IABSE Bulletins

Structural Engineering Documents

17

Extradosed Bridges

Mike Schlaich (Editor)
Andreas Apitz
Thierry Duclos
Akio Kasuga
José Romo Martín
Serge Montens
Chithambaram Sankaralingam
Juan Sobrino

International Association for Bridge and Structural Engineering (IABSE)

Structural Engineering Documents

Objective:

To provide in-depth information to practicing structural engineers, in reports of high scientific and technical standards on a wide range of structural engineering topics.

Topics:

Structural analysis and design, dynamic analysis, construction materials and methods, project management, structural monitoring, safety assessment, maintenance and repair, and computer applications.

IABSE Bulletin Board:

Publisher:

The International Association for Bridge and Structural Engineering (IABSE) is a scientific / technical Association comprising members in 90 countries and counting 54 National Groups worldwide. Founded in 1929 it has its seat in Zurich, Switzerland. IABSE's mission is to promote the exchange of knowledge and to advance the practice of structural engineering worldwide. IABSE organizes conferences and publishes the quarterly journal Structural Engineering International, as well as conference reports and other monographs, including the SED series. IABSE also presents annual awards for achievements in structural engineering.

For further Information:

IABSE
Jungholzstrasse 28
8050 Zurich
Switzerland

Phone: +4143 443 97 65
E-mail: secretariat@iabse.org
Web: www.iabse.org

ISBN 978-3-85748-168-0

Publisher:
IABSE
Jungholzstrasse 28
8050 Zurich
Switzerland

Phone:	Int. +4143 443 97 65
E-mail:	secretariat@iabse.org
Web:	www.iabse.org

Table of Contents

Introduction 1

1 **General** 3

1.1 Definition of "Extradosed," Potential, Advantages 3
1.2 History, Bibliographic Report 6
1.3 Formal and Cultural Aspects 7

2 **Conceptual Design** 9

2.1 General Layout and Structural Scheme 9
2.1.1 General 9
2.1.2 Elevation 9
2.1.3 Transverse View 11
2.1.4 Plan View 13
2.1.5 Single Decks or Separate Deck 14
2.1.6 Railway Extradosed Bridges 15
2.2 Deck 16
2.2.1 Deck Material 16
2.2.2 Deck Cross Section 16
2.2.3 Connection Between Pylon, Deck and Pier 18
2.3 Pylons 19
2.3.1 Pylon Layout 19
2.3.2 Pylon Material 20
2.3.3 Pylons Cross Section 20
2.4 Arrangement of Extradosed Cables 22
2.4.1 Extradosed Cable Design 22
2.4.2 Extradosed Cable Layout 22
2.4.3 Concrete Fin Bridges 24

3 **Analysis** 27

3.1 Structural Behavior and Analysis 27
3.2 Initial State 28
3.2.1 Permanent State for Classic Cantilever Construction 28
3.2.2 Permanent State with Concordant Prestress 31
3.3 Ultimate Limit State Considerations for Extradosed Bridges 32

3.3.1 Standard Method 33
3.3.2 Alternative Method 33
3.4 Cable Design 34
3.4.1 Introduction 34
3.4.2 Stay Cable Design 34
3.4.3 Differences in Codes and Regulations 37
3.5 Dynamics of Cables 38
3.5.1 Introduction 38
3.5.2 Dampers 39

4 **Structural Design – The Japanese Experience** 41
4.1 General 41
4.2 Structural Components and Details 41
4.2.1 Deck Cross Sections 41
4.2.2 Pylon Types 42
4.3 Connection Details 44
4.3.1 Stay Cable and Pylon 44
4.3.2 Stay Cable and Deck – Selected Examples 45
4.4 Reinforcement and Prestressing Details 46
4.4.1 Pylon 46
4.4.2 Deck 47

5 **Technology of Extradosed Cables** 49
5.1 Foreword 49
5.2 State of Technology Through Lessons Learned on Projects 50
5.2.1 Extradosed Bridges with Stay Cables Technology 50
5.2.2 Extradosed Bridges with External Prestressing Technology 50
5.3 Durability 51
5.3.1 Design Life and Maintenance 51
5.3.2 Protection 52
5.3.3 Fire 54
5.3.4 Fatigue 55
5.3.5 Dynamics 55
5.3.6 Tolerances 56
5.4 System Installation 56
5.5 Anchorages 56
5.6 Anchorage Layouts at the Pylon 57
5.6.1 Classical Cable stay Anchorages (Type A and B) 57
5.6.2 Steel Tension Elements (Type C) 58
5.6.3 Saddle as Anchorage in Pylon (Type D) 59
5.6.4 An Alternate Layout Between C & D 63
5.6.5 Choice of a Saddle 63
5.6.6 Friction 64
5.6.7 Saddle Fatigue 65
5.6.8 Current Developments 66
5.7 Conclusion 66

6 **Construction Issues** 67
6.1 Introduction 67

6.2 Pylon 68
6.2.1 Introduction 68
6.2.2 Concrete Pylon 68
6.2.3 Steel Pylon 70
6.3 Deck Girder 71
6.3.1 Deck Girder Erection 71
6.3.2 Cable Stay System 81
6.3.3 Camber Control 84
6.4 Case Studies 86
6.4.1 Odawara Port Bridge, Japan 86
6.4.2 Narmada Bridge, Bharuch 86

7 **Cost/Economics** 89
7.1 Introduction 89
7.2 Estimate of Quantities 91
7.2.1 Concrete 91
7.2.2 Reinforcement and Pre-Stressing in Concrete Elements 92
7.2.3 Extradosed Stays Quantities 92
7.3 Comparison to Girder Bridges Built in Balanced Cantilever 94
7.4 Conclusions 95

8 **Bridge Data and Case Studies** 97
8.1 General 97
8.2 Geometric Properties 98
8.2.1 Introduction 98
8.2.2 Length of Main Span 98
8.2.3 Main Span to Pylon Height Ratio 100
8.2.4 Side Span to Main span Ratio 102
8.2.5 Girder Depth 103
8.2.6 Cable Interval 105
8.3 Case Studies 106
8.3.1 Introduction 106
8.3.2 Case Studies 107
8.4 Bridge Data Table 127

9 **List of Symbols** 135
10 **List of Tables** 136
11 **List of Figures** 137
12 **References** 142

Introduction

While the term “Extradosed” was coined in France in the 1980s, the first extradosed bridges were all built in Japan and today more than 200 of them can be found all over the world. Commonly agreed principles or helpful design guidelines about these bridges, however, do not exist in publications.

In 2014 an international group of engineers, all of them experienced in the field of cable-supported bridges were inducted into Working Commission 3 of the International Association for Bridge and Structural Engineering (IABSE) to address this information gap. The members of the group exchanged their views and worked together via e-mails, and meetings in person took place during the IABSE conferences in Geneva 2015, Stockholm 2016 and Vancouver 2017.

Extradosed bridges with their shallow stay cables and stiff decks can be an economic and attractive alternative to girder and cable-stayed bridges for spans between 100 and 250 m.

The structural system of the extradosed bridges can be described as ‘in-between’ or ‘hybrid’ between balanced-cantilever-type girder and cable-stayed bridges. Therefore, there is not “one” narrowly defined extradosed bridge. Rather, there is a smooth transition from girder to extradosed to cable-stayed bridge and this is reflected in this report.

This state-of-the-art report is the collective experience of the group. All aspects that specifically refer to extradosed bridges are covered here. Typical values and, with equal weight, exceptions will be presented. The reader will find helpful information about all aspects that are relevant for designing and constructing such bridges. Not all the experts shared the same opinions. Some of the controversial issues will be highlighted, in order to identify fields for further research. The reader will also find, that in the text certain subjects are described in different ways. Also this reflects the fact that several authors were involved in the preparation of this report.

The report is aimed at not just practicing bridge engineers but also teachers and researchers in the field of extradosed bridges. First the general aspects and the history of this bridge type are presented. Conceptual and Structural Design, Analysis, Cable Technology, Construction issues and Cost Considerations are presented in separate chapters.

Numerous documents on specific subjects which relate to extradosed bridges have been published world-wide. At the end of this document, all the literature known to the group on all subjects that relate to extradosed bridges are summarized. It is a collection of codes, regulations, books, dissertations and technical papers published at IABSE, fib as well as other institutions and journals. The reference numbers in this text refer to this list.

On behalf of the Authors, I would like to thank the External Reviewers who, along with the Bulletin Editorial Board, spent valuable time in reviewing this document: Naeem Hussain, Marcos Sanchez, Roman Geier and Jiri Strasky; and guest reviewers Andreas Apitz, Koneru Bhavani, Werner Brand, Araby El-Shenawy, Kris Mermigas and Hermann Weiher.

Special thanks to contributors and authors of chapters (members of Working Commission 3): Nirmalya Bandyopadhyay, Don Bergman, Alok Bhowmick, Xu Dong, Thierry Duclos*, Akio Kasuga*, Zhao Liu, Serge Montens*, Guido Morgenthal, José Romo*, Chithabaram Sankaralingam*, Robin Sham, Juan Sobrino*, José Turmo and Edo Vonk.
*Chapter Authors

Mike Schlaich, Chairman WC3, IABSE

Chapter

1

General

Mike Schlaich, Germany

1.1 Definition of "Extradosed," Potential, Advantages

Extradosed Bridges are a new bridge form that offer a competitive alternative to more traditional forms such as girders, arches, and cable-stayed systems used in spans ranging from 100 to 250 m. Since its origin in the 1980s, more than two hundred such bridges have been built all over the world.[1] For spans up to 100 m, concrete or composite girders bridges are the most common choice,[2,3] but for spans longer than 100 m, the depth of the boxes above the piers is becoming increasingly unacceptable for aesthetic constructability and cost reasons. For spans longer than 250 m, cable-stayed bridges are usually the most economical solution. Conceptual and structural designs of this bridge type with bridge decks made of steel, concrete, or a combination of the two materials have been widely published.[4,5] For spans shorter than 250 m, cable-stayed bridges often cannot fully exploit their strength and lose their economic advantage.

There are four suggested definitions of the Extradosed Bridge:

- An Extradosed Bridge can be considered as a hybrid concept in the region of transition between girder bridges and cable-stayed bridges. A large number of options are available to designers for configuring such a bridge type to suit specific constraints for a particular project. The large number of Extradosed Bridges built over the last two decades is testament to that.
- Extradosed Bridges are considered as "in-between" girder bridges and cable-stayed bridges. In a cable-stayed bridge, the loads (permanent as well as live loads) are globally carried predominantly by the stay cables. In a girder bridge, loads are carried by shear and flexure of the girder and internal pre-stressed or post-tensioned cables, which produces permanent stresses that act opposite to those produced by self-weight and moving loads.
- With a stiff deck and shallow angled cables, an extradosed girder behaves like a pre-stressed concrete girder although it has similarity in looks with cable-stayed bridges. The shallow angle of the cables ensures that the extradosed cables directly carry only a small portion of the live load. This is the basic behaviour of an Extradosed Bridge. However, the actual configuration of the girder and cables decides how close its behaviour is to a prestressed girder bridge.
- An Extradosed Bridge has its deck partially supported by a system of cables, which are connected to a pylon of small height (pylons have two legs, and masts only one. For simplicity

in this document, only the word pylon will be used). The pylon's height measured above the bridge deck level is between 0.07 and 0.13 L, L being the main span (unlike the classic cable-stayed bridge, where the pylon has a height between 0.2 and 0.25 L), thus making it easy to build. Having this geometrical arrangement, Extradosed Bridge stays have a small inclination with respect to the roadway and, therefore, provide less vertical stiffness to the deck compared with a cable-stayed bridge. Extradosed Bridges are suitable for spans between 100 and 250 m, depending on specific site constrains. For medium spans, they compete with continuous pre-stressed concrete or steel (closed or truss) girders and with arches. For larger spans (longer than 250 m), cable-stayed structures are likely more economical than Extradosed Bridges depending on site conditions.

In *Fig. 1.1*, the three bridge types mentioned above and their typical dimensions are shown. When comparing them, the characteristics of a typical Extradosed Bridge become apparent:

- Shallow cables: They are often anchored closely spaced in groups, which subdivide the main span length L into portions of 0.2 L. Carrying the permanent loads of the bridge with such shallow inclined cables leads to high cable forces and high compression in the girder. At the anchorages, the horizontal components of these cable forces are introduced into the girder, thus contributing to the post-tensioning of the girder. For larger spans the forces increase and pose the risk of overloading the girder. While the cables can be tensioned so that they carry part of the permanent loads (the rest is carried to the supports by the girder itself), due to their shallow inclination, their stiffness is small. Therefore, the cables are hardly activated by live loads. A deck girder of sufficient stiffness is, therefore, a necessary component of an Extradosed Bridge.
- Deck girder stiffness: Most Extradosed Bridges built so far have very bending-stiff and strong girders compared with cable-stayed bridges. They generally carry live loads primarily in

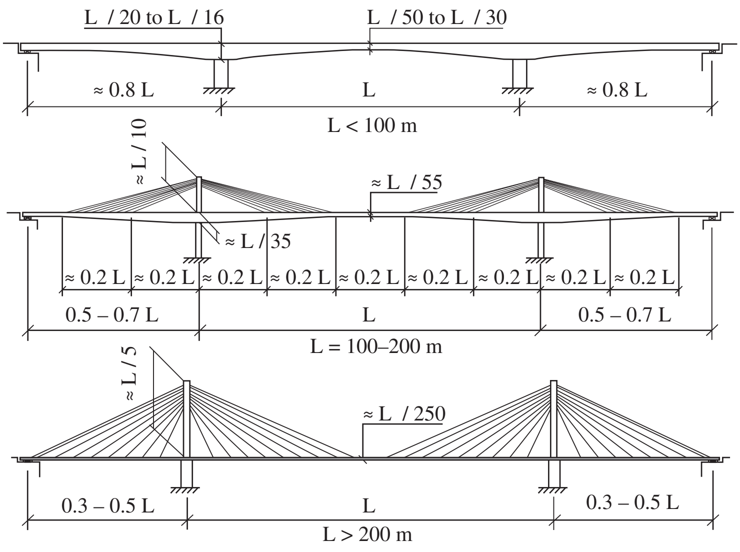


Figure 1.1: Typical dimensions of the three bridges types[6]

bending. For girders of constant depth, typical values are L/40 to L/45. The elegant Sunniberg Bridge by Christian Menn in Switzerland with its slender concrete slab deck is certainly an exception to this.[7]

- Low height pylons: The shallow cables allow for very low height and robust pylons, which often are placed in the centre of the girder (single cable plane), because the already strong box girder decks are also able to take torsion resulting from eccentric traffic loads.
- Deck girder and pylon connection: Often, deck girder and pylons are monolithically connected. This way, the low-height pylon can be made slender and stabilized against buckling more economically than through the cables. Such monolithic connection between the pylon and the deck girder further improves robustness and durability. Some Extradosed Bridges are fully integral, that is, the deck girder and piers are also monolithically connected. For the span lengths considered here, strains due to temperature and time-dependent effects can be handled, at least if the piers are high or longitudinally flexible or the soil is soft. Longitudinal jacking at mid-span can help to mitigate the temperature- and time-dependent effects. This approach has been used successfully in numerous Extradosed Bridges.
- No back stays: It is striking that, contrary to cable-stayed bridges, Extradosed Bridges rarely have backstays to connect the pylon tops to the abutments or anchor piers. Cables are typically anchored in the deck girder short of the abutments or anchor piers, thus lessening the stresses in the cables from stresses due to live load but exposing the deck girder and pylon to bending, which would otherwise typically be resisted by back stays in a cable-stayed bridge.

All these properties, which appear to be disadvantageous to the engineer of cable-stayed bridges, result in live loads basically carried by the deck and, therefore, very small stress changes occur in the cables due to live loads. The cables of Extradosed Bridges are mainly there to carry permanent loads, that is, to reduce bending moments in the deck due to these loads. Actually, in a typical Extradosed Bridge, stress changes due to live loads are between 50 to 100 N/mm^2 as compared to up to 200 N/mm^2 in cable-stayed bridges. Usually, parallel strands are used for Extradosed Bridges, as they can be anchored easily and, depending on the manufacturer, even be replaced strand by strand. Because of the small stress changes in the cables and small dimensions of pylons, saddle systems, which pass the cables through the pylon top, become possible and desirable, especially if the pylon is placed in the middle of the deck girder and a small space is available. More information on saddles is given in Chapter 5.

The small stress changes in the cables due to live loads reduce the risk of fatigue and, therefore, the utilization of the cables can be higher than 45% of the Guaranteed Ultimate Tensile Strength (GUTS) which is usually applied for cable-stayed bridges. This represents a better use of the cable capacity in Service compared with cable-stayed bridges. However, rather than simply raising the allowable stress to say 60% of GUTS, it is desirable to provide structural rationale. Thorough investigation of many existing bridges confirms the fact that there is no clear boundary between extradosed and cable-stayed bridges[8,9,10] and shows that the maximum allowable stresses in service should be based on the level of stress changes due to live load. This issue is dealt with in more detail in Chapters 3 and 5.

What advantages does an Extradosed Bridge have compared with a girder bridge? Neglecting the costs of the bridge pylons, the cost of the deck girder of an Extradosed Bridge can often be less than the cost of a deck girder of a girder bridge. Since concrete accounts for a significant portion of the superstructure cost, Extradosed Bridges are at an advantage as the span increases. While the presence of pylons and stay cables of Extradosed Bridges clearly poses an additional cost compared with post-tensioned cantilever bridges, their shorter height and adoption of saddles instead of anchorages increase their economy over those used for traditional cable-stayed bridges.

What advantage does an Extradosed Bridge have compared with a cable-stayed bridge? Starting with the more obvious, thanks to lower fatigue due to lower stress range in the stay cables, the stay cables can be subjected to a much higher stress and it is possible to use simpler and more economical anchorages. For the same reason anchorages of Extradosed Bridges are typically subject to fatigue test requirements, which are less demanding than those for cable-stayed bridges.

Certain boundary conditions may also lead to the Extradosed Bridge being a preferred option when compared to the traditional post-tensioned cantilever or cable-stayed alternatives. The reduced pylon height has been successfully employed to accommodate height restrictions (e.g. for aviation), aesthetic or other restrictions imposed on the design. The reduced deck depth has been successfully exploited to meet vertical clearance requirements for waterway navigation or for flyovers. There is much freedom to vary the relative stiffness of deck, cables, and pylons to optimize the behaviour of the bridge. This results in a range of structural characteristics lying between an internally post-tensioned balanced cantilever bridge and a cable-stayed bridge, with ample room for innovation and reducing cost for different boundary conditions. As noted above, unlike cable-stayed bridges, Extradosed Bridges do not require back stays to limit the horizontal movement at the top of the pylon caused by uneven live loads, when the short pylon is monolithically fixed to the deck and since the live loads are primarily carried in bending by the deck girder. Thus, Extradosed Bridges are inherently well suited to multi-span structures.

The issues touched upon in the two paragraphs above will be illustrated by numerous examples in Chapter 2.

1.2 History, Bibliographic Report

The wording “extrados prestressed cables” first appeared in the paper by Jacques Mathivat “The recent evolution of prestressed concrete bridges” published in a booklet called “Symposium Paris-Versailles – September 2–4, 1987 – International Association for Bridge and Structural Engineering – Contributions of the French Group,”[11] which was distributed to those attending this IABSE symposium in France. Reading this paper, we understand the original idea of Extradosed Bridges. External prestressing (prestressing tendons not included in the concrete, but located inside a box-girder) was developed during the preceding years for concrete box-girders, starting in USA and France. One of the advantages of external prestressing was the replaceability of the prestressing tendons. At that time, the following prestressing tendon layout was developed for box-girder bridges built by balanced cantilever: internal cantilever tendons located in the top slab, internal continuity tendons in the bottom slab near mid-span, and external draped tendons all along the span. But the cantilever tendons, which represent a large part of the prestressing tendons, were not replaceable. The idea of Jacques Mathivat was that the cantilever tendons could become replaceable, if they were put outside the deck. “Extrados” means in French the top part of a bridge deck, this is why he named these tendons “extrados prestressed cables.” In his paper “Recent developments in prestressed concrete bridges” in FIP Notes 1988/2,[12] Jacques Mathivat used the wording “extradosed prestress.” Then the wording “Extradosed Bridge” appeared. This last wording is less explicit than the original one, because we lose the explanation about the fact that the prestressing cables are extradosed.

Jacques Mathivat intended to propose this concept for the Arrêt Darré bridge, the Mirabeau bridge, and the Joinville bridge, all of them in France. None of them was built as an Extradosed Bridge.

The first “real” Extradosed Bridge to be built was Odawara bridge[13] in Japan, completed in 1994. It was built by a joint venture of Sumitomo Construction Company Ltd. and Kajima Corporation. It should be noted that the 1926 Tempul Aqueduct by Eduardo Torroja, one of the first examples of a modern cable-stayed bridge, with its shallow angle cables and heavy deck, formally also qualifies as an Extradosed Bridge.

1.3 Formal and Cultural Aspects

Economy alone will not make a bridge successful. What counts in the end will be its usefulness and its appearance. Each bridge is a prototype and its design stems from the local context and constraints which are always different.

In some countries or locations codes specify very high live loads. Elsewhere, such as in Japan, it is the seismic loads which often govern the design and in places like Hong Kong it might be the typhoon winds. It can also be severe winter conditions as found in Canada or the heat of the Nubian desert which leads to the final solution. In India the sandy river soils combined with the strong scour caused by monsoon rains have led to a culture of well and caisson foundations while in other countries the construction industry relies on pile foundations for similar conditions.

In some countries precast segmental construction is common whilst in others such as Germany it is not even allowed. High strength structural steel is punished with high import taxes in one country and produced economically in another. Sometimes bridge decks are partly welded on site, others are welded segment-wise in a shop and field-bolted, the practice also depending on local experience and customs.

It is obvious that, for these reasons, bridges will not look the same everywhere (which is good) and naturally this also applies to Extradosed Bridges. They all look different for many reasons but they will only look good if well trained engineers treat them with care, if each detail is designed consciously and if the client pays a fee that allows for spending the time needed for this important task.

Extradosed Bridges with their shallow-angle fans of cables harmoniously distributed along the deck can look very elegant. As opposed to cable-stayed bridges, the heavy deck girders of Extradosed Bridges, however, run the risk of looking clumsy especially if they are of constant depth and two cable planes are anchored in strong outer concrete webs. Concrete barriers at the edge of the deck to protect the cables from vehicular impact make the problem even worse. If the cables are anchored in one central plane, it is easier to hide the heavy deck in the shadow of slender cantilevers. “The more slender, the better” is not necessarily the message here, but care must be taken. The same applies to the short pylons. They must be carefully shaped in order not to look too heavy. It should also be noted, that the cable anchorages at the pylons are rather close to the users and, therefore, well visible. The monolithic connections of deck girder to pylons or even to the piers below the deck are an advantage of the Extradosed Bridge as less unsightly joints and bearings are needed. All these issues must be considered from the very start of the design. Adding ornament at the end to an ill-conceived concept will not rescue it.

Are there cultural differences which affect the appearance of an Extradosed Bridge? The perception of beauty differs from country to country. When a European engineer talks about “elegance,” which can be defined as effortless beauty, does he mean the same as his or her Arab colleague who would use the word أن اق ة (anaka)? In Japanese and Chinese, the word for elegance 優 雅 is the same and just pronounced differently, “youga” in Japanese “you ya” in Chinese. These interesting questions are beyond the scope of this discourse but we should bear them in mind when looking at the examples given in the next chapter.

Chapter

2

Conceptual Design

Serge Montens, France

2.1 General Layout and Structural Scheme

2.1.1 General

As for cable-stayed bridges, there is a wide variety of possible solutions for the general layout, static scheme, deck type and pylons/masts of Extradosed Bridges. The typical value for the main span length of Extradosed Bridges is between 100 and 250 m.

The general layout of Extradosed Bridges will be described in this section. Decks and pylon/mast will be described in greater detail in Sections 2.2 and 2.3. For reasons of simplicity, only the term pylon will be used for the rest of the document even so sometimes mast, that is, single-leg supports would be correct.

2.1.2 Elevation

Although most Extradosed Bridges have two or more supports with pylons along the bridge (i.e. pylons located above two different piers), they may have only one (i.e. a pylon located above only one pier), or also multiple pylons (i.e. pylons located above more than two piers).

Bridges with only one pylon are not frequent. They generally have a symmetrical layout for the spans and the extradosed cables (Miyakodagawa Bridge,[14] Saint-Rémy-de-Maurienne Bridge in France[15]). However, the layout can be asymmetrical, continuous with another span on one side only, for special topographical situations (Yumekake Bridge in Japan,[16] *Fig. 2.1*). The design of an Extradosed Bridge with only one pylon can be a valid solution for an asymmetrical topographical layout, or if it is impossible to place an intermediate pier in two adjacent long spans. It could also be a good solution, for example, in case of an island in the middle of a river.

Bridges with pylons located on two piers are the most frequent type. Generally, they have three spans. Side spans are then approximately 50–70% the length of the main span. If the side spans are longer, the bending moments will be excessive. If they are shorter, there could be some uplift reaction at the end support. If the bridge is longer, it is also possible to create a continuity between

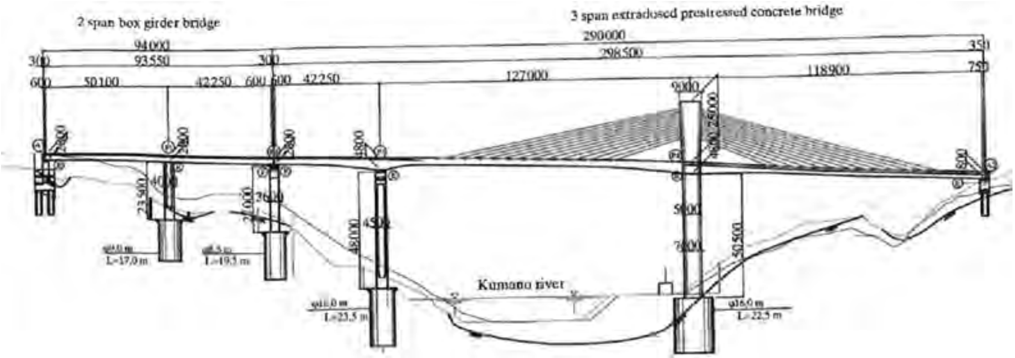


Figure 2.1: Yumekake Bridge[16]

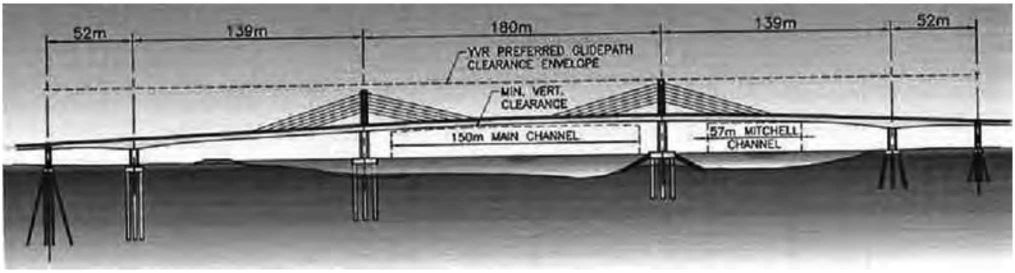


Figure 2.2: North Arm Vancouver Bridge[17]

a three-span Extradosed Bridge and the adjacent spans (North Arm Vancouver Bridge,[17] Canada, *Fig. 2.2*).

Longer Extradosed Bridges may have pylons located on more than two piers. Such multi-span Extradosed Bridges do not have the same design problems as multi-span cable-stayed bridges, consisting of high deck bending moment when only one long span is loaded, because the bending stiffness of an Extradosed deck is higher than that of a cable-stayed deck.

Some Extradosed Bridges have pylons located on many piers (Second Vivekananda Bridge[18] in Kolkata with 8 piers, the third Narmada Bridge in Gujarat with 10 piers, Arrah-Chhapra Bridge in Bihar with 16 piers, all of them in India). In this case, the deck can be either continuous along the full length, if this is not too long, or with intermediate expansion joints located at mid-span of one or several spans (e.g. every third span). These intermediate expansion joints require maintenance. It is necessary also to check the rotation under live loads and long-term dead loads at these expansion joints. To prevent excessive angular deformation at the joint, sometimes some steel beams are placed inside the box-girder deck on both sides of the expansion joint (with bearings between these steel beams and the box-girder), in order to create a moment-transmitting connection between both cantilevers, but this requires special maintenance. Another solution is to place an intermediate pier at the expansion joint (in the centre of the corresponding span), if this is acceptable by other constraints (e.g. clearance over a navigation channel).

2.1.3 Transverse View

Pylons for Extradosed Bridges can be either “lateral” or “central.” “Lateral” means that there are two pylons located on either side of the deck girder. “Central” means that there is only one pylon, located at the centreline of the deck girder. Both options have advantages and disadvantages (*Table 2.1*).

Layout	Lateral	Central
Advantages	Allows non-torsionally stiff deck	Better appearance
Disadvantages	Needs more extradosed cables, visually more obtrusive	Needs torsionally stiff deck, more difficult access for maintenance

Table 2.1 Advantages and disadvantages of lateral and central position of the pylons

With central pylons, there is only one central plane for extradosed cables, so the view of the bridge for the user is less obstructed by extradosed cables. Cable fatigue can also become an issue for the lateral case with low torsional stiffness. Even though the stress change due to live loads in the extradosed cables is small, the following should be considered:

- If the suspension is central, when one lane is loaded with fatigue loading F (generally, according to most design codes, only one lane on the deck should be loaded with fatigue load, whatever the total number of lanes) and the central cable area is A, then the fatigue stress is F/A.
- If the suspension is lateral and if the centre line of the loaded lane is located at say 30 to 70% distance from the deck edges, the fatigue load going to the nearest side, to the lateral extradosed cables plane, is 0.7 F. As the cable area on each side is 0.5 A, the fatigue stress is then 0.7 F/0.5 A = 1.4 F/A, which is more than with the central location of the cables.

In the case of lateral pylons, they are sometimes connected together by a transverse beam located at the top or at an intermediate level, which provides a frame action (Shin-Karato Bridge[19] in Japan (*Fig. 2.3*), Mandaue Bridge[20] in Philippines). This is generally not favoured for aesthetic reasons and is not structurally necessary except if the deck girder is curved in plan with a small radius, in which case high transverse forces, coming from the extradosed cables, are imposed on the pylon.

Figure 2.3: Shin-Karato Bridge[19]

For wide decks supporting both a railway and a highway, it is possible to place twin pylons laterally within the deck, between the railway located centrally and the two highway carriageways located outside (Vidin-Calafat Bridge[21] between Bulgaria and Romania, *Fig. 2.4*). Sidewalks can also be located outside the lateral pylons (Ptuj lake Bridge in Slovenia, Pakse Bridge[22] in Laos). The advantage of these layouts is to reduce the transverse bending effects in the deck girder.

For the Warta River Bridge, the deck is made from three longitudinal girders (*Fig. 2.5*), so it is logical to have a central pylon and two lateral pylons located above the longitudinal girders. We can note that, in order to locate the extradosed cables in vertical planes (to prevent any permanent transverse bending of the pylons), the lateral longitudinal girders are moved slightly inwards, instead of being aligned with the lateral pylons, so that the extradosed cable anchorages

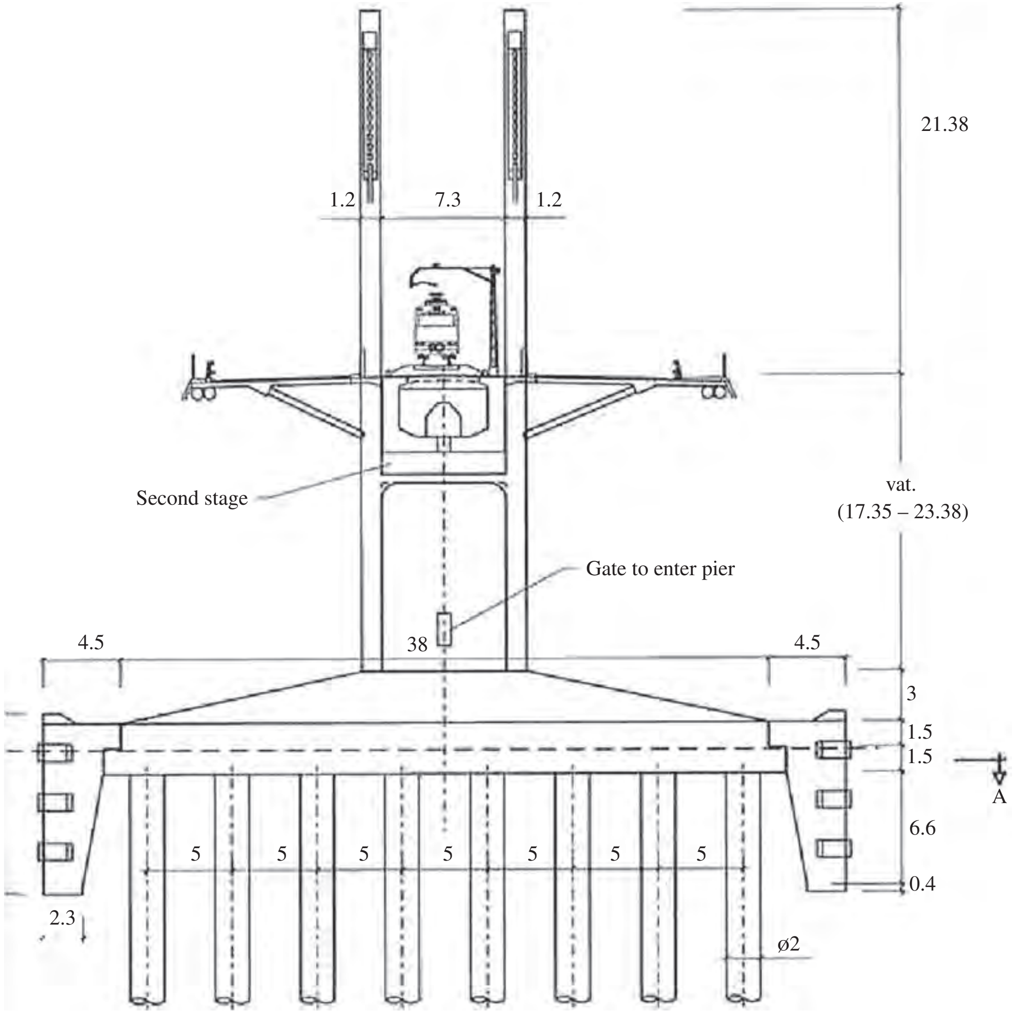


Figure 2.4: Vidin-Calafat Bridge cross section[21]

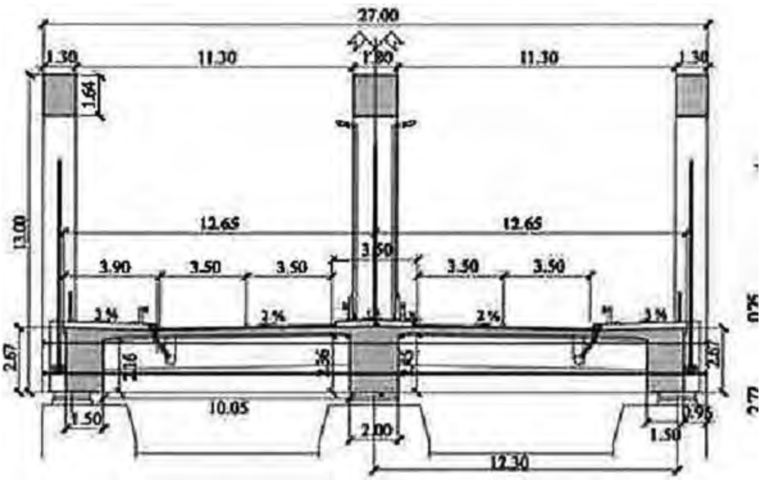


Figure 2.5: Warta River Bridge[23]

do not interfere with the functional requirements of the bridge and in such a way that the anchorages create a fascia on the face of the edge girders. It would also have been possible to have the lateral longitudinal girders aligned with the pylons, with an increased width, incorporating the extradosed cables anchorages.

Since the girder depth of an Extradosed Bridge is already relatively deep, coupled with the common use of box sections, an appropriate cross-section is already available for central support. On the other hand, the height of the pylons is small, which means they are not as costly as the pylons of a cable-stayed bridge. Thus, the designer should feel free to choose the support planes that best meet the functional requirements of the bridge. This contrasts with cable-stayed bridges, where the decision is an optimization between the costs of transverse bending and the cost of an additional plane of cables and an extra pylon.

2.1.4 Plan View

Extradosed Bridges can accommodate a curved deck layout in plan. Some examples are given as follows (L is the main span, and R is the curvature radius):

- Ptuj lake Bridge, Slovenia: L = 100 m R = 460 m.
- Second Agatsumagawa Bridge,[24] Japan: L = 167 m R = 600 m.
- Shin-Karato Bridge,[19] Japan (*Fig. 2.6*): L = 140 m R = 400 m.
- Sunniberg Bridge,[7] Switzerland: L = 140 m R = 512 m.

Similarly to a cable-stayed bridge, the plan curvature will create deck lateral bending moments, due to the transverse components of the extradosed cables. There will also be large transverse bending moments in the pylon. The deck girder and pylon must be checked for these lateral and transverse bending moments, both in terms of strength and displacements.

Conflicts between the highway or railway clearance and the extradosed cables, caused by the curve, must be checked carefully.

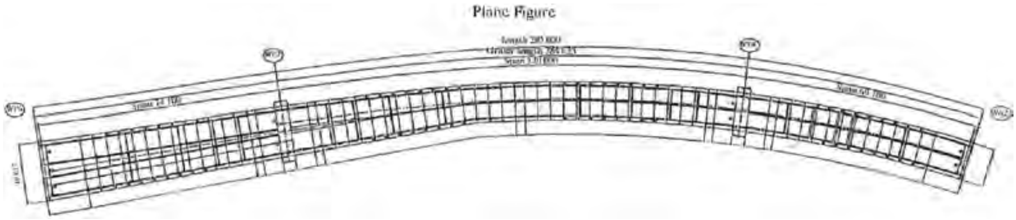


Figure 2.6: Shin-Karato Bridge[19]

Although most Extradosed Bridges are not skewed, it is possible to design an extradosed cable with a skew (Korong Bridge[25] in Hungary, *Fig. 2.7*). The transverse analysis of the deck should be studied carefully, because the extradosed cable anchorages in the deck will not be located in the same transverse section on either edge of the deck.

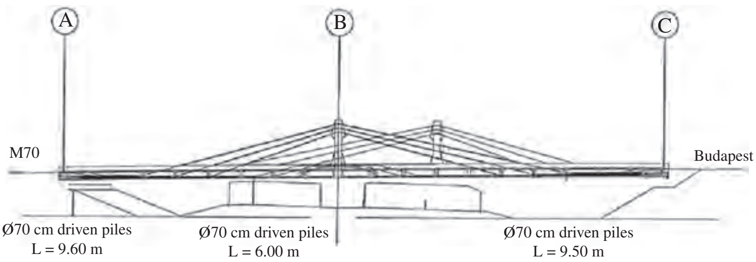


Figure 2.7: Korong Bridge[25]

2.1.5 Single Decks or Separate Decks

For Extradosed Bridges, the configuration of the deck and the positions of the pylons transversely are important considerations. For wide decks, some countries (e.g. Germany) prefer to build motorway bridges with two separate decks, in order to be able to replace one more easily without completely interrupting the traffic. But for economic reasons (smaller initial building cost), it is generally preferred to build a single wide deck, with a central pylon (Keong An Bridge,[26] Korea, *Fig. 2.8*), or with lateral pylons (St Croix Bridge,[27] USA), or with both central and lateral pylons (Pearl Harbour Memorial Bridge[28] in USA, Warta River Bridge[23] in Poland).

The Keong An Bridge utilizes a central support (extradosed cables located at the centre of the deck width). The pier is composed of a central shaft located below the pylon and, because the deck is very wide, two laterally inclined shafts assist in carrying the loads from the deck to the foundations. The three shafts are connected to a common pier cap, which balances the transverse horizontal component of the loads.

Sometimes two separate parallel Extradosed Bridges have been built (Shin-Karato[19] and Tsukuhara[29] Bridges, Japan, *Fig. 2.9*). In this case, the twin decks accommodate the highway which enters a tunnel with two separate tubes near the end of the bridge.

It is also possible to support these two decks with three pylons placed transversally, one of them located between the two decks (Miyakodagawa Bridge[14] in Japan, *Fig. 2.10*). This concept minimizes the transverse bending of the deck.

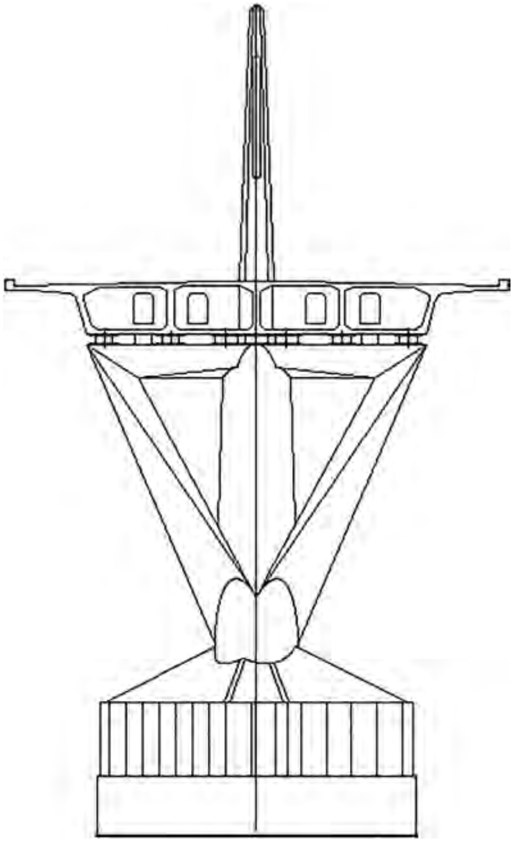


Figure 2.8: Keong An Bridge[26]

2.1.6 Railway Extradosed Bridges

Although most Extradosed Bridges carry roads, there are a few Extradosed Bridges which carry heavy rail or commuter rail (e.g. Shinkotonikouka Shinkawakadou Bridge[31] and Yashiro Bridge in Japan, North Arm Vancouver Bridge[17] in Canada, Moolchand Bridge[32] in Delhi, India, *Fig. 2.11*), or combined highway-railway Extradosed Bridges (Vidin-Calafat Bridge,[21] between Romania and Bulgaria). Similar to road bridges, the risk of trains, in this case, hitting the extradosed cables in case of derailment must be considered: concrete barriers or steel rails can be used to protect the extradosed cables.

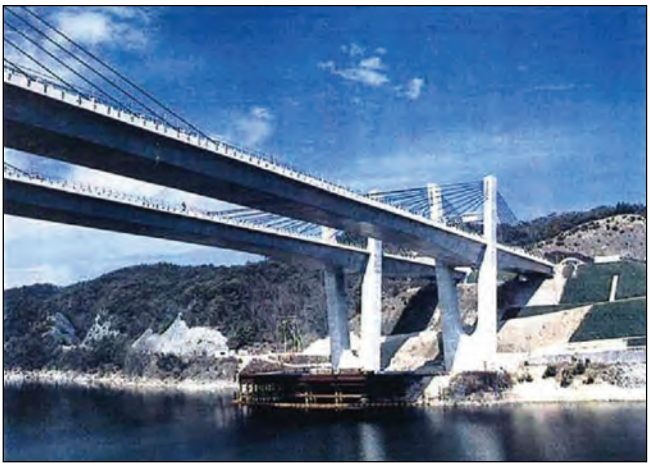


Figure 2.9: Tsukuhara Bridge[29]

2.2 Deck

2.2.1 Deck Material

Most Extradosed Bridges have a concrete deck. However, it can be made of steel, or composite steel–concrete (Riga Southern Bridge in Latvia, Himi Bridge[33] in Japan with corrugated steel webs), or even hybrid, with a concrete part and a steel section placed longitudinally (Japan-Palau Friendship Bridge[34] in Palau, *Fig. 2.12* and Ibigawa and Kisogawa Bridges[28] in Japan, *Fig. 2.13*). As with cable-stayed bridges, the longer the span, the more advantageous it is to reduce the self-weight of the deck by using steel instead of concrete. Therefore, concrete is generally used up to 180 m span. For spans longer than 200 m, composite or steel may be used, sometimes with steel only for the central portion of the span.

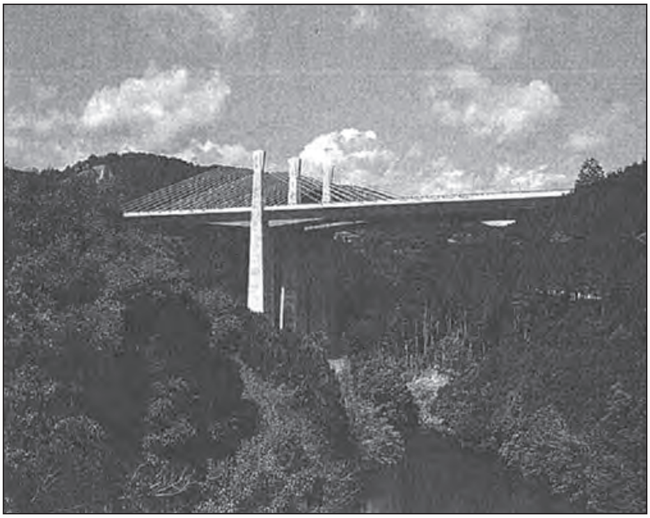


Figure 2.10: Miyakodagawa Bridge[30]

Figure 2.11: Moolchand Bridge[32]

2.2.2 Deck Cross Section

Two families of deck cross sections can be used:

- Decks with lateral beams or slab decks (only if there are lateral extradosed cables, in order to provide sufficient torsional stiffness).
- Box-girders (with lateral or central extradosed cables).

Typical conventional sections are described in Chapter 4 (see also *Fig. 4.1*). The deck can also be U-shaped if there are lateral extradosed cables (Dien-Bien-Phu Bridge[36] on Ho-Chi-Minh City metro in Viet-Nam, *Fig. 2.14*).

The choice of the deck type depends on the functional requirements of the cross section, on the deck width, and potentially on the preferred type of deck for the adjacent spans on both sides of the Extradosed Bridge, for aesthetic continuity.

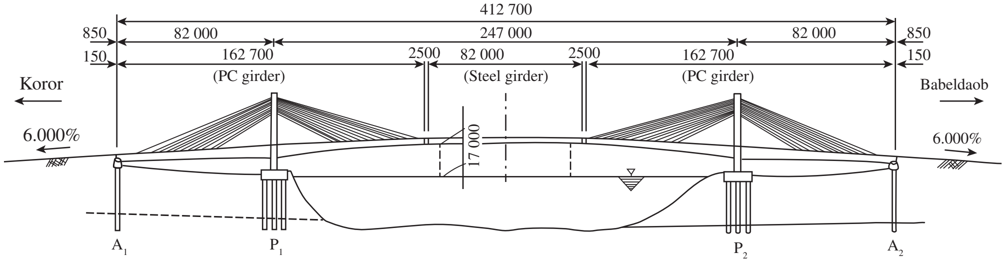


Figure 2.12: Japan-Palau Friendship Bridge[34]

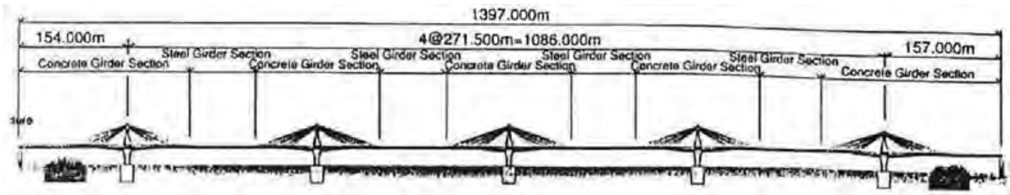


Figure 2.13: Ibigawa Bridge[35]

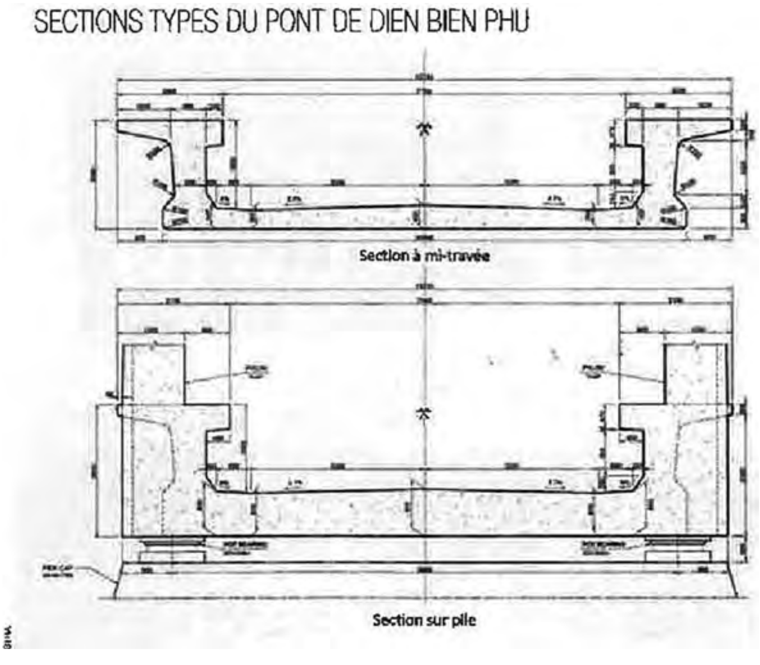


Figure 2.14: Dien-Bien-Phu Bridge[36]

The deck can have a constant or a variable depth, deeper near the piers. The first Extradosed Bridges that have been built generally had a variable deck depth. This can be considered as logical, as those bridges were built by cantilever method. However, it is possible to design Extradosed

Figure 2.15: Vidin-Calafat Bridge[21]

Bridges with a constant depth. The typical value of the ratio of girder depth to main span length is about 1/45 to 1/55 in the main part of the span, and 1/30 to 1/35 at the main pier location, if the deck has a variable depth.

There is a necessary relationship between the deck bending stiffness and the stiffness of the supporting cable system. The smaller the deck's bending stiffness, the greater must be the stiffness of the cable system, consisting of pylons and extradosed cables. The stiffness of this system can be enhanced by increasing the height of the pylons and the size of the extradosed cables.

Some projects include two longitudinally inclined struts supporting the deck from below, at some distance from the piers (Vidin-Calafat Bridge[21] between Bulgaria and Romania, *Fig. 2.15* and Bridge over the Guadalquivir river in Cordoba, Spain). This allows the span to be increased, without increasing the deck depth.

For railway bridges, deflection criteria under live loads are generally strict, so the deck stiffness typically needs to be much higher than for highway bridges. The dynamic behaviour must also be checked to confirm track safety and passenger comfort. Cable fatigue will be of more concern on an Extradosed Rail Bridge than an Extradosed Road Bridge. More generally, if the deck self weight and the superimposed dead loads are low, and if the live load are comparatively high, cable fatigue can govern the design.

Similar to cable-stayed bridges, when designing the deck, attention must be paid to access, for tensioning and replacing extradosed cables: access for jacks, and to new cables at the anchorage points.

2.2.3 Connection Between Pylon, Deck and Pier

There are many possibilities: pylon monolithic with pier but deck on bearings, a monolithic pylon-deck on bearings at the pier, and pylon-deck-pier monolithic. Examples for the latter are Odawara[13] and Tsukuhara[29] Bridges in Japan and Sunniberg Bridge[7] in Switzerland. The main parameters to decide on bearings or not are the same as for classic concrete bridges built by balanced cantilever. The avoidance of bearings is of course in line with the concept of durability, since it reduces maintenance, and no bearing replacement will be necessary over the life of the structure. In the case of an Extradosed Bridge with a single pylon located longitudinally on one pier, it is preferable to make the deck girder, the pylon and the pier integral, making this location the point of longitudinal fixity for the bridge. In the case where bearings are provided between the deck girder and the pier, it is possible to use two rows of bearings in order to transfer longitudinal bending moment to the pier (Povazska Bystrica Bridge,[37] Slovakia, *Fig. 2.16*) and at the same time allow longitudinal displacement of the deck with longitudinally sliding bearings. This arrangement can be used to minimize or eliminate internal forces between multiple piers which result from temperature and time-dependent deck effects.

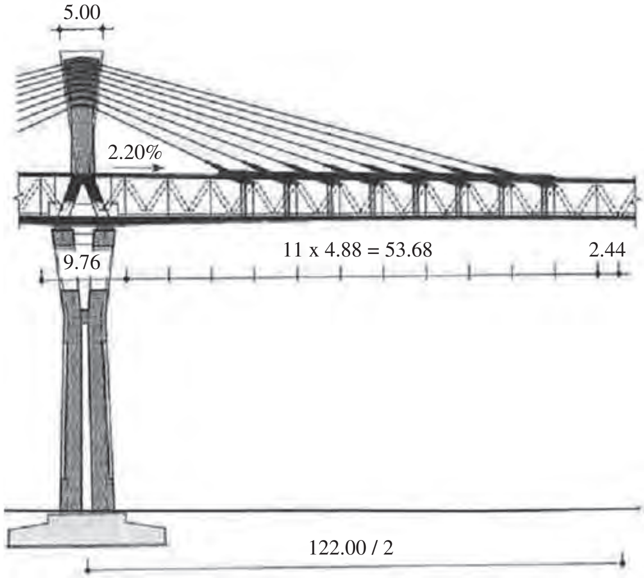


Figure 2.16: Povazska Bridge[37]

2.3 Pylons

2.3.1 Pylon Layout

Pylons are generally vertical. Pylon located at the edge of the deck, could be inclined outwards for aesthetic reasons (bridge over the Segre river in Lerida, Spain; Sunniberg Bridge,[7] Switzerland, *Fig. 2.17*). The extradosed cables are then transversally inclined, which produces a transverse tension in the deck, which must be considered for the deck transverse design. The pylon transverse inclination can be extended to the form of the pier below the deck, again in order to improve the aesthetics of the bridge. This arrangement has been used for several of the Japanese Extradosed Bridges. This transverse inclination of extradosed cables increases the clearance/safety against vehicle impact on the extradosed cables. In case of a deck that is curved in plan, it allows in obtaining the necessary clearance to traffic.

The ratio of pylon height to main span length can vary between 1/15 and 1/8, but is most typically between 1/12 and 1/10. For railway bridges, due to stringent deflection criteria, it is necessary to have more vertical stiffness, so it is recommended to choose a ratio between 1/10 and 1/8.

For Extradosed Bridges with pylons located on two piers (classical case), both pylons are generally of the same height. However, it is possible to have two pylons of differing heights in order to optimize the structure for special topographic constraints (Ravine des Trois Bassins Bridge,[38] France, *Fig. 2.18*).

Some special pylon layouts have been used, such as two vertical or longitudinally inclined legs placed in a longitudinal plane, with a wall or some struts between them, including the extradosed cable saddles (WD22 Bridge[39] near Zgorzelec in Poland, the Second Agatsumagawa Bridge[24] in Japan, *Fig. 2.19* and Doushan Bridge in Taïwan).

2.3.2 Pylon Material

Most pylons for Extradosed Bridges are in concrete. As the pylons are mainly in compression, it can be interesting to make them in high strength concrete, in order to decrease their cross-sectional dimensions. There are some steel pylons (Lyna river Bridge[39] in Olsztyn and WD22 Bridge[39] near Zgorzelec in Poland, *Fig. 2.20*, Waschmühl valley Bridge[40] in Germany, Southern Bridge[41] in Riga, Latvia), and composite steel–concrete pylons (North Arm Vancouver Bridge,[17] Canada). The advantage of steel pylons is the reduced weight and the possibility, if they are not too tall, to prefabricate them and set them in place quickly with a crane.

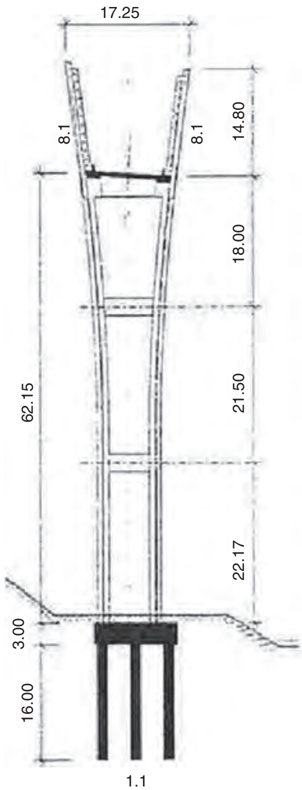


Figure 2.17: Sunniberg Bridge cross section[7]

2.3.3 Pylons Cross Section

The cross section of the pylon can be in any shape. Generally, its longitudinal dimension (the dimension measured in the direction of the bridge axis) decreases upwards, following the decrease of the longitudinal bending moment due to the differential tensions of extradosed cables in the main and side spans under live loads. Pylon longitudinal dimensions tend to be governed by the saddle or double anchor dimensions. If extradosed cables are deviated in the pylons by saddles, and if the longest cables have a larger cross section, they sometimes need a larger curvature radius. In this case, due to the minimum saddle radius requirement, it can be necessary to increase the longitudinal dimension of the top part of the pylon, where the saddles are located, in order for the saddles to fit into the pylon section.

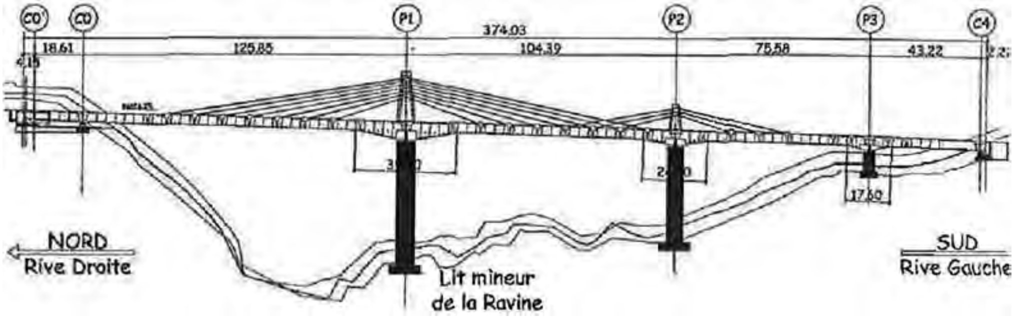


Figure 2.18: Ravine des Trois Bassins Bridge[38]

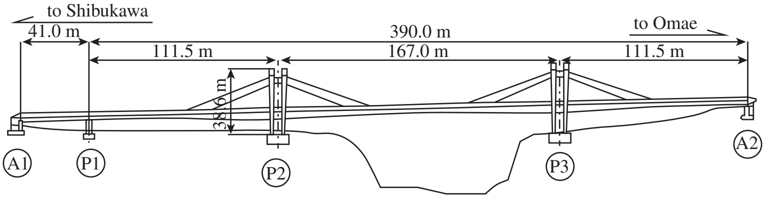


Figure 2.19: Agatsumagawa Bridge[24]

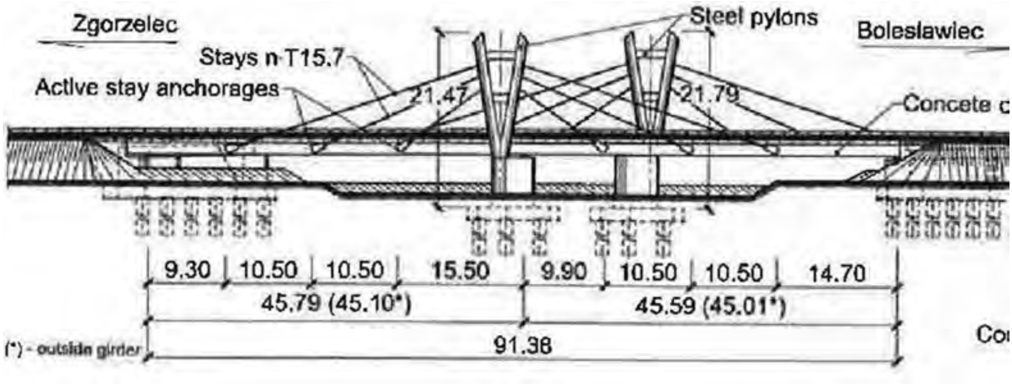


Figure 2.20: WD22 Bridge near Zgorzelec[39]

In the transverse direction (perpendicular to the bridge axis), the pylon dimension is generally constant. However, it sometimes also decreases upwards, following the decrease of the transverse bending moment under wind loads. The minimum width is about 1.2 m, in order to allow enough concrete on both sides of the saddle pipes. In this case the cross-sectional area of the pylon at the saddle can be critical for the design. Steel–concrete composite sections can also be used.

If the deck has a plan curve, the pylons will be subjected to transverse forces coming from the extradosed cables. It is then necessary to check the transverse behaviour of the pylons (bending,

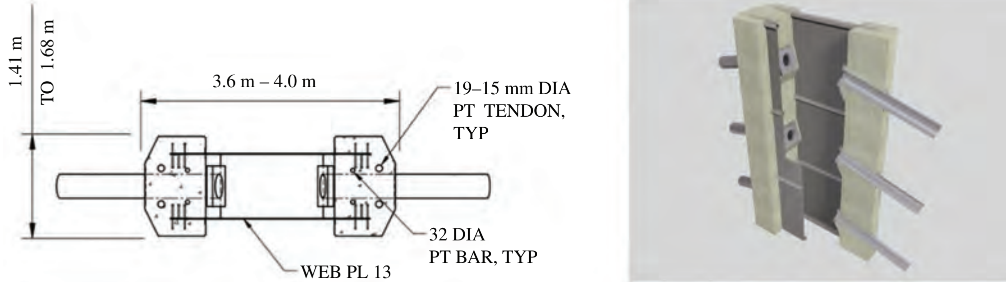


Figure 2.21: Pylon cross section of North Arm Vancouver Bridge[59]

shear) under the dead load and live load transverse forces. It may be necessary to provide the pylon with some vertical pre-stressing tendons with transverse eccentricity, to resist this transverse bending.

For extradosed cables that have anchors located at the pylon (no saddles), it is necessary to pay attention to the tensile forces generated in the cross section. The North Arm Vancouver Bridge in Canada[17] utilized a composite cross section in the area of the cable anchorages with steel webs to resist the tensile forces generated by opposing cables with concrete anchorages to resist vertical loads (*Fig. 2.21*).

In the case where anchorages are provided in the pylon, it is also necessary to provide the access necessary for jacking and replacement of the extradosed cables in case this should be required over the life of the structure. To minimize the access requirements in the cross section it may be also decided to tension the cables at the deck and not at the pylons.

2.4 Arrangement of Extradosed Cables

2.4.1 Extradosed Cable Design

Extradosed cables carry less of the deck loads compared with cable stay bridges. The percentage of the permanent deck load carried by the extradosed cables is typically in the range of 60–70%, the remaining part being carried by the deck in bending. However, it is possible to design Extradosed Bridges with a higher percentage of the permanent deck load carried by the extradosed cables.

2.4.2 Extradosed Cable Layout

Extradosed cables can have a harp pattern (parallel cables) or a semi-fan pattern (*Table 2.2*).

Layout	Harp	Semi-fan
Advantages	Visually better, especially with lateral pylons and two planes of cables; constant size of extradosed cables; consistent details at the deck and pylon extradosed cable connections	Provides more compact anchorage zone which is more efficient
Disadvantages	All cables as shallow as the longest one: less global stiffness Short cables at pylon tend to be inefficient The pylon will bend severely in case of cable failure	With two planes of lateral cables, various angles of cables create some visual confusion

Table 2.2 Advantages and disadvantages of harp and semi-fan patterns

Figure 2.22: Pure fan cable arrangement – Riga Southern Bridge

As for cable-stayed bridges, the harp pattern requires more steel in the extradosed cables to achieve the same vertical stiffness from the cables as compared with a semi-fan arrangement, for a given pylon height. The additional cable force (their horizontal component) increases the pre-stress in the girder compared to a semi-fan arrangement.

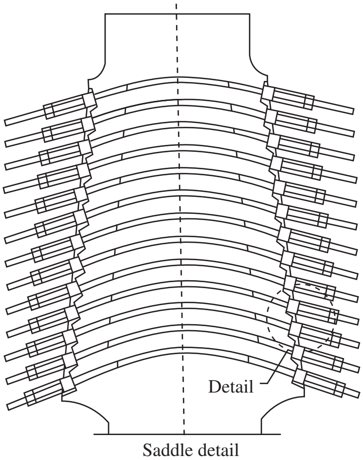


Figure 2.23: Vidin–Calafat Bridge[21]

The Riga Southern Bridge in Latvia (*Fig. 2.22*) has a pure fan layout, with all the extradosed cables deflected by saddles located at the top of the pylons, but this type is not common.

Generally, the semi-fan pattern is preferred, as it provides for ease of construction and a more compact and efficient anchorage zone for the cables or saddles. This is the case for Vidin–Calafat Bridge between Bulgaria and Romania (*Fig. 2.23*).

The vertical spacing between the extradosed cables at the pylon is typically between 0.5 and 1.0 m and is governed by the saddle or anchorage size. For a given pylon height, the shorter spacing gives globally a larger eccentricity for extradosed cables, but the beneficial effect of eccentricity is lost due to the increased cable inclination and therefore lower horizontal force component from the cables. The distance between the extradosed cable anchorages on the deck is generally between 4 and 7 m for a concrete deck. However, this cable distance should be chosen as a function of deck width, extradosed cable capacity, segment length during construction and the number of extradosed cable planes. The choice of cable spacing may also be influenced by the expected construction method whether it be precast segmental, cast in situ segmental or other methods. Some studies[42] have shown that the most economical design corresponds to the first extradosed cable anchorage being located at 20% of the main span length, measured from the pylon location. The

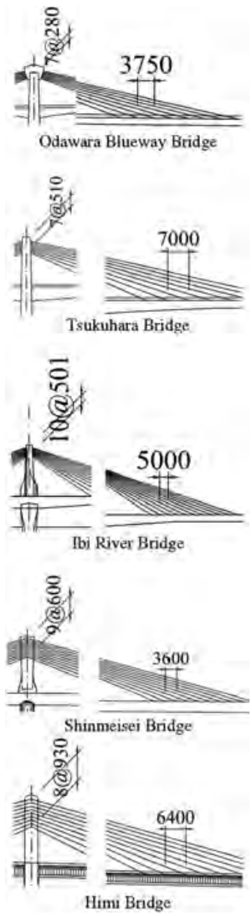


Figure 2.24: Various extradosed cable arrangements

first extradosed cables are often started away from the pylon for economy. Cantilever internal tendons in the deck section can adequately resist the bending moments in this part of the deck, but short extradosed cables have higher relative costs associated with anchorages and corrosion protection. Around 20% (and not more than 50%) of the deck within the middle of the bridge span should not be supported by extradosed cables.

If the pylons are located centrally (in the centre of the deck width), there can be one plane of extradosed cables. But there can be also two parallel planes of extradosed cables attached to the same pylon, in order to decrease the size (diameter) of the cables, and facilitate the replacement of any cable without totally interrupting traffic. In such a case, the risk of wake galloping, caused by twin cables close to each other, must be considered.

Figure 2.24 shows the cable layout of typical Extradosed Bridges. Odawara Blueway Bridge and Tsukuhara Bridge use saddles and the rest of them use steel box anchorages in pylons. The cable spacing at the pylon is typically 0.5–1.0 m. The shorter distance gives larger eccentricity for extradosed cables. And the cable distance at the girder is affected not only by structural considerations but also by construction-related factors as well.

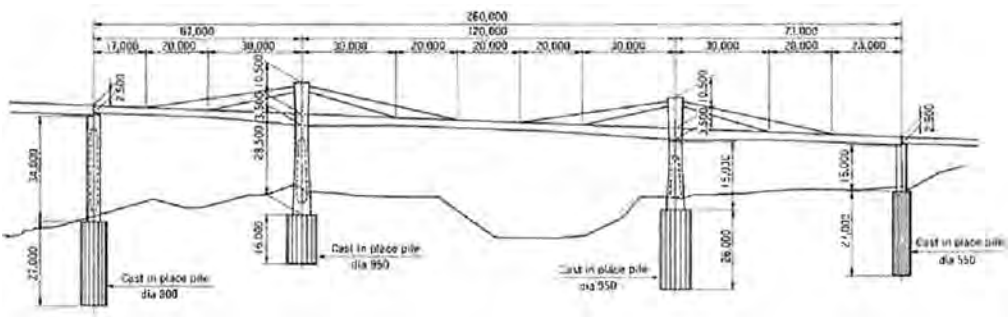


Figure 2.25: Okuyama bridge[44]

2.4.3 Concrete Fin Bridges

For some Extradosed Bridges, the extradosed cables have been encased in concrete fins (Ganter bridge[43] in Switzerland, Barton Creek bridge in the USA, Third Mekong River bridge between Laos and Thailand, Okuyama bridge[44] in Japan, *Fig. 2.25*).

Figure 2.26: Second Agatsumagawa bridge[24]

Although the aesthetic is questionable, this has sometimes been done, particularly for railway bridges, in order to increase the vertical stiffness of the bridge to fulfil the more stringent deflection requirements under railway live loading. This is particularly relevant in the case where the deck for other reasons has an open U-shaped section with low vertical and torsional stiffness (Second Agatsumagawa bridge,[24] *Fig. 2.26*, Narusegawa bridge[45] in Japan and Pragati Maidan bridge[46] in Delhi, India, *Fig. 2.27*).

Figure 2.27: Pragati Maidan bridge[46]

This specific type of Extradosed Bridge is sometimes called a fin back or cable panel bridge. When applying the balanced cantilever construction method, the extradosed cables can be tensioned at a lower value before concreting these walls, and then tensioned to their final permanent value once the walls have been concreted. As a result, tension stresses in the concrete walls under serviceability limit state (SLS) loading can be avoided.

Chapter

3

Analysis

José Romo Martín, Spain

3.1 Structural Behavior and Analysis

Extradosed Bridges are highly redundant structures; their structural behaviour varies with the relative stiffness of the deck, pylon, and cable system.

As discussed in previous chapters, the use of a stiff deck in Extradosed Bridges allows live loads to be transferred to piers mainly in bending of the deck girder. This results in little activation of the cables under live loading and thus a corresponding small stress variation range. Hence, with regard to vertical loading, the cables in an Extradosed Bridge are designed to carry predominantly dead loads from the deck.

The Extradosed Bridges built to date have varied in the locations of cable anchorages along the span. In several recent Japanese designs, the anchorages can be found concentrated towards the mid span, while the deck nearer to the pylon supports its weight and superimposed permanent load though the vertical stiffness and capacity of the deck section. This can be further enhanced by increasing the deck depth or haunching the deck near the pylon location.

Conversely, it is possible to use a slender deck by stiffening the pylon and increasing the extent of live load taken by the cables. This is desirable for longer spans, which tend towards a conventional cable-stayed system, but it should be noted that the cable tensioning in this case may need to be reduced to accommodate the increased live load stress range.

There is a clear interaction between design and construction of Extradosed Bridges. As a result, the selection of the structural type, span arrangement, and materials should consider the method of construction at the design stage. Construction stage considerations such as the changing structural system during construction, temporary supports, erection equipment on the structure, movements of form traveller, and sequence of post-tensioning tendons and stays, among others, may influence the design choices for the structure.

The structural analysis of the bridge should consider the changing structural system during the course of construction, including varying internal effects and stresses at the construction stages. The analysis must consider the redistribution of bending moments in the deck due to long-term creep and shrinkage in the concrete.

Before starting the analysis of the bridge, the designer has to make decisions, not only regarding the size and general layout of spans, pylons height, and cables disposition, but also regarding the percentage of the permanent load that will be resisted by the stay system, and the influence of the construction sequence to achieve that goal.

As an intermediate structural type between a cable-stayed bridge and a continuous classic girder, the design of an Extradosed Bridge is generally subject to interpretation within that range by the designer. As a result the analysis of an Extradosed Bridge combines analysis considerations from both those types of bridges.

The following aspects, however, make the analysis of an Extradosed Bridge different.

- Initial State and relation of deck pre-stress to stay tensioning.
- Safety considerations for Extradosed Bridges.
- Stay cable design.
- Dynamics of cables.

The following sections cover those aspects.

3.2 Initial State

As noted, the permanent state of an Extradosed Bridge is highly dependent on the construction sequence and on the decision of the designer regarding the percentage of the permanent load taken by the stays. Two different approaches to this decision are described here.

3.2.1 Permanent State for Classic Cantilever Construction

The first method is based on cantilever construction, where the permanent loads are only partially compensated by the stays. In this method each stay is normally only stressed once after its installation in the corresponding segment when the deck is built by cantilevering.

Extradosed Bridges behave as a continuous girder partially supported on the stay-cables.

In general, the stays are designed to take only a part of the dead load, creating bending in the deck. Extradosed Bridges are typically built as balanced cantilevers. During the cantilever construction, the deck is subjected to negative moments (*Fig. 3.1*)

In service, the bending moment diagram is similar to the one obtained for a continuous girder but with lower values (*Fig. 3.2*).

The arrangement of the internal top post-tensioning of the deck is typically governed by the demands during construction, whereas the continuity post-tensioning is typically governed by demands during service. The amount of the top post-tensioned steel depends on the amount of dead load carried by the post-tensioning cables. Typically, the cables are dimensioned to carry between 60 and 80% of the dead load.

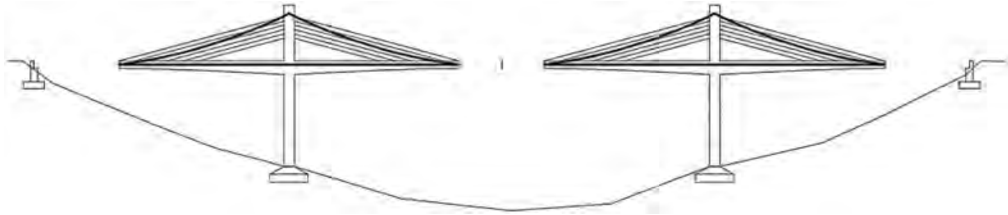


Figure 3.1: Schematic bending moment diagram during construction

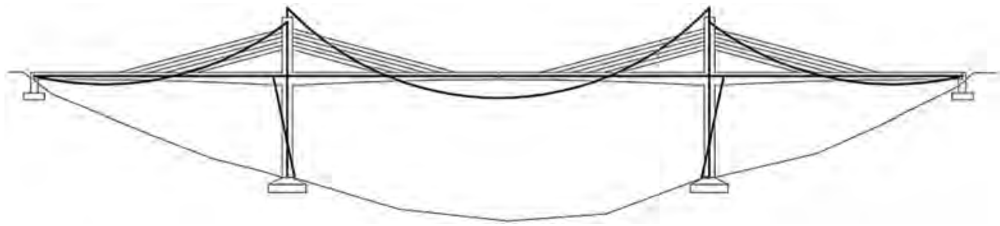


Figure 3.2: Schematic bending moment diagram due to dead loads at service

In standard cantilever construction, it is necessary to use longitudinal pre-stress located in the upper flanges of the section to counter the bending moment caused by the self-weight of the cantilevered deck. That pre-stress is introduced in a statically determinate structure and therefore does not cause any secondary bending moment in the system.

For permanent sagging bending moments in the deck, it is necessary to use a continuity pre-stress, typically located at the bottom slab or as external tendons led by deviators from the top slab at the pier to the bottom slab at midspan. This pre-stress is introduced in the system after the closing of the span in typical cantilever construction.

In general, due to small inclination of extradosed cables, the stiffness provided by the deck is significantly higher than that provided by the stay system. Therefore, the continuity pre-stressing affects mainly the deck, and, therefore, the distribution of primary and secondary effects is very similar to that of an equivalent continuous deck without stays.

Figure 3.3 shows a typical extradosed bridge with a span of 140 m with a deck with span/radio of 1/35, cable average angle of 16 degree, and stay quantities of 15 kg/m^2. This figure shows how the stress of a deck continuity cable in the bottom slab of a cantilever bridge produces almost the same bending moment as in the case of an extradosed structure. Note that this depends on the relative stiffness between deck and cables.

In a similar way, the redistribution of the bending moment due to creep occurs much like a continuous deck and, therefore, the structure has to be checked at long term situation.

In the case of cantilever construction, the structural system changes after the closing of the central segment, which entails a redistribution of the bending moments in the deck, by increasing the hogging bending moments at pylons and decreasing the sagging bending moment at the span

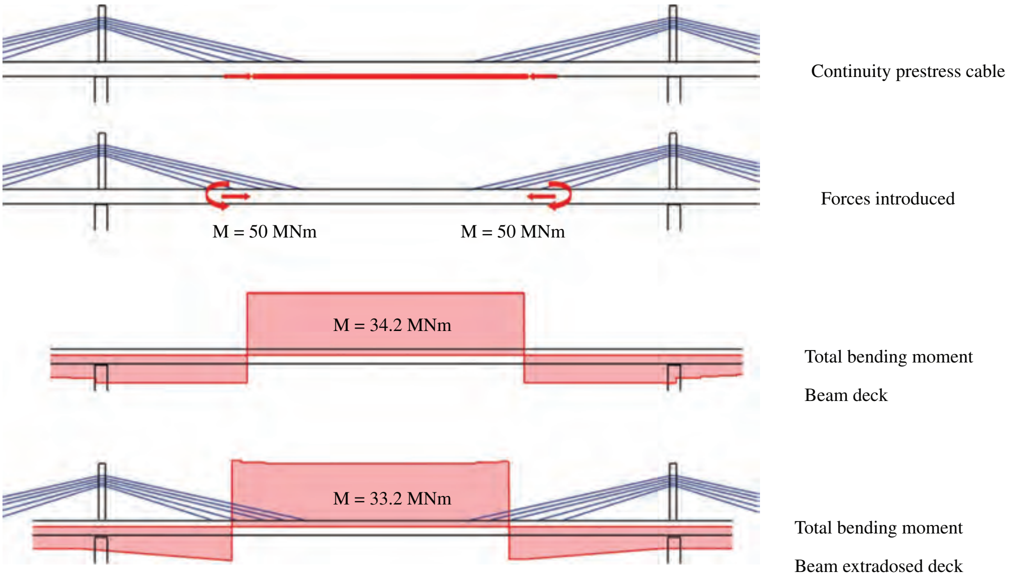


Figure 3.3: Primary and total bending moment caused by a continuity pre-stress in the case of a continuous deck without stays and in the case of an Extradosed Bridge

centre, even though those values are also modified by the pre-stress of the deck. Furthermore, the presence of the stays modifies that redistribution and the forces in the stays also decrease with the corresponding change in the bending moments in the deck.

As an example, *Figs. 3.4 and 3.5* show the bending moments due to permanent loads and extradosed cable tensioning before and after creep, and how the percentage of forces taken by the deck increase due to the long-term effects.

In the Odawara Blueway Bridge, the percentage of dead load bending moment in the deck compared with a continuous deck, before creep, which means just after construction, is about 69%.

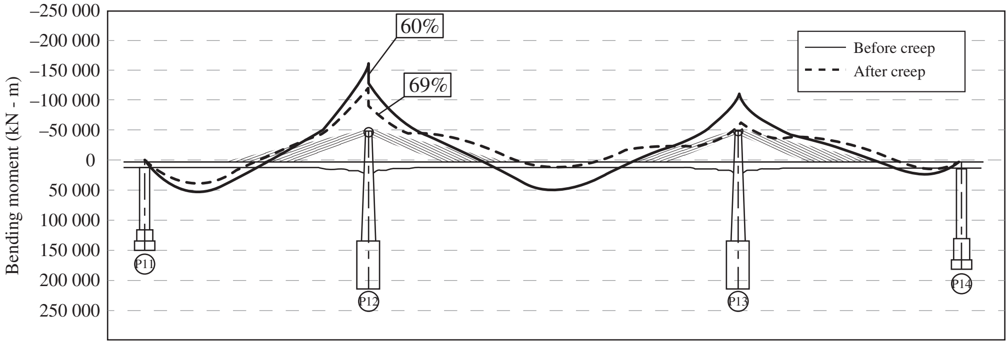


Figure 3.4: Bending moment under dead load (Odawara Blueway Bridge)

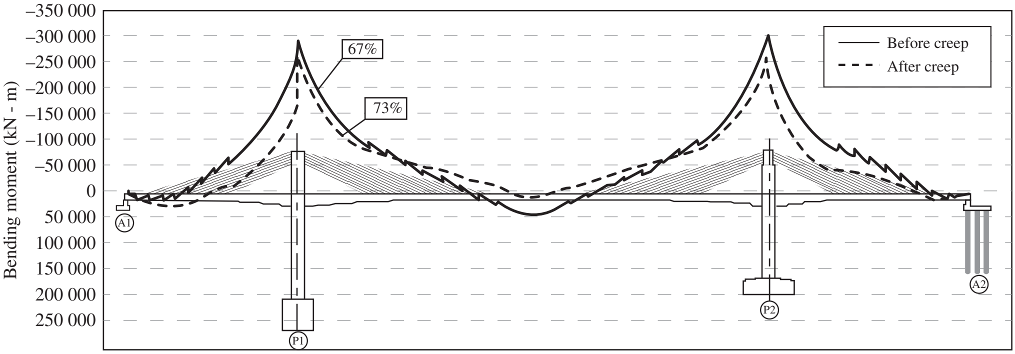


Figure 3.5: Bending moment under dead load (Tsukuhara Bridge)

However, this value changes to 60% after creep. And in the Tsukuhara Bridge, this value reduces from 73 to 67%.

The percentage of stay cable forces is determined by the balance between pylon height and girder stiffness. Usually, the amount of stay cables and internal or external tendons is determined by the girder and pylon height and construction process. In the free cantilevering method, cantilevering tendons are needed before stay cable tensioning. And the amount of these tendons is based on the girder height. Usually, the distance of stay cable is related to the segment length, for example, one stay in each segment or one stay in each two segments. The dimension of stay cable depends on the longitudinal spacing and girder width and depth. There is no typical cable size. Cables from 19 to 93 strands of 15 mm diameter have been used.

3.2.2 Permanent State with Concordant Pre-stress

The second possibility is an adaptation of the traditional approach used in cable-stayed bridges. In this case, the bending moment in the deck for the permanent stage is set to follow the distribution of a continuously supported beam (at the piers and in the area where the deck is supported by the cables). When this approach is used and the construction is by cantilevering, the stays have to be restressed to achieve the desired bending moment distribution in the deck.

In both cases, it is important to consider the effects of pre-stressing on the deck and the time-dependent effects of creep and shrinkage that result in a final condition that can be significantly different from the initial condition.

The stay cables for modern cable-stayed bridges with closely spaced stays are normally designed such that the bridge deck will behave as a continuously supported beam on rigid supports (the cable anchorage locations). Bending moments in the deck due to permanent loads are hence minimized. Consequently, the creep effects due to bending of the deck elements under dead load are minimal and can generally be ignored. Stay cable forces and bridge geometry in a typical cable-stayed bridge will generally vary in time due mainly to time-dependent axial forces in the concrete elements of the deck. The methods for the determination of the stay cable forces under dead load for cable-stayed bridges are well understood and documented.

The methods used for cable-stayed bridges are not necessarily applicable to Extradosed Bridges; however, some similar approaches can be applied. First, the length of the side span should be chosen so that the deck under permanent loads will not rotate at the pier supports. This way, the cables can be placed symmetrically around the pylons and stressed equally. For the standard span distribution of *Fig. 3.6* the length of the side span would be approximately 0.615 L. The cables can then be stressed so that for permanent loads, the centre of each cable group acts like a support, that is, no vertical deflection, and the entire deck shows a bending moment distribution similar to a seven-span continuous girder. The resulting bending moments in the deck might still be large but can be addressed by post-tensioning of the deck. By applying concordant pre-stressing, the bending moments can theoretically be eliminated altogether without changing the cable forces.

This layout of pre-stressing can be directly used when the deck is built by scaffolding. In the case of cantilevering construction, it is necessary to re-stress the stays to achieve the bending moment distribution desired.

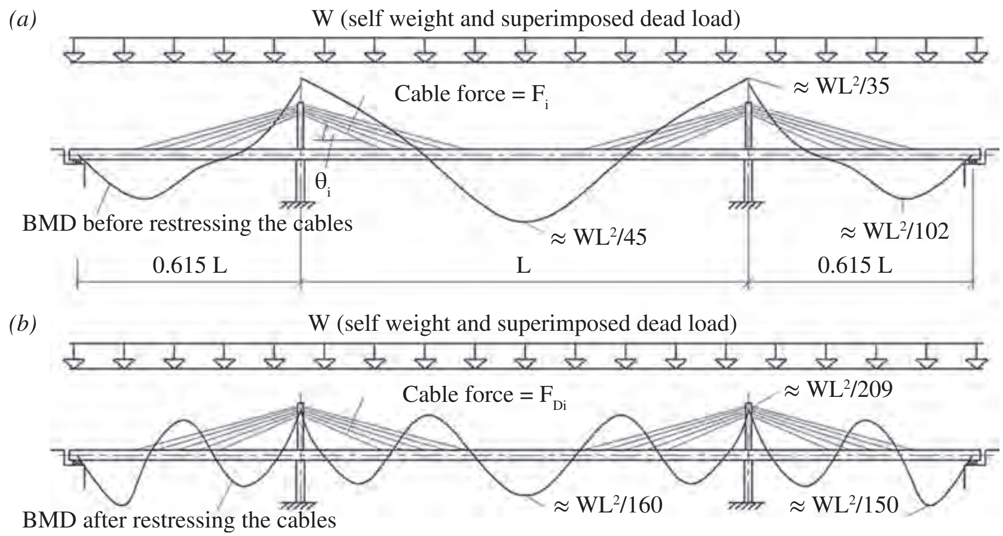


Figure 3.6: Bending moment load for permanent stage before and after the restressing of stays[47]

3.3 Ultimate Limit State Considerations for Extradosed Bridges

One of the main differences in terms of the safety considerations in Extradosed Bridges compared with cable-stayed bridges is the treatment of the permanent loads at the ultimate limit state (ULS).

In cable-stayed bridges, the Permanent State includes, "G" the self-weight of the structure and other superimposed dead loads (pavement, safety barriers, etc.), and also the effect of cable forces "P." In cable-stayed bridges with flexible decks, G and P have a direct relationship, because all the permanent load is generally carried by the stay system. The correct structural geometry would not be obtained without this consistency between G and P. For this reason, the design of a cable-stayed bridge can use the same partial load factor for G and P, and it is normal to consider the permanent effects of G + P as a single action in the analysis (*Fig. 3.7*).

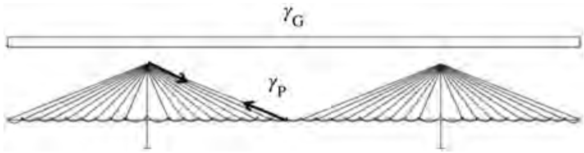


Figure 3.7: Permanent loads + pre-stressed cable forces in ULS in cable-stayed bridges

The same principles do not apply to Extradosed Bridges. Combination of P and G into a single action (G + P) is not appropriate for cable-stayed bridges with stiff decks, externally post-tensioned bridges and guyed pylons, because standard site monitoring of deflections and adjustment of cables will be insufficient to guarantee that there is no significant unintended imbalance between G and P. Note that this is treated differently by different standards.

The pre-stress of the cables on the deck plays an important role in the consideration of the behaviour at SLS and safety at ULS. That includes not only the primary effects but the secondary effects of the pre-stressing when the pre-stress forces are introduced in the complete structural system as is the case for the continuity pre-stress for sagging bending moments in the centre of the spans. For this effect, the behaviour of an Extradosed Bridge is quite different, being closer to that of a classic continuous girder deck not supported by cables.

3.3.1 Standard Method

As the effectiveness of the cables are limited in Extradosed Bridge, a conservative approach to check the deck and pylons in ULS could consist on considering the permanent loads G factored as usual (typically $\gamma_g = 1.35$ in Eurocodes for example), and the forces in the stays factored by 1.00, in other words P with $\gamma_p = 1.00$.

The method is very conservative, since the tension in the stays will rise somewhat when the applied external loads increase.

3.3.2 Alternative Method

As can be understood, the behaviour of an Extradosed Bridge in ULS is highly complex.[48] For ULS checks, if the permanent and live loads are factored with the usual values (1.35 and 1.50), the forces at the stays will reach unrealistic values higher than the conventional yield value 80% GUTS. Thus, the load distribution of the structure at ULS would require a non-linear material analysis, but that is normally unpractical for design purposes. As a simplified method, the bending moment, shear and axial forces of the deck in ULS check could be calculated assuming that all the stays have reached the conventional yielding point 80% GUTS and the permanent load and live loads are factored as usual (*Fig. 3.8*).[48]

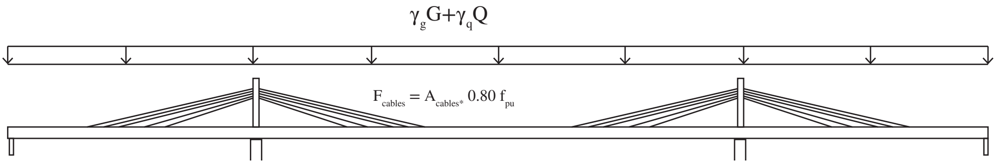


Figure 3.8: Simplified analysis at ULS for stiff Extradosed Bridges

In Extradosed Bridges, stay-cable forces can reach their yield point easily under ultimate limit state if the initial stress is almost 60% GUTS. This means that construction errors of stay cable forces disappear, which indicates partial safety factor of an Extradosed Bridge could potentially be reduced compared to cable-stayed bridges. This subject may be covered in future specifications following further research.

General rules for the behaviour of Extradosed Bridges at ULS cannot be given since it depends on the relative stiffness of deck and cables as well as the deck design itself. Also material factors at ULS in the codes differ. Eurocodes (EN1993-11) prescribe a reduction factor of 1.5 for the cables while PT has a much lower factor (1.1).

3.4 Cable Design

3.4.1 Introduction

For the cable-stayed bridges and Extradosed Bridges constructed up to now, plotting the value β, which expresses the percentage of permanent load carried by the stays (ratio of the vertical component of the forces in the stays due to permanent load of the deck to the total permanent load of the deck) versus the value for maximum stress change of the stay cables due to design live loads, reveals that there is a considerable correlation between these values (*Fig. 3.10*). Two things can be concluded from this figure. First, it is difficult to clearly distinguish Extradosed Bridges (EDBs) and cable-stayed bridges (CSBs) in terms of structural mechanics, since many of the cable-stayed bridges constructed up to now are relatively similar to Extradosed Bridges. Second, in designing stay cables, the stress change due to design live loads provides an effective index that can be easily determined and used as part of the design process.

3.4.2 Stay Cable Design

In the design of stay cables live load stresses are significant and the fatigue limit state is critical. In the design of post-tensioning tendons, live load stresses are typically insignificant and the fatigue limit state is not critical. As noted herein the stays of Extradosed Bridges will be exposed to

Cable type	Live load stress change $\Delta\sigma_L$ (MPa)	Approx max allowable stress
Stay cables	~100 ($60 < \Delta\sigma_L < 160$)	40–45% GUTS
Extradosed cables	~50 ($30 < \Delta\sigma_L < 100$)	Dependent on $\Delta\sigma_L$
PT tendons	~15 MPa	70–75% GUTS

Table 3.1 Live load and dynamic characteristics of cable types

significantly less live load stress than the stays of a cable-stayed bridge. *Table* 3.1 below gives typical expected live load stress changes $\Delta\sigma_L$ for the range of different cable types.

Table 3.1 also indicates the maximum working stress or allowable stress that has typically been used for cable stays and post-tensioning tendons. Based upon this, it is rational to consider using a varying allowable stress for extradosed stays depending upon the live load stresses to which the stay is exposed. Therefore, when designing structures that use cable stays, rather than defining in advance whether the bridge will be a cable-stayed bridge or an Extradosed Bridge and then determining the allowable stress for the stays, the more rational approach is to design the stays by focusing on the stress change caused by live loads that affect fatigue. This approach makes it possible to design each stay separately and enable the allowable stress to be set individually for each stay based upon the level of live load stress in the stay.

The live load stress change in the stays of a cable-stayed or Extradosed Bridge will differ depending on the characteristics of the structure, so it is not rational to define the allowable stress using a single fixed value. Rather the maximum stress should be set based upon the live load stress in each stay. This knowledge is reflected in modern specifications for the design of cables such as “PTI—Recommendations for Stay Cable Design, Testing and Installation”[10] and “Specifications for Design and Construction of Cable-Stayed Bridges and Extradosed Bridges.”[9]

The Japanese Specifications for example[9] allow two design methods, described as follows.

3.4.2.1 Normal Fatigue Design

First method is normal fatigue design using fatigue loads and the target design life of the bridge (Method A).

The design value of stress range to be used for the fatigue assessment is normally the stress ranges corresponding to 2×10^6 cycles: $\Delta\sigma_{2E6}$. The fatigue verification is fulfilled if the design value of the stresses $\Delta\sigma_{2E6}$ is less than the fatigue strength (f_{scrd}) divided by a safety factor (γ_b).

The main difficulty of the method is the estimation of the amount of future traffic and heavy trucks, especially on local roads, and the corresponding stress ranges.

3.4.2.2 Simplified Fatigue Design Method

The simplified method (Method B) simply compares the stay cable stress change due to design vehicular live loads with the maximum allowable stress of the stays *Fig. 3.9*.

Figure 3.9 shows the relationship between the allowable tensile stress for stays of highway bridges and the stress change due to live load $\Delta\sigma_L$ regulated in the specifications. The difference regarding fatigue strength between prefabricated wire type and strand type is considered.

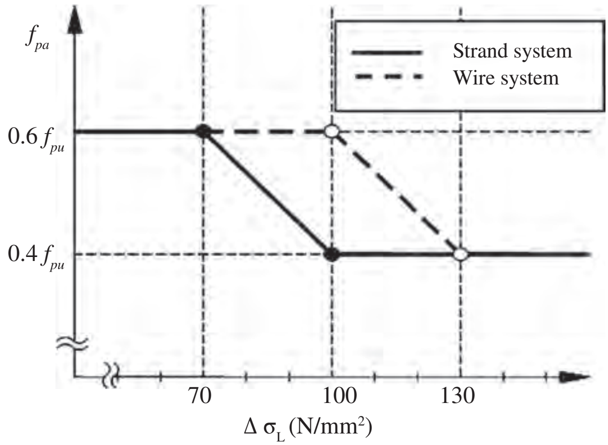


Figure 3.9: Allowable stress of stay cables versus stress change due to live load

Background of the simplified (Method B)

Based upon experience in Japan with cable-stayed and Extradosed Bridges having spans of up to about 250 m, Method B is defined to ensure adequate safety in comparison with bridges designed using Method A.

To establish Method B, fatigue design was performed for the estimation line of stress range for two million cycles ($\Delta\sigma_{2E6}$), including secondary flexural bending due to girder deflection. The fatigue design is carried out by considering a design service life of 50 years and average daily traffic of 70 000 mixing 50% trucks, based upon the structural models of the Odawara Blueway Bridge, the Tsukuhara Bridge, and the Ibi River Bridges, as shown in *Fig. 3.10*.

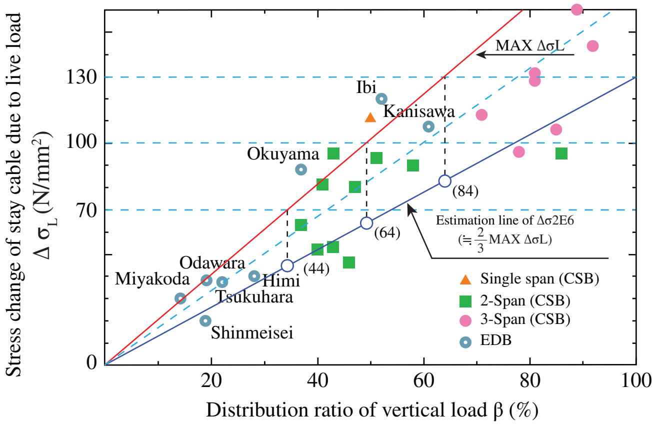


Figure 3.10: Maximum stress change due to live load versus distribution ratio of vertical load and stress change of stay cable

In Japan, based on the calculations, the stress change due to fatigue load is about one-third of that due to the design live loads, and the stress level due to secondary flexural bending is the same as that due to axial forces of stay cables. The estimated level of $\Delta\sigma_{2E6}$ is, therefore, assumed to be 2 (1/3) (Max $\Delta\sigma_L$). The safety margin of Method B can be confirmed compared with $\Delta\sigma_{2E6}$ and fatigue strength (f_{scrd}) divided by a safety factor (γ_b).

For a strand stay cable fabricated on site using wedges, the relationship between f_{scrd}/γ_b and the $\Delta\sigma_{2E6}$ estimation line is shown in Fig. 3.11. It is based on a system with fatigue stress range of at least 120 N/mm^2 at 60% GUTS or at least 200 N/mm^2 at 40% GUTS. In this situation, γ_b is 1.3. The shaded section of the figure is the range determined by Method A with the fatigue design conditions indicated above, and since this is 2/3 of the $\Delta\sigma_L$, as prescribed by Method B, there is a safety factor of around 2.0 with respect to f_{scrd}/γ_b.

For a galvanized wire stay cable made at a factory as a cold-cast cable or as a cable with buttonhead anchorages, the relationship between the f_{scrd}/γ_b and the $\Delta\sigma_{2E6}$ estimation line is shown in *Fig. 3.11*. The line is based on a system with fatigue stress range of at least 180 N/mm^2 at 60% GUTS or at least 230 N/mm^2 at 40% GUTS similar to the figure for the cable fabricated on site. It can be seen from the figure that the factory-made cable also has a safety factor of around 2.0 with respect to f_{scrd}/γ_b.

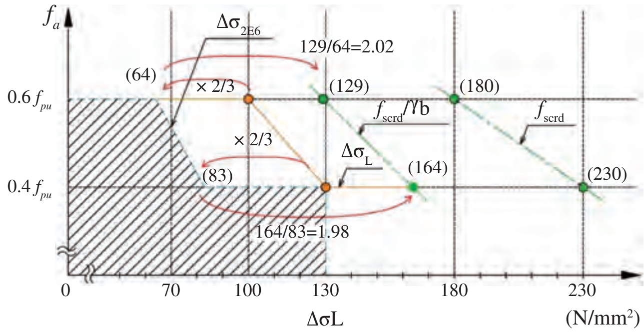


Figure 3.11: Allowable stress of wire stay cables versus stress change due to live load

As described above, stays designed by Method B require a safety factor of about 2.0 for $\Delta\sigma_L$ with respect to f_{scrd}/γ_b. This is done in consideration of the fact that the method includes more uncertainties than Method A, and to ensure that the resulting safety of stays does not vary greatly from that of cable-stayed and Extradosed Bridges constructed to date.

In most of the Extradosed Bridges and some cables of cable-stayed bridges, 60% GUTS can be used as the maximum allowable tensile stress because the live load stress changes are low, in the order of 20 to 50 N/mm^2. The most important aspect of this specification is that the designer can choose the maximum allowable tensile stress in each stay cable continuously from 40 to 60% GUTS depending upon the live load stress in the cable. The resulting specification therefore reflects the concept that adopting one value of allowable tensile stress for all bridges in not a rational approach.

3.4.3 Differences in Codes and Regulations

Figure 3.12 shows the allowable stress for stay cables according to the Japanese[9] and French Recommendations SETRA.[8] The Japanese Specifications have a linear change between 60 and 40% GUTS and two types of stay cable systems, strand and wire, which are defined with different allowable stress in the stress range from 70 to 130 N/mm^2. The stress range considered is the variation in cable stress due to live load. The Japanese Specifications[9] takes into account the flexural bending of stay cables due to girder deflection under live load. However, this graph is an approximate method when using Japan's L-25 live load. When the magnitude of fatigue loading and number of load cycles during the design life time are defined, a more precise fatigue design for stay cables can be performed instead of this simplified method. For comparison, the French Recommendations feature a nonlinear change after 50 N/mm^2 with the allowable stress deceasing gradually to 45% GUTS.

Verification of cables at Ultimate Limit State requires a partial safety factor to be applied. This is specified by several Codes and Regulations, with values of 0.55 (CHBDC), 0.65 (PTI)[10] and 0.67–0.75 (SETRA), see *Fig. 3.12*. All of the estimated values lie within the range of an interpolation between the partial safety factor used for pre-stressing tendons (0.95) and stay cables (0.55).

For SLS the limit stated in CIP-SETRA:

$$F_{SLS} \leq 0.46\left(\frac{\Delta\sigma_{Live}}{140}^{-0.25} F_{GUTS}\right)$$

For the Japanese Pre-stressed Concrete Association the SLS limits are:

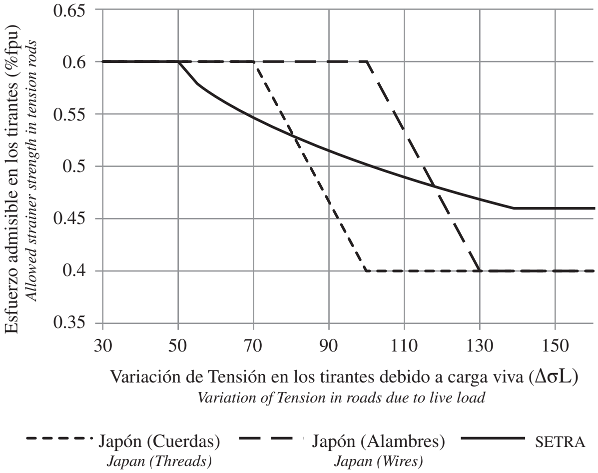


Figure 3.12: Allowable stress in stays versus variation of tensile stress due to live load. Japanese standards versus SETRA (France)

$$F_{SLS} \leq (1.067 - 0.00667\Delta\sigma_{Live})F_{GUTS} (\text{on-site strand assembly})$$
$$F_{SLS} \leq (1.267 - 0.00667\Delta\sigma_{Live})F_{GUTS} (\text{prefabricated wire assembly})$$

The allowable stress range conforms with Serviceability Limit State loading and is adopted for cable sizing. Appropriate dynamic analysis and testing should be carried out to ensure stress cycles do not result in fatigue failure. PTI specifies that design must satisfy both Strength and Fatigue Limit State criteria.

SETRA[8] recommends the application of the most adverse safety factor to all cables. On the other hand, the Japanese Prestressed Concrete Association specifies the option to apply individual tension limits to each cable.

3.5 Dynamics of Cables

3.5.1 Introduction

Stays in Extradosed Bridges are shorter and work at higher stresses than in cable-stayed bridges. Therefore, transverse bending due to wind is less important. Also, their periods of vibration are shorter. Consequently they are typically less sensitive to the dynamic vibrations induced by wind than cable-stayed bridges. But other vibrations initiated by live loads can happen generating a significant stress range with fatigue effects (e.g. train load). The use of dampers is then required to mitigate the vibration of cables.

3.5.2 Dampers

There are many types of dampers for stay cables, for example, viscous dampers, friction dampers and oil dampers are available to mitigate cable vibrations. Detailed information can be found in the IABSE document on cable vibrations in cable-stayed bridges.[49]

Chapter

4

Structural Design – The Japanese Experience

Akio Kasuga, Japan

4.1 General

Structural design of Extradosed Bridges is similar to conventional girder bridges with extradosed cables, which are essentially large eccentricity external tendons. Compared with cable-stayed bridges, the height of pylon is almost a half. Therefore, variation of pylon configuration is more limited than is the case for cable-stayed bridges. The ratio of the back span to centre span is similar to typical girder bridges. Although Extradosed Bridges utilize extradosed cables, the features of the Extradosed Bridge are different from cable-stayed bridges. The girder is much stiffer and is not carried entirely by the cables. The back span of Extradosed Bridges is generally slightly longer than half the main, typically in the range of 0.55–0.60 of main span length. The significance of partially pre-stressed concrete was that it successfully combines pre-stressed concrete and reinforced concrete into a single concept. A similar success has been achieved with the development of Extradosed Bridge technology, which is significant because it enables engineers to combine design principles already established for cable-stayed bridges and ordinary girder bridges. The Extradosed Bridge is a revolutionary high-performance structural system that greatly increases the degree of freedom for the design of cable-stayed and cable-supported structures. Building on J. Mathivat's ideas and the achievement of the Odawara Blueway Bridge, Extradosed Bridges have written a major new page in the history of bridge engineering.

4.2 Structural Components and Details

4.2.1. Deck Cross Sections

The sectional configurations of some of the important Japanese Extradosed Bridges are shown in *Fig. 4.1*. The deck cross sections are of many types, for example, single box, multiple boxes, and twin boxes. Moreover, as shown in *Fig. 4.1e and f*, there are special structures, such as corrugated steel web or butterfly web to reduce the deck weight. These special solutions are effective in earthquake-prone areas. The arrangement of stay cable is deeply related to the deck cross sections. There are two options of stay-cable arrangement: single plane and double planes. For structural efficiency, stay-cable anchorage should be located near the webs. In the deck cross section where the stays are anchored near the web, the vertical component of stay cable forces is low and is transmitted immediately to the main girder, making it unnecessary to install structural diaphragms at the stay anchorage positions as would be the case for a cable-stayed bridge. This feature greatly increases the ease with which this type of superstructure for Extradosed Bridges can be constructed.

In the case where a single plane of stays is utilized, the most important consideration for the section configuration is how to efficiently transmit the stay-cable forces to the main girder. For the Ibi River Bridge, the deck has a width of 33 m, a central set of closely spaced internal webs, combined with upper deck ribs, web ribs, and three structural diaphragms in each cantilever, to ensure the rigidity of the main girder section. For the Shin-Meisei Bridge, the internal webs were spaced as closely as possible and arranged in the form of an inverted trapezoid to concentrate the shear forces at the internal web enabling the elimination of ribs and structural diaphragms to stabilize the section.

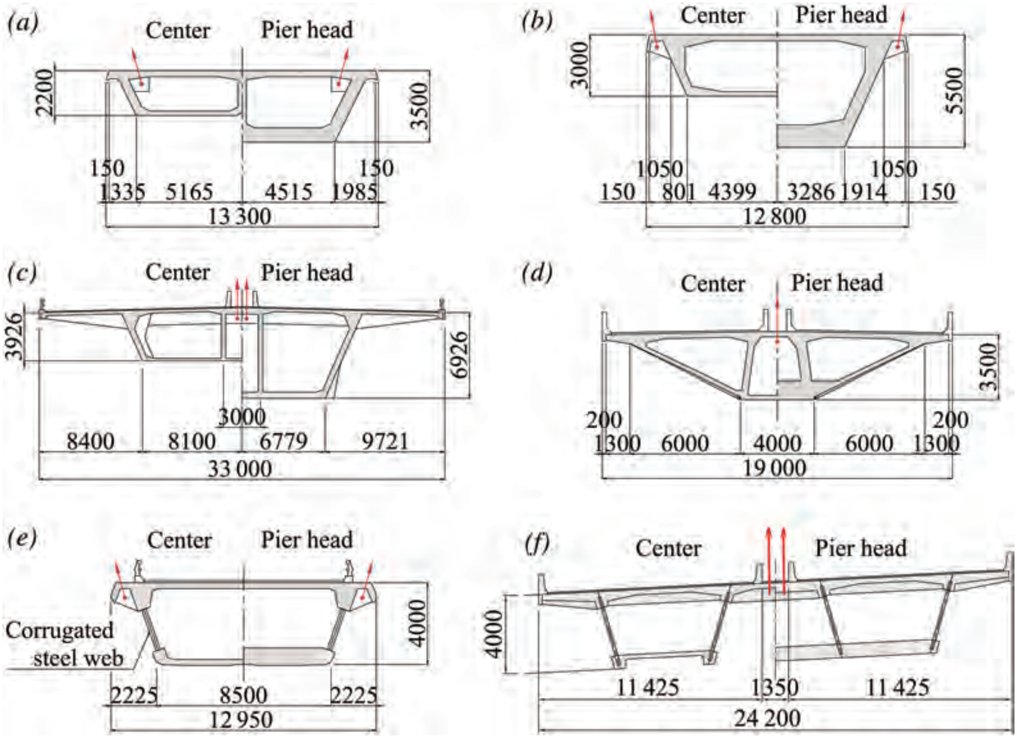


Figure 4.1: Variety of deck cross sections

4.2.2. Pylon Types

Figure 4.2 shows the pylons of the Extradosed Bridges noted above. The pylons of the bridges are low in height, so the variations in form are limited. The interface between the pylons and the

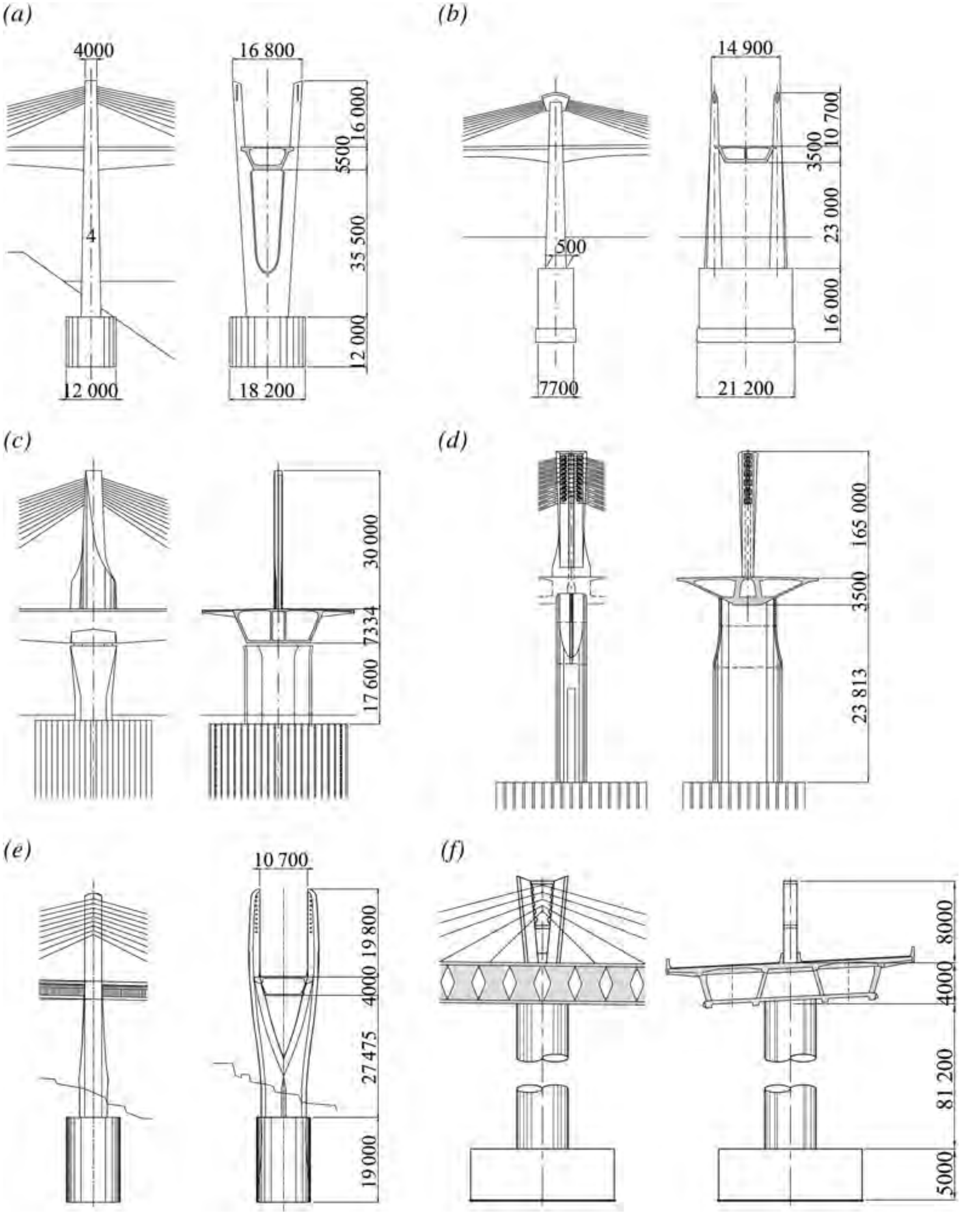


Figure 4.2: Variety of pylon types

bridge piers significantly affects the bridge aesthetics, particularly in the case where two planes of stays are used. On the Odawara Blueway Bridge and Tsukuhara Bridge, the pylons are connected directly to the two-legged bridge piers. Then the resulting structural form has a high degree of purity. On the Ibi River and Shin-Meisei Bridges, the pylons are located on the centre of the girders. To limit the width of unusable deck lost to accommodate the central pylon, the transverse pylon dimension is minimized. The towers beneath the deck therefore need to be widened in order to sufficiently address transverse seismic demand.

In terms of stay-cable anchorage configuration of the pylon, the Odawara Blueway Bridge and Tsukuhara Bridge use saddles, while the other bridges use steel box anchorages. In the case of saddles, there is no access to the pylon. The use of a steel box anchorage makes it possible to inspect the stay cables from inside the pylon during maintenance. *Figure 4.3* shows the inspection path on the Ibi River Bridge. The later Shin-Meisei Bridge and Himi Bridge have similar access provisions.

Figure 4.3: Special pylon solution

Because the pylon width is narrow in the Mukogawa Bridge, the special solution was applied as shown in *Fig. 4.4*. The single-thickness plates, which is arranged at the centre, carry the horizontal component of stay cable forces. The stay cables are located at both sides of the plates.

4.3 Connection Details

4.3.1. Stay Cable and Pylon

The saddle system at the pylon was firstly developed for the Odawara Blueway Bridge. However, saddles can be used under the conditions that stress variation generated by the design live loads is less than 50 N/mm^2 in the Japanese Specifications for Design and Construction of Cable-stayed and Extradosed Bridges. This is based on the research for fretting fatigue test data up to the tendon system of 37 strands of 15.2 mm diameter. Note, that according to fib bulletin 89,[50] saddles are tested with a 180 N/mm² range and an upper limit of 55% GUTS for Extradosed Bridges.

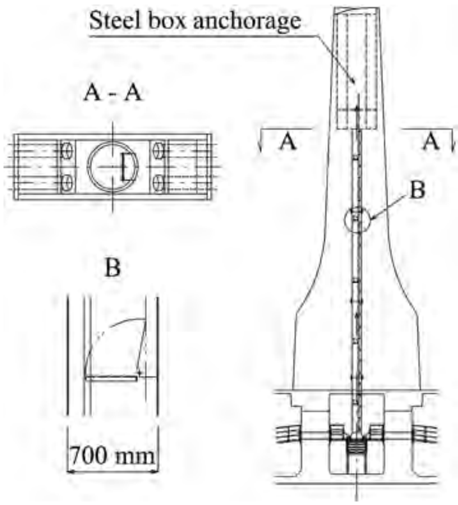


Figure 4.4: Example of inspection path

In case of saddles, stay cable force difference on either side of the pylon due to creep and earthquakes should be considered (*Fig. 4.5*, see also Section 6.6.8). From an ease of maintenance perspective, the steel anchorage box structure in the pylon is useful. In this case, stays are anchored inside a steel box in the same way as steel pylon of cable-stayed bridges (*Fig. 4.6*). In this method, the horizontal component of stay cable forces is carried only by steel, and vertical forces are carried by steel and concrete. Each steel anchorage boxes have corrosion protection.

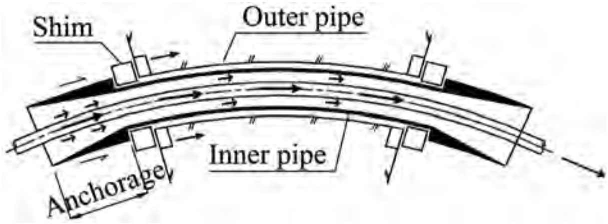


Figure 4.5: Saddle anchorage concept

Another solution of connection between stays and pylons is shown in *Fig. 4.7*. In case of steel box anchorage cannot be formed because of the limited transverse dimension of pylon, a single plate anchorage can be used.

4.3.2. Stay Cable and Deck – Selected Examples

Stay cable anchorages in the girder are basically designed in the same way as for normal external pre-stressing tendons. They are located near webs to minimize reinforcement of anchorage zone and diaphragms. In the special structures, such as corrugated steel web or butterfly web, anchorage design should be considered carefully.

In case of corrugated steel web, a major problem is how to ensure that the vertical component of the stay forces would not be applied directly to the joint between the corrugated steel and the concrete deck. It was decided to adopt a steel diaphragm anchorage structure like that shown in *Figs. 4.8 and 4.9*. The concept behind this structure is that the steel frame would carry the vertical

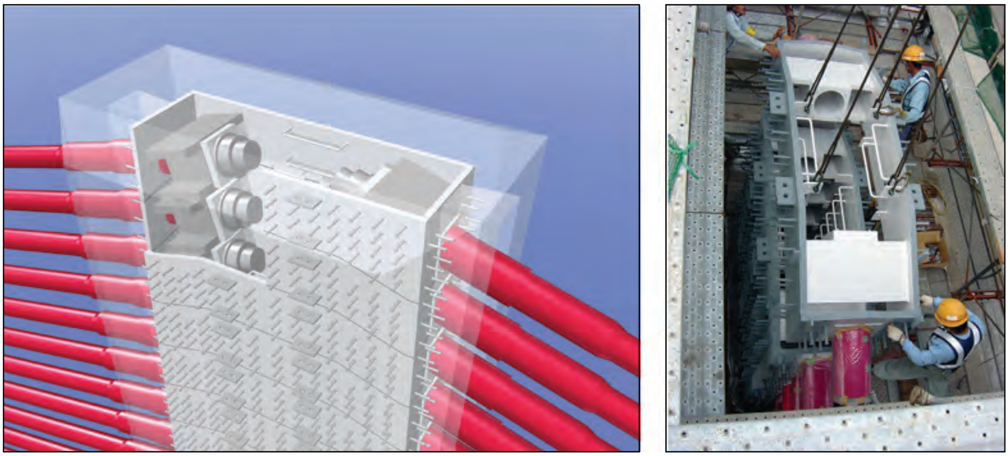


Figure 4.6: Composite pylon structure for anchor stay cables (Shin-Meisei Bridge)

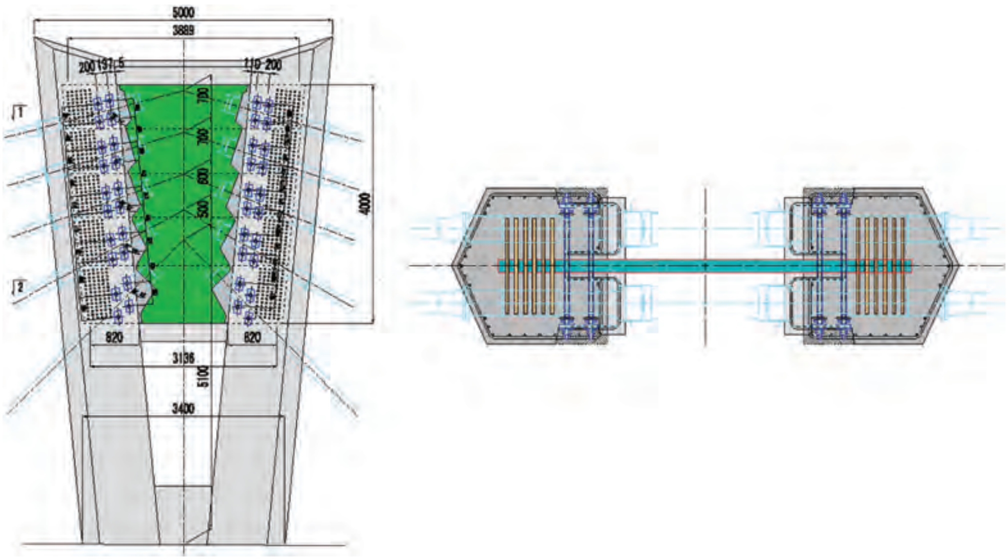


Figure 4.7: Single plate anchorage (Mukogawa Bridge)

component of the stay forces and the shear forces from the corrugated steel, while the concrete slab would cater for the bending moment and the horizontal component of the stay forces. At the same time, this diaphragm would also function as a rib reinforcing the upper and lower decks.

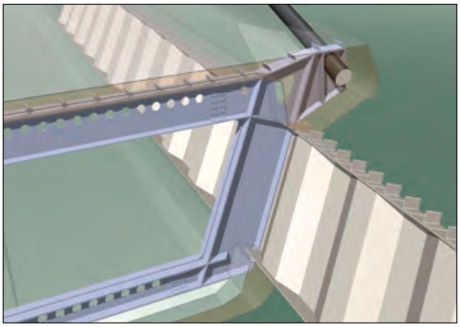


Figure 4.8: Diaphragm anchorage system

In case of butterfly web, structural consideration is the same as corrugated steel web. The important point is how to transfer the vertical component of the stay forces. Stiff diaphragms are utilized in Mukogawa Bridge to anchor stay cables shown in *Fig. 4.10*.

4.4 Reinforcement and Pre-stressing Details

4.4.1. Pylon

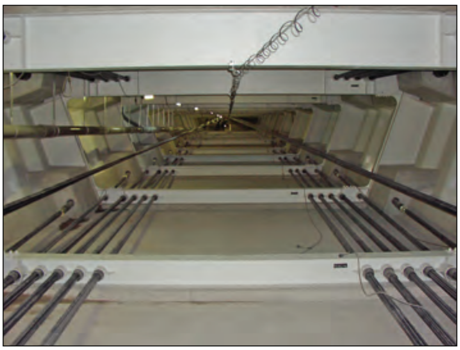


Figure 4.9: Inside view of Himi Bridge

When saddles or composite stay cable connection systems are utilized in the pylon, tension in concrete should be considered carefully. Sometimes pre-stressing bars are arranged to suppress the tension in concrete as shown in *Fig. 4.11*.

Figure 4.10: Diaphragm of stay cable anchorage in butterfly web

Figure 4.11: Pre-stressing bars

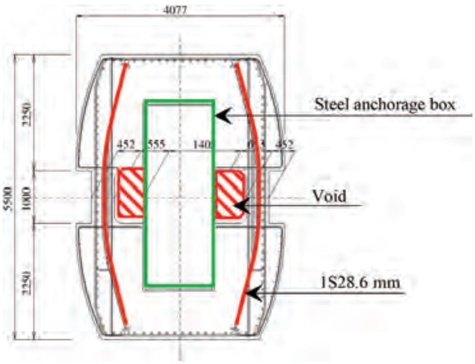


Figure 4.12: Composite stay and pylon connection

Otherwise, some parts of concrete could be cast after tensioning of all stay cables to avoid tension in concrete. If the section of pylon is large, the solution shown in *Fig. 4.12* can be used in order to make the concrete section flexible by arranging void near the steel anchorage boxes.

4.4.2. Deck

Extradosed Bridges are constructed in the same manner as conventional girder bridges. Therefore, the layout of pre-stressing for cantilevering is the same as for typical girders (*Figs. 44.13 and 44.14*). On the other hand, the pre-stressing tendon layout of cable-stayed bridges is shown in *Fig. 44.15*. It is almost constant because maximum bending moments occur in the tip end of cantilevering before introducing the stay forces. This means that the girder of a cable-stayed bridge is shallow and most of the girder weight is supported by stay cables. In this case, pre-stressing bars are usually used and connected by couplers in each segment.

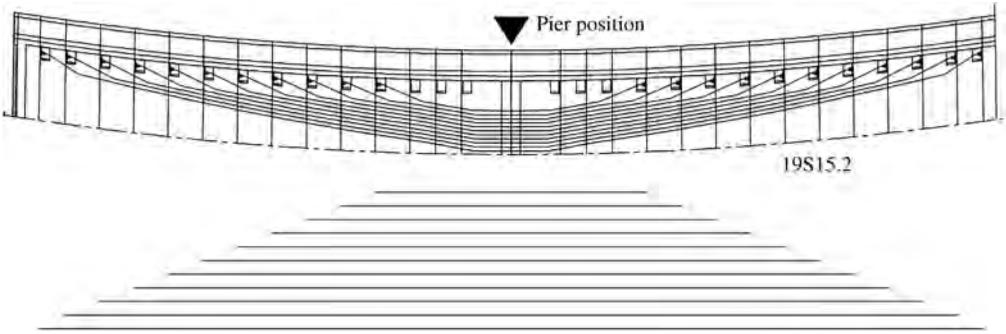


Figure 4.13: Cantilevering tendon layout of Extradosed Bridge (plan view of half top slab)

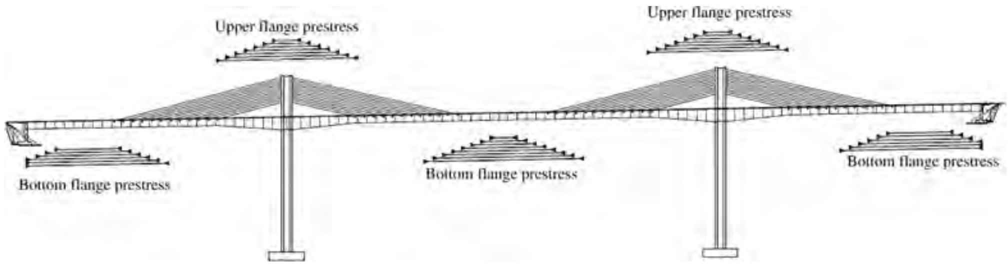


Figure 4.14: Cantilevering tendon layout of Extradosed Bridge

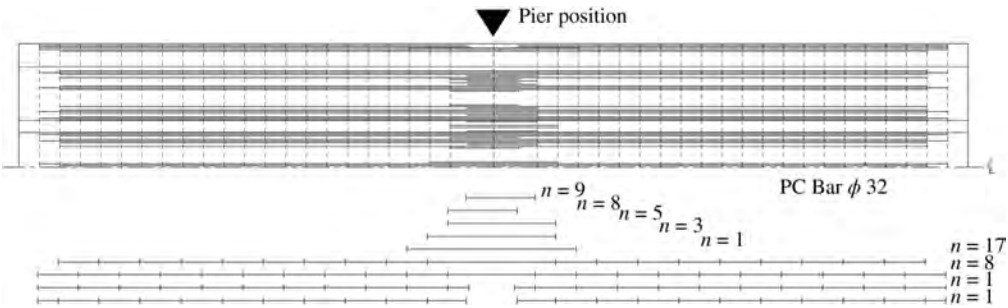


Figure 4.15: Cantilevering tendon layout of cable-stayed bridge (plan view of half top slab)

Chapter

5

Technology of Extradosed Cables

Thierry Duclos, France

5.1 Foreword

Since the emergence of Extradosed Bridges as a concept, the majority have utilized stay cable technology for the extradosed cables. A few have utilized a different type of cable that mixes stay cable technology with the external prestressing technology. Regardless, the aim has been to provide a durable system for the design life of the bridges and to provide the most cost-effective solution given the demands on extradosed stays.

A few publications currently provide the accepted international standards for the basis of design, testing, and installation of stay cables and, to a lesser extent, extradosed cables. These standards address the important technical issues for stay cables and extradosed cables, including durability, fatigue, dynamic response, and technical system details. These standards are as follows:

- SETRA—CIP Stay Cable Recommendations, 2002.[8]
- PTI Recommendations 6th edition, 2012.[10]
- fib Bulletin 89, 2019 (supersedes fib Bulletin 30, 2005).[50]
- JPCEA 2000 Japanese Recommendations: Specifications for Design and Construction of Cable-stayed Bridges and Extradosed Bridges.[9]

These publications continue to evolve as stay-cable technologies and materials have evolved. Some have advanced further than others and the standards vary in some cases among the publications. These publications have evolved primarily to address stay cable technology, often developing progressively to address the learned through actual design, testing, and installation. The development of standards for extradosed stays has been more recent and has lagged that for stay cables. This has resulted in more varied, and in many cases, less robust guidance available to designers for extradosed stays. Work groups with the standards organizations continue to improve and progress updates to the documents and newer more robust recommendations are expected to be incorporated in future versions of these documents.

International bridge design standards typically do not adequately address stay cables. AASHTO does not specifically address stay cables so designers in North America typically rely upon the PTI Recommendations. Similarly, Eurocode[51] gives general requirements for the design of tension elements, does not adequately address stay cables, and specifically does not mention extradosed cables. Designing extradosed cables to the SLS/ULS provisions of Eurocode would be excessively conservative for Extradosed Bridges. European designers, therefore, look to fib bulletin 30, now superseded by fib bulletin 89,[50] for the design of stay cables and extradosed cables. Currently the requirements for extradosed cables are still developing in the international standards.

The use of the stay cable technology is costly and the performance required of stay cables is not necessarily required for extradosed cables. However, the use of the external prestressing technology does not adequately address the issues of waterproofing, fatigue, and dynamic effects etc. An analysis of state-of-the-art specifications related to each cable technology is therefore required to identify the limits of what can be done.

5.2 State of Technology Through Lessons Learned on Projects

Virtually all the Extradosed Bridges use one of two types of technology for the stays, stay cable technology, or an adaptation of external prestressing technology. Cable anchorages of the latter are usually simpler than those for cable-stayed bridges as fatigue and bending issues tend to be less relevant. The following sections provide a brief review of these technologies.

5.2.1 Extradosed Bridges with Stay Cables Technology

In many Extradosed Bridges, stay cable systems are used to provide durability. Referring to Refs. [8, 10, 50], the most common stay cable technology comprises a main tension element (MTE) with parallel strands. Sheathed parallel strands are installed in HDPE duct or steel tube, anchored in the deck, and deviated through pylons with saddles or simply anchored in the pylons. These proprietary systems generally provide a level of corrosion protection which satisfies the requirements of the international standards listed in Section 5.1. The corrosion protection strategy is generally to protect the stay cables with complementary or nested protective barriers. Further details on the typical corrosion protection systems are provided below in Section 5.3.

5.2.2 Extradosed Bridges with External Prestressing Technology

External prestressing technology is sometimes favoured for smaller Extradosed Bridges with spans between 50 to 100 m. In this case, the strand bundle is installed in a HDPE duct and the voids between duct and strands are generally filled with cement grout, wax, or grease. Grease is less frequently used. Wax and grease are identified as flexible filler. Contrary to stay cable systems, restressing and strand replacement are more difficult or even impossible.

5.2.2.1 Flexible Filler

The St Remy de Maurienne Bridge was designed by Jean Tonello and built in 1997.[16] It is a small Extradosed Bridge which utilizes external prestressing technology. The strands are protected by a waterproof steel pipe filled with a flexible filler material. A secondary layer of protection was installed to provide fire protection. This system is described further in Section 5.3.3. The system is reported to have performed adequately since construction.

5.2.2.2 Cement Grout Filler

There are a several bridges which cement grout as a filler material to protect the strands:

- Pragati Maidan Bridge in India.[46]
- Second Vivekananda Bridge, Kolkata, in India.[18]
- Bridge in Konin in Poland.[53]
- The first Extradosed Bridge in Slovenia.[54]
- Several bridges in Japan including Odawara Blueway Bridge.[55]

Cement grout protection is often found in classical external prestressing systems. The use of this system for Extradosed cables is discussed further in the following section.

5.3 Durability

5.3.1 Design Life and Maintenance

As the extradosed cable is typically external to the deck and pylon, it must be able to cope with all environmental conditions at the bridge site. The durability of the cable is dependent not only on the corrosion protection details and waterproofing details but also on the owner's inspection and maintenance strategy. The ability to replace the cable or its components must be integrated into the design. Specific attention needs to be paid to accessibility and replaceability of components as a necessary part of the design. Information on design life and maintenance of stay cables can be applied to extradosed cables since the durability issues are essentially the same. Current relevant international standards offer the following guidance on design life (or service life) and maintenance:

- CIP[8] proposes in its chapter 3 a maintenance-free period of 15 years for the accessible parts (external pipe, external transition zone at pylons, external transition zone at deck, the anchorage…) of the cable and 50 years for the inaccessible parts. For the expected service design life, it could be fixed as 50 years as a standard maintenance free period.
- The fib recommendations[50] do not set maintenance free life and refer instead to the national codes. But in Section 4.5.2 about corrosion protection, Table 4.4 sets the maintenance period in accordance with replaceability and access possibilities, with a design life of the stay cable system of 100 years. The fib recommendations refer to a "reference system" used in terms of corrosion protection. The reference system consists of metallically coated tensile elements

(mainly galvanised), individually protected with wax and PE sheathing, encased inside a HDPE stay pipe without filler or alternatively to individual tensile elements protected with a general injection of wax inside a HDPE stay pipe.

- PTI[10] gives in its commentary C5.3.5, the design life of the cables need not equal the design life of the bridge which may be between 100 and 200 years rather it is suggested that 75 years would be appropriate for cables.

The maintenance strategy must define regular inspection of the accessible components of the cable. The owner and the designer should specify the design life (service life) and the maintenance period in accordance with an agreed maintenance strategy.

5.3.2 Protection

The technology adopted to provide reliable corrosion protection for stay cables has been developing for more than 20 years and there has been substantial progress compared with the protection provided with the very early stay cable systems.

Three specific zones are of interest in the design of stay cable systems for water tightness: the anchorage zone at the deck, the free length between the deck and pylon, and the anchorage zone in the pylon where two concepts exist, classical dead/live end anchorages or saddles. The transition zone between the free length and anchorages or saddle is particularly important and must be designed to assure water-tightness to avoid water ingress into the anchorages or saddle.

The technology and details of stay cable protective barriers have evolved over many years now. This evolution has led to the use of the term "reference system" described in the current fib bulletin 89.[50] This reference system has been used to define an accepted standard or basic level of protection for a stay system. The fib reference system is based on a design life of 100 years, and a class of exposure C5 according to the ISO 12944-2 standard. Three differing types barriers are defined for the reference system are as follows:

- An external barrier which is exposed to the outside environment.
- An internal barrier which is applied directly to the main tension element (MTE).
- An interface between these barriers consisting of wax, grease or grout can be injected.

The external duct need not provide the external barrier. The external barrier can be provided by a sheath around each individual strand in the bundle. Two different types of stays systems, which comprise most of the modern stay cable systems installed, can be used for the reference system:

System 1 Galvanized or plain seven wire strands individually greased or waxed encased in PE and bundled inside a steel or HDPE duct.

System 2 Galvanized or plain parallel wires or strand bundled in a steel pipe or HDPE duct and the pipe or duct filled with corrosion inhibiting wax, grease or other blocking compound.

Modern stay cable systems that utilize these barriers and are tested for water-tightness have performed well.

System 1 above represents the most commonly used system for modern stay cables. In this parallel strand system, the external barrier is the PE sheathing around the individual strands, the internal barrier is a metallic coating applied on each wire and the wax or grease filler between the wires of the strand provides the interface between the external and internal barriers. The metallic coating is generally galvanizing applied to the wires of the strand which works sacrificially to directly protect the MTE against corrosion. The galvanizing is applied directly on the steel wires at the production stage. The bundle of sheathed individual strands is generally surrounded by a stay pipe (duct) made from high-density polyethylene (HDPE). The HDPE pipe would also provide any surface treatment required for aerodynamics of the cable and would provide UV protection. System 1 typically utilizes expansion sleeves to allow for expansion and contraction of the external pipe relative to the MTE so humidity can get into the pipe interior but the MTE remains protect by the nested barriers within the external pipe. In the case of parallel strand systems, the ends of the strands, which must be exposed for anchoring with wedges, are typically protected within in a sealed "stuffing box" and grease or was filled anchor end caps.

For System 2 above, the external barrier is the external pipe or duct, which may comprise high density polyethylene (HDPE) or steel pipe sections welded together. The external barrier in this case must not only provide UV protection and any surface treatment required for aerodynamics, but also provide the external watertight barrier. Similar to System 1, the internal barrier is provided by a metallic or other form of coating applied directly to the wires or strands of the MTE. To provide the interface between the two barriers, the intermediate space is filled with a protective material. Various filler concepts have been adopted for the interface in System 2, including the following:

- Wax or grease.
- Filling with nitrogen or other protective gas.
- Filling with dehumidified air.

The use of gas or dehumidified air of course requires details for an airtight pipe and anchorage system.

Consistent with conventional external prestressing technology rather than stay cable technology, cement grout has been used as the filler material for the external pipe. In this case suitable precautions as mentioned in Ref. [50] must be taken. It must be noted that with some exceptions, grouted stay cables, which were used more extensively in the early developmental stages of stay cable technology, provide less assurance of water-tightness, and have in several cases not performed well over time. If the external pipe or duct is breached in a grouted system, the corrosion protection relies heavily on the cement grout filler. Cement grout filler in cable stays has been shown to exhibit shrinkage cracks and bending cracks which allow ingress of water and initiation of corrosion of the MTE. Cracking of the cement grout filler has also been shown to initiate damage to the external pipe due to differential coefficients of expansion under temperature variations. Modern parallel strand systems that keep the corrosion protection system at the strand surface are readily replaceable strand by strand if necessary. The use of cement grout as a filler impairs replaceability of the cable.

5.3.3 Fire

Stay cables, including extradosed cables, can be vulnerable to fire. Extradosed cables usually have a shallow inclination so that their height typically does not exceed 15 to 20 m. The vertical spacing of extradosed stays is usually smaller than that of conventional stay cables. Extradosed cables are therefore generally more exposed to accidental fire[56] than conventional stay cables. Several extradosed cables may be affected by a fire, threatening the structural integrity of an extradosed bridge. The threat of fire should be assessed as part of the design and fire protection may be required by the owner.

Measures to address the threat of fire are offered in fib bulletin 89[50]:

- Removal of flammable materials from the deck facilitated by proper drainage.
- Limit the fuelling of fire by flammable products on/in the structure or stay cables. Avoid the filling of stay cables with hydrocarbon-based products such as wax.
- Retard temperature rise in the MTE for the time needed to control or extinguish the fire (determined by availability of responders). Special insulating materials may be added to surround the MTE inside or around the stay pipe, guide pipe or anti-vandalism pipe.

The PTI 6th edition[10] specifies the performance of the fire protection. The MTE must be protected so that the time required to reach a temperature of 300°C is not less than 30 min under an external fire of 1100°C. The section 4.5 of PTI 6th edition[10] defines a qualification testing by two stages: a first test of insulating fire protection materials followed by a load test of high-temperature strength. These tests were reviewed by W. Brand in a paper for the fib congress in 2014.[57]

Stay cable system providers have in recent years begun to develop proprietary fire protection systems. Regardless, relatively few references exist for tested and installed fire protection systems. Two design types are offered by providers today. In the first type, the external duct is enclosed in a second duct and the space between the ducts is filled with insulating materials. In the second type, the bundle of strands is surrounded with an insulated fabric which is then enclosed within the external duct.

In France two projects were concerned about fire which lead to development of specific fire protection solutions with certain conditions.[15,58] The philosophy proposed by PTI for addressing the threat of fire has been applied in these projects. The first one was the bridge of St Rémy de Maurienne on the Highway A43.[15] The outer duct of the cable was enclosed within with another duct in lacquered aluminium (baked enamel finish). The space between the two ducts of about 25 mm was filled with refractory ceramic fibre blankets, Kerlane type giving a 1-h protection against the ISO fire with a reference temperature of 950°C. In this solution, the external duct diameter is greater than the usual duct diameter. The second one was the Bridge of the Ravine of 3 Bassins[58] at the La Réunion Island. The extradosed cable strands were protected by an insulating mattress covered by a HDPE pipe. In this test a temperature of not more 100°C for 1 h in the cable under a fire of 1000°C was attained.

A risk analysis is generally required to assess the need for fire protection and to assess the level of protection required, if any for the stay cables. Factors to be considered include the following:

- Cable system construction and materials.

- Bridge and cable geometry relative to the roadway.
- The type of fire threat – car fire, tanker fire, pool fire.
- Availability and response time of fire services.
- Fire standards for the location of the bridge.
- Risk tolerance of the owner.

These factors will all vary depending upon specific project circumstances and will result in varied requirements for the design of fire protection required for extradosed cables.

5.3.4 Fatigue

As their length is generally short, extradosed cables tend to be less susceptible than conventional stay cables to stress variations resulting from wind and other dynamic effects. Live load stress variations in Extradosed Bridges cables are also characteristically lower than those of conventional stay cable bridges. Extradosed Bridge cables are therefore generally less susceptible to fatigue than conventional stay cables. Most Extradosed Bridges are highway bridges where the stress variation in the cables is relatively small. There are examples of Extradosed railway bridges, however, where fatigue due to the larger live load stress variations must be carefully considered in design.

For parallel strand stay cable systems fatigue effects are critical at the entrance to the anchorage. Various proprietary details are used to ensure that no bending effects are transmitted to the area of the wedge anchors. This is typically done by providing lateral support to each strand at a short distance before the strand enters the wedges. Various means such as curved or elastomeric lateral supports are used to control the strand bending at this location (see cables supplier advice and aforementioned standards[8,10,50]). Due to the characteristically stiff decks and short cables of Extradosed Bridges, bending effects due to relative angular rotations of the cables and deck are generally small when compared to those of slender cable-stayed bridges.

Bending effects can also be produced by incorrect angular orientation of the cable anchorages in the structure. When axial live loads are imposed on a stay cable, added bending stresses are imposed on the stay by incorrect angular orientation. Bending also occurs when cables are deviated via saddles. This is treated in Section 5.6.7.

International standards for stay cable design[8,10,50] contain requirements for testing of parallel strand stay cables, anchorages, and saddles to demonstrate fatigue performance. The testing is generally done by imposing combined bending and axial stresses at the anchorages and/or saddle. These parallel strand test arrangements are not well suited to conventional external prestressing technology, where the strand bundle is protected by cement grout. Specific test arrangements are required for grouted cables in order to properly represent the operating conditions.

5.3.5 Dynamics

As extradosed cables tend to be short and work at higher tensions than stay cables, they are less prone to dynamic vibrations. Their periods of vibration are shorter than stay cables. Often, the vibrations effects are neglected. Still, as cable vibrations have many sources, such as wind or parametric excitation, they can make fatigue analysis necessary. In accordance with the results, the use of dampers could be necessary so as to mitigate the vibration of the cable.

Many types of dampers for stay cables are available nowadays, for example, high damping rubber, viscous dampers, and friction dampers. Analysis techniques permit calculation of the required level of damping to suppress the expected vibration mechanisms. Once the required level of damping is known, the designer must work with the damper providers to determine the most appropriate and cost-effective damping devises to deliver the required level of damping.

5.3.6 Tolerances

Construction and fabrication tolerances play an important role in the fatigue behaviour of cable stays. Tolerances must be carefully specified for construction and must be considered as part of the design of the cable stays. The positioning of saddles and anchorages in the pylon and deck anchorages must be controlled with care in construction. Errors in construction or fabrication, improper anchorage, or saddle positioning, which result in bending of the cables can contribute to fatigue due to increased stress variation and fretting at the anchorages or deviation points.

5.4 System Installation

Each cable supplier has typically developed specific technology and methods to install and stress the stay systems for different types of bridges. These methods and cable design are described in the documents.[8,10,50] The designer should refer to these documents for more information.

5.5 Anchorages

Two types of anchorages have been used in Extradosed Bridges to date. The first is a typical anchorage used with conventional external prestressing technology. Generally, this type of system is less robust for water-tightness, corrosion protection and replaceability. Due to fretting fatigue, the fatigue stress range for this type of system is limited to 80 MPa. This type of system is less suited to making cable adjustments during construction or in operation. The system lacks design features of typical cable stay anchorages to control bending stresses at the anchorage so is susceptible to construction tolerances at the anchorages. A high level of precision is therefore necessary with this prestressing technology if selected for an Extradosed Bridge. The second type of anchorage is the stay cable anchorage. As discussed above this anchorage system is designed for water tightness, corrosion protection, replaceability of strands, and control of bending due to construction tolerances and lateral loads on the free length. This type of anchorage allows adjustment of the tension strand by strand or globally with the appropriate multi-strand jacking system and allows for replaceability of the cable system.

5.6 Anchorage Layouts at the Pylon

Two basic arrangements exist for the transition through the pylon head. The first arrangement sees the cables either side of the pylon anchored separately at the pylon head. The continuity of the stay cable is interrupted in this arrangement. In this case there are 3 anchorage types which are defined below as cases A, B, and C. The second basic arrangement maintains the continuity of the cable using a saddle in the pylon head and referenced as case D (*Table 5.1*).

5.6.1 Classical Cable Stay Anchorages (Type A and B)

In Japan, few Extradosed Bridges experience more than 50 MPa as stress range under live load. The results of the research for fretting fatigue test data up to the tendon system of 37 strands of 15.2 mm diameter[59] led to keep a design with anchorages in the pylons. As reported by Akio Kasuga in Ref. [55] the use of a steel anchorage box structure at the pylon head allows for easy maintenance. For construction there are a number of methods to enable the erection of this anchorage system. The space required in the pylon by this arrangement increases the pylon head size and also all the equipment for maintenance: access, stairs, platforms, and so on (*Fig. 5.1*).

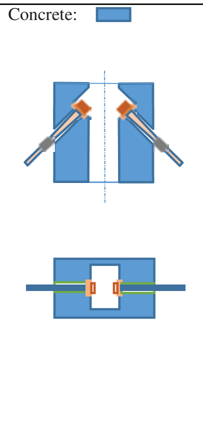
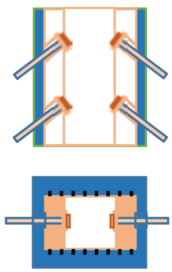
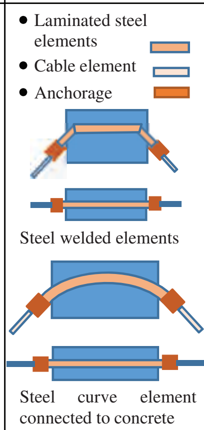
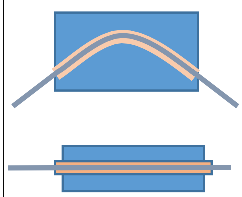
TYPE	A	B	C	D
cross section of pylon head Elevation Plane view				
Comments	Cables anchored inside concrete pylon	Cables anchored inside composite element or in pylon head steel	Cable anchored to a cast-in steel element	Continuous cables running through saddle in concrete pylon

Table 5.1 Type of anchorages in pylon

5.6.2 Steel Tension Elements (Type C)

The anchorages of stay cables at pylons can be done in an external manner using laminated steel elements installed in the pylon head before concreting and completed by external steel nose element on which the cable is anchored. In the completed anchorage, the tension from stay cables creates compression in the concrete introduced via the inclined steel plates in the corners. The steel elements carry the tension across the pylon head. Shear connections deliver differential cable forces to the pylon head (*Fig. 5.2*).

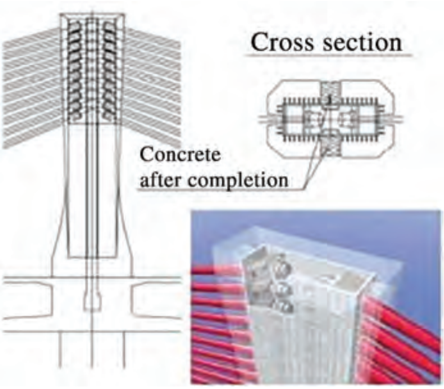


Figure 5.1: Composite anchorage at pylon (Shin-Meisei Bridge)[55]

A few suppliers have developed proprietary composite systems where the extradosed cable is not continuous and is anchored on each side of the pylon head. The details permit the use of standard cable anchorages at either side of the pylon head. Lateral openings can be provided to allow strand by strand cable replacement or even complete cable removal and replacement (*Fig. 5.3*). For that solution, usually the dead anchor is placed at the pylon and the stressing is done at the deck.

This design has the following advantages:

- Shear studs at the steel beams inside the pylon easily transfer differential loads within the concrete, although a proper check of elongation versus shear stud loadings needs to be performed.

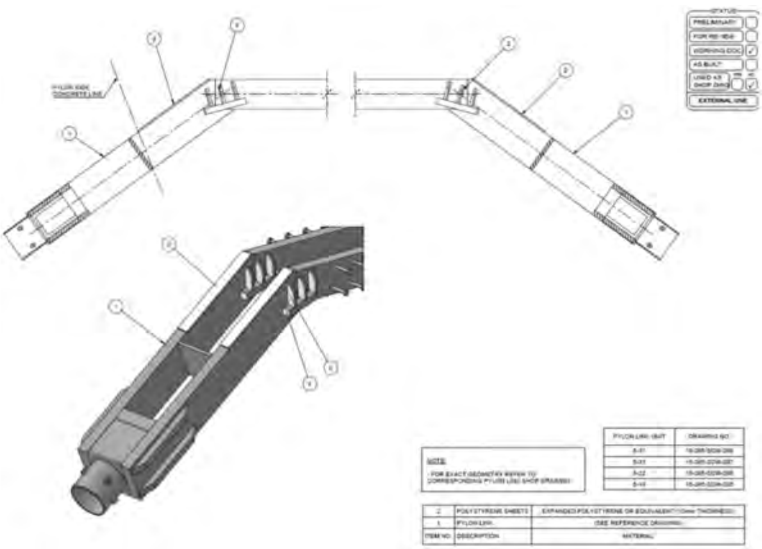


Figure 5.2: Sketch of internal steel link at mast

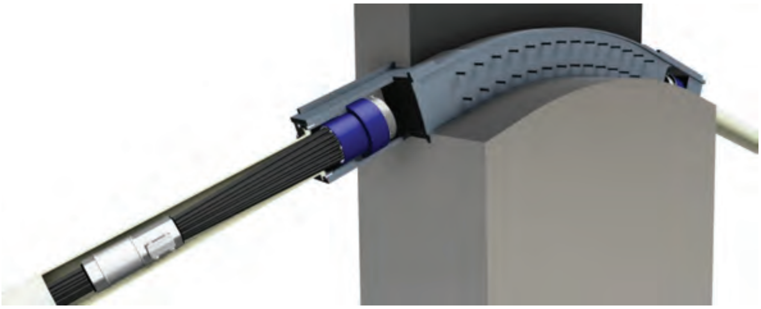


Figure 5.3: DYNA® link curved anchor box with two DYNA Grip® stay cable anchorages

- There is no need of sliding, and bond stresses and mechanical wear due to strand deviation are avoided.
- An unbalanced installation sequence as well as the use of two different cable sizes at one anchor box becomes possible (*).
- The left- and right-side cables of one box can be differently inclined, and there are no limits in the respective deviation radii (*).
- The design of the anchor box is fully in accordance with conventional steel construction standards with element always in tension.
- If any inspection and maintenance of the corrosion protection of the cable anchorage is required, it can be performed easily from outside the pylon structure.
- Possibility to design a slender and elegant pylon.
- No necessity of an access inside the pylon.
- (*) note: these possibilities have an impact on the pylon forces that the designer must consider and adjust the slenderness of the pylon.

This concept was used for the Nonthaburi Bridge in Thailand (*Fig. 5.4*). The contractor installed the steelworks in the formwork taking necessary measures to respect tolerances. The anchorages in this particular case were not external to the pylon head but were included in a pocket of the concrete in the pylon. The pockets were protected by steel covers along each row of anchorages which were installed after cable installation. The cover plates are removable for survey or inspection of the anchorages.

For this relatively new type of anchorage, fatigue and strains in steel and concrete need to be checked.

5.6.3 Saddle as Anchorage in Pylon (Type D)

Saddles have been used in several Extradosed Bridges worldwide. The saddle arrangement allows continuity of the tensile elements without a break and helps to limit the sizes of the pylons. The saddles resolve the tensile forces either side of the pylon while resisting slippage due to differential cable forces. There is a saddle for each cable. The advantages provided by saddles are:

Figure 5.4: Nonthaburi Bridge

- Reduced number of cable anchorages.
- No tension elements buried in concrete.
- No necessity for access inside the pylon.
- Ability to design a slender/elegant pylon.

Details have been developed for saddles to ensure water-tightness and corrosion protection. Saddles in general offer an excellent solution which efficiently meets the strength serviceability, fatigue, and durability requirements for extradosed cables.

There are two basic types of saddles: monotube and multitube.

5.6.3.1 Monotube Saddle Technology

The basic goals of the saddle are to deviate the cable, to provide anchorage of the cable, avoid slip resulting from differential loads on either side of the pylon, protect the cable through the deviation zone, and permit the replacement of the cable if required. The first layout was derived from the conventional external post-tensioning systems. An outer steel tube was installed in the pylon concrete and a smaller second tube (in steel or in HDPE) inserted. This second tube contained the strands that were unsheathed to assure anchorage once the second tube was filled with cement grout. This solution is referred to as a “monotube saddle.”[56] The following durability issues have been documented for monotube saddle[56]:

- Grout in the saddle increases fretting fatigue and damages the wire coating.
- Grout cracks due to the shrinkage or thermal effects expose the strands to corrosion.
- It is impossible to replace the cable one strand at a time, rather the full cable must be replaced.

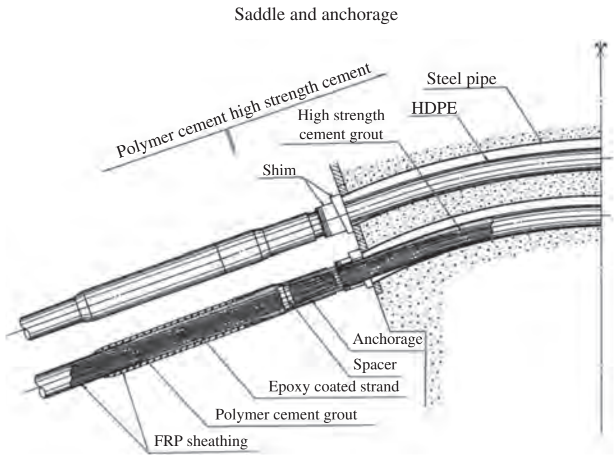


Figure 5.5: Odawara—saddle and anchorage

The stress range in the stay cables must be limited. A thermal variation of 10°C gives a strain of 10^{-4}, which brings $\Delta\sigma = 3$ MPa in short term, 1 MPa in long term. A stress range of 80 MPa imposes a strain of 4×10^{-4}. These tensions are also induced in the grout. To assure the integrity of the cement grout requires specific technical arrangements to limit thermal variation, and to increase the tension capacity of the grout.

The system of the Odawara Bridge (*Fig. 5.5*) follows this design with a small change at the pylon. The shim detail at the pylon (*Fig. 5.6*) face is installed to block any sliding of the stay cable, which remains continuous.

Figure 5.6: Odawara—shims at mast anchorage

5.6.3.2 Multitube Saddle Technology

More recent developments in saddles have led to systems where the protection of the strands is maintained with multiple nested barriers similar to modern stay cable systems. This saddle systems permit the replacement of each strand without changing the cable and has less fatigue issues as compared to monotube systems. Cable suppliers provide various proprietary multi-tube saddle with different shapes and characteristics.

The multitube saddle system shown in *Figs. 5.7, 5.8, and 5.9* permit each strand to be threaded individually with the following properties:

- This saddle system is a steel box with several steel tear-shaped tubes installed through several combs in a high resistance concrete surround.
- The space between the strand sleeves is filled with grout (*Fig. 5.8*).

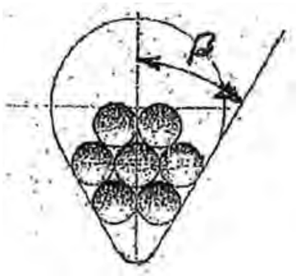


Figure 5.7: Hole section (copyrights: VSL)

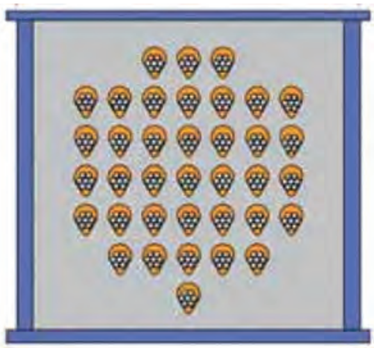


Figure 5.8: Cross section of the saddle (copyrights: VSL)

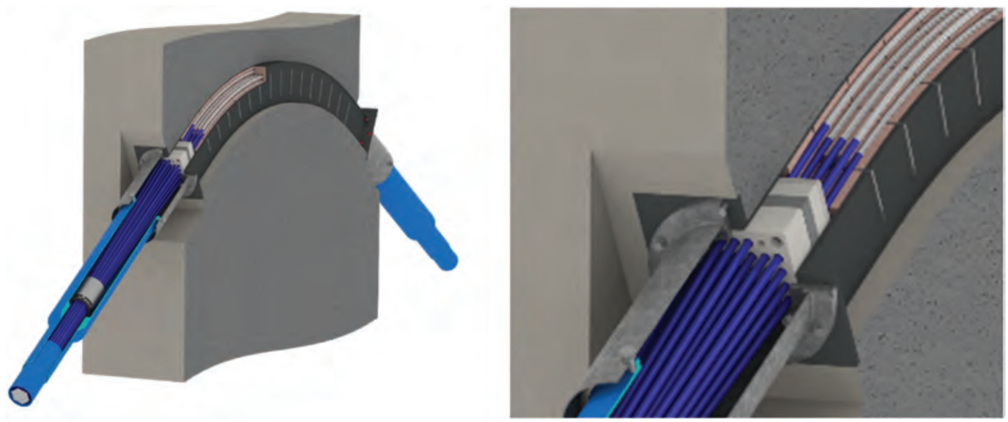


Figure 5.9: CAD view (copyrights: VSL)

- The anchorage of the strands that are locally unsheathed ensures the friction of the strands in each groove (*Fig. 5.9*).
- A final injection with a resipoly filler (a modified urethane epoxy slurry) is needed within each tube to ensure the corrosion protection of the strands.

Figure 5.10: View of multitube saddles (copyrights: Freyssinet)

Another multitube saddle shown in *Figs. 5.10 and 5.11* below utilizes a specific technology of sheathed strand avoiding the sliding of the wires in the HDPE sheath, which keeps its integrity without interruption. The saddle is cast into the pylon concrete. The saddle with recesses allows the strands to pass through and is filled with UHPF concrete. Each strand is replaceable.

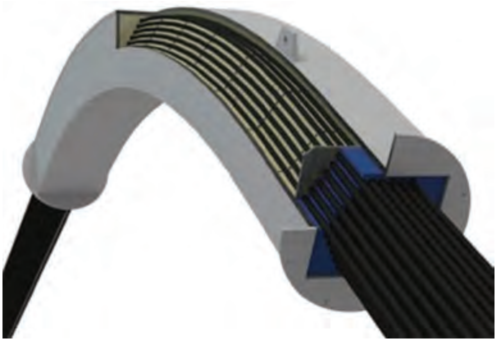


Figure 5.11: Cross section of the saddle (copyrights: Freyssinet)

5.6.4 An Alternate Layout Between C and D

This alternate concept utilizes stay cables which are not continuous through the pylon but connected with a short cable anchored on each side of the mast. This multi-tube saddle system is shown (*Fig. 5.12*) below. The system is buried within the pylon concrete. It combines post tensioning system with the stay cable system. So sliding is avoided. The system offers a continuity of the compression force applied to the pylon. Each strand is replaceable. A specific coupling detail is installed to ensure the continuity of the tensile element.

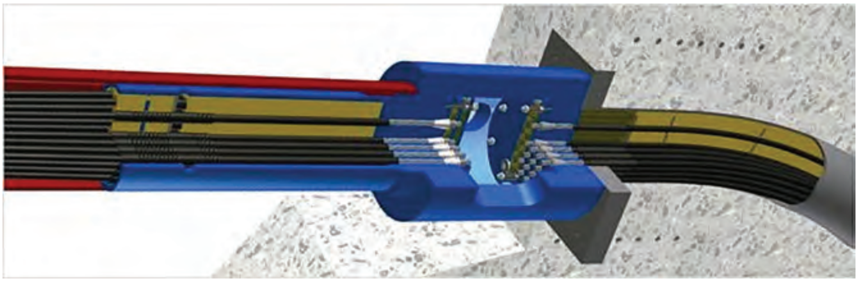


Figure 5.12: Saddle system with coupling details-BBR®

5.6.5 Choice of a Saddle

The C and D type pylon anchorages have specific characteristics that can be considered in choosing the type of saddle. These characteristics are as follows:

- Each system has an impact on the appearance of the bridge. This should be studied along with the owner to assess the visual quality of the solution.
- Stay cable assembly requires simultaneous work cycles at both sides of the pylon during construction of the deck girder. This has impacts on the cycle times for cantilever construction and the overall construction schedule.
- In some systems with galvanized, waxed and PE-coated 7-wire strands, the PE sheathing of the strand needs to be removed at the saddle to transfer differential forces into the pylon by friction. As a consequence, the maintenance free and factory-made corrosion protection of the strand is not continuous anymore and needs to be substituted with another system inside the saddle area.

- Saddle details must undergo time-consuming qualification tests for fatigue, sliding coefficient etc.
- The replacement of individual strands is sometimes not possible or only possible through a full cable replacement.
- Inspection of the saddle is possible from the outside of the pylon only. However, strand inspection in the deviation area can only be conducted by replacements of individual strands or of the complete cable.
- Installation and tensioning of each cable entirely in the C system creates alternative forces in the pylon possibly inducing cracks and are more aggressive than in systems A or B where the pylon is less narrow (the same is true for the median system between C and D).
- In some systems, differential forces are only transferred by friction. To avoid slippage the demand must be checked and confirmed by qualification testing such as those described in the revisions of PTI[10] and the recently published fib bulletin 89[50] and future versions to come. Many of these systems have been tested and suppliers are able to provide the declared value of this friction coefficient. In other cases, tests will be necessary.

The comments above need to be considered in comparing the relative merits of each solution.

5.6.6 Friction

The prevention of slippage is a fundamental design requirement for saddles with mono-tubes or multi-tubes. All loads or events must be considered. The following discussion will enable the designer to discuss the issue of friction with the cable suppliers.

The relationship between stress range and friction coefficient *f* and the deviation α through a saddle is given by the following formula:

$$\sigma = \sigma_0 e^{-f \times \alpha} \qquad (1)$$

With *f* as friction coefficient and α as deviation angle. This formula is without consideration of a safety factor. With respect to friction coefficient, the stress range can be analysed in accordance with the deviation angle for values between 25° and 65° (*Fig. 5.13*).

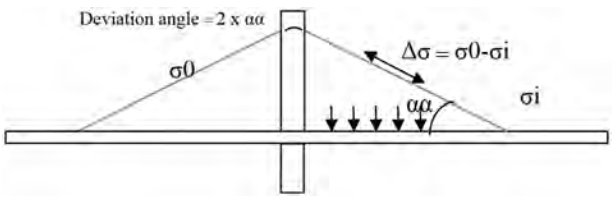


Figure 5.13: Deviation angle definition

To avoid slip of strands, the stress range under live load $\Delta\sigma_L$ must be less than the factored value for a given deviation angle and a given friction coefficient (*Fig. 5.14*) nominally speaking.

The friction coefficient analysis should be carried out at SLS and ULS. In the EC3-1-11,[51] this analysis is limited to ULS but this can be extended to SLS in adding specific requirements. This standard adapts formula (1) giving a partial factor for friction resistance $\gamma_{M,fr} = 1.65$, thus reducing the value of the friction coefficient. This standard proposes the following condition:

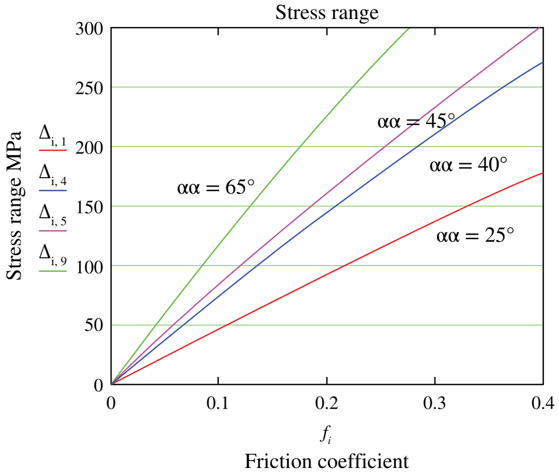


Figure 5.14: Range stress in accordance with friction coefficient and deviation angle

$$max\left\{\frac{F_{Ed1}}{F_{Ed2}}\right\} \leq e^{\left\{\frac{\mu\alpha}{\gamma_{M,fr}}\right\}} \quad (2)$$

where μ is the friction coefficient, α the angle in radians of the cable passing through the saddle, $\gamma M{,}fr$ is the partial factor for friction (recommended value is 1.65), $F_{Ed1 \text{ or } 2}$ the design values of maximum and minimum forces, respectively, on each side of saddle. The recently released seventh edition of PTI addresses this.

The cable supplier will provide friction values for saddle system giving different deviation angles. The designer needs to fix the required geometry of the cables to achieve the required friction value for the structure. It is typical to require testing to demonstrate the friction value for the saddle system. The reader is invited to refer to the revisions of both PTI and fib bulletin 89 to understand the proper assessment and qualifications of friction coefficients. fib bulletin 89 recommends a specific range of friction tests for extradosed bridges (section 6.4.2.1 of Ref. [50]).

5.6.7 Saddle Fatigue

According to fib bulletin 89,[50] the objective of the saddle fatigue test is to confirm the performance of the saddle in terms of fretting fatigue at the entrance into saddle. Stress range and maximum stress proposed by the new fib document are 180 N/mm² and 0.55 GUTS (the previous version did not include specific recommendations for Extradosed Bridges, only cable-stayed values were given).

As mentioned in the article “Simple Model for Contact stress of Strands Bent over Circular Saddles” in the proceedings of Stockholm IABSE symposium 2016,[60] an over tension appears at the position where the cable has its first contact with the saddle. A formula is given and tested in the above-mentioned article. This can induce fatigue effect due to stress range and fretting fatigue. Test of a 55 strands saddle system was related in Ref. [61]. It was completed later by a parametric analysis with test relying on the fretting map in Ref. [62]. The conclusion was that the current recommendations of the fib bulletin 30 for saddle fatigue tests do not guarantee the testing of the critical loading scenario for assessing the durability of the strands. Adaptions to the condition during the design live of the cable and for testing where made for bulletin 89.[50]

5.6.8 Current Developments

Some current developments involve the use of external post-tensioning tendons with an external anchorage allowing for the replaceability of cable.[63] To assure the replacement of the cable which is grouted, the saddle is not buried in the concrete but inserted within an outer steel box, which is buried in the concrete. Water-tightness must be assured at the junctions of the free length at the anchorage zone and at the saddle. The full system is grouted. In this technology, the stress range due to fatigue loading should be lower than $\Delta\sigma = 80$ MPa referring to Ref. [8]. So this solution is not far from the technology used on Odawara Blueway Bridge as exposed by A. Kasuga in Ref. [55].

5.7 Conclusion

The technology for extradosed cables has evolved in recent years as the Extradosed Bridge form has become more prevalent. Similar to cable stays it is expected that the systems for extradosed cables will continue to develop, resulting in effective systems for anchorages, saddles and cables that provide the required performance of extradosed stays in the best possible manner.

Chapter

6

Construction Issues

Chithabaram Sankaralingam, India

6.1 Introduction

The Extradosed Bridges usually have main spans between 100 and 250 m and side spans between 50 and 150 m. For such type of bridges, the precast/cast in place segmental construction method is considered most suitable. Segmental construction for both side and main spans may be constructed by the balanced cantilever method. Alternatively, the side span may be constructed by span by span erection method, by using temporary trestles if necessary and the segmental construction of the main span may follow using the free cantilevering method.

The method of erection is influenced by the stiffness of the pylon cable anchorage system, viability of installing temporary supports, maximum unsupported spans permitted, ease of transporting materials, and so on. However, since stability of the system largely depends upon transferring the horizontal component of the force in a cable through the girder, it is imperative to have girder continuity between each pair of stays.

Several Extradosed Bridges have been successfully built in the last two decades with differing spans, pylon heights and stay cable arrangements. The various erection techniques and construction methodologies are discussed under the forthcoming headings.

The type of deck plays a vital role in the construction stage analysis of Extradosed Bridges. The stay cables of concrete decks are normally installed in such a way that the deck under dead load will not deflect vertically. Accordingly, the bending moment distribution along the deck length will be similar to the bending moment distribution of a continuous beam on rigid supports and, consequently the effect of creep is minimised and the achieved desired alignment does not change with time. The stay cables may then be installed to predetermined unstressed lengths to achieve the desired alignment and the desired bending moment distribution under dead load regardless of the construction method/sequence.

For bridges with composite deck cross sections (concrete/concrete or steel/concrete), the installation of the cables to predetermined lengths, similar to the case of conventional bridges with concrete-only sections or steel-only sections, does not lead necessarily to the desired forces and alignment under dead load. The achievement of the desired results will be influenced by the time dependant forces due to the creep and shrinkage effects of the concrete.

The erection of deck by the use of the precast segments started in the 1960s and the first prestressed concrete bridge consisting of precast segments with match cast epoxy joints was Choisy-Le-Roi bridge over Seine River/France.

6.2 Pylon

6.2.1 Introduction

Pylon construction is not very different from pier construction. However, the accurate location of all components is the key. The pylon of an Extradosed Bridge can be built either in steel or in concrete. For concrete pylons slip form or climbing formwork methods are used. In the case of steel pylons, the segments are usually fabricated in the shop and transported to the project site for erection and installation. These are normally connected by high-strength bolts and/or welding and may be erected by floating or climbing or tower cranes. When using inclined towers, these pylons can be installed either horizontally first and then rotated vertically or vertically installed and then rotated horizontally.

6.2.2 Concrete Pylon

6.2.2.1 Lower Pylon Construction

The construction of lower pylons commences after construction of the pile cap. Pour height, the number of pours, and sequence of pouring are worked out in detail depending on the design.

Climbing-form itself provides workspace for rebar work. Specially designed forms prevent large deformation of formwork during concreting thereby enabling advanced precision management (*Fig. 6.1*).

Figure 6.1: Pylon and pier-table casting

Figure 6.2: Tower crane for pylon construction

Tower cranes of suitable capacity and operating radius are deployed for pylon construction, deck activities, cable installation and cladding works (*Fig. 6.2*).

6.2.2.2 Casting of Pier Table

Typical sequence of construction is:

- Erect bracket support on top of lower pylon to pier table during concreting.
- Erect partial precast pier table over supporting bracket using crane barge.
- Assemble shuttering, tie reinforcement and complete concreting of pier table.

6.2.2.3 Upper Pylon Construction

Upper pylon is usually constructed by using climbing form of lifts with separate forms for each wing. *Fig. 6.3* shows the construction stages of pylon arms. Climbing formwork brackets are fixed onto the completed portion of the pylon using high tension bars. Precise selection of formwork material and design of formwork ensure that the stripped form surface achieves the required degree of quality finish. Climbing brackets are designed for horizontal loads to withstand wet concrete pressure and for part self-weight of legs due to inclination. Pylon construction requires vertical (internal and external) access and working platforms at every level that meet all safety requirements.

Figure 6.3: Upper pylon construction

6.2.3 Steel Pylon

6.2.3.1 Erection

The erection of pylon is extremely critical as unanticipated locked-in stresses due to faulty erection process may lead to partial or full collapse. This would also lead to late completion of the bridge. However, it can be prevented by having a detailed erection scheme.

6.2.3.2 Pylon Erection

Although ordinary pylons can be erected without much difficulty, thin, curved, or inclined pylons or those temporarily supporting or resisting erection forces or loads require detailed erection schemes. The best person to design an erection scheme is the bridge designer, as he/she knows the structure intimately, has done the design, and develops a bridge model that can also be used to develop all erection stages for the structure. Once this is done, the specifications allow the contractor the freedom to modify that scheme or develop a separate erection scheme. If the specifications require the contractor to develop the erection scheme, the bridge designer should ideally check and approve the scheme before erection commences. During the concept-design phase, many different pylon forms and cable arrangements may be considered and each of them evaluated for aesthetics, constructability, and cost. Every alternative considered should have at least one erection method developed during the concept-design phase to ensure that it is constructible. The costs of erecting unusual pylon designs such as inclined pylons, or curved elements, are difficult to estimate and may add significantly to project costs.

The pylons and deck girder should be constructed according to an erection plan. Pylons constructed of structural steel are usually fabricated in a shop by welding together steel plates and rolled shapes to form cells. Cells must be large enough to allow welders and welding equipment, and if the steel is to be painted, painters, cleaning and painting equipment should be accommodated inside each cell. The steel pylon components are transported to the bridge site, erected by cranes and either welded or bolted together with high-strength bolts. For bolting, the contractor should use a method of tensioning the high strength bolts for consistent results to achieve the required tension. The difficulty in field welding lies in holding the component rigidly in position while the weld is completed. Controlling field welding in windy conditions can be tricky.

Field welding should be conducted under a protective covering to keep out water and wind. Full-penetration welds require backing strips that must be removed carefully if the weld is subject to fatigue loading.

Pylons constructed of reinforced concrete are usually cast in forms that can be removed and reused, or “jumped,” to the next level. Placing height for concrete is to limit pressure from the freshly placed concrete.

Reinforcing bar cages are usually preassembled on the ground, or on a work barge, and lifted into position by cranes. This requires the reinforcing bars to be spliced with each lift. Lapped splices are the easiest to make, but these are not permitted in seismic areas.

For shorter pylons, precast concrete segments can be stacked together and steel tendons tensioned to form the pylons. Pylon designers should consider the method of erection that contractors may use in constructing the pylons. Often the design can reduce construction costs by incorporating more easily fabricated and assembled steel components or easily assembled reinforcing bar cages and pylon shapes that can be easily formed. Of course, the pylon design cannot be compromised just to lower erection cost.

Some engineers and several architects design pylons that are angled longitudinally toward or away from the main span or those that are curved or kinked. These are possible if such a design can be justified structurally and aesthetically, and the extra cost can be covered within the project budget. These types of pylons require special erection methods though are not as difficult to construct as the longitudinally inclined ones. The sloping concrete forms can be supported by vertical temporary supports and cross struts that tie the concrete forms of each leg together. This arrangement braces the partly cast concrete pylon legs against each other for support.

As the sloped legs are erected, the inclination may induce bending moments and lateral deflection in the plane of the slope of the legs. Both secondary effects must be adjusted by jacking the legs apart by a calculated amount of force or displacement to release the locked-in bending stresses. If the amount of secondary stress is small, the solution could be in cambering the leg to compensate for the deflection and adding material to lower the induced stress. Neglecting this important construction detail can "lock-in" stresses and deflections can lower the factor of safety of the pylon and, in extreme cases, cause failures.

6.3 Deck Girder

The various methods of deck girder construction are:

- Erect on temporary props.
- Free cantilever with progressive placing.
- Balanced cantilever.
- Push-out

6.3.1 Deck Girder Erection

The various techniques adopted for erection are illustrated below:

6.3.1.1 Erect on Temporary Props

Prefabricated deck

This method is appropriate when the pylon is not designed with full end fixity to the pier or cannot be temporarily fixed, that is, the pylon is not stable unless the anchor cable is held in position. Temporary piers are first installed and the deck units are progressively placed one-by-one (and then welded together in case of steel sections at centre of main span) to form short free cantilevers (*Figs. 6.4 and 6.5*).

Figure 6.4: Erection of prefabricated segments on scaffolding

A derrick-type crane mounted on a rail track is commonly used for lifting and thus the weight of the unit will have to be significantly less than the derrick capacity. It may sometimes even be necessary for the assembly to be carried out in sections. Prefabrication normally takes place off site, and units are erected in lengths of 5–15 m. The length of free cantilever possible during the construction phase depends on the deck characteristics and must be carefully determined for the temporary conditions though un-propped sections of over 50 m have been successfully achieved. A similar procedure using precast concrete can be used but because of the much heavier weights involved, either shorter sections or specialized lifting carriages will be necessary until the stays are in position. On completion of the deck, all the stays are connected, tensioned and the temporary piers are then dismantled. However, some extension of the cable is unavoidable as the self-weight of the deck has to be taken up. The temporary propping should therefore be erected at a height calculated to allow for this movement.

Figure 6.5: Erection of precast segments on scaffolding; all precast segments are lifted at one location by a lifting frame and slid on top of the scaffolding longitudinally

Supporting the deck on temporary propping is a very economical solution for low heights and shallow rivers with adequate soil conditions and requiring no navigation. This method is also appropriate when the pylon is not designed with full fixity to the pier or cannot be temporarily fixed, that is, the pylon is not stable unless the anchor cable is held in position.

Cast-in-situ deck

Formwork for cast-in-situ decks can be supported by propping. During the pre-stressing of the stays, the deck can be supported by continuous or discrete propping (*Fig. 6.6*). The latter, however, is highly recommended for the sake of construction control. Indeed, for an accurate control of the stay pre-stressing, a real time control of the stiffness of the supporting system is required and the continuous propping renders it very difficult to identify the part of the structure that is supported and the part that has already been lifted.[64]

Figure 6.6: Concrete deck supported on temporary continuous or discrete propping

6.3.1.2 Free Cantilever with Progressive Placing

In many situations, where the installation of temporary supports is difficult and expensive, cantilever construction can be considered as an alternative. The side spans are constructed on temporary propping followed by the pylon. This part of the bridge is often situated on the embankment where cranes can be employed at ground level. The centre span is thereafter erected unit-by-unit working out as a free cantilever from the pylon. Like in the previous method, short concrete box sections or steel box sections of up to 20 m length are commonly lifted either by derrick or by mobile lifting beams and welded into place. Then, the permanent stays are fixed on to each side of the pylon. The provision of temporary stays is particularly important with precast concrete segments where units weighing up to 300 tons are occasionally erected. The normal procedure is to match cast adjacent segments and subsequently glue the joints with epoxy resin and bring the two elements together by temporary post-tensioning. The permanent cable is tensioned simultaneously as the temporary stay is released. The stay cable technique using temporary stays only has proved successful for multi short span bridges of the precast type. This progressive erection method allows units to be transported along the previously constructed deck, which are then rotated and attached to the lifting equipment such as swivel arm. Stays are usually tensioned with built-in hydraulic jacks, and the whole device moved forward from pier to pier as each span is erected and post-tensioned.

6.3.1.3 Balanced Cantilever

The need to have clear uninterrupted space below the bridge has forced designers and contractors to develop the balanced cantilevering technique, where very few props are required, as shown in *Fig. 6.7* below.

Figure 6.7: Balance cantilever technique

Erection proceeds simultaneously on each side of the pylon, with the first few sections over the piers temporarily supported on false work until the pylon has been erected and the cables attached. Like the other methods, a degree of cantilevering beyond the last attached cable may be possible depending upon the capability of the section to resist bending moments, the potential for this possibility being much better for steel segments than heavy precast concrete segments. When form travellers are used, interference with the shallow cables can become an issue.

An important feature of this technique is the need to have a stiff pylon and fixity between the deck, pylon, and its foundations, because of imbalances caused by construction plant, variation in segment dead weight, and tension in the cables. Wherever possible, the pylon should be designed to accommodate this requirement, otherwise substantial extra staying, temporary anchor cables or a heavy deck pylon fixing clamp has to be provided. Cantilever spans of over 150 m on either side of the pylon are commonly erected, but wherever possible, some propping is desirable to aid stability.

Superstructure (deck girder) construction

The stages of construction are usually as follows:

a. Stage 1
- Erection frame is erected on top of pier table using crane barge.
- Segments are lifted and erected in position using erection frame.
- Post-tensioning tendons and stay cables are installed after erecting each pair of segments (*Figs. 6.8–6.14*).

b. Stage 2
- Repeat the above stage for the remaining extra dosed cantilevered portions.
- Cast the closure joint and stress continuity cables by post-tensioning.
- Install barriers and overlays.
- Re-stress the stay cables.

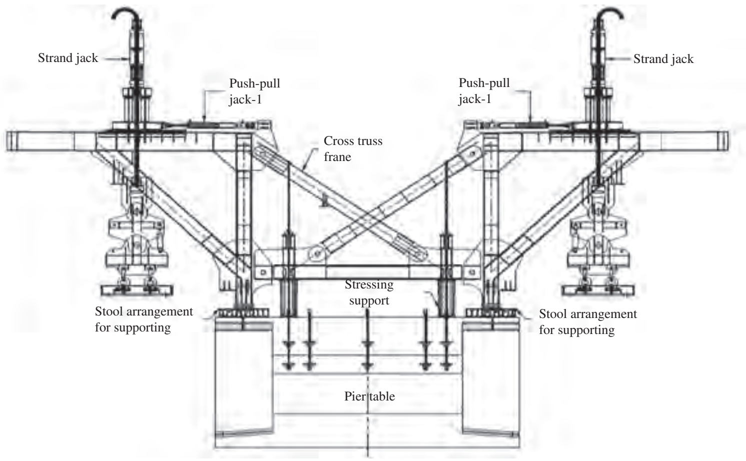


Figure 6.8: Bridge Builder frame is erected on pier table

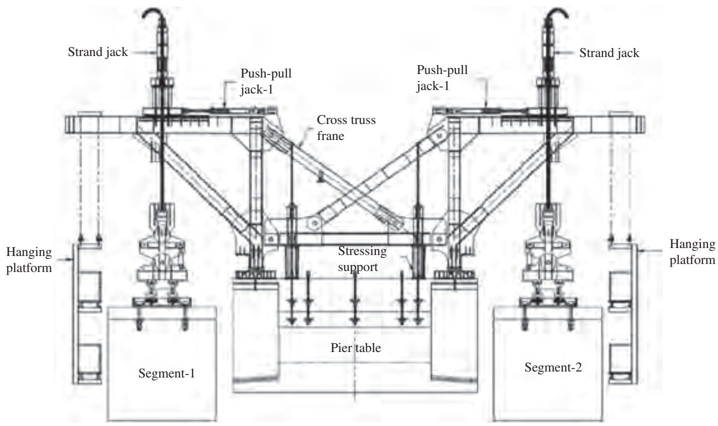


Figure 6.9: Segment 1 and 2 is lifted after rear macalloy bar is prestressed

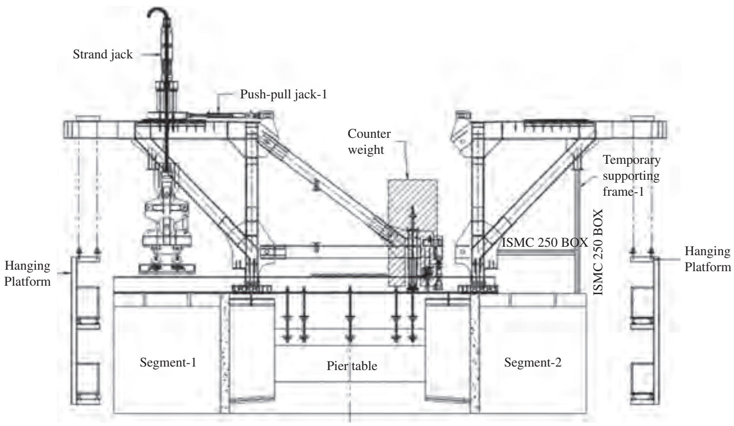


Figure 6.10: Rear portion of Bridge Builder frame is erected

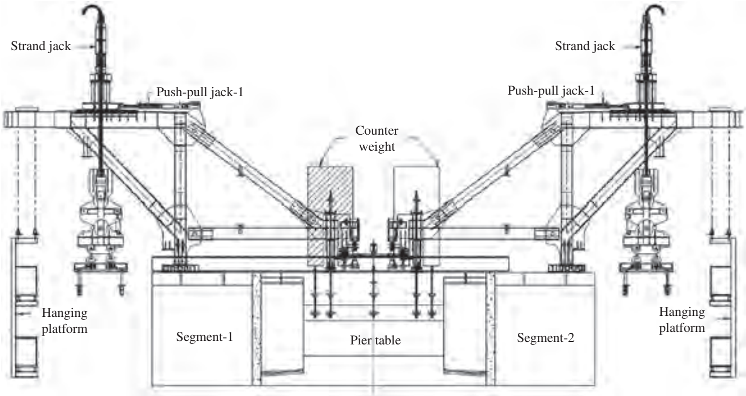


Figure 6.11: Lift and Tilt beam arrangement is mounted on Bridge Builder

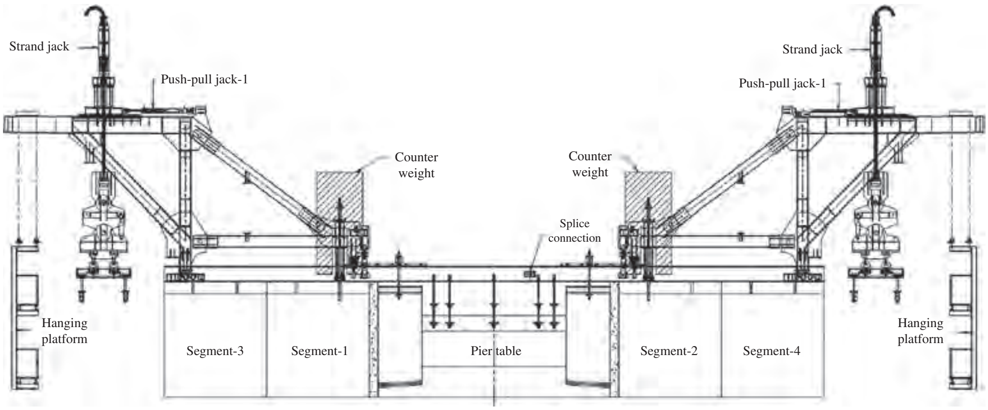


Figure 6.12: Bridge Builder frames are shifted forward and tied down with segment 1 and 2 respectively

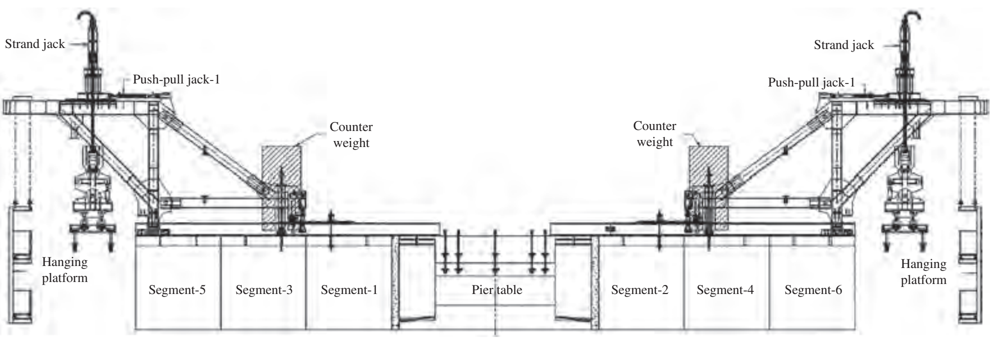


Figure 6.13: Rail beam and Bridge Builder frames

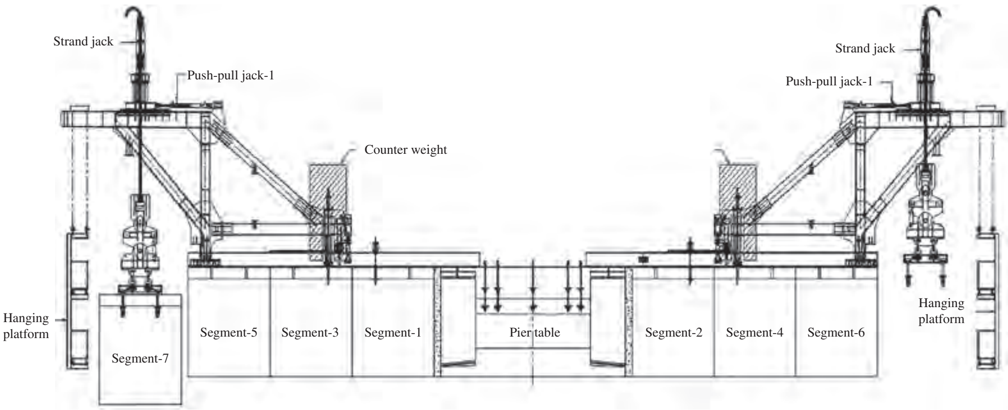


Figure 6.14: Erection of segment 7 (Extradosed Bridge across river Narmada)

c. Stage 3 Back span erection

The segment at the end of cantilever span, that is, after the stitch segment and behind abutment is considered as the back span, Back span can be erected by temporary portal arrangement and the construction sequence is as follows:

- Erect the temporary portal arrangement for back span.
- Lift segments with a barge crane and place it on the temporary portal arrangement.
- Segments are then assembled from the abutment side to the end of the cantilever span before stitch segment.
- Segments are temporarily stressed and finally stressed after the construction of stitch segment (*Figs*. 6.15–6.17

Construction of a new long Extradosed Bridge over River Ganga at Patna, India, will provide an alternate crossing and become a critical link connecting Northern Bihar to Patna and Southern Bihar.

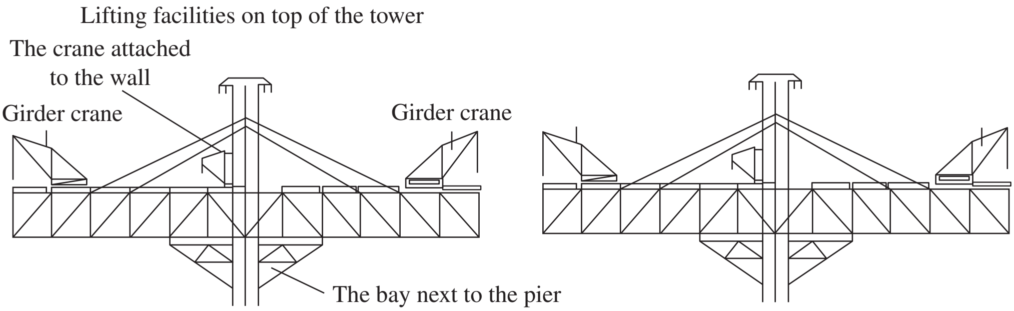


Figure 6.15: Steel girder erection. (From Zhou, M. B., Bridge Construction, (2): 14–19 (in Chinese), 2000a.)

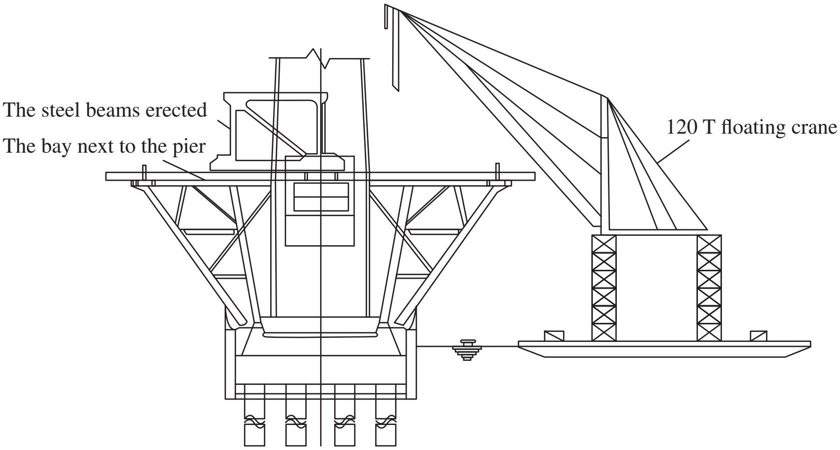


Figure 6.16: Inter-panel steel truss segment erection over pier top. (From Hu, H.Z., Bridge Constr., 3, 1–4, 2007.)

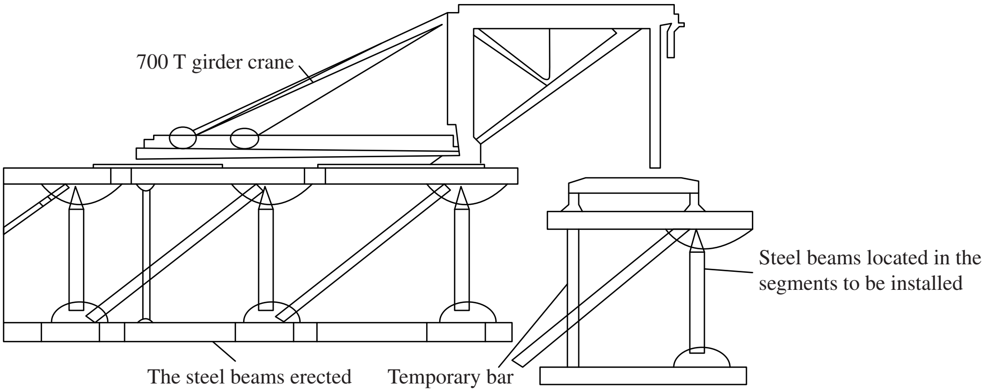


Figure 6.17: Whole segment lifting.

Construction methodology of six lane Extradosed Bridge on river Ganga (From Hu, H.Z., Bridge Constr., 3, 1–4, 2007.)

The construction methodology of the typical block of main bridge is the precast segmental balanced cantilever method using derrick crane. The pier table is considered cast-in-situ method. The precast segment of the deck will be fabricated at facility yard first, transported to the site and erected by the derrick crane in balanced cantilever method. The construction scheme of the main bridge is as per the following (*Figs*. 6.18–6.22).

6.3.1.4 Incremental Launching Method

In some situations, access beyond the abutment may not be available or deck units cannot be transported to the pylon over adjoining property. To overcome these difficulties, a few bridges have used the incremental launching method as illustrated in the *Fig. 6.23* below. The deck is

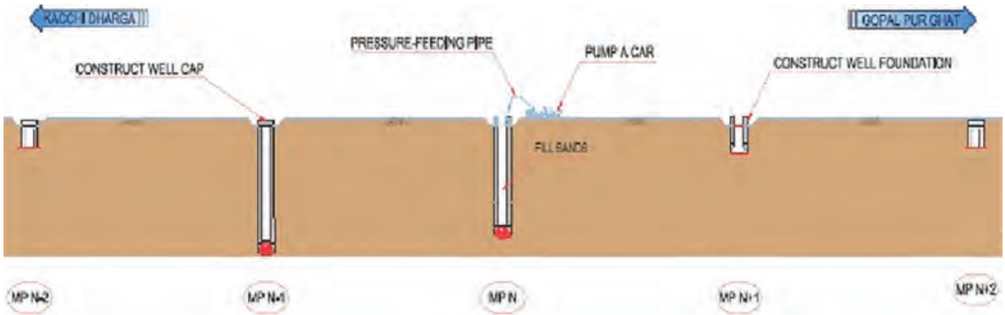


Figure 6.18: Construction of caisson foundation (sinking and concreting for steining is a staggered process)

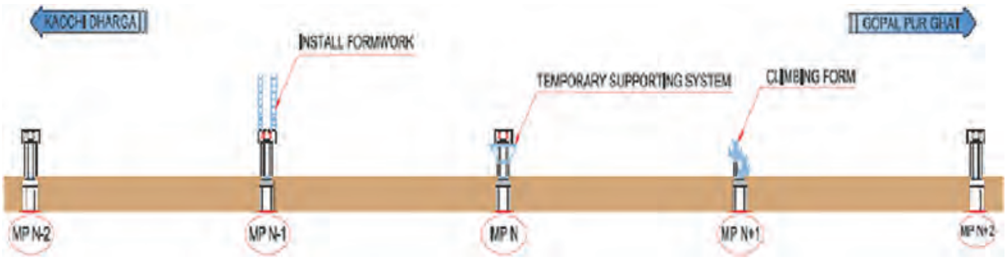


Figure 6.19: 1. Fabrication of reinforcing bar and casting concrete for lower pylon. 2. Installation of the temporary support system for pier table. 3. Construction of pier table and pylon.

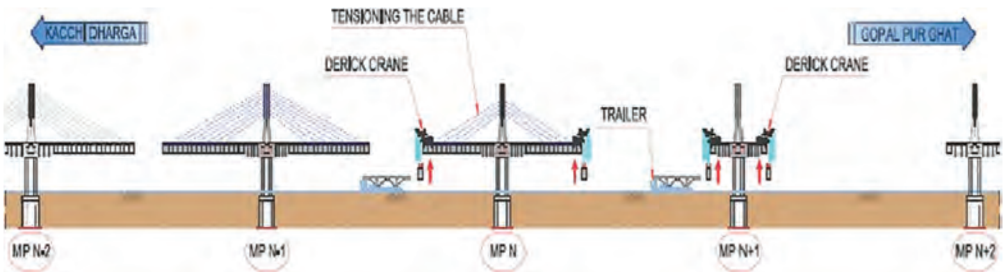


Figure 6.20: 1. Transport of the segments by trailer. 2. Installation of the segment in due sequence by derrick crane. 3. Tensioning of the cable and tendon in due sequence.

assembled at one of the abutments and simply winched out over the rollers or Teflon pad bearings. A similar technique has been used with incremental launching when temporary stay cable are used rather than props.

6.3.1.5 Erection Feasibility

The bridge designer is responsible to his client and to the public with respect to the erection of the bridge that includes (a) making certain, during the design stage, that there is a feasible and

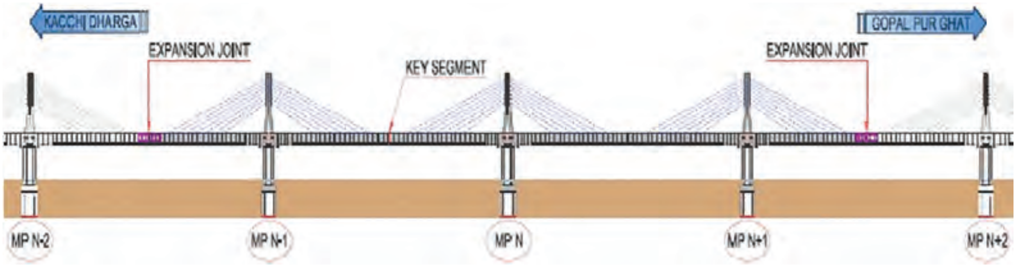


Figure 6.21: 1. Lifting of key segment formwork by derrick crane. 2. Lifting the internal movement joint. 3. Construction of the key segment. 4. Installation of the internal movement joint (Needle Beam)

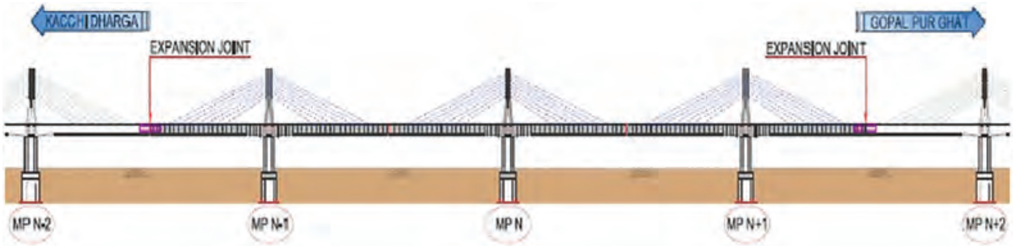


Figure 6.22: Installation of the expansion joint. 2. Installation of the wearing surface and road

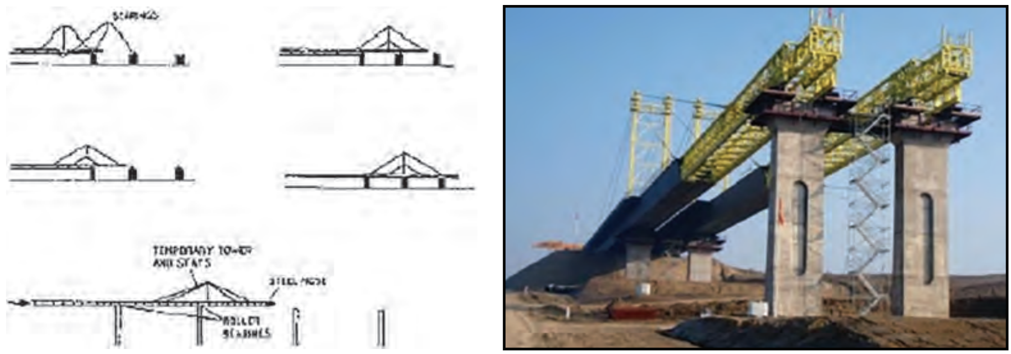


Figure 6.23: Pecs bridges the flat valley of the Szebenyi–river. The straight superstructures were erected by the incremental launching method. The launching nose was supported by stay cables from a temporary tower, located on the deck.

economical erection method; (b) setting forth in the contract documents all necessary erection guidelines and restrictions; and (c) reviewing the contractor's erection scheme, including any strengthening that may be needed, to verify its suitability. It may be noted that this latter review does not relieve the contractor from the responsibility for the adequacy and safety of field operations. Bridge annals include several cases where the design engineer failed to consider erection feasibility.

6.3.2 Stay Cable System

6.3.2.1 Installation and Stressing of Stay Cable

Stay pipe erection

The stay pipe or duct is lifted with the first strand to be installed that is connected at both ends to anchorages and then stressed. The stay pipe is then raised for erection using a tower crane and secured to the pylon.

Strand installation—initial stressing of stay

One or two strands that are connected to anchorages are pulled through the stay pipe and then stressed individually, using an automatic, computer-controlled system that ensures that all the strands are parallel.

Strand installation—final stressing

Strand installation methods are engineered to avoid any kind of cable de-tensioning. All the stressing operations, including fine-tuning, are carried out using an automatic mono strand jack. A compact multi strand jack is used only when final stressing/de-tensioning is unavoidable (*Fig. 6.24*).

The strands are often tensioned one by one thereby eliminating the need for heavy pre-stress equipment. The method has several variants depending on the patent used by each company. It involves pre-stressing first single strand up to the defined stress level. Once the strand is anchored, the rest of the strands are successively pre-stressed one by one until their stresses match the stress of the first one which could, however, lose its tensile stress when the others strands are pre-stressed, due to the elastic shortening. This aspect has to be considered in the design of tensioning process along with the fact that often the first strand is carrying the weight of the cable duct on its own.

Right through the process of structural analysis of the construction process, temperature is usually assumed to be constant and equal for all structural elements. However, at every stage, it is possible to find differing temperatures in various parts of the structure on site depending on their different thermal properties and colours. Pre-stressing operations are performed under varying ambient temperature. Although this phenomenon is often ignored by the contractor, there is no doubt that taking cognizance of it will reduce the differences between the designed forces and

Figure 6.24: stay cable system

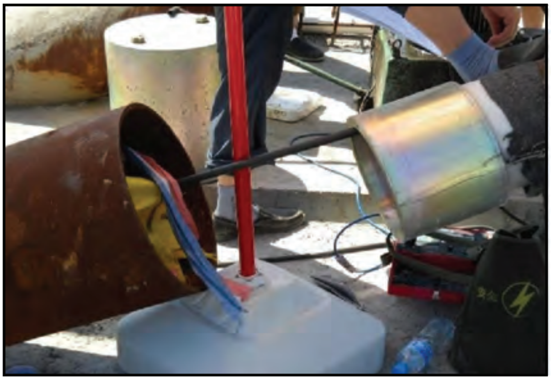


Figure 6.25: Stay duct weight born by the first strand in the strand by strand tensioning technique

actual ones at the end of the pre-stressing process. Hence, recording the temperature of stays, sheathes, deck and pylon and modifying the tensioning strategy according to the actual situation on site is strongly recommended.[65]

Anchorages usually allow for a global re-tensioning of the full stay. However, heavy jacks and oil pumps are required. A site engineer therefore has to carefully design the initial stress of the first strand[66] to avoid re-tensioning operations (*Fig. 6.25*).

The cable end connections

The cable is normally connected to the pylon with pin-type joints as shown in the *Fig. 6.26* or alternatively placed in the groove or guide tube of a saddle, depending upon the design requirements. The cable ends for the pin-type connection have either swaged or filled sockets. Swaging involves the squeezing of a socket into the wire in a hydraulic press and is generally used for strands having diameters in the range of 10–40 mm. Filled sockets are more suited for the larger diameter parallel wire type cables that have bundles of wires.

Several other types are manufactured differing slightly in the form of dead ending of wires and the type of filling material. In its simplest form, the wires are led through a plate at the base of the

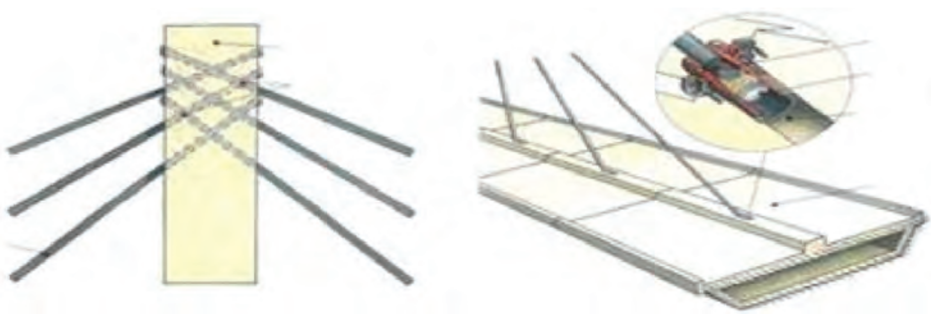


Figure 6.26: Cable end connections- Pin Type Joint

socket and finished with a button head or sockets and wedge. The inside of the conical-shaped socket is subsequently filled with an alloy of zinc, copper, aluminium, or lead, or sometimes with a cold casting compound such as epoxy resin. Thus, when the cable is subjected to tension, a wedging action develops that increases the grip on the wires. The deck- to-cable connection is usually of the "free" type to accommodate adjustments. A flared arrangement is required for multi-strand cables while mono-strand cables require only a single socket. Initial tensioning of the cable to remove slack is generally carried out with a hydraulic jack similar to that used in pre-stressed concrete. The socket is therefore often manufactured with an internal thread for the jack connection and external thread and nut to take up the extension and other adjustments. The cable is normally housed inside a protective covering, pulled to the calculated tension, and then the tube is filled with grout material to further protect the cables. A hydraulic ram is used to apply the correct amount of tension from the top of the pylons.

6.3.2.2 Temporary Stay Cable

It is also important to note that temporary stay cables may be feasible to erect the segments in the central part of the main span where there are normally no permanent stay cables (*Fig. 6.27*). The use of temporary stay cables is easier and more economical as compared to the use of a large number of temporary pre-stressing bars between the deck segments. The anchorage of the temporary stay cables at the deck and the pylon head levels may be arranged as shown in *Figs. 6.28 and 6.29*.

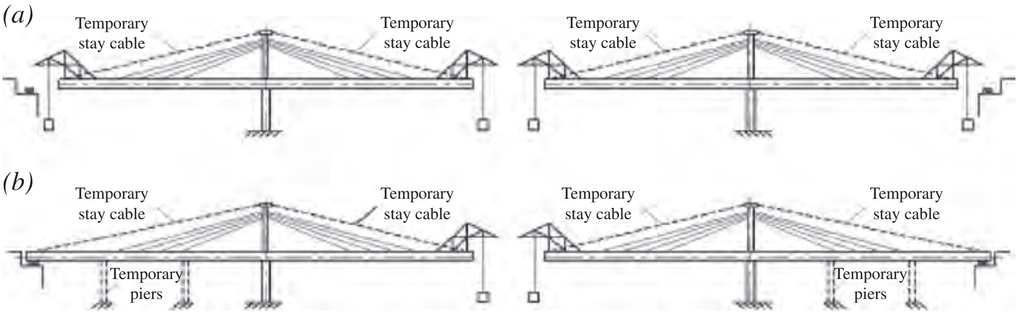


Figure 6.27: Possible construction method of Extradosed Bridges using temporary stay cables

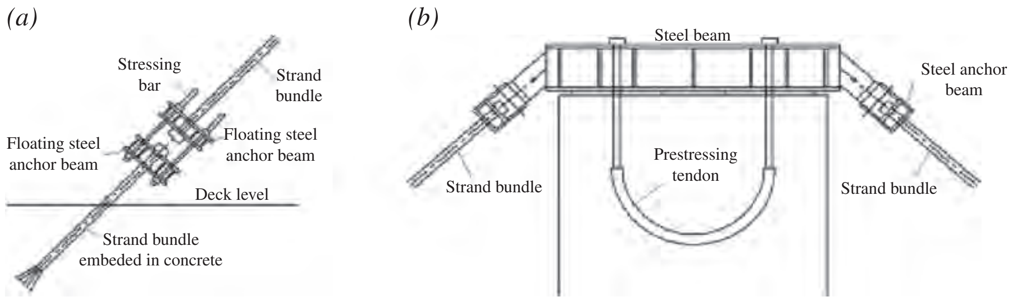


Figure 6.28: Anchorage of the temporary stay cables at the deck and pylon head levels

The stressing of the temporary stay cables can be carried out at the deck level using stressing bars (*Fig. 6.28*a) and floating steel anchor beams.

Figure 6.29: Use of temporary stay cables for the construction of Wadi Abdoun Bridge in Jordan

The anchorage of the temporary stay cable at the pylon head should be provided with two independent pin assemblies allowing the anchor to rotate vertically and horizontally to orient the temporary stay cable to different anchorage locations at the deck level. Such arrangement was used for the construction of Wadi Abdoun bridge in Amman/Jordan (*Fig. 6.29*).

6.3.2.3 Cable Erection

These days, majority of the Extradosed Bridges are designed with mono-strand cables of either the parallel wire or locked coil wire types. A complete stay is manufactured in its polyethylene tubing and delivered to site on reels. The simplest erecting procedure is to unreel the cable along the deck and hoist or lift it up to the top of the pylon. Unfortunately, the natural sag tends to be quite large and therefore considerable take-up has to be provided in the tensioning jack. A more satisfactory procedure is to install a guide rope and pull the cable up with a hauling rope. Intermediate supports to reduce sag have to be provided through intermittently spaced sliding hangers. Tensioning is initially carried out at the deck connection end to take up the slack, final tensioning to remove bending moment in the deck and transfer dead load into the cable being supplied after all work on the newly erected section is complete (i.e. welding, post-tensioning of concrete segments, etc.). The jacking equipment is similar to that used for prestressed concrete with the threaded bar system. Finally, the duct is filled with pumped grout for protection against corrosion.

6.3.3 Camber Control

It is important to control camber and to achieve the required road geometry. Deck finish level of a concrete bridge built with the balancing method can be lower than expected due to concrete creep and shrinkage. A construction camber helps to counteract this type of deformation (*Fig. 6.30*).

Factors effecting the camber:

- Self-weight, Form Traveler (F/T) weight, pre-stressing, temperature, erection load.
- PS tendon property: elastic modulus, relaxation.
- Concrete property: elastic modulus, creep, and shrinkage.
- Deflection of Form Traveller (F/T).

Figure 6.30: Form traveller erection arrangement

6.3.3.1 Form Traveler Deflection

The amount of camber by such kind of non-structural deformation is not included in camber calculation. Therefore, before erection work starts, camber should be adjusted based on calculations during construction. However, the estimation is based on ideal assumptions. Actual field values can be different from the calculated result. Therefore, the actual amount of displacement should be carefully monitored and corrected at the early stages of erection. When the Form Traveler is assembled, gaps between the connection parts make additional displacement. This value generally will be assumed at 20 mm and adjusted at first or second segment erections.

6.3.3.2 Additional Camber

Prestressing the bottom sections with continuity PT, long-term deformation by creep and shrinkage and unexpected vertical displacement should be included in the camber. Additional camber is applied to compensate for any additional displacement and distributed at each segment of the erection stage. Generally, the amount of additional camber at each stage is 50% of the difference between the initial segment level and target geometry of the segment. After the entire erection is completed, creep and shrinkage deformation is supposed to increase up to a final finish level. 50% of long-term displacement is generally included in the additional camber value.

6.3.3.3 Error Correction

Unexpected errors between calculations and field values can be distributed over 2 or 3 segment erections to adjust the camber. Camber lines are monitored and measurements conducted as early as possible at the same time. Errors can be distributed over several stages of erection. If there is a change of sectional properties, measurements have to be done carefully.

6.4 Case Studies

6.4.1 Odawara Port Bridge, Japan

The Odawara Blueway Bridge is the first extradosed precast box girder bridge in the world and was completed in 1994. This bridge was designed with a three-span continuous box girder with the extradosed prestressing, having a middle span length of 122 m length, a pylon 10.5 m high, and a girder with depth of 3.5 m at supports. An allowable stay cable stress of O.6/pu was adopted in this bridge. Stay cables are anchored outside the saddle at the top of the pylon to satisfy the requirement of not allowing them to slip, which would create a difference in cable force on either side of the pylon. Moreover, high damping rubber dampers were installed at the bottom of each stay cable to suppress rain induced vibrations. The validity of strength of the saddle was first confirmed by testing on a full-scale model. Then the flexural fatigue test of the stay cables and the performance test of the dampers were carried out. Construction method adopted was similar to that of free cantilevering of cable-stayed bridges. Cable force adjustment during and after construction was not required, since all forces on stay cables decrease with the progress of creep and shrink-age, just as when prestressing steel is placed inside concrete girders. The anchorages for the stay cables are identical to those for the prestressing tendons in the girders as no cable force adjustment was needed and the stress change due to the live load is low. The highly damped rubber dampers were used to absorb stay cable vibrations. This damper is installed between the pipe and the stay cables, so that the dampers can be hidden inside the cable cover. This arrangement also has advantages from the point of view of the aesthetics of the bridge (*Fig. 6.31*).

6.4.2 Narmada Bridge, Bharuch

Narmada bridge is India's longest extradosed cable stayed bridge which was opened to traffic in March 2017. The length of the main bridge is 1344 m with two end spans of 96 m and eight internal spans of 144 m. The superstructure consists of 20.8-m-wide precast segmental concrete box

Figure 6.31: First Extradosed Bridge–Odawara Blueway Bridge.

Figure 6.32: Narmada Bridge, Bharauch

girder, with a carriageway to accommodate four lanes of traffic. Narmada bridge design and construction comprises of bored cast-in-situ pile foundation, pier-cap, Y-shaped pylons, and extradosed cable deck which consist of 3–5 m varying length segments having three-cell precast segmental box girder cast with short-line method and erected with balance-cantilever underslung method by Bridge Builder, a custom built erection gantry. Stay-cables and post tensioning systems are provided to have structurally sound decking system (*Fig. 6.32*).

The bridge has segmental precast girders erected in balance cantilever manner on both side of pylons supported on pile foundation. Concept is more or less modular based design, where nine numbers of pylons with balance cantilever extradosed design along with two abutments makes the bridge. Bored cast-in-situ piles of 1.5 m diameter have been provided with over 2 m thick pile-cap. Top of pile cap is at low water level. Vertical pile capacity is adjusted for scour condition by considering overburden pressure from scour level. The typical pylon for the main bridge substructure is chosen to have a Y-shaped with rounded corners to improve aesthetics and to reduce wind and water current loads. To cast this kind of shape, special steel formwork with adequate scaffolding and false-work system was designed and provided to have geometric control during construction.

The super-structure of the main bridge is a three-cell precast segmental box girder with depth of 4.0 m. Sloping outer webs connect the top slab and the inner vertical webs to stabilize the top slab in transverse direction and transfer stay force to the bottom of inner/vertical webs. Soffit corners are rounded due to presence of transverse tendons in outer sloping webs. It also reduces the wind drag. The length of typical segment is limited to 3.550 m to limit the weight of the segments during handling. Segments are match-cast. Integral connection at the pylon location of substructure and the superstructure is provided by pier tables. Anchor saddle boxes are provided at upper pylon which provides individual support for each strand and avoids lateral pressures due to grouping of strands. Balanced cantilever construction method was used to erect the box girders with epoxy joints between segments. For service and ultimate load condition adequate internal post-tensioning is provided.

Construction of the bridge involves different phases shown schematically in *Fig. 6.33*.

The precast segmental "Bridge Builder" (*Fig. 6.34*) was designed for a maximum precast segment length of approx. 3–5 m and load capacities between 100 and 300 tons. It is equipped with

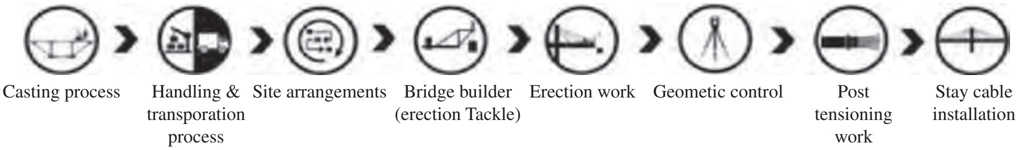


Figure 6.33: Bridge span constriction phases

Figure 6.34: Bridge Builder arrangement on pylon-span

two hoists for lifting the precast segments as well as for adjusting the cross- fall. A manipulator permits adjustment of the longitudinal fall and hydraulic cylinders launch the device forward. The Precast Segmental Bridge Builder is designed and adopted for balance cantilever precast segment erection. The Bridge Builders on both end of the spans where initially erected on pier-table only. Then one by one precast segments are lifted simultaneously and fixed as motioned in erection section involving prestressing and stay cable system.

The erection procedure includes four-cyclic activities, which are listed in *Fig. 6.35*.

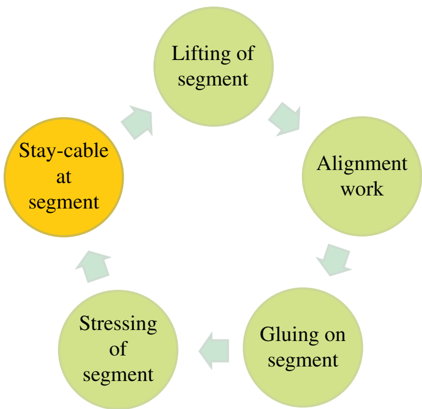


Figure 6.35: Erection work flow

Chapter

7

Cost/economics

Juan Sobrino, Canada

7.1 Introduction

Selection of the most appropriate structural type for a specific bridge is very much dependent on the cost of the bridge, its character, context, environmental and sustainability considerations, site conditions and constraints, life-cycle cost, and construction methods. Construction cost among others is one of the most relevant considerations. Therefore, this chapter provides some guidance on preliminary estimate of the construction cost based on unit quantity ratios and assesses the cost-effectiveness compared to other structural types, such as girders and cable-stayed bridges.

Extradosed Bridges, as other cable-supported structures, are perceived as an expensive structural type for medium-span bridges, probably because these bridges require a more sophisticated design and construction process. However, it has been proved in Japan, and more recently in Europe and America, that Extradosed Bridges are cost-effective for span lengths ranging from 150 to 250 m. For this span length range, Extradosed Bridges can be a competitive alternative compared with steel girders, concrete girders built in balanced cantilever, arches, steel trusses, and cable-stayed bridges, depending on site conditions and project requirements.

Based on the statistics and typical price units (Florida-based) it is possible to make cost comparison between both alternatives. *Figure 7.1* illustrates the results of a cost estimate per unit deck surface area versus the span length. The comparison should be interpreted in a qualitative way, as unit cost depends on site conditions. According to this study, construction cost of Extradosed Bridges is similar to balanced cantilever bridges for span lengths varying between 110 and 150 m and more economical for longer span lengths.

Reliable comparable construction cost to a specific bridge are difficult to find and interpret because each bridge project is unique. Construction cost are not only related to material quantities, labour, and unit prices, but to construction methods, geotechnical, environmental, seismic, construction site constrains, design codes, future maintenance provisions, finishes and many other factors to be considered. For this reason, this chapter summarizes the statistics of quantities that would allow the Transportation Agencies and bridge engineers to conduct a first rough construction cost estimate during the preliminary phases of the design and alternative bridge cost comparison. The curves have been prepared from data provided by designers and contractors[67]

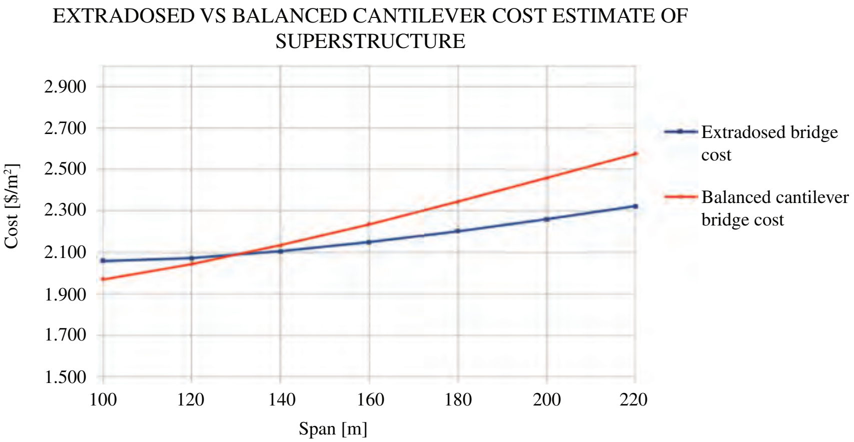


Figure 7.1: Estimated construction cost comparison of Extradosed Bridges and balanced cantilever bridges designed with AASTHO Codes and unit material price of Florida in 2019.[68]

and are assumed to derive from optimized Extradosed Bridge solutions. However, they should be critically interpreted and compared with those derived from a preliminary design based on a specific structural design to account for project specific considerations.

A research study conducted in 2012 and published in 2016[69] collected information on 120 Extradosed Bridges around the world, including cost information on 58 bridges. The statistics on the construction costs show a significant dispersion. *Table 7.1* summarizes the statistical analysis expressed in US dollar at its present value in 2012. The authors also noted that the average construction unit costs of superstructures (deck and pylons, including stays, but without piers and foundations) are much higher than the construction cost of concrete girder superstructures built in balanced cantilever, which ranges between US$ 540 per m^2 and US$ 2700 per m^2 of deck surface area.

Construction cost is highly dependent on the region. While the average construction of small and medium span bridges (typically made up of either concrete or steel girders) in the USA range between US$1400 per m^2 and US$2000 per m^2; in Canada, the construction cost of river crossings varies between US$1900 and US$3500 per m^2. Historical data of long-span bridge construction cost vary widely with the span, structural type, or site conditions. *Figure 7.2* depicts the historical data of Japanese bridge construction cost of various structural types expressed in terms

	Count	Mean (US$/m^2)	Max (US$/m^2)	Min (US$/m^2)	Standard deviation (US$/m^2)	Coefficient of variation (%)
Total cost	58	9964	58 545	1163	12 543	126
Total cost superstructure	8	7397	12 013	1302	4668	63

Table 7.1 Statistics of construction cost of Extradosed Bridges around the world based on bridge deck surface area. Value in US dollar at its present value in 2012.[69]

of a Cost Index. A Cost Index 1 means the average cost of a 100-m-long span box girder bridge. This figure is in line with the general statement that EDB are competitive for spans over 150 m, but does not show that they are more economical than conventional girder bridges.

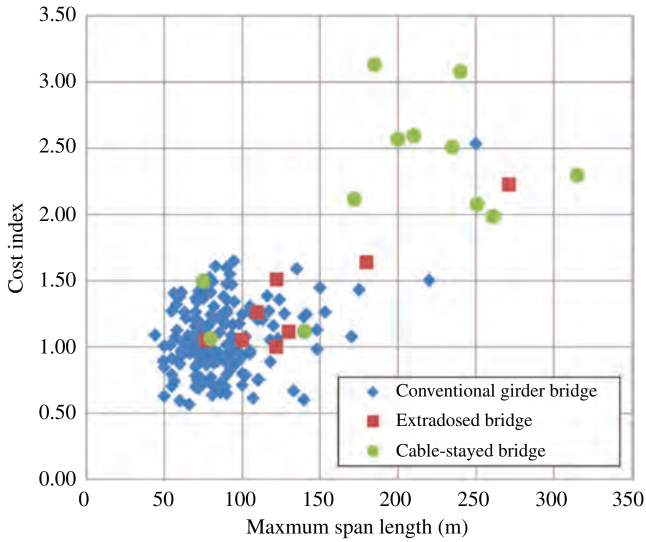


Figure 7.2: Construction Unit Cost Index of concrete and Hybrid structures in Japan (Courtesy of Sumitomo Mitsui Construction).

7.2 Estimate of Quantities

This chapter summarizes the historical data collected by the authors on 54 bridges all over the world. The data have been provided by the designers and extradosed stays' suppliers and include road and a few railway bridges. The results of the bridges identified are summarized in the following sections. Most of Extradosed Bridges are built in concrete, although there are a few built using a composite steel and concrete deck. The average main span length of these bridges is 131 m with a coefficient of variation of 34% (C.O.V. = standard deviation/mean).

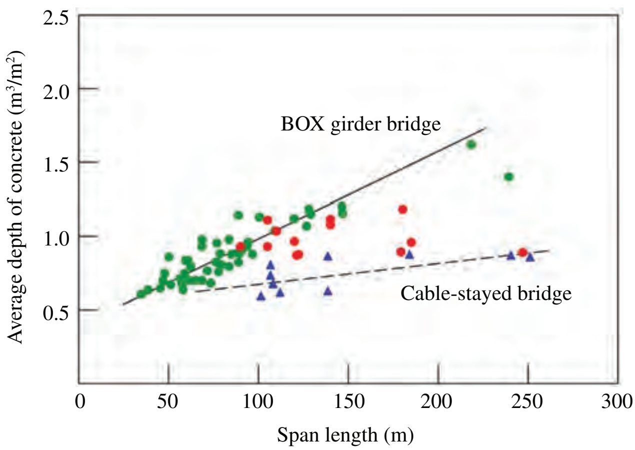


Figure 7.3: Span length versus average depth of girder concrete (Courtesy of Sumitomo Mitsui Construction). (red = Extradosed Bridges, green = box girder bridges, blue = cable- stayed bridges).

7.2.1. Concrete

The volume of concrete used for the superstructure (deck) ranges between 0.4 and 1.4 m^3 of concrete per m^2 of deck surface area, with an average value of 0.9 m^3/m^2 and a coefficient of variation of 33%. Most of the bridges are in seismic regions. It should also be mentioned that when compared to box girders Extradosed Bridges might require higher strength concrete, because the shallow cables cause high deck compression. *Figure 7.3* shows a comparison of concrete consumption of Extradosed Bridges with other bridge types

Material	Mean	Coefficient of variation (%)
Reinforcing bars (kg/m^3 in concrete deck)	168	22
Reinforcing bars (kg/m^3 in pylons)	290	33
Post-tensioning steel (kg/m^2 of deck area)	42	50

Table 7.2 Statistics of material quantities of Extradosed Bridges collected for this report

7.2.2. Reinforcement and Pre-stressing in Concrete Elements

The statistics of ratios of reinforcing bars and prestressing steel from the Extradosed Bridges collected in this report are summarized in *Table 7.2*. The values are slightly higher than those reported for concrete bridges built in balanced cantilever; likely because the collected data for this chapter includes a significant part of bridges located in high seismic regions or because smaller stay cables were used.

The historical collected data are in line with the results of a recent study on Extradosed Bridges with spans varying from 60 to 160 m. The bridges were designed with the AASTHO Code and the range of prestressing steel ranges between 20 and 30 kg per m^2 of deck area, *Fig. 7.4*.[70]

7.2.3. Extradosed Stays Quantities

The amount of steel in stay cables is one of the most significant factors that affect the cost of a cable supported bridge. summarizes a comparative study of the total amount of prestressing tendon used either as internal, external, or stay cables. The total tendon weight required in Extradosed Bridges is in between the typical ratios of box girders and cable-stayed bridges. However, the construction and maintenance cost of internal post-tensioning steel is much lower than

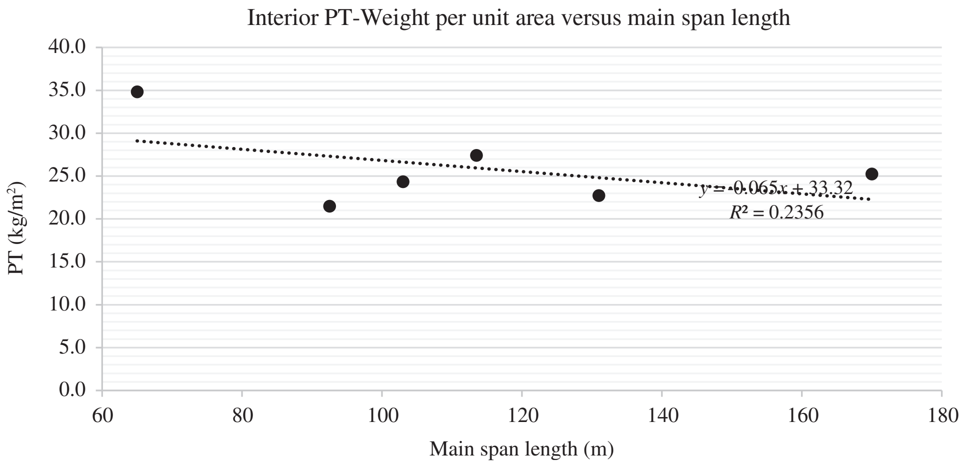


Figure 7.4: Span length versus pre-stressing steel weight.[70]

the overall cost of stays. The data of Extradosed Bridges show a wide scatter as it includes bridges with one and two planes of extradosed stays, as well wide decks (*Fig. 7.5*).

The data collected for this report on 54 Extradosed Bridges are plotted in *Figure 7.6*. The mean ratio of extradosed stays weight per unit bridge deck area is 16.7 kg/m^2, with a coefficient of variation of 55%. The amount of steel is larger in bridges with two planes of stays (average weight ratio of 19 kg/m^2) compared with bridges with one central plane of stays (average ratio of 13 kg/m^2).

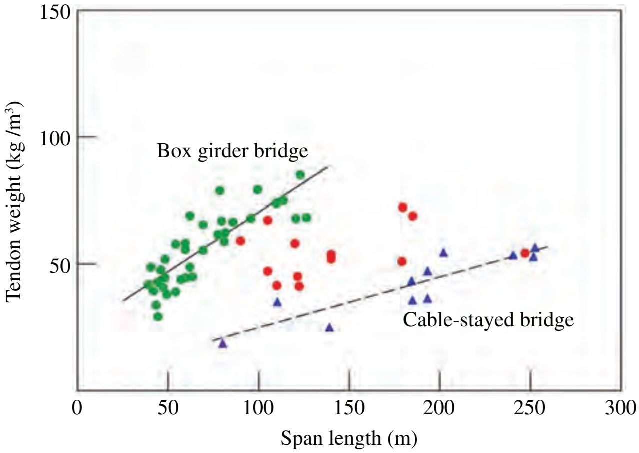


Figure 7.5: Span length versus tendon weight per concrete volume of girders (Courtesy of Sumitomo Mitsui Construction). (red = Extradosed Bridges, green = box girder bridges, blue = cable-stayed bridges)

The historical data include all type of Extradosed Bridges. It includes Extradosed Bridges with different loads, site conditions, as well as designs with different bridge codes and hence shows

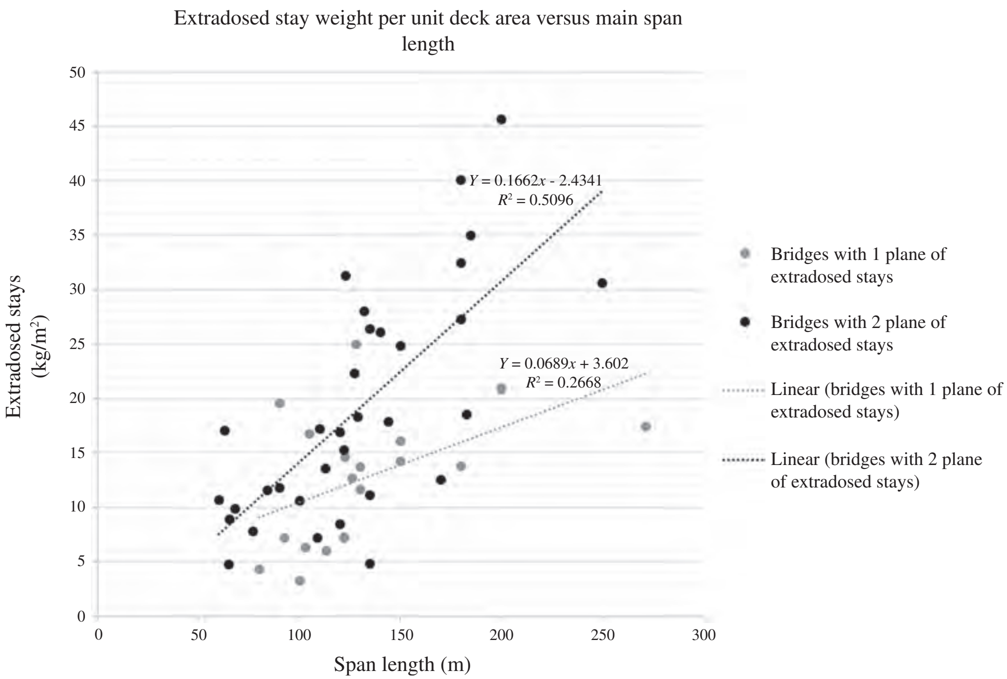


Figure 7.6: Span length versus extradosed stay steel weight per deck surface area.

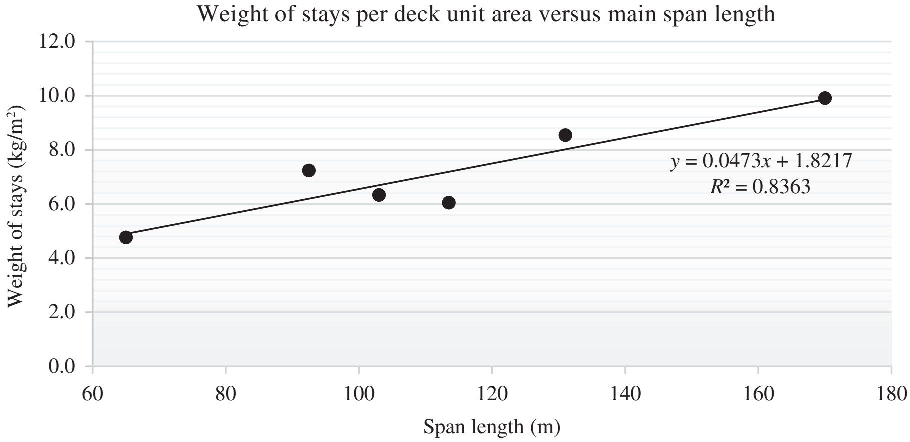


Figure 7.7: Span length versus extradosed stay steel weight per deck unit area.[70]

wide scatter. However, the statistical data produce a reasonable first evaluation of quantities and maybe applicable for preliminary design cost estimates.

A more reliable estimate of the material ratios requires the consideration of the actual design requirements and unique site conditions. As an example, the study developed in Ref. [70] for one- plane Extradosed Bridges designed with the AASTHO Code (*Figure 7.7*) provides stay steel ratios slightly smaller than those collected in *Fig. 7.6*.

7.3 Comparison to Girder Bridges Built in Balanced Cantilever

Segmental concrete bridges built in balanced cantilever with spans over 100 m are very competitive either cast-in-place or with precast segments, depending on the site constraints, project size and construction schedule. This construction technique is generally applied up to 200 m (world span record over 300 m). The use of stays working as eccentric external tendons (Extradosed Bridges) reduces the typical depths of continuous girders as stays provide more stiffness and strength to the system. The amount of materials (concrete and both reinforced and pre-stressing steel) on the deck and substructure can be reduced in Extradosed Bridges, as well as erection equipment and Form Traveler load capacities, but higher unit cost of the stays may not always make this technique appropriate or competitive.

Continuous girder bridges built in balanced cantilever consume more concrete, reinforcing steel and post-tensioning than Extradosed Bridges built with the same erection process but the cost of the stays and a longer cycle for the erection of a typical segment can offset the savings on the rest of the materials. Also, maintenance and inspection cost should be considered. *Figure 7.8* summarizes the material quantities of various balanced cantilever bridges cast-in-place designed with the AASTHO Code.[68] The values are compared with the limits defined in the Spanish Recommendations for road bridges.

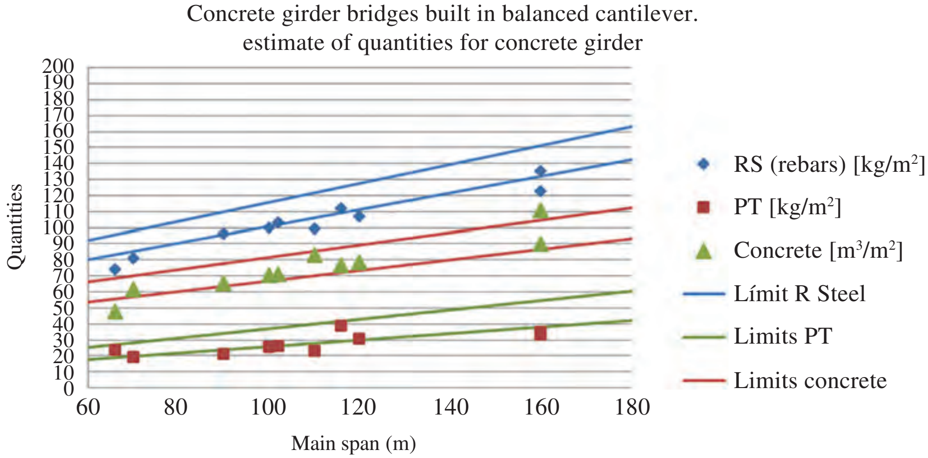


Figure 7.8: Quantities estimate for continuous three-span girder bridges built in balanced cantilever and comparison with estimates provided by Spanish Guidelines for bridge design of the Ministry of Public Works[68]

Material	Typical range – bridges designed with AASTHO code Span length between 80 and 150 m[71]	Typical range—standard bridges in France[72]
Concrete for deck (m^3/m^2 of deck surface area)	0.4 + 0.004 L	0.4 + 0.0035 L
Reinforcing bars (kg/m^3 of concrete)	100–125	• With transverse Prestressing: 110–130 • Without transverse Prestressing: 30–170
Post-tensioning steel (kg/m^2 of deck surface area)	30–45	• Longitudinal Prestressing: 40–50 • Transverse Prestressing: 5–7

Table 7.3 Typical range of material consumption for balanced cantilever concrete bridges

For balanced cantilever concrete bridges, the typical range of material consumption for the superstructure is summarized in *Table 7.3*.

7.4 Conclusions

This chapter summarizes historical data on material ratios and cost of Extradosed Bridges and estimate of quantities from different studies. The statistical analysis of the data can be useful for preliminary design purposes in the bridge selection process, although the engineer should consider the unique site conditions and design specifications for an accurate cost estimate.

The results show that the amount of materials required by Extradosed Bridges is in between concrete girder bridges and cable-stayed bridges. In terms of construction cost, while concrete or steel girder bridges may be appropriate for spans up to 120 m, and cable-stayed bridges are likely more appropriate for spans over 200 m, Extradosed Bridges have been proved competitive for spans ranging between 120 and 180 m. In the selection of the preferred bridge alternative, the engineer shall consider construction and the overall lifecycle cost, among other equally important aspects, such as context awareness, aesthetics, construction constraints, sustainability, and environmental impact.

Chapter

8

Bridge Data and Case Studies

Andreas Apitz, Germany

8.1 General

This chapter provides a general overview of the Extradosed Bridges (EDBs) built so far. With the help of the members and guests of the IABSE Working Commission 3 and by reviewing existing literature,[7,16,28,40,42,53,54,73–79] it was possible to collect data of about 241 EDBs worldwide. The bridge data table in Section 8.4 is perhaps the largest database of its kind until now even though it is certainly not complete. The geometric properties of EDBs are evaluated in Section 8.2. Section 8.3 shows the great diversity of Extradosed Bridges by presenting Case Studies of some selected bridges.

Around 59 % of the Extradosed Bridges in the list have two pylons in longitudinal direction (= three spans). Nearly 19 % have one and 22 % have three or more pylons (multi-span EDBs). Some facts reveal the large field of application of the bridge type:

- Longest main span: 312 m (Wuhu Yangtze River Bridge).
- Shortest main span: 43.5 m (Viaduct over the S8 Expressway in Olesnica).
- Longest multi-span EDB: 9759 m (Kacchi Dargah Bridge, completion expected 2022, see case studies).
- Highest pylon above bridge deck level: 57 m (New Yanggang Bridge).
- Lowest pylon above bridge deck level: 3.2 m (Deba River Bridge).
- Widest bridge deck: 61 m (Guijiugou Bridge).
- Narrowest bridge deck: 7.7 m (Hinase Bridge).

As can be seen from *Fig. 8.1*, the world´s leading countries in building Extradosed Bridges are China and Japan. The latter can be considered the motherland of EDBs, since the first "real" EDB (Odawara Blueway Bridge, see Section 8.3) was built there in 1994. After further development of EDBs and its technology in Japan, the first EDBs were built in Europe and in some Asian countries in the late 1990s. In the year 2000, the first EDB was built in China. Currently, most EDBs are being built in China and India.

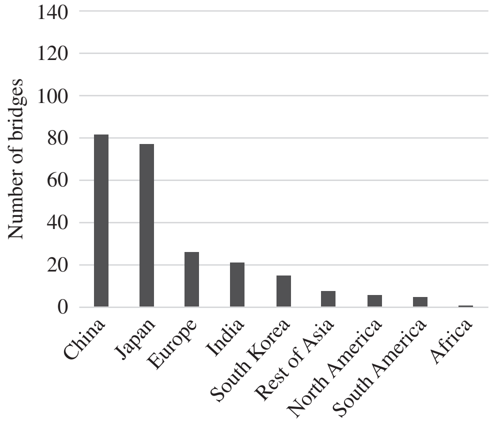


Figure 8.1: Extradosed Bridges built worldwide (as of August 2018)

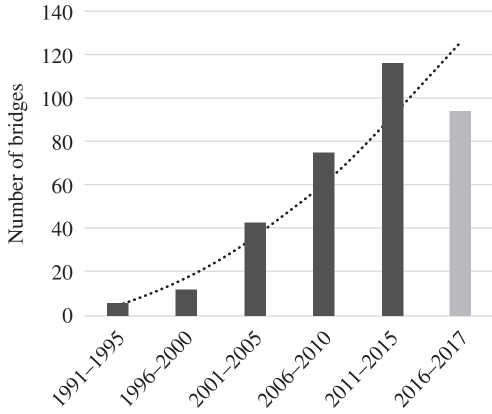


Figure 8.2: Extradosed Bridges built per time periods

Figure 8.2 shows the construction activity of EDBs as a function of time until the year 2017. As an average from 2011 to 2015, 23.2 bridges where built worldwide per year and the tendency indicates a growing activity in the coming years.

8.2 Geometric Properties

8.2.1 Introduction

In this section, the geometric relations and dependencies of existing Extradosed Bridges will be discussed. It can be seen as an elaboration and verification of values provided in Chapters 1 and 2. The following statements result from statistical investigations and by no means should be taken for granted. Each bridge is different and its design stems from given boundary conditions. However, the statements can serve as rough reference values.

As mentioned earlier, a large amount of data was collected in order to provide statistically significant information. To produce clear and meaningful diagrams, the data were partially adjusted. That is to say, statistical outliers have been taken out in some cases. The method of using data of existing bridges to investigate the typical geometric properties of EDBs has been utilized by other researchers,[1,42,80,81] but the amount of data here is greater. The investigated properties are shown in *Fig. 8.3*. In many cases, road and rail bridges are separately displayed.

8.2.2 Length of Main Span

Figs. 8.4, 8.5 and 8.6 show the frequency distributions of main spans for bridges with one (= two spans), two (= three spans), or three or more (row of) pylons(s) in the longitudinal direction. EDB with one (row of) pylon(s) naturally do not have a main span but two side spans and hence the spans found are smaller in comparison. Typical span lengths here are 50 m to 100 m. The typical

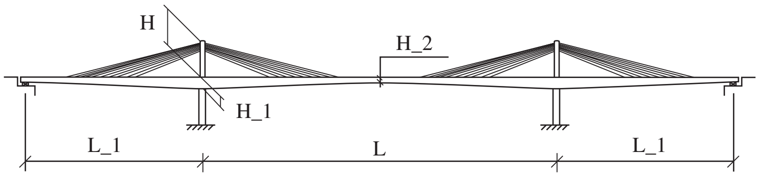


Figure 8.3: Typical Extradosed Bridge and its geometric properties

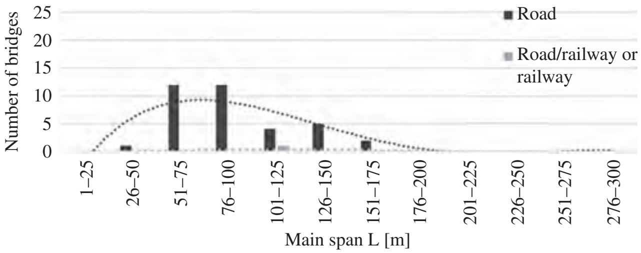


Figure 8.4: Main spans of EDBs with one pylon

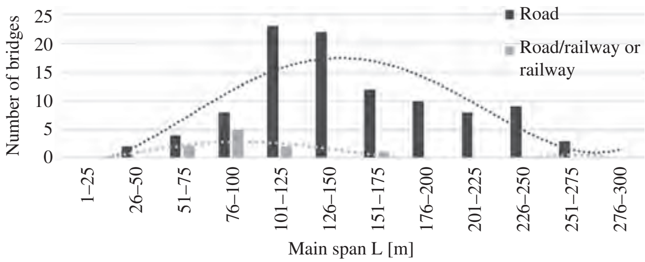


Figure 8.5: Main spans of EDBs with two pylons

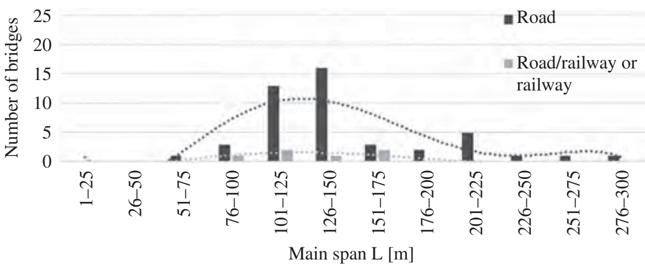


Figure 8.6: Main spans of EDBs with three or more pylons

range for EDB with two and three or more (row of) pylons are 100 m to 200 m, which confirms the specifications given in *Fig. 1.1* (L = 100–200 m). However, there is a high number of bridges with main spans of about 100 m to 150 m. There seems to be no difference between the classic EDB with three spans and those with multiple spans EDBs in this regard. EDBs with railway traffic in general seem to have smaller main spans; the typical range may be assumed to be between 75 m to 150 m. Spans lengths below 50 m were not found; the maximum span (312 m) is not taken into account in the diagrams.

8.2.3 Main Span to Pylon Height Ratio

8.2.3.1 EDBs with One Pylon

It can be observed from *Fig. 8.7* that the span to pylon height ratio (L/H) for EDBs with two spans is between 2 and 6. The ratios seem to be influenced by the span length as can be seen from *Fig. 8.8*. On average, the pylon is higher compared to the span for longer bridges.

8.2.3.2 EDBs with Two Pylons

According to *Fig. 2.1* for a typical three span EDB, the ratio L/H can be assumed to be 10. In contrast, *Fig. 8.9* reveals that pylons are often higher with ratios of 5 to 10. This applies for road as well as railway traffic. This is almost independent of the span lengths (*Fig. 8.10*).

8.2.3.3 EDBs with Three or More Pylons

Figure 8.11 shows typical ratios from 4 to 10, independent of the type of traffic. A slight increase with longer span length can also be detected (*Fig. 8.12*).

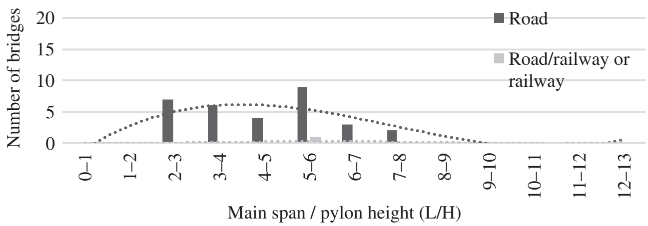


Figure 8.7: Main span to pylon height ratio for EDBs with one pylon

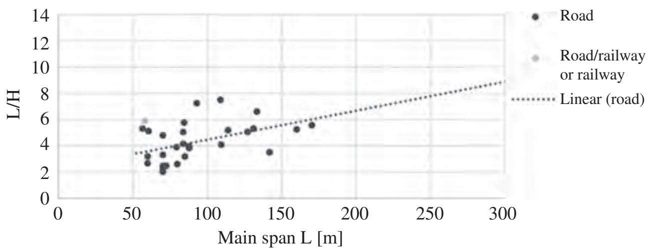


Figure 8.8: Main span to pylon height ratio for EDBs with one pylon as a function of main span

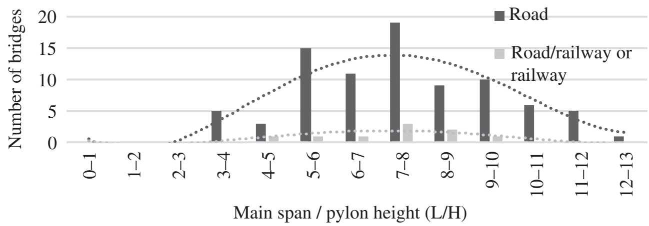


Figure 8.9: Main span to pylon height ratio for EDBs with two pylons

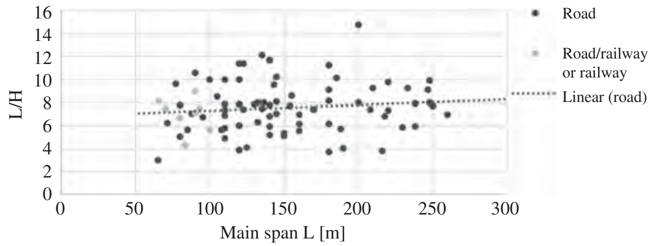


Figure 8.10: Main span to pylon height ratio for EDBs with two pylons as a function of main span

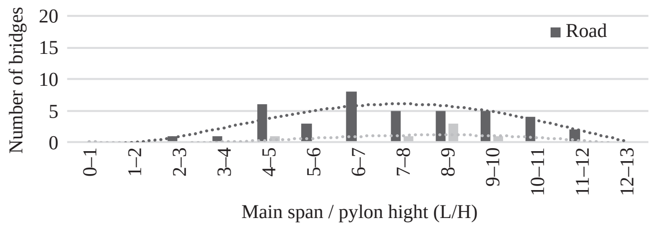


Figure 8.11: Main span to pylon height ratio for EDBs with three pylons

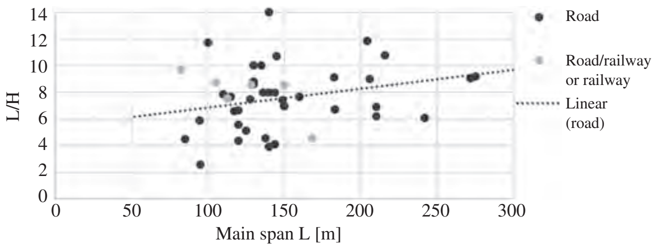


Figure 8.12: Main span to pylon height ratio for EDBs with three pylons as a function of main span

8.2.4 Side Span to Main Span Ratio

8.2.4.1 EDBs with One Pylon

Extradosed Bridges with one pylon only have two spans, which usually have same length (L_1 = L) resulting in a ratio of L_1/L = 1. Due to the specific boundary conditions of each bridge, there are exceptions (*Fig. 8.13*).

8.2.4.2 EDBs with Two, Three or More Pylons

The typical values given in *Fig. 1.1* (L_1/L = 0.5–0.7) can be confirmed for both, EDBs with three spans and those with multiple spans, when observing *Figs. 8.14 and 8.15*. For bridges with three

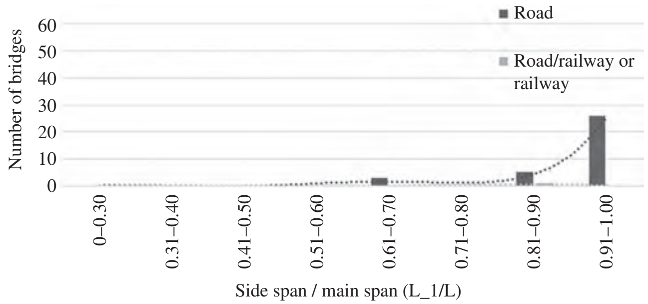


Figure 8.13: Side span to main span ratio for EDBs with one pylon

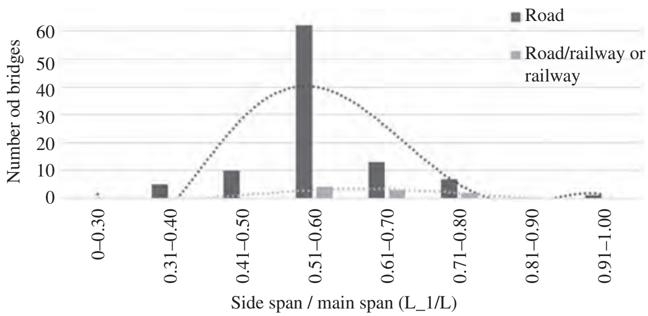


Figure 8.14: Side span to main span ratio for EDBs with two pylons

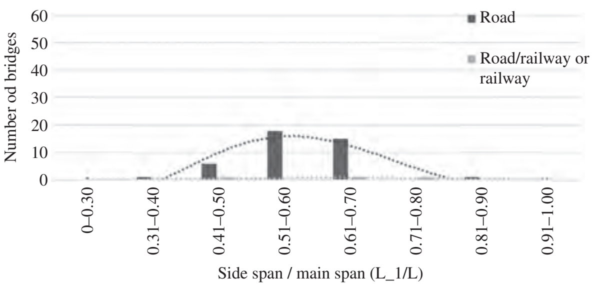


Figure 8.15: Side span to main span ratio for EDBs with three or more pylons

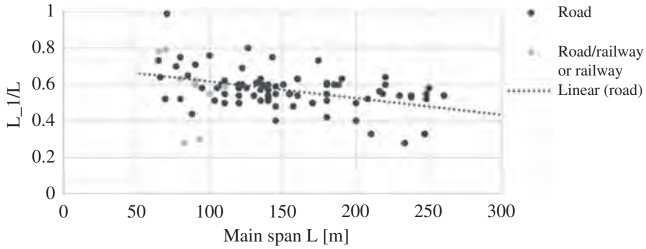


Figure 8.16: Side span to main span ratio for EDBs with two pylons as a function of the main span length

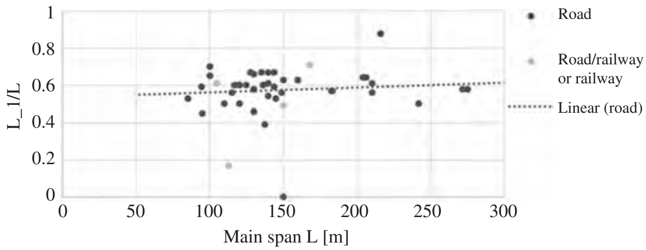


Figure 8.17: Side span to main span ratio for EDBs with three or more pylons as a function of main span

spans (two pylons), the ratio decreases with increasing main span length (*Fig. 8.16*). For bridges with multiple spans, the opposite seems to be the case (*Fig.8.17*).

8.2.5 Girder Depth

The depth of the superstructure is highly dependent on the type of cross section and material used. However, to start with the design, some values might serve for getting an idea of typical proportions.

8.2.5.1 Girder Depth at Midspan

The slenderness of the girder at the centre of main span (L/H_2) shows a clear dependency to the main span. The value given in *Fig. 1.1* (L/H_2 ≈ 55) is typical only for longer spans. Using the formulas of the linear trend lines from *Fig. 8.18* for constant and variable girders depths, some mean values have been derived for different main spans (*Table 8.1*).

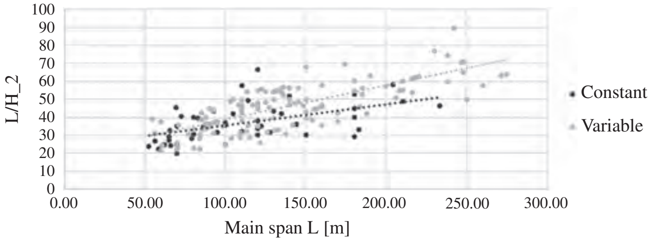


Figure 8.18: Slenderness at centre of main span (L/H_2) as a function of the main span length

	L/H_2	
Span [m]	Constant	Variable
50	30	30
100	36	39
150	42	49
200	48	58
250	54	67

Table 8.1 Mean values for slenderness at centre of main span (L/H_2) in case of different main spans

8.2.5.2 Depth at Pier, Haunched Girder

The slenderness at the pier (*Fig. 8.19*) increases when the main span increases. Thereby it is nearly unimportant if the bridge deck is embedded (fixed in rotation) at the piers or on simple supports. Typical mean values are given in *Table 8.2*. The degree of haunching increases for longer spans, as shown in *Fig. 8.20*.

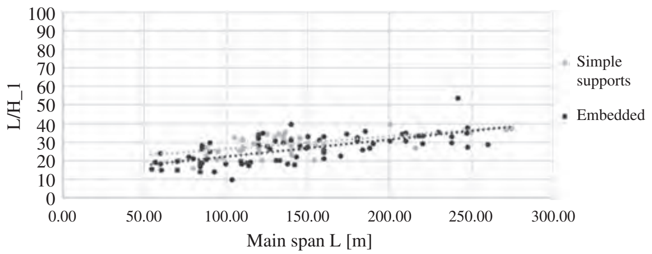


Figure 8.19: Slenderness at pier (L/H_1) as a function of main span

	L/H_1	
Span [m]	Simple supports	Embedded
50	23	18
100	26	22
150	30	27
200	34	32
250	37	36

Table 8.2 Mean values for slenderness at the pier (L/H_1) in case of different main spans

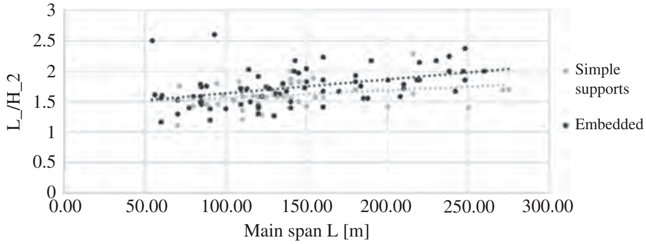


Figure 8.20: Degree of haunching (H_1/H_2 ratio) as a function of main span

8.2.6 Cable Interval

The cable spacing at the bridge deck is mostly around 4 to 8 m and is independent from the main span, as shown in *Figs. 8.21 and 8.22*.

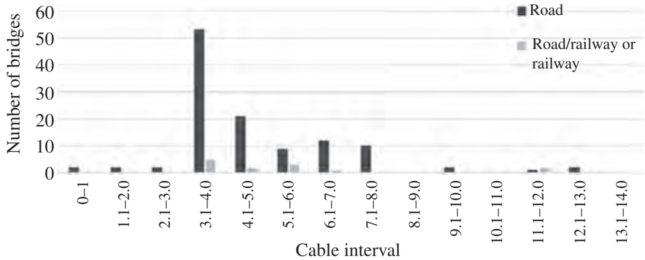


Figure 8.21: Cable spacing of all Extradosed Bridges

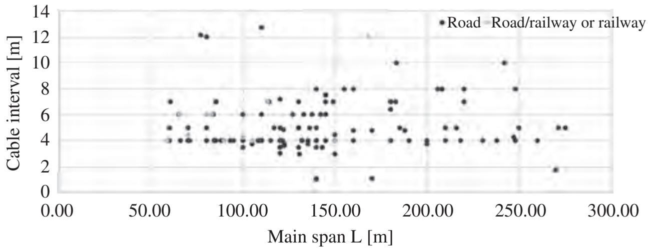


Figure 8.22: Cable spacing of all Extradosed Bridges as a function of main span

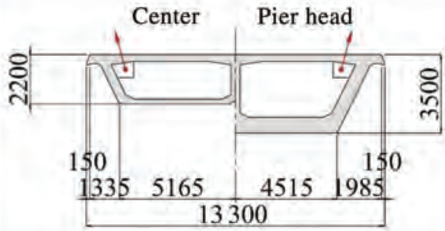
8.3 Case Studies

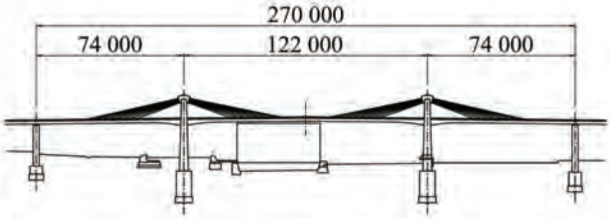
8.3.1 Introduction

In the following, selected Extradosed Bridges around the world are presented. As mentioned before, Extradosed Bridges are highly diverse in terms of dimensions, forms, and materials. Also there is no clear distinction between Extradosed and cable-stayed bridges. All this is reflected in the examples. Beginning with the mother of modern Extradosed Bridges, the Odawara Blueway Bridge, interesting bridges with one, two, or more pylons in longitudinal direction are presented.

8.3.2 Case Studies

First example – Odawara Blueway Bridge (1994)

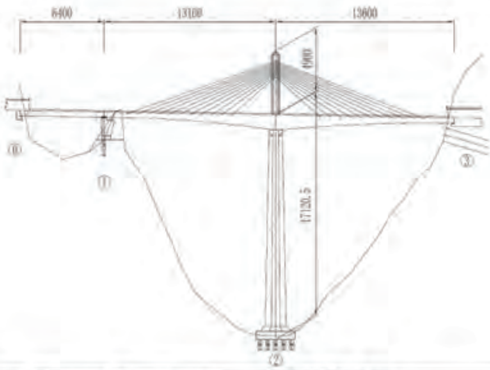




Location **Client** **Designer** **Cost**	Kanagawa, Japan Japan Highway Public Corporation Sumitomo Construction 12.400.000 USD ($)	**Cable type** **Max. stress in cables** **Stress change in cable due to live load**	Epoxy coated strand 1/1.67= 60% GUTS 38 MPa

The Odawara Blueway Bridge [13] is the first application of Extradosed Bridge in the world. Based on the fatigue design of stay cables, a safety factor of 1.67 can be used for the stay cable structure. Prior to the adoption of this safety factor, fatigue tests including flexural bending for stay cable system was done to confirm the safety. And to assure allowable amplitude of stay cables vibrations, high damping rubber damper was developed in this project. At the top of the pylon, a saddle, which has anchorages on both sides to avoid slipping stay cables during earthquakes, is used. This new anchorage system was also tested to assure the strength. Stay cables are made of epoxy coated strands which is the first application in Japan. A polymer cement grout was used to resist the tension of stay cables. Moreover, stay cables are covered by FRP pipes.

One pylon - Xianshenhe Valley Bridge (2009)



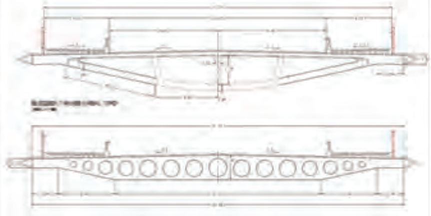
Location	Jincheng City, Shanxi Province, China	**Cable type**	Strand cables
Client	Jincheng Expressway Co., Ltd, Shanxi	**Max. stress in cables**	40% GUTS
Designer	China CCCC Highway Consultants Co., Ltd.	**Stress change in cable due to live load**	65 MPa
Cost	22.000.000 USD ($)		

The bridge has been built ingeniously in between the cliffs and its abutments had to be precisely located within the entrance/exit portals of the flanking tunnels cut through the sheer precipices and deep pathless mountains along the southern foot of Mt. Taihang. The bridge required a main pier founded in the valley bottom, hence condition of pier construction site is good, quality and feasibility of construction are guaranteed, and the environmental impact is not severe.

The box depth is 11 metres on top of the pier and 4 metres at the cantilever tip of uniform section. Each box segment is 4 metres long and was built by cantilever cast-in-situ construction. A 4.4 metres long bracket cast-in- situ segment is set at bridge's start end, and a 7.4m bracket cast-in-situ segment located at the finish end. In the middle, there is a 2m long closure segment.

The main pier is octagon shaped with wall thickness of 1.5m. In order to strengthen the pier to resist possible floating load during flood season, internal stay for pier wall and stiffening ring beams are used in the hollow pier.

One pylon - Príncipe de Viana Bridge over the Segre River (2005)



(View: Total length = 197 m. Main span = 86 m. Typology: Extradosed Bridge, prestressed concrete deck)

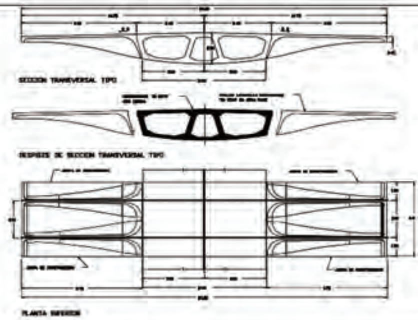
Location	Lérida, Spain	**Cable type**	Multi strand
Client	TIFSA-ADIF	**Max. stress in cables**	60% GUTS
Designer	CFC, S.L.	**Stress change in cable**	≤ 50 MPa
Cost	N/A	**due to live load**	$\leq 50\ MPa$

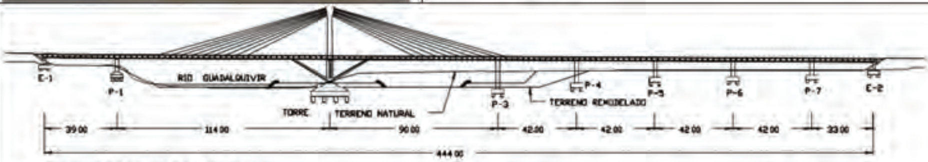
The aim of this bridge is to cross the Segre River and the Segre Avenue [82]. The bridge is placed over the alluvial soil of the Segre River, which involves deep foundations by means of piles. It has to be noticed that the request of both the Local and Railway Administration was to achieve a unique bridge, with prominent technical and aesthetic characteristics. The designed solution is an Extradosed Bridge, formed by four spans of 19.40m + 16.00m + 86.00m + 75.00m, and a total length of 197.00m. The width of the deck is 21.20m, corresponding to four urban traffic lanes, each 3.30m wide, and two sidewalks, which are 4.00m wide.

The deck in the main spans was designed to be formed by a central box girder, composed by two beams joined along their central vertical wall. Each one of the beams is precast, with a "U" shape with curved bottom flange. The precast beams are 2.13m high and 4.00m wide. During the construction, these precast beams were substituted by a cast in situ girder.

The height of the vertical piers is 38.60 m. Their cross section varies from a rectangular section (2.40m x 3.00m) to a section composed by two 0.40m x 1.40m rectangles. The aim of the second rectangle is to lighten visually the shape of the pylons in longitudinal direction.

One pylon - Andalucía Bridge over the Guadalquivir River (2005)





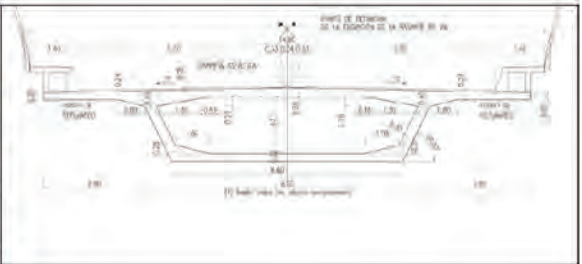
Location **Client** **Designer** **Cost**	Córdoba, Spain SENER/Junta de Andalucía CFC, S.L. N/A	**Cable type** **Max. stress in cables** **Stress change in cable due to live load**	Multi strand 60% GUTS ≤ 50 MPa

This is an Extradosed Bridge, 243m long, followed by an approach viaduct, 201m long, along the right bank of the river [83].

The two main cable-stayed spans have a length of 114 + 96 m, supported by a single pylon with a height of 28m from the deck and 40m from foundations. The shape of the main support is formed by the pylon leg and two diagonal struts, which play a double role. They reduce the main span length and increase the stiffness of the deck, so the effect of the extradosed cable stays is more effective despite having the pylon more than L/10 high. This increase in the height of the pylon involves a reduction in the area and force of the cable stays, due to their bigger inclination, while the oscillation of forces in the cable stays is also smaller.
The deck is constant depth along the whole bridge, in the continuous 42m long approach spans and also in the long main spans, by means of the cable stays. This design is different from the usual cable-stayed bridge designs, in which the depth and inertia of the deck section is bigger in the approach spans than in the main cable-stayed spans. An extradosed solution allows the design of the deck without changing its dimensions along the bridge.

The deck has a curved cross section, formed by a curved central box 10m wide, completed with transverse ribs, also curved, that allow to achieve a total width of 29 m. The depth of the deck is 2.30 m. The erection of the deck was developed with scaffolding in the approach spans, and with a movable scaffolding system and a temporary pier in the river crossing. First, the cross section inner core was built and after that, by means of a form traveler rolling along the central box, the transverse ribs were assembled. Finally, the cable stays were installed, lifting the deck over the temporary supports. The pier was extended upwards with a steel pinnacle, but only for ornamental purpose.

Two pylons - The Triplets (2010)



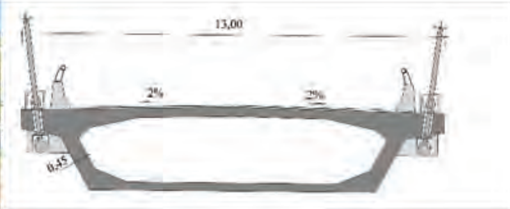
Location	La Paz, Bolivia	**Cable type**	Strand cables
Client	N/A	**Max. stress in cables**	55% GUTS
Designer	PEDELTA	**Stress change in cable due to frequent live load as per EC**	< 100 MPa
Cost	N/A		

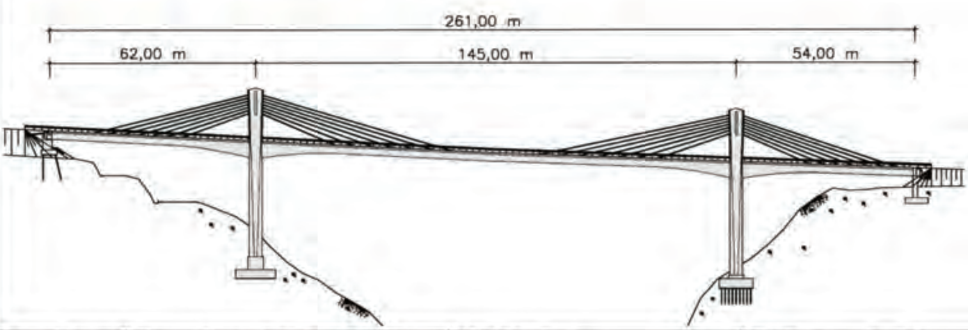
The elevated road crosses three parallel valleys with signature bridges [84, 85]. All three-span structures are made of concrete, with maximum span of 113.5m, featuring extradosed pre-stressed concrete with a single plane of stays to allow a more transparent view. The bridge deck is 14.8m wide and carries 4 lanes for road traffic and is connected monolithically with the two piers, forming a portal frame, and rests on neoprene bearings at the abutments. The cast-in-place deck, built in balanced cantilever, has internal cantilever post-tensioned tendons placed in the top slab, and continuity tendons in the bottom flange of the central section of the mid-span.

The size of the pylon increases with height to place the deviation saddles with minimum radius of 3 m. The relationship between the length of the central span and the height of the pylon is approximately L/7.5. A lower ratio, of about L/11, would have been ideal for the optimum use of the material but not appropriate from aesthetic considerations in the urban environment.

The stays are anchored to the deck at both ends and deviated at a pylon saddle. Wires are galvanized, protected by an individual layer of high density polyethylene (HDPE) and with grease between wires. Stays can be re-stressed, adjusted and are interchangeable. The deviation saddle in the pier is different for each stay and has a double steel tube, a minimum radius of 3m and a locking system. Stays are bare at the deviation saddle and protected by a cement grout. At the anchorage they are protected by grease.

Two pylons - Teror Viaduct (2011)





Location	Gran Canaria Island, Spain	**Cable type**	Galvanized cable strands
Client	Canary Islands Governement	**Max. stress in cables**	50% GUTS
Designer	EIPSA	**Stress change in cable due to live load**	100 N/mm^2
Cost	6.000.000 EUR (€)		

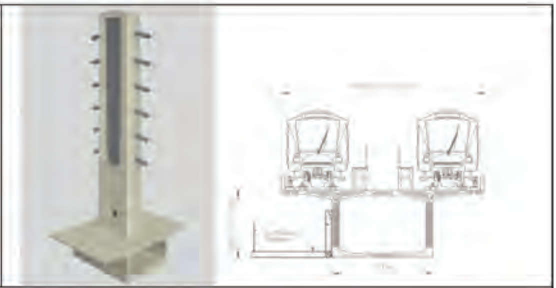
The Teror Viaduct [86] is a bridge located over a difficult-to-reach gorge and was designed keeping in mind the aesthetic requirements so as to achieve the structure's full integration with the existing landscape.

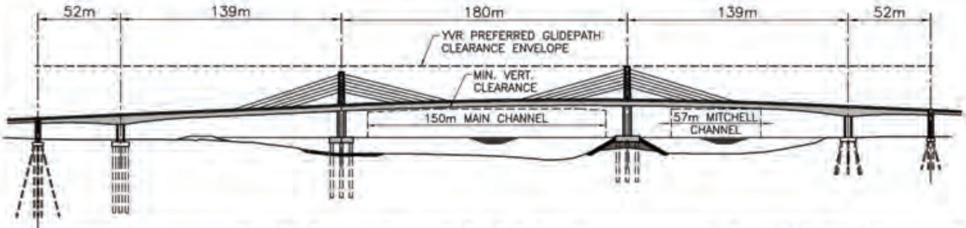
The deck is rigidly fixed to piers. The "V" shaped pylons laterally 'embrace' the deck, forming a monolithic whole, as they extend above. Deck depth ranges from 2.57 m to 5.07 m and the free standing height of the pylons is 15 m.

The cables supporting the deck penetrate through the towers without considering friction under construction and are locked in service to withstand the effect produced by differential forces in the stays on both sides of the saddles due the traffic load. The stays have several layers of protection. They were made out of individual wax-wrapped galvanized steel strands embedded in individual high-density polyethylene casings which were grouped into high-density polyethylene sheaths, between pylon and anchor. Moreover, the deviation pipes embedded in the pylons were injected with wax.

The deck was built using the balanced cantilever system starting from the two piers. Following the deck's construction and prior to the stitching of the two cantilevers, the deck was left with a gap in the centre of the main span in order to partially reduce the long term effects due to concrete time dependent strains (shrinkage and creep).

Two pylons - North Arm Bridge (2010)

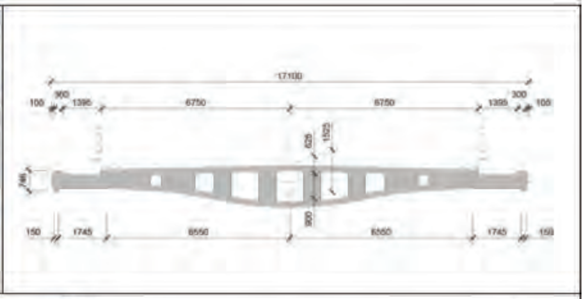


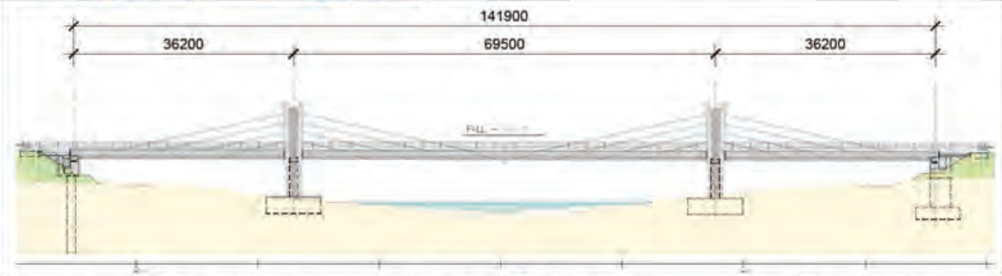


Location	Vancouver, BC, Canada	**Cable type**	Parallel strand system
Client	Translink BC	**Max. stress in cables**	54% GUTS
Designer	Buckland & Taylor Ltd	**Stress change in cable due to live load**	75 MPa
Cost	Aprox. 50.000.000 CAD ($)		

The North Arm Bridge [17, 87] is an extradosed precast segmental box girder bridge with continuous deck of 562m (1844 ft.) for light rail transit. The 180m (590 ft.) main span provides comfortable clearance to the 150m (492 ft.) wide main navigation channel. The 139m (456 ft.) wide side span easily clears the separate 57m (187 ft.) wide north navigation channel. The locations of the sub-structures for these spans avoid direct encroachment upon the environmentally sensitive shore-line areas of the Fraser River. The Crossing includes two centrally located main pylons (45 m/143 ft. high) and two approach piers on each side of the North Arm. The composite steel/concrete pylons have steel webs with concrete flanges that provide anchorage for the extradosed cables. The bridge also carries a pedestrian/bikeway located at the level of the bottom flange of the box girder under the downstream deck cantilever. The bearings at the two end piers were custom designed sliding hold down bearings to control relative movements between the main span structure and the approach structure in order to control relative movements between the main span structure and the approach structure to satisfy tight tolerances on rail joint movements at these locations.

Two pylons - River Erne bridge (2013)





Location	Cavan County Council, Ireland	**Cable type**	Strand cables
Client	NRA	**Max. stress in cables**	45% GUTS
Designer	Ferrovial-Agroman SA	**Stress change in cable due to live load**	90 MPa
Cost	4.200.000 USD ($)		

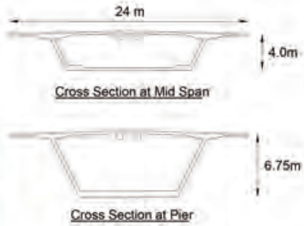
The Extradosed Bridge over the River Erne [88, 89] was required as part of the proposed N3 road improvement project in Cavan, Ireland. The structure is a three span concrete continuous bridge (36.2m + 69.5m + 36.2 m) with low pylons and cables anchored in two cantilevers located at both ends of the cross section.

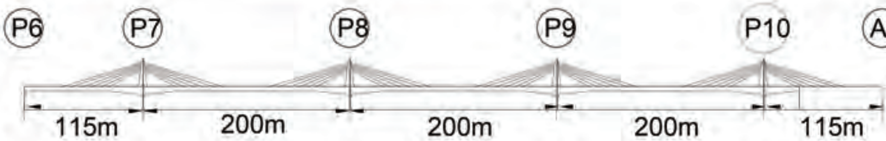
Extradosed and bonded prestressing techniques are used to design a relatively slender deck with a curved soffit and quadratic voids. The maximum depth of the deck is 1.525m at the centreline, with a total width of 17.1 m. The central 12.3m strip is intended for traffic use while raised verges, parapets and extradosed cable anchorages have been provisioned for the 2.4m remaining width along both edges.

A saddle, by means of friction, allows every extradosed cable to pass through the pylons without the need of an anchorage system. The slippage on the saddles decisively determined the number of stages needed to carry out the tuning of stay cable forces, as well as the dimensioning of the shortest cable, since this is most sensitive to the unbalanced loads at either side of the saddle due to indirect actions such as temperature or creep.

The bridge was cast in-situ over falsework and scaffolding. Both halves of the bridge were constructed from their corresponding side of the river with the help of a temporary embankment and two temporary piers in the center of the main span

Multiple pylons - Third Karnaphuli Bridge (2010)



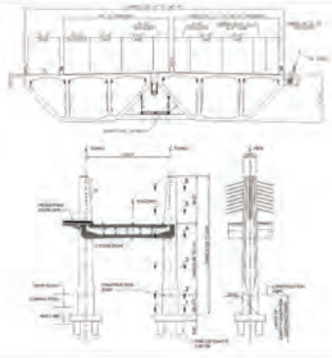


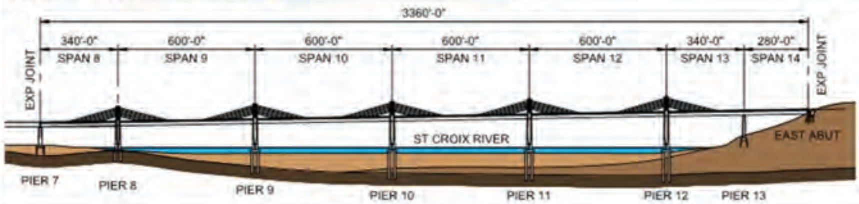
Location	Chittagong, Bangladesh	**Cable type**	Individual sheathed strands
Client	RHD, Bangladesh		
Designer	High-Point Rendel, UK	**Max. stress in cables**	0.45% GUTS
Cost	48 million USD ($) (2005), including 120m Viaduct	**Stress change in cable due to live load**	25 MPa

The Third Karnaphuli Bridge [90, 91] located near the port city of Chittagong in Bangladesh was opened to traffic in 2010. The 830m main single plane Extradosed bridge has a span configuration of 115+3x200+115m. The single box four lane deck is 24m wide with depth varying from 6.75m at pier to 4.0m at midspan. The pylons are 25.75m tall. The bridge has a gentle 3200m horizontal curvature. The cables consists of individual replaceable strands, which are individually galvanized, greased and sheathed.

The deck girder was constructed by cast-in-situ free cantilevering method. The bridge is founded on soft alluvial strata subject to high and non-uniform scour. Bored concrete piles of 3m diameter and more than 70m length were adopted in the foundation. The superstructure box girder along with the pylon is connected with the piers through an elaborate arrangement of POT bearings. As the bridge is located in high seismic zone, shock transmission units as well as backup shear keys were provided to transmit large horizontal forces to the foundation.

Multiple pylons - St Croix River Crossing Stillwater (2017)



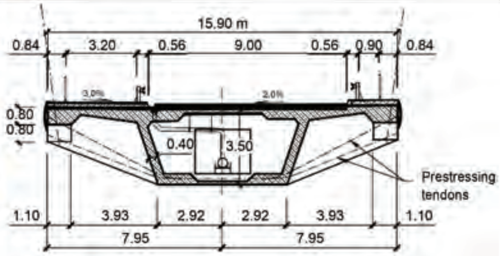


Location	Stillwater, MN, USA	**Cable type**	Parallel strand system
Client	Minnesota DoT	**Max. stress in cables**	0.50% GUTS
Designer	COWI | Buckland & Taylor Ltd	**Stress change in cable due to live load**	75 MPa
Cost	Aprox 450.000.000 USD ($)		

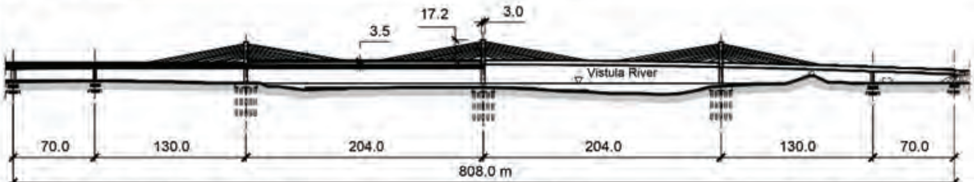
This project over the St Croix River [27, 92] between Minnesota and Wisconsin replaces the 80-year-old Stillwater Lift Bridge with a new 1024m (3,360 ft) four-lane bridge. The centre piece of the crossing is the extradosed precast segmental concrete structure with four main spans of 182m (600 ft), two back spans of 104m (340 ft) and a jump span of 85m (280 ft). The form of the bridge incorporates rounded organic shapes intended to be aesthetically consistent with the natural setting of the bridge. The Extradosed Bridge is continuous end to end. The five main pylons utilize thin twin wall legs beneath the deck to provide flexibility necessary to absorb longitudinal time dependent and thermal displacements of the deck. The deck consist of twin 5.5m (18 ft) deep three cell precast post-tensioned segmental box girders connected by a cast in place concrete stitch along the centre of the roadway deck. Discrete transverse post-tensioned struts connect the two boxes at the bottom flange level at each of the extradosed cable stay anchorage locations. The two planes of extradosed cables are anchored in concrete corbels on the outer edges of the deck and on steel anchor boxes cast into and composite with the tops of the main pylons.

Multiple pylons - Road bridge in Kwidzyn (2013)

photo: Budimex SA



cross-section in span



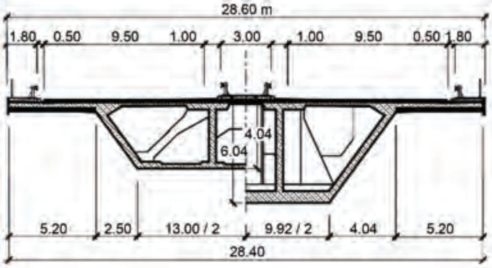
Location	Kwidzyn, Poland	**Cable type**	Strand cables with HDPE coating filled with grease or wax, external HDPE pipe
Client	General Directorate for National Roads and Motorways (GDDKiA)		
Designer	Transprojekt Gdański	**Max. stress in cables**	1/1.82= 55 % GUTS
Cost	24.000.000 EUR (€), (main bridge)	**Stress change in cable due to live load**	90 MPa

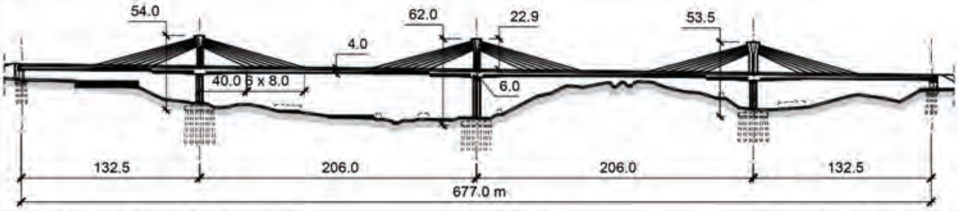
Construction of the bridge over the Vistula River in Kwidzyn, located along the new section of the national road DK90, was one of the most important investments in the transportation network of Pomerania region (northern Poland). This bridge consists of five structures with a total length of 1867m. The main 808m long bridge crossing the riverbed is a six-span extradosed continuous deck girder with spans of 70.0 + 130.0 + 2 × 204.0 + 130.0 + 70.0 m.

The superstructure is a single-cell concrete box girder with inclined webs and a constant depth of 3.50 m. External stays are anchored in the edge beams stiffened by transverse diaphragms. The webs with a typical width of 0.40m are locally widened in the support zones to 0.60m and the thickness of the bottom plate is increased to 0.80m. The small structural depth of the box girder is constant along the whole length of the bridge, which is untypical for such long spans. Therefore it was required to use high strength concrete (C70/80). Short concrete pylons acting as deviators of external stays are fixed in the superstructure and are also made of high strength concrete. Each pylon has a shape of two inclined arms with a height of 17.2m above the deck level, which is only about 1/12th of the span. The post- tensioning system consists of internal cables and external cables and stays. External stays consisting of 75 strands are located outside the cross-section of the span. The strands are deflected in steel saddles (VSL SSI 2000 type) placed in the pylons and are anchored in concrete blocks in the edge beams of the deck. The spans of the bridge in Kwidzyn were constructed section-by-section using underslung movable scaffolding. For this purpose it was necessary to build 13 temporary concrete supports, six of which were located in the river current.

Multiple pylons - Expressway bridge in Ostróda (2017)

photo: Budimex SA

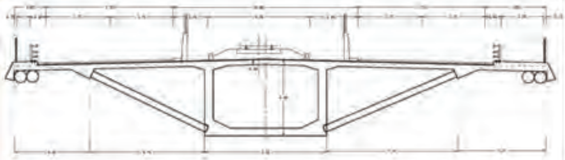




Location	Ostróda, Poland	**Cable type**	Strand cables with HDPE coating filled with grease or wax, external HDPE pipe
Client	General Directorate for National Roads and Motorways (GDDKiA)		
Designer	Transprojekt Gdański	**Max. stress in cables**	1/1.82=55% GUTS
Cost	26.000.000 EUR (€)	**Stress change in cable due to live load**	70 - 120 MPa

The bridge near Ostróda (Poland) carries the southern ring road of the city which is a new section of the national road DK16. It is a four-span structure with a total length of 677m and spans of 132.5 + 2 × 206.0 + 132.5 m. Construction of such long spans resulted from the nature protection requirements. The superstructure is a prestressed concrete (class C60/75) three-cell box girder with a width of 28.6 m. Depth of the girder varies from 4.04m in the span to 6.04m in the support zone. The girder is stiffened every 8.0m by transverse diagonal prestressed concrete struts. External stays are arranged in a single vertical plane along the centre line of the girder. The number of 7-wire high strength steel strands in a single stay varies from 135 up to 167. The stays are deviated in VSL SSI 2000 type steel saddles installed inside the low height pylons. Height of the pylons is only 22.9m above the deck level. Due to the obstacle configuration (a deep valley is crossed), the bridge was cantilevered with simultaneous use of six sets of travellers. In the first stage 15.2m long starting segments were cast on scaffolding. Four initial cantilevered segments with variable depth were 3.6m long and the remaining segments were 4.0m long. The shortest external stays were anchored in the girder at a distance of 42m from the pylon axis. The cantilevers extended 102m before casting the closure segments. The Extradosed Bridge was founded on precast reinforced concrete piles.

Multiple pylons – Vidin-Calavat Bridge (2012)



Total length: 1790 m. Main span: 180 m.

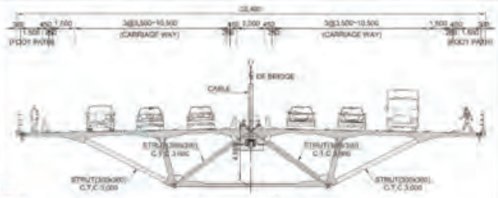
Location	Vidin (Bulgaria) - Calafat (Romania)	**Cable type**	Multi strand
		Max. stress in cables	56-60% GUTS
Client	FCC/Government of Bulgaria	**Stress change in cable due to live load**	≤ 50-75 MPa
			$\sigma_{max} = 0.56 - 0.6 f_{pu}$
Designer	CFC, S.L.		
Cost	140.000.000 EUR (€)		

The crossing of the River Danube [78, 93] presents at this point two clearly differentiated areas. The first one lies between Vidin and an island situated in the centre of the river (non- navigable part) and the other lies between the island and Calafat (navigable part). A 150m horizontal clearance was required in the navigable part, thus determining that the configuration of the bridge was 124m + 3x180m + 115m. On the non- navigable area 80m spans were adopted, and repeated for 612m.

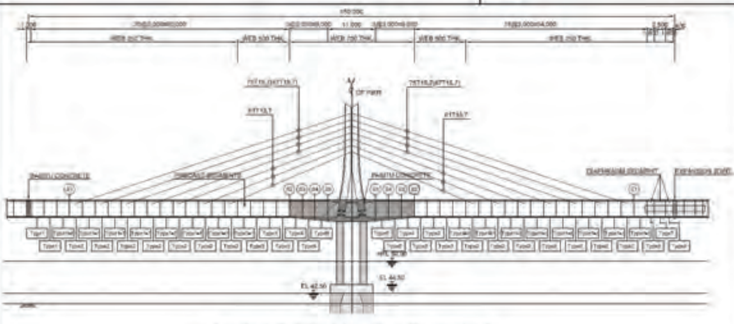
The transverse cross section of the entire viaduct, excepting the mentioned final railway spans, is constant. It consists of a central box-girder of 4.5m depth plus two cantilevers on either side, supported by inclined struts resting on the central box girder. The overall width thus totals 31.35m. The railway is located at the middle of the bridge deck.

The construction on the non-navigable part of the river was carried out using precast segments to build the central box girder. The strutted lateral cantilevers were built subsequently by means of a form traveler travelling along the completed box girder. The 180m span over the navigable part of the river keeps the same cross section assisted by an extradosed cable staying system supported from small-height pylons erected over the piers.

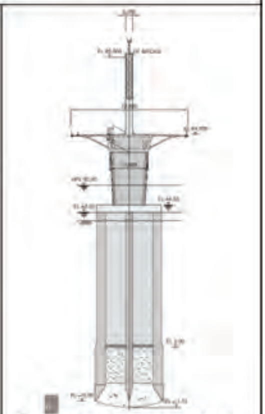
Multiple pylons – Kacchi Dargah Bridge (expected completion 2022)



Cross section of girder



Elevation of upper pylon, stay cable and girder

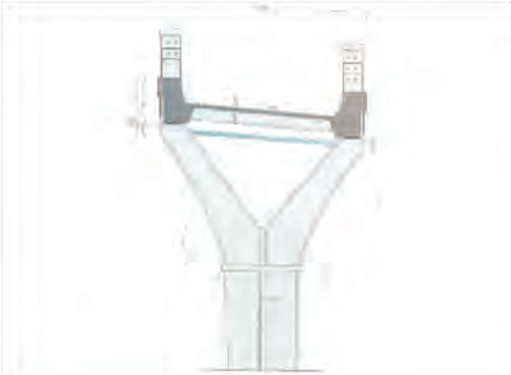


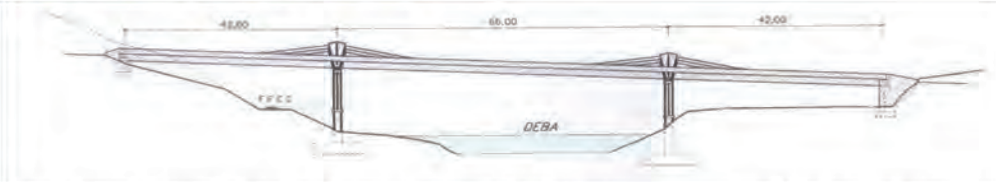
Cross section of pier, pylon and girder

Location	Patna (State of Bihar, India)	**Cable type**	Strands, PE sheathed, waxed and galvanized
Client	Bihar State Road Development Coporation Ltd. Daewoo-L&T JV (50:50)	**Max. stress in cables**	55-60% GUTS
Designer	PEC (Extradosed bridge)	**Stress change in cable due to live load**	58 MPa
Costs	425.000.000 EUR (€)		

The Kacchi Dargah Bridge is currently under construction. It will span the river Ganga connecting Kacchi Dargah in Patna and Bidupur in Hajipur in the Indian state of Bihar. Upon completion in 2022, the bridge will provide an easy roadway link between the northern and southern parts of Bihar and will connect two major national highways, linking NH 30 to NH 103. The total length of the bridge will be 9,759m with spans of 150m between the pylons making it by far the longest Extradosed Bridge in the world. The bridge will have 65 pylons with a height of 22,5m above deck level. The cables are connected to the pylons by the proven DYNA®LINK Anchor Box System. The pier heights will be 18,4 m. The main girders will be precast box girder from 5m to 4m depth, 32.5m width and inner and outer struts of concrete.

Curved deck - Deba Bridge (2004)





Location	River Deba. Guipúzcoa, Spain	**Cable type**	Unbonded strands
Client	Guipúzcoa Provincial Council	**Max. stress in cables**	75% GUTS
Designer	EIPSA	**Stress change in cable due to live load**	Negligible
Cost	N/A		

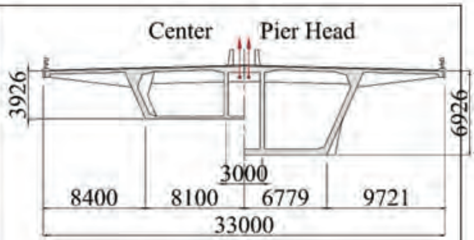
The bridge over the River Deba [94] forms part of the Malzaga connection on the Vitoria – Eibar Motorway. Aligned in a curve, the bridge has a succession of spans measuring 42m + 66m + 42m. It crosses a railway line, the river Deba and a road.

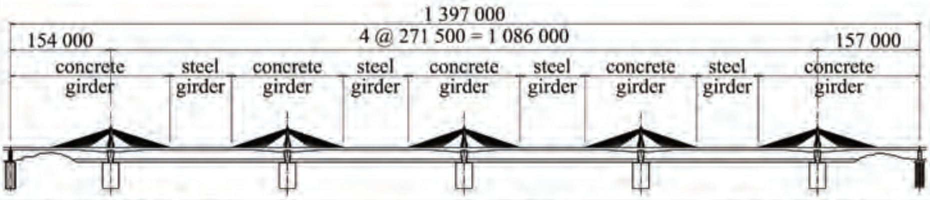
The strict clearance conditions imposed by the route underneath required developing a special solution for this bridge. Hence, the deck was designed with a "U" shape concrete cross- section with edges which prolong over the road platform.

A special feature of this bridge is that the three pairs of extradosed cables (3*2*24ø0,6") are continuous all along the deck. Extradosed prestressing springs from the anchorage located at the upper part of the deck at one abutment to the other one, running along the deck and going through the towers, which are integrally connected to the deck. The post-tensioning is complemented by one pair of conventional tendons (1*2*31ø0,6"), located inside the beams along its whole length. Extradosed prestressing is unbonded along its whole length. Individual wax-wrapped galvanized steel strands were used. These strands run within a polyethylene duct along the deck and within a steel duct in the rest of their alignment, including the towers. Moreover, these ducts are injected with wax. This would allow a future replacement of the extradosed system. As life load is hardly affecting the steel stress, cable fatigue is not a concern in this bridge.

The deck is supported by "Y" shaped piers. The solution, whose special features are compelled by the clearance requirement, provides the attractive shape deriving from its uniqueness.

Composite deck - Ibigawa Bridge (2001)

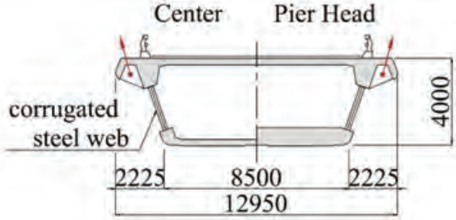


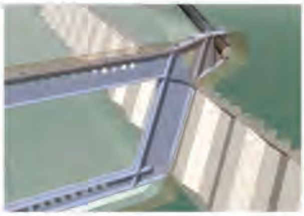


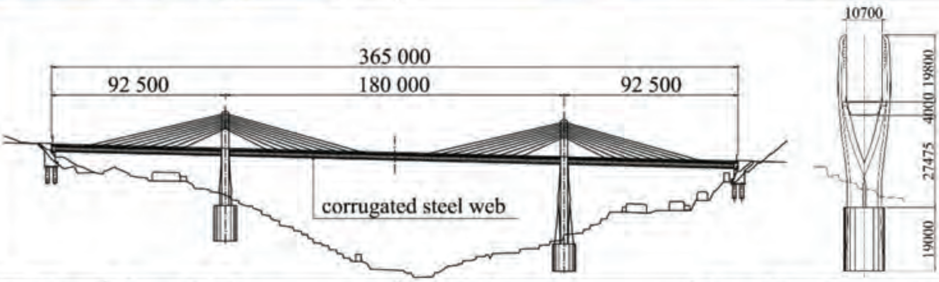
Location	Mie, Japan	**Cable type**	Wire (DINA)
Client	Japan Highway Public Corporation	**Max. stress in cables**	1/1.67 = 60 % GUTS
Designer	Sumitomo Construction, Mitsubishi Heavy Industries, DPS JV & PS, Taisei Corp, Yokogawa JV	**Stress change in cable due to live load**	112 MPa
Cost	286.520.000 USD ($)		

The Ibigawa Bridge [95] is the first application of Extradosed Bridge using precast segmental construction in Japan. The girder is the hybrid type which has 100m steel girder in the centre portion. This 2000 tons steel girder is lifted by lifting jacks. The maximum weight of three- cell concrete segment is 400 tons, which has 7m depth, 5m length and 33m width. The stay cable is prefabricated type with galvanized wires. High damping rubber dampers are installed to avoid stay cable vibration. The anchorage of stay cable in the pylon is composite type. Steel anchorage box, which weighs 80 tons, is encased in concrete. The 100,000m^2 prefabrication yard of concrete segments was located 10km away from bridge construction site. Segments where transported by barge. The Kisogawa Bridge which has 275m spans and the Ibigawa Bridge are twin bridges. These bridges span over the parallel rivers on Nagashima Island.

Corrugated steel web - Himi Bridge (2004)



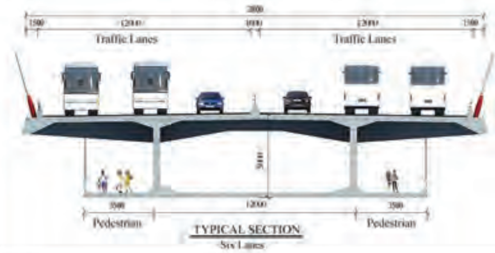


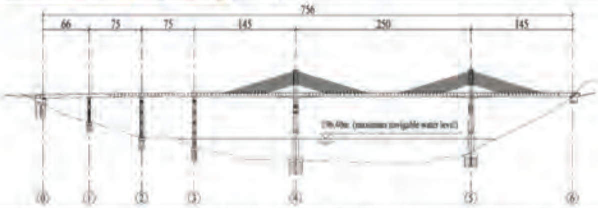


Location **Client** **Designer** **Cost**	Nagasaki, Japan Japan Highway Public Corporation Sumitomo Construction 34.780.000 USD ($)	**Cable type** **Max. stress in cables** **Stress change in cable due to live load**	Strand (prefabricated) 1/1.67 = 60 % GUTS 50 MPa

The Himi Bridge [33] is the first application of Extradosed Bridge with corrugated steel web in the world. Because the corrugated steel web cannot carry the stay cable forces, the steel diaphragm anchorage system is developed and tested by a half size beam model. The 180m span is relatively flexible. Therefore, a wind tunnel test of girder was done to confirm stability against wind. A composite anchorage system of stay cables in the pylon is used. However, the steel box anchorage is divided in each stay cable because of the crane capacity. After assembly of the steel boxes, anchorage is encased in concrete to behave as a composite structure. This anchorage area can be inspected using access from the inside of box girder.

Mixed use deck – Jia-Yue Bridge (2010)



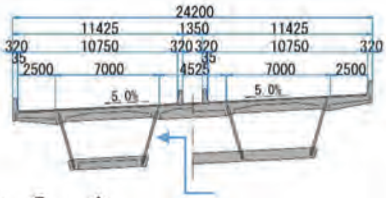


Location	Liangjiang New Area, China	**Cable type**	Strand cables
Client	Chongqing land group	**Max. stress in cables**	1/2.222 = 45% GUTS
Designer	T.Y. Lin International	**Stress change in cable due to live load**	71.28 MPa
Cost	35.466.000 USD ($)		

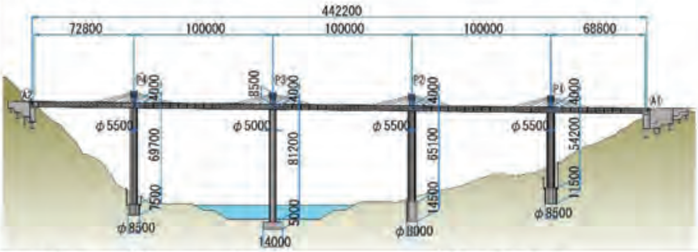
Jiayue Bridge spans the Jialing River valley in Chongqing at about 40km upstream of the junction of the Yangtze River and Jialing River. The basic requirements requested by the client were bridge aesthetics, cost-effective and collision-resistance. After a careful survey of the deep river valley terrain, an Extradosed Bridge with a span arrangement of 145m+250m+145m was proposed, and rigid frame-continuous PC spans where applied as its west approach.

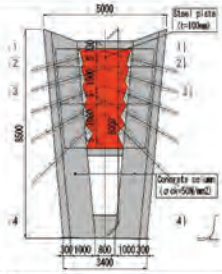
Two Y-shaped pylons have overall heights of 120m and 126.5m respectively, of which the inclined upper parts (over the deck) are 32m high. The box girder depth varies from 5m to 7m. By optimizing the traffic flow, a double deck girder is designed to separate the vehicle lanes and two pedestrian sidewalks on the upper and lower levels respectively. This arrangement can provide safer traffic routes, and reduces the deck width as well as the self- weight of superstructure. The 28m wide deck accommodates 6 traffic lanes, which is supported by the crossbeams at spacing interval of 5m. The bridge has 52 pairs of stay cables, and each one comprises of 85 to 91 seven-wire strands with epoxy-coated protection. The maximum cable tension is up to 11000kN. The pylon anchorage zones of each cable are a steel box encased in the concrete walls, and shear studs are welded on the interfaces. The deck anchorage zones of cable are located at the tips of the overhangs. The construction procedure was conducted by the balanced cantilever method.

Butterfly web – Mukogawa Bridge (2017)



Butterfly web

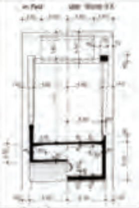




Location	Hyogo, Japan	**Cable type**	Epoxy coated strand
Client	NEXCO West	**Max. stress in cables**	1/1.67 = 60 % GUTS
Designer	Sumitomo Mitsui Construction	**Stress change in cable due to live load**	35 MPa
Cost	36.920.000 USD ($)		

The Mukogawa Bridge [96] is the first application of Extradosed Bridge with butterfly web in the world. Precast butterfly panels are used for girder webs to improve seismic behavior due to light weight and increase the productivity of construction. Therefore, compared with conventional box girder type, concrete volume of substructure is reduced up to 50%. To reduce the time of pier construction, precast skin segments which contain hoop tension reinforcing bars are applied. The speed of construction is double compared with usual cast-in- situ type. The pylon anchorage system for stay cables is unique. Single 100mm thick steel plates are arranged at the centre of the pylon. And double stay cables are anchored on the both sides of the plate. Therefore, horizontal component of stay cable force is carried by the steel plate and vertical component is carried by concrete only. The strength of this new system is confirmed by testing. In addition, the openings made between the butterfly webs provide easy access into the box girder components and inside brightness during inspection, making a great contribution to the ease of maintenance.

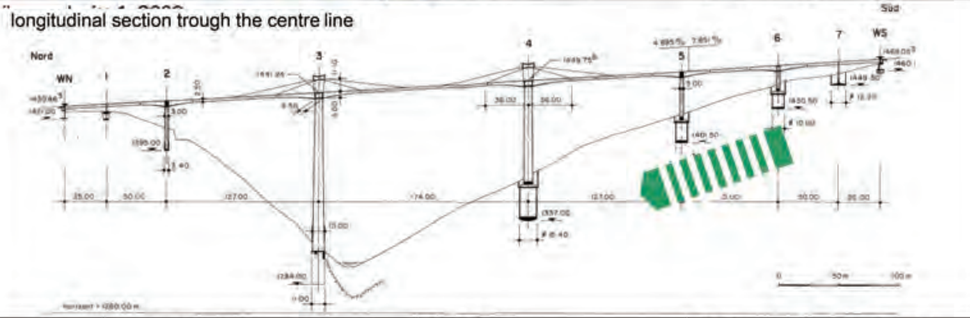
Triangular concrete walls – Ganterbridge (1980)



bridge cross-section

plan view / ground plan

longitudinal section trough the centre line



Location	Swiss Highway A9 Simplon	**Cable type**	0.5" strands with bond
Client	ASTRA / DFM Kt. Wallis	**Max. stress in cables**	N/A
Designer	Christian Menn	**Stress change in cable due to live load**	N/A
Cost	24.000.000 CHF		

The Ganterbridge is not a "real" Extradosed Bridge as the cables are embedded in the concrete, but the look and the structural behaviour are very similar to EDBs. It was built between 1976 and 1980, and carries A9 highway in a S-shape across a deeply-carved valley. The 678m long bridge construction consists of post-tensioned concrete and has a span configuration of 35m – 50m – 127m – 174m – 127m – 80m – 50m – 35m from north to south. The 10m wide bridge girder and all main columns are designed as reinforced concrete hollow boxes. The northern abutment and the columns S1, S2, S3 are founded in the stable rock on the right valley flank. The middle main span crosses a large part of the steeply sided mountain slope prone to sliding. The foundation shafts S4, S5, S6, S7 and the southern abutment is located in the sliding prone mountain slope on the Berisal site and follows the erosion deformation of the valley flank. The columns S4 and S5 are simply supported at the base with the shaft. The columns S6, S7 at the base and the bridge girder on the southern abutment are supported on movable bearing constructions.

The bridge system with the solid tension plate, the box girder and the hollow box columns exhibit consciously an appropriate overall stiffness, in order to resist the sliding effects from the mountain slope Berisal. The structure is designed for a maximum deflection of the column base S4 up to 150 mm. The permissible bearing displacements are more than 500 mm and the usable space on the bearings is generally more than 700 mm, so that several adjustments are possible over the service life. For the design of the bridge construction these topographic and geological boundary conditions were significant. Because of that, column S3 is consciously placed at the foremost stable building ground in the rock. The required high stiffness of superstructure and the S-shaped alignment led to the inclusion of the stay cables in the solid tension plates.

8.4 Bridge data table

In the following, the data of existing Extradosed Bridges are shown. The bridges are categorised by country, traffic type and year of completion. Some definitions of the values are as follows:

- bridge length: only the length of the extradosed bridge spans, spans with other bridge types are not considered
- main span: average of all extradosed main spans (when more than one main span), in case of two spans the longer one
- side span: average of the two side spans, in case of one pylon in longitudinal direction the smaller one
- pylon height above bridge deck level: height from the upper edge of the bridge deck until the top of the pylon (average, when different heights)
- pylon height above ground: height from ground until the top of the pylon (average, when different heights)
- cable interval: interval between the connection of the cables to the bridge deck (average, when different intervals)

No.	Bridge name	Completion	Location	Type of traffic	Bridge length (m)	Span length (m)	Main span (m)	Side span (m)	Number of masts in longitudinal direction	Width (m) Total width	Effective width	Number of planes	Girder support on pier	Girder height (m) At pier	At centre of main span	Tower height above bridge deck level (m)	Tower height above ground (m)	Extradosed cable arrangement Shape	Cable interval (m)
							l	l_1		w				h_1	h_2	h			
1	Shah Amanat Bridge	2010	Bangladesh	road	950.00					24.47									
2	Triplets-Kantutany	2010	Bolivia	road	229		113.50			14.80		1		3.50	2.10				
3	Triplets-Choqueyapu	2010	Bolivia	road	191.5		92.50			14.80		1		3.50	2.10				
4	Triplets-Orkojahuira	2010	Bolivia	road	218.8		103.00			14.80		1		3.50	2.10				
5	Kazungula Bridge	2017	Botswana	road	923		129.00			19									
6	Third Bridge over Rio Branco	2006	Brazil	road	198.00	54 + 90 + 54	90.00	54.00	2	17.40				2.50	2.00				
7	Brazil-Peru Integration Bridge	2007	Brazil	road/railway or railway	240.00	65 + 110 + 65	110.00	65.00	2	16.80				3.35	2.35				
8	Vidin-Calavat Bridge	2012	Bulgaria-Romania	road/railway or railway	779	124 + 3 × 180 + 115	180.00	119.50	4	31.35	21.72	2	rigid	4.50	4.50			harp	
9	North Arm Bridge	2008	Canada	light railway	458.00	139 + 180 + 139	180.00	139.00	2	10.31				3.40	3.40				
10	Deh Cho Bridge	2012	Canada	road	1045	112 + 190 + 112	190.00	112.00	2										
11	Golden Ears Bridge	2009	Canada	road	968.00	121 + 3 × 242 + 121	242.00	121.00	3 +	31.50		2	rigid	4.50	2.70	40.00	69.00	harp	10
12	Zhangzhou Zhanbei Bridge	2001	China	road	293.60	80.8 + 132 + 80.8	132.00	80.80	2	27.00		1	bearing	3.80	2.40	16.50	32.00	fan	4
13	Tongan Yinhu Bridge	2002	China	road	160.00	80 + 80	80.00	80.00	1	27.00		1	rigid	3.80	2.40	30.25	45.00	harp	4
14	Xiaoxihu Yellow River Bridge	2003	China	road	298.40	81.2 + 136 + 81.2	136.00	81.20	3 +	27.50		1	bearing	3.80	2.40	17.00	32.00	fan	5
15	Changzhou Canal Bridge	2003	China	road	260.40	70.2 + 120 + 70.2	120.00	70.20	2	40.00		1	bearing	4.10	2.60	31.00		harp	
16	Shijingshan 5th Loop Viaduct	2003	China	road	245.00	45 + 65 + 95 + 40	95.00	42.50	3 +	28.80		1	rigid	3.00	3.00	37.00		fan	
17	Lishi Viaduct	2004	China	road	305.00	85 + 135 + 85	135.00	85.00	2	26.00		1	bearing	4.20	2.40	18.00	37.00	fan	4
18	Linfen Fenhe Bridge	2004	China	road	330.00	90 + 150 + 90	150.00	90.00	2	26.00		1	rigid	4.50	2.20	28.97	56.00	fan	4
19	The 3rd Aodang Macao Bridge	2004	China	road	400.00	110 + 180 + 110	180.00	110.00	2	32		2	bearing	6.13	6.13	48.00	85.00	fan	
20	The 1st Zhanhe Bridge	2005	China	road	160.00	88 + 72	88.00	72.00	1	30.00		1	rigid	4.21	2.41	22.70	34.40	fan	4
21	Shangyu Caoejiang Bridge	2005	China	road	230.00	60 + 110 + 60	110.00	60.00	2	48.00		2	bearing	3.48	2.88	15.00	36.00	fan	4
22	Kunshan Wusongjiang Bridge	2005	China	road	200.20	100.1 + 100.1	100.10	100.10	1	33.00		1	rigid	5.00	3.00	14.70	42.10	harp	4
23	Shilihe Bridge	2005	China	road	145.00	72.5 + 72.5	72.50	72.50	1	28.50		1	bearing	1.80	1.80	29.00	36.50	fan	
24	Suqian South 2nd Loop Canal Bridge	2005	China	road	242.00	66 + 110 + 66	110.00	66.00	2	20.00		1	bearing	3.50	2.20	14.00	24.50	fan	4
25	Changxin West Loop Viaduct	2005	China	road	165.90	39 + 88 + 38.9	88.00	38.90	2	32.00		1	bearing	3.20	2.20	12.45		fan	
26	Huiqing Yellow River Highway Bridge	2006	China	road	486.00	133 + 220 + 133	220.00	133.00	2	20.00		1	rigid	7.50	3.50	30.00	61.00	fan	8
27	Pushang Bridge	2006	China	road	364.00	72 + 2 × 110 + 72	110.00	72.00	3 +	37.50		2	rigid			27.00		2 Plane	4
28	Chaobaihe Bridge	2006	China	road	384.00	72 + 2 × 120 + 72	120.00	72.00	3 +	29.26		1	rigid	4.20	2.20	21.50	43.00	fan	5
29	Zhongshan Qijiang Bridge	2006	China	road	293.60	80.8 + 132 + 80.8	132.00	80.80	2	31.00		1	bearing	3.80	2.40	21.00		fan	4
30	Kaifeng Yellow River Bridge	2006	China	road	1010.00	85 + 6 × 140 + 85	140.00	85.00	3 +	37.40		2	bearing	5.00	2.50	36.00		fan	4
31	Yumenkou Yellow River Bridge	2006	China	road	400.00	75 + 2 × 125 + 75	125.00	75.00	3 +	28.00		1	rigid	4.48	2.58	24.50		fan	
32	Deqing Yingxi Bridge	2006	China	road	160.00	35 + 65 + 60	65.00	47.50	2	32.50		1	rigid	2.40	2.40	21.50	28.00	fan	6
33	Hemaxi Bridge	2007	China	road	480.00	125 + 230 + 125	230.00	125.00	2	28.30		1	rigid	6.50	3.00	39.00	67.00	fan	4
34	Liuzhou Sanmenjiang Bridge	2007	China	road	360.00	100 + 160 + 100	160.00	100.00	2	41.00		2	rigid	6.17	2.77	22.80	53.00	fan	
35	The 4th Zhuzhou Xiangjiang Bridge	2007	China	road	430.00	75 + 2 × 140 + 75	140.00	75.00	3 +	29.00		1	bearing	4.35	2.80	17.50	39.00	harp	1.1
36	Liuzhou Jinglan Bridge (Reconst)	2008	China	road	583.50	56 + 5 × 94.3 + 56	94.50	56.00	3 +	30.00		1	bearing	4.50	2.50	16.04		harp	
37	Shenxianhe Bridge	2008	China	road	267.00	131 + 136	136.00	131.00	1	26.00		1	rigid	11.00	4.00	48.86		fan	
38	Nanan Bridge	2008	China	road	300.00	82 + 136 + 82	136.00	82.00	2	30.50		1	bearing	4.50	2.60	17.00			
39	Jiyang Yellow River Highway Bridge	2008	China	road	836.00	120 + 190 + 216 + 190 + 120	216.00	190.00	3 +	21.00		2	bearing	6.50	4.00	20.00	32.00	fan	5

Table 1: Continued

No.	Bridge name	Completion	Location	Type of traffic	Bridge length (m)	Span length (m)	Main span (m)	Side span (m)	Number of masts in longitudinal direction	Width (m) Total width	Effective width	Number of planes	Girder support on pier	Girder height (m) At pier	At centre of main span	Tower height above bridge deck level (m)	Tower height above ground (m)	Extradosed cable arrangement Shape	Cable interval (m)
							l	l_1		w				h_1	h_2	h			
40	Yudaohe Bridge	2008	China	road	240.00	60 + 120 + 60	120.00	60.00	2	12.20		2	rigid	1.80	1.80	15.30	38.00	fan	7.2
41	Kangqiaolu Bridge	2008	China	road	242.00	66 + 110 + 66	110.00	66.00	2	32.90		2	bearing	4.00	2.40	16.00		fan	4
42	Jiangdu Qizha Bridge	2009	China	road	195.00	55 + 85 + 55	85.00	55.00	2	15.40		2	rigid	3.20	2.20	15.00	32.00	fan	7
43	Jia-Yue Bridge	2009	China	road	540.00	145 + 250 + 145	250.00	145.00	2	28.00		2	bearing	7.00	5.00	32.53	120.00	fan	5
44	Dazhenggang Bridge	2009	China	road	345.00	90 + 165 + 90	165.00	90.00	2	34.76		1	bearing	6.50	3.00	33.00	49.50	fan	
45	Xiangang Bridge	2009	China	road	120.00	60 + 60	60.00	60.00	1	16.25		2	rigid	2.50	1.60	22.00	37.86	fan	5
46	Qiligou Hanjiang Bridge	2009	China	road	270.00	72.5 + 125 + 72.5	125.00	72.50	2	30.00		1	bearing	3.60	2.30	30.50		harp	
47	Zhoushan Sanjiaogang Bridge	2010	China	road	655.00	120 + 210 + 210 + 115	210.00	117.50	3 +	12.00		2	rigid	6.00	3.50	34.00	50.80	fan	5
48	Wanxihe Bridge	2010	China	road	176.00	88 + 88	88.00	88.00	1	25.50		1	bearing	3.80	2.60	22.00		harp	
49	Changzhou East Loop Canal Bridge	2010	China	road	260.40	70.2 + 120 + 70.2	120.00	70.20	2	28		1	bearing	4.10	2.60	31.00		fan	
50	Yongdingxinhe Bridge	2010	China	road	315.00	85 + 145 + 85	145.00	85.00	2	43.00		4	bearing	4.00	3.00	20.00		fan	7.5
51	Xijiang Bridge on Jiangzhao Expressway	2011	China	road	886.00	128 + 3 × 210 + 128	210.00	128.00	3 +	38.30		1	rigid	6.80	3.80	30.50	52.50	fan	4
52	Huaian Tongfulu Canal Bridge	2011	China	road	360.00	100 + 160 + 100	160.00	100.00	2	28.00		2	rigid	4.80	3.40	26.00		fan	
53	Guangzhou Shawan Bridge	2011	China	road	523.00	137.5 + 248 + 137.5	248.00	137.50	2	34.00		2	rigid	6.50	3.50	25.00		fan	4
54	Shanghai Dazhihe Bridge	2011	China	road	300.00	80 + 140 + 80	140.00	80.00	2	13.64		1	bearing	5.60	3.00	20.50		harp	
55	Jialingjiang Double-line Railway Bridge	2011	China	road	464.00	118 + 228 + 118	228.00	118.00	2	13.10		2	rigid	7.80	7.80	28.50	38.00		
56	Jinhulianluoxian Bridge	2011	China	road	359.20	64.6 + 115 + 115 + 64.6	115.00	64.60	3 +	14.40		2	rigid	6.00	4.00	15.00	31.00	fan	4
57	Qiupuhe Bridge	2012	China	road	300.00	80 + 140 + 80	140.00	80.00	2	34.00		1	bearing	4.50	2.50	23.50	44.00	fan	5
58	Nanyanghu Bridge	2012	China	road	384.00	72 + 120 + 120 + 72	120.00	72.00	3 +	43.00		2	bearing	5.00	3.40	27.65	38.00	fan	3.5
59	Qiangcao Yangtze River Bridge	2012	China	road	504.00	128 + 248 + 128	248.00	128.00	2	37.00		2	rigid	9.00	3.80	31.00	85.00	fan	8
60	Qianjinjie Viaduct	2012	China	road	140.00	70 + 70	70.00	70.00	1	27.50		1	rigid	4.60	3.00	21.00	42.00	harp	
61	Changchuhe Bridge	2012	China	road	163.20	41.6 + 80 + 41.6	80.00	41.60	2	32.00		1	rigid	2.00	2.00	15.80		fan	5
62	Changshan Bridge	2013	China	road	540.00	140 + 260 + 140	260.00	140.00	2	23.00		2	rigid	9.00	4.50	37.40	72.00	fan	4
63	Ningjiang Songhuajiang Bridge	2013	China	road	640.00	95 + 3 × 150 + 95	150.00	95.00	3 +	27.50		1	rigid	5.50	3.00	21.50	37.00	fan	4
64	Guijiugou Bridge	2013	China	road	140.00	70 + 70	70.00	70.00	1	61.00		2	bearing	2.80	2.80	33.50	41.00	harp	5
65	Hezhou Bridge	2013	China	road	140.00	70 + 70	70.00	70.00	1	27.00		1	bearing	3.50	3.50	28.10		harp	4
66	Sifang Bridge	2013	China	road	170.00	85 + 85	85.00	85.00	1	27.00		1	rigid	4.48	2.58	26.50	56.00	fan	4
67	Nanpanjiang Bridge	2013	China	road	396.00	108 + 180 + 108	180.00	108.00	2	27.30		1	rigid	5.80	3.00	29.00	90.00	fan	7
68	Lasa Najin Bridge	2014	China	road	374.00	70 + 117 + 117 + 70	117.00	70.00	3 +	33.00		1	bearing	4.00	2.50	17.70	35.00	fan	5
69	Xinbianhe Bridge	2014	China	road	120.00	60 + 60	60.00	60.00	1	28.00		1	rigid	3.25	2.80	18.50	40.00	harp	4
70	Xinbianhe Bridge in Suzhou	2014	China	road	315.00	85 + 145 + 85	145.00	85.00	2	43.5		4	bearing	4.50	3.00	20.50	42.00	fan	7.5
71	Shiwan Bridge	2014	China	road	331.00	90.5 + 150 + 90.5	150.00	90.50	2	33.50		1	bearing	5.00	3.50	28.00	45.00	fan	4.5
72	Changshan Bridge	2014	China	road	238.00	65 + 108 + 65	108.00	65.00	2	28.60		1	bearing	4.20	2.30	19.15	34.00	fan	4
73	Shuofang Helanshanlu Bridge	2014	China	road	140.00	70 + 70	70.00	70.00	1	60.00		2	rigid	2.00	2.00	30.00		fan	
74	Wenchangnanlu Bridge	2014	China	road	154.00	79 + 75	79.00	75.00	1	29.00		1	rigid	2.79	2.79	20.00		fan	
75	Jinbao Railway Bridge	2014	China	road	140.00	84 + 56	84.00	56.00	1	23		2	rigid	6.00	3.80	20.00	36 +	fan	6
76	Hanjiang Bridge on Yunshi Expressway	2014	China	road	494.00	128 + 238 + 128	238.00	128.00	2	26.50		1	rigid	7.20	3.20	39.90	122.50	fan	4
77	Danyang Qilianglu Canal Bridge	2015	China	road	260.00	70 + 120 + 70	120.00	70.00	2	43.00		2	bearing	4.50	3.40	19.80	35.00	fan	

Table 1: Continued

No.	Bridge name	Completion	Location	Type of traffic	Bridge length (m)	Span length (m)	Main span (m)	Side span (m)	Number of masts in longitudinal direction	Width (m) Total width	Effective width	Number of planes	Girder support on pier	Girder height (m) At pier	At centre of main span	Tower height above bridge deck level (m)	Tower height above ground (m)	Extradosed cable arrangement Shape	Cable interval (m)
							l	l_1		w				h_1	h_2	h			
78	Chaoyanggou Shuiku Bridge	2015	China	road	414.00	118 + 188 + 108	188.00	113.00	2	35.00		2	rigid	7.00	4.50	33.00	67.00	fan	4.8
79	Sanguan Hanjiang Bridge	2015	China	road	430.00	120 + 190 + 120	190.00	120.00	2	33.50		1	rigid	6.50	3.00	47.00	74.00	harp	4
80	Jiangmen Shenglinanlu Bridge	2015	China	road	240.00	65 + 110 + 65	110.00	65.00	2	33.50		1	bearing	4.20	2.50	19.00	35.00	fan	6
81	Haian Qianxi Bridge	2015	China	road	205.00	55 + 95 + 55	95.00	55.00	2	29.50		1	bearing	3.80	2.60	14.00	23.00	fan	4
82	Yudaihe Bridge	2015	China	road	260.00	45 + 85 + 85 + 45	85.00	45.00	3 +	33.50		2	rigid	3.00	2.00	19.00	31.10	fan	7
83	Chencun Bridge	2015	China	road	458.00	120 + 218 + 120	218.00	120.00	2	36.00		1	rigid	6.50	3.50	31.80	53.00	fan	4
84	Huanglongdai Bridge	2015	China	road	424.00	108 + 208 + 108	208.00	108.00	2	42.00		3	rigid	6.00	3.80	28.00	56.00	fan	8
85	Longjinghe Bridge	2015	China	road	332.00	86 + 160 + 86	160.00	86.00	2	28.00		1	rigid	6.50	3.50	28.50	94.00	fan	8
86	Nanao Bridge	2015	China	road	490	126 + 238 + 126	238.00	126	2	14.40		2	rigid	8.00	4.00	29.90			
87	New Yanggang Bridge	2015	China	road	456	120 + 216 + 120	216	120	2	22.5		4	bearing	8.00	3.50	57.00		fan	
88	Dongmen Bridge on Bazhong	2015	China	road	256	68 + 120 + 68	120	68	2	32.50		2	bearing	4.63		23.50		fan	
89	Yongzhou Chengnan Bridge	2016	China	road	380	70 + 2 × 120 + 70	120	70	3	34.00		1	bearing	4.80	2.50	21.00		fan	
90	Yihe Bridge	2016	China	road	228.9	65.4 + 4 × 98.1 + 65.4	98.1	65.4	5	38.60		1	bearing	4.50	2.80	25.50		fan	
91	Wuhan-Jiujiang railway Bridge	2017	China	road	324	82 + 154 + 88	154	85.00	2	10.00		2	bearing	7.50	4.00	20.00		fan	
92	Wuhu Yangtze River Bridge	2000	China	road/railway or railway	672.00	180 + 312 + 180	312.00	180.00	2	23.40	18	2	bearing	14.00	14.00	35.00	106.00	fan	12
93	Zhengzhou Yellow River Bridge	2010	China	road/railway or railway	1080.00	120 + 5 × 168 + 120	168.00	120.00	3 +	32.5	22.5	1	bearing	14.00	14.00	37.00	68	fan	12
94	Domovinski Bridge	2006	Croatia	road	264.00	72 + 120 + 72	120.00	72.00	2	34				3.55	3.55				
95	Saint-Rémy-de-Maurienne Bridge	1996	France	road	100.90	52.4 + 48.5	52.40	48.50	1	13.40				2.20	2.20				
96	Trois Bassins Viaduct	2008	France	road	306.00	126 + 104.4 + 75.6	126.00	100.80	2	22				7.00	4.00				
97	Waschmühltalbrücke	2013	Germany	road	226.5	45.4 + 68.1 + 68.1 + 44.9	68.10	45.15	3 +	20.88						16			
98	Korong Bridge	2004	Hungary	road	114.24	52.26 + 61.98	61.98	52.26	1	15.85				2.50	2.50				
99	Ganga Bridge(Between Arrah and Chhapra)	2017	India	road	1900.00	60 + 15 × 120 + 60	120.00	60.00	3 +	20.50		1		3.17	3.17	18.00	37.50	fan	3.04
100	B.P. Mandal	u.c.	India	road	290.00	75 + 140 + 75	140.00	75.00	2	12.68	11.59	2		2.70	2.70	27.00	40.45	fan	1.05
101	Hatania-Doania(Across River Hatania-Doania)	u.c.	India	road	340.00	85 + 170 + 85	170.00	85.00	2	19.00	15.60	2		5.00	4.00	23.00	38.88	fan	1.09
102	Ganga Bridge(Between Sultanganj and Aguwani Ghat)	u.c.	India	road	550.00	140 + 270 + 140	270.00	140.00	2	25.50		2		5.00	1.50	27.50		fan	1.75
103	Kacchi Dargah Bridge	u.c.	India	road	9759		150.00		3 +	32.50		1	rigid	5.00	5.00	21.50		fan	3
104	Kalyani		India	road															
105	Aunta Simaria		India	road															
106	Sangam. Allahabad		India	road															
107	Bridge at Siddapura, Karnataka	2006	India	road	56.00	56	56.00			9.50		2		2.10	2.10	5.28		fan	
108	Nivedita Setu	2006	India	road	880.00	55 + 7 × 110 + 55	110.00	55.00	3 +	29		1		3.40	3.40	14		fan	
109	Keri Tiracol	u. c.	India	road	350.00	70 + 210 + 70	210.00	70.00	2	13		2		4.30	4.30	22.70			
110	Durgam Cheruvu Lake		India	road	365.00	66 + 233 + 66	233.00	66.00	2	25.70		1		5.00	5.00	25.20		fan	
111	Ha Lania	u. c.	India	road	340.00	85 + 170 + 85	170.00	85.00	2	15.60		2		5.00	4.00	23.00		fan	
112	Barapulah Bridge	u. c.	India	road	552.50	85 + 3 × 127.5 + 85	127.50	85.00	3 +			2		4.00	4.00	17		fan	
113	Narmada Bridge		India	road	1200.00	96 + 7 × 144 + 96	144.00	96.00	3 +	20.80		2		4.00	4.00	18			

Table 1: Continued

No.	Bridge name	Completion	Location	Type of traffic	Bridge length (m)	Span length (m)	Main span (m)	Side span (m)	Number of masts in longitudinal direction	Width (m) Total width	Effective width	Number of planes	Girder support on pier	Girder height (m) At pier	At centre of main span	Tower height above bridge deck level (m)	Tower height above ground (m)	Extradosed cable arrangement Shape	Cable interval (m)
							l	l_1		w				h_1	h_2	h			
114	Mandovi Bridge	u. c.	India	road	620.00	85 + 3 × 150 + 85	144.00	85.00	3 +	21		1		4.00	4.00	35			
115	Kamla Park Bridge	u. c.	India	road	220.00	55 + 110 + 55	110.00	55.00	2			2		1.90	1.90	22.50			
116	Mumbai Metro-WEH	2013	India	road/railway or railway	132	2 × 23 + 86 + 2 × 23	83.00	23.00	2	11.75	8.85	2	bearing	2.09	2.09	19.09	38.04	fan	6
117	Moolchand Crossing	2010	India	road/railway or railway	167.5	51 + 65.5 + 51	65.50	51.00	2	9.36	7.7	1	rigid	2.00	2.00	8.00		fan	6
118	Pragati Maidan	2006	India	road/railway or railway	196.35	31.25 + 93 + 24.8	93.00	28.03	2	9.36		2	rigid	2.14	2.14	12.43		fan	
119	Kacchi dargah bidupu	u. c.	India		9760														
120	Musi 4	2018	Indonesia	road	312		140.00			13.50									
121	River Erne Bridge	2013	Ireland	road	141.9	36.2 + 69.5 + 36.2	69.50	36.20	2	17.10		2	bearing	1.53	1.53			harp	
122	Muko River Bridge	2017	Japan	road	442.2	71.8 + 3 × 100 + 67.8	100.00	69.80	3 +	24.15	21.50	1	rigid	4.00	4.00	8.50	93.20	fan	6.04
123	Odawara Blueway Bridge	1994	Japan	road	270	73.3 + 122.3 + 73.3	122.30	73.30	2	13.00	9.50	2	rigid	3.50	2.20	10.70	37.20	fan	3.75
124	Tsukuhara Bridge	1998	Japan	road	323	65.4 + 180.0 + 76.4	180.00	76.40	2	12.80	9.25	2	rigid	5.50	3.00	16.00	57.00	fan	7
125	Shoyo Bridge(Kanisawa Bridge)	1998	Japan	road	380	99.3 + 180.0 + 99.3	180.00	99.30	2	17.50	15.50	2	bearing	5.60	3.30	22.10	43.20	fan	4
126	Karato Bridge (west)	1998	Japan	road	285	74.1 + 140.0 + 69.1	140.00	74.10	2	11.50	8.70	2	rigid	3.50	2.50	12.00	31.40	fan	4
127	Karato Bridge (east)	1998	Japan	road	260	66.1 + 120.0 + 72.1	120.00	72.10	2	11.50	8.70	2	rigid	3.50	2.50	12.00	43.50	fan	4
128	The second Mactan Bridge	1999	Japan	road	410	111.5 + 185.0 + 111.5	185.00	111.50	2	21.00	18.00	2	rigid	5.10	3.30	18.20	42.60	fan	5
129	Santani River Bridge(Second Bridge)	1999	Japan	road	152	57.9 + 92.9	92.90	57.90	1	20.40	8.5	1	rigid	6.50	2.50	12.80	48.30	fan	4
130	Matakina Bridge	2000	Japan	road	200	109.3 + 89.3	109.30	89.30	1	11.30	8.00	2	rigid	6.00	3.50	26.40	59.40	fan	4
131	Ashikita Bridge	2000	Japan	road	225	60.8 + 105.0 + 57.5	105.00	60.80	2	11.00	9.25	2	bearing	3.20	2.10	12.30	25.00	fan	3.75
132	Yukisawa third Bridge	2000	Japan	road	177.1	70.3 + 71.0 + 34.4	71.00	70.30	2	15.80	12.50	2	bearing	3.50	2.00	11.30	20.30	fan	4
133	Suriage Dam Bridge	2000	Japan	road	110	84.15	84.15			9.20	7.00	2	rigid	5.00	2.80	16.50	21.50	fan	4
134	Shikari Bridge	2001	Japan	road	610	94.0 + 140.0 × 3 + 94.0	140.00	94.00	3 +	23.00	22.00	1	bearing	6.00	3.00	10.00	32.70	harp	3.5
135	Nakanoike Bridge	2001	Japan	road	123	60.6 + 60.6	60.60	60.60	1	21.40	17.40	1	rigid	4.00	2.50	11.80	26.00	fan	7
136	Miyakoda River Bridge	2001	Japan	road	268	133.0 + 133.0	133.00	133.00	1	19.91	16.50	2	rigid	6.50	4.00	20.00	83.00	fan	6
137	Hozu Bridge	2001	Japan	road	252	76 + 100 + 76	100.00	76.00	2	15.30	14.50	2	rigid	2.80	2.80	10.00	31.50	fan	3.5
138	Kiso River Bridge (Kisowaga Bridge)	2001	Japan	road	1145	160 + 275 × 3 + 160	275.00	160.00	3 +	33.00	29.00	1	bearing	7.30	4.30	30.00	54.30	fan	5
139	Ibi River Bridge (Ibigawa Bridge)	2001	Japan	road	1397	154 + 271.5 × 4 + 157	271.50	157.00	3 +	33.00	29.00	1	bearing	7.30	4.30	30.00	54.80	fan	5
140	Japan Palau Friendship Bridge in Republic of Palau	2001	Japan	road	412.7	82 + 247 + 82	247.00	82.00	2	11.60	8.00	2	rigid	7.00	3.50	27.00	40.70	fan	4.25
141	Hukaura Bridge	2002	Japan	road	218.1	62.1 + 90.0 + 66.0	90.00	64.05	2	13.70	10.80	2	rigid	3.00	2.50	8.50	40.50	fan	4
142	Gotoh River Bridge(Sashikubo Bridge)	2002	Japan	road	230.3	114.0 + 114.0	114.00	114.00	1	11.33	8.00	2	rigid	6.50	3.20	22.00	73.00	fan	7
143	Tobiuo Bridge	2002	Japan	road	386	90.0 + 130.0 + 80.5	130.00	85.25	3 +	25.80	20.50	1	bearing	4.00	2.40	13.00	29.00	fan	3.5
144	Akatonbo Bridge(Shinmeisei Bridge)	2004	Japan	road	294.32	88.3 + 122.3 + 81.2	122.34	84.78	2	21.50	16~20	1	rigid	3.50	3.50	16.50	23.80	fan	3.6
145	Himi Bridge	2004	Japan	road	365	91.8 + 180.0 + 91.8	180.00	91.75	2	12.95	9.75	2	rigid	4.00	4.00	19.80	42.48	fan	6.4
146	Sannohe Bohkyo Bridge	2005	Japan	road	400	99.3 + 200.0 + 99.3	200.00	99.25	2	13.45	10.25	2	rigid	6.50	3.50	25.00	66.00	fan	3.75
147	Noikura Bridge	2005	Japan	road	273	62.2+135.0+74.2	135.00	68.20	2	11.20	9.00	2	rigid	4.50	2.50	11.10	35.00	fan	3.75
148	Nanchiku Bridge	2006	Japan	road	248	68.1 + 110.0 + 68.1	110.00	68.05	2	20.55	19.75	2	bearing	3.50	2.60	11.00	17.20	fan	12.75
149	Satonojo Bridge	2006	Japan	road	186	54.9 + 77.0 + 52.9	77.00	53.90	2	13.25	12.25	2	rigid	3.50	2.50	8.00	23.80	fan	12.15
150	Asagiri Bridge	2006	Japan	road	166	80.2 + 84.2	84.20	80.20	1	17.80	15.00	2	rigid	4.50	3.00	14.50	16.50	fan	4

Table 1: Continued

No.	Bridge name	Completion	Location	Type of traffic	Bridge length (m)	Span length (m)	Main span (m)	Side span (m)	Number of masts in longitudinal direction	Width (m) Total width	Effective width	Number of planes	Girder support on pier	Girder height (m) At pier	At centre of main span	Tower height above bridge deck level (m)	Tower height above ground (m)	Extradosed cable arrangement Shape	Cable interval (m)
							l	l_1		w				h_1	h_2	h			
151	Tokunoyama Hattoku Bridge	2006	Japan	road	503	139.7 + 220.0 + 139.7	220.00	139.70	2	8.20	7.00	2	rigid	6.50	3.50	22.50	101.00	fan	7
152	Ohmi-Otori Bridge(Ritto Bridge)	2007	Japan	road	495	137.6 + 170	170.00	137.60	1	19.63	16.50	2	rigid	7.50	4.50	30.50	65.00	fan	4.8
153	Ohmi-Otori Bridge(Ritto Bridge)	2007	Japan	road	505	152.6 + 160	160.00	152.60	1	19.63	16.50	2	rigid	7.50	4.50	30.50	61.50	fan	4.8
154	Yanagawa dam 9th Bridge	2007	Japan	road	264	130.7 + 130.7	130.70	130.70	1	17.40	15.00	2	rigid	6.50	4.00	24.60	57.00	fan	3
155	Hedase Bridge	2008	Japan	road	285	69.2+145+69.2	145.00	69.15	2	12.40	9.00	2	bearing	5.50	3.00	14.18	48.00	fan	4
156	Rades La Goulette Bridge	2009	Japan	road	260	70.0 + 120.0 + 70.0	120.00	70.00	2	23.50	18.00	1	rigid	3.70	2.60	20.00	20.90	fan	3.5
157	Yumekake Bridge (Route1681-1Bridge)	2010	Japan	road	290	127.0 + 118.9	127.00	118.90	1	14.20	10.51	2	rigid	4.80	2.80	25.00	50.50	fan	6
158	Shin yokoyama Bridge	2010	Japan	road	232.6	88.2+142.0	142.00	88.20	1	11.85	8-11	2	rigid	8.00	4.00	40.00	95.00	fan	6
159	Surikamigawa Bridge	2000	Japan	road	166	84 + 82	84.00	82.00	1	9.20				5.00	2.80	16.50			
160	Tatekoshi Kosen Bridge	2004	Japan	road	113	56.3 + 55.3	56.30	55.30	1	19.14	15.50	1	rigid	2.90	1.80	10.50	24.40	fan	
161	Shirosuna River Bridge	2015	Japan	road	210.8	109.00 + 98.75	109.00	98.75	1	18.3	17.50	2	rigid	5.50	3.80	14.50	41.00	fan	
162	Hinase Bridge	2015	Japan	road	765	86.8 + 170 + 155 + 2 × 135 + 80.8	148.75	83.80	3 +	7.72	6.50	2	rigid	5.50	3.20	20.00	53.50	fan	7
163	Fudo Bridge	2010	Japan	road	590	65.0 + 125.0 + 2 × 155.0 + 90.0	145.00	77.50	3 +	16.00	15.50	2		6.00	5.00	13.50	99.50		
164	Koshigaya Lake Town Bridge	2010	Japan	road	105.8					18.55	17.75	2	bearing	1.80	1.20	12.00	22.20	fan	
165	Ege Bridge	2011	Japan	road	129.65	53.0 + 76.65	76.65	53.00	1	8.00	5.00	2				15.30	32.40	harp	
166	Negai Bridge(Basen River Bridge)	2012	Japan	road	181	91.5 + 87.55	91.50	87.55	1	18.90	16.00	2		4.80	2.80	20.00	37.00		8
167	Komenichigou Bridge	2014	Japan	road	214		135.00			14.30									
168	Jintsuugawa Bridge	2012	Japan	road	428		128.00			14.50									
169	Ohnogawa Bridge	2008	Japan	road	285		113.00			12.40									
170	Hirano Bridge	2008	Japan	road	132		63.00			8									
171	Kita 4bancho Daikousen Bridge	2006	Japan	road	111	54.50 + 54.50	54.40	54.50		25.80	25.00	1	rigid	3.50	1.40	16.30	30.80		
172	Kanazawatagami Bridge	2007	Japan	road	166		84.00			19.80									
173	Arakogawa Bridge	2003	Japan	road	246		90.00			12.70									
174	Fukaura	2001	Japan	road	140		90.00			13.50									
175	Sashiki Bridge	2001	Japan	road	223		105.00			13.90									
176	Okuyama Bridge (Shin-Karato Bridge)	1998	Japan	road	285	74.1 + 140 + 69.1	140.00	71.60	2	16.5	14.1	2	bearing	3.50	2.50	12.00			
177	Mitanigawa Daini Bridge	1999	Japan	road	152	92.9	92.90			17				6.00	3.00	12.80			
178	Chuo Bridge	2014	Japan	road	356	63 + 121 + 121.5 + 59.5	121.25	61.25	3 +	17.8	17.00	2	rigid	4.70	2.50	14.50	37.20	fan	4.00
179	Komen 1st Bridge	2014	Japan	road	494	48.5 + 94.0 + 135. + 135.0 + 78.5	121.33	63.50	3 +	16	16.80	2	rigid	4.00	3.00	13.00	94.50	fan	
180	Onahama Marine Bridge	2015	Japan	road	510	75.0 + 3 × 120.0 + 75.0	120.00	75.00	3 +	16	12.25	2	rigid						
181	Sakanomachi Bridge (Fukushima Bridge)	2007	Japan	road	104.6	29.9 + 43.0 + 29.9	43.00	29.90	2		20.00			2.00	1.00	14.20	48.00		
182	Nga Tu So Bridge	2006	Japan	road	93	24.0 + 45.0 + 24.0	45.00	24.00	2	17.5		1	rigid	1.10	1.10	9.50	18.10	fan	
183	Haneji Bridge		Japan	road	200	109.3	109.30			8			rigid	6	3.50	26.40			
184	Sajiki Bridge		Japan	road	225	105	105.00			9.30				3.20	2.10	12.30			
185	Hodu Bridge		Japan	road	368	100	100.00			14.50			rigid	2.80	2.80	10.00			

Table 1: Continued

No.	Bridge name	Completion	Location	Type of traffic	Bridge length (m)	Span length (m)	Main span (m)	Side span (m)	Number of masts in longitudinal direction	Width (m) Total width	Width (m) Effective width	Number of planes	Girder support on pier	Girder height (m) At pier	Girder height (m) At centre of main span	Tower height above bridge deck level (m)	Tower height above ground (m)	Extradosed cable arrangement Shape	Extradosed cable arrangement Cable interval (m)
							l	l_1		w				h_1	h_2	h			
186	Yubikubo Bridge		Japan	road	230.3	114	114.00			8				6.50	3.20	22.00			
187	Sinkawa Bridge		Japan	road	386	130	130.00			20.50				4.00	2.40	13.00			
188	Surikamigawa Bridge	2000	Japan	road															
189	Yashirominami Bridge	1995	Japan	road/railway or railway	340	64.2 + 105.0 × 2 + 64.2	105.00	64.20	3 +	12.80		2	rigid	2.50	2.50	12.00	28.80	fan	4
190	Yashirokita Bridge	1995	Japan	road/railway or railway	200	54.3 + 90.0 + 54.3	90.00	54.30	2	12.80		2	rigid	2.50	2.50	10.00	30.60	fan	4
191	Shinkawa Bridge	1999	Japan	road/railway or railway	111	51.4 + 58.4	58.40	51.40	1	13.20		2	bearing	2.60	2.60	9.90	15.10	fan	4
192	Sakai River Bridge	2003	Japan	road/railway or railway	182.9	55.3 + 70.0 + 55.3	70.00	55.30	2	12.50	9.30	2	bearing	3.50	3.15	9.30	19.85	fan	4.5
193	Arako River Bridge	2004	Japan	road/railway or railway	200.92	54.42 + 90 + 56.5	90.00	55.46	2	12.70	9.10	2	rigid	3.60	2.60	12.60	18.90	fan	4
194	Sannaimaruyama Over Bridge	2008	Japan	road/railway or railway	450	74.2 + 150.0 + 150.0 + 74.2	150.00	74.18	3 +	13.85		2	rigid, bearing	8.00	3.80	17.50	40.50	fan	4
195	Ono River Bridge	2009	Japan	road/railway or railway	286	29 + 113 + 113 + 29	113.00	19.00	3 +	11.85	10.45	2	rigid	6.00	3.50	15.00	33.00	fan	7
196	Amarube Bridge	2011	Japan	road/railway or railway	270.1	50.1 + 2 × 82.5 + 55.0	82.50	52.55	3 +	7.55	7.25	2	rigid		3.50	8.50	45.00		
197	Jinzu River Bridge	2013	Japan	road/railway or railway	428	86 + 2 × 128 + 86	128.00	86.00	3 +	13.70	11.30	2	bearing	6.00	3.50	15.00	38.20		
198	Second Agatsumagawa Bridge		Japan	road/railway or railway	390	111.5 + 167 + 111.5	167.00	110.40	2										
199	Pakse Bridge	2000	Laos	road	357.5	123 + 143 + 91.5	143.00	107.25	2	14.60	13.80	2	rigid	6.50	3.00	15.00	27.00	fan	3.5
200	Riga Southern Bridge	2008	Latvia	road			110.00		3 +	34.28		1						fan	
201	Earthquake Memorial Bridge	2012	Pakistan	road	246		123.00			15.60									
202	Koror Babeldoap Bridge	2002	Palau	road	411.00	82 + 247 + 82	247.00	82.00	2	11.60				7	3.50				
203	Second Mandaue–Mactan Bridge	1999	Philippines	road	408.00	111.5 + 185 + 111.5	185.00	111.50	2	18				6	3.50				
204	Viaduct over the S8 Expressway in Olesnica	2012	Poland	road	83.4	43.5 + 39.9	43.50	39.90	1			2	bearing			6.65	15.21	harp	
205	Bridge over the Motława River in Gdansk	2012	Poland	road	290	77.5 + 135 + 77.5	135.00	77.50	2			2	bearing	3.20	3.20	22.80	32.30	harp	
206	Bridge along the A1 Motorway in Mszana	2014	Poland	road	380.00	60 + 2 × 130 + 60	130.00	60.00	3 +	43.02		1		4.00	2.60	15.10		fan	5
207	Bridge over the Vistula River (M4) in Kwidzyn	2013	Poland	road	674.5	130 + 2 × 204 + 130	204.00	130.00	3 +	15.9		2	bearing	3.50	3.50	17.20		harp	
208	Bridge (MS-3/DK-16) near Ostróda	2017	Poland	road	677	132.5 + 2 × 206 + 132.5	206.00	132.50	3 +	28.6	18	1		6.04	4.04	22.90	57.75	fan	8
209	Olsztyn	2012	Poland	road	240.00	60 + 120 + 60	120.00	60.00	2					4.00	4.00				
210	Bridge over the River Warta in Konin	2007	Poland	road	200.00	60 + 80 + 60	80.00	60.00	2	25.10		2	bearing	2.60	2.60	10.30			12
211	Socorridos Bridge	1993	Portugal	road	268.90	73.3 + 122.3 + 73.3	122.30	73.30	2	13				3.50	2.20				
212	Ziarnoviva	2009	Slovakia	road	240		80.00			26.10									
213	Viaduct in Povazska Bystrica	2010	Slovakia	road	871	71 + 6 × 122 + 68	122.00	69.50	8	30.65	26.5	1	bearing	6.00	4.70	14.10	32.71	fan	4.88

Table 1: Continued

No.	Bridge name	Completion	Location	Type of traffic	Bridge length (m)	Span length (m)	Main span (m)	Side span (m)	Number of masts in longitudinal direction	Width (m) Total width	Width (m) Effective width	Number of planes	Girder support on pier	Girder height (m) At pier	Girder height (m) At centre of main span	Tower height above bridge deck level (m)	Tower height above ground (m)	Extradosed cable arrangement Shape	Extradosed cable arrangement Cable interval (m)
							l	l_1		w				h_1	h_2	h			
214	Ptuj	2007	Slovenia	road	430	65 + 3 × 100 + 65	100.00	65.00	3 +	18				2.70	2.70				
215	Aam	2009	South Korea	road	308	84 + 140 + 84	140.00	84.00	2	17.10		2	rigid	4.50	3.00	18.00	36.35	fan	8
216	Yaro	2015	South Korea	road	760	105 + 180 + 190 + 180 + 105	183.33	105.00	3 +	27.04		2	rigid	7.00	4.00	27.25	148.95	fan	10
217	Pyeongtaek	2014	South Korea	road	1160	110 + 6 × 160 + 90	160.00	100.00	3 +	30.90		2	bearing	5.50	3.50	20.80	71.60	fan	4
218	Unam	2011	South Korea	road	670	75 + 4 × 130 + 75	130.00	75.00	3 +	23.00		2	rigid	4.20	3.30	14.80	52.10	fan	7
219	Kyeongin Unha #3–Sicheon	2012	South Korea	road	305	80 + 145 + 80	145.00	80.00	2	33.50		2	bearing	4.50	3.00	18.00	42.20	fan	7
220	Unnam	2012	South Korea	road	325	85 + 155 + 85	155.00	85.00	2	23.90		2	bearing	5.50	3.50	18.00	42.00	fan	8
221	Heainji	2010	South Korea	road	360	80 + 200 + 80	200.00	80.00	2	14.50		2	bearing	5.00	3.50	13.50	45.70	fan	4
222	2nd Pyeongyeo	2005	South Korea	road	250	65 + 120 + 65	120.00	65.00	2	21.00		2	rigid	3.50	2.70	10.50	59.30	fan	4
223	Noksan	2005	South Korea	road	140	70 + 70	70.00	70.00	1	22.35		2	rigid	3.50	2.70	14.50	32.50	fan	4
224	Naerinchheon	2016	South Korea	road	382	45 + 120 + 155 + 62	137.50	53.50	3 +	27.15		2	rigid	7.50	4.50	30.00	84.50	fan	6
225	Cheongpyeong	2017	South Korea	road	720	90 + 4 × 135 + 90	135.00	90.00	3 +	14.34		2	bearing	4.00	2.50	13.50	32.20	fan	
226	Kyong-An Bridge	2012	South Korea	road	270	70 + 130 + 70	130.00	70.00	2	30		1	bearing	3.00	3.00	16.50	45.33	fan	5
227	Keong-An Bridge	2013	South Korea	road	270	70 + 130 + 70	130.00	70.00	2	30.00		1	bearing				40.00	fan	
228	Chowol	2014	South Korea	road/railway or railway	210	55 + 100 + 55	100.00	55.00	2	13.50		2	rigid	5.50	4.00	17.60	39.10	fan	4.3
229	Jeongjigoga	2012	South Korea	road/railway or railway	180	50 + 80 + 50	80.00	50.00	2	13.02		2	bearing	5.00	3.50	12.00	31.20	fan	6
230	Teror Viaduct	2010	Spain	road	261	62 + 145 + 54	145.00	58.00	2	13.00	11.2	2	rigid	5.08	2.58	16.0	51.0	fan	6
231	Deba River Bridge	2003	Spain	road	150	42 + 66 + 42	66.00	42.00	2	13.90	10.8	2	bearing	2.70	2.70	3.20	22.8	fan	4
232	Principe de Viana Bridge	2010	Spain	road	161	86 + 75	86.00	75.00	1	21.20	19.57	2	rigid	2.50	2.50		38.60	fan	
233	L-9		Spain	road/railway or railway	274.3		65.00			12.70		2		2.22	2.22				
234	Bridge over the Guadalquivir River in Cordoba	2005	Spain		444	114 + 90	114.00	90.00	1	29	29	1	rigid	2.30	2.30	28.00	40.00	fan	
235	Sunnibergbrücke	1998	Switzerland	road	526	59 + 128 + 140 + 134 + 65	140.00	62.00	3 +	12.38						15.00	77.00		
236	Ganter Bridge	1980	Switzerland	road	428.00	127 + 174 + 127	174.00	127.00	2	10				5	2.50				
237	Third Mekong River Bridge between Laos and Thailand		Thailand	road															
238	Chao Phraya River Crossing Bridge	2012	Thailand	road	460		200.00			32.40									
239	Barton Creek Bridge	1987	USA	road	209.10	47.6 + 103.6 + 57.9	103.60	52.75	2	17.70				10.70	3.70				
240	Pearl Harbor Memorial Bridge	2012	USA	road	308.80	75.9 + 157 + 75.9	157.00	75.90	2	33.70				5	3.50				
241	St Croix Bridge	2017	USA	road	938	103.6 + 4 × 182.8 + 103.6	182.80	103.60	3 +	30		2	rigid	5.50	5.50	20.00	65.00	harp	7.0

Table 1 Design data of extradosed bridges in Japan

Chapter

9

List of Symbols

Nomenclature

CSB	Cable-Stayed Bridge
EDB	Extradosed Bridge
GUTS	Guaranteed Ultimate Tensile Strength of steel
HDPE	High Density Polyethylene
MPa	Mega-Pascal, 1 MPa = 1 N/mm^2
MTE	Main Tension Element
PE	Polyethylene
SLS	Serviceability Limit State
ULS	Ultimate Limit State
β	Ratio of the vertical component of the forces in the stays due to permanent load of the deck to the total permanent load of the deck
$\Delta\sigma_L$	Stress change of stay cable due to live load
σ	Stress

Chapter

10

List of Tables

Table 2.1 Advantages and disadvantages of lateral and central position of the pylons 11

Table 2.2 Advantages and disadvantages of harp and semi-fan patterns 22

Table 3.1 Live load and dynamic characteristics of cable types 35

Table 5.1 Type of anchorages in pylon 57

Table 7.1 Statistics of construction cost of Extradosed Bridges around the world based on bridge deck surface area. Value in US dollar at its present value in 2012.[69] 90

Table 7.2 Statistics of material quantities of Extradosed Bridges collected for this report. 92

Table 7.3 Typical range of material consumption for balanced cantilever concrete bridges 95

Table 8.1 Mean values for slenderness at centre of main span (L/H_2) in case of different main spans 104

Table 8.2 Mean values for slenderness at the pier (L/H_1) in case of different main spans... 105

Chapter

11

List of Figures

Figure 1.1 Typical dimensions of the three bridges types[6] 4
Figure 2.1 Yumekake Bridge[16] 10
Figure 2.2 North Arm Vancouver Bridge[17] 10
Figure 2.3 Shin-Karato Bridge[19] 11
Figure 2.4 Vidin-Calafat Bridge cross section[21] 12
Figure 2.5 Warta River Bridge[23] 13
Figure 2.6 Shin-Karato Bridge[19] 14
Figure 2.7 Korong Bridge[25] 14
Figure 2.8 Keong An Bridge[26] 15
Figure 2.9 Tsukuhara Bridge[29] 15
Figure 2.10 Miyakodagawa Bridge[30] 16
Figure 2.11 Moolchand Bridge[32] 16
Figure 2.12 Japan-Palau Friendship Bridge[34] 17
Figure 2.13 Ibigawa Bridge[35] 17
Figure 2.14 Dien-Bien-Phu Bridge[36] 17
Figure 2.15 Vidin-Calafat Bridge[21] 18
Figure 2.16 Povazska Bridge[37] 19
Figure 2.17 Sunniberg Bridge cross section[7] 20
Figure 2.18 Ravine des Trois Bassins Bridge[38] 20
Figure 2.19 Agatsumagawa Bridge[24] 21
Figure 2.20 WD22 Bridge near Zgorzelec[39] 21
Figure 2.21 Pylon cross section of North Arm Vancouver Bridge[59] 21
Figure 2.22 Pure fan cable arrangement – Riga Southern Bridge 23
Figure 2.23 Vidin–Calafat Bridge[21] 23
Figure 2.24 Various extradosed cable arrangements 23
Figure 2.25 Okuyama bridge[44] 24
Figure 2.26 Second Agatsumagawa bridge[24] 24
Figure 2.27 Pragati Maidan bridge[46] 25
Figure 3.1 Schematic bending moment diagram during construction 29
Figure 3.2 Schematic bending moment diagram due to dead loads at service 29

Figure 3.3 Primary and total bending moment caused by a continuity pre-stress in the case of a continuous deck without stays and in the case of an Extradosed Bridge 30
Figure 3.4 Bending moment under dead load (Odawara Blueway Bridge) 30
Figure 3.5 Bending moment under dead load (Tsukuhara Bridge)... 31
Figure 3.6 Bending moment load for permanent stage before and after the restressing of stays[47] 32
Figure 3.7 Permanent loads + pre-stressed cable forces in ULS in cable-stayed bridges 33
Figure 3.8 Simplified analysis at ULS for stiff Extradosed Bridges 34
Figure 3.9 Allowable stress of stay cables versus stress change due to live load 36
Figure 3.10 Maximum stress change due to live load versus distribution ratio of vertical load and stress change of stay cable... 36
Figure 3.11 Allowable stress of wire stay cables versus stress change due to live load 37
Figure 3.12 Allowable stress in stays versus variation of tensile stress due to live load. Japanese standards versus SETRA (France) 38
Figure 4.1 Variety of deck cross sections 42
Figure 4.2 Variety of pylon types 43
Figure 4.3 Special pylon solution.. 44
Figure 4.4 Example of inspection path 44
Figure 4.5 Saddle anchorage concept.. 45
Figure 4.6 Composite pylon structure for anchor stay cables (Shin-Meisei Bridge) 45
Figure 4.7 Single plate anchorage (Mukogawa Bridge)... 46
Figure 4.8 Diaphragm anchorage system.. 46
Figure 4.9 Inside view of Himi Bridge 46
Figure 4.10 Diaphragm of stay cable anchorage in butterfly web... 47
Figure 4.11 Pre-stressing bars 47
Figure 4.12 Composite stay and pylon connection 47
Figure 4.13 Cantilevering tendon layout of Extradosed Bridge (plan view of half top slab) ... 48
Figure 4.14 Cantilevering tendon layout of Extradosed Bridge... 48
Figure 4.15 Cantilevering tendon layout of cable-stayed bridge (plan view of half top slab) .. 48
Figure 5.1 Composite anchorage at pylon (Shin-Meisei Bridge)[55] 58
Figure 5.2 Sketch of internal steel link at mast 58
Figure 5.3 DYNA® link curved anchor box with two DYNA Grip® stay cable anchorages.... 59
Figure 5.4 Nonthaburi Bridge 60
Figure 5.5 Odawara—saddle and anchorage.. 61
Figure 5.6 Odawara—shims at mast anchorage... 61
Figure 5.7 Hole section 62
Figure 5.8 Cross section of the saddle 62
Figure 5.9 CAD view 62
Figure 5.10 View of multitube saddles 62
Figure 5.11 Cross section of the saddle 63
Figure 5.12 Saddle system with coupling details-BBR® 63
Figure 5.13 Deviation angle definition.. 64
Figure 5.14 Range stress in accordance with friction coefficient and deviation angle............. 65
Figure 6.1 Pylon and pier-table casting... 68
Figure 6.2 Tower crane for pylon construction.. 69

Figure 6.3 Upper pylon construction 69
Figure 6.4 Erection of prefabricated segments on scaffolding 72
Figure 6.5 Erection of precast segments on scaffolding; all precast segments are lifted at one location by a lifting frame and slid on top of the scaffolding longitudinally 72
Figure 6.6 Concrete deck supported on temporary continuous or discrete propping 73
Figure 6.7 Balance cantilever technique 74
Figure 6.8 Bridge Builder frame is erected on pier table 75
Figure 6.9 Segment 1 and 2 is lifted after rear macalloy bar is prestressed 75
Figure 6.10 Rear portion of Bridge Builder frame is erected 75
Figure 6.11 Lift and Tilt beam arrangement is mounted on Bridge Builder 76
Figure 6.12 Bridge Builder frames are shifted forward and tied down with segment 1 and 2 respectively 76
Figure 6.13 Rail beam and Bridge Builder frames 76
Figure 6.14 Erection of segment 7 (Extradosed Bridge across river Narmada) 77
Figure 6.15 Steel girder erection. (From Zhou, M. B., Bridge Construction, (2): 14–19 (in Chinese), 2000a.) 77
Figure 6.16 Inter-panel steel truss segment erection over pier top. (From Hu, H.Z., Bridge Constr., 3, 1–4, 2007.) 78
Figure 6.17 Whole segment lifting. (From Hu, H.Z., Bridge Constr., 3, 1–4, 2007.) Construction methodology of six lane Extradosed Bridge on river Ganga 78
Figure 6.18 Construction of caisson foundation (sinking and concreting for steining is a staggered process) 79
Figure 6.19 1. Fabrication of reinforcing bar and casting concrete for lower pylon. 2. Installation of the temporary support system for pier table. 3. Construction of pier table and pylon 79
Figure 6.20 1. Transport of the segments by trailer. 2. Installation of the segment in due sequence by derrick crane. 3. Tensioning of the cable and tendon in due sequence. 79
Figure 6.21 1. Lifting of key segment formwork by derrick crane. 2. Lifting the internal movement joint. 3. Construction of the key segment. 4. Installation of the internal movement joint (Needle Beam) 80
Figure 6.22 Installation of the expansion joint. 2. Installation of the wearing surface and road 80
Figure 6.23 Pecs bridges the flat valley of the Szebenyi–river. The straight superstructures were erected by the incremental launching method. The launching nose was supported by cable stays from a temporary tower, located on the deck. 80
Figure 6.24 Cable stay system 81
Figure 6.25 Stay duct weight born by the first strand in the strand by strand tensioning technique 82
Figure 6.26 Cable end connections- Pin Type Joint 82
Figure 6.27 Possible construction method of Extradosed Bridges using temporary stay cables 83
Figure 6.28 Anchorage of the temporary stay cables at the deck and pylon head levels 83
Figure 6.29 Use of temporary stay cables for the construction of Wadi Abdoun Bridge in Jordan 84
Figure 6.30 Form traveller erection arrangement 85

Figure 6.31 First Extradosed Bridge–Odawara Blueway Bridge. 86
Figure 6.32 Narmada Bridge, Bharauch........ 87
Figure 6.33 Bridge span constriction phases........ 88
Figure 6.34 Bridge Builder arrangement on pylon-span........ 88
Figure 6.35 Erection work flow 88
Figure 7.1 Estimated construction cost comparison of Extradosed Bridges and balanced cantilever bridges designed with AASTHO Codes and unit material price of Florida in 2019.[68] 90
Figure 7.2 Construction Unit Cost Index of concrete and Hybrid structures in Japan (Courtesy of Sumitomo Mitsui Construction). 91
Figure 7.3 Span length versus average depth of girder concrete (Courtesy of Sumitomo Mitsui Construction). (red = Extradosed Bridges, green = box girder bridges, blue = cable-stayed bridges). 91
Figure 7.4 Span length versus pre-stressing steel weight.[70] 92
Figure 7.5 Span length versus tendon weight per concrete volume of girders (Courtesy of Sumitomo Mitsui Construction). (red = Extradosed Bridges, green = box girder bridges, blue = cable-stayed bridges) 93
Figure 7.6 Span length versus extradosed stay steel weight per deck surface area. 93
Figure 7.7 Span length versus extradosed stay steel weight per deck unit area.[70] 94
Figure 7.8 Quantities estimate for continuous three-span girder bridges built in balanced cantilever and comparison with estimates provided by Spanish Guidelines for bridge design of the Ministry of Public Works[68] 95
Figure 8.1 Extradosed Bridges built worldwide (as of August 2018) 98
Figure 8.2 Extradosed Bridges built per time periods 98
Figure 8.3 Typical Extradosed Bridge and its geometric properties 99
Figure 8.4 Main spans of EDBs with one pylon 99
Figure 8.5 Main spans of EDBs with two pylons 99
Figure 8.6 Main spans of EDBs with three or more pylons 99
Figure 8.7 Main span to pylon height ratio for EDBs with one pylon 100
Figure 8.8 Main span to pylon height ratio for EDBs with one pylon as a function of main span 100
Figure 8.9 Main span to pylon height ratio for EDBs with two pylons 101
Figure 8.10 Main span to pylon height ratio for EDBs with two pylons as a function of main span 101
Figure 8.11 Main span to pylon height ratio for EDBs with three pylons 101
Figure 8.12 Main span to pylon height ratio for EDBs with three pylons as a function of main span 101
Figure 8.13 Side span to main span ratio for EDBs with one pylon 102
Figure 8.14 Side span to main span ratio for EDBs with two pylons 102
Figure 8.15 Side span to main span ratio for EDBs with three or more pylons 102
Figure 8.16 Side span to main span ratio for EDBs with two pylons as a function of the main span length 103
Figure 8.17 Side span to main span ratio for EDBs with three or more pylons as a function of main span 103
Figure 8.18 Slenderness at centre of main span (L/H_2) as a function of the main span length 104

Figure 8.19 Slenderness at pier (L/H_1) as a function of main span 104
Figure 8.20 Degree of haunching (H_1/H_2 ratio) as a function of main span 105
Figure 8.21 Cable spacing of all Extradosed Bridges 105
Figure 8.22 Cable spacing of all Extradosed Bridges as a function of main span 105

Chapter

12

References

[1] A. Kasuga, “Extradosed Bridges,” in *Bridge Engineering Handbook. Superstructure Design*, Boca Raton, CRC Press, 2014, p. 437–462.

[2] N. R. Hewson, *Prestressed Concrete Bridges: Design and Construction*, 2nd edn, London: ICE, 2012, pp. xiii, 412.

[3] J. Schlaich and H. Scheef, *Concrete Box-girder Bridges*, vol. **1e**, Zürich Switzerland: International Association for Bridge and Structural Engineering, 1982, p. 108.

[4] H. Svensson, *Cable-stayed Bridges*, Berlin: Ernst & Sohn, 2012.

[5] N. J. Gimsing and C. T. Georgakis, *Cable Supported Bridges*, 3rd edn, Chichester: J. Wiley, 2012, pp. viii, 590.

[6] El Shenawy EA, Schlaich M. Stay cable forces under dead load for cable-stayed and extradosed bridges," *IABSE Symp. Rep.*, vol. **101**, no. 13, p. 1–8, 2013.

[7] H. Figi, C. Menn, D. J. Bänziger and A. Bacchetta, "Sunniberg bridge, Klosters, Switzerland," *Struct. Eng. Int.*, vol. **7**, no. 1, p. 6–8, 1997.

[8] SETRA. *Cable Atays: Recommendations of French Interministerial Commission on Prestressing*. Bagneux Cedex, 2002; 197.

[9] Japan Prestressed Concrete Engineering, *Japanese Recommendations - Specifications for Design and Construction of Prestressed Concrete Cable-Stayed Bridges and Extradosed Bridges*. 2009 (English version).

[10] PTI. *DC45.1-12: Recommendations for Stay-Cable Design, Testing, and Installation*, 2012.

[11] J. Mathivat, Évolution récente des ponts en béton précontraint," in *IABSE Reports = Rapports AIPC = IVBH Berichte*, vol. **55**, 1987.

[12] J. Mathivat, *Neuere Tendenzen in der Entwicklung von Spannbetonbrücken*, vol. **41**, Zürich: IVBH, 1988, pp. S.33-S.48.

[13] Kasuga A, Shirono Y, Nishibe G, Okamoto H. Design and construction of the Odawara Port Bridge - the first extradosed prestressed bridge. *FIP International Conferences*, vol. XII, Washington; 1994; 56–62.

[14] T. Kato, N. Terada, Y. Fukunaga and Y. Uehira, "Design and construction of extradosed bridge - Miyakodagawa bridge - in the New Tomei Expressway," *IABSE Symp. Rep.*, vol. **84**, no. 4, p. 9–16, 2001.

[15] A. Grison and J. Tonello, "Le pont de St Rémy de Maurienne sur l’A 43, un parti original: la précontrainte extradossée," *Revue Ouvrages d’art*, vol. **27**, 1997.

[16] H. Akiyama, M. Nakayama and H. Kayanoki, "Extradosed prestressed concrete bridge with high-strength concrete, Japan—Yumekake Bridge," *Struct. Eng. Int.*, vol. **21**, no. 3, p. 366–371, 2011.

[17] A. Griezic, C. Scollard and D. Bergman, "Design of the Canada line extradosed bridge," *IABSE Congress Rep.*, vol. **17**, no. 15, p. 268–269, 2008.

[18] B. C. Roy and D. M. Tassin, "Second Vivekananda Bridge, Kolkata, India," *IABSE Symp. Rep.*, vol. **101**, no. 12, p. 1–9, 2013.

[19] M. Tomita, K. Tei and S. Takashi, "Shin-Karato bridge in Kobe, Japan," *Struct. Eng. Int.*, vol. **9**, no. 2, p. 109–110, 1999.

[20] Kimishima M, Maeda H, Yamasaki K. *Extradosed prestressed concrete bridge in the Philippines – Second Mandaue - Mactan bridge*, Japanese National Report ed., 2012.

[21] F. Florez Llanos, J. I. Diaz De Argote and J. R. Jimenez Aguilar, "Danube Bridge II. Vidin-Calafat. Construction methodology," *IABSE Symp. Rep.*, vol. **102**, no. 17, p. 1882–1887, 2014.

[22] T. Nakamura, "Lao-Nippon bridge, Laos," *Struct. Eng. Int*, vol. **11**, no. 2, p. 104–106, 2001.

[23] A. Berger, M. Lewandowski, A. Nadolny, J. Onysyk, K. Sadowski and P. Hawryszków, "Warta river crossing 130 m of ready construction per week with a crowing achievement – extradosed bridge," *IABSE Congress Rep.*, vol. **17**, no. 1, p. 608–609, 2008.

[24] Higashi R, Tatsuki S, Osato T. Design and construction of the second Agatsumagawa bridge. *IABSE Symposium Report*, vol. 98, no. IABSE-IASS 2011 London Symposium Report, 2011.

[25] J. Becze and J. Barta, "Korong prestressed extradosed bridge," *Struct. Eng. Int.*, vol. **16**, no. 1, p. 28–30, 2006.

[26] Montens S, Akraa M. An extradosed prestressed concrete bridge: The Keong An bridge (Korea). *fib Symposium*, Mumbai, 2014.

[27] D. Bergman, "Development of the St Croix River extradosed bridge form," *IABSE Symp. Rep.*, vol. **102**, no. 17, p. 1888–1895, 2014.

[28] J. E. Chilstrom, *United States Federal Highway Administration and Connecticut Department of Transportation, Extradosed bridge technology in Japan and the new Pearl Harbor Memorial Bridge*, United States: National Government, 2001.

[29] A. Ogawa, T. Matsuda and A. Kasuga, "The Tsukuhara extradosed bridge near Kobe," *Struct. Eng. Int.*, vol. **8**, no. 3, p. 172–173, 1998.

[30] Terada N, Kato T. *Prestressed Concrete Extradosed Bridge – Miyakodagawa Bridge*, Japanese National Report ed.

[31] Tsukamoto A, Okagawa H, Yoshino S, Konishi Y. *Extradosed PC Bridge by Incremental Launching Method – Shikotonikouka Shinkawakadou Bridge*, Japanese National Report ed.

[32] S. Montens, G. Mauris, P. Jain, M. Shahid, A. Mhedden and R. Kataria, "Moolchand extradosed bridge for Delhi metro," *IABSE Congress Rep.*, vol. **18**, no. 2, p. 1890–1897, 2012.

[33] Maeda Y, Imaizumi Y, Kasuga A, Tazoe K. Design and Construction of the Himi Bridge – Extradosed Bridge with Corrugated Steel Web. *fib Congress*, Osaka, 2002; 95–100.

[34] Oshima H, Suzuki N, Kashiwamura T, Oda I. *3-Span Extradosed Bridge – Japan Palau Friendship bridge*, Japanese National Report ed.

[35] A. Ogawa and A. Kasuga, *Extradosed Bridges in Japan: FIP Notes*, Lausanne: Internat. Federation for Structural Concrete, 1990.

[36] M. Muls, P. Eschard and G. Mauris, "Première ligne de métro de Ho-Chi-Minh-Ville au Viet-Nam," *Revue Travaux*, **909**, 2014.

[37] J. Strasky, M. Matascik, R. Novak and K. Taborska, "Urban Viaduct in Povazska Bystrica, Slovakia," *Struct. Eng. Int.*, vol. **21**, no. 3, p. 356–359, 2011.

[38] J. Frappart, "L'ouvrage d'art exceptionnel de franchissement de la Ravine des Trois Bassins," *Revue Travaux*, **823**, no. 2005.

[39] Biliszczuk J, Onysyk J. Pioneering concrete cable-stayed bridges in Poland. *fib symposium*, Tel Aviv, 2013.

[40] W. Eilzer, R. Lutz, V. Priebe, V. Angelmaier and R. Stockmann, "Neubau der Waschmühltalbrücke - Harmonische Symbiose Baudenkmal-Neubau," *Stahlbau*, vol. **79**, no. 5, p. 370–388, 2010.

[41] Gutmanis A. Latvian link. In *Bridge Design & Engineering*, vol. 16, no. 59, 2010; 43–44.

[42] Mermigas KK. *Behaviour and Design of Extradosed Bridges*. Master Thesis, University of Toronto, Toronto, 2009.

[43] Menn C, Rigendinger H, Ganterbrücke, *Schweizer Ingenieur und Architekt*, Vol. **97**, 1979.

[44] NSE GROUP JAPAN, *Okuyama viaduct, booklet* (Japanese version).

[45] Kobayashi H, Takaki Y, Kobayashi H, Koshiishi M. *The first PPC fin-back bridge for railway – Narusegawa bridge*, Japanese National Report ed.

[46] Gauthier Y. Pragati Maidan Bridge – First extradosed LRT Bridge in India. In *Future Technology for Concrete Segmental Bridges*, San Francisco, 2008.

[47] El Shenawy EA, *Form finding for cable-stayed and extradosed bridges*. Dissertation, TU Berlin, Berlin, 2013.

[48] Kutsuna Y, Kasuga A, Morohashi A. Non-linear behavior of the Ibi River Bridge under ultimate loads. In *First fib Congress, Concrete Structures in the XXI Century*, Osaka, Japan. 2002.

[49] E. d. S. Caetano, *Cable Vibrations in Cable-stayed Bridges*, vol. **9**, Zürich: IABSE, 2007, p. 188.

[50] fib Bulletin 89. *Acceptance of Stay Cable Systems Using Prestressing Steels, (Supersedes fib bulletin 30). Recommendation*, Lausanne: Internat. Federation for Structural Concrete, 2019.

[51] *Eurocode 3: Design of steel structures - Part 1-11: Design of structures with tension components*.

[52] Y. Gauthier, S. Montens, R. Kataria and A. Touat, "Pushing the limits of U shaped viaducts," *IABSE Symp. Rep.*, vol. **93**, no. 21, p. 36–43, 2007.

[53] J. Biliszczuk and J. Onysyk, "Examples of Polish extradosed bridges – design and construction," *IABSE Congress Rep.*, vol. **18**, no. 1, 2016.

[54] V. Markelj, "The first extradosed bridge in Slovenia," *Struct. Eng. Int.*, vol. **20**, no. 4, p. 462–467, 2010.

[55] Kasuga A. Extradosed Bridges in Japan. In *Future Technology for Concrete Segmental Bridges*, San Francisco, 2008.

[56] E. Mellier, "State-of-the-art extradosed cable technology," *IABSE Symp. Rep.*, vol. **101**, no. 12, p. 1–8, 2013.

[57] Brand W, Märzluft A. Fire Protection For Stay Cables. In *fib Congress*, Mumbai, 2014.

[58] P. Charlon and J. Baumgartner, "Protection au feu des câbles extradossés et des haubans (Fire protection of curved cables and câble stays)," *Revue Travaux*, vol. **843**, p. 30–36, 2007.

[59] Fujita M, Umezu K, Arai H, Ueda T. Fretting Fatigue Characteristics in the Curved Layout of Large-capacity Prestressing Strands. In *U.S. -Japan Bridge Engineering Workshop*, November, 1999; 205–222.

[60] S. Mohareb, A. Goldack and M. Schlaich, "Simple model for contact stress of strands bent over circular saddles," *IABSE Congress Rep.*, vol. **18**, no. 1, p. 2016.

[61] Schlaich M, Abdalsamad A, Annan R, Fatigue and Tensile Tests of a 55 Strands Saddle System. In *fib Congress*, Washington, 2010.

[62] Mohareb S, Goldack A, Schlaich M, Walbridge S. Effect of Relative Displacement of Strands Bent over Circular Saddles on Fatigue Life under Fretting Conditions. In *fib symposium*, Mumbai, 2017.

[63] VSL. VSL Strand post-tensioning systems, 2018. Available from: http://www.vsl.com/-brochures/post-tensioning-strand-systems.html (Accessed 6 March 2019).

[64] J. A. Lozano-Galant, X. U. Dong, I. Payá-Zaforteza and J. Turmo, "Direct simulation of the tensioning process of cable-stayed bridges," *Comp. Struct.*, vol. **121**, p. 64–75, 2013.

[65] J. A. Lozano-Galant, I. Payá-Zaforteza, D. Xu and J. Turmo, "Forward algorithm for the construction control of cable-stayed bridges built on temporary supports," *Eng. Struct.*, vol. **40**, p. 119–130, 2012.

[66] J. A. Lozano-Galant, I. Payá-Zaforteza, D. Xu and J. Turmo, "Analysis of the construction process of cable-stayed bridges built on temporary supports," *Eng. Struct.*, vol. **40**, p. 95–106, 2012.

[67] Dywidag Systems International. *Stay cable systems - refernce list, internal data*, not published, 2018.

[68] J. A. Sobrino, "Cost-effectiveness of balanced cantilever girder bridges versus cable supported bridges," *IABSE Symp. Rep.*, vol. **101**, no. 5, p. 1–8, 2013.

[69] Hu J, Kim YJ. *Synthesis on cost-effectiveness of extradosed bridges: Technical Report*, Texas State University, San Marcos.

[70] Patiño O. *Estudio del potencial técnico y de costos de puentes extradosados para vías de doble calzada en Colombia*. Master Thesis, Pontificia Universidad Javeriana, Bogota, 2015.

[71] Villegas Felix NF. *Herramienta para el pre-dimensionamento y estimacion de cantidades del tablero para puentes por voladizos sucesivos*. Diploma Thesis, Universidad de los Andes, Bogota, 2016.

[72] SETRA, *Prestressed Concrete Bridges Built Using the Cantilever Method*, 2003.

[73] T. Vogel and K. Schellenberg, "The impact of the Sunniberg bridge on structural engineering, Switzerland," *Struct. Eng. Int.*, vol. **25**, no. 4, p. 381–388, 2015.

[74] Schueller M, Singh PR. Design and Construction of the Deh Cho Bridge. In *Conference of the Association of Canada Fredericton*, New Brunswick, 2012.

[75] A. Kasuga, "Extradosed bridges in Japan,"*Struct. Concr. J.*, no. **3**, p. 91–103, 2006.

[76] J. Biliszczuk, J. Onysyk, W. Barcik, R. Toczkiewicz and A. Tukendorf, "Extradosed bridges in Poland—design and construction," *Front. Built Environ.*, vol. **2**, p. 285, 2017.

[77] Biliszczuk J, Teichgraeber M, Toczkiewicz R. Large bridges recently built in Poland. In *IABSE Conference*, Kuala Lumpur, Malaysia, 2018.

[78] J. Manterola, A. Martínez, J. A. Navarro, J. L. Álvarez and J. I. Díaz De Argote, "Construction and design features of the Bridge over the Danube River. Bulgaria," *IABSE Symp. Rep.*, vol. **102**, no. 21, p. 1216–1220, 2014.

[79] Strasky J, Matascik M, Novak R, Taborska K. Multi-span extradosed viaduct in Povazska Bystrica, Slovakia. In *fib Congress*, Washington, 2010.

[80] Stroh SL. *On the Development of the Extradosed Bridge Concept*. Dissertation, University of South Florida, 2012.

[81] Collings D, Gonzalez AS. Extradosed and cable-stayed bridges, exploring the boundaries. In *Proceedings of the Institution of Civil Engineers - Bridge Engineering*, vol. 166, no. 4, 2013; 231–239.

[82] Manterola J, Montero J, Fernàndez S. Puente Príncipe de Viana sobre el río Segre en Lleida. In *V Congreso ACHE*, Barcelona, 2011.

[83] Manterola J, Martínez A, Muñoz-Rojas J. The bridge over the Ebro in Zaragoza and The bridge over the Guadalquivir in Córdoba. In *L'Industria Italiana del Cemento*, 2005.

[84] J. A. Sobrino, "Trillizos bridges (The Triplets) in La Paz, Bolivia," *IABSE Symp. Rep.*, vol. **97**, no. 22, p. 63–70, 2010.

[85] Sobrino JA. Design and construction of three Extradosed Bridges in La Paz (Bolivia). In *Proceedings of the International Bridge Conference*, Pittsburgh, PA, US, 2011.

[86] J. A. Llombart, J. Revoltós and J. Fernández, "Project design and construction of the Teror Viaduct in Gran Canaria (Spain)," *IABSE Symp. Rep.*, vol. **102**, no. 18, p. 1825–1829, 2014.

[87] Griezic A, Scollard C, Bergman D. Design of the Canada Line Extradosed Transit Bridge. In *Seventh International Conference on Shorty and Medium Span Bridges*, 2006.

[88] Sánchez JJ. The bridge over the River Erne. In *PCI Convention and National Bridge Conference*, Grapevine, Texas (USA), 2013.

[89] Sánchez JJ. Bridge over the river Erne. In *fib International congress (Bombay*), vol I, 2014; 373–376.

[90] Astin D. Design of the Third Karnaphuli Bridge. In *Proceedings of the IABSE-JSCE Joint Conference on Advances in Bridge Engineering-II*, Dhaka, Bangladesh, 2010.

[91] Bandyopadhyay and Mitra. Third Karnaphuli Bridge in Bangladesh – Design & Construction. In *IABSE Symposium*, Kolkata, 2013.

[92] Western K, Bergman D, Alca N. Design of the St Croix Extradosed Bridge. In *International Bridge Conference*, 2014.

[93] Manterola J. The Vidin (Bulgaria) – Calafat (Romania) Bridge over the Danube. In *Multi span Large Bridges Congress*, Oporto, 2015.

[94] Llombart JA. Extradosed Bridge over the river Deba. In *Hormigón y Acero*, vol. 231, 2004.

[95] Hirano M, Ikeda H, Kasuga A, Komatsu H. Composite Extradosed Bridge. In *fib Symposium*, Prague, 1999; 661–666.

[96] Mizuno K, Samizo J, Kasuga A. Design and Construction of the Mukogawa Bridge. In *Proceeding of fib Symposium*, 2015.

[97] J. A. Torroja Cavanillas, José Ma De Villar Luengo, J. Gamino Palomo and Á. Mazariegos Borobio, "Origen e historia de los puentes extradosados," *Jornada Técnica Anual de ACHE*, no. **11** 2015.

[98] K. Takami and S. Hamada, "Long-time behavior of extradosed composite bridges," *IABSE Symp. Rep.*, vol. **84**, no. 11, p. 57–64, 2001.

[99] Sumida T, Makita J, Arai H, Saito K. Rebar cage application to cast-in-place box-girder, Japanese National Report ed.

[100] Stroh SL. *On The Development Of The Extradosed Bridge Concept*, University of South Florida, Tampa, Florida.

[101] S. L. Stroh, W. R. Stark and J. E. Chilstrom, "First of its kind: the crossing that has been designed to replace the Pearl Harbor Memorial Bridge, in New Haven, Connecticut, has the appearance of a cable-stayed bridge with unusually short towers and a deep deck," *Civil Eng.*, vol. **73**, no. 8 (2003), pp. 54-59, 87, 2003.

[102] Stroh SL. *Extradosed Prestressed Bridges*, 2008.

[103] Sogabe M, Watanabe T, Goto K, Tokunaga M, Kanamori M, Tamai S. *Performance Verification for Railway Extradosed Bridges by Dynamic Interaction Analysis*, 2014; 1203–1210.

[104] Sobrino JA. Comparison of long-span girder bridges versus extradosed bridges built in balanced cantilever (in French). In *Proceedings of the International Bridge Conference*, Pittsburgh, PA, US, 2013.

[105] N. K. Sinha, G. Chattopadhyay and M. Dasgupta, "Extradosed bridges for major river on Vadodara Mumbai Expressway," *IABSE Symp. Rep.*, vol. **101**, no. 12, p. 1–7, 2013.

[106] M. Schueller, "Design and construction of multi-span extradosed bridges," *IABSE Symp. Rep.*, vol. **101**, no. 1, p. 1–10, 2013.

[107] M. Schlaich and E. A. El Shenawy, "Extradosed Brücken - Tragverhalten und Einstellen der Seilkräfte für ständige Lasten," *Bautechnik*, vol. **90**, no. 7, p. 410–420, 2013.

[108] Sasaki H, Ikeda T, Ichinomiya T. *Prestressed concrete extradosed bridge – Shikari-Ohashi bridge*, Japanese National Report ed.

[109] F. Saad, "Structural optimization of extradosed bridges," *IABSE Symp. Rep.*, vol. **88**, no. 6, p. 212–223, 2004.

[110] J. Radic, Z. Savor, M. Pipenbaher and G. Hrelja Kovačević, "New alternative solutions for Pelješac bridge in Croatia," *IABSE Symp. Rep.*, vol. **102**, no. 17, p. 1896–1903, 2014.

[111] B. Pauw and P. van Bogaert, "New continuous 3-span concrete railway decks with extradosed post-tensioning cables in widening an existing multiple arch bridge," *IABSE Symp. Rep.*, vol. **99**, no. 5, p. 1792–1799, 2013.

[112] Ohba M, Koshiishi I. Designing of a bridge over the Natori river, the first PC stayed bridge for railway use in the world. *Japan. Railway Eng.*, vol. **127**, 1994.

[113] A. Ogawa and A. Kasuga, *Extradosed Bridges in Japan*, Lausanne: Internat. Federation for Structural Concrete, 1998.

[114] Mutsuyoshi H, Witchukreangkrai E. *Recent Techniques of Prestressed Concrete Bridges in Japan*, 2004.

[115] S. Montens, Le pont extradossé de Moolchand pour le métro de Delhi," *Revue Travaux*, vol. **879**, 2011.

[116] C. Menn, "An approach to bridge design," *Eng. Struct.*, vol. **13**, no. 2, p. 106–112, 1991.

[117] M. Matascik, R. Novak, J. Strasky and M. Chandoga, "*The "In-town Flyover" in Povazska Bystrica*," 2014.

[118] F. F. Llanos, J. R. J. Aguilar and J. I. D. Argote Cervera, "New bridge over the Danube," *Struct. Eng. Int.*, vol. **24**, no. 3, p. 414–419, 2014.

[119] Liu C, Xu D, Sun Y. Full-scale model test for behaviour of saddle of a extradosed bridge. *IABSE Symposium Report*, vol. 98, no. IABSE-IASS 2011 London Symposium Report, 2011.

[120] H. K. Kim, J. K. Lyoo, Y. K. Ahn and J. K. Cheong, "Geum-gang 1st bridge-extradosed bridge with V-shape pylon," *IABSE Congress Rep.*, vol. **18**, no. 1, p. 2067–2073, 2012.

[121] Kawasaki H, Higa H, Yamauchi A, Ohmura K, *Extradosed bridge with a high tower*, Japanese National Report ed.

[122] Kasuga A, Kimizu T, Matsui Y. Testing of High Damping Rubber Damper for Stay Cables. In *EASEC Proceedings*, vol. 5, no. Building for the 21st Century, 1995; 243–246.

[123] Iida J, Morimoto M, Wakasa T, Kasuga A, Virlogeux M. *Planning and Design of the Shin-Meisei Bridge*, 2004.

[124] J. Iida, H. Nakayama, T. Wakasa, A. Kasuga and K. Mizuno, "*Design and Construction of Shinmeisei Bridge*," 2002; 557–562.

[125] S. Grogan and P. Mahar. L'ouvrage de Pakse au Laos," *Revue Travaux*, vol. **749**, 1999.

[126] Gläser C, Brand W. *Requirements on Tendons for Extradosed Bridges*, 2011; 439–442.

[127] J. Gawlista, M. Kasper, M. Schraml and W. Brand. *DYNA® Link Anchor Box*, 2017.

[128] C. Cremona and J.-C. Foucriat, *Comportement au vent des ponts*, Paris: Presses de l'Ecole nationale des ponts et chaussées, 2002, p. 491.

[129] Chio G. *Comportamiento Estructural y Criterios de Diseño de los Puentes con Pretensado Extradosado*, Universidad Politécnica de Cataluña, Barcelona, 2000.

[130] P. Casteleyn, "Le « Kisosansen project » au Japon, deux viaducs sur les rivières Ibi et Kiso," *Revue Travaux*, vol. **749**, 1999.

[131] Brand W. *Cable Supported Bridges From Cable Stayed in Germany to Extradosed in Slovakia*, 2010.

[132] J. Benjumea, G. Chio and E. Maldonada, "Structural behavior and design criteria of extradosed bridges: general insight and state of the art, Vol 25 N°3, December 2010," *Rev. Ing. Constr.*, vol. **25**, no. 3 (2010), 2010.

[133] Beneitez H, Ramirez S, Areoso L. *Bridge over the River Erne, Ireland*, 2014.

[134] K. Baumann and J. Däniker, "Sunniberg bridge, Klosters, Switzerland," *IABSE Symp. Rep.*, vol. **82**, p. 211–223, 1999.

[135] T. An, O. Kiyomiya and V. Hoang, "Vibration behaviour and seismic performance of a steel cable-stayed bridge with low-rise pylons," *IABSE Symp. Rep.*, vol. **102**, no. 40, p. 356–363, 2014.

[136] Maric Z. *Contest for the Homeland Bridge in Zagreb*, Croatian National Report ed, 1998.

References [97-136] are for information but are not used in the text.

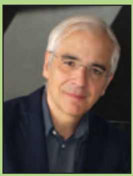
About the Authors

Editor
Mike Schlaich
Germany

Co-ordinator
Andreas Apitz
Germany

Thierry Duclos
France

Akio Kasuga
Japan



José Romo Martín
Spain

Serge Montens
France

Chithambaram Sankaralingam
India

Juan Sobrino
Canada

Extradosed Bridges

Extradosed bridges can be an elegant and economic solution for bridges with spans ranging between 100 and 250m. This novel type of cable-supported bridges has become quite successful in recent years first in Japan and then all over the world.

Experienced members of the international bridge community have come together in Working Commission 3 of IABSE to share their knowledge and to prepare an SED which provides the reader with guidance and practical advise that was not available so far. This book contains useful information regarding conceptual and structural design, analysis, construction, cost and typical properties of Extradosed Bridges.

ISBN 978-3-85748-168-0

9 783857 481680