# Shock Transmission Units in Construction

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This book is dedicated to my beloved parents who educated me to enable me to write this book.

*Dinesh Patel*

This book is dedicated to the memory of Dinesh Patel by his wife, Bhavini Patel.

# Contents

# Preface

Recent earthquakes in many countries around the world have confirmed the potential for the occurrence of unexpected very large seismic events. The levels of disturbance and destruction of highways, bridges and lives suffered during these large earthquakes were far greater than structural engineers and the design codes had predicted. One important lesson that has been learnt is that it is vital that bridges which connect major transportation routes must continue to function after an earthquake. In the quest to achieve this goal, new technologies and design efforts by engineers have resulted in the development of seismic protection systems (see Chapter 2), which include shock transmission units (STUs) and various seismic-isolation and energy-mitigation devices.

Bridge engineers have always wanted to tie the components of a bridge structure together to distribute seismic or suddenly applied impact loads between the various substructural elements of a bridge, but were unable to do so as a bridge must be allowed to move due to slowly applied loads such as expansion and contraction due to temperature variation, and shrinkage and creep. When a bridge is impacted by a suddenly applied load or shock loading, STUs lock up momentarily, making the structure rigidly connected, but they allow the movement of the bridge structure due to a slowly applied load. STUs can also be used to share, reduce and transfer suddenly applied shock loads (see Chapter 3). Therefore, the STU is a perfect device for use by bridge engineers.

The early STUs were rather complex and bulky, and required frequent maintenance, which discouraged their use by engineers on bridge structures. However, development has led to the modern STU (see Chapter 1), which is compact and maintenance free.

STUs are now used mostly in Southeast Asian and Western countries, being relatively unknown to the engineering profession in many other countries due to a lack of appropriately documented technical literature. Therefore, the purpose of this book is to acquaint the reader with the various applications of STUs in structural engineering, and to guide future users in procuring, selecting, testing and installing the most appropriate STUs for their projects. It is hoped that, by creating greater awareness among engineers, this book will also lead to an unleashing of the potential uses of STUs in the future. Chapters 4 to 9 include a number of case studies of the application of STUs supplied by various manufacturers known to the author. The case studies include new and existing highway and rail bridges, suspension and cable-stayed bridges, buildings, and nuclear power plants and pipelines.

Modern STUs (see Chapter 3) have been used since 1970. Unfortunately, however, some engineers are still not happy to specify STUs until they have been proven to work satisfactorily in the field for the expected lifespan of a bridge structure, which is 75–100 years, even though extensive laboratory tests have proved STUs to be reliable devices. It is hoped that this book will enlighten the critics with regard to the safe use of STUs, and will encourage their use of these devices in the various applications possible in structural engineering. Undoubtedly, there are other, as yet unknown, STU applications to be exploited; it is anticipated that, as engineers become aware of the potential that these devices have to offer, increasingly varied and innovative applications of STUs will be discovered (see Chapter 10). Furthermore, some devices derived from the STU (see Chapter 16) will be further developed and used in various other structural applications.

One of the most important applications of STUs is for the retrofitting of existing bridges in many countries where the existing bridges were not designed and constructed for recently revised design codes for higher intensity earthquakes or braking loads. The installation of STUs on such existing bridges involves little disruption to bridge traffic during installation, and is therefore a simple, safe and highly economical solution (see Chapter 12). While STUs are most commonly used for seismic applications, they can also be used to distribute braking and traction forces due to vehicles and railway trains on highway and railway bridges, and for restricting the movement of the deck caused by earthquakes and hurricane winds on suspension and cable-stayed bridges. STUs can also be used on structures in conjunction with other seismic protection systems (see Chapter 17).

There are only a few countries in the world where STUs are manufactured at present. Unfortunately, engineers in many countries remain somewhat confused, as each manufacturer offers somewhat different designs and available options, as STUs are proprietary devices. In addition, before the introduction of the American Association of State Highway Transportation Officials (AASHTO) interim specifications for STUs in 2002, manufacturers were trying to push their own proprietary specifications, which were written for their own brand of STU and were, to some extent, occasionally to their own advantage. The author has visited most of the manufacturers of STUs around the world and discussed the dilemma facing engineers in the present environment. He also took this opportunity to stress the importance of this book in creating better understanding between the manufacturers and the users of STUs. While this book contains case studies of STUs produced by various manufacturers, the purpose is to create awareness among engineers about the various brands of STUs available today, and not to favour any manufacturer of a particular brand.

In 2006, the AASHTO interim specifications for STUs released in 2002 were revised and approved, and the specifications are now included in the AASHTO American Bridge Design Code, Section 32. Eurocodes also now include STU specifications. It should be realised, however, that these AASHTO and Eurocode specifications for STUs are not for design purposes. Until such time as design principles are included in the specifications, users must test every STU manufactured to ensure that the intended design purpose of the STU can be achieved under the site environmental conditions in accordance with the contract specifications, and the devices can only be installed once every STU to be used on the project has successfully passed all the relevant tests specified. The installation of untested STUs or STUs not found to be satisfactory in load tests (see Chapter 13) could lead to a disastrous outcome for the structure.

While present-day STUs are robust, they can become problematic if they are not checked for correct performance and corrosion of the steel components. It is of utmost importance that with the installation of STUs on bridges in developing countries the maintenance aspect of bridges is stepped up for STUs, just like those for expansion joints and bearings and other bridge accessories. STUs should be inspected, maintained and painted regularly (see Chapter 14) in accordance with the inspection and maintenance manual normally supplied by the manufacturer.

The installation of STUs on a structure can be done in various ways (see Chapter 11), but it is of great importance that they are installed in the most economical, safe and simple manner possible, as the cost of these connections can be considerable. In addition, an STU must be installed on a structure in such a way that it can be removed and replaced in the future if it is found to be behaving in an unexpected manner due to damage or other factors.

Finally, while the STU is a magnificent device, its as-designed performance can only be guaranteed if its stiffness is made compatible with the stiffness of the structural element it is installed on. If the deflection of the STU when acted on by a suddenly applied load is greater than the deflection of the pier together with its foundation of a bridge structure on which the STU is installed, the as-designed load distribution cannot be achieved. It is therefore very important during load testing to evaluate the maximum deflection of the STU when acted on by a suddenly applied load, as an STU will not be able to link the deck instantaneously with the pier if its deflection is greater than the deflection of the pier together with its foundation. This aspect of design, together with other relevant design guidelines, is covered in Chapter 15.

To the best knowledge of the author, this is the first book published on the subject of STUs, and it is hoped that it will be useful to practicing engineers, academic researchers and engineering students. For the benefit of STU users, Appendix 1 provides the contact information for various manufacturers known to the author, and Appendix 2 gives the addresses of some major laboratories around the world that are equipped for load testing STUs.

# Acknowledgements

The author is grateful to the following employees of the STU manufacturers who have supplied STU case studies, technical information and most of the illustrations of their projects included in this book. Their contribution is substantial, and without it this book would have not been complete.

Mr Francesco Tomaselli and Mr Paolo Baldo, FIP Industriale, Italy
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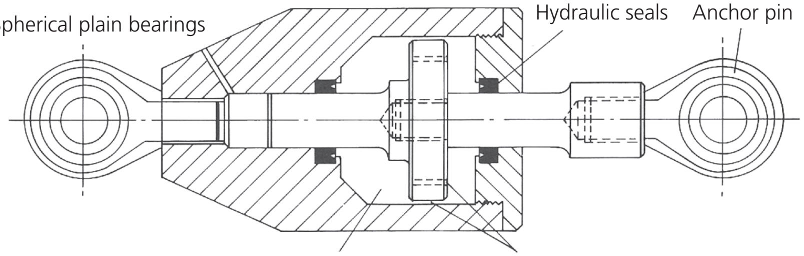
Mr Herald Lange, Lisega Inc., Germany
Mr Steve Brown and Mr Warren Brown, TechStar Inc., USA
Mr Antoine Domange, Jarret, France
Mr Andy Gill and Mr Craig Winters, Taylor Devices Inc., USA
Dr Marioni, Alga S.p.A., Italy
Mr Nigel Townsend and Mr Kerry Scudamore, Colebrand, UK

# About the author

D.J. Patel BSc(Eng), MEngSc, MICE, MIHT, MIE Aust, CPEng, FIIBE India

Dinesh J. Patel graduated from the University of London, UK, with a degree in Civil Engineering, and obtained his Master's in Structural Engineering from the University of New South Wales, Sydney, Australia. He then went on to supervise the design and construction of bridge structures across the world, including projects in the UK, Canada, Australia, Nigeria, Saudi Arabia, Indonesia, India, British Guyana and the Philippines, and worked with various international consulting engineering firms on bridge projects funded by the World Bank, the Asian Development Bank and the Japanese Government.

Mr Patel supervised the construction of the Badiwan Bridge, Baguio – the first bridge in the Philippines to use shock transmission units (STUs) – an experience that enabled him to procure, select, test and install the STUs on the Second Bassein Creek Bridge, Mumbai, India. He was employed by N.D. Lea International, Canada, as Resident Engineer/Team Leader to supervise the construction of this project, the first bridge in India to use STUs. In 2001, the Second Bassein Creek Bridge was awarded the first prize for innovation in construction engineering by the Indian Institution of Bridge Engineering.




A silicone putty STU

Mr Patel has written and presented many papers on bridge engineering and STUs. His paper entitled 'Shock transmission units (STUs) for earthquake load distribution on the World Bank funded Second Bassein Creek Bridge in Maharashtra', published in the *Journal of the Indian Roads Congress*, was awarded a certificate of merit by the Indian Road Congress in 2002.

Mr Patel's involvement in the Second Bassein Creek Bridge made him realise the potential for retrofitting STUs to bridges in countries with revised and upgraded earthquake zones in order to strengthen bridges for the revised earthquake loadings. In the absence of technical literature and a specification in the Indian bridge design code on the subject of STUs, Mr Patel hoped to use this book to share his experience and best practice guidance to help future bridge engineers to procure, test and install STUs to a high standard, and to ensure that they select the most appropriate STUs for their projects.

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# Chapter 1
# Evolution of shock transmission units (STUs)

## 1.1. Introduction

Once since engineers started to design multi-span bridges, and especially once they started to design continuous bridges, after Hardy Cross published his theory on moment distribution theory in 1930, there was the need for a device that could distribute suddenly applied horizontal forces (due to earthquake, wind storm, vehicle braking, etc.) on a bridge structure between its various substructure elements, while allowing the movement of the deck due to slowly applied loads (produced by temperature, shrinkage and creep). This dream became a reality for engineers in 1927, when D.B. Steinman, the designer of the Carquinez Strait Bridge in California, USA, used shock transmission units (STUs) for the first time to distribute seismic loads between the various piers of the bridge.

## 1.2. History of the evolution of STUs

STUs were developed from fluid dampers, which had been used since the 1860s in the field of artillery (Taylor, n.d.-a), where an effective device was needed to restrain the recoil of large cannons. New technology developed during World War II (radar and similar electronic systems) required the creation of specific isolation techniques so that the equipment could resist the effects of shock. The guided missile became the preferred weapon during the Cold War, and fluid dampers were used by the military as the most economical way of protecting missiles against the detonation of conventional and nuclear weapons. Thus, in the 1900–1945 period, the technology of fluid dampers became known to the armies and navies of many countries. However, as the technology was mostly classified, it was not widely publicised.

In the late 1980s, with the end of the Cold War, much of this fully developed defence technology became available for sale to the general public. In the USA, Taylor Devices, which was the supplier to the US Government of dampers and shock absorbers, took the initiative and teamed up with the State University of New York at Buffalo to study the application of these devices to buildings and bridges to improve their seismic performance.

Because of the similarity in design between a damper and an STU, early hydraulic devices used to damp or limit motion in structures such as the aforementioned Carquinez Strait Bridge, could conceivably be classified as either type of component. Until the end of World War II, most hydraulic components lacked precision fits and sealing techniques, and thus could not be considered as anything other than crude dashpots. To produce a realistic STU, high velocity orifices must be bored in relatively large components, with controlled orifice flow areas of less than one-millionth of the cylinder bore area. This was not possible until the introduction of computer-controlled machinery. During the 1960s, a fluid shock-transmission unit was developed by the United States Navy (Taylor, n.d.-b) to restrict the motion of large electronics drawers on ships. While under normal conditions the drawers could be easily opened and closed manually by the ship's staff, under conditions of severe sea storm or weapon explosions they became loose or floating, injuring crew and breaking equipment. This potential hazard was avoided by fixing a small STU to the drawer which locked the drawer in position until the temporary event had passed. These STUs were manufactured for capacities of up to 5000 lb, and movements of up to 30 in. During the period 1966–1980, large numbers of these devices were manufactured for this application in the shipping industry.

In the 1960s, a similar technology to STUs was developed by various industrial hydraulic manufacturers (Taylor, n.d.-b). The subsequent product, known generally as a ‘snubber’, was used on nuclear power plant equipment and pipelines to restrict motion in the event of pipe failure, surges and earthquakes. Snubbers were manufactured from a variety of hydraulic parts, generally including a hydraulic cylinder, an accumulator and numerous spring-loaded flow control and check valves.

As stated in Section 1.1, the first known use of STUs on a bridge structure was on the Carquinez Bridge in California, USA, designed by D.B. Steinman in 1927 (HITEC, 1999). Four units were installed, one each in the lower chords of the suspended span expansion joints in two 1100 ft main spans. The units functioned simply by moving oil from one side of the tight piston in a cylinder to the other through a very small tube. Under seismic loading the internal oil could not flow fast enough, and the piston ‘locked up’. The next application of STUs in the USA was on the San-Mateo Hayward Bridge, also in California, in 1967. Four units were installed, two in each of the twin box girders at the expansion joint in the 750 ft main span. These units were similar to those used on the Carquinez Strait Bridge, and were composed of a tight piston in a cylinder moving oil from one side to the other. Since the mid-1990s, STUs have been installed on bridges in the USA in California, Pennsylvania, Georgia, Kentucky, Missouri, Illinois, Arkansas, Idaho, Oregon, New York and Washington, and many more are planned or being installed on bridges in other US states. In Canada, STUs were used for the first time on the Arthur Laing Bridge in Richmond, Vancouver, British Columbia.

STUs were also developed in Europe. STUs were used extensively on the 5 km long Oosterschelde Bridge in Holland, completed in 1965. STUs of up to 180 ton capacity were placed at the expansion joints located between the twin cantilever spans forming each 90 m span concrete box girder deck. The STUs transmitted traction and braking across the central expansion joint, and offset the joint rotation movements resulting from heavy vehicles travelling on the cantilevered decks, with only minimal resistance to slow movements.

In Italy, the first device installed on a bridge to protect the structure from seismic action dates back to 1974, before the earthquakes of 1976 in Friuli and 1980 in Irpinia. The event in Irpinia prompted great interest in seismic protection systems, and the Italian road authorities approved a seismic protection system containing STUs, manufactured by FIP Industriale of Padua, Italy, for installation on the Savio Viaduct on the E47 Expressway.

These were the first STUs manufactured by FIP, and they are still functioning satisfactorily after about 35 years of operation and a few earthquakes. These early versions of STUs consisted of a cylinder, with heads joined by tie rods, in which a sliding piston creates two chambers filled with hydraulic oil. The two chambers were connected by means of a calibrated orifice protected by filters. The connection of the pier and the bridge was effected by means of hinges.

STUs were also utilised on the approach spans of the Kingston Bridge, completed in Glasgow in 1970 (Fairhurst *et al.*, 1971), and the units were found to be in good working condition during recent inspections. In 1979, four 25 ton STUs were installed on the abutments of the Stour viaduct, part of the Sandwich bypass, in Kent, England. In 1989, for the upgrading of the London Docklands Light Railway, STUs were installed, linking the joints between adjacent segments of the seven-span viaduct. This warranted the engagement of additional piers from adjacent spans to resist traction and braking forces of a revised longer train on any one span. STUs have been used recently in Australia, the Philippines, India and China, and in many other Southeast Asian countries.

STUs have also been used in several tall buildings to counter wind loads, and for vibration control and traction and braking force distribution on highway and railroad structures. Today, engineers around the world are specifying STUs for suspension and cable-stayed bridges, as they allow the bridge to move due to temperature, shrinkage and creep, while resisting sudden wind storms and seismic events.

## 1.3. Progression of early STUs to present-day, maintenance-free, compact STUs

The early STU was a relatively complex oil-filled or gas-filled device, with a high initial cost and a continuing need for regular and expensive maintenance and adjustment. Most of them failed due to poor sealing techniques. These early STUs used different fluids, generally aviation-type oils, which led to two problems.

- The seal could be guaranteed only through the use of elastomeric gaskets. This involved a regular maintenance regimen, as this type of gasket would not normally last more than 5 years.
- The aviation-type oils used had a high thermal expansion coefficient, so the STUs had to be provided with an oil-filled expansion pot incorporated in the valves (accumulator) in order to compensate for the variation in the volume of the fluid at different temperatures. This requirement for an accumulator made them bulky and costly.

These complications made the early STUs complex, heavy and unreliable, as their function could not be guaranteed unless regular inspection and maintenance was provided. Therefore, in the early days, engineers did not have a great deal of confidence in STUs, and this restricted the use of this very useful device.

In the 1960s, a unique chemical compound of boron-filled dimethyl siloxane, known as ‘silicone putty’, was developed in the USA and was used in the space exploration programme. It offered special thixotropic properties: deforming readily under slowly applied pressure, but acting as a near-rigid body under impact. John Chaffe and Reg Mander, along with the then UK Ministry of Transport, recognised that the new material was highly suited to STUs, offering a better and simpler filler material than the special oils and gases used previously (Pritchard, 1992). They developed and patented in the UK an STU using the silicone putty (Mander and Chaffe, 1970). Later, the then UK National Research and Development Corporation granted a license to Colebrand of the UK to market and further develop the Chaffe–Mander STU. Improvements in seal and silicone technology, together with full-scale dynamic testing capabilities, led to long-term, guaranteed and verifiable performance of STUs that the first-generation devices employed earlier on the Carquenez Bridge in California, USA did not have. In the USA, the first silicone putty STU was installed in 1969 by the California Department of Transportation (Caltrans) on the Dumbarton Bridge. This bridge has effectively withstood three earthquakes without any sign of distress. In 1996, Caltrans notified STU manufacturers in the USA that the silicone-putty-based STU was an approved product for Californian bridges, and in 2000 the Carquinez Strait Bridge built in 1958, which had oil-based STUs similar to the ones used on the original 1927 Carquinez Strait Bridge, was retrofitted with six 16 000 kN nominal rated capacity STUs. These are the highest capacity STUs used on any bridge in the world to date.

The use of silicon oil or silicon putty in modern STUs overcomes the two problems experienced by the early STUs containing aviation oils (see above) and makes them compact and more or less maintenance free and less costly. Because of this, and their simplicity and tolerance of a wide range of environments, modern STUs are opening up a whole range of possibilities in the construction industry. The military sees them as having interesting possibilities in the many bridging and other logistic devices that underpin all modern military movement on land.

Until 2002, no specifications for STUs were included in any bridge design code. In 1999, the Highway Innovative Technology Evaluation Center (HITEC, 1999) evaluated, by load testing, STUs manufactured by Colebrand of London, and published a technical evaluation report *Guidelines for Evaluation and Testing of Shock Transmission Units* (HITEC, 1999). Then, in 2002, the American Association of State Highways and Transportation Officials (AASHTO) included 'Interim Specifications for Shock Transmission Units' in Section 32 of its *LFRD Bridge Construction Specifications*, based on this evaluation report by HITEC. Also in 2002, Eurocode 8, 'Draft Code: Design of Structures for Earthquake Resistance, Part 2: Bridges', included a specification for STUs. In 2006, the interim AASHTO Specification for STUs, released in 2002, was approved and revised within Section 32 of the AASHTO Bridge Specifications.

STUs are currently manufactured in the USA, Canada, France, Italy, Germany, Switzerland, the UK and a few other countries, and, to the best knowledge of the author, there are about 12 manufacturers in these countries, each offering different designs and options. The author was involved in the procurement and selection of STUs for the New Badiwan Bridge in the Philippines during the early stages of the project before construction started in 1998, which was the first application of STUs on a bridge in the Philippines. Later, in 1999, the author had a major involvement in the use of STUs on the Second Bassein Creek Bridge near Mumbai, India, and faced, in the absence of technical literature and standard specifications, various problems resulting in unwarranted delays in procuring, testing and installing the STUs on the bridge.

However, over the years, the STU has been developed to become a robust, compact, low-cost and virtually maintenance-free device, with no moving parts apart from the piston. The technology of STUs has changed considerably since the earlier applications mentioned above, making these devices more reliable and economical.

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# Chapter 2
# Seismic protection systems

## 2.1. Introduction

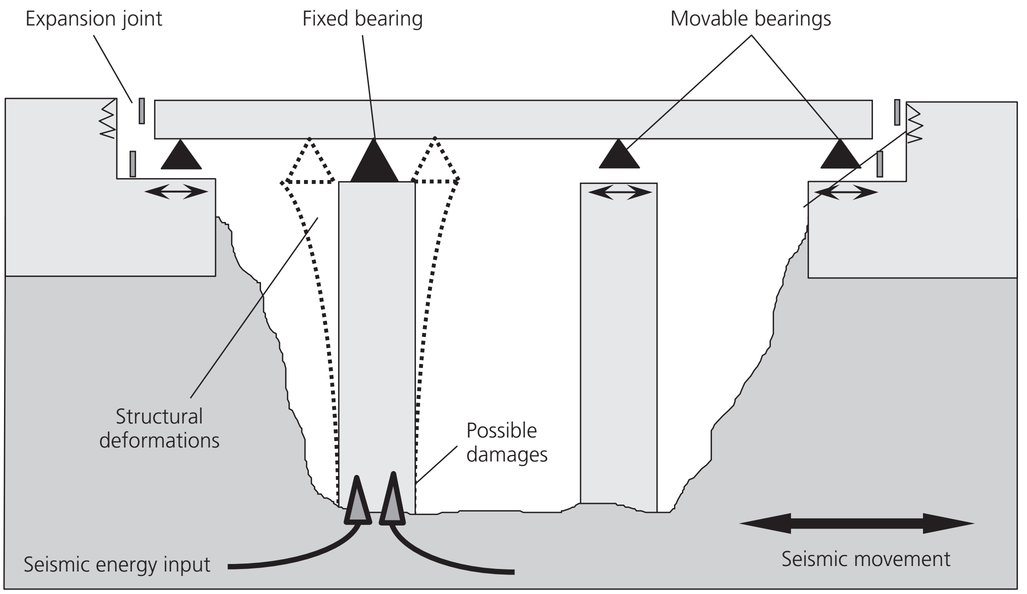
Seismic protection systems generally comprise seismic isolators, hydraulic dampers, STUs and seismic expansion joints. While STUs are used to transfer, distribute and share traction and braking loads from vehicles and trains, and to restrict deck movement due to wind gust, they are more frequently used to withstand the effects of earthquakes. It is therefore appropriate to consider STUs as part of present-day seismic protection systems, which include seismic isolation and energy-dissipation devices.

The first idea that comes to the mind of engineers is to interpret an earthquake in terms of the deformation it induces in a structure. Hence, with regard to seismic protection, there is a tendency to think about increasing the strength of a structure. However, increasing the strength leads to massive structural dimensions, resulting in a much stiffer structure, and the forces within the structure consequent to a seismic event lead to enormous local energy accumulation and plastic hinges. This conventional concept of structural strengthening with the usual bearing arrangements permits plastic deformations, leading to yield stress and cracks.

Figures 2.1 and 2.2 show a conventional bridge system and its behaviour during an earthquake. Under service load conditions, the isostatic system prevents unexpected strains resulting from horizontal movements. However, in the case of an earthquake, the structural deformations occur on the fixed pier between the substructure and the superstructure. As soon as the elastic limit of the fixed pier is exceeded, plastic deformations will lead to structural damage in the most stressed areas at the base of the pier. A further strengthening of the pier will increase the mass, and therefore will result in an increase in the inertial forces in the structure. This means that not only will the seismic resistance be increased, but so too will the amount of energy entering the structure, leading to larger deformations. Furthermore, the large degree of movement in the bridge deck can damage the expansion joints at the bridge ends.

Therefore, in order to protect structures effectively, design strategies must be devised in accordance with the inherent nature of the destructive phenomena. Housner suggested an energy-based design strategy in 1956, but it was only in the last quarter of the twentieth century that the design strategy of seismic isolation and energy dissipation based on an energy approach was adopted. Earthquake engineering has witnessed significant developments during the course of the last quarter century. The progress has been made firstly owing to parallel development of new design strategies (seismic software) and the perfection of suitable mechanical devices to implement such strategies (seismic hardware), and secondly due to an awareness that energy dissipation can be a powerful tool in the hands of the design engineer to control the response of structures struck by earthquakes and windstorms. In other words, as stated earlier, these natural events are being increasingly perceived as phenomena

**Figure 2.1** A conventional bridge system




involving the transmission of mechanical energy, rather than being interpreted only in terms of forces and deformations resulting from the simple application of mathematical equations. However, this awareness has mainly concerned the academic world, and has been appreciated only to a limited extent by civil engineering practitioners.

## 2.2. Energy concepts for seismic protection of structures

There are two other concepts, apart from the conventional design concept noted above, for the seismic protection of structures.

- Energy sharing.
- Energy mitigation.

**Figure 2.2** Bearing arrangement for a conventional bridge system (isostatic system)

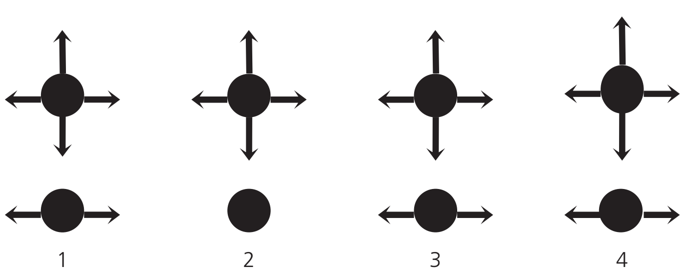
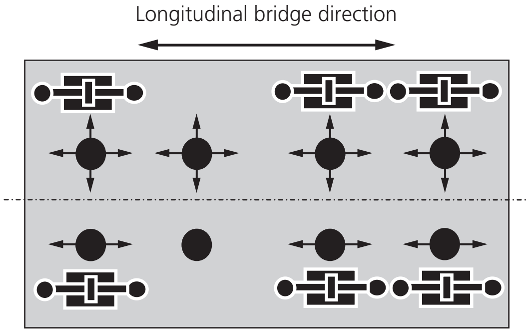


Figure 2.3 Bearing arrangement with integrated STUs




### 2.2.1 Energy sharing

Energy sharing means that a structure is designed such that the seismic energy resulting from an earthquake event enters the structure at as many locations as possible. Thus the distribution of stress will be more or less constant over the entire structure in order to avoid significant energy accumulation at only a few structural locations.

STUs (Figure 2.3) permit slow relative displacements (e.g. due to temperature differences, or creep and shrinkage) between the deck and the substructure during service conditions (Figure 2.4). Only in the case of seismic attack or a suddenly induced traffic force do the STUs behave like a rigid device (Figure 2.5), distributing the horizontal seismic or traffic response forces to several structural elements. Consequently, the capacity of the structure to store elastic and kinetic energy is increased, and the resulting structural displacements are smaller than for those structures without STUs. The ensuing horizontal forces are largely distributed evenly over the structure. Therefore, while the energy resulting from an event is still entering the structure, unequal energy distribution within the structure is avoided. This principle works very well and is proven to be economical for small seismic events and moderate lateral forces.

### 2.2.2 Energy mitigation

To be even more effective, the applied seismic concept has to take into account the nature of the earthquake. The distribution of the energy input to several structural locations is not sufficient to protect the

Figure 2.4 Bridge during service load, with inactive STUs

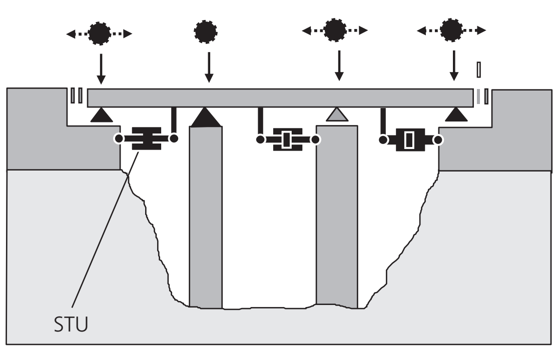
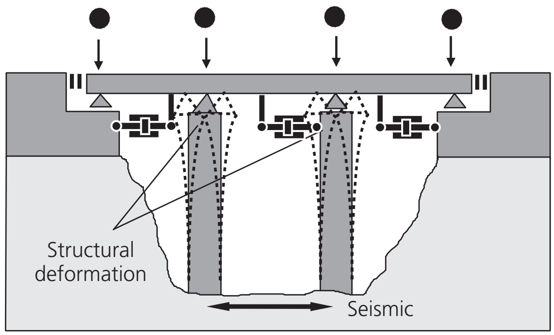


Figure 2.5 Bridge with active STUs during ultimate load, such as an earthquake




structure in case of a severe seismic attack, and for a medium to severe seismic attack the concept of energy mitigation for seismic protection needs to be applied.

Energy mitigation is the application of the concept of the 'energy approach' while in addition considering the 'energy disposition' of earthquakes. Two methods are applied simultaneously: seismic isolation and energy dissipation.

### 2.2.2.1 Seismic isolation

In seismic isolation, the superstructure is decoupled from the ground. This automatically limits to a minimum the amount of energy that enters the superstructure during an earthquake. Because of this, the natural period of the structure is increased, and the spectral acceleration during a seismic attack is reduced (Figure 2.6). Multidirectional seismic isolators transfer the vertical loads and actively support the re-centring of the superstructure during and after an earthquake (Figures 2.7 and 2.8). Re-centring means that the displaced bridge deck (due to the seismic energy input) is automatically shifted back to its original position by means of the seismic isolators.

### 2.2.2.2 Energy dissipation

The remaining part of the seismic energy that still enters the structure can effectively be dissipated by means of additional damping devices. The passive energy dissipation (energy transformation)

Figure 2.6 Characteristic response spectrum of a bridge

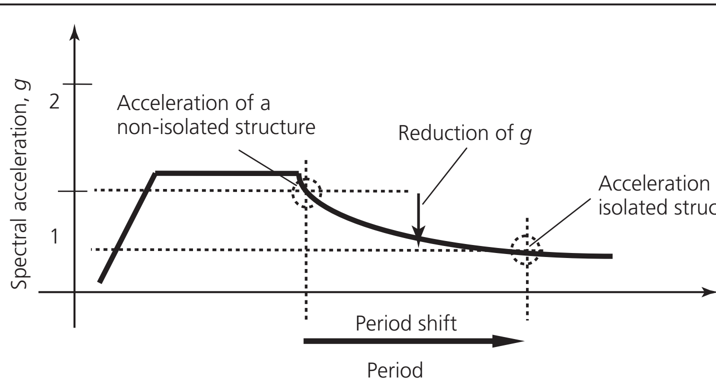
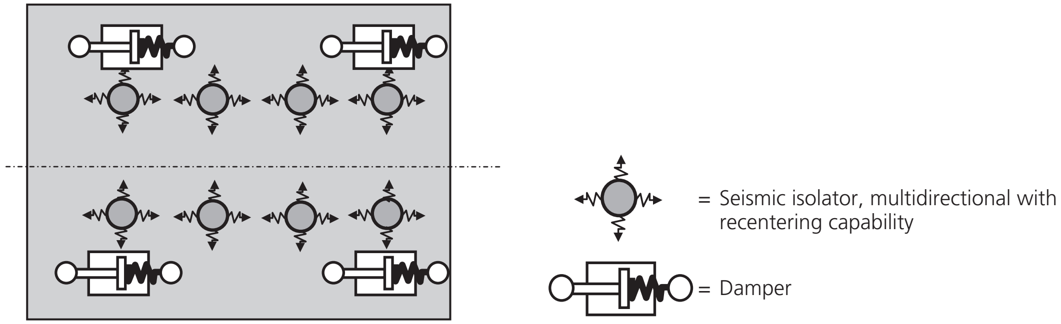


Figure 2.7 Bridge deck for a seismic isolator-and-damper arrangement




protects the entire structure from additional strain (see Figures 2.7 and 2.8). Dampers can be installed in the longitudinal as well as the lateral direction. This combination of seismic isolation and energy dissipation leads to the best possible seismic protection for structures.

The penetration of the energy resulting from an earthquake into a bridge structure designed using conventional design concepts and with the above two energy-sharing concepts is shown in Figures 2.9 to 2.11.

## 2.3. Energy approach

In order to appreciate the above development of seismic protection systems, let us start from the general concept of:

$$\text{Demand} < \text{Capacity} \tag{2.1}$$

where the terms Demand and Capacity assume, from time to time, their usual meaning.

Figure 2.8 Bridge with seismic isolation and passive energy dissipation (for the arrangement shown in Figure 2.7)

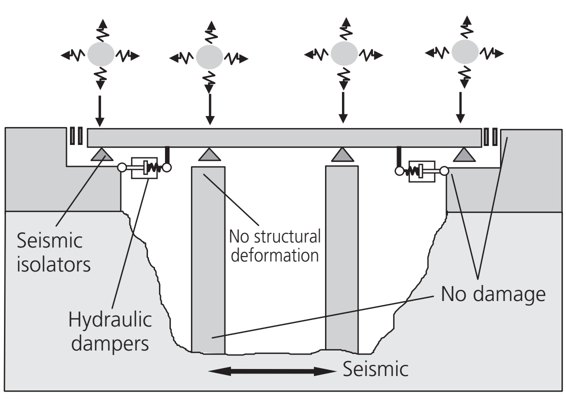


**Figure 2.9** Without a seismic protection system the energy penetrates the structure in a concentrated manner

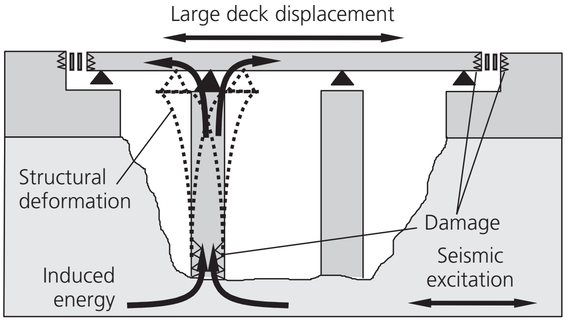



**Figure 2.10** Seismic energy can be distributed to several piers by using STUs to enable energy sharing

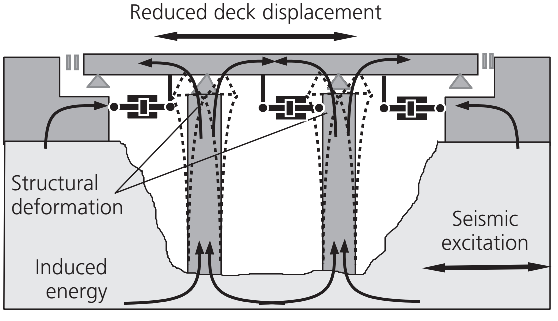



**Figure 2.11** Minimised penetration of seismic energy by implementing seismic isolation (isolators) and energy dissipation (dampers)

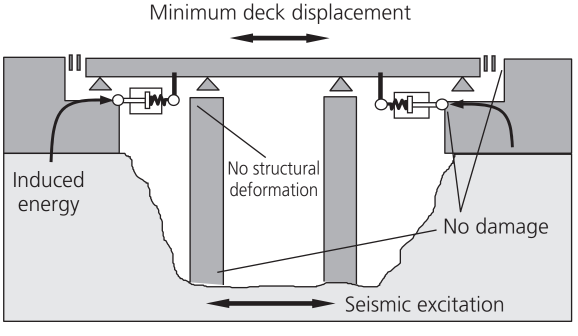
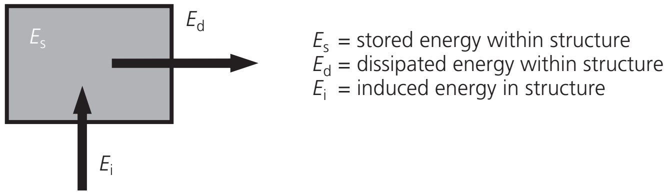


Figure 2.12 Physical system and energy exchanges in accordance with the principle of energy conservation




Equation 2.1, sometimes referred to as the 'design equation', is valid in earthquake engineering, even when the energy concepts are applied in the sizing of the structural members.

Let us consider a generic physical system that interacts with the external environment through energy exchange (Figure 2.12):

$$E_i = E_s + E_d \tag{2.2}$$

where $E_i$ is the energy input to the structure, $E_s$ is the stored energy in the structure and $E_d$ is the dissipated energy in the structural system.

If one considers a structure such as a building or bridge experiencing a seismic event, then the term $E_i$ represents the mechanical energy transmitted to the structure through its foundations by the ground motion. The energy $E_s$ can be stored in the structure in two ways, one of which depends only on deformations and the other only on the velocity:

$$E_s = E_e + E_k \tag{2.3}$$

where $E_e$ is the elastic strain energy stored into the structure and $E_k$ is the kinetic energy.

The energy $E_d$ can be dissipated by two distinct mechanisms, one of which depends only on the deformations and the other only on the velocity:

$$E_d = E_h + E_v \tag{2.4}$$

where $E_h$ is the energy dissipated by hysteretic (or plastic) deformation and $E_v$ is the energy dissipated by viscous damping.

The energy $E_v$ is associated with the seismic forces $\boldsymbol{F}$ that depend only on velocity $\boldsymbol{V}$ through a constitutive law of the type

$$\boldsymbol{F} = \boldsymbol{C}\boldsymbol{V}^n \tag{2.5}$$

where the exponent $n$ ranges from 0 to 1.8 depending on the type of device.

By substituting Equations 2.3 and 2.4 in Equation 2.2, we derive the energy-balance equation in the following form, which is valid for structures:

$$E_i = E_e + E_k + E_h + E_v \tag{2.6}$$

In this equation, $E_i$ can be interpreted as Demand, and the four terms on the right-hand side represent the possible capacities of the structure.

When structures are designed adequately for a seismic event according to the conventional approach, in practice the use of only the terms $E_e$ and $E_k$ is required. However, it should be noted that, even though it remains within elastic limits, the structure has intrinsic dissipating capacity of the viscous type, and thus the term $E_v$ comes into play. However, the above approach often represents an illusion, as, when using this approach, seismic protection can only be ensured for slender structures subject to modest intensity earthquakes.

When the energy transmitted to the structure by the earthquake exceeds the structure's capacity to store it elastically, portions of the structure typically yield or crack. In other words, it can be stated that, in such cases, the structure automatically resorts to the third term $E_h$ of the energy-balance equation.

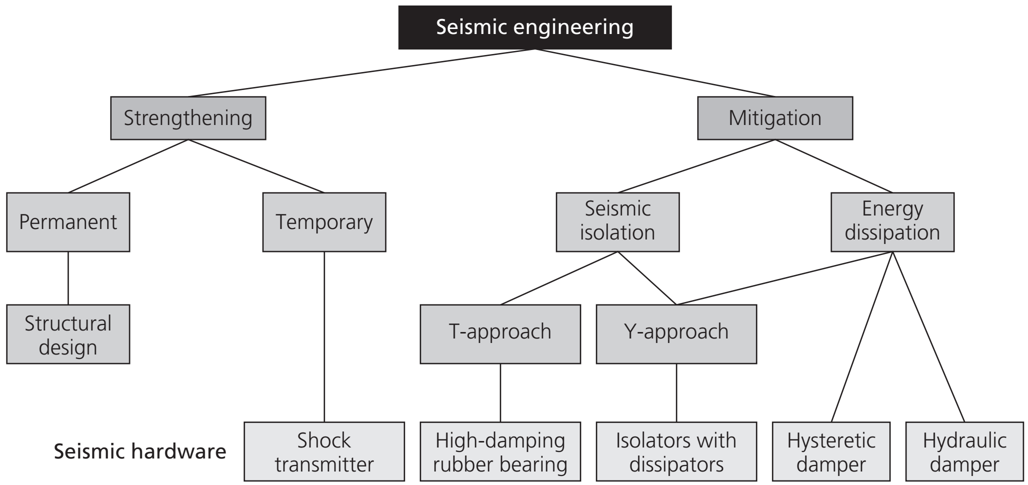
Unfortunately, structures are still being designed using the term $E_h$, implying that it is accepted that structural members will undergo deformation beyond their elastic limit, resulting to their ductility. In fact, accepting deformations beyond the elastic limit means resorting to a dissipating mechanism that induces permanent structural damage, and thus accepting the need for costly repairs and the fact that the structure will be temporarily out of service. This design approach, termed 'traditional' or 'conventional', is still accepted by most existing anti-seismic standards. However, the drawback is the risk of structural collapse if there is a seismic event that exceeds the design earthquake intensity.

Only in recent years it has been recognised that it is possible to significantly increase at will both $E_h$ and $E_v$, and thus to fully control the response of the entire structure by placing energy-dissipating devices at properly determined strategic locations. This is referred to as the 'passive energy dissipation' approach. If this approach is not technically feasible or is economically disadvantageous, there is the option of balancing the energy input $E_i$ using the terms $E_e$, $E_k$, $E_h$ and $E_v$ by decreasing the energy input itself. This design approach is called 'seismic isolation', and essentially entails decoupling the prevailing structural mass from its foundations (this approach is, therefore, sometimes improperly referred to as 'base isolation').

From the above, one can conclude that the most rational approach is to utilise all the terms in the energy-balance equation (Equation 2.6), which can be achieved by means of a combination of seismic isolation and passive energy dissipation. Seismic isolation was proposed over a century ago (Kelly, 1982) but it has found extensive application only during the last three decades, the delay being due to a lack of adequate hardware. Seismic isolation is generally achieved by means of isolators such as high-damping rubber bearings, friction sliders with hysteretic dissipaters, lead–rubber bearings, etc., and passive energy dissipation is achieved generally by means of hysteretic and hydraulic dampers.

Seismic isolation can be achieved in two ways. One system comprises mainly high-damping rubber bearings, which also have a modest energy-dissipating capacity, and this is known as the T-strategy. The other system comprises numerous devices such as friction pendulums, friction sliding bearings, lead–rubber bearings and friction sliding bearings with steel hysteretic dissipaters, and this is known as the Y-strategy. In this system, a high level of energy dissipation also takes place, and so it represents a combination of the two strategies of seismic mitigation described above.

Figure 2.13 Flow chart to show some concepts that interrelate design choices, and anti-seismic devices suitable for their practical application




Seismic isolation and energy dissipation represent the most efficient tools currently available to design engineers working in seismic areas to limit both relative displacements and transmitted forces between adjacent structural elements to desired values. This means being able to control, at will, the structure's response, and at the same time ensure the required degree of protection.

## 2.4. Seismic hardware

Today, engineers can rely on a number of solutions and various types of seismic device that have been used with success over the last three decades. The various solutions can be grouped into two main types.

- Those that provide structural members with sufficient flexibility, strength and ductility to absorb and dissipate the energy input – these solutions are part of the 'conventional' design approaches.
- Those that aim to protect the structure against earthquake damage by limiting the effects of a seismic attack (rather than resisting it) through the use of seismic devices properly inserted into the structure – these solutions are usually referred to as 'seismic mitigation'.

The flow chart in Figure 2.13 illustrates some concepts that interrelate design choices, and anti-seismic devices suitable for their practical application. It can be seen from the chart that the following two alternatives are possible for the conventional design technique, which essentially consists of strengthening the structure as follows.

- Provide the structure with permanent restraints only, and its members with adequate flexibility, strength and ductility.
- Insert temporary restraints (STUs) at strategic points in the structure.

The superior seismic behaviour of hyperstatic structures, and bridges in particular, is well known. The simple explanation for this is that in hyperstatic structures all the structural members are forced to work together at a critical moment such as an earthquake event. However, particularly in the case of bridges, the required construction techniques and the risk of differential settlement of foundations often suggest the choice of an isostatic arrangement. During an earthquake event STUs create temporary restraints and tie the structure together, but also allow slow displacements due to temperature variation, shrinkage and creep. As a consequence, the structure remains isostatic under service loads but becomes hyperstatic during a seismic event through the creation of temporary restraints. In the energy-based approach, by forcing all the structural members to tie together, STUs increase both the overall capacity of the structure to store energy and its capacity to dissipate the energy through the intrinsic viscous mechanism.

The seismic hardware currently used to implement the T-strategy mainly comprises high-damping rubber bearings. These are essentially laminated rubber bearings manufactured from dissipative elastomer. The devices developed for the Y-approach comprise many different types, the most important being the following.

- Friction pendulum.
- Friction sliders.
- Lead–rubber bearings.
- Various types of slider with steel hysteretic dissipaters.

In the first two devices, the dissipation of energy is achieved through friction, whereas in the last two it is achieved by means of the plastic deformation of a metal.

It is important to point out that, in both new and retrofit seismic projects, the selection of the type of seismic devices to be employed must not necessarily be a single type of device. In many cases, the adoption of combinations of devices can result in a significant advantage. Several combinations of bearings (elastomeric, pot or spherical) and steel hysteretic dissipaters and STUs are possible, and the choice of an optimal combination depends on the requirements to be fulfilled (elastic stiffness, post-elastic stiffness, amount of displacement, dissipative efficiency, etc.). Today, seismic engineers are showing increased interest in the adoption of different types of seismic device within the confines of a single project. Chapter 17, 'STUs in conjunction with other seismic-protection devices', includes some applications of STUs used in conjunction with other seismic protection systems based on seismic-isolation and energy-dissipation concepts.

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# Chapter 3
# Shock transmission units and their applications

## 3.1. Introduction

Bridge engineers had always hoped to tie the elements of a bridge structure together, but had been unable to do so because a bridge must be permitted to move in response to slowly applied loads such as temperature, shrinkage and creep. This dream became a reality for engineers with the advent of STUs.

STUs are known by various other names such as lock-up devices (LUDs), displacement-control devices (DCDs), rigid connection devices (RCDs), seismic connectors, buffers and snubbers. An STU is designed to be connected between structural members to form a rigid link under rapidly applied loads, but to allow the structure to move freely under slowly applied loads. Such a temporary fixed connection facilitated by an STU allows load sharing in response to a rapidly applied force. Longitudinal traction, braking forces, vehicle impact, seismic and hurricane winds, etc. are examples of such short-duration horizontal loads applied suddenly to structures, transmitting short-duration shocks or impact forces.

## 3.2. Modern STUs

The present-day STUs shown in Figures 3.1 and 3.2 are very simple and compact devices, in which the piston is the only moving part, and it is virtually maintenance free. It comprises a cylinder filled with a silicon compound, either fluid or putty, with a piston mounted on a transmission rod through the cylinder. The components of a modern STU are described below.

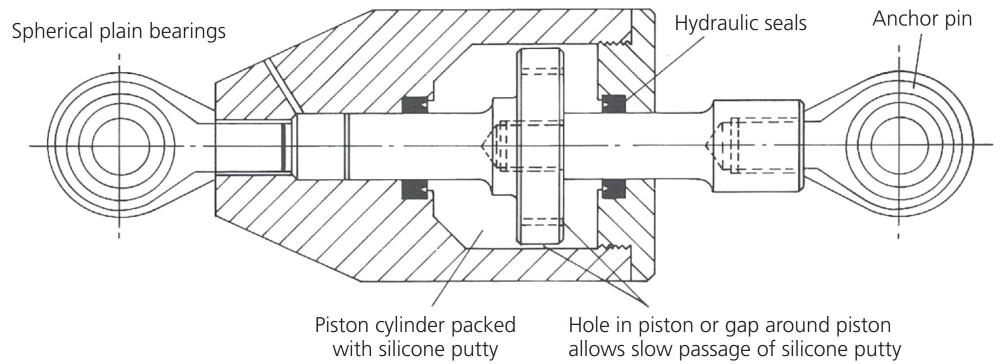
### 3.2.1 Cylinder

The cylinder is normally made from seamless steel tubing, as welded or cast construction is prone to cracking. It contains the silicon compound (fluid or putty) and must be adequate to carry the pressure vessel loading, and therefore it is designed for a minimum proof pressure loading equal to 1.5 times the internal pressure under the anticipated maximum loading. The cylinder is normally made from corrosion-protected steel and painted to meet the projected environmental requirements. If the site environment is extremely corrosive, the cylinder can be manufactured from stainless steel, but this is rather costly.

### 3.2.2 Piston rod

The piston rod is relatively slender and must support the load acting on the piston, which acts like a column, and it is therefore normally manufactured from high-strength stainless steel. Stainless steel is preferred as any rust on the rod surface can cause seal failure. It is sometimes necessary to chrome plate the piston rod for compatibility with some types of seal material used to prevent the internal medium

Figure 3.1 A modern STU – Colebrand, putty based. © Colebrand International Ltd




leaking from the cylinder. The piston rod passes through the entire length of the cylinder, so that the volume of the cylinder and the silicone-based compound remains constant at all piston positions. The piston rod at the moving end is covered by a flexible gaiter made of a durable material.

### 3.2.3 Piston head

The piston head is attached to the piston rod, and is also made from high-strength stainless steel. The head has orifices allowing the internal medium to pass from one side of the piston to the other.

### 3.2.4 Orifices

The orifices control the flow of the internal medium under pressure across the piston head. They can be finely drilled holes less than one-millionth of the cylinder bore area, which are made using computer-controlled machinery, or alternatively they can be special valves.

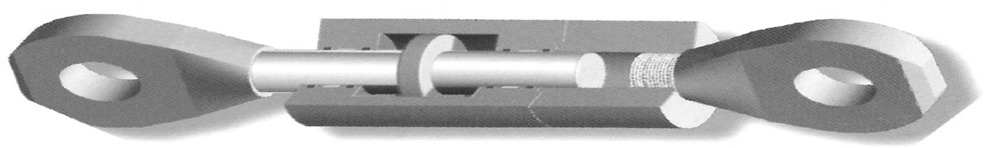
### 3.2.5 Seal

The seal closes the open ends of the cylinder. While seals are available commercially, most STU manufacturers have their own proprietary machined seal, as the quality is of paramount importance for the long-term performance of the STU (it can make or break the STU for a manufacturer). The seal must not degrade with age, must be compatible with the internal medium and must have a long service life without requiring periodic replacement.

### 3.2.6. Internal medium

Two types of internal medium are normally used in STUs today: oil or putty. Both types are silicone-based materials. The American Association of State Highways and Transportation Officials

Figure 3.2 Cut section of a modern STU – ALGA, oil based. Reproduced courtesy of Antonio Marioni, ALGA, 1991



(AASHTO) specification for STUs stipulates an Occupational Health and Safety Act (OSHA) approved, non-toxic, inflammable, silicon-based fluid or silicon putty medium. The precise composition of the internal medium is generally not revealed by manufacturers. Each manufacturer has its own formula to keep the properties of the internal medium constant over time and variations in temperature; it is generally proprietary in nature and patented by the manufacturer.

Silicone oil is cosmetically inert and thermally stable, and has a flashpoint in excess of 340°C. It is produced by distillation, so is uniform in nature and does not settle over the long term or degrade with age. Silicone oil has been used in dampers for a long time, even before STUs evolved, and has performed satisfactorily in the field.

Silicone putty is also inert. In the early 1960s, when silicone putty was first developed, it was common to use Freon- or benzene-based materials to enhance low-temperature operation. The use of such materials would be potentially disastrous to the environment, and is illegal today. However, STUs containing silicone putty have been installed in hot and cold climates in many countries, and it may be that manufacturers have now overcome the initial problems. That said, users are advised to evaluate such STUs by load testing for any of the two filler mediums used, under specific site environmental conditions.

The biggest disadvantage of STUs containing silicone oil is that the medium can leak out from the seals very easily and rapidly compared with putty. There is still some doubt, though, in the minds of many engineers about the durability of silicone putty over time and with variations in temperature. The time-dependent properties of silicone putty, while confirmed by accelerated ageing tests to some extent, have only been proven in the field for some 35 years, as STUs containing silicone putty have only been used since the 1970s. The first silicone putty STUs were installed on Dumbarton Bridge in California in the 1970s by the California Transport Department (Caltran), which only approved the use of silicone putty in STUs in 1996 after monitoring this bridge for some 25 years. Although Caltran found the silicone putty to be acceptable, some engineers in the USA have expressed doubts and still think that STUs containing a silicone putty-based medium should be accepted only once the STU has been shown to remain fully functional for the 75–100 years of service life of this structure in the field.

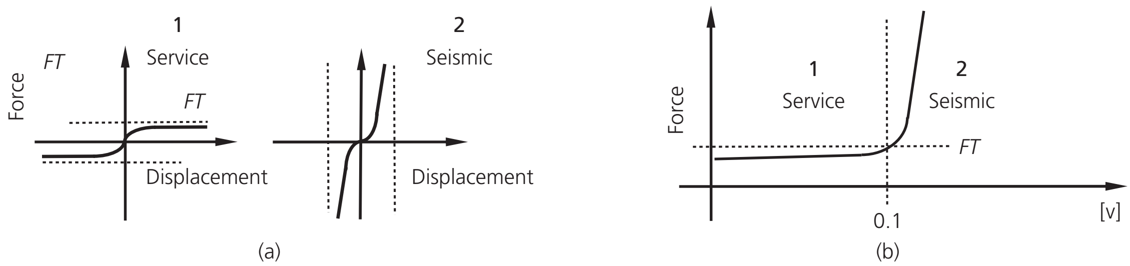
### 3.2.7 Fixing eyes with spherical bearings

The STU is attached to the structure or structural elements by means of fixing eyes located on the cylinder at one end and the transmission rod at the other (see Figure 3.1). Spherical bearings are provided at the mounting interfaces to the structure to align the STU in the direction of the movement. The use of spherical bearings rectifies any misalignment in the installation of the STU by allowing multi-axis end rotation of the STU, thereby preventing bending and providing ease of rotation of the STU by about 3°. These bearings can be made from stainless steel, plated steel or polymer/poly coated steel, depending on the project requirements. Stainless steel clevis pins through the spherical bearings connect the STU to the structure mounted on clevis-type brackets at each end.

## 3.3. Principle of operation of STUs

STUs work on the principle that rapid passage of viscous fluid through a narrow gap/orifice or valve generates considerable resistance, while slow passage generates only minor resistance. Depending on the displacement velocity, the STU reacts with a certain nominal response force. Very slow displacements (due to temperature changes and creep/shrinkage) cause minor response forces $FT$ within the STU (Figure 3.3). The silicon medium inside the STU flows from one side of the piston to the other within the hydraulic cylinder without causing any appreciable response force.

Figure 3.3 Plots of (a) force versus displacement for an STU and (b) force versus velocity for an STU




During sudden-impact accelerations (due to earthquakes, braking actions of vehicles, etc.), which result in greater relative displacement velocities between the superstructure and the substructure (above approximately 0.1 mm/s), the STU reacts with an intense increase in its response force (see Figure 3.3). The silicon medium inside the STU is not able to flow fast enough from one side of the piston to the other, and the device locks up and blocks any relative displacement between the connected structural parts.

## 3.4. Function of STUs

An STU is designed to perform in either tension or compression. As a reversal in motion occurs during an earthquake, the device locks almost instantaneously in each direction.

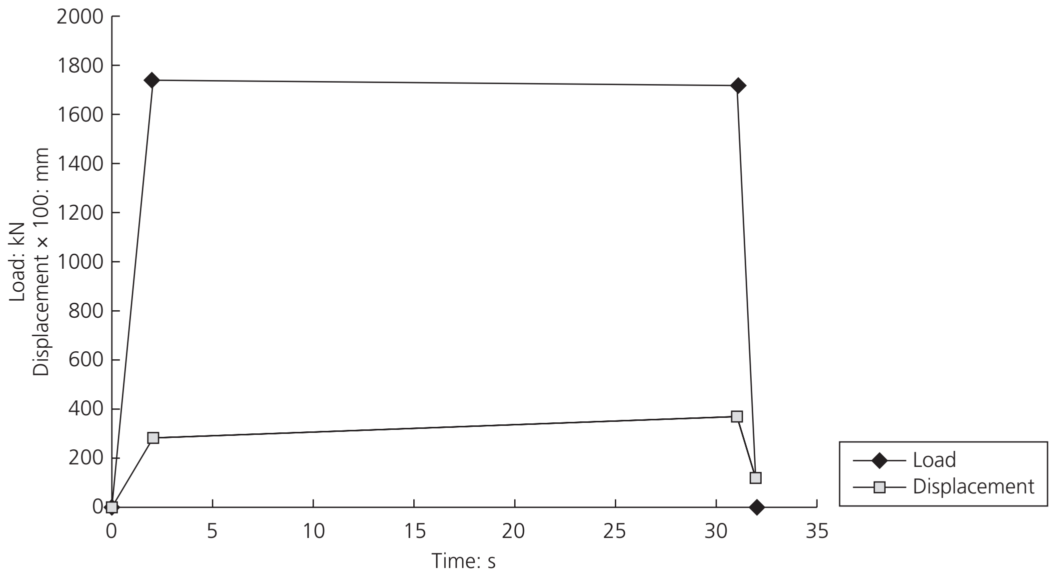
The load versus displacement characteristic for a Colebrand STU of rated capacity 1750 kN in response to a rapidly applied load equal to its design load being held for just over 30 s is illustrated in Figure 3.4.

During the first 2 s of load application in the piston displaces, due to elasticity of the steel cylinder under pressure, less than 3 mm (in a linear time versus displacement manner). After an elapsed time of 2 s to approximately 30 s, the load is maintained at a constant value of 1750 kN. During this period there is a small additional movement of the piston of just over 1 mm, due to creep, bringing the total deformation of the STU over the duration of the period to 4 mm. After an elapsed time of 30 s the load is removed, and there is a near instantaneous recovery of the piston to within 1.5 mm of its original position.

In the installed situation, the STU would, after the elapse of a further short period of time, return close to its original neutral position due to the strain energy stored in other structural elements such as supporting piers or elastomeric bearings. The stroke of an STU is normally established by matching it to the anticipated relevant movement of the superstructure to which it is connected.

In Figure 3.5, the normal operation of a typical 50 ton capacity STU is shown and the graph depicts resistance typical of what might be applied by a STU during the slow movement of the structure. When a short-duration sudden force is applied to one structure, there is negligible lock-up displacement and the dynamic tensile or compressive force is passed along the load path of transmission rod/piston head/ silicone putty/cylinder to the second structure. The rating of a unit defines the maximum impact force which can be transmitted, and the length of the transmission rod can be varied to suit the expected

Figure 3.4 Load and displacement versus time for a Colebrand 1750 kN STU (±60 mm stroke). © Colebrand International Ltd

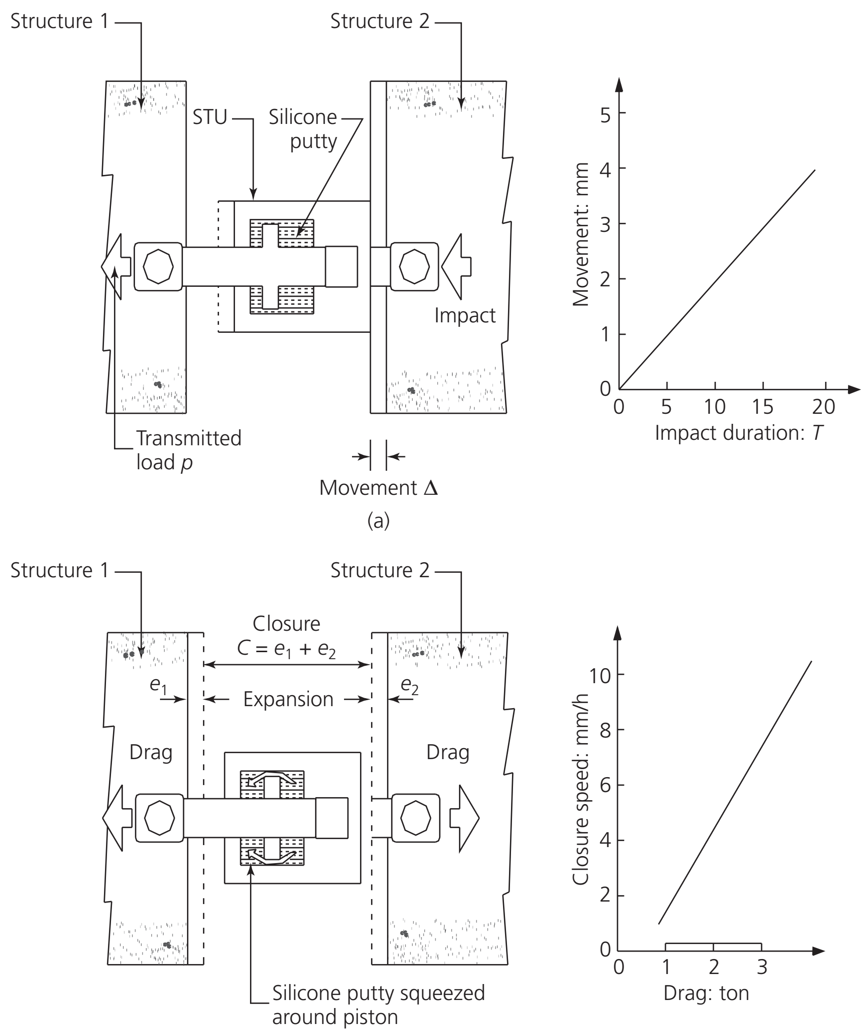



long-term axial movements (stroke) due to temperature, shrinkage and creep between the fixing eyes attached to the separate elements of a structure.

From Figure 3.5b it can be seen that the maximum STU restraint force or drag to deck movement is generated by the fastest long-term deck movement up to 10 mm/h. For a steel deck the maximum temperature rise can be as high as 4°C/h. The maximum speed of expansion and contraction is at the expansion joint, where an STU would be subjected to some 5 mm/h movement for, say, every 100 m length of the deck ($100 \times 4 \times 0.0125 = 5$ mm/h). A similar concrete deck would undergo around half this movement. From the same figure it can be seen that the maximum drag force generated by a 50 ton STU fixed across the expansion joint of a 200 m long steel deck, moving at up to 10 mm/h, would be less than 3 ton. This force is negligible in terms of deck articulation. With regard to the transmission of short-duration sudden traffic and earthquake load (see Figure 3.5a), when the STU locks up, the individual impact load duration is unlikely to exceed 10 s, and it can be seen that a 50 ton STU moves by no more than 2–3 mm (lock-up displacement) during this period of load transmission, and is thus acting as a rigid link.

The lock-up speed for an STU is defined as the translational velocity at which the STU will put out its rated force. Typical lock-up speeds for seismic- or wind-induced motion are between 460 and 910 mm/h for bridge and building applications. The typical maximum thermal motion speed is about 10 mm/h, with allowable STU output forces being 3–10% of the maximum rated force of the STU. However, the lock-up displacement which is due to the elasticity of the STU parts, and the elasticity (bulk modulus) of the fluid or putty can be altered by increasing or decreasing the diameter of the STU for a specific project.

**Figure 3.5** STU force versus movement characteristics: (a) short-term impact transmission; (b) long-term movement

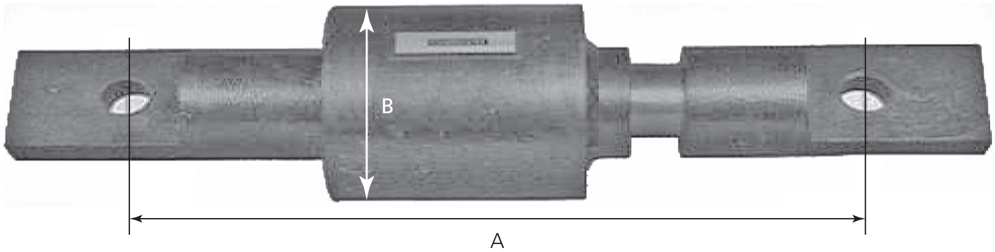



## 3.5. Different brands of STUs and their dimensions

### 3.5.1 Colebrand putty-based STUs

The author has visited ten manufacturers of STUs around the world. These are listed in Appendix 1 together with contact details for the benefit of readers. Of these ten manufacturers, there are two in Germany, two in Italy, three in the USA, two in France and one in the UK. There are a few more manufacturers in the USA, Switzerland, Canada, China and other countries. The STU described earlier and illustrated in Figure 3.1 is a typical putty-based STU manufactured by Colebrand of London, UK (Figure 3.6). While all brands of STUs perform in the same way, there are some

Figure 3.6 The Colebrand silicone putty-based STU (see Table 3.1). © Colebrand International Ltd




minor differences in the construction and dimensions of STUs produced by different manufacturers. Table 3.1 gives the typical dimensions of Colebrand's silicon putty-based STUs.

From Table 3.1 it can be seen that a 3000 kN STU weighs only 850 kg and has dimensions of 1.2 m × 0.55 m and so is particularly compact and lightweight. If larger impact forces have to be catered for it is usually best to provide groups of the units having up to 3000 kN capacity. However, STUs are available for force levels of 10–20 000 kN and displacement range of ±25 mm to ±1500 mm.

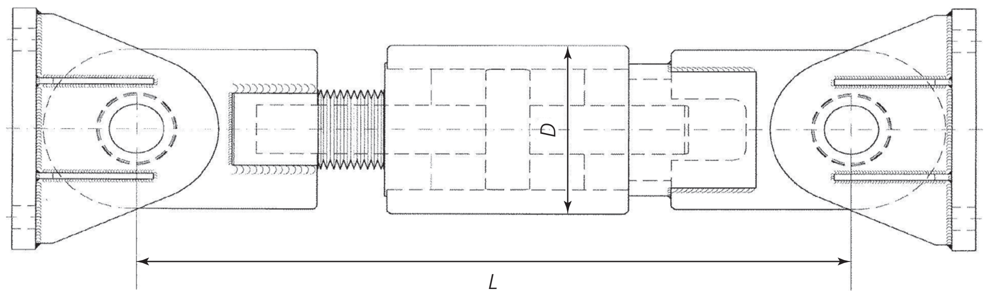
### 3.5.2 FIP oil-based STUs

Most of the STU manufacturers visited by the author, apart from two, use silicone oil. Figure 3.7 shows a typical silicone oil-based STU manufactured by FIP Industriale of Padua, Italy. It is similar to the Colebrand silicon putty-based STU. However, oil-based STUs for greater temperature ranges may require an accumulator to allow for the variation in the volume of the silicon oil. Table 3.1 shows the dimensions of the oil-based STUs manufactured by FIP Industriale.

Table 3.1 Typical dimensions of Colebrand silicon putty-based STUs (see Figure 3.6). Reproduced courtesy of FIP Industriale SpA

| Unit size: kN | *A*: mm | *B*: mm | Weight: kg |
|---|---|---|---|
| 100 | 400 | 110 | 8 |
| 200 | 470 | 145 | 28 |
| 300 | 555 | 172 | 40 |
| 400 | 555 | 206 | 65 |
| 500 | 600 | 208 | 75 |
| 600 | 630 | 240 | 100 |
| 750 | 730 | 265 | 130 |
| 1000 | 800 | 300 | 200 |
| 1250 | 850 | 325 | 260 |
| 1500 | 900 | 385 | 300 |
| 1750 | 950 | 410 | 350 |
| 2000 | 1000 | 450 | 500 |
| 2500 | 1050 | 460 | 650 |
| 3000 | 1200 | 550 | 850 |

Figure 3.7 Details of the FIP Industriale silicone oil-based STU (see Table 3.2). *D*, external diameter of the body; *L*, length when in the midposition. Reproduced courtesy of FIP Industriale SpA




### 3.5.3 Freyssinet oil-based STUs

Freyssinet of Paris, France, manufactures two basic types of hydraulic STU: the Transpec SHT Standard (Figure 3.8, Table 3.3) and the Transpec SHT Compact (Figure 3.9, Table 3.4). While they both operate in the same manner, the Transpec SHT Compact has smaller dimensions and is welded to fixing brackets at the bottom, and so can be installed directly at the top of a pier as shown in Figure 3.9. Freyssinet has also patented a mechanical STU which has been used on the southern section of the Vasco De Gama Bridge in Lisbon, Portugal.

## 3.6. Applications of STUs

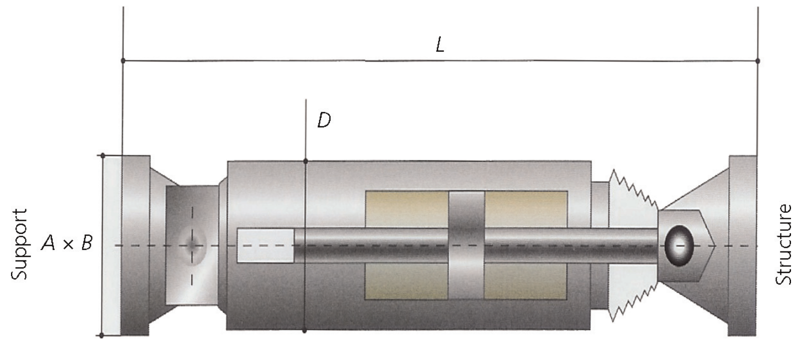
STUs can be used to take loads wherever you want them to. There are three possible ways in which STUs can be used for structures.

- They can share loads.
- They can reduce loads.
- They can transfer loads.

Table 3.2 Typical dimensions of FIP Industriale oil-based STUs (see Figure 3.7). Reproduced courtesy of FIP Industriale SpA

| Model | Axial force: kN | *D*: mm | *L*: mm |
|---|---|---|---|
| OT 100/100 | 1000 | 254 | 885 |
| OT 125/100 | 1250 | 301 | 960 |
| OT 150/100 | 1500 | 339 | 1050 |
| OT 200/100 | 2000 | 402 | 1270 |
| OT 250/100 | 2500 | 416 | 1380 |
| OT 300/100 | 3000 | 454 | 1500 |
| OT 400/100 | 4000 | 550 | 1570 |
| OT 500/100 | 5000 | 605 | 1850 |
| OT 750/100 | 7500 | 705 | 2120 |
| OT 1000/100 | 10 000 | 760 | 2360 |
| OT 1500/100 | 15 000 | 840 | 2780 |
| OT 2000/100 | 20 000 | 920 | 3040 |

Figure 3.8 Details of the Freyssinet Transpec SHT Standard STU (see Table 3.3). Permission requested from Freyssinet




### 3.6.1 STUs can share loads

This normally applies to multi-span continuous bridges where all the longitudinal forces are carried by one fixed pier. Figure 3.10 shows this aspect of load sharing.

Many bridge designs are restricted by the maximum size of the fixed pier. The longer the overall span, the greater the traction or braking forces, seismic force, etc. applied to the fixed pier. These high forces result in large piers and foundations, which, in turn can cause the following problems.

- Restriction to traffic during both construction and operation.
- River flow.
- Increased seismic force due to the increased size of the pier.
- Additional reinforcement needed in piers and piles, leading to increased costs.
- Aesthetic issues.

Table 3.3 Dimensions of Freyssinet Transpec SHT Standard STUs with a stroke of 100 mm (see Figure 3.8). Permission requested from Freyssinet

| Model | Nominal force: kN | Average length, *L*: mm | Body diameter, *D*: mm | Height of plate, *A*: mm | Width of plate, *A*: mm |
|---|---|---|---|---|---|
| SHT 200–100 | 200 | 820 | 106 | 150 | 100 |
| SHT 300–100 | 300 | 895 | 125 | 170 | 120 |
| SHT 500–100 | 500 | 985 | 140 | 200 | 170 |
| SHT 1000–100 | 1000 | 1235 | 210 | 260 | 200 |
| SHT 1500–100 | 1500 | 1415 | 245 | 330 | 240 |
| SHT 2000–100 | 2000 | 1565 | 290 | 390 | 300 |
| SHT 3000–100 | 3000 | 1875 | 355 | 420 | 380 |
| SHT 4000–100 | 4000 | 2110 | 405 | 490 | 490 |

**Figure 3.9** Details of the Freyssinet Transpec SHT Compact STU (see Table 3.4). Permission requested from Freyssinet

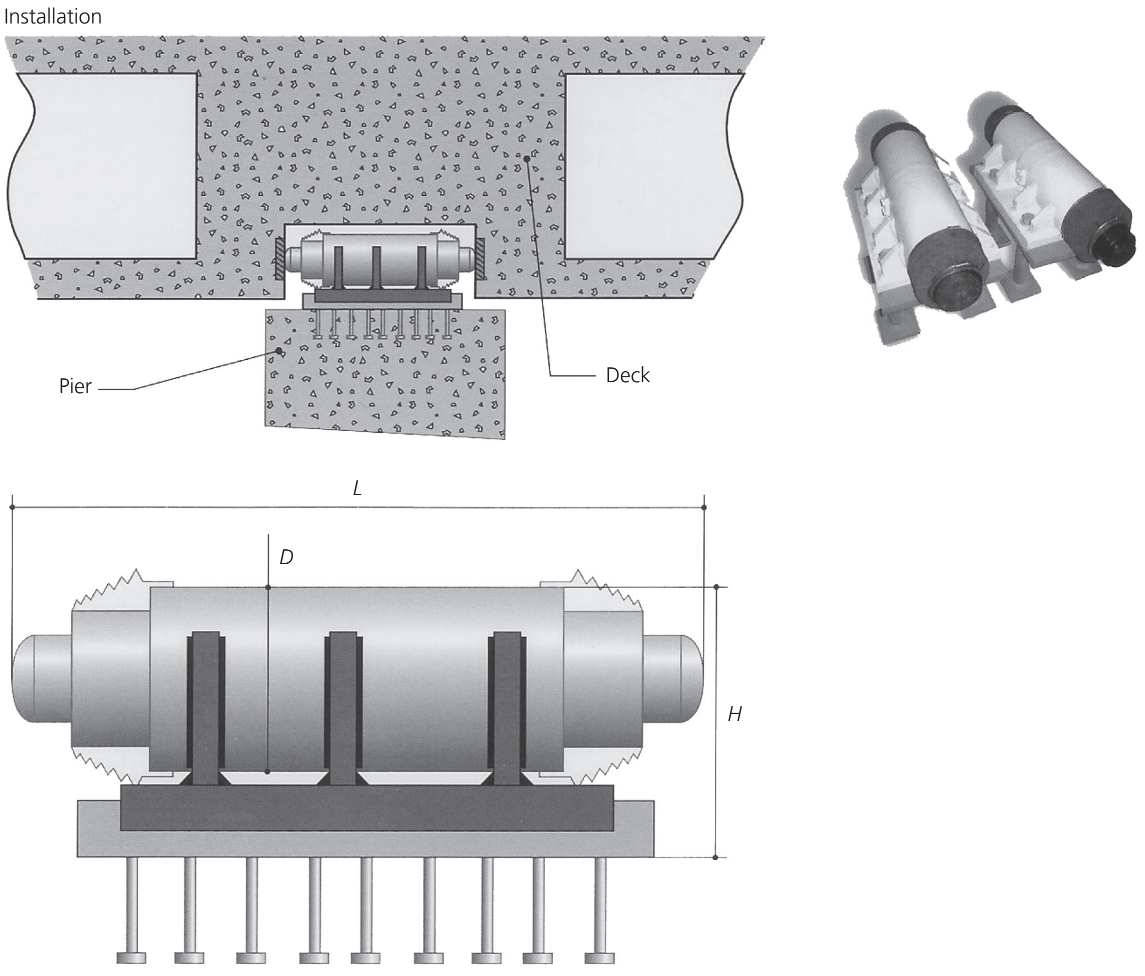
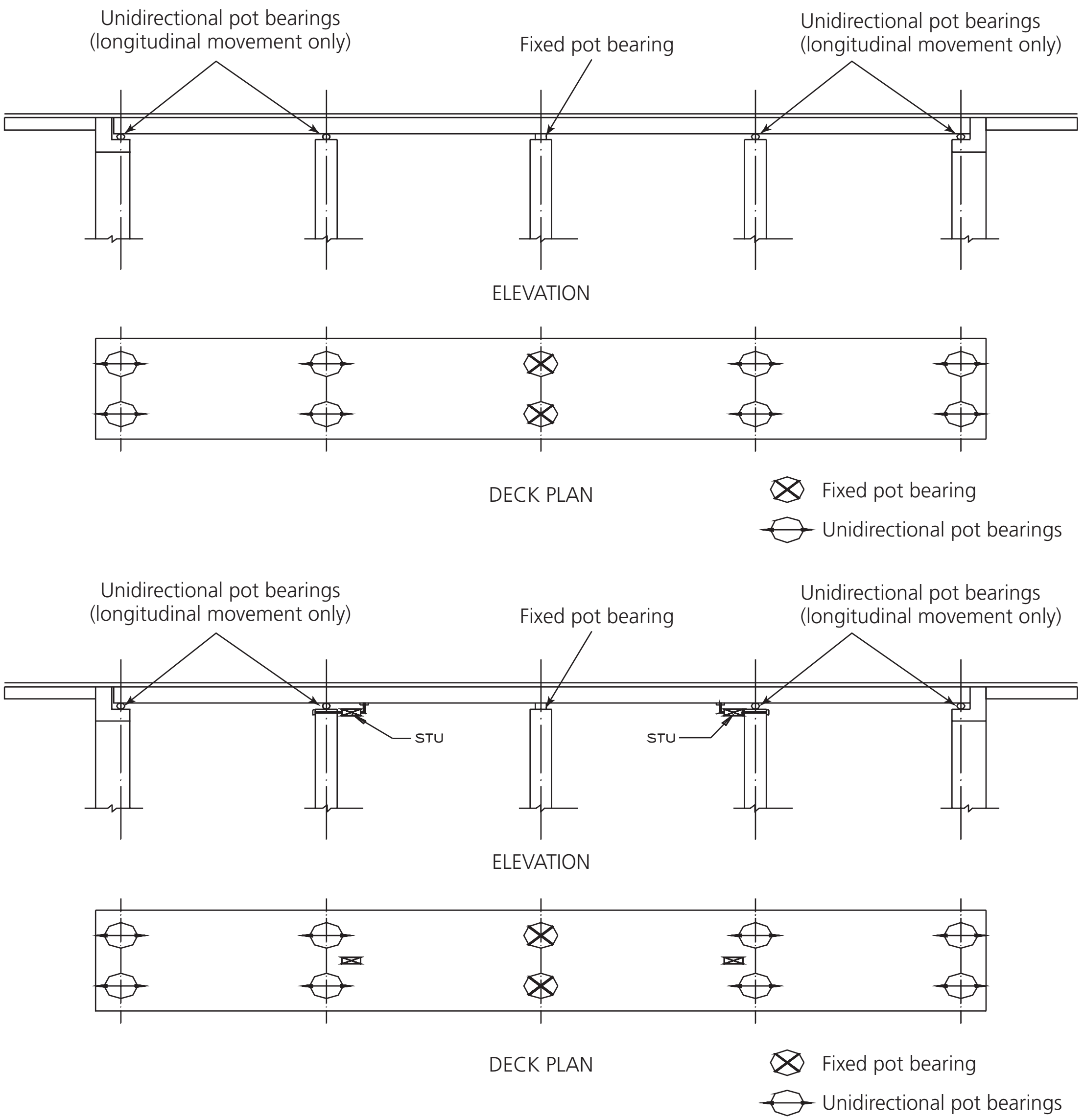



**Table 3.4** Dimensions of Freyssinet Transpec SHT Compact STUs with a stroke of 100 mm (see Figure 3.9). Permission requested from Freyssinet

| Model | Nominal force: kN | Length, *L*: mm | Total height, *H*: mm | Body diameter, *D*: mm |
|---|---|---|---|---|
| SHT Compact 200-100 | 200 | 575 | 200 | 85 |
| SHT Compact 300-100 | 300 | 595 | 215 | 105 |
| SHT Compact 500-100 | 500 | 620 | 240 | 130 |
| SHT Compact 1000-100 | 1000 | 670 | 305 | 190 |
| SHT Compact 1500-100 | 1500 | 720 | 340 | 230 |
| SHT Compact 2000-100 | 2000 | 750 | 375 | 260 |

**Figure 3.10** Sharing of loads by means of STUs




When STUs are installed on the free piers adjacent to the fixed pier where unidirectional pot bearings are installed (see Figure 3.10), the STUs lock up when a sudden horizontal load is applied to the bridge structure; this means that the free piers temporarily become fixed piers. The longitudinal load is now shared by all three piers (rather than one fixed pier) in a manner depending on the stiffness of the piers and foundations.

The benefit of this aspect of distributing the sudden horizontal force has been exploited in many new and existing bridges (see Chapters 4 and 5).

### 3.6.2 STUs can reduce loads

This normally applies to simply supported and continuous spans in multi-span arrangement. Figure 3.11 shows this aspect of reducing loads for a typical multi-span, simply supported bridge. The piers under each simply supported span carry the usual fixed bearings for one span and the free bearings for the adjacent span. The design longitudinal traction and braking force must be applied individually to each deck span for such a viaduct. The majority of the resistance against the longitudinal acceleration and braking forces is offered by the pier or the abutments carrying the fixed bearings of that particular span.

A small additional frictional resistance generated at the free bearings is normally not permitted to be included in the analysis of the bridge. This requires that a substructure of this type with, say, abutments and piers of equal stiffness, has to carry a total designed capacity of five times the required design braking and traction longitudinal loads of the deck. As shown in Figure 3.11, by placing five STUs at bearing level on the free abutments and piers it is possible to share out the traction and braking load acting anywhere on the viaduct's deck between all four piers and both abutments. This results in a significant reduction in the required pier and fixed abutment sizes and foundations to provide a total designed horizontal load capacity that is only about 20% of that required initially.

The other advantage is that, by installing the STUs as shown in the Figure 3.11, the simply supported bridge becomes tied together when a seismic event occurs, thereby considerably increasing the stability of the deck. This is a requirement stipulated in the AASHTO seismic code. Furthermore, as the top end of column for each pier is also fixed during a seismic event, the column can be designed as fixed at both ends for impact loading, which may allow for a more economical design. The benefit of this aspect of load sharing has been exploited in many new and existing highway and railway bridges (see Chapters 4 and 5).

### 3.6.3 STUs can transfer loads

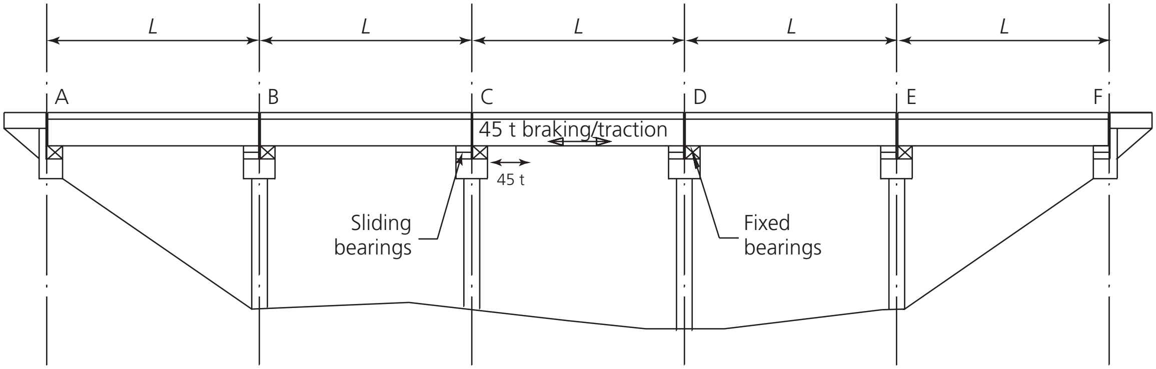
This normally applies to simply supported and continuous spans in a multi-span arrangement with varying pier heights. Figure 3.12 illustrates the principle of transferring horizontal loads from tall piers in the river water to shorter piers and abutments on the land.

When a bridge is built across a river valley, the height of the piers can vary greatly from one to another. If each pier has to carry a high proportion of the longitudinal forces, the tall piers can be subjected to high overturning moments, giving rise to a need for larger and more costly foundations. If, somehow, the longitudinal forces can be transferred away from the tall piers located in the middle part of river valley to the shorter piers and the abutments, the pier and foundation design for the bridge can be made more economical.

If all the spans for a simply supported bridge are linked together by means of STUs to give a total continuous span under a suddenly applied load, then under any given longitudinal force the total span will move as a single unit. Now, as each pier will deflect by the same amount, the applied longitudinal force carried at the top of each pier will be directly proportional to the relative stiffness of the piers.

If the three piers of the bridge have heights of $1h$, $2h$ and $3h$ and have the same cross-section, as shown in Figure 3.12, and are linked together by means of STUs, as each pier will deflect equally, the proportion of the longitudinal load that will be carried by pier 1 is 86%, by pier 2 is 11% and by pier 3 is 3%. In this way the longitudinal force can be transferred to the shorter piers which are located

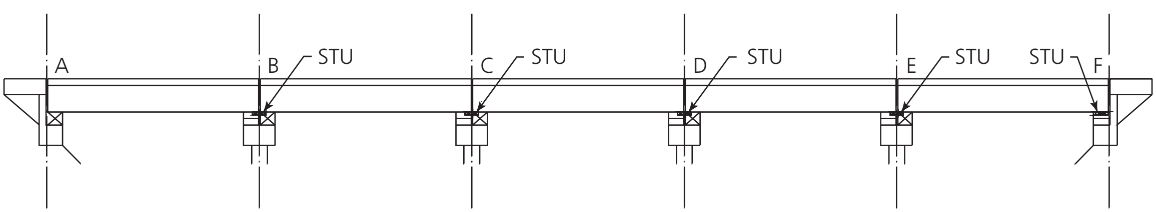
**Figure 3.11** Reducing of loads by means of STUs: a multi-span, simply supported bridge




Typical 5-span, simply supported bridge

| 45 t traction/braking in span | Horizontal load on support: t | | | | | |
|---|---|---|---|---|---|---|
| | A | B | C | D | E | F |
| AB | 45 | – | – | – | – | – |
| BC | – | 45 | – | – | – | – |
| CD | – | – | 45 | – | – | – |
| DE | – | – | – | 45 | – | – |
| EF | – | – | – | – | 45 | – |

Total support horizontal design capacity required for
traction/braking = 45 t + 45 t + 45 t + 45 t + 45 t = 225 t

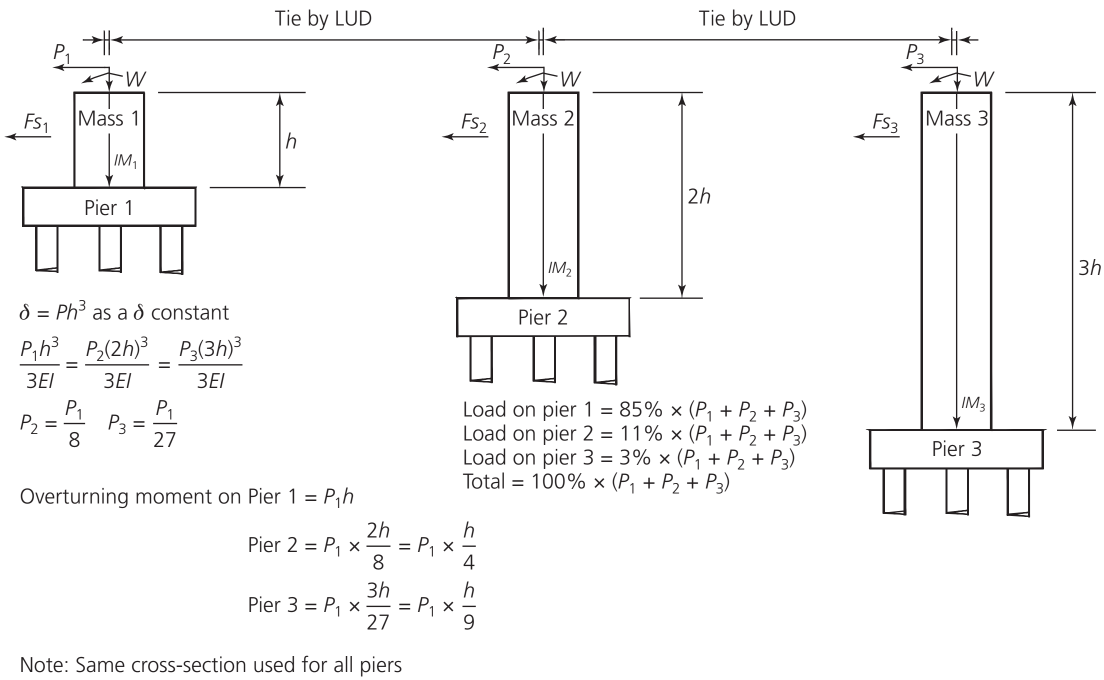



Addition of 5 STUs – equal stiffnesses at all supports assumed

| 45 t traction/braking in span | Horizontal load on support: t | | | | | |
|---|---|---|---|---|---|---|
| | A | B | C | D | E | F |
| AB | 7.5 | 7.5 | 7.5 | 7.5 | 7.5 | 7.5 |
| BC | 7.5 | 7.5 | 7.5 | 7.5 | 7.5 | 7.5 |
| CD | 7.5 | 7.5 | 7.5 | 7.5 | 7.5 | 7.5 |
| DE | 7.5 | 7.5 | 7.5 | 7.5 | 7.5 | 7.5 |
| EF | 7.5 | 7.5 | 7.5 | 7.5 | 7.5 | 7.5 |

Total support horizontal design capacity required for
traction/braking = 7.5 t + 7.5 t + 7.5 t + 7.5 t + 7.5 t + 7.5 t = 45 t

Figure 3.12 Transferring of loads by means of STUs. © Colebrand International Ltd




nearer to the land where the water is not deep. This will not only make the construction of the bridge quicker and easier, but it will also lead to substantial savings in the foundation design.

## 3.7. Service life of STUs

The service life of an STU can be around 50–75 years, depending on the following factors.

- The cycle life of the seals.
- The ageing or degradation per cycle of the working viscous medium in the cylinder.
- The fatigue life of the component parts.
- The corrosion of metallic parts.

STUs must be tested for a particular service-life requirement and, depending on the required service life, a seal wear test should be carried on the seal material. The most severe applications for STUs is on highway and railroad bridges, where at least one measurable cycle of thermal expansion and contraction occurs daily, along with traffic braking and train braking loads. Assuming a 75-year service life with a minimum of maintenance, this is equivalent to a minimum of $75 \times 365 = 27\,375$, say 30 000 cycles, when only the thermal motion is considered. Therefore, the STU must be able to withstand a minimum of 30 000 full displacement cycles with no degradation of seals, which are designed to stop silicon putty/oil leakage for a life of 75 years.

Ageing or cycle degradation of the working viscous medium in fluid-filled devices can be simply evaluated using the cyclic test unit. However, for putty-filled devices accelerated ageing tests must be performed to ensure that the putty will not harden due to cross-linking of its molecules or the out-gassing of solvents. Some manufacturers have carried out accelerated life tests in the laboratory on by heating and cooling the putty-filled STU from as low as −40°C to +50°C and cyclically loading it for many thousands of cycles, and have reported no effect on the properties of silicone putty used.

STUs on a railroad or a highway bridge can withstand many cycles of loads from braking and traction forces, with frequent heavy vehicle braking. However, a fatigue test for the required service life of the STUs should be carried out. The worst-case scenario for service loading of an STU is an application of braking loads to lock up the STU four times a day for the AASHTO LFRD specified design life of 75 years. This is roughly equal to $75 \times 4 \times 365 = 109\,500$, say 100 000 cycles. If the project design requirements do not include braking loads of this frequency, the number of cycles should be adjusted accordingly, or, if fatigue is not a consideration, say for earthquake applications, then this test can be waived at the discretion of the design engineer.

However, a corrosive environment may well determine the longevity of an STU if it is not properly protected by galvanising and painting the steel components to protect against the conditions expected over the 50–75 years of service life. In addition, any applied protective system must be maintained regularly over the service life of the STU.

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# Chapter 4
# STUs for new highway bridges

## 4.1. Introduction

The most effective use of shock transmission units (STUs) is in bridge engineering. The devices offer the bridge engineer a robust, low cost and virtually maintenance free mechanism that can be used cost-effectively on both new and existing highway and railway bridges. STUs provide the following characteristics, which can be used to advantage in bridge engineering.

- They move freely in both directions to cater for the long-term movements of the bridge.
- They act as a temporary fixed link between the structural elements of the bridge to beneficially transmit, transfer or share suddenly applied loads for the duration of the application of such loads.

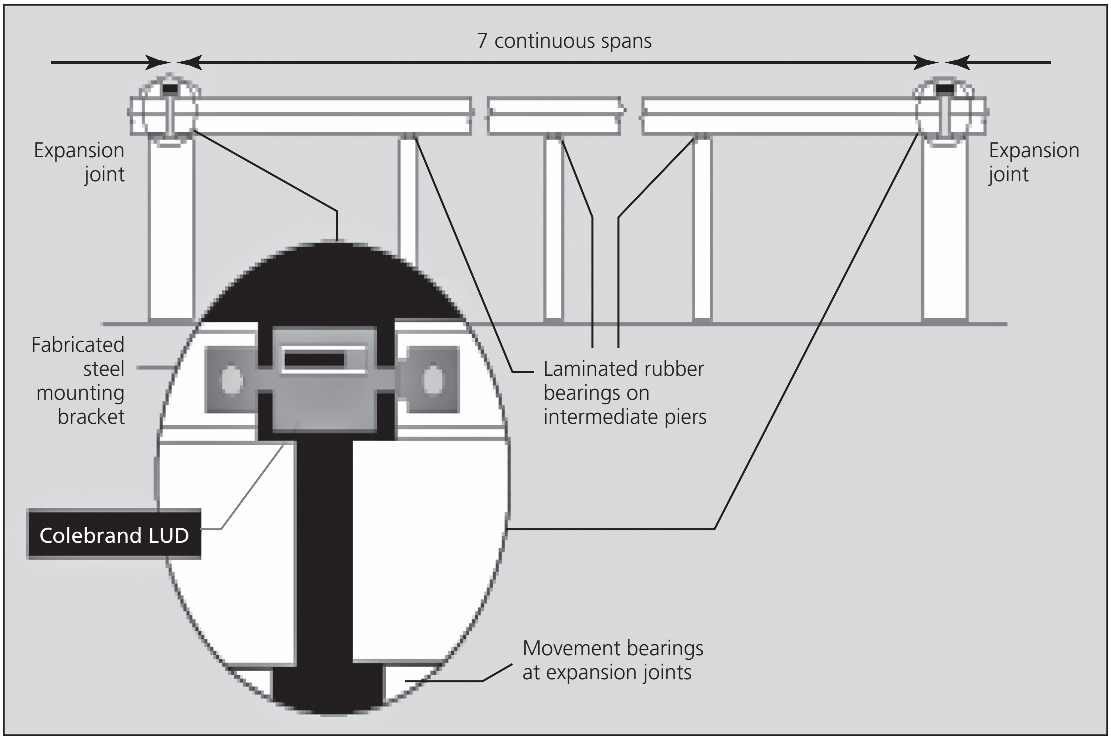
STUs can be applied to bridge structures subject to the following effects.

- Significant long-term movements, such as concrete shrinkage, creep and temperature or differential temperature expansions and contractions.
- Significant short-duration shock loads, such as earthquake loading, road or rail vehicle traction and braking, nosing, centrifugal force, wind storm and accidental impact.

For the design of substructures and foundations of new multi-span highway bridges or continuous viaducts, the incorporation of STUs is beneficial in the following three particular situations.

- **New simply supported multi-span bridges.** Multi-span bridges or viaducts are often made up of a series of simply supported spans rather than continuous spans. An anticipation of future large settlements in poor ground or areas of mining subsidence may be the reason for this option. However, regardless of the merits of simply supported spans for the deck in such situations, the overdesign of the substructure will be required. By placing an STU at every expansion joint of a typical simply supported bridge, the bridge structure can be tied together in the event of one of the significant short-duration shock loads mentioned earlier. Such an applied load is then distributed according to the stiffness of each substructure element, as explained in Chapter 3, and therefore results in much smaller substructure elements and foundations for the bridge than for the simply supported bridge without incorporated STUs.
- **New continuous viaducts with expansion joints between adjacent viaducts.** Load sharing by means of STUs can be used in new continuous viaducts with expansion joints between the adjacent viaducts. STUs can be installed at the expansion joints between two adjacent continuous viaducts as shown in Figure 4.1. The suddenly applied longitudinal load, which was resisted only by the piers of one viaduct before installing the STUs, can be now shared by the piers of all the connected viaducts, thereby reducing the horizontal load resisted by the piers. Thus, STUs

**Figure 4.1** Continuous viaduct with STUs installed at the expansion joints




allow a continuous deck structure to be designed lighter, giving potential savings in piers and foundations.

- **New continuous bridges.** To appreciate the benefits that can be gained by the use of STUs, it is necessary to understand the interrelated design requirements for continuous bridge decks and their supports. Multi-span continuous decks are supported on substructures formed by a series of piers and abutments. Each substructure must be designed to cater not only for the vertical loads due to the dead load of the deck and the road or rail traffic carried, but also for the dynamic short-duration horizontal loads arising from the traffic, such as vehicle traction, braking, skidding and centrifugal and nosing effects. Other short-duration horizontal loads may arise from accidental impacts, wind gusting and, in many global locations, earthquakes.

Ideally, these horizontal loads would be shared out among the substructure piers and abutments to provide the optimum and most economic design by attaching the entire substructure to the deck superstructure, either by building it in or by using fixed bearings. Unfortunately, bridge decks are always on the move, dimensional changes being due mainly to the rise and fall temperature. Further movements will arise as a result of shrinkage, pre-stressing and creep in concrete bridge deck superstructures. Such movements, which are normally largest in the deck longitudinal direction, can generate very large horizontal restraint forces in a fully attached substructure due to its stiffness. To avoid these unnecessary restraint forces, which are usually considerably greater than the dynamic horizontal loads, the designer inevitably detaches most of the multi-span deck superstructure from the substructure. This is done by installing moving bearings that offer as little resistance as possible to movement at the tops of the pier and abutments; these are normally designated as ‘free supports’.

To resist the horizontal loading from the traffic, wind and, where appropriate, earthquakes the deck is usually attached by fixed bearings or by building in to either one abutment or one or two central piers; these are normally designated as ‘fixed supports’. Thus, the beneficial sharing of the short-duration traffic and earthquake loading by fixing all supports is normally lost, and this loss increases with the number of non-participating free piers. This means that all resistance must come from the fixed piers or abutment only.

The use of STUs can readily restore this short-duration horizontal load-sharing advantage without building in any large restraint forces due to deck movements. By installing STU links between the detached deck and the substructure at the free supports (i.e. the movement joint or movement bearing locations), all the substructure is virtually fixed to the deck for the short-duration loading described above, with full load sharing. This means considerable horizontal load reduction on the fixed piers or abutments at the expense of smaller load increases, which can be readily absorbed by each of the free piers. After the temporary fixity, each STU reverts to its free state, offering virtually no restraint to the longer-term deck movements.

The intensity of earthquake forces is a function of deck mass and generally results in forces well in excess of design braking and traction of traffic when the spans are large. The high earthquake loading necessitates the development of considerable longitudinal restraint from the substructure at bearing level. Such a high earthquake force on a continuous bridge structure would overload the fixed piers or abutments. As explained above, STUs can be installed on the other piers of a continuous bridge having moving bearings in order to advantageously distribute the earthquake forces between the fixed piers or abutments and the piers installed with STUs. This results in considerable savings with regard to the substructure elements and foundations. Apart from seismic loading, load relief can be also used to distribute the braking and traction forces of vehicles on new continuous bridges.

STUs can be attached between the deck and the piers with moving bearings of a long continuous viaduct to relieve the longitudinal loading on the central fixed pier, which may have to be founded on poor soil or in deep water in the middle of a river. Two long viaducts designed by Kent County Council in the UK, completed in 1979 and 1982, used STUs attached to the decks at abutments to reduce the longitudinal loading on the central fixed piers, which were founded on particularly poor estuarial soils.

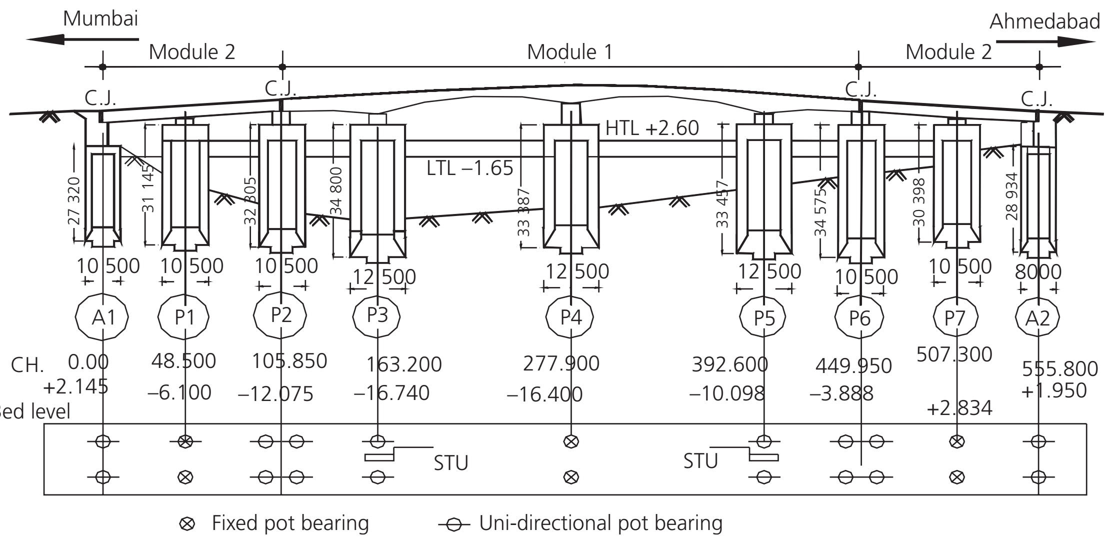
The following are case studies of the use of STUs on new highway bridges.

## 4.2. Second Bassein Creek Bridge, Mumbai, India

The Second Bassein Creek Bridge is a good example of a new multi-span continuous bridge where the application of STUs has saved time and money in the construction of the caisson foundations. The new bridge is located 25 m upstream of the existing bridge on Mumbai Ahmedabad National Highway No. 8, just outside Mumbai, near Ghodbunder at the confluence of the Ulhas river and the Arabian Sea.

This World Bank financed bridge was designed by Span Consultants, New Delhi, India, and proof-checked by Kinhill Engineers of Australia. It was constructed by Bhagheeratha Engineering Ltd of Kochi, India, and was supervised by the World Bank appointed supervision consultants N.D. Lea

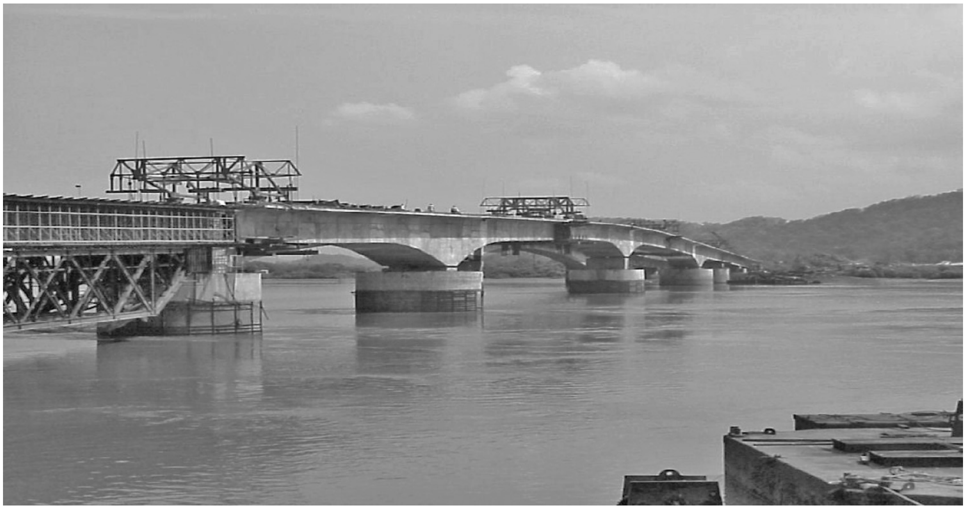
Figure 4.2 The general arrangement of the Second Bassein Creek Bridge, Mumbai, India




International of Canada in a joint venture with Louis Berger International of the USA in association with Lea Associates S.A. Pvt Ltd and ICT, Delhi, India.

Figure 4.2 shows the general arrangement of the Second Bassein Creek Bridge. The eight-span, prestressed, post-tensioned bridge comprises two navigational spans in the middle, each of length 114.7 m, with two adjacent spans of length 57.35 m constructed as balanced cantilevers (Figure 4.3) and made continuous by pouring the closure segment. The end spans comprise two continuous

Figure 4.3 Construction of modules 1 and 2 of the Second Bassein Creek Bridge



spans, with one span of 57.35 m and the other of 48.5 m, making the total length of the bridge 555.8 m. The cantilever box girder decks vary in depth from 7.0 m to 3.5 m and were constructed simultaneously using three travelling formworks, one on each of the piers P3, P4 and P5. The box girder end spans have a constant depth of 3.5 m and were constructed using a conventional method of ground-supported staging and steel trusses over the creek water, supported on the piers. The total width of the bridge is 11.0 m, and comprises a roadway of 7.5 m and a footway on either side of 1.25 m. The bridge is supported over nine caisson foundations varying in diameter from 8.0 m to a maximum of 12.5 m. Three of the caissons (A1, A2 and P7) are ground caissons, and the other six (from P1 to P6) are floating caissons sunk 30 m through the creek water to a basalt rock stratum.

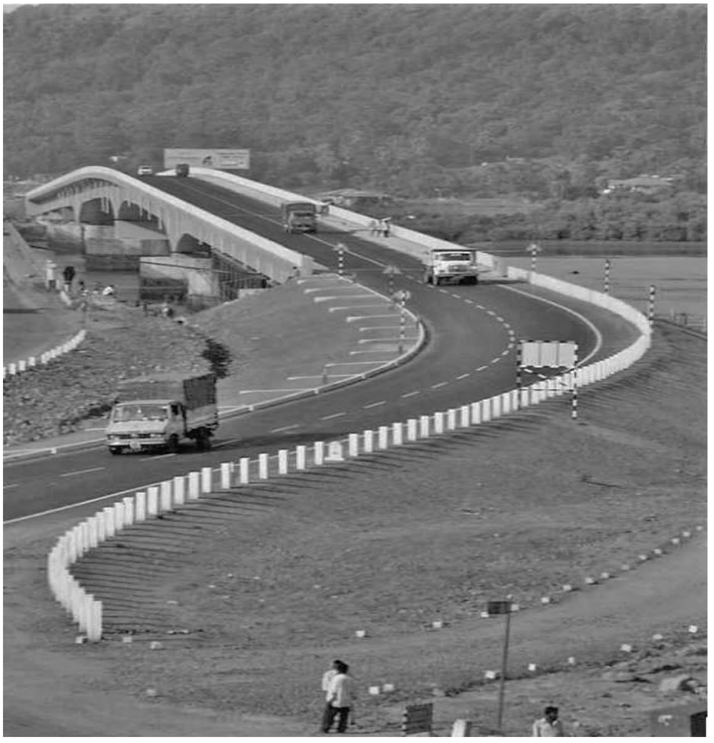
It was very difficult to construct the caissons at this site, as there are two low tides and two high tides daily, one tide followed by the other every 6 hours. There are also occasional windstorms, which create high water currents with a velocity of 3 m/s, and during the monsoon heavy floods are encountered. The depth of the water under the two central navigation spans is around 20 m, and the average daily tidal variation is of the order of 4.25 m with an average velocity of 2.4 m/s. All these site conditions made construction of the caissons a difficult and risky task for the contractor. The existing bridge was originally designed as a ten-span simply supported pre-stressed concrete girder bridge, the eight middle spans being 57.35 m and the two end spans being 48.5 m long. However, during construction of the bridge the design of the six internal spans had to be changed: the two middle spans were increased from 57.35 m to 114.7 m due to submergence of the two middle caissons during construction in the monsoon flood of 1964. Most of the other caissons experienced heavy tilt and shift, which had to be corrected by means of kentledge and sinking methods. The existing bridge took 7 years to complete due to the difficulties encountered in sinking the caissons.

While the caissons of the new bridge suffered lesser problems of tilt and shift due to improved modern methods, the construction process had its fair share of problems. Two of the floating caissons were swept away during a storm in January 1999, but were recovered later. One 12.5 m outer diameter caisson had to be abandoned as it was submerged on 8 December 1999 due to water entering the caisson during the high tide while the steel liners required as permanent formwork were being fabricated in the creek near the launching platform.

The new bridge (Figure 4.4) is designed for a lifespan of 100 years for a design live load of one lane of IRC Class 70R or two lanes of IRC Class A, whichever governs. It is also designed for vessel collision with an impact force of 500 ton at well cap mid-depth for the main span piers (P3, P4 and P5) and a force of 200 ton for the end span piers (P1, P2, P6 and P7). The site is located in a moderate seismic zone and the bridge has been designed accordingly for a 0.075 horizontal seismic coefficient, including an importance factor of 1.5. However, the horizontal forces generated in a bridge deck subjected to an earthquake shock are a function of the deck mass, and so for this bridge, which has heavy navigation spans each of 114.7 m, the caisson foundation for pier P4 with the fixed bearings had to be designed initially for a 930 ton seismic horizontal force.

During the preliminary design stage it was found by the design and proof-check consultants that the size of the caisson required for pier P4, with the fixed pot bearings, to resist the full seismic loading of 930 ton was 16.5 m diameter and that the size of the adjacent caissons for piers P3 and P5, with unidirectional pot bearings, was 11.5 m diameter. They realised the difficulty of constructing such a large 16.5 m caisson in the difficult creek conditions and the corresponding high cost, and decided to incorporate STUs on piers P3 and P5 to create temporary fixity at these piers during seismic episodes. The STUs on piers P3 and P5 would lock up during an earthquake event, thereby distributing

Figure 4.4 The completed Second Bassein Creek Bridge



the seismic loading of about 930 ton between piers P3, P4 and P5 according to their respective stiffnesses. This resulted in a reduction in the diameter of the caisson for pier P4 from 16.5 m to 12.5 m and an increase in the diameter of the caissons for piers P3 and P5 from 11.5 m to 12.5 m. The horizontal seismic force became distributed equally across the three piers, which had more or less the same stiffness, from 930 ton initially on pier P4 alone to 310 ton on each of the three piers P3, P4 and P5. Not only did this make the caisson construction manageable, it also appreciably reduced the cost of the caisson foundations, resulting in considerable overall cost savings.

The bridge contract was to be completed in 3 years but the contractor experienced a 6-month delay due to the problems involved in the construction of the 12.5 m diameter caissons. It can be concluded that the contractor would have experienced greater difficulty in constructing a 16.5 m diameter caisson in the difficult creek conditions, and the project would have been delayed much longer. The use of STUs on this bridge not only decreased the cost of the foundations by about 11 million Indian rupees but, above all, it made the caisson foundations for the bridge practicable by reducing the unmanageable size of the caisson for pier P4 from 16.5 m to 12.5 m diameter.

## 4.3. New Paksey Bridge, Bangladesh, India

Paksey Bridge is the first segmental bridge over the River Padma, and is the second largest in Bangladesh, with 15 spans 109.5 m long and two 72.0 m end spans, having a total length of 1786 m. It provides a strategic road connection across the River Padma (Ganges) and links directly to road connections with India and Nepal. It is also one of the longest segmental concrete bridges in the world constructed as a single continuous structure from end to end between two abutments. The four-lane bridge directly connects south-western and north-western Bangladesh and the Mongla seaport with the rest of the country. It carries two lanes of 7.5 m wide roadways and two 1.0 m wide walkways.

The owner is the Bangladesh Roads and Highways Department and the bridge was funded by the Japan Bank for International Cooperation, formerly the Overseas Economic Co. Fund, under a soft loan to the Government of Bangladesh. PB, USA was the prime consultant in coordination with: Worley, New Zealand; Kiljian Corporation, USA; Sarm Associates Ltd, Bangladesh; and KS Consultants Ltd, Bangladesh. The contract for the construction of the bridge and the river training works was awarded to the Major Bridge Engineering Bureau of the People's Republic of China in August 2000.

The superstructure was designed as a continuous, precast, pre-stressed, single-cell box girder to be erected using the segmental balanced cantilever construction method. The piers for the bridge are almost all of the same height, with similar stiffness characteristics. That being the case, the decision was made to incorporate STUs at all moveable piers to share the longitudinal seismic forces that would otherwise be imposed on the fixed pier. By using STUs, all piers would share any seismic load more or less equally, and thus the total force on the only fixed pier would be greatly reduced.

The bridge is a series of spans, 17 in total, with a fixed shear key connection at pier 8, near the middle of the bridge. Due to the extreme length of the structure, the STUs at piers 1 and 16 were required to have a total movement in excess of 750 mm, with an ultimate limit state (ULS) capacity of 11 500 kN. Figure 4.5 shows one of the STUs. These devices are among the largest STUs ever fabricated for a

**Figure 4.5** A 11 500 kN ULS capacity STU with 750 mm stroke for Paksey Bridge (Hongbing *et al.*, 2004). © D.S. TechStar, Inc.

Figure 4.6 STUs being installed on one of the piers of the Paksey Bridge (Hongbing *et al.*, 2004).
© D.S. TechStar, Inc.

bridge, and are comparable in weight to the devices produced by TechStar/Alga for the Carquinez Bridge in California, which had an ULS capacity of 25 000 kN (Marioni and Dalpedri, 2001).

The STUs were provided by a TechStar/Alga joint venture, the same joint venture which provided the highest capacity STUs to date of 15 000 kN for the Carquinez Bridge. In addition to the design considerations for the bridge, there were special design considerations for the STUs. The STUs are equipped with spherical bearings at each end to assist in installation and to permit articulation during seismic events. The STUs are adjustable, so that the distance between the pin connections can be accommodated within a reasonable range. The corrosion protection of the outside clevis and cylinder is a special polychlorinated paint system, specially designed for an extremely harsh, brackish environment. The pin connections are stainless steel so that corrosion will not affect their integrity. Figure 4.6 shows the installation of one of the STUs on the bridge.

## 4.4. New Goff Bridge, Riggins, Idaho, USA

The Goff Bridge is in Riggins, Idaho, and is located on a landfall that is part of a seismic zone (Figure 4.7). The bridge is one of the first tied-arch bridges approved by the Federal Highway Administration, and is the longest tied-arch bridge built in Idaho. In a tied-arch bridge the outward directed horizontal forces of the arch are borne by the bridge deck rather than the ground or the bridge foundations. The bridge deck ties the ends of the arch together and is under tension, much like the string of a bow. The elimination of horizontal forces at the ends allows a tied-arch bridge to be constructed with less robust foundations. This type of bridge can be easily situated on top of elevated piers or in areas of unstable soil. The bridge runs parallel along the landfall as it crosses the Salmon River in central northern Idaho. The bridge is owned by the Idaho Department of Transportation. It was built by Harcon Construction, and the steel was erected by Stone River Construction.

Figure 4.7 Goff Bridge, Riggins, Idaho, USA

TechStar supplied four STUs, which are installed transversely across the bridge, two at each abutment, as shown in Figure 4.8. The STUs permit slow movement by the landfall, which is expected to be 0.25–0.5 inches/year. The landfall movement is strictly in one direction. The STUs behave as shear keys for horizontal forces but allow for the gradual creep of the bridge. Extra-wide abutments (Figure 4.9) are built into the bridge to permit the continued slide of the bridge.

Figure 4.8 Two STUs per abutment installed transversely at the abutment of the Goff Bridge

Figure 4.9 The extra-wide abutment of the Goff Bridge

Once the lock-up devices (LUDs) have moved to the end of their movement capacity they are removed, recentred and reinstalled. Steel shims are then reversed to allow continued motion control over an anticipated 50-year period. After this, new shims can be modified and/or removed to provide LUD functionality for an indefinite period.

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# Chapter 5
# STUs for existing highway bridges

## 5.1. Introduction

The most advantageous use of STUs is to be found in the strengthening of the substructures of existing bridges. In many countries, the plan for the assessment and strengthening of existing highway and railway bridges will, for various reasons, require substructure strengthening. Some of the integrity assessments will indicate that the supporting piers are inadequate due to increases in braking and traction loading since the original design and also due to code revision. It may also be found that there is damage to the structural integrity due to carbonation or alkali–silica reaction and general ageing.

Recent earthquakes in many countries around the world have shown that the intensity of earthquakes is much higher than the earthquake zone specified in the local code for the design of structures. In some countries, engineers have now upgraded the earthquake zone and revised such codes. However, it still remains to strengthen existing bridges that are inadequate to resist the higher seismic forces computed using the revised design code. As a result, significant retrofitting of many bridges is required to reduce the probability of catastrophic collapse and loss of life. Because of the very large number of these existing bridges, it is imperative that this work be done by the most cost-effective means possible, while at the same time not using unproven ‘magical’ solutions that provide a false sense of security. STUs have been found to be a very economical means of retrofitting such existing bridges.

Repair and strengthening procedures for substructures are less disruptive to road traffic than are deck repairs, as the work for substructure repair takes place under the deck. However, any cutting away and subsequent strengthening by means of conventional procedures will definitely cause some disruption to the bridge traffic. Besides, such repairs and strengthening to the substructure of the bridge may disrupt any heavily trafficked roads, railways and waterways under the bridge. The cost of disruption when using conventional methods can be significant for such strategically located bridges, and so the bridge designer should decide on repair methods that cause the least disruption. STUs provide a means of strengthening existing bridges by load sharing between the piers and, possibly, the abutments during traction, braking and seismic events. STUs installed at the expansion joints or over the piers on such bridges cause virtually no disruption to traffic. In addition, as the installation does not require any amendment to the structural bearings, expensive jacking of the structure is not required.

The load sharing achieved will depend on the stiffness of individual piers and possibly abutments, but will considerably reduce the individual traction and braking or seismic forces on the substructure elements. This reduction in shear and bending on each pier and its foundations due to the installation of STUs may mean that any increased traction and braking or seismic load requirement can be absorbed immediately without any need for pier or foundation strengthening by conventional strengthening procedures, which would disrupt traffic. A long viaduct spanning a reservoir in Eire has been strengthened by installing new STUs at the drop in span deck joints. This simple method of sharing out the required increase in traction and braking was preferred to the alternative of column strengthening by means of conventional methods under water. In India, many long-span bridges constructed on major rivers such as the Ganga, Jamuna and Brahmaputra require retrofitting for the higher seismic loads stated in the revised bridge design code for seismic loading, and the use of STUs can solve this problem in the most cost-effective way and without interruption to traffic.

STU links can be used in a retrofit application on bearing plinths to prevent simply supported decks from falling off the plinths during an earthquake event, a problem that was seen during the earthquake in Los Angeles in 1994. The consequent and current strengthening of over 1000 bridges in California included a review of available methods, which introduced an interesting new definition of an STU. It was said that an STU acts as a ‘lock-up’ device at joints and bearings to hold the bridge together under the shock impact of an earthquake, ensuring that the seismic loading is shared beneficially around the structure in order to greatly reduce the possibility of collapse. This definition has given the STU the appropriate new name ‘lock-up device’, and this term is used by some manufacturers and engineers.

It is the capability to resist dynamic forces, yet provide virtually no increased resistance to the slow normal structural movements that occur due to temperature, shrinkage and creep that makes STUs the ideal choice for retrofitting existing bridges. For many bridges, the removal and replacement of bearings to provide sufficient damping of seismic forces is not practicable due to the substantial traffic disruption that would occur while physically lifting the deck to replace the bearings. For concrete bridges, the weight of the concrete deck and the limited space available on top of the piers to lift the deck and replace the bearings make bearing replacement too costly.

STUs can make a major contribution to the modernisation and modification of the existing stock of viaducts. A large number of these viaducts feature a long sequence of simply supported deck spans, often supported on a series of high substructure piers. This is particularly evident on major river crossings, where high navigation clearances require long approach viaducts (Figure 5.1).

The piers under each simply supported span carry a fixed bearing for one span alongside free bearings for the adjacent span. This means that the design longitudinal traction and braking forces must be applied individually to each deck throughout the viaduct. The main resistance is offered by the pier supporting the fixed bearings of that particular span, generally with a small additional resistance from the friction generated at the free bearings carrying that span, located over the next pier. This means that a substructure of this type, with, say, 10 equal-height piers has a total resistance capacity of 10 times the deck design traction and braking longitudinal loads, a capacity that is, unfortunately, not available because of the simply supported articulation. This large extra resistance capacity can be realised by placing STUs across the joints between the simply supported span at either deck or bearing level.

The following are some case studies of the strengthening of existing highway bridges by means of STUs.

## 5.2. Carquinez Bridge, California, USA

The Carquinez Bridge in California (Figure 5.2) is a good example of the strengthening of an existing bridge by means of STUs for a revised (higher) seismic loading. The Carquinez crossing, which carries Interstate 80 over the Carquinez Strait, consists of two bridges – one dating back to 1927 designed by Steinman (on which STUs were installed for the first time), and the other a more recent structure built

**Figure 5.1** A typical major river crossing

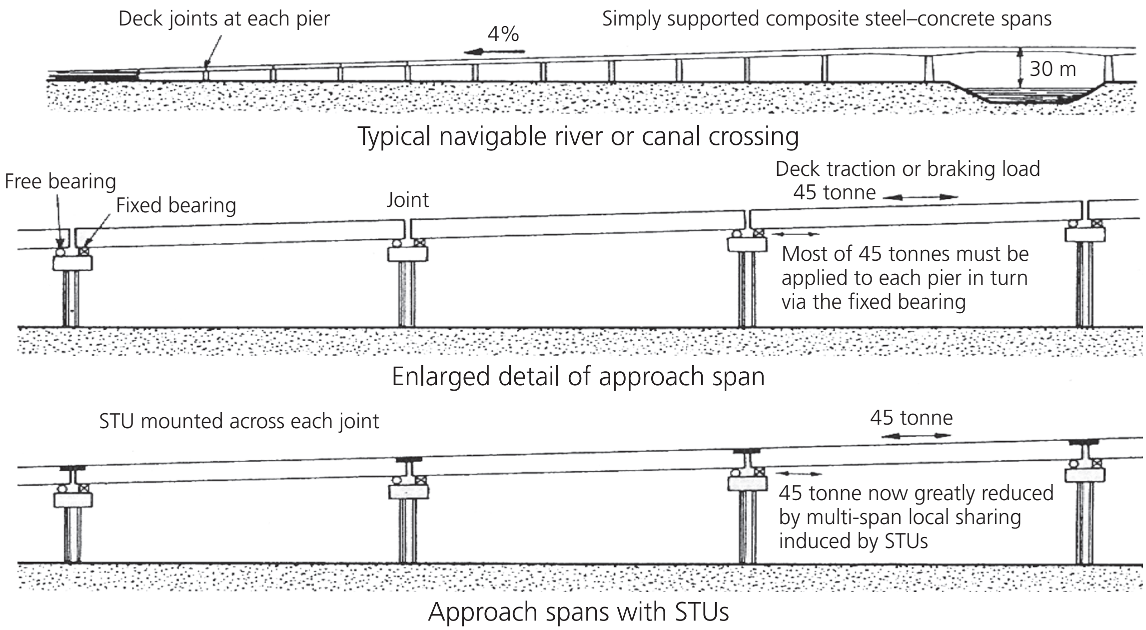



**Figure 5.2** The original Carquinez crossing over San Francisco Bay (*Bridge Design and Engineering, Third Quarter 1999*, p. 54). © Helena Russell/Bridge Design & Engineering

Figure 5.3 The new suspension bridge and the two older Carquinez crossings

in 1958. The two bridges carry more than 100 000 vehicles a day between Contra Costa and Solano County, and are a vital link in the highway system connecting San Francisco and Sacramento.

When the link across the Carquinez Strait was first constructed it was the first major bridge to be built over the deep water of San Francisco Bay, and its long-span cantilevered steel truss was regarded as state of the art at the time. The later structure was designed to match the original bridge in looks, and so the two bridges are identical in terms of their structural form, length, span arrangement and overall geometry. The only difference in geometry is that the new bridge is 18 m wide compared with the original structure's width of just 12 m. Both bridges have four spans (two central spans of 335 m and two side spans of 152 m) and a main central pier support, and both consist of a steel through cantilever truss superstructure on steel-braced truss substructure supports, with concrete caissons and steel piles as foundations. Both bridges have suspended spans between the cantilever sections, and STUs to transmit longitudinal loads across interior expansion joints. These hydraulic buffers were designed to allow the expansion joints to move during normal temperature variations, but provide structural continuity during an earthquake. They have a capacity of approximately 2850 kN.

During its condition study of the bridge crossings, the California Department of Transportation (Caltrans) decided that the most economical option was to replace the 1927 bridge with a new one and to retrofit the 1958 structure. The then 42-year-old 1927 bridge was considered to be at risk of serious damage during a major earthquake along the Franklin fault, which is located within 1.5 miles of the bridge. It was decided to replace this bridge with a suspension bridge located just on the west side of the 1927 structure and to demolish the original bridge once the new bridge had been completed. Figure 5.3 shows all three bridges, including the 1927 structure before it was demolished, and Figure 5.4 shows the 1958 retrofitted bridge with the new suspension bridge after the 1927 structure had been demolished.

Figure 5.4 The new suspension bridge and retrofitted 1958 steel truss bridge after demolition of the 1927 bridge at Carquinez

In addition to strengthening the superstructure and the foundations, the retrofitting works for the 1958 bridge structure included replacement of the existing hydraulic STUs. The existing STUs, located at the expansion joints on the 335 m spans (Figure 5.5), did not have enough capacity to resist the probable yield capacity of the truss chords, and were replaced with six large STUs, each of 1600 ton capacity

Figure 5.5 Elevation of the Carquinez Bridge (*Use of Lock-Up Devices for Structural Strengthening of the Carquinez Straits Bridge* by Warren M. Brown). © D.S. TechStar, Inc.

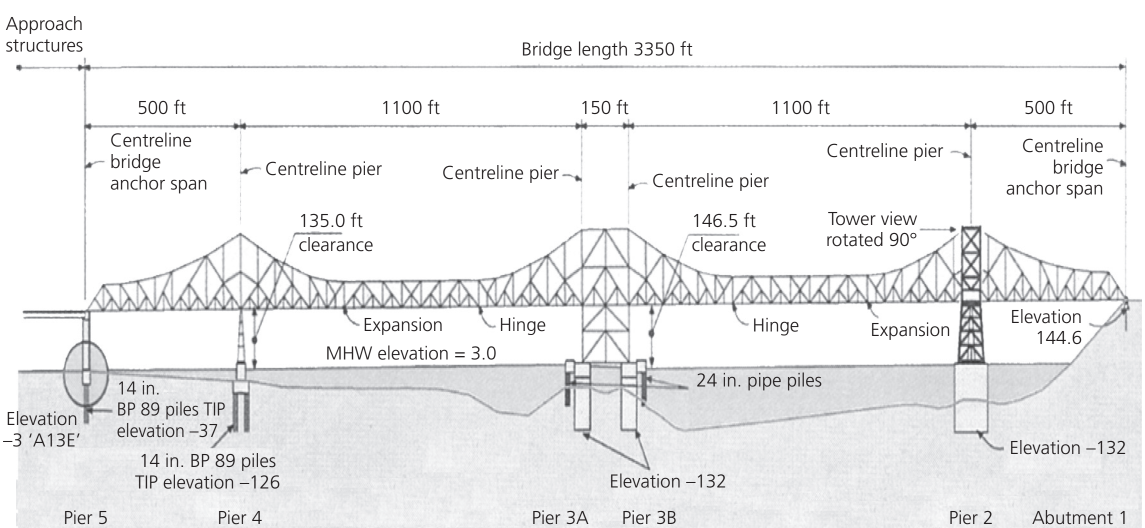


**Figure 5.6** The 16 000 kN nominal rated capacity STUs for the Carquinez Bridge (*Use of Lock-Up Devices for Structural Strengthening of the Carquinez Straits Bridge* by Warren M. Brown). © D.S. TechStar, Inc.

(Figure 5.6), manufactured and installed by TechStar, USA, and Alga, Italy, in a joint venture, set up specifically for the project. These new 1600 ton capacity STUs are the highest capacity STUs installed on any bridge structure in the world to date.

The complete retrofit of the eastbound 1958 bridge was designed by consultant CH2M Hill, and the UK contractor Balfour Beatty carried out the retrofitting works. The retrofit required seven 16 000 kN nominal rated capacity STUs, 2.9 m long and 0.86 m in diameter, with $\pm 152$ mm movement. Six of the STUs were installed on the bridge, while the seventh was kept as a spare to replace one of the installed STUs if found necessary during future testing. The total retrofitting cost was US$ 70 million.

## 5.3. Poplar Street Bridge, St Louis, USA

The Poplar Street Bridge over the Mississippi River at St Louis is a 660 m long bridge comprising two parallel five-span continuous steel box girders. Each deck facilitates four lanes of traffic and carries more than 130 000 vehicles per day across the river via US 40/I-64 and I-55/70. The span lengths from west to east are 91, 152, 183, 152 and 81 m. Two variable-depth steel box girders support each roadway. The deck is made of orthotropic steel plate, and consists of a deck plate, trapezoidal longitudinal ribs and transverse floor beams. The reinforced concrete substructure from west to east consists of a hollow shear-wall-type structure at pier 1, solid shafts with rectangular columns and continuous cap beams at piers 2, 3, 4 and 5, and a hollow shear wall structure at pier 6. The substructure is founded on 1.8 m diameter caissons to the rock at piers 1, 3, 4, 5 and 6, and a spread footing on rock at pier 2. The superstructure is supported on the substructure by spherical bearings moving in only the longitudinal direction at piers 1, 2, 4, 5 and 6, and spherical fixed bearings at pier 3.

The bridge was originally designed by Sverdrup and Parcel and Associates in 1963. Jacobs Civil Inc. (formerly Sverdrup Civil) evaluated and designed retrofits for this bridge for the Mississippi Department of Transportation (MoDOT) using the guidelines current in 2001 (FHWA, 1983, 1995). The analytical model used in the seismic evaluation consisted of 15 000 degrees of freedom, and included representation of the deck, box girders and piers 1 and 6, with shell elements, and the bearings, cross-frames and piers 2, 3, 4 and 5 were modelled as beam elements. Linear spring elements were included in the model to represent the stiffness of the soil–foundation system, and mass was applied to account for other components such as parapets. Major sign structures and one lane of traffic live load per roadway, etc. were not included in the analytical model. Dynamic analysis was performed using the linear elastic response spectrum method and spectra based on the parameters, return periods of 475 years (American Association of State Highway Transportation Officials (AASHTO, 1996)), 2500 years (Federal Emergency Management Agency (FEMA) design principles) and the maximum credible earthquake (MCE) design level (USACE, 1995). In addition to the three design spectra, the analysis included cases with completely rigid soil springs and with linear soil springs, and with existing bearings and with moving bearings supplemented by rigid longitudinal restrainers. The results of the dynamic analysis were evaluated using the capacity/demand (C/D) ratio method presented in the Federal Highway Administration (FHWA) guidelines (FHWA 1983, 1995). According to these guidelines, C/D ratios approximate the percentage of the design earthquake at which a particular component can be expected to fail, and therefore ratios less than 1.0 indicate insufficient capacity.

The C/D ratios for the existing bridge structure identified significant deficiencies, at the AASHTO design level, in the bearings, the lap splices at the base of the columns and the lap splices at the base of the piers, in spite of design acceleration coefficients as low as 0.12 g. Evaluation of the structure with engaged restrainers (modelled for STUs) at the moving bearings 1, 2, 4, 5 and 6 showed significant improvement in the critical C/D ratio of pier 3 at the AASHTO design level, but indicated the requirement for higher overturning capacity at pier 1. The analysis indicated that similar types of retrofit would be required to meet both the AASHTO 475-year and FEMA 2500-year design levels, while major changes in the structural response, which could conceptually be provided by isolation bearings and/or major increases in capacity, would be needed to satisfy the MCE design level.

MoDOT selected the AASHTO 475-year spectrum as the design level based on the findings and recommendations of the phase 1 evaluation. Accordingly, the retrofit strategy involved: adding STU-type longitudinal restrainers to moving bearings at piers 1, 2, 4, 5 and 6; increased transverse force capacity of the bearings at piers 1, 4, 5 and 6, using reinforced shear blocks and steel bumpers; strengthening of the longitudinal and transverse capacity of the pier 3 bearings; adding rock anchors to the pier 1 foundations; confinement of lap splices in the reinforcement at the base of the columns; and reinforcement of the lap splices at the base of piers 2, 3, 4 and 5.

During phase 2, the retrofit strategy was further refined by: evaluating viscous dampers against STUs, investigation of using longitudinal restrainers at piers 2, 4 and 5 with bearing seat extensions at piers 1 and 6; investigation of the longitudinal shear capacity of piers 1 and 6; and investigation of alternatives for reinforcing the lap splices at the base of the piers.

The study of retrofitting the bridge with viscous dampers, which are designed to dissipate significant amounts of energy as the bridge moves longitudinally under seismic loading, included a time history

Figure 5.7 The STUs installed on the Poplar Street Bridge

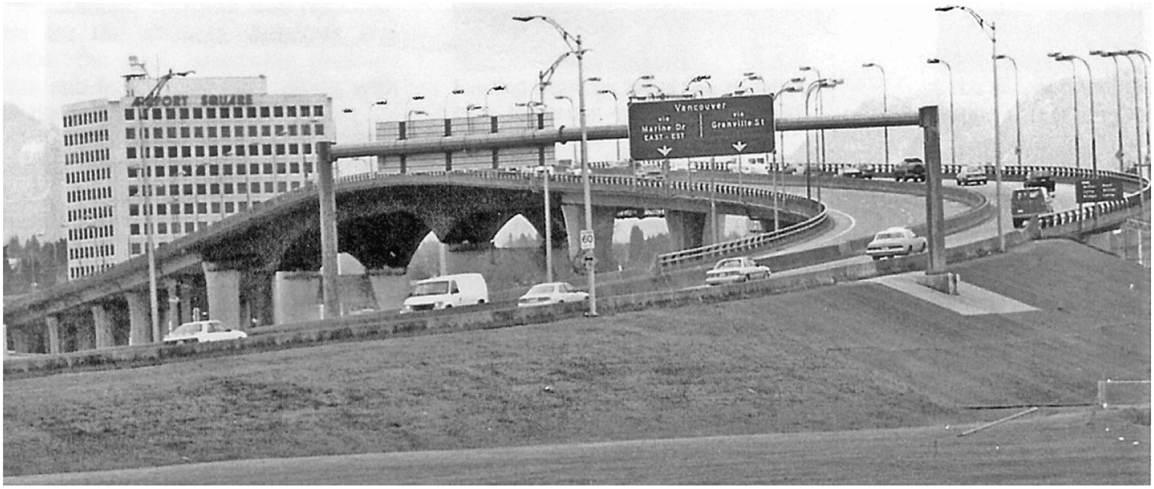
dynamic analysis of the bridge structure. The analyses were based on AASHTO spectra compatible synthetic ground motions, and damper characteristics developed from recommendations from damper manufacturers. STUs were finally selected for the retrofit primarily because obtaining force reductions with the viscous dampers was dependent on the ability of the existing bearings to displace through multiple cycles under seismic loading. Because the actual displacement characteristics of the existing bearings could not be readily quantified, STUs, which do not require significant bearing displacements, were believed to provide a more reliable alternative to viscous dampers for this bridge. The analysis to compare retrofitting with longitudinal STUs at all piers versus retrofitting with STUs at only the interior piers showed that, as expected, the first of these alternatives provided greater longitudinal displacement control and greater capacity for resisting seismic loading.

The final longitudinal restrainer configuration comprised four 1560 kN STUs at each of the moving bearings at piers 2, 4 and 5, and two 2590 kN STUs at each of the moving bearings at piers 1 and 6. The STUs were attached through steel brackets at the bottom of the steel girders and the pier cap beams (Figure 5.7), with high-strength rods through the cap beams holding the steel brackets.

Piers 1 and 6 were found to have inadequate shear capacity to resist the design restrainer forces, and this was remedied by providing reinforced-concrete shear walls inside these piers. The overturning capacity of pier 1 was increased by adding rock anchors through the grade beams at the base of the pier, as the rock was at less than 8 m deep. Confinement of lap splices near the base of the $4\ \text{m} \times 3.7\ \text{m}$ rectangular columns was achieved by steel plates with closely spaced post-tensioned steel bars. Tension in steel bars which were extended into the core of the column provided the desired confining pressure.

The major construction issue was to minimise traffic disruption, as the bridge is a major transportation link in the region. The strategy of retrofitting the bridge with STUs at the piers was therefore a very appropriate choice compared with the conventional method of strengthening the piers. The contractor

Figure 5.8 The Arthur Laing Bridge (*Bridge Design and Engineering*, February 1997, p. 50). © D.S. TechStar, Inc.




carried out all construction activities from below the bridge, using barges on the river, and worked from the ground when possible during low water. This required coordination with barge operators in the river, but the construction was completed with minimum disruption and without any disruption to the traffic on the bridge. The construction of the retrofit was completed in late 2002.

## 5.4. Arthur Laing Bridge, Vancouver, Canada

The Arthur Laing Bridge in Richmond, Vancouver, Canada (Figures 5.8 and 5.9), owned by the Vancouver International Airport Authority, is a concrete box girder bridge with 22 spans (not including ramps). The main bridge comprises 11 two-span continuous modules with an expansion

Figure 5.9 Structural elements of the Arthur Laing Bridge

joint between each module. There are 37 piers supporting the superstructure of this main bridge, consisting of single and multiple free-standing or cantilever columns. The bridge is located on the main arterial route into and out of the city's international airport, and the airport authority was aware that any damage to the bridge necessitating a closure could have a serious effect on the airport operations. It was decided to instigate a full-scale seismic retrofit to protect against earthquake damage in accordance with the *Seismic Retrofitting Manual for Highway Structures – Bridges* (FHWA, 1995).

Sandwell Engineering and Construction Services Group, Vancouver, was commissioned to prepare documents for the seismic upgrade, and they recommended that the works be carried out in three phases. In phase one the pier footings would be retrofitted to 130% of the minimum compression capacity of the existing piles or soil, the elastic strength of the columns, or the redistribution forces after the bearing retrofit. Phase two would provide a complete bearing retrofit to resist seismic shears, computed using a 3D elastic–plastic spectral analysis.

The initial phase of the contract was completed satisfactorily. During the initial inspections and analysis in phase two it was found that the existing bearings were not adequate to resist the seismic shear developed during an earthquake. The consultants developed a full scheme of bearing retrofitting, which would strengthen the bridge under seismic loads but leave unchanged the behaviour of the bridge under non-seismic loads, such as static vertical loads, temperature, shrinkage, creep and wind deflection.

The existing bridge bearings located on either side of every expansion joint were unidirectional bearings allowing movement in the longitudinal direction only. Due to this arrangement, if a seismic event should occur, the piers in the centre, having fixed bearings only, would resist any seismic force in the longitudinal direction. The bridge is in a seismic zone 4, which carries a design acceleration coefficient of approximately 20%.

Retrofitting this bridge required that the transverse and longitudinal forces generated by the inertia of the superstructure had to be transmitted to the piers. In addition, the substructure units had to be capable of withstanding the forces generated by this transfer in both directions. To provide adequate seismic resistance, the bridge required retrofitting in both the transverse and the longitudinal direction. To transmit the seismic forces in the transverse direction, 61 new shear pins were provided to connect the deck to the pier caps. As this part of the work involved coring through the concrete bridge deck and through the box into the pier cap, the bridge was closed each evening to allow access for the contractors. Once the 250 mm core holes had been drilled, 200 mm shear pins were inserted, and the holes capped.

Shear pins can transmit the transverse forces after a small movement. Because of the need to provide for the longitudinal movements due to temperature, shrinkage and creep, etc., the pins were fixed through a larger hole, allowing their free movement in the longitudinal direction. Additional resistance against the longitudinal seismic forces was provided by installation of 11 STUs to piers at the expansion joints (Figure 5.10), 10 of these having a rated capacity of 2100 kN and one having a rated capacity of 700 kN. These two rated capacities satisfied the demand forces of 2013, 1965, 1883, 1642, 1627, 1550, 1436, 1358, 1357, 1339 and 684 kN.

All but one of the STUs has a far greater capacity than required, providing resistance against seismic events greater than the design event. Reducing the number of different types of STUs from four to two offset the cost of having oversized units. The existing piers that are fixed in the longitudinal direction already transmit that force component through the bearings to the pier. However, this did not provide adequate resistance for the longitudinal seismic forces. Therefore, 11 STUs were installed on the piers

Figure 5.10 An STU being installed at the expansion joint of a pier on the Arthur Laing Bridge (*Bridge Design and Engineering*, February 1997, p. 51). © D.S. TechStar, Inc.

at the expansion joints to transmit the seismic forces while maintaining the capability of the deck to expand and contract. The STUs were fixed to the pier face by 32 mm high tensile steel rods drilled through the pier and post-tensioned. Figure 5.11 shows the Arthur Laing Bridge completely retrofitted with all the STUs installed.

Figure 5.11 The Arthur Laing Bridge, showing the installed STUs

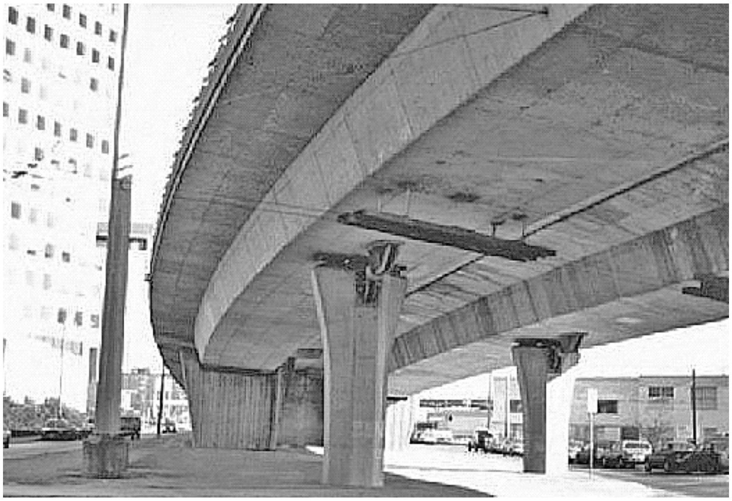
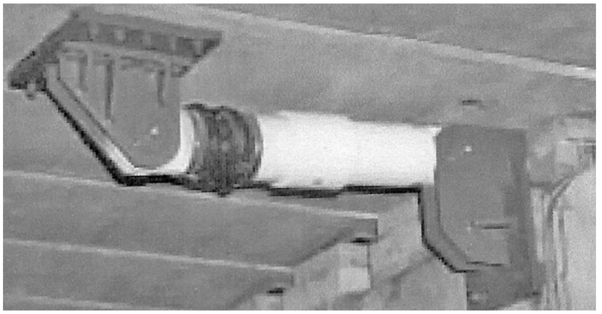


Figure 5.12 An STU installed on the Hamborok Viaduct, UK. © Freyssinet



## 5.5. Hambrook Viaduct, UK

Hambrook Viaduct is situated near Junction 19 of the M5 motorway where it meets the M32 spur into Bristol, UK. The piers and abutment needed to be strengthened to withstand sudden longitudinal shock loading from vehicle braking on the deck, and to transfer the forces to the newly strengthened piers and abutments. Transpec SHT 550-100 STUs having a load capacity of 550 kN and a movement capability of $\pm 100$ mm, manufactured by Freyssinet, were installed to absorb the braking force.

The STUs are mounted within a bracket arrangement fixed to the structure (Figure 5.12), and designed such that any subsequent repair or maintenance to the unit can be made with minimum disturbance to the assembly. Prior to delivery, each unit was rigorously tested at Bochum University in an extensive programme specified by the client.

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# Chapter 6
# STUs for new and existing railway bridges

## 6.1. Introduction

Railway bridges are generally similar to highway bridges, but trains apply high traction and braking forces to the deck and the supporting substructures. Transportation Technology Centre, Inc. (TTCI), USA, carried out a test in December 1998 to measure the longitudinal forces transmitted to a 60 ft single-span, ballast deck plate girder bridge. It was found that very little of this horizontal force was taken up by the rails, and that a new AC locomotive significantly increased horizontal forces into bridge piers and abutments. TTCI concluded that for a ‘typical train’ with three SD70MAC locomotives in front, 117 fully laden loaded coal cars and two SD70MAC locomotives at the rear, the dynamic braking and traction forces into the bridge equalled up to 115 kilopounds (kips) per pier and abutment. The TTCI test results confirmed concerns already voiced by several prominent bridge design consultants that the American Railway Engineering and Maintenance of Way Association (AREMA) design standards should be updated for the handling of these new large forces. STUs can provide an economical and easily installed solution to distribute these forces across the entire bridge structure and to maximise the bridge’s load-carrying capacity. Besides, if the bridge is small and the train is very long, the time for which the traction and braking forces are applied can be much longer than on a highway bridge. STUs are subjected to a small permanent displacement under the application of long-duration sudden-impact loading due to creep, and therefore such railway bridges can sustain progressive creep or ‘ratcheting’ of the adjacent decks under successive loading events caused by frequent trains crossing the bridge. For such railway bridges it is important to provide a special type of STU called a ‘crawl connector STU’ (see Chapter 16), which can bring the deck back to the original position with recovery of the creep movement and stop progressive ratcheting of adjacent decks. This function can also be achieved by fixing springs with STUs, as described in Section 6.5.

STUs can be used on new and existing railway bridges, just as described for highway bridges in Chapters 4 and 5. On light-railways, it is sometimes necessary to increase the length of the trains by providing more cars due to an increase in traffic. The trains then apply considerably higher traction and braking forces to bridge structures than the original longitudinal and horizontal forces designed for the smaller number of cars. The existing substructures of such railway bridges are not designed for this increased loading and, while conventional strengthening by means of adding concrete and reinforcement to piers and foundations is a solution, the addition of STUs at the deck joints provides for load sharing and avoids the costly and disruptive strengthening achieved by conventional methods. The strengthening of such railway bridges by the addition of STUs can be simple and comparatively significantly more economical than conventional methods, and more so if such a bridge is located on a strategic location over a river or on a heavily trafficked road or railway line. Generally, the saving generated just by keeping the bridge open during strengthening by means of STUs is considerable.

The traction and braking forces may be applied many times over the life of a railway bridge structure, and there are now many examples around the world of railway bridges that have been strengthened by the addition of STUs which are functioning satisfactorily. These bridges provide a very convincing argument to the critics of the use of STUs on such structures.

Economy of retrofit can also be achieved by the addition of STUs on a railway bridge located over a valley or a river, where the height of the middle piers may be much greater than the bankside piers, as illustrated by the example discussed in Section 6.2.

Some case studies of new and existing railway bridges strengthened by means of STUs are given in the following sections.

## 6.2. Railway bridges over rivers in Indonesia – new bridges

In 1990, Foster Wheeler World Services Ltd, based in the UK, was contracted to provide basic and detailed design, procurement of specialised equipment and material, and advice during the construction and commissioning phases for a rail system project by the government of Indonesia. The route for the new rail project, which is known as the Citayam–Cibinong Railway, crossed three major rivers, two of which were fairly remote and difficult to access. Two of the crossing lengths were about 180 m and the third was about 210 m. The bridge decks were to be constructed between 21 m and 30 m above the river bed. The rainy season in Java, Indonesia, typically lasts 4–5 months, and during this period high levels of flood water cover the valleys. Because of these conditions, it was agreed that the wisest course was to keep the construction time in the valleys to an absolute minimum.

Construction of the bridge foundations in the river valleys was to be completed during the region's dry season in order ro be able to get out of the valley bed and above the water line as quickly as possible. The final choice for the bridge superstructure was 30 m long simply supported steel trusses (Figure 6.1), built using the cantilever method, starting from the abutments. The size of these bridges and their importance to the operation of the railroad necessiated dynamic analysis to predict their behaviour during an earthquake. It was determined that, because of the predicted differential movements of the supports of varying height, the use of special design techniques, such as STUs, was necessary if the design was to be economical.

The STUs are fixed to the pier heads and longitudinally connect the spans of the normally simply supported steel trusses. They allow the thermal expansion and contraction of the individual spans but, upon receiving a shock from an earthquake or from locomotive braking and traction, the units 'lock', making the whole length of the bridge act as a tie or a strut. The superstructure and substructure thus move as a coupled unit. The piers, which have the same cross-section, would deflect by the same amount. The result is that the shock loading would be transferred from the tall piers located in the river to the short piers located on the river banks. As long bridges flex under vertical train loading to cause a cyclic movement at bearing level, the STUs for these railway bridges were installed on the pier plinths at the neutral axis of the trusses. The use of STUs not only helped to complete the piers and foundations in the dry season, but also generated considerable economy in the construction of the piers and the foundations.

## 6.3. Taiwan high-speed rail project – a new bridge

Taiwan High Speed Rail operates between Taipei in the north of Taiwan to Kaohsiung in the south on a route that is 345 km long. This project was one of the largest transport infrastructure initiatives in the

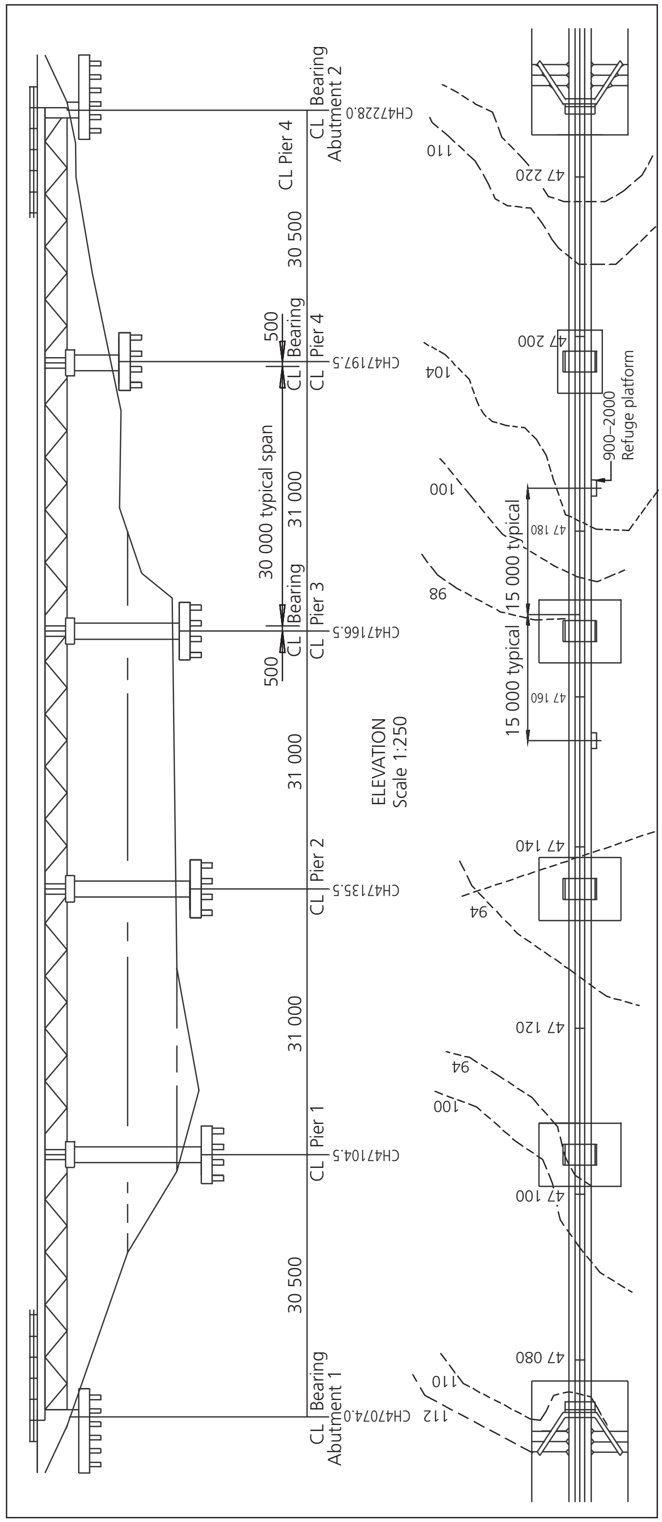



**Figure 6.1** Raiway bridge over the Ciliwung River in Java, Indonesia

Figure 6.2 Taiwan High Speed Rail viaduct with STUs at the expansion joints

world to date. The US$ 16 billion scheme enables 300 000 people a day to travel at speeds of up to 300 km/h between Taipei and Kaohsiung, along the island's western corridor. It comprises a number of seismic-resistant viaducts designed by Faber Maunsell consulting engineers for a train speed of 300 km/h. The project was scheduled for completion by October 2005.

Project C270, one of the 12 major civil works design and construct contracts, comprised a 38 km long standard viaduct through Changhua, Yunlin and Chiayi counties (Faber Maunsell, n.d., 2004). The viaduct comprised 35 m long precast, post-tensioned, simply supported box girders supported on a single column type of substructure, as shown in Figure 6.2. The box-girder deck is supported on two free-sliding pot bearings at one end and fixed pot bearings at the other for each span. Shear keys are provided at each end of the span to provide transverse restraint. The foundations for the viaduct comprise 2 m diameter reinforced-concrete bored piles up to 65 m in length.

The island of Taiwan sits on an active tectonic region that has a seismic activity among the highest recorded in the world. Thus the criteria for the performance and safety of the high-speed railway under different levels of seismic ground acceleration were important for the purpose of design. Therefore, the viaduct was designed to meet the following three requirements (THSRC, 1999).

- The trains must run smoothly for strict ride-performance criteria set by the client for normal operating conditions.
- The structure should remain within its elastic range and restrict movements to specified values during a significant earthquake event, so that a train may stop safely from a speed of 300 km/h.
- The viaduct should support the design loads and suffer only minor repairable damage from the maximum design earthquake.

The viaduct structure was analysed using a finite-element computer program called LUSAS Bridge. The design ground acceleration for the anticipated earthquake could exceed 0.6*g*, and it had

Figure 6.3 A typical 3900 kN STU installed at the expansion joint of the simply supported viaduct

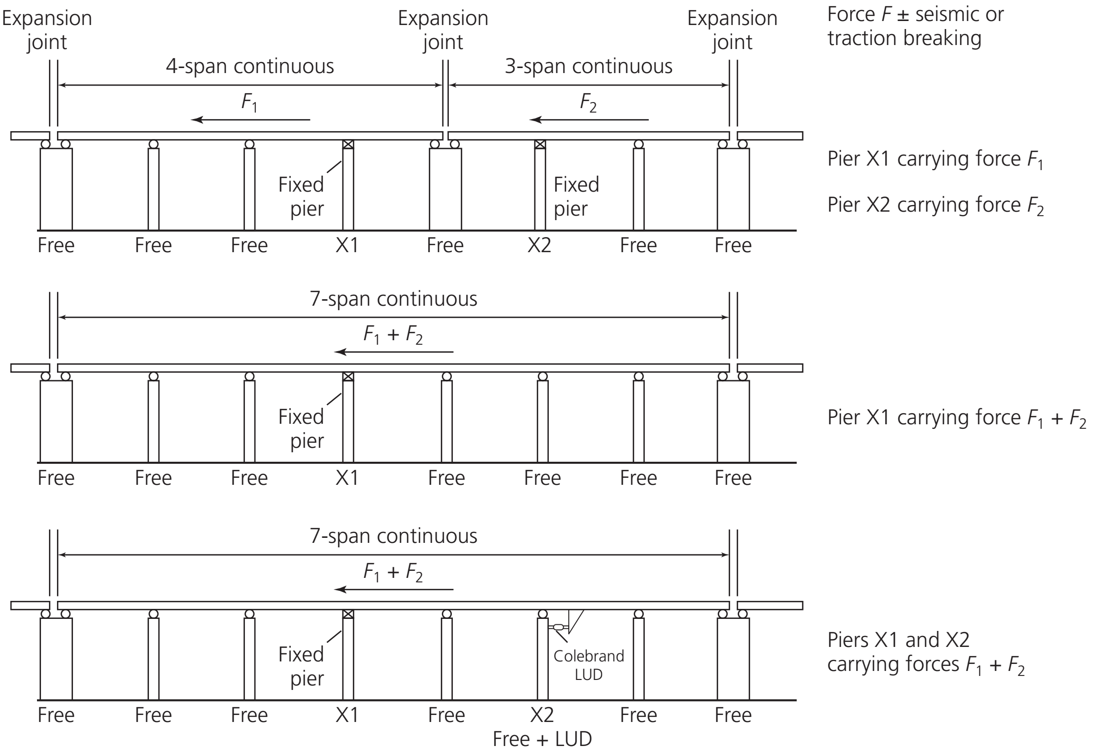
to be demonstrated that the substructure was sufficiently stiff to ensure safe operation of the railway during frequent earthquakes and to allow safe stopping of a train should the need arise. The versatility of the LUSAS Bridge program helped demonstrate these criteria, and made it possible to complete the design within a demanding timescale. The analysis showed that load sharing by means of STUs connected at the expansion joints of the viaducts (Figure 6.3) satisfied the above three requirements, and provided an economical and safe design by restricting the movements to the specified values during an earthquake event. STUs having a 3900 kN capacity and $\pm 125$ mm movement capability were provided at the expansion joints of the viaduct to protect it against a 900-year return period earthquake.

## 6.4. Light rail transit project, Kuala Lumpur, Malaysia – new bridge

The design arrangement for the light rail transit system was for a three-span continuous bridge followed by a four-span continuous bridge, with both continuous spans having one fixed pier with all the others being free piers (Figure 6.4).

Due to track requirements, it was not possible to have an expansion joint between the two bridges, and therefore it was decided to change the span arrangement into a seven-span continuous bridge (see Figure 6.4). As the foundations and piers were already under construction, the revised structure could now accommodate only one fixed pier with the others being free piers. However, a single fixed pier was inadequate to carry all the increased longitudinal forces, and it was therefore necessary to introduce a second fixed pier to share these forces. This was made possible by using an STU (see Figure 6.4). The STU was connected between the pier cap of the originally designed fixed pier and

**Figure 6.4** Span arrangement for the light rail transit system, Kuala Lumpur



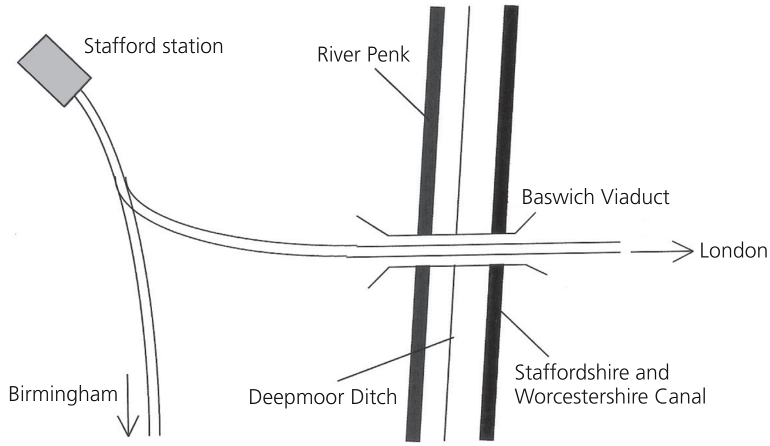

the underside of the bridge deck. The bearings were changed from a fixed-bearing type to a free-sliding bearing type.

## 6.5. Baswich Viaduct, UK – a new deck with an existing substructure

Baswich Viaduct is situated on the West Coast Main Line between Stafford and Lichfield, UK (Figure 6.5). The viaduct had been carrying fast trains since the 1890s, and is located parallel to another viaduct carrying slow trains. It was revealed during an inspection of the viaduct that the deck was in a rundown state and had come close to its life expectancy, although the substructures were in reasonably good condition. A feasibility study showed that a new deck construction supported on the existing substructure would be economical compared with a major repair option, as the new viaduct must allow for a new generation of high-speed trains travelling at up to 200 km/h.

The existing viaduct was an eight-span composite steel girder with a concrete deck, continuous structure measuring 110 m in length on a 1200 m radius curve supported by flexible reinforced-concrete piers and abutments. Demolishing the old viaduct and replacing it with a new one would have involved closure of the railway track for a long time. Therefore, in order to cause the least disruption to train services, the alternative was to build a new deck alongside the original one and then, during a 10-day closure of the track, demolish the old one and move the new deck into place over the existing substructure.

Figure 6.5 Plan showing the location of the Baswich Viaduct. © Jarret Structures




Works were divided into three parts.

- **Temporary works.** A temporary road and bridge were built across the Staffordshire and Worcestershire Canal for access to the site, and temporary foundations were placed to support the steel cradles on which the viaduct deck was to be constructed.
- **Construction of the new deck.** The bolted steel girders were placed in position and a reinforced-concrete composite deck constructed. The deck was then jacked up and equipped with rollers.
- **Rolling of the new deck in place.** The fast-line railway track was closed for 10 days while the track and power cables were removed and the old composite deck was demolished. The new deck was then rolled into place over the existing substructure and the track replaced.

As the existing piers were flexible, the displacement of the deck was going to be as much as 105 mm due to the application of a dynamic braking force of 700 ton generated by high-speed trains. It was therefore decided to use STUs at both the abutments to control the displacement due to the dynamic braking force to a maximum of about 10 mm.

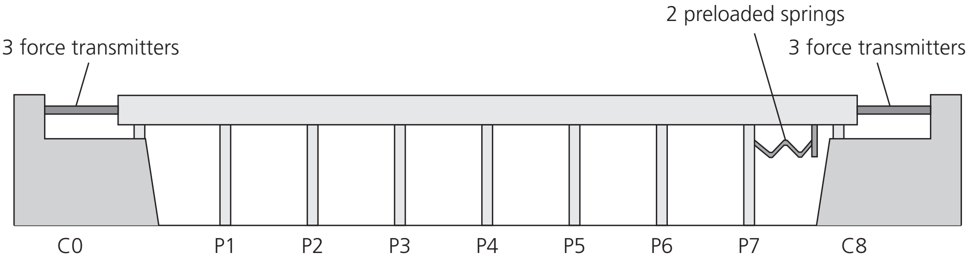
However, as STUs can be subjected to a small permanent displacement due to creep every time a train passes over a bridge, the deck can be out of its original position. Thus the deck must be centred in order to prevent it from hitting the abutment. This function was realised by using a device that works as a spring fixed between the deck and the last pier column (P7 in Figure 6.6).

The deck is rigid and supported on seven flexible-column piers. Three STUs located between the deck and each abutment were installed as shown in Figure 6.6. Two centring spring devices were fixed on pier 7 and created a fixed point. Neglecting the stiffness of the piers, the stability of the system implies

$$F(\text{braking}) = 700\ \text{ton} = (6 \times \text{Force STU}) + (2 \times \text{Force Spring})$$

where Force STU and Force Spring depend on the displacement of the deck.

Figure 6.6 Elevation of the Baswich Viaduct showing the location of the STUs and the springs. © Jarret Structures




For safety reasons, the STUs were designed to handle the braking force in case of failure of one of the six STUs. Under the normal static force due to the expansion and contraction of the deck, the STUs allow a maximum of $\pm 80$ mm displacement, 65 mm of this being due to temperature and shrinkage and 15 mm being due to the stroke of the preloaded springs. While under the dynamic braking force (the maximum acceptable displacement is 10 mm), the STUs allow the deck to move by less than 8 mm under the dynamic braking design load of 700 ton. Again, should one of the STUs fail under the dynamic braking load application then the maximum displacement would be 9.3 mm to $<$10.0 mm.

## 6.6. Docklands Light Railway (DLR), London, UK – an existing bridge

An interconnecting new light-rail service for the rapidly developing London Docklands area, located just east of the highly congested financial area of the City of London, was completed in 1987. The 12.1 km system, linking Tower Hill in the city with the Isle of Dogs and Stratford in the Docklands, included 2 km of new steel composite decks. Even before the 1987 opening, a decision had been made to locate a massive new building complex, Canary Wharf, to straddle one of the DLR stations. With a projected eight-fold increase in passenger traffic, it was necessary to provide doubled trains and increase trip frequencies when the complex was part completed in 1991.

A typical viaduct for the DLR is continuous over seven spans between deck expansion joints, as shown in Figures 6.7 to 6.9. The train traction and braking horizontal loading, together with any wind

Figure 6.7 Sketch showing a typical seven-span continuous viaduct on the DLR (Pilgrim and Pritchard, 1990). © Institution of Civil Engineers

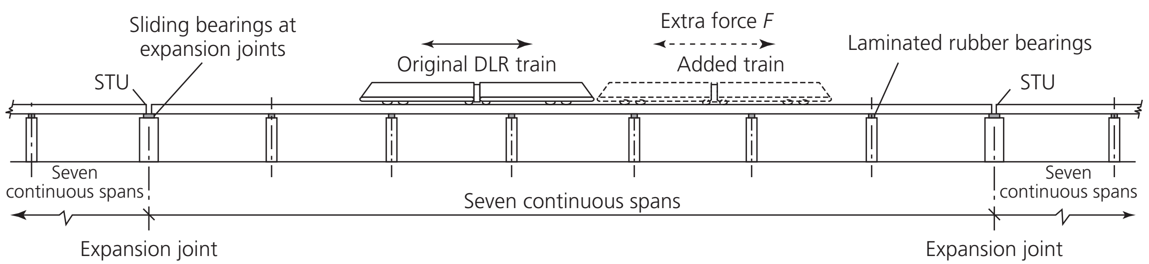


Figure 6.8 Cross-section showing typical details of the DLR (Pilgrim and Pritchard, 1990). © Institution of Civil Engineers

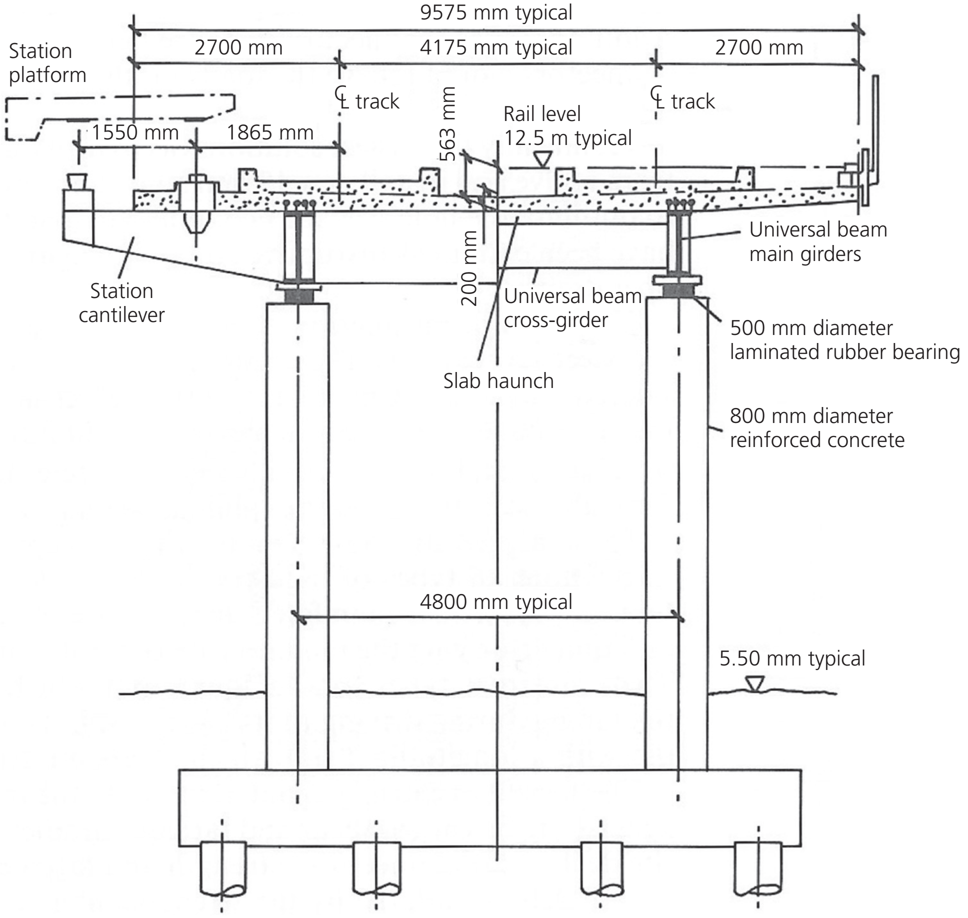



Figure 6.9 Typical DLR viaduct. © Institution of Civil Engineers

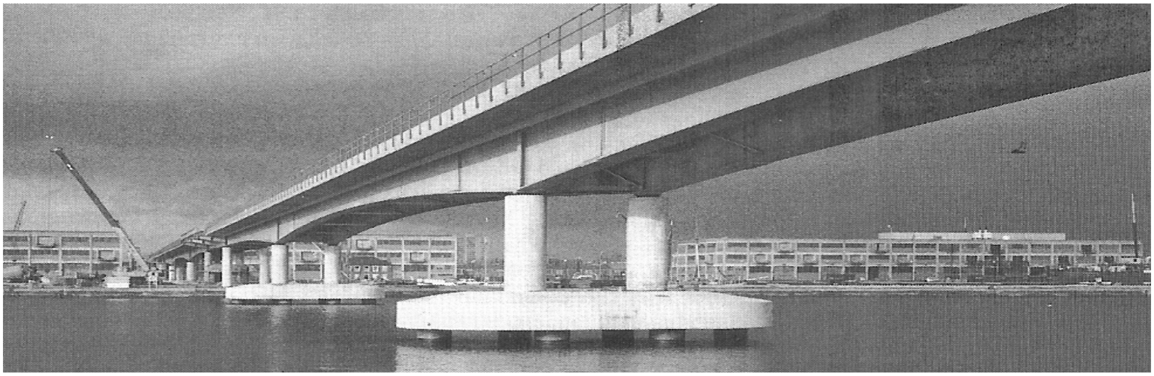
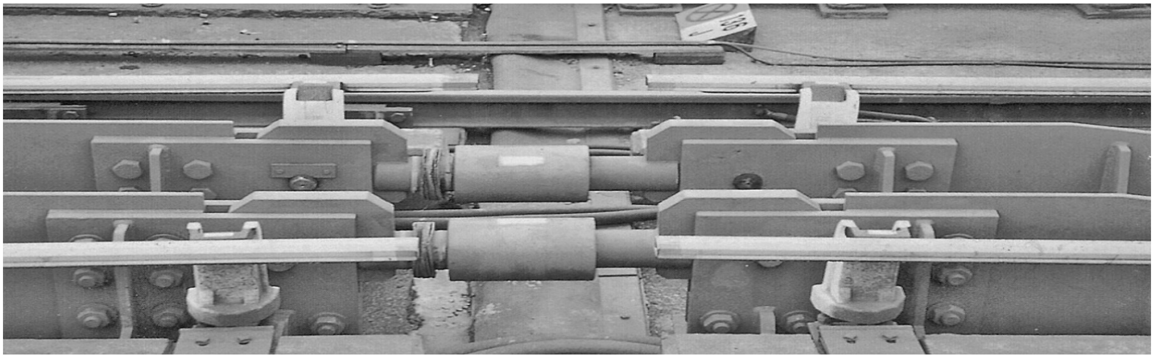


Figure 6.10 Installation of STUs at the expansion joints of the DLR



loading, is shared between the slender reinforced-concrete piers, which generally support the deck by way of rubber bearings. STUs were installed across the expansion joints on top of the reinforced-concrete deck at rail level, linking the joints between the adjacent seven-span viaducts (Figure 6.10). When the increased longitudinal traction and braking load is applied to one particular viaduct, the load is transmitted and shared with the adjacent unloaded viaducts. Installation of the STUs eliminated the need for costly strengthening of the piers and foundations by conventional means, and allowed the train service to continue without interruption during installation of the STUs (Figure 6.11) (Pilgrim and Pritchard, 1990).

Figure 6.11 The DLR in operation with added trains after installation of STUs

## 6.7. Putney Bridge, London, UK – an existing bridge

This Victorian bridge (Figure 6.12), which carries the District Line of the London Underground over the River Thames, needed to be upgraded and repaired as it had come near the end of its design life. Eight successive simply supported steel spans of approximately 45 m (Figure 6.13) required urgent repairs due to cracks in the old rubble-filled cast-iron piers. At each alternate pier

Figure 6.12 Putney Bridge over the River Thames

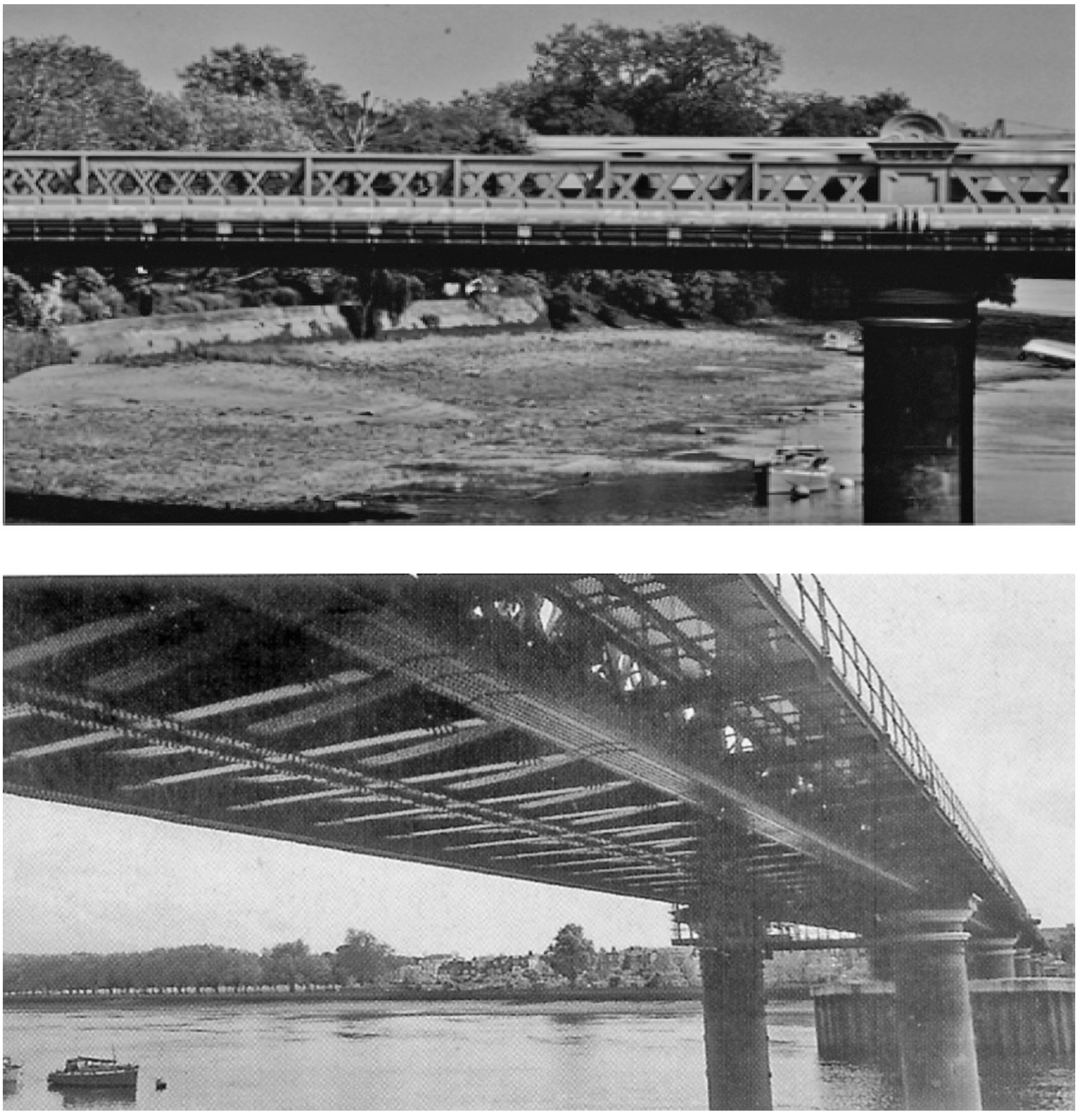
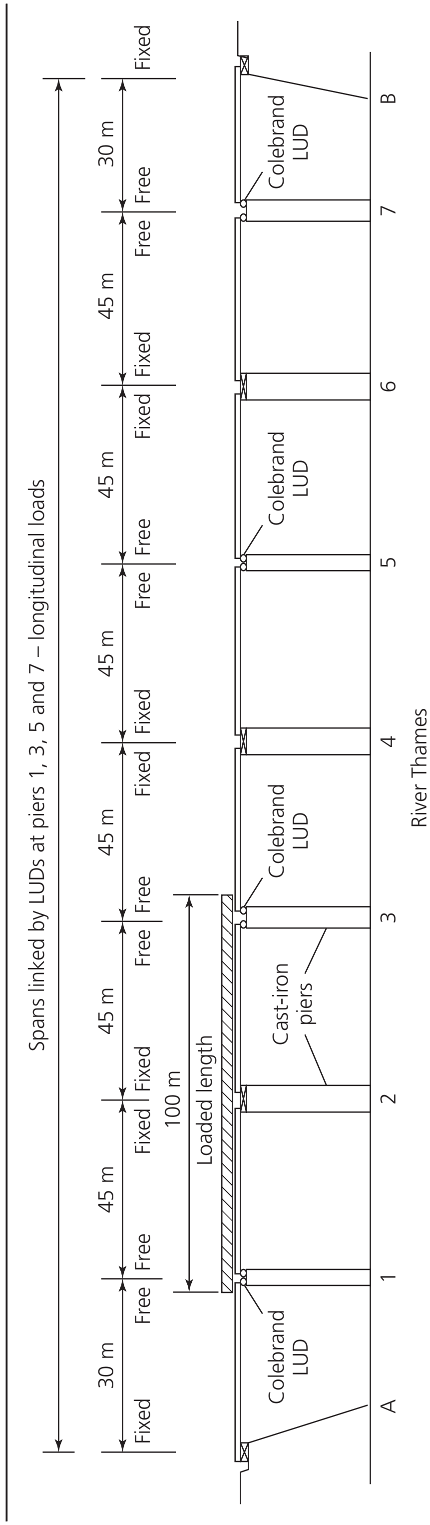
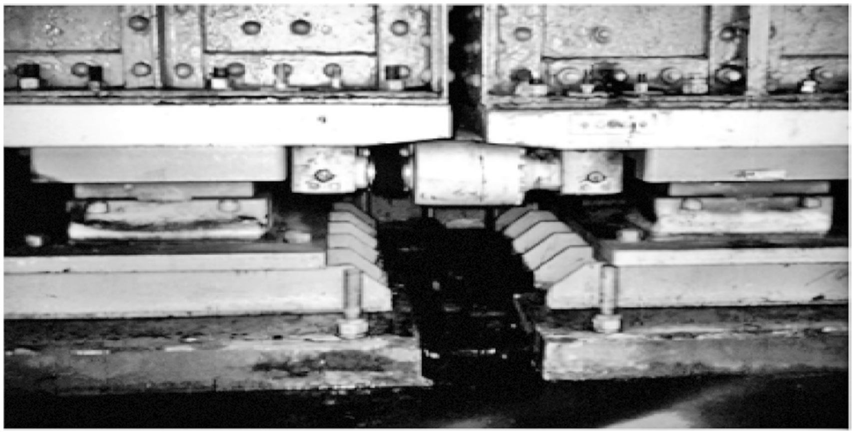


**Figure 6.13** Sketch showing the load distribution on the substructure of Putney Bridge due to rail loading before the installation of STUs




Loaded length 100 m now load 800 kN (braking)
Load now acting on piers 2, 4 and 6, by computer analysis = 120 kN max. (16.7%)
Load acting on abutments equivalent to 400 kN
Load distribution dependent on relative stiffness between piers and abutments
NB: Abutments upgraded to carry 400 kN

Figure 6.14 An STU installed at an expansion joint



there were pairs of fixed bearings, while at the intermediate piers there were pairs of free bearings (see Figure 6.13).

The operator, London Underground, wanted to upgrade the line with longer trains, which would give rise to higher traction and braking forces on the already cracked piers. A design requirement was to ensure that the columns would remain in compression so that the cracks would not increase. Therefore, the horizontal longitudinal forces on the fixed piers needed to be reduced. Strengthening of the piers was considered uneconomical; however, there was sufficient strength within the abutments. A scheme of deck-to-deck STUs at the expansion joints (see Figures 6.14 and 6.15) was devised which provided continuity to transfer the enhanced longitudinal forces and reduced the braking force on the fixed piers from 720 kN to 129 kN. This reduced the bending moment on the base of the pier, resulting in compression stress at the bottom of the pier, with the direct compression stress being greater than the tensile stress caused by the reduced bending moment. This also transferred load to the abutments, the load at each abutment being in the range 300–400 kN. The abutments were upgraded to carry this new design load of 400 kN.

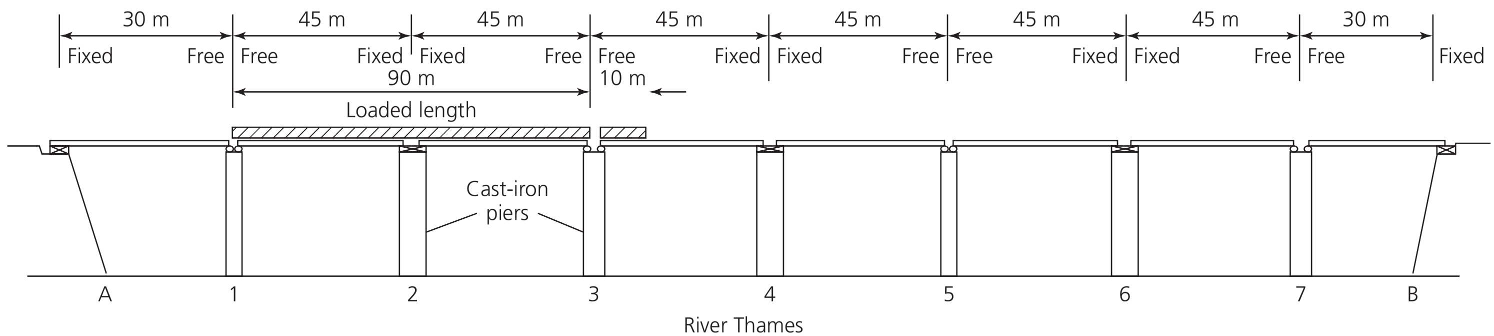
## 6.8. Neath Railway Bridge, South Wales, UK – an existing bridge

This four-span Victorian continuous curved bridge of approximate total length 80 m (Figure 6.16) is located on the London to Swansea main high-speed railway line. The central fixed piers were found to be inadequate to take the longitudinal braking/traction forces of modern trains. One abutment was adjacent to a trunk road and had no capacity for its strength to be enhanced. Therefore, strengthening had to be concentrated at the other abutment, which had reserve capacity.

The articulation of the bridge prevented a fixed bearing being provided at the abutment. Therefore, in order to achieve the transfer of all longitudinal forces, the designer elected to install four STUs, each with a rated capacity of 1000 kN (Figure 6.17). A particular innovation of this scheme was the use of ball-and-socket STU end connections which, because of the skew relationship between the deck and the abutment, proved to be a compact means of providing rotation in both the longitudinal and vertical directions.

**Figure 6.15** Sketch showing the load distribution on the substructure of Putney Bridge due to rail loading after the installation of STUs

Simple span arrangement – longitudinal loads




**Piers**
Total loaded length 100 m from BS 5400
RL loading
Loaded length which acts on fixed piers = 90 m
(i.e. two 45 m spans)
Therefore max. load acting on piers 2, 4 or 6 is
90 m × 8 kN/m = 720 kN

**Abutments**
30 m load 300 kN (traction)
Load acting on abutment A and B = 300 kN

**Problem**
Cast-iron piers cracked at base
Requirement to reduce tension in piers to zero, i.e. reduce bending moment so that direct compression is greater than tension stress

Figure 6.16 Neath Railway Bridge

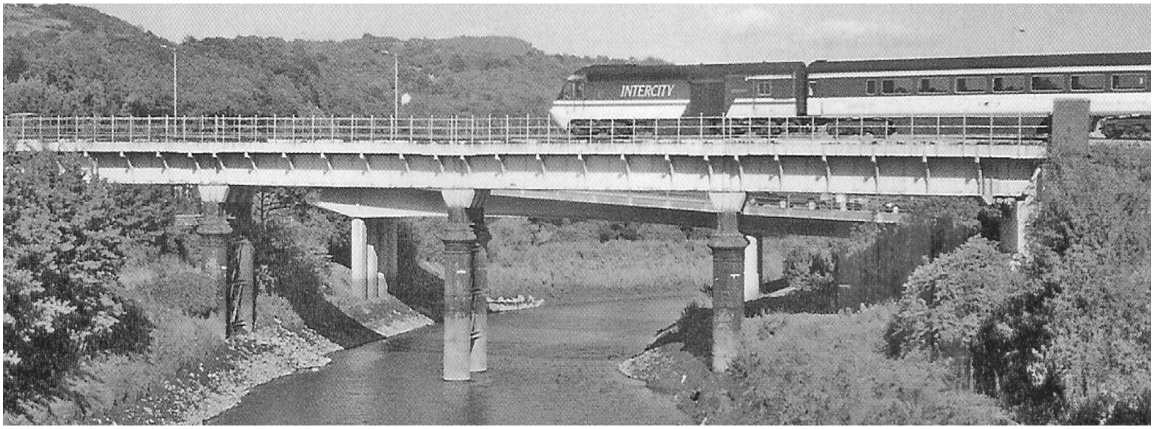
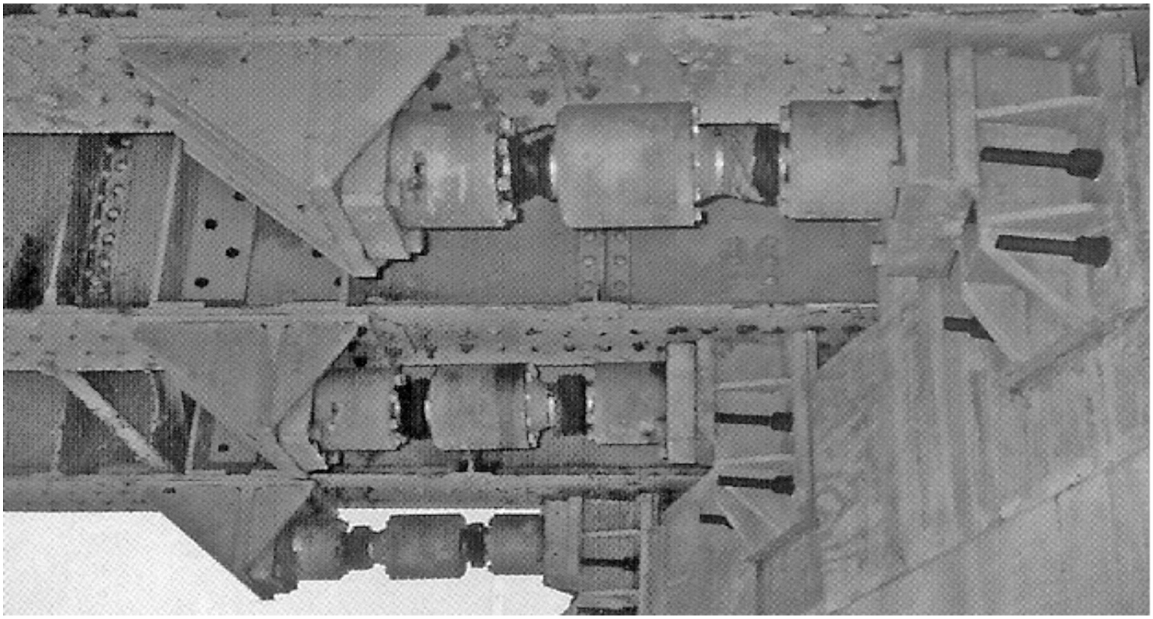



Figure 6.17 Four STUs of 1000 kN capacity installed at the abutment of Neath Railway Bridge

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# Chapter 7
# STUs for cable-stayed and suspension bridges

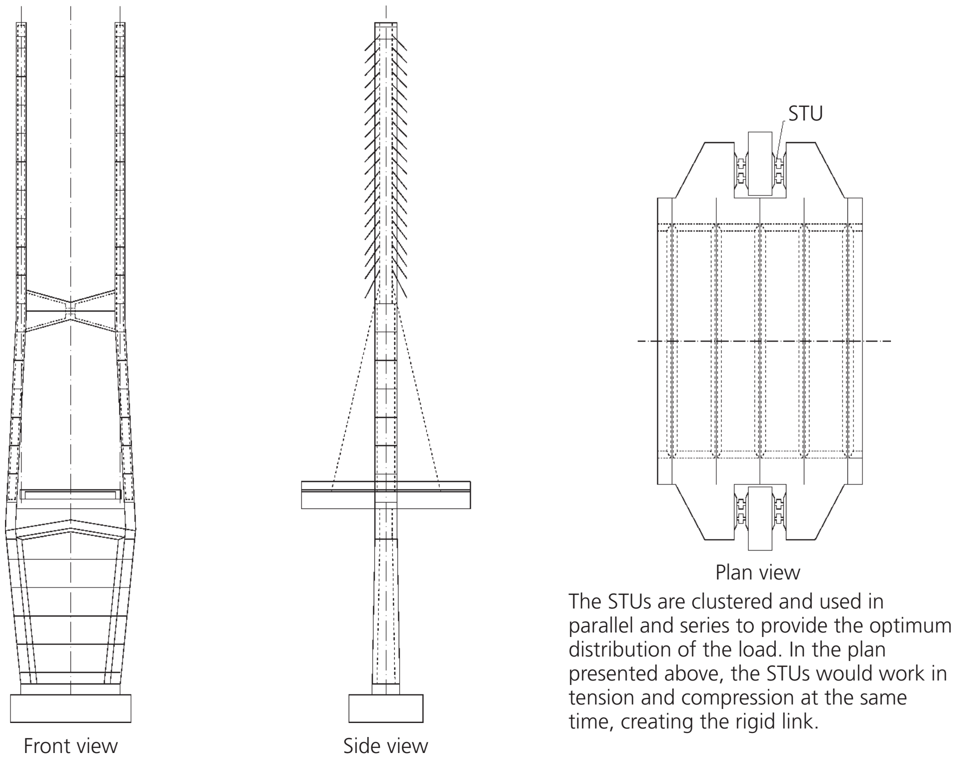
## 7.1. Introduction

The scientific seismic assessment of cable-supported bridges has been given serious attention by the bridge engineering profession only in recent decades. Before that period, many engineers involved in the engineering of such bridges apparently believed that long-span cable-supported bridges in general, and suspension bridges in particular, were for some reason immune to the effects of earthquakes. An important exception to this belief of immunity was avowed by the late Professor Frank Baron (Baron, 1979), whose proposed procedures for performing seismic investigations of suspension bridges. His recommendations went largely unheeded for 10 years until the 1989 Loma Preita earthquake damaged many bridges in northern California. Since 1989, seismic investigations of many major west-coast suspension bridges in the USA have been completed, with scopes of work similar to those in Baron's proposal. Some 57 long-span suspension bridges, varying in main span length from 126 m to 1299 m and in age from 19 to 153 years, have been identified as being in service in the USA. Most of the 57 lifeline suspension bridges were designed and built in previous technological periods, when the bridge engineering profession did not have adequate technical knowledge or resources and had a common belief that such bridges are not vulnerable to earthquakes. Many of these bridges are aged and have seismic vulnerabilities that stem from deterioration, inherent weaknesses in components and details, and inadequate dynamic-response characteristics. Seismic evaluation studies have now been completed on most of these bridges, and retrofitting methods have been finalised for many of these. The use of STUs (Figure 7.1) and energy-dissipation devices has been recommended.

With the advent of STUs and energy-dissipation devices, STUs have come to be used in cable-stayed and suspension bridges for protection against seismic events and windstorms. In suspension and cable-stayed bridges, the deck is often connected to the tower so that the large displacement of the deck in the longitudinal direction during a seismic event or a hurricane can be eliminated. However, in concrete-girder cable bridges, the tower is influenced by the forces of creep and shrinkage of the deck, as well as by fluctuation in temperature. In many cases, this results in a large increase in the structural capacity of the tower. The question, therefore, was how to connect the deck to the tower to allow for seismic events, and yet allow the movement of the deck at the tower to allow for shrinkage and creep and temperature variation over the life time of the structure.

Today, bridge engineers in seismic zones and heavy wind storm areas are using STUs on both steel and concrete cable-stayed and suspension bridges. In such cases, the towers are free from the deck during the normal operations of the bridge, but during a seismic or hurricane event the STUs between the towers and the deck lock up and create a fixed connection, so that the resulting forces

Figure 7.1 Typical location of STUs on a cable-stayed bridge




associated with the movement of the deck are resisted by both towers. This is a great benefit to the bridge engineer, and can realise a great cost saving in the design of the towers. In some cases, the bridge engineer may elect to retain one tower in the normal configuration and allow the STUs to function at the other tower, while at other times it may be deemed beneficial to have both towers connected by STUs. Suspension and cable-stayed bridges have long spans between the towers, and seismic phenomena can be of a different nature at each tower location. In such cases, it may be wise to install STUs on only one tower, as installing STUs on both towers may lead to more damage as the STUs on the two towers may not work in unison.

STUs and dampers are both used to avoid pounding problems due to earthquakes and wind gust between the stiffening trusses and towers of suspension bridges. The selection between the two devices depends mainly on the end conditions. In the case of the Golden Gate Suspension Bridge, because the stiffening trusses are connected to flexible towers, dampers were used to dissipate the energy. However, in the case of the suspended spans of the San Francisco–Oakland Bay Bridge, STUs were used between the stiffening trusses and the towers, because there are no tall towers and both abutments are on rock (ASCE, 1996), to which the pounding forces are transferred directly.

Today there are many cable-stayed bridges and suspension bridges throughout the world on which STUs have been installed to control displacement under the application of dynamic loads and for economy in tower design. Some case studies of cable-stayed and suspension bridges that have STUs installed are given below.

## 7.2. Storebaelt Suspension Bridge, Denmark – a new bridge

This road bridge across the eastern Storebaelt channel (Figure 7.2) has a total length of almost 6800 m, of which nearly 2700 m comprises a suspension bridge with a main span of 1624 m and two end spans of 535 m each (FIP Industriale, 2002). It is one of the longest suspension bridges in the world. The approach ramp spans are 62, 64 and 140 m, with 17 others of 193 m. It rests on two 250 m high pylons, with two anchor blocks and 19 pier shafts beneath the approach spans. The bridge is supported on free-sliding bearings at the anchor blocks, and transverse displacement is controlled by end-stroke elements in the bearings that prevent the displacement from exceeding $\pm 10$ mm. The longitudinal displacement is controlled by STUs, which limit longitudinal oscillations of the bridge induced by wind or normal service loads. The STUs also to reduce fatigue effects caused by both traffic and wind on the joints and suspension cables. The significant reduction in deck longitudinal oscillations has enabled the containment of the expansion width to the necessary minimum.

Four STUs, two at each anchor block, were installed. At about 10 m long they are rather long STUs (Figure 7.3), and provide a deck displacement of $\pm 1100$ mm. The STUs are designed for a 5000 kN longitudinal dynamic service load, with an ultimate capacity of 15 000 kN, and were supplied by FIP Industriale, Italy. A maximum rotation of $\pm 5°$ is provided at the hinges. Figure 7.3 shows one of the four STUs used on the bridge.

Various tests were carried out to simulate the extreme temperature variation of –35°C to + 60°C to which the devices are subjected, and to ensure adequate performance for the structure to maintain its function safely and efficiently for its design life of 100 years. The main difficulty encountered in load testing these devices was due to their length and the high load capacity, and a special rig had to be built for this load testing.

Figure 7.2 Storebaelt Suspension Bridge, Denmark

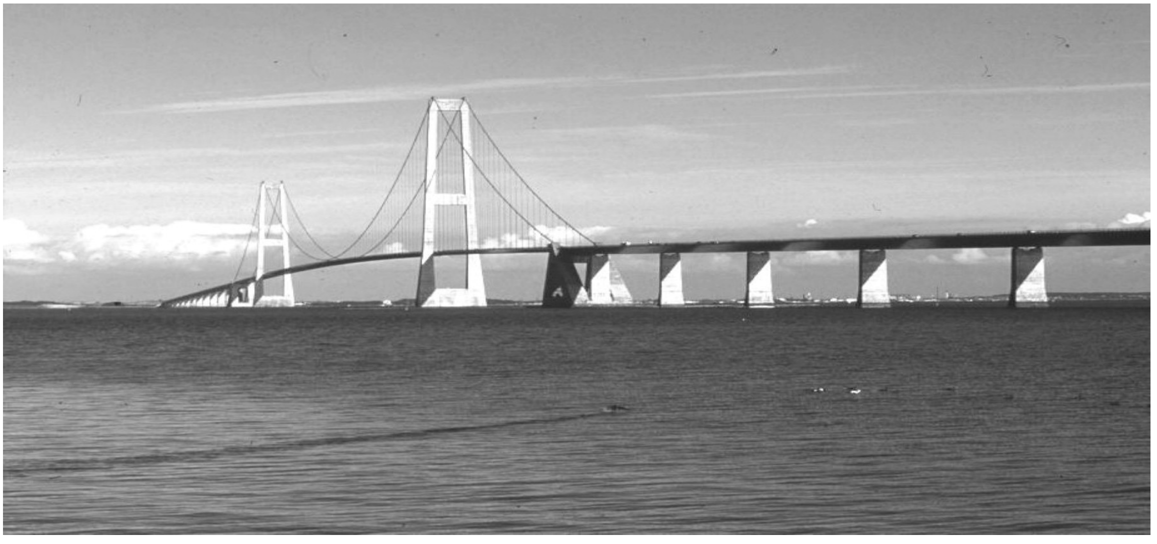


Figure 7.3 One of the 10 m long STUs used on the Storebaelt Suspension Bridge. Reproduced courtesy of FIP Industriale SpA

## 7.3. Sidney Lanier Bridge, Georgia, USA – a new bridge

The existing Sidney Lanier Bridge crossed the Brunswick River near Brunswick, Georgia, and had a lift span of only 250 ft, which was deemed far too small for the ships that needed to pass. In the past, ships had collided with the piers, which had then needed major emergency repairs. The new Sidney Lanier Bridge has replaced the existing structure to ensure navigational safety.

The new Sidney Lanier Bridge (Figure 7.4) is 7780 ft long. The main spans are cable-stayed structures, with two 625 ft long side spans and a 1250 ft long centre span. The bridge is designed to carry two traffic lanes in each direction. The navigational channel has a minimum horizontal clearance of 400 ft and a vertical clearance of 185 ft measured from the mean high water (MHW) level of +4.30 ft. Sand islands protect the main piers, and the minimum clearance between the sand islands is 1040 ft. Thus the vertical and the horizontal clearances can permit the passage of any ship in the world today.

The deck consists of an 11 in. thick concrete deck supported by two longitudinal edge girders and transverse floor beams. The parallel edge beams are 4 ft 9 in. wide by 5.0 ft deep. The transverse floor beams, spaced at 27 ft 6 in. in the side spans and 27 ft 2 in. in the main span, have a 1 ft 9 in. wide web, which is variable in depth. Including the slab, the depth of the floor beams is 5 ft at the edge girders and 7 ft at the centreline of the deck. The width of the floor beam is 81 ft between the outer edges, and 70 ft and 6 in. between the inside faces of the kerbs.

Two planes of stay cables with a fan configuration support the bridge. The stay cables consist of Grade 270, 0.6 in. diameter seven-wire strands placed inside high-density polyethylene ducts. The space in the ducts was filled with cement grout after final stressing. The stay cables are stressed at the top of the towers and dead-end anchored in the edge girders.

**Figure 7.4** The Sydney Lanier Bridge

The H-frame towers are 467 ft tall measured from the base. Each tower consists of two hollow rectangular columns connected together at two locations by the lower and upper cross-ties.

The girders are monolithically connected to the anchor piers at both ends of the main bridge. At these locations, the deck is about 168 ft above the water level. Each anchor pier has a hollow rectangular pier shaft, 24 ft by 10 ft. At the top of the pier shaft is a hammerhead, which cantilevers out transversely from the pier shaft. The top of the hammerhead is recessed to receive the precast girders of the approach span. There is an expansion joint between the main span and the approach span to allow relative movement of these two elements.

The deck girder is supported at the towers vertically by elastomeric bearings. There are two large bearings measuring 4 ft long by 3 ft 2 in. and 1 ft 6 in. thick at each tower, one under at each edge girder.

The bridge site is not in a highly seismic area, being designated a seismic Zone 1–2, compared with Zone 4 for Los Angeles and San Francisco. Cable-stayed bridges are usually very flexible and are not sensitive to earthquakes. In the case of the Sidney Lanier Bridge, the earthquake load was not significant and was not a controlling load in the design. However, because this is a major bridge structure, it was designed for an increased seismic requirement. Lateral reinforcement was increased to allow the formation of plastic hinges at the top and bottom of the tower legs between the tower cross-tie and the base. However, the bridge designers had to perform extensive studies to ensure that the new bridge would be safe during conditions of hurricane-force winds and low-level seismic transient events. An area of concern was longitudinal wind and/or seismic motion of the concrete deck of the main span. Under adverse conditions, the deck could impact the approach spans, causing damage to the costly expansion joints and the bridge structure itself. Calculations indicated that the application of STUs was a good solution to the problem, and four STUs of 250 ton force each provided the required restraint force. Longitudinal STUs (Figure 7.5) were used to connect the girder to the towers in order to activate the support of both towers under seismic loads. The STUs allow slow movement due to creep, shrinkage and temperature, but will lock up when they are subjected to sudden

Figure 7.5 The four 250 ton STUs used on the Sidney Lanier Bridge

movement, as in the case of an earthquake. The basic specifications of the STUs manufactured by Taylor Devices Inc., USA, were as follows (Taylor, n.d.).

- Rated capacity 250 ton.
- Available stroke 8.0 in.
- Thermal expansion 4 in. in 10 h with an applied force of less than 25 ton.
- Lock-up speed/displacement: sudden application of 250 ton of force in less than 0.5 s shall not cause more than 0.25 in. total deflection, and not more than an additional 0.125 in. deflection during a 5 s sustained load.

At the anchor piers all movements are restrained due to monolithic connections, and so no devices were required for restraint.

## 7.4. Second Severn Bridge, UK – a new cable-stayed bridge

The Second Severn Bridge over the Severn River estuary (Figure 7.6) was constructed during 1992–1996 as part of the M4, which links England and South Wales. The existing bridge built in 1966 had been subjected to a very heavy loading due to an increase in road traffic of 63%. The overall length of the new structure is 5126 m, of which the cable-stayed main span is 456 m. The cable-stayed deck is supported on pot bearings, while all the other spans are supported on reinforced rubber bearings.

STUs were required to restrain the deck movement due to wind and seismic loading. Eight STUs were installed at the piers of the cable-stayed bridge on each cross-beam, forming a linkage between the steel

Figure 7.6 The Second Severn Bridge. Reproduced courtesy of FIP Industriale SpA

deck superstructure and the reinforced-concrete substructure. The specifications for the STUs, supplied by FIP Industriale SpA of Italy, are as follows (FIP Industrial, n.d.).

- Serviceability limit state (SLS) load 1900 kN.
- Ultimate limit state (ULS) design load 2625 kN.
- Maximum force at high translational velocity = 1900 × 1.1 = 2090 kN at SLS and 2625 × 1.1 = 2887 kN at ULS.
- Maximum force at low translational velocity 190 kN.
- Required longitudinal stiffness 9.375 kN/mm (–5%, + 15%).

The STUs consisted of oil-filled piston/cylinder units surrounded by barrel-shaped elastomeric sleeves (the elastic element). At each end the STUs connect to the bridge deck and cross-beam by way of spherical bearings which give the devices the required rotational capability (Figures 7.7 to 7.9).

The standard STU provides a very stiff connection during impact loading, and an almost free connection under slowly applied loads. In suspension and cable-stayed bridges where the dynamics of the flexible deck–cable structure require a finite stiffness, this type of STU gives measurable, controlled restraint to thermal movements and also enables non-shock loadings to be transferred between structural elements. Under quasi-static service loads, the hydraulic fluid flows through the piston orifice with negligible reaction, and the rubber spring, by undergoing a shear deformation, allows relative movement between the substructure and the superstructure while providing the necessary restoring force. After removal of the load, the superstructure is brought back to its initial position. Under dynamic loading due to an earthquake the reaction from the restricted fluid flow locks up the unit and prevents movement of the superstructure.

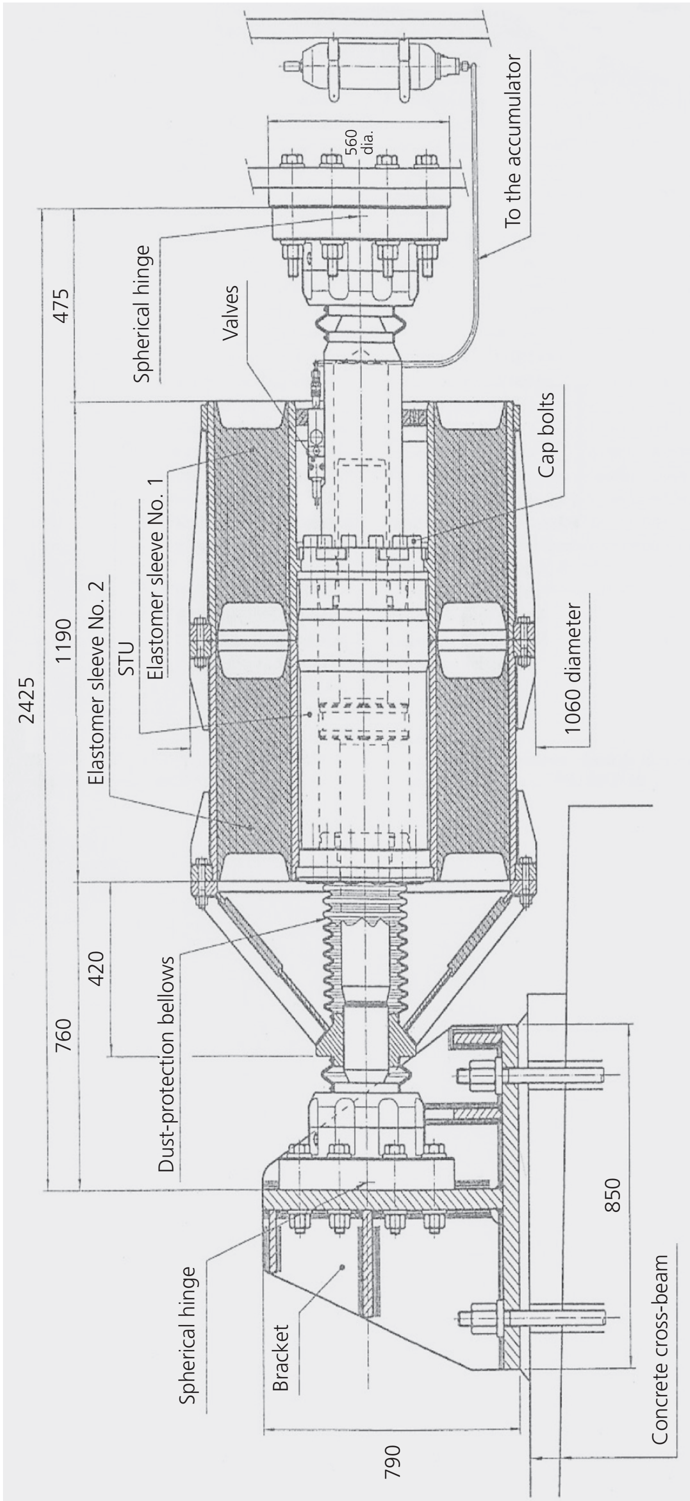



**Figure 7.7** Schematic diagram of the STUs used on the Second Severn Bridge

Figure 7.8 The STUs for use on the Second Severn Bridge

Figure 7.9 STUs in place on the Second Severn Bridge

## 7.5. Maysville Bridge, Ohio, USA – a new bridge

The Maysville cable-stayed bridge (Figure 7.10) carries the US 62/68 highway across the Ohio River and connects the towns of Maysville, Kentucky and Aberdeen about 60 miles south of Cincinnati. The bridge was designed in 1996 by American Consulting Engineers PLC, USA, and Michael Baker Jr Inc., USA; it was constructed in approximately 3 years and 3 months and opened to traffic in 2000 (Caroland and Suarez, 2000).

The bridge structure comprises a cable-stayed portion of length 2100 ft and a three-span approach bridge of length 311.9 ft. The cable-stayed portion is a five-span symmetrical structure with a main channel span of 1050 ft, two anchor spans of 125 ft each and two flanking spans of 400 ft each. The bridge comprises a precast concrete deck supported by two composite welded steel plate girders, with floor beams spaced at 16.67 ft. A single diaphragm is located between the edge girders at the crown point of the deck. The cable stays are composed of two planes in a semi-harp configuration, with 40 stays in each plane spaced at 50 ft intervals. The stays are anchored in the top section of the hollow tower legs to support the superstructure. There are two H-frame type towers that are 332 ft from the bottom of the footing to top of legs, supported by sixteen 72 in. diameter drilled shafts embedded 10 ft into solid rock. The towers are reinforced and post-tensioned with multi-strand tendons. The tower shafts were solid from the footing to the bottom strut and hollow to the top of the legs to facilitate anchoring of the stays in the walls.

The deck carries two 12 ft traffic lanes with two 12 ft shoulders, with a total width of 58.5 ft. It is made up of 9.75 in. thick precast concrete panels with cast-in-place splices. The deck slab is continuously post-tensioned longitudinally with 1.375 in. diameter bar tendons. The superstructure design was based on the balanced cantilever construction method.

**Figure 7.10** The Maysville Bridge

**Figure 7.11** One of the eight 1300 kN STUs installed at the two towers of the Maysville Bridge

The bridge is located in a low seismic zone having an acceleration of 0.06*g*, but the decision was made to fix both the towers against earthquake and wind forces. Eight 1300 kN load STUs with $\pm 305$ mm movement were installed at the two towers, connected at one end to the vertical web of the girder and at the other end over the bottom strut of the towers (Figure 7.11). The STUs allow both towers to share the wind and earthquake forces by locking the bearings, while allowing the daily movement of the deck due to temperature variation and creep. This approach led to a considerable cost saving in the design and construction of the towers.

## 7.6. Seohae Grand Bridge, South Korea – a new bridge

The Seohae Grand Bridge crosses the Asian bay about 65 km south of Seoul. It is about 9.4 km long and consists of several kilometres of concrete box girder spans and the main Seohae Grand Bridge (Figure 7.12), which is a 990 m long cable-stayed bridge. This is the longest bridge in South Korea, and its total width of 34 m comprises six traffic lanes together with a 3 m outside shoulder, a 1.2 m inside shoulder and a 0.8 m median barrier.

The main Seohae Grand Bridge consists of a 870 m long cable-stayed structure and two 60 m long end spans of simply supported composite girders (see Figure 7.12). The cable-stayed portion has three

Figure 7.12 The Seohae Bridge

spans: a 470 m centre span, which is the longest span in the South Korea, and two 200 m side spans. The centre span provides a 62 m high navigation above the bay.

The 60 m end spans of the bridge are connected to the cable-stayed spans by a hinge at piers 40 and 41 (Figure 7.13). In this way the rotations in both the 60 m spans and the cable-stayed spans are independent of one another. However, the hinge connection does not allow relative movement longitudinally. Large expansion joints provided at the transition between the end spans and the approach spans accommodate the total longitudinal movement of the entire main span.

The deck of the bridge consists of two longitudinal steel girders spaced 34 m apart, with steel floor beams, 4.10 m apart, running transversely between the edge girders. The roadway deck consists of precast concrete panels, 310 mm deep, which are supported on edge girders, and a steel stringer in the centre of the roadway and the floor beams, which are running perpendicular to the two edge girders.

The pylons of the Seohae Grand Bridge, which are 180 m in height from the base, consist of two hollow rectangular columns (Figure 7.14). The exterior dimensions vary from 15.7 m at the base to 6.6 m at the top of the tower in the direction of the roadway, and from 6 m below the deck to 4 m above the deck in the other direction (see Figure 7.12). Hammerhead shaped-anchor piers support the cable-stayed bridge end beam, and the 60 m end-span girders are supported by hinge bearings on the end floor beam.

Dual-plane stay cables support the bridge, fanning from the top of the pylons and anchored to the steel girders. There are 72 cables in each cable plane, ranging in size from 37 to 91 strands, each 15 mm in

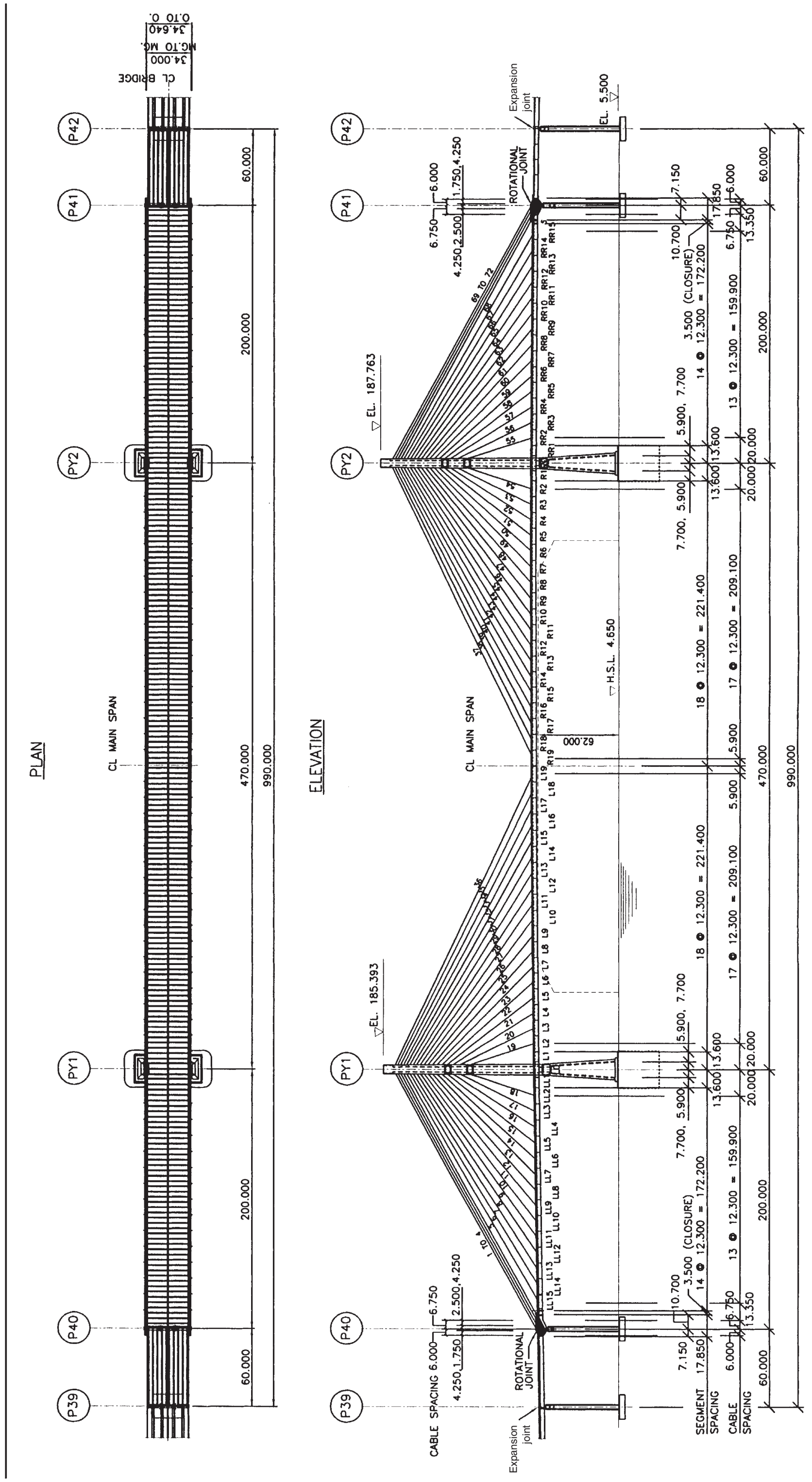
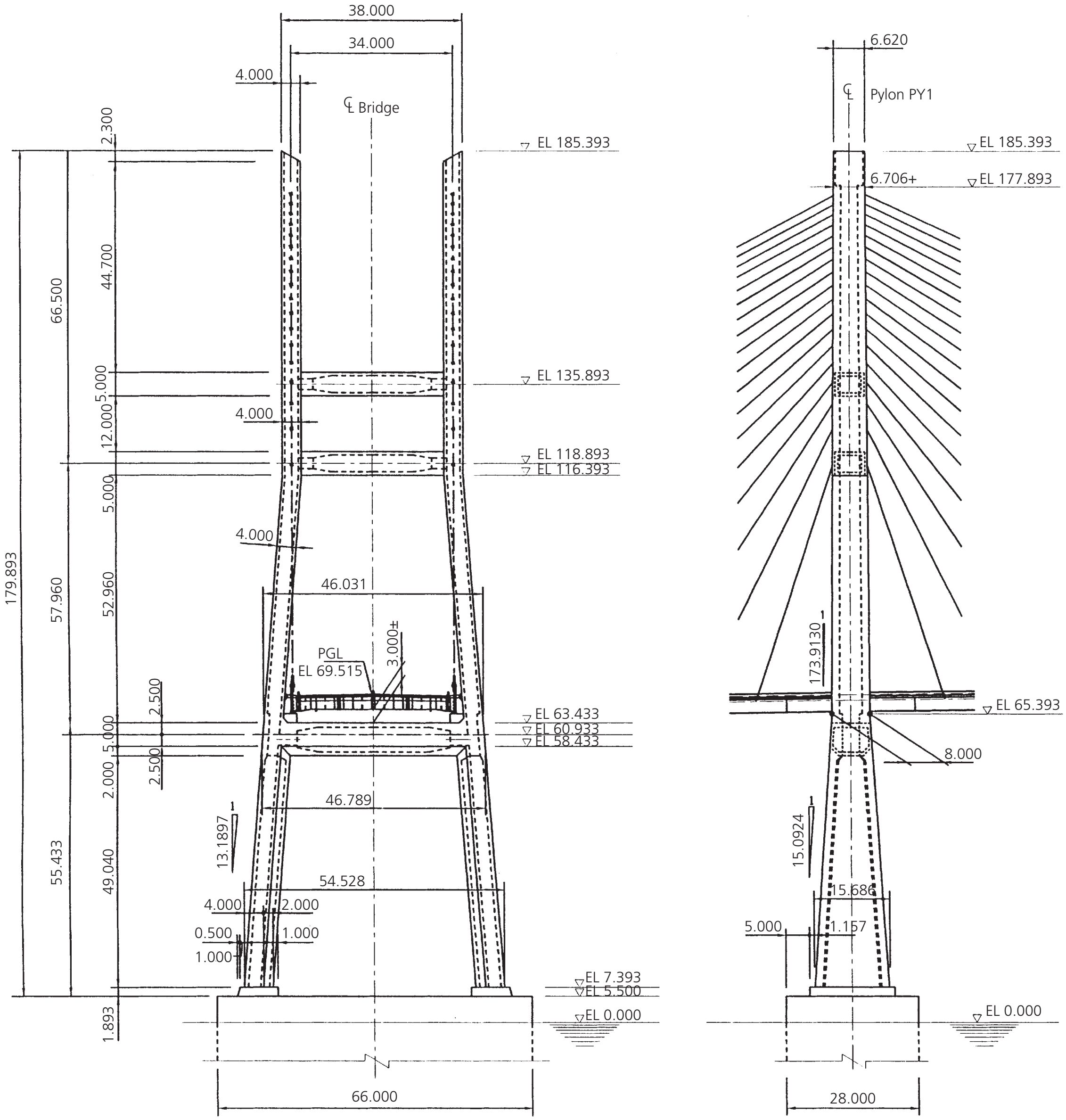



**Figure 7.13** Elevation of the Seohae Bridge. © T.Y. Lin International

**Figure 7.14** The pylons of the Seohae Bridge. © T.Y. Lin International




diameter. The strands are galvanised, covered with wax, individually sheathed and then placed inside a plastic pipe without grout.

Elastomeric bearings placed under the edge girders at the two pylons restrain the deck girder in the transverse direction, while lateral bumpers located at the edge of the edge girders further restrain the maximum relative transverse movement between the girders and the pylons. The bearings are sized to provide approximately the same stiffness as the adjacent cables, so the girder behaves more like a floating member. This allowed the design to do without a hard point support, which would have caused a very high bending moment in this area of the girder.

A primary design challenge on this cable-stayed bridge was its location in an area of high winds. To control the longitudinal displacement of the bridge under live loads, wind gust and other dynamic loading, while allowing movement due to creep, shrinkage and temperature variation, the girder is hinged at one pylon and is movable at the other. However, to activate the horizontal support of the latter pylon under such dynamic loads as aerodynamic motions and earthquakes, STUs are installed to connect it to the girder (Tang, 2001). The STUs allow the slow motions to take place but lock up under rapid motions. Accordingly, the structural system responds differently to static and dynamic loads. This arrangement distributes the dynamic forces between both the pylons, rather than to be resisted only by the pylon having the hinged connection, and also generates economy in the cost of the pylons.

## 7.7. Stonecutters Bridge, Hong Kong – a new cable-stayed bridge

The construction of this cable-stayed bridge began in May 2004 and the bridge was opened in 2009. It is one of the longest span cable-stayed bridges in the world. It has a clear span of 1018 m and back spans of 289 m each, giving a total length of 1596 m spanning across the Rambler Channel at the entrance to the busy Kwai Chung Container Port (Figure 7.15). The bridge forms a major part of Route 8 between Tsing Yi and Cheung Sha Wan.

The deck comprises two 18.4 m wide longitudinal girders separated by a 14.15 m wide central gap. The main span and the first 49 m of each back span are fabricated from steel, and the remaining 240 m section of each back span is constructed from post-tensioned concrete. Each half of the main span is supported by 28 pairs of stay cables, spaced at 18 m intervals, matched by 28 pairs of cables in the adjoining back span, spaced at 10 m intervals.

**Figure 7.15** Stonecutters Bridge spanning the Rambler Channel, Hong Kong

Figure 7.16 One of the pylons of Stonecutters Bridge during construction

The main feature on the twin three-lane highway is the 1018 m long cable-stayed span, which is punctuated by two gleaming, stainless-steel capped, tall concrete towers (Figure 7.16). Each main tower is approximately 298 m high and is of single-shaft, circular hollow section type, passing through the central gap in the deck. The towers are made of reinforced concrete up to the start of the cable-anchorage zone in the upper part, and thereafter the construction is of composite steel and reinforced concrete. An outer layer of stainless-steel reinforcement is incorporated in the tower concrete to increase its durability and thus achieve the 120-year design life.

In order to protect this strategic structure from seismic activity, wind actions and traffic braking loads, two sets of four 8000 kN STUs (Figure 7.17) are provided on each pylon. This dynamic protection system of STUs is installed along the longitudinal bridge axis, connecting the deck to the circular pylon (Figure 7.18).

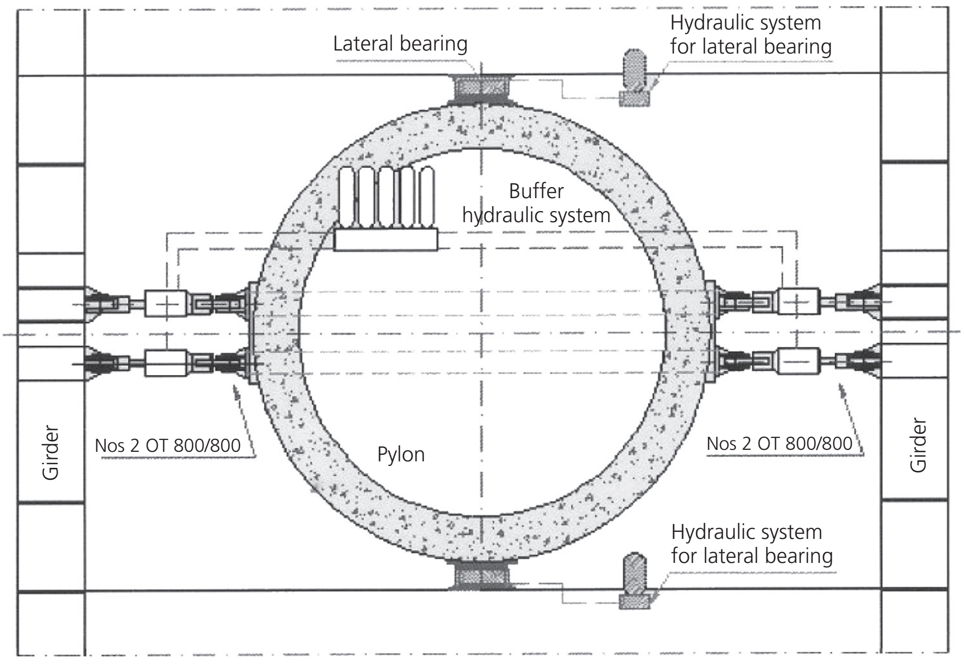
The bridge is designed to include devices also in the transverse direction. These lateral bearings are intended to control and mitigate the transverse movements of the main bridge girders. Hydraulically preloaded spherical bearings connect the steel deck girder to the main pylons, with a reaction that depends on the dynamics of the imposed loads.

Figure 7.17 The STUs used on Stonecutters Bridge. Reproduced courtesy of FIP Industriale SpA

Each dynamic protection system comprising STUs can accommodate thermal expansion and contraction as well as any other low-velocity displacements without appreciable reaction, while at the same time ensuring an almost rigid connection between the deck box girder and the pylon. Each group of STUs is linked by a hydraulic circuit with a flow-control valve. This arrangement guarantees uniform loading of all units, eliminating any torque effect induced by differential reaction between the individual STUs. The objective was to achieve a system that locks during dynamic load application, and hence particular attention was paid to the stiffness characteristics of the STUs. A well-designed STU behaves like a very stiff spring, locking within a short piston stroke, and so the compressibility of the fluid medium had to be carefully selected in order to minimise the stroke that was needed to develop the design reaction of the bridge.

Another important aspect was the behaviour of the STUs over a wide range of temperature variation. The specific design and the special oil used ensure consistent behaviour of the STUs over the design temperature range (−5°C to +70°C). The change in volume of the hydraulic fluid over this range had to be taken into account, and this was achieved by providing suitable accumulators with nitrogen-filled bags that can expand or contract in order to accommodate the variation in volume, and hence maintain the required pressure level. In order to maintain maximum control over the behaviour of the entire system, the flow control can be adjusted within a certain range. Any pipe damage would allow hydraulic oil to leak out and damage the structure, and so special check valves are incorporated in each STU. These valves are normally opened by a hydraulic-controlled pressure, which is established according to the basic pressure of the accumulator. When the valve closes due to a

Figure 7.18 An STU connected between the deck and a single pylon on Stonecutters Bridge




rapid reduction in pressure in the circuit, the hydraulic system of each STU is isolated and leakage thus avoided. In order to protect the structure and allow the girder to move during this piping damage phase, the chambers of the STUs in the group are connected by means of pressure-relief valves, which come into operation at a pressure of 60 MPa. The same valves are activated even during a dynamic event that generates loads creating a pressure of more than 60 MPa in the circuit.

A stiffness test was carried out in order to measure the compressibility of the fluid into the hydraulic cylinder loaded to 8000 kN with the valves closed. The STUs were designed to withstand this load with a minimum factor of safety of three. The hydraulic cylinder was also tested with a static pressure on both chambers simultaneously at 40 MPa for 24 h. A static loading test was carried out to determine the resistance of the system to the thermal movement. In addition, a dynamic loading test was carried out to verify the maximum reaction at high velocities. This test was conducted by applying a thrust velocity of 2 mm/s to check whether the predicted maximum reaction of 8000 kN was reached.

To ensure that the system is working properly, a monitoring system was installed comprising pressure, temperature and displacement transducers. The data obtained are sent to a data-transmission unit, then to a peripheral data monitoring unit, and finally to the master monitoring station, with relevant channel identifications. The acquisition system can visualise the data and send out a signal in real time if safety thresholds are exceeded.

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# Chapter 8
# STUs for buildings

## 8.1. Introduction

In addition to their use on bridges, STUs are also used on buildings, especially multi-bay car park structures and tall buildings. Car-parking structures are generally simple span multi-bay structures, and in many cases the shear capacity provided by the walls is insufficient to resist the seismic forces. STUs can provide additional rigidity when such structures are connected together, with the STUs placed at the expansion joints, as shown in Figures 8.1 and 8.2 for a multi-bay building in Udine, Italy.

When there is a wide-base building comprising sections of different heights, it is necessary to provide expansion joints to allow for the deflection due to temperature variation. In such cases, STUs can be placed between the expansion joints, to tie them together during a seismic event but allowing them to move together due to temperature effects. Figure 8.3 shows a wide-base building with STUs installed at the expansion joints, allowing expansion and contraction due to temperature variation under the service load conditions. In the wide-base building in Figure 8.4, there are no expansion joints, as the STUs would lock up, providing much greater shear area to resist dynamic loads.

Tall buildings located close to one another in a seismic or wind storm area are subjected to 'pounding' problems. In such buildings, having either a fixed-base or a base-isolated structure, pounding can be avoided by connecting the buildings with STUs at the top (Figure 8.5).

STUs have also been used for seismic protection of historic structures and monuments to provide the stiffness needed to withstand earthquake forces without inducing undesired forces under service conditions. The use of STUs in historic structures was first proposed in Italy for the dynamic connection of a new steel roof structure to the masonry walls of a church. To avoid the danger of percolation of silicon oil from the STUs onto the ornamental parts of such an historical structure, it was preferred to use silicon putty, as this medium has a higher viscosity than silicone oil.

Some case studies of building structures installed with STUs are given in the following sections.

## 8.2. Roof structure of Rome Stadium, Italy

STUs can be used to connect a large roof structure having many sections separated by expansion joints. Expansion joints are necessary in such a large roof structure to allow expansion and contraction due to temperature variations. Unfortunately, the presence of expansion joints reduces the rigidity of a roof, and hence its capacity to resist wind storm and seismic forces. When connected by STUs, such a roof structure can expand and contract without any interference, while holding the roof sections together as a single unit during an earthquake or wind storm event, and thereby making it structurally stronger to resist such dynamic forces.

**Figure 8.1** An STU at the expansion joint of a multi-bay building at Udine

**Figure 8.2** An STU installed in the multi-bay building at Udine shown in Figure 8.1

Figure 8.3 Wide-base building comprising sections with STUs installed at expansion joints, allowing temperature variations under service load conditions

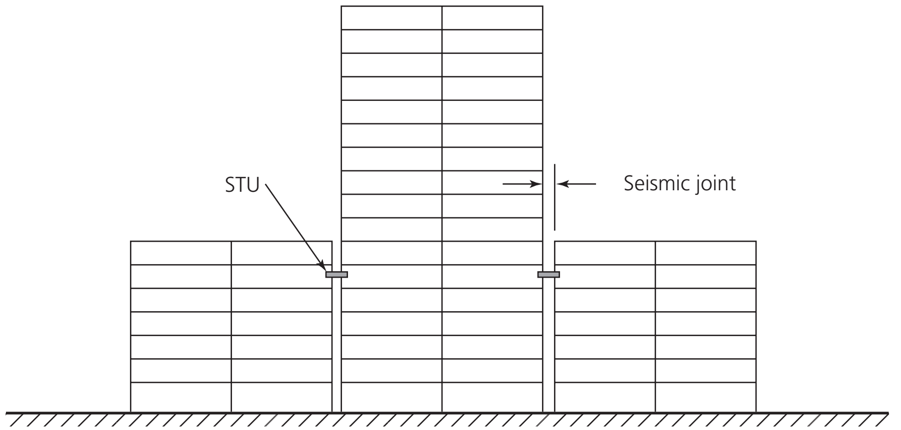



Figure 8.4 Wide-base building with no STUs or expansion joints. The building acts as a single tied structure under dynamic load conditions

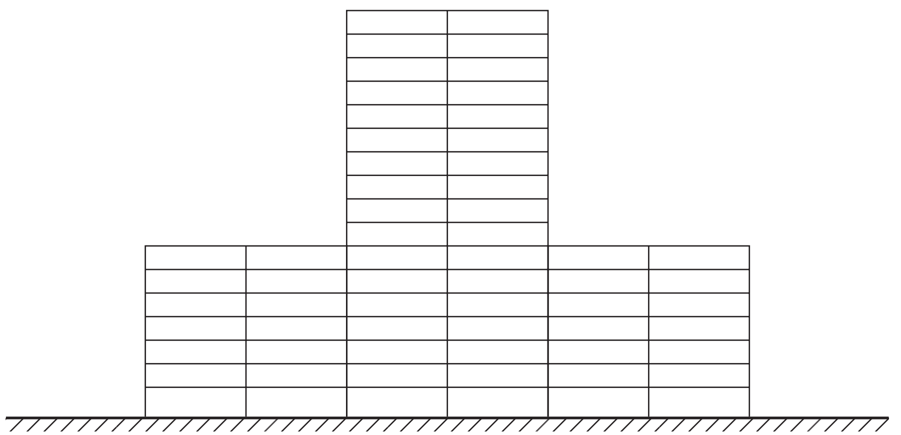


Figure 8.5 Buildings subjected to pounding problems

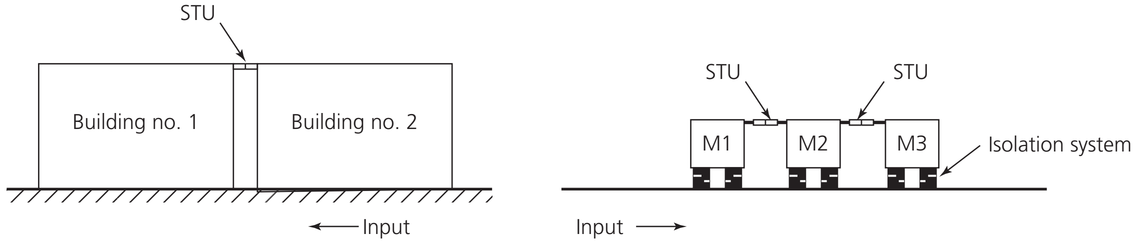


Figure 8.6 Roof structure of the Rome Stadium

Such a light steel roof has been constructed for the Rome Stadium (Figure 8.6). The steel roof needs to expand and contract due to temperature change, and to resist effectively the constant strong wind blowing over the city of Rome. STUs were installed at the expansion joints to allow the movement due to temperature variation, and at the same time to enable the whole roof structure resist wind storms by linking the roof sections separated by the expansion joints.

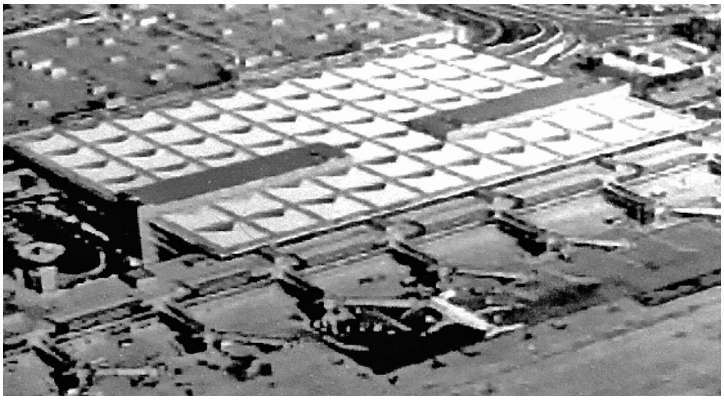
## 8.3. Retrofit of Ataturk International Airport terminal building, Istanbul, Turkey

On 19 August 1999, when it was nearing completion, the new Ataturk International Airport terminal building was struck by a devastating earthquake of intensity 7.4. The airport building is located some 25 km from the centre of Istanbul and about 70 km from the fault rupture plane. The new terminal building, approximately 240 m by 168 m in size, is a three-storey reinforced-concrete structure with a space-frame roof (Figure 8.7).

The lowest storey of the building houses mechanical and baggage-handling services, and the second and third storeys contain the arrivals and departures halls, respectively. Figure 8.8 shows a cross-section through the building, showing typical framing. The building is framed in reinforced concrete. Above the third floor, cantilever columns on 24 m centres support a three-dimensional steel space-frame roof, 800 ft long by 500 ft wide. The roof is provided with sleeved expansion joints to permit movement of the roof due to expansion and contraction as the temperature changes.

The terminal building was shaken by the earthquake and sustained some damage. The maximum horizontal ground acceleration recorded at the airport was approximately 0.1*g*, while the maximum

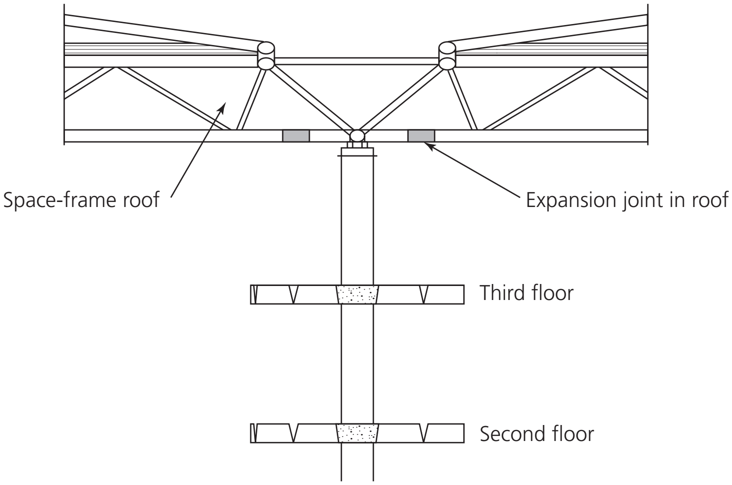
Figure 8.7 The Ataturk International Airport terminal building. © Michael Constantinou, Andrew S. Whittaker and Emmanuel Velivasakis, University at Buffalo



vertical acceleration was less than 0.5$g$. The following damage was identified by the engineers inspecting the building soon after the earthquake.

- Some spalling of the concrete cover.
- Buckling of the longitudinal rebar at the base of the third-storey columns.
- Loss of concrete at the underside of the roof truss–cantilever column connection.
- Slippage of the roof truss base plates atop the columns.
- Splitting cracks and spalled concrete in the beam–column joints at the third-floor level.

Figure 8.8 Cross-section of the Ataturk International Airport terminal building. © Michael Constantinou, Andrew S. Whittaker and Emmanuel Velivasakis, University at Buffalo



It was imperative that the terminal building be retrofitted for another large-magnitude earthquake with an epicentre closer to the airport, to ensure the safety of people inside (Constantinou *et al*., 2001). The third-storey columns that support the roof were the most vulnerable link in the existing building, and the objective of retrofitting was therefore to reduce earthquake forces on both the supporting columns and the roof.

The Multidisciplinary Center for Earthquake Engineering Research (MCEER) located at the University at Buffalo, The State University of New York, was part of a team engaged in the seismic upgrade of the airport building. It was decided to carry out a detailed analysis of the building structure to determine the earthquake damage and to establish the options available for the retrofit. Non-linear static and dynamic analyses were conducted using procedures set out by the US Federal Emergency Management Agency (FEMA, 1997) and included numerous contributions from the MCEER research team. Pushover analyses were performed using inelastic damage analysis of reinforced concrete (IDARC) procedures developed with MCEER support. Dynamic analyses of the roof – an isolated structure considering the inelastic frame behaviour – were carried out using computer software based on modifications of the 3D-BASIS software developed with MCEER support.

Several possible retrofit schemes were then investigated, including conventional methods and materials, and new innovative technologies. The conventional retrofit options considered included steel braced frames, special shear walls and special reinforced-concrete moment frames. All these conventional method options included

- new foundations under the new lateral force resisting components
- repair and reconstruction of the third-floor columns and roof trusses
- elimination of the expansion joints provided for the reinforced-concrete construction of the two storeys
- joining and jacketing the peripheral columns
- jacketing of the interior third-storey columns.

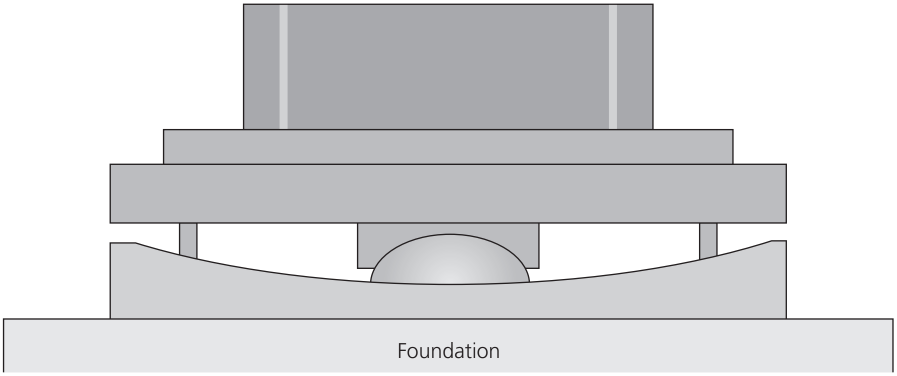
The addition of a new lateral force resisting system was not practical, as it was not feasible to install new walls and braced frames in or below the first storey.

Innovative seismic isolation and supplemental damping were also studied as a retrofit option. However, the addition of supplemental damping would have required the addition of braces or new wall panels in the lower two storeys of the building, and so a detailed retrofit design for this option was not prepared. Two isolation schemes were considered: base isolation of the complete building and isolation of the roof structure. Building isolation was the best choice, but was rejected by the owner as it required the demolition and reconstruction of the ground floor, and removal and reinstallation of the mechanical and baggage handling systems located in the first storey. The second isolation scheme was considered and studied in detail, and was finally selected for the retrofit of the building by the owner.

This second retrofit scheme involved

- isolation of the roof trusses to reduce the demand on the third-storey columns and the framing at the lower levels
- introduction of STUs to the roof trusses to lock up the space-frame trusses during an earthquake event
- strengthening of the reinforced-concrete construction.

Figure 8.9 A friction pendulum bearing




The third-storey columns could not be increased in size because of architectural considerations, and so the existing strength of these columns as cantilever components dictated the inertial force that could be developed at the roof level. To this end, friction pendulum isolation bearings (Figure 8.9) were installed at the top of the third-storey columns to support the roof structures.

A friction pendulum bearing comprises two horizontal steel plates that can slide over each other. Because of their shape and an additional articulated slider, they allow structural components to swing gently from side to side, like a pendulum. Such bearings can isolate light components and structures. A friction pendulum bearing providing an isolated period of 3.0 s, a design friction coefficient of 0.09 and a displacement capacity of 260 mm was provided over each of the third-storey columns carrying the roof trusses (Figure 8.10). The strengthening of the reinforced-concrete construction included, among other things, increasing the strength of the second- and third-storey columns by steel jacketing. STUs, supplied by Taylor Devices Inc. were installed between the expansion joints of the roof segments, tying together the segments of the 800 ft long roof, which is supported on the friction pendulum isolation bearings. This gave the roof the ability to swing by as much as 300 mm with respect to the columns during an earthquake event, thus protecting the columns.

The performance of the retrofit design was evaluated using non-linear dynamic and static analysis, and the predicted damage was found to be completely consistent with the observed damage to the building. The entire evaluation and subsequent retrofit was completed in about 12 weeks, which was an outstanding achievement.

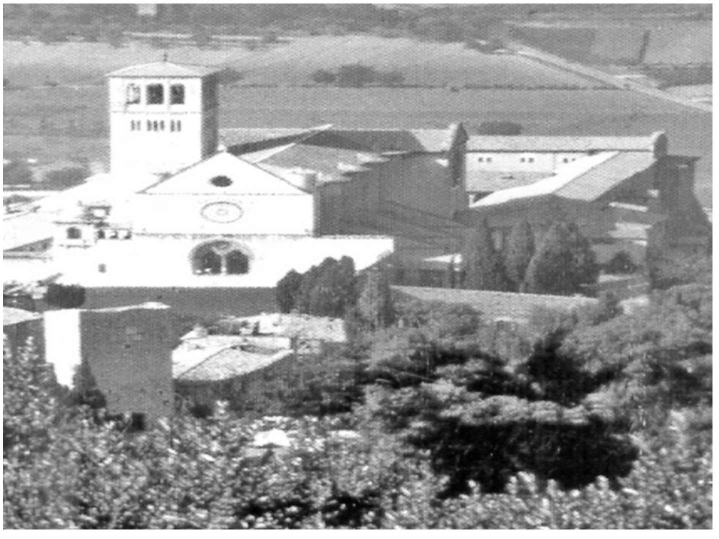
## 8.4. The upper basilica of San Francesco, Assisi, Italy

The Basilica of San Francesco (Figure 8.11) comprises two distinct basilicas, the upper and the lower, one on top of each other. During the earthquake that struck the Italian central region of Umbria in September 1997, the Basilica of San Francesco in Assisi suffered heavy damage, with partial collapse of the frescoed vault and detachment of numerous stones from the tympanum of the left transept due to pounding of the roof (Croci, 1998).

Vertical cracks near the centre of each bay of the side walls of the upper basilica had opened and been enlarged many times by the past earthquakes, and these were found to have been widened by the 1997 earthquake. Each crack starts at floor level and continues up to the level of the tall window base.

Figure 8.10 A typical third-storey column carrying the roof trusses over a friction pendulum bearing. © Michael Constantinou, Andrew S. Whittaker and Emmanuel Velivasakis, University at Buffalo

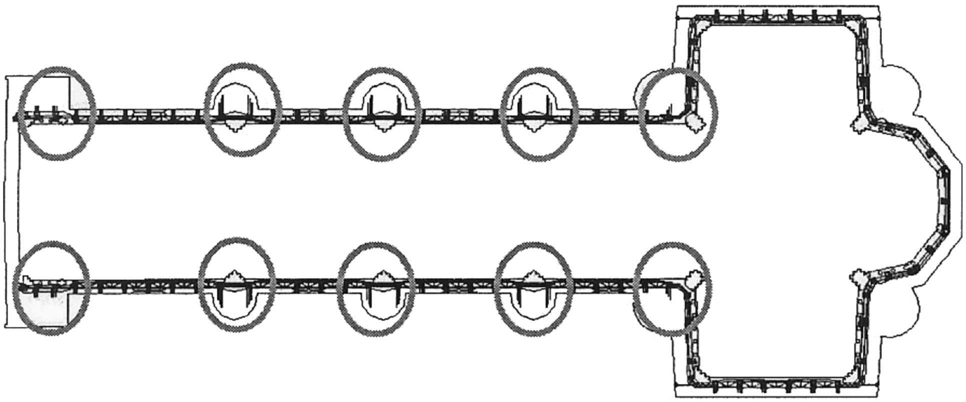
Figure 8.11 The Basilica of San Francesco



In many cases, the window cornices themselves were cracked, and these cracks were visible on both the inside and outside of the wall. In each bay of the lower basilica, there is a large and tall opening for access to the lateral chapels and, as the top of the windows of the upper basilica are close to roof level, the nave of the basilica as a whole is divided into vertical portions weakly connected to one another at the middle of each bay. The cracks in the side walls below window level in the upper basilica affected the Giotto frescoes (Figure 8.12).

**Figure 8.12** The damaged Giotto frescoes in the upper basilica of San Francesco

Figure 8.13 The horizontal stainless steel girder along the entire length of the upper basilica of San Francesco. Reproduced courtesy of FIP Industriale SpA



A traditional technique to increase the strength of masonry walls or to improve the connection between different structural elements is the use of reinforcing injections. However, the technique was ruled out in this case because of the presence of Giotto's frescos. In such cases, additional stiffening, such as a steel beam, can be introduced. However, this solution is not without problems, as it could have obstructed the natural breathing of the masonry walls of the basilica. It was therefore decided to use a chain of trussed steel beams placed over the internal cornice and located at an intermediate height just above the Giotto's frescoes and under the windows. The transverse stiffness of one bay of the masonry side wall between two external buttresses was increased by placing a steel beam, which was connected to the beam in the adjacent bay by means of STUs, which allowed for the natural breathing of the masonry wall. The steel girder was placed along the entire length of the basilica from the façade to the apse, and passing through the narrow chinks behind the pillars (Figure 8.13).

The section profile of the girder was shaped in order not to be seen by people standing in the nave. The steel girder was fixed locally at many points along the walls. Thus, in the nave, the steel girder was divided into portions corresponding to the bays, with the portions being connected to one another, behind the pillars, by a couple of STUs (Figure 8.14). These STUs allow movement due to temperature variation, etc. under normal service conditions, and a global stiff linking during seismic events.

Tympanum walls were directly supporting the main reinforced beams of the roof structure, and so there was no possibility of controlling the forces exchanged between the tympanum and the roof or the beams pounding against the walls during a seismic event. This had caused the south tympanum masonry wall of the transept of the upper basilica to partially collapse in the 1997 earthquake. Traditional connections by steel bars between the basilica roof and the tympanum walls would be too stiff, and dangerous for the tympanum itself. Therefore, it was decided to use shape-memory alloy devices to connect the roof to the walls of the two tympana of the transepts (Figures 8.15 and 8.16). This was the world's first application of shape-memory alloys in a building to improve its earthquake resistance.

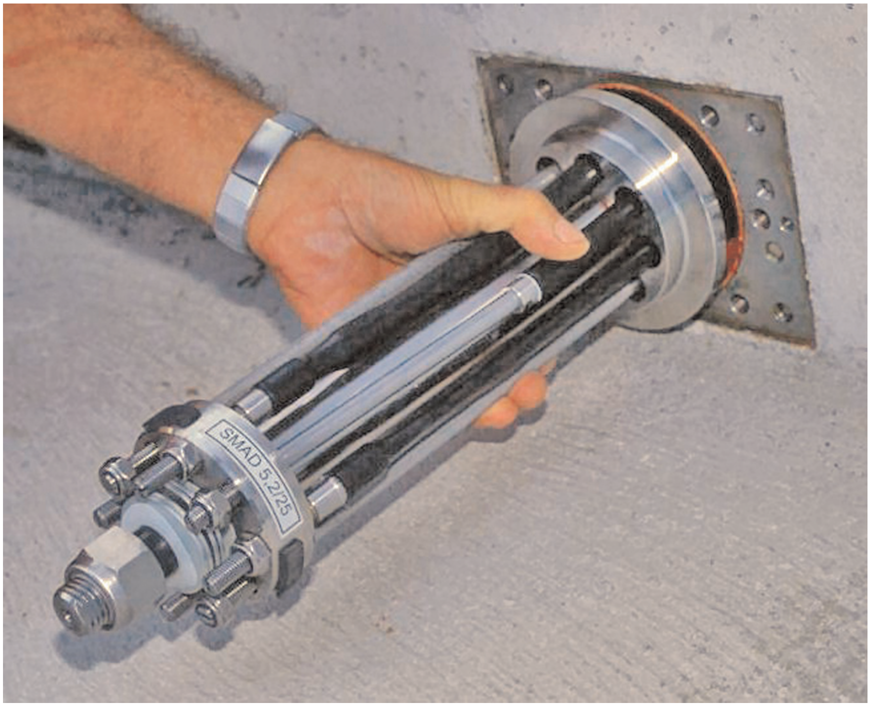
Shape-memory alloys are metals endowed with very unusual thermo-mechanical properties. They have the ability to recover from large deformations (greater than ten times that of conventional metals) during loading–unloading cycles. Loading and unloading paths generate an hysteresis loop, which

Figure 8.14 Two STUs connecting the steel girders in the upper basilica of San Francesco. Reproduced courtesy of FIP Industriale SpA


results in energy dissipation. The superelastic behaviour of shape-memory alloys makes them particularly suitable for creating force-limiting devices. They work as very flexible ties, capable of allowing controlled displacement, and of limiting forces to below a pre-established value. Shape-memory alloy devices use the alloy in wire form, this being suitably connected such that it always works under tension, giving the device a symmetric behaviour regardless of the direction of the displacement. The devices behave differently under different intensities of external action.

- The device remains stiff, like a traditional steel connection, and does not displace significantly under slow horizontal action due to wind, low intensity earthquakes, etc.
- Under higher horizontal forces, such as those due to a high intensity earthquake, the stiffness of the device reduces, and controlled displacement occurs. This allows the masonry to dissipate part of the energy transmitted by the earthquake.
- For forces higher than the design earthquake, the stiffness of the device increases in order to prevent excessive displacements, thereby avoiding instability of the structure.

Figure 8.15 A shape-memory alloy device used in the upper basilica of San Francesco. Reproduced courtesy of FIP Industriale SpA




Significant improvement in the seismic response of masonry façade walls has been observed when the walls are connected to the structure by shape-memory devices rather than steel bars (Castellano and Martelli, 2000). The shape-memory alloy that has the best performance is nickel–titanium (NiTi) alloy, which also has a very high resistance to corrosion. High durability of shape-memory alloy devices has been observed for devices using NiTi wires with stainless steel.

In order to modify the roof–tympanum interaction, the roof structure was disconnected from the tympanum wall and a new concrete truss built to support the roof and bring the roof vertical loads directly onto the transept lateral walls. At the same time, the collapsed portion of the left tympanum was rebuilt, and the deflection of both the transept tympana was removed. Shape-memory alloys spaced 50 cm apart were used to connect the new concrete walls to the tympanum walls in such a way as to better distribute the exchanged seismic forces. Three different groups of devices were installed in order to allow for the different properties required with increasing distance from the lateral walls of the transept to the roof top. The upper basilica was reopened on 28 November 1979 after the retrofitting works were completed.

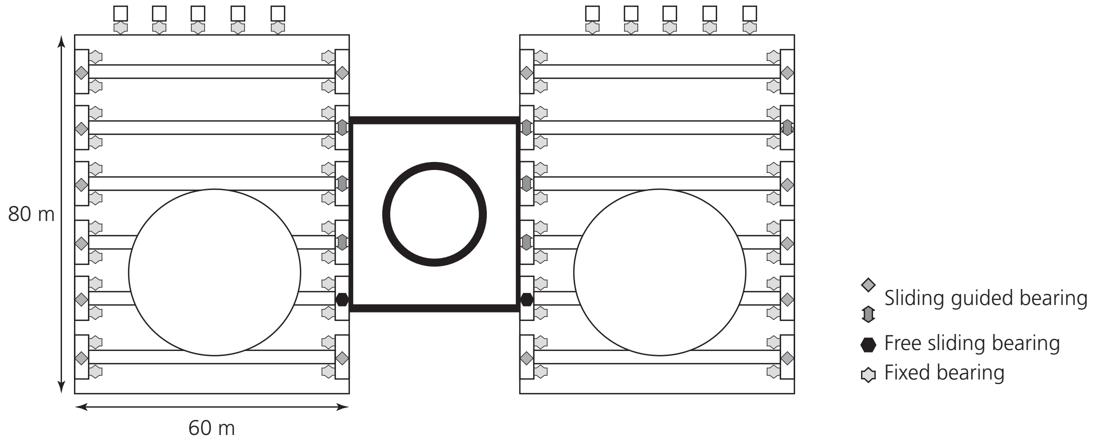
## 8.5. Official reception building, Riyadh, Kingdom of Saudi Arabia

The top slab of the two halls of the official reception building in Riyadh (dimensions 60 m × 80 m) is supported by 12 prestressed concrete 60 m long U-beams. The beams are connected to the support columns (3 m × 1.8 m × 25 m) by sliding pot bearings in order to allow for thermal deformation. The seismic connection is realised by means of STUs (Transpec SHT hydraulic connectors supplied

Figure 8.16 Shape-memory alloy devices connecting the roof of the upper basilica of San Francesco. Reproduced courtesy of FIP Industriale SpA

by Freyssinet, France). The building was also designed to withstand the effect of an explosion of a bomb along the exterior wall. In order to withstand the blast of the explosion, the wall is connected to the top slab by STUs. Figure 8.17 shows the layout of the building and the location of the STUs and bearings.

Figure 8.17 Plan of the official reception building, Riyadh, showing the location of the STUs. © Freyssinet



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# Chapter 9
# The snubber – a special type of STU for nuclear power plants and pipelines

## 9.1. Introduction

In the early 1950s in the USA, the National Advisory Committee for Aeronautics (NACA) developed STUs, and in the 1960s a similar technology to STUs, known generally as a 'snubber', was developed by various industrial hydraulics manufacturers and used on nuclear power plants and pipelines. These early snubbers, manufactured from a miscellaneous variety of hydraulic parts and pipe fittings, generally comprised a hydraulic cylinder, an accumulator and numerous spring-loaded flow control and check valves. However, they performed very poorly in service, largely due to an overcomplex design and the poor radiation resistance of the gaskets and the seals. Generally, in the nuclear industry snubbers are divided into small-bore hydraulic snubbers (SBHSs), which have a rated load capacity of less than 50 kilopounds (kip), and large-bore hydraulic snubbers (LBHSs), which have a rated load capacity greater than 50 kip.

Prior to the 1980s, the reliability of LBHSs, which are used primarily to restrain large components such as steam generators and reactor coolant pumps during seismic events, was a safety concern in nuclear power plants. Initially, there were no machines capable of testing LBHSs for functionality, so they were exempted from surveillance testing prior to 1980. When LBHSs were later tested, numerous deficiencies were found, many of which would render the LBHSs inoperable. Relatively high failure rates of LBHSs were common during the early 1980s, with the largest number of reported failures being due to seal failure and wear-related problems (Nitzel *et al*., 1992).

The safety of snubbers is of great safety significance for nuclear power plants. In 1992, the Nuclear Regulatory Commission (NRC) of the USA developed Generic Issue 113 (GI-113), *Dynamic Qualification and Testing of Large Bore Hydraulic Snubbers*, to address this issue of adequacy, with the objective of evaluating the reliability of this type of device in operating commercial nuclear power plants. The NRC contracted the Idaho National Engineering Laboratory (INEL) to carry out the research plan. The NRC found that there were 12 companies manufacturing LBHSs but only five companies supplying them. The research project that addressed GI-113 was subdivided into several tasks and subtasks that dealt with the various facets of the generic issue, such as LBHS design, operating experience, qualification testing, etc. Fifteen potential improvements in LBHS reliability were identified, covering the areas of design, environmental factors (including dynamic qualification), functional testing, visual inspection and human factors.

LBHSs are used to support steam generators, reactor coolant pumps and large piping systems (main steam and feed water). SBHSs are used for smaller equipment in refineries and nuclear power plants. Hydraulic snubbers are generally preferred in areas of higher vibration, such as in feed-water systems.

Although they are not designed to be vibration dampers, they are believed to wear less than mechanical snubbers under a low amplitude, high-cycle vibration environment.

The snubber is a special type of STU and its design is governed by radiation and the much higher temperatures present in nuclear power plants compared with those experienced by STUs used on bridges and buildings. In addition, snubbers are exposed to the operational vibrations of equipment. High temperatures can break down the internal fluid medium, causing the oil and entrained water to separate, and can also degrade the seals of the snubbers. The high cumulative radiation can cause the fluid to gel and embrittle the seals. Therefore, the seal and the oil generally used as the internal medium for snubbers are not as long lasting as those used for STUs, and they need to be replaced, on average, every 20 years. Unlike STUs, but like viscous dampers, snubbers require a reservoir or an accumulator to deal with changes in hydraulic fluid volume caused by temperature changes. While the STU is a compact device and is relatively maintenance free, the snubber, with its various valves and the severe operating conditions in power plant, must be inspected regularly, with a visual inspection at least once a year and functional tests at least every 12 years by means of on-site mobile test units. The internal medium of STUs can be either silicone oil or silicone putty, while only oil can be used in snubbers due to the high temperature and radiation environment.

## 9.2. General description

A hydraulic snubber consists of a cylinder containing fluid, a piston internal to the cylinder, control valves and a fluid reservoir. One end of the cylinder through the piston rod is connected to the equipment to be protected and the other end is connected to a fixed object, such as a wall. A schematic diagram of a snubber is shown in Figure 9.1 (Nitzel *et al.*, 1992).

**Figure 9.1** Schematic diagram of a snubber

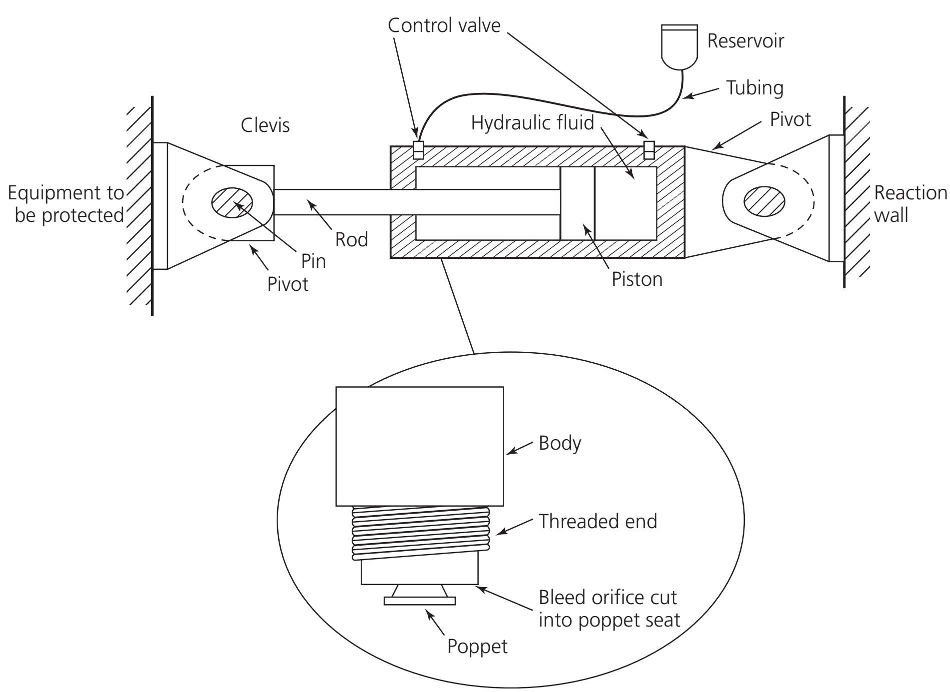
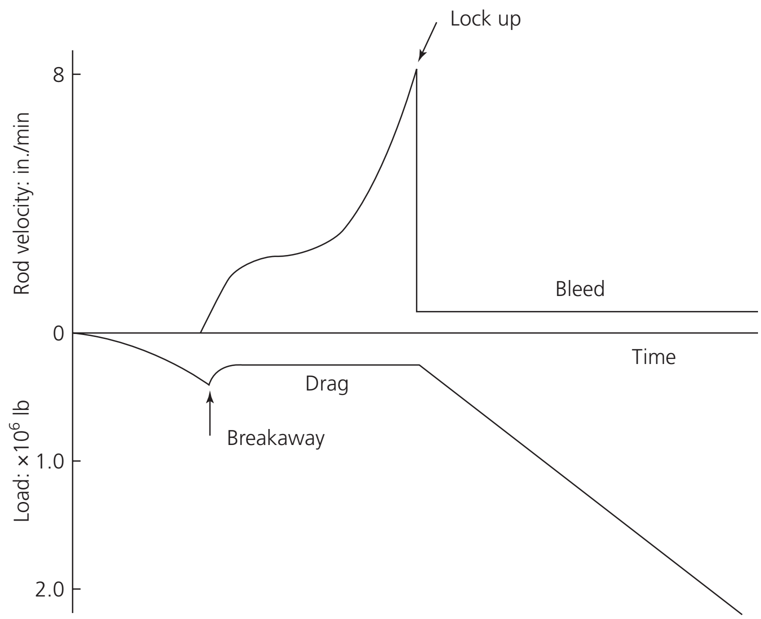


Figure 9.2 The force and velocity characteristic of snubbers



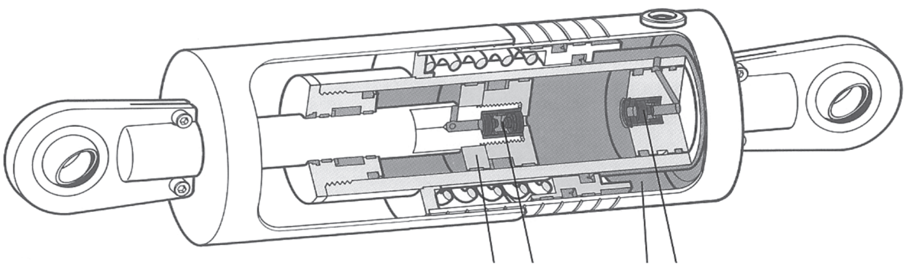

Under normal operating conditions, thermal expansion causes the equipment to move relative to the reaction wall at a comparatively low velocity, and the piston pushes the hydraulic fluid through the control valve to the other end of the cylinder with little resistance. However, during a dynamic event such as an earthquake or a pipe breaking, the piston velocity is considerably higher. Under these conditions the piston movement causes the pressure behind the cylinder to build up, forcing the control valve to close. The closed control valve blocks the flow, causing the hydraulic snubber to lock up and thus resist the dynamic motion. A small bleed orifice is supplied that allows the hydraulic snubber to move slowly after locking. The rate of movement is called the 'bleed velocity' or 'bleed rate'. The bleed orifice also allows the pressure acting on the control valve to return to normal after the dynamic event has ceased, thus unlocking the control valve.

Figure 9.2 shows the characteristic applied load and velocity versus time relationship for a hydraulic snubber during a dynamic event. When the force is initially applied during a dynamic event, the hydraulic snubber does not move until a threshold force caused by the internal resistance of the snubber, termed the 'breakaway force', is overcome. As the movement continues, the snubber moves at increasing velocity. The force that is required to keep the snubber in motion is called the 'drag force'. The snubber will lock up when the velocity reaches the lock-up velocity, and it will then move at the bleed velocity. The drag and breakaway forces are resisted by the piping or the equipment connected to the snubber, but these forces are relatively low compared with the force generated due to lock up of the snubber.

## 9.3. Types of snubber

There are a few snubber manufacturers, each of which has its proprietary seal, internal fluid, and valve and reservoir arrangement. Figures 9.3 to 9.6 show snubbers manufactured by Lisega Inc., Germany.

Figure 9.3 A small-bore hydraulic snubber (Lisega Inc., Germany)




### 9.3.1. Small-bore hydraulic snubber (SBHS)

Lisega's type 30 snubber has a load capacity ranging from 0.675 kip to 123.5 kip and can be considered as an SBHS. The function of an SBHS is controlled by the main control valve (B), axially mounted within the hydraulic piston (A), as shown in Figure 9.3.

During the piston movement (<2 mm/s) the valve is kept open by spring pressure, and hydraulic fluid flows from one side of the piston to the other. During rapid piston movement (approximately above a velocity of 2 mm/s), the resulting fluid flow pressure on the valve plate closes the main valve, and the flow of the hydraulic fluid is stopped and movement is blocked. For movement in the compressive direction, the compensating valve (D) closes almost synchronously with the main valve. If the pressure on the closed valve subsides, for example, through reversal of the direction of the movement, the main control valve opens automatically when the fluid force falls short of the spring force.

A bypass system is provided to prevent the valves from jamming in the blocking position. This allows limited piston movement under continuing load, and ensures safe opening of the valves by rapid equalisation of pressure in the two cylinder chambers. The compensating valve works synchronously with the main valve (B) in the same way.

A coaxially mounted reservoir (C) is provided for volume compensation due both to variable piston-rod positions and to changes in the volume of the hydraulic fluid volume arising from temperature changes. The link between the reservoir and main cylinder is regulated by the compensating valve (D).

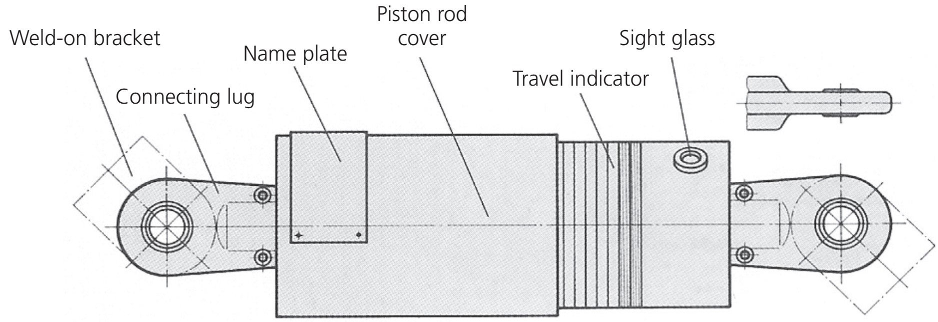
The fluid level of the reservoir is indicated by the position of the reservoir piston. A sight glass is provided (Figure 9.4) to check the minimum fluid level in the SBHS.

### 9.3.2. Large-bore hydraulic snubber (LBHS)

The function of an LBHS (Figure 9.5) is the same as that of an SBHS. However, the dimensions of an LBHS require a different arrangement of the reservoir (C), and the valve assembly also differs. The valves (B) themselves operate similarly to those in the SBHS shown in Figure 9.3 – the circulation of the fluid is blocked by the closure of the corresponding valve in each direction of movement. This happens whenever a flow speed limit is exceeded. However, no special compensating valve is required, as the valves are linked directly to the reservoir.

The control valve system in an LBHS is designed such that it can be removed while the snubber is in place, to facilitate routine maintenance.

Figure 9.4 External detail of the small-bore hydraulic snubber shown in Figure 9.3 (Lisega Inc., Germany)




To check the fluid level in the reservoir, the LBHS has a marked indicator rod attached to the base of the external reservoir (Figure 9.6).

LBHSs are fitted with exchangeable valves for on-site testing (Figure 9.7).

In both types of snubber, the end of the piston rod is threaded to accept a paddle end for attachment to a pipe clamp or structural bracket. The back end of the cylinder has a threaded stud to accept a second paddle end, or an extension piece for attachment to a pipe clamp or structural bracket. Each paddle includes a spherical bearing to allow angular movement and to prevent the creation of moments between the pipe and the structure. The connection to the structure is formed by special brackets supplied by the manufacturer. The piston position of the snubber can be viewed from all sides by checking the scale rings on the body of the cylinder. The sturdy stainless-steel shroud connected to the piston rod protects the device from mechanical damage, dirt and heat, and also serves as an indicator.

## 9.4. Design specification and load testing

Present day snubbers are precision made, safety-related components designed in accordance with the specification developed by the American Society of Mechanical Engineers (ASME) Boiler and

Figure 9.5 A large-bore hydraulic snubber (Lisega Inc., Germany)

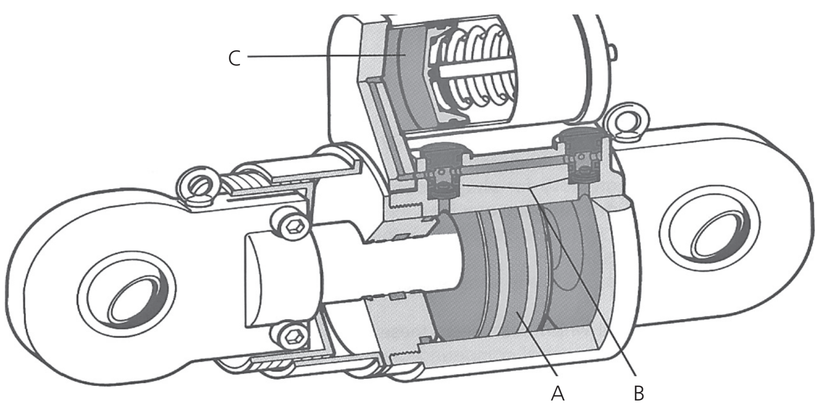
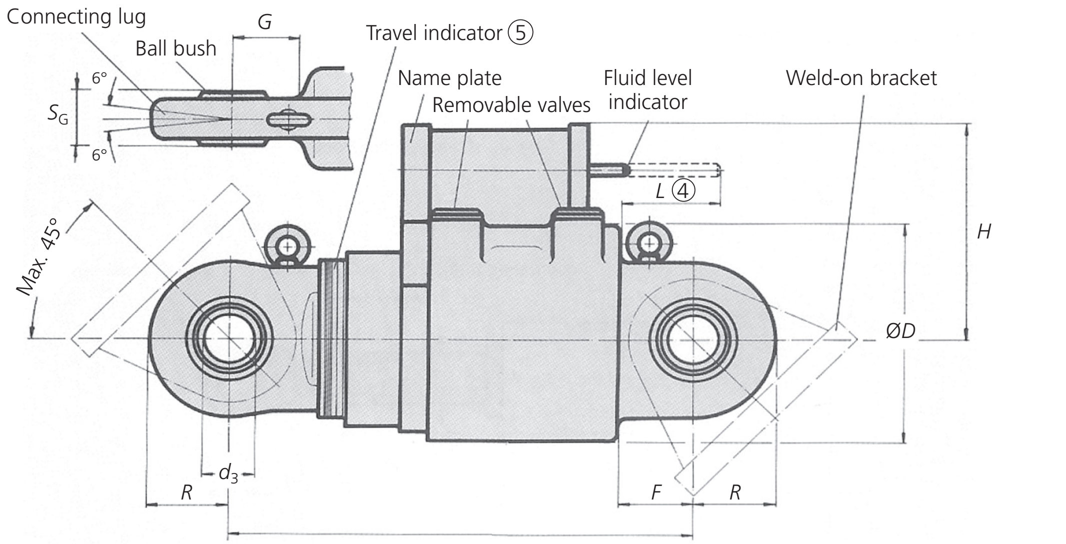


**Figure 9.6** External detail of the large-bore hydraulic snubber shown in Figure 9.5 (Lisega Inc., Germany)




Pressure Vessel Code. Snubbers offer protection to the piping system and other components in nuclear power plants against dynamic overloading caused by unplanned load events. As such events are unpredictable, the complete functional safety of the snubbers must be guaranteed at all times. The particularly stringent requirements within the nuclear industry demand flawless proof concerning the functional parameters of snubbers. Therefore, testing of snubbers, both before and after installation, is of paramount importance.

The ASME recommends three tests to be done on snubbers to check the following pre-service and in-service characteristics (IMF 5300 and IMF 5400).

- During low-velocity displacements the specified maximum drag or free movement force will initiate motion of the snubber rod in both tension and compression.

**Figure 9.7** The exchangeable valve facility on a large-bore hydraulic snubber

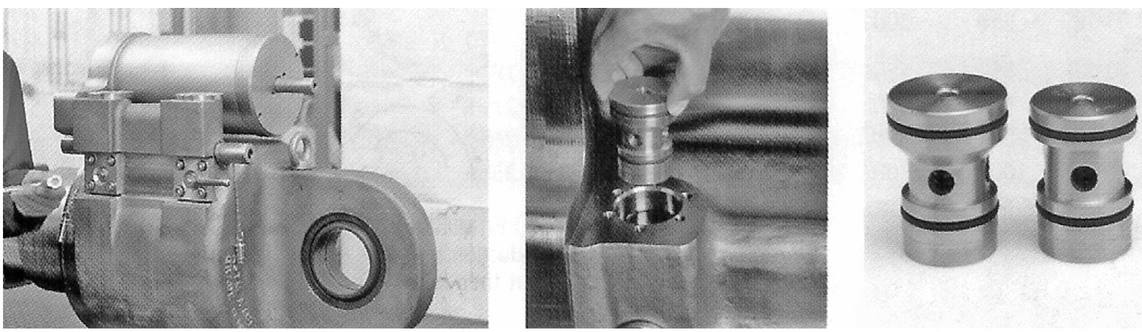
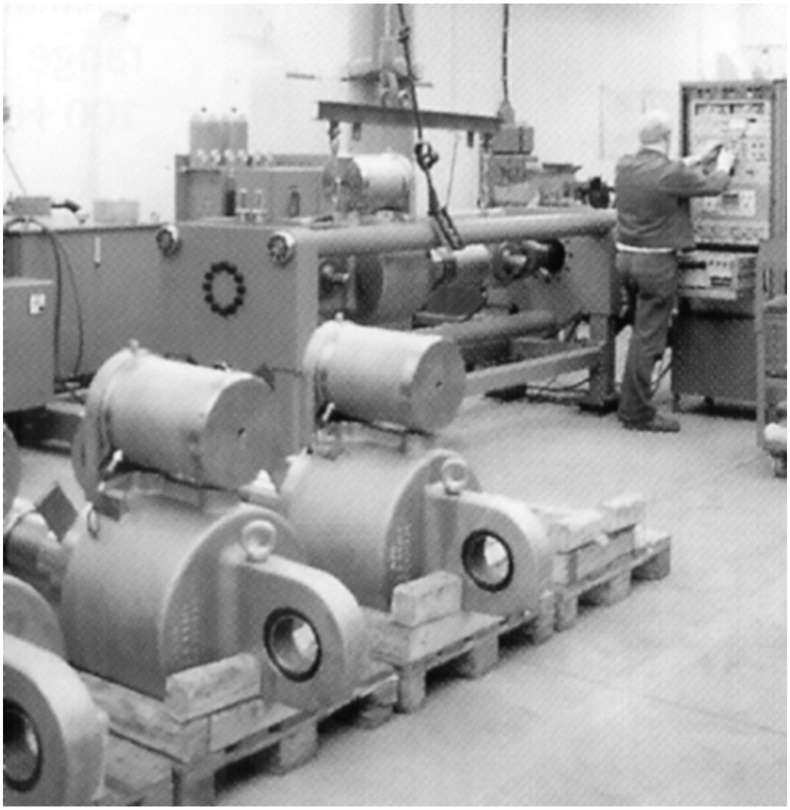


Figure 9.8 Pre-service testing of snubbers in a manufacturer's laboratory



- Activation (restraining action) is achieved within the specified range of velocity or acceleration in both tension and compression.
- Snubber bleed, or release rate, where required, is within the specified range in compression and tension. For units specifically required not to displace under continuous load, the ability of the snubber to withstand load without displacement shall be demonstrated.

Figures 9.8 and 9.9 show the pre-service testing of snubbers in the manufacturer's laboratory, and Figure 9.10 shows testing of an installed snubber in a nuclear power station, with the help of a mobile testing bench.

## 9.5. Installation

Snubbers can be installed in virtually any conceivable position. Standard extensions (Figure 9.11) are available to bridge larger installation lengths, thus avoiding structural adjustments on site. Connection to the snubber is made at the cylinder base. In addition, a range of special connections is also available (Figure 9.12) from manufacturers to replace an existing snubber installed by another manufacturer who is no longer supplying these devices.

Figure 9.9 Computer-controlled test equipment to achieve optimum testing of snubbers

Figure 9.10 In-service testing of installed snubbers

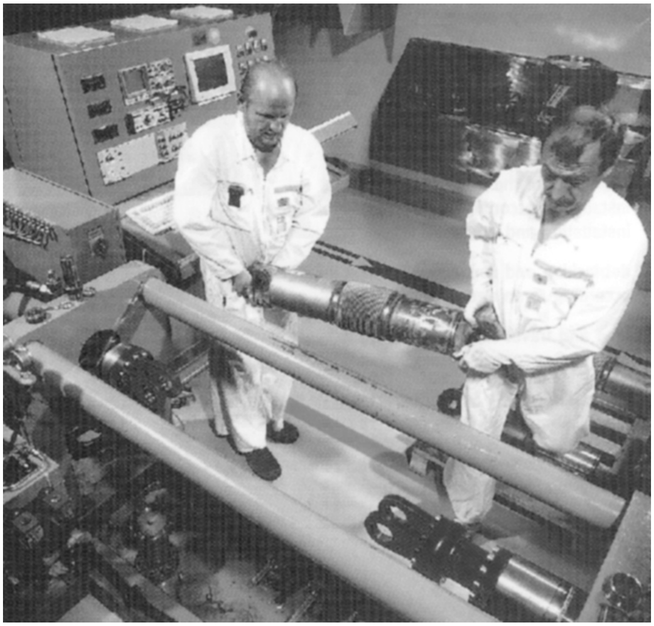
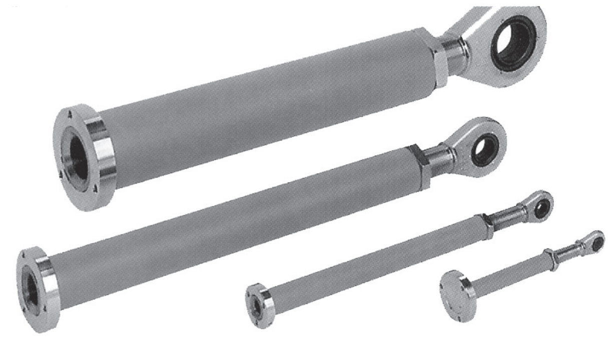


**Figure 9.11** Installation extensions



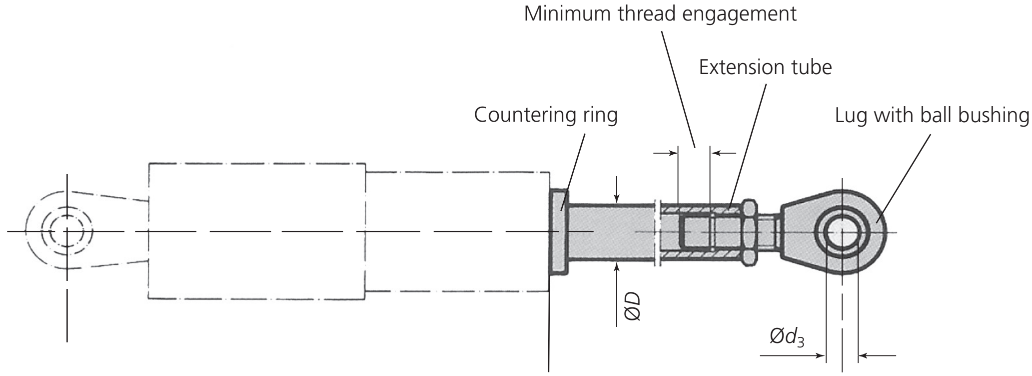



**Figure 9.12** Special installation connections

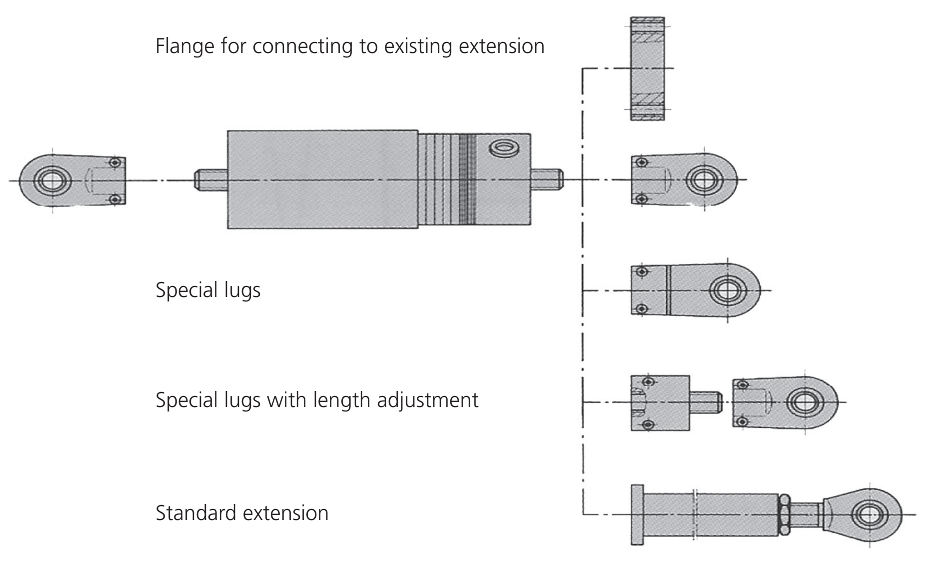
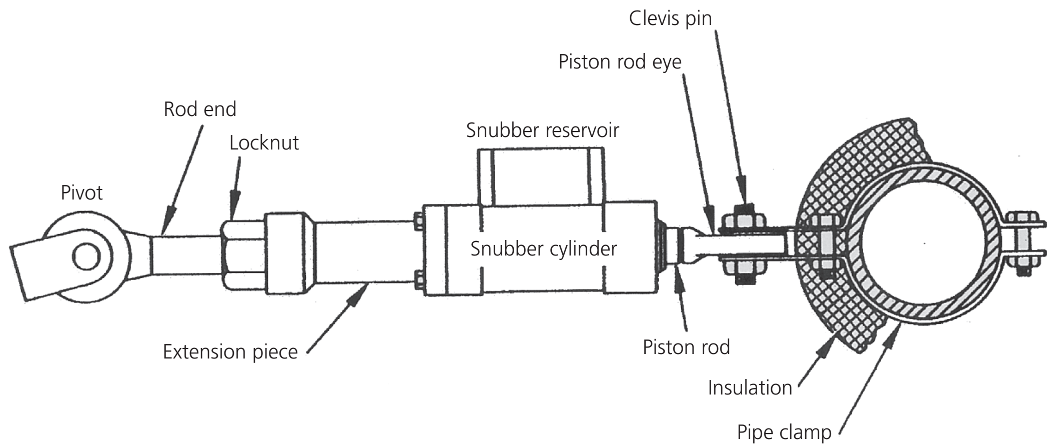


Figure 9.13 Typical connection of a snubber to a pipe carrying fluid in a nuclear power plant




The position of installation of snubbers should be selected so as to offer access to the sight glass for fluid inspection during service checks. The connections to the various attachment structures must be form fit for load actuation. All threaded connections in the flux force must be tightened with sufficient torque. Figure 9.13 shows a typical connection of a snubber to a pipe in nuclear power plant.

Just before installation, the connection lugs must be checked for a tight fit, and the snubber must be inspected for any damage. The structural attachments and weld-on bracket (see Figure 9.6) must be completely welded on site. The arrangement of the weld on the brackets must be such that the maximum angular displacement takes place in the direction of the greatest thermal expansion in service. Lateral displacement is limited to a maximum of $\pm 6°$. All welding at the connections or in their vicinity should be completed before installing the snubber. However, if any welding needs to be carried out on the attachment structure after snubber installation, care must be taken that no welding current strays through the body of the snubber. After installing the complete system for the snubber, each unit must be checked for correct fitting of form-fit connections and freedom of movement during thermal variations.

Figures 9.14 and 9.15 show the connection of a snubber to a pipe carrying liquid in a nuclear power plant, to control movement of the pipe in the longitudinal and vertical direction, respectively.

LBHSs are used to support large equipment and piping. Pressurised water reactors, steam generators, reactor coolant pumps, and large piping systems (main steam and feed water) are usual locations for LBHSs. However, according to a 1989 survey conducted to develop the *Technical Evaluation of Generic Issue 113* (Nitzel *et al.*, 1992), configurations in US nuclear plants varied from plant to plant, and some plants did not have LBHSs at one or more of these locations. Combustion engineering plants generally had large LBHSs on their reactor coolant plant. For instance, there were two 850 kip LBHSs on the Palo Verde plant, as shown in Figure 9.16.

## 9.6. Inspection and maintenance

For normal operating conditions, snubbers are usually designed to function for the entire 40-year life of a nuclear power plant. The seals and hydraulic fluid should be checked at least once during this period, at least after 20 years, or as recommended by the manufacturer.

**Figure 9.14** Snubber connection to a pipe in a nuclear power plant to control movement of the pipe in the longitudinal direction

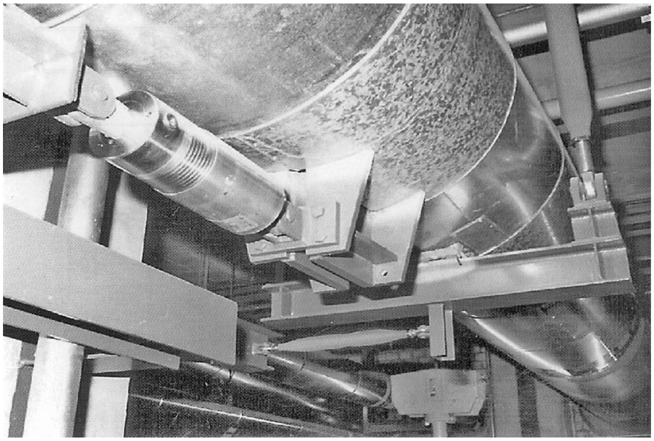


**Figure 9.15** Snubber connection to a pipe in a nuclear power plant to control movement of the pipe in the vertical direction

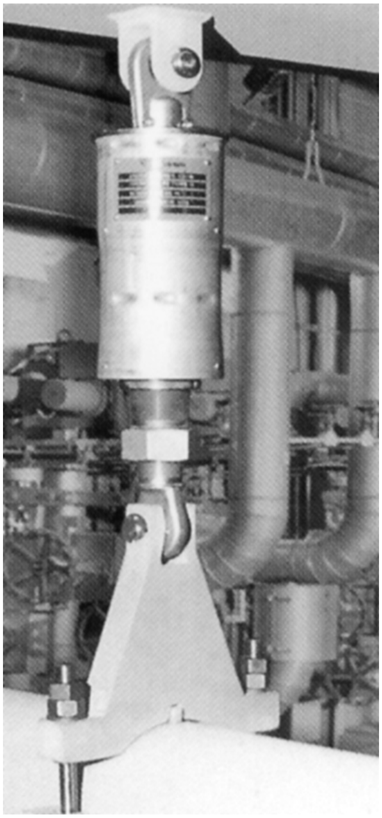
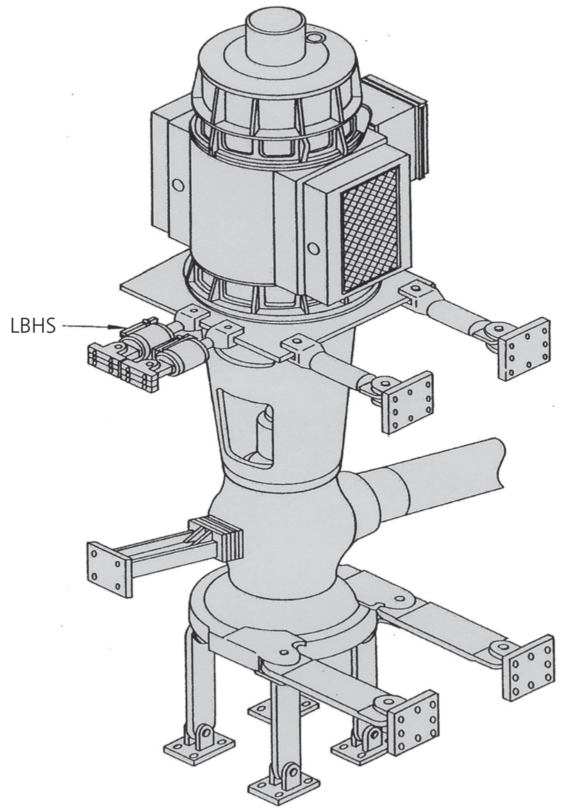


Figure 9.16 Large-bore hydraulic snubber installed on the Palo Verde reactor coolant pump



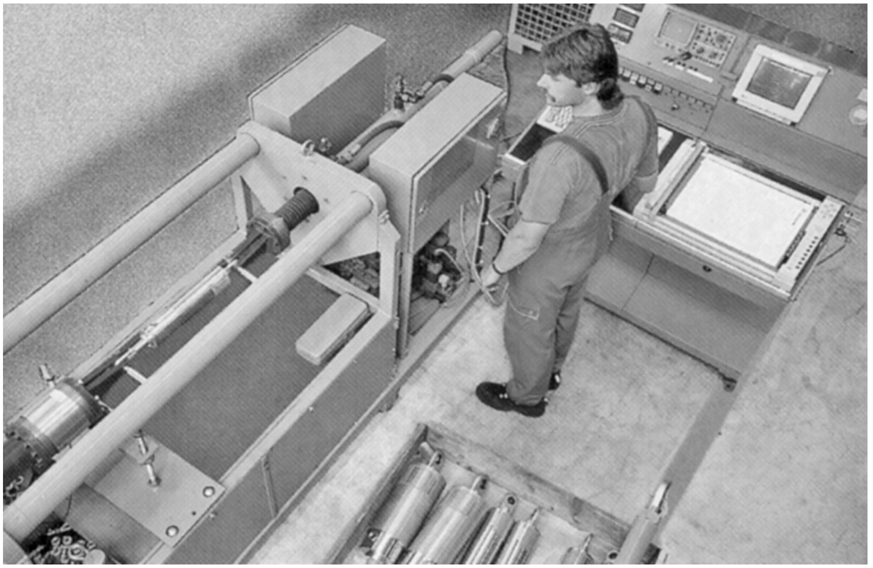

However, under certain conditions, snubbers can experience premature ageing and increased mechanical wear. Preventive maintenance is recommended in order to make sure that the snubbers remain fully operational and reliable. This maintenance is the responsibility of the plant operator.

Regular inspection should involve a visual inspection carried out once a year on all installed units. The first inspection should take place immediately before commissioning. The regular inspection should include inspection of the snubbers and a check on the environmental conditions and installation situation.

As long as the reservoir piston cannot be seen through the sight glass there is sufficient fluid in the reservoir. If the piston is visible, the unit must be checked to see if fluid has leaked out.

A more rigorous inspection is carried out after 12 years, whereby a number of installed snubbers (minimum of two units of each type) are subjected to an additional function test. Mobile on-site computer-controlled benches for carrying out the function tests are available from the manufacturers (Figure 9.17). The extended inspection and maintenance should be carried out by specially trained personnel supplied by the manufacturer.

Figure 9.17 On-site test bench in a nuclear power plant



After approximately 20 years of operation, at the latest, it is recommended that the hydraulic fluid and seals in all installed snubbers are replaced, and the snubbers then subjected to function testing by trained personnel employed by the manufacturers. The Snubbers can be then used for a further 20 years period.

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# Chapter 10
# Miscellaneous applications of STUs

## 10.1. Introduction

International bridge engineering projects in the twenty-first century will present engineers with the ever more demanding challenge to find solutions to often conflicting design constraints, such as elegance, robustness, seismic resistance and, above all, economy. It is hoped that, in the years to come, STUs and their derivatives will provide solutions to these complex problems. Because of their simplicity, their tolerance of a wide range of environments and their lack of need for maintenance, modern STUs are opening up a whole range of possibilities in the construction industry. For example, their ability to absorb the natural diurnal slow movement of large structures yet provide load transfer under heavy impact loading makes them ideal for use in the design of rocket launch pads. The military sees STUs as having interesting possibilities in the many bridging and other logistical devices that underpin all modern military movement on land. Undoubtedly, there are other applications of STUs yet to be exploited, and among possible avenues worthy of consideration are

- STUs assisting in maintaining structural integrity and stability during the settlement of structures subjected to sudden ground settlements
- STUs used in conjunction with various energy-absorbing devices on floating tunnels to transfer all longitudinal loads and a large proportion of longitudinal movements
- STUs used in conjunction with energy-absorbing protective devices on bridge piers at risk of ship, train or vehicle impact.

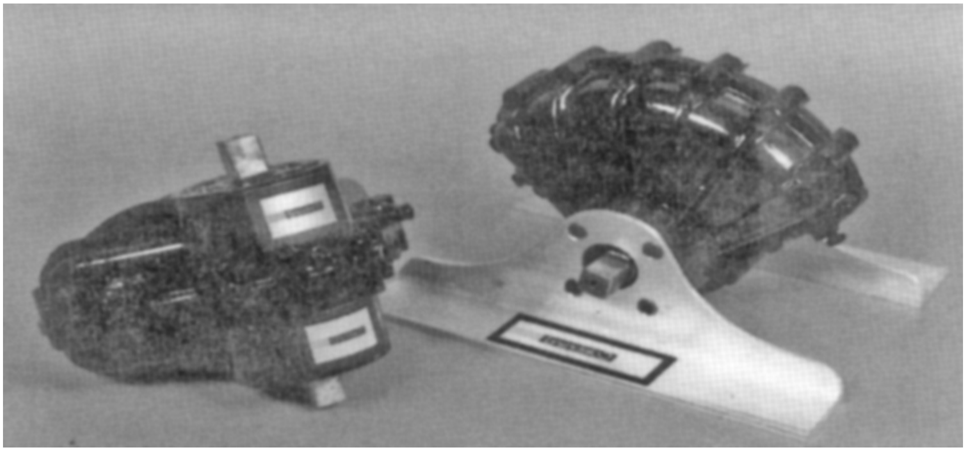
There is a great deal that can be achieved by using the simple robust STU once engineers set their minds to exploit its potential. Apart from their use on bridges and buildings, there are various miscellaneous applications of STUs. While STUs are normally used on bridges in the horizontal longitudinal direction for load transfer, etc., they can be also used in a transverse direction for bridge widening, and in a vertical direction for impact transmission, etc. Some unusual applications of STUs are included in this chapter, but it is anticipated that, as awareness about the application of STUs grows among engineers, more and more structural applications of these devices will be found in the future.

## 10.2. Rotational STUs for floating pontoons

The STU principle can be extended to rotational applications to provide a mechanism between two separate structures that will allow free slow-acting relative rotations but transmit bending moment and shear during fast-acting relative rotations. Figure 10.1 shows a model of a rotational STU that was developed for military pontoon bridging (Pritchard, 1992).

STUs can be installed to connect each floating pontoon of the bridge and the bridging elements attached to the river banks. The slow rise and fall of the river bridge is accommodated by free rotation of the bridging element joint units. The slow individual rotations of each pontoon due to

Figure 10.1 A rotational STU



currents and river swell are similarly accommodated by free rotation of the units at the pontoon joints.

However, when a heavy military vehicle such as a tank, drives onto the floating bridge, the relative rotation between the bank and the pontoon elements is minimised by the temporary fixity of the joint unit, usefully reducing the nose drive impact as the tank crosses onto the first pontoon unit. More important is the shear and moment transfer offered by the temporarily locked pontoon joint units. This allows increased spread of the tank load between pontoons, to the extent that the numbers of pontoon units required for a river crossing can be usefully reduced.

The rotational STU could offer highly beneficial stiffening and load spread under traffic for floating bridges for long water crossings.

## 10.3. STUs for bridge parapets

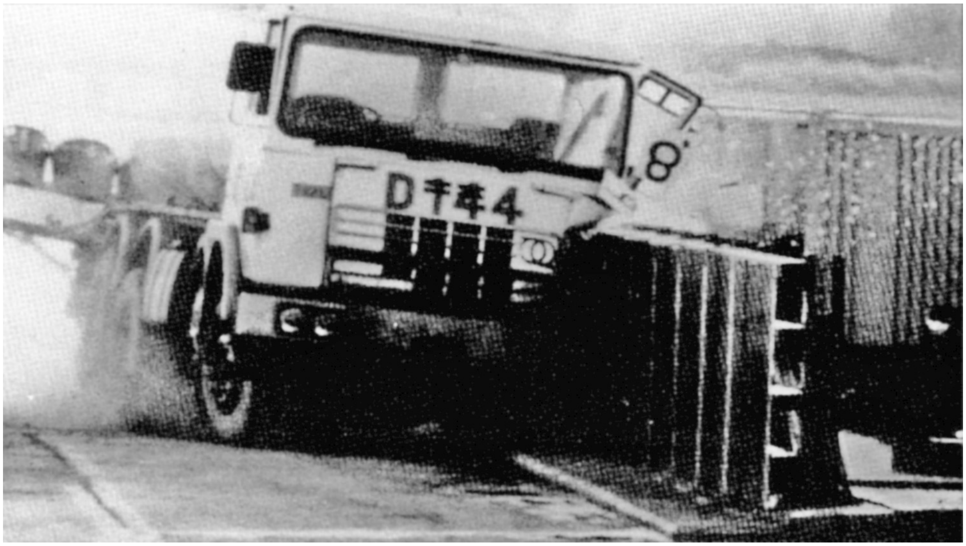
In Part 1 of BS 6779: 1992, it is stated that STUs may be used as one of two methods for dealing with expansion joints in high containment parapets (Pritchard, 1992). The STU is defined as being able to transmit not less than 500 kN, and to be incorporated in the joint between the top and second effective rail at as a high a position as is practicable (BSI, 1992).

In general, the method has not been economically competitive with the alternative of strengthened parapet bays adjacent to the joint. Nevertheless, some interest has been shown in the use of the method for large joint movements. Figure 10.2 shows testing of a high containment parapet with an STU connection at the expansion joint.

## 10.4. STUs for vertical movement and impact transmission

It should be noted that the STU has been designed primarily to function in a horizontal position to resist horizontal impact forces. Nevertheless, it will normally work equally well at inclinations of a few degrees if differential out-of-plane movements occur between anchorages. It can readily be adapted

Figure 10.2 Testing of a parapet with an STU connection at the expansion joint

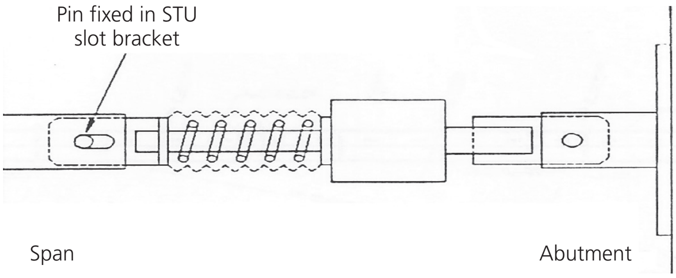



for vertical movement and impact transmission through the incorporation of an internal spring to counter gravity and to return the piston to the neutral position.

## 10.5. STUs as unidirectional struts or ties

The use of a spring can also change a horizontal STU into an unidirectional strut or tie, if required, rather than the usual two-way strut or tie (Pritchard, 1996). This type of STU has been used to upgrade an existing multi-span bridge carrying London's Docklands Light Railway (DLR). The STUs were installed to an abutment in order to share the increased train traction and braking forces with the adjacent piers. However, the extra loading on the abutment had to be directionally limited to push against the backfill only, as pull would have caused abutment overload. The extra loading in that direction was shared with another substructure element. This was achieved by combining a standard STU with a helical spring unit (Figure 10.3).

Figure 10.3 An STU with a helical spring for unidirectional strut action to push against an abutment



## 10.6. STUs in a transverse direction on a bridge

While STUs are generally used in a longitudinal direction, they can also be used in a transverse direction for some bridges that require widening. One such example is the widening of the Yuen Long Highway in Hong Kong. The highway is a trunk route connecting Tuen Mun and Yuen Long, and it is constructed mainly on an embankment over an alluvial flood plain. At its original completion in 1993, it was a dual two-lane road.

To cater for future traffic growth it was necessary to widen the highway to a dual three-lane road, with 3.65 m hard shoulders on either side. The width of the carriageway widening required on either side ranged from 4.7 m to 9.3 m. This widening of the highway also required the widening of nine bridges and the extension of two underpasses and six subways.

When widening a bridge a longitudinal movement joint would normally be provided between the existing bridge deck and the new bridge deck. However, the use of such joints is undesirable with regard to future maintenance and riding quality. Therefore, a cast in situ link slab was provided for the connection. The slab was designed for the vertical vehicle loading and was made relatively thin compared with the thickness of the bridge decks in order to minimise the transfer of bending moments from one bridge deck to another. The slab was cast 60 days after completion of the new deck in order to reduce the stress caused by the shrinkage of the concrete in the new bridge.

With the construction of the link slab, transverse loads on the new bridge deck, such as wind load and seismic load, will be transmitted to the existing bridge due to the set up of the bearings on the existing bridge (Figure 10.4). The existing fixed and longitudinally guided bearings on the substructure of the existing bridge did not have sufficient additional capacity to resist this additional transverse loading, and replacing the existing bearings with larger capacity bearings would have created serious disruption to traffic.

This problem was resolved by making use of STUs – the first time these devices had been used on a bridge in Hong Kong. As shown in Figure 10.4, an STU was placed on the fixed abutment of the new deck to connect the substructure to the superstructure. With the STU installed, the decks could move due to temperature variation, but in the case of a seismic or wind gust event the STU would lock up and act as a rigid link, thus transmitting the transverse loads from the new bridge deck to the new bridge abutment on which the STU is installed.

## 10.7. STUs for strengthening against collision forces

The existing Barham Road Bridge on the A14 in the UK needed to be strengthened against the effects of accidental collision forces due to increased road traffic. The existing guided bearings in a longitudinal direction were inadequate to take the superstructure collision loads. A reinforced concrete infill wall (Figure 10.5) was added between each pair of columns, and this resulted in adequate resistance to the specified forces for the modified columns themselves. However, the column bases were still inadequate to withstand loads occurring parallel to the carriageway. To remedy this situation STUs were installed to assist in transferring the deck loads to the piers. The STU connection between the top end of the pier and the superstructure essentially transformed the new piers into propped cantilevers, thereby making the modified piers adequate to withstand substructure collision loading.

Under normal conditions the STUs allowed free relative lateral movement between the pier and the deck. One STU was installed on each of the three piers formed by the paired original columns and the new infill walls (see Figure 10.5). The STUs are fixed to the top of the infill walls by way of

**Figure 10.4** Widening of the bridges on the Yuen Long Highway using an STU in the transverse direction on the abutment

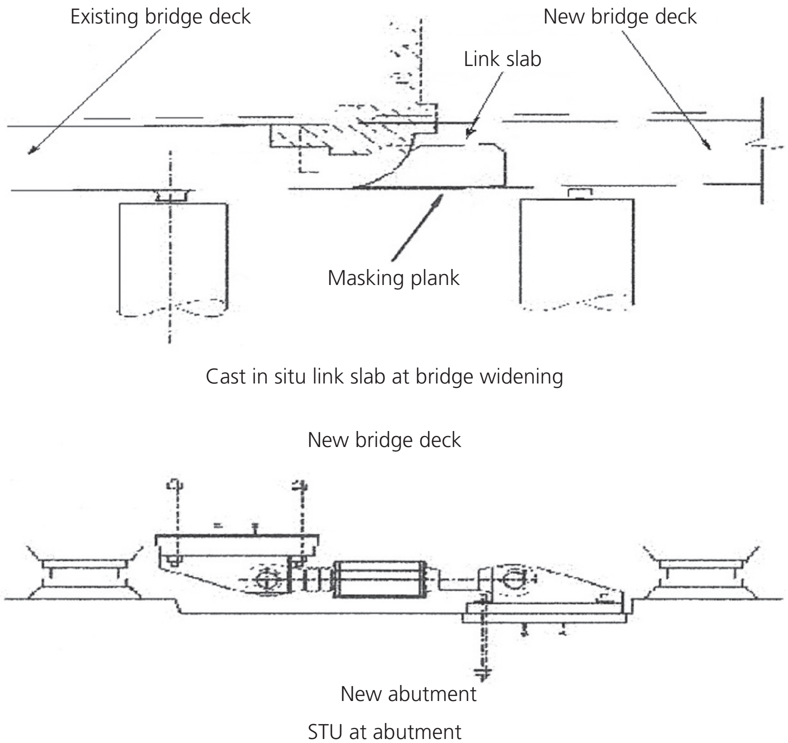
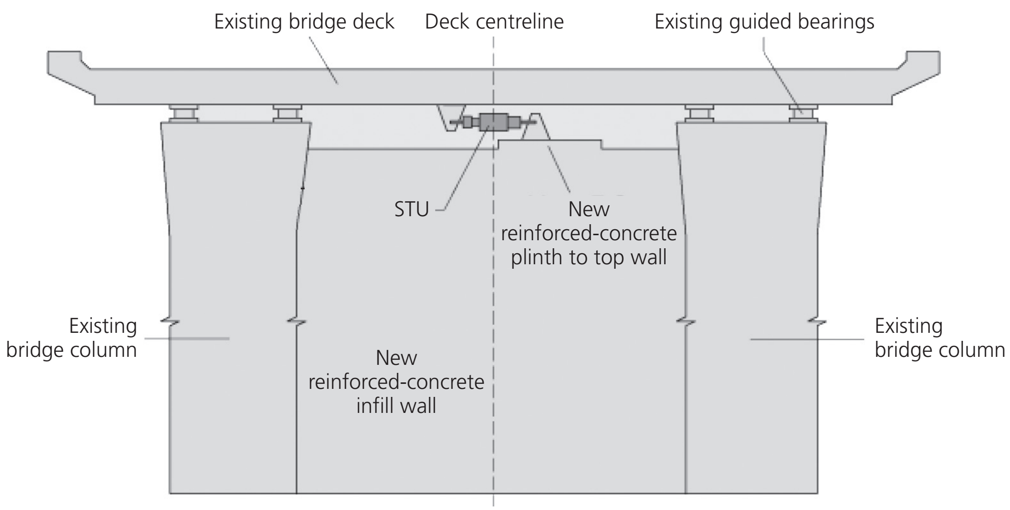



**Figure 10.5** The strengthened pier with the reinforced-concrete infill wall and an STU installed in the transverse direction on the Barham Road Bridge. © Colebrand International Ltd



cast-in sockets, and to the soffit of the slab via drill-in anchor bars. STUs of 500 kN were installed to transmit the forces along the axis of the units in the transverse direction. The units were designed to allow for a thermal movement of $\pm 50$ mm across the bridge deck. A longitudinal movement of $\pm 25$ mm along the bridge deck was accommodated by the design of the mounting bracket.

## 10.8. STUs used temporarily during construction

STUs can be used temporarily for bridge repair works or during construction. For example, in the early 1980s temporary STUs were installed to provide stability during the construction of a continuous precast beam/in situ concrete crosshead viaduct at Sandwich, UK.

During construction, both putty-filled and oil-filled STUs can be used for the closure of a segmental deck cable-stayed bridge. For a cable-stayed bridge, oil-filled STUs similar to those used on the Second Severn Bridge (see Chapter 7, Section 7.4), with the cylinder surrounded by a barrel-shaped elastomeric sleeve, can be used as an actuator. Consider, for example, the closure of the main span of a segmental deck cable-stayed structure. Insertion of the final unit can be greatly eased if the deck ends can be jacked apart slightly, allowing the final segment to be adjusted into place. Permanent STUs are ideally suited to this task and, in particular for cable-stayed bridges, these STUs can be used later to control the longitudinal movement of the deck due to wind gust or earthquake. This would save considerable time and money in hiring, placing and removing additional, and sometimes large, jacks for the closure of the span. Finding locations to jack horizontally between the deck and the substructure can also prove difficult. With this type of STU, a permanent compression or tension can be developed in the deck by activating the unit hydraulics to shear the elastomer. The shear in the elastic element is then balanced by the corresponding longitudinal forces in the deck.

Putty-filled STUs can also be used for the closure of a segmental deck cable-stayed bridge by providing internal or external pre-compressed springs. The deck segments could be built to leave the required oversized gap for the placement of the central unit. STUs mounted on the free pier would incorporate internal or external springs which, on release, would overcome the bearing friction acting through the STUs to slowly close, or even compress, the remaining gap. The STUs would then revert to their normal in-service behaviour.

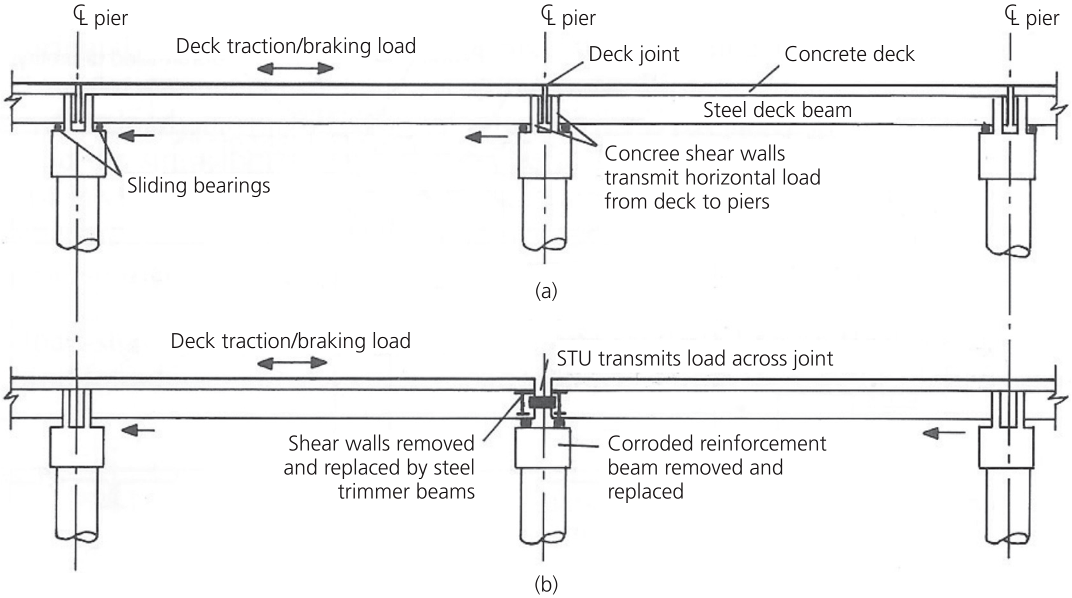
## 10.9. STUs for the replacement of a reinforced-concrete cross-head

Figures 10.6 and 10.7 show the STUs used in a complex replacement of a salt-corroded reinforced-concrete cross-head supporting a section of a heavily trafficked deck of the M5/M6 Midlands Link motorway viaduct on the outskirts of Birmingham, UK.

The viaduct comprises simply supported steel composite reinforced-concrete decks supported by reinforced-concrete cross-beams constructed on reinforced-concrete circular columns. The joints were leaking, which allowed road salts to contaminate the cross-beams beneath the joints with chlorides. This resulted in corrosion and delamination. It was decided to replace one of the worst affected cross-beams. The problem was that the simply supported steel composite decks, located on the temporary supports for cross-beam replacement, and then permanently located on the new cross-beam, could not accommodate the traction loading on the permanent supports unless the decks could share the traction forces. This was achieved by means of installing STUs between the deck ends, this solution still permitting thermal movement of the structure.

The removal of the corroded cross-head included the removal of twin shear walls, which had served not only to support the deck edges either side of the deck expansion joint, but also transmitted the

Figure 10.6 STUs transmitting the traction and braking force on the M5/M6 Midlands Link viaduct: (a) bridge before repair; (b) bridge after repair




deck traction and braking loads down to the cross-head and pier. The new steel trimmer beam visible in Figure 10.7 took over the slab-edge support duty, while the traction and braking loads were transmitted through the strut-tie STU links between the decks either side of the expansion joint and the shear walls and supporting piers either side of the cross-head pier (Figures 10.7 and 10.8).

Figure 10.7 STUs for cross-head replacement on the M5/M6 Midlands Link viaduct

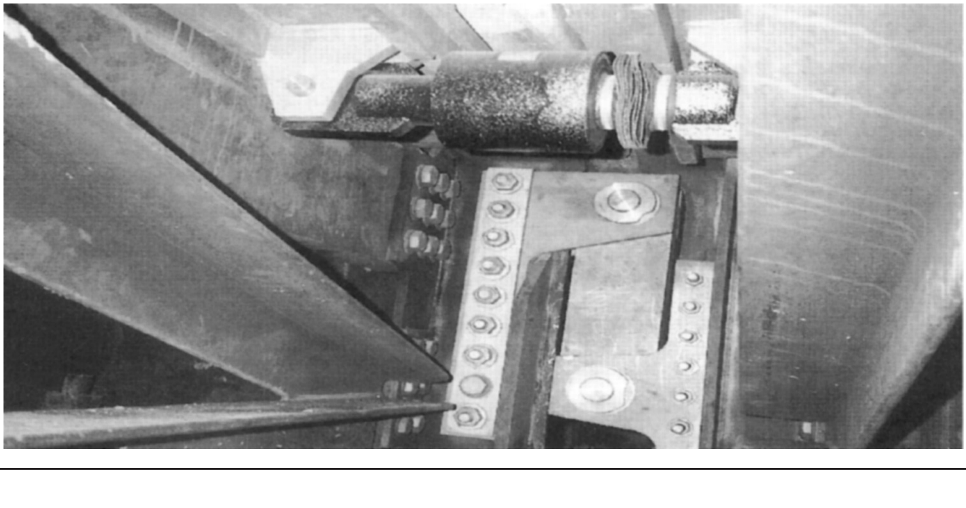
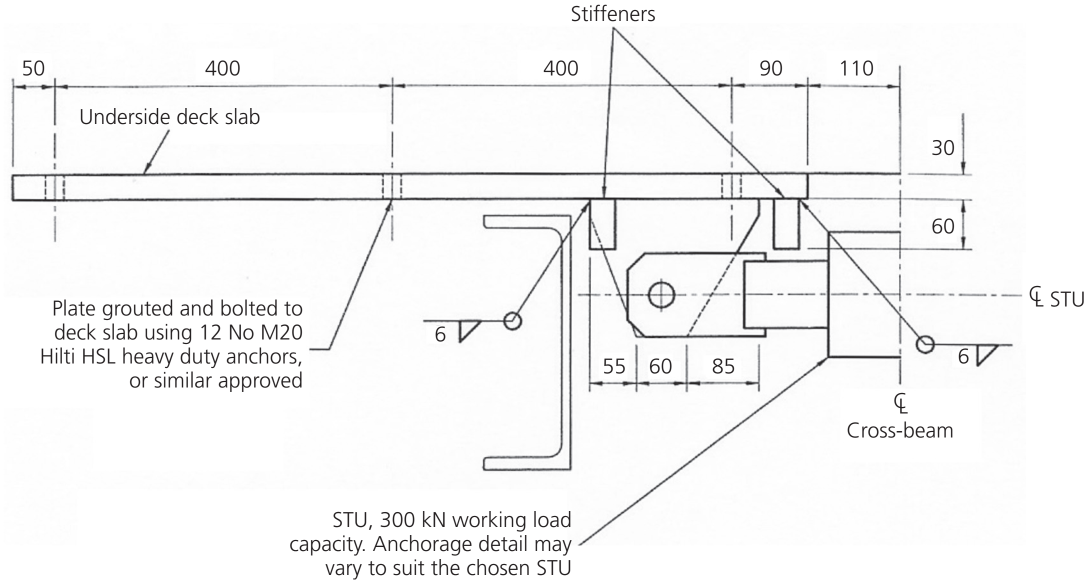


**Figure 10.8** STU connection detail for the Midlands Link viaduct. © Institution of Civil Engineers




## 10.10. STUs integrated in pot bearings

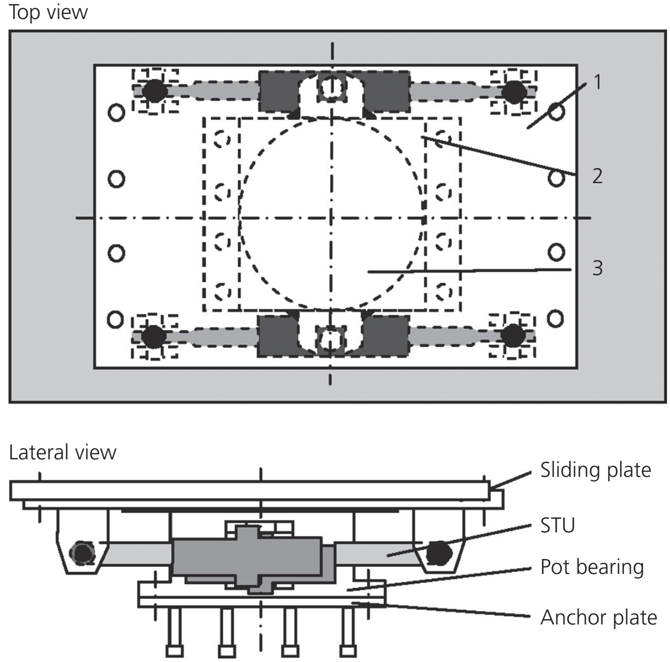
Manufacturers in Germany and Italy have developed STUs integrated in moving bearings, which provide a very compact connection. In addition, these devices do away with separate installation of the STU, which provides a cost saving. The heavy brackets and holding-down bolts generally required for the installation of STUs are quite expensive, and also require maintenance for corrosion protection. Integrating STUs into moving bearings (Figure 10.9) is the state of the art in compact installation, and generates economy in installation costs. These integrated STUs have been installed on the Stura Di Demonte viaduct in Italy (Figures 10.10 and 10.11).

The viaduct is located on a highway connecting Torino to Savona over the river Stura Di Demonte near the village of Fossano. The 2.73 km long viaduct is a continuous composite steel box girder bridge comprising 31 spans (29 are 90 m long and two are 60 m). It was constructed using the incremental launching method, because the viaduct crosses a deep river valley with the maximum pier height of 90 m at the centre. The piers are of wall type, and three central piers are built into the deck, making the bridge fixed at these piers. Eight integrated STUs having a vertical bearing capacity of 17 000 kN, a horizontal longitudinal force capacity of 1700 kN and $\pm$370 mm movement were used on this viaduct.

## 10.11. STUs for access stairs and escalator supporting structures

A novel application of STUs was made on the access stair and escalator supporting structures at the East India Dock Station of the Docklands Light Railway (DLR) Beckton extension, London, UK. The viaduct carrying the station platform runs at a high level and required correspondingly high structures on either side of the platform to support the stairs and escalators. Some means of providing support to the stairs and escalator structures in the longitudinal direction was necessary for their stability against

Figure 10.9 An STU integrated in a pot bearing




impact in that direction. Small STUs were attached to the structure and the platform. The linkage to the side of the viaduct offered longitudinal impact transfer and stability benefits to the stair and escalator structures, while at the same time allowing the viaduct to move in the longitudinal direction due to thermal effects.

Figure 10.10 The Stura Di Demonte viaduct

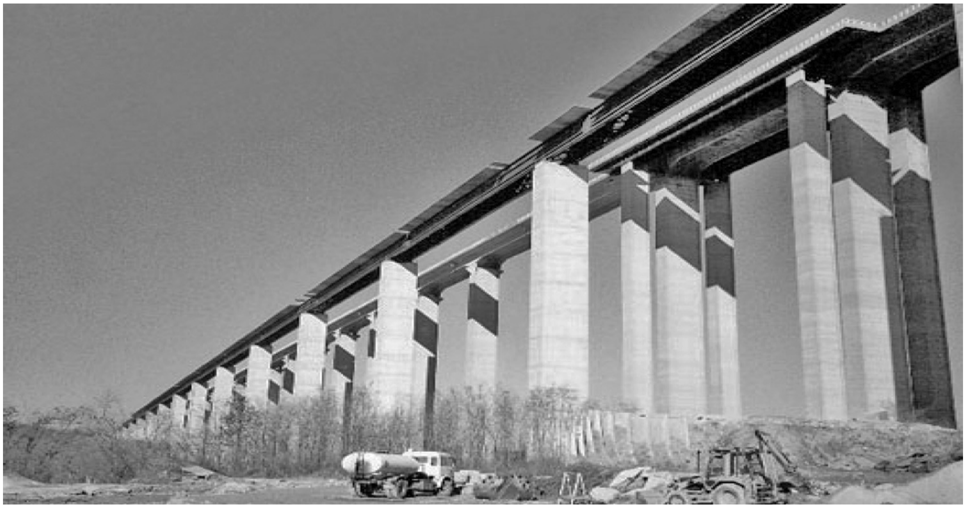
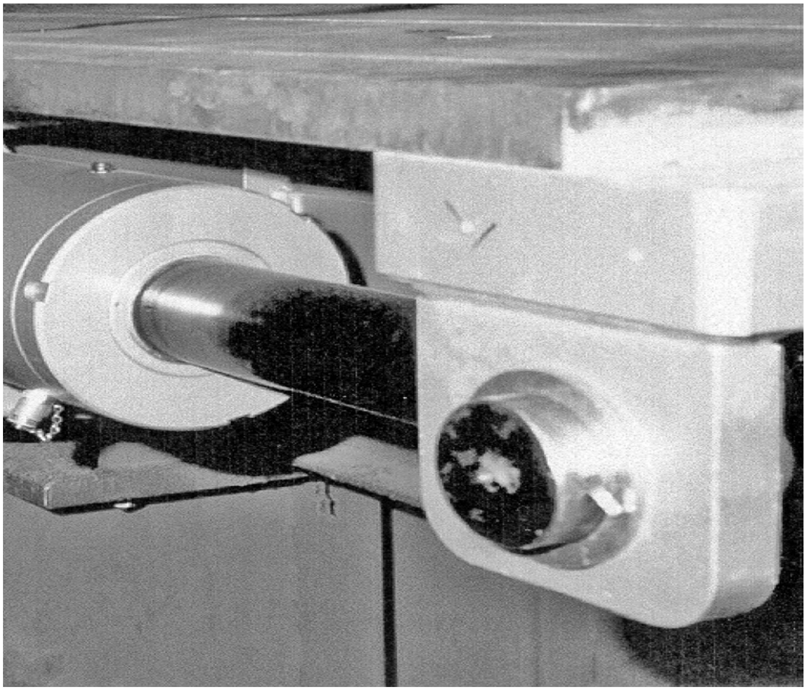


Figure 10.11 An STU integrated within a bearing installed on Stura Di Demonte viaduct. © Maurer Söhne GmbH & Co. KG

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# Chapter 11
# Installation of STUs on structures

## 11.1. Introduction

While the installation of STUs on a new structure can be carried out during the construction or after the structure is completed, it is better to install them during construction, similarly to bridge bearings. There is also the advantage of ease of installation compared with installation of the STU after construction in a confined space. However, the STUs may need to be preset accurately and the stroke required may need to be somewhat larger if they are to be installed during construction, to allow for the larger creep and shrinkage movements of, for instance, a continuous concrete bridge. However, if the STUs are to be installed after the structure has been completed, the holding-down anchor bolts, and maybe the brackets required to fix the STUs, must be installed in the structure during its construction. Time can be saved if the holding-down anchor bolts and the brackets are not imported but are fabricated locally as per the specifications. The installation of STUs, if planned to be done during the construction of a new structure, may become a critical activity and cause delay due to the time involved in procurement and load testing.

While installation of STUs on a new structure is a fairly simple task, the installation of STUs on an existing structure can be tricky and may require a bespoke solution due to an uncompromising and complex arrangement of the structural elements. STUs can be manufactured and load tested to provide a lifespan of about 50–75 years, but it may become necessary during the life of the structure to replace some of the STUs because of damage or performance problems, or the structure may outlive the STUs. Therefore, it is of utmost importance to provide sufficient space at the point of installation of an STU so that it can removed and replaced in the future.

There should be a provision in the tender documents for the manufacturer to send an experienced engineer for the installation of the STUs. The first step in the installation of an STU is to compare the gap between the pin connection points of the attachment brackets with the preset length of the STU (pin-to-pin distance). The STU pin-to-pin length can be altered through the threaded end connection clevis, but this is usually limited to about $\pm 10$–50 mm depending on the size and the movement requirement of the STU. When the total movement of the STU is about 1 m or more, further adjustment above the tolerance of $\pm 50$ mm may be required. To do this, the STU must either be jacked open using specialist equipment, or hung in a manner that permits gravity to slowly open the device. Obviously, the opening or closing of the STU takes time, as it can only be opened at a very slow velocity that will not cause it to lock up. Once the pin-to-pin distance of the attachment brackets and the STU pin-to-pin length are exactly the same, the STU can be lifted into position for installation.

Once the holding-down anchor bolts have been installed in a new structure, the installed length of the STU may depend on the site conditions prevailing at the date of installation. Therefore, the manufacturer must be advised, before the STUs are transported, if any adjustment in length above the tolerance provided is required due to the normal day-to-day movement of the deck structure, otherwise specialist equipment may have to be brought to the site, which may prove expensive.

STUs can be installed in groups to meet a high capacity requirement, and in many cases it may be more economical to install a lower capacity, say 150 ton capacity, STU in groups than to manufacture a single special high-capacity STU, for which the cost of load testing may be prohibitive. However, there may be circumstances, such as the case of the Carquinez Bridge, where space for the installation of STUs is limited and there is no alternative but to install a single high-capacity STU. Small STUs having a capacity of up to 150 ton are easy to transport and install, and can be tested in most of the laboratories equipped for such testing. However, each STU requires fittings, which can be also expensive, and a cost–benefit study should be done to determine the most cost-effective solution, taking into account the costs of the STU, load testing, fittings, and transportation and installation (if space for installation is not a criterion). However, if it is decided that STUs should be installed in groups, either in series or in parallel, then it is of utmost importance that the stiffness and behaviour characteristics under loading of each STU should be as near as possible the same.

## 11.2. Design of connections

Normally, when an STU is to be installed across an expansion joint, the fasteners need to be designed for direct shear only. However, when an STU is installed between the superstructure and the substructure of a bridge, in addition to the direct shear forces, there is an additional moment acting on the anchorage components due to the small installation gap required between the soffit of the superstructure and the top of the substructure. The fasteners for such a connection must be designed for the direct shear forces generated by the rated capacity of the STU in addition to the axial loads on the fasteners generated by the moment.

As an STU responds to dynamic-type impact loadings by locking, there is a possibility of load amplification due to dynamic force. It is recommended that related components, such as the substructure, anchorage and all fasteners, are designed for an amplification of 1.25 times the rated STU load.

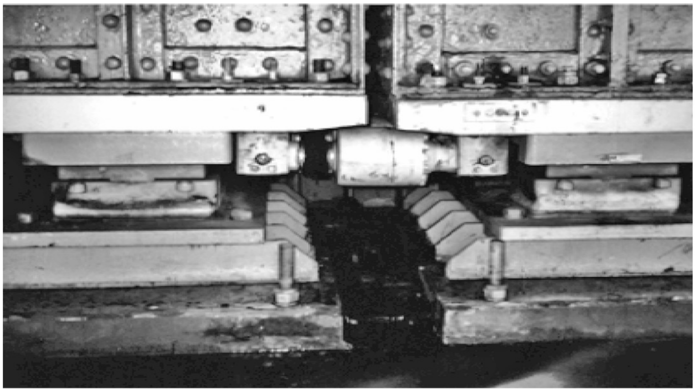
## 11.3. Standard STU connections for highway and railway bridges

There are generally two ways of connecting STUs to new highway and railway bridges. One way is to connect the STU at the expansion joint by fixing each end to the superstructure elements (Figure 11.1). The other way is to connect the STU in the small gap between the superstructure and the pier, generally in conjunction with unidirectional bearings (Figure 11.2). The selection of the connection type depends on what is to be achieved by using the STUs.

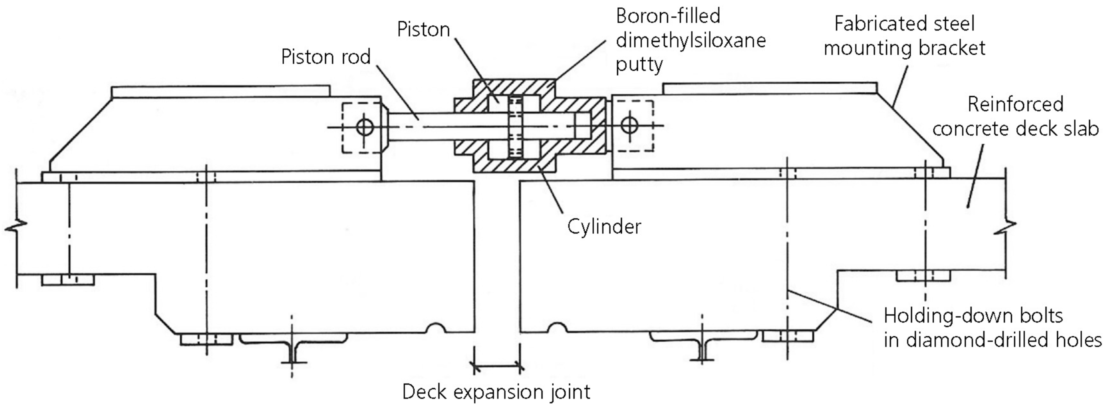
The use of an expansion-joint type of connection is very common on railway bridges where, when an increased longitudinal traction and breaking load is applied to one particular continuous viaduct due to an increase in the number of cars or speed, the load is transmitted through STUs connected across the expansion joints and shared with adjacent unloaded continuous viaducts. STU connections of this type are shown in Figure 11.1. On the Putney railway bridge, London, the STUs are bolted to the underside of the steel girders (see Figure 11.1a), while for the Docklands Light Railway (DLR), London, the STUs are bolted to the upper surface of the concrete deck (see Figure 11.1b), making for easy inspection and maintenance.

The second type of connection where STUs connect superstructure elements to substructure elements is used for load distribution in continuous bridges. Here, the STUs can be connected, depending on the

Figure 11.1 STU connected at the expansion joint of: (a) Putney railway bridge; (b) the DLR



(a)



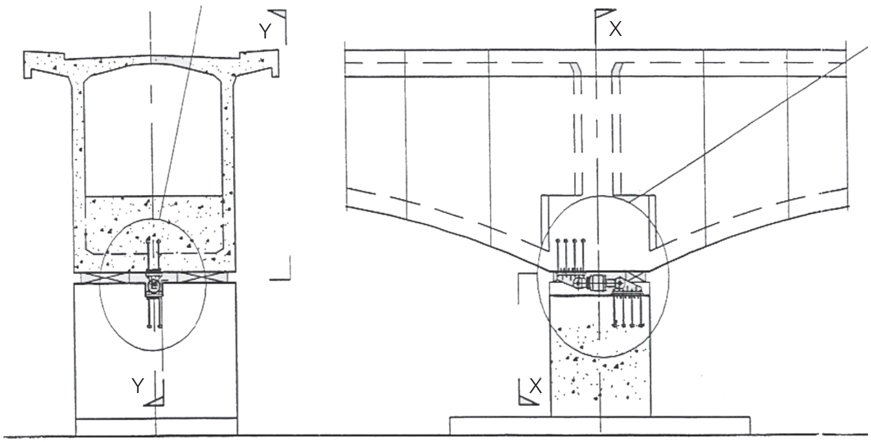

(b)

space available, between the soffit of the deck and the top of pier. There are generally three ways in which this connection can be made, as described in the following paragraphs.

Figure 11.2 shows the STU connection on the Second Bassein Creek Bridge in Mumbai, India. Here the STU is placed in a central gap between the soffit of the deck and the top of the pier located between the unidirectional pot bearings. One end of the STU is connected to the underside of the deck diaphragm by means of the bracket, shear plate and anchor bolts, and the other end is connected to the top of the pier.

The STU connection on the Arthur Laing Bridge in Vancouver, Canada, is shown in Figure 5.10. Here one end is connected to the pier face by means of a bracket held by high-strength pre-stressed steel bars through the pier, and the other end is connected to the soffit of the deck. This connection can be used when there is not enough space on the pier for a bracket connection, or when the gap between the pier top and the deck soffit is smaller than the STU height. However, while it may be easier to install and to remove and replace the STUs on this bridge than those on the Second Bassein Creek Bridge, the

Figure 11.2 The STU connection below the deck diaphragm of the Second Bassein Creek Bridge. © Colebrand International Ltd




disadvantage is their full visibility, inviting theft, especially in developing countries where every piece of steel can be converted into money.

Figure 11.3 shows the STU connection on the Second Badiwan Bridge, Baguio, the Philippines. Here one end is connected to the pier top, and the other end is connected to the soffit of the bottom slab of the box girder. However, on this bridge, as there was more than one STU and more than one bearing on each pier (four 1600 kN STUs on each pier), the STUs could not be placed directly under the solid diaphragm. The only possibility was to connect the STUs to the superstructure by means of brackets anchored into the thinner bottom slab of the box girder deck. To transmit the shear forces through the slab in the event of an earthquake, the deck slab at the connection needed to be strengthened by providing additional reinforcement.

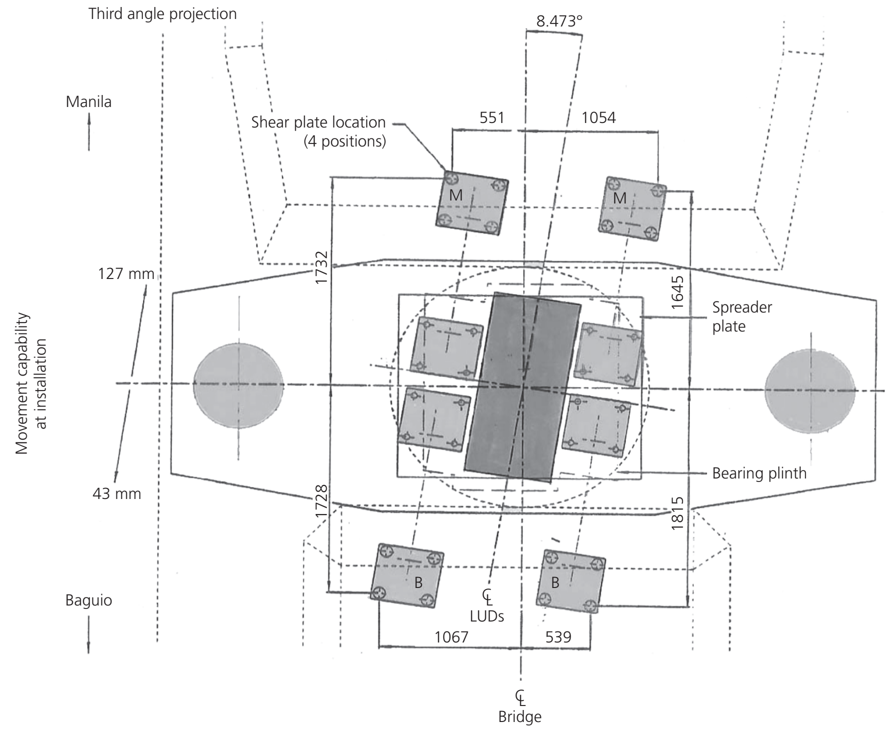
Some case studies of the installation of STUs on new bridges are given in Sections 11.4 and 11.5, and of the installation of STUs on existing bridges in Sections 11.6 and 11.7.

## 11.4. Second Bassein Creek Bridge, India

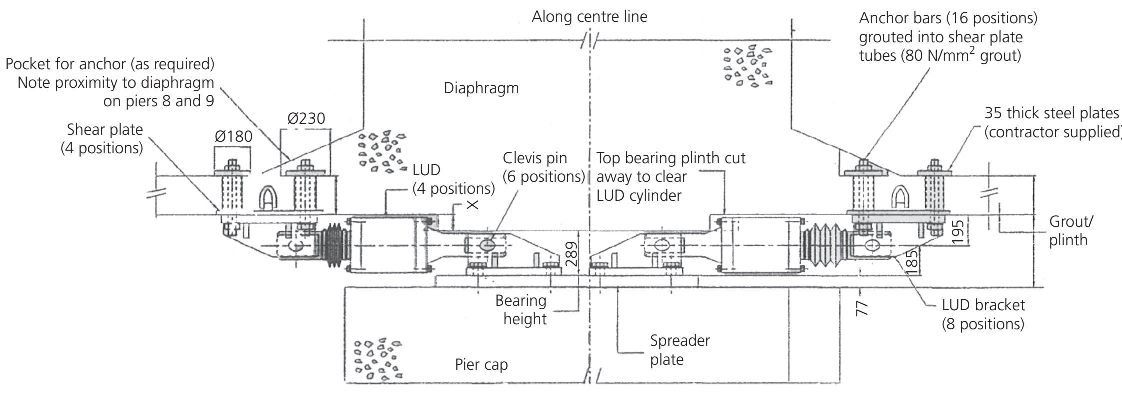
It is important that STUs are installed level. A tolerance of ±10–50 mm at the clevis pin locations is within the capability of the system. Tolerance for small positional errors is catered for in the STU clevis pin connections through the inclusion of spherical bearings; however, it is extremely important that the STUs are installed parallel to the longitudinal bridge central line at each pier on a straight bridges. On the Second Bassein Creek Bridge at Ghodbunder, Mumbai, in order to achieve this particular care was given to installing the anchor bolts and the attachment plates. The holding-down anchor bolts were installed during the construction of the pier and the first segment of the box girder deck in 1999/2000. A gap of 793 mm (Figure 11.4) between the soffit of the box girder diaphragm and the top of the pier was available, which was sufficient to install the brackets together with the STU at a later date (June 2001) when the deck was planned to be completed.

The STU is installed in the gap between the pier and the deck, as in the case of the Badiwan Bridge (see Section 11.3). However, on the Badiwan Bridge the connection of the STU is to the deck slab soffit,

**Figure 11.3** The Second Badiwan Bridge: (a) plan view of pier top, showing the STUs and bearings; (b) cross-section at the longitudinal centre line, showing the STUs



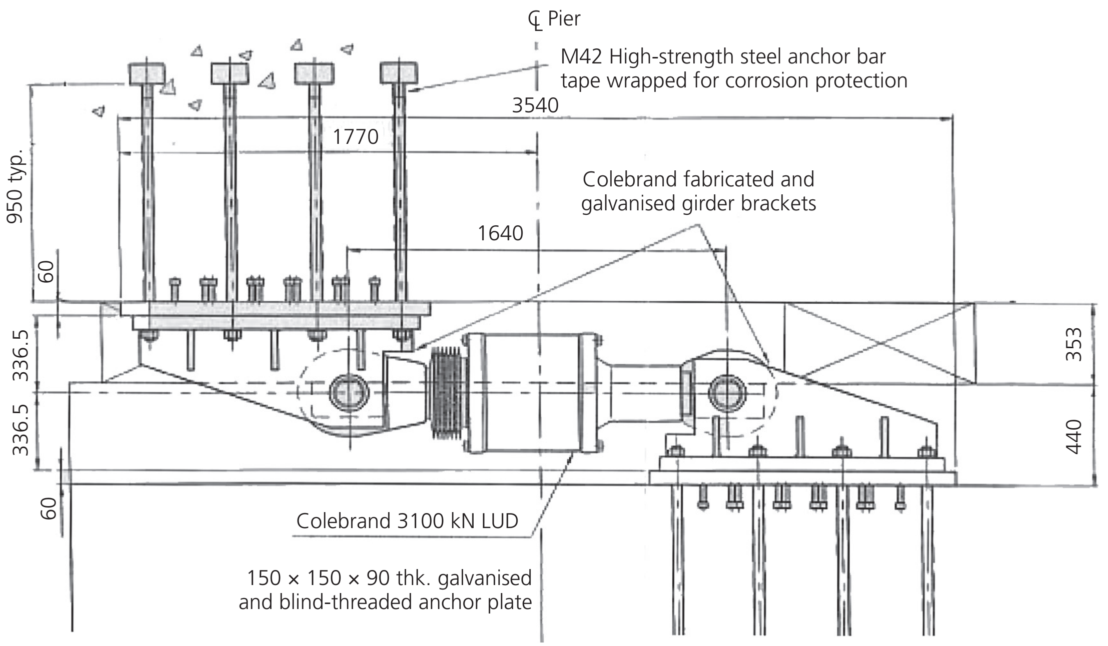

(a)




(b)

while on the Second Bassein Creek Bridge the connection of the STU is to the deck diaphragm, which is fairly rigid compared with the deck slab, and therefore does not require any additional strengthening to transfer earthquake shear forces. This connection may be somewhat more difficult to make but it is the best of the three connections for box girder bridges described in Section 11.3, both aesthetically and in terms of safety.

Figure 11.4 STU connection on the Second Bassein Creek Bridge. © Colebrand International Ltd




Two steel brackets, weighing about 0.25 ton each, were connected to the anchor bolts previously installed in the soffit of the diaphragm and the pier top through shear plates. After aligning the brackets parallel to the longitudinal bridge centreline, the holding-down bolts were tightened. The STU, weighing about 1.0 ton, was adjusted to match the pin-to-pin distance, and was connected to the brackets at each end by means of clevis pins (Figure 11.5). During construction a rebate was formed in the pier head concrete to facilitate the insertion of these pins.

## 11.5. Installation of STUs on the Mekong River Bridge, Laos

This new bridge over the Mekong River, linking Thailand with Laos, is a multi-span continuous prestressed box girder viaduct (Figure 11.6). STUs were introduced to transmit an earthquake force of 9000 kN. Pier 11, with fixed bearings, had the capacity to resist 4500 kN, and the remaining 4500 kN force was to be carried by STUs placed on one or more of piers 8–10 and 12–15, which had free bearings moving in the longitudinal direction only.

As there was not sufficient space to install six STUs of 750 kN capacity on any other free piers, they were installed on pier 12 in groups where there was sufficient space for the installation of two banks of three STUs. Similar to the STUs on the Second Badiwan Bridge, these STUs were also installed with the bracket attached to the bottom slab of the box girder bridge.

## 11.6. CR111 Bridge, Suffolk County, USA

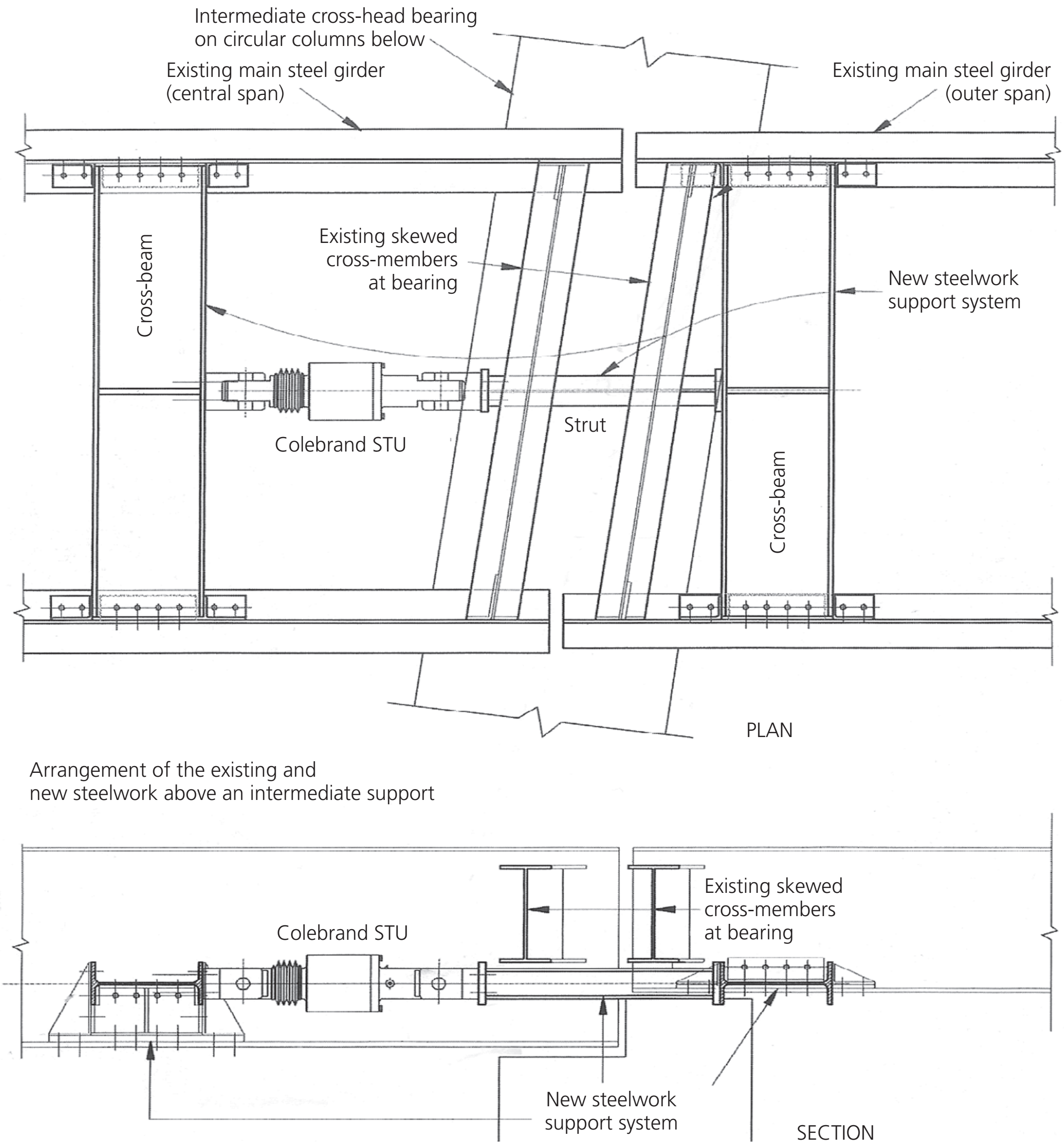
Existing bridges may have an uncompromising and complex arrangement of structural elements for which innovative installation solutions have to be found. This case study of a bridge located in Suffolk County, New York, USA, illustrates this aspect (Brown, 1996).

**Figure 11.5** Installation of STUs on the Second Bassein Creek Bridge

**Figure 11.6** The Mekong River Bridge during construction, with pier 12 ready for the installation of two banks of three 450 kN capacity STUs

The State Structural Engineers wanted to provide seismic resistance to a four-span composite bridge on trunk route CR111 by splicing the centre joint and installing deck-to-deck STUs at the two intermediate joints. The STUs therefore link the adjacent spans together while still enabling thermal movement to take place unhindered. The bridge actually consisted of two virtually identical four-span bridges, side by side on an 8° skew, one carrying the eastbound carriageway and the other the westbound carriageway. Each bridge deck comprised seven equispaced steel plate girders carrying an in situ concrete top slab. The two outer spans and two central spans were 11 m and

**Figure 11.7** Innovative installation of STUs on the CR111 Bridge



23 m long, respectively, and were carried on central and intermediate supports composed of circular concrete piers and crossheads. At the intermediate supports, where the STUs were to be installed, a change in girder depth occurred, resulting in a step in the flange soffits, which was accommodated in the plinths on which the sliding bearings were mounted. Fixed between the plate girders were stiffening cross-members, which were skewed to the main girders at the intermediate supports.

On the face of things, a considerable challenge was presented in finding a suitable way of mounting the STUs within an initially unpromising and complex arrangement. However, working in close collaboration with the State Transport Department, the engineers developed an innovative system of additional support steelwork (Figure 11.7) consisting of new cross-beams between the girders on both spans on either side of the intermediate bearings. The addition of a short longitudinal strut to one of the cross-beams then reduced the excess distance between the new cross-beams, thereby facilitating installation of the STU.

## 11.7. Carquinez Bridge, California, USA

The 1600 ton capacity STUs that were installed on the Carquinez Bridge in 2000 were at that time the highest capacity STUs installed on any bridge in the world (Brown, 1996) (see Chapter 5 the for a full description). The installation programme was originally scheduled to start at pier 5, sited on the western end of the bridge, and then to fix the STUs to the expansion joint ends at the eastern end. However, due to other factors in the construction sequence, the STUs for the eastern end expansion joints were installed first.

Due to their size (2.9 m long by 0.6 m diameter), these STUs were not simple to install. Two STUs each were installed on pier 5 and at each expansion joint of the two suspended spans. The STUs were transported to the site by truck for installation during the evening when the temperature was low and the opening between the connection points was the largest. As part of the retrofit contract, the contractor had to fabricate the brackets to be placed at the ends of the bottom chords where the STUs were to be installed (Figure 11.8). The sequence of installation of an STU at the expansion joint of the suspended span of the Carquinez Bridge is shown in Figures 11.9 to 11.13.

**Figure 11.8** Fabricated brackets at the end of the bottom chords of the steel girder of the Carquinez Bridge

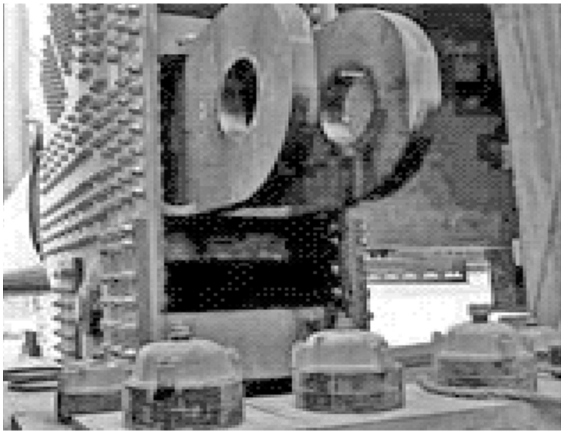


Figure 11.9 The STU is lifted using eye bolts threaded into tapped holes in the end caps of each cylinder

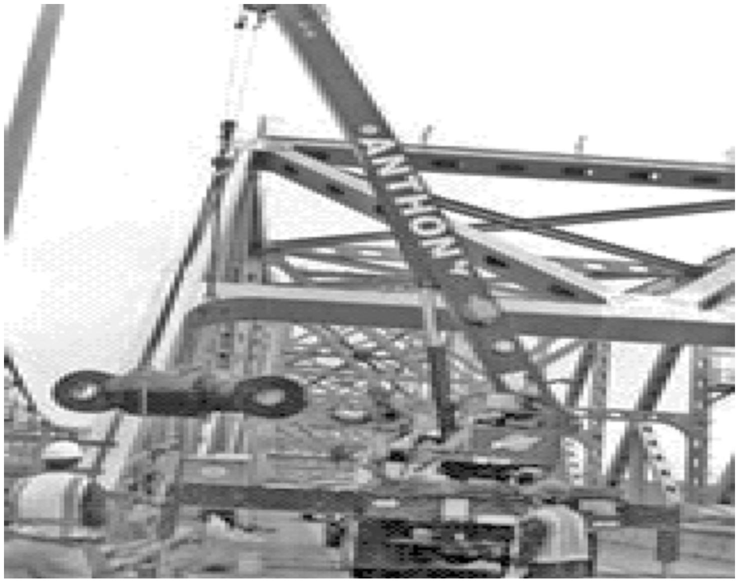



Figure 11.10 The STU is lowered down into the recess in the bottom chord to be attached to the fabricated brackets

**Figure 11.11** The STU is positioned to align with the brackets for connection

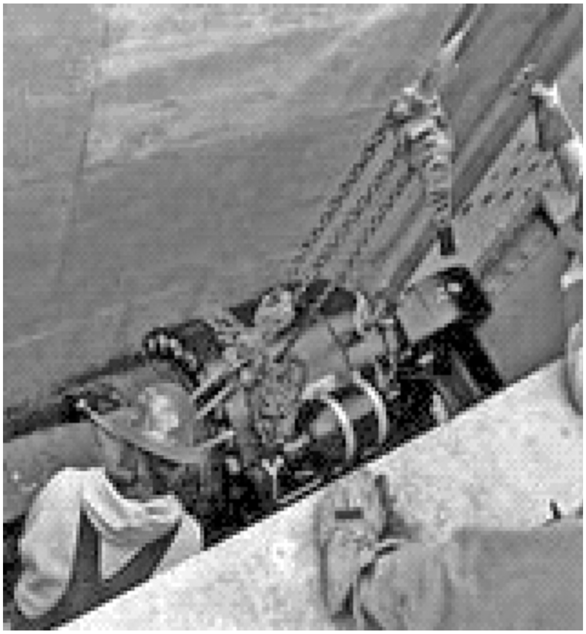


**Figure 11.12** The STU is connected using a stainless steel clevis pin

Figure 11.13 The STU installed at the eastern expansion joint of the suspended span of the Carquinez Bridge

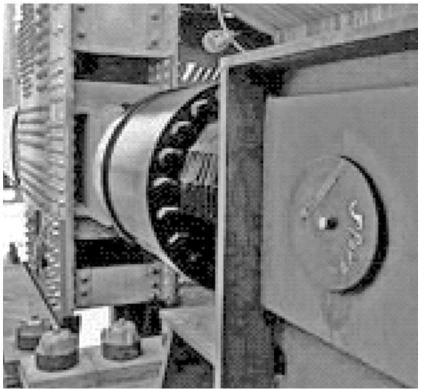
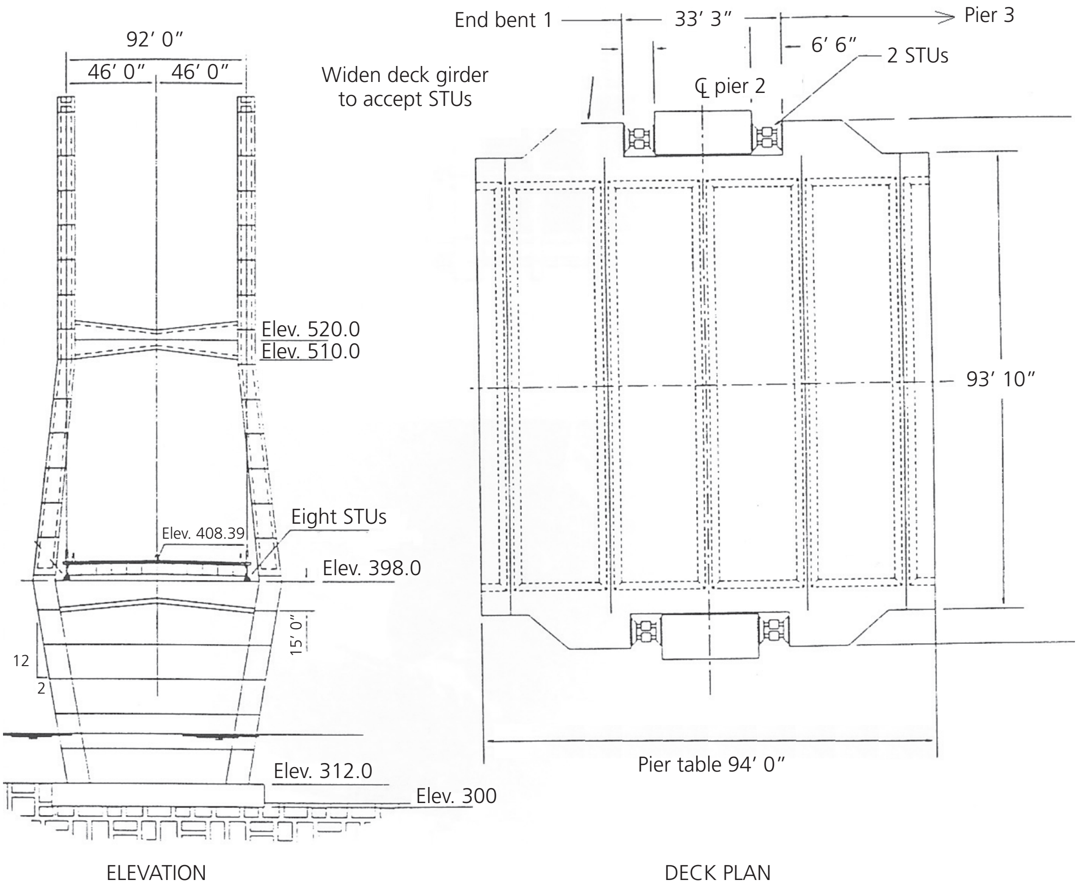


Figure 11.14 Installation of STUs on the Cape Girardeau Bridge



## 11.8. Installation of STUs on cable-stayed and suspension bridges

STUs for suspension and cable-stayed bridges are mounted between the tower and the deck (Figure 11.14) in order to distribute the load throughout the deck as much as possible. While STUs can be manufactured to handle a very large capacity, they are normally used in series and/or parallel so that the load is distributed throughout the structure and no particular element is overloaded.

The Cape Girardeau Bridge in Missouri is a 1150 ft. long cable-stayed bridge crossing the Mississippi River, which has a main channel width of 850 feet. The bridge, designed by Howard, Needles, Tammen and Bergendoff, was originally designed to be unrestrained at the towers. However, in order to restrain seismic movements while at the same time maintaining the original design concept of freeing the towers from thermal loads, sixteen 375 ton STUs were specified to be installed between the span and the towers and eight 420 ton STUs were specified for the bent.

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# Chapter 12
# Cost-effectiveness of STUs

## 12.1. Introduction

Over the last 100 years the structural configuration of long bridge crossings has been governed by optimisation of the bridge spans selected. For low-profile bridge crossings, simple span configurations have generally provided economical solutions, resulting in double bearings, moving and fixed, at each pier. In this type of arrangement, longitudinal forces are resisted by the fixed bearings and longitudinal displacements are allowed to occur at the moving bearings, with rotational flexibility provided for both. In longer structures, supported on tall and flexible piers, the superstructure is generally fixed at the piers, which allow movement due to thermal and other effects, accompanied by horizontal shears resulting from the restraining action of the supports. In long suspended structures, expansion joints are provided at the towers to relieve them of thermal loads at deck level or, if continuity is required, as in the case of cable-stayed bridges, the towers are designed to resist thermal and creep loads from the suspended structure. The introduction of STUs for use on such bridge crossings has provided immense advantages in terms of the capital costs, maintenance costs and riding comfort of such structures.

The use of STUs has been found to be cost-effective for both new bridges and existing bridges. In new bridges, load sharing between the moving bearing piers and the fixed bearing piers, achieved by means of STUs, results in smaller substructure elements and foundations, thereby saving considerable amounts of money. The strengthening of existing bridges by means of STUs provides an economically more feasible solution than the use of conventional structural methods. The introduction of revised codes for increased seismic loading and longitudinal braking loading due to vehicles, and the subsequent structural design assessment often dictates that the substructure be upgraded to cater for the increased loading. This upgrading can be economically better achieved by the installation of STUs compared with the conventional method of strengthening of the piers and foundations by increasing their size. In addition, an existing bridge to be retrofitted with STUs can remain open to traffic while the STUs are installed. This aspect alone, of keeping bridges open to traffic while installing STUs, makes this method cost-effective compared with strengthening bridges by means of conventional methods, which would normally require the bridge to be closed to traffic.

The study of recent earthquakes around the world has provided better knowledge of the magnitude of the seismic forces that must be resisted by bridge structures. Unfortunately, these forces are much higher than was assumed in the past, and this has necessitated an upgrading of earthquake zones. As a result of this upgrading, in a lot of countries significant retrofitting of many bridges is required to reduce the possibility of catastrophic collapse and loss of life. Because of the very large number of bridges involved, it is imperative that this retrofitting be done in a cost-effective manner. This can be achieved by using STUs, which provide a very economical means of retrofitting existing bridges, without closing the bridge to traffic.

As explained in Chapter 3, Section 3.6, the installation of STUs at the expansion joints of a five-span simply supported bridge can result, in the case of new bridge, in a significant reduction in the size of the pier and fixed abutment and foundations such that the final design capacity is only some 20% of that required originally. In the case of an existing five-span simply supported bridge, retrofitting with STUs allows up to a five-fold increase in the originally designed traction and braking forces, resulting in considerable cost savings as well as providing enhanced stability to the bridge deck against earthquake shock.

## 12.2. Second Bassein Creek Bridge, Mumbai, India

The Second Bassein Creek Bridge (see Chapter 4) is a multi-span continuous bridge on which the installation of STUs saved time and money in the construction of the caisson foundations (Deshpande *et al.*, 2002). It was realised during the preliminary design that the construction of any caisson greater than 12.5 m outer diameter (o.d.) would be very hard, risky and costly due to the prevailing difficult site conditions. The introduction of STUs in the design made it possible to revise the original 16.5 m o.d. caisson for the fixed pier and 11.5 m o.d. caisson for each of the adjacent two piers with moving bearings, to 12.5 m o.d caissons for all the three piers (Figure 12.1).

This resulted in a considerable cost saving (Table 12.1). The saving in the cost of the caisson foundations for the final design of the caissons for piers P3, P4 and P5, made possible due to the use of STUs, was 99.6 million – 78.89 million = IR 19.71 million, which is equivalent to US$ 410 000 (Table 12.2). This is a saving of about 20% in the cost of the construction of the three pier foundations. However, the cost of the STUs must be set against this saving.

The cost of supplying, testing, transporting and installing the two STUs, according to the rate in the bill of quantities, was IR 8.269 million, which is equivalent to US$ 172 000. The actual purchase cost of the two 3100 kN capacity STUs was IR 4.8 million, which is equivalent to US$ 100 000, which included full-scale testing and transportation by ship to India, but excluded installation, overheads and profit for the contractor. There is normally a 40% custom duty on cost, insurance and freight (CIF) value, and a 16% countervailing duty and valorem on CIF and custom duty payable to Indian Customs. However, as the bridge project was financed by the World Bank, the duties payable were exempted, as the STUs were not locally available and had to be imported.

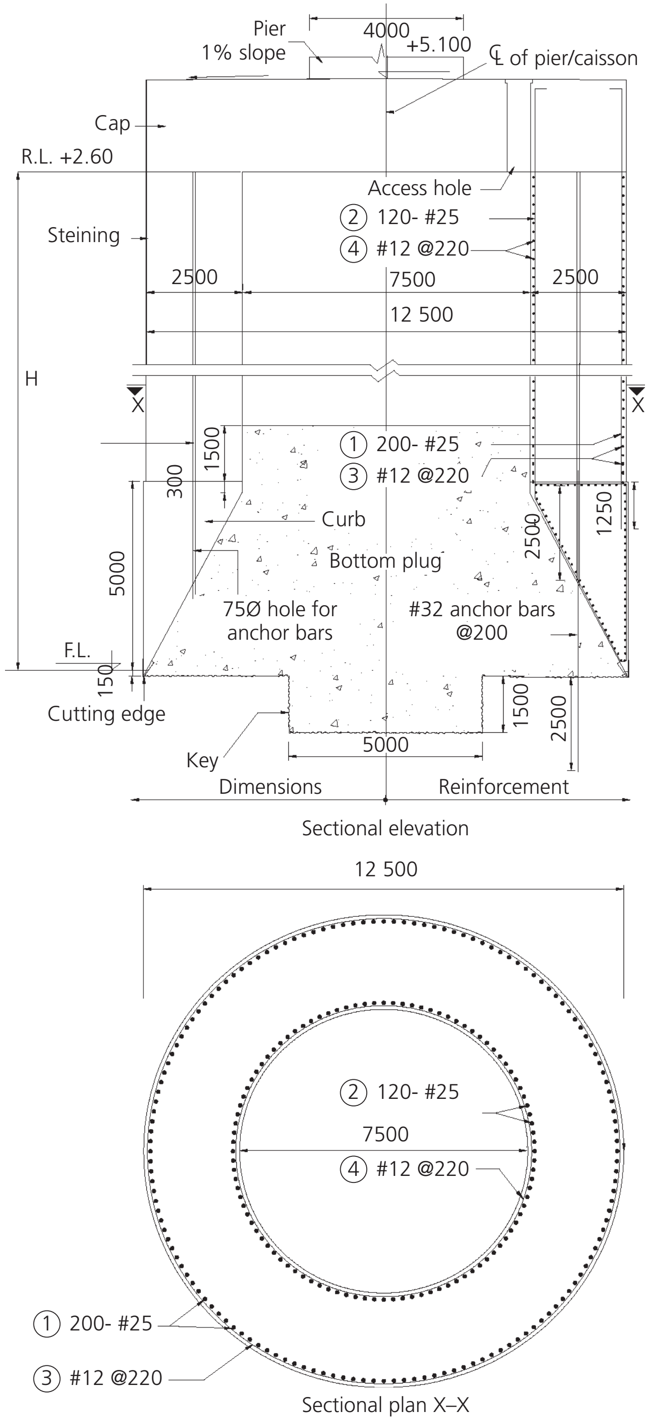
Therefore, the net saving in the cost of the caisson foundations was IR 11.441 million, which is equivalent to US$ 238 000. The total cost of the bridge, including the two approaches, was IR 330 million, equivalent to US$ 6.875 million, and so the saving generated by the use of STUs was about 3.46% of the total cost of the bridge construction project.

Some case studies illustrating the cost-effectiveness of using STUs are given in the following sections.

## 12.3. Paksey Bridge, Bangladesh

This bridge (described in detail in Chapter 4) is an 1800 m long continuous bridge, and is one of the longest continuous concrete bridges in the world. It was elected to strengthen the piers on this bridge by installing STUs, rather than by using conventional strengthening methods. Due to the extreme length of the Paksey Bridge, the largest STU, having an ultimate limit state (ULS) capacity of 11 500 kN, needed to have a total movement of 750 mm, and this resulted in a 3530 mm long device. This is a 17 span continuous concrete bridge, with 16 piers and two abutments. The fixed bearing is located on pier 8, and all the other piers, which have moving bearings, had one STU installed, except for piers 1 and 16, which required two STUs in parallel, each having a ULS capacity

**Figure 12.1** Details of the 12.5 m o.d. caisson




Details of 12.5 m o.d. caisson

Table 12.1 The cost of materials and sinking for the Second Bassein Creek Bridge

| Caisson no. | External diameter: m | Internal diameter: m | Average foundation depth: m | Material cost: IR million | Sinking cost: IR million | Total cost: IR million |
|---|---|---|---|---|---|---|
| P1, P2, P6, P7 | 10.5 | 6.5 | 31.92 | 11.01 | 3.5 | 14.51 |
| P3, P4, P5 | 12.5 | 7.5 | 33.76 | 16.84 | 9.79 | 26.63 |
| P4 | 16.5 | 9.5 | 33.39 | 32.70 | 25.10 | 57.80 |
| P3, P5 | 11.5 | 7.0 | 33.94 | 14.20 | 6.70 | 20.90 |

* Costs for the 10.5 m o.d. and 12.5 m o.d. caissons are from the tender documents (Patel, 2004). The costs for the 11.50 m o.d. and 16.50 m o.d. caissons were evaluated using the values given in the bill of quantities

of 11,500 kN and 750 mm movement. If the strengthening had been done by conventional means, i.e. the application of additional steel reinforcement and concrete, it would have added a tremendous cost to the structure. The STU carries a moderate price tag by comparison, and the savings on the Paksey Bridge project through using these devices were estimated to be of the order of ten times the cost of the STUs.

## 12.4. Carquinez Bridge, California, USA

The STUs provided by TechStar/Alga for the Carquinez Bridge (described in detail in Chapter 5) are unique, not only because of their immense size and design capacity, but also because they represent a tremendous cost saving compared with conventional structural strengthening. Not using STUs would have either made the structure continuous for both thermal loads and earthquake loads, or would have made it discontinuous for both these loads. The first option would have required very large expansion joints at the bridge ends, and very high thermal loads on the piers. The second option would have required very large expansion joints within the bridge, and would have meant that the bridge piers would be subjected to very high earthquake loads. The large expansion joints that would have been required are expensive, and difficult to build and maintain, and the corresponding retrofit of the bridge piers would have been complex. It is estimated that the additional design and construction costs associated with a conventional retrofit would have increased the project costs significantly over the cost of the STUs, including their design, manufacture and testing.

## 12.5. Cable-stayed and suspension bridges

Today, bridge engineers working in seismic zones and heavy wind storm areas are using STUs, also known as lock-up devices (LUDs), on both steel and concrete girder cable-stayed bridges as well as

Table 12.2 Cost saving due to the use of STUs on the Second Bassein Creek Bridge

| Preliminary design | | | | Final design | | |
|---|---|---|---|---|---|---|
| Caisson no. | P4 | P3 | P5 | P4 | P3 | P5 |
| External diameter: m | 16.5 | 11.5 | 11.5 | 12.5 | 12.5 | 12.5 |
| Cost: IR million | 57.8 | 20.90 | 20.90 | 26.63 | 26.63 | 26.63 |
| Total cost: IR million | 99.6 | | | 79.89 | | |

Figure 12.2 The Maysville cable-stayed bridge

suspension bridges. The use of these devices eliminates the requirement to rigidly connect the deck to one of the towers, and thereby generates considerable savings in the design of the tower. The STUs are placed between the tower and the deck so as to distribute the load as much as possible throughout the deck.

The Owensboro Bridge, which crosses the Ohio River in Kentucky, is a concrete cable-stayed bridge designed by DRC Consultants. It has a 1200 ft main span and two 485 ft side spans. It is designed such that shrinkage, creep and thermal deformations can take place in the girder without preventing the towers from resisting longitudinal seismic forces. Two STUs were specified for the design to provide the required type of connection between the girder and each tower. Savings were thus achieved by not having to design the towers to carry additional loads due to the creep, shrinkage and temperature effects on the concrete girders.

Another example of a cable-stayed bridge where considerable cost savings were made by using STUs in the design of a new structure is the Maysville Bridge in Ohio, USA (described in detail in Chapter 7). The STUs were used to control the longitudinal displacement of the bridge in the event of an earthquake (Figure 12.2).

STUs are also used on suspension bridges just in the same way and for the same reasons that they are used on cable-stayed bridges. There are many cable-stayed and suspension bridges throughout the world that have STUs installed. An example of a suspension bridges with STUs is the Storebaelt Suspension Bridge, Denmark, described in Chapter 7.

## 12.6. Railway bridges over rivers in Indonesia

The railway bridges over rivers in Indonesia are described in detail in Chapter 6. It was necessary that the piers in the river be constructed in a short period of 4–5 months during the summer, as it would be very expensive to build these piers and their foundations in the monsoon season when the rivers would be flowing full. This was achieved by installing STUs on each span where there was a free bearing on the simply supported truss railway bridge.

The STUs tie all the spans of the normally simply supported steel trusses together. They allow the thermal expansion and contraction of the individual spans, but upon receiving a shock from an earthquake or from braking and locomotive traction, the units ‘lock’, making the whole length of the bridge act as a tie or a strut. The superstructure and the substructure thus move as a coupled unit. The piers, which all have the same cross-section, deflect by the same amount. The result is that the shock loading is transferred from the tall flexible piers located in the river to the short stiff piers located on the river banks. This considerably reduces the lateral force on the middle piers, which therefore require smaller foundations. Not only did this design enable the construction of the in-river piers and foundations to be completed during the dry season, it also led to considerable costsavings in the construction of the piers and the foundations as a whole. The design of these simply supported bridges with STUs provides a very good solution for improving the resistance to dynamic loads and the stability of the bridges, together with shortening the construction time and providing economical foundations for the bridges.

## 12.7. Conclusions

STUs are a viable and economical means of retrofitting existing structures and designing new structures for dynamically applied loads. They offer the opportunity to upgrade existing structures with virtually no impact on existing traffic conditions. Often, it is this traffic impact that is the largest cost associated with structure rehabilitation.

The application of STUs not only decreased the cost of the Second Bassein Creek Bridge by about 3.46% but, above all, made the caisson foundations for the bridge manageable by reducing the size of the caisson for pier 4 from 16.5 m o.d. to 12.5 m o.d. The bridge contract was to be completed in 3 years but the contractor experienced a 3-month delay due to various problems in the construction of 12.5 m o.d. caissons. It can be concluded that the contractor would have experienced great difficulty in constructing 16.5 m o.d. caissons under the difficult site conditions, and would have been delayed much longer.

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# Chapter 13
# Load testing of STUs

## 13.1. Introduction

STUs are not used as often as bearings on bridges, but they must be load tested, before use, more exhaustively than bearings, as they remain a proprietary item. Unfortunately, load testing of STUs is expensive, as it requires a special testing rig for dynamic load testing. As STUs are manufactured in only a few countries, they may be imported into countries where the special testing rig required is not available and there are no testing engineers having the necessary knowledge required to undertake the tests. In such instances, load testing of STUs is even more expensive, as the devices have to be load tested in the country of manufacture. Sometimes, there is a problem even in countries where STUs are manufactured if the capacity of the STU required is exceptionally high, such as the 1800 ton capacity STUs used on the Carquinez Bridge in California, USA. No laboratory in the USA had the physical capacity to carry out such large-scale dynamic force testing. Even the two CTL Group laboratories in Skokie, Illinois, and San Diego, California, the two largest structural laboratories with shock load experience in the USA, had insufficient hydraulics and testing rig capacity to handling these giant STUs. No manufacturer had the in-house capability either. These STUs had to be tested at the Alga laboratory outside Milan, Italy, which has a 50 000 kN capacity bi-axial press and modern high-power 250 kW equipment for dynamic tests and seismic simulations.

## 13.2. 2002 AASHTO interim specification for STUs

Until 2002 there was no specification for STUs in any design code in the world. In the absence of a standard specification, each manufacturer and quasi-governmental testing agency attempted to push their own proprietary specifications for STUs, which were written in such a way as to restrict competition between manufacturers or to promote the use of a particular testing agency. Thus users were thoroughly confused, and felt that they were being misled by the manufacturers in order to save on tendered testing costs, which are generally significant in comparison with the cost of the STU itself. However, in the absence of a global specification and a lack of technical literature, users had no alternative but to accept the advice of the manufacturer with regard to load testing. Unfortunately, none of this was promoting the best interests of the STU industry. In fact, these ‘hocus pocus’ specifications and associated testing protocols were damaging the STU industry, which was anyway being ignored by engineers in many countries due to their conservative attitude. What was needed, in the interests of both the STU industry and users, was for a major design code agency to establish a uniform specification, based on product performance, material structural integrity, and realistic testing expectations. At last, in 2002, the American Association of State Highway and Transportation Officials (AASHTO) included Section 32: ‘Interim Specification for Shock Transmission Units’, in its *LFRD Bridge Construction Specifications*, based on testing and evaluation carried out by the Highway Innovative Technology Center (HITEC).

In 1996 HITEC convened a panel of experts to develop an evaluation plan for STUs which could be used as a basis to demonstrate the capabilities of STUs with regard to their application to highway and railway bridges. The intention of the programme was to demonstrate that STUs will perform as required during their service life and adequately cope with the extremes of temperature (−40°C to +50°C) that can be anticipated in the USA and other countries around the world. The plan consisted of laboratory tests, design procedures and recommendations for potential users, and detailed manuals of instruction for the installation, maintenance and inspection of STUs over the lifespan of a bridge (i.e. 75 years or more).

Five specific tests were conducted.

- Seal wear.
- Cyclic load.
- Drag load.
- Overload.
- Fatigue load.

Testing criteria were established for each test by which potential users can evaluate their installations to ensure that the devices provided meet the intended design criteria. However, to provide confidence to potential users that STUs will, in fact, meet these criteria, the HITEC panel established the need for advanced testing of four STUs with rated load capacities approximating lower (50 kilopounds (kip)), middle (150 and 300 kip) and upper (400 kip) potential dynamic loading limits. The panel decided that the same STUs should be tested for all the conditions and at all the extreme conditions anticipated. This represented a very severe set of test conditions, and exceed the actual conditions that any STU might be subjected to in the field.

The above test programme was carried out satisfactorily and it gave assurance that the product specified would provide the necessary force resistance and/or redistribution to ensure adequate dynamic load resistance for the extreme event design loadings. The panel of experts from HITEC submitted an evaluation report on the testing of the STUs for discussion to AASHTO for the purpose of developing a specification for STUs. This process finally resulted in the AASHTO 2002 'Interim Specifications for Shock Transmission Units'. In 2006, the interim AASHTO specification, released in 2002, was approved and revised within Section 32 of the AASHTO *LFRD Bridge Construction Specifications*.

The interim 2002 AASHTO specification provided users with information on the preparation of working drawings, material requirements, installation, inspection and maintenance procedures, and packaging, handling and storage requirements, together with testing and acceptance criteria.

There is a wide variation in the properties of the internal medium, generally silicon oil or putty, used by the manufacturers of STUs, and generally this medium is proprietary in nature. Users are advised to obtain detailed information on the properties of the medium, bearing in mind the variation in temperature expected on site, and to make sure that the medium remains inert over the specified temperature range and that its properties do not change over time. The other important component that can make or break an STU is the seal used to prevent leakage of the internal medium from the cylinder. Again this is a proprietary item, and users must ensure, by means of testing for seal wear and other load tests, that the internal medium will not leak out for the specified life of the STU.

According to Clause 32.4 of AASHTO Section 32 – Testing and Acceptance, three types of tests must be performed on an STU.

- Prequalification tests (system characterisation tests).
- Prototype tests.
- Proof tests (quality control tests).

### 13.2.1 Prequalification tests

The interim 2002 AASHTO specification required manufacturers to have prequalification tests carried out on their STUs. These tests are used to establish the properties of new types of STU units and substantially different versions of an existing unit. It is stated that, at a minimum, the following five prequalification tests shall be conducted according to the testing guidelines developed by the HITEC evaluation panel.

- Seal wear test.
- Cyclical load test.
- Drag load test.
- Overload test.
- Fatigue load test.

The guidelines for the prequalification tests are given in the following sections.

#### 13.2.1.1 Seal wear test

This test shall be performed to ensure that the seal will withstand movement over an assumed design life without leakage of the internal fluid or putty.

Each assembled STU, but without internal fluid or putty in the cylinder, shall undergo pushing and pulling of the piston through a travel distance of $\pm 2$ in. (total 4 in.) for 10 000 cycles, at each of the extreme test temperatures. At the end of this cycling, the STU shall be reassembled with the fluid or putty in place, and the rated load, $P_r$, shall be applied and maintained until the piston reaches its limit of travel, or a period of 2 minutes, whichever occurs first. This part of the seal wear test can be conducted at ambient temperature.

The STU shall maintain its rated capacity without leakage of the fluid over the test period. The amount of leakage of the fluid or putty through the seal, and the length of time of application of the rated load, are to be reported for each test. The total distance travelled by the piston (number of cycles $\times$ strokes per cycle) over the test period shall also be reported.

#### 13.2.1.2 Cyclical load test

This test shall be performed to ensure that the STU will function as designed during a simulated application of cyclic loadings. The load is to be applied for a specified number of cycles to simulate an earthquake.

After the seal wear test has been completed, the rated load, $P_r$, shall be applied to the same STU in a cyclic manner (sinusoidally, from $+P_r$ to $-P_r$) for 10 cycles, at each of the frequencies given in Table 13.1 and each of the test temperatures.

The STU shall reach its rated capacity for each cycle, and without leakage of the fluid medium. The results of the cyclic load tests shall be plotted as ‘elapsed time’ against ‘applied load’ and, on the

Table 13.1 Test frequencies

| STU with minimum rated capacity | STU with maximum rated capacity |
|---|---|
| 0.5 Hz | 0.1 Hz |
| 1.0 Hz | 0.25 Hz |
| 2.0 Hz | 0.5 Hz |
| 4.0 Hz | 1.0 Hz |

same plot, as ‘elapsed time’ against ‘total piston movement’, for each STU and each test temperature and frequency specified. The total movement of the piston shall be reported for each cycle (load–displacement curves are required). The amount of leakage of the internal fluid medium through the seal, if any, shall also be reported for each test.

#### 13.2.1.3 Drag load test

This test shall be performed to determine the load applied to the structure and its anchorages by the STU as it moves through simulated normal daily temperature cycles.

After completion of the cyclic load tests, and with one end of the STU anchored, forces of 2%, 5% and 10% of the rated load, $P_r$, shall be applied at each test temperature, and the rate of movement of the STU shall be recorded. Alternatively, the STU shall be loaded at the specified rate of movement of the structure, and the drag force resistance shall be reported.

For prequalification testing to be acceptable, the rate of movement of the STU shall equal or exceed the minimum specified for the project under consideration, and the load resistance shall be less than the maximum allowable for the project under consideration.

Resistance of an STU can be varied by the manufacturer for specific rates of movement if the site-specific requirements indicate that the load/rate interaction obtained from these tests is not acceptable. The minimum practical drag force is 2% of the rated load, $P_r$. If the rate of movement of the bridge is not known, a default value of 0.5 in./h may be used.

#### 13.2.1.4 Overload test

The purpose of this test is to ensure that the STU will perform properly should the rated capacity be exceeded while it is in service.

After the conclusion of the drag force test, the STU shall be loaded three times to an overload of 1.5 times the rated load, $P_r$, at a rate fast enough to make it ‘lock up’, and the load shall be held each time at $1.5P_r$ for 30 s. This test can be conducted at ambient temperature.

A graph of ‘elapsed time against ‘applied load’ for the time period for which the overload is applied (30 s minimum) shall be plotted. Leakage, if any, shall also be recorded.

#### 13.2.1.5 Fatigue load test

The purpose of this test is to determine if the STU can withstand many cycles of loads as could occur from braking and/or traction forces on a railroad or a highway bridge that experiences high vehicle braking actions.

After completion of the overload test, the rated load, $P_r$, shall be applied to the STU, cycling the load at least 100 000 times at any desired frequency. This test can be conducted at ambient temperature. Leakage, if any, shall also be recorded.

The HITEC panel of experts determined that a worst-case scenario for service loading of STUs is an application of braking loads equal to the lock-up load four times a day, for the AASHTO specified design life of 75 years. This is roughly equivalent to 100 000 load cycles (4 cycles/day $\times$ 365 days/year $\times$ 75-year service life = 109 500; say, 100 000). If the project design requirements do not include braking actions of this frequency, the number of cycles should be adjusted, or, if fatigue is not a consideration for earthquake or wind gust application, this test can be waived at the discretion of the engineer.

### 13.2.2 Prototype tests

The objective of these tests is to evaluate the performance of STUs for two design conditions consisting of slow movements that will not lock-up the device, and fast movements that will lock-up the device, within temperature and loading conditions at least equal to those at the project site. All testing shall be performed at an independent testing laboratory approved by the engineer. All testing shall be performed in the presence of the engineer, unless otherwise approved in writing by the engineer. These tests can be carried out at ambient temperature. Prototype tests are necessary when a number of different capacity STUs are required for a project. The following six prototype tests shall be performed on at least one STU of each type in accordance with the AASHTO specification, Section 32, Clauses 32.4.2.2 to 32.4.2.7.

- Hydrostatic pressure test.
- Slow movement test (thermal).
- Fast movement test.
- Simulated dynamic test.
- Overload test.
- Fatigue load test.

### 13.2.3 Proof tests (quality control)

The following three proof tests shall be performed on every STU that is going to be incorporated in the structure. All testing shall be performed at an independent testing laboratory approved by the engineer. All testing shall be performed in the presence of the engineer, unless otherwise approved in writing by the engineer. These tests can be carried out at ambient temperature in accordance with the AASHTO specification, Section 32, Clauses 32.4.3.2, to 32.4.3.4.

- Hydrostatic pressure test.
- Slow movement test (thermal).
- Fast movement test.

## 13.3. Second Bassein Creek Bridge, Mumbai, India

Colebrand Ltd, the manufacturer of the STUs used on the Second Bassein Creek Bridge, was pre-qualified as its STUs had been evaluated by HITEC successfully. It was therefore necessary to have proof testing for each of the two STUs. In the absence of any specification at that time, the supervising engineers prepared the specification for testing the STUs based on the specification for the STUs used on the Arthur Laing Bridge, Vancouver, Canada, which were installed as seismic protection. The following three tests were specified (Deshpande *et al.*, 2002).

- Impressed deflection test (drag load test).
- Simulated dynamic force transfer test (fast load test).

- Simulated cyclical force transfer test (simulated dynamic test).

Three engineers, one each from the employer (PWD, Maharashtra), the supervision consultants and the contractor, travelled to Manchester, UK, to supervise the load tests, as this was the first time that imported STUs were going to be used on a bridge in India, and each stakeholder had to be satisfied with regard to the performance of the selected STUs.

The objective of the test schedule was to prove the operation of the STUs against the specified design loading conditions for the seismic and thermal loading expected at the site, and also to establish the performance characteristics. The test requirements, acceptance criteria and results of the three tests, as specified above, are described below.

### 13.3.1 Impressed deflection test (drag load test)

The STU shall have an impressed deflection of 0 to +40 mm to 0 to −40 mm to 0 in not less than 10 h and not more than 24 h. During the impressed deflection cycle the STU shall develop no more drag force than 10% of the maximum design capacity of 3100 kN.

This test was specified to prove that excessive load could not be transmitted to the structure by the STU as it moves through simulated daily temperature cycles. The worst-case scenario would be for the structure to move through its entire movement in one diurnal cycle, that is to say through +40 mm in 24 h, which gives the conservative rate of movement of $40 \times 4 = 160/24 = 6.67$ mm/h or 0.262 in./h. This would be due to the temperature rising and falling to its maximum and minimum, which were determined by referring to meteorological records. In addition, the structure's thermal inertia would be zero, which is, obviously, impossible. Therefore, the test could be deemed to be conservative. The maximum drag load criterion was established by consideration of the corresponding resistance of the bearings. Ten per cent of the rated capacity ($3100 \times 0.1 = 310$ kN) is less than the total resistance of two bridge bearings installed with each STU ($2 \times 525 = 1050$ kN), and therefore if the STU met these criteria it could be regarded as having negligible effect on the structure in normal service.

#### 13.3.1.1 Results

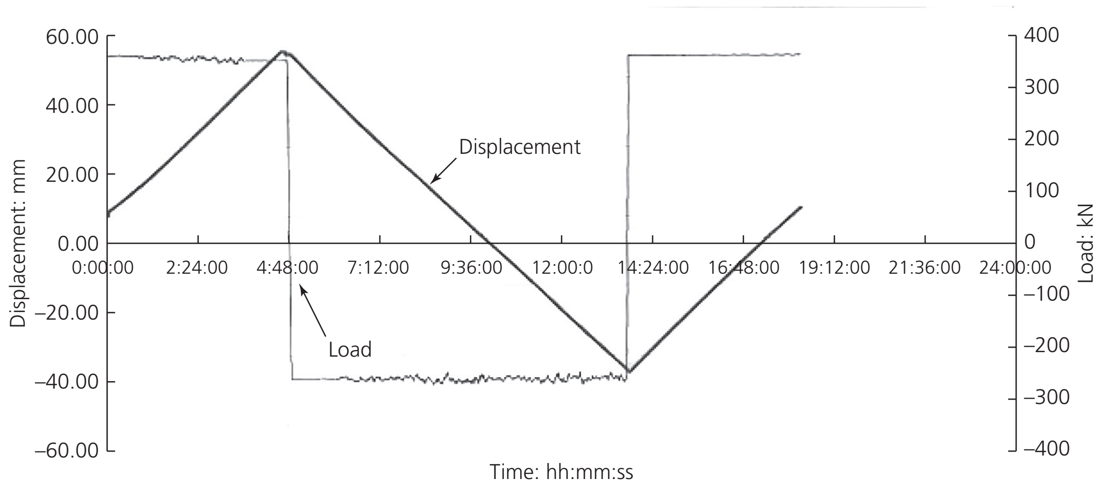
It can be seen from Figure 13.1 that the displacement of the STU piston was constant with a constant resistance, or drag force. The test was accomplished in approximately 18 h. The load trace shows some noise, which is due to the fact that, as the load is so low, the rig is required to work near the bottom of its range. During the final quarter of the test, the rig was periodically manually adjusted, resulting in a smoother plot. The small irregularity at the end of the first quarter is due to being taken up backlash in the rig. The duration of the test exceeded the minimum specified period of 10 h and not more than 24 h. During the cycle, the force required to move the STU was about 300 kN, which is not greater than 10% of its design capacity of +3100 kN.

### 13.3.2 Simulated dynamic force transfer test

The STU shall be loaded in tension from zero to the full design load of 3100 kN in less than 0.50 s and the force sustained for 5 s. The load shall then be reversed to the full design load of 3100 kN in less than 0.50 s and held for 5 s.

The acceptance criteria for this test shall be that the deflection during the loading of the positive force and the negative force shall be no greater than 6 mm, and the deflection during the sustained load portions shall not exceed 3 mm, and there shall be no leakage of the medium.

Figure 13.1 The load and STU displacement recorded during the impressed deflection test of the STUs to be used on the Second Bassein Creek Bridge. © Colebrand International Ltd

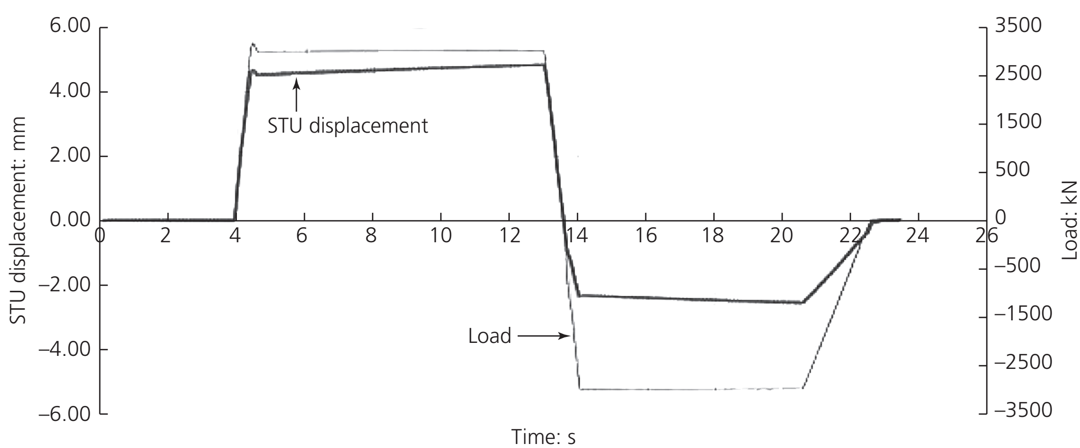



This test was specified to prove the lock-up capability of the device and establish the stiffness under impulse and constant forces. It gives a clear demonstration of any creep under high constant force.

### 13.3.2.1 Results

Figure 13.2 shows the response of the STU to the applied load. The load is applied in less than 0.50 s after about 4 s of logging. The sharp crest at the top of the two traces is an overshoot, which was necessary on this particular test machine to achieve the loading rate of 6200 kN/s. The STU piston response (the displacement trace) follows the load trace exactly. There is no noise on the load trace

Figure 13.2 The load and STU displacement recorded during the simulated dynamic force transfer test of the STUs to be used on the Second Bassein Creek Bridge. © Colebrand International Ltd



recorded during this test because the test machine is working in the middle of its range wherein it is impressively stable. After approximately 13 s of logging, the load was released and applied in the opposite direction, again in less than 0.5 s. The graph obtained is not completely symmetrical, for two reasons. First, due to the vertical test set-up, gravity will ensure that the weight of the test machine's piston and all its fixtures result in the application of a small compressive force on the unit when the rig is at rest. Therefore, when the load is applied there will be a greater displacement in tension than in compression, due to the backlash being taken up. There is also a very small amount of strain in the fixtures that is only possible in tension and a very small amount of free movement in the STU. Second, at the end of the tension loading phase the piston has displaced by a small amount due to creep. Therefore the trace is shifted vertically.

During the compression loading the load trace is not completely straight – a result of the control system reacting to and controlling the rate of load. Again, the STU piston response copies the load trace exactly. A small degree of creep can be seen during the sustained-load portion of the test. This is, however, only 15% of the allowable value of 3 mm.

The maximum displacement recorded as the load was applied in tension was 4.65 mm, which increased to 4.84 mm during the dwell period. The maximum displacement recorded as the load was applied in compression was −2.39 mm, which increased to −2.60 mm during the dwell period.

### 13.3.3 Simulated cyclical force transfer test

The STU shall be tested by applying 50 sinusoidal cycles of load ranging between the maximum (100%) design tension and compression forces (+3100 kN) at a frequency of 1 Hz. There shall be no visible signs of distress or degradation as a result of 50 cycles of loading.

This test was specified to prove that the STU would function as intended during a seismic event. A 1 Hz sinusoidal response of the structure to the ground motion was considered to be acceptable for test purposes. This would result in more than double the load application rate of the simulated dynamic force transfer test and prove the robustness of the STU under the most demanding circumstance that it is possible to test. The duration of the test was considered to be appropriate as it greatly exceeded the likely demand on the STU in service.

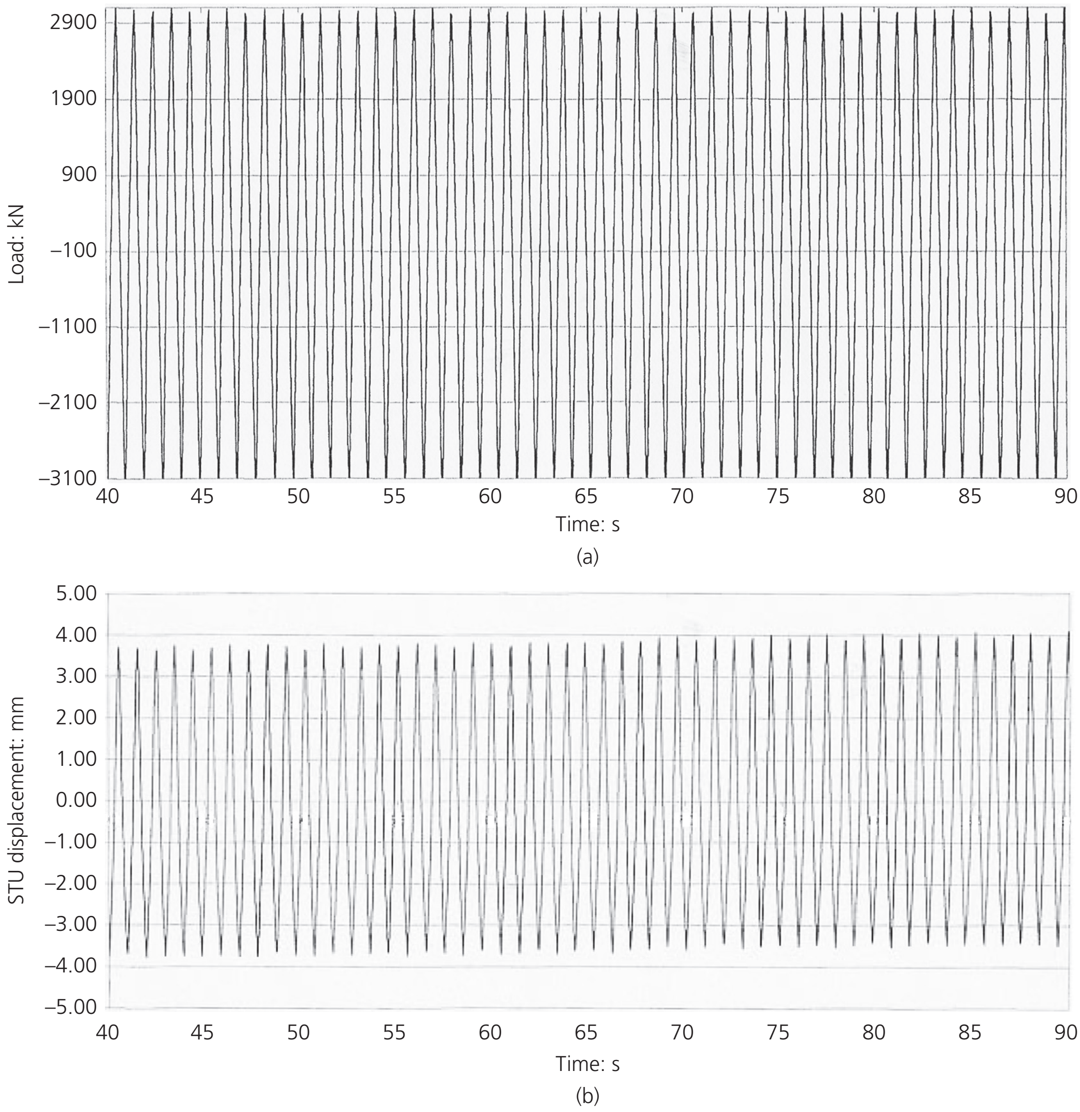
#### 13.3.3.1 Results

The +3100 kN sinusoidal load input is shown on Figure 13.3a. The graph commences at 40 s because the data logging was started before the test machine was switched on. Also, the load was increased from zero to +3100 kN over a few cycles, and these are not included in the plot. The displacement graph is symmetrical in this case because a small constant tension load was superimposed on the sinusoidal load to offset the gravitational effects.

The peak-to-peak displacement measured across the STU (Figure 13.3b) was typically 7.6 mm. No visible signs of distress, leakage or degradation of the STU was observed.

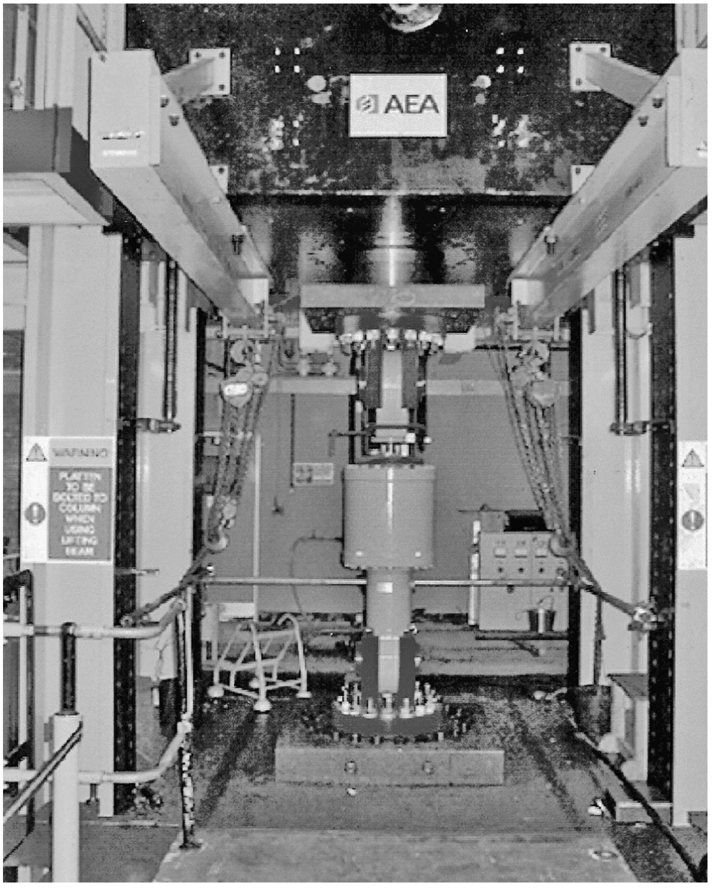
After installation of the STU its performance with regard to slow movements due to temperature, shrinkage, creep, etc. can be verified by measuring the pin-to-pin distance between the brackets holding the STU. The locking up of the STU due to braking of vehicles and traction due to railway trains can be verified by monitoring the STU when such loads are applied on the particular bridge. However, for the Second Bassein Creek Bridge, the STUs are installed for an earthquake force of 3100 kN, and it is only possible to check their performance for this sudden/dynamic load if such a force can be

Figure 13.3 Results of the simulated cyclical force transfer test on the STUs to be used on the Second Bassein Creek Bridge: (a) applied load plotted as a function of time; (b) STU displacement plotted as a function of time. © Colebrand International Ltd




applied to the deck in the horizontal direction. However, the performance and effectiveness of the STU (lock-up device) under the severe performance criteria imposed in the laboratory tests was found to be satisfactory and, as there are no moving parts in the STU apart from the piston, and the patented silicone-putty medium had been thoroughly tested for its performance over a wide range of temperature, the chances of the STU not performing under the cyclical earthquake force could be ruled out. The device exceeded the prescribed parameters without degradation, thus verifying its usefulness as a tool for the effective distribution of forces.

Figure 13.4 Load testing at the AEAT laboratories of the STUs to be used on the Second Bassein Creek Bridge




Colebrand had in the past used the laboratory services of Bodycote Materials Testing Ltd at Daventry, UK, for load testing of its STUs. However, at this laboratory the maximum testing capacity was only 2000 kN. Colebrand, therefore, had to arrange for the testing to be done at the Atomic Energy Authority Technology (AEAT), an independent laboratory that was originally part of the UK Atomic Energy Authority, located at Risley, UK. The AEAT laboratory has a servo-hydraulic fatigue machine (Figure 13.4) manufactured by Schenck, Germany, which has a maximum load testing capacity of 6300 kN and one of the largest test spaces of any machine in the world. In addition to the above two independent laboratories for testing STUs in the UK, there are others located in Europe and America and other countries (see Appendix 2).

## 13.4. Paksey Bridge, Bangladesh

The Paksey Bridge is described in detail in Chapter 4. As the rated capacity of each STU to be used on this bridge was 11 500 kN, which is close to the highest capacity (18 000 kN) STUs installed on the Carquinez Bridge, a high-capacity testing rig was required. The testing of the STUs to be installed on the Paksey Bridge was carried out by Alga at their facility south of Milan, Italy (as for the Carquinez Bridge). The tests were supervised by the engineer's representative and the Major Bridge

Figure 13.5 Test assembly with an STU in place during the test phase for the Paksey Bridge

Engineering Bureau (MBEB). Alga constructed a testing assembly that enabled all tests to be conducted. The full evaluation was over 3 months of continuous testing. The concept involved two actuators, one at the top and one at the bottom of the assembly. The two actuators alternately pushed on the cylinder, so that a cyclical action resulted. Figure 13.5 shows the test assembly and one STU in place during the test. The testing protocol agreed between the engineer, MBEB, and the supplier was similar to the one used for the Carquinez Bridge STUs completed in 2000 for the California Department of Transportation (Caltrans) and in accordance with the AASHTO specification, Section 32.

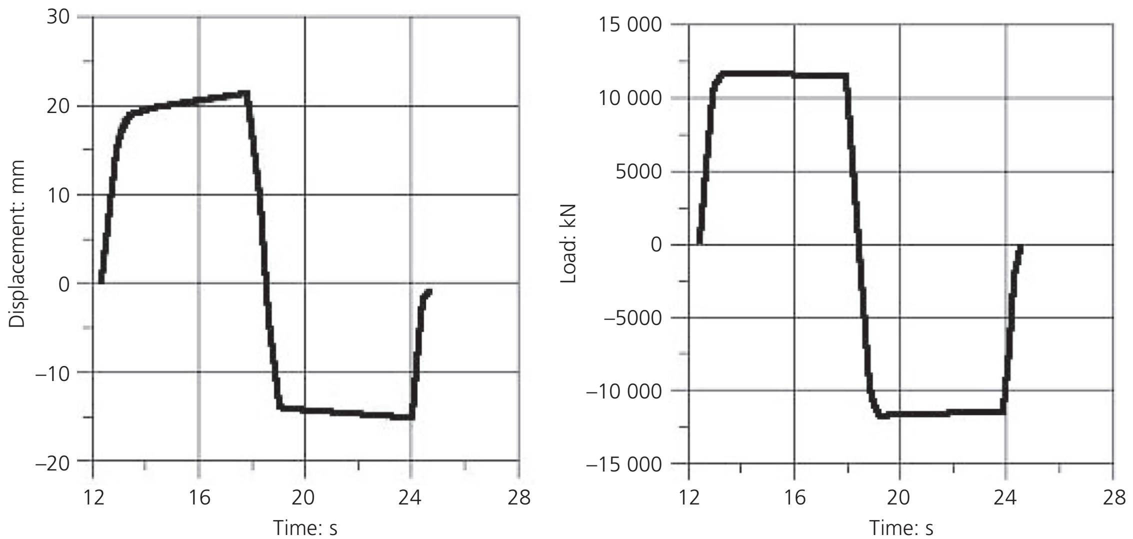
The following three tests were conducted on each STU to be used on the Paksey Bridge.

- Hydrostatic test (ultimate pressure test).
- Shock load test (simulated dynamic force transfer test (SDFTT)).
- Slow velocity test (impressed deflection test).

### 13.4.1 Hydrostatic test (ultimate pressure test)
This test determines whether the STU can be loaded in excess of the operating pressure within the cylinder at the ultimate load. Normally, the cylinder is loaded to 150% of the operating pressure.

### 13.4.2 Shock load test (simulated dynamic force transfer test (SDFTT))
This test determines if the device is capable of holding the ultimate design capacity. The load is applied in the following sequence.

Figure 13.6 Typical output from a simulated dynamic force transfer test for the STUs to be used on the Paksey Bridge




- Apply a tension load 11 500 kN in 0.5 s.
- Maintain the ultimate design load 11 500 kN for 5 s.
- Unload and apply a compression load of 11 500 kN.
- Maintain the ultimate design load (11 500 kN) for 5 s, and then unload.

The movement between the point of zero and the maximum design load, as well as the movement during the sustained load period, is not to exceed 25 mm. Figure 13.6 shows the typical output of a SDFTT.

### 13.4.3 Slow velocity test (impressed deflection test)

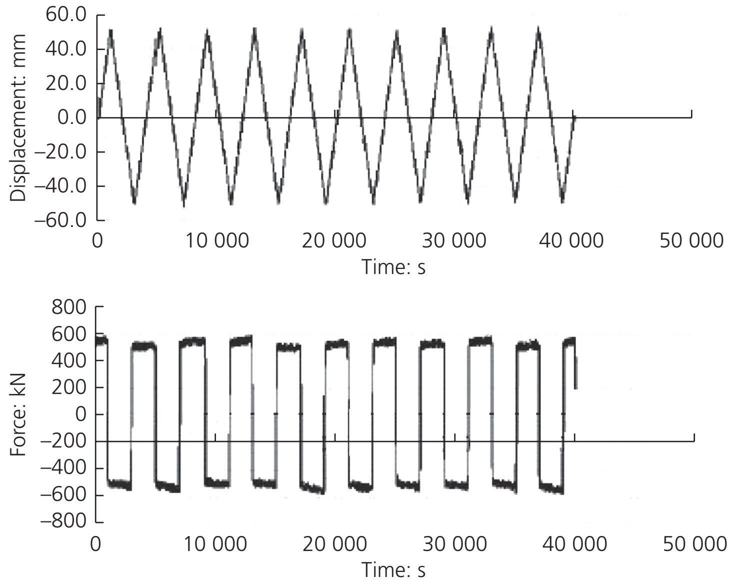
During this test each device was cycled through its full stroke at a rate of between 0.02 mm/s and 0.05 mm/s for 10 cycles in order to prove that the device will move through the entire movement without restriction. The reaction generated did not exceed 10% of the nominal rated force, as stipulated in the Paksey Bridge STU testing protocol. Figure 13.7 shows a typical output from an impressed deflection test.

The devices to be used on the Paksey Bridge were large, and with extremely big movements, and there are few facilities that are able to handle the testing of such devices. The results of the testing of all the devices to be used on the Paksey Bridge showed that the testing protocol was satisfied and all devices rated at the same capacity were within 2% of each other in performance.

## 13.5. Carquinez Bridge, California, USA

The Carquinez Bridge is described in detail in Chapter 5. In choosing TechStar/Alga to supply the 2100 ton ultimate limit state, $\pm 6$ in. stroke STUs, there was an assumption that STUs of this size could be performance tested to comply with the specification requirement of Caltrans. This was no small assumption, as, at the time of contract tender, there was no laboratory in the USA that had

Figure 13.7 Typical output from an impressed deflection test for the STUs to be used on the Paksey Bridge

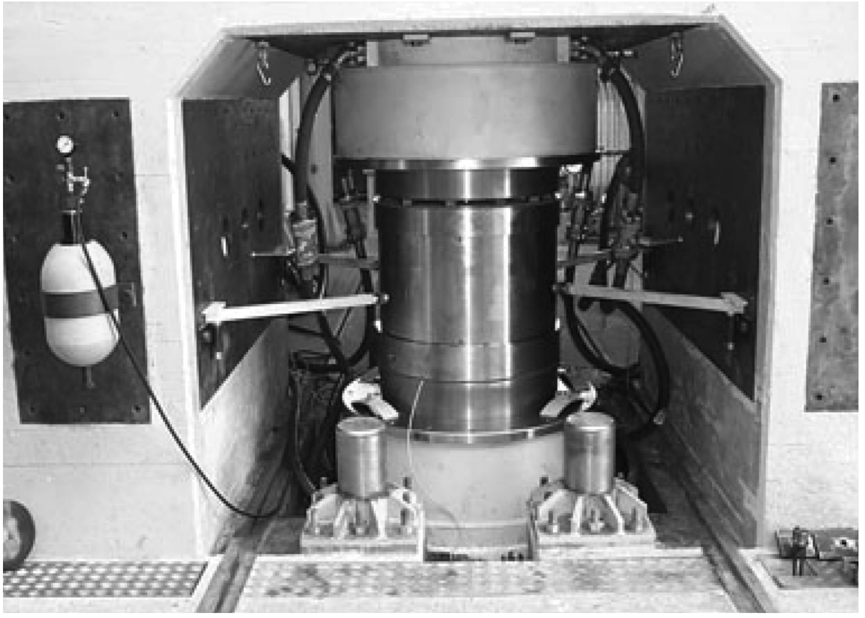



the physical capacity to undertake such large-scale dynamic force testing. Caltrans insisted that each device be fully tested, and supervised the development of an acceptable test plan to be carried out at the Alga laboratory outside Milan, Italy. Additional tests beyond the original scope of the testing specification were added to verify further the expected in-service performance of the bridge. Extensive modifications and improvements had to be made to the testing laboratory to handle the large loads required for these tests. Independent supervision of the load testing was carried out by the University of Pavia, which performed the calibration of the testing rig loads, and the Polytechnical Institute of Milan, which monitored and certified the testing. The total testing costs of the TechStar/Alga proposal were over 50% of the total price of the lock-up device (LUD) supply package to the contractor (Balfour Beatty).

Approval of the TechStar/Alga LUD design took 6 months from the first submission, and required several revisions to satisfy all levels of Caltrans and CH2M Hill. Manufacturing began in early spring 1999, after a Caltrans inspection for quality-control compliance and welding certification. Testing (Figure 13.8) commenced in May 1999 and finished in September 1999. Caltrans and CH2M Hill team members were onsite for the entire 4-month period of manufacturing inspection and testing verification. The testing comprised dynamic shock load testing, high velocity testing, and hydrostatic testing of the STUs, and testing of all the structural connection elements of the devices. Testing approval was not given on site at the laboratory by the inspectors, but only after all tests had been completed, documented, and submitted to Caltrans for review and analysis.

The results of the LUD testing showed that, even on large-stroke, extremely large force STUs, lock-up can be accomplished within less than 0.5 in. total movement (plunge). The performance of each of the STUs met or exceeded every condition stated in the testing requirements. The six STUs to be used were paired in groups of two based on the test results, so that the members of each pair were matched as

Figure 13.8 One of the TechStar/Alga STUs to be used on the Carquinez Bridge within the test rig for shock load testing



closely as possible in terms of their performance when installed side-by-side in the three bridge locations.

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# Chapter 14
# Inspection and maintenance of STUs

## 14.1. Introduction

The present-day STU is a compact and maintenance-free device, with the piston being the only moving part. However, like other civil engineering devices, such as bearings, expansion joints, etc., STUs need to be regularly inspected for wear and tear and leakage of the medium, and should be properly maintained. This is especially important in many developing countries, where maintenance of structural devices is often overlooked, thus reducing the lifespan of a structure. More often it is not the mechanism of the STU that is causing trouble because of the lack of maintenance, but the effect on its body and its attachment hardware of the corrosive environment in which STUs are normally installed, especially on bridges. The lifespan of present-day STUs is about 50–75 years, and in order to have trouble-free performance while they are in service, it is of utmost importance to ensure that the corrosion protection provided is of the highest quality and is periodically touched up in order to keep the STU rust free.

The STU manufacturer is required to provide an inspection and maintenance manual that will include specific instructions to ensure proper inspection and maintenance procedures for the STU while it is in service. The contents of a typical manual (the maintenance and inspection manual supplied by Colebrand Ltd, the manufacturer of the STUs used on the Second Bassein Creek Bridge, Mumbai) are summarised in Sections 14.2 and 14.3.

## 14.2. Inspection

The STUs are protected against corrosion due to environmental factors by a project-specific corrosion protection system. This system must be checked for damage or deterioration, and made good as necessary.

The flexible gaiter material covering the moving end side piston is selected for its durability, but care must be taken not to damage it, especially during the installation of the STUs. The material must be inspected for degradation and, if necessary, replaced with a split gaiter.

The STUs must be checked for leakage of working compound in situ by drawing back the gaiter and inspecting the area where the piston rod enters the body of the STU, and by removing the blanking plug from the fixed end of the STU and inspecting inside the threaded hole. If leakage has occurred it will be evident as a high-viscosity material exuding from around the piston rod, and, in this case of the Colebrand STUs, the manufacturer must be contacted immediately.

The midstroke pin-to-pin distance at the installation is arrived at from the maximum movement specified in the design. During normal service the pin-to-pin distance will vary, but it must not be greater than the pin-to-pin distance set at installation plus the maximum design movement specified,

Figure 14.1 Measuring the pin-to-pin distance of an STU to on the Second Bassein Creek Bridge

or less than the pin-to-pin distance minus the maximum design movement. The pin-to-pin distance must be measured, as shown in Figure 14.1, during every inspection, and any difference in this distance from the previous inspection is proof that the STU is moving with the deck and is functioning, but if the pin-to-pin distance is outside the limits stated above, the STU may have been damaged and the manufacturer must be contacted.

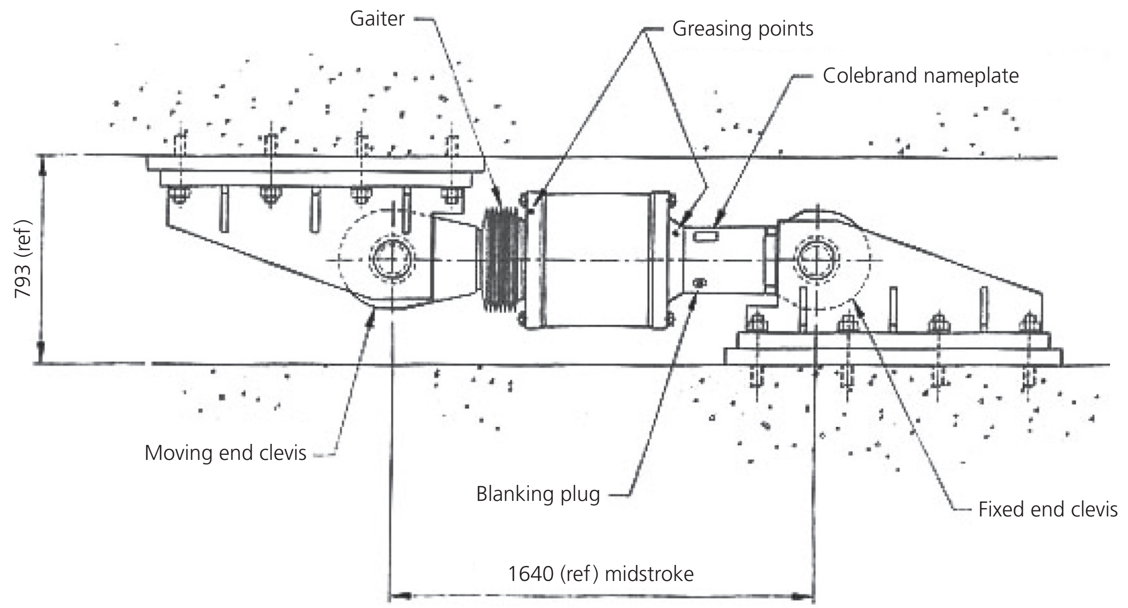
The STU clevis pin locking nut and the nut for the anchor bars must be checked and, if necessary, tightened. The STU must be inspected for any unexpected change in alignment or condition that would indicate that the STU has undergone an unexpected force application.

In general, the STU will not require maintenance relating to its dynamic performance if the rated load, stroke and fatigue rating have not been exceeded.

## 14.3. Maintenance

The parts of the STU must be properly lubricated. The piston rod under the gaiter should be greased by loosening one clip and, holding the gaiter in position, pulling it back. The moving end bush should be greased at the moving end through a grease nipple mounted on the circumference of the STU end

Figure 14.2 A diagram showing the inspection holes and greasing points. © Colebrand International Ltd




flange. The fixed end bush should be greased through a nipple mounted on the tapered portion of the end flange. The clevis pins should be smeared with a light coating of grease over their entire length before being inserted, and grease should also be brushed liberally over the spherical bearings with the pins in situ.

A diagram is generally included in the inspection and maintenance manual supplied by the manufacturer showing the inspection holes, greasing points, etc. The diagram shown in Figure 14.2 is from the manual for the Second Bassein Creek Bridge, supplied by Colebrand Ltd, the manufacturer of the STUs.

## 14.4. Removal and replacement

The anticipated lifespan of a present-day STU is about 50–75 years. However, the lifespan of the structure on which the STUs are installed may be 50–100 years, and therefore it may be necessary to remove and replace an existing STU. In addition, sometimes STUs may need to be replaced if they are found not to be functioning properly. It is necessary for the designer of the bridge and the manufacturer of the STUs to discuss this requirement for possible removal and replacement, and provide sufficient clearances for the STU to be installed after the construction of the bridge and for it to be removed in the future for replacement. The manufacturer should prepare a drawing showing these clearances and anchorage details, and include it in the inspection and maintenance manual. Average STUs are reasonably light and can be carried by workmen, but very long and heavy STUs may have to be lifted using a small crane and rolled into place for installation.

## 14.5. Maintenance period

The above maintenance regimen should be carried out at least every 10 years. However it is recommended that the STU maintenance procedure be incorporated in, and carried out simultaneously with, the inspection and maintenance programme specific to the bridge. The inspection should be carried out every time the bridge is inspected, say every 2 years. In addition, the STUs must be inspected after every significant seismic event.

## 14.6. Load testing of installed STUs

There are many critics, even in developed countries such as the USA, who do not advocate the use of STUs. They want to be assured of the satisfactory performance of STUs in the field for their life expectancy of 50–75 years. However, this is rather a long period of time to wait. Some authorities provide for quality assurance of installed STUs after 15–20 years of usage on a structure. The California Department of Transportation (Caltrans) planned for such a load testing by storing an additional 1600 ton STU for the Carquinez Bridge (see Chapter 5). It intended to remove one of the six 1600 ton capacity STUs installed on the bridge after 10 years or so service and replace it with the stored, unused STU. The removed STU would be then fully load tested for dynamic performance, and the medium inside the STU would be tested for various properties, and the results compared with the original property tests. This evaluation would help establish the performance of the STUs and their medium.

## 14.7. Load testing of installed STUs for the Stour Viaduct, Kent, UK

In 1997, a long-term evaluation was carried out by the bridge owner, Kent County Council, UK, and the STU supplier, Colebrand Ltd, UK, of the 250 kN rated capacity STUs installed on the Stour Viaduct, Kent.

### 14.7.1 Introduction

In the 1970s, STUs of 250 kN nominal capacity were fitted between the deck and the abutment on the Stour Viaduct. Their purpose was to enhance the longitudinal capacity of the structure to take account of additional braking forces resulting from an increase in allowable heavy traffic load from 45 ton to 70 ton. The STUs were also to provide a secondary function of transferring load from the deck to the substructure in the event of a collision involving heavy vehicles being abruptly brought to a standstill.

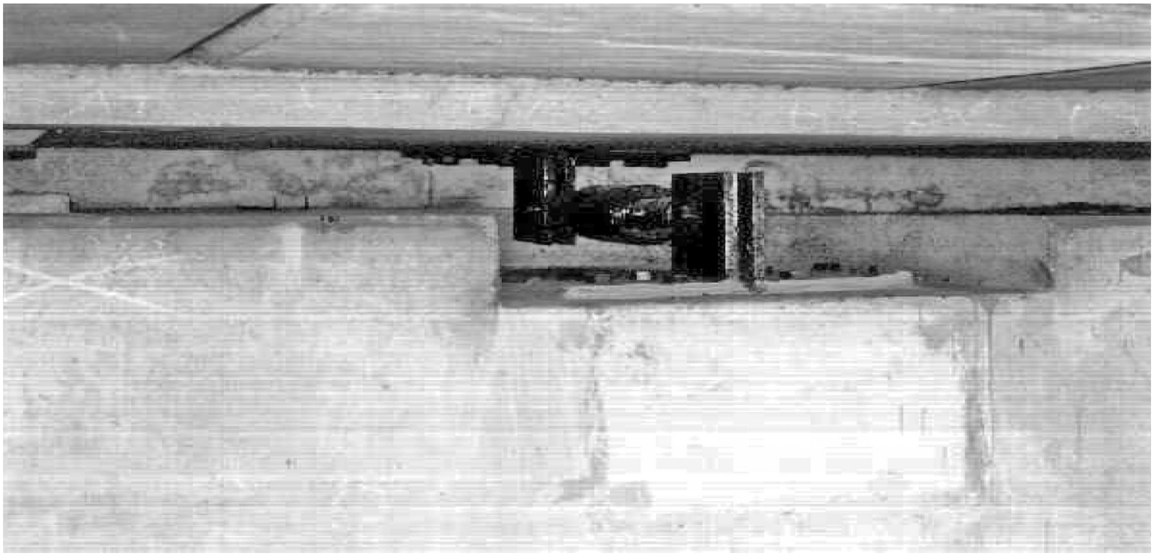
The STUs had been inspected periodically by the Kent County Council maintenance team and also by Colebrand, who provided the council with condition reports. The condition of the units had always been found to be satisfactory, and they were apparently providing the necessary enhancement to the structure that the council required. Given the age of the STUs and the fact that condition monitoring had been recorded, a single STU was removed and disassembled to evaluate its condition and performance after approximately 20 years of service.

### 14.7.2 Site conditions

The STUs are located under the deck between the soffit and bearing shelf. As the STUs and mounting brackets were slightly larger than the bearings, a channel was formed in the bearing shelf to provide adequate clearance. The fabricated brackets attaching the STUs to the structure were bolted into cast-in sockets. Figure 14.3 shows the STU in place before removal.

The local environment has moderate temperature extremes due to its proximity to the sea. However, it is fairly wet and is affected by salt in the atmosphere, which provides a moderately corrosive environment. It should be noted that the client's original specification for corrosion protection did not involve any metallic coating (e.g. zinc or aluminium). A conventional three-coat system comprising zinc phosphate and epoxy undercoats with a polyurethane topcoat had been specified.

Figure 14.3 The STU in place on the Stour Viaduct



Although no recorded data on loading conditions are available, the viaduct is on a main trunk route, which carries heavy traffic. Therefore, the structure would certainly be subjected to heavy service loads on a regular basis.

### 14.7.3 Removal

The STU was removed by first taking off the abutment fixing bracket and then removing the clevis pin in the deck fixing bracket. The fasteners were manufactured in stainless steel and the clevis pin was zinc plated, so no problems due to corrosion were observed. A new STU was put in place and the bracket reinstalled with the original fixings.

### 14.7.4 Inspection

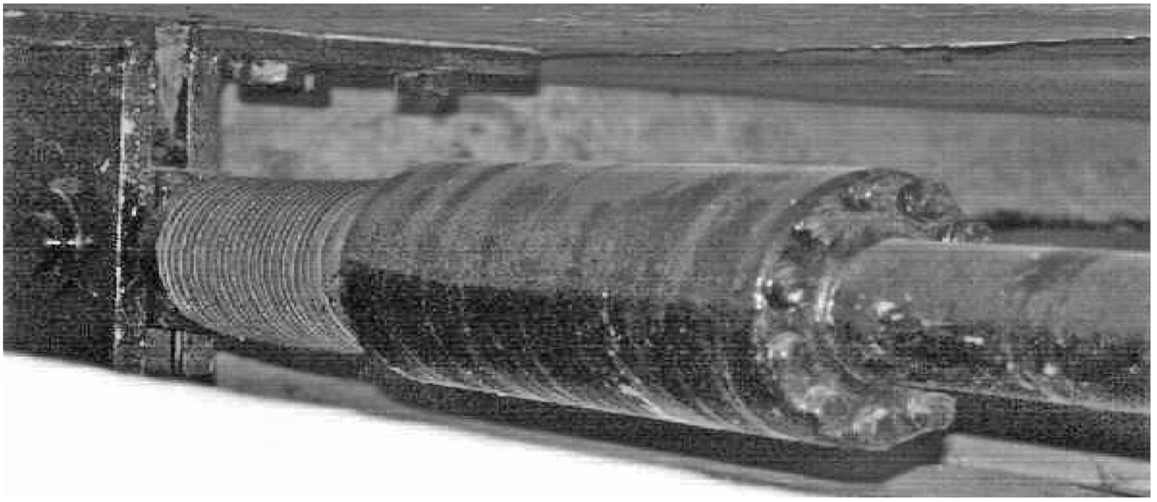
Inspection of the STU was carried out at Colebrand's factory. External corrosion had occurred, but this was superficial. Fasteners were also corroded, but again this was not detrimental to the functioning of the unit. The gaiter covering the piston rod as it emerges from the cylinder was in perfect order, with no deterioration of the material or stitching. Figure 14.4 shows the condition of the STU before disassembly.

### 14.7.5 Load testing

The STU was taken to Bodycote Materials Testing Ltd, Daventry, an independent National Measurement Accreditation Service (NAMAS) accredited laboratory experienced in with lock-up devices and mechanical testing in general. The laboratory was contracted to test the STU against the following pass/fail criteria, taken from the original specification.

- Move at a minimum rate of 5 mm/h under the action of thermal movement.
- Withstand an applied proof load of 250 kN.

Figure 14.4 The condition of the Stour Viaduct STU before disassembly



The specimen was loaded into a Losenhausen 400 kN servo-hydraulic universal testing machine equipped with a Bodycote computer control system and data logger. The test machine's hydraulic grips were used to secure the STU to the fixed platen and hydraulic actuator.

The specimen met the criteria given above when tested in both tension and compression, with a drag force simulating thermal effects of 25 kN.

### 14.7.6 Disassembly

The moving end clevis was unscrewed from the piston rod and the cylinder fastenings removed. The cylinder assembly was then dismantled. There was no internal corrosion of any sort or any other age-related degradation. The seals were intact and no leakage of the working fluid had occurred.

The internal compound was examined and its physical properties tested against the Colebrand standard and the performance data from the original material. There was no measurable difference, confirming that no change had occurred in the compound over time.

### 14.7.7 Conclusion

The performance of the STU fitted to the Stour Viaduct had not deteriorated since installation. The corrosion protection system in the absence of galvanising or other metallic coating required some maintenance, but the deterioration was superficial and a lifespan of 50 years was achievable. The performance and condition of the internal working fluid and components were virtually indistinguishable from when new. A projected lifespan of far in excess of 50 years was perfectly possible.

#### FURTHER READING

Colebrand UK (n.d.) LUD *Maintenance and Inspection Manual for the Second Bassein Creek Bridge*, *Mumbai*, *India*. Colebrand Ltd, London.

Colebrand UK (n.d.) *Stour Viaduct, Kent, UK Technical Report for Load Testing of Installed STU*. Colebrand Ltd, London.

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# Chapter 15
# Design guidelines for procurement and selection of STUs

## 15.1. Introduction

STUs were used for the first time in India on the Second Bassein Creek Bridge on National Highway No. 8 at Ghodbunder, near Mumbai, in the year 2000. The author was involved in the supervision of the construction of the bridge and, in the absence of technical literature, specification, etc., the procurement, selection, load testing and installation of STUs proved rather difficult and very time-consuming. However, the STUs were thoroughly tested and installed successfully, and a paper was presented at the 2002 annual conference of the Indian Road Congress (IRC) at Kochi (Deshpande *et al.*, 2002). The paper was awarded a certificate of merit by the IRC and the bridge itself was awarded first prize for innovation in construction engineering by the Indian Institute of Bridge Engineers (IIBE) in the year 2002, for the successful use of STUs for the first time in India. STUs were used also on the Second Badiwan Bridge at Baguio, Philippines, in the year 2000 and during the early stage of construction supervision, the author was involved in the selection of the STUs for the bridge. The main purpose of writing this book, as already stated, is to help engineers procure and select the most appropriate STUs for their projects, without causing any undue delay in construction. The other purpose is to make engineers aware of the various applications of STUs in practice today. When engineers are well informed the results are always outstanding, and it is hoped that that this book will encourage engineers to unleash the vast potential of STUs.

## 15.2. Design guidelines

A critical factor in determining the successful application of an STU on a given structure is a complete understanding on the part of the designer as to the capabilities of these devices. The following guidelines are cautionary notes to assist engineers in ensuring a full-lifespan performance of installed STUs.

1 STUs are unidirectional devices for load transmission. If out-of-plane forces will be acting on the STU, the angular limits of these loads must be determined to ensure that the STU can withstand them. Generally, spherical bearings and/or additional devices can be incorporated to transfer these forces properly.
2 The load testing requirements and service life selected by the designer can be in the range 25–75 years, subject to appropriate maintenance being carried out by the owner. If a particular application requires a significantly longer or shorter service life, appropriate modifications life can be made to the test requirements for fatigue. However, the designer must provide sufficient working space on the structure at the STU locations such that the devices can be dismantled with ease if they need to be replaced in the future.
3 For railroad bridges, the traction forces may equal or exceed seismic forces in magnitude, and may be applied many, many times over the life of the structure. The designer must ascertain the

magnitude of loads, their rate of application and their duration prior to specifying STUs to distribute them.

4 The corrosion protection system used on STUs should be the same as that used on the bridge. Uncoated weathering steel is preferred, to reduce maintenance requirements. However, STUs can be furnished with a standard galvanised finish on all exposed parts. The boot protecting the piston rod is made of a durable reinforced neoprene material. The stainless piston rod is protected by grease packed into the neoprene boot. It is the responsibility of the designer to determine if this protection 'package' is satisfactory for the exposure conditions expected over the service life of the STU. If additional protection is required, it shall be noted in the project special provisions. If specific durability testing is required as a condition of acceptance, the requirements shall be specified by the designer as a part of the order. Particular attention must be directed to whether corrosion protection beyond that provided for other bridge components is necessary for the STU anchoring system.
Environmental conditions to be considered when selecting the corrosion protection system are
- coastal (marine salts)
- industrial
- potential for periodic immersion due to flooding
- de-icing salts and mud (under bridge joints)
- inadvertent sandblasting during structure repainting
- pigeon nests
- ultraviolet light and ozone exposure of the neoprene material for the boot.

5 The designer should know that the STU is a custom-made unit that is designed, manufactured and tailored to meet both minimum and maximum site-specific load limits. An STU is designed assuming that bridges react to temperature changes at a given rate of movement per hour. This is the basis for determining the 'drag' force that the substructure will be subjected to over the service life of the STU. In addition to the required rated load for each STU, the designer must specify in the contract documents the maximum rate of movement or the maximum acceptable drag force, to ensure a proper STU design. The rate of movement is a function of many variables, such as the rate of temperature change, type of bearings, type of superstructure, etc. If the rate of movement is not known, a value as determined by rational analysis, or $0.0001L$ per hour can be assumed for single and multi-span deck girder bridges with a design length, $L$, of up to 500 ft. The design length is the expansion length that a particular STU must accommodate. The designer should be aware that 2% of the rated load is a practical limit for the minimum drag force that can be achieved. A limiting factor for the drag force could be fatigue loading of the substructure or superstructure units.

6 Typically, only STUs exposed to direct sunlight will reach a temperature of 120°F. Units that will be in the shade of the bridge deck will be subjected to a maximum of 100°F. If units are exposed to direct sunlight, they can be painted a light colour to reflect heat, or be insulated to reduce the effect of solar radiation. A maximum prequalification test temperature of 100°F would be appropriate for STUs that are thus protected.

7 STU can be used to attach superstructure elements together such as at an expansion joint, or to attach superstructure elements to substructure elements. The designer must determine where and how the device can be attached to the structure. The actual connections can be designed by the designer or manufacturer, as specified in the contract documents.

8 The designer shall determine the maximum movement limits for the proposed location of the STU.

9 The STU responds to impact-type loadings by locking. Thus there is a possibility of load amplification due to the impact force. For purposes of analysis, the designer should assume an

amplification of the rated design force, and should check the design of all related components (substructure, anchorage and all fasteners) for an amplification of 1.25 times the rated load.

10 The internal medium used in STUs can be either oil based or putty based, and is generally proprietary in nature. The manufacture must demonstrate, through the load-testing programme, that the design requirements of the STU can be met.

## 15.3. Procurement and selection

As it may be necessary to import STUs, the procurement and selection of the STUs for a particular project should be put in hand at an early stage in the project. Once selected the STUs must be load tested before being transported by sea or air. This selection and procurement may take a considerable time and, if not done early, could cause some delay in the project, especially if the STUs are required to be installed before the completion of the structure. To the knowledge of the author, there are about 10 manufacturers of STUs in the world today, and their names and contact addresses, etc. are given in Appendix 1. As the use of STUs become more widespread, new manufacturers will enter the market, especially in developing countries where the cost of manufacture of STUs can be lower due to cheaper raw materials and labour. There is keen competition between the few present STU manufacturers, and one can negotiate for the best price, bearing in mind that the required specification for the design and testing must be strictly followed by the manufacturer. An STU is a proprietary device and therefore there differences in size, shape, materials, material protection system, internal medium etc. between different devices. Therefore, it is best to obtain a proposal from more than one manufacturer when selecting the STU.

A few manufacturers should be contacted during the early stages of the project and supplied with the following information in accordance with the information given on the design drawings for the structure on which the STUs are to be installed.

- The number of STUs specified on the design drawings and their rated capacity and movement requirement.
- A diagram showing the STU locations on the design drawings.
- Notes on corrosion protection etc. given on the design drawings.
- The specification to be followed.
- The purpose of STUs on the project.
- Any special requirements for the project, including any additional load testing requirements, other than those in the specification, the project completion date, the anticipated date for the supply of the STUs, etc.

This information is required by the manufacturer so that it can submit a detailed proposal, including cost and evidence of prequalification tests required in accordance with the AASHTO specification. The manufacturer should generally submit a detailed proposal based on the above information, together with a brochure giving technical information about the STUs, their track record, evidence of the STU performance, etc.

Once a manufacturer has been selected based on the information submitted in his proposal to supply the STUs, the selected STU manufacturer should prepare and submit working drawings for the STUs and anchorages. Such drawings shall show the external details and dimensions of the STUs and anchorages proposed for use and shall be approved by the engineer prior to fabrication. Such approval shall not relieve the manufacturer of any responsibility under the contract documents for the successful completion of the work.

The following (and more, if appropriate) shall be specified on the working drawings.

- The total number of STUs required, grouped according to rated capacity and rated travel limits.
- The mass of each STU.
- A plan view and section elevation view showing all relative dimensions, including dimensions for the assumed installation temperature, of each STU and associated attachment hardware.
- The minimum and maximum design temperatures for the STUs.
- The maximum drag force at the specified rate of movement caused by the specified temperature changes for each STU.
- The total movement capacity of each STU.
- The maximum rated load capacity for each STU.
- The maximum movement anticipated for each cycle during dynamic loadings.
- The type of materials to be used for all STUs and associated attachment hardware.
- Painting or coating materials to be applied.
- Alignment plans for the STUs, showing tolerances for alignments in which the STUs must be installed.
- Installation and future dismantling and replacement of STUs.
- Design calculations for the attachment hardware verifying conformance with the loading requirements, if required by the contract documents.
- Anchorage details of the attachment hardware.
- The place of manufacture of the STUs and the fabricator of the hardware.
- The manufacturer's name and the name of the representative who will be responsible for coordinating production, inspection, sampling and testing.

Apart from the above information, the manufacturer should also provide the following.

- *Quality control and assurance*. The manufacturer is required to furnish a quality-control certificate and an inspection certificate to confirm that an adequate system of continuous quality control is operating at its plant.
- *Load testing of STUs*. The manufacturer must submit a load-testing programme in accordance with AASHTO Section 32, Clause 32.4, after discussion with the contractor/engineer, stating the name and location of the independent laboratory where the testing will be undertaken. The laboratory selected must be acceptable to the engineer.
- *Packaging and handling*. The manufacturer must submit its plan for packaging, shipment and handling in accordance with AASHTO, Section 32, Clause 32.3.3.
- *Manuals*. The manufacturer must provide an installation manual and maintenance and inspection manuals in accordance with AASHTO, Section 32, Clause 32.5.

## 15.4. Final approval

It is the responsibility of the manufacturer to satisfy the owner of the project in every respect, including load testing. Approval of the STUs must be given to the manufacturer only after it has been satisfactorily demonstrated, through the load-testing programme, that all the STUs are adequate for the project.

### REFERENCE

Deshpande DB, Patel DJ and Sakhadeo RV (2002) Shock transmission units (STUs) for earthquake load distribution on the World Bank funded Second Bassein Creek Bridge in Maharashtra. *Proceedings of the Indian Road Congress Conference*, Kochi, Vol. 2.

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# Chapter 16
# Derivatives of STUs

## 16.1. Introduction

The STU was developed from the fluid damper, and is nothing more than a damper with orifices in the piston that are so small that the device does not move more than a few millimetres when a dynamic load is applied. The STU has such a small lock-up displacement that it does not absorb or dissipate the energy applied due to dynamic load, but rather acts inherently as a hydraulic lock. The orifices in the piston of the STU are specifically designed to provide an output force that varies with velocity, usually to the first power or higher. This, coupled with a very low speed of piston-rod movement for maximum force, ensures that the STU will have virtually no ability to dissipate energy. The diameters of the orifices in the piston of an STU must to be very small in order to achieve a low speed of piston-rod displacement. In general, the orifices in STUs have a very large length/diameter or length/width ratio, and such design offers a high flow resistance yet is relatively free from clogging during long-term service.

The main difference between a damper and an STU is that a damper is designed to displace substantially, and in so doing absorbs and dissipates the force being applied, while, when under a rapidly applied load, an STU acts like a rigid link, dissipating virtually none of the applied force from one end and directly transmitting the applied force to a structural element connected at the other end. Dampers absorb the energy, resulting in a less stiff structure and an increase in the period of the structure, and, in some cases, they reduce the total seismic forces generated. In doing this, however, large displacements could become a problem, and seating restrainers may be necessary, especially in retrofits. STUs are a form of ‘reverse damper’ – they stiffen up the structure and decrease displacements, making the use of restrainers unnecessary. The trade-off is higher forces. However, in general, basic STUs are fully functional over a wide range of frequencies, and in structural applications STUs will easily respond to input frequencies ranging from zero to more than 50 Hz.

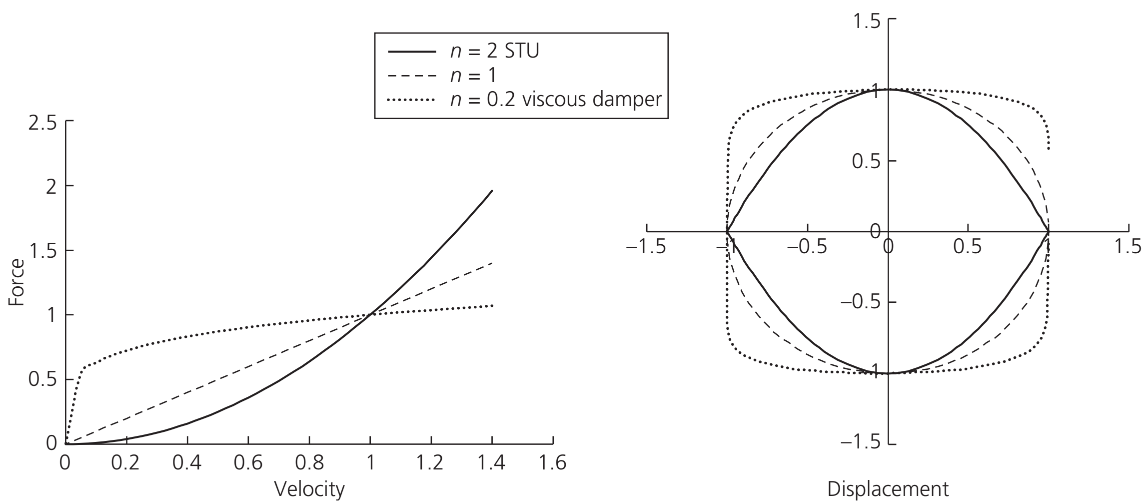
The behaviour of a viscous damper can generally be described by the equation

$$F = CV^n$$

where $F$ is the force applied to the piston, $V$ is the velocity at which a piston is moved, and $C$ and $n$ are constants that depend on the properties of the fluid and the hydraulic circuit.

The force–velocity and force–displacement diagrams are plotted for different values of the exponent $n$ in Figure 16.1. From these diagrams it is obvious that the difference between the force at low velocity and the force at high velocity is maximum when $n = 2$. When this is the case the device allows slow movements, and becomes rigid in the case of dynamic actions. These devices are commonly known as ‘shock transmission units’.

Figure 16.1 Plots of force versus velocity and force versus displacement for a sinusoidal excitation of hydraulic devices for different values of the exponent *K*. Courtesy Agostino Marioni, Alga




When the dissipation of energy is the main requirement, $n = 0.2$ or smaller is required, as is evident from the force–displacement diagram, the energy dissipation (which is proportional to the area of the plot) increases as the value of *n* decreases. In this case the devices are commonly known as 'viscous fluid dampers'.

## 16.2. Hydraulic dampers

A hydraulic damper is similar to an STU and consists of a sliding piston that divides the hydraulic cylinder into two chambers. Both chambers are filled with nearly incompressible silicon oil, and the two chambers are connected by small, calibrated gaps. When the piston starts to move, the resulting pressure difference between the two chambers forces the silicon oil to flow from one chamber to the other. The pressure difference resulting from very slow movement velocities (e.g. resulting from creep and shrinkage or temperature variation) can immediately be balanced without producing any considerable response force (Figure 16.2). In the case of faster velocities (e.g. >0.1 mm/s), the gaps are too small to allow an immediate balancing of the pressures in the two chambers. The pressure in the chamber from which the oil is being pushed thus increases, and the piston cannot move any more, and the system becomes rigid (Figure 16.3). As soon as a predefined pressure is exceeded in the chamber, a special mechanism allows the silicon oil to flow under very high pressure from one chamber into the other. As the damper moves (see Figure 16.3) the high oil pressure leads to heating of the oil. This transformation of kinetic energy into heat represents the process of energy dissipation.

Figure 16.2 A hydraulic damper. © Maurer Söhne GmbH & Co. KG

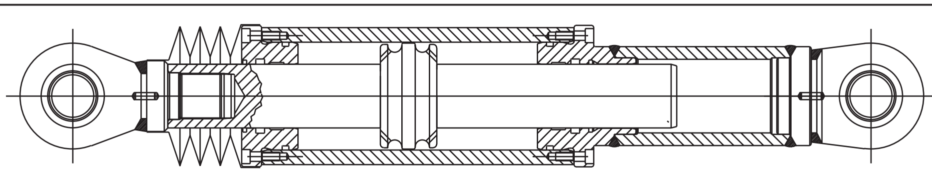
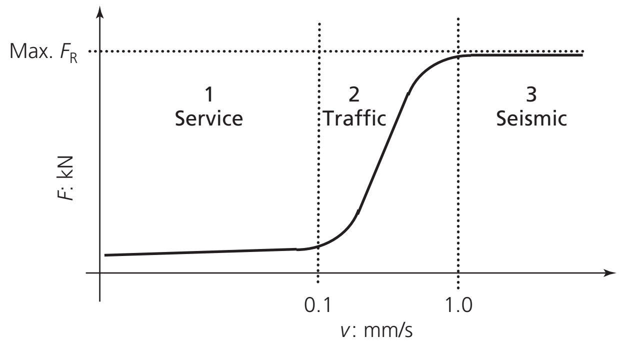


Figure 16.3 Relationship between the movement velocity and the response force for a hydraulic damper




The response force of the damper follows the equation

$$F = CV^n$$

A very low damping exponent of $n = 0.015$ allows the creation of a response force that is nearly independent of the movement velocity. This means that the predetermined response force of the damper will never be exceeded, and overloading of the structure is prevented.

The purpose of the accumulator is to allow for the volumetric displacement of the piston rod as it enters or exits the damper during excitation. The second purpose is to compensate for thermal expansion and contraction of the fluid as heat is generated when a damper is in action under dynamic force.

Dampers are fixed between the substructure and superstructure. In an earthquake event, the piston begins to move and mechanical energy is transformed into heat. In this way the maximum horizontal movements of the superstructure are limited, and the seismic energy cannot cause any damage to the structure. Viscous fluid dampers, like STUs, are used extensively in the structural engineering field.

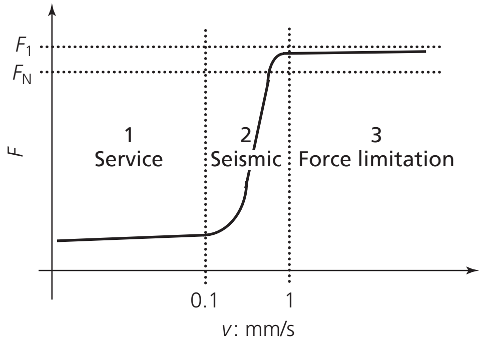
There are various derivative types of STUs available, and some of the important ones are described in the following sections.

## 16.3. Crawl connector STUs

When a dynamic load is applied to a standard STU and it locks up, there is a small displacement due to the elasticity of the STU elements and to creep. The shortening due to elasticity is reversed once the load is removed, but a very small creep component remains. In seismic applications the creep component is rather insignificant, as the load is applied in both directions and is not held steady for any appreciable time. However, in the case of STUs installed on a railway bridge at the expansion joints, the braking and traction load is applied and holds steady for a longer duration, which gives rise to higher creep movement. A crawl connector is an important derivative that resists all further creep movement once the initial load has been applied (Gill *et al.*, 1999).

The special design and construction of a crawl connector gives it a particular behaviour response. In a typical crawl connector, as the initial load is being instantaneously applied, the piston displaces by approximately 5 mm. As the rated load is held steady, there is virtually no further displacement of

**Figure 16.4** Plot of force ($F$) versus velocity ($V$) for a force limiter STU. © Maurer Söhne GmbH & Co. KG



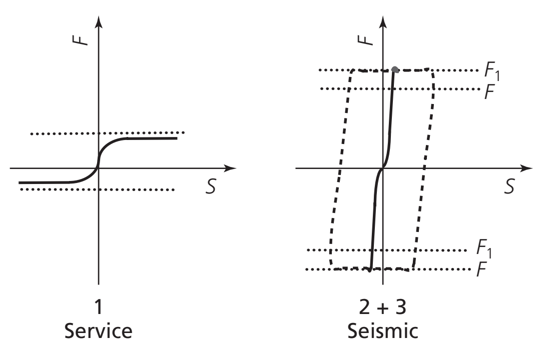

the piston. When the load is removed, the piston undergoes a simultaneous recovery of 2.5 mm from the starting position, followed by a complete return to its starting position over the following 10 s or so. The use of crawl connector STUs is of particular benefit in multi-span structures where deck-end connectivity is required under the traction and braking of a train or a vehicle. The crawl connector facilitates the transfer of force between adjacent deck spans, with no movement beyond the initial displacement while the force is being applied. In this way successive loading events can be sustained without the risk of progressive creep, or 'ratcheting', of the adjacent spans, with recovery to the original position and freedom to sustain thermal and other secondary movements.

## 16.4. Force limiter STUs

The force limiter STU is a further variant of the standard STU that has been developed primarily for use in seismic design applications. Compared to the standard STU, which has a theoretically unlimited blocking force, when a force limiter STU experiences an unexpected infinite force it reacts with a well-defined maximum response force $F_1$, as shown in Figures 16.4 and 16.5. This response force limit is normally defined slightly above the nominal blocking force (see Figures 16.4 and 16.5).

If the maximum nominal response force of the load limiter STU is exceeded by unforeseen dynamic seismic structural behaviour or too great a seismic energy input, an 'intelligent' control device

**Figure 16.5** Plot of force ($F$) versus displacement ($S$) for a force limiter STU



allows displacement of the force limiter STU. The response force is kept constant (see Figures 16.4 and 16.5) by the 'intelligent' control device, whereas the displacement velocity plays no important part, meaning that the response force is independent of the displacement velocity during a seismic event or traffic impact.

The force limiter STU provides the designer with the confidence that the maximum response force of the STU is well defined and independent of the amount of impact energy. This brings the significant advantage that the structure can be designed exactly for a defined response force. When using a standard STU the structure has to be designed with greater safety margins, which is more expensive, or the STU itself or the structure could be damaged if the seismic force is greater than expected.

## 16.5. Load and displacement equaliser STUs

The load and displacement equalising STU is a special variant for application typically on multi-span structures that have significantly different adjacent support stiffnesses (Gill *et al.*, 1999). An example of such a structure would be a viaduct across hilly terrain, with multiple single simply supported spans between piers.

Consider a structure where pier 1 is 5 m high and pier 2 is 10 m high. Because longitudinal stiffness is given by the expression $3EI/L3$, if the plan section of the two piers is the same pier 1 will be eight times stiffer than pier 2, and will therefore attract a highly disproportionate share of any local uniformly distributed longitudinal load, whether this is a service load or a load due to an earthquake. This could result in excessive construction costs for pier 1 and its foundations, together with higher frequency oscillation than is desirable during an earthquake event.

The load and displacement equalising STU incorporates a separate chamber, connected in series with the STU piston, which has a predetermined load–displacement characteristic. In all other aspects it performs precisely the same function as a standard STU. The load–displacement function can be set to reintroduce flexibility in tension at short stiff piers so that they behave in a similar manner to their taller neighbours, thereby inducing a uniform force and displacement distribution. The principle can clearly be extended to viaducts or bridges of any type having any number of piers of varying height, so that the designer achieves a supporting substructure of uniform longitudinal stiffness throughout the length of the structure.

The stiffness $K_s$ of the STU required at any location can be readily determined by considering the strain compatibility of the pier and foundation, $K_p$. If the STU is connected between the deck and the pier, the spring stiffness of the STU can be ascertained as

$$K_t = (K_s + K_b)K_p/(K_s + K_b + K_p)$$

Because of its ability to control and decrease the stiffness and the period of oscillation of multi-span bridge structures, the load and displacement equalising STU is expected to become a significant tool in the effective and economic seismic design of these types of structure in the twenty-first century.

## 16.6. Shock absorber STUs

The shock absorber STU is, in essence, an elastic buffering device with an approximately linear spring characteristic: $F = Kd$, where $F$ is the applied force, $K$ is the spring stiffness and $d$ is the displacement (Gill *et al.*, 1999). The device has many seismic protection applications, where absorption of energy is not required, and can be designed for most combinations of force, stiffness and displacement.

This shock absorber STU is a special type of STU snubber, which is used on nuclear power plants and pipelines (for more detail see Chapter 9).

## REFERENCE

Gill AC, Townsend GH, Snook V and Minta MT (1999) Lock up devices, their derivatives and their applications in the design of engineering structures. *Taiwan Construction Institute Conference*. Colebrand Ltd, London.

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# Chapter 17
# STUs in conjunction with other seismic-protection devices

## 17.1. Introduction

Seismic-protection devices that can be installed at the supports of a structure to gain specific design benefits can be grouped into the following three types based on their functional characteristics.

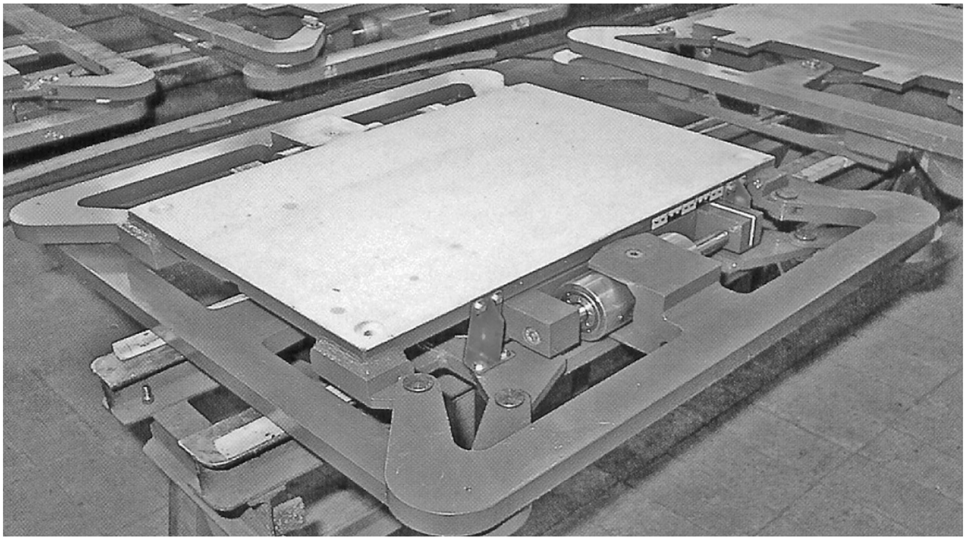
- *Energy-dissipating systems*: the purpose of these systems is to reduce the dynamic response of a structure to external forces by dissipating part of the energy imparted to it. This is most often achieved by mobilising the plasticity or ductility of solids, the viscosity of fluids and by friction.
- *Base-isolation systems*: the primary function of these systems is to increase the flexibility of a structure and shift its period to a range of lower response. This is achieved by the use of elastic devices inserted between the structure and its foundation.
- *Temporary strengthening systems*: under normal circumstances these systems allow relative movement between the superstructure and substructure, but create a condition of rigid support during shock or dynamic loading events, in effect, temporarily restraining the structure during such transient loads. Devices performing this function are known as shock transmission units (STUs) or lock-up devices (LUDs).

Multifunction systems incorporating two or all of the above systems to complement or supplement each other are also possible. A purely energy-dissipating system, for example, lacks the ability to restore the structure to its initial position after the dynamic event, but a purely elastic system can reduce the inertial loads arising from dynamic base excitation at the expense of large relative motion. Combining the two systems, therefore, presents distinct advantages in terms of both the safety and the serviceability of a structure. Such systems can be designed by combining various mechanical and hydraulic devices to address the specific load-response requirements of any major structure.

When supplying seismic protection in both new and retrofit projects, the selection of the type of anti-seismic device must not necessarily fall upon a single type, as in many cases the use of combinations of devices can result in a significant advantage. Several combinations of bearings (elastomeric, pot or spherical), steel hysteretic dissipaters and STUs are possible, and the choice of an optimal combination depends on the requirements to be fulfilled (elastic stiffness, post-elastic stiffness, amount of displacement, dissipative efficiency, etc.). There is at present a trend towards an increased interest among seismic engineers in employing different types of anti-seismic device on a single project.

Various types of hysteretic damper used to protect bridge structures in highly seismic zones have STUs and pot bearings integrated within them. A very common application of STUs is in series with

Figure 17.1 An hysteretic damper integrated in a pot bearing, and two STUs



hysteretic dampers. A hysteretic damper dissipates energy through the cyclic yield of the steel elements. Figure 17.1 shows an hysteretic damper integrated in a pot bearing, with two STUs, one on each side of the bearing. Such an integrated device performs the following functions.

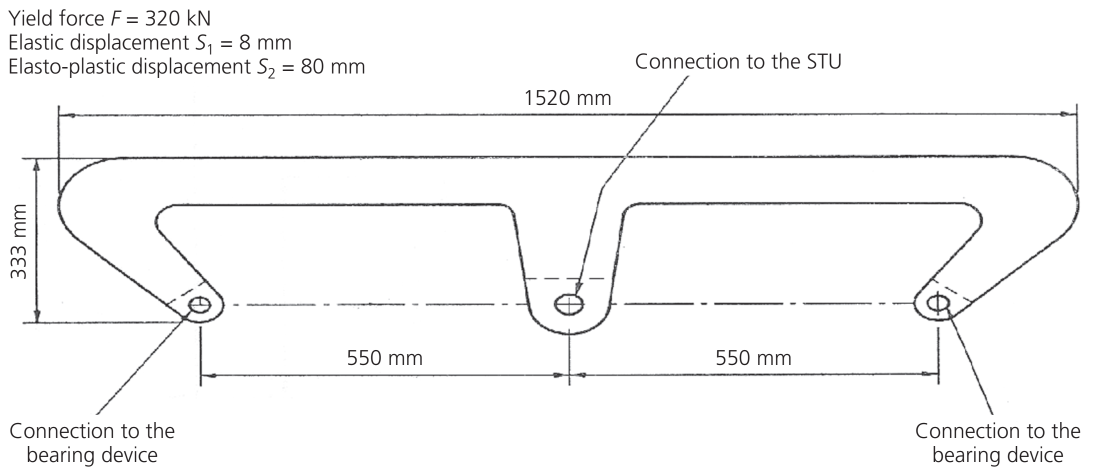
- The pot bearing transmits the vertical loads and allows rotation and slow movements.
- The STUs ensure simultaneous reaction of all the bearing devices belonging to a structural section (STUs are not used with an integrated hysteretic damper with a fixed bearing, which must remain fixed for slow effects).
- The hysteretic damper dissipates the seismic energy.

Figure 17.2 shows an E-shaped two-bay portal-type hysteretic damper. For every integrated device of the type shown in Figure 17.1 there are four E-shaped hysteretic dampers, two in each orthogonal direction. The dampers are made of special steel that has a particularly low percentage of carbon and a very high impact resistance at low temperatures, and this increases their low cycle fatigue life.

This hysteretic damper must function for only about 60 s in a structure, the lifespan of which can be 50–100 years. If it does not function during these 60 s of a seismic event, the purpose of providing seismic protection to the structure is defeated, and all the efforts made and costs borne will have been in vain. Therefore, this device is designed to be simple in concept, extremely reliable and to not require regular maintenance. Such a device solves the problem of making large engineering structures ‘quake proof’ by means of the simple and highly reliable method of dissipating the seismic energy through the flexural yielding of its components.

Under normal conditions, the integrated hysteretic device allows for a variation in the length of the structure, due to shrinkage and creep, and expansion and contraction due to temperature variations of concrete, without causing appreciable resistance. In the event of an earthquake, the device distributes the seismic forces throughout the support structures of the bridge, and keeps the effects

**Figure 17.2** An E-shaped two-bay portal-frame-type hysteretic damper. Courtesy of Agostino Marioni, Alga




of the seismic force down to a predetermined tolerable level by way of the yielding of the steel components. The benefits of such integrated hysteretic dampers are two-fold: they substantially increase the structure's safety in the event of an earthquake, and they reduce the size and the cost of the structure's substructures and foundations.

Figure 17.3 shows a typical application of an integrated hysteretic device on a six-span continuous bridge located in a highly seismic area. An hysteretic damper integrated in a unidirectional bearing, with two STUs, one on each side, is installed on the internal piers 1, 2 and 4, 5, and an hysteretic damper with a fixed bearing is installed on the middle pier 3. In the event of an earthquake, the STUs will distribute the earthquake load to all piers instead of just to the single fixed pier. The integrated hysteretic dampers installed on all piers then dissipate some of the earthquake load by allowing yielding of the steel elements, thereby reducing significantly the amount of the earthquake load on the piers.

**Figure 17.3** A continuous bridge installed with hysteretic dampers integrated in bearings, and with STUs

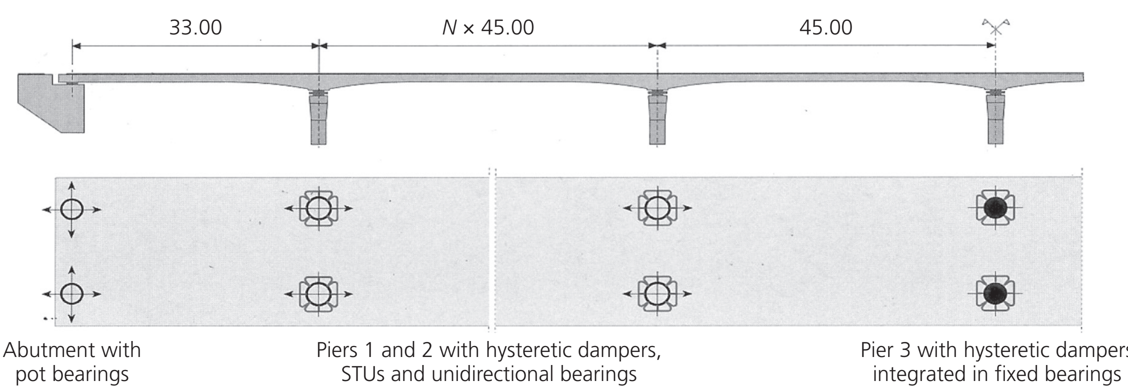
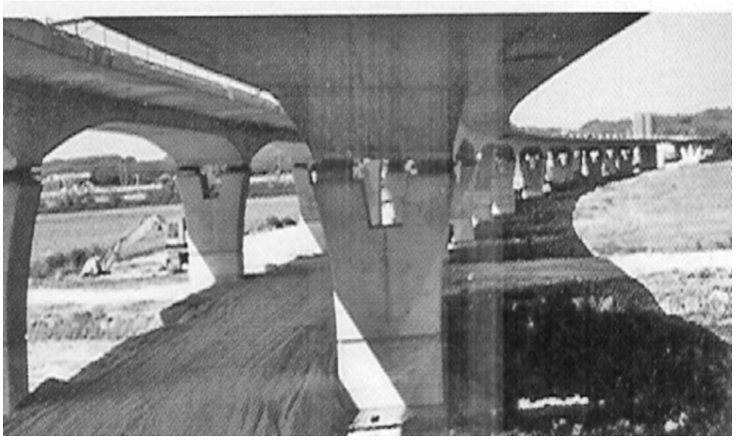


Figure 17.4 The Mortaiolo Viaduct. Courtesy of Agostino Marioni, Alga



Some case studies of the use of STUs in conjunction with other seismic-protection devices are given in the following sections.

## 17.2. Mortaiolo Viaduct, Italy

The Mortaiolo Viaduct is located on the Livorno–Civitavecchia superhighway near Livorno, and is in a zone 2 seismic area (Marioni, 1991). The viaduct comprises a continuous pre-stressed concrete box girder deck composed of 10 spans of 33 m, eight spans of 45 m and one span of 33 m, giving a total length of 426 m (Figure 17.4).

The module shown in Figure 17.5 is repeated with little variation over the total 8 km length of the structure. The length of this viaduct makes it the most important one of the entire highway.

Figure 17.5 A typical module of the Mortaiolo Viaduct. Courtesy of Agostino Marioni, Alga

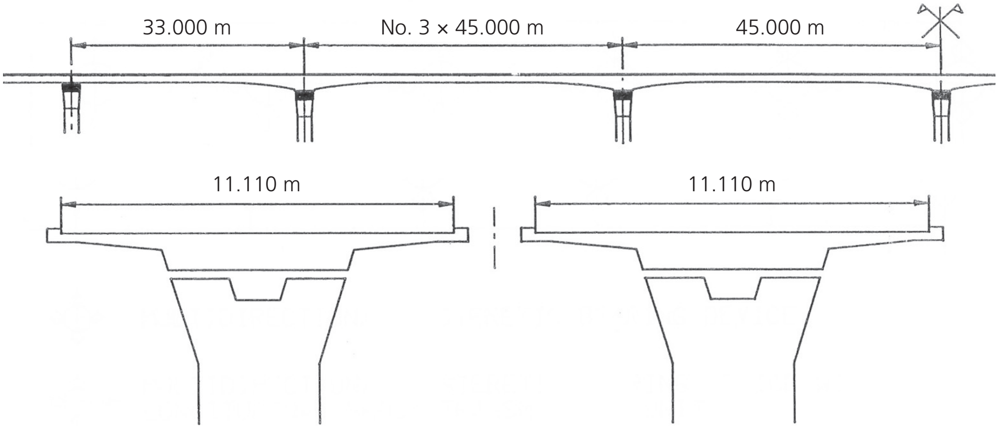
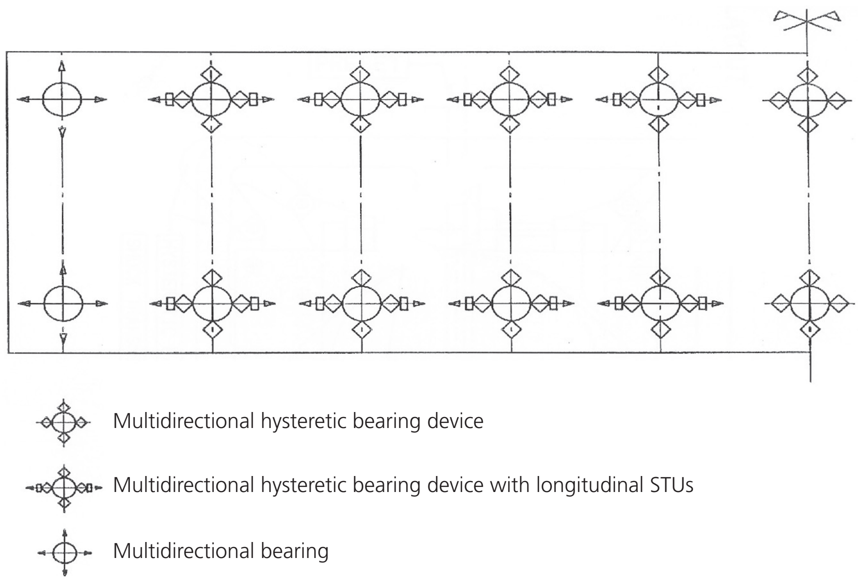


**Figure 17.6** A typical layout of pot bearings and integrated hysteretic bearings on the Mortaiolo Viaduct module. Courtesy of Agostino Marioni, Alga




Multidirectional hysteretic bearings with longitudinal STUs, multidirectional hysteretic bearing and pot bearings are provided for each module as shown in Figure 17.6.

The four piers with multidirectional hysteretic bearings with longitudinal STUs, and one pier with multidirectional hysteretic bearing devices, in a typical module of the Mortaiolo Viaduct (see Figure 17.6) satisfy the following objectives.

- They perform the function of normal bearings under working conditions. The horizontal forces are unloaded on the central pier having multidirectional hysteretic bearings. The remaining bearings are sliding, and allow for the variation in structure length due to slowly applied loads of creep, shrinkage and thermal variation.
- In the event of an earthquake the seismic force is uniformly distributed to all the internal piers of the module due to STUs integrated in the hysteretic bearing device, which stop the movement of the sliding bearings. All the hysteretic bearings show hysteretic behaviour during a seismic event and limit the effects of the earthquake through the flexural yielding of their components.

Figures 17.7 to 17.9 show the working scheme of the multidirectional hysteretic bearing under slowly applied and dynamic loads.

The hysteretic damper for the viaduct has the following properties.

- Yield force 320 kN.
- Elastic deformation 8 mm.
- Elasto-plastic deformation 80 mm.

Figure 17.7 Working scheme of a multidirectional hysteretic bearing under slow longitudinal movement. Courtesy of Agostino Marioni, Alga

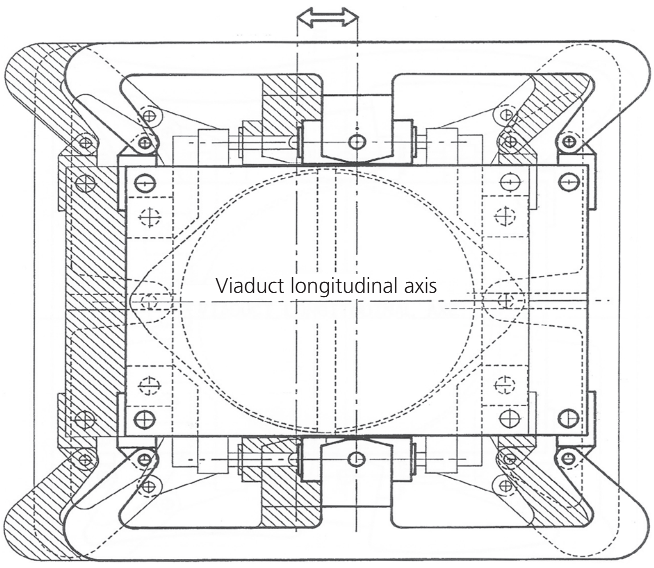



Figure 17.8 Working scheme of a multidirectional hysteretic bearing under dynamic longitudinal movement. Courtesy of Agostino Marioni, Alga

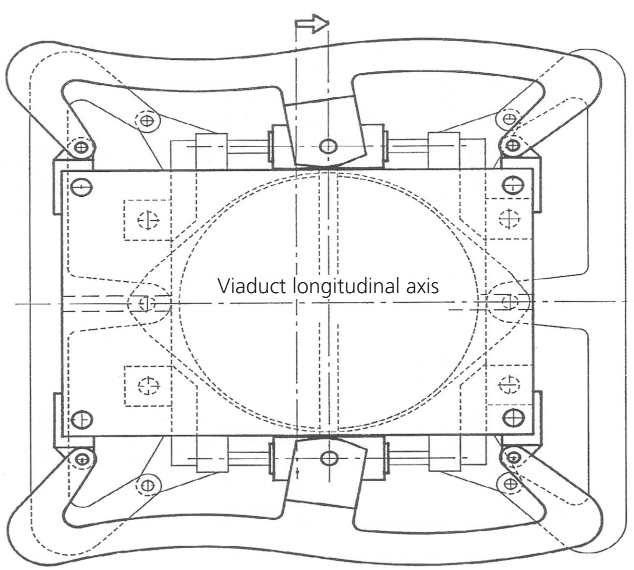
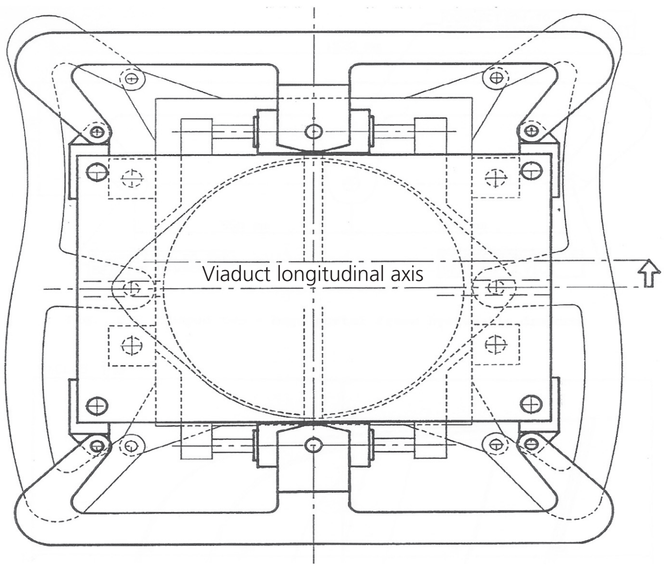


Figure 17.9 Working scheme of a multidirectional hysteretic bearing under dynamic transverse movement. Courtesy of Agostino Marioni, Alga




Test results have demonstrated that the hysteretic damper can sustain more than 50 cycles at a displacement ductility of 10; after that the dampers are still capable of undergoing at least 10 more cycles at a ductility of 15.

As in this application the STUs are installed in series with the hysteretic damper, the damper limits the maximum force to its yielding force of 320 kN, thereby avoiding the need to introduce a pressure limiter valve in the STU.

## 17.3. Jamuna multipurpose bridge, Bangladesh

The bridge lies 120 km north of the capital Dhaka, and comprises a 4.9 km viaduct (FIP Industriale, n.d.). It is a multi-span continuous Gerber-type bridge, with a typical module comprising girders that are 6 or 7 spans long (99.75 m) (Figure 17.10). It has a 12 000 m constant-radius curve in plan view. The deck comprises a box girder which was constructed by the on-site connection of prefabricated pre-cast reinforced-concrete girder elements. The deck slab is 18.5 m wide and its height varies from 2.75 m to 5.5 m. It is a multipurpose bridge carrying highway traffic, rail traffic, long-distance power lines and gas pipelines.

The bridge is located approximately 30 km from an active fault line in a highly seismic zone having a ground peak acceleration of 0.47*g*, and thus its design includes anti-seismic devices. The deck rests on only free-sliding pot bearings of 33 000 and 30 000 kN vertical load capacity, and so the design of the anti-seismic devices had to meet the following requirements.

- A static scheme under normal service conditions, providing fixed points for horizontal forces (transverse and longitudinal) and guiding devices to permit longitudinal deck movements with respect to the piers.

Figure 17.10 The Jamuna multipurpose bridge. Reproduced courtesy of FIP Industriale SpA

- A multi-directional damper system (able to withstand 15 cycles at the design displacement) under seismic conditions.
- A lateral stop-block, acting at a displacement of 250 mm, designed for 4500 kN capacity.
- Anti-seismic devices and all bearings need to be easily replaceable without excessive deck jacking.

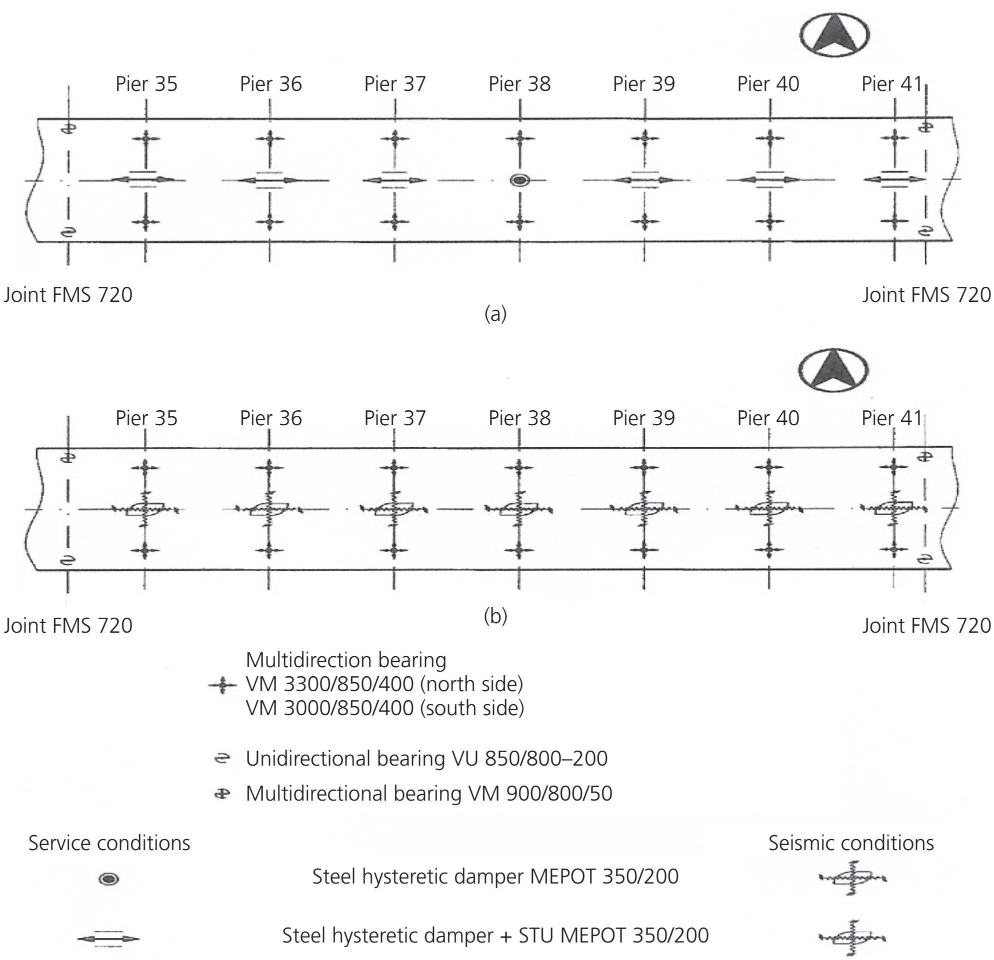
These seismic design requirements were met by FIP Industriale of Italy through the design of multidirectional steel hysteretic dampers comprising double-tapered spindles as dissipating elements. These dampers, the sole purpose of which is to control horizontal forces, are combined with free-sliding pot bearings that serve to transmit vertical loads and permit horizontal movements in all directions.

Figure 17.11 shows the bearing system under both service and seismic conditions of a typical seven-span continuous module of the bridge. Under service conditions there is a fixed point at pier 38 where an anti-seismic MEP-type device is located. MEPOT-type anti-seismic devices are located on the other piers of the module. The MEPOT device has an integrated hysteretic damper with two STUs coupled in series (Figure 17.12), and is capable of accommodating longitudinal deck displacements due to slowly applied loads.

During a seismic event, all anti-seismic devices behave identically, dissipating a great amount of energy transmitted by the seismic event through plastic deformation of their steel spindles. In fact, the STUs lock up, transmitting the force to the spindles.

Each damper, in both the MEP- and MEPOT-type devices, comprises 42 double-tapered spindles working in parallel, designed for a yield force of approximately 80 kN and a maximum force of 100 kN each at 200 mm displacement. The spindle is one of the dissipating elements used in steel hysteretic dampers. One of its main advantages is its intrinsic multi-directionality. Each MEPOT

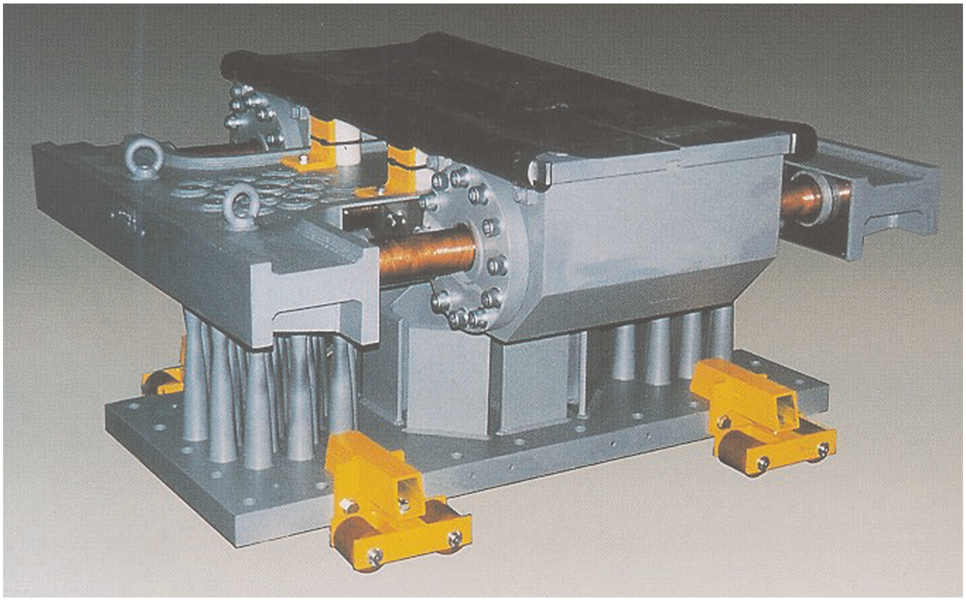
Figure 17.11 The bearing system of a typical seven-span module of the Jamuna multipurpose bridge under (a) service and (b) seismic conditions




device contains two STUs having a nominal maximum force capacity of 2100 kN each. Figures 17.13 and 17.14 show elevation views of a MEPOT anti-seismic device along the bridge transverse and longitudinal directions, respectively. The MEP-type anti-seismic device is identical to MEPOT except for the absence of STUs.

Service, wind, braking, etc. loads do not stress the dissipating elements owing to the use of ‘sacrificial restraints’ which are designed to fail at a 500 kN horizontal load. The sacrificial restraints impede any displacement in the dampers of the fixed type, and any transverse movements on those of the expansion type. In the event of a strong earthquake, the sacrificial restraints fail and the dampers are activated. The anti-seismic isolation devices provided can also withstand greater displacement than the design value of $\pm 200$ mm. The spindles themselves can withstand significantly higher displacements but, regardless, during an earthquake a stop-block intervenes at the onset of transverse displacements of $\pm 250$ mm.

Figure 17.12 A MEPOT-type device: an integrated hysteretic damper with STUs. Reproduced courtesy of FIP Industriale SpA



The seismic isolation of the Jamuna multipurpose bridge represents a successful example of the application of anti-seismic devices consisting of multiple spindle steel hysteretic dampers, STUs and steel bearings. The adoption of the seismic-isolation approach provides the bridge with complete protection against the design earthquake, and at a significant saving in construction costs compared with a traditional approach, particularly with regard to the foundations.

Figure 17.13 Elevation view along the transverse direction of a MEPOT anti-seismic device – an hysteretic damper with STUs

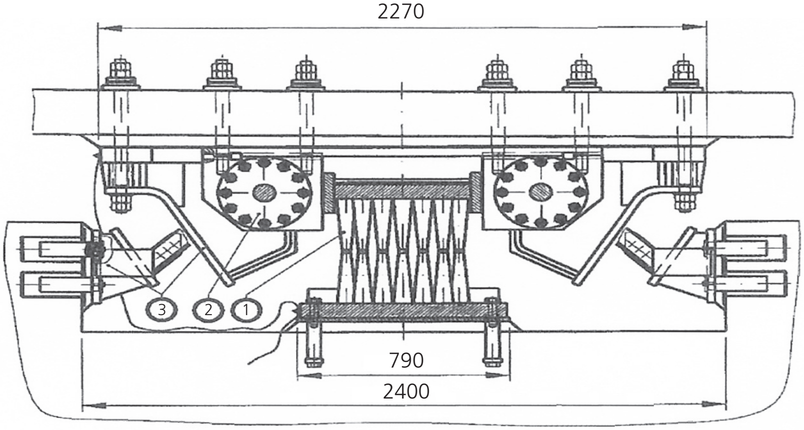
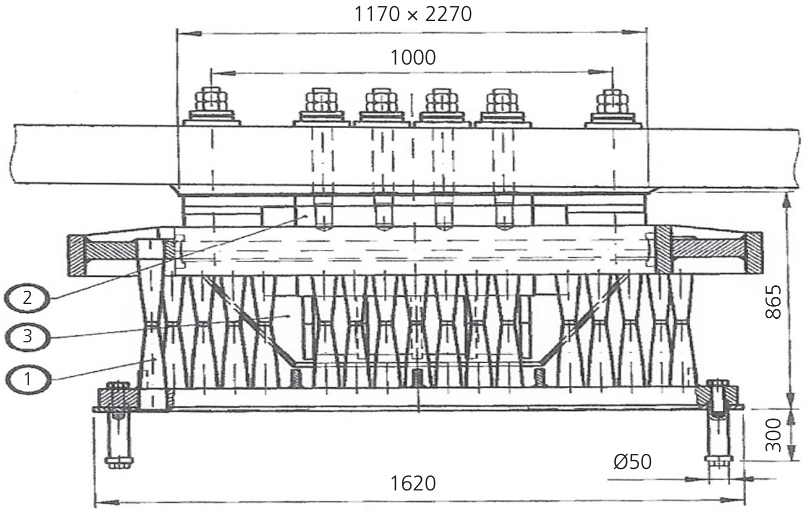


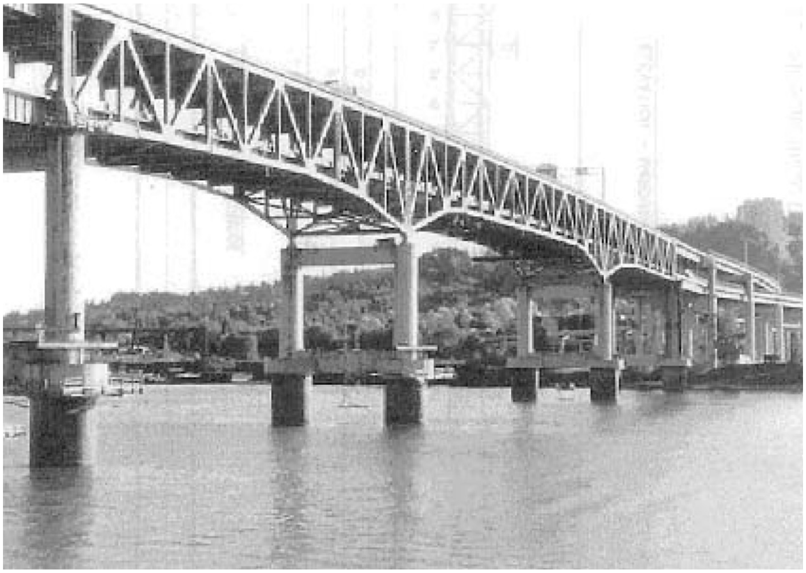
Figure 17.14 Elevation view along the longitudinal direction of a MEPOT anti-seismic device – an hysteretic damper with STUs




## 17.4. Marquam Bridge, Oregon, USA

The Marquam Bridge is a major bridge on interstate I-5 constructed in 1966 over the Willamette River in Portland, Oregon (Griffith *et al.*, 2000). The bridge has a typical Gerber girder configuration (Figure 17.15), in which the two end girders have a cantilever beam toward the centre of the bridge,

Figure 17.15 The Marquam Bridge. Image courtesy of the Oregon Department of Transportation



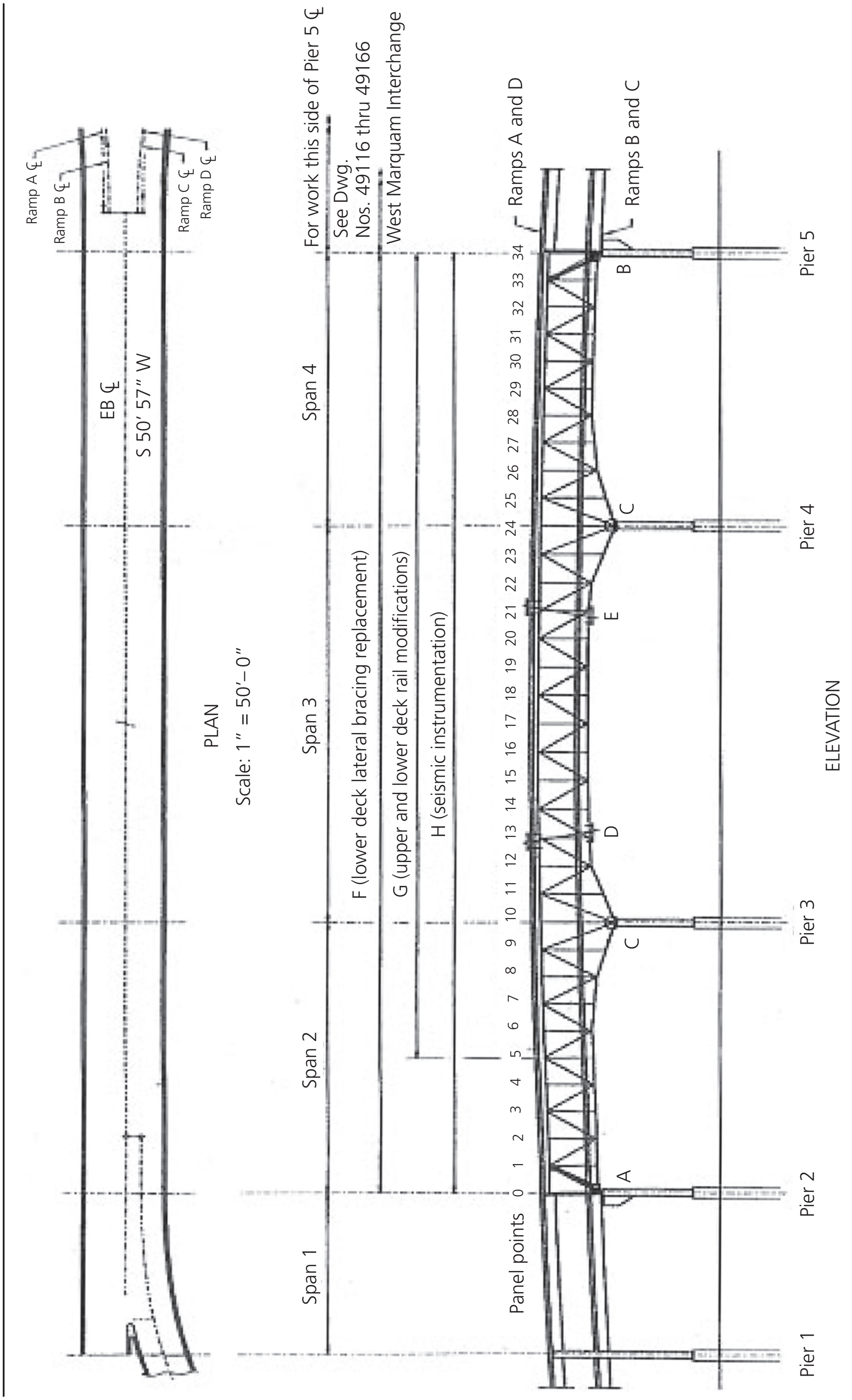



**Figure 17.16** Plan and elevation of views of the Marquam Bridge. Image courtesy of the Oregon Department of Transportation

upon which the suspended central truss is supported. The main spans of the bridge are steel trusses carrying two levels of roadway for a length of 318 m. The two end spans measure 92 m and consist of steel trusses cantilevered 27.5 m into the middle length, providing support for the 79 m suspended span. The bridge rests on four reinforced-concrete piers.

Since the 1990s a greater emphasis has been placed on seismic protection of structures by the Oregon Department of Transportation (ODT). The preliminary project study included the investigation of the adequacy of the bridge to meet the seismic design forces. Accordingly, a multimodal analysis was performed, which showed that the existing bridge was incapable of resisting the design maximum expected earthquake forces. It was decided to retrofit the Marquam Bridge to achieve the following objectives.

- To attain seismic protection for superstructure pull-off in the event of seismic movements arising from the design event.
- To minimise any future strengthening of the existing structural elements.
- To maintain the existing system of service-load transfer.
- To limit seismic displacement to values compatible with the existing configuration.

To achieve these four objectives, ODOT selected FIP-EAS, a joint venture between FIP Industriale, an Italian-based provider of seismic management technologies, and Energy Absorption Systems Inc., Chicago, a manufacturer of highway safety hardware. Using a multidisciplinary approach, FIP-EAS designed a system that replaced the pre-existing vulnerable steel bearings with high-energy dissipating isolators.

The retrofit involved installation of proprietary bearing/restraint devices and STUs supplied by FIP Industriale. The following anti-seismic devices were installed at piers 2 to 5 and at the panel points L13 and L21 (the expansion joint of the suspended span) (Figure 17.16).

- Hysteretic bearings with STUs were installed at piers 2 and 5.
- Hysteretic bearings were installed at piers 3 and 4.
- Two STUs were installed, one on each side of panel point L13 and L21, at the expansion joints for the suspended span (Figures 17.17 and 17.18).

The existing bridge had fixed bearings on piers 3 and 4 and free bearings on land piers 2 and 5. The suspended span was kept free to move at both the expansion joints.

The isolators used for the expansion joints at each end of the bridge at piers 2 and 5 were unique in that they incorporated an STU that is active between the superstructure and the steel dissipating elements. The concept allows for thermal and other service load movement without engaging the dissipating elements.

The isolators for piers 3 and 4 are of the sliding type and combine a free-sliding pot bearing to transmit the vertical loads and a series of steel dissipating elements to control horizontal actions (Figure 17.19). Figure 17.20 shows such an hysteretic bearing (without STUs) installed at pier 3.

Therefore, the original fixed points remain fixed and the original expansion points remain flexible after the installation of the anti-seismic system, thus maintaining the existing system of service load transfer as specified in the objectives.

Figure 17.17 An STU at panel L13 of the Marquam Bridge. Image courtesy of the Oregon Department of Transportation

To accomplish the objective of maintaining the existing bearing system, which called for expansion bearings on land-side piers 2 and 5 and fixed bearings on the two internal piers 3 and 4, sacrificial restrainers (shear keys) were provided at each pier-bearing location. These shear keys ensured that normal service loads, including wind, breaking actions and even moderate earthquakes, do not

Figure 17.18 A closer view of the STU at panel L13 of the Marquam Bridge. Image courtesy of the Oregon Department of Transportation

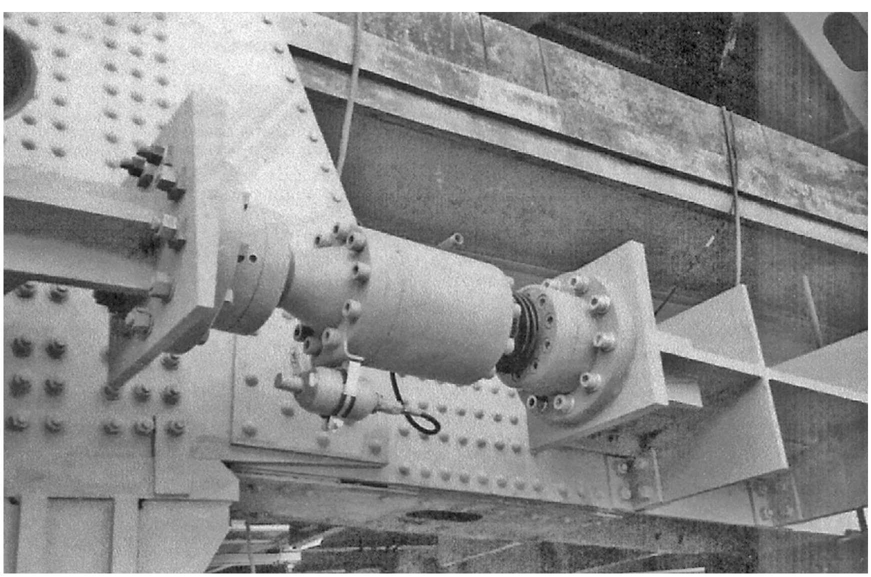


**Figure 17.19** An hysteretic bearing on the Marquam Bridge. Image courtesy of the Oregon Department of Transportation

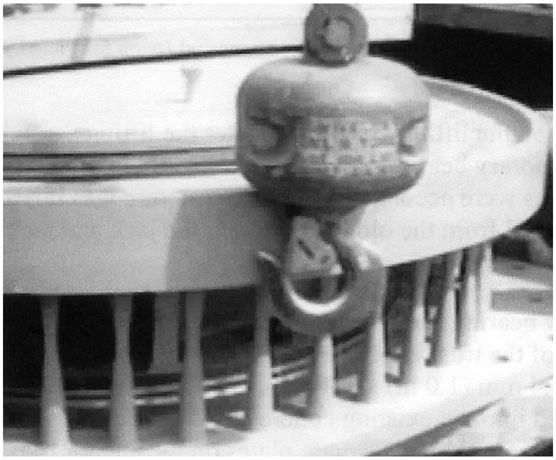
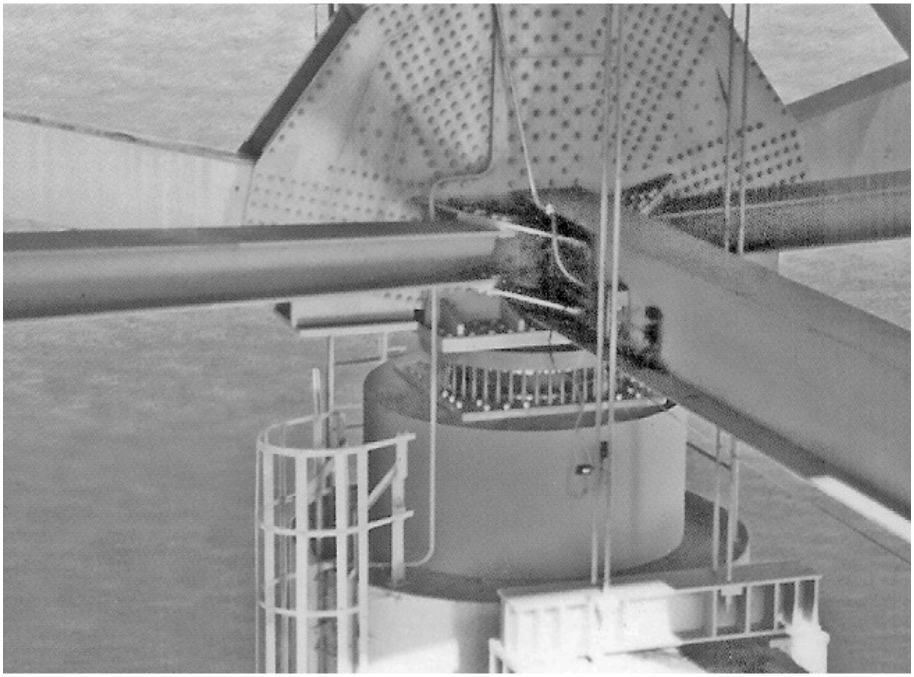


**Figure 17.20** The hysteretic bearing installed at pier 3 of the Marquam Bridge. Image courtesy of the Oregon Department of Transportation



unnecessarily stress the dissipating elements and displace the isolators. In addition, these restrainers serve to impede displacement in the isolators at what were the fixed piers, and also limit transverse movements in the free piers 2 and 5.

In the event of a design maximum earthquake, the sacrificial restrainers, as the name implies, will fail and the STUs at each end of the bridge will lock up. When the STUs are locked, the bridge in effect becomes a multi-span continuous girder bridge (hyperstatic structure), which has proven to be the most effective structure type with regard to resisting earthquakes.

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# Appendix 1
# Manufacturers of STUs

To the best knowledge of the author, shock transmission units (STUs) are manufactured only in the UK, the USA, France, Italy and Germany. The names and addresses of all the manufacturers are given below. Four of these manufacturers have local representatives in India, the details of which are also given, and denoted by an asterisk (*).

**Alga SpA**
Via Olona n.12, 20123 Milan, Italy
Phone: 39 02 48569.1, Fax: 39 02 48569.245, Email: alga@alga.it, Web: www.alga.it

**Colebrand Ltd**
18–20 Warwick Street, London W1R 6BE, UK
Phone: +44 (0)20 74391000, Fax: +44 (0)20 77343358, Email: adeng@colebrand.com,
Web: www.colebrand.com

**D.S. TechStar Inc.**
1219 West Main Cross Street, Findlay, OH 45840, USA
Phone: 1 419424 0888, Fax: 1 419 4245959, Email: engineering@techstar-inc.com,
Web: www.techstar-inc.com

***Z-Tech (India) Pvt Ltd**
J-1867, Chittaranjan Park, New Delhi-110 019, India.
Phone: 011-6227271, 022-7701265, Fax: 011-6227273, Email: ztech@airtelmail.in,
Web: www.ztech-india.com

**FIP Industriale SpA.**
Via Scapacchio, 35030 Selvazzano (PD), Italy
Web: http://www.fip-group.it

**Freyssinet Group**
1 Bis, Rue Du Petit-Clamart, 78148 Velizy, Villacoublay Cedex, France
Phone: 331-46018484, Fax: 331-46018585, Web: www.freyssinet.com

**Jarret**
4 Four Coins Drive, Canonburg, PA 15317, USA
Phone: 1 724 746 1600, Fax: 1 724 7460660, Email: contact@jarret.com

*__METCO__
42B Motilal Basak Gardens, Calcutta-700 054, India
Phone: 033-3582740, Fax: 033-3346308, Web: www.metcocal.com

**Lisega Inc.**
375 Lisega Boulevard, Newport, TN 37821, USA
Phone: 01 423 6252000, Fax: 01 423 6259009, Email: info@lisega.com, Web: www.lisega.com

**Maurer Söhne**
PO Box 440145, D-80750 Munchen, Germany
Phone: 49 89 323940, Fax: 49 89 32394338, Web: www.maurer-soehne.com

*__J. Sons Engineering Corporation Ltd__
E-341 Mayur Vihar, Phase-II New-Delhi-110 091, India
Phone: 011-2476997, Fax: 011-2472547, Web: www.jsonsbearings.com

**Taylor Devices Inc.**
90 Taylor Drive, North Tonawanda, NY 14120–0748, USA
Phone: 1 716 4240888, Fax: 1 716 6956015, Web: www.taylordevices.com

*__Sanfield (India) Ltd__
62 Zone-II, M P Nagar, Bhopal-462011, India
Phone: 0755-576661, 022-7824683, Fax: 0755–576663, Web: http://sanfieldindia.in

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# Appendix 2
# Laboratories for testing STUs

**AEA Technology plc**
Risley, Warrington, Cheshire WA3 6AT, UK
Phone: +44 (0)925253393, Fax: +44 (0)925252285

**Alga SpA**
Via Olona n.12, 20123 Milan, Italy
Phone: 39 02 48569.1, Fax: 39 02 48569.245, Email: alga@alga.it, Web: www.alga.it

**CTL Group**
5420 Old Orcahd Road, Skokie, IL 60077-1030, USA
Phone: +1 847 965 7500, Fax: +1 847 965 6541, Email: amehrabi@ctlgroup.com,
Web: www.ctlgroup.com

**EXOVA Ltd** (formerly **Bodycote Materials Testing Centre**)
See website for contact details of offices around the world: www.exova.com

**FIP Industriale SpA.**
Via Scapacchio, 35030 Selvazzano (PD), Italy
Web: www.fip-group.it

**Taylor Devices Inc.**
90 Taylor Drive, North Tonawanda, NY 14120–0748, USA
Phone: 1 716 4240888, Fax: 1 716 6956015, Web: www.taylordevices.com

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