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Figure 1.13 Quebec Bridge, Canada

1886 The Fraser River Bridge, Canada – believed to be the first balanced cantilever truss bridge to be built. All the truss piers, links and lower chord members were fabricated from Siemens–Martin steel. It was dismantled in 1910.

1890 The Forth Rail Bridge, Edinburgh, Scotland – when finished, the world's longest spanning bridge at 1700 ft.

1891 The Cincinnati–Newport Bridge, Cincinnati – with its long through-cantilever spans and short truss spans, this was the prototype of many rail bridges in the USA.

1902 The Viaur Viaduct, France – this rail bridge between Toulouse and Lyon was an elegant variation of the balanced cantilever, with no suspended section between the two cantilever arms.

1919 The Quebec Bridge – completion of the second Quebec Bridge, the world's longest cantilever span.

1927 Carquinez Bridge – the last of the long cantilever truss bridges to be built in the USA, although a second identical bridge was built alongside it in 1958 to increase traffic flow.

The suspension bridge

The early pioneers of chain suspension bridges were James Finlay, Thomas Telford, Samuel Brown and Marc Seguin, but they had only cast and wrought iron available in the building of their early suspension bridges. It was not until Charles Ellet's Wheeling Suspension Bridge had shown the potential of wire suspension using wrought iron that the concept was universally adopted. Undoubtedly, the greatest exponent of early wire suspension construction and strand spinning technology was John Roebling. His Brooklyn Bridge was the first to use steel for the wires of suspension cables.

Suspension bridges are capable of huge spans, bridging wide river estuaries and deep valleys and have been essential in establishing road networks across countries. They have held the record for longest span from 1826 to the present day, interrupted only between 1890 and 1928, when the cantilever truss held the record.

1883 Brooklyn Bridge – following the completion of the Wheeling Suspension Bridge, pioneered by Charles Ellet, John Roebling went on to design the Brooklyn Bridge, the first steel wire suspension bridge in the world.

1931 George Washington Bridge – the heaviest suspension bridge to use parallel wire cables rather than rope strand cable, and the longest span in the world for nearly a decade (Figure 1.14).

1950 Tacoma Narrows Bridge – the second Tacoma Narrows Bridge, rebuilt after the collapse of the first bridge with a deep stiffening truss deck, set the trend for future suspension bridge design in the USA.

1957 Mackinac Bridge – Big Mac is the longest overall suspension bridge in the USA.

1965 Verazzano Bridge-Narrows – the last big suspension bridge to be built in the USA, also held the record for the longest span until 1981.

Figure 1.14 George Washington Bridge, New York

Chapter 6

Seismic response and design

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Bridges are the transportation network component most vulnerable to damage from natural disasters, compared with roads and railway lines. It is therefore of priority to adequately design new bridge structures and reassess the response of existing bridges in areas subjected to earthquake hazard. This chapter briefly addresses a number of topics related to seismic response and design of bridges, namely damage observations in previous earthquakes, conceptual design and modern seismic codes. Commonly observed bridge failure modes following damaging earthquakes are presented. This shows that despite the advancement in seismic design practice, there are repetitive damage patterns, owing to the increased number of bridges of complex configurations and the heightened consequences of bridge damage in developed societies. Features of layout and configuration that are favourable to controlled and predictable seismic response of bridges are also discussed. Various options available, from foundations through to the superstructure, and connections between various components, are presented and their likely effects on the response are discussed. Finally, a brief review of seismic design codes in Europe, the USA and Japan is presented. The review highlights the differences and their origin, which is an important step towards improved understanding of seismic design procedures.

Introduction

An efficient transportation system plays a vital role in the development of a modern society, mainly owing to the inter-reliance of various industries and the increased trend for outsourcing various necessary ingredients within a single activity. Hence, transportation networks are referred to as lifelines, the integrity of which has to be protected, alongside water supply, electricity and gas networks. While roads are a most important component of transportation networks, bridges are both more important and sensitive to damage from natural disasters, since roads are more easily repairable and may also be readily bypassed. The closure of a bridge that represents the only or most important link between two areas separated by water or some geological feature (e.g. gorges) would potentially cause very severe consequences for industry, commerce and society as a whole. Recent examples abound as to the effects of earthquake damage to bridges, as discussed in subsequent sections. Two examples are quoted herein of the consequences of the closure of the Oakland Bay Bridge on traffic between San Francisco and Oakland (Loma Prieta earthquake, 1989) and the closure of several of the crossings between Kobe and Port Island (Hyogoken-Nanbu earthquake, 1995), among several others. Not only did such closures affect the communities in the immediate vicinity of the bridge, but they also had knock-on effects on many other communities, owing to loss of business and delays in the delivery of essential goods. Table 6.1 gives estimates of economic loss as the result of bridge damage in three major earthquakes. These do not include indirect loss arising from business interruption and lost revenue; however, they serve to confirm the economic significance of bridge damage.

If the economic loss arising from closure of a main arterial bridge is assessed alongside the cost of seismic retrofitting of the structure, the case for the assessment and redesign of bridge structures in seismic areas will be immediately apparent. To emphasise this point, the effect of the San Fernando earthquake of 1971 is considered. Many of the cases of collapse of spans were attributed to the short seating length allowed at seismic joins. The cost of design and installation of restrainers (assuming that other failure modes would not be triggered) would have been a very small fraction of the direct cost of repair, and an even smaller proportion of the total cost including business interruption and loss of revenue. It is therefore of priority to reassess bridge structures in areas subjected to seismic hazard with a view to minimising earthquake damage.

One serious problem facing the earthquake engineering community in reducing the earthquake risk to bridges is that whereas, in general, engineers tend to have a feel for vulnerable parts in buildings and frequently encountered failure modes are common knowledge, they are often less familiar with bridge structures. Therefore, increasing bridge designers'

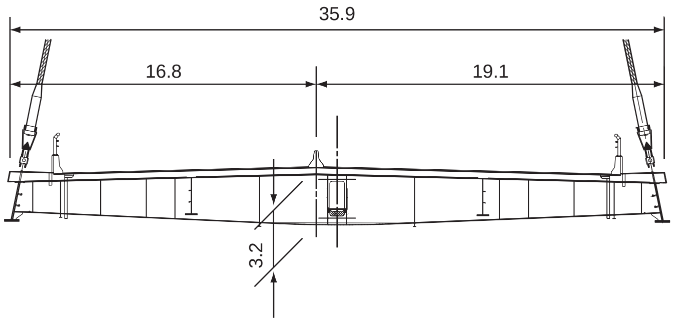

Figure 14.30 Twin girder composite deck – Industrial Ring Road Bridges, Bangkok (all dimensions in metres)

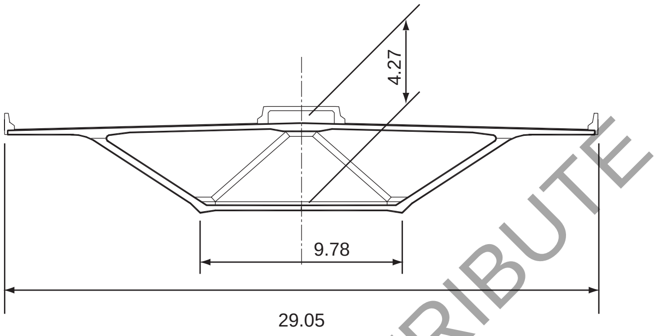

Figure 14.31 Concrete torsion box deck – Sunshine Skyway Bridge, USA (all dimensions in metres)

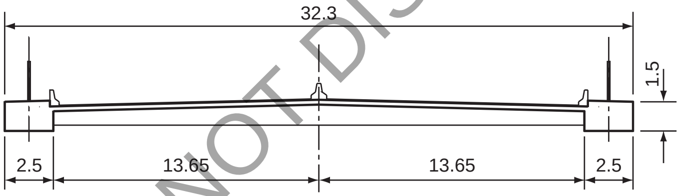

Figure 14.32 Twin beam concrete deck – Dames Point Bridge, USA (all dimensions in metres)

Concrete deck section

Concrete deck sections were subject to developments similar to those for steel deck sections. The torsion box deck sections are commonly used in conjunction with a single central plane of stays. The first such design was the Brotonne Bridge crossing the River Seine near Rouen, France. A similar design, using precast segmental units for the deck, was adopted for the Sunshine Skyway Bridge, USA, as illustrated in Figure 14.31. In common with a number of major bridge projects in the USA, this design was selected after a process of competitive pricing between alternative steel and concrete designs.

Concrete designs, in common with the evolution of the composite deck section, have developed a simplified deck form. Examples of this deck construction are the Dames Point Bridge over the St. Johns River in Florida, USA, which is illustrated in Figure 14.32, and the Helgeland Bridge, Norway (Svenssen and Jordet, 1996). In a typical arrangement, the transverse floor beams are at 3–5 m centres supporting an in situ concrete road deck. The transverse beams on the Dames Point Bridge took the form of precast T-beams. The longitudinal beams are located at each edge of the deck, centrally beneath the cable planes, and incorporate the stay anchors. Erection is by casting the deck in segments as a free cantilever using a form traveller. The stay is initially stressed against the form sufficiently to minimise

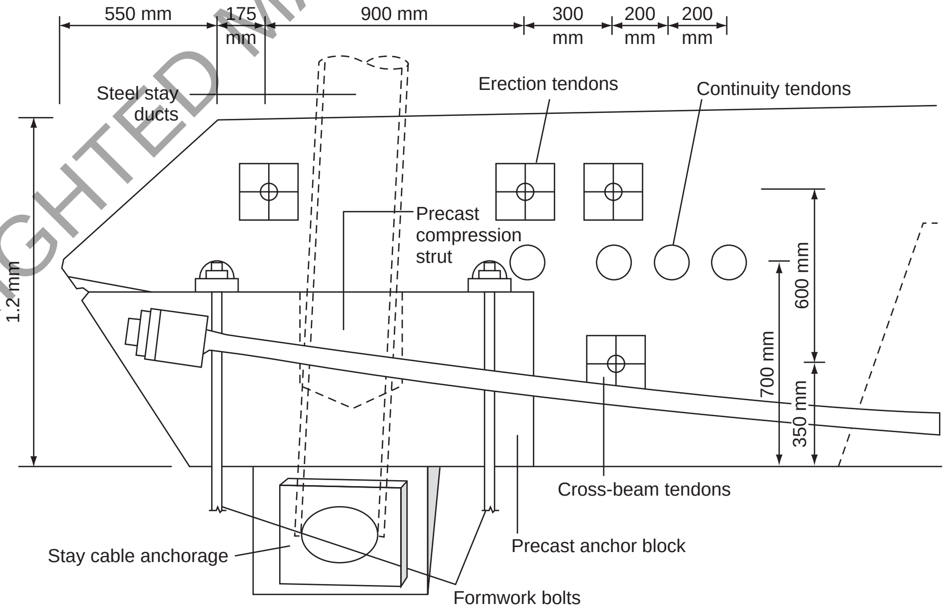

Figure 14.33 Precast anchorage and edge beam – Helgeland Bridge

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Inverted T-beam decks

Inverted pre-tensioned T-beam decks are generally analysed on a strip basis or, for larger or higher skew structures, by using grillage. Transverse moments are rarely correctly determined using grillage and reduced transverse properties are introduced. The yield-line method is rarely used.

Shear key and filler beam decks

Grillage analysis is generally used, or the strip method if appropriate. Transverse effects are covered by limiting tensile stresses. Use is made of national highways CS 457 (HE, 2020f).

Single-span and multispan brick and masonry arches

Standard CS 454 (HE, 2020d) supersedes DMRB documents BD 21 (Highways Agency, 2001) and BA 16 (Highways Agency, 1997). It consolidates the material relating to masonry arch assessment in a single chapter with clearer requirements that are not as focused on the use of the modified MEXE method and gives additional restrictions on the use of the modified MEXE method.

The assessment of arch bridges is only as good as the accuracy of the dimensional survey and the condition survey carried out on-site to inform the assessment. The rise at the crown, the rise at the quarter points, the arch span (skew and square) and the springing height should be recorded, as well as the depth of the arch barrel and the depth of fill over the arch barrel. For arches with a non-uniform profile, additional measurements may need to be taken along the profile of the arch barrel. If intrusive investigations are carried out, the depth of the arch barrel can be established, either by excavating a trial pit from above the crown of the arch and taking a level survey to the extrados or by drilling pilot holes from the underside of the arch (the intrados). If no intrusive investigations are carried out, it is common practice to use a lower bound and upper bound approach where the lower bound assessment assumes that the arch barrel is 60% of the depth of the facing stones and the upper bound assessment assumes that the arch barrel is 100% of the depth of the facing stones. The condition survey of the bridge helps the inspector to assign condition factors to the bridge and considers mortar loss, friability of the mortar joints, materials, thickness of the mortar joints and direction and spacing of any cracks that may be present in the arch barrel.

The initial live load assessment is normally carried out using the modified MEXE method (for spans ranging between 5 and 18 m). However, where the arch is flat (the span/rise ratio exceeds 8), appreciably deformed or with skew greater than 35°, or where the depth of fill over the arch barrel is greater than the thickness of the arch barrel, the MEXE method is not deemed suitable. In addition, for brick arches where ring separation has been identified as likely, the MEXE method may not be used. If the results of the MEXE method need to be verified, if the MEXE method gives unsatisfactory results or if the MEXE method is not deemed appropriate, alternative methods of assessment are carried out using computer software packages. The most common analysis packages use either the three-hinge limit analysis method or the rigid block method. Alternatively, finite-element packages have been used successfully for the live load assessment of arches using non-linear analyses. For multispan arches, MEXE has been used by assuming fixity at springing, particularly where piers are stocky (height/width ratio of 2) and where arches and piers are restrained by cross-walls at each end of the arch barrel. The software packages used for single-span arches, that is, the three-hinge limit analysis method and the rigid block method are also generally capable of modelling multispan arch structures.

The recommended phasing of the assessment is to first assess the arch structure considering the existing condition of the bridge with the assigned appropriate condition factors. If the results are not satisfactory, the arch structure is then assessed considering improved condition factors, assuming that repairs are carried out (such as repointing of the arch barrel). If the live load capacity of the structure is still below the required live load rating, backing can be considered in the analysis and confirmed on-site by visual inspection or by intrusive investigations (pilot holes through the wing walls or spandrel wall and or trial pits).

For arches, MEXE is sometimes used to give the loadings; these are then used in a space frame analysis program. Sometimes, if the arch is thin and flexible, soil–structure interaction can be modelled using a finite-element program. This gives much reduced bending effects and hence greater capacity.

In general, there is far less concern about arches, as they have strong inherent strength and distress can be observed. They tend to be condition-dominated and so poor-condition arches tend to be repaired or replaced.

Concrete post-tensioned beams or slabs

Grillage analysis is normally used, particularly if a strip analysis has been used and failed. For voided post-tensioned slabs, a shear flexible grillage has been used with a reduced shear area for transverse members. Finite-element shell analysis has also been used, but it may be more difficult to interpret the results.

Concrete post-tensioned box girders

A line beam with a shear flexible grillage and a transverse plane frame or a full grillage analysis model is generally used. An analysis package that covers post-tensioned construction can be used to determine stressing sequence and losses. The grillage or line beam approach will generally require a separate transverse model to quantify the transverse frame action occurring in the structure, potentially combined with Pucher charts or similar. Alternatively, a finite-element shell analysis may be used to quantify these transverse effects.