



Guidelines for the design of footbridges

Guidelines for the design of footbridges

Guide to good practice prepared by
Task Group 1.2

November 2005

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Foreword

The members of *fib* Task-group 1.2 'bridges' (Convener: Klein, Members: Astiz, Bakhoun, Clark, Gauvreau, Kasuga, Schlaich, Strásky, Subba Rao T., Virlogeux, Wilson), during their meeting in July 1999 in Geneva identified footbridges as a subject that deserved special attention by the group. The group decided to start a Working-Party with the aim to develop internationally valid Guidelines for the Design of Footbridges.

Work started by forming a group of internationally acclaimed experts who, by sharing their experience in the field, would contribute to the guidelines. In parallel, several diploma students dedicated their theses to specific footbridge related topics and collected all available information.

From the initial outline of this document its present form took more than five years to complete. This long period can be explained by the authors' other obligations as practitioners and the international membership of the group which required additional coordination efforts. The outcome, however, is up-to-date and constitutes internationally valid guidelines that will assist designers with their decisions regarding footbridges. The ideas that govern the contents of these guidelines are described in Chapter 1.

Stuttgart/Geneva, October 2005

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This document would not have been possible without the work of several diploma students from the universities of Stuttgart and Darmstadt, Germany, namely Carsten Block, Jürgen Gläsle, Michael Müller, Ulrike Schatz and Sven Wörner. Most valuable input came from Chris Rieser of Schlaich Bergermann und Partner as well as Annette Bögle and Achim Bleicher from the Technical University of Berlin who spent considerable time in helping to give the document its final shape and contents. Drawings were prepared by Miguel Paredes.

Finally, it is gratefully noted that Hugo Bachmann, Switzerland looked through the chapter on dynamics.

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1 Introduction

The intention of this document is to present guidelines for the design of footbridges as well as bridges accommodating cyclists and bridleways (horses). The necessity of these guidelines comes from the fact that today, the structural engineer as designer of a footbridge has to spend considerable time and energy collecting information from numerous documents, codes and recommendations to take his design decisions. There seems to be no international document dedicated solely to the design of footbridges.

These guidelines attempt to provide a concentrated source of information regarding all design issues specific to footbridges. They are meant to be a 'liberal' document in the sense that they promote new, innovative and bold yet prudent designs by sharing the experience of the authors; by summarizing what codes do offer and by presenting a collection of examples of well designed structures or structural details from around the world.

These guidelines must not be seen as an international code which presents limits and admissible values leading to designs which are only timid and conservative repetitions of the approved and tested. Perhaps the very fact that no international code exists specifically for footbridges enables the design of the wide variety of ingenious structures that can be found these days. This margin of freedom, which allows for innovative design and nurtures a certain 'joy of engineering', must not be constricted.

There are numerous guidelines, codes and books on bridge design in general. Such information, i.e. information on road bridges that can be applied to footbridges as well, will not be repeated here.

The chapters of these guidelines all follow the same pattern:

- An introduction to the subject, general guidelines as well as do's and don't's.
- A summary of information found in existing international codes, recommendations, experience of the authors, and built examples with comparison and comments on this information.
- Examples.

Plenty of illustrations and photographs help to visualize the themes of this work. The last chapter 'Case Studies' contains footbridges each with a short summary of main structural data and references for further reading.

2 Conceptual Design

Footbridges are walked upon, touched and looked at by the slow moving pedestrian. This means that they are more directly experienced than road and railroad bridges, a fact that certainly influences their design as a whole and even more so in detail. They must be on a human scale.

In comparison to road bridges (and, to a larger extent, railroad bridges), which must usually connect two points in the most direct way, footbridges offer a multitude of possibilities to escape this one-dimensionality. Pedestrian ‘desire lines’ can be reflected in the design and heavily influence the layout of the bridge. Movable bridges, curved and cable-supported decks or the intersection of several decks can generate a spatial experience.

Contrary to road and railway bridges, in footbridge design location, length and elevation of the bridge are not usually given parameters. They should instead be carefully investigated by the designer. Furthermore, bridges can prevent or foster future urban or environmental developments by the way they are inserted in existing surroundings. Concerning such aspects, the contribution of architects can be very helpful.

Footbridges therefore offer the designer a wide variety of design choices. This freedom is due to the following characteristics of footbridges:

- In plan, the deck shape can be formed more freely and with stronger curvature than the deck of road bridges. The deck of a footbridge is also narrow. A width of 3 to 4 m is usually sufficient.
- There are fewer restrictions to the deck inclination (relative to highway/railway bridges), which permits unusual structural types such as stress-ribbons or arches that can be walked upon.
- Besides the usual spectrum of asphalt and concrete surfacing, wood, steel, aluminum or even glass can sometimes be used as pavement.
- The shape and the materials for railings can be chosen more freely.
- The footbridge can be treated differently with regards to loading. Even if local codes specify high live loads, deformations are not usually as limited as they are for bridges that support fast moving vehicles. This allows the design of a much more slender and elegant structure. However, slenderness leads to liveliness and dynamics must be considered.
- New materials and structural types may be easily introduced in footbridges due to their relatively low cost.

Considering the human scale of footbridges, it is important to take into account the needs of the users, passers-by and nearby residents to the design. These may include able-bodied people, the physically handicapped, the elderly, infants and pregnant women. Some issues addressing the relationship between footbridge structures and the users and nearby residents are provided below:

- The effects of vibrations should be considered taking into account the type of user (elderly, handicapped, etc.)
- Users walking on a narrow elevated pathway may feel uncomfortable when seeing vehicles passing below. Special consideration should thus be paid to the perspective of users.

- Rest stations (benches, etc.) can be set up on an elevated pathway when an open space is available.
- Spiral staircases may interfere with the passage of the elderly and physically handicapped.
- Gently sloping stairs are preferable, though steep stairs are generally adopted in congested cities. For very busy urban footbridges, the use of escalators or elevators may be considered.
- Footbridges, when constructed across narrow roads, leave little space in the areas around their entrances. In such cases, alternative plans should be developed – e.g., exploitation of neighbouring alleys or elimination of the entrances by linking footbridges to adjacent buildings.
- When housing is located near footbridges, specific measures should be adopted to avoid overlooking. Likewise, users may feel the eyes of someone below when going up and down the stairs or walking on footbridges. Board screens are commonly used in Japan, but there are many problems to be solved with such a system (fig. 2.1).

The approach to the design of a footbridge depends on the location and the surroundings of the bridge. Each site and each task is different and rarely can the same design fit different contexts. For a bridge in an unspoiled natural environment, the designer will generally attempt to create a light and transparent design. If the designer chooses to create a bolder structure, he wants to create a symbol or landmark in these natural surroundings. In an urban environment, it is often necessary to counteract the mass of the surrounding buildings with a strong and dominant design. On other occasions, it might be advisable to humbly adjust the design to conform to the adjacent cityscape.

Heavily congested urban areas, such as in East Asia, may provide more difficult constraints. Special considerations for this type of setting are provided below:

- At large intersections, horizontal and vertical travel distance should be reduced as much as possible through measures such as installation of multiple ‘entrances’ and links to adjacent buildings.
- During rush hours, a large number of users pass through from mass transit systems (trains) to smaller transit systems (buses, taxis, etc.). The congestion due to this type of traffic can be prevented by widening pathways, reducing vertical travel distances and creating networks of elevated pathways (fig. 2.2).
- Current urban redevelopment projects may be based on a long-term perspective. It is therefore desirable that the aisles of the upper floors of buildings be linked to footbridges to minimize vertical travel distance.
- There is generally a high incidence of traffic accidents at large intersections in densely populated areas. To avoid this, footbridges are often constructed. During the planning of footbridges in such areas thorough consideration should be given to the location of the bridge piers and entrances (fig. 2.3). Alternative access positions may also be provided.
- The locations of entrances are particularly important when constructing footbridges across narrow intersections. In this case, the entrances of adjacent buildings could be used as entrances to footbridge, if the structure is linked to the buildings.
- Extension of pedestrian malls is a preferred option when linking footbridges to adjacent parks.



Fig. 2.1 Board screens on a footbridge in Japan



Fig. 2.2 Pedestrian deck at Station Plaza



Fig. 2.3 Entrance and remaining width on sidewalk

Footbridges show a larger diversity in materials than other bridge types. The superstructure may be made of either steel, concrete, timber, masonry, stainless steel, aluminum alloy or fiber-reinforced plastics among others, or as a composite or hybrid structure that uses a combination of those materials. Materials with which there is little experience or newly developed materials are often tested in the footbridge. Such materials can be introduced more easily for footbridges being that it is easier to convince a client if the structure, and thus related cost, is small. However, all materials to be used must conform to the requirements of the appropriate international standards or the relevant national standards, unless sufficient test results and/or analytical results have already been obtained.

In the choice of materials, the close relationship between the structure and its users as well as the large impact on its surroundings must be taken into account. In the design, the idea of proposing a new regional character or harmony with the existing regional character may be considered, and the aesthetics of individual materials should be utilized sufficiently. For example, appropriate materials should be chosen for different types of structures whether they are free forms with soft curved surfaces, or structural forms in which lightness and slenderness are emphasized. By considering texture, colour and lustre, it is also possible to produce designs with warmth and grace. Designs with an historic atmosphere can also be created through the aging of the materials and surfaces.

Also for footbridges, sustainability is an important issue. The responsible designer should try to use materials of low environmental impact. The application of a highly recyclable material, or the quantitative reductions caused by high-performance materials requires consideration.

The superstructure materials should suit the materials of the substructure and non-structural members. In examining the suitability of a superstructure material with substructure materials, the effect of the structure on the landscape should also be considered. Also the suitability of all non-structural materials needs to be studied carefully considering the users of the bridge.

Who is the designer? Bridges are structures that require profound knowledge of the pertinent engineering concepts. Therefore, it should be a team lead by the engineer and, depending on the case, with the support and advice of an architect. The engineers and architects should start working together at an early stage and try to find a carefully detailed solution that satisfies structural as well as formal requirements. If the engineer or architect works alone and knows nothing of his partner's field, one risks creating a:

- Bridge design based solely on formal 'visions' without consideration of structural function. The structural engineer is called in later to make structural sense of an essentially sculptural design. This leads to changes in form and the necessity of additional structural elements not to mention higher costs.

- Bridge design based solely on utilitarian and cost considerations. The architect is called in later to ‘apply beauty’ to an essentially unimaginative design. Neither approach can lead to a harmonious whole.

Footbridges are permanent structures that, in addition to providing a necessary path connecting two points, may positively contribute to their environment or create a symbol for a place. They may therefore provide structural, social, aesthetic and visual value. These opportunities must be taken into account during the design or one risks creating a structure devoid of inspiration where the only value taken into consideration is minimizing construction cost. The most successful designs arise from the understanding that the true value of a bridge consists of a combination of positive assets in addition to cost.

3 Geometric Conditions

3.1 Introduction

This chapter provides geometric guidelines related to footbridge design. Factors influencing deck width and bridge inclination are discussed as well as the relationship between bridge access and bridge geometry. An overview of the international regulations and a comparison between the codes is provided.

3.2 Deck Width

One of the first decisions to be made when designing a bridge for pedestrians is the deck width, i.e. the capacity of the bridge. The width is dependant on local conditions and the expected density of pedestrians. It may also depend on the location of the footbridge, be it on a trail, in a park or in an urban setting. Cultural setting may also play a role, as the width of Japanese footbridges is usually narrower than bridges built in North America or Europe. The information given in the table below that has been prepared for pedestrian walkways [66] and has proved helpful for the determination of deck width. The sketches in fig. 3.1 define pedestrian density and the corresponding live load.

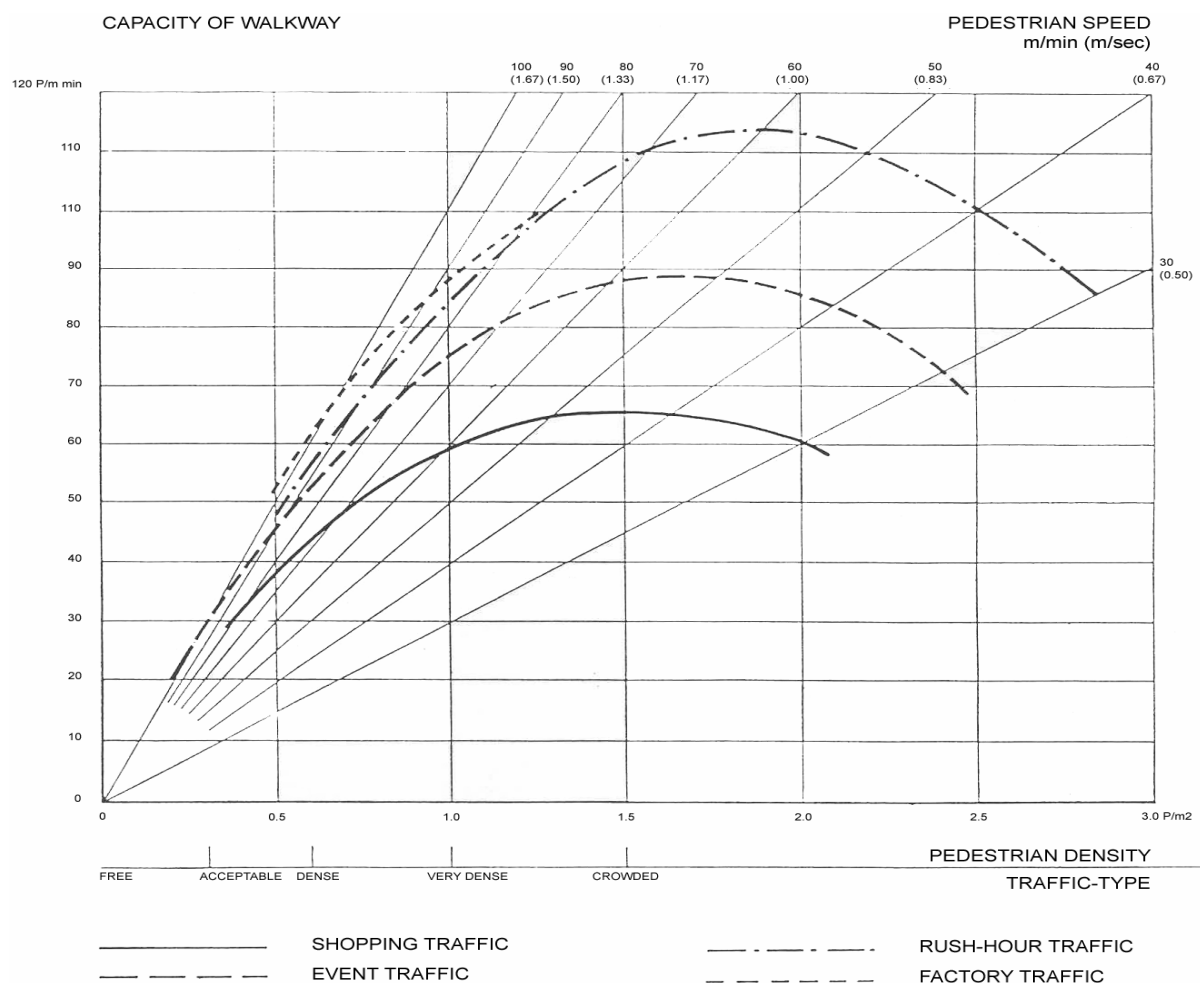


Fig. 3.1 Capacity of pedestrian walkway dependant on traffic type and pedestrian density, see [66]

Fig. 3.1 permits to determine the capacity of a pedestrian walkway (in persons per meter per minute) depending on the type of pedestrian traffic and the acceptable pedestrian density. Note that if the bridge is used also for cyclists the width may need to be increased. If a footbridge is to be used by pedestrians only, general practice suggests a minimum width, W_1 , of 2.50 - 3.00 m. Should a footbridge also be used by cyclists, the minimum width, W_2 , should be at least 3.50 m [87].

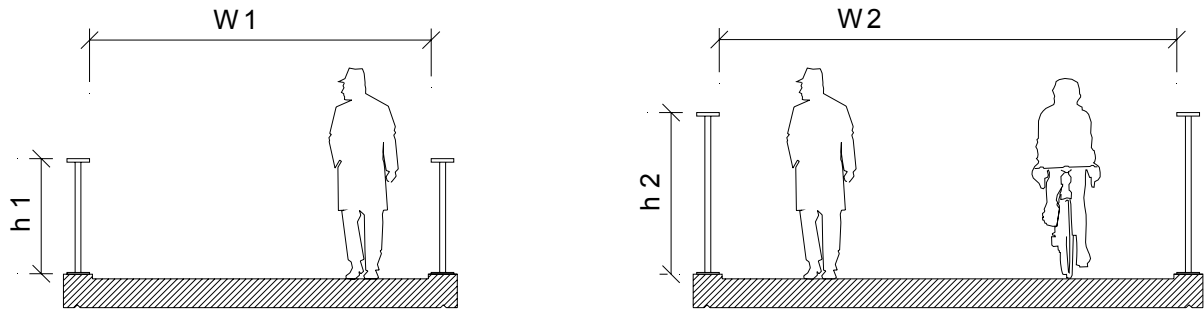


Fig. 3.2 Minimum widths of footbridges [87]

3.3 Grades

In general the grade of the bridge deck can be freely chosen. Bridge grade requirements depend on the location of the footbridge. A bridge on a mountain trail may have slopes of more than 20 % while footbridges in urban settings must accommodate the disabled.

The designer should bear in mind that for wheelchair users grades of more than 6 % are difficult to handle. If steeper slopes or even stairs are necessary, alternative routes should be made available. Table 3.1 gives an overview on typical values for minimum width and maximum inclination of the bridge deck as specified by different national codes.

Bridge inclination and length must be considered together. A slope of 8 % over a length of 5 meters will be easier for wheelchair users to overcome than a slope of 5 % over a length of 200 m. It is therefore the opinion of the authors that the permissible slope should not be determined by the maximum slope at one point in the structure but should be derived from the conditions of the potential energy the disabled must overcome [87].

The height of the deck directly affects the length of the ramps that lead to the bridge. Therefore, in order to avoid long and costly ramps, minimising the deck height should be considered (fig. 3.3). Ramp length is one of the reasons why footbridges are often suspended structures with slender decks.

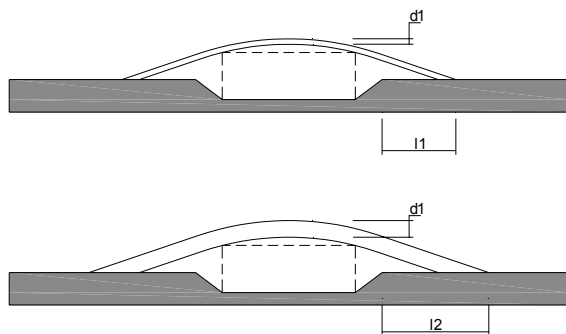


Fig. 3.3 Deck height d and ramp length l

3.4 Stairways

Densely built urban areas often force the bridge designer to provide stairways to provide the necessary roadway clearance. Steep stairways and spiral staircases, while minimizing overall stairway length, are difficult for the disabled and elderly to handle. Elevators and rest stations may often be required to ease use for the elderly and disabled. Providing multiple entrances to elevated footpaths reduces detours for pedestrians but may complicate the design, as does providing a connection to second floors of neighboring buildings.



Fig 3.4 Typical pedestrian footbridge in Japan

3.5 Layouts in Plan

As previously stated, the slow speed of traffic on footbridges leads to a great choice of possible layouts in plan. Among the many options are strongly curved decks, decks that split and ramps which form spirals to shorten their total length. Fig. 3.5 shows the freedom of design and several layouts in plan for footbridges.

This freedom of form calls for special attention to bridge access. Although creating the shortest possible route between two points is a criterion for plan layout, other issues must be taken into account. The ease of access as well as the image of the structure as perceived by the user are important factors. Transitions from ramps or stairways to the bridge itself must be well thought out, allowing for ample space for the user and easy connection between structure and path.

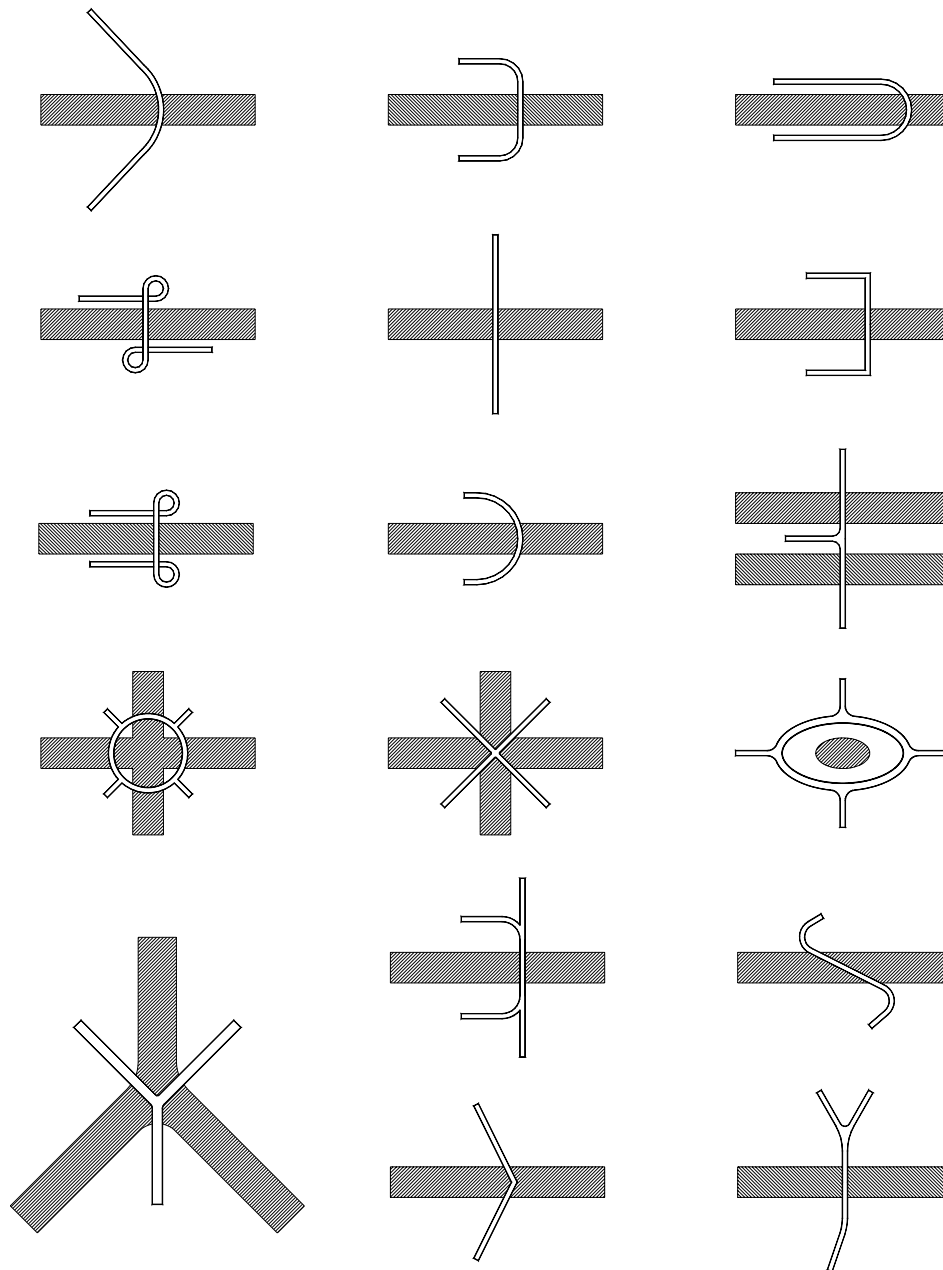


Fig. 3.5 Some of the many possible layouts in plan for footbridges

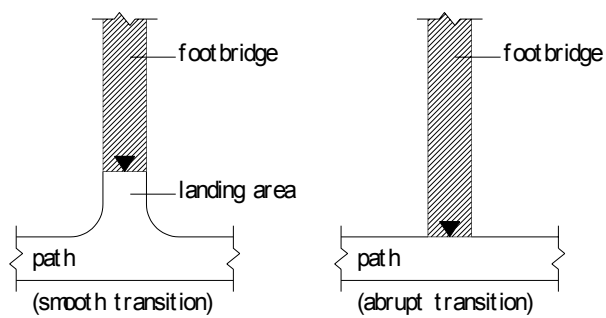


Fig. 3.6 Example of providing landing area for smooth transition from footbridge to path

3.6 Comparison of Codes

The regulation of footbridge geometry in the codes varies from country to country and is summarized in table 3.1. The designer must attempt to fulfill the requirements of aesthetics and economy while at the same time meeting these geometric constraints.

Code / Spec	Country	Min. Deck Width [m]	Clearance [m]	Max. Inclination [%]
Austroads 13, 14, 92	Australia	1.5-1.8 (pedestrians) 1.5-2.0 (cyclists 1 lane) 2.5-3.0 (cyclist 2 lanes) 2.5-3.0 (mixed traffic)	2.1-2.4 (pedestrian) 2.5-3.0 (cyclists)	12.5 (pedestrians) 5.0 (cyclists) 3.0 (mixed traffic)
DIN 18024-1	Germany	1.8 (pedestrians) 2.0 (mixed traffic) 2.7 (pedestrians/cyclists separated)	-	6.0
Structures Design Manual	Hong Kong	2.0 3.0 (at metro stations)	-	5.0-8.3 (pedestrians) 4.0-8.0 (cyclists)
Japanese Footbridge Design Code (1979) [28]	Japan	1.5 Pedestrians 2.0 Cyclist and Wheelchair Users	2.5	12.0
Japanese Footbridge Design Guidelines for Pedestrians (1998)	Japan	3.0	-	5.0
Design specifications of road structures	South Korea	1.5 – 3.0 (pedestrians) 3.0 (cyclists)	2.5	-
BS 5400	United Kingdom	1.8 (pedestrians) 2.0 (mixed traffic) 2.7 (pedestrians / cyclists separated)	-	5.0 –8.3 (pedestrians)

Table 3.1 Geometry in the codes

The Australian Bridge Design Code, **Austroads 92 Pt 1**, General Code gives both required and absolute minimum dimensions for pedestrian and cyclist bridges. Required widths are 1.8 m for exclusively pedestrian paths, 2.0 m for one-lane cyclist paths, and 3.0 m for two lane bike routes with absolute minimum of 1.5 m, 1.5 m, and 2.5 m respectively. The hand rail heights are given as 1.1 m for pedestrians and 1.2 m for cyclist routes, and as 1.0 m and 1.1 m absolute minimum. The maximum diameter of hand railings is given as 50 mm and a maximum distance between railing posts of 130 mm.

In the Australian Guide to Traffic Engineering Practice – **Austroads Pt 13 and 14** regulates the maximum slopes of all pedestrian and cycle paths, and footbridges in particular. For footpaths, 12.5 % is the steepest slope allowed and 5 % is the maximum for cycle paths. If both pedestrians and cyclists are sharing a path, 3.0 % is the maximum slope. Pt 13 and 14 also give minimum heights to be kept free from obstructions or 2.4 m for pedestrians and 3.0 m for cyclists (2.1 m and 2.5 m are absolute minimums).

The German code, the **DIN 18024-1** [9], regulates the geometry of footbridges. The code allows a maximum slope of 3 % for all surfaces carrying pedestrian traffic, which, in the authors' opinion, is too conservative. The code does allow a slope of up to 6 % for slopes with platforms every 10 m with a slope of 3 %. The lateral slope of the surface may not exceed 2 %. The minimum width of a lane is given as 1.5 m and is determined from the needs of the handicapped in wheelchairs. The value is valid for pedestrians and cyclists. Constructions near schools, malls, hospitals, etc. call for a minimum width of 3.0 m. Footpaths and bicycle lanes must be separated by 50 cm. 2.3 m must be kept free from obstructions for both pedestrian and cyclist lanes.

General practice is to allow slopes of up to 6 %. This would prevent the construction of such innovative bridge design types such as walkable arches or stress-ribbon bridges. Alternatives to this requirement should be provided such as locally allowing slopes greater than 6 % to allow flexibility in design.

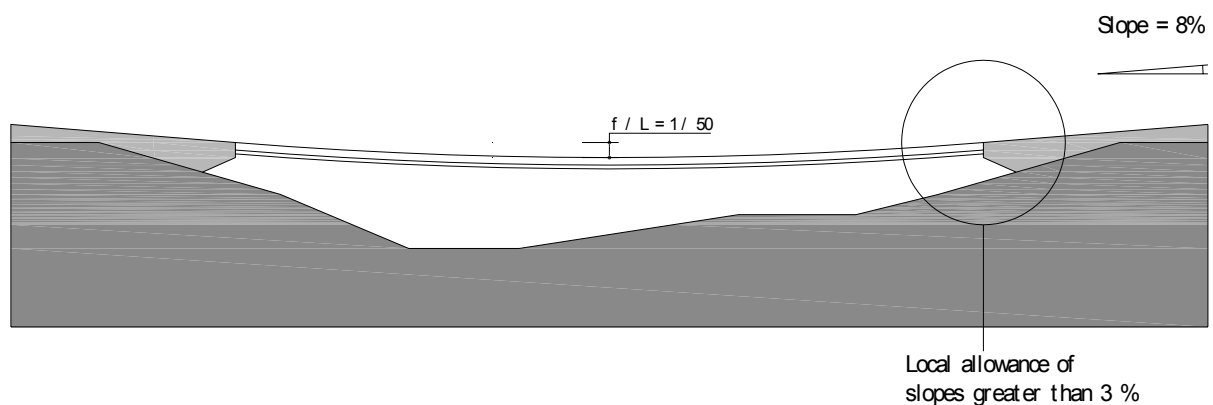


Fig. 3.7 Even with the very small sag to span ratio of 1/50, a stress-ribbon bridge exceeds the slope of 6 %. The author's proposal of locally allowing steeper slopes is depicted here.

The **Structures Design Manuals** [24] for Highways and Railways from the Highway Department Hong Kong Government requires a height of 2.0 m to be kept free from obstruction above all footpaths. The width of footpaths may not fall below 2.0 m. This limit increases to 3.0 m if the footpath is near a metro station.

The Highway Department Hong Kong Government also limits the slope of pedestrian ramps to 8.3 %, which may be increased to 10 % if little space is provided. As in the UK standard, platforms are to be built every 3.5 m in height. Cycle routes should be no steeper than 4 %, generally, and 8 % is allowed only in exceptional cases.

According to the **Japanese Footbridge Design Code (1979)** [28], footbridges are required to have a minimum width of 1.5 m for pedestrians only and 2.0 m for bridges accommodating wheelchair users and cyclists. The clearance is set at 2.5 m. The maximum slope is given at 12 %.

The **Japanese Footbridge Design Guidelines for Pedestrians (1998)** [29] recommends that the deck width be greater than 3.0 m and that the maximum slope be less than 5 %.

In **Spain**, regional governments regulate the accessibility parameters in urban spaces to guarantee the access of disable people. As a summary of these codes, some of the most relevant parameters are as follows:

Maximum grades (p) depend on the length (Lr) of the ramp (not explicitly bridges):

$L_r \leq 3 \text{ m}$	$p_{\text{maximum}} < 12 \%$	$p_{\text{recommended}} = 10 \%$
$3 \text{ m} < L_r \leq 10 \text{ m}$	$p_{\text{maximum}} < 10 \%$	$p_{\text{recommended}} = 8 \%$
$10 < L_r$	$p_{\text{maximum}} < 8 \%$	$p_{\text{recommended}} = 6 \%$

The maximum length of the ramp without landing areas is 20 m. The length of the landing should be at least 1.5 m. The width of the ramps to permit the crossing of two wheelchairs should be at least 2.0 m.

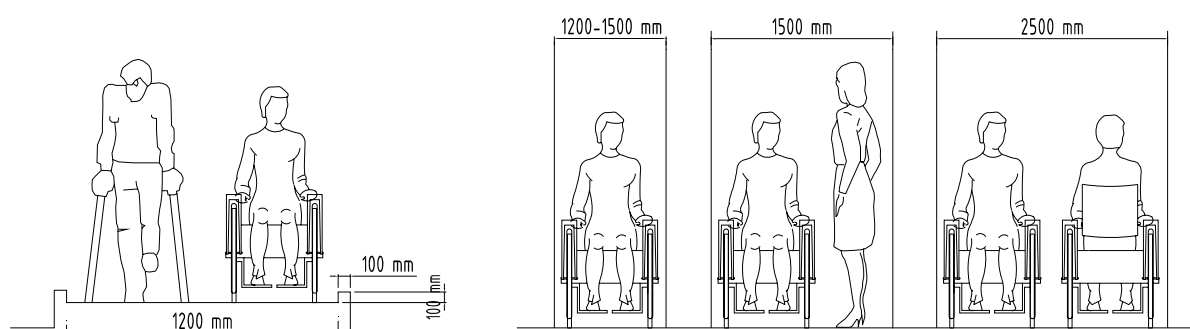


Fig. 3.8 Width of ramps for different types of pedestrian traffic according to Spanish regulation.

The **Design Specifications of Road Structures and Facilities** give the required width of cyclist and pedestrian lanes in South Korea. It stipulates that cycle paths may not be narrower than 3.0 m. By low side clearance, this value may be reduced to 1.5 m. Requirements for walkways vary according to location. In rural areas and along smaller city streets, lane width must exceed of 1.5 m. For main roads the minimum width is 3.0 m and for other important roadways, the width must exceed 2.25 m. Height clearance is set at 2.5 m. The minimum required width and radius for paths is also given in the Design Guide for Roadway Structures and Facilities. These requirements are dependant on the volume of pedestrian and cycle traffic.

The **UK Highway Agency Standard BD 29/03** [23]: Design Criteria for Footbridges regulates the geometry of footbridges. Footbridges are to have a minimum width of 2.0 m. Footbridge widths are to be derived from the peak pedestrian flows and the gradient of the bridge. For a level surface to gradients of 1 in 20, 300 mm of width should be given per 20 persons per minute. For gradients steeper than 1 in 20, 300 mm of width must be provided per 14 persons per minute. If pedestrians and cyclists are separated by markings, the minimum width becomes 2.7 m, see fig. 3.9.

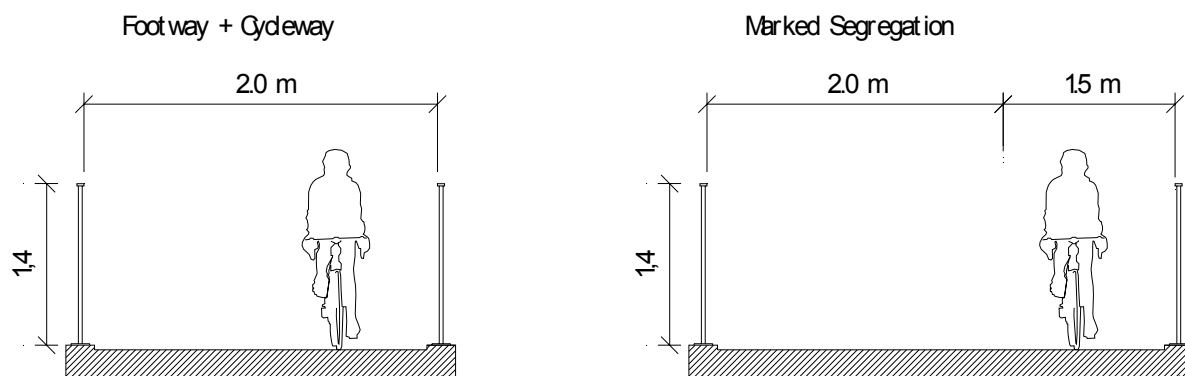


Fig. 3.9 Minimum widths for pedestrian and cyclist routes according to UK standard. The figure to the left is for bridge decks without a marked segregation between cyclists and pedestrians, the figure to the right with a marked segregation.

A height of 5.7 m must be kept free above roadways, and 4.64 m above railways. Bridge piers must be kept 4.5 m from the roadway to avoid any impact. The British Department of Transport advises the use of ramps instead of steps to allow those in wheelchairs to access the bridge deck. A maximum slope of 5 % should not be exceeded. Slope up to 8.3 % are allowed if space for the ramp is limited. Straight ramps must have level platforms every 3.5 m in height. The radius of a spiral ramp may not fall short of 5.5 m and the slope of such a ramp cannot exceed 5 %.

4 Loads

4.1 Introduction

Footbridges are subject to different loads than highway or railroad bridges. Although appropriate loading would seem instinctively lower for footbridges, most codes call for a live load that is comparable to the values for highway bridges. Nevertheless, high wheel loads need not be considered in footbridge design, although consideration should be made to the use of the structure by maintenance or emergency vehicles. The ‘hands-on’ nature of footbridges should also be taken into account and is important for railing loads and vandalism considerations.

4.2 Loads as Function of Deck Length

Surprisingly, international codes specify live loads for footbridges that tend to be as high as those of road bridges or higher. In Germany, for example, the old **DIN 1072** [7] specifies a live load of 5 kN/m^2 for footbridges for the entire deck. For road bridges it prescribes 5 kN/m^2 for the main traffic lane while the other lanes must support only 3 kN/m^2 . Although footbridges are not subjected to the weight of large trucks, with wheel loads of up to 100 kN , the distributed loads are lower for road bridges in the German code. In Switzerland however, small footbridges in rural or alpine settings can be designed for a live load of only 2.5 kN/m^2 in accordance with the owner. A summary of live load in international codes is provided in table 9.2, fig 4.1, and fig 4.2.

Many, but not all, codes allow a reduction of the live load as spans increase. For structures near sports stadiums this reduction should not be used. Other codes hold live load values constant regardless of span, as does the **Dansk Standard**, the **Brazilian NBR-7188** [3], the **Swiss SIA 160** [32] and the **South African SABS 0160-1989** [35]. The **Australian Austroads 92 Pt 2** admits a dependency of the live load values on the surface area of the bridge deck, which is similar to the ‘American Guide Specification for Design of Footbridges’ reduction of live loads by greater deck surface areas.

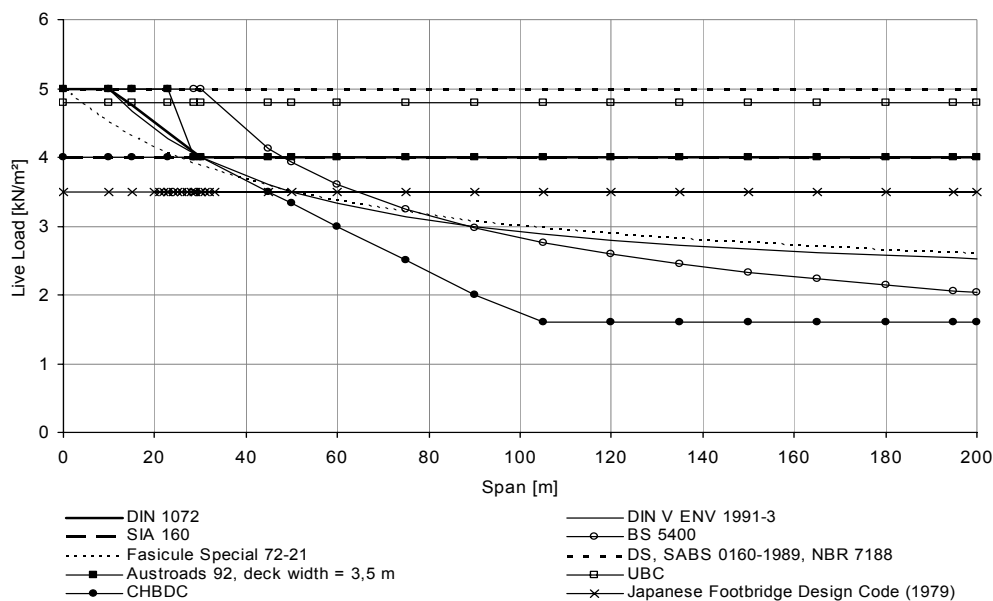


Fig. 4.1 Live load (nominal) as a function of span length for spans of 0-200 m, service load.

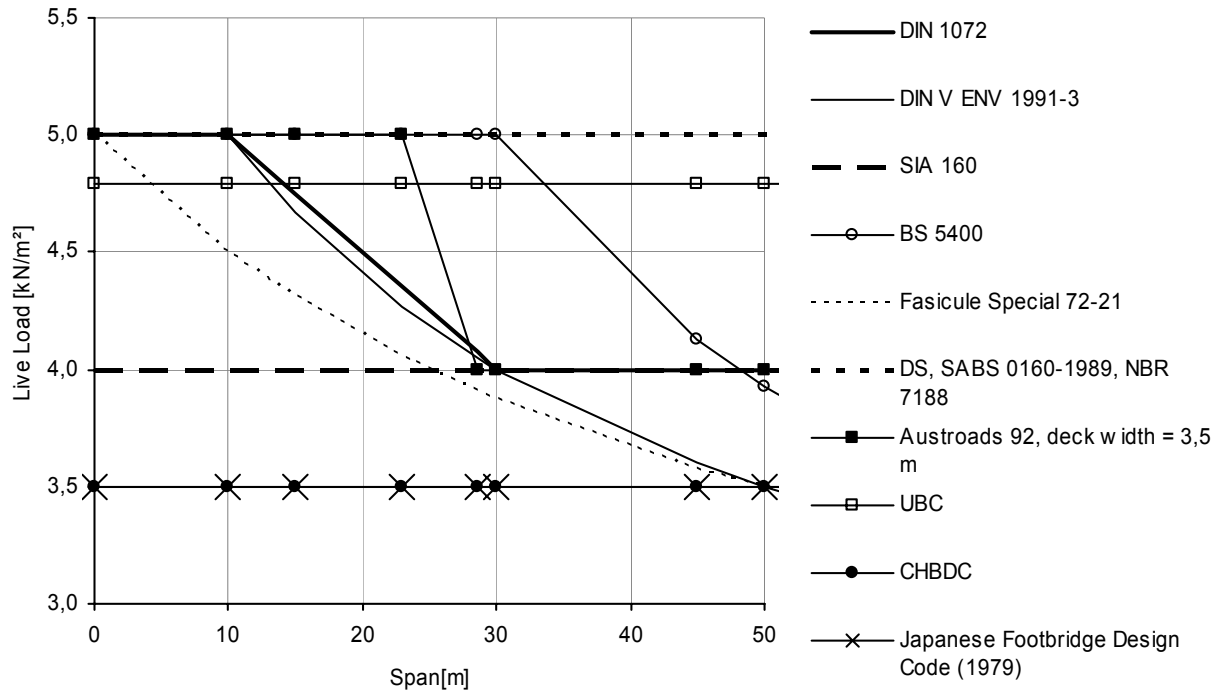


Fig. 4.2 Live load (nominal) as a function of span length for spans of 0-50 m, service load.

4.3 Asymmetric Loading

One-sided live loads are of special concern in footbridge design. Unbalanced loads can occur when numerous spectators stand at the railing to one side of a narrow footbridge during an event. Common practice may lead the designer to apply the full live load to one half of the bridge deck while applying only one half of this value to the other half of the bridge deck. This depends on how, for a given task, the designer interprets codes like the Spanish IAP or Eurocode 1 which state that ‘for each individual application, the models of vertical loads should be applied anywhere within the relevant areas so that the most adverse effect is obtained’. For railing loads see chapter 7.

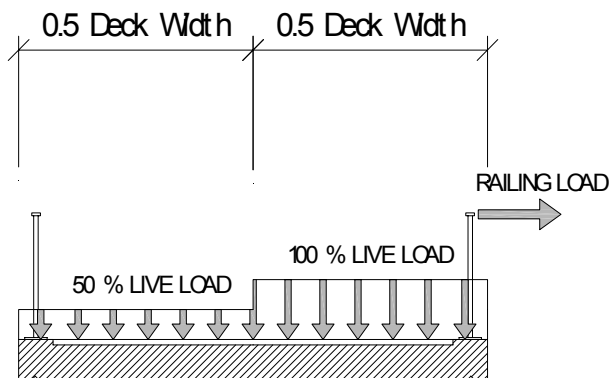


Fig. 4.3 Proposed treatment of one-sided live loads

4.4 Inspection, Emergency, and Cleaning Vehicles

At an early stage the designer should enquire whether emergency or maintenance vehicles would be allowed on the bridge. The use of footbridges by maintenance and emergency vehicles is required by some codes, while others allow the client to decide if such vehicles are permitted on the structure. The distributed load generated by maintenance, emergency, or cleaning vehicles will generally be less than 5 kN/m^2 . Still, wheel loads (for an 8-ton-vehicle around 30 kN) make it necessary to verify that there is no punching of slender concrete decks or local damage of steel decks. For footbridges without any road traffic, some codes require point loads of up to 10 kN to be applied. However, this requirement only poses a problem for very light deck surfaces such as steel gratings.

The **DIN V ENV-1991-3; Eurocode 1, Part 3** [12] and **DIN-Fachbericht 101** [10] examines loads due to inspection, cleaning, or emergency vehicles only if the client specifies that the bridge is to be designed for such vehicles. The load is represented by pairs of forces, each representing an axle of a vehicle. The wheelbase is given as 3.0 m and the distance between wheels is considered as 1.3 m . The force is then spread over the surface generated by a square with 0.2 m sides. The point force of $Q_{\text{fwk}} = 10 \text{ kN}$, normally considered in the live load, is then no longer required to be taken into account.

The **SIA 261** [33] requires the verification of the structure subjected to a force representative of cleaning and inspection vehicles. This force is set equal to $Q_r = 10 \text{ kN}$ and is spread over the surface area formed by a square with 0.1 m sides or a circle with a 0.11 m diameter. For service catwalks, this value is reduced to $Q_r = 2.0 \text{ kN}$.

If higher loads than pedestrians are expected in the **Austroads 92 Pt 2**, such as emergency and maintenance vehicles, then a concentrated force of 20 kN is to be considered in the design.

4.5 Accidental Load and Vandalism Loads

The **DIN V ENV 1991-3; Eurocode 1, Part 3** [12] and **DIN-Fachbericht 101** [10] requires the examination of the results of vehicle impact due to traffic underneath the bridge on bridge piers and supports. The impact load is given as the combination of two forces, 1000 kN in the direction of traffic and 500 kN perpendicular to traffic. These forces are applied in a height of 1.25 m above the road surface.

The **Japanese Footbridge Design Code (1979)** [28] requires that 1000 kN be applied in the direction of traffic or 500 kN perpendicular to the direction of traffic. These forces should be applied at a height of 1.8 m above the roadway.

Due to the hands-on nature of footbridges, vandalism is a particular problem. In critical areas, easily accessible structural members should be protected from potential vandalism. This may also have effects on the choice of material. Graffiti can damage the image of the structure and vandals' access to the bridge structure should be prevented, often for their own safety. In some environments, construction materials with high recycling values, such as aluminum, must be protected from potential thieves.

4.6 Wind Loads

Footbridges are subject to wind actions. Wind loads are particularly important for movable and covered footbridges. In many countries in which wind gusts are a particular problem, a detailed analysis of wind loads is required. Wind occurrence must be examined carefully as it varies greatly from location to location. Flexible, long-span suspended structures may also be subject to wind-induced phenomena such as flutter and galloping.

In the **Canadian Highway Bridge Design Code CSA-S6-00** [6], the reference wind load q is taken from Appendix A 4.1. The Canadian code also examines wind load forces in all three directions in space. The horizontal and vertical wind loads are calculated as follows:

$$F_h = q \cdot C_e \cdot C_g \cdot C_h$$

$$F_v = q \cdot C_e \cdot C_g \cdot C_v$$

For footbridges the value of C_g is given by $C_g = 2.5$. C_e is a modification factor that takes into account the structure's elevation over ground level. It ranges from 1.0 to 1.6 and can be calculated using the following formula:

$$C_e = (0.1 \cdot H)^{0.2}; C_e \geq 1.0$$

where H is the height of the construction above ground level. C_h is to be taken as 2.0 and C_v as 1.0. The height above the bridge deck of the wind surface to be considered for lateral wind loads is taken as 1.50 m for open footbridges.

In the **Dansk Standard**, there is no explicit mention of footbridges in regard to wind loads and wind load values for footbridges are to be taken from those referring to building construction. In the Australian Design Loads Code, wind load values can be taken as the same as those for buildings.

The **DIN V ENV 1991-2-4; Eurocode 1, Part 2-4** [13] considers forces due to wind pressures laterally (y), transversely (x), and vertically (z). The ratio of structure width to height and the height of the construction above the ground determine the wind pressure values which range from 1.2 kN/m² to 4.1 kN/m². These values are to be found in Table N.1 in Appendix N of the code. This basis value of wind pressure is multiplied by a coefficient of force c_f . This coefficient of force consists of a basis coefficient $c_{fx,0}$, which takes into account the shape of the bridge section and a factor $\psi_{\lambda,x}$ takes into account the slenderness of the structure l/b . The coefficients of force in the z direction $c_{f,z}$ and y direction $c_{f,y}$ are to be taken from diagrams in the code.

The French code **Fascicule Special 72-21** [30] makes a distinction for wind loads during and after construction. For structures constructed in less than one month, the wind load to be considered is $w = 1.0$ kN/m², for a construction period over one month, this value increases to $w = 1.25$ kN/m². If the structure is covered during construction, the wind load is given by $w = 2.0$ kN/m². After construction, the wind load to be considered is $w = 2.0$ kN/m², which is applied along the bridge axis.

The **DIN 1072** [7] handles wind loads. Generally, wind pressure is to be applied horizontally, laterally, and transversely. The values for wind pressures are given in table 4 of the code.

The **Japanese Footbridge Design Code (1979)** [28] calls for the effective exposed area of a structure (without live load) to be subject to a wind load of 2.0 kN/m² for windward surfaces and 1.0 kN/m² for leeward surfaces.

The South African **SABS 0160-1989** [35] also makes no explicit reference to bridge constructions, however with a construction width to height ratio from 0.5 to 80, wind pressure coefficients range from $c_f = 1.2$ to 2.0.

The **SIA 261** [33] gives values of 0.9 kN/m² to 1.3 kN/m² for the referential stationary wind pressure, depending on the location of the construction as indicated in the map of appendix E in the code. For exposed hills and mountain peaks this value can increase to 2.4 kN/m² (Jura) and 3.3 kN/m² (Alps). A wind pressure coefficient c_h is multiplied with this reference value to take in account for earth surface roughness and height of the structure above ground. Values for c_h range from 0.75 until 2.35.

Wind loads for all bridges are also determined in the British Standard **BS 5400 Part 2** [4] Paragraph 5.3.

The **Structures Design Manual for Highways and Railways** [24], valid in Hong Kong, uses the same procedure as the British Standard for calculating wind loads. Wind forces are considered in all three directions of space. The stationary wind pressure q can be calculated according to the gust speed, which is modified by coefficients to take into account the height above ground level and the reoccurrence. The Spanish design code IAP uses a similar procedure as the BS 5400, with 4 different types of exposure being defined.

5 Dynamics

5.1 Introduction

To the authors' knowledge, in recent history, dynamic effects have rarely caused any structural failure of a footbridge. A sad exception is the footbridge failure in 1994 in Canton, China [72]. Even though collapse or even damage are quite seldom, effects on the comfort and emotional reactions of pedestrians need to be considered. This is an issue of increasing importance, as modern footbridges tend to become more and more slender and lighter.

This means that during the design the engineer should try to determine the dynamic behaviour of the bridge. Will it be lively? Will it even move or vibrate too much under certain loads? It should be clearly noted that with today's analytical tools these questions cannot yet be answered with total accuracy. Nevertheless, analytical tools can help to estimate the dynamic behaviour in a first step.

There are several effects that can cause a footbridge to oscillate. Pedestrian induced vibrations are the most common one. The type of pedestrian traffic that is to be expected can greatly influence the design of footbridges. Footbridges in more remote locations with sparse pedestrian density are not subject to the same dynamic loading as those structures in city centres with heavy commuter traffic. The type of user is also a factor. For footbridges near hospitals and nursing homes, the user may be more sensitive to dynamic oscillations than in the case of a footbridge along a hiking trail. The general approach for determining such vibrations at the design stage is as follows:

- build a numerical model (or an equivalent single-degree-of-freedom oscillator of the structure) and select realistic damping characteristics.
- choose a load model (single person or group, walking or running)
- analyse the frequencies and the accelerations caused by the chosen load model.
- compare it to the limit values, which correspond the chosen load model.

Several ingredients to the above procedure add considerable uncertainties:

- damping is the most problematic one. The damping behaviour of a footbridge will only be known once it is built and even then it will change with time. Damping depends on many parameters such as chosen materials, complexity of the structure, the type of deck furniture, surfacing, bearing conditions and railings, the mode shape under consideration and even the number of individuals on the bridge.
- there are many possible loads to be applied and many ways to model them. They will all give different results. A single person walking will naturally cause only minimum effects on the bridge while "vandal" load cases can cause tremendous oscillations.
- the limit values should not only depend on the type of load applied, they should also consider the location of the structure and depend on the type of user. Also it makes no sense to limit only frequency or only acceleration. Acceleration limits should depend on the frequency.

The current practice of footbridge design regarding dynamics consists of several steps:

- Determination of the sensitivity of the location, type and intensity of loading, etc.
- Discussions with the owner are held to address possible dynamic oscillations, establish reasonable expectations and limits on the dynamic response, and address the potential use of damping systems.
- A preliminary dynamic calculation is carried out by hand calculation or using a finite element model. This allows the bridge designer to get a rough overview of the dynamic response of the structure and locate possible problems in dynamic behaviour. Due to uncertainties in the dynamic calculations (see above), these results should be taken with a 'grain of salt'. Still, they can lead to initial considerations on possible damping systems and design provisions to counteract the oscillations.
- In case of doubt, provisions for dampers are made. This allows subsequent installation of the dampers on the structure without exceeding stress or stability requirements.
- The real dynamic behaviour of the finished structure is observed. Should a damping system be required, this can easily be carried out.

Several studies¹ are ongoing presently to shed more light on the issues mentioned above. This section summarizes the present status of know-how and what is given in the available codes. The following points will be discussed:

5.2 Pedestrian Traffic and Action

5.3 Pedestrian-Induced Actions and Relevant Models

5.4 Analysis of Accelerations

5.5 Comfort Criteria and Limit Values

5.6 Preventive Measures and Damping

5.7 Dampers

¹ French footbridge guideline;
SYNPEX: Advanced load models for synchronous pedestrian excitation and optimised design guidelines for steel footbridges

5.2 Pedestrian Traffic and Actions

The type of pedestrian traffic acting on the structure influences the dynamic response. At high pedestrian densities, the individual's freedom to choose his pace is restricted. Also, at higher pedestrian densities, group effects may occur, causing many pedestrians to walk at the same frequency of pace as their neighbours. Given the right conditions, the so-called lock-in effect may occur causing many individuals to walk with the resonance frequency of the structure. In this section a classification of pedestrian density will be given and the effects on dynamic load will be discussed.

5.2.1 Pedestrian Density

The pedestrian density greatly influences the speed of the individual and is therefore important for the dynamic analysis. The relation between velocity and pedestrian density is given in the figure 5.1.

The pedestrian density can be calculated according to the following formulas:

$$q = \frac{\lambda}{v_s \cdot b_{eff}} \text{ [Pers/m}^2\text{]} \quad (5.1)$$

$$q = \frac{N_r}{b_{eff} \cdot L} \text{ [Pers/m}^2\text{]} \quad (5.2)$$

where:

λ	=	rate of pedestrian arrival [Pers/s]
v_s	=	velocity of pace [m/s]
b_{eff}	=	effective width [m]
N_r	=	number of pedestrians on bridge deck
L	=	length of bridge [m]

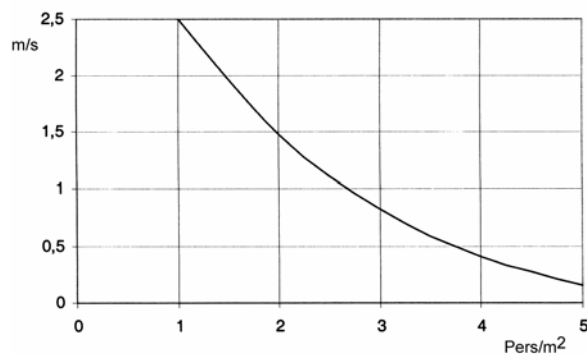


Fig. 5.1 Relation between pedestrian density and velocity [67]

5.2.1.1 Low-Density Pedestrian Stream

The main characteristic for a low-density pedestrian stream is an occupancy of the bridge that does not restrict the freedom of movement. According to Oeding [66], in a pedestrian stream with a density of 0.3 pers/m² to 0.6 pers/m², the pedestrians are able to move undisturbed with their individual step frequency. The time of arrival of the pedestrians on the bridge is random. Hence description of the resulting bridge vibration should take account of the randomness of the pedestrian traffic.

5.2.1.2 Dense Mass Traffic

If the pedestrian density increases, the single pedestrian is no longer able to walk with his individual step frequency and walking velocity. The synchronisation of pedestrians regarding step frequency, phase and velocity takes place that can also occur in combination with the bridge movement.

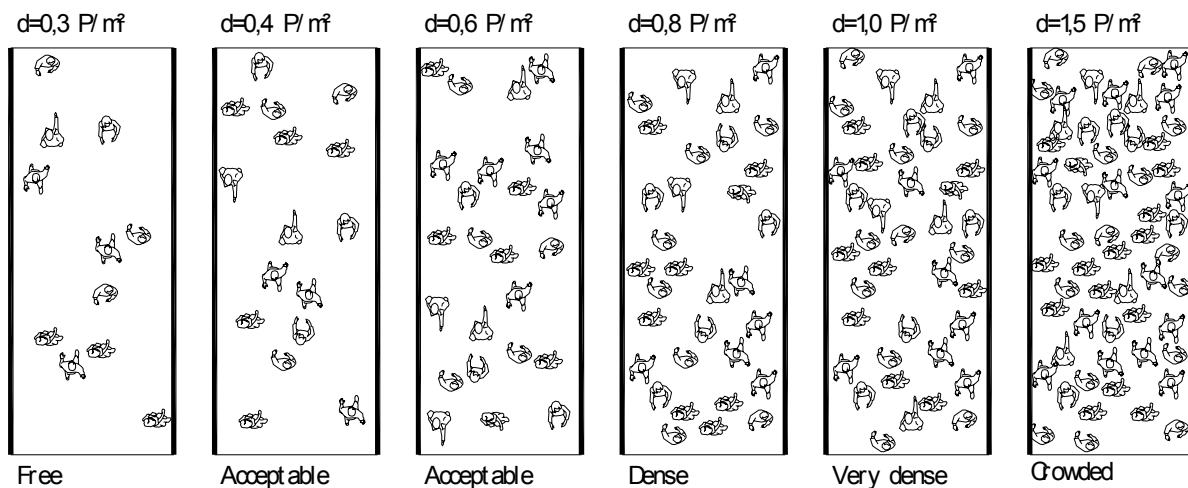


Figure 5.2 Different types of pedestrian densities [66]

In general the walking velocity reduces with increasing traffic density. The single pedestrian has to adjust his walking velocity to the movement of the mass. First restrictions occur at a pedestrian density of 0.6 pers/m² as passing becomes more difficult. From a pedestrian density of 1.0 pers/m² the freedom of movement is greatly inhibited. The pedestrians must adjust their velocities and step frequencies to each other. If the density is about 1.5 pers/m², columns dependent on the direction of walking with a very low velocity develop. 2.0 pers/m² result in a very crowded stream where only a sliding movement with very small steps is possible. The pedestrian is not able to move on his own.

It should also be noted that by dense pedestrian traffic, the modal characteristics of the structure might be changed due to an increase in mass and an increase in damping caused by their vertical movements. Horizontal movements may experience negative damping with the presences of large numbers of pedestrians.

Stage	Pedestrian Density [Pers/m ²]	Characteristics
1	$0 < q < 0.3$	One can walk comfortably and freely
2	$0.3 \leq q < 0.6$	Freedom of movement is intermittently inhibited
3	$0.6 \leq q < 1.0$	Freedom of movement restricted
4	$q \geq 1.0$	Dense crowd, one can no longer freely choose pace

Table 5.1 Classification of pedestrian density according to Oeding [66]

Having seen the effects of pedestrian density on the velocity and freedom of movement, density dependent effects, such as groups of pedestrians and structure – pedestrian interaction, will be discussed.

5.2.2 Actions of Groups of Pedestrians

Although the way of considering the bridge crossing by groups of pedestrians has only recently been defined in the structural engineering codes [14], it is sometimes important to predict its effect at the design stage.

This problem may be treated at three different levels: (i) the passage of small and synchronized groups; (ii) the passage of small groups not synchronized; (iii) the passage of a continuous flow of pedestrians.

Many authors have addressed the issue of loading due to groups of pedestrians (see [46], [53], [57], [58], and [63]). The effects of pedestrian groups only according to Grundmann [54] and Kramer [57] will be addressed here for the sake of brevity. Grundmann has developed a coefficient S on probabilistic considerations to take into account groups of pedestrians for various pedestrian densities on the bridge structure. This coefficient depends on an acceptable acceleration $a = 0.7 \text{ m/s}^2$ according to BS 5400 [4]

$$a_{gr} = S \cdot a \quad (5.3)$$

where: S = factor of synchronization (see fig. 5.3)
 a = horizontal or vertical acceleration due to an individual pedestrian calculated by hand (see chapter 5.4)

5.2.2.1 Bridges with Low Pedestrian Density

For footbridges with an expected maximum of ten or less pedestrians at a time, the factor of synchronization can be found according to the following graph.

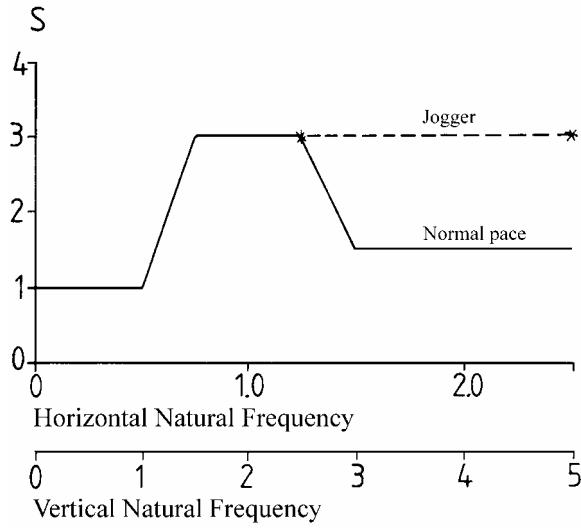


Fig. 5.3 Factor of synchronization for low pedestrian density according to eq. 5.3 [54]

5.2.2.2 Bridges with a Random Stream of Low Density Pedestrian Traffic

If the pedestrian density is low ($q \leq 0.6 \text{ Pers/m}^2$) and pedestrians are able to move freely on the bridge deck, the factor of synchronization can be found according to the following formula for a structure with first vertical frequency between 1.50 Hz and 2.50 Hz:

$$S = 0.225 \cdot N_r \quad (5.4)$$

For frequencies under 1.50 Hz and over 2.50 Hz, the factor can be conservatively estimated as:

$$S = \sqrt{N_r} \quad (5.5)$$

For frequencies between 3.50 Hz and 4.50 Hz:

$$S = 0.225 \cdot 0.5 \cdot N_r \quad (5.6)$$

$$N_r = \lambda \cdot T_c \cdot K = q \cdot L \cdot b_{eff} \cdot K \quad (5.7)$$

where:

N_r	=	number of pedestrians on the bridge
λ	=	rate of pedestrian arrival [Pers/s]
T_c	=	duration of bridge crossing [s]
K	=	weighting factor to take into account the variable point of application for loading (simple beam: $K = 0.6$ according to [54])
L	=	length of bridge [m]
v_s	=	velocity of pedestrian [m/s]
q	=	pedestrian density [Pers/m ²]
b_{eff}	=	effective width of bridge deck [m]

5.2.2.3 Bridges with a Stream of High Density Pedestrian Traffic

For bridges with pedestrian density $q > 0.6$ Pers/m², the factor of synchronization can be calculated according to (5.4) till (5.6).

5.2.2.4 Synchronization to Kramer

For vibration excitations of a bridge due to pedestrians by a time-varying force $F(t)$, Kramer [57] is using a coefficient N_f to take similar effects of synchronization based on tests into account.

$$F_N(t) = N_f \cdot F(t) = N \cdot S \cdot R \cdot F(t) \quad (5.8)$$

where:

$F(t)$	=	time-varying force of one pedestrian (see section 5.3.3.1)
N	=	number of pedestrians on the bridge
S	=	0.275: empirical factor of synchronization (difference between freedom of movement and lockstep)
R	=	0.465: empirical factor of reduction (difference between distributed load to concentrated load)

5.2.3 Lock-In Effect

If vibrations are perceptible on a bridge, some pedestrians will attempt to counteract the vibrations by slight lateral motions to keep their balance (the so-called sailor's walk). This instinctive behaviour causes an adjustment of pedestrian step frequency and phase to coincide with the natural frequency of the bridge. The pedestrian-induced forces will be exactly in resonance with the structure. This is referred to as the lock-in effect.

The lock-in effect increases with the bridge's amplitude of vibration, i.e. the number of pedestrians participating in this corrective motion rises with the amplitude of vibration of the structure [43], [44], [82]. As the bridge vibration increases, the initially random excitation of a group develops into a mass resonant excitation. This increases until a critical number of pedestrians create an unacceptable level of vibration.

The lock-in effect develops more rapidly for lateral vibrations than for vertical vibrations. Small lateral amplitudes are sufficient to throw a pedestrian off balance. A lateral amplitude of 5 mm and a vibration frequency of 1 Hz produce a 40 % probability of pedestrian resonance [41].

Baumann and Bachmann [41] observed that with a vertical amplitude of 10-20 mm and a step frequency close to the vibration frequency a pedestrian is unable to move further with his initial frequency. He adjusts more or less his movements to the movement of the ground. This threshold has not been confirmed experimentally. For lateral vibrations, Bachmann [40] gives a limit displacement of about 2 mm, which was clearly confirmed on Millennium and Solférino footbridges.

For a footbridge with the eigenfrequency of 2 Hz, a vertical displacement of 10-20 mm corresponds to an acceleration of 1.6 m/s², which is very uncomfortable and should therefore be avoided due to comfort criteria. For vertical vibrations, this threshold for lock-in is therefore generally not governing.

In different experiments on the Millennium Bridge [44] the force of the pedestrian dependent on the measured velocity of oscillation of the structure was studied. In fig. 5.4 the correlation between lateral ground reaction force due to the pedestrian and lateral bridge response is shown.

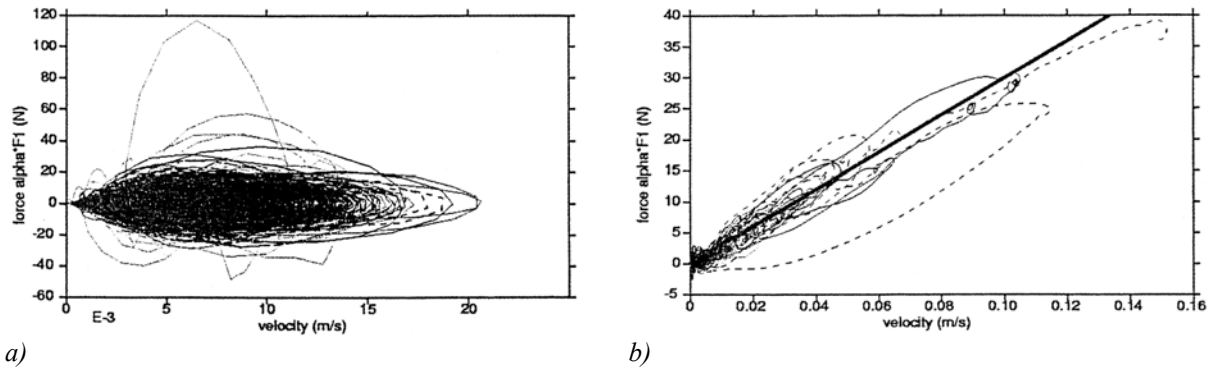


Fig. 5.4 Dependence of pedestrian force on local bridge velocity, a) vertical component, b) lateral component [44], [69]

The linear increase of the lateral force of the pedestrian with the amplitude of bridge response shows that the ground reaction forces of a pedestrian are larger on a vibrating ground than on a stationary surface. As the linear increase of the lateral force of the pedestrian with increasing local bridge velocity accelerates and amplifies the synchronisation, the pedestrian-structure-interaction is a self-driven excitation.

Lock-In Effect According to Findings from the Millennium Footbridge

During investigations on the Millennium Bridge [44], the correlation between lateral force components and local bridge velocity has become evident (see fig. 5.4 b), this can be formulated as:

$$F_{lat} = k \cdot v_{local} \quad (5.9)$$

where:

F_{lat}	=	correlated lateral ground reactions force due to a single person
k	=	300 Ns/m
v_{local}	=	local lateral bridge velocity

The correlation factor k corresponds to a structural damping. But while damping dissipates energy, the pedestrian produces 'negative damping' i.e. he applies energy to the system.

ARUP developed an equation for the approximate determination of the expected value of the acceleration amplitude of the bridge [44]. Based on this analytical investigation of the modal response of the Millennium Bridge during the walking tests ARUP developed an expression for the determination of the number of pedestrians that are necessary for lateral instability. The expression is developed for a sinusoidal mode shape.

$$N_L = \frac{8\pi \cdot c \cdot f \cdot M}{k} \quad (5.10)$$

where:

- N_L = critical number of persons
- c = damping of structure
- f = bridge's natural frequency
- M = modal mass of eigenform (can be determined from numerical model)
- k = coefficient of force (to be determined experimentally: 300 Ns/m lateral for Millennium Footbridge)

The necessary damping factor for prohibiting synchronization can be calculated:

$$\xi > \frac{N_L \cdot k}{8\pi \cdot f \cdot M} \quad (5.11)$$

A correlation between vertical pedestrian force and bridge vibration has not been observed. But it is possible that a correlation exists for higher amplitudes.

5.3 Pedestrian-Induced Actions and Relevant Models

Pedestrian induced loads may be due to walking, running, or jumping, the so-called vandalism loading. Each of these types of loading produces a different loading curve over time as well as frequencies in which the oscillations can occur. The loading due to walking will be examined in detail here with only a short discussion of running and jumping loads.

The frequency range for normal walking is roughly between 1.5 and 2.5 Hz. Ranges of typical step frequencies for running, walking and jumping are given below.

Activity	Group Size N ^{er} of people	Range of activity rate, Hz (Footsteps/s)	
		Normal Range	Measured Range
Pedestrian Movements			
Walking	1, 2 and 4	1.6 – 2.2	1.0 – 3.0
Jogging	1, 2 and 4	2.2 – 3.2	1.6 – 4.0
Rhythmic Exercises			
Jumping	1, 4 and 8	2.0 – 3.0	1.4 – 4.0
Stride Jumps	1, 2 and 4	2.0 – 2.6	1.6 – 3.4
Running-on-the-spot	1, 2 and 4	2.2 – 3.2	1.4 – 4.0

Table 5.2 Group size and range of activity [70]

In spite of sharing the same step frequency, individuals' step length can differ because of different body height and hence leg length. The leg length governs the step length and hence some individuals walk slower or faster than others [82], [92].

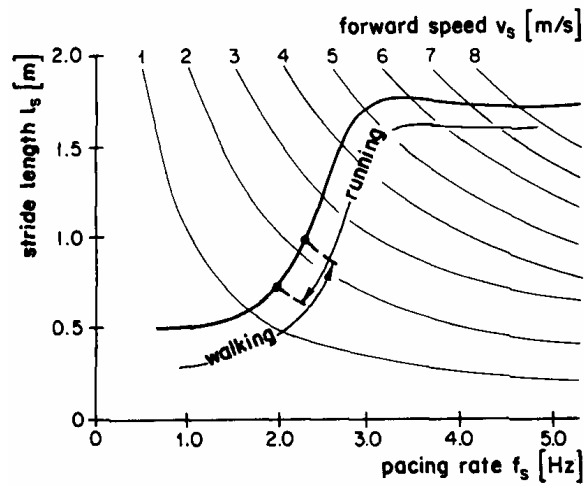


Fig. 5.5 Relation between step frequency, velocity and step length [92]

	f_s	v_s	l_s
	[Hz]	[m/s]	[m]
slow walking	1.7	1.0	0.60
normal walking	2.0	1.5	0.75
fast walking	2.3	2.3	1.00
normal running	2.5	3.1	1.25
fast running	>3.2	5.5	1.75

Table 5.3 Typical values for step frequency, velocity and step length [37]

Fig. 5.5 shows for undisturbed walking and running the relationship between step length, velocity and step frequency. Some relevant values for step frequency, velocity and step length are assembled in table 5.3.

5.3.1 Actions from Walking

The loading induced by walking is a periodic excitation and its intensity mainly depends on the individual's step frequency and body weight. As one foot is always in contact with the ground, the loading does not disappear completely at any time. During the transition period of the motion sequence the body weight is shifted from one foot to the other and the two load curves for each foot overlap. The vertical load component is larger than the horizontal components, but the lateral and longitudinal components can also cause vibration problems of slender bridges especially if a pedestrian-bridge-interaction develops.

5.3.1.1 Vertical Force Component

Normal walking induces a vertical force with a butterfly shape having two dominant force maximums. The first one is caused by the impact of the heel on the ground, while the second one is produced by the push off. The maximums increase with increasing step frequency.

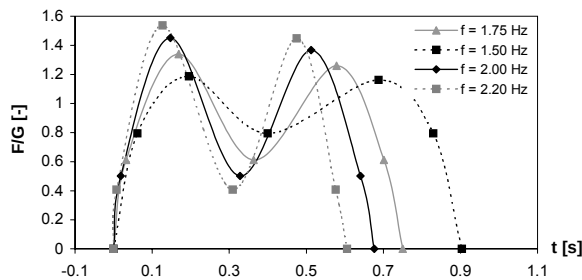


Fig. 5.6 Vertical loads for different walking frequencies [59]

In case of hard soles the heel-strike causes a very sharp peak additional to the described butterfly load curve.

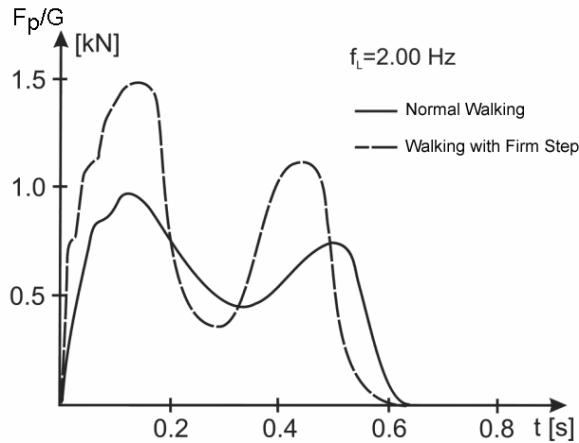


Fig. 5.7 Vertical ground reaction forces: normal walking and walking with firm step [37]

The relationship between step frequency, time period of the ground contact and the load maximums is shown in fig. 5.8.

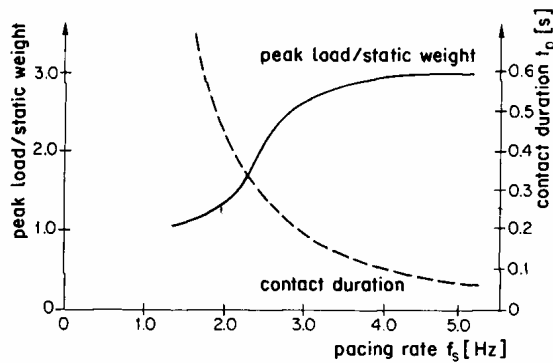


Fig. 5.8 Contact period and ratio maximum load to weight dependent on f_s [92]

In case of very slow walking with a step frequency less than 1 Hz (a long period of ground contact) the dynamic loads are equal to the static load due to the body weight. For fast walking with 2 to 2.5 Hz the dynamic loads rise to 1.5 times the static load. For very fast running (> 3.5 Hz) the maximum is about three times the body weight.

5.3.1.2 Horizontal Force Component

The horizontal force components in longitudinal and lateral direction are much smaller than the vertical component. The longitudinal force (x-direction) is characterised by the retarding and the pushing walking period. The lateral force (y-direction) is caused by the lateral oscillation of the body.

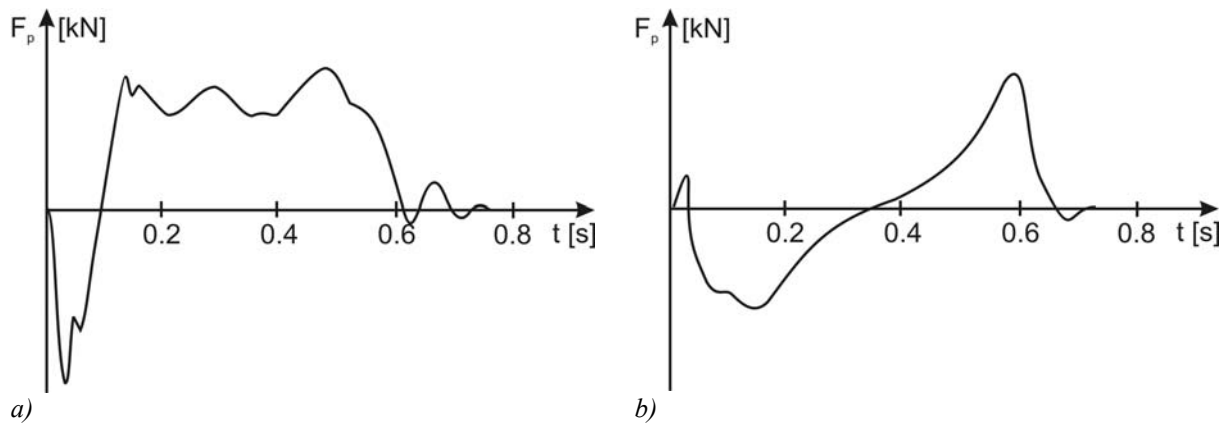


Fig. 5.9 Ground reaction forces from walking: a) lateral, b) longitudinal [82]

Fig. 5.9 shows the time histories of the horizontal ground reaction forces on a non-moving surface. Unlike the vertical force, the horizontal forces are periodic with only half the walking frequency.

5.3.2 Actions from Running and Jumping

Running is characterised by a complete lift-off. Ground contact is interrupted and hence the force is zero. In comparison to walking, running-induced forces depend more on the individual running type and shoes.

5.3.2.1 Vertical Force Component

Whereas the walking induced vertical load is characterised by two load maximums, the running-induced load has only one maximum. It is characterised by a steep increase and decrease. The maximum increases and becomes more narrow with increasing step frequency and hence decreasing contact phase. The heel strike may rise to 3 to 5 times the body weight.

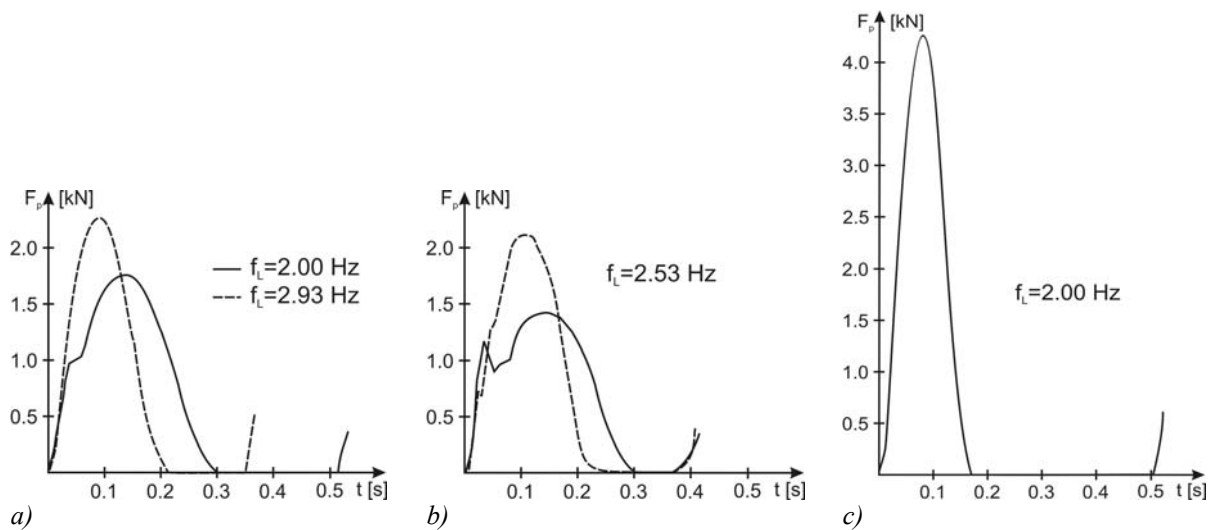


Fig. 5.10 Ground reaction forces: a) running with different frequencies, b) running dependent on running style, c) jumping [37]

For comparison the forces induced by jumping are shown in fig. 5.10. They are similar to the running-induced ones but have higher amplitudes. Jumping is not a normal type of activity on a footbridge but for systematic excitation (vandalism) it should be taken into account. Jumping induced vibrations provide a wider frequency of excitation. The frequencies fall between 1 Hz and 3.5 Hz for jumping loads.

5.3.2.2 Horizontal Force Component

According to Schneider [82] due to the larger lateral stability during running the lateral force component is half as large as during walking. Also the risk of lock-in is negligible as the link between pedestrian and structure is less for running in comparison to walking. Like walking, the lateral ground reaction force is periodic with half of the step frequency, as the force changes direction with each step.

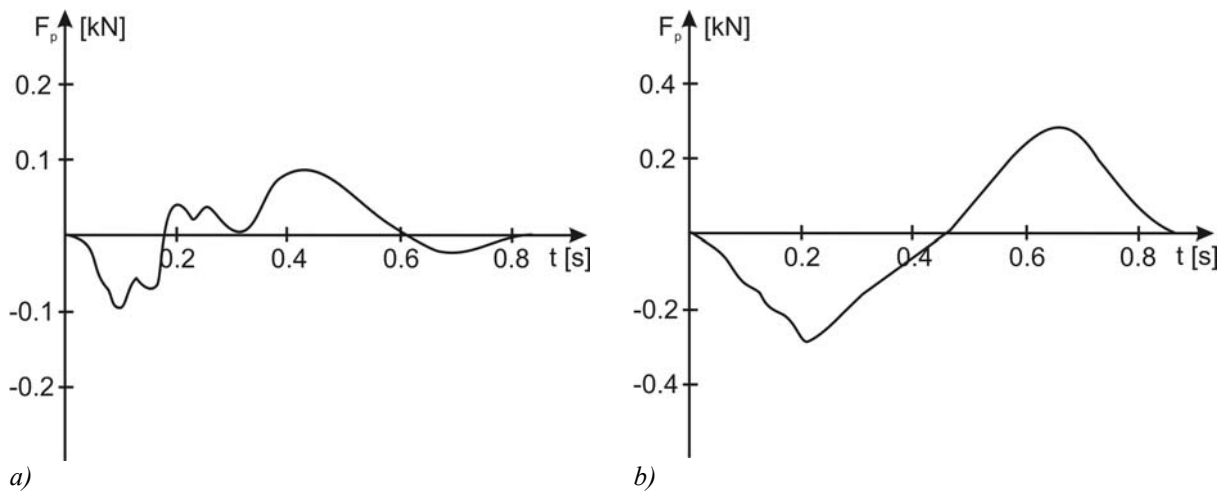


Fig. 5.11 Ground reaction forces due to running [82], a) lateral component, b) longitudinal component

5.3.3 Load Models of Pedestrian Loading from Literature

5.3.3.1 Walking Induced Vertical Loads

The induced ground reaction forces are periodic function dependant on time. If the force components of both feet are added together, the load can be divided into different sinusoidal oscillations by a Fourier transformation.

$$F(t) = F_0 + \sum_i F_i \cdot \sin(2\pi \cdot i \cdot f_s \cdot t - \varphi_i) \quad (5.12)$$

where:

F_0	=	mean or static load
F_i	=	load component for frequency $i \cdot f_s$
f_s	=	step frequency
φ_i	=	phase angle of load component F_i

Fig. 5.12 shows, for the vertical load induced by walking, the resulting time history and the amplitude spectrum and different load coefficients. It can be shown that there are parts acting with two or more times the walking frequency.

The number of harmonics that have to be considered depends on their amplitude and their dynamic influence. Bridges can be excited by even the second or third harmonic.

The ratio of the force amplitude to the person's weight is defined as dynamic force factor. The results of different investigations show that these values scatter greatly ([40], [59], and others). The force factors are determined for walking on a rigid ground. It can be seen that the first force factor increases noticeably with increasing step frequency, while the second do not show as great a dependence on frequency. It is said that the first three components are relevant for the design.

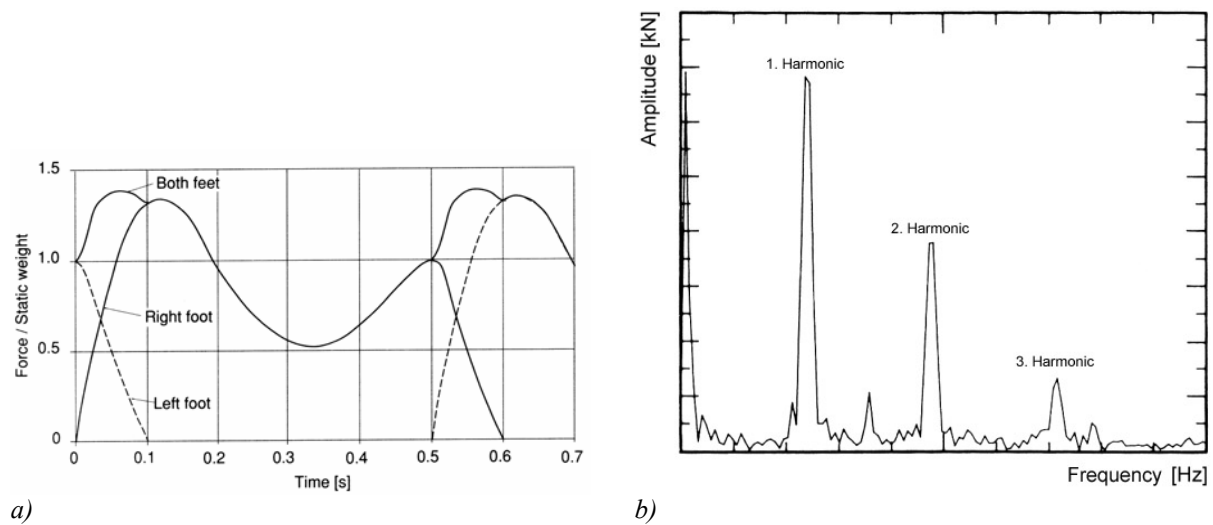


Fig. 5.12 a) resulting vertical force caused by walking [38] b) Fourier-Spectrum obtained from a) by fast Fourier transformation [74]

Various authors have studied the Fourier coefficients for pedestrian loading. For the sake of brevity, only the loading coefficients according to Bachmann will be handled here. Bachmann [37] describes the vertical load:

$$F_v = F_0 + F_{1,v} \cdot \sin(2\pi \cdot f_s \cdot t) + F_{2,v} \cdot \sin(4\pi \cdot f_s \cdot t - \varphi_2) + F_{3,v} \cdot \sin(6\pi \cdot f_s \cdot t - \varphi_3) \quad (5.13)$$

where:

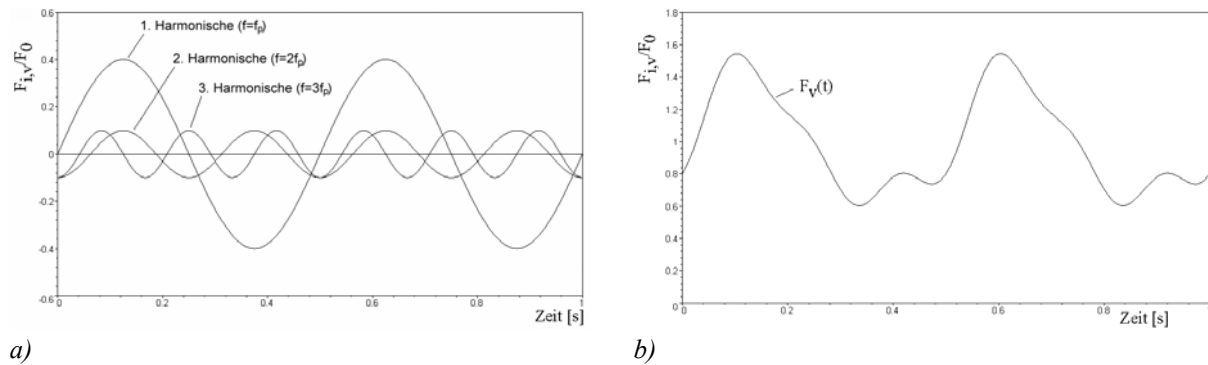
- F_0 = dead load of the pedestrian (800 N)
- $F_{i,v}$ = participation of the i-th harmonic to the resulting load
- f_s = step frequency
- φ_i = phase angle of the i-th harmonic

In table 5.4 the force factors for the vertical force according to Bachmann are given.

Force factor [-]	$F_{1,v} / F_0$	$F_{2,v} / F_0$	$F_{3,v} / F_0$
Bachmann [37]	0,4 for $f_s = 2\text{Hz}$ 0,5 for $f_s = 2,4\text{Hz}$	0,1	0,1

Table 5.4 Recommended force factors for vertical loads

According to experiments [37], [59], [82] the phase angles depend on different parameters and scatter greatly. Bachmann decided to take them as approximately $\varphi_2 = \varphi_3 = \pi/2$.

Fig.5.13 Walking induced vertical forces: a) first 3 harmonics with $\pi_2 = \pi/2$ and $\pi_3 = 3\pi/8$, b) resulting force [39]

5.3.3.2 Walking Induced Lateral Loads

The spectrum of the lateral force component shows that the load components are applied with half the walking frequency and multiples of it. Under consideration of the first 3 harmonics of the Fourier coefficients the lateral force component can be described as:

$$F_h = F_{1,h} \cdot \sin\left(2\pi \cdot \frac{f_s}{2} \cdot t\right) + F_{2,h} \cdot \sin\left(4\pi \cdot \frac{f_s}{2} \cdot t - \varphi_2\right) + F_{3,h} \cdot \sin\left(6\pi \cdot \frac{f_s}{2} \cdot t - \varphi_3\right) \quad (5.14)$$

where: $F_{i,h}$ = participation of the i-th harmonic to the resulting load
 f_s = step frequency
 φ_i = phase angle of the i-th harmonic

Recommended values for the lateral component are assembled in table 5.5.

Force factor [-]	$F_{1,h} / F_0$	$F_{2,h} / F_0$	$F_{3,h} / F_0$	$F_{4,h} / F_0$
Eurocode 1 [14]	0,1	-	-	-
Other sources	0,05	0,01	0,05	0,01

Table 5.5 Recommended force factors for lateral loads

The phase angles should be taken as $\varphi_2 = \varphi_3 = \pi/2$. Hence the resulting force response is shown in fig. 5.14.

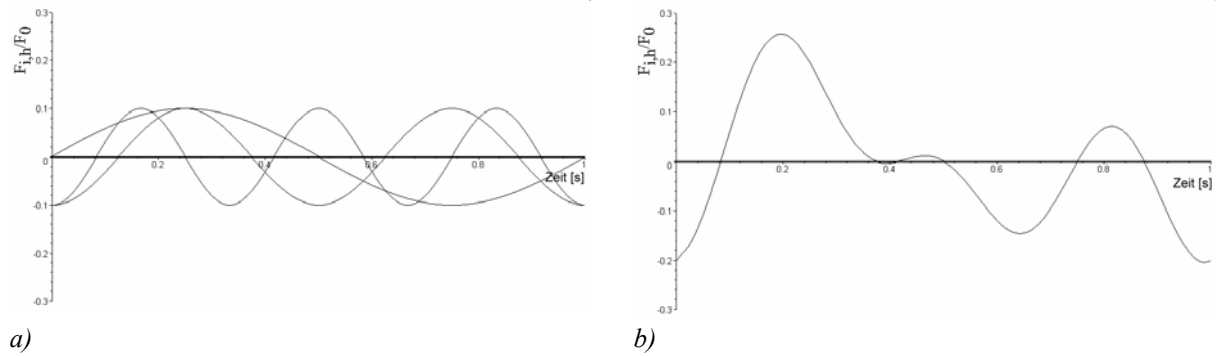


Fig. 5.14 Walking induced lateral loads: a) first 3 harmonics for $F_{i,h}/F_0 = 0,1$, b) resulting force

The **French Footbridge Guideline** (presently in preparation) values will be based on generally accepted test results. The guideline recommends taking into account the first four terms of the series to achieve a loading function sufficiently close to measured pedestrian loading.

5.3.4 Load Models in International Codes

The application of the theory of pedestrian dynamic loading can be found in the international codes and regulations. An overview of these codes is provided here.

In the **prenorm of Eurocode 1, Part 1** ‘live loads on bridges’ a dynamic design load model is included that allows for synchronisation effects (see chapter 5.1). The load model is divided into three parts, so that different pedestrian configurations are considered in the design. The **British Standard BS 5400, Part 2** [4] and the **Ontario Highway Bridge Design Code OHBDC ONT 83** [31] contain the same loading model as the single pedestrian load model.

5.3.4.1 Single Pedestrian Load Model (DLM 1)

The dynamic action consists of a pulsating stationary force with two components. It accounts for the effect of a walking pedestrian with a weight of 700 N and a walking velocity [m/s] of 0.9 m times the step frequency f_s by considering the first harmonic of the ground reaction force (vertical force factor $0.4 \cdot 700$ N, lateral force factor $0.1 \cdot 700$ N).

$$\text{vertical component:} \quad Q_{pv} = 180 \cdot \sin(2\pi \cdot f_v \cdot t) \text{ [N]} \quad (5.15)$$

$$\text{lateral component} \quad Q_{ph} = 70 \cdot \sin(2\pi \cdot f_h \cdot t) \text{ [N]} \quad (5.16)$$

where: f_v = vertical bridge natural frequency close to 2 Hz
 f_h = lateral bridge natural frequency close to 1 Hz

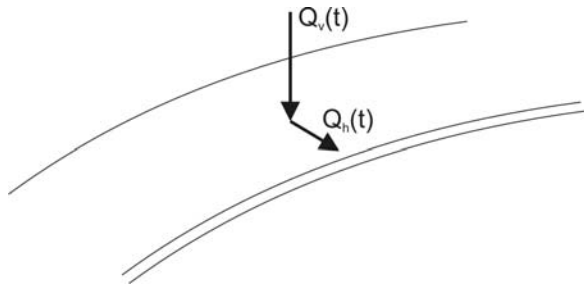


Fig. 5.15 DLM 1 acc. to Eurocode

In general the frequencies should be chosen to match the fundamental frequencies and the action should be applied on the most unfavourable position on the bridge.

The vertical component of the Single Pedestrian Load Model (DLM 1) is also contained in DIN-Fachbericht 102 [11].

5.3.4.2 Load Model for a Pedestrian Group (DLM 2)

The stochastic approach of load model DLM 2 is to describe the effect of a group with a limited number of unsorted walking persons (8-15 persons). The synchronisation of step frequencies and phases is taken into account by coefficient k_v and k_h .

$$\text{vertical component} \quad Q_{gv} = 180 \cdot k_v(f_v) \cdot \sin(2\pi \cdot f_v \cdot t) \text{ [N]} \quad (5.17)$$

$$\text{lateral component} \quad Q_{gh} = 70 \cdot k_h(f_h) \cdot \sin(2\pi \cdot f_h \cdot t) \text{ [N]} \quad (5.18)$$

where: f_v = vertical bridge natural frequency close to 2 Hz
 f_h = lateral bridge natural frequency close to 1.5 Hz
 k_v, k_h = synchronisation factor (see fig. 5.17)

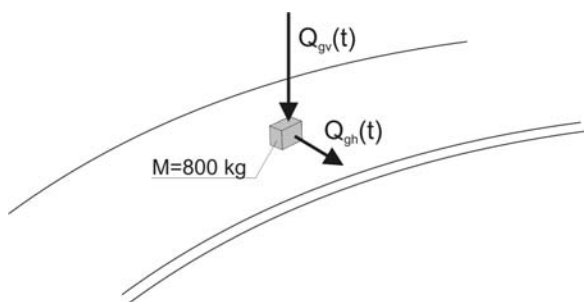


Fig. 5.16 DLM 2 acc. to Eurocode

The dynamic load of a pedestrian group should be applied as stationary pulsating force with two components that should be considered separately. In the same way the load should be applied on the most unfavourable position on the bridge. To allow for the influence of the pedestrians on the dynamic properties of the bridge (natural frequencies), a mass of 800 kg should be applied at the same position as the force.

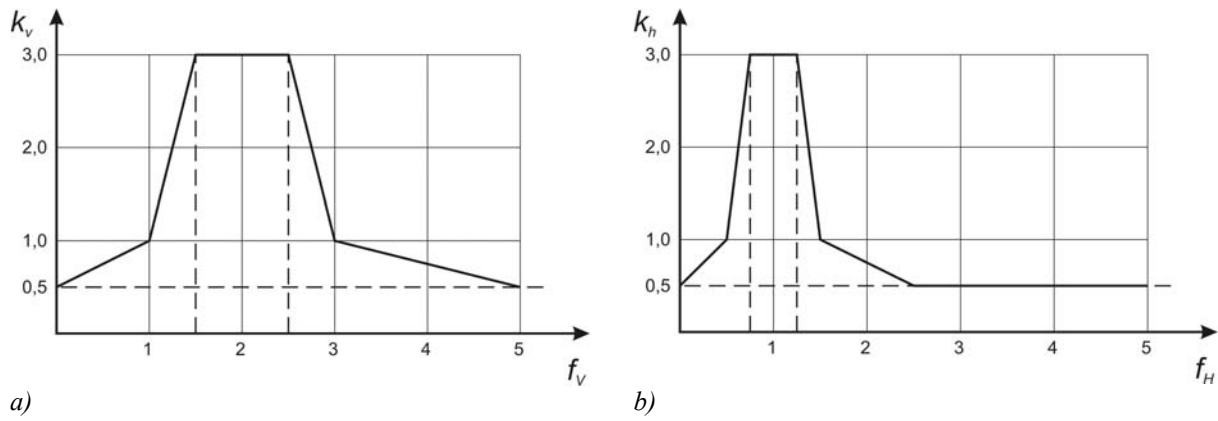


Fig. 5.17 Dependence of k_v (a) and k_h (b) on natural frequencies f_v and f_h

The coefficients k_v and k_h are introduced to account for random synchronised pedestrians within the group.

Only the first harmonics are considered (vertical load factor 0.4, lateral load factor 0.1) at the moment, but discussion is continuing on adding the second harmonics (vertical load factor 0.2, lateral load factor 0.05).

5.3.4.3 Load Model for a Continuous Pedestrian Stream (DLM 3)

The dynamic load model DLM 3 stands for the excitation forces due to a continuous pedestrian stream with a density of 0.6 pers/m² (see fig. 5.2). It should be used separately to DLM 2. The total number of pedestrians is $N = 0.6 B L$, where B is the effective bridge width and L the length. The continuous pedestrian stream is applied as a uniformly distributed pulsating area load existing of a vertical and a lateral component.

$$\text{vertical component} \quad q_{s,v} = 12,6 \cdot k_v(f_v) \cdot \sin(2\pi \cdot f_v \cdot t) \text{ [N/m}^2\text{]} \quad (5.19)$$

$$\text{lateral component} \quad q_{s,h} = 3,2 \cdot k_h(f_h) \cdot \sin(2\pi \cdot f_h \cdot t) \text{ [N/m}^2\text{]} \quad (5.20)$$

where: f_v = vertical bridge natural frequency close to 2 Hz
 f_h = lateral bridge natural frequency close to 1.5 Hz
 k_v, k_h = synchronisation factor (see fig. 5.17)

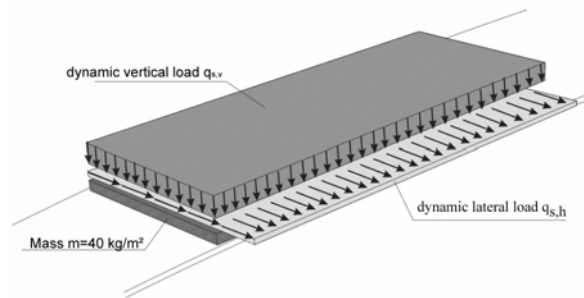


Fig. 5.18 DLM 3 acc. to Eurocode

To produce the most unfavorable loading case the load should be applied to the relevant areas of the bridge. Depending on the mode shape the total span or as well half of the span should be loaded. In accordance to DLM 2 an additional evenly distributed mass of 40 kg/m² should be applied on the same area if unfavorable.

The load coefficients are derived by assuming 40 kg/m² as the weight of 0.6 pers/m² if a single person weighs 66.67 kg. A synchronisation factor with the movement of the bridge is assumed to be $1/10 k_v$ resp. k_h . As the pedestrians are moving along the bridge, a reduction factor of 0.75 is incorporated in the load model.

5.4 Analysis of Accelerations

There are several methods to calculate the accelerations caused by pedestrian induced loads (see chapter 5.3). The modal analysis is one possibility and can be found in the literature, see [71]. Analytical methods to calculate resonant response according to Grundmann [54] and Rainer [74] are shown.

The calculation method by Rainer determines the maximum vertical acceleration resulting from the passage of one pedestrian walking / running with a pace rate equal to the fundamental natural frequency of the bridge. Therefore, it is modelled as an equivalent single-degree-of-freedom oscillator.

$$a = 4 \cdot \pi^2 \cdot f^2 \cdot y \cdot \alpha \cdot \Phi \text{ [m/s}^2\text{]} \quad (5.21)$$

- where:
- f = vertical natural frequency of the bridge [Hz]
 - y = static deflection at mid-span for a force of 700 N [m]
 - α = $F_{i,v} / F_0$: fourier coefficient of the relevant harmonic of the walking or running rate (see table 5.4.)
 - ζ = $(\Delta/2\pi)$ = damping ratio
 Δ = logarithmic decrement
 - Φ = dynamic amplification factor for one pedestrian moving across simple span (see fig. 5.19)
number of cycles per span = number of i-th harmonic · span / step length (see table 5.3)

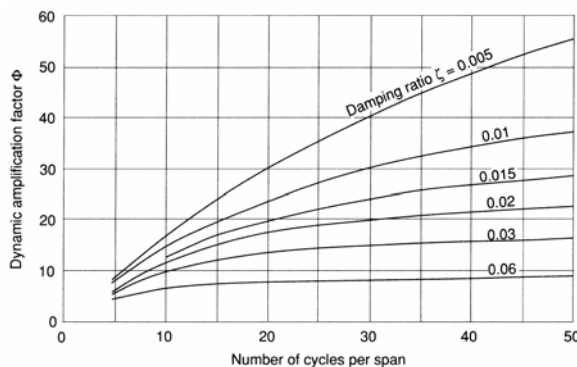


Fig. 5.19 Dynamic amplification factor Φ for resonant response due to sinusoidal force moving across simple span [74]

The approximation by Grundmann acts on the following assumptions:

- single-degree-of-freedom (SDF) oscillator with stationary excitation
- step frequency f_s = natural frequency of the bridge f
- coefficient 0.6 due to changing excitation point

The maximum acceleration a in case of $f = f_s$ is defined by:

$$a = 0.6 \cdot \frac{F}{M_{gen}} \frac{\pi}{\delta} (1 - e^{-n\delta}) \text{ [m/s}^2\text{]} \quad (5.22)$$

where:

$$F = \alpha \cdot G; \quad \alpha = F_{i,v} / F_0 \text{ (see table 5.4)}$$

$$G = 0.7 \text{ kN: dead load of the pedestrian}$$

$$M_{gen} = 0.5 \cdot M: \text{ mass [t] of equivalent SDF oscillator for simple span [37]}$$

$$\delta = \text{logarithmic decrement (= } A \text{)}$$

$$n = \text{number of cycles per span = span / step length (see table 5.3)}$$

5.5 Comfort Criteria and Limit Values

The human perception of vibrations is subjective and depends on individual characteristics and psychological influences. The perception is influenced by the physical factors vibration frequency, acceleration and the time period of exposure. The discomfort depends much on the environmental conditions and the attitude towards the vibration cause. Sound caused by rattling or resonating of bridge equipment as well as visual influences may provoke discomfort. The height above ground of the bridge or the traffic below the bridge may influence the perception also. But over the time pedestrians get used to the vibrations and the acceptance with regard to the vibrations can rise. Stochastic vibration in general induces more discomfort than periodic ones. An investigation by Capozzo showed that vibrations caused by walking were judged by standing or seating persons as unacceptable while walking persons were not bothered by the vibration. Hence one is less sensitive to self-induced vibrations. Dieckmann provides a discussion of the effects of oscillations and resonance vibrations on the human body. It has been shown that the pedestrian is more easily brought off balance with horizontal oscillations than with vertical ones and is therefore much more sensitive to horizontal vibrations.

Many studies on the human perception of vibrations have been carried out, see the fib-document on dynamic behavior of structures as well as [54], [60], [63], [88], [82], [26] and [27]. The limit values according to international codes and regulations will be handled here. Comfort requirements are either handled by providing natural frequency ranges to be avoided or by providing limit accelerations. Some codes also provide simplified rules, e.g. the Spanish Bridge Design Code recommends a limit value of deflection ($L_{span}/1800$) due to the frequent live loads (2 kN/m^2) to avoid dynamic problems. If this admissible deflection is exceeded a dynamic analysis should be carried out.

In practice, many lightweight footbridges exceed the limits of comfort requirements found in the literature and codes but have never been the source of complaints by users over a long service life. In many cases the users particularly enjoy these structures, maybe in part due to their liveliness.

Comfort requirements in codes fall mainly into two categories:

- Limit values for structural frequencies: Pedestrian induced loading falls within certain frequencies (see table 5.6). Structures whose natural frequencies fall outside these pedestrian loading frequencies will generally not be at risk of resonance loading. For this reason, many international codes provide frequency ranges for which no further dynamic calculation is required.
- Limit values of accelerations: Should the natural frequency of a structure fall within the range of pedestrian induced loading frequencies, the international codes call for a dynamic calculation. The resulting structural accelerations are then limited to ensure pedestrian comfort (see table 5.7).

5.5.1 Limit Values for Structural Frequencies According to International Codes

In this section only those codes and standards offering limit values for frequencies or defining critical areas will be handled. Table 5.6 offers a summary of frequency limits for pedestrian induced oscillation found in the international standards and codes. Most international codes observe on the first harmonic from the Fourier analysis of pedestrian loading, leading to limits of the bridge natural frequency of at or just below 3 Hz. The **Eurocode 5** and the **BS 5400** provide higher limits at 5 Hz, taking into account the higher-order harmonics on pedestrian excitation. These higher order harmonics of pedestrian excitation can be an issue for lighter structures. Petersen [71], however, holds that the higher order harmonics will not produce important oscillations in a structure, due to the stiffness of the bridge (with natural frequencies above 2.4 Hz) and the lower force component for the higher harmonics.

Code / Standard	Limit values	
	Vertical	Horizontal
American Guide Spec.	< 3 Hz	
Eurocode 2 (ENV 1992-2)	1.6 Hz - 2.4 Hz	0.8 Hz – 1.2 Hz
DIN-Fachbericht 102	1.6 Hz - 2.4 Hz, 3.5 Hz - 4.5 Hz	
Eurocode 5 (ENV 1995-2)	< 5 Hz	< 2.5 Hz
SBA (former East Germany)	1.0 Hz – 3 Hz	
SIA 260 (Switzerland)	1.6 Hz – 4.5 Hz	< 1.3 transverse < 2.5 longitudinal
BS 5400 (Great Britain)	< 5 Hz	
Austroroads (Australia)	1.5 Hz– 3 Hz	
Japanese Footbridge Design Code (1979)	1.5 Hz– 2.3 Hz	

Table 5.6 Summary of critical frequencies in international codes.

5.5.2 Limit Values for Accelerations According to International Standards

Limit values for accelerations in the international codes are directly linked to pedestrian comfort. Due to the plethora of studies on such a subjective matter as pedestrian comfort, there are many different acceleration limits in the international codes. An overview of the acceleration limits in the codes and relevant literature is provided in table 5.7. Some of these limit accelerations are dependent on the natural frequency while others are constant for the whole range of pedestrian induced loading frequencies.

Vertical acceleration $a_{V,\max}$ [m/s ²]		
ISO 2631	$1.9 \cdot \sqrt{f_1}$	f_1 = fundamental natural frequency of the bridge
AISC Guide 11	0.5	
Eurocode 1	$\text{Min} \begin{cases} 0.50\sqrt{f_h} \\ 0.70 \end{cases}$	for $f = 1$ to 3 Hz for $f = 3$ -5 Hz: check dependant on case from $f = 5$ Hz: no check necessary
DIN-Fachbericht 102	$0.5 \cdot \sqrt{f_1}$, vert.	for $f_1 \leq 5$ Hz; f_1 = fundamental natural frequency of the unloaded bridge
VDI 2057	$0.6 \cdot \sqrt{f_1}$, vert. 0.214 , hor.	f_1 = fundamental natural frequency of the bridge
SBA	0.39	
BS 5400	$0.5 \cdot \sqrt{f_1}$	f_1 = fundamental natural frequency of the bridge
Ontario Bridge Code ONT83	$0.25 \cdot f_1^{0.78}$	f_1 = fundamental natural frequency of the bridge
Eurocode 5 (ENV 1995-2)	0.7	
Bachmann [40]	0.5 - 1.0	
Japanese Footbridge Design Code (1979)	1.0	
Lateral acceleration $a_{L,\max}$ [m/s ²]		
Eurocode 1	$\text{Min} \begin{cases} 0.14\sqrt{f_h} \\ 0.15 \end{cases}$	for $f = 0.5$ to 1.5 Hz for $f = 1.5$ -2.5 Hz: check dependant on case from $f = 2.5$ Hz: no check necessary
Eurocode 5 (ENV 1995-2)	0.2	for $f < 2.5$ Hz (for standing individuals)

Table 5.7 Acceleration limits as comfort criteria

The Association of German Engineers guidelines, **VDI Richtlinie 2057**, covers the effects of mechanical vibrations on the human body. Different body positions such as standing or sitting were studied. Dependency of individual perception on the duration of oscillation and other factors were taken into account the VDI guideline. A level of stress on the pedestrian can be determined from the frequency and acceleration of the vibration. The stress level is dependant on the direction of the oscillation, vertical or horizontal. With the calculated level of stress, the level of individual perception can be determined from a table in the guideline.

The **ISO 2631** [26], [27] treats accelerations of the human body and not necessarily refers to bridge structures.

Even though, as one can see from table 5.7, the allowable values for vertical accelerations vary greatly between different international standards and in the literature. The authors emphasize that the values in the codes, even though they vary, are conservative. There are footbridges that exhibit vertical accelerations above 1.0 m/s^2 when excited by 2 walking pedestrians or vertical accelerations greater than 1.5 m/s^2 when passed by 2 joggers that have never caused complaints.

Vandalism loading, or repetitive jumping in order to excite the natural frequency of a bridge, can lead to very high values of acceleration. Some authors state a maximum limit of acceleration for footbridges of 0.7 g to 0.8 g [40], [54], even under vandalism loads. This acceleration should thus be used to dimension the structure but not as a threshold for comfort.

5.5.2.1 Lock-In and Acceleration Limits

The acceleration limits in the international codes are based on the comfort of the pedestrians using the bridge. Lock-in can nevertheless be an important issue with regards to acceleration limits. Lock-in may arise for accelerations well below those limits for pedestrian comfort. Lock-in inevitably leads to strong amplitudes, inadmissible for pedestrians. The lock-in threshold for lateral vibrations corresponds to an acceleration of 0.08 m/s^2 for a frequency of 1 Hz , that is to say rather clearly below the comfort threshold (at about 0.15 m/s^2). This phenomenon is therefore more governing than comfort criteria for lateral vibrations.

5.5.2.2 Comfort Criteria in Displacement and Speed

Displacement criteria can be interesting especially when vibrations are rather slow and when it is not the rapidity of the movement that is disturbing but its amplitude. Nevertheless, literature is not abundant in examples concerning this matter. The only known displacement criteria are in fact static criteria, as deflection limits.

On the other hand, as the accelerations, speeds and displacements are linked, an acceleration threshold can be expressed as a displacement or speed threshold:

$$\text{Acceleration} = (2 \cdot \pi \cdot \text{frequency})^2 \cdot \text{Displacement}$$

$$\text{Acceleration} = (2 \cdot \pi \cdot \text{frequency}) \cdot \text{Velocity}$$

E.g., for a frequency of 2 Hz :

- Acceleration of 0.5 m/s^2 corresponds to a displacement of 3.2 mm and to a speed of 0.04 m/s ,
- Acceleration of 1 m/s^2 corresponds to a displacement of 6.3 mm and to a speed of 0.08 m/s .

But, for 1 Hz:

- Acceleration of 0.5 m/s^2 corresponds to a displacement of 12.7 mm and to a speed of 0.8 m/s,
- Acceleration of 1 m/s^2 corresponds to a displacement of 25.3 mm and to a speed of 0.16 m/s.

The link between acceleration, velocity and displacement is only valid for harmonic vibration, which is not always the case.

5.6 Preventive Measures and Damping

While pedestrian loading in combination with the ‘right’ natural frequencies theoretically can cause disturbing oscillations and accelerations, certain structural properties result in damping, which in practice may be sufficient to eliminate unacceptable oscillations without the need for additional measures. Inherent natural damping may be in the form of material damping, such as cracking of concrete, and heat dissipation in the material.

The dynamic response of a structure can be affected by:

- Choice of surfacing and railing

The choice of building materials affects the damping qualities of the structure. The choice of deck surfacing and railing may also have an effect on the damping or amplitude of dynamic response of a structure. The use of heavy concrete deck slabs can decrease the dynamic response of stress-ribbon structures due to pedestrian loading by increasing dead load mass.



Fig. 5.20 Heavy deck surfacing and chain-link railing at the Nordbücke in Rostock, Germany

- Choice of connections and joints

Connections also play a role in the dynamic behaviour of a structure. Bolted connections tend to provide higher damping than welded connections in steel structures due to friction and play in the connections. Frictional damping in connections can play an important role. Elastomer pads used to support glass surfacing may also add to structural damping. However these effects might decrease with time, as the elastomer materials become brittle.

If it is unlikely that such inherent properties will be sufficient in keeping the oscillations of the structure within allowable limits, dampers may be installed. This eventuality should be considered in the design phase.

5.6.1 Modification of Vibration Eigenfrequencies

An eigenfrequency is always proportional to the square root of the stiffness and inversely proportional to the square root of the mass. In general, an increase in the frequency of vibration is desired. In order to achieve this, it is necessary to increase the stiffness of the structure, but in reality that increase is often followed by an increase of the mass, so the solution is therefore not easy.

Vertical vibrations

Different cases can be considered:

- Steel box-girder deck: If the depth of the girder can be increased, the stiffness can be increased without increase of the mass (with thinner webs). If that is not the case, the increase of thickness increase the stiffness but also the mass and the vibration frequency does not change considerably. The only possibility is then to change the structural system.
- Composite girder deck: The increase of the thickness of the lower flange can be effective as the mass does not increase proportionally being essentially due to the concrete slab.
- Truss girder deck: It is interesting to increase the depth in order to increase the vibration frequency. The stiffness varies with the square of the depth and the mass less than proportionally.
- Cable-stayed deck: The increase of stay section permits in general the increase in stiffness without the considerable increase of the mass. That solution is effective but not very economical. Furthermore, a fan cable arrangement provides a system that is stiffer than harp arrangements. Higher pylons also increase the stiffness without the increase of the vibrating mass.
- Suspension deck: Vibration frequency increases as the square root of the tension in cables divided by the linear mass of the cables and the deck. Therefore, there is no interest to simply increase the cable section. Most of all, it is necessary to reduce the cable sag.

For all systems, it is also possible to increase the stiffness by including the handrail to participate in the stiffness.

5.6.2 Torsional Vibrations

The frequency of the torsional vibrations is proportional to the square root of the torsional stiffness and inversely proportionate to the square root of the polar mass of the deck. The interest is therefore to increase the deck stiffness in torsion.

There are different means to increase the torsional vibration frequency of a deck:

- Steel box-girder deck: This type of structural system provides higher torsional stiffness than a deck with open girder. Increasing area enclosed by the box may increase torsional stiffness.
- Two-beam girder deck: The adding of the lower chord wind bracing allows for an increase of the torsional stiffness. This is nevertheless not as effective as the use of a box girder.
- Two truss girder decks: Vertical truss girders with wind bracing and triangular truss girders exhibit low torsional stiffness and therefore not recommended for long spans.
- Cable-stayed deck: If there are two planes of stays, its torsional stiffness can be increased by a factor of 1.3 by anchoring the stays in the central plane of the bridge (pylons of the A-shape) in comparison with the case where the stays are anchored in two parallel lateral independent pylons.

5.6.3 Horizontal Vibrations

The horizontal vibration frequency is proportional to the square root of the horizontal stiffness and inversely proportionate to the square root of the deck mass. An evident means to increase the horizontal stiffness is to increase the width of the deck, but that can be expensive.

In the case of very narrow (compared to their span) cable-stayed or suspension footbridges, lateral cables can be placed in, in order to stiffen the deck horizontally.



Fig. 5.21 Footbridge in Nepal with lateral cables

5.6.4 Damping of Footbridges

Damping, the way oscillations decrease with time, of a footbridge depends on many factors. Structurally less important elements such as wire mesh railings might have a tremendous effect on damping. Generally, damping depends on material properties the type of bridge structure and the bearing conditions.

The logarithmic damping of different materials, structural systems, and bearing conditions according to Petersen [71] is given in table 5.8 till table 5.10. More values are given in [38]. The total damping of the structure can be calculated according to the following formula:

$$\Lambda = \Lambda_1 + \Lambda_2 + \Lambda_3 \quad (5.23)$$

where: A = $2\pi\zeta$: logarithmic decrement
 ζ = damping ratio = % critical damping
 A_1 = logarithmic decrement of material damping (see table 5.8)
 A_2 = logarithmic decrement of structural damping (see table 5.9)
 A_3 = logarithmic decrement of bearing conditions (see table 5.10)

Material Damping Λ_1 (logarithmic decrement)	Range	Mean value
Ferritic Steel	0,005...0,012	0,008
Austenitic Steel	0,008...0,018	0,013
Aluminium alloys	0,010...0,025	0,015
Hardwoods	0,030...0,040	0,035
Softwoods	0,040...0,050	0,045
Laminated wood	0,025...0,035	0,030
Fibre glass	0,035...0,045	0,040
Reinforced Concrete Uncracked	0,025...0,040	0,030
Reinforced Concrete Cracked	0,035...0,055	0,045
Prestressed Concrete	0,020...0,030	0,025
Lightweight Concrete	0,035...0,055	0,045

Table 5.8 Material damping [71]

Structural Damping Λ_2 (logarithmic decrement)	Range	Mean value
Steel Bridges		
Steel and asphalt deck	0,020...0,030	0,025
Concrete and composite deck	0,025...0,040	0,035
Wooden deck	0,030...0,050	0,050
Wooden Bridges		
Laminated	0,015...0,025	0,020
Mechanical connections – nails, bolts, and pegs	0,035...0,050	0,040
Concrete bridges	0,015...0,025	0,020
Cable stayed bridges	0,030...0,050	0,040
Suspension bridges	0,025...0,035	0,030

Table 5.9 Structural damping [71]

Damping due to bearing conditions Λ_3 (logarithmic decrement)	Range	Mean value
Steel sliding bearings	0,012...0,018	0,015
Roller bearing	0,004...0,006	0,005
Pot bearings	0,008...0,012	0,010
Elastomeric	0,010...0,025	0,015

Table 5.10 Damping due to bearing conditions [71]

For existing footbridges not too much data has been collected and published. Dynamic tests could be carried out as proof load tests just after erection of light-weight footbridges to check their correct behaviour or to define preventive measures. In these load tests, damping coefficients, frequencies and accelerations due to pedestrians per seconds crossing at different speeds should be measured. These dynamic tests are very easy to be carried out and they have low cost.

	Name	Type	Frequency		Damping (logarithmic decrement)
			Vertical	Horizontal	
	Kochenhofsteg Stuttgart, Germany	Suspension bridge	1.0 Hz, 1.99 Hz	-	0.033
	Enzsteg II Pforzheim, Germany	Suspension bridge	2.30 Hz	-	0.016
	Deutsche Museum Bridge Munich, Germany	Suspension bridge	1.65 Hz, 1.76 Hz	2.29 Hz	0.048 / 0.040 damping dependent on amplitude
	Glacis Bridge Ingolstadt, Germany	Stress-ribbon bridge (footpath)	1.1 Hz	-	0.005-0.009 before handrail and surfacing, 0.017-0.021 after handrail and surfacing
	Enzsteg III, Pforzheim, Germany	Stress-ribbon bridge	1.307 Hz, 2.05 HZ, torsion: 1.626	-	0.005404 0.007023 0.024121

Table 5.11 Damping and vibrations from measurements on footbridges

5.7 Dampers

Although the control of excessive vibrations in footbridges can be based on active, semi-active or passive control techniques, the use of passive control devices is presently the most common, reliable and economic technical solution.

Among many passive control devices that are commercially available for implementation in footbridges, tuned mass dampers (TMDs), tuned liquid mass dampers (TLMDs) and fluid-viscous dampers are the most efficient and frequently used solutions.

Table 5.12 summarizes some information concerning the implementation of control devices in different interesting footbridges susceptible to elevated levels of vibration.

General principles for the design of a damping system have been discussed by Petersen [72]:

- The damping system should be accessible
- The damping system should be low maintenance
- Corrosion should be taken into account in the design
- Buffers for high amplitude oscillations are required
- The damping system should allow adjustment later
- The damper design should be accompanied with experimental tests

Different types of possible damping systems will be discussed here.

Bridge	Damped frequencies [Hz]	Number of spans/ Length [m]	Characteristics	Predominant vibration direction	Type of damping system implemented	Effect of the damping system on the overall behaviour	Ref
T-Bridge, Japan	0.93	2 spans 45+134	Cable-stayed, continuous steel box girder	Lateral	Tuned liquid dampers of sloshing type, installed inside box girder. A total of 600 containers were used, with a mass ratio of 0.7% of the generalised mass of the girder in its lateral vibration mode.	Lateral girder displacement reduced from around 8.3mm to 2.9mm.	[52]
Millennium Bridge, London	0.8 (main) 0.5 1.0	3 spans 108+144+8	Suspension tension-ribbon	Lateral	Viscous dampers and tuned mass damper were used to control horizontal movements. Vertical mass dampers were also used to control vertical oscillation, for frequencies between 1.2 to 2.0Hz	Vibrations became imperceptible for user	[52]
Britzer Damm footbridge, Berlin	5.6	1 span 33.83	2 hinged steel arch, with orthotropic plate cross section	Vertical	2 vertical tuned mass dampers, each weighting 520kg were fitted on the bridge		[52]
Schwedter Straße Bridge, Berlin	1.9	1 span 209	Cable stayed steel deck, by means of a steel arch	Vertical	4 vertical tuned mass dampers, each weighting 900kg were fitted on the bridge		[52]
Mjølmesundet bridge, Norway	0.8	3 spans	Steel box girder	Vertical	1 vertical tuned mass damper, weighting 6000kg was fitted on the bridge		[52]
Footbridge on large atrium	4.3	1 span 28	Steel beams	Vertical	2 vertical tuned mass dampers, each weighting ≈ 1000 kg; mass ratios of $\approx 5\%$ of structural modal mass		[52]
Simply supported footbridge	1.84	1 span 47.4	Steel box girder	Vertical	1 vertical tuned mass damper; mass ratio of 1.0% of structural modal mass	Increase of structural damping by 12.7 times	[52]
Bellagio to Bally's footbridge, Las Vegas	1.7 to 2.2	1 span	Steel beam girder	Vertical	6 vertical tuned mass dampers	Increase of structural damping by ≈ 16 times	[52]
Forchheim footbridge, Germany	1.0 to 3.0	117.5	Cable-stayed	Vertical	1 semi-active TMD, fitted with MR damper		[52]
Minden Footbridge, Germany	0.25 to 1.05	103	Suspension bridge	Vertical, Horizontal, and Torsional	2 telescoping hydraulic dampers	Increase in structural damping	
Duisburg Footbridge, Germany	0.80 to 1.33	1 span	Suspension bridge	Horizontal	Frictional dampers in deck connections	Increase in structural damping	

Table 5.12 Bridges where damping systems have been implemented

5.7.1 Tuned Mass Damper

In the case of tuned mass dampers, the mass of the structure is connected to a damper mass with damping and spring elements. The dissipative quality of the damping system must be tuned to the structure. Petersen [72] provides a method design for tuned mass dampers.

The natural frequencies of a structure should be investigated by measurements.

The relevant natural frequency f_h and the modal mass of the main structure m_h is then determined.

The relation between the damper mass and modal mass of structure is chosen:

$$\mu = \frac{m_d}{m_h} \quad (5.24)$$

It is current practice to use values between $\mu = 0.03$ to 0.05 , otherwise the damping mass becomes too large.

Calculation of damper mass

$$m_d = m_h \cdot \mu \quad (5.25)$$

Calculation of optimum tuning κ_{opt} and of optimum damping ratio ξ_{opt}

$$\kappa_{opt} = \frac{1}{1 + \mu} \quad (5.26)$$

$$\xi_{opt} = \sqrt{\frac{3 \cdot \mu}{8 \cdot (1 + \mu)^3}} \quad (5.27)$$

Calculation of the optimum damping frequency, f_d

$$f_d = \kappa_{opt} \cdot f_h \quad (5.28)$$

Calculation of the spring constant of the damping element, k_d

$$k_d = (2 \cdot \pi \cdot f_d)^2 \cdot m_d \quad (5.29)$$

Calculation of the damping coefficient of the damping element, d_d

$$d_d = 2 \cdot m_d \cdot (2 \cdot \pi \cdot f_d) \cdot \xi_{opt} \quad (5.30)$$

The tuned mass damper should be installed after the completion of construction and surfacing. The tuned mass damper must be tuned exactly to the natural frequency of the structure. The natural frequency can be best determined by measurements on the completed structure. For very light structures, the mass of the damping system should be taken into account in the determination of the natural frequency of the structure. The tuned mass damper should be accessible to allow for later adjustments.

5.7.2 Pendulum Dampers (Horizontal Tuned Mass Damper)

Pendulum dampers are used on structures for which horizontal oscillations are a problem. Pendulum dampers use a mass connected to the structure by a pendulum to create vibrations that interfere with a horizontal natural frequency of oscillation for a structure. If one substitutes the structural system as an oscillating mass, the addition of the pendulum mass changes the dynamic system. The new dynamic system is that of two masses coupled by a spring. This new system has two new natural frequencies, one above the original natural frequency of the single mass oscillator and one below. For the first of these two natural frequencies, the pendulum mass and mass of the structure oscillate with the same frequency in the same direction. For the second of these two natural frequencies, the pendulum mass and the mass of the structure oscillate with the same frequency but the direction of motion of the two masses is opposed. The idea of a pendulum damping system utilizes the relative motion between the structure and the pendulum mass by connecting a damping element between the two. This damping element is essential for pendulum damping systems for footbridges. In the following, we will discuss the determination of the necessary pendulum length.

For the addition of pendulum damping system, the horizontal natural frequency of the structure must be known with precision. The horizontal frequency can be best determined by measurements on the completed structure.

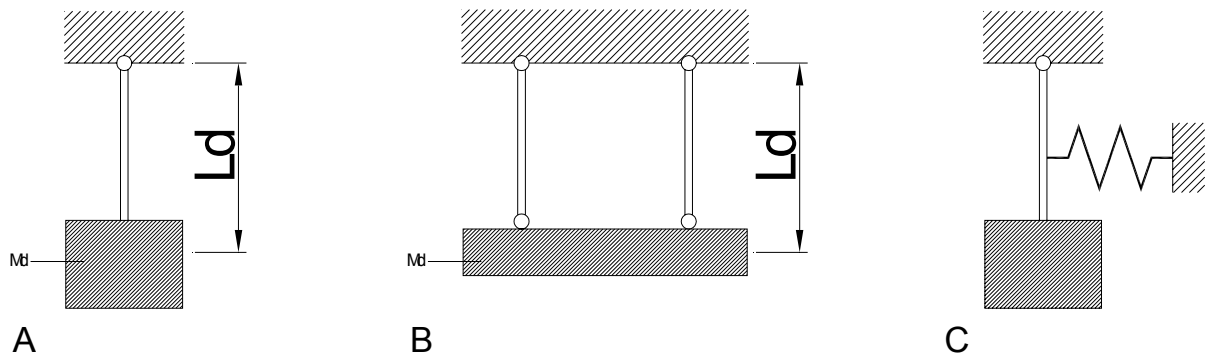


Fig. 5.22 Various pendulum systems

Neglecting the rotational inertia of the pendulum mass, the pendulum frequency can be calculated as follows:

$$f_d = \frac{1}{2 \cdot \pi} \cdot \sqrt{\frac{g}{l_d}} \quad (5.31)$$

where:

f_d	=	pendulum frequency
l_d	=	pendulum length
g	=	acceleration of gravity

Assuming an optimum tuning κ_{opt} of 0.95 [72], the pendulum length can be calculated as follows:

$$l_d = \frac{27.53}{f_h^2} \text{ [cm]} \quad (5.32)$$

Pendulum length becomes rather small for frequencies greater than 1 Hz, which can lead to problems constructing the damping system. The use of a spring can be used to overcome these potential problems. In order to minimise the stresses in the pendulum rod, the spring may be connected directly to the pendulum mass. The pendulum length can be calculated according the following formula:

$$l_d = \frac{g}{(2 \cdot \pi \cdot f)^2 - \frac{c}{m}} \quad (5.33)$$

For a given pendulum length, the spring constant c of the damping system can be derived:

$$c = \left[(2 \cdot \pi \cdot f)^2 - \frac{g}{l_d} \right] \cdot m \quad (5.34)$$

5.7.3 Viscous Dampers

Viscous elastic dampers or dry friction dampers use the action of solids to dissipate the oscillatory energy of a structure. It is also possible to use a fluid for obtaining the same goal.

The immediate device is the one derived from the ‘dashpot’. In such a device, the dissipation is obtained by the conversion of the mechanical energy into heat with the help of a piston that deforms and displaces a very viscous substance such as silicon. Another family of dampers is based on the flow of a fluid in a closed container. The piston not only deforms the viscous substance but also forces the passage of the fluid through calibrated orifices. As in the preceding case, the dissipation of the energy results in a development of the heat.

The main difference between these two techniques is the following. In the ‘dashpot’ damper, the dissipative force is function of the viscosity of the fluid, while in the other one that force is principally due to the volumic mass of the fluid. That means that the dampers with orifices are more stable against temperature variations in comparison with the ‘dashpot’ ones.

In the case of footbridges, the movements are small, and one must assure that the dampers are effective even for small displacements of the order of the millimetre. Taking into account the compressibility of the fluid, the friction of the joints and the play in the joints, this is not easy to obtain.

Viscous dampers must be placed between two points of the structure with a differential displacement between them. They can be either on an element linking a pier and the deck or on the horizontal wind bracing of the deck (see fig. 5.22).

The damping characteristics of a viscous damper must be determined experimentally, but there are approximations for their analytical estimation from the material characteristics

Such dampers have also been used on the Millennium Bridge in London.



Fig. 5.23 Example of viscous damper on the footbridge in Minden, Germany. Viscous damper installed between pylon and deck slab

5.7.4 Viscous Elastic Dampers

The use of viscous elastic materials for the control of vibrations goes back to the 50 s. Their application in structural engineering dates back to the 60 s.

The viscous elastic materials are principally polymers dissipating the energy by shear. The characteristics of viscous elastic dampers depend not only on frequency, but also on temperature. The damping coefficient is expressed by:

$$C = \frac{W_d}{\pi \cdot \Omega \cdot x^2} \quad (5.35)$$

where:

W_d	=	energy dissipation in the structure per cycle
Ω	=	circular frequency of excitation
x	=	(piston) displacement

This type of damper has been rarely used for footbridges.

To calculate the effect of dampers in the structure a time history dynamic analysis could be carried out. Visco-elastic dampers behaviour could be represented with the following simplified model:

$$F = F_0 + Kx + C v^\alpha \quad (5.36)$$

where:

F	=	force transmitted by damper
F_0	=	preloading force
x	=	(piston) displacement
v	=	velocity
K	=	spring parameter of the damper
C	=	viscous constant of the damper
α	=	parameter of the damper, α usually varies between 0.1 and 0.4.

Parameters should experimentally calibrated by the supplier.

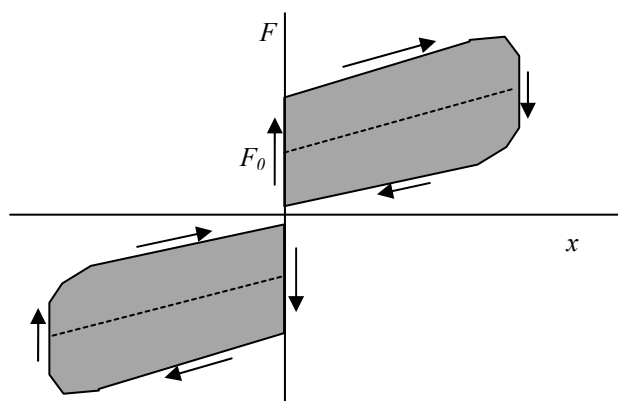


Fig.5.24 General behaviour of visco-elastic dampers, force-displacement relationship

5.7.5 Tuned Liquid Dampers

Tuned liquid dampers are fluid filled containers and provide an interesting possibility for footbridge damping systems. Accelerations of the container cause inertial and damping forces that can be used as system damping. The damping forces are dependant on the viscosity of the fluid and the texture of the container walls. Fig. 5.24 provides an overview of the different types of tuned liquid dampers. Figures A-D are referred to as tuned liquid column dampers while figures E and F are referred to as tuned sloshing dampers.

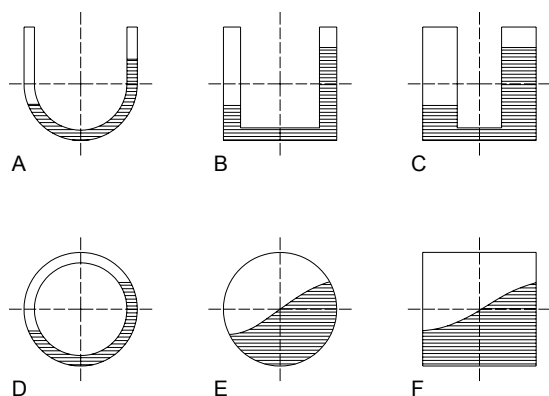


Fig. 5.25 Various types of Tuned Liquid Dampers

The natural frequency of the liquid dampers as in figure 5.24 A and B can be expressed:

$$f = \frac{1}{2 \cdot \pi} \cdot \sqrt{\frac{2 \cdot g}{L_f}} \quad (5.37)$$

where: L_f = total length of fluid column
 g = acceleration of gravity

The natural frequency of the liquid dampers as in figure 5.24 E and F can be expressed:

$$f = \frac{1}{2 \cdot \pi} \cdot \sqrt{\alpha \cdot \frac{g}{L} \cdot \tanh\left(\alpha \cdot \frac{H}{L}\right)} \quad (5.38)$$

where: $\alpha = \pi/2$ for a rectangular container and 1.84 for a cylindrical container

H and L can be determined from figure 5.25. The radius of the cylinder can be replaced by L in equation 5.38 for cylindrical containers.

The effectiveness of a fluid damper depends on the ratio between modal mass of the damper and the structure as well as the detuning. The dimensions of the container, height of fluid, and the viscosity of the fluid also play an important role. For larger accelerations, the behaviour of the fluid damper is non-linear. Petersen [71] recommends an experimental tuning of the fluid damper. Contrary to a number of high-rise buildings, it seems that no footbridge has been equipped with this type of damper yet.

Frictional damping systems

Frictional damping systems use friction between surfaces to achieve a damping effect. Frictional damping systems have been used in the footbridge at Duisburg, Germany. A total of eight dampers were installed near the bearings, provided frictional damping in the vertical and longitudinal direction.

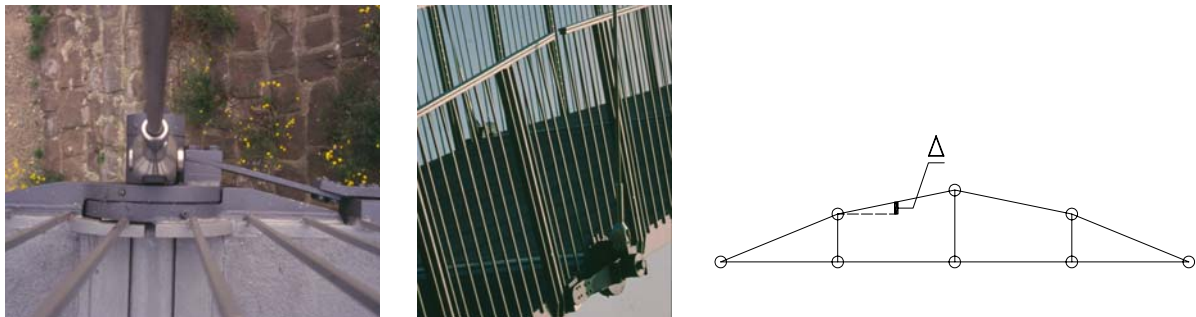


Fig. 5.26 Frictional damping system against vertical deflections for the footbridge at Duisburg, Germany

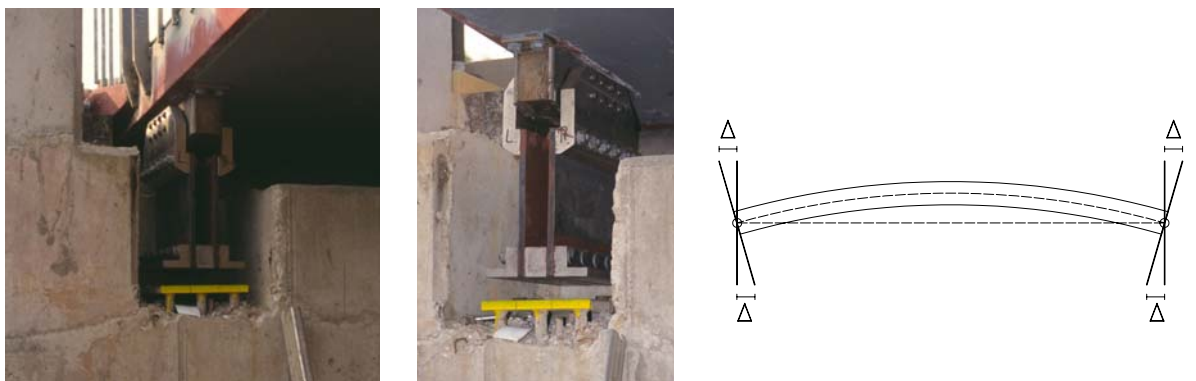


Fig. 5.27 Frictional damping system against horizontal deflections for the footbridge at Duisburg, Germany

5.7.6 Comparison of Various Damping Systems

Damper type	Field of application	Advantages	Disadvantages
Tuned mass dampers (Pendulum damper)	Much used	Easy to dimension	Additional mass to consider Dampens one mode only Demands frequency adjustment
Viscous with orifices	Little used	Independent of temperature Dampens several modes	Non-linear calculation
Viscous "dashpot"	Little used	Dampens several modes	Sensitive to temperature No-linear calculation
Viscous elastic	Very little used	Dampens several modes	Demands shear assembling
Frictional damper	Very little used	Dampens several modes	May demand complicated assembly
Fluid tuned mass dampers	Very little used	Dampens one mode only	Additional mass to consider Demands frequency adjustment

Table 5.13 Comparison of damping systems

6 Deck Surfacing

This section deals with structurally independent decking materials and surfaces. The functional requirements of deck surfacing and the influencing factors on performance are discussed. A synopsis of alternative decking materials and their construction is included as well as a short discussion regarding deck drainage.

6.1 Functional Requirements

The primary functions of a deck surface are to define the traffic surface and provide suitable traction for the user. Applied surfaces may also act to protect the structural deck from weathering and wearing. Segregation, for instance between pedestrians and cyclists, can be facilitated by a change of surface specification. The choice of surface decking may have a dramatic effect on aesthetic criteria and issues such as lighting.

6.1.2 Anti-Slip Characteristics

Deck surfaces must provide adequate traction to inhibit slipping. Note that cyclists and horses display different characteristics in terms of speed, stability and area of surface contact to pedestrians and should be considered accordingly. Where the pedestrian is concerned, traction is influenced by factors including the type of user and footwear, environmental conditions and the condition of the deck surface. A broad range of issues contribute to the technical condition of the traffic surface, including the gradient of the bridge deck, system porosity, drainage strategy, surface texture, continuity of surface and so on. The surface material should provide adequate friction and be sufficiently resistant to polishing through use. Whilst a level deck is less prone to cause slipping, good drainage must also be provided in order to prevent deterioration of the wearing surface or ponding, leading to ‘aquaplaning’ through reduced friction.

Traction is not only an objective characteristic but subjective as well, with perception playing a key role. Pedestrians tend to change their pace and gait if they are aware that a surface is slippery and tend towards parapets and handrails for support. Lighting, color and pattern may additionally affect the confidence of the pedestrian and should be considered accordingly.

There have been many studies regarding pedestrian slippage. Research by *Perkins* from the American Society for Testing Materials’ ‘Walkway Surfacing Report’ suggests that most slips occur as the foot is landing and the heel is in contact with the ground [2]. This tends to indicate that a coefficient of static friction would be the most relevant factor in surfacing. The heel does undergo a rotation as it lands, so the relevance of the dynamic coefficient of friction cannot be completely discounted. The study also suggests that patterns on shoe soles may have little effect on slipping, due to the fact that only a small part of the heel is in contact with the ground when the slipping begins.

The use of profiled materials to increase grip on surfacing may not work in all cases. Surface profiling increases traction by causing deformations in the shoe sole. Hard soles show limited deformations and therefore profiling has less effect. Profiled surfaces with rounded profiling and low surface roughness are to be avoided.

6.2 Surfacing Materials

6.2.1 Asphalt Paving

Mastic asphalt is commonly used in footbridge surfacing, often to provide protection against water ingress a thickness of 5 to 6 cm is generally sufficient for asphalt surfacing whose flexibility allows it to follow the deformations of the structure although one must take into account the inclination of the bridge deck, which may be too high for effective covering. During paving no oils shall be present that would impede the binding process and steam build up must be avoided. A mat of woven fibreglass may be used to allow steam on the underside to escape. The effects of the heat of asphalt paving should be noted to avoid excessive additional deformations to the structure.

At low temperatures, mastic asphalt may crack if deformations become too large. Swedish engineers have studied the use of polymer modified mastic asphalt to prevent this cracking. At lower temperatures, this asphalt performs better than conventional mastic asphalt. High temperatures may nevertheless cause changes in the polymer and thus cancel any positive effects.

In the example of the Glacis Bridge in Ingolstadt, Germany, asphalt surfacing was used. High temperatures and the slope of the bridge deck made paving difficult. The problem of asphalt flow was taken into account using a mastix asphalt mix with an appropriate consistency to avoid flow. Thorough control of paving was necessary to ensure the necessary asphalt thickness with a deck curved in elevation and the transverse slope necessary for drainage. Also, paving near the deepest point of the ribbon had to be done very carefully, to ensure that the water would not accumulate.

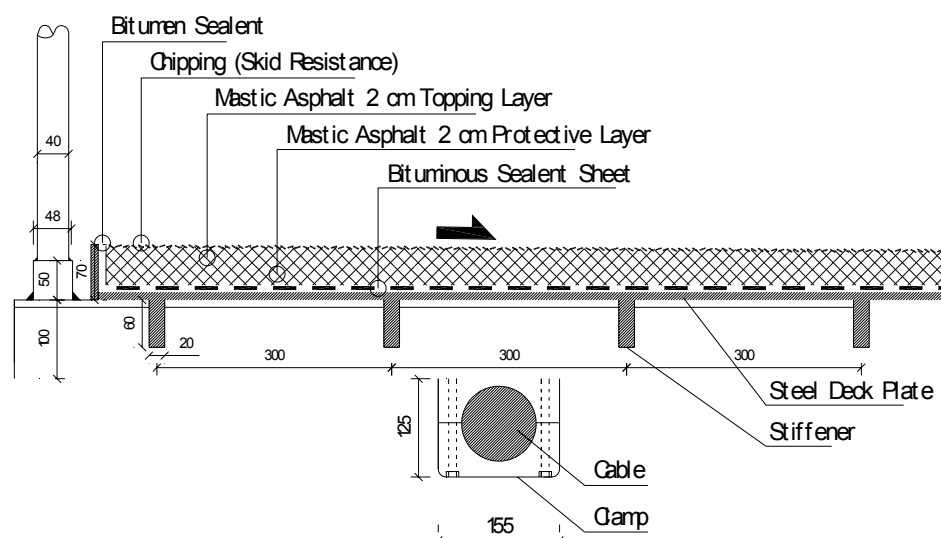


Fig. 6.1 Example of asphalt surfacing for the Glacis Bridge in Ingolstadt, Germany

The steel plate of the bridge deck was sandblasted to obtain the necessary surface smoothness. A waterproof layer of welded asphalt sheet was then applied to the steel plate. A total of 4 cm thick mastic asphalt was then applied, in two layers of 2 cm each. The longitudinal joint between steel profile and asphalt was poured with hot bitumen with a width of 25 mm. Drainage was provided to the middle of the bridge deck, to avoid water running into the interface between asphalt and steel at the outside edges. The asphalt surface was finally chipped to provide the necessary skid resistance.

6.2.2 Open Grating Decks

Most commonly comprising steel or aluminium in a variety of profiles forming one way or two-way open grids with variable loading and spanning capabilities. Gratings are generally panellised and material options include plastics, composites, and timber derivatives. UK departmental code BD29/03 notes that aluminium or other alloys with a high scrap value should be avoided where there is a high likelihood of damage or theft.

The main advantage to open gratings is their permeability to water and light, preventing the creation of ‘sterile’ overshadowed areas under the bridge and avoiding the need for of elaborate secondary drainage systems. Modular gratings are easily replaceable and relative lightweight but provide an undesirable surface for some pedestrians e.g. bare feet and narrow-heels, and other bridge users e.g. cyclists and pushchairs etc. It is not uncommon to provide alternative ‘closed’ surfacing strips adjacent to open grid areas to provide free access. This was the solution at the Mahlbusenbrücke in Rostock, Germany.



Fig. 6.2 Mahlbusen Bridge in Rostock, Germany

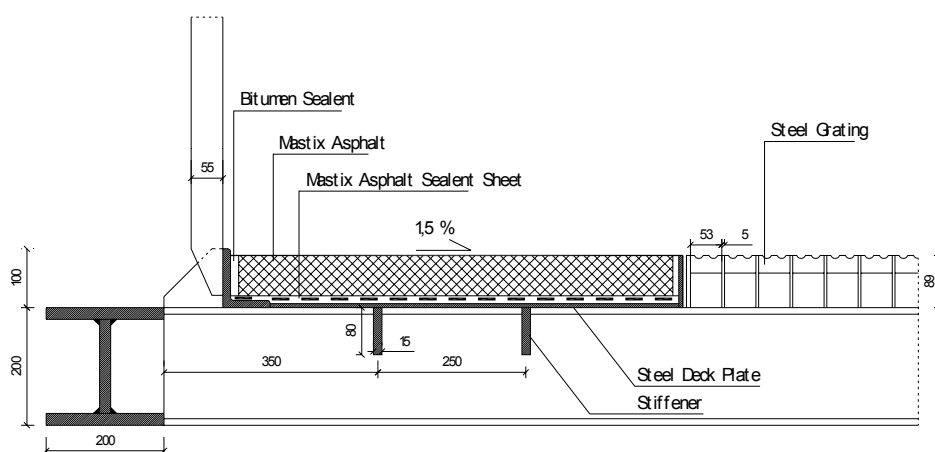


Fig 6.3 Deck section Malbusen Bridge in Rostock, Germany

Dimensions of the traffic surface and in particular the gaps between them should be considered with regard to shoe/heel/tyre sizes and the avoidance of creating trip hazards. Note that gaps sized to prevent heels falling through may prevent a finger trap hazard in the event of a fall.

Generally the traffic surface of gratings is ridged, knurled or otherwise profiled to provide anti-slip characteristics. Because open grids are 'directional' the anti-slip surfacing must be placed perpendicular to the direction of flow. This may be difficult on curved alignments. On metal decks in particular non-slip profiling should avoid being abrasive.

6.2.3 Concrete Surfacing

Epoxy resin and polyurethane coating can be used as surfacing for concrete decks. Sandblasting of the concrete surface is usually a necessary surface preparation. It is important that the coating material is able to bridge cracks that naturally occur on the surface of concrete. Fine sand must be added to ensure sufficient bond between layers. A final layer of sand or silicon carbide, which provides an attractive surface finish, should be provided to provide skid resistance. Should excessive wear be expected, a thin polyurethane sealant layer atop the sand should be added.

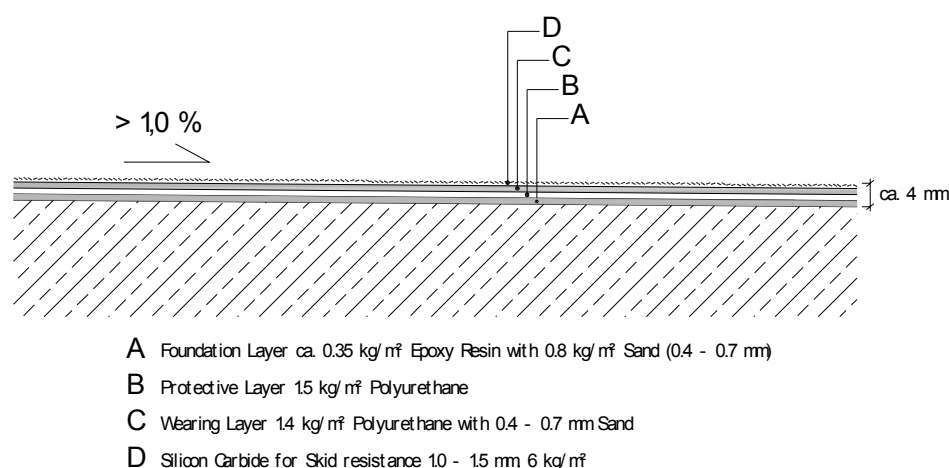


Fig 6.4 Example of concrete surfacing with epoxy resin - polyurethane coating

6.2.4 Wood Surfaces

Wood surfacing can provide an aesthetically pleasing and light deck surface but is generally slippery when wet. There are weather-protected surfaces in *covered bridges* and weathered surfaces in *open structures*. In both cases wood surfacing may be either *open* or *closed*. In any case, wood surfaces should be regarded as non-structural parts that wear out on the long run, either by weathering or by abrasion. It must be easy to replace them.

Open and closed surfaces offer both limited protection for the underlying structure (it is difficult to tighten joints in closed surfaces) and there must be a water-tight layer between surface and bridge structure, even in covered bridges where water and snow may penetrate into the inner space of the bridge by wind, or by being carried in by vehicles and people. Surfaces of diagonal planks used as a wind bracing structural element only make sense for temporary bridges (e.g. on building sites) with an expected lifetime of a few months.

The durability of the surface depends largely on the type of wood. In Switzerland, the core wood of larch, Douglas fir and oak is often used. Chestnut and black locust have excellent weather resistance, but are not yet common in bridge surfaces. Such types can be used without any impregnation. There are some examples of surfaces made of impregnated timber such as European silver fir, but the utility of impregnated wood has to be considered under ecological criteria as well as for the danger of corrosion of adjacent material when the chemicals are washed out.

The biggest difference between covered and exposed surfaces is the degree of variation in moisture content which can vary roughly from 12 % to 18 % for covered surfaces and 10 % to 28 % for exposed surfaces. Of course these values depend in great measure on the local climate, material type, exposure, wood thickness, ventilation etc.

A change in moisture content of 1 % changes the dimensions of wooden planks transversally to their fibres between:

0.16 % (radial) and 0.33 % (tangential), mean value 0.25 % for conifers

0.19 % (radial) and 0.31 % (tangential), mean value 0.25 % for oak

0.21 % (radial) and 0.41 % (tangential), mean value 0.30 % for beech

parallel to the fibres:

0.01 % is valid for all the mentioned materials (source: **Swisscode SIA 265** [34]).

Open surfaces allow for water to drain easily. Swelling and shrinking of the wood changes the width of the joints and therefore remain a local effect. Warping of planks can be avoided by a careful grading of boards with rather parallel annual growth rings, which means the 'radial' part of the tree. Controlled drainage and ventilation of the underlying space is crucial for the durability of these parts.

Closed systems may be more comfortable to the user in some cases, but the effects of the changing dimensions of the planks transversally to their fibres must be taken into careful consideration.

The use of laminated elements (plywood or glulam beams) for weather-exposed surfaces cannot be recommended for elements that are expected to have a long lifetime.

For horizontal or slightly inclined surfaces rough-sawn planks normally deliver enough traction for the comfort of pedestrians. In other cases, hardwood planks shaped in a diagonal saw-tooth profile 3 mm deep give excellent traction performance as the water is immediately pushed out between the shoe sole and the ridge of the wood. On the contrary, when there are single grooves in the planks, a remaining film of water still can cause a slipping effect. Surfacing may also be done with an elastic mixture of epoxy and sand or rubber. These elastic materials follow the deformation of the wood. These layers generally wear out within 2-3 years.

Slabs made of post-tensioned wooden boards are also used. Post-tensioning bars or strands act transversally to the fibres, thus forming an orthotropic slab distributing concentrated wheel loads efficiently. Swelling and shrinking of untreated wood does not allow to maintain a reasonable degree of post-tensioning. In Switzerland the use of tarred soft wood combined with side elements in oak (to distribute concentrated stresses near the anchors) has proved successful, as movements due to a variation in moisture are almost completely prevented. This impregnation at the same time provides an excellent weather protection so that the structure itself can be the walking surface. The post-tensioning system must be protected against corrosion by the use of stainless steel bars and anchors or sheathed systems.

6.2.5 Synthetic Materials

Synthetic materials are commonly used to increase surface profile on the deck while improving the coefficients of friction. Mats made from recycled synthetics may be used to allow for efficient drainage of the walkway. These materials generally are highly elastic and follow the deformations of the underlying structure.

Plastic lumber and wood composite lumber have been used in several applications where conventional timber would be too vulnerable to rotting and insect attacks. Wood composite lumber consists of approximately a 50-50 mix of recycled waste plastics and wood fibres or sawdust. Plastic lumber is entirely synthetic. Both of these materials have a lower modulus of elasticity than wood and experience larger longitudinal thermal expansions. Gaps allowing for this expansion must be provided. Their mechanical properties depend more heavily on temperature than conventional wood. These materials do not contain any of the toxins of treated lumber and may be used where environmental considerations are extremely important. Conventional woodworking tools may be used on these materials.

Fibre reinforced plastic materials can exhibit outstanding mechanical and chemical properties (high resistance, low weight, impact resistance, durability and corrosion resistance, etc.). FRP plates or gratings could be used as a floor plate providing a long lasting maintenance-free material. These plates could be gritted to improve slip behaviour and they can also be coloured to present an attractive appearance.



Fig. 6.5 FRP surfacing, Bridge in Lleida, Spain, Engineer: M. Dolores G. Pulido, Juan A. Sobrino

For bridleway bridges, rubberised interlocked paddock blocks, generally comprising recycled vehicle tyres, provide the necessary tactility for horses.

6.2.6 Glass

The use of glass as a decking surface is, for obvious reasons, more commonplace on internal or weather protected bridges. In general it is used to allow views through the deck and create a light-weight appearance. Most applications have involved the use of laminated, tempered glass plates.

Slip resistance, confidence of the user and privacy from views below the bridge present technical challenges. A wide range of integral or surface applied systems may be utilised in combination to resolve these issues, but are most often tackled on plate glass applications by textured *fritting* of a dotted or other pattern. Variations in the size, density and colour of

fritting have a marked effect on traction and visual permeability. Proprietary systems in which the fritting is coloured dark on the upside and light on the underside, exploit optical perception to encourage views through the glass in one direction and inhibit them in the other.

Lamination provides scope for inclusion of visually or technically functional interlayers. Applications are used in particular to contribute to the integrity of the system and may include any combination of 'secondary' uses such as means of defrosting the traffic surface, lighting, solar generation, performance monitoring systems etc.

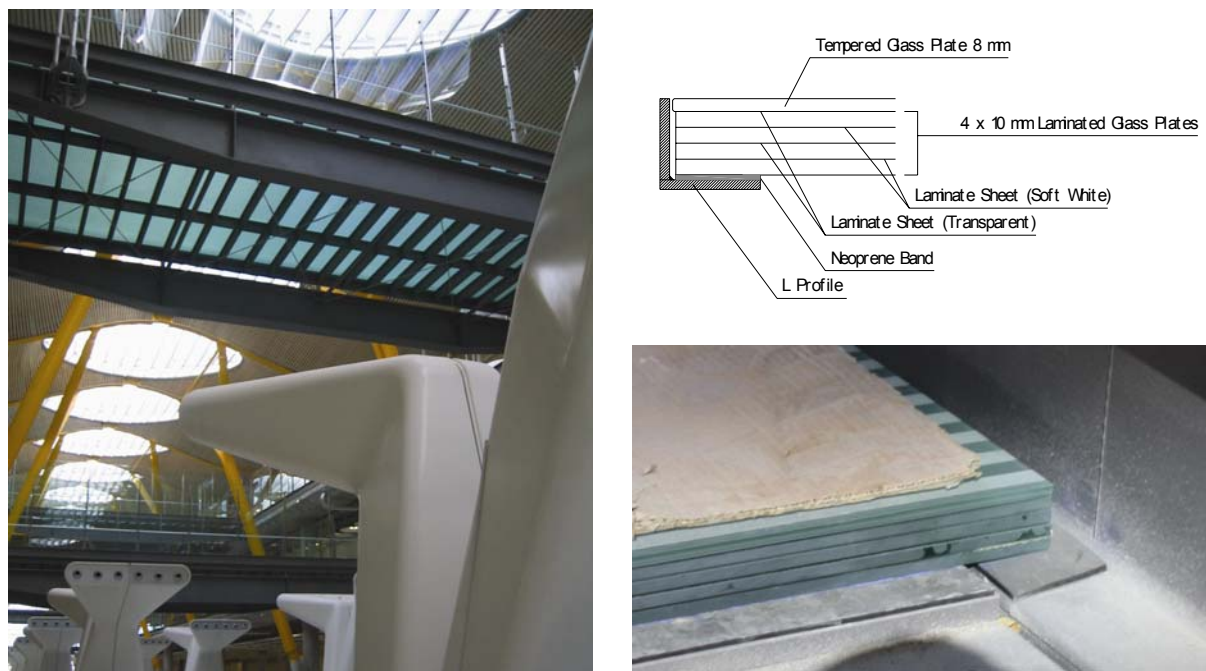


Fig.6.6 Glass surfacing for the footbridges at Madrid-Barajas Airport, Spain

Glass surfacing was used for the footbridges at the Madrid-Barajas airport. The glass plates are 1800 mm wide by 900 mm. The top layer of glass is formed by an 8 mm thick tempered glass plate with beveled edges. The top layer is laminated to atop 4 layers of 10 mm thick laminated glass panels creating a total thickness of 51.8 mm, taking into account the interlayers. Some interlayers are coloured soft white to provide the necessary opacity, while still allowing a light to pass. The glass panels are supported by a 5 mm thick and 80 mm wide neoprene strip forming a continuous support for the panels in the longitudinal direction. A neutral silicon band is used between the joints of the glass panels. To obtain the necessary skid resistance, a surface treatment is provided in 20 mm bands spaced every 40 mm in the longitudinal direction.

6.2.7 Drainage

Drainage in footbridges is often different on footbridges than on road or railway bridges. Keeping drainage gullies on the sides of the deck surface is a common solution for road and railway bridges but on a footbridge, this configuration leaves pedestrians walking in drainage channels. A more elegant solution would be to place the drainage in the middle of the bridge deck although this configuration may lead to drainage constructions interfering with the bridge structure.



Fig. 6.7 Possible drainage schemes

Wherever possible, water should be drained directly to the river below the bridge in order to avoid costly and unsightly drainage pipes. Drainage is also an issue for pedestrians passing below the bridge deck. Drainage should be so thought out as not to trench the pedestrian passing below during rainfall. In colder conditions, an insufficient drainage system may cause icicles to form on the structure, creating a safety hazard for pedestrians below.

An interesting solution is through-drainage that has been applied on the Miho Bridge in Japan [77]. The porous surface material allows water to drain right through the deck.



Fig 6.8 Deck drainage for the Miho Bridge in Japan

6.2.8 International Codes

The **British Departmental Standard BD 29/03 Design Criteria for Footbridges** [23] calls for drainage for the footbridge surface. Adequate falls must be provided for water run-off and water is not allowed to spill over the edge of a structure with the exception of perforated decks. The code designates that all decks, stairs and ramps of footbridges are to be waterproofed if made of concrete. The surfacing of all footbridges must have a slip resistance of not less than 65 units under wet conditions for all types of shoe sole. A portable skid resistance pendulum may be used to check the skid resistance of the surface.

7 Railings

This chapter deals with railings for footbridges. Pedestrian safety and the formal aspects of footbridge railings are discussed and railing construction is reviewed. A summary of railing regulation in international codes is also provided.

7.1 Safety Aspects

Railing loads for footbridges are much lower than those for safety barriers on road bridges. The shape and material for the railing can, therefore, be chosen more freely. Footbridge railings prevent people from falling off the bridge deck. Minimum height h_1 for railings varies in the international codes from 1.0 m to 1.15 m. For footbridges carrying cycle traffic, the required height of the railing h_2 might have to be increased to 1.20 m [87].

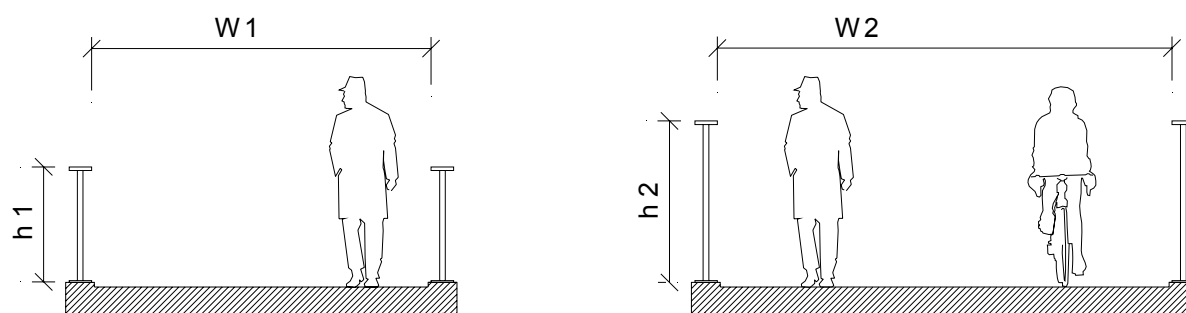


Fig. 7.1 Railing height and cyclist traffic [87].

One must take into account children on the bridge and railing posts should be designed so as to prevent children from slipping between posts with a maximum distance of 15 cm between filling posts. Children also have a tendency to climb on the railings and the railings should be designed to inhibit climbing by using vertical infilling, grating, inclining the railing in the case of horizontal bars, or similar preventive measures. Railings may also be needed to prevent debris from falling on traffic below the bridge.

The ends of footbridge railings also play a role in bridge safety by preventing pedestrians and cyclists from toppling over steep embankments and onto roadways and into waterways. Rail endings must clearly mark the continuation of path and funnel pedestrians and cyclists safely onto the bridge deck.

Railings also play a psychologically important role, since the fear of being suspended high above a waterway or roadway is somewhat curbed by the presence of a robust railing. This aspect of footbridge design is becoming more important. As the attempt to construct lighter and more transparent structures leads designers to choose lighter handrail constructions in order to emphasize the slenderness of the structure. These two opposing aspects of railing design, slenderness and feeling of safety, oblige the designer to find a sometimes difficult balance between the two.

7.2 Load Bearing Railings

Railings may be designed and considered as a structural element to help to reduce the visual depth of the deck. Some examples of footbridges with load bearing railings are provided below:



Fig. 7.2 Sant Feliu Footbridge, Girona, Spain, Engineer: M. Dolores G. Pulido, Juan A. Sobrino, Architect: A Blazquez, Ll.Guanter, P. Sola

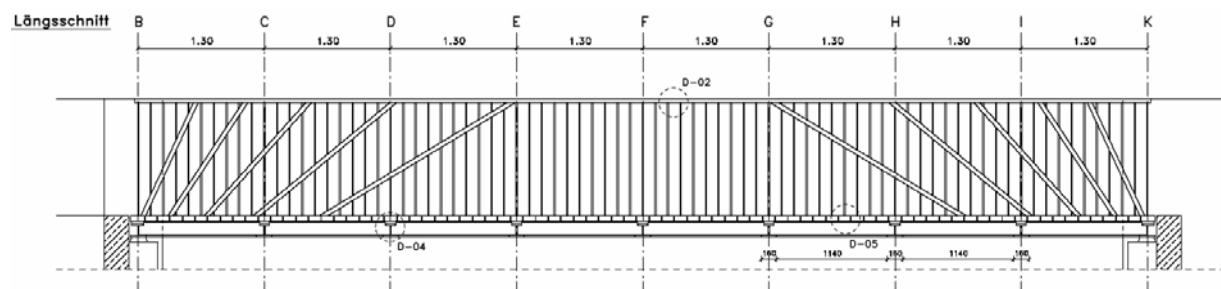


Fig 7.3 Döllnitz Footbridge, Oschatz, Germany, Engineer: ifb frohloff staffa kühl ecker, Architect: Martin Sauerzapfe Architekten [91]



Fig. 7.4 Katherinenbrücke, Grossenhain, Germany, Engineer: ifb frohloff staffa kühl ecker, Architect: Martin Sauerzapfe Architekten [91]

7.3 Formal Aspects

Railing form influences the architectural presence of a footbridge much more than a highway bridge and is closer to the user who can touch it. The more slender the section and form of a footbridge, the more it can be dominated by a poor choice in railing. It is therefore imperative that attention be placed on the interaction between railing and structure to achieve the desired formal effect.

All railings contain an inner and outer side, which play two different roles. The inner side is part of the bridge space and defines its limits. The outer side takes part in the external image of the bridge structure in its environment.

For the pedestrian crossing the footbridge, the railing helps to define distance. The railing guides the direction of the user's path and defines the traffic situation. It also links the bridge with its access paths and may narrow, collect, or widen the route.

The railing emphasizes the three dimensional aspect of the bridge structure. The railing should not dominate the structure, but emphasize its architectural strengths. Choices of colour, form, and material determine the interaction between the bridge and its surroundings.

7.4 Construction

Many modern footbridges strive for transparency. It follows that railings should become lighter as well. Railings may be made from virtually any building material but, due to the achievable transparency, steel (and sometimes timber or aluminium) is most commonly used.

Handrail materials should correspond to the needs of the user. In very cold or hot climates, metallic handrails may prove out of place due to their high thermal conductivity.

Special consideration must be paid to metallic railings with welded connections. All welds on metallic railings must be reground. This prevents pedestrians from cutting themselves on weld splatter.

Special attention must be paid to post footings. Since rail footings may pierce the deck surfacing, metal footings must be protected from corrosion in this area. If an aluminium or stainless steel railing is employed, contact corrosion must be avoided. Wooden railings must also be protected from water damage.

Expansion joints in railings should correspond with those in the bridge structure to allow for sufficient expansion. A judicious selection of the placing and distance of railing joints is required as footbridges can be lively structures and be affected by thermal and other imposed movements (shrinkage, creep).

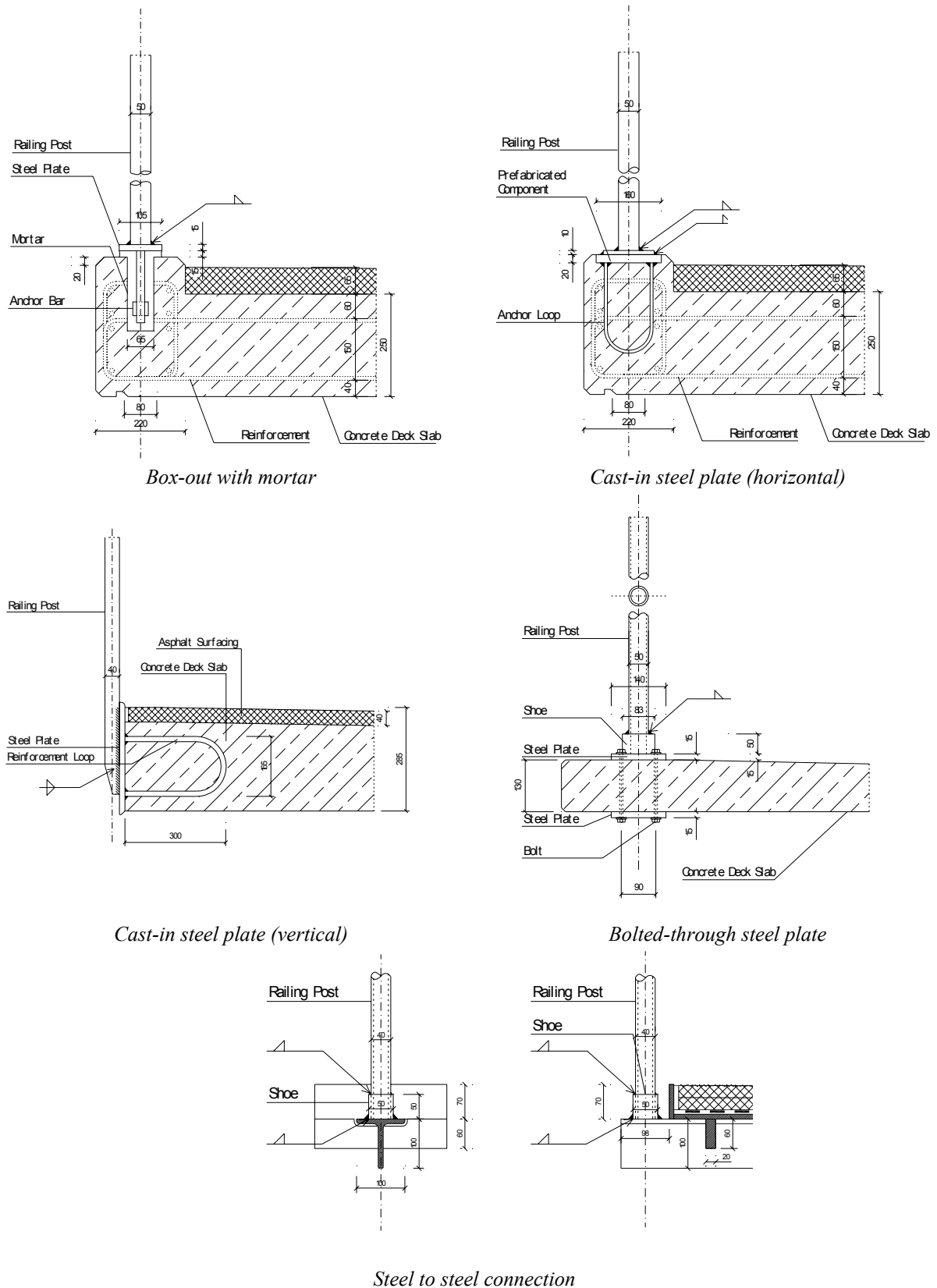


Fig 7.6 Some of the many possible anchorages of railing posts.

7.5 Railings and Codes

Those codes that handle railing loads have quite different approaches.

7.5.1 German Codes

The **DIN 1072, 4.7** [7] Straßen- und Wegbrücken Lastannahmen and the **DS 804** – Deutsche Bundesbahn – Vorschrift für Eisenbahnbrücken und sonstige Ingenieurbauwerke regulate Railings in Germany. Railings must withstand a uniformly distributed load of 0.8 kN/m applied transversely to both sides of the handrail. The **DIN V ENV 1991-3** [12] calls for a load of 1.0 kN/m to be applied horizontally and vertically along the railing.

The German railway code **DS 804** gives a maximum gap between infill posts of 0.12 m. The Bridge Code **DIN 18809** calls for a maximum gap between infill posts of 0.14 m. Constructive guidelines for railings are given in the Richtzeichnungen des Bundesministers für Verkehr.

7.5.2 The Hong Kong Highways Department

The Hong Kong Highways Department – Structures Design Manual for Highways and Railways regulates footbridge railings and reflects Hong Kong's unique boundary conditions. It provides the minimum design load values given in Table 7.1. The horizontal gap between infill members must not exceed 0.10 m.

Handrails and Rubrails	Other Rails	End and 90° Corner Posts		Other Posts		Infilling
		Parallel	Normal	Parallel	Normal	
0.7 kN/m	1.4 kN/m	1.4 kN	1.4 kN	1.4 kN	2.8 kN	1.0 kN

Table 7.1 Minimum design loads for parapets according to the Structures Design Manual for Highways and Railways in Hong Kong

7.5.3 Spanish Regulations

The Spanish bridge design code IAP dictates a constant pressure of 1.5 kN/m and 1.2 kN/m respectively to be applied to the railing

Railings should have two handrails, the highest at 0.95 m (+/- 2 cm) from the transit surface and the other at 0.75 m (+/- 2 cm). Handrailings should have a diameter between 45 to 55 mm and be separated of the vertical supports at least 40 mm.

A lateral railing load of value equal to 1.5 kN/m could be applied on the handrailing in order to obtain the local effects needed for the design of the railing and its anchorages.

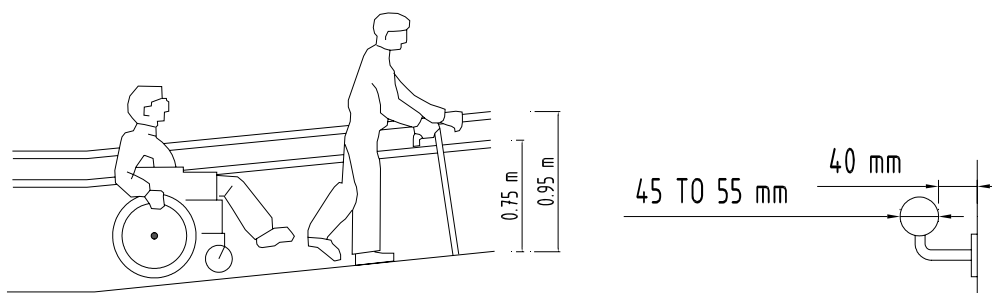


Fig. 7.7 Railing height and handrail arrangement according to Spanish regulations

7.5.4 British Standard

The Department of Transportation Highway and Traffic – Departmental Standard **BD 29/03** – Design Criteria for Footbridges provides requirements for footbridge railings. The handrail must withstand a load of 0.7 kN/m in the vertical and transverse direction.

Handrails must be present on both sides of stairs, ramps and ramp approaches. If the width of the stairway or ramps is less than 3.00 m, a central handrail need not be provided. Handrail height must be between 0.84 m and 1.00 m.

The standard height of parapets on footbridges is 1.15 m. If there are strong winds or if clearance under the bridge is greater than 10.00 m, parapet height may be increased to 1.30 m with the agreement of the technical authority. The minimum parapet height for cyclists is 1.40 m.

The British Standard **BS 5400 – Part 2** calls for a load of 1.4 kN/m to be applied at the height of one meter from the bridge surface to the railing.

7.5.5 AASHTO

The **AASHTO Standard Specifications** for Highway Bridges calls for a minimum railing height of 3'6" (1.07 m) for footbridges. For bridges carrying cyclists, a minimum railing height of 4'6" (1.37 m) is required.

A minimum design load of $w = 50 \text{ lbs/ft}$ (0.73 kN/m) is required to act laterally and vertically on all horizontal railing elements. Posts are not to be more than 5 ft (1.52 m) apart, and posts are required to withstand a force of $w \cdot L$, where L is the distance between posts, acting at the center of the top rail. For cyclist bridges, the design load for posts is to be applied no higher than 4'6" (1.37 m). If the railing requires higher posts, the designer is at his own discretion for determining the load.

The infilling of the railing should be so designed to prohibit the passage of small objects. The requirements for exclusively footbridges are as follows:

- For a height of less than 27 in. (0.69 m) above the bridge deck, a 6 in. (0.15 m) sphere may not pass through the railing.
- For heights between 27 in and 3 ft 6 in. (0.69 to 1.07 m) above the bridge deck, an 8 in. (0.20 m) sphere may not pass through the railing.
- Horizontal elements are maximum 1 ft. 3 in. (0.38 m) apart.
- Vertical elements are maximum 8 in. (0.20 m) apart.

- For infilling with vertical and horizontal elements, only one of the last two constraints apply.

The requirements for cycle bridges are as follows:

- For a height of less than 27 in. (0.69 m) above the bridge deck, a 6 in. (0.15 m) sphere may not pass through the railing.
- For heights between 27 in and 4 ft 6 in. (0.69 to 1.37 m) above the bridge deck, an 8 in. (0.20 m) sphere may not pass through the railing.
- Horizontal elements are maximum 1 ft. 3 in. (0.38 m) apart.
- Vertical elements are maximum 8 in. (0.20 m) apart.
- For infilling with vertical and horizontal element, as before, only one of the last two constraints apply.

7.5.6 Japan Codes

The Japanese Footbridge Design Code (1979) gives rather high railings loads, with 2.5 kN/m to be applied horizontally to the handrail. Furthermore, the Footbridge Design Guidelines for pedestrians (1998) recommends that the railing should be used as shown in fig. 7.8. In this code, two steps railing are recommended in stairways or slopes. In addition, it has also been indicated to be useful for the visually handicapped as the guide if the railing is continuous in all routes from the origin to the end point of bridge.

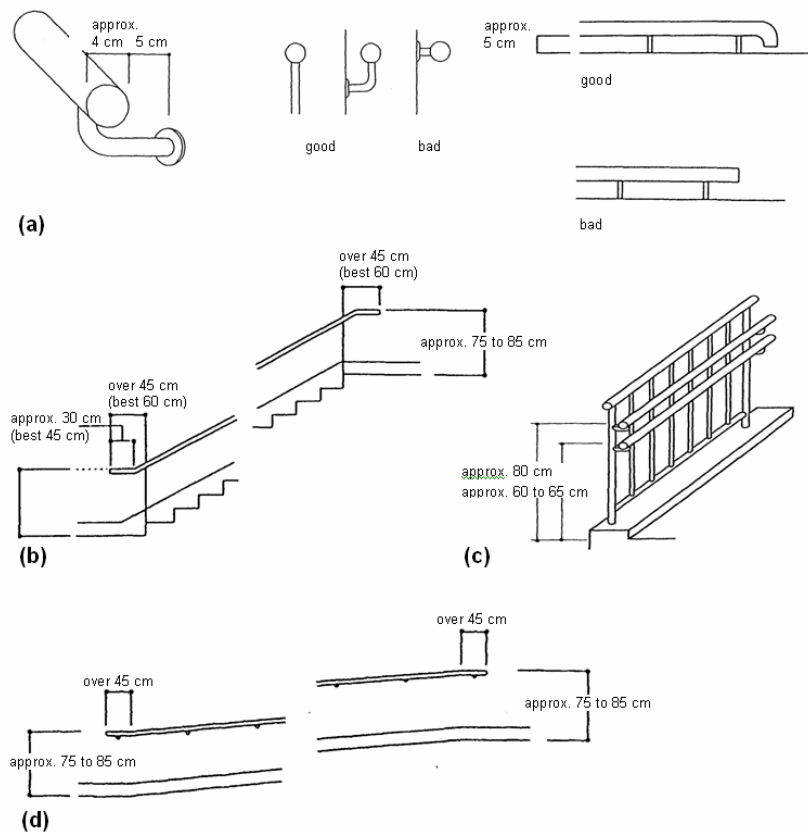


Fig. 7.8 Railing shape and dimension recommended by the Japanese Code: (a) Railing shape and dimension, (b) Dimension in stairway, (c) Two steps railing, (d) Dimension in slope

7.5.7 Canadian Codes

The Canadian CSA-S6-00 dictates a constant pressure of 1.5 kN/m and 1.2 kN/m respectively to be applied to the railing.

7.5.8 Australian Codes

A load of The Australian Austroads 92 Pt 2, Design Loads Code requires the application of vertical and horizontal loads of 0.75 kN/m simultaneously. The Australian code also provides maximum values for deformations (1/800 for the railing and 1/500 for the posts).

8 Illumination

This chapter discusses lighting of footbridges. A discussion of the role and function of footbridge lighting is provided. Some general definitions of lighting theory are provided to better understand lighting requirements. The lamp and lighting fixtures are also discussed. A short discussion of lighting in international codes is also provided.

The lighting of footbridges plays many roles. The lighting should be uniform and provide a safe and agreeable path for those crossing over the bridge and not interfere with other traffic. Lighting also helps the structure fit into its urban or natural environment and can be used to highlight the structure. The color of the lighting plays an important role and must provide for adequate rendering.

8.1 Several Definitions

Luminous flux Φ can be considered as the power of the electromagnetic radiation from a point light source as perceived by the eye. Luminous flux is measured in lumen [lm] and is an important factor in bridge lighting design.

Due to the fact that light is not radiated equally in all directions from light sources, it is necessary to measure the luminous flux for a given solid angle Ω . This is defined as the luminous intensity I , whose unit is the candela [cd].

$$I = \Phi / \Omega \quad (8.1)$$

The illumination E is average amount of luminous flux spread over a given area. Its unit is the lux [lx].

$$E = \Phi / A \quad (8.2)$$

The luminous efficiency describes the ability of a light source to transfer electrical power into light. It is the ratio of luminous flux to expended electrical energy. The luminous efficiency of a light source is a factor in its long-term costs.

A measure for uniformity for a lighting system is the quotient of the minimal to the maximal illumination for the bridge.

$$g_2 = E_{\min} / E_{\max} \quad (8.3)$$

8.2 Guiding the Way

Footbridges have no set point or direction of view, unlike the roadway bridge with its set lanes. This leads to footbridge lighting facilities to be graded according to their illumination as opposed to their luminous intensity.

The lighting must ensure that cyclists and pedestrians are able to recognize the continuation of their path. Curves and path intersections must be clearly marked and may call for additional lighting in these areas. This is also the case for interfaces between stairways and bridge access routes and the bridge itself.

On and near stairways, there is a higher risk of accidents. The German code DIN 5044 suggests a minimal illumination $E_{\min}$ of 15 lx and a value of g_2 greater than 0.3. The first and last steps, as well as the entrances and exits of the stairway must be highly visible, given that most accidents occur in these areas. This can be achieved by either increasing the illumination in these areas or using more visible colors and materials.

Although the perception of the traffic lane at speed is not as much an issue as for footbridges, it is also a factor of design, in particular for those bridges carrying cyclists. An ISO bicycle light provides a horizontal illumination of 0.5 lx at 10 meters. At a speed of 10 to 20 km/h, this is insufficient to make out the path and any obstacles in the way. The “Forschungsgemeinschaft für Straßen und Verkehrswesen” [17], [18] provides recommendations for minimal illumination $E_{\min}$ and uniformity g_2 that can be seen in the following table.

	Paths directly next to unlit streets and roads	Paths within 8 m of unlit streets and roads	Paths near lit streets and roads
Illumination $E_{\min}$	≥ 3 lx	≥ 1.5 lx	≥ 3 lx
Uniformity g_2	$\geq 0,3$	≥ 0.15	≥ 0.15

Table 8.1 Recommended values of cyclists' paths [17], [18]

Glare has both an effect on comfort and our ability to see. Glare can be caused by either a very high luminance or by large differences in luminance in the field of view. High luminance can be due to the lighting system itself or reflection. The choice of materials and surfaces and drainage has an effect on reflection and should be taken into account during design. A greater uniformity of lighting, i.e. a higher g_2 value, can limit the differences in luminance thus reduce glare. The luminance of the bridge lighting should be similar to its surroundings in an urban environment, thus reducing large differences in luminance. In the ‘Richtlinien für die Beleuchtung in Anlagen für Fußgängerverkehr’ [20] recommended limits on luminous intensity $I_{\text{allowable}}$ are given and are dependant on the height of the light source L_{ph} . These values can be found in table 8.2.

$I_{\text{allowable}}$		
γ (luminous emittance)	$L_{ph} \leq 1,5$ m	$L_{ph} > 1,5$ m
100°	10 cd/klm	
90°	30 cd/klm	70 cd/klm
80°		150 cd/klm
60°		3000 cd

Table 8.2 Limit values on luminous intensity [20]

Colors of light can also be used to increase visibility or create an architectural effect. The choice of lighting color should not be confusing for traffic passing below the bridge. Recognizing color contrast is also affected by the lighting color. For increased visibility warm-white or neutral-white is often used on footbridges. The warm-white and neutral-white light permits good color contrast [18].

8.3 Lamps and Lighting Fixtures

Lamps must meet several criteria. They must provide good contrast and lighting color should blend in with the surrounding area. Warm white lights are therefore recommended. Maintenance costs must also be considered. Lamps with high luminous efficiency have lower energy costs. Also the life span of lamps is a large factor in choosing a lamp. Maintenance costs for lamps often outweigh the initial investment cost for an economic design.

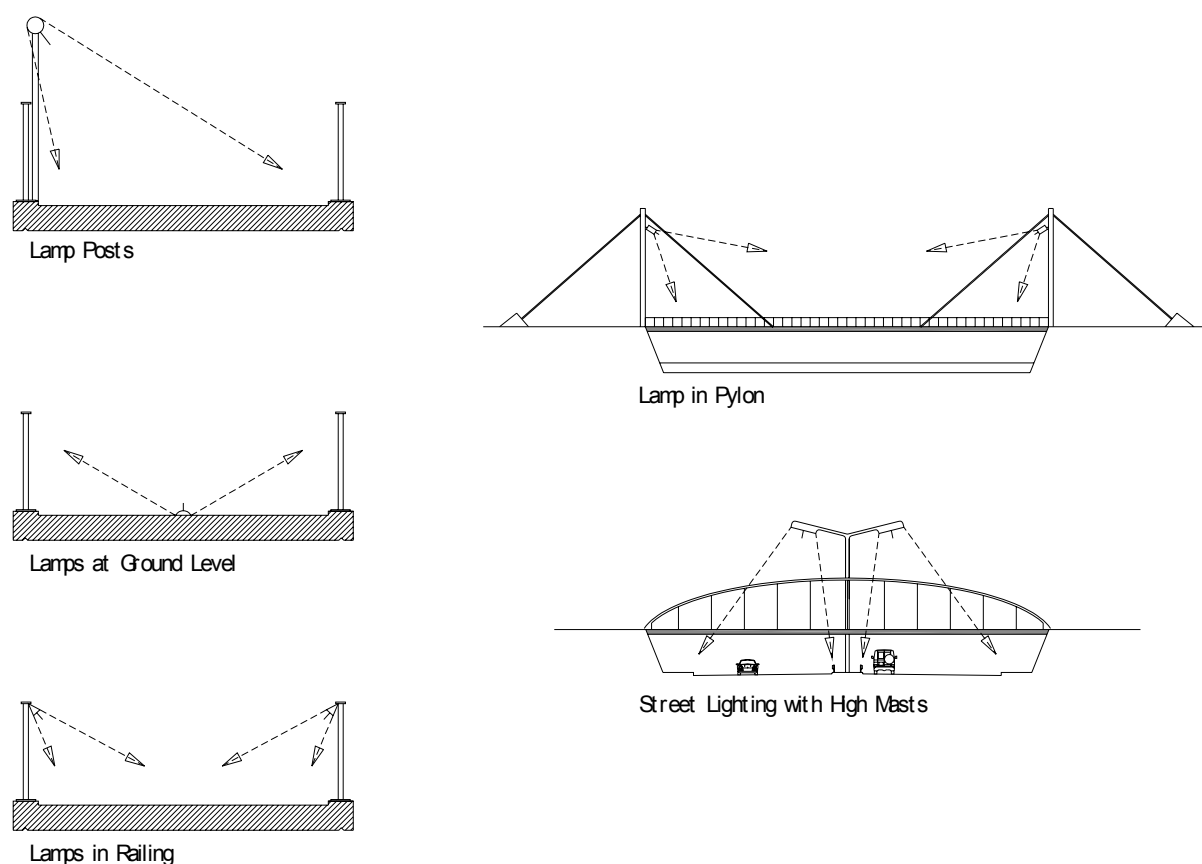


Fig. 8.1 Different lighting systems for footbridges

Lighting fixtures are exposed to the environment and vandalism. This affects the choice of materials. Lighting fixtures must not corrode easily and be robust to avoid damage due to vandalism. This leads to the recommendation that plastics be used in place of glass fixtures where vandalism is a particular problem. Compact lamp designs are also less prone to vandalism.

Lighting with high masts is a design option that provides uniform lighting. This in turn limits glare. Mast lighting also provides a good overview of the bridge area thus increasing orientation. A great luminous flux is needed for this type of lighting. Lighting may also be provided at the pylons of suspension or cable stayed structures.

Lighting from posts is also a design option. Post lighting, while not providing as great an overview as mast lighting, requires less luminous flux. Attention must be paid to lamp spacing to provide uniform lighting and signal any changes in path. Post height can vary for different lamp systems. A post height of 1 – 2 m for spherical white luminaries, 3 m for conical white luminaries, and 4 – 5 m for metal cylinder lamps with mirrors are recommended [17].

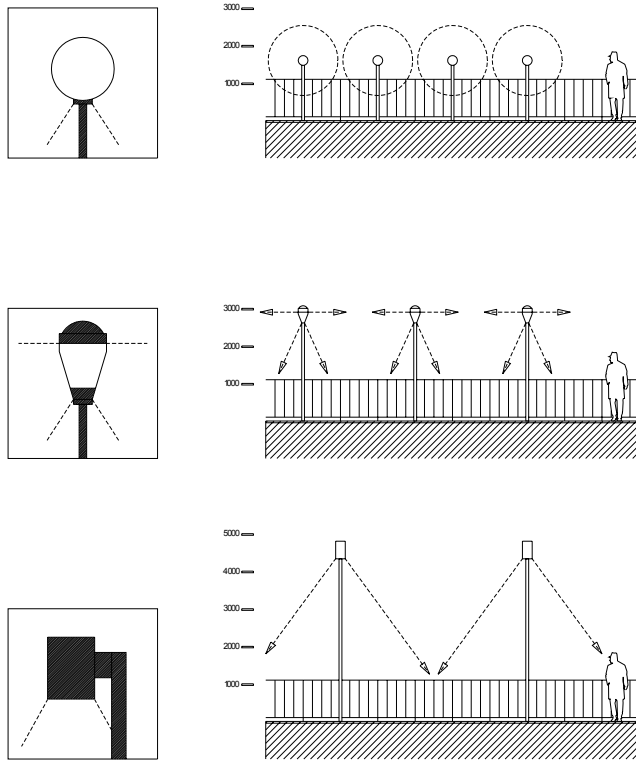


Fig. 8.2 Post height for different lamp systems from [18]

Placing lighting in the handrail is an interesting yet expensive solution to bridge lighting. Rail lighting provides a uniform vertical illumination although the luminance of the walkway may vary greatly. This lighting option is usually constructed using continuous or individual tube lamps. When using individual tube lamps, flickering may occur. Special considerations of vandalism must be made.



Fig.8.3 Handrail lighting at the Nordbrücke in Rostock, Germany, Engineer: Schlaich Bergemann and Partner, Architect: WES & Partner

Placing Light Emitting Diodes (LED) bulbs in railing post is an option used on the Inner Harbor Bridge in Duisburg, Germany. The advantages of the LED lighting are its low energy costs, small size, and long life.

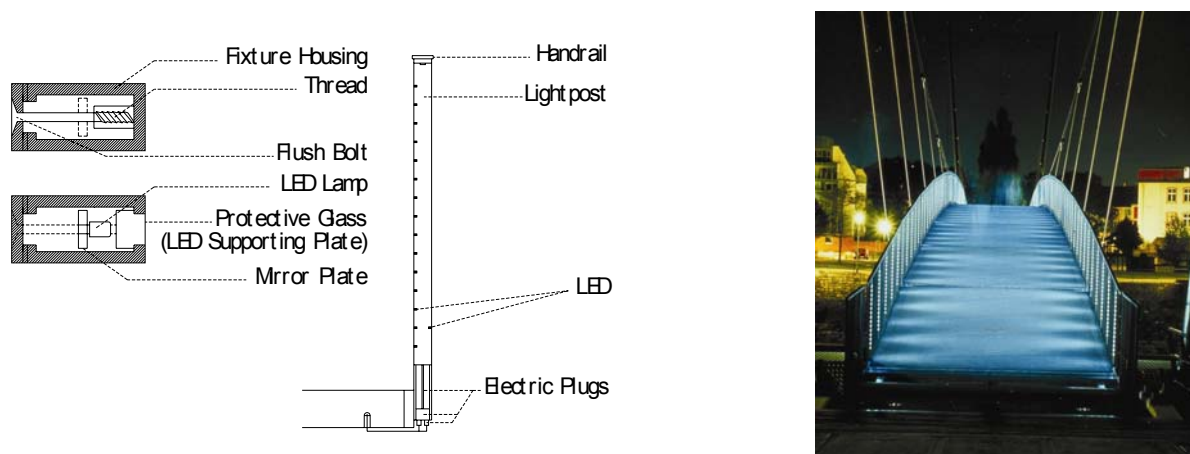


Fig. 8.4 Lighting system for footbridge in Duisburg, Germany, Engineer: Schlaich Bergermann and Partner

Airport runway lights fitted with frosted lenses and powered by reduced current voltage may also light the deck walkway. These lamps shed dispersed light onto the parapet causing it to glow and create ambient effects at night.

8.4 Lighting and Codes

The German **DIN 5044** regulates the lighting for road and highway traffic. While it does not regulate lighting for exclusively pedestrian and cycle paths, it does offer some regulations and suggestions that are relevant to our topic. The German code stipulates that lighting of pedestrian and cycle bridges over roads and highways must not interfere with signals for the traffic below the bridge. The lighting system is not to cause confusion for road traffic beneath it. Due to pollution and ageing, an additional 25 % of luminance should be added to the required named rated illumination. It is then time to replace the lamps, when illumination has dropped to 70 % of its named rated value.

The British **BD 29/03** Design Criteria for Footbridges stipulates that lighting must be provided where public lighting is provided. Bridges may be lit using an addition to existing ground level lighting such as columns and lanterns.

Some Spanish regulations specify a minimum illumination of 10 lux at the beginning and end of ramps to improve accessibility. The **CEI** and **IDEA** regulations suggest $E_{\min} = 20$ lux and a uniformity of $g_2 = 0.40$. These regulations also provide recommendations for minimum maintenance and energy consumption.

8.5 Examples of Lighting Systems

Lighting systems especially in urban surroundings may become quite complex and may require input from lighting experts.

For the footbridge in Leer (Fig. 5.5) the lighting system for the bridge is threefold. First, three floodlights are installed at each of the mastheads to illuminate the cantilever section and the mid span. Secondly, the bank pathways below the bridge are also lit by floodlights in the abutment wall. These floodlights illuminate the underside of the bridge deck, thus illuminating the bank pathway by reflection. Floor lamps near the bridge piers form the third lighting system and illuminate the inside surface of the bridge piers, thus providing ambient lighting. The approach paths are lit by the existing lighting near the paths.

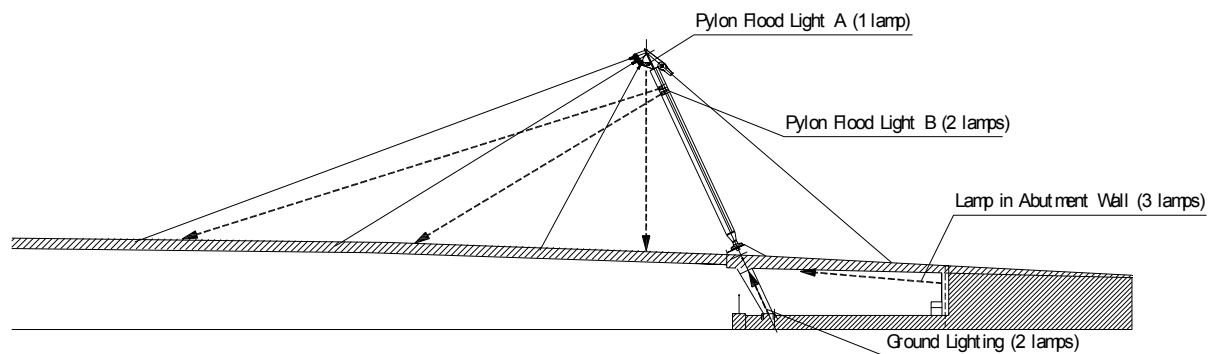


Fig 8.5 Lighting system for footbridge in Leer, Germany, Engineer: Schlaich Bergermann and Partner

The footbridge in Bad Homburg, Germany (Fig. 8.6 to 8.9) also demonstrates a multi level lighting system. The structure is a cable-stayed structure with a forked stone mast that penetrates the bridge deck. There is a stairway to provide access to the bridge structure on one side and ends at the bridge abutment.

Six LED lamps are installed in the vertical surfaces of each of the stairs to provide orientation and emphasize the stairway. The schematic drawing of the lamps and a photo of the stairway lighting are provided below.

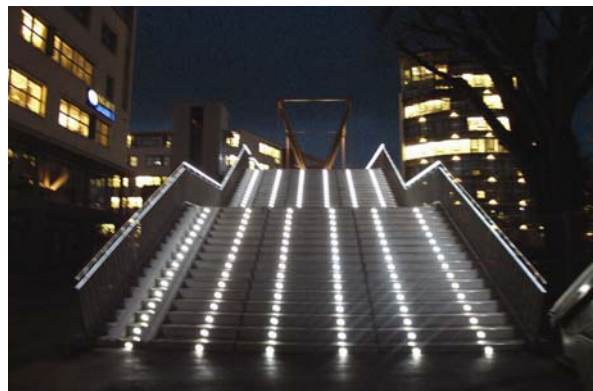
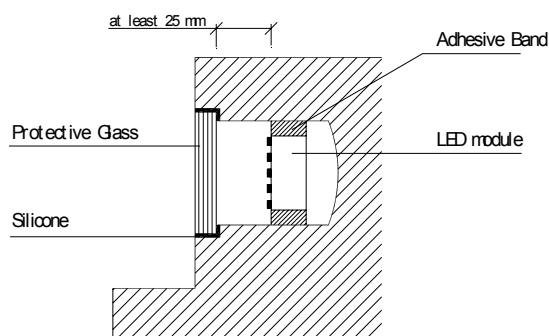


Fig 8.6 Stairway lighting system, footbridge across the Hessenring, Bad Homburg, Germany, Engineer: Schlaich Bergermann and Partner

Ambient lighting of the structure is provided by eight floor lights that accentuate the bridge mast.

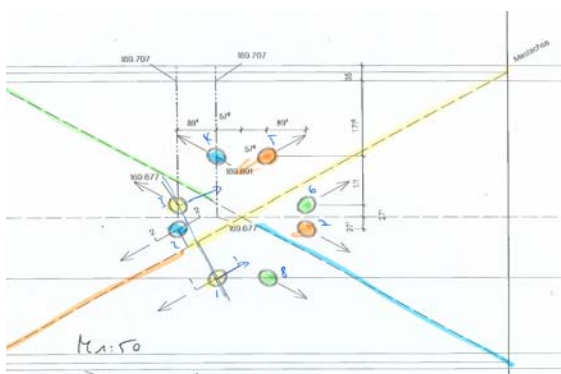


Fig. 8.7 Plan layout of floor lights, lighting of the bridge mast

To accentuate the bridge abutments, a glass curtain wall is placed in front of the abutment wall. The abutment wall surface is white mat. The abutment wall is then lit by a series of lamps and reflects the light to the glass curtain wall. This provides warm, uniform lighting of the curtain wall.

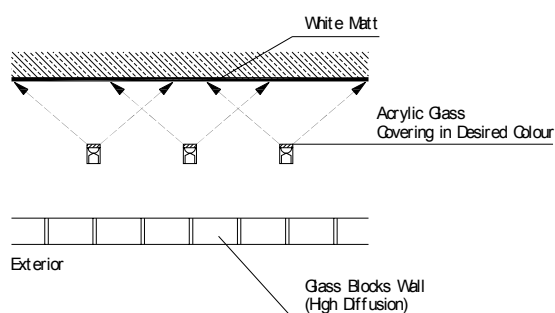


Fig. 8.8 Abutment lighting

Bridge deck lighting is provided by LED lamps provided in the handrails. Details from the handrail lighting system are provided below.

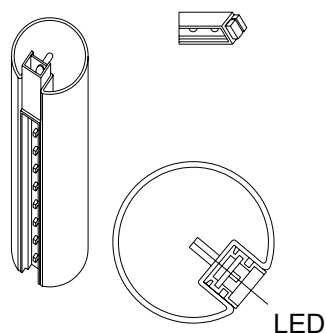


Fig. 8.9 Handrail lighting system

The minimum illumination for the bridge deck was found to be 5 lx. This is higher than the requirements of the German norm **DIN 5044** of 3 lx.

	Values at bridge deck	Requirements of the German codes
Mean Illumination, E_m	11 lx	> 3lx
Maximum Illumination, E_{max}	23 lx	
Minimum Illumination, E_{min}	3 lx	
Uniformity, $g_1 = E_{min} / E_m$	0.24	>0.10
Uniformity, $g_2 = E_{min} / E_{max}$	0.11	

Table 8.3 Values for illumination and requirements according to German code [20]

The semi cylindrical illumination at 1.5 m height (Fig. 8.10) was found to be greater than 1 lx for the entire bridge deck. A diagram of the semi cylindrical illumination at a height of 1.5 m is given below.

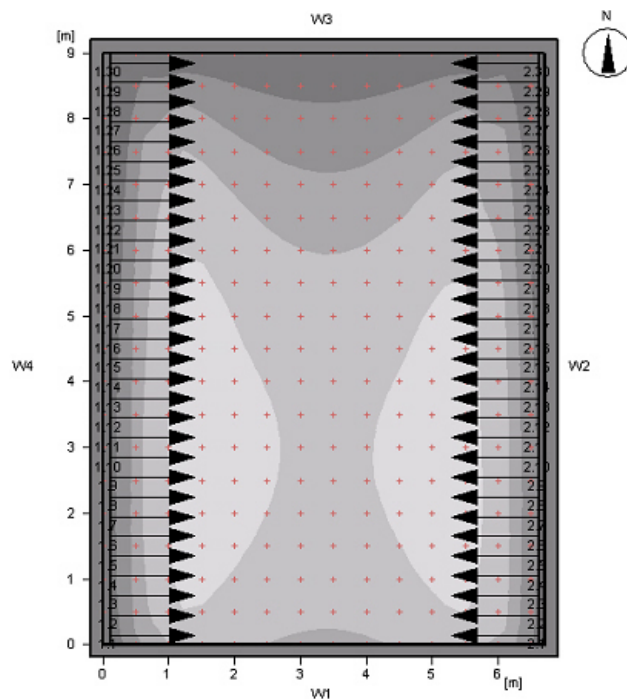


Fig. 8.10 Semicylindrical illumination $E_{h_z,min}$ at 1.5 m height for the footbridge at Bad Homburg, Germany

9 Summary of Information Provided in International Codes

There are numerous codes dealing with footbridges. Some codes, such as the Structures Design Manual of Hong Kong, provide extensive information pertaining solely to footbridge design. Others offer little information on pedestrian and cycle bridges in particular, leaving the engineer to fend for himself and find the relevant information in highway or building codes. Table 9.1 offers an overview of the various codes as well as a guide to their organization. The values of static live, wind and railing loads for footbridges in international codes are summarized in Table 9.2.

Code/Regulation	Country	Geometric Conditions		Static Loads		Dynamic Behavior		Moveable Foot-bridges	Covered Foot-bridges
		Max. Width	Max. Slope	Live Loads	Railing Loads	Frequency Restrictions	Max. Acceleration		
Austroads 13,14, 92	Australia	✓	✓	✓	✓	✓			
NBR 7188	Brazil			✓					
CSA-S6-00 Section 4	Canada			✓	✓				
Canadian Highway Design Code, Section 13	Canada							✓	
OHBDC	Canada						✓		
Dansk Standard	Denmark			✓					
EC 1	Europe			✓	✓				
EC 2	Europe					✓			
EC 5	Europe						✓		
Fascicule Special, 72-21	France			✓					
DIN 18024-1	Germany	✓	✓						✓
DIN 1072	Germany			✓	✓			✓	✓
DIN-Fachbericht 101, 102	Germany			✓	✓	✓	✓		
Structures Design Manual	Hong Kong	✓	✓	✓	✓	✓	✓		✓
Japanese Footbridge Design Code 1979	Japan	✓	✓	✓	✓	✓	✓		
Nederlandse Norm NEN 6788/A1	Netherlands							✓	
Transit New Zealand 1994-Bridge Manual	New Zealand			✓					
SABS 0160-1989	South Africa			✓	✓				
Design Specifications of Road Structures	South Korea	✓							
SIA Code	Switzerland			✓		✓			
BS 5400 Part 2	UK			✓	✓	✓	✓		
AASHTO Guide Specifications for Design of Pedestrian Bridges	USA			✓					
AASHTO Standard Specifications for Moveable Highway Bridges	USA							✓	
UBC and Structures Design Guideline	USA			✓					

Table 9.1 Overview of information in international codes and regulations

Code	Country	Live loads [kN/m ²]	Railings/Parapet [kN/m]	Wind loads [kN/m ²]	Other (snow,...) [kN/m ²]
Austroads 92	Australia	4.00 to 5.00	0.75		
		5.0; up to 80 m ² 4.0, >100 m ² linear interpolation between			
NBR-7188	Brazil	5.00			
CHBDC Section 4, Canadian Standard CSA S-269.11975	Canada	1.60 to 4.00	1.20		
		$p = 5.0 - \frac{s}{30}$ 1.6<p<4.0		Traffic height h=1.50 m	
DS	Denmark	5.00			
EC 1	EU Countries	2.5 to 5.0	1.00		
		q=5.0; for $L \leq 10$ m $q = 2.0 + \frac{120}{L + 30}$; 2.5>q>5.0 for spans L>10 m	(horizontally or vertically at handrail level) 0.80 for service catwalks	→ Bridge code	
Fasicule Special	France	2.00 to 5.00 $a_1 = 2.0 + 150 / (l+50)$ l= bridge span a_1 = surface load			
DIN 1072	Germany	4.0 to 5.0	0.8	wind loads and snow loads as defined in the building code snow: 0.75 to 5.5 snow + wind simult.: snow+0.5*wind or wind +0.5*snow	
		5.0; up to l=10 m $p = 5.5 - 0.05 \cdot l$ >4.0 for structural elements l>10 m	applied horizontally at handrail level	Traffic height of h=1.80 m	0.75 to 5.5
DIN-Fachbericht 101	Germany	see EC 1	0,8		
Structures Design Manual Hong	Hong Kong	1.50 to 5.00			
		As in BS 5400	-	→ Bridge code	
Japanese Footbridge Design Code (1979)	Japan	5.0 (for Deck) 3.5 (for Girder)	2.5	1.0 – 2.0 kN/m ²	-
Transit New Zealand Bridge Manual	New Zealand	2.00 to 5.00			
		$P = 6.2 - s / 25$ $2.0 < p < 5.0$			
SABS 0160-1989	South Africa	5.00 to 7.00			
		7.00 for exposed situation (e.g. railway stations)			
IAP	Spain	4.00	1.5 kN/m at railing		
SIA 260,261	Switzerland	4.00			
		q=4.0 2.5 for service catwalks; different values for SLS calculations			
UBC / IBC and Structures Design Guideline	USA	4.79			
AASHTO (1997): Guide Specifications for Design of Pedestrian Bridges	USA	4.10 Reduction: for footbridges >37m ² $w = 85 \cdot (0.25 + (\frac{15}{\sqrt{A_1}}))$ $> 3,1 \frac{kN}{m^2} = 65 psf$ A_1 in square feet		→ Bridge code	

Table 9.2 Overview of loading in international codes and regulations (Railing loads are given in chapter 7)

Appendix

Any bridge type can be considered when designing a footbridge. However, due to their characteristics the following bridge types are more often used for pedestrian than for road or rail traffic:

- Stress-ribbons: Some (un-built) stress-ribbon bridges have been designed for road traffic, the most famous perhaps being the design for a bridge over the Bosphorus by Finsterwalder. However, most stress-ribbons are footbridges. For pedestrians it is easier to cope with the vibrations of these lively structures and with the rather strong inclinations at the abutments that are necessary to keep the sag and the corresponding horizontal forces of the stress-ribbon within reasonable limits.
- Covered bridges: Modern footbridges are often covered to protect the user. For the user of a road bridge this is less important as the vehicle provides shelter. Examples for covered footbridges include elevated walkways to connect buildings for weather protection or “airport fingers” at airports for additional noise protection.
- Movable bridges: The deck of a pedestrian bridge is lighter than that of a road bridge. Therefore pedestrian bridge decks can be moved more quickly and with less effort. Several rather unique movable footbridges have been designed in recent years.

The Appendices I to III treat these three “exclusive” bridge types in more detail. The structure of the sections is the same:

- Introduction to the structural behaviour in general and reasons for using such bridges.
- Existing types: different structural solutions for each bridge type are described.
- Special considerations: design issues that especially apply to these bridges.
- Examples giving an overview of the great number of such bridges already built.

I Stress-Ribbon Bridges

This chapter deals with the stress-ribbon bridge type. A discussion of existing types of stress-ribbon bridges, design guidelines, and a list of examples are provided. Further detailed information can be found in [49], [84], [85], [86], [87].

A stress-ribbon bridge can be considered as a suspension bridge in which the bridge deck, suspension cable, and stiffening element are combined into one element, the stress-ribbon. The stiffening element works only locally, not in the traditional sense of suspension bridges. Because the structure uses tension forces almost exclusively to carry any load, a very thin ribbon can be used. The slenderness of the structure comes with a price in that bridge foundations must be designed to transfer the immense tension forces to the soil, usually through prestressed anchoring or heavy, massive foundations. The structure is also very sensitive to bearing displacement, which causes a large vertical displacement of the ribbon, as in the system of a stressed cable.

Even a stress-ribbon needs some stiffness and there are several ways to stiffen the cable, i.e. ribbon.

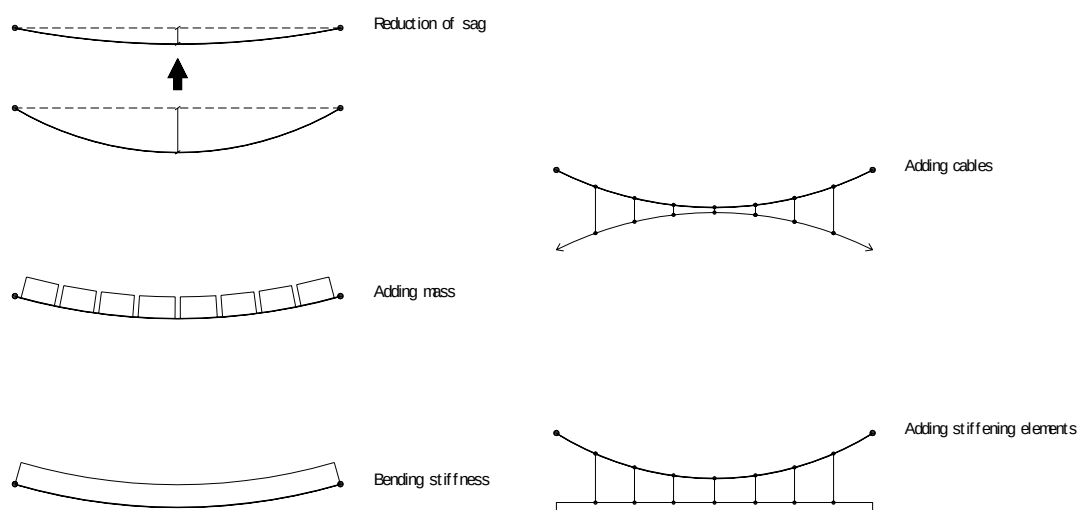


Fig. I.1 Various methods for stiffening cables

There are three main types of stress-ribbon bridges: prefabricated concrete slabs suspended on steel cables, prestressed concrete systems, and systems using steel bands to bear the tension forces.

I.1 The System of Prefabricated Concrete Slabs Supported by Steel Cables

The system of prefabricated concrete slabs laid on steel cables was first used on a series of bridges built between 1979 and 1985 in Czechoslovakia under the supervision of Jiri Strasky. The stress-ribbon consisted of prefabricated concrete segments under prestress. During construction of the bridges, temporary supporting cables were used to bring the prefabricated segments into place. The permanent supporting cables were then pulled through the segments and details requiring on-site concrete were filled. The final prestress was applied once the on-site concrete reached the necessary strength in the joints between segments [85].

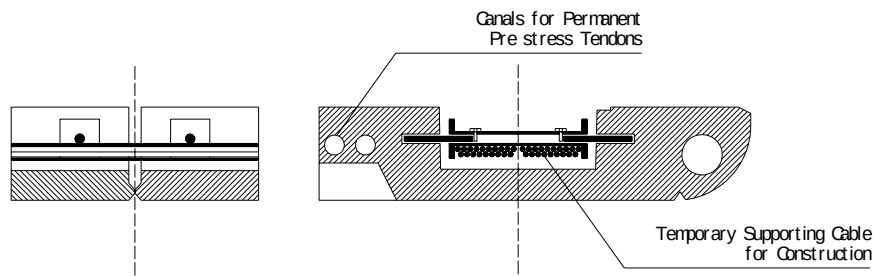


Fig. I.2 Joint and section of prefabricated segment with temporary supporting cable

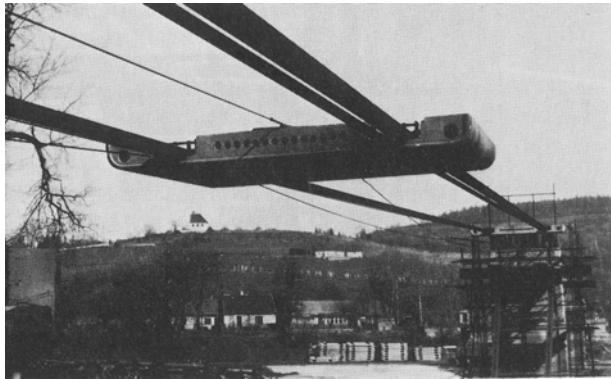


Fig. I.3 Erection of a prefabricated segment

I.2 Prestressed Concrete Stress-Ribbons

Prestressed concrete stress-ribbons were first used in footbridge construction in 1970. The Bicherweid footbridge over the N3 near Pfäffikon Switzerland was designed by R. Walther. The foundations and bridge piers were poured, prestressing tendons were put into place, and the concrete section was finally poured. For the on-site concreting, wagons supported by the prestress tendons already in place were used for the formwork. No scaffolding was necessary.



Fig. I.4 Bicherweid Footbridge, Switzerland, designed by R. Walther.

I.3 Stress-Ribbons Using Steel Bands

One method of construction of stress-ribbon bridges uses steel bands. Concrete or stone may be used as a deck surface, which is then bolted to the supporting metallic band. This method was employed in the Pforzheim III constructed in 1992 in Pforzheim, Germany. Lightweight concrete slabs were bolted to the prestressed steel bands. At the bridge bearings, curved saddles were used to attach the steel ribbons. A minimum radius for the steel bands must be respected to avoid bending the bands which could lead to problems of fatigue in the steel.



Fig. I.5 Enz Footbridge; State Garden Show in Pforzheim, Germany (1992), Engineers: Schlaich Bergermann and Partner, Architects: Knoll, Reich, Lutz

I.4 Saddle Radius and Length

An aid in pre-dimensioning the appropriate saddle radius can be found using the following considerations. For a stressed cable, the axial load in the cable due to a constant vertical load can be given by:

$$H = \frac{q \cdot l^2}{8 \cdot f}$$

where: H = cable force
 q = constant vertical load
 l = length of span
 f = sag

The resulting stress in the section can be found as follows:

$$\sigma_n = \frac{H}{A} = \frac{H}{b \cdot d} = \frac{q \cdot l^2}{8 \cdot f \cdot b \cdot d}$$

where: b = ribbon width
 d = ribbon thickness

The curvature of a beam can be expressed as follows in terms of the bending moment:

$$\frac{1}{r} = \frac{M}{E \cdot I}$$

where: E = Young's modulus for the material
 I = Inertia of section
 r = radius of curvature

It follows:

$$M = \frac{E \cdot I}{r}$$

The stress due to bending can be found as follows:

$$\sigma_m = \frac{M}{W} = M \cdot \frac{d/2}{I} = \frac{E \cdot I}{r} \cdot \frac{d/2}{I} = \frac{E \cdot d}{2 \cdot r}$$

For a maximum design stress of $f_{y,d}$, the necessary saddle radius can be found in dependence on the sag, ribbon thickness and width as follows:

$$r = E \cdot d \left(\frac{1}{2 \cdot f_{y,d}} - \frac{4 \cdot f \cdot b \cdot d}{q \cdot l^2} \right)$$

The length of the stress-ribbon can be found using the following formula:

$$L = 2 \cdot \pi \cdot r \cdot \alpha / 360$$

where: α = $\alpha_{deadload} + 2\Delta\alpha$
 $\Delta\alpha$ = the change in angle at the saddle taking into account deformation due to live loads and construction tolerances

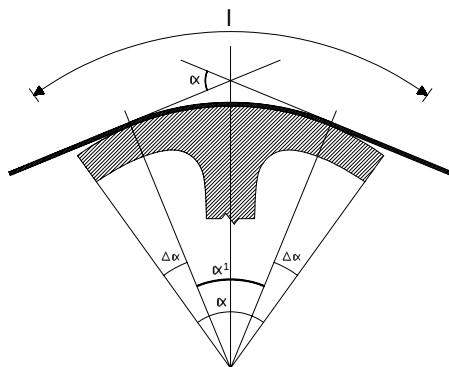


Fig. I.6 Saddle length and change in angle at a stress-ribbon pier

I.5 Special Considerations

As mentioned in the introduction, stress-ribbons do have some aspects that must be addressed. Particular attention must be paid to the foundations and bearings. Foundations must be designed to withstand the high-tension forces resulting from the system. Foundations must also be so conceived so that there is minimal displacement of the abutments. This causes high vertical displacement in the ribbon. A thorough and detailed study of soil conditions is needed to ensure that the stress-ribbon system is economical and appropriate for the site.

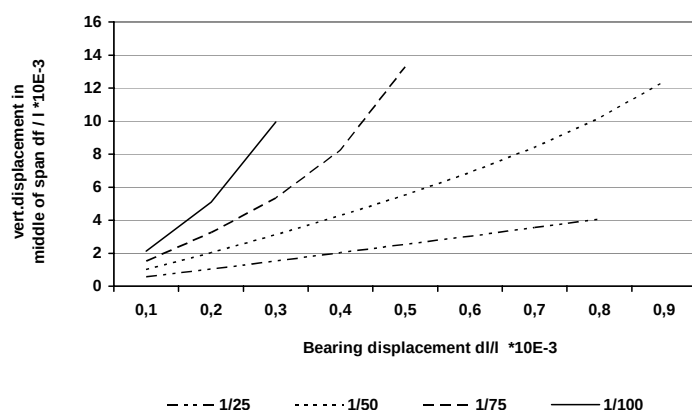


Fig I.7 Relationship between bearing displacement and vertical displacement for different values of sag

Special consideration must be made regarding dynamics. Because these are usually thin constructions, their flexibility makes them susceptible to harmonic vibrations. A detailed dynamic analysis is usually necessary to take this into account. Sufficient damping, for example from handrail systems, must be provided in the system and this may be very difficult to model in the dynamic analysis. In particular, analysis of horizontal oscillations should be undertaken if there is insufficient horizontal rigidity.

The bearing-ribbon interface is critical for the structure. In order to avoid high bending moments at this junction, the appropriate ribbon tangent must be determined. Ribbon saddles must also respect minimum radius requirements to avoid bending and fatigue problems in the metal bands or tendons. In design of the stress-ribbon bridges at the IGA in Rostock, Germany, the problems of saddle radius were solved using laminated springs.

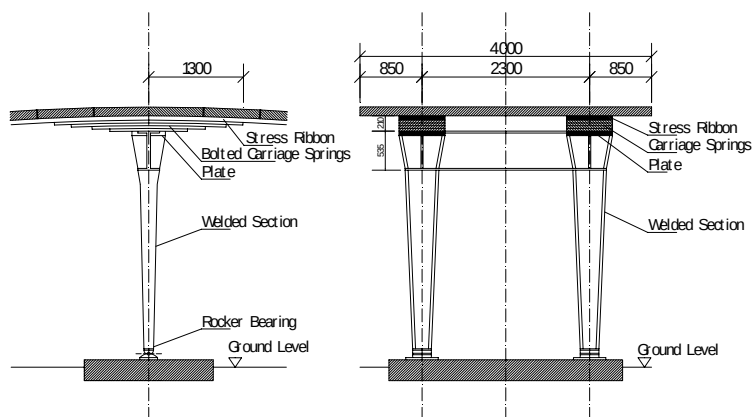
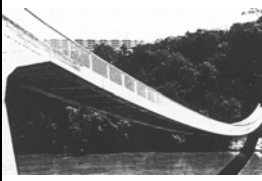
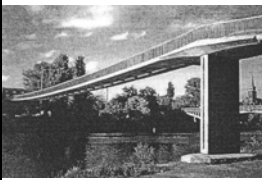


Fig. I.8 Bearing using laminated spring for the IGA in Rostock, Germany; left: elevation; right: section

Examples

	Bridge	Architect/Engineer	Year	Span / Deck thickness	f / L-Ratio; max. Slope	Notes
	1. Transport Bridge of the Cement Producer Holderbank-Wildeck, Germany	Züblin & Chie. AG, Zürich	1963 / 64	Spann: 216 m Ribbon thickness: ~ 25 cm	1/15	
	2. Pedestrian Bridge, Bircherweid near Pfäffikon, Switzerland	Ing.-Büro R. Walther u. H. Mory	1967	Span: 40 m	1/100 19,5 %	First stress-ribbon bridge open to pedestrian traffic. The bridge was constructed using a falsework hung on prestressing tendons.
	3. Footbridge in Freiburg, Germany	Dyckerhoff & Widmann AG	1970	three spans: 25,5 m; 30 m; 34,5 m Ribbon thickness: 25 cm	1/60 14 %	The middle bearing in a linear rocker bearing. The prestressed concrete band was poured using falsework.
	4. Rhône, Geneva, Lignon, Switzerland	H. Weisz, Geneva; O. Wenaweser + Dr. R. Wolfensberger, Zürich	1971	Span: 136 m Ribbon thickness: 8 cm – 40 cm	1/25 17,5 %	
	5. Prag – Troja, Czech Republic	Jiri Strasky	1984	Three Spans: 85,5 - 96 - 67,5 m	1/56,8	Typical "Strasky Bridge" using prefabricated segments hung on supporting cables and then pre-stressed. Monolithic middle bearing
	6. Nymburg, Czech Republic	Jiri Strasky	1985	Three Spans: 46,5 – 102 - 70,5 m	1/51,5	Supporting cables cast into concrete
	7. Sacramento River Pedestrian Bridge, USA	Charles Redfield; Jiri Strasky	1989	Span: 127,5 m	1/44 9 %	Prefabricated segments hung on supporting cable

	8. Pforzheim III – Bridge (Enzauenpark in Pforzheim, Germany)	Schlaich, Bergermann und Partner	1991	Span: 50 m Slab thickness of prefabricated segment: 10 cm Plate thick-ness: 4 cm	1/70	Stress – ribbon constructed using light concrete slabs bolted to steel plates. Abutment constructed using unwinding saddle
	9. Mosbach Pedestrian Bridge, Germany	Schlaich, Bergermann und Partner	1995	Two spans: 23 - 30 m Stress-ribbon thickness: 20 cm	1/60	Construction is continuous and monolithic with the exception of one expansion joint in the ramp. Stress-ribbon widened to follow higher bending moments at bearings.
	10. Unterer Grund Footbridge, (Stuttgart, Germany)	Mayr & Ludescher, Stuttgart	1996	Length: 37 m Span: 23 m	1/100 5,18 %	Stress-ribbon constructed using concrete poured on top of Steel bands.
	11. Punt da Suransuns; Viamala Gorge, Thusis-Zillis, Switzerland	Conzett, Bronzini, Gartmann AG; Chur	1999	Span: 40 m Granite slabs: 60 x 250 x 1100 mm	1/40 20 %	Constructed using granite plates bolted to 4 prestressed steel bands 15 x 60 mm
	12. Bad Oeynhausen/ Löhne, Germany	Schlaich, Bergermann und Partner	2000	Length: 80 m Stress-ribbon spans: 35 – 25 m Section depth: 17 cm	7 %	Stress-ribbon bridge whose middle bearing consists of an arch.
	13. Rogue River Bridge, Grants Pass, Oregon, USA	OBEC, Jiri Strasky	2000	Length: 200.55 m 73.15 m +84.73 m +42.67 m		Three span prestressed concrete stress-ribbon
	14. Blue Valley Ranch Bridge, Colorado, USA	Huitt-Zollars; Charles Refield, Jiri Strasky	2001	76.80 m	1/75	Composite stress-ribbon bridge
	15. Nordbrücke, Rostock, Germany	Schlaich Bergermann und Partner	2003	Three Spans: 27 – 38 – 27 m	1/750	Multi-span steel stress-ribbon
	16. Kent Messenger Millennium Bridge, Maidstone, UK	Studio Bednarski, Strasky Husty and Partners		101.50 m		Prestressed concrete stress-ribbon

Table I.1 Examples of stress-ribbon bridges

II Covered Bridges

The function and structure of covered footbridges are handled here. Design guidelines and international bridge codes are discussed.

Covered pedestrian bridges have long provided lines of communication as well as protection from the elements. Covered bridges have been places of business, housing, and leisure. Covered bridge tollhouses provided an important source of income in early cities as well as housing for officials. Ponte Vecchio in Florence first provided an exclusive plaza and housing for the Duke of Florence and his court and later served as a centre for goldsmiths, jewelers, and moneychangers.

Today many covered bridges provide important links between neighboring buildings. They may become part of the total expression of a building complex, as in the Petronas Towers in Kuala Lumpur. For short spans, a new use of materials, such as glass, and structural systems, such as cable net tubes, is currently being used in projects.



Fig. II.1 Chapel Bridge in Lucerne, Switzerland

II.1 Functional Types

Covered bridges are used to transfer passengers and goods to ships, such as airport fingers and cargo bridges. They provide protection against extreme exposure and sometimes they are even movable (see Appendix III). One can find truly interesting variations in design due to functional requirements; telescoping airport fingers are one example.

The covered bridge, while providing a link between points, also protects the user from exposure to the elements. The type of exposure depends on the geographic necessities of the region or the bridge function. In Switzerland, the roofing protects the bridge structure from high snowfalls, while in Hong Kong they protect the user from heavy rains or winds. The enclosure may also provide protection from sunlight, whereby adequate ventilation must be provided to avoid any greenhouse effects. Airport fingers protect passengers from the elements as well as the high noise pollution created on airport runways. The covered footbridge may also protect traffic passing below from falling objects.

II.2 Existing Structural Types

Covered bridges may be split into three categories: bridges in which the roofing plays no role in the structural system, those in which the roofing takes part in the structural system and those in which the roofing system is the structural system.

Historically, the roofing has played little or no part in the structural system of covered bridges. In the example of early wooden bridges, one can witness the development of the science of engineering. Designers began to perfect trusses and arches and used them in covered bridges. Some bridges began to incorporate the top chord of a truss or an arch into the roofing area, but the roofing system was strictly speaking left out of the structural function. One of the earliest remaining examples of this first category of covered bridge is the Chapel Bridge in Lucerne built in 1333.

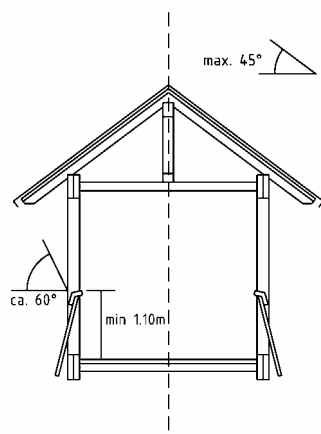


Fig. II.2 Typical cross section for wooden covered bridges

Many modern covered pedestrian bridges incorporate the roofing in the structural system. The Mur walkway in Murnau, Austria by Conzett is a timber structure in which the roofing system acts as a chord for a simple Vierendeel beam.

Dieter Sengler's and Erich Milbrandt's pedestrian and cyclist bridge in Remseck, Germany is another example of roofing playing an important structural role. The bridge acts as a simple beam consisting of a three-dimensional truss where two intersecting planes of the truss covered with glass form the roof enclosure.

II.3 Design Guidelines

Existing information specific to covered bridges in codes:

The **DIN V ENV 1991-3 Eurocode 1** makes a distinction between open and covered footbridges in Annex D. The design load combination differs in the two bridge types. Wind, snow, and live loads are to be considered simultaneously whereby a combination factor specific to pedestrian bridges is applied. In the case of open bridges, these loads must not be applied simultaneously.

According to **DIN V ENV 1991-2-3 Eurocode 1**, the snow load for covered bridges is handled as in the case of building roofing, with all the problems of snow accumulation along edges depending on the shape of the roof.

The German bridge code, **DIN 1072**, stipulates the loads to be applied for footbridges. These values can be found in the Chapter 4 of these guidelines. In the case of wind loads on covered bridges, the surface considered is dependant on the load combination. In combinations where live loads are not considered, the surface subject to wind pressure is that of the structure's projection and any side enclosure. By load combinations in which live loads are considered, a side enclosure with a height of 1,8 m is to be applied even if no side enclosures are present. If side enclosures higher than 1,8 m are present, this height of the enclosures is to be applied. For elements that are not part of the structural system, the wind loads are to be taken from DIN 1055 Part 4. For covered footbridges, the DIN 1072 refers to DIN 1055 Part 5 for the required snow loads.

The **Hong Kong Special Administrative Region Government** has required that all pedestrian bridges be covered or be so constructed as to allow retro-fit roofing. In addition to the required consideration of high wind loads, the Structures Design Manual for Highways and Railways refers to the BS 5400 which calls for a live load of $0,5 \text{ kN/m}^2$ to be applied to the roofing.

The **SIA 261** stipulates that the additional surface on roofs caused by snow must be taken into account for wind loads.

The **British Departmental Standard BD 29/03** specifies that pedestrian bridges with a high risk of objects being thrown from the footbridge or where there is a high suicide risk should give consideration to a partial or complete enclosure depending on site conditions. Adverse weather such as high winds and above average rainfall should lead to consideration of a covered bridge system. The minimum headroom of such a bridge is to be 2.30 m.

II.4 Examples

Covered pedestrian bridges in which the roofing takes part in the structural system

	Bridge	Architect, Engineer	Year	Structure	Max. Span	Length
	1. Lederer, Footbridge, Germany	Julius Natterer	1979	Folded Plate Structure with three Chords	24.2 m	24.2 m
	2. Mercedes Benz Customer Footbridge, Rastatt, Germany	Daniela Kurz, Dietmar Kirsch	1997	Steel Truss	36.0 m	50.4 m
	3. Mur Footbridge, Murnau, Austria	Marcel Meili & Markus Peter, Branger & Conzett	1995	Laminated Timber Beam	46.8 m	~ 57 m
	4. Pedestrian and Cyclist Bridge, Remseck, Germany	Dieter Sengler, Erich Milbrandt	1988	Three Dimensional Truss	80 m	80 m
	5. Airport Finger, Hamburg, Germany	n/A	n/A	n/A	n/A	n/A
	6. Inflated Bridge, Delft, Netherlands	Eventstructures Research Group	1972	Inflatable Pipe	n/A	n/A
	7. Kunsthalle Entrance Bridge, Vienna, Austria	Adolf Krischanitz	1991-1992	Three Dimensional Truss	~ 32 m	~ 69 m
	8. Corporate Street Footbridge, Manchester, England	Hodder Associates, Ove Arup & Partners	1999	Hyperbolic Bar and Cable Structure	18 m	18 m

Table II.1 Examples of covered footbridges for which the roofing takes part in the structural system

Covered pedestrian bridges where the roofing does not take part in the structural system

	Bridge	Architect, Engineer	Year	Structure	Max. Span	Length
	9. London Bridge, London, England	n/A	1176 - 1209	Stone Arch	n/A	n/A
	10. Kramerbrücke, Erfurt, Germany	n/A	1293	Stone bridge	n/A	n/A
	11. Chapel Bridge, Lucerne, Switzerland	n/A	1333	Trestle Bridge	7.65 m	222 m
	12. Ponte Vecchio, Florence, Italy	Taddeo Gaddi	1345	Stone Arch	30 m	84 m
	13. Bridge over the Brenta, Bassano del Grappa, Italy	Andrea Palladio	1569	Strutted Frame	13 m	n/A
	14. Rialto Bridge, Venice, Italy	Antonio da Ponte	1588 - 1591	Stone Arch	27 m	n/A
	15. Bridgeport Bridge, Yuba, USA	David Wood	1862	Truss and Arch	71 m	71 m
	16. Cornish- Winsor, USA	James Tasker, Bela Fletcher	1866	Truss	61.92 m	140 m
	17. Neckar Footbridge, Stuttgart, Germany	Dieter Sengler, Julius Natterer, Holzapfel & Rüd	1977	Truss	72 m	136.75 m

	18. Footbridge in Herrenburg, Germany	Schlaich Bergemann und Partner	1992	Truss Bridge	n/A	n/A
	19. Japan Bridge, Paris, France	Kisho Kurokawa, RFR Paris	1993	Arch and Truss Hybrid Structure	100 m	100 m
	20. Enclosed Walkway, Rotterdam, Netherlands	Kraaijvanger, Urbis, ABT Velp	1994	Glass Beam	3.2 m	3.2 m
	21. Hoffmeister Steg, Bietigheim, Germany	Schlaich Bergemann und Partner	1994	Suspension structure	n/A	n/A
	22. Enclosed Walkway Petronas Towers, Kuala Lumpur, Malaysia	Cesar Pelli & Associates, Thorton Tomasetti Engineers	1992 - 1997	Steel Beam	29 m	29 m
	23. Skywalk, Hannover, Germany	Schulitz + Partner, RFR Paris	1997 - 1998	Suspended Truss	28 m	28 m
	24. Royal Ballet School Bridge	Wilkinson Eyre Architects, Flint & Neill Partnership		Beam Bridge	9.5 m	9.5 m
	25. Passenger Bridge, Hamburg, Germany	n/A	n/A	n/A	n/A	n/A

Table II.2 Examples of covered footbridges for which the roofing does not take part in the structural system

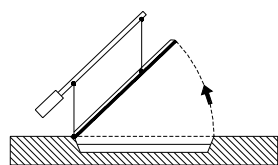
III Movable Bridges

This chapter deals with movable pedestrian bridges. A summary of different types of movable bridge types is provided. The international regulations concerning pedestrian bridges are summarized and a table of examples is provided at the end of the chapter.

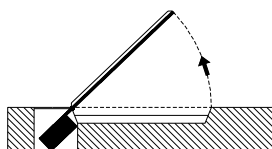
Movable bridges offer an array of different structural and mechanical systems. Movable bridges are considerably more expensive than fixed bridges. Therefore they are employed only if the necessary clearance cannot be reached with a fixed bridge, i.e. if the ramps / approaches would be too long or complex. The design decision may be also dictated by other functional necessities, as with airport fingers.

III.1 Types of Movable Bridges

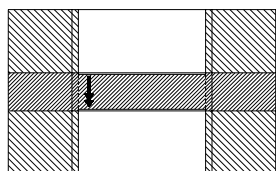
Because site conditions and bridge functions vary considerably, designers' responses have also varied. This has led to the development of numerous types of movable bridges. This section will explain the traditional types of movable bridges. Possibilities seem unlimited, as many recent designers have offered variations or even new types of movable bridges.



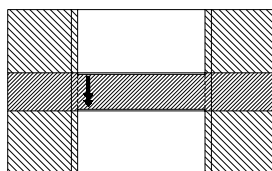
Draw Bridge with Arm of Balance



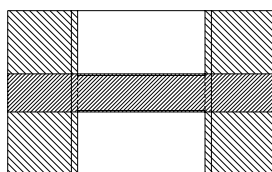
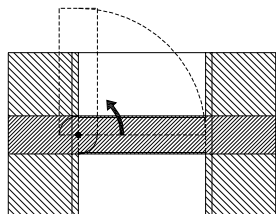
Bascule Bridge



Swing Bridge



Vertical Lift Bridge



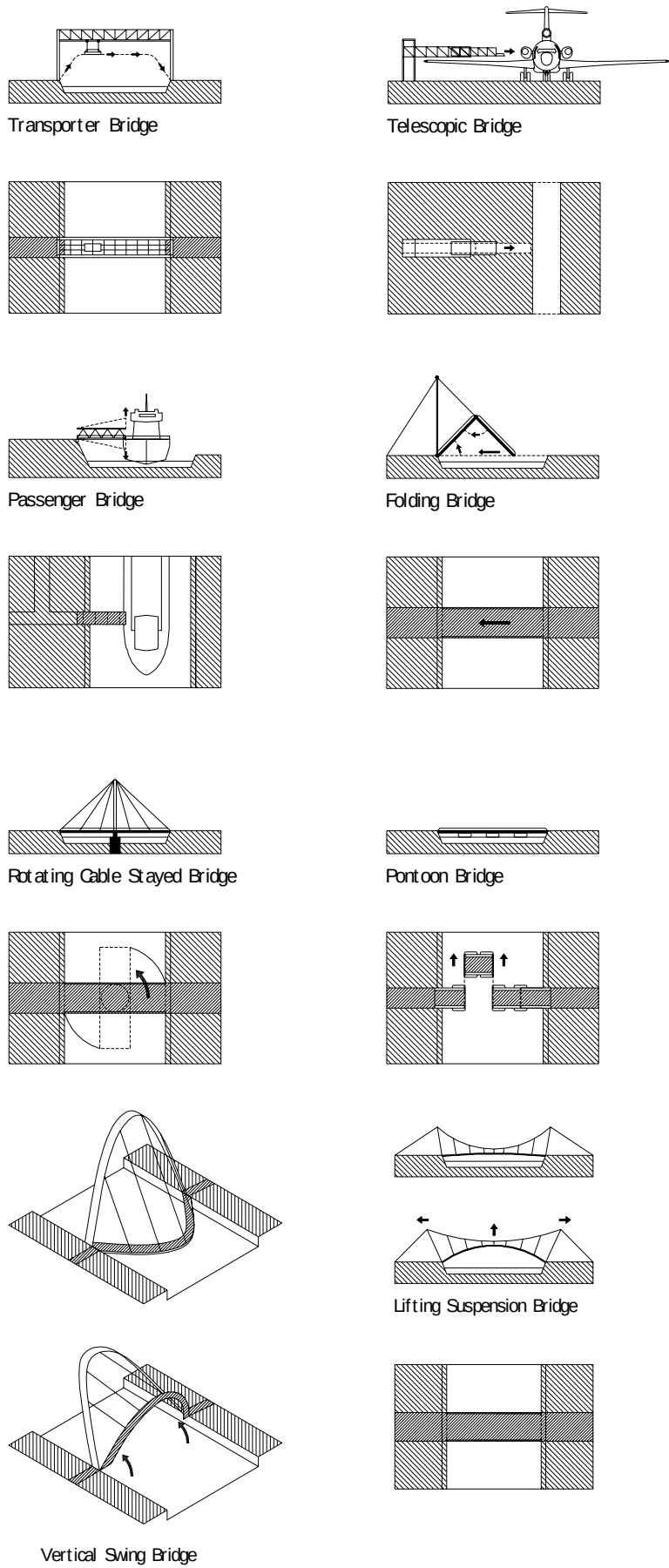


Fig III.1 Types of Movable Bridges

III.2 Bascule Bridges

Bascule bridges rotate around a horizontal axis. Bascule bridges only require additional space above the deck to open which may lead designers to opt for this type of bridge when space to either side of the bridge is limited. Although they take up less space when completed, bascule bridges might inhibit waterway traffic during construction, which is not necessarily the case with swing bridges.

Bascule bridges can be constructed with one rotating element, or two rotating elements and a fixed or rotating connection joining these elements when the bridge is closed. The segments act as cantilevers when the bridge is open. This cantilever system may lead to foundations subject to large moments and thus more complex abutments.

Double bascule bridges are connected in the middle of the closed span. This connection can either be a fixed or freely rotating. Care must be taken to ensure that this connection is able to close without complication and a sufficiently even deck surface is provided. Sufficient drainage must be available and corrosion problems must be addressed for these connections. Play in this connection will cause abrasion that may eventually lead to locking of the connection.

All of the bridge joints must be designed to take into account of water and road salts that may cause corrosion in the joints and interfaces and in particular all mechanical elements and cables must be sufficiently protected from such effects.

The axis of rotation of the bridge divides the structure into two arms or cantilevers. This axis may require transverse girders to improve the torsional stiffness of the structure and ensure equal rotation of the main girders. The ratio of fore-cantilever to aft-cantilever lengths vary from 1:1 to 6:1. With an increase in length in the forward cantilever, rotating the bridge about its axis becomes more and more difficult. This leads to the necessity of a counterweight attached to the aft-cantilever. Due to the cantilever system of the bascule bridge, and attempts to diminish weight, longitudinal bridge girders often have a tapered section.

Counterweights are usually made of concrete or, if space is limited, steel. The mass of the counterweight should not exceed that of the forward cantilever; otherwise the bridge may have a tendency to open when subject to non-uniformly distributed live loads. Bascule bridges whose center of gravity is found in the forward cantilever will have a tendency to close in their open state.

Counterweights may be located above or below the deck surface. Counterweights below the deck surface must take into account flooding of the counterweight chamber and usually call for more complex abutments that must be designed to house the counterweight chamber.

Drawbridges are another type of bascule bridge and have been used since the Middle Ages. They are often employed for spans less than 40 m. For very light constructions and short spans, Drawbridges may be moved without the aid of a counterweight. If a counterweight is needed, these bridges often use balance-arms to counteract the weight of the bridge deck. Drawbridges with balance beams often necessitate a transversal girder or bracing between them, which helps to stabilize the system in the transverse direction and ensure equal displacements.

III.3 Vertical Lift Bridges

In a vertical lift bridge the deck is left horizontal and lifted vertically with the use of levitation towers. These towers may be free standing, joined to form a frame, or they may sink into the foundations to close the bridge. Towers that sink into the foundation are usually more costly, calling for very deep foundations to house them. Vertical lift bridges are often used for bridges with long spans, due to the fact that the bridge rests on two supports, which is

more advantageous than the cantilever system of bascule bridges. Care must be taken to ensure that the levitation towers undergo the same settlement, otherwise there may be locking during opening and closing.

III.4 Swing Bridges

Movable bridges that rotate along a vertical axis are referred to as swing bridges. Sufficient space to the side of the bridge must be available for the opening. This is a disadvantage for this form of bridge in relation to bascule or vertical lift bridges. Swing bridges do offer the advantage that they can be built without impeding water traffic, unlike some bascule bridge forms.

The system consists of a continuous beam over three supports when closed and two cantilevers balanced on a central support when opened. These cantilevers may be of equal or unequal length, for which a counterweight may be necessary. The bridge structure rotates around a bridge bearing which must be specially conceived to allow this rotation. These bearings must also provide sufficient stability when the bridge is opened and subject to asymmetric loads. The bearings must be able to bear horizontal forces.

III.5 Folding Bridges

Folding bridges consist of two or three elements that are connected with horizontal axes of rotation. The bridge deck is connected to pylons by rods. These rods pull on the bridge deck to cause a folding of the deck. This bridge type requires more complicated connections between deck segments and sufficient protection against corrosion must be provided.

III.6 Transporter Bridges

A truss constructed high above the waterway is used to carry a transport platform that is hung from its underside. The transported platform is then driven across the truss using a system of cables. The height of water traffic is only limited by the underside of the truss.

III.7 Telescoping Bridges

Telescoping bridges consist of several tubes or plates. These elements are connected in such a way as to allow them to glide over each other thus regulating the bridge length. Telescoping bridges are often used as airport-fingers.

III.8 Passenger Bridges

Used to allow access to ships, passenger bridges must allow for a variable ship levels. These bridges are usually connected to the shore with a connection that serves as an axis of rotation. The structure is then hung via a movable connection on a fixed bearing. The variation in height is caused by a regulation of this movable connection, often using cables.

III.9 Pontoon and Swimming Bridges

Bridges over waterways may be supported on ships, rafts or pontoons. Modern movable bridges employ pontoons due to their low maintenance costs. Pontoons should be of such a form as to minimize drag, which may cause horizontal displacement. Sections of the pontoon bridge may be moved out of the way in the case of ship traffic.

III.10 Movable Bridges in Codes and Standards

There are several codes dealing exclusively with movable bridges. Many countries prefer to mention design guidelines for movable bridges in their standard bridge codes, while others make no specific mention of movable bridges, leaving the bridge designer to fend for himself. The plethora of ways of handling movable bridges may leave the designer bewildered. For example, the Austrian regulation makes an interesting distinction in the case of movable bridges. When closed, the designer must apply the bridge code, but when the bridge is open, the Austrian building code. While the bridge is opening or closing, the bridge is to be treated as a machine. For an overview of the load codes for movable bridges see Table 9.1.

The American regulations are contained in the **AASHTO Standard Specifications** for Movable Highway Bridges 1988 Edition and Appendix A of the proposed LRFD specifications.

Locks or safeties must be provided to ensure safe positioning and prevent unwanted horizontal and vertical displacements during motion.

Counterweights are handled in the American code. They are assumed to be necessary for vertical lift bridges of over 80 ft height difference. The mechanisms for all movable bridges must be designed taking into account loading outside of an equilibrium state in spite of a counterweight. Swing bridges with unequal arm lengths should be balanced using a counterweight. Counterweights are usually constructed on concrete poured into a steel container. Concrete counterweights without steel containers must be reinforced.

Two gates are to be provided for movable bridges. The first gate is a warning gate; the second gate acts as a barrier.

Lights are obligatory for movable bridges, unless the bridges may be opened manually. Lights may be supplemented with sirens.

Live loads for movable pedestrian bridges are set at 4.15 kN/m² (85 lbs/ft²).

Wind loads must be applied axially, transversely, and diagonally at an angle of 45° for tests of structural integrity.

Wind direction relative to bridge axis	Transverse	Axial	Diagonal
Pressure	$p_T = \frac{1}{2} \cdot C \cdot \rho \cdot V^2$	$p_L = 0,5 \cdot p_T$	$p_D = 1,05 \cdot p_T$

Table III.1 Wind Load on bridges according to AASHTO Standard Specifications for Movable Highway Bridges

If the bridge is closed, the wind loads are calculated as a conventional bridge according to AASHTO. During opening and closing and in the opened state, wind loads are given in Table III.2.

State	Bridge during opening or closing	Bridge is open
Load	30 lbs/ft ² (1.46 kN/m ²)	50 lbs/ft ² (2.44 kN/m ²)

Table III.2 Wind Load on Movable bridges according to AASHTO Standard Specifications for Movable Highway Bridges when opening and in the opened state

Bridges with grid decks are subjected to 85% of the wind load of a solid deck.

Swing bridges are subject to a simultaneous load combination of 50 lbs/ft² (2.44 kN/m²) on one arm and 35 lbs/ft² (1.7 kN/m²) on the other.

Earthquake loads are to be applied in the opened and closed state. If the bridge is in a certain state for less than 10 % of the time, earthquake loads may be reduced by 50 % for this state.

Movable bridges must also be examined for fatigue.

Swing bridges must be examined for the following load combinations:

1. Dead load while opened or closed without locks
2. The closed bridge is loaded with the dead load. Depending on which load is more unfavorable, the dead load is combined with either a displacement of bridge ends of 1 in. (2.54 cm) or 150 % of the temperature load, the live load and a load at the connection.
3. The bridge is closed without locks and subject to a live load on one arm and an impact load.
4. The bridge is closed and subject to a live load on both arms and an impact load.

Maximal and minimal stresses can be calculated using 120 % of the first load combination, the first load combination combined with the third load combination, or the second and fourth load combinations combined.

Particular attention must be given to the bridge pier. Any sinking or displacement of the pier can render the movable bridge useless. Bridge piers must be able to bear horizontal forces and twisting moment due to the rotation of the bridge.

Temperature differences between upper and lower chords of 20°C for trusses and 15°C for girders must be taken into account.

Single bascule bridges must have an additional bearing on the end of the cantilever. Double bascule bridges must have a lock in the middle of the complete span that transfer transverse force and ensure the same deflection of the cantilevers in the middle of the span under non-uniform live loads.

Bascule bridges must be subjected to the following base load combinations:

1. Dead load on the open or opening bridge.
2. Dead load on the closed bridge.
3. Dead load on the closed bridge with consideration of a simply joined counterweight.
4. Dead load, live load and impact load on the closed bridge.

Maximal and minimal stresses can be calculated using 120 % of the first load combination, the second combination combined with the fourth load combination, or the third and fourth load combinations combined.

The following load combinations are to be considered for vertical lift bridges according to the American code:

1. Dead load on the opened bridge
2. Dead load on the closed bridge
3. Dead load without counter weight
4. Dead load and live load on the closed bridge

Maximal and minimal stresses can be calculated using 120 % of the first load combination, the second combination combined with the fourth load combination, or the third and fourth load combinations combined.

Towers and lift mechanism must be loaded with 120 % of the dead load. The wind load for the towers is given by 50 lbs/ft² (2.44 kN/m²). Lift towers are either partly or completely fixed. Bridge abutments must therefore be subjected to all live loads (except impact loads).

Movable bridges are handled in the standard German bridge code, **DIN 1072**. According to Section 3.1.1, an additional dead load of 0.25 kN/m² must be applied to the bridge deck surface. This additional load is meant to take deviations in paving or surfacing into account and must be applied to bascule bridges in all the various stages of their movement.

Wind loads for opened movable bridges are taken as 70 % of the conventional maximal wind load acting on the structure in its closed position. The wind surfaces to be considered for bascule bridges are the total surface subject to wind along the bridge axis in the opened position, as well as horizontal wind loads acting on the structure. For swing bridges, the wind load is to be applied to only one arm of the bridge. The wind load is also decreased during the opening and closing of the bridge, see Table III.4.

	1	2	3
Height of surface subject to wind pressure above ground	Load combination without live load: Superstructure without piers, columns and Sound Proof Wall	Final position of opened bridge: 70% of the values in Column 1	Intermediate positions during opening: 30% of the Values in Column 1
0 to 20 m	1.75 kN/m ²	1.225 kN/m ²	0.525 kN/m ²
20 to 50 m	2.10 kN/m ²	1.47 kN/m ²	0.63 kN/m ²
50 to 100 m	2.50 kN/m ²	1.75 kN/m ²	0.75 kN/m ²

Table III.4 Treatment of wind loads for movable footbridges in the German Code DIN 1072

Snow loads of 0.75 kN/m² must be applied to opened bridges in the most unfavorable position. The snow load is vertical and based on the horizontal projection of the bridge in its opened position.

Additional loads due to accelerations of the bridge during opening and closing must also be considered according to Section 4.6 of the German bridge code.

The Dutch Nederlandse Norm **NEN 6788/A1** regulates movable bridges. It makes a distinction between movable bridges that:

- Rotate around a horizontal axis, such as bascule bridges, and those that rotate around a vertical axis, such as swing bridges.
- Can be displaced along a horizontal translation or a vertical translation, such as vertical lift bridges.
- Rotate around a horizontal axis and simultaneously undergo a horizontal translation.

The Dutch code examines three states of the bridge, open, closed, and in motion while opening or closing. All loads are applied to the superstructure, the mechanics, and the foundations.

Wind loads are determined under consideration of the following factors: the bridge size, a wind pressure coefficient dependant on bridge type and wind directions, the dynamic wind pressure dependent on the bridge height, and a dynamic load factor.

Snow loads are set relatively low due to meteorological conditions in Holland. Snow load values are set at 100 N/m² for movable bridges using mechanics or hydraulics and 50 N/m² for bridges that open by hand.

A provision is made for temperature differences for both the superstructure and the bridge mechanism. Temperature differences range from -25°C to +45°C.

An addition to the dead weight of the structure is made to take into account the effects of the system change during opening and closing. This additional load must be applied to the bridge mechanics.

A load due to friction and dependant on the type of bearings used is also taken into account.

Dynamic loads due to an emergency braking of the structure are also taken into account. The time of acceleration and brake time must not exceed a certain value. These range from 3 to 6 sec for bridges with surface areas below 125 m², and 6 and 12 sec for larger bridges.

The **Canadian Highway and Bridge Design Code** Section 13 is a regulation dedicated exclusively to movable bridges. The code mostly offers rules for the design of movable bridge elements and installations although mechanical elements are also handled. Live loads are to be taken from the conventional bridge code.

The Canadian code offers many design considerations for movable bridges. It stipulates that the superstructure should be light, opening procedure should be quiet, and mechanical elements should be protected. There must be fire and smoke protection for the engine house if there are any flammable materials.

The procedure of opening and closing the bridge must not exceed 2 min. Signals announcing the opening of the bridge may only stop working when the bridge has completely shut. Gate prohibiting passage to the bridge must also be kept down until the bridge is fully closed.

Connections and locks must be in place and prevent displacement of the bridge under horizontal and vertical loading. This displacement is limited to 15 mm. Loads due to ship collisions must be considered in the fully open and fully closed state.

Wind loads are handled according to Table III.5

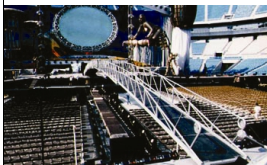
Type of bridge	Swing bridge			Bascule bridge		Vertical lift bridge	
Wind direction	horizontal		vertical	horizontal		horizontal	
Surface subject to wind load/ wind direction relative to bridge length	Arm 1	Arm 2	Arm 1	transverse	axial	transverse	axial
Load (kPa)	1.20	1.70	0.25	1.50	1.50	1.44	1.50

Table III.5 Wind loads and directions according to Canadian Highway and Bridge Design Code Section 13

Earthquake loads are decreased by 50 % for the bridge in the open position.

A temperature difference between upper and lower chords must be taken into account for swing bridges. This difference is given as 10°C for trusses and 15°C for girders.

III.11 Examples

	Bridge	Architect, Engineer	Year	Structure	Total Width Total Length	Movable Length Clearance
	1. Locomotive Bridge, Huddersfield, Great Britain		1865	Vertical lift Bridge		
	2. De la Fusta Dock, Barcelona		1981 - 1987	Drawbridge with Arm of Balance		
	3. Limekiln Dock Footbridge, London	Anthony Hunt Associates	1996	Swing Bridge	3 m	34 m
	4. St. Savior's Dock Bridge, London	Nicolas Lacey & Partner, Whitby & Bird	1996	Swing Bridge	15.2 m	9 m
	5. West India Quay Footbridge, London	Future Systems, Anthony Hunt Associates	1996	Bascule and Swim Bridge	2.4 m - 4.8 m 94 m	2 m x 6.6 m
	6. Bascule Bridge, Wolgast, Germany		1997	Bascule Bridge		
	7. Bridge over the Amtsgraben, Berlin	Senatsverwaltung Bauen, Wohnen und Verkehr	1997	Drawbridge with Arm of Balance	3.0 m 7.79 m	7.79 m
	8. Bridges to Babylon	Atelier One	1997	Telescoping Bridge	120 m	120 m

	9. Footbridge over the Flaz River; Pontresina, Switzerland	Inst. für Hochbautechnik, ETH Zürich Prof. Dr. O. Künsle	1997	Truss Bridge	1.9 m 2 m x 12.5 m	2 m x 12.5 m
	10. South Quay Footbridge, London	Wilkinson Eyre Architects; Jan Brobowski & Partner	1997	Swing Bridge	180 m	90 m
	11. Three Segment Folding Bridge, Kiel	Gerkan, Marg & Partner; Schlaich Bergemann und Partner	1997	Folding Bridge	5.0 m 120 m	25 m
	12. Lowry Centre Footbridge, Manchester	Carlos Fernandez Casado, Parkman	1998	Vertical Lift Bridge	7 m 92 m	92 m
	13. Royal Victoria Dock Bridge, London	Lifshutz Davidson	1998	Transporter Bridge	6 m 135 m	6 m x 9 m +/- 14 m
	14. Waluliso Bridge, Vienna	Valentinitsch Design; Öhlinger und Metz	1998	Swimming bridge		
	15. Katzbuckel Bridge, Duisburg	Schlaich Bergemann und Partner	1999	Suspension Bridge	1.5 m 73.7 m	77.4 m +/- 10 m
	16. Backpack Bridge		2000	Foldable Bridge	10 m	
	17. Bascule Bridge over Versacker Harbor, Bremen	DesignLabor Bremerhafen, Arup GmbH Berlin	2000	Bascule Bridge	2.75 - 6.20 m 42 m	18.2 m

	18. Forton Lake Opening Bridge, Gosport, Great Britain	Mamsell Ltd., Univ. of Southampton	2000	Drawbridge	5.0 m 170 m	18 m
	19. Gateshead Millenium Bridge, Newcastle	Wilkinson Eyre Architects; Gifford & Partners	2000 15 Mio. Pounds	Tilting Bridge	5.6 - 7.0 m 105 m	105 m +/- 20 m
	20. Airport Finger			Telescoping Bridg		
	21. Drawbridge over the Castell, Lagos, Portugal			Drawbridge		
	22. Marina Bridge, Lagos, Portugal			Drawbridge		
	23. Drawbridge, Roskilde, Denmark			Drawbridge with Arm of Balance		
	24. Harbor Passenger and Cargo Bridges					
	25. Offshore Rig Bridge			Telescoping Bridge		77.4 m

Table III.5 Examples of movable bridges

Case Studies

Introduction

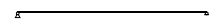
The guidelines would not be complete without examples of good bridges: footbridges that show beauty and well-functioning structural concepts; examples of the joy of engineering and the good collaboration of architects and engineers.

Thirty-three cases from all over the world, all built during the last ten years, are studied each on one page. Admittedly, this is a rather subjective and, therefore, incomplete collection. Still, the variety displayed here demonstrates that in footbridge design there are few limits to our imagination.

Illustration Credits

Pictures for the Case Studies were provided by the designers.

33 Case Studies



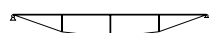
Mursteg Murau, Austria (1995)
Saint Feliu Footbridge, Spain (1996)
West India Quay Bridge, United Kingdom (1996)
Sakaemachi Greenwalk, Japan (1998)
Footbridge Rapperswil - Hurden, Switzerland (2001)
La-Ferté-Steg, Germany (2001)
Shibuya 21 Bridge, Tokyo, Japan (2001)
Pasarela Peatonal de Mondragón, Spain (2002)
Bridge of Aspiration, United Kingdom (2003)



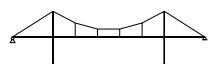
Footbridge over Rhein Herne Canal, Germany (1997)
Passerelle Solférino, France (1999)
Gateshead Millennium Bridge, United Kingdom (2001)
Footbridge across the "Ronda de la Hispanidad", Spain (2002)



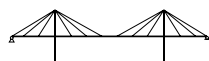
Pünt da Suransuns, Switzerland (1999)
Kent Messenger Bridge, United Kingdom (2001)
Rogue River Bridge, USA (2000)
Nord Bridge, Germany (2003)



Inachus Bridge, Japan, (1994)
Shiosai Bridge, Japan (1995)
Traversinasteg I, Switzerland (1997)
Glass Bridge in the Basilica of Aquileia, Italy (1998)
Footbridge Gaißau, Austria (1999)
Glass Bridge Haute-Provence, France (2003)

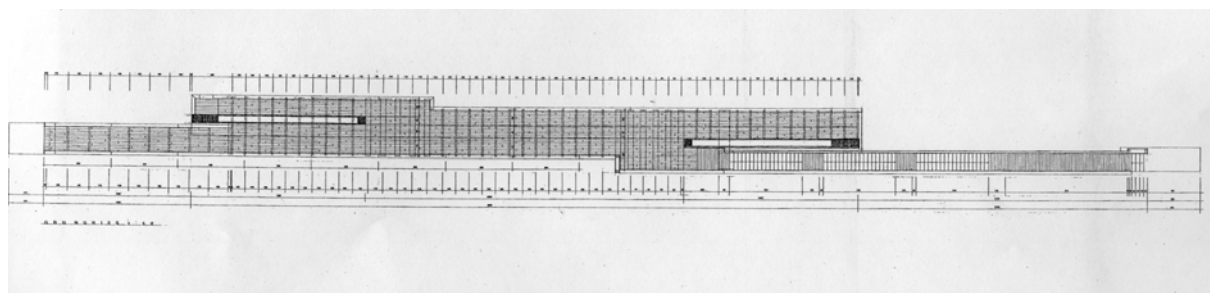
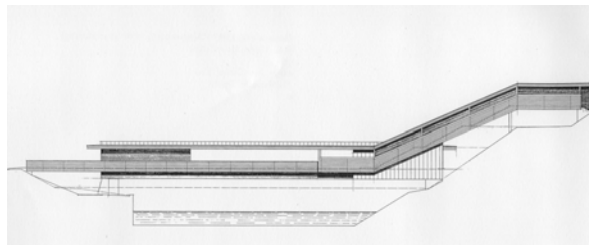


Footbridge across the Rhone, Switzerland (1998)
Katzbuckel Bridge Duisburg, Germany (1999)
Millennium Bridge, London, United Kingdom (2000)
Halgavor Bridge, United Kingdom (2001)
Footbridge over the Gahlensche Straße, Germany (2003)
Traversinasteg II, Switzerland (2005)



Challenge of Materials Bridge, United Kingdom (1997)
Royal Victoria Dock Footbridge, United Kingdom (1998)
Manzanares River Footbridge, Spain (2003)
Nesse Bridge, Germany (2005)

Mursteg Murau, Austria (1995)



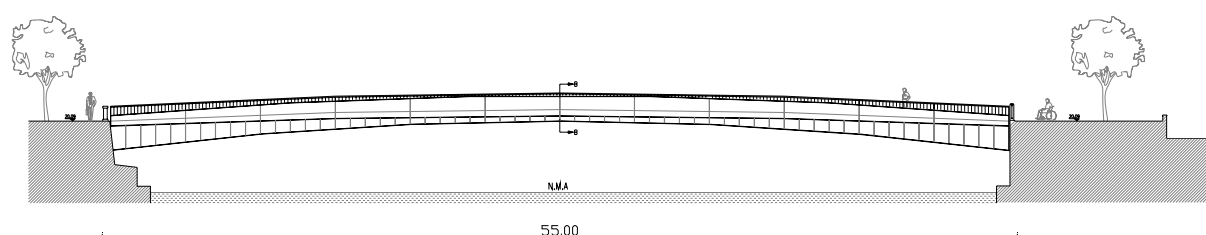
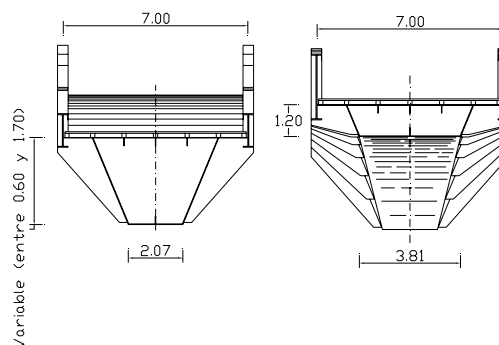
Location	Murau (Steiermark), Austria
Client	Stadtgemeinde Murau und Landesregierung Steiermark
Designer	Conzett, Bronzini, Gartmann AG (Engineers); Meili, Peter Architekten, Zürich
Cost	7'000'000 ATS

The bridge connects a variety of routes in an almost urban situation. On the south bank a narrow covered stairway rises from the main walkway to the railway station situated above it, while on the opposite bank a stair leads down to the promenade along the river. The structure is defined by two basic concepts: one structural in nature, the other spatial. *Structural* is the central girder (span 47 m), which provides the optimum protection against the elements for a covered bridge. *Spatial* is the decision to construct the bridge using walls and slabs so as to form a variety of spatial experiences characterised by the contrast between steep stair and level path (width 3.20 m), narrow passageway and open space, or directed and distant view. These apparently contradictory elements became so strongly intertwined in the course of the project's development that the bridge as built may be considered equally as a spatial sequence or as a technical construction with its own intrinsic value. Thus the spatial decision to displace the shear wall elements from the central longitudinal axis opened up the technical possibility of fixing these elements to the side of the chord, which in turn permitted a very simple connection with screw rods and ductile dowels. At the same time this created, almost by itself, a large bearing surface over the abutments, which helped stabilise the central framework against overturning. The dimensions of the chords were determined by the large torsional moments, but their size also created – by their flexural resistance – the long central “window” of the bridge.

References

- werk, bauen + wohnen, December 1995 (in German).
- Architektur Aktuell, December 1995 (in German).

Saint Feliu Footbridge, Spain (1996)



Location Girona, Spanien
Client Ministerio de Fomento (Ministry of Public Works)
Designer Pedelta (Juan A. Sobrino, M^a Dolores Gómez-Pulido, Engineers); Blázquez & Guanter (Architects)
Cost 325'000 USD

The objective of the new structure was to cross the Onyar River, with a footbridge connecting the oldest part of the city, in the area of the Sant Feliu Church, and the Devesa Park. The better balance of aesthetics, cost, constructability and serviceability behaviour (vibrations) was achieved selecting a two-hinged frame of one span of 58.4 m (L) where the two extreme concrete supports are below existing road level and keep occult.

The bridge deck is a weathering steel structure ($f_y=355$ MPa) accommodating a 3.5 m wide walkway. The typical cross-section is a unicellular box girder a top flange of 2.4 m wide with varying depth between 0.6 m (L/97) at centre span and 1.7 m (L/34) at supports. The top flange is a steel plate of 10 mm thickness, the bottom flange 12 mm and the webs varying between 10 to 12 mm. The box girder is stiffened by transverse diaphragms each 2.25 m. The railings are designed as structural elements. Each railing is a double T beam, the web is formed by a vertical steel plate stiffened each 1.6 m and the two flanges are made by longitudinal plates of 200 x 20 mm. The railing is connected to the box girder throughout the top cantilever flange and transverse ribs each 2.25 m.

Reference

– ‘Sant Feliu footbridge in Girona, Spain’. Proceedings of the footbridge – 2002 conference. Paris, 2002.

West India Quay Bridge, United Kingdom (1996)



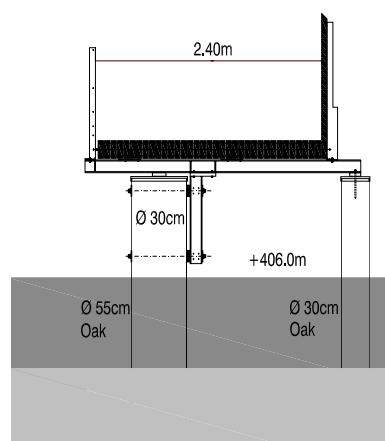
Location West India Quay, London Docklands, United Kingdom
Client London Docklands Development Corporation
Designer Anthony Hunt Associates (Engineers); Future Systems (Architects)
Cost 1'700'000 GBP

The bridge is a floating structure, in response to a design competition condition that no loads be imposed on the waterfront at the two ends. An aluminium deck supported on four sets of splayed tubular steel legs rest on largely-submerged pontoons secured by tension piles. The insect-like structure lies low on the water, avoiding visual competition with the buildings of Canary Wharf and West India Quay. The deck is divided into three, with a 6.6 m long central section that can be lifted hydraulically to allow boats to pass through. The opening action is by a simple cantilever counterbalance. The steel structure comprises a 750 x 300 U - shaped spine beam with tapered angles set diagonally to the spine beam to cross-brace it. The splayed legs are in 244.5 CHS and are bolted to the spine beam. They terminate in solid, cone-shaped connections welded to circular plates that are bolted to the pontoons. Only 500 mm of the 2800 mm diameter foam-filled pontoons are above the water surface. The use of pontoons allowed the bridge to be workshop fabricated and floated upstream into position. It was fabricated in two sections and transported by road to a dock side assembly yard and towed to the site where the pontoons were positioned and jacked down on to the piles.

References

- Detail - Zeitschrift für Architektur + Baudetail, December 1999, n. 8 v. 39 (in German).
- Watanabe, E. 'Floating Bridges: Past and Present' 'Structural Engineering International', May 2003, n. 2 v. 13.
- Pearce, Martin 'Bridge Builders', Wiley, 2002; pp. 22-25.

Footbridge Rapperswil - Hurden, Switzerland (2001)



Location Rapperswil, Switzerland
Client Municipalities of Rapperswil and Freienbach
Designer Huber & Partner AG; Walter Bieler AG
Cost 3'050'000 CHF

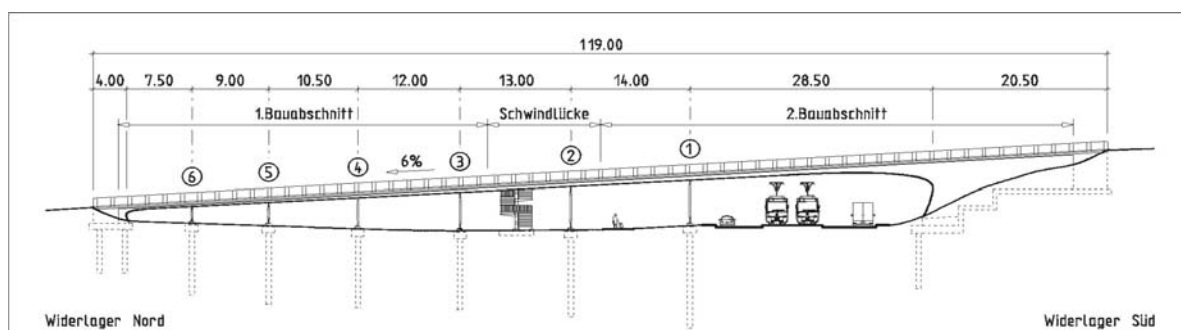
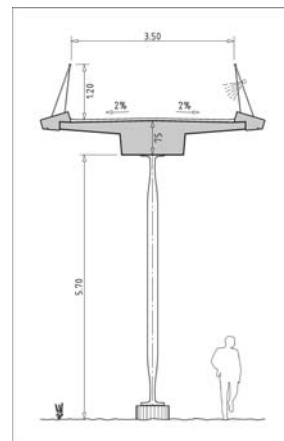
The history of this timber bridge between Hurden and Rapperswil reaches back to the early Bronze Age culture. With a length of 841 m it is today the longest wooden catwalk in Switzerland. The bridge itself is composed of three simple structural elements in principle: the foundation made of wooden piles – the crosspieces made of steel sections as supports – the bridge deck made of wooden planks reaching between the supports. Oak was chosen because of its well-known durability and resistance.

Each plank reaches from one pier to the next and has a length of 7.50 m, whereby the planks do not simply lie on the crosspieces, but are extended into each other in a refined way: with the deck made of oak, every second plank is offset by 1.25 m with respect to the next in order to achieve an interlocking/dovetail effect with the next plank. The coupling forces are taken with a horizontally prestressed steel rod.

The modernity of the bridge concept is to be found in its simple static idea in connection with its refined detail solutions.

The landscape and the rich world of flora and fauna as well as the cultural context of this historical pilgrimage route unite to a unique setting for this footbridge.

La-Ferté-Steg, Germany (2001)



Location Haldenrainstraße, Stuttgart-Zuffenhausen, Germany
Client Landeshauptstadt Stuttgart
Designer Peter und Lochner, Stuttgart (Engineers); Arat und Siegel, Stuttgart (Architects)
Cost 1'200'000 EUR

The design of the 'La-Ferté-Steg' takes into account the given location with a slope on one side and a wide, almost flat meadow on the other side, resulting in an asymmetric bridge. A special feature is the T-beam superstructure, being curved in plan view without any joints and bearings, supported by slender columns.

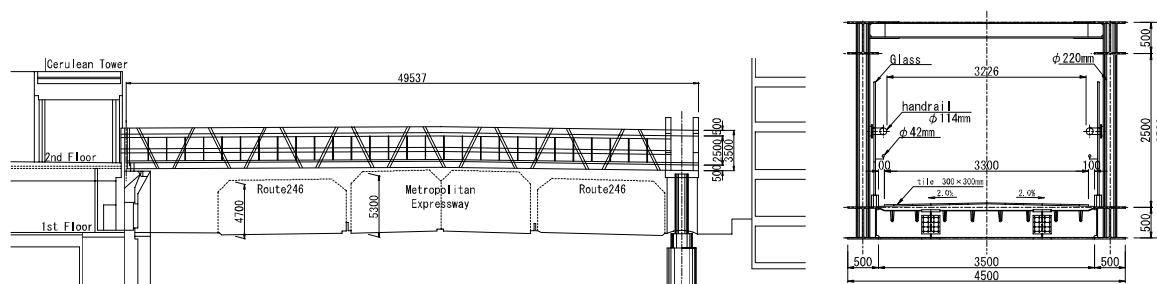
The lower T-beam contour follows a parabolique gradient towards the abutment and ties with a continuous transition into the terrain with a final arch in order to emphasize the outgrowing of the structure as well as the monolithical aspect. Due to their shape, the slender columns sustain primarily normal forces, even when the superstructure will expand or contract due to temperature variations. In this case, the curved integral superstructure moves outside or inside with the slender columns following the movement with neglectable resistance. This structural behaviour of the columns is achieved by the reduced cross section at the column ends made of high strength cast steel, which avoids moment interaction between beam and column.

The different structural elements demonstrate the contrast in material and function. The columns keep the superstructure in a 'suspended floating state', an appearance of lightness which is unusual for a concrete superstructure.

Reference

– Schüller, M., Peter, J.: 'Fuß- und Radwegbrücke über die Haldenrainstraße in Stuttgart – Entwurf und Konstruktion einer fugen- und lagerlosen Betonbrücke', Beton- und Stahlbetonbau 97 (2002), Heft 11, S. 609 (in German).

Shibuya 21 Bridge, Japan (2001)



Location Tokyo, Japan

Client Ministry of Land, Infrastructure and Transport

Designer R. Umezawa (Engineer), E. Horikoshi + Architect 5 (Architect)

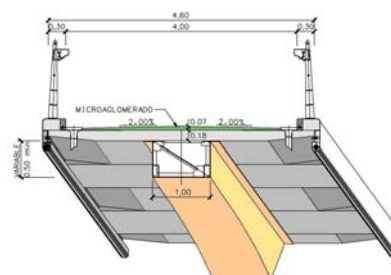
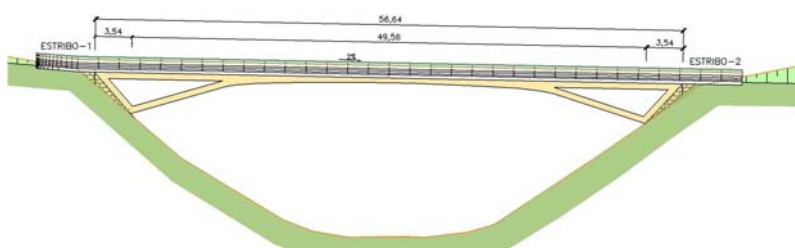
The Bridge Shibuya 21 was built as a part of redevelopment of a central town of Tokyo. The bridge features on innovative structure and a sophisticated design. The structural system of the bridge was aimed at an intermediate between Warren truss and Vierendeel girder (span 49.5 m). More or less intricate parts of nodes were simplified, and decrease the number of the diagonal members, thus, a transparent slender structure was achieved.

The design of the truss structure is based on setting the intersections of adjacent diagonal axes on the chord axis in order to prevent applying a bending moment to the members. However, a node would be too complicated and difficult to weld if the chord axis and the two diagonal axes were to intersect at one point. Therefore, gusset plates are generally used to solve this problem. Nonetheless, the gusset plates are inappropriate for the design and should be avoided if possible. For this reason, the intersections of the two diagonals with the chord seem to be separated to simplify the node and to avoid using gusset plates. Leaving spaces between nodes also simplifies erection on site.

Reference

– Takenouchi, K. and Ito, M. (2002). 'Function and Development of Pedestrian Bridges in Japan', Proceedings of the footbridge 2002, pp.55-56.

Pasarela Peatonal de Mondragón, Spain (2002)



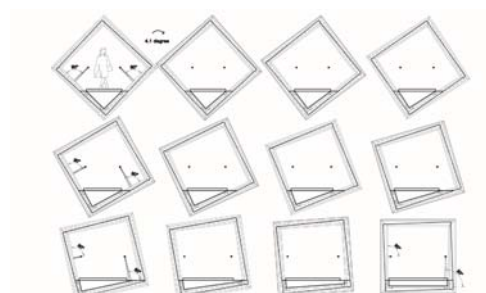
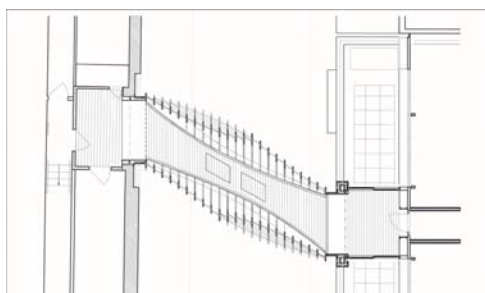
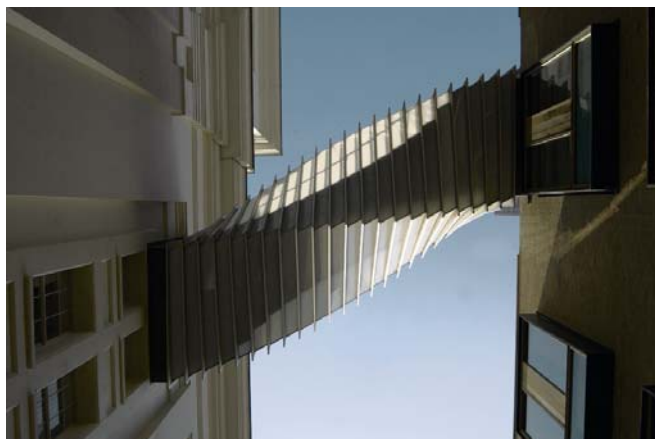
Location Mondragón, Gipuzkoa, Spain
Client Council of Mondragón
Designer IDEAM. S.A.
Cost 207'565 EUR

The footbridge is located over the Kanpazar hill, a selected environmental area in Euskadi, at the North of Spain. The main span of the bridge is 50 m between the bottom of the inclined piers. The structural form consists of a central deck elastically restrained at both sides by a triangular cell. This cell is constituted by the inclined piers, the upper lateral deck and a back strut which connects them and nearly follows the slope of the ground near the abutments. The footbridge is supported vertically only at the bottom of the main piers, without any vertical bearing of the deck at abutments. Lateral forces from wind are transferred to small lateral bearings at abutments, which also help for torsional response of the footbridge together with lateral restraints at the bottom of piers. The depth of the deck at the middle of the central span is 0.68 m. A box girder 1 meter wide and 0.50 m deep supports the upper slab 4.60 m wide and 0.18 m thick, to constitute a steel concrete composite solution for the deck. The slenderness of the deck at the middle of the central span results $L/73$. This high value for the slenderness is actually relevant for the aesthetical impact of the bridge, which seems to fly full of lightness between the adjacent hills. The action of the triangular cells is essential to allow this slenderness, as well as the double composite action designed for the hogging cross sections of the deck. Inclined piers and back struts of the lateral cells have also double composite action. The main frequency of flexural mode is 2.66 Hz, and the second mode 2.82 Hz.

Reference

– Miguel Ortega, Javier Pascual, Francisco Millanes; ‘Pasarela Peatonal de Mondragón en el Proyecto de Alameda Peatonal Monterrón – Ergüin’.

Bridge of Aspiration, United Kingdom (2003)



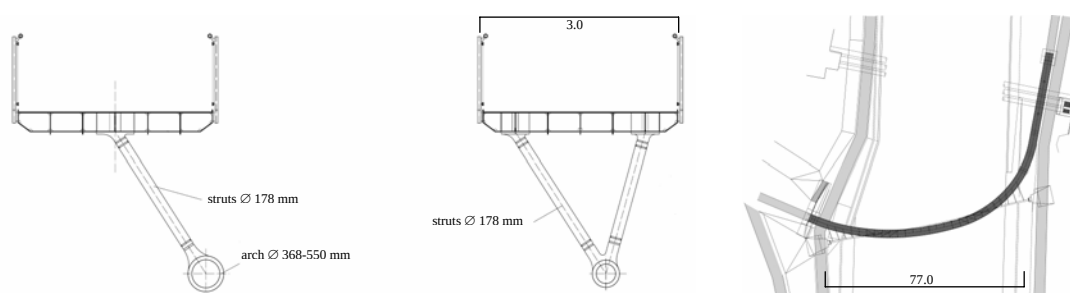
Location Royal Ballet School, Floral Street, Covent Garden, London W1, United Kingdom
Client The Royal Ballet School
Designer Flint & Neill Partnership (Engineers); Wilkinson Eyre Architects
Cost 450'000 GBP

The bridge spans 9.5 m between the existing Grade 1 listed Royal Opera House and the new Royal Ballet School providing an internal high level crossing. The openings to each building are not aligned in plan nor elevation, and to ameliorate the visual effect of the geometric skew the structure performs a 90 rotation across its length. A series of glazed portal frames rotate incrementally by about 4° each. The enclosure forms a light filled space that ‘pirouettes’ in reference to its users –the dancers of the RBS. There were strict time constraints surrounding the construction and installation of the bridge to avoid disruption to the Opera House and the street below. The whole bridge was prefabricated and lifted into position by mobile crane. There were also limits on the loads that could be imposed on the RBS structure. The spanning structure consists of an aluminium box beam of constantly varying cross-section, chosen to minimise maintenance and reduce weight. The requirements for the internal environment and the largely transparent façade led to integration of mechanical services equipment within the structural beam which acts as a plenum for the heating and ventilation system.

References

- Firth, I.P.T, ‘New Materials for Modern Footbridges’ – AFGC & OTUA Footbridge 2002, Paris, 20-22 November 2002.
- Firth, I.P.T, ‘Landmark Footbridges in the Urban Environment’ – IABSE Symposium 2004, Metropolitan Habitats and Infrastructure, Shanghai, China, September 22-24 2004.

Footbridge over Rhein Herne Canal, Germany (1997)



Location	Haus Ripshorst, Rhein-Herne-Kanal, Oberhausen, Germany
Client	Kommunalverband Ruhrgebiet KVR Essen
Designer	Schlaich Bergermann and Partner, Stuttgart; Ingenieurbüro Dr. Pelle, Dortmund (Engineers); Diekmann + Lohaus, Hannover (Architects)
Cost	1'600'000 EUR

The bridge smoothly connects with a generous curve two approaches which in plan are almost perpendicular to each other. An arch beneath was chosen to support the walkway slab, the challenge for the engineers: to combine the bent layout of the pathway with the bent curve of the arch when seen above the canal. The steel arch has a span of 77 m and individual struts positioned centrally under the walkway or V-supports every 3 m.

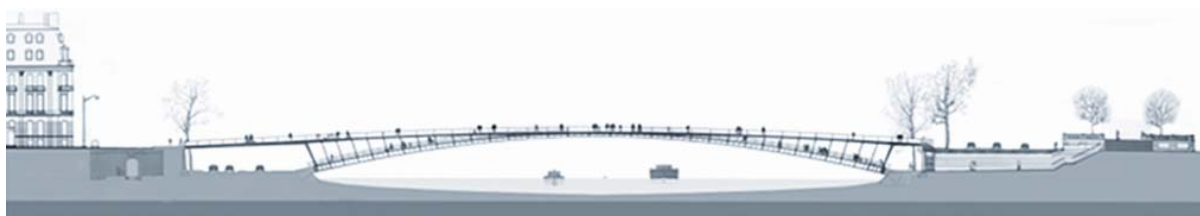
The arch runs in a spatial curve between the two opposite footprints such that it functions as a thrusting line for the walkway, the plan of which features an appreciable bend when compared to the slight curve in the elevation. The course of the arch can easily be seen as the inversion of a suspension bridge with a bearing cable and hangers along the centre of the walkway – and the geometry was computed precisely this way.

Cast-steel joints were thus the obvious choice for the arch. To ensure easy and swift assembly, the slab was made as a hollow steel section and the two sections on shore were erected first. Then using a crane the 60 m long middle section was lowered into place in-between and welded to them.

References

- Bögle, A.; Schmal, P.; Flagge, I. (Hrsg.), leicht weit – Light Structures, Jörg Schlaich, Rudolf Bergermann, Prestel Verlag, München 2003.
- Schlaich, J.; Moschner, T.: Die Ripshorster Brücke über den Rhein-Herne-Kanal, Bautechnik 76 Heft 6, Ernst und Sohn Verlag, 1999 (in German).

Passerelle Solférino, France (1999)



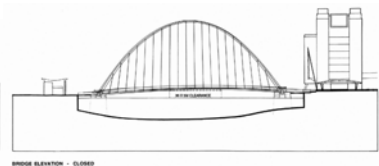
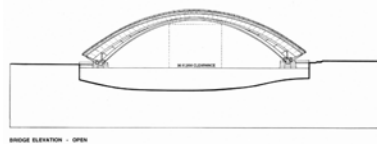
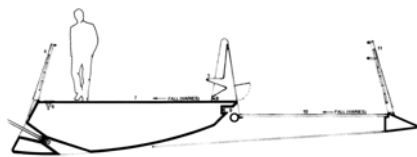
Location over the Seine, Paris, France
Client EPMOTC
Designer Marc Mimram
Cost 9'800'000 EUR

The Solférino Bridge is integrated into the built environments on each bank of the river. The design is based on the continuity of the pathways thus produced. The riverside walkways are extended smoothly onto the arch structure with a span of 102 m, at low level via the sunken passage on the Tuileries side and at the upper deck level onto the Quai Anatole France on the opposite bank: an asymmetrical route across a symmetrical structure. The bridge deck becomes a promenade in itself. In the unique arch structure, the deck divides into two. Gently curving, the two pathways are like twin balconies over the river, terraces looking out onto the Paris cityscape.

References

- Françoise Fromonot: Marc Mimram / Passerelle Solferino Paris / Solferino Bridge Paris, Birkhäuser, 2001.
- Bulletin 'Ouvrages Metalliques N°1', OTUA, 2001 (in French).

Gateshead Millennium Bridge, United Kingdom (2001)



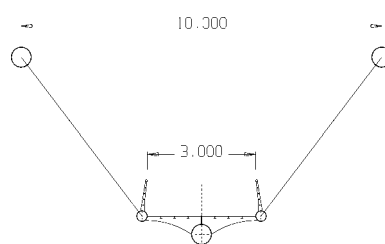
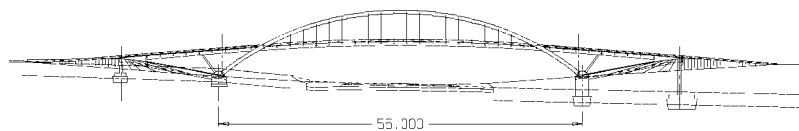
Location Gateshead, United Kingdom
Client Gateshead Metropolitan Borough Council
Designer Wilkinson Eyre Architects
Cost 17'700'000 GBP

This opening foot and cycle bridge across the River Tyne in Gateshead has a curved deck (total length 126 m, span 105 m) with grade separated pedestrian and cycle lanes. The pedestrian deck is a closed fabricated steel torsion box with cantilevered ribs, which supports an open cycle deck on the outside curve. The curved alignment was developed due to a lack of available land for approach ramps. A curved deck provided additional deck length at shallow gradients to clear the required navigation envelope in the bridge-closed position. The deck is stayed from an inclined 50 m high arch to the inside of the deck curve. The bridge opens with a 'tilting' motion, where the support arch and deck are rotated 40 degrees through the vertical axis by hydraulic ram sets at each end. The movement translates the deck into a high arch, spanning a 35 m wide x 25 m high navigation channel.

References

- Johnson, J & Curran, P, Gateshead Millennium Bridge – an eye-opener for engineering, Civil Engineering 156, February 2003, Paper 12885.
- Eyre, J & Clark, G, The Gateshead Millennium Bridge, RIBA, February 2001.

Footbridge across the ‘Ronda de la Hispanidad’, Spain (2002)



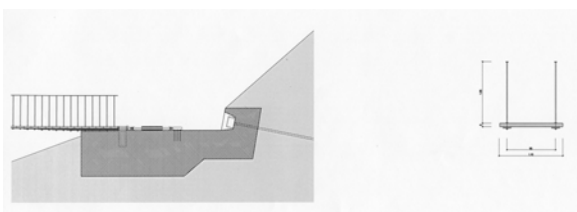
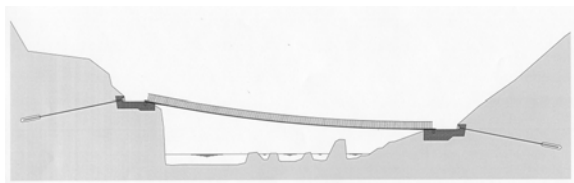
Location Ronda de la Hispanidad, Zaragoza, Spain
Client Ministry of Public Works
Designer Carlos Fernández Casado S.L. & SERS
Cost 260'000 EUR

This pedestrian bridge crosses across the “Ronda de la Hispanidad” (an urban peripheral motorway) and it connects two park areas. As the motorway level is lower than both parks, it is possible to design a promenade with an almost horizontal alignment. The arch with an intermediate deck fully matches the geometrical constraints and it has the advantage of reducing the apparent depth of the full structure making it visually more slender. The central part of the deck is supported by two inclined arches by means of hangers while each lateral span is supported by a single inclined strut. The arches are located in inclined planes leaving a space between both planes for the deck. The static configuration is not the most efficient but this is not important for such a small span (56 m) and the gain in terms of visual impact is worthwhile. Pedestrians using the footbridge may appreciate the openness of the promenade since both arches are well apart from their path. Tubular elements are used throughout the whole bridge. The arches are made out of circular hollow sections with 560 mm diameter. The deck has a central spine which is also a 560 mm diameter tube and two edge beams which are 300 mm diameter tubes. Finally the inclined struts in the lateral spans are 219 mm diameter CHS.

Reference

– Astiz M.A., Manterola J. & Gil M.A., ‘The Ronda de la Hispanidad pedestrian bridge in Zaragoza (Spain)’, Tubular Structures X, Jaurrieta, Alonso & Chica (Eds.), pp. 25-31, Swets & Zeitinger, 2003.

Pùnt da Suransuns, Switzerland (1999)



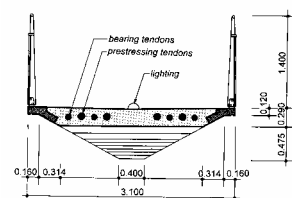
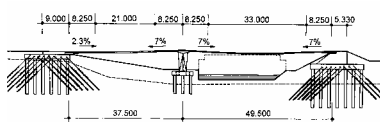
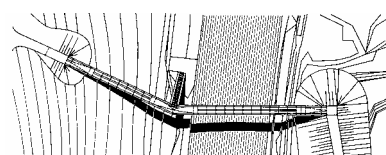
Location Viamala (Graubünden), Switzerland
Client Verein KulturRaum Viamala
Designer Conzett, Bronzini, Gartmann AG
Cost 265'000 CHF

This structure was derived from two ideas. A stress-ribbon system (span 40 m, width 0.85 m) was persuasive on both technical and aesthetic levels, as it accommodated the height differences between the river banks and allowed for flooding. In addition the bridge was to be made from stone (Andeer gneiss), as a material manifestation of the cultural divide between north (wood) and south (stone) along the Viamala trail. The weight of the stone slabs provides sufficient stiffness against vibrations, furthermore, the stone pieces are posttensioned by the underlying rectangular metal bars to act statically like one big single stone piece. This adds greatly to the bending and torsional stiffness. The posttensioning was effectuated by two temporary hydraulic jacks at the lower abutment, acting on hooks welded to the end blocks of the main steel bars. V4A stainless steel or duplex steel was chosen for all steel parts, as the bridge lies within the range of salt-spray mists from the national trunk road above it. The butt joints of the stone slabs were filled with aluminium strips as a substitute for mortar and as a levelling layer against brittle failure due to small-scale unevenness of the two stone surfaces touching each other directly. To reduce the fatigue-stresses in the steel bars near the abutments, additional metal bars are locally stapled like leaf-springs.

References

- SI+A Schweizer Ingenieur und Architekt, January 2000 (in German).
- Structural Engineering International, May 2000.

Kent Messenger Bridge, United Kingdom (2001)



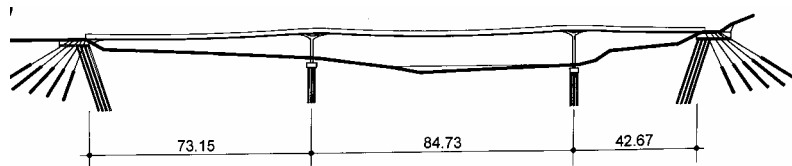
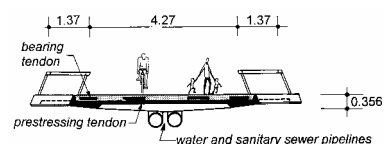
Location River Medway, Whatmans Field, Maidstone, Kent, United Kingdom
Client Maidstone Borough Council
Designer Flint & Neill Partnership, Strasky Husty & Ptrs, (Engineers); Cezary Bednarski (Architects)
Cost 1'760'000 GBP

The structure is the world's first 'cranked' stressed ribbon bridge. The deck comprises a set of post-tensioned pre-cast concrete planks, resting on two sets of bearing cables whose geometry is pre-set to achieve the desired profile after deck erection. The bridge is very slender and has no bearings or expansion joints but nonetheless creates substantial horizontal forces which need to be anchored at the abutments, and a high proportion of the cost is related to the foundation conditions. The two-span cranked arrangement adds complexity to the engineering; the inwards horizontal force at the crank is resisted by the concrete staircase acting as a strut, with a stainless steel tie to deal with out of balance forces under live load. The prop accommodates large horizontal, vertical and torsional loads arising from the prestress and the varying thermal and live load conditions. The design was strongly influenced by the landscape of the river which is lined with mature trees. The bridge has a restrained sculptural form with no overwhelming structural elements to interrupt views along the river. Both engineering and aesthetic beauty derive from the use of the suspended walkway itself as the structure, without masts or stays, but gaining stiffness and stability from its geometry.

Reference

– Bednarski, C. M., Kent Messenger Millennium Bridge, Maidstone, UK, presented at Footbridge 2002, Paris, 20-22.11.2002.

Rogue River Bridge, USA (2000)



Location Grants Pass, Oregon, USA

Client City of Grants Pass

Designer OBEC, Consulting Engineers, Eugene, Oregon, USA & Jiri Strasky, Consulting Engineer, Greenbrae, USA

Cost 1'320'000 USD

The Rogue River Bridge connects a major park on one side of the Rogue River and the County Fairgrounds on the other side of the river. The bridge is formed by a stressed ribbon of three spans (73.2 m + 84.7 m + 42.7 m) for an overall length of 200.6 m. The corresponding sags at mid spans are 1.10 m, 1.55 m and 0.31 m. The bridge of the width of 4.7 m provides a 4.3 m wide multi-use path. Observation areas located on widened deck segments at mid-span above the river and wetland offer the users a location to stop and enjoy the river.

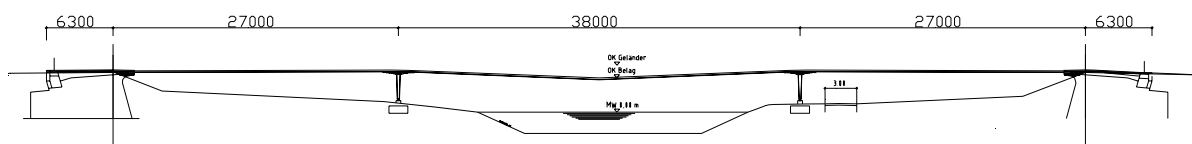
At the abutments and above the intermediate supports the stress ribbon is supported by concrete saddles. The bridge deck is formed from precast concrete segments with a coffered soffit that are composite with a cast-in-place deck slab. The segments are suspended on bearing tendons; the composite deck is post-tensioned by prestressing tendons. Both bearing and prestressing tendons are situated within the deck slab. Above the saddles the segments are 1.0 m long and have a solid cross section. Before the erection of bearing tendons these segments were post-tensioned by short internal tendons.

To reduce the dynamic response of the structure to seismic load, the transverse stiffness of piers was reduced as much as possible.

Reference

– Strasky J.: Stress Ribbon and Cable Supported Pedestrian Bridges, Thomas Telford Ltd, London, 2005.

Nord Bridge, Germany (2003)



Location Rostock, Germany
Client IGA GmbH, Rostock
Designer Schlaich Bergermann und Partner
Cost 700'000 EUR

The Nordbrücke is a multi-span stress-ribbon bridge. It is located on the grounds of the International Garden Exhibition, IGA 2003 where it crosses a flood plain a few meters above the plain surface. It has spans of 27 m, 38 m and 27 m. The high strength, grain refined steel ribbons (S690 QL1) are anchored in massive abutments that are founded on pressure-grouted piles. The intermediate bearings are made using a flat carriage spring system, which provides an elastic saddle for the stress ribbon. This spring system, which allows for optimising the saddle length, is in turn mounted on a rocking pier. Pre-fabricated concrete slabs (thickness 12 cm) are bolted on the steel ribbon to provide the deck surface. The resulting deck height is only 15 cm, thus creating an exceptionally transparent structure.

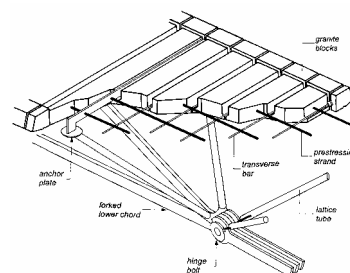
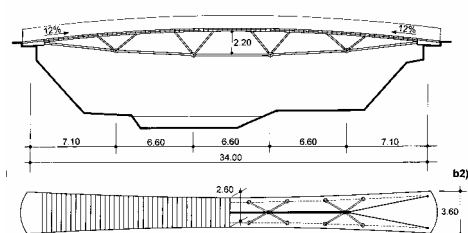
The bridge is in total 4 m wide, while the usable width is 3.70 m. The sags of the stress ribbons (0.56 m, 0.72 m, and 0.56 m) were chosen so that the horizontal components of the stress ribbon forces at the piers are equal under constant loading. The weight of the concrete slabs and the small sags provide sufficient stability for the structure.

The stainless steel tubes that form the handrails encase the lighting system of the bridge, consisting of LED strips that effectively emphasize the undulating form of the bridge at night.

References

- Schlaich, M.: „Die Fussgängerbrücken auf der internationalen Gartenschau IGA 2003 in Rostock“, Bauingenieur Nr. 10, Springer-Verlag, 2003 (in German).
- Russel, H.: Five modest bridges make economic sense for garden show, Bridge Design and Engineering No. 4, London, 2003.

Inachus Bridge, Japan (1994)



Location Oita, Japan
Client Beppu City
Designer Kawaguchi & Engineers

The bridge was designed to have a lenticular shape with an arched upper chord and a suspended lower chord, which is sometimes called a suspended arch (span 34.0 m). The deck is made from granite quarried from China.

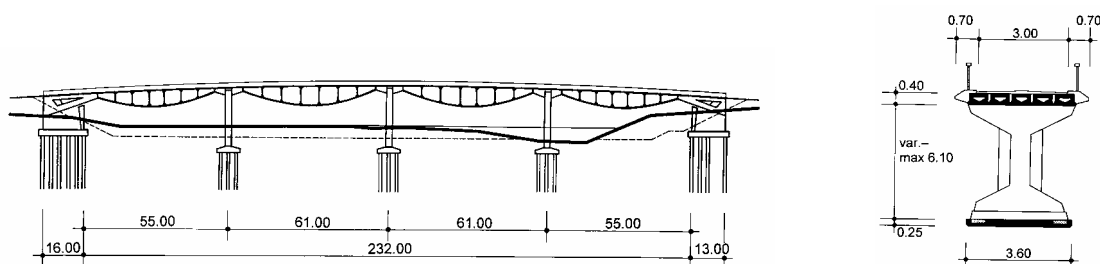
The longitudinal shape of the upper chord is a circular arc with the maximum slope of 12 % at the ends which is the possible steepest shape specified in pedestrian bridge code in Japan. For aesthetic reasons the upper chord of granite has such a shape in plan that it is narrower at the center and wider toward the ends.

It consists of 78 blocks of granite 40 cm wide and 25 cm deep with a varying length from 2.6 m to 3.6 m. Through the hole drilled in the center of the depth of the granite blocks, 5 prestressed cables are arranged in the longitudinal direction, parallel to each other. After the joints between the adjacent granite blocks are secured with filling mortar, the whole upper chord is prestressed to produce a literally monolithic structural member. The lower chord has the longitudinal shape of a funicular polygon, which is almost symmetrical with the upper chord. It consists of steel plates arranged into a chain. The upper and lower chord are connected to each other by means of web members consisting of steel tubes arranged to form inverted pyramids.

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- Kawaguchi, M.(2002). 'On how concrete spatial structures can be beautiful', Proceedings of the 1st fib Congress, Session 14, pp.1-12.
- Strasky J.: Stress Ribbon and Cable Supported Pedestrian Bridges, Thomas Telford Ltd, London, 2005.

Shiosai Bridge, Japan (1995)



Location Kikugawa River, Shizuoka Prefecture, Japan,
Client Shizuoka Prefectural Office
Designer Shizuoka Construction Technology Center

The Shiosai Bridge built in 1995 across the Kikugawa River in Shizuoka Prefecture, Japan, is formed by a prestressed concrete deck of four spans that is supported by a continuous suspended stress ribbon.

The stress-ribbon is assembled of precast segments of variable depth – from 0.25 m at mid-span to 0.48 at piers. To minimize the horizontal force in the stress-ribbon the sag to span ratio was set 1/10. For the same reason a lightweight concrete of specified concrete strength 40 MPa and density of 18.5 kN/m^3 was used.

The slab decks consisted of 0.40 m deep hollow prestressed concrete girders. These girders were rigidly connected to the tops of the piers and columns. Two columns situated close to the abutments are pin-connected both with the stress-ribbon and the decks. Between the deck and abutment horizontal neoprene bearings and prestressing tendons were installed.

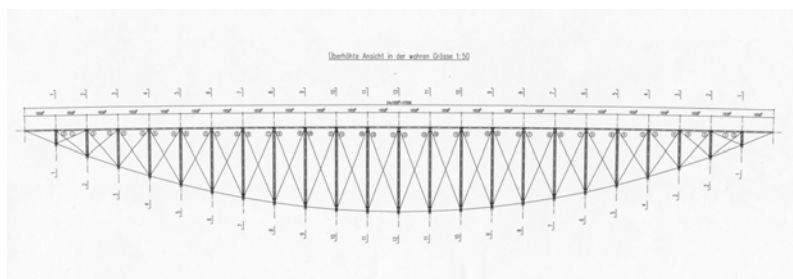
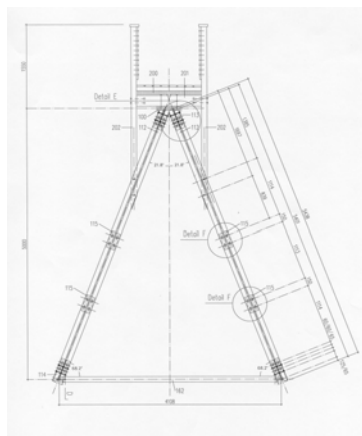
The construction of the bridge started by casting of the abutments and piers. Then the bearing tendons were installed and precast segments of the stress ribbon were erected. After casting the joints between the segments the stress ribbon was post-tensioned. Then the columns and precast girders were erected.

The logarithmic decrement of damping varied from 0.04 to 0.06 for the longitudinal bending modes.

Reference

– Strasky J.: Stress Ribbon and Cable Supported Pedestrian Bridges, Thomas Telford Ltd, London, 2005.

Traversinasteg I, Switzerland (1997)



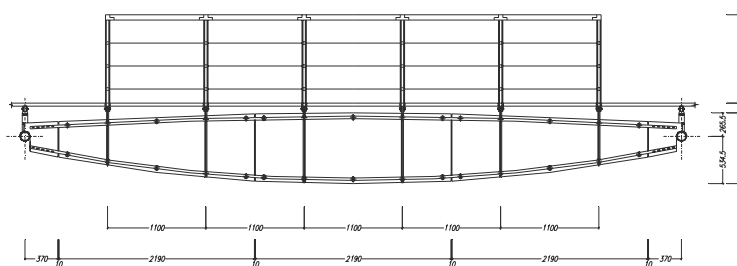
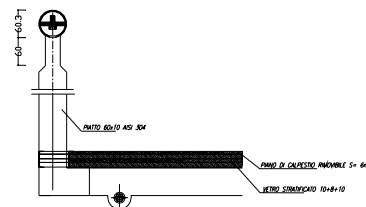
Location Traversiner Tobel, Viamala (Graubünden), Switzerland
Client Verein KulturRaum Viamala
Designer Konzett, Bronzini, Gartmann AG
Cost 291'000 CHF

The bridge connects two fragments of an old Roman trail at a remote place. All parts had to be transported to the building-site by helicopter and thus the bridge is divided in a light substructure (spanning 47 m) and a heavier stiffening and protecting superstructure. The balustrades act as plate girders and stabilize the two point-supported substructure against torsional motion by a pair of vertical forces. The weight of the three-chord truss for the substructure is 4.3 t, equal to the load-carrying capacity of the most powerful helicopter available. The superstructure could be assembled in several pieces by a lighter helicopter. Wind resistance determines the geometry of the cross section, as there is the risk of the lower chord cable on the upwind section losing its tension on a strong gale. Safety therefore depends mainly on the ratio of wind load to permanent load. The main structural elements (the three chords of the substructure) are designed for a long lifetime, whereas the weather-exposed timber elements allow for an eventual replacement. Thus each of the struts is made of four individually replaceable larch boards. The upper chord of larch glulam timber is well protected from rain and snow by the laterally protruding watertight walkway and the parabolic main cables are made of V4A stainless steel. The Traversina footbridge was destroyed by a rockfall in march 1999.

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- Structural Engineering International, May 1997.

Glass Bridge in the Basilica of Aquileia, Italy, 1998



Location Aquileia, Italy

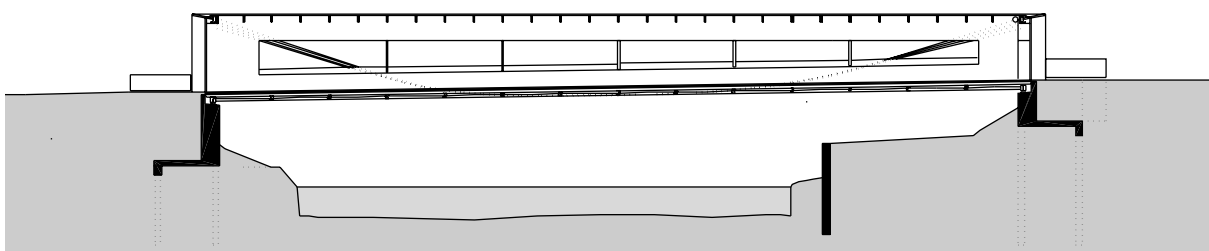
Designer Facero & Milan ingeneria, Meran (Engineers); Ottavio Di Blaso Associati, Milano (Architects)

The important mosaics in the Basilica of Aquileia attract 300'000 visitors a year and therefore required some form of protection. A steel construction was designed that allowed a system of transparent walkways to be suspended from the new roof structure. These permit an unimpeded view of the mosaics, while at the same time protecting them against wear and tear. The walking surface consists of sheets of laminated glass (three 12 mm layers, with a 6 mm top layer that can be replaced at regular intervals). The dead and live loading is borne by a slender stainless-steel structure. Vertical sheets of glass at all corners of the walkways solve the problem of horizontal bracing.

Reference

– Detail. Zeitschrift für Architektur + Baudetail, „Bauen mit Glass“ 3/2000, Page 364-366 (in German).

Footbridge Gaißau, Austria (1999)



Location Gaißau, Vorarlberg, Austria
Client Land Vorarlberg
Designer Frank Dickbauer (Engineer); Hermann Kaufmann (Architect)
Cost 6'000'000 ATS

This “timber” bridge connects the towns of between Gaißau (Austria) and Reineck (Switzerland). The erection of a covered timber bridge at this place, assigned symbolically the entry in the wooded scenery of Vorarlberg.

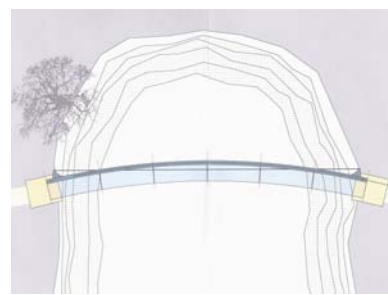
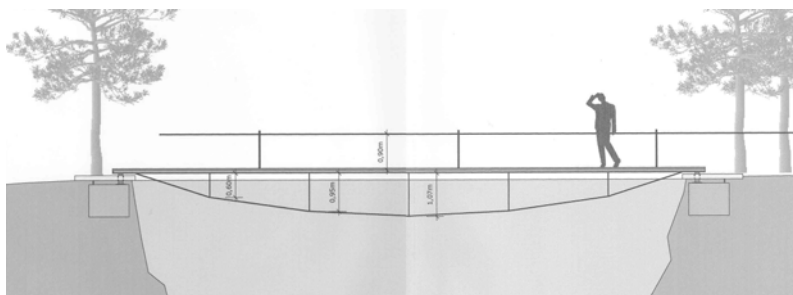
Timber bridges are often the cheapest construction for client. The load carrying system consists of two laminated beams (span 44 m, width 4.5 m) with intermediate stressed ribbons (flat steel) at each side in the side wall. This design enables a very open bridge and a free view.

When crossing the river and looking through the wide openings in the side wall, the bridge offers space and a feeling which is known from crossing old covered timber bridges. To protect the bridge construction, it is fully covered.

References

- Pro Holz Zuschnitt 2/2001, S 16-17 (in German).
- archi, 5/2004, S. 30-33 (in German).

Glass Bridge Haute-Provence, France (2003)



Location	Haute-Provence, France
Client	Private
Designer	Johannes Liess (Engineer); Renate Fehling (Architect)
Cost	80'000 EUR

The dream of the client was to have only a single line in the landscape instead of a massive construction. This idea was leading us quickly to a light bridge made out of glass. The bridge is crossing a deep valley, connecting two parts of a private park. The walkway is made out of 6 plane glass panels. Each panel is made out of 3 layers of glass, two 10 mm partially prestressed glass and one 20 mm fully prestressed glass.

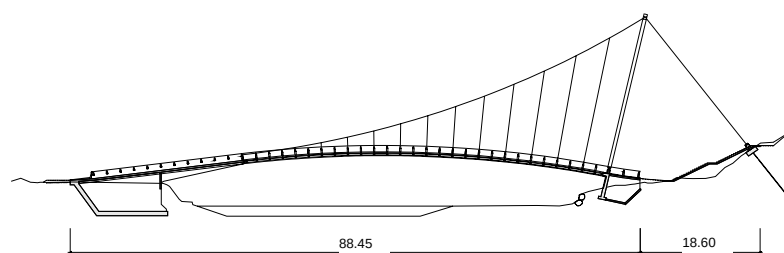
The panels are supported on one side in a curved beam. The curve is following the direction of the path. A prestressed cable supports the beam. On both sides the cable is eccentrically connected with the beam, so that the normal loads not influence the curve of the beam. There is only one handheld on the lower side of the valley.

Even if normally not many people are crossing the bridge it is calculated for normal live load. So the number of people passing the bridge at the same time did not have to be limited. Of course the construction is not fulfilling all safety requirements or national building codes. It is a piece of art showing how beautiful a construction can be.

References

- Fehling, Liess: in Glas, Architektur und Technik, Page 5-8, 3/2000.
- Fehling, Liess: in Glasforum, Page 1-4, 5/2004.

Footbridge across the Rhone, Switzerland (1998)



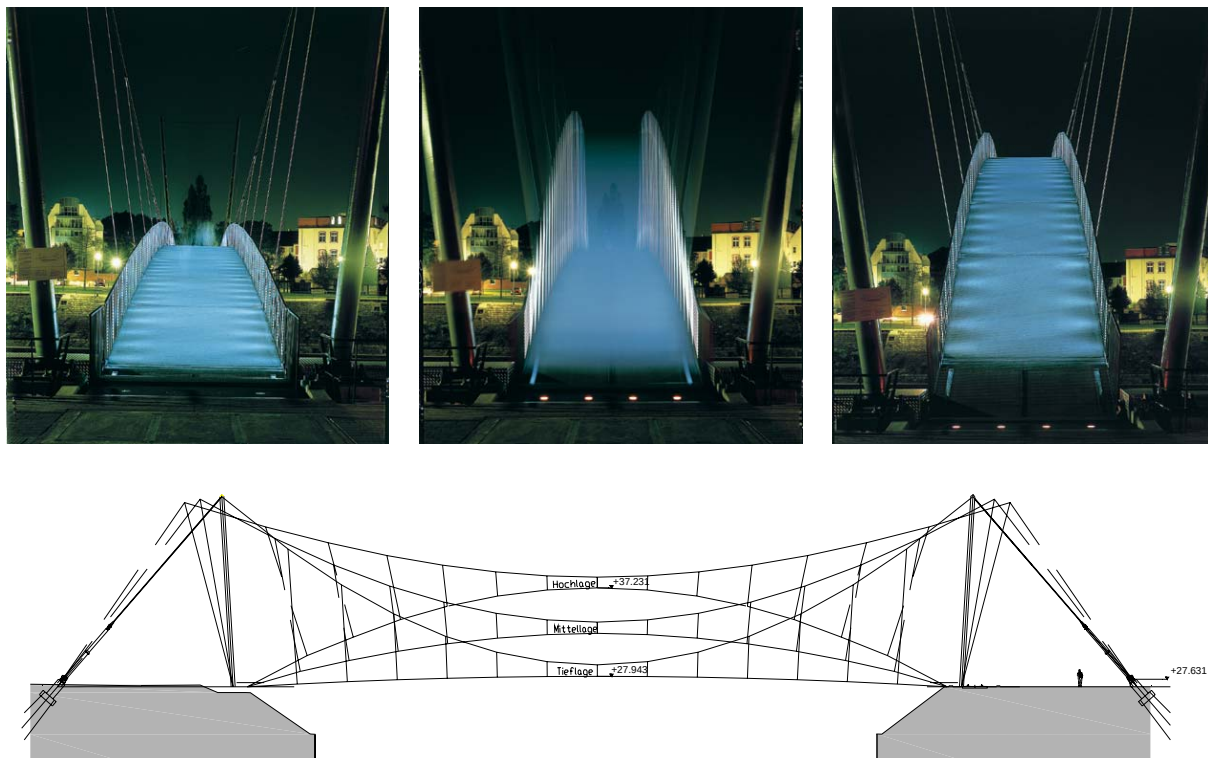
Location Sierre, VS, Switzerland
Client Routes nationales du Valais
Designer DIC SA
Cost 1'200'000 CHF

In order to restore the pedestrian access between Sierre and the valley of Anniviers a new footbridge was required. At the site, the Rhone curves several times between the plain and the steep mountain slopes. An asymmetrical suspension bridge with the pylon on the mountain side was chosen. The abutment on the opposite side curves gracefully from the plain like a water jet, and the movement is taken over by the suspension cables and anchored in the mountain.

To accentuate the crossing movement, the bridge is vertically curved. The span of the bridge is 68 m, the width of the deck is 3.0 m. The towers for the suspension cables rise 26 m. Every 4.0 m a vertical hanger supports the walkway, which consists in steel girders as supporting members and wooden deck planks.

The concrete abutment was cast in situ and later prestressed to resist to the horizontal tension of the suspension cables. The steel girders and steel towers were transported in several pieces and welded together on site. With the help of a mobile crane the tower and the girders were erected on temporary supports. Then the main suspension cables were installed. After the hangers were fixed, the main suspension cables were stressed to their final positions. The combination of a suspension bridge with an arch bridge reduces considerably the vulnerability of the bridge due to vibrations introduced by pedestrians.

Katzbuckel Bridge, Germany (1999)



Location Inner Harbour, Duisburg, Germany
Client Innenhafen Duisburg Entwicklungsgesellschaft mbG
Designer Schlaich Bergemann und Partner
Cost 1'800'000 EUR

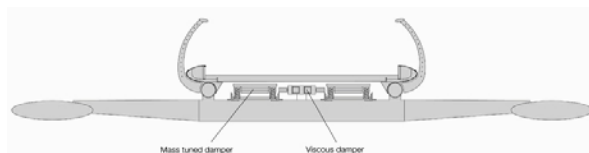
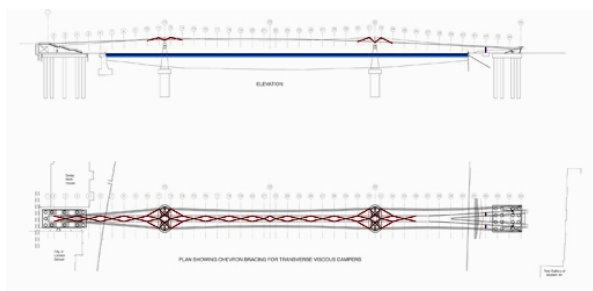
As graceful as a cat arching its back, the bridge curves up over the central basin of the Inner Harbour in Duisburg to allow the occasional larger ship to pass under it. It can be raised to any height up to its maximum arch height of 9.2 m – and need only five minutes to do so. Three positions are pre-set and in 90 % of all cases the middle position allows ships a clear passage. The four slender masts, each 20 m high and 42 cm in diameter, as well as the thin walkway slabs are hardly visible when the bridge is in its normal state. With its 73-meter span and pleasant sweep it links the old town park and newly planned greenbelt areas.

The elegant movement is achieved with minimal input: each of the four stay cables has a hydraulic cylinder that can be extended or retracted a maximum 3 meters. When retracted, the stays shorten and the masts tilt outwards. This circular movement of the mast heads causes the position of the main cables and hangers to change, lifting the bridge deck in such a way that it assumes an affine shape, something clearly noticeable even with a minor alteration of the masthead positions. The change in form caused by lifting leads to a strong arching of the bridge deck, which is designed as an articulated chain to enable this. The total of 14 prefabricated concrete slabs, each 3.5 m wide, are placed in steel frames and joined at the corners via eye bars and hinged bolts to neighbouring elements.

References

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- Architectural Record: Duisburg Pedestrian Bridge, Architectural Record 3/2000, New York.
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Millennium Bridge London, United Kingdom (2000)



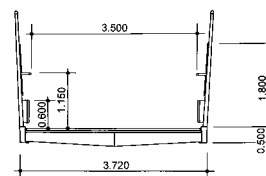
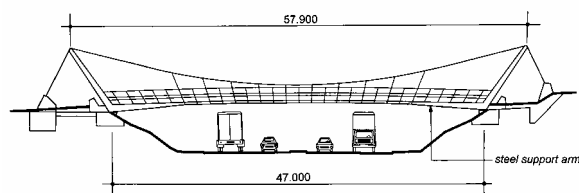
Location	London, United Kingdom
Client	London Borough of Southwark
Designer	Arup (Engineers); Foster & Partners (Architects)
Cost	18'200'000 GBP (including 5'000'000GBP retro fit of dampers)

The 325 m long bridge spans the River Thames, London, UK connecting St. Pauls Cathedral to the Tate Modern art gallery. The structure is a suspension bridge with a span to tip ratio of 63:1 (main span 144 m, sag 2.3 m), which is around 6 times shallower than a conventional suspension structure. Stressed ribbon bridges are limited in their ability to achieve useable pedestrian gradients whilst maintaining structural efficiency. The Millennium Bridge achieves some of the aesthetic minimalism of a ribbon structure but disconnects the deck from the catenary cables to provide shallower gradients. The cables rise above the deck at supports and dip below at centre span to provide clear views down river from the bridge. The suspension structure is gravity anchored with cables terminated above ground at each end. On the south side the deck bifurcates and the approach span returns between the two halves. The bridge experienced deflection problems on opening subsequently diagnosed as synchronous lateral excitation ie. excessive dynamic horizontal movement of the structure caused by crowd footfall inducing, and then 'locking in' to the swaying motion of the deck. The bridge was closed temporarily pending a retro-fit programme of dampers.

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- Detail - Zeitschrift für Architektur + Baudetail, Dezember 1999, n. 8 v. 39 (in German).
- Wells, Matthew, 20 Bridges, Laurence King, 2002.
- Millennium Bridge, London: problems and solutions, in The Structural Engineer, 17 April 2001, n. 8 v. 79.

Halgavor Bridge, United Kingdom (2001)



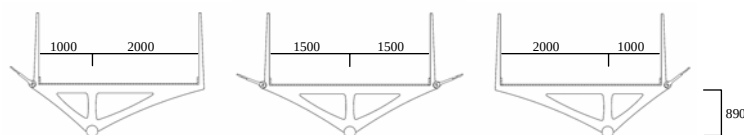
Location A30, Bodmin, Cornwall, United Kingdom
Client The Highways Agency
Designer Flint and Neill Partnership (Engineers); Wilkinson Eyre Architects
Cost 600'000 GBP

The Halgavor Bridge is the first publicly funded GRP bridge in the UK and carries pedestrians, cyclists and horses across a 48 m suspended glass fibre reinforced plastic deck. The need for easy maintenance and rapid construction to minimise disruption to road traffic led directly to the selection of lightweight self-finished GRP materials which are durable and require little or no maintenance. The deck is hung from conventional steel suspension cables, with a radial pattern of stainless steel hangers and parapet posts. The inclined steel masts are anchored to the ground by pairs of steel back-stays. The bridge deck is 3.5 m wide and 350 mm deep, but as a bridleway required 1.8 m high parapets. The design adopts a transparent mesh system in the parapet to avoid giving the bridge a heavy or bulky appearance. The finished deck surface is made from re-cycled rubber car tyres, suitable for horses, pedestrians and cyclists, and also requires little or no maintenance. From a distance, the bridge appears to emerge abruptly from the wooded embankments on either side, with the delicate suspended deck seeming to float between the branches. From closer up the inclined steel masts are seen through the trees, which together with the back stays form a “gateway” to the bridge at each end.

References

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- Firth, I.P.T, ‘A Tale of Two Bridges – A case study on the Lockmeadow and Halgavor bridges’ – Institution of Structural Engineers, 21st March 2002.

Footbridge over the Gahlensche Straße, Germany (2003)



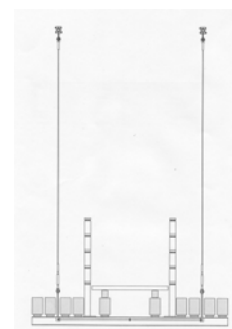
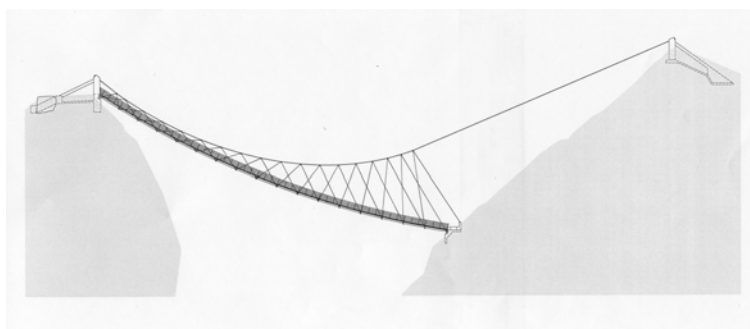
Location	Westpark, Bochum, Germany
Client	Kommunalverband Ruhrgebiet
Designer	Schlaich Bergemann und Partner
Cost	2'700'000 EUR

Pedestrians and cyclists can cross the road and rail track in Bochum with some verve by way of an S-bend in order to get to the re-naturalized West Park. The opportunity to make the path over the bridge a real experience was seized here, and the fact that a bridge that is circular in the plan needs bearings or suspension on one side only was played up to great effect. The curved path seemed the obvious choice given the natural and existing routes and consists of two segments of a circle that merge, each some 66 m long and with a radius of 46 m. Two inclined masts inside the circle segments unilaterally bear the bridge only on the inner side. In line with the switch-over of the cable suspension, the cross-section of the circular ring girder changes along the length of the bridge.

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Traversinasteg II, Switzerland (2005)



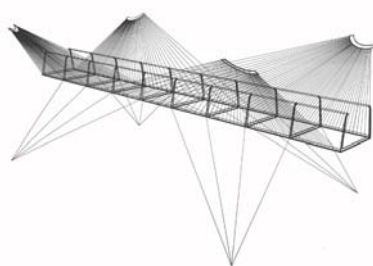
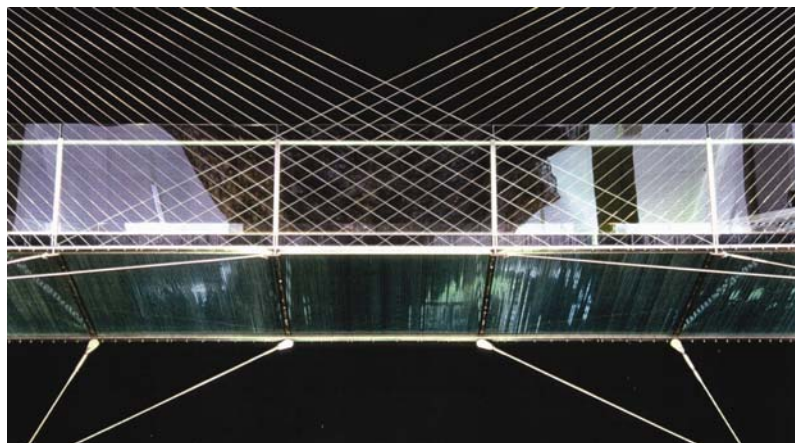
Location Traversiner Tobel, Viamala (Graubünden), Switzerland
Client Verein KulturRaum ViaMala
Designer Conzett, Bronzini, Gartmann AG
Cost 490'000 CHF

This bridge replaces the destroyed first Traversina footbridge. Its location was found after a long investigation about safety and feasibility of a new bridge. At the finally chosen position the span of the walkway could be minimized by building a suspended stair following the shortest possible line between the two flanks of the valley. The structural system is a suspension bridge (main span 95 m, span of pathway 56 m) with hangers arranged in a diamond truss form. The main cables are fixed to concrete anchorages with small pylons, using the advantages of the terrain profile. Posttensioning of the main ropes creates a compression force in the curved beams along the walkway which improves its stiffness against vibrations. Structural elements consist of larch glulam beams, treads and handrails are made of fir. Timber elements are weather-exposed and exchangeable. All timber was sawn from locally grown trees. Psychologically, the dip of the walkway makes the stair seem less steep than it actually is, when seen from above. The structure of the walkway (width 2.60 m) is much wider than the treads of the stair (width 1.0 m) itself, providing lateral stiffness and improving the comfort of the user (as a pair of 'blinkers' they prevent a direct view down into the gorge). The form of the main cable and the position of the nodes for the diamond truss were determined according to Culmann's Graphische Statik to obtain a constant tension force in the main cable under maximum snow load.

Reference

– Tec21, Zürich, September 9, 2005 (in German).

Challenge of Materials Bridge, United Kingdom (1997)



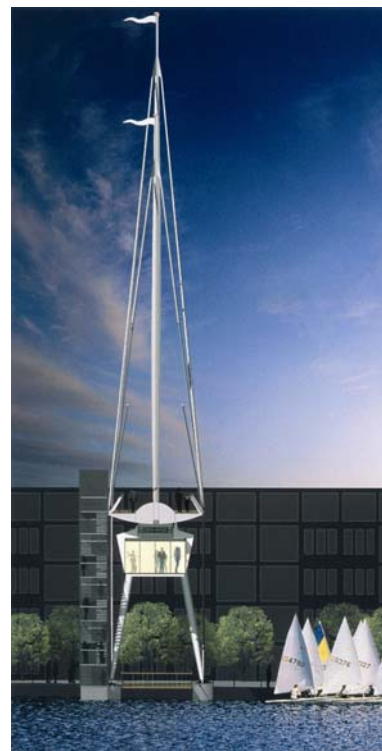
Location The Science Museum, London, United Kingdom
Client The Science Museum
Designer Wilkinson Eyre Architects
Cost 200'000 GBP

This internal footbridge spanning an atrium of 16 m of the Science Museum in London is the centrepiece of the Challenge of Materials Gallery, showcasing material diversity, applications and capabilities. The structure accordingly uses materials to legibly demonstrate their properties and potential, acting as both a link bridge and an exhibit in its own right. The deck consists of 828 glass planks, laminated in groups of five with the laminated edge forming the traffic surface of the deck. This arrangement partially obscures transparency through the structure. Clear plate glass parapets on each side complete a visually minimal composition. The deck is supported by an array of 186 ultra-fine gauge stainless steel wires in an overlapping semi-fan arrangement. The cables are stressed at a stainless steel 'sickle' fixed back to the building structure via a stress gauge in each location. The bridge responds to applied live load through computer generated light and sound effects of variable intensity, informed by the stress gauges.

Reference

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Royal Victoria Dock Footbridge, United Kingdom (1998)



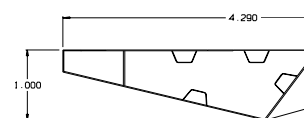
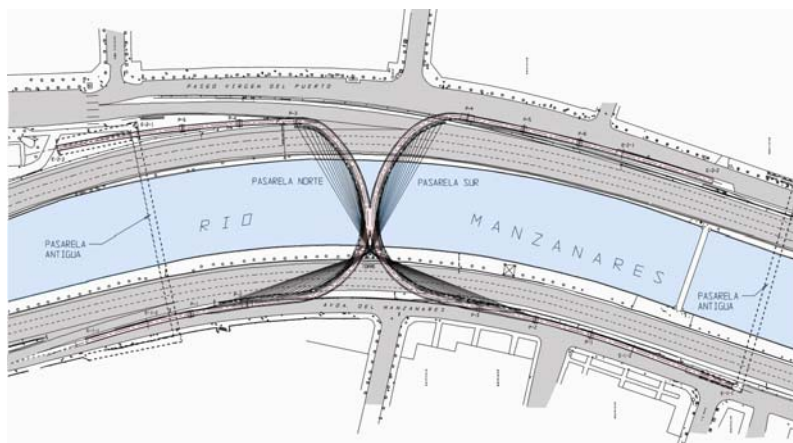
Location	Royal Victoria Dock, Silvertown Way, London, United Kingdom
Client	London Docklands Development Corporation
Designer	Techniker Ltd (Engineers); Lifschutz Davidson Architects
Cost	5'000'000 GBP

The Royal Victoria Dock Bridge is a 150 m long transporter bridge spanning one of the largest docks in London. The dock is a national sailing centre and so the bridge deck is raised 15m above water level and has an efficient aerodynamic profile to reduce wind turbulence and allow tall boats to pass underneath. Lifts and stairs at either end give access to the upper deck and the structure allows for the future installation of a forty person gondola travelling between dock-level stations in a parabolic arc reaching 11 m above the water. The structure is an inverted Fink truss spanning 130 m onto a pair of trestles. Six tapering conical masts are cable-linked to tie down points and at each end a further cable carries tension forces to ground via a distinctive 'bowsprit'. Materials, including hardwood decking and perforated stainless steel cladding, were chosen for low maintenance and endurance combined with richness and quality. The bridge is illuminated to complement the structural form with mast top projectors highlighting pylons and cables and concealed strip lighting accentuating the deck ribs. The visual appearance of the bridge reflects its context and responds to a history of high masted, cabled stayed structures on the ships and dock cranes which once lined the quayside.

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Manzanares River Bridge, Spain (2003)



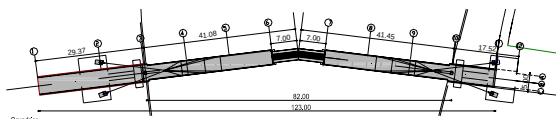
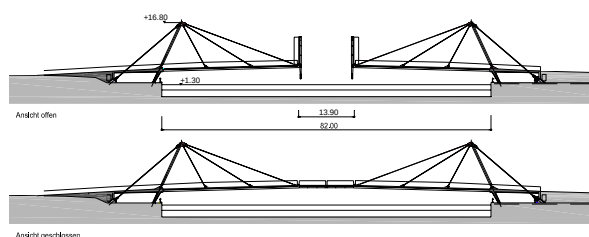
Location Manzanares River, Madrid, Spain
Client City of Madrid
Designer Carlos Fernández Casado S.L.
Cost 3'970'000 EUR

This double U-shaped pedestrian bridge connects both banks of the Manzanares River across the river and the M-30 urban motorway. Consequently, the horizontal and vertical clearances are great (about 85 and 6 m respectively) and the landing space is very much reduced by the presence of two streets running parallel to the river. The location of the tower is eccentric since it has to be located in one of the riverbanks and this fact strongly conditions the static scheme of the bridge. Horizontal cable force components create important bending effects on the deck, which is stiffened by a horizontal plate extending beyond the closed box combined with a 508 mm diameter tube. These force components are also compensated by important horizontal reactions at the abutments. The tower, 42 m high, has a circular cross section with a variable diameter ranging from 1.5 m at the base to 0.3 m at the top. The stays are locked-coil cables with diameters ranging from 20 to 40 mm; they come prefabricated with their exact length and they are anchored at both ends by means of eye bars connections. Because of important modifications in the motorway, the footbridge will be moved to another location in the near future.

Reference

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Nesse Bridge, Germany (2005)



Location Leer, Germany
Client Bauamt, Stadt Leer
Designer Schlaich Bergemann und Partner
Cost 1'100'000 EUR

This cable-stayed bridge of 82 m main span crosses the former trade harbour of Leer without piers in the water in order not to obstruct the frequent rowing regattas at this location. The central part of the deck of 12 m length is movable.

In plan the two bridge parts follow the directions of the roads leading to it which results in a “kinked” plan layout with a change in direction at the movable central part of the span. This not only allows for interesting views when crossing the bridge. It also fits the concept of the cable-stayed bridge which works as two fully loadable cantilevers when the bridge is open and which converts to a continuous girder of additional transverse stiffness when the bridge is closed.

From the centre of the bridge, where the movable steel-only deck is 4 m wide, deck width increases to 5 m along the fixed part of the main span (composite section with a 20 cm concrete slab and total height of 60 cm) and to even 6 m at the massive concrete abutments, that support the inclined mast.

Inclining the masts towards the water minimises their height and moves the mast heads away from adjacent buildings. Usually, single masts lead to transversally inclined cables which reduce headroom for the user. Here, spreader beams are introduced which by putting the stays straight eliminate this clearance problem and create an interesting space above the deck.

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Codes and Standards

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- [3] Associação Brasileira de Normas Técnicas (ABNT): NBR 7188, Carga Móvel em Ponte Rodoviária e Passarela de Pedestres, São Paulo, 1982.
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