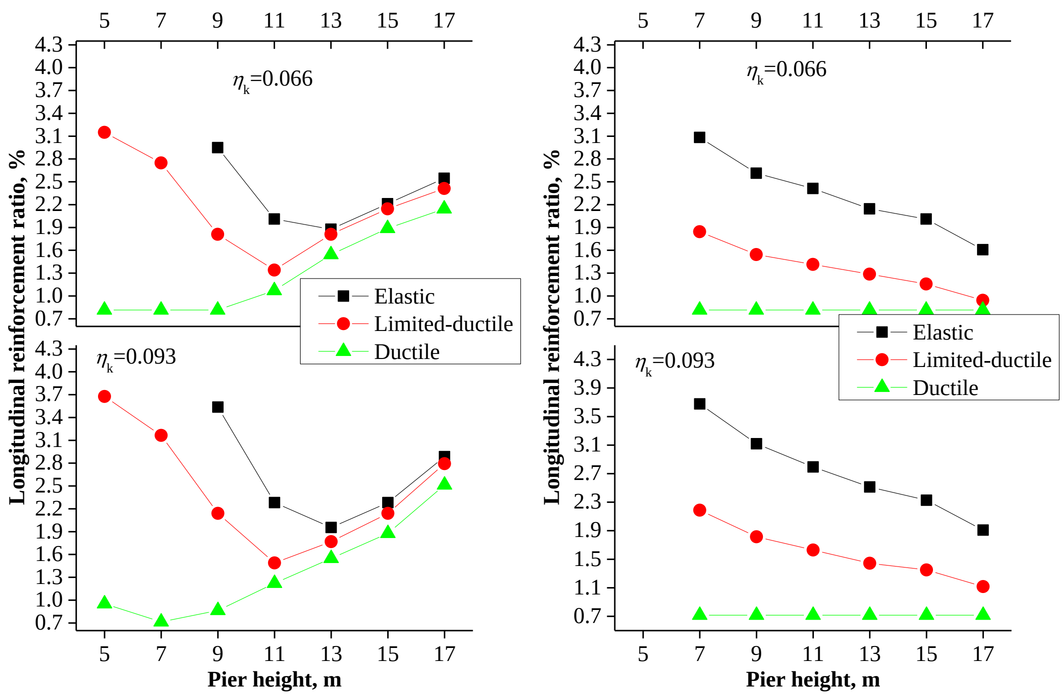
Figure 4.4(b).



Bearing connection

Monolithic connection

(a) Importance Class I and Ground Type A

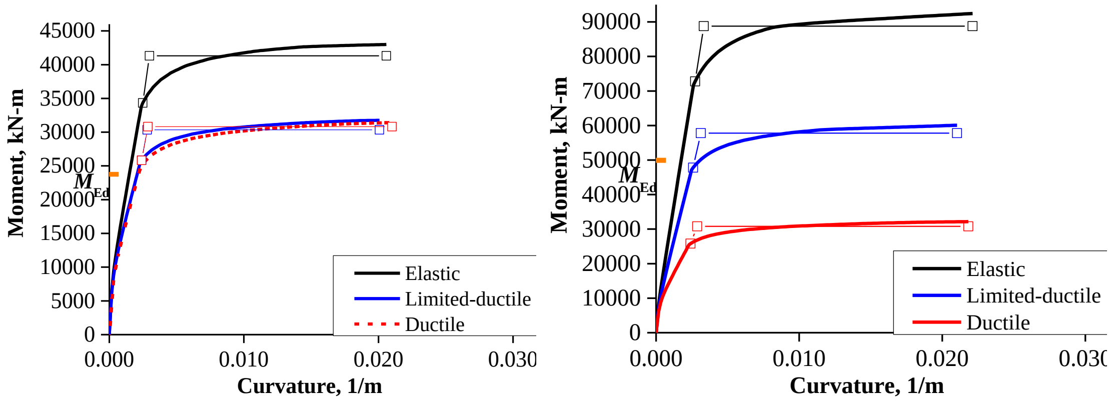


Bearing connection

Monolithic connection

(b) Importance Class II and Ground Type C

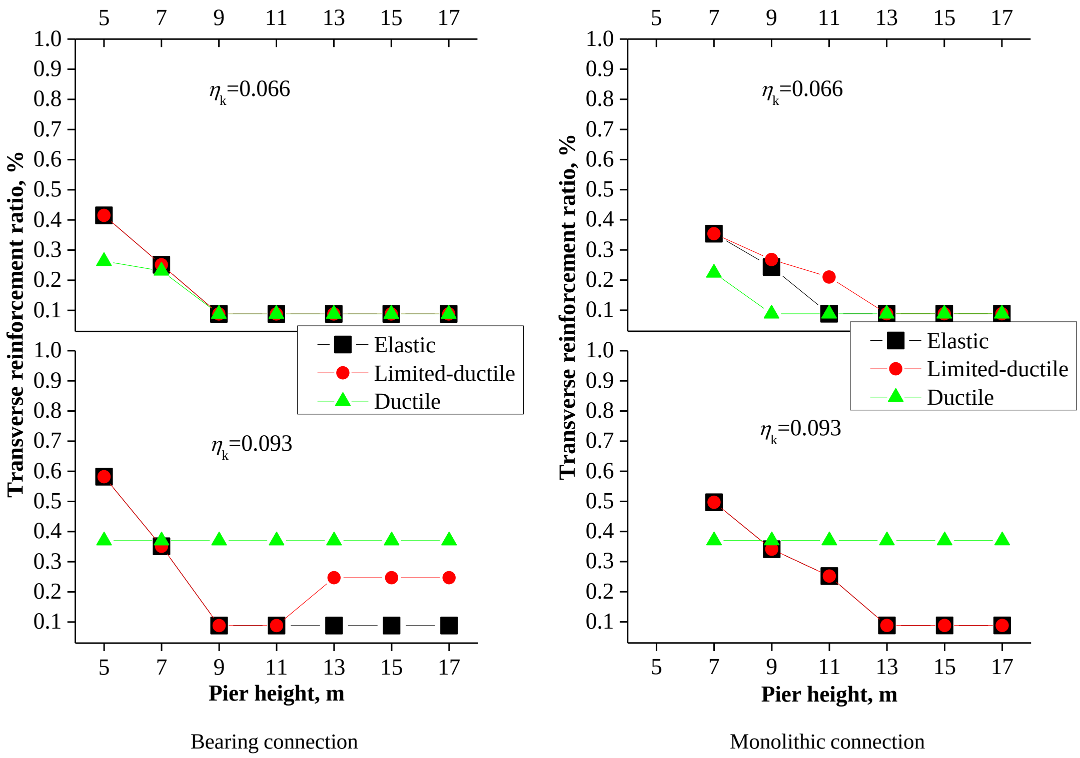
Figure 4.2 Longitudinal reinforcement ratio intended for different behaviour



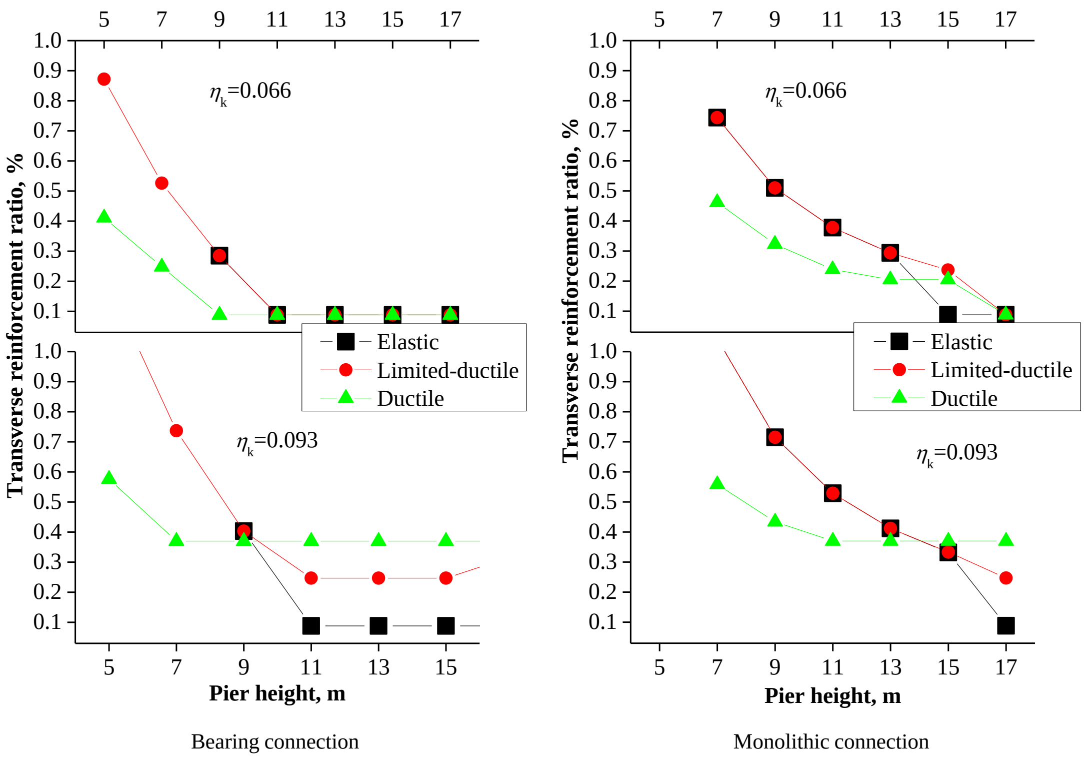
(a) Importance Class I and Ground Type A (b) Importance Class II and Ground Type C

Figure 4.3 Moment-curvature relationship of base section for the pier with normalized axial force of 0.093, height of 9 m and monolithic connection to the deck

In summary, on the choice of suitable intended behaviour in the design of a bridge in Hong Kong, it is preferable to design the bridge for ductile behaviour if the bridge is considered to be of critical importance and/or the site condition is relatively poor. If the design conditions are more favourable, it may be better to opt for limited ductile behaviour for economic reasons. Particular attention should be paid to bridges having either piers shorter than 9 m or tall piers exceeding 13 m in height in the earthquake resisting system. The former group will generally be subjected to strong seismic motions due to their short periods so that the selection of ductile behaviour is usually expedient, regardless of the importance class of the bridge and the site conditions. For the latter group, the seismic resisting systems are quite flexible and it is usually justifiable to design them for limited ductile behaviour irrespective of the importance class and ground type. In most cases, designing for elastic behaviour is wasteful and should not be accepted.



(a) Importance Class I and Ground Type A



(b) Importance Class II and Ground Type C

Figure 4.4 Transverse reinforcement ratio intended for different behaviour

The AASHTO Guide Specifications (AASHTO, 2011) have developed the concept of seismic design category (SDC) to determine different requirements for the methods of analysis, column design details, etc. Four SDCs are established based on the seismic risks, using the horizontal response spectral acceleration coefficient at period of 1.0 sec (S_{D1}) (where the abbreviation “sec” denotes "second" in this chapter in order to be consistent with terms used in the AASHTO Guide Specifications), and the seismic design requirements are adjusted for each of the SDCs. When the 1-sec period design spectral acceleration coefficient is smaller than 0.15, the bridge shall be assigned to SDC A. The seismic analysis for bridges in SDC A is generally not required for such low seismic hazard level. Instead, default values are used as minimum design forces in lieu of rigorous analysis. A clearly identifiable earthquake resisting system (ERS) is not necessary either and minimum detailing requirements for support length and column transverse steel are deemed adequate. For bridges in SDC B, where the 1-sec period design spectral acceleration coefficient is between 0.15 and 0.30, the identification of an ERS selected to achieve adequate seismic performance should be considered. The designer should ensure that an ERS is present and capable of providing a reliable and uninterrupted load path for transmitting seismically induced forces to the surrounding soil and sufficient means of energy dissipation and/or restraint are available to reliably control seismically induced displacement. In addition, to avoid unintentional weak links in the ERS, capacity design should be considered for column shear. The seismic analysis should be conducted for bridges in SDC B and a displacement demand to capacity check is required for individual piers or bents. Furthermore, SDC B level of detailing should be provided. More stringent detailing rules may apply to bridges in SDC C and above.

The SDC concept may also be adopted for local use. Actually, the design requirements for bridges in SDC B and above according to the AASHTO Guide Specifications, including identification of ERS, capacity design and sufficient detailing, are essentially the same for the bridges designed for ductile behaviour based on BS EN 1998-2. These requirements may be lessened or avoided for bridges in SDC A of the AASHTO Guide Specifications, which is also considered in BS EN 1998-2 for the design for limited ductile behaviour. In view of the similarities, the SDC strategy may be used to assist in the selection of intended seismic behaviour at the conceptual design stage. The horizontal 1.0-sec period design spectral acceleration coefficients for different seismic design situations in Hong Kong are listed in Table 4.4. Three SDCs are identified by reference to the AASHTO Guide Specifications, with SDCs A and B forming the main SDCs applicable to Hong Kong owing to the low-to-moderate seismicity. It is found that SDC B has included those bridges of critical importance and/or on relatively poor sites recommended by the present study to be designed for ductile behaviour, and SDC A is limited to those bridges with more favourable conditions for which essentially elastic behaviour is recommended. The use of the SDC strategy can provide a rational basis according to findings of the present study.

Table 4.4 1-sec period design spectral acceleration coefficient for Hong Kong

Importance Class	Ground Type				
	A	B	C	E	D
I	0.075	0.101	0.113	0.120	0.162
II	0.105	0.142	0.158	0.168	0.227
III	0.173	0.233	0.259	0.276	0.373

Note: In accordance with the AASHTO Guide Specifications, the seismic design situations marked by the "blue", "green" and "red" zones, where the 1-sec period design spectral acceleration coefficients satisfying $S_{D1} < 0.15$, $0.15 \leq S_{D1} < 0.30$ and $0.30 \leq S_{D1} < 0.50$, are assigned to SDCs A, B and C, respectively.

4.2.2 Full-range behaviour of pier under different design

The seismic performance of the pier is more of a concern than the cost or others. Since shear capacity is known to decrease as plastic deformation develops (Kowalsky and Priestley, 2000), unintentional shear failure can occur in piers under earthquakes that are more severe than the design events. It is thus of interest to evaluate the full-range behaviour of piers designed based on intended ductile and limited ductile behaviour.

(1) Description of the piers

Five rectangular reinforced concrete bridge piers under the same normalized axial force (η_k) of 0.093 are designed to resist the seismic inertial forces with seismic coefficients corresponding to conditions of Importance Class II and Ground Type C. The tributary masses of the deck and quasi-permanent traffic load are listed for the piers in Table 4.5, together with the depth of the cross section in the direction of the force (h), clear height (H), and aspect ratio (α_s) that is related to the cross-sectional depth as well as end rigidity. Both ductile design and limited ductile design are performed for each of the piers. For the ductile design case, the value of the behaviour factor is taken in accordance with Table 4.1 of BS EN 1998-2. Besides, the natural vibration period of the pier is calculated based on the tributary mass and cracked stiffness of pier that is initially assumed to be 30% of the flexural stiffness of uncracked gross section. Thus the structural design is conducted in an iterative manner. For the limited ductile design, the value of the behaviour factor is taken to be 1.5, and the natural period of the pier is calculated based on the flexural stiffness of uncracked gross section as suggested by Clause 2.3.6.1(3) of BS EN 1998-2. The longitudinal and transverse reinforcement ratios at the base section (i.e. ρ_L and ρ_w, respectively) determined under different design schemes are also summarized in Table 4.5. It is noted that the transverse steel has met the minimum detailing requirement specified in BS EN 1998-2 for ductile and limited ductile reinforced concrete piers.

Table 4.5 Pier design intended for ductile and limited ductile behaviour

Column	Properties of column					Ductile design			Limited ductile design		
	h (m)	H (m)	α_s	η_k	m (kg)	T (s)	ρ_L (%)	ρ_w (%)	T (s)	ρ_L (%)	ρ_w (%)
1	3.00	5	1.7	0.093	7653	0.26	0.95	0.58	0.16	3.68	0.61
2	1.80	9	2.5	0.093	3826	0.84	0.72	0.44	0.53	1.82	0.72
3	3.00	9	3.0	0.093	3826	0.64	0.72	0.37	0.40	1.82	0.38
4	1.80	15	4.2	0.093	3826	1.81	0.72	0.37	1.15	1.35	0.33
5	3.00	15	5.0	0.093	3826	1.38	0.72	0.37	0.87	1.35	0.25

(2) Shear-displacement capacity analysis

Pushover analysis is performed for each of the piers. The resulting shear-displacement curve may be represented by bilinear approximation schematically as shown in Figure 4.5, where the retention of the shear capacity with development of plastic deformation is decided by reference to Priestley *et al.* (2007), which accounts for not only the contribution of the transverse reinforcement, longitudinal reinforcement and axial force to the shear resistance, but also the reduction effect of the plastic deformation. The column can fail in flexure, flexure-shear and shear under the design-level earthquake, depending on the shear capacity and demand relation as depicted in Figure 4.5. In respect of the full-range behaviour, the column displacement capacity (d_c) can be significantly reduced from the ultimate displacement (d_u) if shear-type failures dominate. As a measure of this feature, a normalized displacement capacity factor ($\tilde{d}_p$), defined as the ratio of the plastic part of displacement capacity (d_c-d_y) to the plastic part of ultimate displacement (d_u-d_y) is proposed, where d_y denotes the yield displacement. Note that the value of this factor can be negative, between 0.0 and 1.0, or equal to 1.0, indicating full-range behaviour of shear, flexure-shear and flexure, respectively.

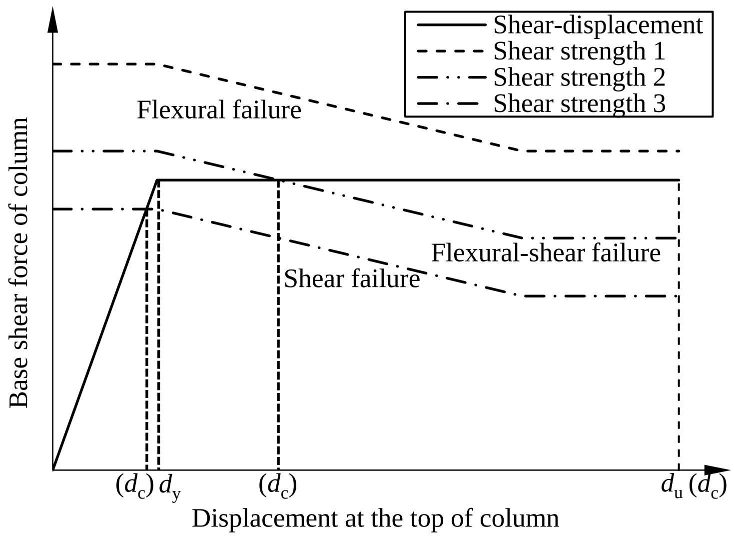


Figure 4.5 Shear capacity and demand relation of pier

(3) Results and discussions

The calculated values of $\tilde{d}_p$ for these piers under different design schemes are plotted in Figure 4.6 against the aspect ratios. It shows that piers with aspect ratios not exceeding 3.0 are susceptible to flexural-shear failure and the pier with an aspect ratio as small as 1.7 is prone to shear failure based on the design for limited ductile behaviour, while the ductile design enables piers to fail in ductile mode regardless of the aspect ratio. The occurrence of either flexural-shear failure or shear failure for limited ductile design is attributed to that BS EN 1998-2 does not require capacity design for bridges based on the design for limited ductile behaviour. As opposed to limited ductile design, ductile design has been equipped with capacity design strategy to avoid brittle shear failures of any kind.

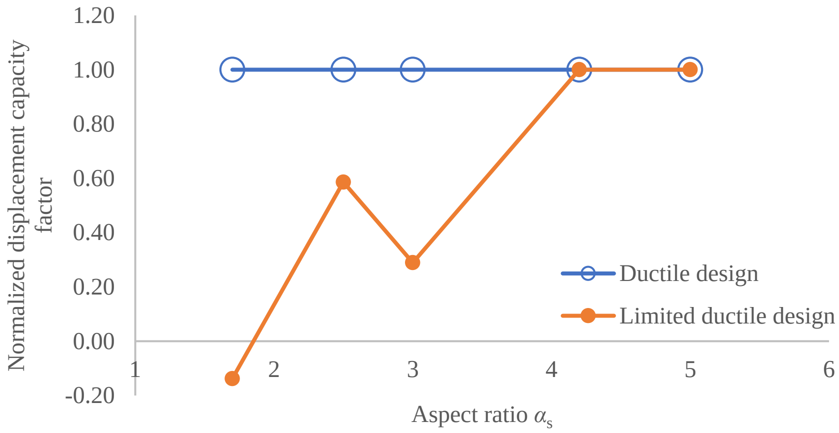


Figure 4.6 Full-range behaviour of ductile and limited ductile piers in terms of normalized displacement capacity factor

4.2.3 Piled foundation under different design

The intended behaviour of bridge will also affect the magnitude of seismic actions transmitted to the foundation. This section further illustrates the effect of intended seismic behaviour of bridge on the design of foundation.

(1) Description of the problem

A piled foundation is designed to support a reinforced concrete bridge pier with rectangular cross section of 3.0 m × 1.8 m and height of 9 m. The base of the pier is fixed to the pile cap while the top of pier is assumed to be free to move and rotate. The tributary deck mass and the mass associated with traffic load are lumped at the top of pier for simplicity. The total weight for seismic design situation consists of a permanent weight of 14000 kN and a quasi-permanent weight of 1000 kN, leading to a normalized axial force in the pier defined in Clause 5.3(4) of BS EN 1998-2 of approximately 0.1. The seismic response of the superstructure will generate lateral forces at the centre of mass, and these forces will be transmitted through the pier to the piled foundation.

The piled foundation consists of a group of four bored piles arranged on a square grid at a centre-to-centre spacing of 3 times the pile diameter (d) as shown in Figure 4.7. The subsoil is assumed to be uniform. The piles are founded on bedrock at 30.5 m below ground level. The allowable bearing pressure of bedrock (q_b) is taken to be 5000 kN/m^2, which corresponds to moderately strong rock with a total core recovery of more than 85% of the grade as given in Table 6.6 of GEO 1/2006 (GEO, 2006). The minimum penetration of the piles into rock is 0.3 m to conform to the requirement set forth in Note (3) to Table 6.6. The 2 m thick pile cap has a soil cover of 1.5 m.

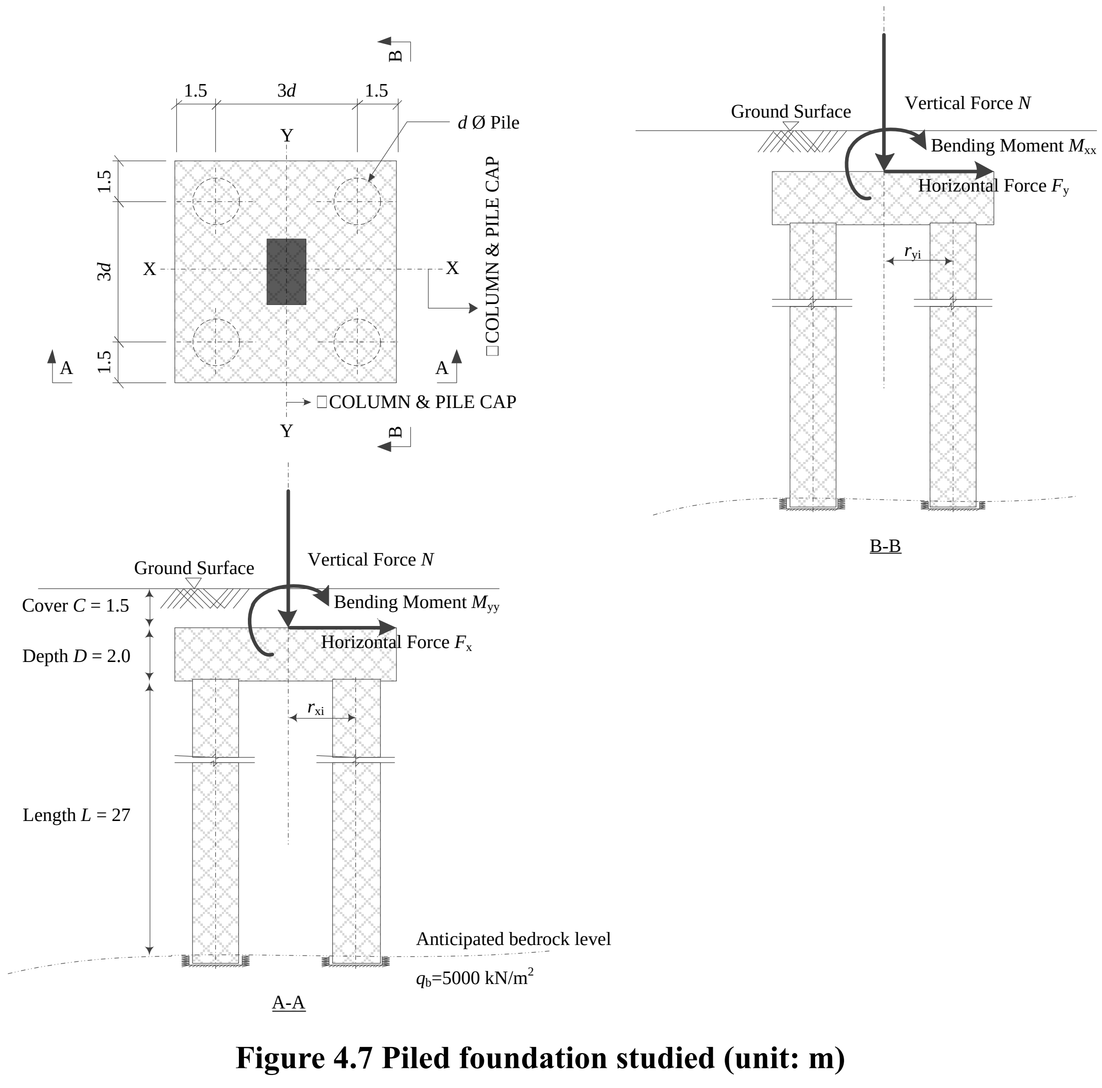


Figure 4.7 Piled foundation studied (unit: m)

The following seismic design situations with various combinations of bridge importance class and ground type are investigated: (a) Importance Class I and Ground Type B; (b) Importance Class I and Ground Type C; (c) Importance Class II and Ground Type C; and (d) Importance Class II and Ground Type D.

In accordance with the definition of Ground Type B as given in Table 3.1 of BS EN 1998-1, stiff clay with undrained shear strength c_u of 350 kN/m^2 is considered. The adhesion factor α for the soil / pile interface is estimated to be 0.4. For Ground Type C, a medium dense residual soil with weight intensity γ_s of 17 kN/m^3 and undrained shear strength c_u of 200 kN/m^2 is considered. The friction angle of shearing resistance φ' is taken to be 35° and the shaft resistance coefficient β is estimated as 0.2. The poorer site of Ground Type D consists of soft clay with undrained shear strength c_u of 50 kN/m^2. The adhesion factor α for the soil / pile interface has a value of 0.8.

(2) Design of foundation

Bridge foundations shall not be intentionally used as sources of hysteretic energy dissipation and therefore shall, as far as practicable, be designed to remain elastic under the design seismic actions. In accordance with Clause 5.8.2 of BS EN 1998-2, the design shear loads and moments on the foundation shall be either those at the base of pier intended for limited ductile behaviour multiplied by the behaviour factor or the "capacity design effects" of pier intended for ductile behaviour based on the actual reinforcement and accounting for overstrength, but not exceeding those resulting from elastic analysis. It is expected that the intended limited ductile behaviour of bridge will result in the same design action effects on the foundation as the elastic behaviour will do. The axial compressive load on the foundation shall include the weight of the pier.

The relation between the design shear at the pier base and the design action effects on the foundation for intended elastic, limited ductile or ductile behaviour of bridge corresponding to Important Class II and Ground Type C, for instance, are shown in Table 4..

Table 4.6 Design action effects for pier and foundation (Importance Class II and Ground Type C)

Intended seismic behaviour of bridge	Pier		Foundation			
	Longitudinal	Transverse	Longitudinal		Transverse	
	F_x (kN)	F_y (kN)	M_{yy} (kNm)	F_x (kN)	M_{xx} (kNm)	F_y (kN)
Elastic	6249	5846	56241	6249	52614	5846
Limited ductile	4166	3897	56241	6249	52614	5846
Ductile	2520	1056	33615	3735	33264	3696

The piles must be verified to have sufficient axial and lateral resistances. To simplify the design, the following simplifying assumptions are made:

- The moments acting on the foundation are assumed to be kept in equilibrium solely by the axial loads of the piles.
- The weight of overburden can be balanced by that of the pile, allowing both to be excluded from the bearing resistance and bearing load respectively.
- The lateral loads on the foundation are assumed to be distributed to the piles equally.
- The deflection, bending moment and shear along a pile subjected to lateral loading can be determined using the generalised solutions of Matlock and Reese (1962) included in Geotechnical Engineering Office Publication No. 1/2006 (GEO, 2006). The constants of

horizontal subgrade reaction n_h for the soils considered for Ground Types B, C and D are taken to be 17.6 MN/m^3, 6.6 MN/m^3 and 2.2 MN/m^3, respectively.

- The lateral load capacity of a pile is either the maximum resistance that can be offered by the soil or the ultimate structural resistance (i.e. bending moment and shear) of the pile section, whichever is the lesser. The allowable lateral deflection of the pile is limited to 0.1 m.

The pile diameter is determined when any of the three criteria is met: (a) the axial compressive load on a pile reaches the bearing resistance of the foundation; (b) the tensile pile reaches the uplift capacity; and (c) the allowable lateral deflection of the pile is attained. Adequate longitudinal and transverse reinforcement should be provided to satisfy the maximum bending moment and shear force in the pile, and to meet the detailing rules set forth in BS EN 1992-1-1 (BSI, 2004a) and BS EN 1536 (BSI, 2000). In particular, the clear distance between longitudinal bars as measured along the periphery of the pile should not exceed 200 mm as specified in Clause 9.8.5(4) of BS EN 1992-1-1.

(3) Results and discussions

The diameters and structural design of the piles are summarized in Table 4.. Figure 4.8 shows the sizes and reinforcement amounts of the piles for ductile design of bridges, normalized by those for elastic and limited ductile design of bridges. The results show that the pile diameter can be reduced by 5% to 29% and the steel volume in pile can be reduced by 4% to 41% if the bridge is designed for ductile behaviour instead of elastic and limited ductile behaviour. The reduction effects are more significant in cases of poor soil conditions.

Table 4.7 Results of pile design for elastic, limited ductile and ductile design of bridge

Design situations		Intended seismic behaviour of bridge	Diameter (m)	Longitudinal reinforcement	Transverse reinforcement
Importance Class	Ground Type				
I	B	Elastic / Limited ductile	1.642	26 T32	T20 - 200
		Ductile	1.562	26 T32	T20 - 200
I	C	Elastic / Limited ductile	1.681	28 T32	T25 - 250
		Ductile	1.568	26 T32	T20 - 200
II	C	Elastic / Limited ductile	1.820	32 T32	T25 - 200
		Ductile	1.620	26 T32	T20 - 200
II	D	Elastic / Limited ductile	2.770	46 T32	T25 - 200
		Ductile	1.970	32 T32	T25 - 250

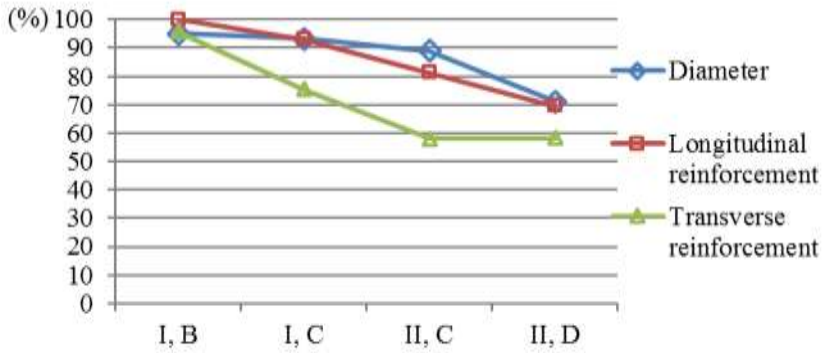


Figure 4.8 Relation of the size and reinforcement amount of pile between ductile design of bridge and limited ductile or elastic design of bridge

The use of the concept of SDC has been explored to determine the intended seismic behaviour of bridge as presented in Section 4.2.1(3). The seismic design situations for bridges in Hong Kong can be classified primarily into two categories as shown in Table 4.4 based on the design spectral acceleration coefficients at a period of 1.0 sec. The previous work shows that the bridges classified as SDC B should generally be designed for ductile behaviour. Bridges classified as SDC A should be designed for elastic or limited ductile behaviour based on considerations of economic efficiency, with the exception of very stiff bridges such as those with short piers. These conclusions are expected to remain valid even when the effects of various intended behaviour of bridges on foundation design are considered. Figure 4.9 shows the normalized sizes and reinforcement amounts of the piles intended for ductile behaviour of bridges by those for elastic and limited ductile behaviour of bridge plotted against the design spectral acceleration coefficients at a period of 1.0 sec for the design situations. The design for intended ductile behaviour of bridge can reduce the size and total amount of reinforcement of the piles for bridges in SDC B. However, the reduction effects are generally not as significant for bridges in SDC A. The plot confirms that the SDC can provide guidance on the design of bridges from the economic point of view.

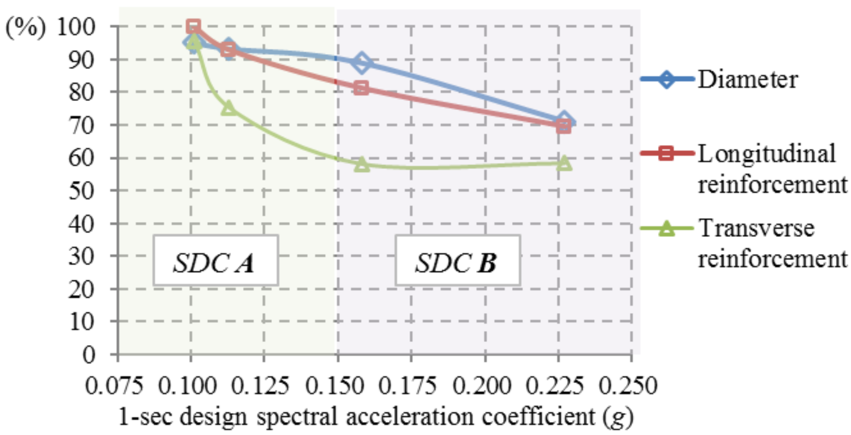


Figure 4.9 Reduction effects on size and reinforcement amount of pile by ductile design of bridge with the seismic risk

The bridges having either piers shorter than 9 m or tall piers exceeding 13 m in height in the earthquake resisting system may not be completely described by the SDC as discussed in Section 4.2.1(3). The same bridge pier with the height taken as 7 m is considered with the design situation of Importance Class I and Ground Type C, which falls into SDC A. On the other hand, the design situation of Importance Class II and Ground Type D in SDC B is considered for the bridge pier with the height taken as 13 m. The results are presented in Figure 4.10 and compared with those for pier height of 9 m taken from Figure 4.9. By adopting ductile design for the 7 m pier in SDC A, the pile diameter is reduced by 13% and the volume of reinforcement is reduced by more than 40%. The pile design for the 13 m pier in SDC B, however, is only slightly affected by the ductile design of bridge.

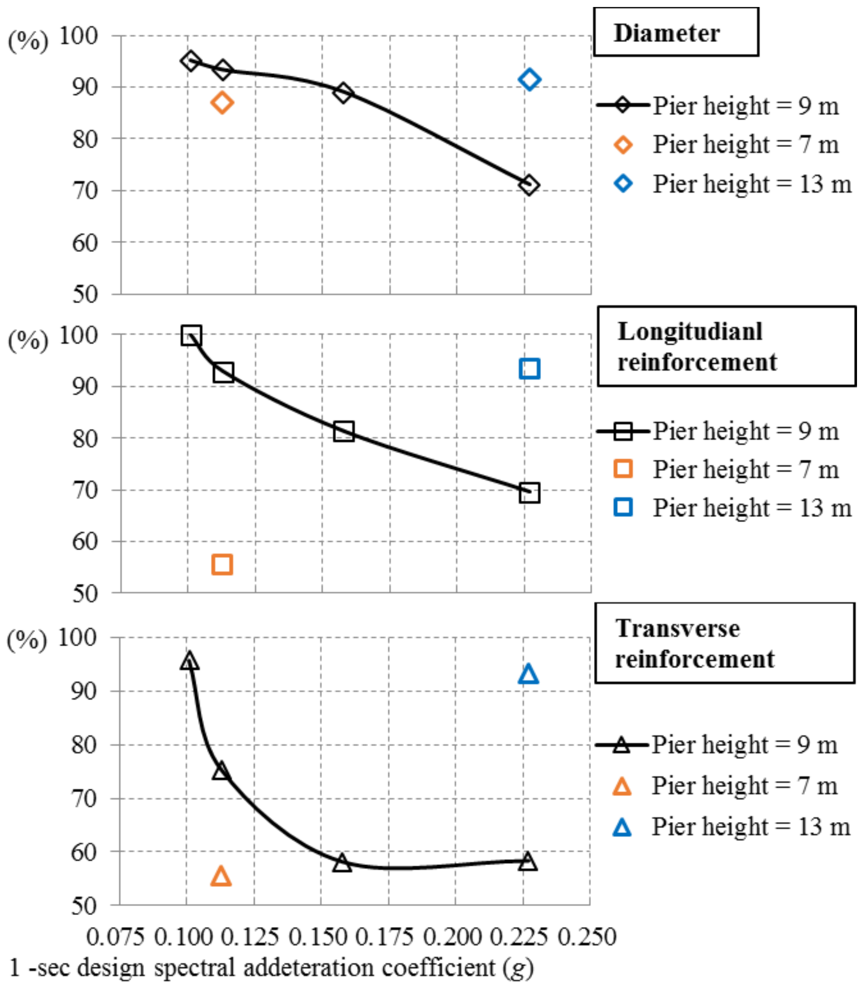


Figure 4.10 Reduction effects on size and reinforcement amount of pile by ductile design of bridge with the bridge stiffness

4.2.4 Conclusions

In summary, on the choice of suitable intended behaviour in the design of a bridge in Hong Kong, it is preferable to design the bridge for ductile behaviour if the bridge is considered to be of critical importance and/or the site is relatively poor. If the design conditions are more favourable, it may be better to opt for limited ductile behaviour for economic reasons. The SDC defined based on seismic risk can be used to determine the intended seismic behaviour of the bridge for a design situation. The classification of the SDCs applicable to Hong Kong is given in Table 4.4. Bridges of SDC A can be designed for essentially elastic behaviour and those of SDC B shall be designed for ductile behaviour. Special attention should be paid to bridges having either piers shorter than 9 m or tall piers exceeding 13 m in height in the earthquake resisting system. The former group will generally be subjected to strong seismic motions due to their short periods so that the selection of ductile behaviour is usually expedient even if the situation is classified as SDC A. Besides, piers with aspect ratios not greater than 3.0 are susceptible to flexural-shear or even shear failure during earthquakes more severe than the design events if designed for limited ductile behaviour. For the latter group, the seismic resisting systems are quite flexible and it is usually justifiable to design such bridges in SDC B for limited ductile behaviour.

CHAPTER 5 SEISMIC ASSESSMENT OF EXISTING BRIDGES

As one of the most densely populated metropolises, Hong Kong has a highly developed and sophisticated transportation network with bridges forming critical links. Traffic disruption caused by natural hazards such as earthquakes or super typhoons could lead to enormous economic losses, apart from potential loss of lives. The current stock of highway bridges in Hong Kong has an average age of over 30 years. Since their construction, there have been significant improvements in bridge design practice, particularly in seismic bridge design and analysis, based on lessons learnt from the subsequent earthquakes and more laboratory investigations around the world. Sophisticated bridge design codes incorporating advanced methodologies and procedures for design for earthquakes resistance have emerged worldwide since the release of Structures Design Manual for Highways and Railways 3rd Edition, i.e. SDMHR 3rd (Highways Department, 2006). As opposed to SDMHR 3rd, the recent SDMHR 2013 Edition (Highways Department, 2013) has mainly made reference to BS EN 1998-1 (BSI, 2004c, 2005b) and BS EN 1998-2 (BSI, 2004c, 2005b) for seismic bridge design provisions. The revisions to the design manual are quite significant in respect of adoption of ductility design and dynamic analysis as the basis. Moreover, knowledge of the seismic hazard in the region has grown. On the basis of a comprehensive seismic hazard analysis for Hong Kong as described in Chapter 2, the reference peak ground acceleration is raised from $0.05g$ in SDMHR 3rd to $0.12g$ in SDMHR 2013, where g is the acceleration due to gravity. Hence some of the existing highway bridges designed to earlier standards may be vulnerable to damage during a moderate-to-high seismic event. It is therefore essential to examine the compliance of existing bridges with the current design manual by appropriate seismic assessment.

This chapter firstly summarizes the arrangements and structural details of some typical existing highway bridges followed by discussions of their compliance with SDMHR 2013 including the design seismic action and detailing rules. The vulnerable components, such as piers, pier-deck connections, deck and abutment, and their possible damage and failure modes will be identified. The available methodologies for seismic assessment of a specific bridge are then elaborated with a focus on strain-based evaluation method. This method will be demonstrated with an example. Lastly, an approach to obtain the vulnerability of a class of bridges by means of fragility analysis is introduced, which is particularly useful for optimization of bridge retrofitting programmes and development of pre-earthquake action plans. The fragility curves for typical classes of highway bridges in Hong Kong are presented.

5.1 Compliance of Existing Bridges

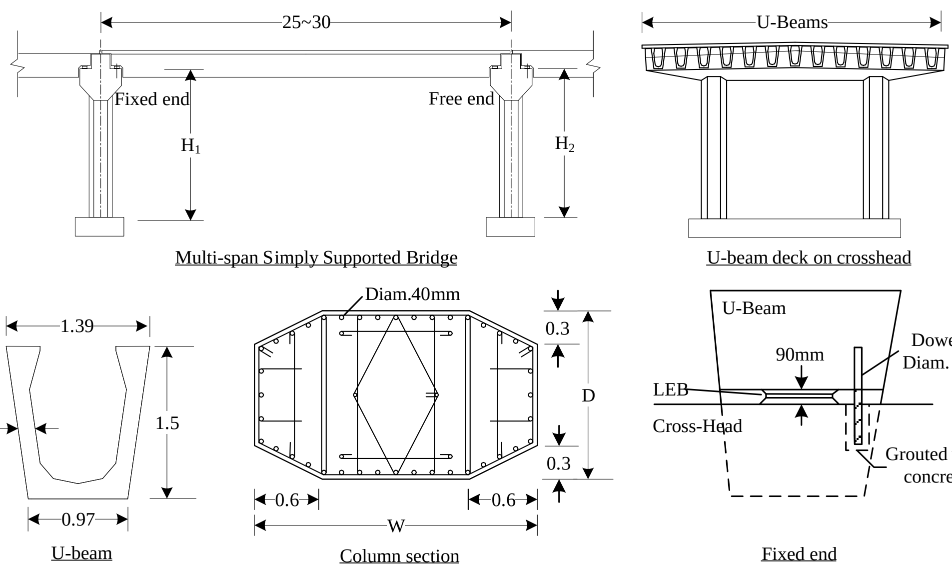
5.1.1 Typical characteristics of existing bridges

After an initial survey of major highway bridges in Hong Kong, it is found that the bridges in Hong Kong are predominately concrete girder bridges. As-built bridge data have been collected and examined for general details of piers, superstructures and their connections. Based on the types of superstructure and connection surveyed, the typical existing highway bridges in Hong Kong comprise mainly two classes of concrete girder bridges: multi-span simply supported bridges (MSSB) and multi-span continuous bridges (MSCB), which are prevalent in the 1980s and 1990s, respectively. Figure 5.1 shows the typical configurations of these two classes of bridges. It should be noted that some generalizations have been made as necessary. Although all details are not identical, the details shown here are considered to be representative of their respective bridge classes.

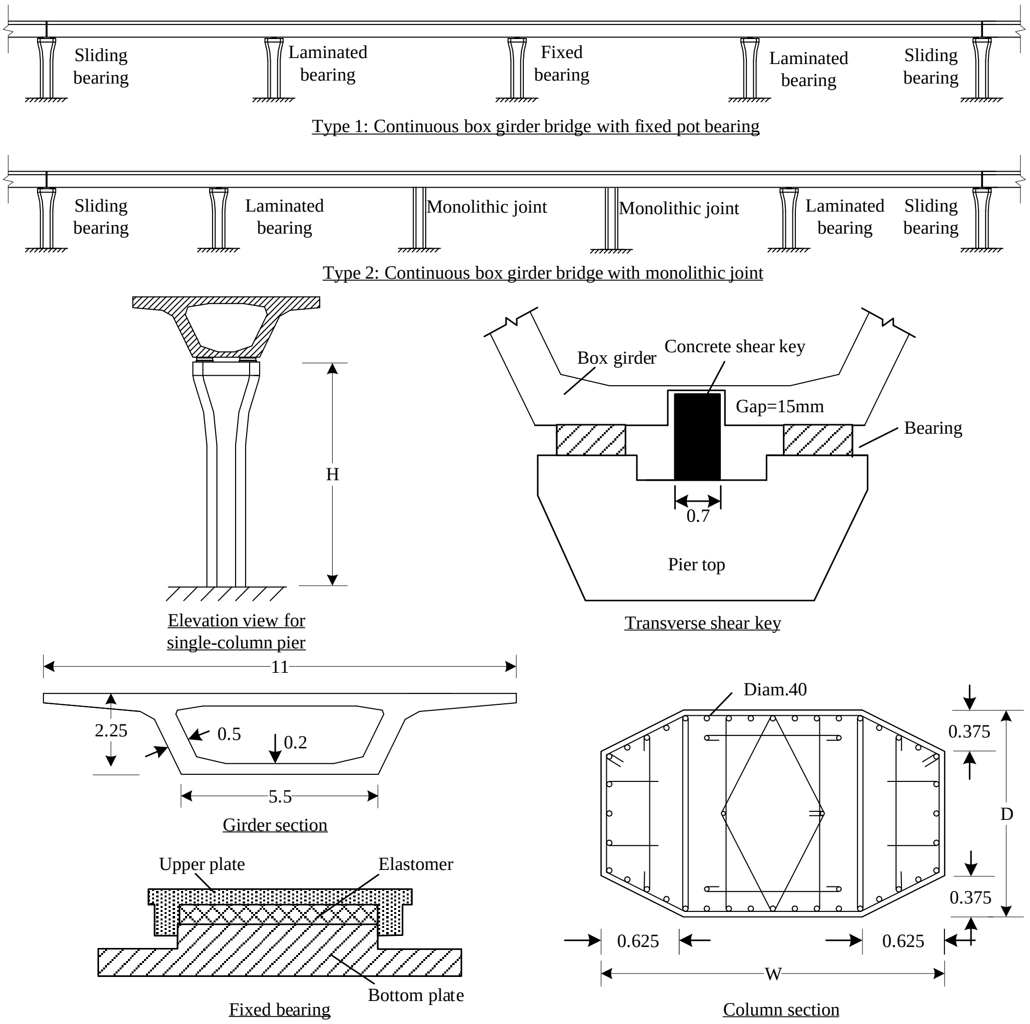
The MSSB bridges consist of multiple spans of 25 m to 30 m that are simply supported on bents as shown in Figure 5.1(a). The superstructure usually consists of a number of precast U-beams resting on crossheads, each of which is cast with either single or multiple octagonal columns. The width of the deck and the arrangement of columns depend on the number of U-beams used, i.e. often 9 to 13 U-beams supported by two columns and 4 to 5 U-beams supported by one column. Laminated elastomeric bearings are used to support the deck under each U-beam. At one end, the connection of the beam to the pier crosshead consists of an elastomeric bearing pad with a ϕ50 mm galvanized dowel bar cast in the precast beam and grouted in a hole left in the crosshead to restrain relative displacement between the girder and the pier, i.e. equivalent elastomeric fixed bearing. No dowel bars are provided at the other end so as to allow expansion movements. All bearings are installed between an upper plinth and a bearing plinth on the crosshead, both of roughly the same size as the bearing.

For the MSCB bridges, the superstructure is continuous across the pier crosshead. There are 4 or 5 roughly uniform continuous spans between expansion joints with the span length ranging from 30 m to 50 m. The superstructure consists of a single- or multi-cell box girder and the pier usually consists of a single octagonal column as shown in Figure 5.1(b). The MSCB bridges can be further divided into two classes depending on the type of principal structural girder-pier connection: using either fixed bearings on one central pier, or monolithic joints over two consecutive piers close to the middle of the bridge. Laminated elastomeric bearings are used for the rest of the connections to allow longitudinal deck movements. In addition, additional plane sliding bearings mainly consisting of polytetrafluoroethylene (PTFE) are provided on the top of the elastomeric bearings at the expansion joints to accommodate large deck movement as shown in Figure 5.1(b). The movable bearings are provided with a concrete shear key system that allows the deck to move freely in the longitudinal direction while excessive movement in the transverse direction is to be restrained as shown in Figure 5.1(b). The fixed bearings are often fixed pot bearings, each of which consists of an upper steel plate with keeper plates, a bottom steel pot and an elastomer in-between. All bearings are installed between an upper plinth and a bearing plinth on pier crosshead, both of roughly the same size as the bearing.

The columns usually have octagonal sections for both classes of bridges and the longitudinal reinforcing bars are mostly 40 mm in diameter. The transverse reinforcement typically consists of ϕ12 mm stirrups at a spacing of 300 mm for the MSSB and at a spacing of 250 mm for the MSCB.



(a) Typical MSSB bridges



(b) Typical MSCB bridges

Figure 5.1 Typical configurations of existing bridges in Hong Kong
(Dimensions are in metres unless otherwise stated)

5.1.2 Seismic action based on SDMHR 3rd and SDMHR 2013: Parametric study

The seismic actions defined in SDMHR 3rd and SDMHR 2013 are fundamentally different with the former based on static treatment in terms of nominal earthquake load equivalent to 5% of the total vertical load together with a partial factor of 1.4 for ultimate limit state (i.e. an equivalent response acceleration of 0.07*g*) and the latter based essentially on dynamic consideration using the response spectrum. The seismic action may be underestimated under the SDMHR 3rd and the effects of design parameters including deck width, span length, pier height, and number of spans on the seismic action are evaluated below via a comprehensive parametric study.

(1) Key design parameters and representative ranges

A total of 219 hypothetical sample bridges were generated from the MSSB and MSCB classes with 3, 4 and 6 equal spans. The values considered for the other design variables were decided so as to cover as broadly as possible the existing bridges. They are summarized here:

- Three span lengths were studied, i.e. 30 m, 40 m and 50 m, in order to approximate the span lengths that are commonly used in small- to medium-span bridges.

- The deck width was from one of the six values, i.e. 6.15 m, 7.40 m, 8.80 m, 12.18 m, 18.80 m and 22.50 m, to cover a variety of traffic lane layouts from one-way one lane to dual-way six lanes.
- The cross-sectional area of deck is associated with the span length and deck width, and it is supposed to increase with the increase of span length and deck width. The cross-sectional area of deck of the sample bridges were decided by drawing on existing bridges with similar span lengths and deck widths to make sure that the sample bridges studied are reasonable.
- The pier with a single straight rectangular column was considered for both the MSSB and MSCB classes. The pier height was assumed to be one of the following values, i.e. 5 m, 7 m, 9 m, 11 m, 13 m, 15 m, 17 m, 19 m and 21 m. The dimensions of pier cross-section were also drawn from existing bridges with similar span lengths, deck widths and pier heights. The dimensions were between 1.5 m and 3.0 m.

According to SDMHR 2013 and the relevant Eurocodes, the seismic action is related to the ground type. As the soil gets softer, the seismic action becomes larger. In addition, the seismic action is varied to take into account various levels of structural importance. The results for Importance Class I with Ground Type A and Importance Class III with Ground Type D would provide the lower bound and upper bound estimates of the seismic action in accordance with SDMHR 2013.

Other assumptions made for the calculations are as follows:

- The permanent masses for consideration of seismic action included self-weight and superimposed dead load. The superimposed dead load mainly consisted of road surfacing and parapet. Solid reinforced concrete (RC) parapet with a sectional area of 0.5 m^2 and 130 mm thick bituminous road surfacing with a unit weight of 23 kN/m^3 were assumed. The other associated masses for consideration of seismic action included traffic actions of Load Model 1 from BS EN 1991-2 (BSI, 2003) with a combination factor of 0.2 as required in Clause 4.9(a) of SDMHR 2013 and 1/3 type HA highway loading from BD37/01 (Highways Agency, 2001) as required in Clause 2.6(b)(i) of SDMHR 3rd.
- The dynamic responses of the sample bridges were approximated by equivalent single-degree-of-freedom systems without accounting for soil-structure interaction. Apart from the mass, the stiffness of the system was taken to be the sum of the stiffnesses of the resisting members. In the transverse direction, the seismic resistance of the bridge was considered to be provided by all the piers. In the longitudinal direction with some piers rigidly connected to the deck either monolithically or through fixed bearings which have a major contribution to the seismic resistance, only the longitudinal stiffnesses of such piers were accounted for.
- Elastic behaviour was assumed for all of the sample bridges. Therefore, the elastic stiffness of the pier corresponding to the uncracked gross concrete section and a behaviour factor of 1.0 were adopted when determining the stiffness of pier and design seismic action respectively based on SDMHR 2013.

(2) Results and discussions

The fundamental periods of the sample bridges are shown in Figure 5.2. The ranges of period encompassing the horizontal branch (constant acceleration response-control zone) and first descending branch (constant velocity response-control zone) of Type 2 response spectrum of BS EN 1998-1 (BSI, 2004c), i.e. 0.05 s to 0.3 s and 0.3 s to 1.2 s, respectively, are also marked in the figure by pink and blue, respectively. It can be seen that the fundamental periods of the majority of sample bridges containing monolithic joints are covered by the first descending branch in both the longitudinal and transverse directions. The majority of sample bridges with the structural deck-pier connection of fixed bearings have fundamental periods covered by the first descending branch in the transverse direction but the second descending branch (constant displacement response) in the longitudinal direction. The different structural connections seem to have major effect on the longitudinal fundamental period only.

The fundamental period is significantly influenced by the pier height. Taller piers can result in longer fundamental periods in both horizontal directions. In addition to the pier height, the weight of bridge also has some effect on the fundamental period in the longitudinal direction. As longitudinal sliding or flexible mountings must be provided between the deck and some piers to reduce the stresses induced by deck deformations due to thermal actions, shrinkage, etc., the overall stiffness is unlikely to increase as much as that of the weight in the longitudinal direction. The longitudinal fundamental period of bridge then tends to increase as the total length and hence weight increase.

Considering the most favourable seismic design situation with Importance Class I and Ground Type A, bridges that are subjected to elastic response spectral acceleration (RSA) not greater than 0.07*g* shall have fundamental periods longer than 1.07 s based on SDMHR 2013. This critical fundamental period is also marked by a red line in Figure 5.2. It can be seen that a large portion of sample bridges are susceptible to RSAs above 0.07*g* for having fundamental periods below 1.07 s.

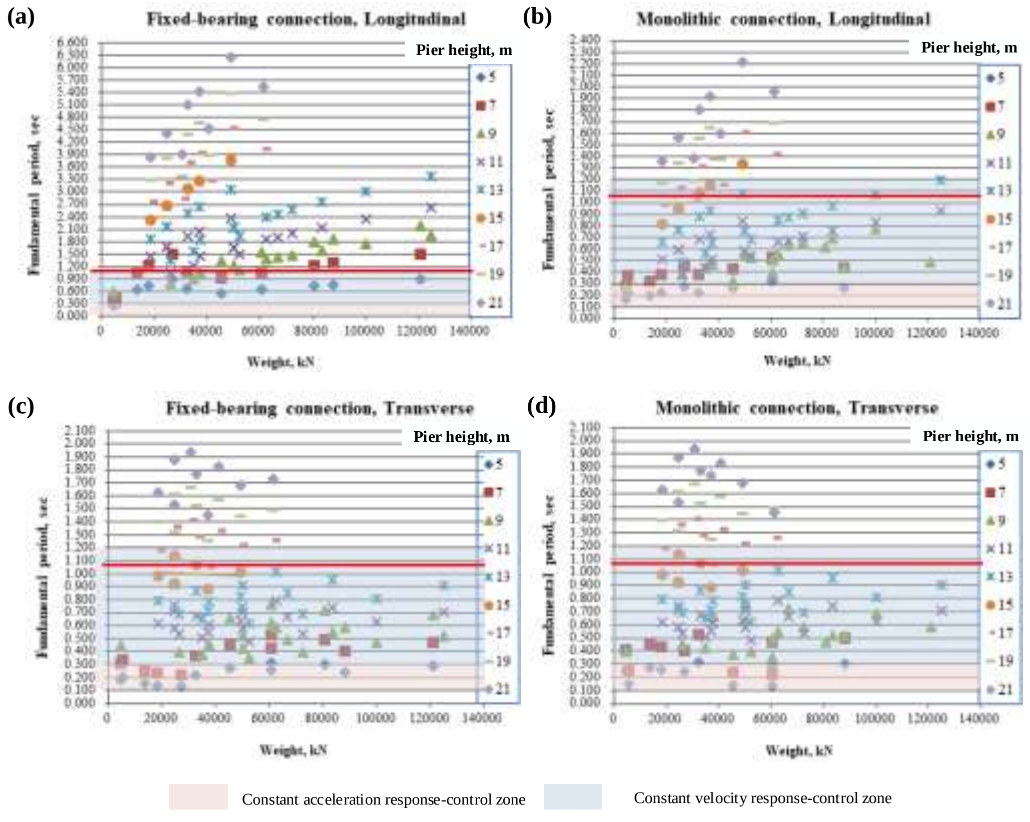


Figure 5.2 Fundamental periods of sample bridges

The RSAs for these sample bridges corresponding to Importance Class I with Ground Type A and Importance Class III with Ground Type D based on SDMHR 2013 are plotted in Figure 5.3 and Figure 5.4, respectively, giving the following observations:

- The RSAs for the sample bridges with monolithic joints are rarely below the equivalent response acceleration of 0.07*g* as adopted in SDMHR 3rd in both horizontal directions, except for those with piers taller than 17 m. When the design conditions get more stringent up to Importance Class III with Ground Type D, the RSAs for this particular type of bridges are very likely to exceed 0.07*g*. The most likely upper and lower bounds of RSAs for this type of bridges are found to lie in 0.03*g* - 0.30*g* and 0.14*g* - 1.26*g*, respectively. Similar trends have also been found for sample bridges with structural deck-pier connections of fixed bearings in the transverse direction of the bridge.
- The bridges having fixed bearings as the structural deck-pier connections are normally subjected to much smaller RSAs than their counterparts with monolithic joints in the longitudinal direction of the bridge. The RSAs of the former in the longitudinal direction can be lower than the equivalent response acceleration of 0.07*g* as adopted in SDMHR 3rd for some MSCBs of this type especially when the piers are taller than 7 m. However, only part of those with piers taller than 15 m can still have RSAs not exceeding that specified by SDMHR 3rd under the most unfavourable circumstances with Importance Class III and Ground Type D. In the longitudinal direction, the most likely upper and lower bounds of RSAs for MSCBs of this type are found to lie in 0.03*g* - 0.14*g* and 0.07*g* - 0.70*g*, respectively, while those for MSSBs are found to lie in 0.03*g* - 0.24*g* and 0.07*g* - 1.20*g*, respectively. The MSSBs are generally stiffer than the MSCBs with fixed bearings and are thus susceptible to higher RSAs.

In summary, the elastic seismic coefficient defined in accordance with SDMHR 2013 is higher than 0.07*g* as adopted in SDMHR 3rd for common short- to medium-span highway bridges under most circumstances and can most often be up to 2 to 18 times higher. Therefore, the existing bridges are expected to yield under the design level earthquakes rather than to remain elastic. For these bridges, the seismic coefficients can theoretically be reduced by a behaviour factor not exceeding 3.5 as allowed by BS EN 1998-2 (BSI, 2005b) on condition that the structural details comply with the minimum requirements as set out in BS EN 1998-2 to ensure the required levels of ductility, which is rather unlikely. This will be discussed next. For those incurring extremely large seismic coefficients, the flexural resistance of the piers of these bridges may need enhancement so as to lessen the ductility demand imposed on the piers by seismic actions within the available levels of ductility as allowed by BS EN 1998-2.

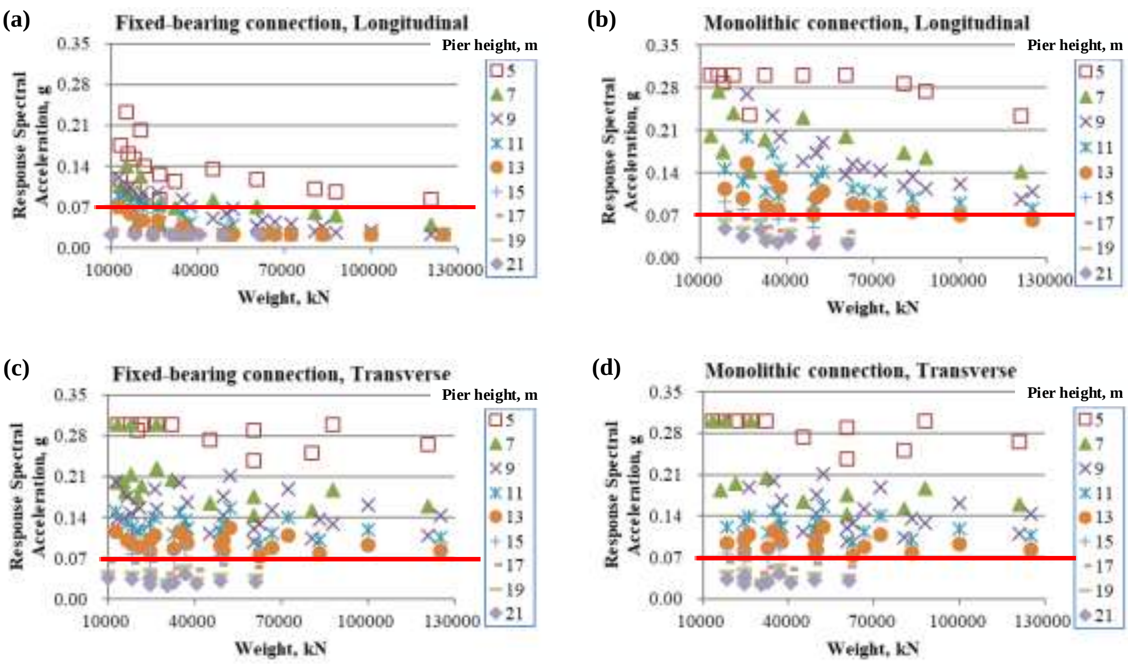


Figure 5.3 RSAs of sample bridges with Importance Class I and Ground Type A

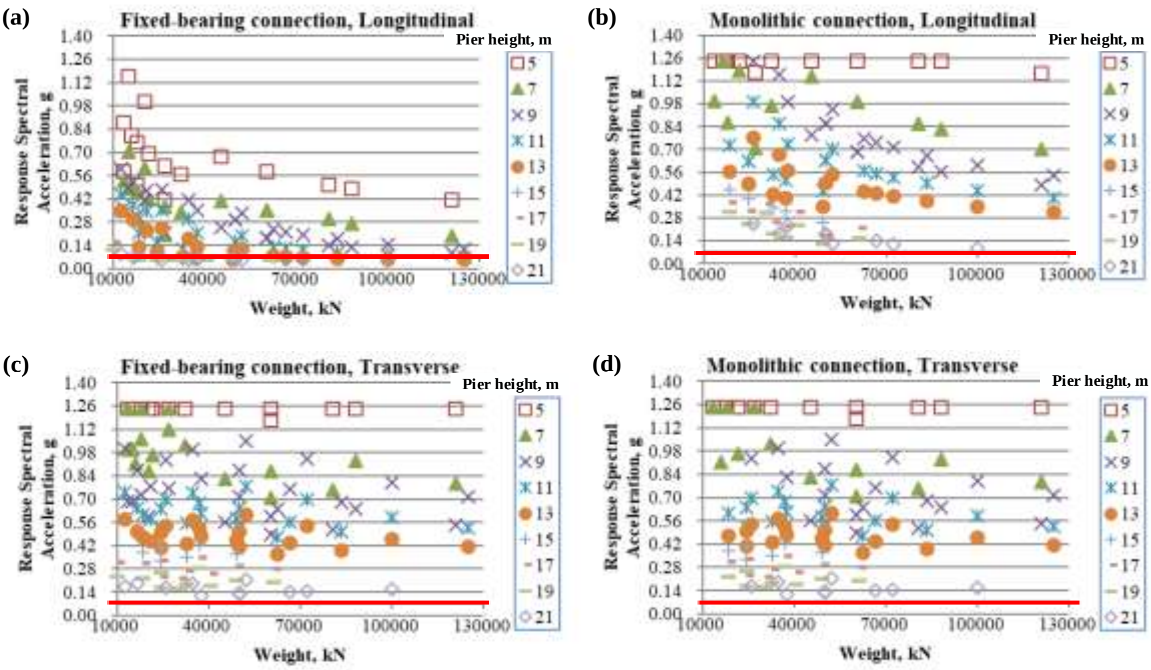


Figure 5.4 RSAs of sample bridges with Importance Class III and Ground Type D

5.1.3 Assessment of structural details in existing bridges

BS EN 1998-2 has incorporated ductility requirements into the structural design to avoid collapse during the design earthquake by ensuring the structural integrity of the bridge at connections and providing at least a minimum level of deformability at critical sections. For concrete piers, BS EN 1998-2 requires that the transverse reinforcement be closely spaced especially over the potential plastic hinges to guard against outward buckling of main longitudinal bars and that the longitudinal bars develop as far as possible into the bent cap and pile cap among others. At the moveable connections, BS EN 1998-2 has specified a minimum overlap (seat) length to avoid unseating at abutments and expansion joints. The relevant details in the existing bridges will be checked for their compliance and potential failure modes of non-compliant components will be discussed below. Other prevalent details that are vulnerable to seismic actions will also be presented.

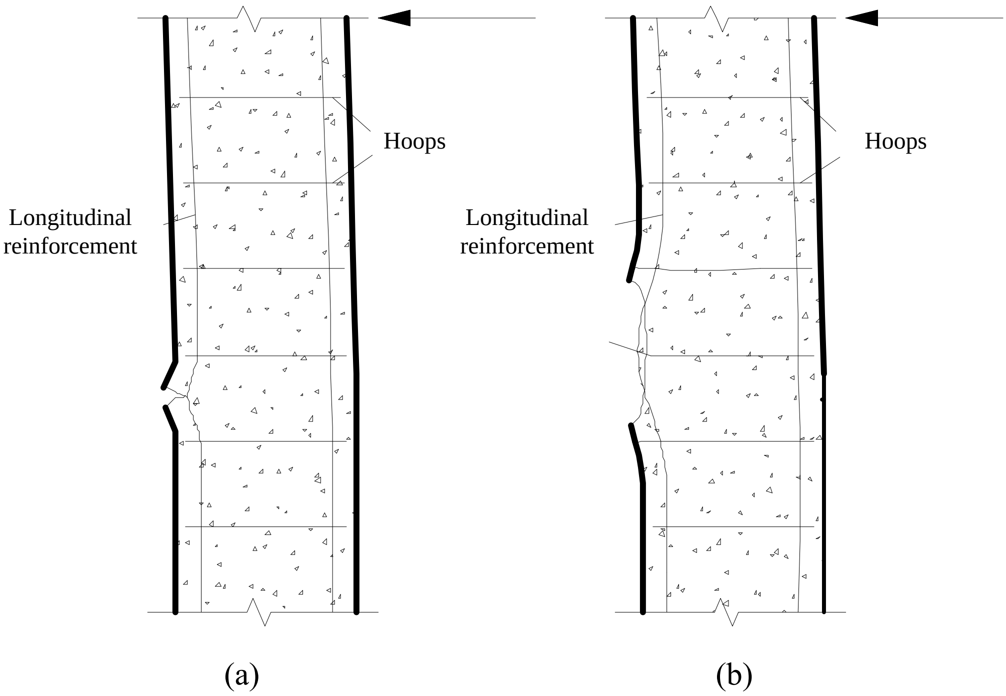
(1) Confining reinforcement

The intended ductility is largely dependent on the structural details that have been proven in the laboratory to provide at least the intended ductility, and the effective confinement of the core concrete is found to be of particular importance to the concrete piers. The requirements as set out in BS EN 1998-2 for the spacing between hoops or ties in the longitudinal direction s_L and the transverse distance between hoop legs or cross-ties s_T over the entire length of plastic hinge are as follows:

- s_L shall not exceed 6 times the longitudinal bar diameter or 1/5 of the smallest dimension of the confined concrete core to the hoop centreline; and
- s_T shall not exceed 200 mm or 1/3 of the smallest dimension of the concrete core to the hoop centreline.

As there is no explicit ductility requirement according to the previous versions of design manual, it is uncertain if both the values of s_L and s_T in the existing bridges can fulfil the current standards, possibly resulting in deficient confinement of concrete. For instance, SDMHR 3rd adopts a maximum longitudinal spacing of 12 times the size of longitudinal bar and a maximum transverse distance of 300 mm for the transverse reinforcement, which are both larger than those specified in BS EN 1998-2.

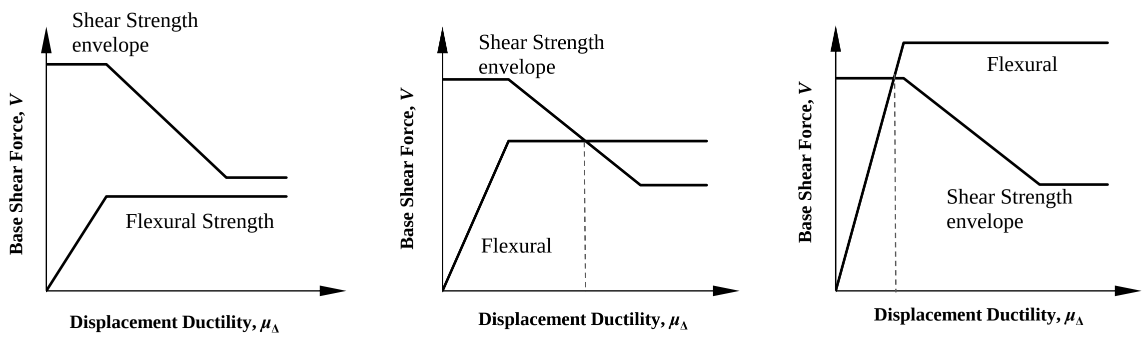
One of the important functions of the transverse reinforcement is to provide lateral support to the longitudinal reinforcement in the bridge pier. If the transverse reinforcement is inadequate, buckling of longitudinal bars may occur and cause premature brittle failure of the pier. Figure 5.5 shows two types of premature buckling of longitudinal reinforcement in a pier. Figure 5.5(a) illustrates the buckling of the longitudinal bar between adjacent sets of transverse reinforcement, while Figure 5.5(b) shows the buckling across multiple sets of transverse reinforcement. The former is related to the excessive spacing while the latter is generally caused by the inefficient lateral binding action provided by the hoops and/or cross ties.



(a) (b)

Figure 5.5 Premature buckling of longitudinal reinforcement in piers: (a) buckling between adjacent hoops; and (b) buckling across several hoops

Another important function of the transverse reinforcement is to provide shear resistance. As shear failure is brittle in nature, it should be avoided. The capacity design strategy has been adopted by BS EN 1998-2 to ensure that potential plastic hinges form before shear and/or other brittle failures. The capacity design principles require that the shear resistance of the plastic hinges and all other regions shall be designed to resist the shear forces developed when all the plastic hinges have attained their overstrength moments. Considering the relationship between shear resistance and moment resistance of a pier, there are three possible damage modes under extremely strong earthquake: flexural failure, flexural-shear failure and brittle shear failure, as illustrated in Figure 5.6 (ATC, 1981). Adequately confined concrete piers will behave according to Figure 5.6(a). In the previous versions of design manual, no capacity protection accounting for formation of plastic hinges was prescribed and the base shear force could be underestimated, and hence the piers in some existing bridges may behave as shown in Figure 5.6(b) and Figure 5.6(c).



(a) (b) (c)

Figure 5.6 Failure modes of RC column: (a) flexural failure; (b) flexural-shear failure; and (c) brittle shear failure (ATC, 1981)

(2) Splicing of longitudinal reinforcement

It is also common in some existing bridges that the longitudinal reinforcement was lap-spliced at critical sections for construction convenience. However, this practice is discouraged today, since "lap-splice failure" is probable even at small to moderate displacement ductility unless very large amount of transverse reinforcement is provided. BS EN 1998-2 has allowed no splicing by lapping or welding of longitudinal reinforcement within the plastic hinge region.

The "lap-splice failure" is described as "failure that involves relative longitudinal movement of the spliced bars and that requires the formation of a fracture both perpendicular and parallel to the member surface in order to permit the bars to slide relative to the RC-member core" (Priestley *et al.*, 1996). Theoretically the splice failure can occur before or after any yielding of the longitudinal bars, depending on the spliced bars, the concrete and reinforcement, but mostly on the splice length (Priestley *et al.*, 1996). If the splice failure occurs before any yielding of the bars, the section will not be able to develop its intended flexural strength. Even if the splice is able to develop the flexural strength, its competence may gradually degrade under cyclic loading. Eventually the residual moment capacity is sustained by the concrete stress in the compression zone of the column with no contribution from the longitudinal bars.

Not only will lap-splice affect the moment capacity, but it will also have obvious influence on the deformability of plastic hinges. Four types of moment-curvature behaviour have been proposed by Priestley *et al.* (1996) as shown in Figure 5.7. Curves (1) and (2) denote the moment-curvature model for confined and unconfined RC sections, respectively, both without lap-splices. Curves (3) and (4) denote the models for lap-spliced section that the intended flexural strength can and cannot be attained, respectively. These idealised models have been verified by various studies (Chail *et al.*, 1991; Lynn *et al.*, 1996; Melek and Wallace, 2004). It can be seen from Figure 5.7 that the curvature ductility drops significantly due to the presence of lap-splice. However, if there is ample lap length and the section stays elastic under the design seismic action such that the flexural strength is not exceeded, the existing bridge may be free from lap-splice failure.

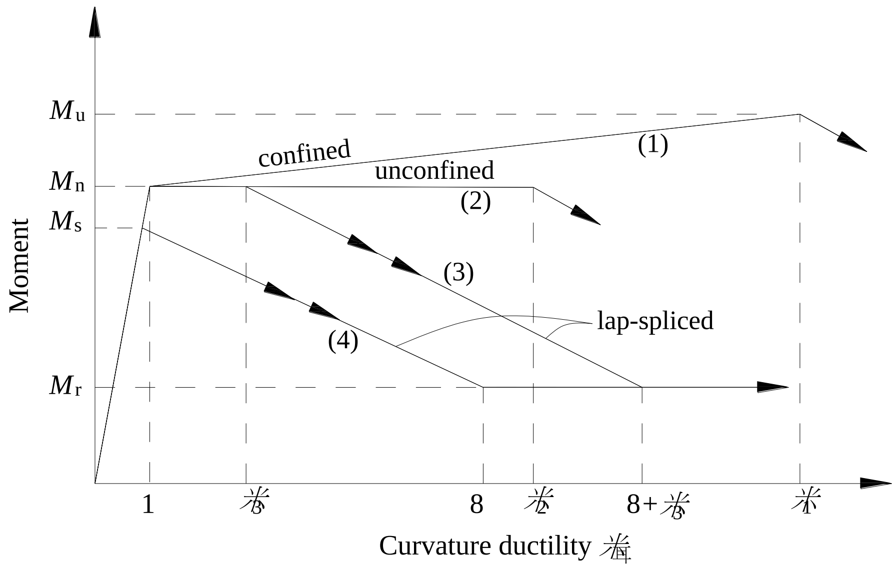


Figure 5.7 Curvature ductility of column section with and without lap-splicing of main bars (Priestley *et al.*, 1996)

(3) Bearings and monolithic joint

Unless the superstructure and substructure are connected monolithically, bearings are commonly used on the piers and abutments to support the superstructure. Aside from transferring vertical gravity loads, the bearings can also facilitate longitudinal movements and rotation due to live load deflection, expansion and contraction. The bearings can be important components for transferring seismically induced forces from the superstructure to the substructure and accommodating seismic movements. They are vitally important to the seismic performance of the whole bridge.

The bearings can be classified by their horizontal behaviour as either fixed, moveable or deformable. Moreover, the bridge articulation can be different in the longitudinal and transverse directions. BS EN 1998-2 has specified different criteria for the design of different bearings. Fixed bearings shall be designed as capacity protection components and damage of shear keys and anchor bolts shall be avoided. Moveable bearings shall be able to accommodate without damage the total displacement in the seismic design situation. Elastomeric bearings shall be designed to resist the maximum shear deformation.

However, these present design approaches most likely have not been applied to the design of existing bridges. In view of the higher seismic hazard level, bearings may be vulnerable in earthquakes, particularly if they are also required to carry transverse shear. Fixed bearings are vulnerable to shear failures in case the design shear capacity is exceeded. Elastomeric bearings that have deflected beyond their design shear movement may be ruptured. For sliding bearings, if the design capacity for sliding movement has been exceeded, damage is likely.

Monolithic connection as another type of connection between the superstructure and substructure is able to transfer shear forces as well as bending moments. The performance of such joints depends on proper structural detailing. For example, the reinforcement may be pulled out of the joint due to insufficient development length of reinforcement through the connection. Besides, joint shear damage is often caused by insufficient confinement at the joint region. BS EN 1998-2 has required capacity design in shear for joints, and continuation of pier stirrups and "beam" stirrups into the joint is recommended.

(4) Movement joint

Movement joints are normally provided in bridges to allow movements due to thermal, creep and shrinkage deformations of the superstructure. The movement joints have been a source of extensive damage in past earthquakes due to the pounding at joints. Although they are often treated as non-critical structural components and expected to be damaged under the design seismic action, they should still cater for a predictable mode of damage and provide the possibility of permanent repair. For this reason, BS EN 1998-2 requires that the clearances should accommodate appropriate fractions of the design seismic displacement and of the thermal movement after allowing for any long-term creep and shrinkage effects. In this manner, any damage under frequent earthquakes for much shorter return periods than that for the design seismic event is avoided. Appropriate values of such fractions may be chosen, based on a judgement of the cost-effectiveness of the measures taken to prevent damage.

During an initial survey of existing bridges, it was found that common clearances at movement joints of 50 mm, 75 mm and 100 mm were provided to bridges in Hong Kong of small- to medium-spans. It is expected that these movement joints may be incapable of relieving possible pounding under extremely strong earthquakes. The current seismic design methodology for bridges calls for movement joints that are capable of accommodating large multi-directional displacements as well as dissipating energy, i.e. the so-called "seismic movement joint".

(5) Overlap length

The unseating at movement joints is one of the most devastating types of bridge damage. The damage due to unseating has been mostly attributed to narrow seats. Sufficient overlap length between the supported and supporting members shall be provided if relative displacement is intended, which includes but is not restricted to

- seismic displacement;
- long-term displacement, such as shortening due to prestressing, and creep and shrinkage of concrete decks;
- thermal expansion and contraction; and
- minimum supported length ensuring the safe transmission of the vertical reaction (not less than 400 mm).

It is necessary to estimate the value of the total displacement in order to provide adequate overlap length. BS EN 1998-2 has provisions for the design value for each of the displacements in seismic design situation, whereas the previous versions of manual have not provided any guidance on estimation of the seismic displacement.

The typical overlap lengths in existing bridges were found to be 0.4 m to 0.8 m for the MSSBs and 1.1 m to 2.5 m for the MSCBs. It is unlikely that the girders will become unseated with such ample seats. Nevertheless, detailed assessment should be carried out. If the required minimum overlap lengths are not met, seat extenders can be provided, or alternatively positive linkages such as catchers and restrainers between supporting and supported members may be used.

(6) Half joint

The half joint as shown in Figure 5.8 is a typical feature in many bridges built until mid- to late 1990s around the world and, with no exception, it applies to Hong Kong. In accordance with BS EN 1998-2, seismic links shall always be provided at intermediate separation joints located within the span to prevent unseating of the suspended beams and to resist large shock forces caused by unpredictable pounding between adjacent sections.

Half joints were initially introduced into concrete bridges as a means to simplify design and construction operations. However, in addition to potential pounding and unseating damage during earthquakes, there are usually leakage problems causing concrete deterioration and corrosion of reinforcement at the joints. Moreover, they are not easily accessible for inspection and maintenance. The use of half joints should be discouraged now.

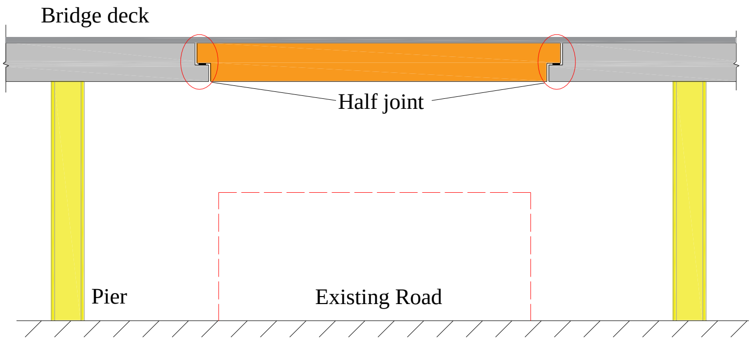


Figure 5.8 Bridge with half joints

(7) Dowel bar

After reviewing some existing bridges in Hong Kong, it is found that dowel bars were widely used in precast-beam-and-slab bridges in the past as shown in Figure 5.1(a). The dowel bar is embedded in the precast beam and inserted into the keyhole cast at the cap beam so as to restrict the horizontal movements of the deck. This practice is usually adopted at one end of the precast beam, i.e. the fixed end. At the other end, the beam is only supported by the laminated elastomeric bearing that is designed to cope with rotational and horizontal movements in all directions, i.e. the free end.

However, this supporting system can be detrimental to the seismic resistance of bridges. Owing to the presence of the dowel bar, the horizontal shear stiffness at the fixed end is much higher than that at the free end, causing torsional effect as shown in Figure 5.9. The torsional effect is mainly balanced by shear forces of the dowel bars and eventually transferred to the pier. Consequently, it is likely that the pier would suffer the effect of shear-bending-torsion interaction during an earthquake, which would adversely affect the seismic resistance of the pier. Besides, the deck may also rotate so much as to break the dowel bars. Once the dowel bars are ruptured, pounding between adjacent components may occur, possibly causing even unseating of the beams.

In view of the risks, the use of dowel bars for shear keys should be reviewed and improved in future. One possible solution is to provide only laminated elastomeric bearings at both ends. To avoid unseating, larger bearing seat and side keeper devices may be adopted. Alternatively, if the deck is made continuous over several spans, the torsional effect can be relieved.

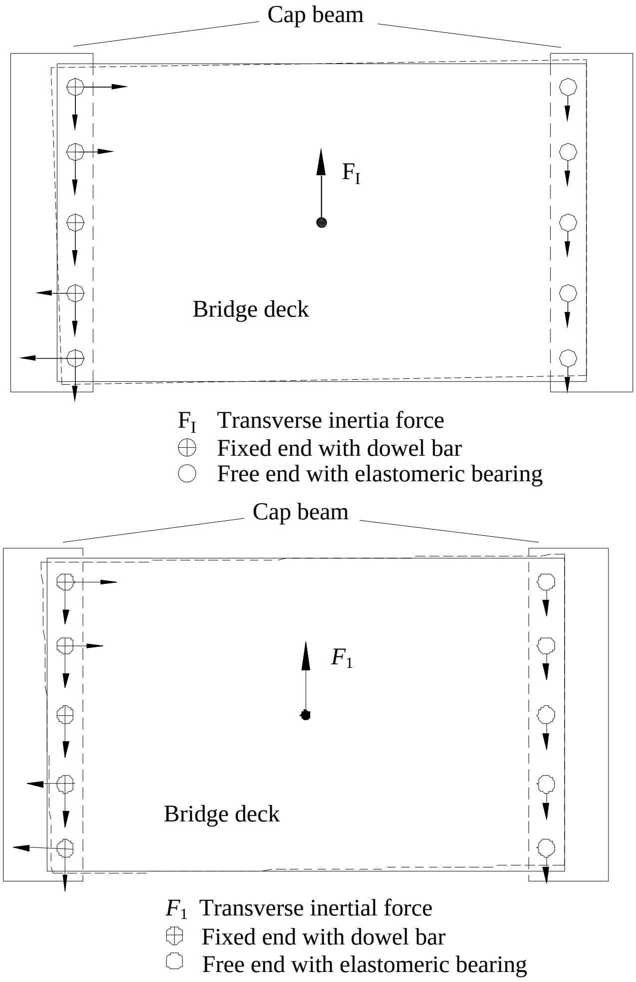


Figure 5.9 Torsional effect of bridge deck under transverse seismic excitation

5.1.4 Summary

In Table 5.1 that summarizes the possible failure modes of various components during earthquake, there are basically two categories of failure in nature: brittle failures such as shear failure and span unseating, and ductile failures such as flexural failure. Owing to the brittle nature of all the possible failure modes of the superstructure and connections, these components are required to remain elastic and shall be protected by capacity design and proper detailing. A problem with foundation damage is that it occurs underground, which may not be detected or accessible for repair. Bridge foundations shall thus be designed to remain elastic as capacity-protected components as far as practicable. The substructure is usually intentionally used as a major means of hysteretic energy dissipation for non-isolated bridges. Brittle shear failure should be avoided in the substructures by applying capacity design strategy and proper detailing. These principles, which were specified in BS EN 1998-2 and adopted by SDMHR 2013, should be adequately considered not only in the design of new bridges but also in the assessment of existing bridges.

Table 5.1 Vulnerable members and possible failure modes

Component	Possible failure modes
Superstructure	Span unseating or pounding caused by large movement (**Brittle**)
Girder-pier connection	Shear failure of bearing or cast-in-place joint (**Brittle**)
Pier	Shear failure (**Brittle)**, or flexural failure (**Ductile**)
Foundation	Shear failure (**Brittle)**, or flexural failure (**Ductile**)

5.2 Seismic Assessment of Existing Bridges

5.2.1 Philosophy

Seismic structural assessment is usually required for non-compliant structures to obtain information on their likely performance so as to facilitate decision making on suitable retrofitting plans. The seismic assessment of existing structures can be conducted in a variety of ways, including the standard force-based assessment, equivalent elastic strength assessment, nonlinear time-history analysis and displacement-based assessment (Priestley *et al.*, 1996).

The most straightforward method of seismic assessment is the standard force-based assessment, which is similar to the strength verification in design of new structures, by comparison of the base shear demand specified by the design code against the estimated base shear capacity. The base shear strength required by the design code is obtained in the usual manner, i.e. reducing the elastic base shear force corresponding to the elastic stiffness of the structure by a code-specified force-reduction or behaviour factor to account for the effect of presumed ductility. The actual assessed base shear strength is then estimated. Comparison of the capacity with the demand will indicate whether the structure is satisfactory. For the case of the capacity being not less than the demand, the structure is satisfactory; otherwise it is unsatisfactory.

The classical approach of using the capacity-demand ratio is primarily based on strength, but no assessment is made regarding the actual displacement or ductility capacity. In view of this, the equivalent elastic strength assessment, relying on a mixed strength and displacement assessment, is a much-improved approach. The characteristic force-displacement response is determined, resulting in an expected strength and displacement capacity. Then, as opposed to the standard force-based assessment, an equivalent elastic strength is derived by multiplying the actual strength by the displacement ductility factor based on the equal displacement approximation and is compared with the elastic base shear requirement for the same elastic stiffness according to the design code. Although this approach still retains elements of the force-based approach, the actual ductility capacity is adopted. However, the method is considered less effective for multi-degree-of-freedom structural systems, as there is hardly any reliable way for including the effects of higher-order modes for determining the displacement capacity. Moreover, owing to the use of displacement-equivalence rule, such as the equal displacement approximation, the accuracy of this method is affected.

At present, the most accurate method for determining the seismic response of an existing structure is by nonlinear time-history analysis. In this method, the effects of higher-order modes and nonlinear properties of components are directly included in the analyses. As a result, it is expected to provide insight into the actual performance of the structure under seismic ground motions and to predict the force and displacement demands at various components more accurately. The procedure of nonlinear time-history analysis is quite straightforward. An appropriate structural model is developed, a suite of spectrum-compatible accelerograms is chosen, the average structural response to the accelerograms is determined, and the critical response parameters are then compared with their respective capacities.

5.2.2 Assessment criteria

(1) Conventional damage index

The damage index (DI) is usually employed to quantify the extent of damage to structures caused by earthquake excitations. Many proposals are currently available to calibrate a DI based on a number of parameters such as deformation, stiffness, energy absorption, etc. The available concepts related to DIs can be divided into two broad categories: non-cumulative DI and cumulative DI.

Non-cumulative DIs are generally simple but they often do not reflect the state of damage accurately because the effects of cyclic loading are not included. The ductility ratio, which is expressed as the ratio of the maximum deformation in the loading time history to the yield displacement, is an example of non-cumulative DI and it is also the simplest available DI.

On the other hand, cumulative DIs are more rational but more complicated than the non-cumulative DIs as they include the effects of cyclic loading. The mechanistic seismic damage model (Park and Ang, 1985) has been widely adopted by researchers up to now, which expresses the seismic structural damage as a linear combination of the damage caused by excessive deformation considering the effect of repeated cyclic loading as

$$\mathrm{DI}=\frac{\delta_{\mathrm{M}}}{\delta_{\mathrm{u}}}+\frac{\beta}{Q_{\mathrm{y}}\delta_{\mathrm{u}}}\int \mathrm{d}E \tag{5.1}$$

where δ_{M} is the maximum deformation under earthquake; δ_{u} is the ultimate deformation under monotonic loading; Q_{y} is the calculated yield strength; dE is the incremental absorbed hysteretic energy; and β is a non-negative parameter. The three parameters included in the damage model shall be calibrated on the basis of extensive monotonic and cyclic test data of RC beams and columns.

A number of modifications to the above damage model of Park and Ang have been developed over time. For instance, Stone and Taylor (1993) used the moment-curvature behaviour and accounted for the effect of recoverable curvature as

$$\mathrm{DI}=\frac{\Phi_{\mathrm{m}}-\Phi_{\mathrm{r}}}{\Phi_{\mathrm{u}}-\Phi_{\mathrm{r}}}+\beta\left(\frac{A_{\mathrm{t}}}{\Phi_{\mathrm{u}}M_{\mathrm{y}}}\right) \tag{5.2}$$

where Φ_{m} denotes the maximum curvature attained during seismic loading; Φ_{u} denotes the ultimate curvature capacity of section; Φ_{r} denotes the recoverable curvature at unloading; M_{y} is the yield moment of section; A_{t} is the total area bounded by the M-Φ loops; and β is a strength deterioration parameter.

The first component in Equation (5.2) is equivalent to the ductility ratio, while the second component is a strength deterioration term that is related to the cumulative normalized energy absorbed by the columns. Besides, Stone and Taylor (1993) proposed three states, i.e. yield, ultimate and failure states, as distinguished by the yielding of longitudinal steel, attainment of ultimate load (moment) capacity and loss of lateral load capacity up to 20%, respectively. The three states form useful delimiters for four possible damage states that might exist in a bridge column following an earthquake, i.e. no damage, repairable, to demolish and about to collapse. Stone and Taylor (1993) estimated the threshold values of the damage index for the three limit states by examining the statistical distribution of calculated values from laboratory tests of 82 spiral-reinforced bridge piers. The tenth percentile of each of the three distributions was found to provide threshold damage index estimates, which are close to the mode and yet are fairly conservative. The tenth percentile threshold index estimates for the three damage states are determined as 0.11, 0.40 and 0.77, respectively.

Hose and Seible (1999) proposed five performance levels for individual components and entire system. The corresponding damage limit states are No Damage, Minor Damage, Moderate Damage, Major Damage, and Local Failure / Collapse. The qualitative and quantitative guidelines for these five performance levels as provided by Hose and Seible (1999) based on results of bridge components and systems tested over 15 years are shown in Table 5.2. The qualitative and quantitative descriptions for these performance levels are based on performance measures that can be observed visually. The first performance level, i.e. Cracking, which corresponds to the No Damage limit state, is described by barely visible hairline cracks that close after a seismic event and require no repair. The second performance level, i.e. Yielding, is defined as the stage when the reinforcement has yielded and it is characterized by cracks that are clearly visible after the seismic event but are less than 1 mm in width, which implies Minor Damage. The third performance level, i.e. Initiation of Local Mechanism, is defined as the onset of inelastic deformation that, depending on the prevalent failure mechanism, consists of development of diagonal cracks or spalling of concrete cover. This performance level is quantified by Moderate Damage with crack widths between 1 mm and 2 mm or lengths of spalled regions greater than 1/10 of the cross-section depth. The Major Damage limit state is correlated with the full development of local mechanism. This performance level can be qualitatively described as the stage when cracks and spalling extend over the full region of the local mechanism region. Quantitatively, cracks greater than 2 mm in width and lengths of spalled regions that extend significantly beyond half of the section depth in the loading direction are commonly observed. The final performance level, i.e. Collapse, is reached when the structural component or system experiences a significant reduction in observed or calculated strength such that the load-carrying capacity of the component can no longer be relied upon. This performance level is reached when buckling of the main reinforcement is initiated, and the hoop or tie reinforcement fails due to anchorage problem or rupture. It can also be caused by crushing of concrete core. For field investigation following a seismic event, this level can be characterized by crack width greater than 2 mm within the concrete core.

Table 5.2 Bridge performance levels and criteria (Hose and Seible, 1999)

Level	Performance level	Qualitative performance description	Quantitative performance description
I	Cracking (Fully operational)	Onset of hairline cracks	Cracks barely visible
II	Yielding (Operational)	Theoretical first yielding of longitudinal reinforcement	Crack widths < 1 mm
III	Initiation of local mechanism (Limited damage)	Initiation of inelastic deformation; Onset of concrete spalling; Development of diagonal cracks	Crack widths 1 - 2 mm; Length of spalled region > 1/10 cross-section depth
IV	Full development of local mechanism (Life safety)	Wide and extended cracks; Significant spalling over local mechanism region	Crack widths > 2 mm; Diagonal cracks extend over 2/3 cross-section depth; Length of spalled region > 1/2 cross-section depth
V	Strength degradation (Collapse)	Buckling of main reinforcement; Rupture of transverse reinforcement; Crushing of core concrete	Crack widths > 2 mm in core concrete

Hose and Seible (1999) adopted three indices that can be evaluated at each performance level: residual deformation index RDI, equivalent viscous damping ratio ξ_{eq} and normalized effective stiffness n_k. The RDI is a non-dimensional index that is obtained by dividing the permanent residual displacement observed at each performance level Δ_r by the ideal yield displacement Δ_y, i.e.

$$\mathrm{RDI}=\frac{\Delta_r}{\Delta_y} \tag{5.3}$$

The parameters necessary for the calculation of the RDI are shown in Figure 5.10. On the other hand, the equivalent viscous damping ratio ξ_{eq} can be calculated based on the equal area approach in terms of parameters E_d and E_s defined in Figure 5.10 as

$$\xi_{eq}=\frac{1}{2\pi}\left(\frac{E_d}{E_s}\right) \tag{5.4}$$

The calculation of ξ_{eq} for cases with symmetric hysteresis loops is also shown in Figure 5.10. In particular, E_d is the area within the inelastic force-displacement response curve, which is a measure of the hysteresis damping or energy-dissipating capacity of the structure, while the hatched region E_s denotes the elastic energy stored in an equivalent linear elastic system. The slope of the equivalent linear elastic system is defined as the effective stiffness K_{eff}. For non-dimensional representation, the effective stiffness parameter is normalized by the initial stiffness K_o to give the normalized effective stiffness n_k as

$$n_k=\frac{K_{eff}}{K_o} \tag{5.5}$$

The parameters necessary for the calculation of the normalized effective stiffness n_k are also shown in Figure 5.10. Some components and systems may experience asymmetric response in the two loading directions under cyclic loading. The concept of taking the average of the push and pull responses can be applied to the determination of the residual displacement index RDI, equivalent viscous damping ratio ξ_{eq} and normalized effective stiffness n_k, as shown in Figure 5.11 and in equation forms, respectively, as

$$\mathrm{RDI}=\frac{1}{2}\left(\frac{\Delta_{r1}}{\Delta_{y1}}+\frac{\Delta_{r2}}{\Delta_{y2}}\right) \tag{5.6}$$

$$\xi_{eq}=\frac{1}{4\pi}\left(\frac{E_{d1}}{E_{s1}}+\frac{E_{d2}}{E_{s2}}\right) \tag{5.7}$$

$$n_k=\frac{1}{2}\left(\frac{K_{eff1}}{K_{o1}}+\frac{K_{eff2}}{K_{o2}}\right) \tag{5.8}$$

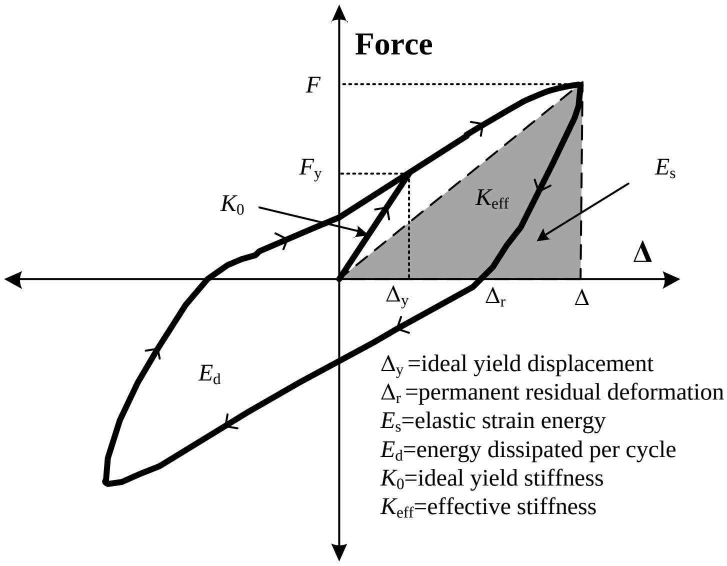


Figure 5.10 Residual deformation index (RDI) and equivalent damping ratio (ξ_{eq}) for symmetric hysteresis loops (Hose and Seible, 1999)

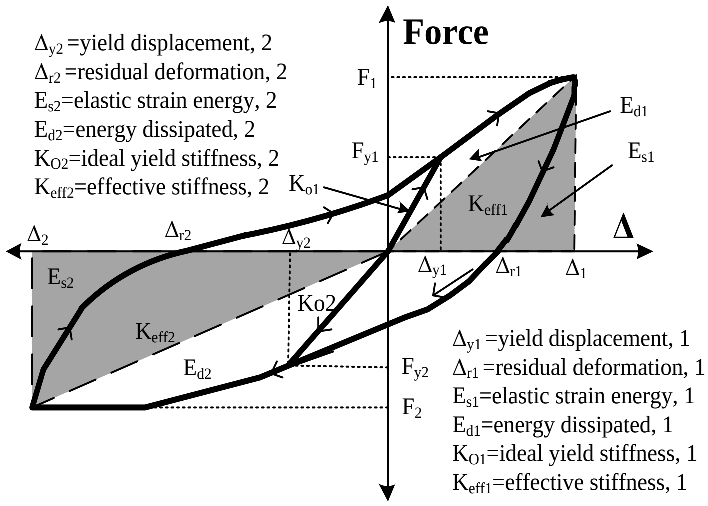


Figure 5.11 Residual deformation index (RDI) and equivalent damping ratio (ξ_{eq}) for asymmetric hysteresis loops (Hose and Seible, 1999)

From the evaluation of results from laboratory tests as well as damage from past earthquakes, the threshold values for these indices at each performance level are estimated as shown in Table 5.3. The indices have been observed to vary depending on the type of failure mode of the structure.

Table 5.3 Threshold values of some indices at selected performance levels (Hose and Seible, 1999)

Behaviour Mode: Brittle					
	I	II	III	IV	V
RDI	< 0.1	0.25	0.5	0.75	1.25
ξ_{eq}	< 8	8	10	11	12
n_k	1.6	1.1	0.8	0.5	0.3
Behaviour Mode: Strength Degrading					
	I	II	III	IV	V
RDI	< 0.1	0.25	0.5	1	1.5
ξ_{eq}	< 8	8	10	13	14
n_k	1.6	1.0	0.5	0.3	0.1
Behaviour Mode: Ductile					
	I	II	III	IV	V
RDI	< 0.1	0.25	0.5	3	5
ξ_{eq}	< 8	8	10	20	25
n_k	1.6	1.1	0.6	0.25	0.175

As observed from the above, there is no universally accepted range for the magnitude of DI. It is now widely accepted that the magnitude of DI should ideally vary between 0 and 1. In other words, a structure suffers no damage when it operates within its elastic limit and hence the DI should be equal to 0 at this stage, while the maximum possible magnitude for DI should be set equal to 1, which denotes the event of total collapse. Obviously, the DIs discussed above have the drawbacks as follows: (a) the magnitude of DI is non-zero when a structure operates within the elastic range; and (b) the magnitude of DI often exceeds 1, i.e. there is no specific upper limit to define the state of collapse.

(2) Strain-based damage index

For RC flexural members such as concrete beams, columns and walls, the strain limits at the material level can be used for the identification of element damage limit states and performance levels. The strain-based damage index has the benefits of cumulative and non-cumulative damage indices. The strain limits for concrete compression and steel tension adopted by Kowalsky (2000) to define selected limit states are shown in Table 5.4. The two limit states considered are "serviceability" and "damage control". According to Kowalsky (2000), "serviceability'' implies that repair is not needed after the earthquake, while ''damage control'' implies that only repairable damage occurs. Consequently, the serviceability concrete compression strain is defined as the strain at which crushing is expected to begin, while the serviceability steel tension strain is defined as the strain at which residual crack widths would exceed 1 mm, thus likely requiring repair and interrupting serviceability. The proposed strain limits for serviceability limit state as shown in Table 5.4 are generally accepted. The damage control concrete compression strain is defined as the compression strain at which the concrete is still repairable. As the ultimate concrete compression strain estimated using the model of Mander *et al.* (1988) is found to be consistently conservative by 50% or more as compared to the test results, Kowalsky (2000) has based the damage control concrete compression strain level on this estimated ultimate concrete compression strain, recognising that failure would not occur until the strain levels increase by at least 50% over the damage control strain. It should be noted that the damage control concrete strain of 0.018 is obtained assuming well-detailed systems typically with the volumetric ratio of transverse reinforcement of around 1% and may not be appropriate for assessment of existing columns with insufficient confinement. The steel tension strain at the damage control level is likely to be related to the point after which incipient buckling of reinforcement will occur during subsequent reversed load cycles. However, there were insufficient data to quantify this limit at that time. Therefore, the steel tension strain at damage control level as shown in Table 5.4 is limited to avoid rupture of reinforcement while allowing for the reduction in strain capacity due to cyclic loading.

Table 5.4 Performance strain limits (Kowalsky, 2000)

Limit State	Concrete Compressive Strain Limit	Steel Tensile Strain Limit
Serviceability	0.004 (Onset of cover concrete crushing)	0.015 (Residual crack widths exceeding 1 mm)
Damage Control	0.018 (Ultimate concrete compression strain estimated according to Mander *et al.* (1988))	0.060 (Avoiding rupture of reinforcement while allowing for reduction in strain capacity due to cyclic loading)

Goodnight *et al.* (2013, 2015, 2016) tested 30 large-scale RC bridge piers with diverse values of axial load, longitudinal steel content, aspect ratio and transverse steel detailing under reversed cyclic loading and real seismic load histories, and made an effort to calibrate the performance strain limits recommended by Kowalsky (2000) through the use of an optical three-dimensional position measurement system to obtain high fidelity strain data of concrete and reinforcing steel. The calibrated performance strain limits are shown in Table 5.5 with major change to the damage control steel tensile strain limit. The peak tensile strain expected to initiate bar buckling in longitudinal reinforcement upon reversal of load was found to be much smaller than 0.060 and an empirical expression was developed based on the dataset.

Table 5.5 Calibrated performance strain limits (Goodnight *et al.*, 2013, 2015, 2016)

Limit State	Concrete Compressive Strain Limit	Steel Tensile Strain Limit
Serviceability	0.004 (Onset of cover concrete crushing)	Not provided.
Damage Control	0.018 (Ultimate concrete compression strain estimated according to Mander *et al.* (1988))	$0.03+700\rho_s\frac{f_{yh}}{E_s}\text{-}0.1\frac{P}{f'_c A_g}$ where ρ_s is the transverse volumetric steel ratio, $\frac{f_{yh}}{E_s}$ is the yield strain of transverse steel, and $\frac{P}{f'_c A_g}$ is the axial load ratio.

Su *et al.* (2017) further proposed damage indices based on the strain-based damage to reinforcing bars and cover concrete. The damage index D_s based on the damage limit states proposed by Hose and Seible (1999) is given in terms of the steel tensile strain ε_s by

$$D_s=\begin{cases}(\varepsilon_s/\varepsilon_y)D_{s1} & \varepsilon_s<\varepsilon_y\\ D_{s1}+[(\varepsilon_s\text{-}\varepsilon_y)/(\varepsilon_{c1}\text{-}\varepsilon_y)](D_{s2}\text{-}D_{s1}) & \varepsilon_y\leq\varepsilon_s<\varepsilon_{c1}\\ D_{s2}+[(\varepsilon_s\text{-}\varepsilon_{c1})/(\varepsilon_{c2}\text{-}\varepsilon_{c1})](D_{s3}\text{-}D_{s2}) & \varepsilon_{c1}\leq\varepsilon_s<\varepsilon_{c2}\\ D_{s3}+[(\varepsilon_s\text{-}\varepsilon_{c2})/(\varepsilon_{bb}\text{-}\varepsilon_{c2})](D_{s4}\text{-}D_{s3}) & \varepsilon_{c2}\leq\varepsilon_s<\varepsilon_{bb}\\ D_{s4}+[(\varepsilon_s\text{-}\varepsilon_{bb})/(\varepsilon_u\text{-}\varepsilon_{bb})](D_{s5}\text{-}D_{s4}) & \varepsilon_{bb}\leq\varepsilon_s<\varepsilon_u\\ D_{s5} & \varepsilon_s\geq\varepsilon_u\end{cases} \tag{5.9}$$

where ε_y denotes the steel yield strain, ε_{c1} and ε_{c2} denote the steel strains corresponding to 1 mm and 2 mm crack widths, respectively, ε_{bb} denotes the peak steel strain prior to bar buckling and ε_u denotes the ultimate steel strain; and the parameters D_{s1} to D_{s5} are damage measures at the limit states, which are set to 0.1, 0.2, 0.4, 0.6 and 1.0, respectively. The damage index D_c is given in terms of the length of spalled region p as a percentage of the cross-section depth by

$$D_c=\begin{cases}(p/p_1)D_{sc1} & p\leq p_1\\ D_{sc1}+[(p-p_1)/(p_2-p_1)](D_{sc2}-D_{sc1}) & p_1<p\leq p_2\\ D_{sc2}+[(p-p_2)/(p_3-p_2)](D_{sc3}-D_{sc2}) & p_2\leq p<p_3\end{cases} \quad (5.10)$$

where the selected damage limit states are determined based on Hose and Seible (1999) with p_1, p_2 and p_3 rounded off to 25%, 65% and 100%, respectively; and the corresponding damage measures D_{sc1}, D_{sc2} and D_{sc3} are set to 0.2, 0.4 and 0.6, respectively. The damage index for the section is then taken as the larger of the concrete and steel damage measures. It should be noted that this damage index varies between 0 and 1.

5.2.3 Performance-based seismic assessment

A variety of performance levels and the associated limit states have been introduced at member-level to evaluate the performance of concrete structures as noted above. Other classifications of performance level are also available. At member-level, Priestley *et al.* (2007) presented five damage limit states: cracking state, first-yield limit state, spalling limit state, buckling limit state and ultimate limit state, while BS EN 1998-3 (BSI, 2005c) specified three limit states, i.e. Damage Limitation (DL), Significant Damage (SD) and Near Collapse (NC), based on chord rotation limits. The strain-based limit states are now widely adopted in performance-based seismic design and assessment of structures. This section provides an overview of the procedures for conducting performance-based seismic assessment based on capacity-demand analysis. This method applies to bridges with piers as major earthquake-resisting components.

(1) Moment-curvature analysis

One of the basic steps for performance-based seismic assessment of structures is moment-curvature analysis. In simple terms, the cross section is discretised into fine strips ("fibre") and the stress in each strip and that in the reinforcement are calculated based on the assumed strain profile that complies with the assumption that plane sections remain plane. Based on the calculated axial force, the strain profile is then updated so that the calculated axial force converges to the applied axial force. The curvature and bending moment at that point are then calculated. The definition of the section is illustrated in Figure 5.12. It is important that the material properties are prescribed separately for the cover concrete, confined core concrete and longitudinal reinforcement. In particular, the compressive properties of confined concrete should take into account the confining effect of as-built transverse steel details and the model of Mander *et al.* (1988) has usually been employed to generate the stress-strain model for confined concrete. At the end of this step, the curvatures at selected limit states when some of the concrete and steel "fibres" have reached the associated strain limits should be determined. For example, in Figure 5.13 that shows the moment-curvature curve of a pier base-section, the curvatures at three limit states, i.e. DL, SD and NC as specified in BS EN 1998-3 (BSI, 2005c), are identified based on steel tensile strain of 0.0025, concrete compressive strain of -0.005 and concrete compressive strain of -0.006, respectively, which correspond to the yielding strain of longitudinal bar, spalling strain of cover concrete and ultimate strain of core concrete in this case.

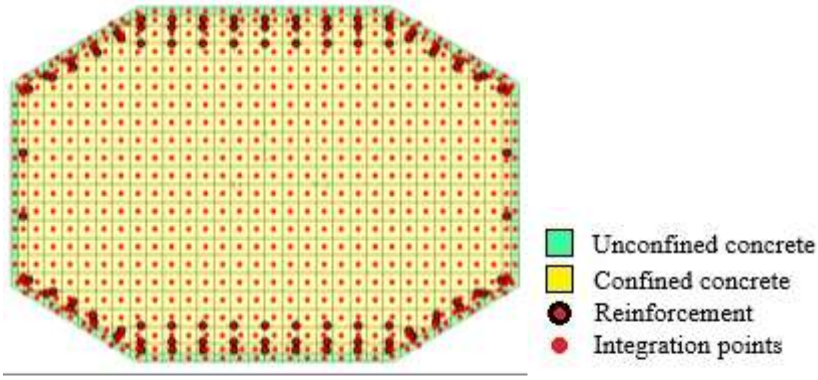


Figure 5.12 The “fibre section” model of a bridge pier

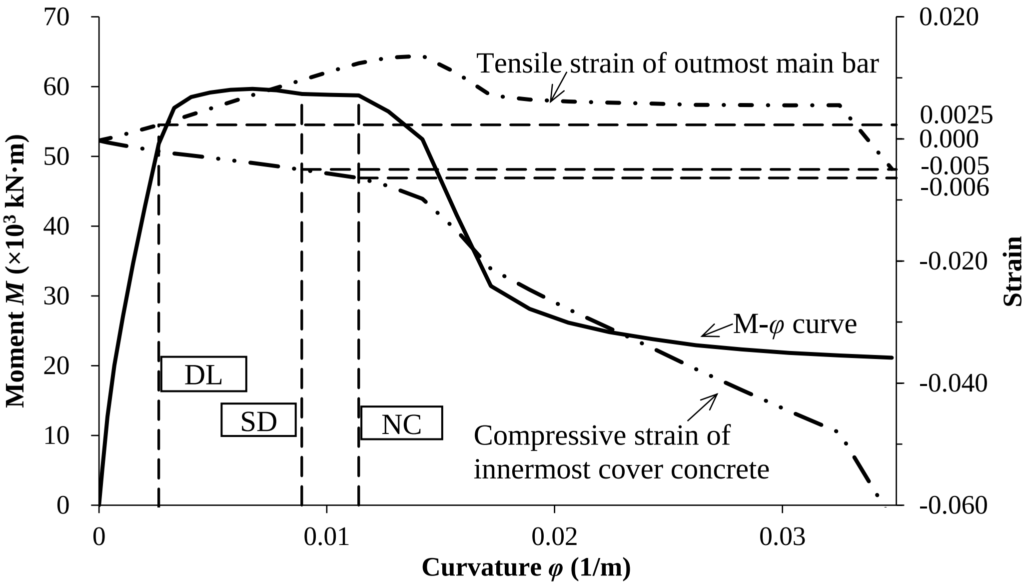


Figure 5.13 Moment-curvature curve of a pier section

(2) Force-displacement response (pushover analysis)

The next step is to relate the selected limit state curvatures to the member response. The main technique currently utilised to convert curvatures to displacements is based on the plastic hinge method presented by Priestley *et al.* (2007). In this method, the elastic and plastic curvature distributions are separated into simplified equivalent shapes, i.e. triangular distribution and uniform distribution, as shown in Figure 5.14. In particular, the plastic curvature is concentrated over a constant height termed the plastic hinge length L_p and the width of the rectangular shape is equal to the actual plastic curvature ϕ_p at the base section. To account for the strain penetration of longitudinal reinforcement into the footing, the curvature distribution is assumed to extend into the footing by a depth termed the strain penetration length L_{sp}.

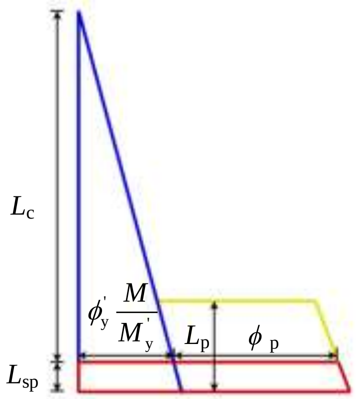


Figure 5.14 Plastic hinge methods (Priestley *et al.*, 2007)

The displacement at the top of column, which is deemed to be fixed at the base, is mainly contributed by the flexural deformation (including both elastic and inelastic components) along the length of the column and fixed-end rotation attributable to the strain penetration. The shear deformation along the length of the column may be neglected as it is commonly much smaller compared with the flexural deformation, especially for columns of aspect ratios greater than 3 (Priestley *et al.*, 2007). Therefore, the displacement at the top of column Δ is obtained in terms of the moment M and curvature ϕ at the base section by

$$\Delta = \Delta'_y \frac{M}{M'_y} + \Delta_p \tag{5.11}$$

$$\Delta'_y = \frac{\varphi'_y \left(H + C_1 L_{sp}\right)^2}{3C_1} \tag{5.12}$$

$$\Delta_p = \left(\varphi - \varphi'_y \frac{M}{M'_y}\right) L_p \left(H + C_1 L_{sp} - \frac{C_1 L_p}{2}\right) \tag{5.13}$$

where Δ'_y denotes the displacement at first yield when the extreme tension reinforcement first attains its yield strain; Δ_p denotes the displacement contributed by flexural plastic deformation including strain penetration; M'_y and ϕ'_y are the moment and curvature at first yield, respectively; H is the effective height of column; and C_1 is a coefficient accounting for end rigidity with $C_1 = 1$ for vertical cantilevers and $C_1 = 2$ for double-bending columns assuming the contraflexure point to be at the mid-height. The plastic hinge length L_p and strain penetration length L_{sp} can be estimated, respectively, by (Priestley *et al.*, 2007)

$$L_p = \frac{kH}{C_1} + L_{sp} \geq 2L_{sp} \tag{5.14}$$

$$L_{sp} = 0.022 f_{yk} d_{bL} \tag{5.15}$$

where f_{yk} and d_{bL} are the yield strength and diameter of main reinforcing bar, respectively; and k is a coefficient associated with ultimate tensile strength / yield strength ratio $\left(f_{tk}/f_{yk}\right)$ of the main reinforcing bar given by

$$k = 0.2\left(\frac{f_{tk}}{f_{yk}} - 1\right) \leq 0.08 \tag{5.16}$$

For the force-displacement curve (also known as the capacity curve here), the lateral force at the top of column F is calculated by

$$F = \frac{C_1 M}{H} \tag{5.17}$$

Furthermore, the flexural response has been found to interact with the shear response (Kowalsky and Priestley, 2000). The shear strength decreases as the flexural plastic deformation increases. Flexural-shear failure may occur and lead to incomplete development of displacement capacity of the column as shown in Figure 5.15. Priestley *et al.* (2007) have also proposed a method for the calculation of shear capacity in this case.

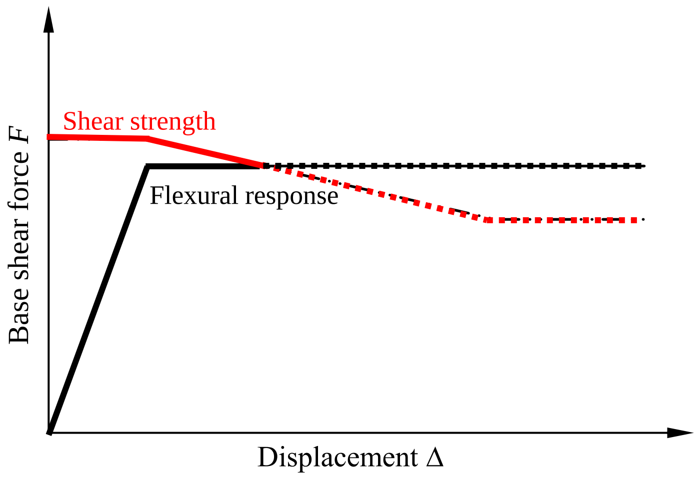


Figure 5.15 Interaction between flexure and shear

(3) Capacity-demand assessment of bridge

As mentioned previously, the earthquake demand for a bridge may be represented by a response spectrum. Both the elastic acceleration spectrum $S_e(T)$ and elastic displacement spectrum $S_{De}(T)$ can be used, but by far the most commonly used is the acceleration spectrum. The elastic displacement spectrum can be obtained by direct transformation of the elastic acceleration response spectrum using (BSI, 2004c)

$$S_{De}(T) = S_e(T)\left(\frac{T}{2\pi}\right)^2 \tag{5.18}$$

These spectra, when scaled by the seismic mass, give the seismic forces acting through the centre of mass of the bridge. To combine the pushover capacity curve and the demand spectrum in a single plot, however, it is convenient to express the spectral acceleration in terms of spectral displacement rather than period T, resulting in the acceleration-displacement response spectrum $S_a(S_{De})$ (also known as the demand curve here). To do so, Equation (5.18) is first written as

$$S_{De}(T) = 2.5 a_g \cdot S \cdot \eta \cdot T_C \cdot \frac{T}{4\pi^2} \tag{5.19}$$

based on the acceleration spectrum defined in BS EN 1998-1 (BSI, 2004c) as

$$S_e(T) = 2.5 a_g \cdot S \cdot \eta \cdot \frac{T_C}{T} \tag{5.20}$$

where a_g denotes the design peak ground acceleration on "rock" site with importance of structure accounted for, S is the soil factor, η is the damping correction factor, and T_C is the upper limit of the period of the constant spectral acceleration branch (BSI, 2004c). Combining Equations (5.19) and (5.20) to eliminate the period T gives

$$S_e = \frac{1}{S_{De}}\left(2.5 a_g \cdot S \cdot \eta \cdot \frac{T_C}{2\pi}\right)^2 \tag{5.21}$$

Figure 5.16(a) shows the spectral acceleration plotted against spectral displacement. It is noted that the probable maximum spectral displacement is solved by setting $T = T_D$ defining the beginning of the constant displacement response range in Equation (5.19).

This spectrum assumes 5% viscous damping in the bridge and it should be modified by η for other damping values. A value of 5% is appropriate for essentially elastic behaviour. However, once yielding occurs, the damping level increases. The effective damping ξ_{eff} is therefore introduced for this purpose, which can be calculated assuming idealized elasto-plastic response as (Kowalsky *et al.*, 1995)

$$\xi_{eff} = \xi_0 + \frac{1}{\pi}\left(1 - \frac{1}{\sqrt{\mu}}\right) \tag{5.22}$$

where $\mu = \Delta/\Delta_y'$ is the ductility factor at the current displacement; and ξ_0 is the damping before the yielding of the bridge, which is taken to be 0.05. The value of the damping correction factor η may be determined as (BSI, 2004c)

$$\eta = \sqrt{\frac{0.1}{0.05+\xi_{eff}}} \geq 0.55 \tag{5.23}$$

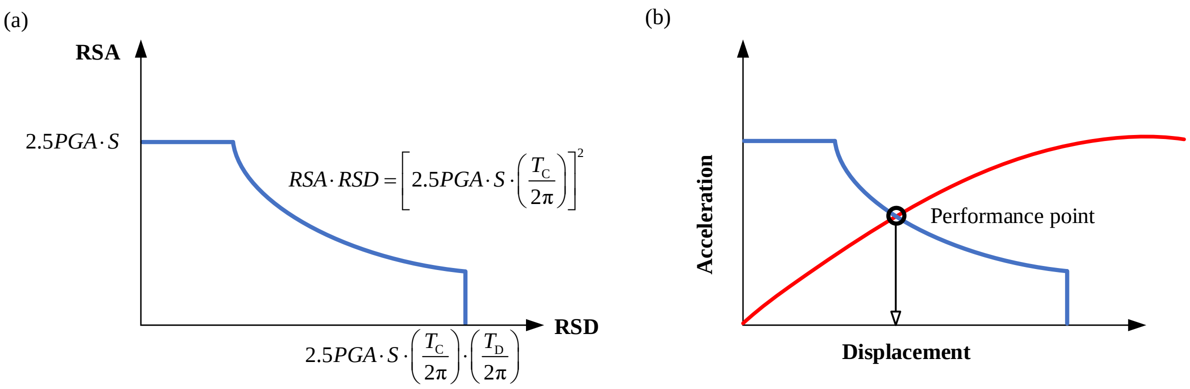


Figure 5.16 Capacity-demand analysis: (a) Acceleration-displacement response spectrum; and (b) Determination of demand

The seismic demand imposed on a bridge by the design earthquake may be determined by the intersection point when its capacity curve is superimposed onto the demand curve as shown in Figure 5.16(b). The seismic performance of the bridge can then be evaluated by comparing the seismic demand with the capacity at selected limit states. The difficulty, however, is that the final displacement is unknown and those damping factors cannot be calculated in advance. Iteration is therefore used, starting with an initial estimate for displacement and iterating until the assumed value and the calculated value are in agreement. The basic steps in this method are listed below:

(1) Start iteration by setting Δ equal to the displacement of the bridge assuming elastic behaviour, and calculate the ductility factor μ;

(2) Calculate the damping factors ξ_{eff} and η;

(3) Calculate the capacity acceleration $a_c = F / M$, where F is the capacity of the bridge at displacement Δ and M is the seismic mass;

(4) Set $S_a = a_c$ and solve for S_{De} in Equation (5.21);

(5) Compare S_{De} with the value for Δ and, if in agreement, go to Step (6); otherwise set $\Delta = S_{De}$ and recalculate μ, and then repeat from Step (2); and

(6) Compare the final Δ with the capacity displacements at various limit states to estimate the likely performance of the bridge.

Numerical Simulation

Two bridges with conditions of Importance Class II and Ground Type C are assessed using the capacity-demand analysis. The structural seismic response of a bridge is mostly related to the fundamental period T, the normalized axial force $\eta_k = N_{Ed} / A_c f_{ck}$ expressed in terms of the axial force N_{Ed}, the gross area of column cross-section A_c, and the characteristic compressive strength of concrete f_{ck}, the aspect ratio $\alpha_s = H / C_1 h$ expressed in terms of the shear span H / C_1 where C_1 has been defined before, and the depth of column cross-section in the direction of flexure h, and the longitudinal and transverse reinforcement ratios ρ_L and ρ_w, respectively. The information of the two example bridges are listed in Table 5.6.

Table 5.6 Information of example bridges

Bridge	Properties of bridge								
	h (m)	H (m)	Connection	C_1	α_s	η_k	T (s)	ρ_L (%)	ρ_w (%)
T026-S095-C058	3.00	5	Fixed-bearing	1	1.7	0.093	0.26	0.95	0.58
T062-S368-C122	1.80	5	Fixed-bearing	1	2.8	0.093	0.62	3.68	1.22

The basic performance requirement for bridges as set out in BS EN 1998-2 (BSI, 2005b) is that the bridge shall retain the structural integrity and possess adequate residual resistance to enable the immediate occupancy of emergency traffic after occurrence of design seismic event and that the damage to some parts of the bridge due to their contribution to energy dissipation shall be easily repairable. A further requirement is that frequent events with much shorter return periods than that of the design event should only cause minor damage and the bridge should remain fully functional after those events. In view of these, the performance requirements as adopted in SDMHR 2013 for bridges are elaborated in Table 5.7.

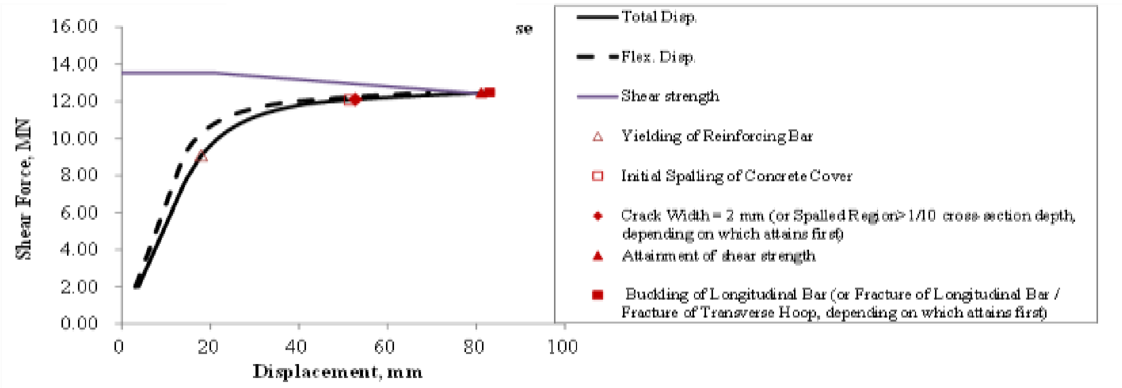
Table 5.7 Performance requirements for bridges based on SDMHR 2013

Return Period (year)	Bridge		
	Importance Class III	Importance Class II	Importance Class I
475	Full service	Full service	Emergency traffic
1000	Full service	Emergency traffic	-
2500	Emergency traffic	-	-

It is known from a number of experimental investigations (Berry *et al.*, 2004; Goodnight *et al.*, 2015; Lehman *et al.*, 2004) that the sequence of damage in RC columns under cyclic excitations is normally as follows: concrete cracking, yielding of longitudinal reinforcement, initial spalling of concrete cover, complete spalling of concrete cover, yielding of transverse reinforcement, buckling of longitudinal reinforcement, fracture of transverse reinforcement, and fracture of longitudinal reinforcement. As either bar buckling, spiral fracture or longitudinal bar fracture will initiate loss of lateral-load resistance and hence prohibit immediate post-earthquake occupancy, these types of damage should technically be avoided and not expected in consideration of the basic performance requirement. The damage limit state for the performance level of restricted service by emergency traffic is taken to be (a) residual crack width of 2 mm, and (b) cover concrete spalled up to 1/10 of section depth by reference to Hose and Seible (1999) and Lehman *et al.* (2004). The corresponding strain limits are defined as follows. The steel tensile strain at which the residual crack width would exceed 2 mm is taken to be 0.02 (Su *et al.*, 2017), and the concrete spalling strain is conservatively taken to be the crushing strain of -0.004.

The results of capacity-demand analysis for Bridges T026-S095-C058 and T062-S368-C122 are shown in Figure 5.17 and Figure 5.18, respectively. Selected damage limit states and the associated performance levels are marked on the capacity curves. Figure 5.17(a) and Figure 5.18(a) show that the lateral displacement is mostly from flexural displacement with little shear displacement even for the columns studied with aspect ratios below 3. Figure 5.17(a) also shows that the displacement capacity of Bridge T026-S095-C058 depends on the flexural ductility, while that of Bridge T062-S368-C122 is limited by the shear capacity well before the full development of flexural ductility as shown in Figure 5.18(a). The demand curves as shown in Figure 5.17(b) and Figure 5.18(b) are determined with $a_g = 0.168g$, $S = 1.5$, $T_C = 0.25$ and $T_D = 1.2$ for bridges of Importance Class II and Ground Type C in Hong Kong. They show that the responses of both bridges under the design earthquake are within the elastic stages and they are likely to remain in full service after the event.

(a) Force-Displacement Capacity



(b) Capacity vs. Demand

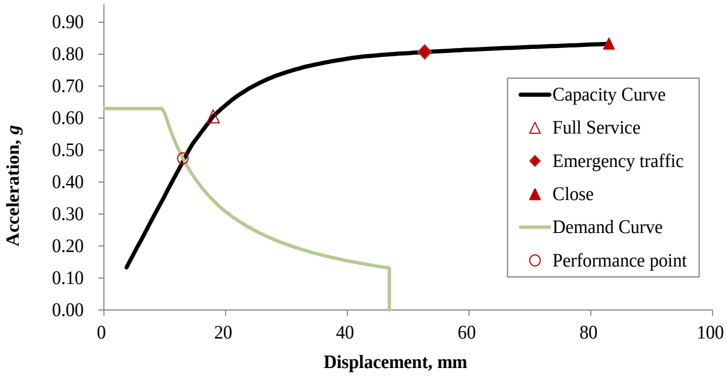
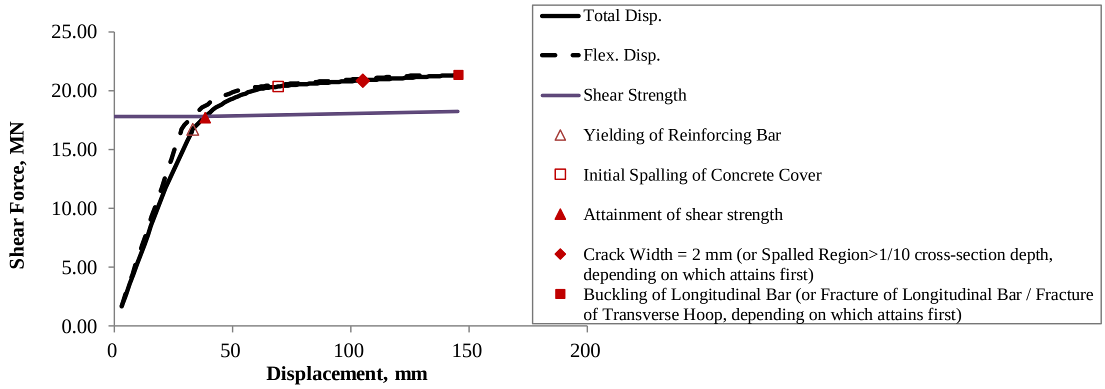


Figure 5.17 Capacity-demand analysis for Bridge T026-S095-C058

(a) Force-Displacement Capacity



(b) Capacity vs. Demand

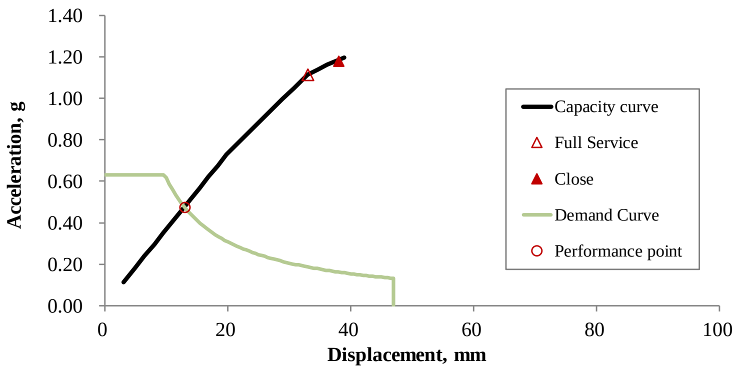


Figure 5.18 Capacity-demand analysis for Bridge T062-S368-C122

5.2.4 Nonlinear time-history analysis

Although nonlinear time-history analysis is quite time-consuming and computationally demanding, it is often preferred for seismic assessment of bridges because it can provide insight into the performance of various bridge components during earthquake including the piers, bearings and shear keys. The use of nonlinear time-history analysis for assessing existing bridges is not much different from that for the design of new bridges, except that the existing bridges may exhibit highly nonlinear behaviour in piers due to impact at movement joints and that some unique details may also need further attention. Some recommendations are presented below.

- For bridge piers under combined compression and bending, which may exhibit significant material nonlinearities, the fibre-based approach is normally implemented with concrete and individual longitudinal bars modelled separately. The confining effect provided by the transverse reinforcement for core concrete may be ignored for the existing bridge piers due to potentially inadequate confinement.
- Bearings are usually modelled using link elements. Since laminated elastomeric bearings used to be installed without effective connection to the piers and girder, the relative frictional sliding between the bearing and the contact surfaces can occur in case of moderate-to-large earthquakes. For unbonded elastomeric bearings, the bilinear hysteresis curve as shown in Figure 5.19 is adopted, which is defined by the elastic stiffness and yield force. The elastic stiffness k_e can be calculated by (Choi, 2002):

$$k_e = GA/h_r \tag{5.24}$$

where G is the shear modulus of the elastomer, and A and h_r are the plan area and thickness of the elastomeric pad, respectively. The yield force F_f that denotes the frictional force at the initiation of sliding can be calculated by

$$F_f = \mu_e N \tag{5.25}$$

where N is the normal force acting on the bearing, and μ_e is the coefficient of friction between the elastomer and the concrete surface, which depends on the contact pressure σ_m and may be estimated in terms of the average of compressive stress σ_m in MPa by (BSI, 2005a)

$$\mu_e = 0.1 + 0.9/\sigma_m \tag{5.26}$$

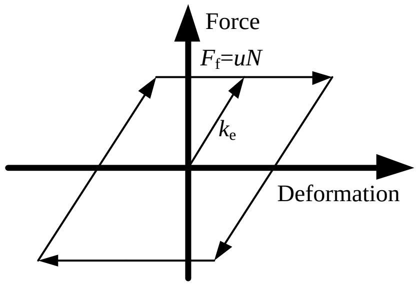


Figure 5.19 Model for the unbonded laminated elastomeric bearing with and without sliding bearing

- At the supports near expansion joints, the elastomeric bearings were often used together with sliding bearings in order to accommodate large deck movements. The sliding bearings usually consist of a low-friction polymer, such as PTFE, sliding against a metal plate. The combined bearing can also be modelled using the bilinear mechanical model used for elastomeric bearings as shown in Figure 5.19. The coefficient of friction μ_p used to determine the yield force for sliding bearings, however, is obtained in a different way due to the distinct sliding surfaces. According to Annex B of BS EN 1337-2 (BSI, 2005a), the recommended design coefficients of friction between PTFE and stainless steel is given in terms of the PTFE contact pressure σ_p in MPa by

$$\mu_p = 1.2/(10+\sigma_p) \tag{5.27}$$

- The steel dowels are simulated as suggested by Vintzeleou and Tassios (1987) using a multi-linear hysteresis model as shown in Figure 5.20. The characteristic stiffness k_i (i = 1, 2, 3 and 4) of the model can be obtained by finite element analysis of a three-dimensional model of the steel dowel. The values as shown in Figure 5.20 are obtained for dowel bars of 50 mm diameter and 75 mm clear height.

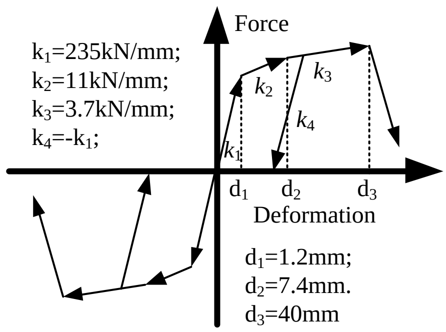


Figure 5.20 Model for steel dowel

- The pounding between the girder and the pier crosshead or the girder and the shear key can be modelled using a compression-only link element. The pounding can be simulated by the Kelvin model (Jankowski *et al.*, 1998) plus a gap g_p to represent the expansion joint as shown in Figure 5.21. This model consists of a linear spring in conjunction with a damper element that accounts for energy dissipation during the impact. The relationship between the impact force F_c and the displacement d can be expressed as

$$F_c = k_k(d-g_p) + c_k\dot{d} \qquad \text{for } d-g_p > 0 \tag{5.28}$$

$$F_c = 0 \qquad \text{for } d-g_p \leq 0 \tag{5.29}$$

where k_k is the pounding stiffness of the Kelvin model, which can be taken as the axial stiffness of the girder; and c_k is the damping coefficient. The damping coefficient c_k is related to the coefficient of restitution e by considering the energy losses during the impact as

$$c_k = 2\xi\sqrt{k_k\frac{m_1 m_2}{m_1+m_2}} \tag{5.30}$$

$$\xi = \frac{-\ln e}{\sqrt{\pi^2+(\ln e)^2}} \tag{5.31}$$

where m_1 is the mass of the girder for the span; m_2 is the effective mass of the pier, which is taken to be half of the mass of the pier; ξ is the equivalent viscous damping for energy dissipation; and the coefficient of restitution e is 0.65 for concrete material (Jankowski *et al.*, 2000).

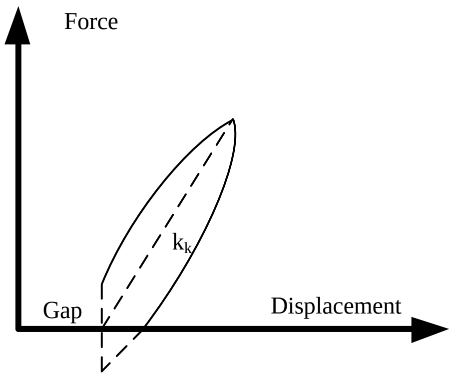


Figure 5.21 Model for pounding

Example

The assessment of an MSCB bridge of Importance Class II using nonlinear time-history analysis is shown here. The superstructure is a twin-cell box girder that is continuous over six spans of 34 m each. The superstructure is supported by Y-shaped piers on bearings. In particular, fixed bearings are used on the central support and sliding bearings are used on the two end supports adjacent to expansion joints. Elastomeric bearings are used in conjunction with RC shear keys in the transverse direction over the rest of the supports. The piers have about the same height of 10 m.

In this study, SAP2000 is used to perform the nonlinear time-history analysis on the three-dimensional line beam model of the bridge model as shown in Figure 5.22. The piers are numbered #1 to #7 from the left end, with Pier #4 connected to the deck through the fixed bearings. In particular, the superstructure is modelled with elastic beam-column elements since it is expected to remain in the elastic stage in its inertial response, while nonlinear fibre beam-column elements are employed over the potential plastic hinge zones in the piers. The superstructure, bearings and piers are tied together using two inclined and two horizontal rigid link elements at each of the connections. Two other vertical link elements are defined to simulate the bearings. These links are fixed in the vertical direction. In the horizontal directions, the links may be either fixed in the cases of fixed bearings or defined with proper hysteresis behaviour depending on the elastomeric and sliding bearings. The foundations are not included in the present model. The piers are thus fixed in all translational directions at the bottom. The effect of the soil is taken into account in terms of 7 spectrum-compatible ground motions for each of the five ground types A to E as specified in BS EN 1998-2, which result in a suite of 35 ground motions.

The responses of various bridge components and of the whole bridge are summarized and evaluated below.

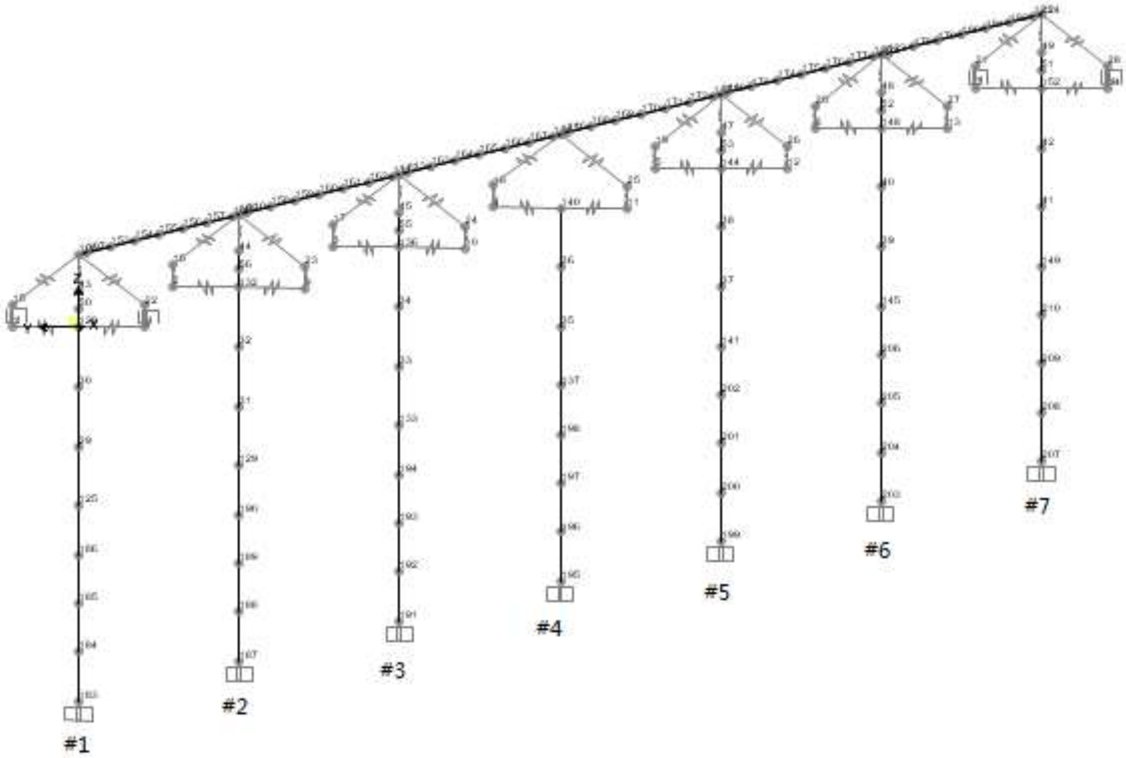


Figure 5.22 SAP2000 line beam model for the bridge

(1) Examination of the bearings

The fixed bearings used include one D3T 1250 and one D3E 1000. The total maximum allowable horizontal force of the fixed bearings is 1600 kN. The shear demand, to be compared to the capacity, is based on the average of the shear forces in the fixed bearings from the analyses. The shear demand due to the ground motions corresponding to ground type A is obtained as 1720 kN, which is taken as the minimum. As a consequence, for the fixed bearings, the seismic shear demand is very likely to exceed its capacity and cause failure.

The maximum deck movement at the elastomeric bearings from all the analyses is found to be 0.045 m in the longitudinal direction of the bridge, occurring at the bearing on Pier #6. Given that the thickness of this bearing is 0.134 m, the equivalent shear strain is obtained by dividing the lateral deflection by the thickness as 0.34. The shear strain of the elastomeric bearing does not exceed the maximum permissible strain of 1.00 as defined in BS EN 1337-3 (BSI, 2005a).

The maximum deck movement at the expansion joint is estimated to be 0.065 m. As the length of the sliding bearing is 0.5 m, the movement capacity is evaluated as a quarter of the length as 0.125 m, which is able to accommodate the seismic displacement.

(2) Examination of possible damage of concrete shear key

For the superstructure supported on elastomeric bearings, the restriction of movement in the transverse direction of the bridge is often achieved by means of RC shear keys against the girder. The keys are thus subjected to shear forces and bending moments in seismic events. The average of the shear forces from analyses with ground motions based on ground type D is used to obtain the probable maximum shear demand on the key. The maximum shear demand is then found to be 545 kN acting on the key at Pier #5. The shear capacity of the keys is estimated to be 840 kN. The keys can therefore resist the seismic shear without failure.

(3) Examination of base shear

Owing to the fixed bearings, Pier #4 carries a large portion of the total seismic shear in both longitudinal and transverse directions of the bridge. The average values of the base shear forces at Pier #4 from analyses based on ground type D are obtained as 2530 kN in the longitudinal direction and 2590 kN in the transverse direction. The corresponding shear capacities of Pier #4 are calculated as 5680 kN and 6610 kN, respectively, in accordance with Clause A.3.3.1 of BS EN 1998-3 (BSI, 2005c), which are adequate to resist the design seismic base shear forces.

(4) Examination of possible formation of plastic hinges

The flexural response of Pier #4 should also be examined. The most unfavourable stress-strain response of concrete at the base section of Pier #4 is shown in Figure 5.23, giving the maximum compressive strain of concrete of about 0.0014, which is below the limiting strain of onset of concrete crushing as suggested by Kowalsky (2000). Besides, the maximum tensile strain of steel reinforcing bar is found to be 0.0016, not even attaining the yielding strain of the reinforcing steel. In other words, the bridge is unlikely to suffer visible damage after the design events.

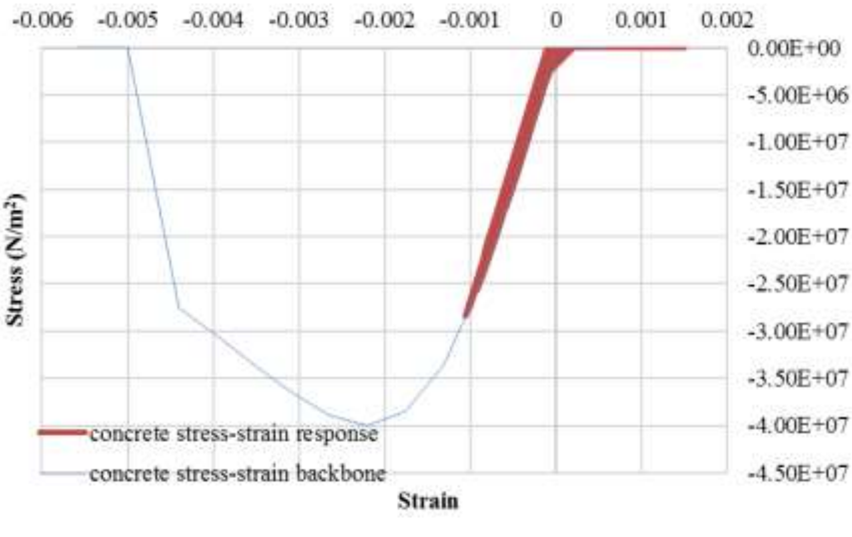


Figure 5.23 Most unfavourable stress-strain response of concrete fibre at Pier #4

(5) Summary

In view of the assessment results of various bearings and piers, the bridge will experience shear failure of fixed bearings when subjected to the design seismic actions specified in SDMHR

2013 for highway structures of Importance Class II. This work may be refined by considering the effects of lap-splice and material deterioration.

5.3 Seismic Fragility Analysis

There are a variety of uncertainties associated with the seismic hazard and properties of the bridge. Thus a probabilistic model is usually more suitable than a deterministic model for seismic risk assessment of bridges. The seismic fragility curves are emerging tools that have been adopted for the development of probabilistic assessment of bridges under earthquake hazards. Fragility curves define the conditional probability, i.e. the likelihood of a structure being damaged beyond a specific damage level for a given ground motion intensity. It has become a useful tool for assessing the vulnerability of a class of bridges in terms of the probability of damage over a range of potential earthquake ground motion intensities. This characterization can be used to optimize bridge retrofitting programmes and develop pre-earthquake action plans as proposed by some bridge design codes (FHWA, 2006).

Fragility analysis can be either empirical or analytical. Empirical fragility curves are usually established by relating the reported and/or observed bridge damage states from past earthquakes to the recorded ground motion intensities. For example, empirical fragility curves have been developed for bridges in California following the 1989 Loma Prieta and 1994 Northridge earthquakes (Basoz *et al.*, 1999; Mander and Basöz, 1999) and in Japan following the 1995 Kobe earthquake (Yamazaki *et al.*, 1999). However, owing to differences in ground motions and bridge characteristics in different regions, empirical fragility curves developed in these regions are generally not applicable to bridges elsewhere. In Hong Kong, since data are lacking for the development of empirical bridge fragility curves, analytical fragility curves are derived instead. The method for deriving analytical seismic fragility curves are presented followed by the development of analytical seismic fragility curves for MSSBs and MSCBs in Hong Kong.

5.3.1 Methodology for fragility analysis

Analytical fragility curves are developed based on the seismic response data from the analyses of bridges by a fragility function. This function defines the conditional probability of the seismic demand (D) placed upon the structure exceeding its capacity (C) for a given level of ground motion intensity (IM) as

$$\text{Fragility} = P\,[D \geq C \mid \text{IM}] \tag{5.32}$$

This evaluation is accomplished by the convolution of the capacity models and the demand models. Normally, in order to describe the uncertainty of the demand, the seismic demand models are assumed to follow a lognormal distribution as

$$\text{P}\,[D \geq d \mid \text{IM}] = 1\text{-}\emptyset\left[\frac{\ln(d) - \ln(S_\text{d})}{\beta_{D|\text{IM}}}\right] \tag{5.33}$$

where $\emptyset(\cdot)$ is the standard normal cumulative distribution function, S_d is the median value of the seismic demand, and $\beta_{D|\text{IM}}$ is the logarithmic standard deviation of the demand conditioned on the IM.

The capacity of the bridge is defined in terms of limit state capacities for various damage states of the bridge component. Similar to the demand models, the capacity models are also described by a two-parameter lognormal distribution with median value S_c and logarithmic standard deviation β_c.

Having described the demand and capacity models, the component fragility as expressed in Equation (5.32) can be computed with a closed-form solution in lognormal form as

$$\text{Fragility} = \emptyset\left[\frac{\ln\left(\frac{S_d}{S_c}\right)}{\sqrt{\beta_{D|\mathrm{IM}}^2+\beta_c^2}}\right] \tag{5.34}$$

This closed-form solution results in fragility curves for component-level failure probabilities. The fragility of the full bridge system can then be assessed. A common assumption is that a bridge damage state is reached if any of the components exhibits the associated level of damage. Assuming that all the components are statistically independent (Choi *et al.*, 2004), it is appropriate to take the upper bound estimate of the system fragility as a conservative estimate on the overall bridge fragility, giving the joint probability of failure of the entire system $P(F_{sys})$ as

$$P(F_{sys})=1-\prod_{i=1}^{m}[1 - P(F_i)] \tag{5.35}$$

where $P(F_i)$ is the probability of failure of the i-th component.

The development of seismic fragility curves for some typical existing highway bridges in Hong Kong is introduced. The procedures adopted are described as follows:

- Classify typical highway bridges in Hong Kong based on the structural types and other details critical to seismic performance.
- Establish appropriate bridge samples representative of their respective bridge classes, considering geometric uncertainties and material uncertainties.
- Generate a suite of earthquake inputs, which cover various levels of ground shaking intensity in terms of the peak ground acceleration (PGA).
- Randomly pair the earthquake inputs with the bridge samples to establish a set of bridge-motion pairs for each bridge class.
- Establish a nonlinear finite element model of bridge and perform time-history analysis for each of the bridge-motion pairs to obtain the maximum demand placed on various components.
- Perform a regression analysis on the simulated response data to determine the probabilistic characteristics of structural demand as a function of the PGA.
- Define component damage states based on repair-related decision and establish the probabilistic characteristics of structural capacity corresponding to each damage state.
- Compute the conditional probabilities that the structural demand exceeds the structural capacity for various levels of ground shaking and plot the component-level fragility curves as a function of the PGA.
- Determine the bridge fragility curves based on the component-level vulnerability.

5.3.2 Seismic fragility analysis of existing bridges in Hong Kong

(1) Bridge classifications

The success of fragility analysis depends largely on the effectiveness of bridge classification based on a limited number of parameters that can provide a reliable description of the bridge characteristics within each portfolio or class. Two prevalent classes of bridges in Hong Kong, i.e. the MSSB and MSCB bridges depending on the continuity of superstructures as depicted in Figure 5.1, are studied for their fragilities. Besides, the variations in significant geometric characteristics should be accounted for, including the number of U-beams and the bent type for the MSSBs, and the principal structural girder-pier connection type and the number of spans for the MSCBs. To this end, ten basic bridge samples with representative geometric configurations are prescribed for each of the two bridge classes as presented in Table 5.8.

Table 5.8 Representative bridge geometry

MSSB Sample #	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**
Nos. of U-beams	4	4	6	6	9	9	11	11	13	13
Elastomer dimensions (*longi.* mm × *trans.* mm × h_r mm)	355 × 610 × 60									
Bent type	Single-column bent				Two-column bent					
Column spacing (m)	-	-	-	-	8.2	8.2	12.2	12.2	16	16
Column sectional dimensions (*W* m × *D* m)	2.0× 1.5	2.5× 1.5	2.5× 1.5	3.0× 1.8	2.5× 1.5	3.0× 1.8	2.5× 1.5	3.0× 1.8	2.5× 1.5	3.0× 1.8

MSCB Sample #	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**
Connection type	Fixed bearing (Tetron D3T 1250)					Monolithic				
Elastomer dimensions (*longi.* mm × *trans.* mm × h_r mm)	550 × 600 × 75									
No. of spans	4	4	4	5	5	4	4	4	5	5
Column sectional dimensions (*W* m × *D* m)	2.0× 1.5	2.5× 1.5	3.0× 1.8	2.5× 1.5	3.0× 1.8	2.0× 1.5	2.5× 1.5	3.0× 1.8	2.5× 1.5	3.0× 1.8

(2) Generation of bridge samples

The uncertainty of bridge geometry, material properties and other random variables affecting the seismic response is further explored. Since a large number of random variables may be considered in the fragility assessment of bridge classes, past studies have indicated that some parameters are less significant than the others and may be neglected (Pang *et al.*, 2013; Tavares *et al.*, 2012). Table 5.9 shows the significant parameters considered in this study and their associated distribution parameters. The upper and lower levels for each of these parameters are also selected such that they encompass only the reasonable values for the parameter.

Table 5.9 Significant parameters considered in the fragility analysis

Parameter	Probability Distribution	Mean	Deviation	Lower level	Upper level	Unit
MSSB						
1. Elastomeric bearing stiffness	Uniform [a]	100	50	50	150	%
2. Fixed bearing stiffness	Uniform [a]	100	50	50	150	%
3. Dowel strength	Normal [a]	100	20	80	120	%
4. Deck mass	Uniform [a]	100	10	90	110	%
5. Damping ratio	Normal [a]	0.05	0.03	0.02	0.08	-
6. Gap between beam end and cross-head	Normal [c]	30	6	24	36	mm
7. Pier height (H_1)	Uniform [c]	10.5	3.5	7	14	m
8. Pier height difference (H_1-H_2)	Uniform [c]	1	1	0	2	m
9. Longitudinal steel ratio	Uniform [c]	0.0225	0.0075	0.015	0.03	-
MSCB						
1. Sliding bearing coefficient	Uniform [b]	0.05	0.03	0.02	0.08	-
2. Elastomeric bearing stiffness	Uniform [a]	100	50	50	150	%
3. Deck mass	Uniform [a]	100	10	90	110	%
4. Damping ratio	Normal [a]	0.05	0.03	0.02	0.08	-
5. Span length	Normal [c]	35	5	30	40	m
6. Gap between shear key and girder	Normal [c]	15	15	0	30	mm
7. Pier height (H_1)	Uniform [c]	10.5	3.5	7	14	m
8. Pier height difference (H_1-H_2)	Uniform [c]	1	1	0	2	m
9. Longitudinal steel ratio	Uniform [c]	0.0225	0.0075	0.015	0.03	-

[a] Nielson and DesRoches (2007)
[b] Dolce *et al.* (2005)
[c] Adapted from Nielson and DesRoches (2007) to suit local situations

For each of the geometry samples as listed in Table 5.8, the uniform design method (Fang *et al.*, 2000) is used to generate 30 statistically different and nominally identical bridge samples by sampling on the significant variables as shown in Table 5.9. Thus, 300 bridge samples are generated for each of the bridge classes. Later each of the resulting bridge samples is paired with a ground motion for use in the probabilistic seismic demand analysis.

(3) Limit state capacity estimates

Fragility analysis requires the knowledge of capacities of various bridge components. These capacities are defined in terms of limit state capacities for selected damage states of the bridge components. The damage states may be repair-related decisions. The selected damage states here are qualitatively described as slight, moderate, extensive and complete damage. The descriptions of these damage states are broadly defined as follows (Ramanathan, 2012):

- Slight damage: aesthetic damage of the component occurs, where the associated repair is primarily aimed at restoring the aesthetics.
- Moderate damage: significant repairs are required to restore component functionality.
- Extensive damage: extensive repairs are required to restore component functionality.
- Complete damage: component replacement is likely to be the most cost-effective means to restore component functionality.

The component limit states at each damage state are selected in an effort to maintain consistency across various bridge components, i.e. reaching a particular damage state for one component should have a similar impact on the functional performance of the entire bridge system as reaching the damage state for another component. Estimates of limit state capacities of the existing bridges are quantitatively presented in Table 5.10, in which the individual limit state capacities are characterized by a two-parameter lognormal distribution with the median S_c and dispersion β_c.

The damage states for columns are quantified by the curvature ductility and those described in Table 5.10 are based on tests on columns (Ramanathan, 2012) similar to those found in the existing bridges. Such definitions have taken into account large spacing of transverse reinforcement and lap-splices at the base of columns that are usually found in non-seismically designed columns.

The damage states for the expansion bearings, i.e. elastomeric bearings with or without sliding bearings on top, are associated with deformation. The deformation may be combinations of shear deformation of elastomer and sliding at unrestrained interfaces. The nominal bearing limit of deformation reaching 100% of the thickness of the elastomer (BSI, 2005a) is defined as the slight damage limit state. With further increase in the deformation, frictional sliding starts to occur at these bearings. Large relative displacement can cause problems of instability and unseating, and elastomeric bearings incorporating a layer of PTFE are also likely to suffer PTFE damage due to knife-edge contact of the sole plate trailing edge on the PTFE (Steelman *et al.*, 2015). Experiments have shown that sliding bearings can tolerate deformation up to approximately the size of the bearing prior to unseating, i.e. elastomer plan size in the direction of horizontal motion (Steelman *et al.*, 2015). This is adopted to define the limit state for complete damage of the expansion bearing. The limit state deformations for moderate and extensive damage are defined by intermediate values before the unseating deformation as 25% and 50% of the elastomer plan size, respectively.

The damage states for the fixed elastomeric bearings are based on the numerical simulation results of the steel dowel as shown in Figure 5.20. At a deformation of 7.4 mm, material nonlinearity develops throughout the dowel bar section, which is defined as slight damage. A deformation of 23 mm results in 100% shear strain in the elastomer and also large permanent deformation in the dowel bar. Thus, this is taken as the moderate damage state. At a deformation of 40 mm, dowel fracture would occur. Replacement of new steel retention dowels is likely required in addition to deck realignment, implying an extensive damage state. After the fracture of the steel dowels, the elastomeric bearings have the same unseating problem as do the expansion bearings and it thus warrants a deformation limit for the complete damage state to be the elastomer plan size in the direction of horizontal motion.

The damage states for the fixed pot bearing are based on the design shear capacity. For the Tetron D3T 1250 fixed pot bearing, the damage state ranges from a horizontal load of 900 kN for slight damage to a horizontal load of 1500 kN for complete damage as provided by the manufacturer. The horizontal load capacity of 990 kN at serviceability limit state is defined as moderate damage. A horizontal load of 1200 kN is adopted for extensive damage.

Table 5.10 summarizes the limit state capacities of various bridge components adopted for the existing bridges. The uncertainty associated with component capacities at each of these limit states is characterized by a dispersion value of 0.35 across the components and damage states (Ramanathan, 2012). This value is a particularly good estimate for columns (Berry *et al.*, 2004).

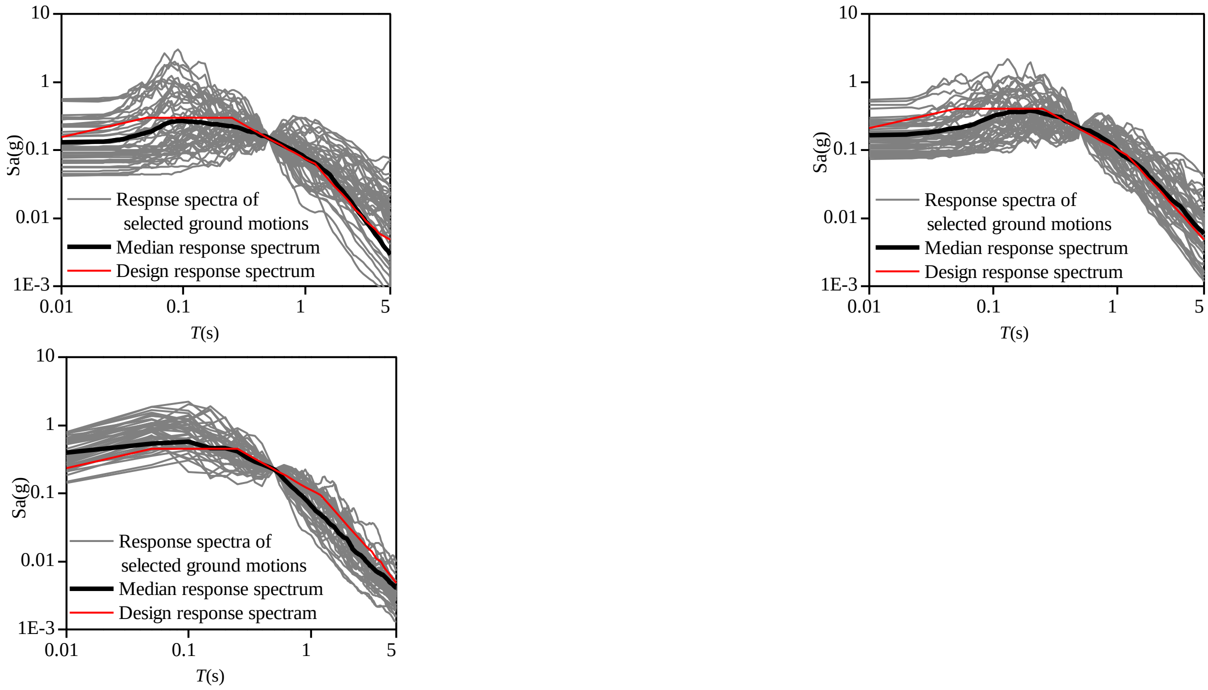
Table 5.10 Definition of damage states for bridge components

Component	Slight damage		Moderate damage		Extensive damage		Complete damage	
	S_c	β_c	S_c	β_c	S_c	β_c	S_c	β_c
MSSB								
Fixed elastomeric bearing, longitudinal (mm)	7.4	0.35	23	0.35	40	0.35	355	0.35
Fixed elastomeric bearing, transverse (mm)	7.4	0.35	23	0.35	40	0.35	610	0.35
Expansion bearing, longitudinal (mm)	60	0.35	90	0.35	180	0.35	355	0.35
Expansion bearing, transverse (mm)	60	0.35	150	0.35	305	0.35	610	0.35
Column (-)	1	0.35	2	0.35	3.5	0.35	5	0.35
MSCB								
Fixed pot bearing (kN)	900	0.35	990	0.35	1200	0.35	1500	0.35
Expansion bearing, longitudinal (mm)	75	0.35	137	0.35	275	0.35	550	0.35
Expansion bearing, transverse (mm)	75	0.35	150	0.35	300	0.35	600	0.35
Column (-)	1	0.35	2	0.35	3.5	0.35	5	0.35

(4) Earthquake inputs

Since there are very few strong ground motion records in Hong Kong available, the Next Generation Attenuation (NGA) database developed by the Pacific Earthquake Engineering Research Center (PEER) (PEER, n.d.) is employed. A suite of 300 ground motion records is selected.

According to the "Final Supplementary Report on the Study for Seismic Actions" (Atkins, 2013), the selected ranges of moment magnitude M_w and epicentral distance are decided to be 5.0 – 8.0 and 10 – 100 km, respectively. Besides, the shear velocity for ground $v_{s,30}$ is taken to be about 180 m/s, covering rock to medium dense soil sites based on ground classifications as prescribed in BS EN 1998-1, which is also typical in Hong Kong. Each of the selected ground motions is scaled so that its spectral acceleration matches the design spectral acceleration at a given period (Baker *et al.*, 2011). This specific period is normally the fundamental period of the structure. Owing to random coupling of ground motions and bridge samples in fragility analysis, the specific period is taken to be 0.5 s after several trials. Figure 5.24 shows that the median response spectra of the selected accelerograms generally agree well with the design response spectra derived in accordance with SDMHR 2013 for the ground types considered at periods above 0.1 s, where the fundamental periods of bridges are most likely to concentrate. Since the selected accelerograms have the magnitudes and source distances consistent with records in Hong Kong and generally good agreement is achieved between the median response spectra and the design response spectra, it is justified to use the selected ground motions for fragility analysis in Hong Kong.



(a) Ground Type A (b) Ground Type B (c) Ground Type C

Figure 5.24 Comparison of response spectra of selected ground motions and the design response spectra (BS EN 1998-2 Type 2 spectra with a return period of 475 years)

(5) Analytical modelling

The 300 bridge samples for each bridge class are then randomly paired with the 300 ground motions to create 300 bridge-motion pairs for each bridge class. A full nonlinear time-history analysis is performed for each bridge-motion pair in the longitudinal and transverse directions separately, and the maximum demand placed on each component is then recorded. In total, 1200 nonlinear time-history analyses are performed for the bridge samples in the two bridge classes considered. The bridges consist of components that may exhibit highly nonlinear behaviour, such as the elastomeric bearings, columns, and the impact between the neighbouring girders and/or the girder and the crosshead. These nonlinearities are incorporated into three-dimensional nonlinear analytical models of the bridge samples as discussed in Section 5.2.4. These models are developed using the OpenSees platform (Mazzoni *et al.*, 2006).

(6) Probabilistic seismic demand models

Probabilistic seismic demand models (PSDMs) are constructed through regression analysis on the computed peak responses of the critical bridge components of the 300 bridge samples for each bridge class. The component responses of concern include the curvature ductility of columns μ_φ, deformation of expansion bearings d_{ex}, deformation of fixed elastomeric bearings d_{fx}, and shear force of fixed pot bearings F_p.

The relationship between the peak component response and the ground motion intensity can be estimated in the form of a power law as (Cornell *et al.*, 2002)

$$S_d = a(\mathrm{IM})^b \tag{5.36}$$

which can be rewritten in logarithmic form as

$$\ln(S_d) = b\ln(\mathrm{IM}) + \ln(a) \tag{5.37}$$

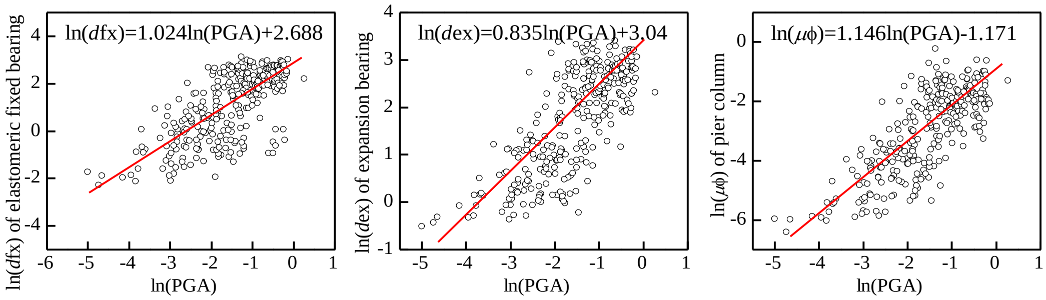
where *a* and *b* are unknown coefficients determined from linear regression in the log space.

The intensity measure adopted for demand modelling in this study is the PGA since it is identified as an appropriate intensity measure for girder bridges (Padgett *et al.*, 2008). In addition to the median value, the uncertainty of the demand model is characterized by a lognormal distribution in which the conditional logarithmic standard deviation is estimated as an average across all PGAs as

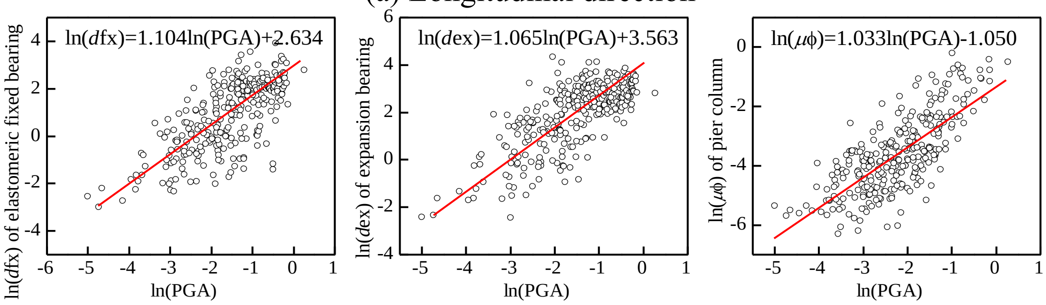
$$\beta_{D|\mathrm{PGA}}=\sqrt{\frac{\sum_{i=1}^{N}(\ln d_i - \ln S_\mathrm{d})^2}{N-2}} \tag{5.38}$$

where N is the number of simulations, and d_i is the peak response of the component of interests from the i-th simulation.

Figure 5.25 and Figure 5.26 show the results of the probabilistic seismic demand analysis for components for the bridge classes MSSB and MSCB, respectively. The component PSDMs for the two bridge classes are summarized in Table 5.11.

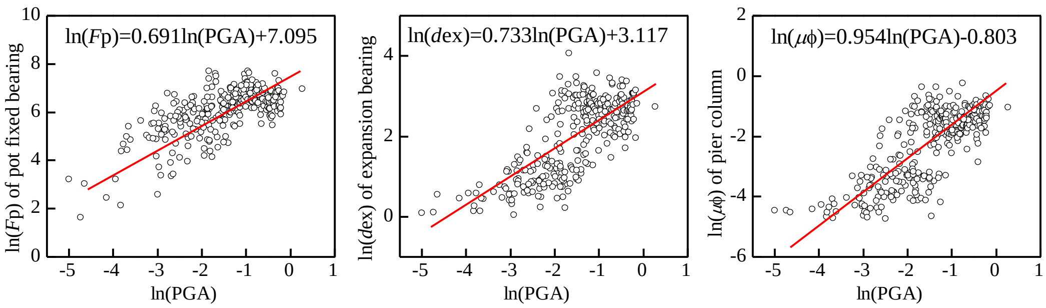


(a) Longitudinal direction

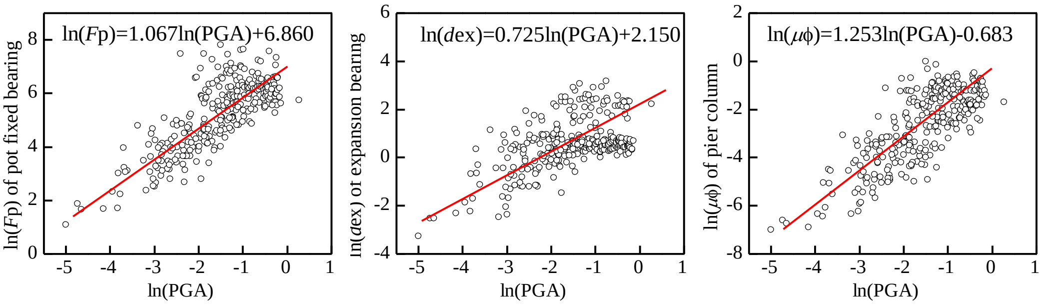


(b) Transverse direction

Figure 5.25 Regression analysis of seismic demands on the bridge components of MSSB



(a) Longitudinal direction



(b) Transverse direction

Figure 5.26 Regression analysis of seismic demands on the bridge components of MSCB

Table 5.11 Estimates of probabilistic seismic demand models

Component	Longitudinal direction			Transverse direction		
	b	$\ln(a)$	$\beta_{D\|\mathrm{PGA}}$	b	$\ln(a)$	$\beta_{D\|\mathrm{PGA}}$
MSSB						
Fixed elastomeric bearing (d_{fx})	1.024	2.688	1.011	1.104	2.634	1.009
Expansion bearing (d_{ex})	0.835	3.044	0.700	1.065	3.563	0.949
Column (μ_{φ})	1.146	-1.171	0.886	1.033	-1.050	0.837
MSCB						
Fixed pot bearing (F_{P}, kN)	0.691	7.095	0.709	1.067	6.860	0.801
Expansion bearing (d_{ex})	0.733	3.117	0.633	0.725	2.150	0.892
Column (μ_{φ})	0.955	-0.803	0.868	1.253	-0.683	0.938

(7) Component fragility curves

The seismic fragility or probability of exceeding a limit state given a PGA is assessed for the four damage states. The probability that the demand on the component will exceed its capacity as expressed by Equation (5.34) can be rewritten by substitution of regression parameters for the demand median as

$$\mathrm{P(LS|PGA)} = \emptyset\left[\frac{\ln(\mathrm{PGA}) - \ln(\lambda)}{\xi}\right] \tag{5.39}$$

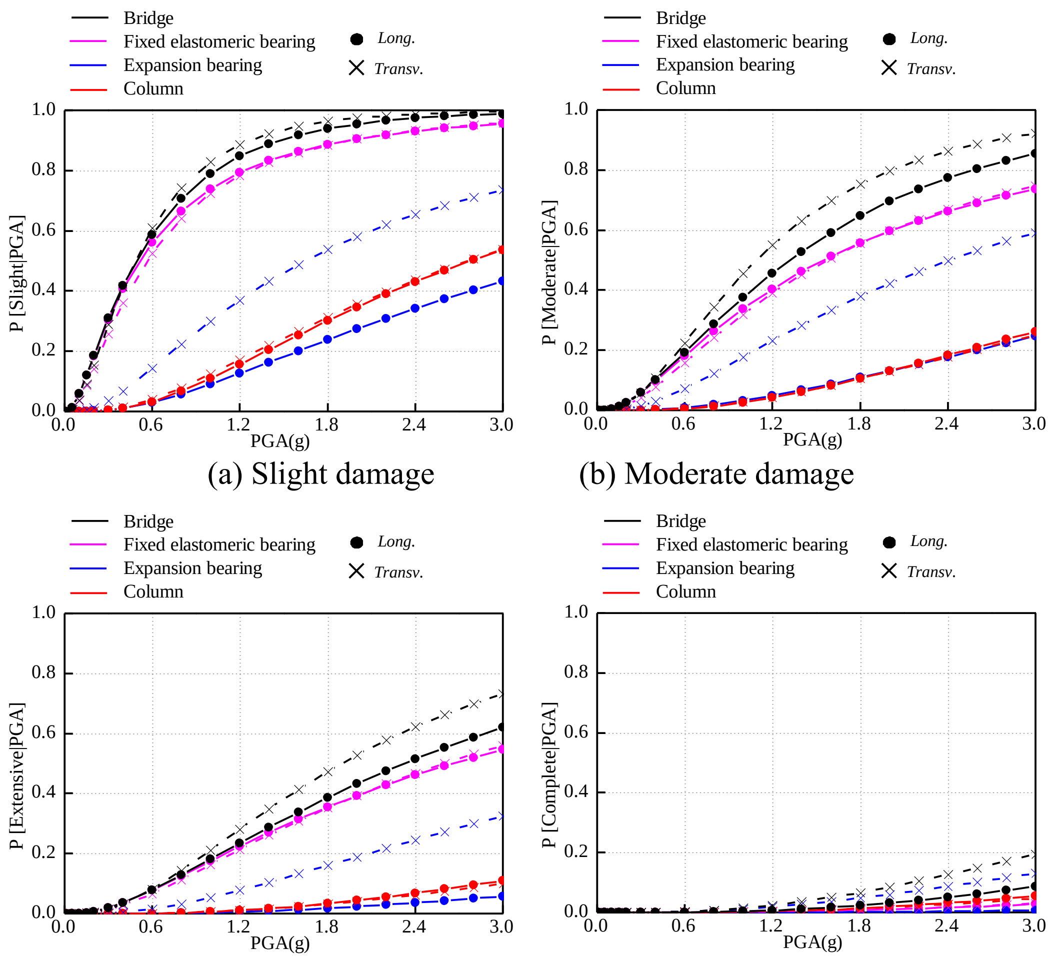
where λ and ξ are the median and dispersion, respectively, of the specific limit state, which are given, respectively, by

$$\lambda = e^{\frac{\ln(S_c) - \ln(a)}{b}} \tag{5.40}$$

$$\xi = \frac{\sqrt{\beta_{D/\mathrm{PGA}}^2 + \beta_c^2}}{b} \tag{5.41}$$

This closed-form analysis results in fragility curves for component-level failure probabilities, which are useful in comparing the relative vulnerability of bridge components within a given class. Figure 5.27 and Figure 5.28 show the component fragility curves for the typical existing MSSB and MSCB bridges at the four damage states, respectively. For the MSSB bridges, the elastomeric fixed bearings are the most vulnerable components at all damage states, including the slight, moderate and extensive damage states, indicating the highest probability of the limit states being exceeded, while the expansion bearings and columns have much lower fragilities. The steel dowels of the fixed bearings have acted as "fuse elements" which are destroyed in the first place, preventing the elastomeric bearings and the substructure from serious damage. For the MSCB bridges, the fixed pot bearings are the most fragile components at all the damage states, followed by the columns and expansion bearings. The fixed pot bearings have significant vulnerability especially in the longitudinal direction. This should be expected as the fixed pot bearings are designed for the main restriction of deck movement and there is thus considerable demand imposed on the bearings, although their failure helps mitigate the force transmitted to the columns thereby resulting in less fragile columns. Overall, the fixed bearings tend to be the controlling fragile components at all damage states for both the existing MSSB and MSCB bridges in Hong Kong.

It is normal to have distinct seismic fragilities for various components of the bridge since they have been designed for multiple load cases and they may thus have diverse redundancies in terms of seismic capacity. Besides, the earthquake-resisting members designed for potential inelastic behaviour will incur damage more easily and are therefore more fragile than those capacity-protected members. It is also desirable to tune the fragility of components in seismic bridge design by assigning sacrificial elements. Forces that can be transmitted to critical structural components can be limited by using yielding elements as “fuses”. If, for example, a column is more fragile and intended to yield by forming plastic hinges, the maximum moment that can be transmitted from the column to the foundation is limited by the yield moments of the hinges. The preferable ductile members should be those that can be easily inspected and repaired following an earthquake and are usually columns, pier walls, seismic isolation and damping devices, bearings, and shear keys. It is advisable to design a bridge system with hierarchical fragility of components based on the complexity and cost of repair or replacement of the components.



(a) Slight damage (b) Moderate damage

(c) Extensive damage (d) Complete damage

Figure 5.27 Component fragility curves for the existing MSSB

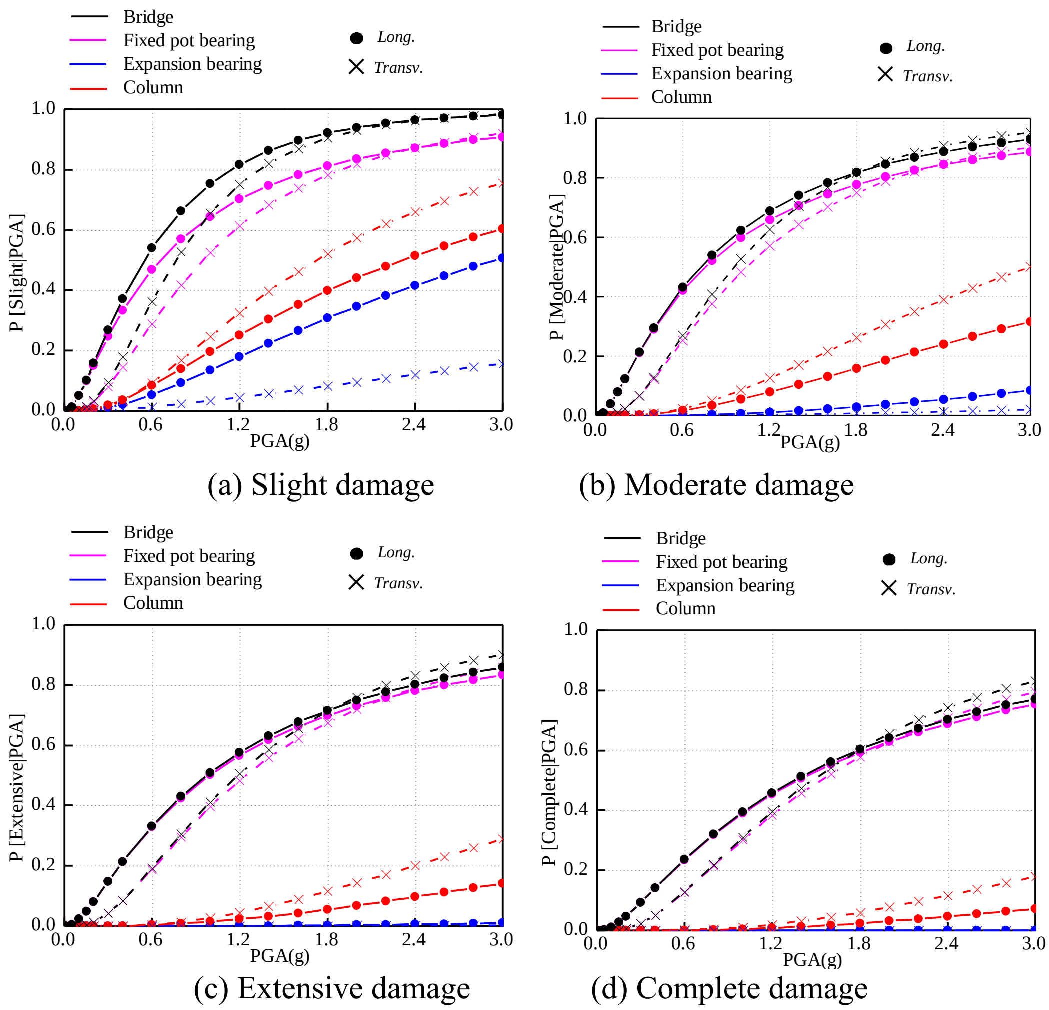


Figure 5.28 Component fragility curves for the exiting MSCB

(8) System fragility curves

To enable comparison of the relative vulnerability of various bridge types, it is essential that the overall bridge fragility be determined. A common approximation method is adopted for the system-level failure analysis, where a bridge damage state is reached if any component exhibits the associated level of damage. As presented in Equation (5.35), the probability that the bridge is at or beyond a particular limit state is therefore the union of the probabilities of each of the components exceeding the same limit state.

The system fragility curves for the existing MSSB and MSCB bridges are also shown in Figure 5.27 and Figure 5.28, respectively. In general, the bridge as a system is more fragile than any of its components. This is a consequence of the underlying series assumption that has been used in formulating these curves. Nevertheless, the transverse fixed bearing response tends to dominate the system fragility in the case of the MSSB, while it is the longitudinal fixed bearing response in the case of the MSCB. For the reference design ground acceleration of 0.12*g* according to SDMHR 2013, the probabilities of exceeding the four damage states for the bridge system for the two bridge classes are tabulated in Table 5.12. The median values of fragility curves for the two bridge classes across the four limit states are also summarized in Table 5.12. These bridges generally show negligible fragility for a PGA of 0.12*g* even at the slight damage limit state. As the ground motion intensity increases, however, the MSCB tends to have a higher fragility than the MSSB. This finding is consistent with those of past studies in other regions, owing to the larger inertial loads from continuous decks which tends to increase demands placed on the key bridge components.

Table 5.12 Bridge system fragility curves for existing bridges

Existing bridges	**Median (unit: *g*)**				**P(LS|0.12*g*) (unit: %)**			
	Slight	Moderate	Extensive	Complete	Slight	Moderate	Extensive	Complete
MSSB	0.66	1.55	2.60	8.75	8.27	0.72	0.15	0.00
MSCB	0.51	0.75	0.99	1.37	2.09	1.25	0.49	0.14

(9) Summary

This section presents the analytical development of fragility curves for two bridge classes commonly found in Hong Kong. Each bridge class was represented by a suite of 300 three-dimensional analytical models subjected to a suite of bi-directional ground motion time-histories for Hong Kong. The resulting probabilistic seismic demand analysis considered the major components including the columns and bearings in assessing the seismic vulnerability. Both component-level demands and capacities were described using lognormal distributions. Fragility curves were derived for the components and the typical Hong Kong bridge systems for slight, moderate, extensive and complete damage limit states defined based on repair-related decisions. The results show that the fixed elastomeric bearings and fixed pot bearings govern the fragility for the typical simply supported and continuous concrete girder bridges, respectively. Representing a bridge system simply by its columns as commonly adopted in the single-degree-of-freedom system approximation is thus likely to underestimate the vulnerability of the bridge. Nevertheless, the fragility curves show that the typical existing bridges in Hong Kong have negligible fragility for ground motions with a peak ground acceleration of 0.12*g*. In cases of greater ground motion intensities, the continuous bridges are more vulnerable than the simply supported bridges, which is consistent with findings in other regions. These curves can be improved if more knowledge of the responses of various bridge components of the real bridges is acquired and additional laboratory testing of similar components as well as input from stakeholders regarding the component capacity limit state estimates are available.

CHAPTER 6 RECENT TREND OF BRIDGE ENGINEERING

Bridge engineering has undergone significant developments towards advanced performance and improved buildability and maintainability during the past decade. This chapter presents a state-of-the-art review of emerging bridge engineering practices particularly in three aspects: material, construction and structural system. Special focus has been placed on the strategies and/or impact that are brought about by these new trends on seismic bridge design and retrofitting.

6.1 Construction Materials

6.1.1 High-strength concrete

High-strength concrete (HSC) refers to concrete with a characteristic compressive strength over 60 MPa. Owing to its high strength, HSC is supposed to significantly reduce the size of component for the same load-carrying capacity and thus reduce the self-weight. As a result, HSC is preferred in tall buildings and bridges with tall piers.

It is necessary to provide not only sufficient strength, but also a minimum level of flexural ductility for steel reinforced concrete (RC) columns in seismic design. For a particular material, very often the ductility decreases when the strength increases. Concrete, of course, is no exception. Figure 6.1 shows the design compressive stress-strain relations for concrete as defined in SDMHR 2013. The figure shows that with the rise of concrete grade, the strain upon reaching the design strength rises sharply while the ultimate strain decreases for concrete grades beyond C60, indicating less ductility for HSC compared with normal-strength concrete.

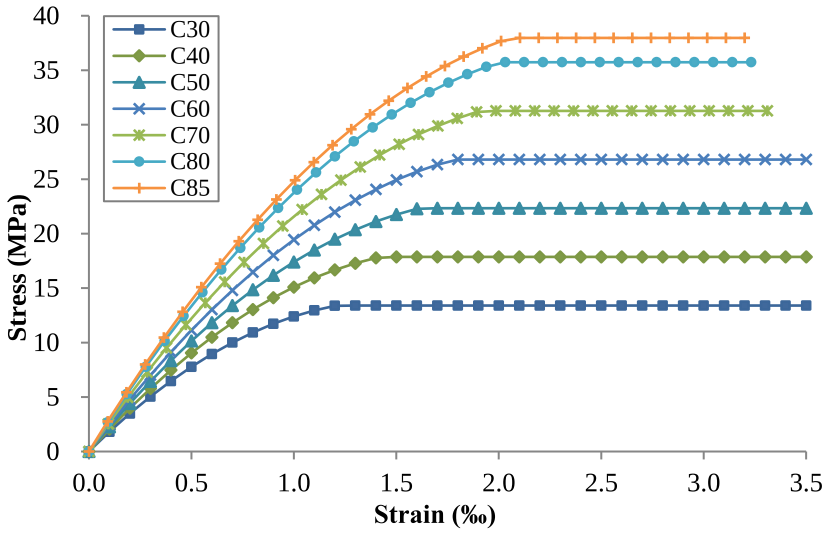


Figure 6.1 Design compressive stress-strain relations for concrete based on SDMHR 2013

In order to explore the flexural ductility of HSC columns, a theoretical study of their full-range flexural behaviour has been carried out based on symmetrical RC columns cast of normal- and high-strength concrete (Bai and Au, 2009). The results indicate that confinement can help HSC column sustain the residual moment up to a much greater curvature at the post-peak stage. Hence, if HSC is used in RC piers, sufficient confining reinforcement should be provided for proper ductility design.

More recently, HSC is often utilised instead of normal concrete in the increasingly popular composite bridge with prestressed concrete flanges and corrugated steel webs as shown in

Figure 6.2. The idea is that the prestressed concrete flanges together mainly carry bending moments, while the corrugated steel webs resist shear without the need for excessive stiffeners. Owing to the "accordion effect" associated with the flexible corrugated steel webs (Jiang *et al.*, 2014), the efficiency of prestressing is much improved. The relative lightness of this type of superstructures will incur less seismic actions during earthquake.

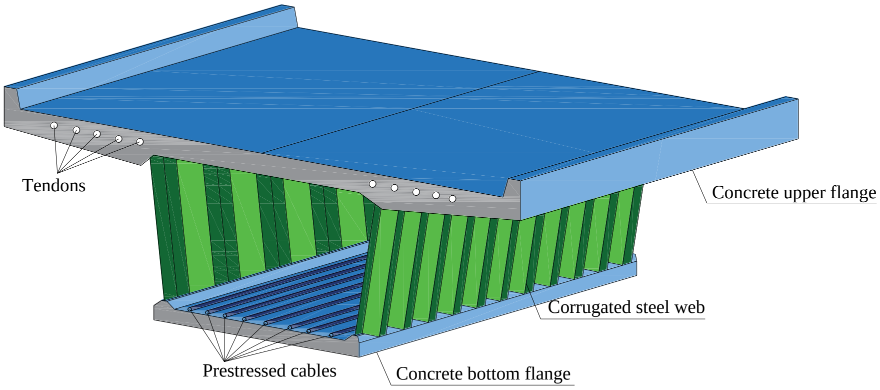


Figure 6.2 Illustration of composite bridge with corrugated steel webs

6.1.2 High-performance concrete

High-performance concrete (HPC) is a relatively new class of concrete, featuring excellent durability, dimensional stability and workability in addition to the high compressive strength. Today, HPC has been widely used in bridges, especially in sea bridges which have stricter requirements for durability. Examples include Confederation Bridge in Canada (Dunaszegi, 2005) and the newly built Hong Kong - Zhuhai - Macao Bridge in South China (Zhao and Li, 2015).

In the event of seismic design situation, ordinary HPC also suffers from low ductility. As an alternative to the common practice of relying on confining steel reinforcement, some special types of HPC with high ductility have been developed.

Engineered Cementitious Composite (ECC), also called "bendable concrete", is a kind of concrete with a matrix reinforced by selected short fibres such as polymeric fibres. As opposed to materials based on ordinary Portland cement with inherent brittle nature, ECC exhibits metal-like tensile strain-hardening property with ultimate tensile strain capacity in excess of 4% (Li, 2003). ECC is most suitable for shear-vulnerable elements (Li, 2003) such as short columns, beam-column joints, in-filled walls, etc., making ECC a very promising construction material for seismic regions. As an alternative to traditional movement joints, ECC has been used in the link slab for jointless bridges (Lepech and Li, 2009). The successful implementation of this strategy also relies on the ample tensile strain capacity of ECC to accommodate the long-term and thermal deformation.

Ultra-high-performance concrete (UHPC) is an upgrade of HPC mixed with steel fibres. It differs from the normal- or even high-strength concrete in compression by its strain hardening behaviour before failure. In addition, it possesses as good tensile resistance and tensile ductility as ECC. Like ECC, UHPC is very damage tolerant and capable of absorbing energy, and thus it is favourable in seismic regions.

6.2 Accelerated Bridge Construction

In parallel with the economic developments in many countries, the volume of traffic has increased so much that the majority of bridges, especially those in urban areas, are now carrying more vehicles and higher loading than they have been designed for. This has placed an increasing demand for upgrading their capacities. In addition, many of them are close to their design lives and the ageing problems are matters of concern. Tremendous resources are needed for repairs, rehabilitation and replacement of existing bridges. However, the biggest challenge for construction activities in congested urban areas like Hong Kong is to minimise the impact on the motorists and neighbourhood due to lane closures, reduced speed zones, noise and air pollution. It is apparent that shortening the construction time is a key factor in improving the buildability of bridges. A relatively new idea for bridge construction, namely Accelerated Bridge Construction (ABC), might be an option. The core of ABC is to save as much as possible the on-site construction time needed when building new bridges or replacing and rehabilitating existing bridges, through the use of innovative planning, design, materials and construction methods in a safe and cost-effective manner (Culmo, 2011).

6.2.1 Prefabrication

Prefabrication is an important method for ABC. A major advantage of prefabrication is that it can reduce the on-site construction time, resulting in reduction of lane closures and traffic detours. Prefabrication can also improve the quality of bridge elements and systems since they are constructed in a controlled environment using high quality materials and standardized production processes. Although there can be a cost premium for using prefabrication during construction, improved quality leads to an extension of service life and reduction in life-cycle costs. Other benefits that can justify the additional costs of prefabrication include improved safety. By reducing the amount of construction that takes place on site, the duration over which construction workers and passers-by are exposed to hazards on construction site is also reduced. With more extensive use of prefabrication, contractors and fabricators will become more proficient in the techniques, thereby boosting the efficiency and economy of prefabrication. Indeed, the past few decades have witnessed the rising trend in adoption of prefabrication in bridge building.

Prefabrication is actually not a new concept. The vast majority of existing bridges today have employed some degree of prefabrication. Steel girders and pretensioned concrete beams are some of the most common prefabricated elements in typical bridges. The techniques of prefabrication of steel bridges are well established and documented (Hayward *et al.*, 2002), but continuous efforts have been spent on prefabrication of concrete bridges for many decades. Precast concrete deck panels have been used for many years. The use of precast segmental construction of box girder bridges in conjunction with post-tensioning has now spread throughout the world since its first appearance in 1960s in France and has made substantial contribution to the continual evolution of prestressed concrete bridge construction. Precast concrete segmental bridge decks have been most commonly used in recent decades for accelerated bridge construction projects due to several distinct advantages. First and foremost, as concrete is the most common material for construction of bridge decks but the conventional cast-in-place (CIP) construction of concrete bridge decks can be very time consuming because of the significant amount of formwork and falsework, prefabrication can minimise or eliminate the need for on-site concreting. Other common prefabricated elements include pier caps. Precast pier caps have been used due to the difficulties of casting large concrete components at relatively high positions over land or water.

The application of ABC to the substructure like precast segmental columns used to be limited. Unlike superstructures, columns are often the most heavily loaded elements during a seismic event. The presence of weak connection between prefabricated elements may affect the seismic resistance of major resisting component, especially in regions with moderate-to-high seismicity. To address this problem, several connection details have been proposed for the application of

ABC to the substructures.

The high demand region in a typical column is at the ends where the column connects to the footings and pier caps. One method to connect the precast columns to adjoining members is through the use of mechanical rebar splices commonly referred to as couplers. The ability of couplers to splice rebars between precast elements to simulate CIP construction (Saito and Terada, 2016) has made them a popular choice for bridge designers. The most common type of rebar coupler for bridge column construction is the grouted sleeve coupler shown in Figure 6.3 (Pantelides *et al.*, 2017) with wide application according to the Federal Highway Administration (FHWA) in USA (FHWA, 2009). The cast steel sleeve is cast into the end of one element and a protruding reinforcing bar is cast in the end of the adjacent element. The elements are connected by inserting the protruding bars from one element into the hollow ends of couplers in the other element. The joint between the pieces is then grouted, and grout is pumped into the couplers to make the connection. The grouted sleeve coupler gains more popularity than other types of couplers, such as threaded and headed couplers, because it does not require as tight construction tolerances and is less time consuming to connect. Research has shown that grouted couplers are able to develop the full rebar strengths with much shorter spliced lengths than the conventional development lengths (Hayshi *et al.*, 1994; Saito and Terada, 2016), which makes this connection especially desirable for substructure connections with large diameter bars. The use of grouted couplers is currently limited to low-to-moderate seismic areas and not recommended for high seismic areas that require plastic hinging of connections (AASHTO, 2011). There is concern that the grouted sleeve couplers are not capable of developing plastic hinges in these high demand connections. However, recent research has shown that the grouted sleeve couplers with proper detailing can be emulative of CIP column-to-footing connections in terms of displacement capacity of columns in moderate-to-high seismic zones (Tazarv and Saiidi, 2014). The researchers thus suggest the removal of the restrictions imposed on grouted rebar couplers by AASHTO in seismic zones. The Texas Department of Transportation, USA has built connections that employ standard post-tensioning ducts to create voids for projecting reinforcing steel (FHWA, 2009). These connections are similar to the grouted splice couplers in that rebars are inserted into a sleeve made up of standard post-tensioning duct. The difference is that the duct is non-structural and therefore additional confining reinforcement is required around the duct to develop a significant connection. More recently, connections with performance emulative of CIP connections in moderate-to-high seismic zones have been developed by researchers by employing ultra-high-performance concrete filled corrugated duct connections (Tazarv and Saiidi, 2015).

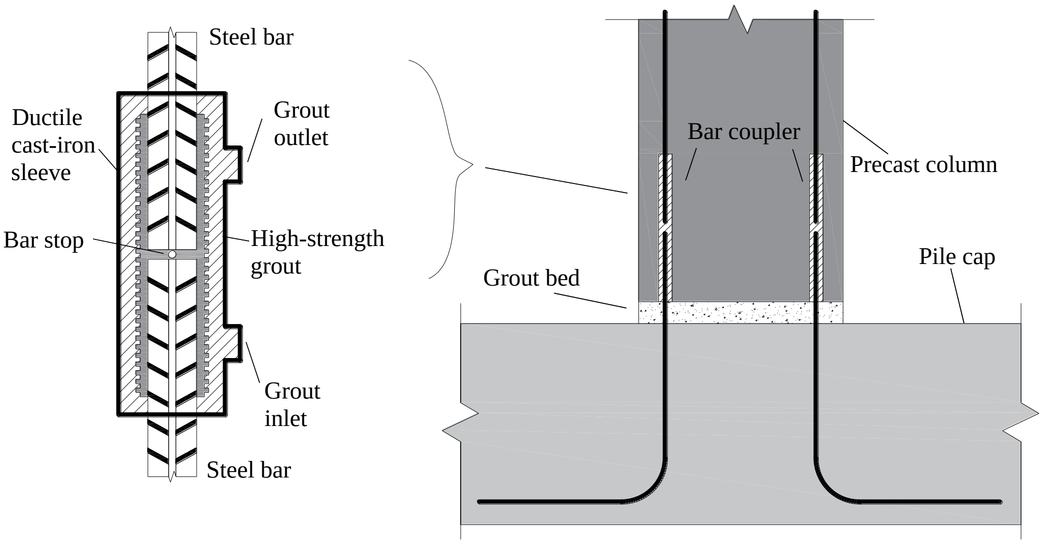


Figure 6.3 Precast column-to-footing connection using grouted sleeve coupler (Pantelides *et al.*, 2017)

The details for column connections that are proposed by FHWA in the Highways for LIFE (HfL) Technology Partnerships Program can also be used for energy dissipation connections in high seismic regions (FHWA, 2013a). The HfL connections, which are used especially for a fully precast bent system, consist of a column-to-footing connection, referred to as the socket connection, and a column-to-cap beam connection, as shown in Figure 6.4. The socket connection is made by placing the precast column in the excavation, placing the footing steel, and then casting the footing concrete around the column. Alternatively, the footing steel may be placed before the column is set. The surfaces of the column are usually roughened to enhance the connection. The socket concept represents a simple way to precast a column and integrate it with the foundation. Compared with the other alternatives, such as the grouted sleeve couplers, the socket system has the advantages that the placement tolerances for the column are significantly greater than those available with a sleeve system and that the connection requires no special or proprietary hardware. For the precast bent system, the column-to-cap beam connection has utilized the grouted post-tensioning duct connection. The lower precast cap beam is placed by lowering the beam over the rebars that protrude upwards from the precast column. Corrugated metal ducts are provided in the cap beam to accept the column dowels. The column rebars are subsequently grouted into the ducts. A socket connection at the base and a grouted duct connection at the top are considered to be practical solutions to the use of assembled substructures (FHWA, 2013a). The socket connection and the grouted duct connection have also been tested in New Zealand (Mashal *et al.*, 2014).

Figure 6.4 Exploded view of HfL precast bent concept (FHWA, 2013a)

In Taiwan, a new connection detail as shown in Figure 6.5 for building segmental concrete columns in seismic regions appeared in 2012 and was used for the Taichung Metro - Area No. 4 Elevated Expressway (Ou *et al.*, 2013). The segmental column has the lower part cast in place with the foundation and the upper part consisting of hollow segments. The CIP region provides U-hoop steel ducts for post-tensioning tendons and a rebar cage protruding into the hollow core of the first segment. Concrete is poured into the CIP region up to the hollow core of the first segment to ensure a sealed connection between them. The connections between the other column segments are made by using the match casting method of construction in conjunction with post-tensioning. The joints are epoxy bonded together and post-tensioning completes the joint. It should be noted that the height of the CIP region and post-tensioning force in the precast region are selected such that the ultimate condition is dominated by the plastic hinging behaviour of the CIP region rather than by significant nonlinear behaviour of precast joints (Ou *et al.*, 2013). In this way, the critical joint of the precast region is protected. Nevertheless, opening of the precast joints is not prevented. Thus, the post-tensioning tendons are designed to be unbonded from the concrete to reduce yielding and the subsequent prestress loss due to joint opening. Besides, the unbonded post-tensioning tendons are also known to provide excellent self-centring capability (Priestley *et al.*, 1999). In order to enhance the hysteretic energy dissipation capacity of the segmental columns, the bonded longitudinal mild steel rebars inside the column segments may be made continuous at the segment joints by using grouted steel corrugated duct connections, forming a hybrid system (Ou *et al.*, 2009). While the tendons provide self-centring and restoring actions, the mild steel rebars act as energy dissipaters and shock absorbers through the opening and closing of the precast joint, which is often termed the rocking mechanism.

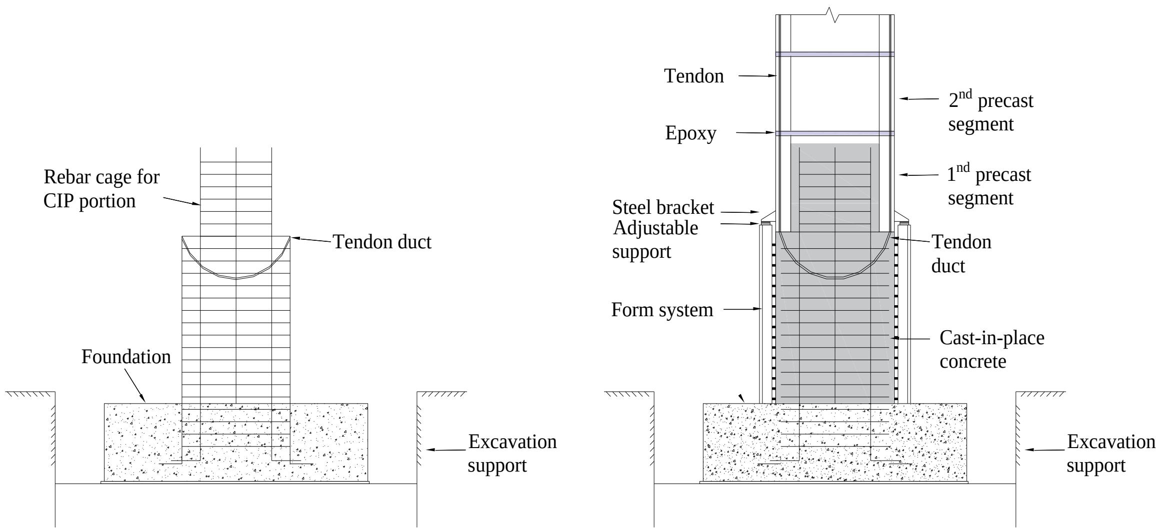


Figure 6.5 Joint construction of the CIP region (Ou *et al.*, 2013)

A new precast segmental bridge pier system has been proposed (Sung *et al.*, 2017) as shown in Figure 6.6. The system contains multiple segmental layers with several small precast modular segments for each layer, and therefore the size of each segment can be kept small to ease transportation and erection. This system has utilised the block-stacking concept. Each of the precast modular segments includes double joint holes and shear keys at the top and bottom surfaces, respectively, so that it can be bonded to two adjoining segments in the neighbouring layer by embedding each of the two shear keys in the corresponding joint holes belonging to two different segments to establish interlocking between segments in the horizontal direction (Sung *et al.*, 2017). Post-tensioning is applied to complete the assembly. It should be noted that shear keys are commonly used in precast segmental bridge piers mainly to ease assembly and their shear resistance need not be taken into account in the design phase. The shear keys adopted in this system, however, are used not only as alignment guides to facilitate assembly but also to provide shear resistance between segments in order to reduce the prestressing force needed (Sung *et al.*, 2017). To use such prefabricated columns in moderate-to-high seismic regions, the connection between segments in the vertical direction utilizes a hybrid connection that contains not only shear keys but also bonded mild steel rebars that are spliced by rebar couplers and unbonded tendons with small prestressing force (Sung *et al.*, 2017). Likewise, the bonded steel reinforcement that runs continuously between segments can provide strength and energy dissipation capability, and the prestressed unbonded tendons can provide re-centring forces to minimize residual displacements. Cyclic loading test results and construction practices of the system have confirmed the seismic performance and constructability of the modular precast segmental column to be satisfactory (Sung *et al.*, 2017).

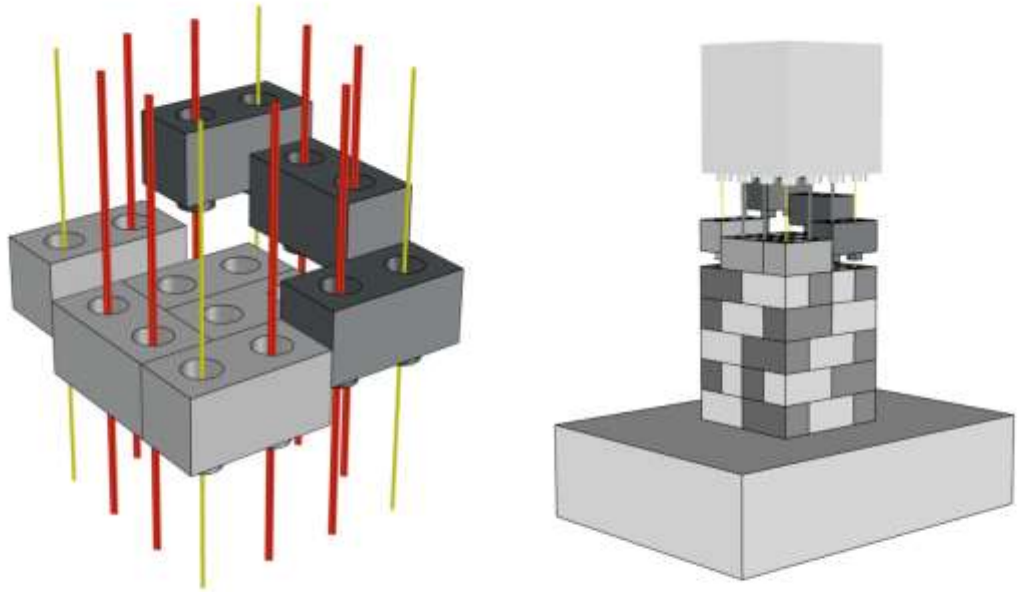


Figure 6.6 Schematic view of modular precast segmental bridge pier (Sung *et al.*, 2017)

6.2.2 Innovative construction methods

For projects based on large and heavy prefabricated elements, a particular timely and economical solution is to rotate or slide bridge structures from the assembly area to its final position using heavy lifting equipment and techniques.

The rotation construction method involves building the superstructure at one side of the obstacle being spanned (normally at 90 degrees with its final position), depending on the bridge layout and surrounding landform, and then rotating the superstructure into place. In the earliest attempts, vertical rotation construction method has been used in arch construction. Horizontal rotation method was accomplished decades later and is applicable to other types of bridges as well (Sun *et al.*, 2011). The highly-mechanized rotation method is efficient when used in the proper circumstances, such as spanning a busy shipping channel or railway. Compared with the balanced cantilever method and incremental launching method, the rotation method shortens the construction time and causes minimal traffic impact. The technique has been quite popular in Mainland China. One notable bridge is Gusao-Tree Road Overpass in Wuhan City (武漢市姑嫂樹路高架跨鐵路橋), which spans over one of the busiest railway routes in China, i.e. the Jingguang Railway (京廣鐵路) that connects the capital Beijing and Guangzhou City. In 2014, a 17,000-ton part of this elevated motorway, after being constructed independently beside the high-speed railway, was carefully swung 106 degrees about a vertical axis into place through a horizontal plane 15 m above ground and connected to the rest of the bridge in 90 minutes.

The slide-in bridge construction method is a special method used to replace an existing bridge with the minimum road closure duration. It is also one of the fundamental ABC techniques being promoted in the USA (Aktan and Attanayake, 2013). In this method, the new superstructure is built alongside an existing bridge on temporary supports. All traffic continues uninterrupted on the existing bridge until the construction of the new superstructure is completed. Then the existing bridge is demolished or removed, and the new bridge can be slid into place (FHWA, 2013b). There is a minimum road closure duration during the lateral bridge slide, which is required to be within 24 hours (SHRP 2, 2013). The old substructure can be retained or a new substructure can be built before the new superstructure is slid in. In some instances, the new substructure is pre-constructed below the existing structure to reduce the overall road closure time (SHRP 2, 2013). This method also provides a safer environment for construction workers and greater ease, as construction work need not be carried out immediately adjacent to the traffic. There is additional room for girder sets, deck concrete placement and equipment access (SHRP 2, 2013).

6.3 Resilient Structural Systems

6.3.1 Self-centring system

Ductility design attempts to prevent catastrophic failure through the formation of reliable plastic hinges. However, the hysteretic dissipation of energy usually causes non-repairable damage to the hinge zones. After a strong earthquake, even though life safety may have been assured, extensive damage may have left the structure unserviceable and costly to repair. Owing to this, Damage Avoidance Design (DAD) based on self-centring system is proposed to be used with the ABC method.

The concept of self-centring system was first proposed in 1991 under the Precast Seismic Structural System (PRESSS) Research Programme at University of California at San Diego (Priestley, 1991). It is a jointed ductile connection system similar to that shown in Figure 6.5, aiming to minimize damage in the pier through dissipating the energy via “controlled” rocking motion at the segmental connections. External dissipative devices that can be easily replaced may be provided at the rocking section to dissipate the energy for easy maintenance (Marriott *et al.*, 2008).

Priestley *et al.* (1999) proved that an effective and viable solution is to allow the rocking motion between structural members, i.e. the beam-to-column or column-to-foundation connection, rather than adopting CIP. Figure 6.7 shows a solution to DAD at the column-to-foundation connection. The rocking motion of the column is “controlled” by an additional restoring force provided either by the inner partially unbonded mild steel bar as shown in Figure 6.7(a) or through external dissipative linkages (reinforcing bars or mechanical dissipative devices) placed at the rocking section as shown in Figure 6.7(b). The latter is preferable in the event of moderate-to-high seismicity.

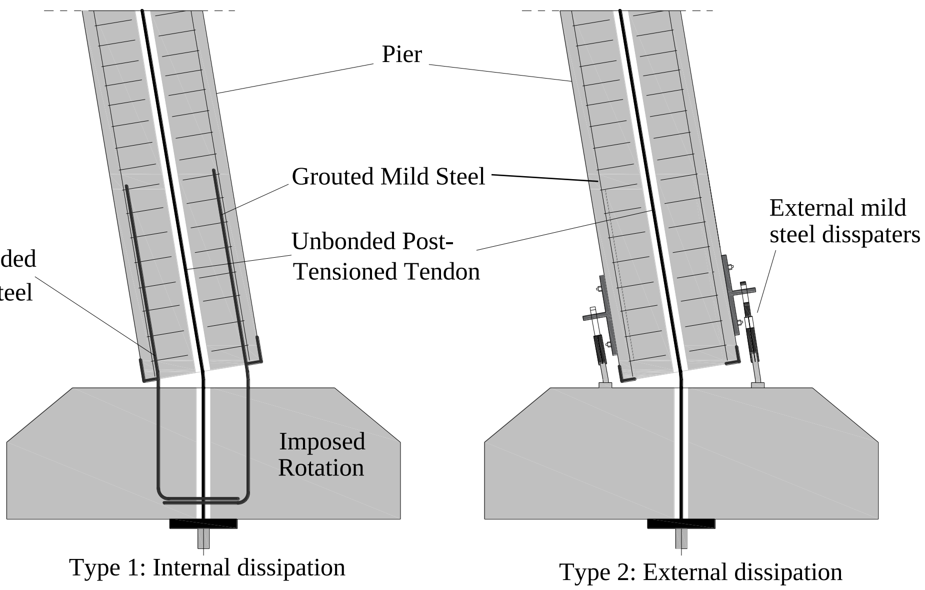


Figure 6.7 Two DADs for column-to-foundation connection: (a) internal dissipation; and (b) external dissipation

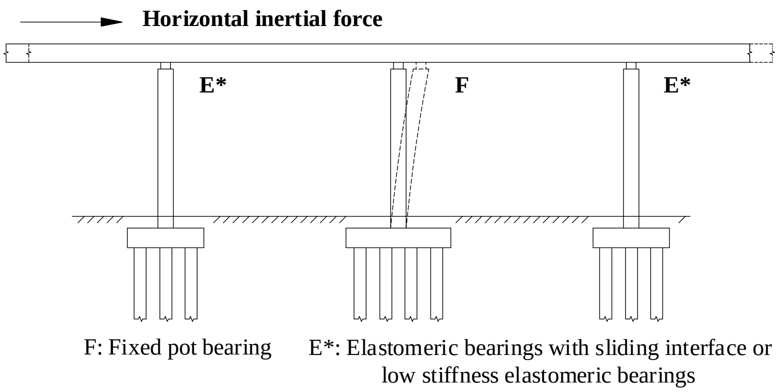
It is expected that controlled dissipative rocking action during an earthquake offers restoring or self-centring capacity plus energy dissipation. This technology is likely to reduce the potential damage to the structure and preserve the functionality of the bridge after the earthquake, representing a promising solution to future bridge seismic design.

In New Zealand, damage avoidance design has been used in a three-span continuous bridge, i.e. Wigram-Magdala Bridge (Routledge *et al.*, 2016). Nevertheless, there are still many problems to be solved before wide application of DAD.

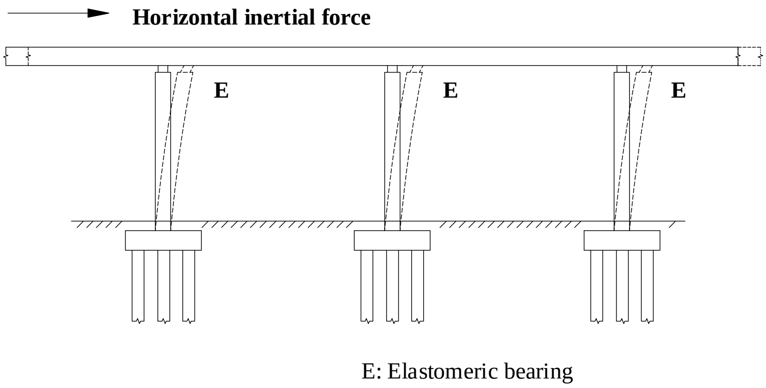
6.3.2 Floating articulation

Laminated elastomeric bearings are commonly used in small- to medium-span bridges around the world for they are relatively cheap and they require low maintenance. In some of the past earthquakes, engineers found that bridges with mixed use of elastomeric bearings and fixed bearings as shown in Figure 6.8(a) performed poorly as compared to those with solely elastomeric bearings on all supports as shown in Figure 6.8(b), i.e. floating bearing articulation. Bridges with the floating bearing articulation have the following advantages in respect of seismic performance:

- Incurring smaller seismic action due to the smaller stiffness and longer fundamental period;
- Enabling more uniform distribution of inertial forces among piers and hence smaller pier sizes; and
- Ease of repair and replacement of bearings after earthquake.



(a)



(b)

Figure 6.8 Bearing articulation: (a) conventional; and (b) floating

The floating articulation system, also known as "seismic isolation", has been widely used in many countries in recent years. In Japan, this practice has been adopted for the majority of bridges built after the 1995 M6.9 Great Hanshin earthquake.

Despite the numerous advantages, the use of floating articulation system should be adopted with caution. Large seismic displacement could occur, making it difficult for the design of movement joints. For bridges founded on soft ground, the stiffness of the elastomeric bearings shall not be lessened by an excessive amount as resonance may occur between the structure and ground.

The adoption of floating bearing articulation in long-span bridges may result in large bearing sizes, which could offset the material savings achieved in the substructure. To solve this, a hybrid bearing system has recently been proposed in Japan (Saito and Terada, 2016). The hybrid bearing system consists of pot bearings and elastomeric bearings on the same pier as shown in Figure 6.9. The pot bearings with sliding interface are aimed for supporting the vertical load, while the elastomeric bearings are for resistance and flexibility in the horizontal direction. Since the elastomeric bearings in the hybrid bearing system do not carry substantial vertical loads due to the shielding effect of the stiffer pot bearing, they can be much more compact.

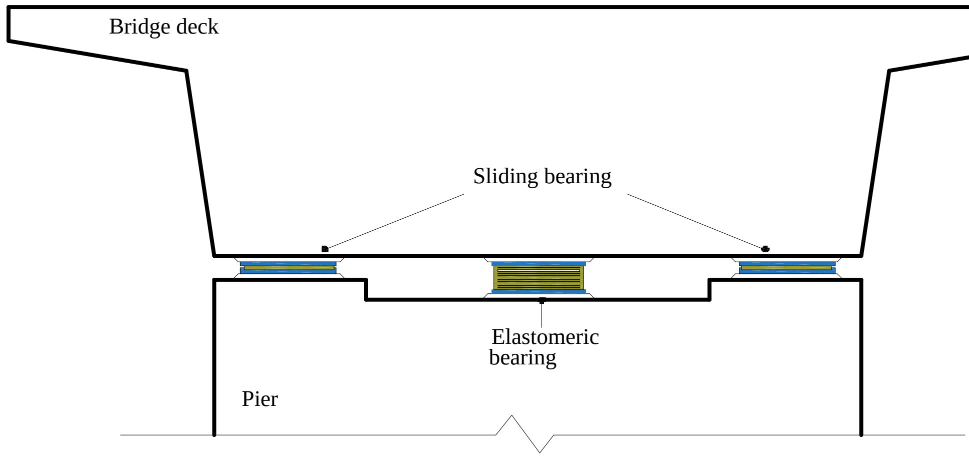


Figure 6.9 Hybrid bearing system

6.4 Design for Maintainability

It is increasingly recognized that many decisions taken during the planning and design of a bridge can significantly influence its service life as well as the costs of future inspection and maintenance, and that maintainability should become one of the primary considerations in the bridge design process. This is a paradigm shift as most planning and design operations in the past were governed by the current needs and initial costs then with less emphasis on the future maintenance, which has resulted in bridge designs that are not conducive to cost-effective maintenance. The basic maintenance aspects include the selection of material and components that are easy to repair, details that can provide easy access to inspect and repair, components that can be replaced easily, and use of features such as integral abutment bridges to avoid joints that can cause deterioration of other components because of their failure and so forth (Alampalli, 2014a). All these aspects have been changing slowly from very reactive to very proactive in nature because of the increasing emphasis on life-cycle costs.

The following considerations are recommended in bridge design for improved maintainability (Alampalli, 2014b; Ceran and Newman, 1992; NYSDOT, 2017):

- It is desirable to reduce the number of expansion joints to an acceptable minimum and avoid half-joints and in-span hinges. The maintenance impacts of deck joint failure include deterioration of the beams, pier caps and bearings as a result of water and chemicals leaking through the joints. A non-functioning or jammed joint, and the resulting pressure, will cause undesirable movement of abutments or piers, and spalling of concrete decks, beams, and abutments or pile caps. As a result, the designer should try to avoid the use of open joints. Most importantly, the designer should accept that joints will leak and therefore waterproof the concrete and steel works below. Elimination of joints may be accomplished by designing for continuity and taking advantage of the flexibility of the structural system.
- The major maintenance impacts associated with deck drainage are retention of water due to inadequate deck drainage and deterioration of the superstructure and substructure elements when water is not directed away from these elements. In this regard, the drains should be designed with adequate gradients and cross falls. Besides, it is better to avoid placing bridges on sag vertical curves, especially when concrete barriers are used, as it will create problems in case the drain becomes clogged. If this cannot be avoided, drainage outlets through the barrier should be provided to prevent ponding. Inlet grates shall be located in front of barriers in such a way that they are easily accessible for daily cleaning to minimize obstruction of deck drains with debris. Flat inlet grates should be provided to

avoid damage to the inlet by pavement cleaning equipment. Moreover, oversized pipes and fittings, inlet openings and catch basins may be adopted to eliminate clogging even though they are not required for capacity purposes. In severe clogging conditions, it is preferable to provide mechanical joints so that the downspout system can be disassembled if necessary. The downspouts shall not be encased in concrete pier where they cannot be maintained or the problems are out of sight. It is also suggested that the downspouts extend below the superstructure so that water is not sprayed onto the superstructure elements.

- The use of elastomeric bearing pads should be the first bearing of choice. These bearings are inexpensive and can be detailed for very simple and fast installations. They are also known for their good seismic performance. Unlike rocker, roller and sliding bearings made of steel, which are prone to debris blocking and corrosion, elastomeric bearings do not seize or corrode, and they require a minimal amount of maintenance. Nevertheless, bearings should be protected from dirt, adverse chemicals and water by proper details. It will help to reduce maintenance and future rehabilitation requirements by eliminating or reducing the number of bearings by designing a continuous superstructure with integral interior piers and/or abutments. However, the continuous superstructure will unfortunately cause an increase in the magnitude of movements at the expansion bearings. Elastomeric pads, in combination with polytetrafluoroethylene (PTFE) and stainless steel, can be provided to accommodate large movements.
- Owing to the corrosion vulnerability of the post-tensioning systems, tendons located in the columns below the highest water splash zone elevation should be avoided, and no precast concrete hollow column section specified below the waterline should be allowed. Multiple levels of protection shall be provided at the anchorages, including the permanent grout cap, epoxy material pour-back and polymer coating over the pour-back. A post-tensioning redundancy system or practical replacement capabilities should be incorporated (FHWA, 2004).
- For inspection and maintenance purpose, access to parts that may require maintenance or replacement during the life of the bridge, e.g. bearings, anchorages, unbonded prestressing tendons, etc., should be included in the plans, and the required clearances for removal of components should be provided. In providing the access, consideration should also be given to provide a dry, comfortable and pleasant environment in which the inspector is to work. Improved accessibility at the end supports may be achieved by providing an inspection gallery to the rear of the bearings as shown in Figure 6.10. The preferable width and headroom clearance of such galleries are recommended to be not less than 1000 mm and 1800 mm, respectively (NRA, 2010). The abutment galleries are also useful for inspection and maintenance of expansion joints, prestressing tendons and anchorages, deck ends and abutment curtain walls. A suitable gap should always be provided between the top of the bearing shelf and the deck soffit. Besides, bearings should always be designed for future removal and replacement. The location for jacking should be considered, especially when it would be difficult to place a jacking bent. Consideration may be given to providing a widened bridge seat where jacks may be placed as shown in Figure 6.11 (DOTMR, 2013).

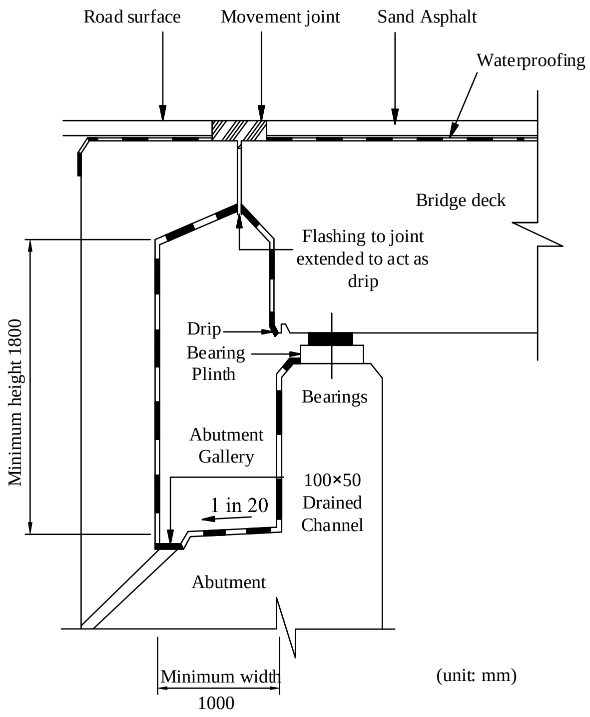


Figure 6.10 Abutment gallery (NRA, 2010)

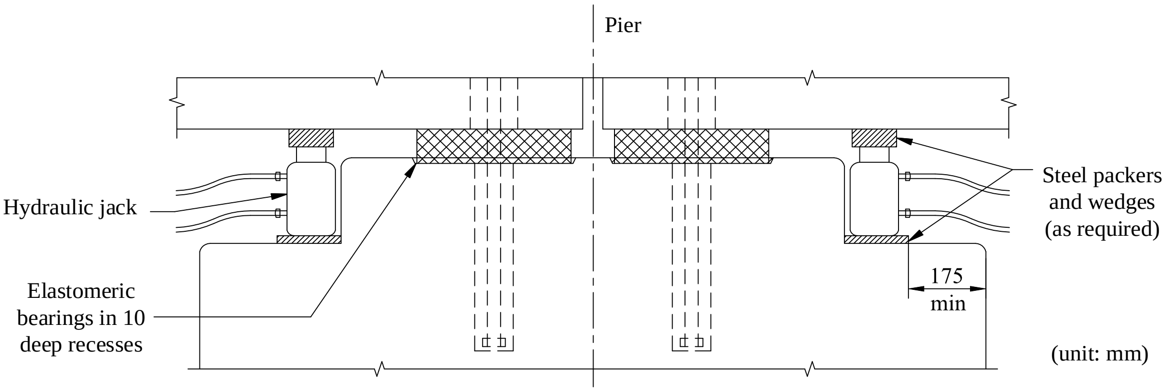


Figure 6.11 Jacking place (DOTMR, 2013)

- Where possible, it is recommended to include with the bridge design equipment to ease inspection and maintenance. Smaller structures may not justify a health monitoring system or permanent cradles and runway beams, but portable, "over the side" inspection and repair platforms can save scaffold erection and be moved as required. With the present practice of using Under Bridge Inspection Truck (UBIT), the need for simple attached platform has been reduced. If a UBIT is required for inspection of a bridge, the fencing along the fascia should be limited or fence gate should be provided to swing the boom and basket of the UBIT to reach outside the fence. Nevertheless, the use of UBIT may not be able to inspect some utilities due to the deep superstructure or other design details.
- It is desirable to ensure that all materials are of a suitable quality so as not to require frequent renewal. It is also important to make sure that the work force has the necessary expertise in the use of the materials.

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