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Bridges and ecological structures

edited by

Krzysztof Śledziewski

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Preface

One of the most important ecological consequences of the development of road infrastructure is its strong impact on the natural environment. In the case of bridges and other civil engineering structures, in particular, it is necessary to reconcile the technical requirements of the structure with the needs of nature conservation at every stage of planning, design, construction and maintenance. Taking into account examples of recent projects (e.g. the Rospuda, discussed in Section 5.3), this is not a simple task.

This monograph presents important current issues related to modern bridge construction – considered mainly in technical and environmental terms. The publication presents a multi-threaded and multi-disciplinary approach of the authors of its chapters.

The intention of the authors is to create a useful and valuable tool for all actors and institutions involved in the preparation of design and environmental documentations or issuing administrative decisions necessary to execute road projects. The publication also aims to fill in the gap in the knowledge about the impact of widely defined transportation infrastructure on the natural environment. The authors present proposals for optimal solutions on the basis of their experience and long-term research as well as the literature.

The creators of the monograph hope that the publication will be a useful source of information in the processes of planning and design civil engineering structures and contribute to a constructive dialogue between engineers, environmental protection services and public administration bodies, thus resulting in the implementation of the solutions that are the best for the environment.

The team of authors, consisting mainly of engineers and scientists of the Department of Roads and Bridges of the Lublin University of Technology would like to thank Mrs Grażyna Łagoda, professor of the Warsaw University of Technology, who reviewed the outline of the work and provided insightful and valuable comments that helped improve the quality of the book.

Krzysztof Śledziewski and the team of authors

Lublin, 2017.

Chapter 1.

Structural forms of bridges and ecological objects

Maciej Kowal, Krzysztof Śledziewski

1.1. Bridges classifications

Bridges are currently one of the main elements of transport infrastructure and can perform a variety of functions, be located over various obstacles (valleys, watercourses or transport routes), have various structures and be made of many types of materials.

The main purpose of bridge classification is to organise their characteristic features. This organisation is necessary because of the existing variety of bridges. The knowledge of the features characterising particular solutions facilitates all kinds of analyses carried out in various stages of design and construction of bridges. It also facilitates the selection of the appropriate bridge option, taking into consideration the conditions (technologies) related to its construction and operation.

The classification of bridges is a kind of overview of known and used solutions. Therefore, it may be a starting point in the search for modern bridge structures.

One of the most important classification criteria is the criterion of the function the facility is to perform. Based on their functions, bridges can be grouped into:

- road bridges,
- railway bridges,
- tram bridges,
- footbridges,
- aqueducts (channel bridges),
- mixed traffic bridges: roads and railways, roads and trams, etc.

This criterion directly affects the shape of the cross section and the road surface as well as impacts the operational load diagrams and values. The way of shaping the cross section also depends on the requirements for the bandwidth, comfort and safety of traffic. All these factors impact the adopted design solutions, such as the number of the main girders, the width of the structure, etc.

Another very important classification criterion is the selection of the material for the basic elements of the structure. The type of material used has a large influence not only on the possible design solutions but also on the operating conditions and thus the maintenance of the structure. The division of bridges according to the span construction material is as follows:

- wooden,
- stone and brick,

- concrete,
- steel,
- combined (steel and concrete).

The material selection criterion is related to the function that the structure is designed to perform. The selection of the material to be used for the construction of the bridge depends primarily on technical and economic reasons which result from e.g. the span length, the reduction of the maximum design height or the conditions related to the construction of the bridge.

Therefore, concrete bridges are heavy structures whose dead load constitutes a large share of its overall load. This results in a number of limitations in the assumption of the span length. For small and medium span lengths, a relatively large dead weight is compensated generally by a simpler solution, lower dynamic excitability and more durable pavement in road bridges [18]. In contrast, steel bridges are generally lighter, which allows for the construction of spans with a longer span length with the same load performance [5]. As for the bridges with wooden spans, these are practically not built nowadays [21]. This does not apply to supports and decks, which are still often made of wood in temporary bridges [22].

Composite structures are currently one of the most popular solutions in bridge engineering. The components of the cross section are made from materials with different Young's moduli which work with each other thanks to the use of various connectors. The elements are connected to make the most of their various properties. The greatest benefits can be provided by composite structures made of steel and concrete [2].

Another important classification criterion is the division of bridges according to the time of their use, i.e.:

- permanent bridges,
- temporary bridges (traffic diversion bridges).

Permanent bridges are those whose lifetime is counted in decades. It requires the use of appropriate design solutions that are resistant to ageing and corrosion of materials. In the case of temporary bridges, the criterion of durability is only one of multiple criteria taken into account during the design. Other important criteria include:

- a low construction cost (compared to a permanent bridge),
- a short construction time,
- easy bridge assembly and disassembly
- the possibility to reuse structural components (portable military bridges).

The operating time of temporary bridges usually does not exceed a few years or a few months in the case of traffic diversion bridges built for the period of reconstruction of a permanent bridge.

Other classification criteria are important for a designer as well, e.g. the criteria that define the technical feasibility of the bridge (related to the limit spread resulting from the type of span structure, the possibility of being constructed in certain terrain conditions and the lead time etc.) and allow for the optimisation of costs and the attainment of the planned operating time. In addition, during a bridge design process, one of the more important issues is the static diagram of the structure. Fig. 1.1 shows the division of bridges according to the applied static diagram: beam bridges (simply supported beams, cantilever beams, continuous beams, continuous beams with joints), arch bridges (hinge-less, two-hinged and three-hinged arches), frames and beam-tension systems (cable-stayed and suspension systems).

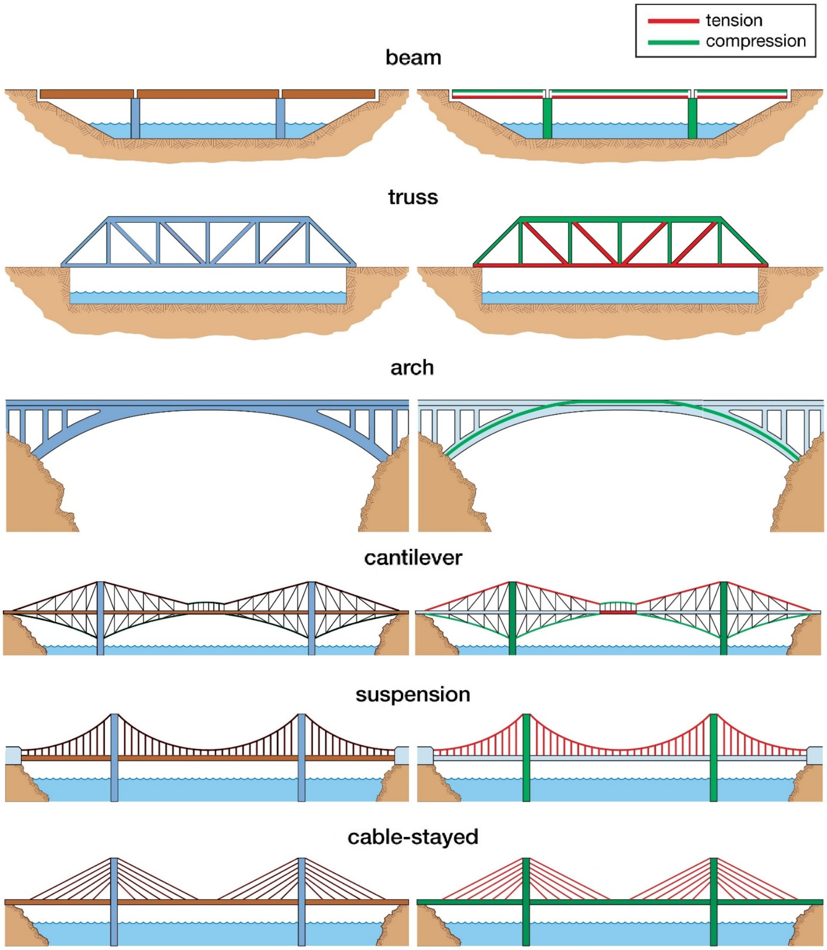

Fig. 1.1. Basic types of bridges

The presented division of bridges applies only to selected bridge features. In addition to these criteria, there are others that have a significant impact on the shape of the bridge in the cross and longitudinal sections, its usefulness, durability and the method of calculating the internal forces and dimensioning. For example, these can be division according to the foundation of supports on the ground, the variability in the positioning of the span relative to the obstacle, the type of spans, the positioning of the deck relative to the main girders, the height of the main girders and their number and the method of their construction [1].

1.2. Bridge elements

Bridges (with the exception of culverts) can be divided into two basic components (Fig. 1.2):

- spans,
- supports.

In addition to these structural elements, all civil engineering structures must also include many additional structural elements without which their operation would not be possible. These auxiliary elements are called secondary elements.

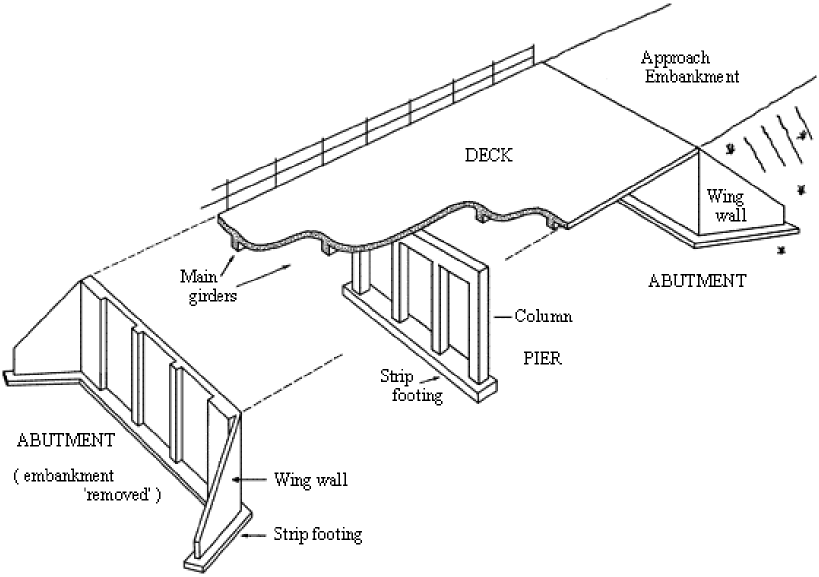

Fig. 1.2. Typical two-span bridge[1]

In some types of bridges, however, it may be difficult to clearly distinguish the essential parts of the structural system (e.g. masonry arch bridge).

1.2.1. Bridge spans

A bridge span is a three-dimensional structure contained between successive supports. A span ensures the free movement of road users over an obstacle and receives and transfers the permanent loads (its dead load and the weight of the secondary elements) and the operating loads to the supports.

[1] http://www.aholgate.com/genimages/girderbr.html

At each end, a span is supported on bearings and consists of:

- the main girders,
- the deck,
- bracings.

The complexity of the spans structure depends on the adopted static assumptions and the type and material used to construct them. The most complex span designs are usually those of steel truss bridges and the simplest – those of stone bridges.

Depending on the number of spans, bridges are called single-span bridges, double-span bridges, etc.

Main girders

The main girders are the proper supporting part of the span structure. They take and transfer the weight of the deck and the operating load to the supports.

The functions of the main girders can be performed by both beams (beam bridges), slabs (slab bridges) and arches (arch bridges). The number of girders can vary from one to several.

The basic construction materials used for spans nowadays are steel and reinforced concrete (with a conventional reinforcement or pre-stressed concrete).

Bridge deck

The bridge deck is present in most bridges, except for open railway bridges, and sometimes its role is played by a carrying plate, as in the case of plate bridges. The bridge deck can be based on main girders or on a grill with longitudinal and cross members, which can cooperate in transferring loads. The bridge deck can also be a self-supporting element. Bridge decks may be made from solid or laminated wood, steel, concrete, glass, aluminium or composite materials. The cross-section of the bridge deck and equipment are shown in Fig. 1.3.

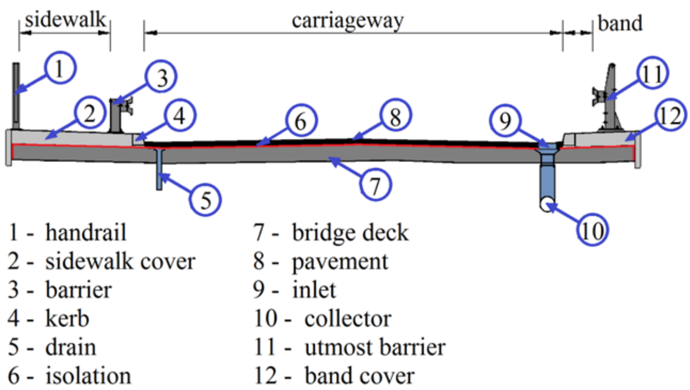

Fig. 1.3. Bridge deck section

Stiffeners

Stiffeners secure the cooperation of the various components of the span, protect span components against the effects of horizontal interactions and ensure the structural stability of the whole span. Stiffeners play an important role, in particular in steel truss bridges made from elements of low stiffness in the transverse direction.

Bearings

Bearings serve two basic functions:

- transfer of the load from the span onto the support,
- enabling free movements of the structure.

Depending on the freedom of movement, bearings are divided into fixed (allowing rotation of a structure, without any possibility of displacements in any direction) and sliding (allowing rotation of a structure with the possibility of displacement) ones. Sliding bearings can be unidirectional and multidirectional.

With regard to materials, bearings are divided into:

- neoprene (reinforced rubber)
- steel (tangent, roller, bowl, pot)
- concrete.

Bearing sets depend on the pressure exerted by the span and the expected direction of displacement. Steel roller bearings are used to transfer pressure no higher than 1000 kN. Bowl and pot bearings should be used to transfer pressure no less than 2000 kN. Individual elements of steel bearings should be adequately protected against corrosion.

Bearings, depending on the type and size, should enable securing of sliding surfaces and bearings from contamination. Bearings should be equipped with:

- the sliding indicators – when displacements of individual bearing parts are larger than 20 millimetres,
- elements that stabilize the components of a bearing during transport,
- and installation brackets – removed after assembling the bearing.

Bearings which operate tensile forces should be equipped with security anchors and also equipped for bearing performance.

1.2.2. Supports (extreme and intermediate)

Supports are divided into two groups, i.e. extreme (abutments, supports in contact with the embankment) and intermediate supports (pillars). Wooden bridge supports are called anchors.

Today, supports are made mostly of reinforced concrete, less frequently with plain concrete or steel. In the past, supports were built of stone or brick, and support bodies often wrapped with bricks or stone elements. Currently, in order to improve the aesthetic perception of supports, concrete surfaces are coated with colored or anticorrosion coatings or applied so called architectural concrete.

Temporary bridges are also carried by supports of wood or steel. Elements of supports can be delivered to site in prefabricated form.

The purpose of supports is to transfer to the ground their own weight, service loads and other loads acting on the structure. Outposts must also ensure the stability of the embankment at the junction with the object. There must be a sufficient space left under the support, resulting from the function performed by the object.

Abutments

Abutments (extreme supports) are divided into massive and sunk.

The main feature of sunk abutments is that their structure largely covers the embankment, penetrating the lower part of the structure. This design reduces soil pressure on the support, reducing material consumption, but also reducing the available storage space under the object. It also can increase the object length because of the requirement of structure horizontal light. In the case of solid wall abutments (massive) the situation is reversed.

The essential elements of the abutment are the foundation body, bearing bench, bearing ashlars, gravel wall, wings and transient plate. The elements of the abutment are shown in Fig. 1.4.

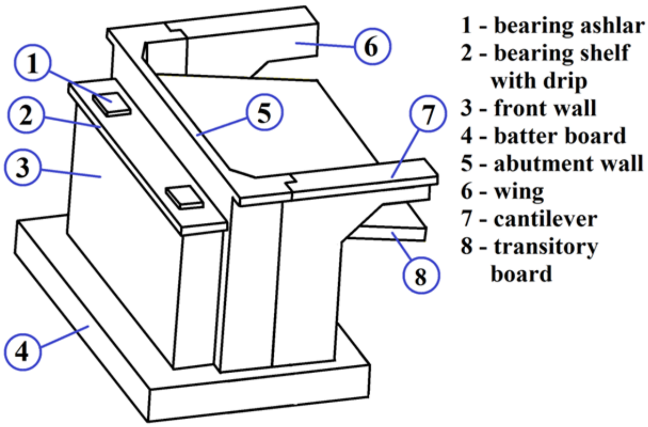

Fig. 1.4. Bridge abutment components

The role of the foundation is the same as in the case of the pillar. The body is designed to transfer the load on the foundation and ensures the stability of the embankment.

Wings also contribute to the stability of the embankment. The wing axis may be arranged in parallel, perpendicularly or at an angle with respect to the object's axis. The wings can be suspended to the body of bridgehead, standing on a footing or mixed construction. The wings must be embedded in the embankment at the depth of at least 1.0 m.

The bearing bench acts the as in the case of the pillar. It must be topped with a cornice which protects the body of bridgehead against dripping water.

The gravel wall is located in the upper of the abutment body. It protects the bearing bench from soil filling. At the junction of the body and the gravel wall the transient plate is usually based.

The transient plate (ramp) will ensure the continuity of changes in the stiffness at the transition from the road to the object. It protects also from the formation of faults at the interface between the embankment and the object. The plate has a length dependent on the height of the embankment, but no less than 4.0 m. The plate is made with a decrease of 10% on the outside of the object. Based on one end on the abutment, the other end stands on an embankment or on the bench embedded in the embankment. The plate must have sufficient stiffness and strength, and its thickness should not be less than 30 cm, while the concrete class not lower than C25/30.

Pillars

Pillars (intermediate supports) can be divided into river and overpass pillars or flyovers. Pillar could be massive or openwork. Massive pillars of the weight above 300 kN are called massive. The essential elements of a pillar are the foundation, body, bearings bench, bearing ashlars and starling in river pillars.

The foundation is designed to provide a secure transfer of load from the structure to the ground and ensure the stability of the structure. Foundations can be put directly on the ground (bench, feet) or under it (piles, wells, formerly well caissons).

The body is designed to carry loads transferred from the structure to the foundation and all kinds of horizontal loads acting directly on the pillar.

The bearing bench is a part of the crowning body which transfers the load of spans onto the body and distributes it evenly. On the bearing bench there can be placed bearing ashlars constituting the fulcrum for the bearings. Ashlars also protect the bearings from dirt and water. The pillar pole bearings bench takes the form of a beam called *head beam*.

Starlings occur in river supports or supports in flood plains, from the side of the influx of water. Their mission is to protect the pillar against the thrust of ice and floating materials that can damage the pillar.

1.2.3. Bridges equipment

An engineering structure, besides the basic structural elements (span, supports), must be equipped with additional non-structural elements which allow the full exploitation of the object and the execution of its assumed functions. The basic pieces of equipment include: insulation, pavement, curbs, cover paving, railings, barriers, expansion joints, control devices (stairs drains and ladders), drainage (drains, filters, drainage, collectors of storm water drainage, and in the case railway bridges also fenderings). Additional equipment can include: inspection trucks, lighting components, anti-electric shock shields, noise barriers, information boards.

Expansion joints are elements designed to overlap the expansion slots while providing the freedom of span movement and rotation. Expansion joints must be chosen depending on the expected displacement of the span.

Expansion joints can be opened (no airtightness) and closed. At present, mainly devices contained in the block and modular systems are used. A block expansion device consists of a block of rubber with steel inserts embedded and secured within the cavity of the expansion by means of steel screws. A modular device consists of a steel profile with a rubber insert inside. The device is mounted in a prepared niche. The next step is connection of the concrete plate reinforcement steel rods with device anchors. At the end, niche is filled with concrete. If the required compensation is a high shift, the device can consist of several modules, steel profile – vulcanized rubber – steel profile.

The facilities where displacements are small (under 1 cm) in order to secure the road surface over the expansion joint from cracking, a joint covering bituminous roof is applied. Along the expansion joint a trough of the width of about 40 cm (20 cm on either side of the expansion joint) to the depth of carriageway pavement thickness (approx. 10 cm) is made (the shape cut and forged mechanically). Then the trough is cleaned of impurities (blows, vacuums), and on the bottom a sheet of stainless steel covering the slot is placed. The trough is filled with aggregate surrounded by hot bitumen. The vertical edges of the trough should be heated before filling. Along the edges of the contact surface can be made a furrow and fill with an elastic sealing compound to ensure tightness.

Insulation (waterproofing) can be thin or thick. Thin insulation protects a concrete object in contact with the ground (footings, abutments from the embankment part of pillars embedded in the ground). Thin insulation is made of materials based on liquid asphalt emulsions by the cold or hot methods. Such insulation may be applied by painting or spraying. The thickness of this kind of insulation may be different, currently a layer 2 mm thick is required. Putting such a layer uniformly on a structure is difficult to achieve, even if painted several times. More and more often, spraying in multiple layers of lesser thickness is applied.

Thick insulation, often with a heat-sealable membrane, is laid directly on the bridge deck (over the entire horizontal surface), the horizontal parts of wings and the gravel wall. Its task is to protect the structure from rainwater and harmful substances that can penetrate into the pavement or sidewalks.

The insulation can be laid on the surface of a bridge deck, which previously must be properly profiled (dips, equality) and prepared (removal of cement milk). The insulation can be placed on a substrate that has an adequate peel strength and moisture. Water on the surface of isolation through appropriate declines flows into the drain, which is fed into drains and discharged outside the building. Insulation of the surfaces is made of heat-sealable roofing, tar paper and adhesive mastics. The minimum thickness of the insulation of a membrane are 5 mm and 2 mm coatings.

On the sidewalks (sidewalk covers) insulation-pavement made from synthetic resins (polyurethane, epoxy), or a modified bitumen emulsion with a thickness of 3–10 mm, which acts both as waterproofing and pavement on the sidewalk is often used.

In the case of the railway bridges with a ballast trunk, the insulation is protected with a protective layer (usually from cement concrete or asphalt).

Pavement is a part of the equipment over which the traffic moves and depends on the type of traffic that is carried along it. Pavements can be divided into road, rail, tram and pedestrian types.

Railway pavement can be open-ended (rails attached to beams based directly on the longitudinal beams) or closed (a track based on the bed of a crushed stone ballast in the trunk).

Road pavements have to meet the same functional requirements as the paving on a trail. The exception will be temporary surface objects which are usually made of wood. Nowadays, bituminous pavements consisting of two layers are mainly used – a layer of equalization (tack, binding) directly covering insulation and wear upon which traffic moves. Bituminous layers have a total thickness from 8 to 10 cm and can be made (binding or both) of asphaltic concrete or modified mastic (hard asphalt). The wear layer may also be made of SMA mixtures.

Tram pavement on bridges with a dedicated junction has a structure similar to the railway's. In the case of bridges with mixed car-tram traffic (track in the roadway), rails are generally fixed directly to the deck or embedded in trays and covered with masses of damping vibrations and noise.

The kerb is an element separating lanes of pavement. It is primarily a safety feature preventing a vehicle's entry onto the sidewalk. If, between the sidewalk and the road there is not a barrier, the curb height should be 14÷18 cm. If, between the sidewalk and the road there is a barrier, the curb height can be 8 to 14 cm.

Kerbs must be made of durable materials, resistant to corrosion and abrasion. Stone kerbs (granite) with dimensions of 20×18 cm and 20×20 cm are frequently used. The same applies to the reinforced concrete curbs polymers or plastics. Kerbs are placed on a layer of grit surrounded by synthetic resins or mortar. The contacts between kerbs and between the kerb and the surface must be permanently sealed by plastic that protects against the water penetration.

Sidewalk covers are pieces of bridge deck equipment, over which pedestrians and cyclists move. On one side a cover is limited by the kerb, and on the other side topped with cornice boards, monolithic or prefabricated. If the sidewalk is separated from the carriageway by a barrier, at the edge of the cover slabs railings are mounted. If the barrier does not exist, one must design an edge barrier. Band covers have a structure similar to sidewalk covers, but do not carry pedestrians or cyclists. They are used to fasten edge barriers.

Railings. Engineering constructions should have railings to prevent people from falling off, if the difference between the level of pedestrian or cycling traffic and the level of the adjacent ground is greater than 0.5 m. The height of railings on the sidewalk should be at least. 1.1 m, on the cycle path min. 1.2 m, the pavement over the railway line min. 1.3 m, and in the railway bridges on the side of the railway track – min. 1.5 m. Railings should be topped with handrails, and the inside filled with vertical cuts of the maximum spacing of 14 cm. Technical railings may be composed of a balustrade handrail and two parallel rods.

Protective barriers have two basic functions, i.e. to protect vehicles from falling off an object and to protect pedestrians from collisions with vehicles. The barriers are made of steel (susceptible) or concrete (rigid). Barriers on the object should be selected in accordance with the anticipated type of traffic. Barriers are classified in [31].

The **drainage system** of a bridge consists of inlets, drainage, sewerage drains and collectors leading water outside the structure to receivers (settling tanks, separators, storm water drainage). Drains are used for the acquisition of surface water from the road or bridge slabs in railway bridges. The distance between inlets depends on the inheritance vertical alignment of a road (from 5.0 m at $i = 0.3\%$; up to 25 m at $i \geq 2.0\%$). A road rainwater runoff should not be greater than 30 m. The inlet must have a pollution settler. Water from drainage collectors is directed outside the object.

A drainage is made of a grit (8÷16 mm) surrounded by synthetic resin or geotextile strips surrounded by grit. The drainage is placed in the axes of dehydration before dilatation devices, and in the places of anticipated stagnant water on the level of insulation under pavement and under the sidewalk covers. They are output to the drains and inlets. They should be protected from damages.

Drains are tubes of steel or plastic, completed with the cup, placed in the axis of drainage. The main role of drains is to lead the water from the drainage. The drains are arranged at distances of 3÷5 m. Drains, despite their small size, are a very important element of dehydration, and errors in their execution may contribute to the corrosion of other parts of the bridge [7].

Lighting should be adapted to the type of objects approaching it. Lighting equipment should be fastened to street lamps around the waist or outside balustrade railings on the edges of the sidewalk covers or extra supports. The power cables of lanterns should be carried in plastic conduits embedded in the construction covers of sidewalks (with the possibility of revision) or suspended from the construction of the span.

Screen barriers. In areas required to be protected from noise, in accordance with the applicable law, or where the noise limit values are exceeded, one mounts protective equipment against noise called screen barriers. Their job is to absorb and/or reflect the noise on the object to reduce it to the required level. Their design must effectively protect against noise, do not impede the access of light by the adjacent buildings and allow the evacuation of fumes.

Guards protect against electric shock from catenary wires used on railways, tram or trolley ways. Guards should be located at the railing or barrier. Guards should be mounted at a distance not less than 2.0 m from elements under voltage.

1.3. Material been used for bridges and ecological objects

1.3.1. Introduction

Over the entire period of the development of bridge engineering, there has been a close relationship between the material used and the adopted design solution. Analysing the relationship between the bridge material and structure throughout history, one can notice a trend. In its initial period, bridge engineering was based directly on the structures observed in nature.

The continuous improvement of materials and their properties, as well as the better understanding of the structure and the improvements in the computational methods (discussed in Chapter 4), have helped reduce the permanent loads and construct bridges with increased spans lengths. These trends have been particularly clear since processed materials started to be used.

It should be remembered that the creation of new materials has not been generally related to the construction industry (the only exception is concrete and the materials associated with it). They have been created for other purposes, and then improved and modified for the purposes of the construction industry, including the bridge industry.

The high requirements for materials used in bridge engineering result from static strength, dynamic and fatigue requirements on one hand and on the other – from the required high resistance to environmental impacts associated with the direct influence of the environment and the use of de-icing agents during operation.

The choice of building materials used in the construction of bridges has an influence, among the others, on the expected service loads, environmental conditions in the vicinity of the object, its estimated durability and often the cost of its construction and subsequent maintenance [1].

Considering the characteristics of service loads, materials for the construction of a railway bridge will be chosen differently than materials for a footbridge. Similarly, bridge deck materials as opposed to bridge supports materials. The bridge location, types of obstacles, hydrological and geological conditions also have an impact on the construction and related material capabilities.

Bridge structures are exposed to weather conditions, i.e. humidity, rain and snow, cyclic changes of temperature, freezing and thawing, insolation and flowing ground and surface waters.

Air, precipitations and water contain various chemical contaminants which may locally adversely affect the stability of embedded materials, and thus may affect the stability of a bridge structure globally. What's more, the bridge environment could change locally. Chemical factors negatively affecting the durability of the materials used to erect an object. Chemicals or more precisely different ions, especially Cl^-, may be introduced by the traffic or by the winter maintenance. De-icing agents or other chemical agents associated with the railway or road traffic can cause corrosion and ultimately destroy the element exposed to an aggressive environment. Finally, the object’s environment affects the structure chemically, physically, biologically, and mechanically contributing to the degradation of used materials. When selecting materials it is essential to know their resistance to environmental influences. It is possible to ensure the assumed and required durability of a structure or its components.

The choice of construction materials also requires decisions in the field of economic planning. Here the following question arises: whether to build with expensive materials of higher quality ensuring low costs of maintenance, or with relatively low-cost, lower quality and durability materials which will inevitably result in higher costs of maintenance. The decision is at the discretion of the investor. Considering the problem in the long run, the investor should look for a solution ensuring minimal interference with the maintenance of the facility. Unfortunately, the lack of investors’ awareness as well as the lack of uncompromising designers can cause misguided choices. On the designer's experience and authority depends whether the structure designed with an initial higher cost of construction, but with lower costs of maintenance in the perspective of a long-term service, will be built.

Construction materials widely used in bridge construction are concrete and steel. They are characterized by considerable immediate and fatigue strength, elasticity, resistance to influence of rheological properties, fracture toughness, impact of the environment (after the application of anti-corrosion treatments). To a lesser extent, aluminium and wood construction are used. In bridge construction there is an increasing interest in the use of composite materials (fiber-reinforced polymers), plastics, glass and recycled materials.

Bridge foundations and supports (abutments and pillars), which are in constant contact with the soil and water, in general are made of concrete. Concrete foundations, i.e. piles, wells and benches, below the frost line do not need to have specific characteristics. The primary role in selecting the characteristics of concrete play in this case, low water absorption and water permeability, reducing the technological scratches and proper resistance to chemical attack of the environment. Higher requirements apply to foundations immersed in water, particularly marine and chemically aggressive soil and aquatic environment.

In the case of the bodies of supports, the choice of material properties is influenced primarily by durability. The durability of supports may be affected by humidity, precipitation, insolation, pollution, surface water, drying and humidification, pressure of ice floes and impact of road vehicles or floating objects. For this reason, one uses concrete with a low water absorption and water permeability, frost resistance, adequate mechanical strength and resistance to chemical compounds that may be found in water.

Lightweight supports (frame, pole, disc) can be made of concrete or structural steel, if raised above water or a dry obstacle. The selection of materials for light supports should be guided by the requirements of both the surrounding environment, strength and of taking into account winter maintenance in the case of road.

Culverts, depending on their width, environment and design load can be made of concrete, corrugated metal sheet or plastic.

Concrete and steel are primarily used in construction of spans. In addition, there may be used materials lighter than concrete and structural steel, such as aluminium and composite materials. Wood is used for temporary structures or less loaded bridge spans, such as footbridges. More and more often, in the construction of footbridges deck glass is used.

When selecting structural materials of spans the main criterion is strength. In the case of the dominance of permanent loads, such as dead load and equipment, the required feature is the strength of materials and their resistance to a possible increase of deflection during operation. Materials of span structures may contribute to the occurrence of dynamic effects and fatigue, especially in the case of road and rail bridges. The most vulnerable to the influence of dynamic and fatigue loads are the decks that directly bear the pressures of vehicle wheels. The use of high-strength materials in order to increase the load capacity at the expense of a reduction in its cross-section, and as a result the weight and stiffness, can cause counterproductive effects. Strength increase usually results in a reduction of the deformation limit, causing the fragility of the material and a reduction of the sensitivity to the dynamic and fatigue loads. Increasing strength shoulb be connected to an increase of the modulus of elasticity. It results in increased rigidity and a reduction of formability. This can lead to cracks and scratches resulting in a dicrease of the load capacity and durability.

In the case of main girders, the choice of materials is determined by both the load, span length and a construction system. The use of materials with higher strength may result in a reduction of the dead load to a significant degree (several-tens of percent). As a result, the value of forces transferred to supports and foundations is lowered. It opens up the possibility of reducing their sections, and the costs of their construction. With an increase of the span length, increases the ratio of the dead loads to service loads, causing a loss of dynamics and fatigue. Moreover, an increase of the span length, impacts on the selection of materials as well as structure types that may reduce span deflection.

Materials and products used in bridges are also vulnerable to environmental influences. They can partly carry live loads (e.g. the surface of a road), and to a certain degree are subject to deformations and vibrations caused by traffic. Moreover, elements of optional equipment may be exposed to vandalism.

Today's technology cannot yet produce all bridge building materials and elements which serviceability would be on the same level and correspond to an assumed period of use. The relevant regulations [28] provide that when using available materials and providing a basic level of maintenance bridge element lives should not be less than:

- 200 years – the supports of bridges in stagnant waters of a stable level, 150 years in the depths of rivers and 100 years on flood plains,
- 100 years – massive abutments, retaining structures, massive arched and plate structures and tunnels,
- 80 years – beam or box carrying systems with massive decks,
- 60 years – overpass supports, lightweight abutments, beam or box carrying systems with lightweight and densely-finned decks, entire cross-section of prestressed carrying systems,
- 50 years – tangential and roller bearings,
- 40 years – culverts, massive decks,
- 30 years – lightweight and densely-finned decks, massive deck waterproof insulation, railings,
- 25 years – drainage facilities,
- 20 years – elastomeric bearings and with sliding pads, waterproof insulations of lightweight and densely-finned decks, paving, railing beams, expansion joints, anti-electric shields and barriers,
- 15 years – new anti-corrosion coatings of steel structures,
- 10 years – road surface, provided that it is not intended as a protective waterproofing layer,
- 5 years – repainted anti-corrosion coatings of steel structures.

1.3.2. Concrete

When designing bridges, in addition to a grade of concrete on must specify additional physical, mechanical and structural characteristics.

Assuming environmental classifications introduced in the European Union, a concrete bridge may be exposed to different classes of exposure [30].

The concrete exposure class XC refers to corrosion by way of carbonation, which is dangerous if the concrete exposed to air and moisture has reinforcement. The exposure class XD refers to concrete corrosion caused by chlorides in fresh waters or the air. The exposure class XS refers to concrete corrosion caused by chlorides in seawater. The exposure class XF refers to corrosion of wet concrete caused by alternating freezing and defrosting. The exposure class XA concerns ground and groundwater contaminated with chemicals.

According to the requirements [30], to ensure at least a 50-year durability of an object, one must apply different strength classes of concrete, cement content and air, depending on the exposure class. In the case of the exposure class XC1, the minimum grade of concrete is C20/25, therefore at least the concrete class C25/30 should be applied in the case of the environments XC2 and XF2, and at least the concrete class C30/37 in the case of the environments XC4, XS1, XF1, XF3, XF4, XA1, XA2. In the relation to the environmental actions the concrete grade C35/45 with exposure class XD3 and XA3 is commonly used. Moreover, in the chemically aggressive environments XA2 and XA3, sulphate resistant cement (HSR) and low heat of hydration cement (LH) for massive pillars, and in the case of alkali aggregate reactivity, low-alkali cement (NA) is demanded.

In the construction of carrying systems, especially for prestressed systems, the use of Portland cement (CEMI), is recommended. For the construction of supports and foundations, it is advisable to use metallurgical cement (CEMIII), because of the low heat of hydration, slow chemical bonding and reduced shrinkage, allowing the reduction or even elimination of shrinkage cracks.

Currently, the relevant regulations [28] require that in the design of bridges only the following concrete classes be used:

C20/25 – in foundations and massive supports (thicker than 60 cm) in a non-aggressive environment,

C25/30 – in lightweight supports, foundations and reinforced concrete spans in an aggressive environment,

C30/37 – in prestressed constructions,

without reference to the above described exposure classes that should be considered in addition. Additional requirements apply to concrete absorption i.e. no more than 5%, sometimes even 4% water permeability of at least W8 and frost resistance of at least F150 [38]. The current standard [30] does not require the water absorption and frost resistance of concrete. This results in a certain inaccuracy associated with the continued requirement for these parameters described in [28].

Tab. 1.1. Bridge concrete classes

Strenght class of concreto								
Non-structural concrete		Structural concrete						
C12/15	C15/20	C20/25	C25/30	C30/37	C35/45	C40/50	C45/55	C50/60
Compressive strength – cylindrical sample [N/mm^2]								
12	15	20	25	30	35	40	45	50
Compressive strength – cubic sample [N/mm^2]								
15	20	25	30	37	45	50	55	60
Tensile strength [N/mm^2]								
1.6	1.9	2.2	2.6	2.9	3.2	3.5	3.8	4.1

Concrete requirements are largely related to the reduction of structure destructing processes, but may also relate to the physical characteristics or mechanical properties (strength, modulus of elasticity, shrinkage, creep). The concrete classes are shown in Tab. 1.1.

1.3.3. Steel

Durability of steel depends primarily on its resistance to corrosion. The corrosion of reinforcing steel is prevented by a suitable concrete cover. Prestressing steel is secured by introducing injectable materials to cable or casing pipes. In justified cases, additional galvanic and plastic coatings on bars are also employed.

In order to ensure the sustainability of prestressing steel and reinforcing steel, the minimum diameters of rods has been introduced, i.e.:

- 4 mm for wire compression,
- 6 mm for reinforcing rods,
- 15 mm for tension rods.

The durability of structural steel is primarily reinforced by protective coatings, painting and/or metallization. Application of protective coatings may be disregarded only in the case of stainless steel. Construction steel of this kind, however, cannot be used in highly industrialized regions (chemically aggressive environment) and with high humidity. In such conditions it does not produce a natural protective layer of corrosion.

The usefulness of structural steel for the construction of a bridge mainly consists in its physical and mechanical properties (strength, yield strength and toughness). They depend on the implementation of the technological process of steel, the chemical composition (primarily carbon) and heat treatment. For the construction of bridges structural steel with reduced sulphur and phosphorus and low carbon content is especially suitable. Structural steel should be ductile and weldable.

Bridge construction steels have a yield strength ranging from 235 to 460 MPa, on the base thickness of steel up to 40 mm [35]. They exhibit plasticity (the ratio of tensile strength to yield strength) of not less than 1.10, elongation at break of not less than 15%, and 15-fold higher than the deformation limit strain at yield. Yield stress (f_y) and ultimate tensile strength (f_u) of the bridge steels are shown in Tab. 1.2.

Construction steels are marked in the following way:

- N are normalized or normalized rolled,
- M are thermo-mechanically rolled,
- Q are quenched and tempered,
- W they have increased resistance to atmospheric corrosion,
- L are intended for use at low temperatures,
- H they have high hardenability steel.

In contrast to concrete, in the case of steel, in the design one uses materials selected from the finite range of products with specific characteristics. If a given type of steel meets the standard requirements, it simultaneously fulfils material requirements.

Tab. 1.2. Nominal values for hot rolled structural steel

Steel grade	Nominal thickness of the element t [mm]			
	$t \leq 40$ mm		40 mm $< t \leq 80$ mm	
	f_y [N/mm^2]	f_u [N/mm^2]	f_y [N/mm^2]	f_u [N/mm^2]
S 235	235	360	215	360
S 275	275	430	255	410
S 355	355	510	335	470
S 450	440	550	410	550
S 275 N/NL	275	390	255	370
S 355 N/NL	355	490	335	470
S 420 N/NL	420	520	390	520
S 460 N/NL	460	540	430	540
S 275 M/ML	275	370	255	360
S 355 M/ML	355	470	335	450
S 420 M/ML	420	520	390	500
S 460 M/ML	460	540	430	530
S 235 W	235	360	215	340
S 355 W	355	510	335	490
S 460Q/QL/QL1	460	570	440	550
S 235 H	235	360	215	340
S 275 H	275	430	255	410
S 355 H	355	510	335	490
S 275 NH/NLH	275	390	255	370
S 355 NH/NLH	355	490	335	470
S 420 NH/NLH	420	540	390	520
S 460 NH/NLH	460	560	430	550
S 235 H	235	360		
S 275 H	275	430		
S 355 H	355	510		
S 275 NH/NLH	275	370		
S 355 NH/NLH	355	470		
S 460 NH/NLH	460	550		
S 275 MH/MLH	275	360		
S 355 MH/MLH	355	470		
S 420 MH/MLH	420	500		
S 460 MH/MLH	460	530		

In addition, steel is marked by letters or letters and numbers characterizing the toughness of steel [J] and the breaking work test temperature. The letter J denotes a value of breaking work 27 J, K – 40 J and L – 60 J. Followed by the letter R or O or digits 2, 3, 4, 5, 6 it refers respectively to a temperature of toughness test 20ºC, 0ºC, -20ºC, -30ºC, -40ºC, -50ºC, -60ºC.

Due to the influence of dynamic fatigue, resistance of steel to fracture to which the structure is exposed at low temperatures is also required. In [34] it is required that the lowest computational design temperature of an element exposed to weather for a foreseeable period, in which there is a representative stress, be not less than the computational temperature of the steel cracking.

In the case of steel – unlike concrete – materials are selected from a limited range of products with specific characteristics. If a steel type meets the requirements set by a standard, it also meets material requirements.

The most important characteristics of reinforcement steels in the design include tensile strength, (f_t), yield strength (f_y), ductility (plastic elongation), the type of surface, weldability and fatigue strength. Yield strength is an essential mechanical characteristic corresponding to the stress at which the permanent unit elongation is 0.2%.

Reinforcement steel can be smooth or deformed. It is also recommended to use weldable killed steel. The classification of reinforcement steel according to [33] is shown in Tab. 1.3.

Tab. 1.3. Classification of reinforcing steel

Steel class	Characteristic yield strength f_{yk} [MPa]	Ratio of tensile strength to yield strength k	Characteristic strain at maximum force ε_{uk} [%]
A – low ductility steel		≥ 1.05	≥ 2.25
B – medium ductility steel	400÷600	≥ 1.08	≥ 5
C – high ductility steel		1.15÷1.35	≥ 7.5

Apart from the specific requirements for chemical composition, mechanical properties and deformations of the reinforcement steel produced from low carbon steel and low alloy steels, bridges can use the steel types listed in Tab. 1.4.

Tab. 1.4. Mechanical properties of reinforcing steel

Steel class	Steel grade	Nominal diameter [mm]	Characteristic yield strength f_{yk} [MPa]	Characteristic tensile strength f_{tk} [MPa]
A	B500A	4÷16	500	550
	RB500 RB500W	6÷40		
B	RB400 RB400W	6÷40	400	440
C	B500SP	8÷32	500	575

The tensioning by means of steel tendons is possible thanks to the use of a high strength steel. Such a steel can be obtained by:

- a heat treatment (patenting, hardening) and cold forming (pull broaching),
- increasing the carbon content in the steel up to 0.9% (1.0%) while adding a small amount of noble metals (manganese, silicon) and reducing the content of impurities in the steel (sulphur, phosphorus).
- the addition of noble metals (e.g. manganese, silicon, nickel) while reducing the carbon content down to ~ 0.3%.

Wires made of high-carbon cold drawn steel can be applied directly or used to produce strands or ropes. In order to ensure maximum durability of bridges, pre-stressing steel should meet a number of requirements, such as:

- have a yield point of not less than 85% of the tensile strength and a yield point at the strain of 0.1%,
- the minimum elongation at rupture of 3.5%,
- be characterised by a low relaxation,
- ensure an adequate fatigue strength (at 2×10^6 of stress change cycles),
- provide a minimum tensile strength of strands in a complex stress state and provide resistance to stress corrosion.

Table 1.5 shows examples of mechanical properties of pre-stressing steel produced nowadays [39].

Tab. 1.5. Mechanical properties of prestressing steel

Types of prestressing steel	Steel name	Nominal diameter [mm]	Cross sectional area [mm^2]	Characteristic value of breaking strength F_{pk} [kN]
Wires	Y 1770C	5.0	19.6	34.7
	Y 1670C	7.0	38.5	64.3
Weaves	Y 1860S7	13.0	100	186
	Y 1770S7	16.0	150	265
Bars	Y 1030H	40	–	1295
	Y 1230H	40	–	1546

Explanations:
Y – prestressing steel, C – cold drawn wire, S –weaver, H – hot rolled bar, 7 – number of wires in weaver.

Pre-stressing steels approved for use in bridge construction available in the European Union include:

- wires with the diameters 4–10 mm and the strength of 1860–1570 MPa,
- stands with the diameters 5.2–16 mm and the strength of 1960–1700 MPa.

Furthermore, it is permissible to use smooth or deformed stainless steel bars with the diameters of 15–50 mm and the strength of 1100–1230 MPa, which must meet the requirements for fatigue strength, resistance to stress corrosion and relaxation.

1.3.4. Timber

Wood is one of the raw materials used by man since ancient times, also as a building material for bridges [21]. It is a material with an uneven structure. Both its appearance and physical and mechanical properties vary depending on the wood grain.

As a building material, wood is characterised by a low density, ease of treatment but also a low durability under changeable atmospheric conditions and the variability of the mechanical properties caused by its anisotropic structure.

The main elements usually use softwood (coniferous wood). Hardwood (deciduous wood) is used for small-sized elements. Connections between various softwood elements should use hardwood of higher class than the elements being connected. The hardwoods that are used the most frequently in Poland are oak, beech and acacia, ash and hornbeam. Among conifers, the most popular type is pine, followed by spruce and fir.

Tab. 1.6. Strength classes of structural timber – characteristic values

Properties	Poplar and coniferous species							Deciduous species					
	C24	C27	C30	C35	C40	C45	C50	D30	D35	D40	D50	D60	D70
Bending	24	27	30	35	40	45	50	30	35	40	50	60	70
Tension paralel	14	16	18	21	24	27	30	18	21	24	30	36	42
Tension perpendicular	0.5	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6
Compression parallel	21	22	23	25	26	27	29	23	254	26	29	32	34
Compression perpendicular	2.5	2.6	2.7	2.8	2.9	3.1	3.2	8.0	8.4	8.8	9.7	10.5	13.5
Shear	2.5	2.8	3.0	3.4	3.4	3.8	3.8	3.0	3.4	3.8	4.6	5.3	6.0

In bridge construction both solid and glued wood can be used. Solid wood is a material that is easy to handle and can be used without processing (round timber) or after treatment in the form of beams or planks (lumber). Logs can be used for foundation supports, head beams, and machined and glued wood in main girders and bridge elements. Wood used to build permanent bridges should meet the requirements of the class wood bending strength above 24 MPa. In order to increase the resistance of wood to moisture, insects, fungi, and fire, impregnation is used.

Tab. 1.7. Strength class of homogeneous glued laminated timber

Properties	Strength classes			
	GL 24h	GL 28h	GL 32h	GL 36h
Bending	24	28	32	36
Tension	16.5/0.4	19.5/0.45	22.5/0.5	26/0.6
Compression	24/2.7	26.5/3.0	29/3.3	31/3.6
Shear	2.7	3.2	3.8	4.3

The strength properties of solid softwood and solid hardwood are presented below in Tab. 1.6 [29]. In contrast, the strength properties of homogeneous laminated timber used in the construction of bridges are shown in Tab. 1.7.

1.3.5. Composites materials

Composite materials used in bridge engineering can be divided into:

- concrete-like composites (fibre-cement),
- plastics reinforced with fibre reinforced polymer.

Fibre-cement is a material in which steel wires or synthetic fibres are added to the concrete mixture to increase the tensile strength. Another concrete-like composite material is the so-called ultra-high strength concrete. The desirability of high-grade concretes in bridge engineering results from their much greater durability and the so-called high early strength, which makes it possible to shorten the construction cycles compared to conventional concretes [3].

Composite materials used in repairs include concretes made of cement modified with synthetic resins in the form of ready-made mortars (PCC) and concretes made of synthetic resins (PC). The composition of these concretes includes only aggregate and monomer emulsion used as a binder.

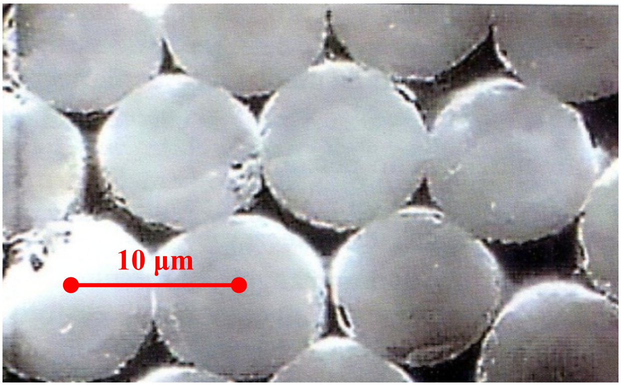

Fig. 1.5. Tape cross section [15]

Among plastics reinforced with various kinds of fibres used both for the construction of new bridges and for the repair or strengthening of the existing structures are [15], [10]:

- carbon fibre reinforced polymers (CFRP),
- glass fibre reinforced polymers (GFRP),
- aramid fibre reinforced polymers (AFRP).

FRP composites are both in the form of rods, fittings, mats and tapes. They consist of a large number of small continuous oriented non-metallic fibres with high tensile properties contained within a resin matrix (Fig. 1.5).

Tab. 1.8. Properties of the fibers used for FRP

Material	Modulus of elasticity [MPa]	Tensile strength [MPa]	Tensile strain [%]	Density [g/cm^3]
C carbon fibres				
High strength	215–235	3500–4800	1.4–2.0	1.7–1.9
Ultra high strength	215–235	3500–6000	1.5–2.3	
Highly modular	350–500	2500–3100	0.5–0.9	
Ultra high modular	500–700	2100–2400	0.2–0.4	
Glass fibres				
E – include boric acid and clays in their composition, AR – alkali resistant	70	1900–3000	3.0–4.5	2.6
S – higher tensile strength and higher stiffness	85–90	3500–4800	4.5–5.5	
A aramid fibres				
Low modular	70–80	3500–4100	4.3–5.0	1.4
Highly modular	115–130	3500–4000	2.5–3.5	

Most often FRP fibre content is in the range of 25–75% (50–75% for tapes and 25–35% for sheets). The properties of the fibres used in the manufacture of FRP composites are shown in Tab. 1.8.

Tab. 1.9. Selected properties of typical composites

Type of composite	Width [mm]	Thickness [mm]	Tensile strength [MPa]	Strain at break [%]	Modulus of elasticity E [GPa]
CFRP tape	50, 60, 80, 90, 100, 120, 150	1.2; 1.4	2800	1.8	150
			2800	1.7	165
			3200	1.8	200
			2400	1.2	210
			1600	1.8	230
			3900	1.5	280
			1300	0.45	300
			1800	0.45	400
			2800	0.4	640
C mats	300, 600, 670	0.065÷0.3	3800	1.2÷1.55	240
			2650	0.4	640
E mats			3400	4.5	73
AR mats			3000	4.3	65
A mats			2900	2.5	120

FRP composites are characterised by a low density and tensile strength greater than steel (even above 2800 MPa) and the value of the modulus of elasticity depending on the adopted composite (Tab. 1.9). Additionally, they have a high resistance to fatigue and corrosion. For this reason, they have been used in the aerospace, automotive and shipbuilding industries for many years, and for approx. 30 years now – also increasingly in the construction industry. In Poland, composite materials have been used so far mainly as reinforcing elements in the form of mats and tapes. It was not until recently that a successful attempt to build a road bridge using FRP composites was made in Blazowa near Rzeszow (designed by Com-bridge).

In the USA, the first road bridge made of composites was built in Kansas in 1996. Over the next eight years, more than 100 similar structures were built or modernised there using FRP materials [20]. The experiences of the designers, investors and contractors of the bridges made of polymer fibre composites around the world encourage to address these issues more widely.

1.3.6. Non-structural materials and fittings for animals passages

Facilities intended for use by animals are constructed with typical materials (steel, concrete) as in the case of ordinary road engineering structures. However, the intended use of these objects makes it necessary to use additional materials and components. These are usually natural materials such as native fertile soil (humus), stones and boulders, natural aggregate – gravel, trees and vegetation (shrubs, trees, perennials, grass).

Transitional land and entrance areas require a surface of fertile soil of thickness up to 1.0 m covered by adequate vegetation.

Culvert bottom slabs for small animals need to be covered with a layer of mineral soil. Culvert bottom slabs for amphibians require a cover layer of soil with a high ability to retain rainwater and to adjust the nature and structure of vegetation to those found in the environment of transition, taking into account potential natural vegetation species and actual vegetation.

Animal crossings require a grass cover on the surface of overpasses and underpasses by sowing a type of grass or a mixture of grasses and legumes. Such objects require the introduction of [11]:

- dense row of shrubs,
- row of vines on protective fences,
- transition surface and entrance areas with shrubs, perennials and trees,
- arrangement on the surface of a transition and on embankment invade rootwood (a few/several pieces),
- arrangement on the surface of overpasses and underpasses at outlets of larger boulders (a few/several pieces).

Introduction of safeguards (stumps, boulders) is needed to reduce of animal areas by people. Should be used boulders, root stumps, logs, piles of branches and planting thorny bushes.

1.4. Aesthetic shaping of bridge

1.4.1. Aesthetics as a notion

The notion of bridge aesthetics is connected with higher-tier feelings, as peculiar to humans since time immemorial [23]. Aesthetics-related considerations are associated with the functionality of structures; they shape the space, arousing the sense of harmony owing not only to outward appearance: they impact man's psyche and culture as well. Construct-ing bridges is an art, one that calls for special attention, since bridge structures become, as a rule, lasting elements within their surroundings and environment, 'accompanying' us for several dozens of years.

Bridge engineering has lately been dominated by technological development and economic drivers. Bridges reflect today the society's civilisational development. 'Aesthetic design' is understood very broadly at present: apart from the beauty of the object as such, the concept seeks to observe the basics of form. In terms of functionality as well as construction and materials used, a bridge ought to be matched with its surroundings, not intervening in its environment in any manner whatsoever. It moreover should be in harmony with the surrounding environment, or even enrich it, in certain cases. Quite importantly, the effect of changes appearing with time on the perception of the bridge ought to be prevented. Bridges are, normally, more astonishing structures compared to other architectural elements; they are prevalent and accessible as works of architecture also for those who are not interested in art. Bridges each have their individual character. Apart from transport and communication, they play a social role as well.

Aesthetics is defined as a science of beautiful objects and arts, part of its scope being considerations of aesthetic experience. European aesthetics originated in ancient Greece and has been developing till our day. This continuous development is not free of moments of severity and resistance, breaks and turns. The most violent turns followed the collapse of the Roman Empire and subsequently appeared in the Renaissance era. Apart from influencing the aesthetics, these changes affected the entire European culture. The developments in question make legitimate the identification of three periods in aesthetics: ancient, mediaeval, and modern [24].

Ancient aesthetics spans around a thousand years, and forms the foundation of European aesthetics. It was prevalently developed by the Greeks; later on, other nations have made their contributions.

The notions and concepts developed within the aesthetics founded by the Greeks were original and, to an extent, took shape before the era of philosophers; as such, they were much different from those commonly used today. 'Beauty' referred to anything that aroused recognition. The idea of beauty was pretty broad, extending not only to views and sounds but also personality traits, for instance. The Greeks considered the concept of beauty a material and intellectual good. It was them, it is generally assumed, to have created the Great Theory. Plato believed that beauty was something worth living for, and placed it on equal footing with truth and good. Thus, the three greatest values were established: truth, good, and beauty – the triad that has ever since remained part of the European thought. Beauty would be based on the matching of proportions, and on a relation of the simplest numbers. In music, the Greeks would fundamentally use the intervals of octave (1:2) and fifth (2:3); it was particularly in the human body that they found the proportions of 1:8 and 1:3; in architecture, 5:8. "Every domain of art has its own, peculiar types of relations between elements constructing a work of art. In architecture, for that matter, the relations are spatial, as opposed to temporal relations in music. In bridge design/construction, like in architecture, the beauty of a bridge is mainly founded upon the number and lengths of spans, the slenderness of supports and of the load-bearing structure" [13].

The Middle Ages have preserved the ancient theory and view of beauty. St Augustine was of opinion that beautiful things are such per se, rather than because they please somebody. According to this thinker, beauty is measure, form, and order (*modus*, *species*, *ordo*). The age of Boethius (fourth century) begot the mediaeval formula of beauty: commensurability of members (*commensuratio membrorum*). The theory of beauty was dualistic in the mediaeval age. Some claimed that proportion is the foundation of beauty; others would say that clarity and appropriate proportion is an inherent part of beauty. Everything comes from God's will, it was believed; a work of art is born in the artist's soul, his tools, and the shaping of matter.

The Renaissance resumed the ancient theories of beauty, perceived as measure, shape, and order. Leonardo da Vinci believed that beauty is not only observable by human senses but also consumed by the mind. Harmony was the most sublime expression of beauty. Many a philosopher considered the issue of beauty by making new observations and drawing new conclusions, as recapitulated and summarised by W. Tatarkiewicz [23].

The nineteenth century saw two theories of beauty emerging: in G.W.F. Hegel's approach, beauty is a revelation of an idea; B. Croce, for a change, saw beauty as an expression of the psyche. The conviction that beauty is subjective led to the formulation of a view whereby it is one's aesthetic experience, rather than beauty, that is the basic concept of aesthetics. Two currents have emerged in the views closer to our day: the aesthetics of expression and the aesthetics of contemplation. It is expressing one's inner life through art that matters the most: as V. Kandinsky wrote, any form is an expression of spiritual content.

The twentieth century saw a departure from the classical rules of beauty and, consequently, quit the idea of masterly performance. Symmetry, balance, cohesiveness and coherence, and unity, have been replaced by asymmetry; balance has turned into unstable equilibrium. Decomposition has become prevalent; genres and types of art have become integrated, and technological development taken advantage of; aesthetics has become generalised. Avant-garde and artistic conventions triggering the sort of reception of aesthetics that is permeated with shock or provocation have become omnipresent.

1.4.2. Experiencing aesthetics

Architecture as well as construction tend to beget feelings or sentiments as part of conceptual and design activities, and in the use of a building or structure. The intensity and scope of this experience is determined by a variety of factors that influence, to a larger or smaller degree, both the designer and the user/consumer. Three processes, describable as 'perceptual image', 'implementation reckoning', and 'classifying opinions', can be specified as far as architectural sentiments are concerned (Fig. 1.6); each of them being, possibly, positive or negative.

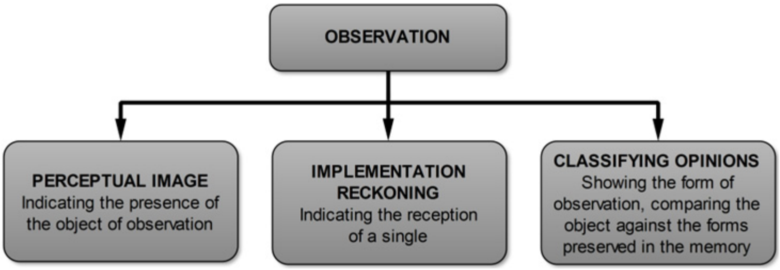

Fig. 1.6. Architecture perception experience

Observations, things spotted/perceived, imply diverse feelings that appear one after another or overlap at various time intervals, contributing altogether to an experience, whilst remaining mutually independent. Reception of a stimulus may trigger a pleasant sentiments whereas the feelings related to the 'implementation reckoning' or 'classifying opinion' may be completely neutral, or even unpleasant.

Referring to architecture-related feelings, or sentiments, their causes call for adequate attention. The potential of responding by feeling and perception may be stimulated by any of the following (and, within each, of any sort of):

The interior effect – having entered into an interior separated from its surroundings, any human feels that such a particular space is, in a sense, part of a personality – his/her own, or someone else's. The illusion is based on the fact that the surrounding environment meets the functionality conditions and, what is more, has been chosen by the perceiving individual and subsequently adjusted to his needs, tastes or inclinations, concepts or perceptions; in particular does it reflect, basically, the man's self-image. This effect is referred to as identification of the environment with the personality. Bridge structures are equipped with a dual interior: upon the bridge's surface and beneath it. Both interior spaces are open, in contrast to other types of construction. The space 'on' the bridge is an antidote of closed spaces; the world is seen from high above and appears completely open.

Contrasts – the contrast between a structure and its surroundings, or between a structure and its elements, sharpens one's perceptiveness, enchains attention, enlarges the scope of stimuli, thus stimulating the response capability, in terms of both sentiments and perception. New technologies, materials, structures of unheard-of scale, and extravagant solutions all trigger the contrast effect. Bridge structures arouse the reaction of contrast perception because of, for instance, the smooth-profiled lines of roads or tracks visible within them. At times, a contrast may be perceived because of an error related to the useful purpose or specific construction conditions. The fact stands out, moreover, that contrast effect tends to fade away if perceived frequently.

Expression – the belief has prevailed since the ancient Greek times that essential about the beauty in architecture is its geometric regularity, which impacts perceptiveness through use of repeated stimuli and impulses. Perceptions seem clearer and the perceiving individual feels more perceptive. Regular though banal arrangements tend, however, to arouse negative feelings – boredom coming to the fore. Hence, regularity, without additional characteristics, cannot fundamentally define the beauty of a construction: any geometrical, dynamic, and functional features of functionality render the impression more powerful.

1.4.3. Rules of aesthetic shaping

Taking into account all the principles of aesthetic design of a bridge is a must, both at the initial phase of design, when the form and general proportions of the structure are being formed (taking shape), and at the final stage, when decisions are made as to details.

No ready-to-use templates or patterns have ever been made available that would clearly describe the aesthetic shaping approach in detail. There are, instead, the general rules whose observance normally has a beneficial effect on how the designed structure is (to be) perceived. Knowledge of these rules considerably facilitates the elaboration of correct solutions. Furthermore, they serve as an instrument with which to verify the architectural regularity at each step of the design process.

Aesthetic designing: criteria and foundations

Based on a review of the existing bridge construction practice, a set of observations can be discerned – by repeated interdependencies between the elements – and certain conclusions based thereon drawn. To bring an order into architectural forms, prevent elements of negligence in the related studies, and in view of bettering the collaboration between designers, 'principles of elaboration of architectural design of bridges' have been worked out. According to [24], these include:

The **form gradation principle** stands for the need to classify by scale and visibility of elements. Gradation ought, namely, to be observed in a manner so as to prevent the attention getting distracted by certain ‘parent’ or ‘child’ elements. Architectural forms of various grades should be supplied with elements as appropriate with them, avoiding the visibility of forms and elements of other grades. To this end, the following form groups have been developed:

- bridge-and-barrier entirety effect: it is characterised by the largest scale and is based on a clarity principle: the structure appears together with the barrier, as a cohesive whole;
- the road on the bridge and approach roads: it is essential that the bridge’s scale not exceed the one of the road; otherwise, the bridge becomes less expressive. A road set above the structure arouses a better impression compared to a road occulted by the structural arrangement;
- supports and spans: it is fine if both are perceptible separately, their constituent elements remaining invisible;
- the interior space underneath the structure: this frequently neglected issue is, in fact, quite of essence. The form of this space is chiefly founded on the dimensions: height, width, and length. Interiors whose height is larger than the width tend to compare favourably with others. The way the bottom sections of spans look is often left unelaborated and limited to the constructional solutions applied. One example of correct solution in this respect is the orthotropic slab (deck) in steel bridges. It is different with girder bridges, where small curvatures of the bottom-deck surface, variable thickness of beams, slabs or midriffs ought to stand out. This allows to avoid the effect of emptiness, as otherwise caused by flat surfaces. Any curvature introduce differences in light refraction;
- bridge details.

This classification ensues from the observation of forms emerging or created resulting from dependencies between the elements involved.

Error rectification in developing a form – otherwise, the ‘form excellence’ rule, meant to make the form free of whatever might be considered irrational.

Elaboration of forms across the bridge elements – leaving aside any of the elements of the elaborated design produces a worse solution than conscientious elaboration across the steps.

Taking advantage of means of expression: architectural forms are not reducible to geometric forms or a play of lights and shadows. Attention should also be paid to the characteristics the observer is sensitive to. It is recommendable that all the means available are made use advantage of; these include: the shapes of the structure(s); the forces and weights; lights, shadows, and colours; the shapes and the development of the barrier and the surroundings; the road-line on the bridge’s surface, approach roads, and underneath the bridge. The features of evenness, shapes and forces, symmetries and eurhythmics, the proportions of individual elements and of the entire structure, the span (range), the height and width of the spans, the distribution of the spans, and the equipment of the bridge should all be taken into account in this respect. All this contributes to what is referred to as 'regularity of composition'.

Elaboration of such regularity includes correction or rectification of contaminated visibility. Contaminations of line consist in perceiving a shape as different than in reality. The same is true for shapes of forms or the visibility of solids, depending on the colour of their surface, background, or other elements adjacent or placed next to them. The most frequent type of visual contamination in bridge engineering is the impression that the span's bottom edge is bent downwards (as in Fig. 1.7).

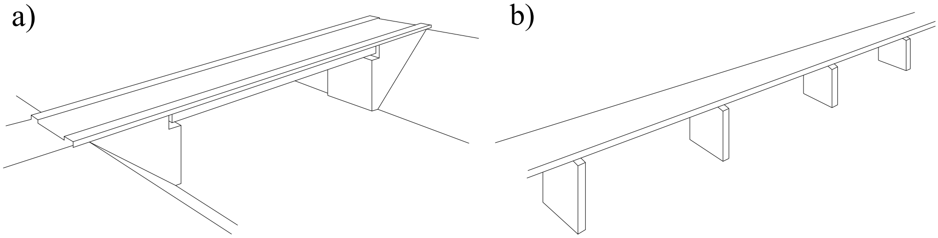

Fig. 1.7. 'Span bent downward' or the Zollner effect as perceptible: a) between the edges of the span and the bridgeheads; b) between the edges of the span and the pillars [24]

Fig. 1.8 shows the appropriate method of removing the span's bent edge effect: the straight lines in the external walls within the bridge's elevation are to be retained, with only the bottom surface of the spans being bent.

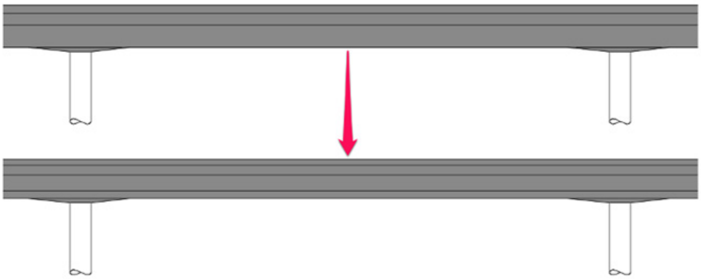

Fig. 1.8. Curved span edge effect rectified [24]

Convexity of the flat walls is a common illusion in visibility of the surface: the most outstanding spots in this respect are the poles and, to a lesser extent, side surfaces of beams and arches. The phenomenon can be counteracted by adding convex depressions, or by introducing an outline of adjacent constructional elements.

The density of creases sufficient for rectification of the illusion of convexity of the surface ought to be of the order of 1/10 to 1/50 of the wall's width, the wider walls always to receive shallower creases.

Primacy of communication (transport-related) forms enables to design with any intensity only the line of the road and the barriers underneath the bridge. It is an erroneous conviction that the construction of bridge is beautiful in itself. Frequently, the construction's form is overly intense and, consequently, interferes with the clarity, or 'legibility', of the whole thing. One arrives at such conclusions through juxtaposing the early and the modern solutions (Fig. 1.9).

Fig. 1.9. Earlier and modern solutions compared[2]

Formerly, bridge structures tended to be rather massive, composed of a large number of materials. Contemporary bridges (Fig. 1.10) are made of modern higher-resistant materials [6], with use of new technologies enabling to make an efficient use of cross-section [19].

Fig. 1.10. Modern bridge designs[3]

Experimental rules in bridge architecture

Experimental rules are findings whose perception influences aesthetic experience. They are formulated based on observations carried out in various condi-

[2] http://www.buzzle.com/articles/pros-and-cons-of-arch-bridges.html
[3] http://www.buzzle.com/articles/famous-bridges-of-the-world.html

tions, by different observers, with respect to a broad scope of objects or things. Being a variety of architectural principles, the rules in question ensue from the general traits of observations and experiences. Experimental rules are uncomplicated and widespread.

Experimental rules may function as guidelines in architectural composition and facili-tate the verification of conceptual solutions – these being the main two objectives of experimental rules of aesthetics. Yet, they ought not to be treated as a must-do, since the experience is the final check. The rules should be considered in a fourfold sense: object-related (objective), psychological (subjective), cognitive, and creative. The first concerns analysis of the forms of material objects; the second, reception of aesthetic experience; the third explains cognitive actions, whilst the fourth makes use of the rules cognised.

Rules based on observation and association of forms

The **entirety principle** finds that "aesthetic experience is determined by noticeability of all the elements of the form and their interdependencies" [4]. What it means is that associating a structure's geometric features with their physical and functional meaning needs being sought. The principle of entirety has many degrees to it, and extends to analysis of the structure together with its surrounding environment, the structure itself, as well as its individual elements. For this very reason, Marzyński [17] discerns the following types of aesthetics:

- large – town-planning and landscape-design;
- medium – architectural;
- small – finishing and details.

Fig. 1.11. Bridges harmonized with the surrounding environment[4]

Large aesthetics seeks how to place a bridge, together with approach roads, and align it with the area. Analysis of large aesthetics leads one to the issues of medium aesthetics. Moving further on, and considering the factors informing the visual perception of bridges, one comes across the small aesthetics, such as se-

[4] http://www.flickriver.com/photos/bridgink/popular-interesting/

lection/matching of materials, or finish, which influence neither the shape or form, nor the structure.

Seeking to observe an order of the constituent elements, which is fundamental to aesthetic reception, cohesion of architectural form must not be neglected. Equilibrium between rational elements and architectural expression needs to be kept; otherwise, wherever any constituent is missing, the overall construction is affected (Fig. 1.11).

The **simplicity of form principle** provides that the number of individual elements of a bridge ought to be small enough, in order that a non-complicated form be preserved. This is directly interrelated with man's capability of perceiving a small number of elements whilst ensuring a simple form. Wherever the form is overcomplicated, or overly complex, one is not capable of seeing it as a whole and gets bogged down in the interrelations, with no aesthetic experience coming out as a result.

As is the case with the entirety principle, similarity of form consists of multiple grades: rather than being limited to the whole structure, including its environment, it extends to individual elements.

The simplicity principle should not be approached in terms of restricted architectural expression or avoidance of essential dependencies but rather as a postulate to respect moderation, or restraint. Bridge structures should display the road's line and the barrier's line in the first place, the other elements of the bridge route being not as outstanding (Fig. 1.12).

Fig. 1.12. An exemplary simple form of bridge structure[5]

The **clarity of form principle** postulates that "in order for the form to arouse aesthetic impression, the associations between its elements ought to be easily perceptible" [24]. The clarity criterion complements the two previously discussed principles. Aesthetic impression is achieved through expression of forms and ease with which they are associated with the cognitive importance of aesthetics. In bridge architecture, the means bringing about this effect is the use of

[5] https://www.gti-usa.net/Saint-Anthony-Falls-Replacement-Bridge.shtml

elements other than constructional – additional or, at times, outright antithetical. Bridge structures tend to show off arrangements that are indicative of the system of internal forces, communication lines, close interdependence between the type of construction and the materials and technologies applied as well as the construction conditions.

The former half of the twentieth century saw a rejection of traditional architectural forms, and focusing, instead, on clarity of lines of communication and force arrangement. This is not to say, though, that arrangements, or systems, of forces are 'legible' in themselves – one example being truss bridges, common to industrial areas (but not only; Fig. 1.13).

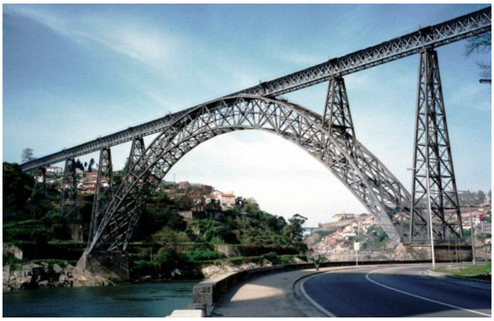


Fig. 1.13. Exemplary truss structures[6]

The forces in the rods are of diverse values and marks, which translates into non-clarity. The multiple grades within the criterion imply that the forms of individual elements ought to indicate whether the item has been bent, squeezed, etc., and how the forces are transferred to the other links. Architectural forms also have to be legible in view of the purpose of the bridge, features of the landscape, natural and economic conditions.

The **avoidance of emptiness principle**. In order for a bridge to be of interest and attract attention, it has to have certain characteristics.


Fig. 1.14. Exemplary methods of preventing the impression of emptiness [14]

[6] http://www.flickriver.com/photos/bridgink/popular-interesting/

Without them in place, a bridge structures can prove outright repellent. Emptiness triggers feelings analogical to boredom, weakening the ability to act due to no emotional impulses present. Lack of (inter)dependencies between the structure's elements, incompetence in comprehension, or lack of clarity imply emptiness (Fig. 1.14).

As a criterion, emptiness prevention is a variety of the criteria of genuineness (denial of cognitive endeavours; emptiness dissembling the truth) and form clarity (an empty form is illegible). Any form that has nothing to say or appears incomprehensible should be rejected.

Principles ensuing from cognitive (inter)dependencies between elements of forms

The **genuineness of form principle** implies the resolute expectation that a form arouse genuine associations, in line with the object's (structure's) purpose, operation conditions, functionality and utility. 'Genuineness' is a relative notion as far as aesthetics of bridge construction is concerned, and is dependent upon cognition. Dependent on the cognitive conditions is also the criterion's permanency, as a given form may turn out to be non-genuine under altered conditions.

The principle in question should remain superior. It extends to the conclusions drawn based on the useful purpose and the conditions of making and operation (actual use) of the structure. The relevant conclusions include as follows:

- the bridge must be aligned with its useful purpose: this works for the traffic on and underneath the bridge and the actual development of the barrier;
- the bridge must be adapted to the natural and physical conditions: this is true for hydrological and climatic conditions as well as the choice of load-bearing structure dependent on the balance of forces;
- the bridge project has to meet the economic conditions; and,
- the bridge project has to be adapted to the social conditions.

Alignment with useful purpose: If aligned with the actual useful purpose, the form of a bridge positively influences aesthetic feelings. The most outstanding elements in the entire construction, and the most important factors, include the shape of the road set along the bridge and the approach roads. The use of straight lines, arches, transition curves, grade-line inclinations, and cross-falls, the shape is adapted to the specific area features and the forecast traffic. Each of these elements is visually perceptible and, if comprehensible, fosters the aesthetic impression. When designing an object or structure in line with its useful purpose, the aforementioned relevant criteria should be borne in mind; in specific:

- the entirety principle: the road to be visible along the whole section where its shape is connected with the bridge;
- the simplicity principle: any unwelcome complication should be avoided with regards to the road. It is important that the solutions applied not be contrary to the natural topography (such as e.g. reverse inclinations/radii);

- the legibility (clarity) principle: the road to be visible all along the bridge line. There is more to this particular rule, though: first, the road should be visible from, potentially, every single point, outside the bridge space and within it. The road's section within the bridge, including the approach roads, should make up a concavity; to enable this, very small slopes, below 1%, definitely suffice. Hard to notice at times, such inclinations do contribute to the aesthetic values through improved visibility.

Structure shaped according to physical conditions: Civil structures normally tend to be subject to certain natural dependencies such as, primarily, geological, vegetal, climatic, and physical conditions. The latter two, in particular, inform the structure's architectural shaping, the other ones influencing the type and quality, or colour and texture, of materials selected/used. The rule whereby the structure's arrangement is made compliant with the physical conditions implies the adaptation to the system (balance) of forces and climatic conditions (Fig. 1.15).

Fig. 1.15. The structure arranged according to the physical conditions[7]

The **optimum form principle** translates, in practice, to actions aimed at the possibly best way to satisfy the needs within the given conditions, this being altogether referred to as optimality. As regards bridge structures, optimality seeks to adapt their expression to communication/traffic, construction, and spatial development conditions. The favourable factors include an optimum way of setting the road up to and all the way through (along) the bridge, as well as matching the bridge's siting, span and height to the system of supports, relative to the barrier. A form of bridge structure that contradicts the development of the surrounding area adversely impacts the aesthetic experience.

[7] http://www.flickriver.com/photos/bridgink/popular-interesting/

The design process ought to endeavour to respect the order, simplicity, selection/matching of appropriate internal proportions, and harmony with the surrounding environment. With these basic principles taken into account, the outcome can be really positive, whilst neglecting them may lead to a dissonant experience. The designer is obligated to act in a conscious and responsible manner, always bearing in mind the rules of aesthetic architectural shaping of bridge structures.

Hence, when it comes to shaping a bridge, slenderness of the entire structure and the supports should be sought, as should lightness combined with (the sense of) stability. Simplicity and variety of forms reduced to a minimum ensues directly from the principle of simplicity ('less' sometimes means 'more'). Massive and heavy-looking bridge structures ought to be avoided, as a rule. It should instead be endeavoured that the object assume its original and unique form, and bear a peculiar character – something that would make it nicely remembered; a view that would render the journey more pleasant and, above all, more interesting.

Among the thousands of structures constructed or under construction these days, it verges on the impossible to give every one of them a unique or original form; thus, repeating decent designs is essentially unavoidable. Otherwise, of high importance is the skill of fine-tuning the details, displaying the elements that improve the overall look, and masking those details which do not quite add to the aesthetic outcome of the solution.

1.5. Animal transitions

Animal crossings are currently being modernized and built on expressways or highways. The use of objects of this category in the construction and modernization of roads lower classes should be considered in appropriate cases. Crossings for animals and their significance are discussed in [8], [9] among others.

Animal crossings have two basic ecological functions, i.e. creating conditions that allow the existence of species whose habitat includes the area of a given road and allowing migration and dispersion of individual animals [11].

Animal crossings in the form of bridges can be divided into independent (only one ecological function) and combined (ecological and economic functions) ones. With regard to the size (introducing the possibility of use by a particular species), transitions are divided into large, medium, small and amphibian ones. With regard to the surface for use by animals, transitions can be divided into lower (animals use the transition moving under the object, between its supports) and upper (animals benefit directly from the bridge object moving around the premises).

Tab. 1.10 and Tab. 1.11 shows the types of transitions and its dimensional requirements.

Tab. 1.10. Types of animal crossings – the ecological function

	B_{min}.(recommended) [m]	H_s [m]	WWC [m]
	Large landscapes transitions		
upper – landscape bridge	≥ 50 (≥ 60)		
bottom – flyover	Span length > 15	≥ 5 (≥ 10)	
	Upper transitions		
large animals	≥ 35 (≥ 50)		
medium animals	≥ 30 (≥ 40)		
	Bottom transitions		
large animals	≥ 15	≥ 3.5 (≥ 5.0)	≥ 1.5
medium animals	≥ 6.0 (≥ 10)	≥ 2.5 (≥ 3.5)	≥ 0.7
small mammals	≥ 1.5 (≥ 2.5)	≥ 1.0 (≥ 1.5)	≥ 0.07
amphibians			
to 20 m	≥ 1.0 m	≥ 0.75 m	
to 30 m	≥ 1.5 m	≥ 1.0 m	
to 50 m	≥ 2.0 m	≥ 1.5 m	
to 80 m	≥ 3.5 m	≥ 1.5 m	

B_{min} – minimal width, H_s – height between the ground level and the bottom of span, WWC – the relative tightness factor (the product of the height and width of a passage divided by the length).

Tab. 1.11. Combined transitions – for combine ecological and economic functions

	B_{min} (recommended) [m]	H_s [m]	WWC [m]	SPZ [m]
	Upper animal transition combined with road			
large animals	≥ 35 (≥ 50)			≥ 2 × 15
medium animals	≥ 30 (≥ 40)			≥ 2 × 12
	Bottom animal transition combined with road			
large animals	2 x 5	≥ 3.5 (≥ 5.0)	≥ 1.5	
medium animals	2 x 3	≥ 2.5 (≥ 3.5)	≥ 0.7	
	Bottom animal transition combined with river or less watercourse			
large animals	2 x B_{kor}	≥ 5.0		
medium animals	2 x B_{kor}	≥ 3.5		
	Combined with railway			
medium, small	≥ 2 x 3	≥ 3.5		
	Bottom small animal transition combined with watercourse, height < 2.5 m			
	≥ 2 B_{kor} ≥ 1.0	≥ 1.5		

SPZ – zone intended for animals, B_{kor} – the width of the river or other watercourse.

Another type of animal crossings, non-structural one, includes passages in the crowns of trees (the main aim: to preserve the continuity of ecological corridors for climbing and arboreal mammals), crossings over the roads for bats and transitions on the level of the road.

1.6. Culverts

1.6.1. Introduction

A culvert is an engineering structure constructed under an embankment for the purposes of carrying water, communication cables or others. A culvert can also serve as a transition for small animals and amphibians under an embankment.

Culverts are made in depressions. The axis of a culvert should be perpendicular to the road, and as far as possible, on the axis of the watercourse, ensuring a good flow of water. If there is a sharp angle between the watercourse axis and the road, the culvert is designed irrelevantly to the road and according to the natural watercourse or perpendicular to the road with rebuilding of watercourse bed. From the point of view of the flow conditions, more favourable is the first solution, however, it is connected with higher culvert construction costs.

The length of a culvert depends on:

- the width of the crown of the embankment,
- the height of the embankment,
- the embankment slope inclination,
- the angle between the culvert axis and the axis of the road.

The length of a culvert is determined by the formula:

$$L_{\mathrm{p}} = \left[B + 2n\left(h_{\mathrm{n}} - h_{\mathrm{p}}\right)\right]\sin\beta, \tag{1.1}$$

where:

L_{p} – culvert length [m],
B – width of the road crown on the embankment [m],
n – embankment slope inclination,
h_{n} – height of the embankment measured from the road crown to the bottom of the culvert [m],
h_{p} – culvert height [m],
$\sin\beta$ – acute angle between the axis of the road and culvert.

A typical culvert is usually composed of a body divided into sections (inlet, outlet, the inner part) and a foundation. The foundation of a culvert is always made in-situ. If a culvert serves as an animal passage combined with a constant or periodic watercourse, it should include a communication shelf for animals inside.

Calculations of the water guiding culverts are carried out according to the rules described below. Calculations of the vertical/horizontal light of a passage for small animals or amphibians is performed on the basis of the required relative tightness factor. It is the product of the ratio of the vertical and horizontal width to the length of a culvert. In the case of a passage for amphibians, the following minimum dimensions are recommended [12]:

- width ≥ 1.0 m, height ≥ 0.75 m – with lengths up to 20 m,

- width $\geq$ 1.5 m, height $\geq$ 1.0 m – with lengths up to 30 m,
- width $\geq$ 2.0 m, height $\geq$ 1.5 m – with lengths up to 50 m,
- width $\geq$ 3.5 m, height $\geq$ 1.5 m – with lengths up to 80 m.

The minimum dimensions of a transition for small animals that can be made in the form of a culvert are: horizontal width $\geq$ 1.5 m (recommended $\geq$ 2.5 m), vertical width $\geq$ 1.0 m (recommended $\geq$ 1.5 m), and the relative tightness factor $\geq$ 0.07.

Minimum dimensions always refer to the inner width of a passage, regardless of the construction type, shape and material from which the object is built.

The bottom longitudinal inclination of a culvert is usually matches the inclination of the watercourse bed. The bottom of a culvert should be designed and constructed with such an inclination which will ensure a rapid flow of water without damage to the culvert. Typically, the designed decrease is about 2%. For longer culverts the bottom inclination is determined on the basis of the following formula [26]:

$$i = \frac{v_0^2}{(C^2 R_\mathrm{h})}, \tag{1.2}$$

where:

i – culvert bottom inclination,
v_0 – water speed in the culvert [m/s],
C – resistance coefficient of the channel on the basis of Bazin and Manning's formula [$m^{0.5}$/s],
R_h – hydraulic radius of the culvert [m].

Manning's formula:

$$C = \frac{1}{n} R_\mathrm{h}^{(1/6)}. \tag{1.3}$$

Bazin's formula:

$$C = \frac{\left(87 R_\mathrm{h}^{0.5}\right)}{\left(g + R_\mathrm{h}^{0.5}\right)}, \tag{1.4}$$

where:

γ – surface roughness coefficient [$m^{0.5}$],
n – surface roughness coefficient [$s/m^{1/3}$].

In construction of a culvert in the area of significant inclination in the direction perpendicular to the axis of the road, the bottom of the passage may be formed as a cascade. In such a situation, an inlet to the culvert may be decreased by means of a trough or a trough threshold.

Upstream and downstream culvert watercourse declines should not be much different. The watercourse water level change from a higher to a lower can cause fouling on culvert inlet or outlet.

Depending on the culvert bottom longitudinal inclination, a passage must be protected from blurring resulting from an increased water flow rate.

If the bottom inclination is too big and the flow rate exceeds the limit value, steps, cascades or other devices to reduce the kinetic energy of water must be introduced.

Dimensioning of the hydraulic parameters of bridges

For a given catchment area, the flow is calculated as follows:

$$Q_1 = \mathrm{F}\chi, \tag{1.5}$$

where:

Q_1 – design flow [m^3/s],
F – the catchment area [km^2],
χ – catchment area characterization ratio [m^3/s].

If the cross-section of a watercourse and the water velocity in the cross-section is known, the water flow is calculated from the following equation:

$$Q_2 = fv, \tag{1.6}$$

where:

Q_2 – the flow from hydrometric measurements [m^3/s],
f – cross-sectional area of a watercourse [m^2],
v – water velocity [m/s].

The flow rate can be calculated by a method based on the empirical formula by Cheza:

$$v = CR_{\mathrm{h}}^{0.5}i, \tag{1.7}$$

where:

v – watercourse water velocity [m/s],
i – watercourse bed inclination (200 m before and after the culvert),
C – surface roughness coefficient [$m^{0.5}/s$],
R_{h} – hydraulic radius of a watercourse [m].

In order to calculate a meaningful level of the high water level, an approximate level is assumed and the flow Q_2 is calculated on this basis. Then it is compared to Q_1 and accepted as valid, if it does not differ by more than 5%. Then the depth of the water in the trough is determined.

The stream of water in an open trough may have a different total energy depending on the depth of the trough. The amount of the energy of the water in the trough is determined by the formula:

$$E_c = h + \frac{\alpha v^2}{2g}, \tag{1.8}$$

where:

E_c – the amount of the stream energy calculated from the culvert bottom level [m],
h – watercourse depth in an undeveloped section [m],
α – Coriolis' coefficient [-],
v – the average velocity of the water in the undeveloped stream [m/s],
g – acceleration due to gravity [m/s^2].

The stream movement of the constant flow rate Q at which the total energy reaches the minimum value, is called critical:

- supercritical, peaceful $h > h_{kr}$; $v < v_{kr}$;
- subcritical, turbulent $h < h_{kr}$; $v > v_{kr}$.

The opening of a bridge is calculated by assuming a water flow with minimum energy (potential and kinetic).

The width of the space under the bridge structure should be such that the accumulation of water cause no damage to the surrounding area and that an increased water velocity does not cause blurring of slopes and bottom of the trough.

The hydraulic calculation of a culvert or bridge consists in determining the minimum width of an object corresponding to v_{max}.

Due to the nature of the non-pressurized flow within the bridge structure, there are two cases:

- flow without accumulation,
- flow with accumulation.

In practice, it is assumed at first that the accumulation does not occur. The condition is met if the designated depth is greater than the theoretical critical depth, which is equal to 2/3 of the amount of the water stream energy.

Sizing of the width of a rectangular culvert and a small bridge

Peaceful movement. The first step is to check the flow assuming a peaceful movement. Assuming the maximum of the water velocity:

$$Q_1 = v_{max} \mu \left(E - \frac{\alpha v_{max}^{\ 2}}{2g} \right) l_1, \tag{1.9}$$

where:

Q – design flow [m^3/s],
v_{max} – maximum water velocity in a culvert [m/s],
μ – coefficient of contraction [-],
α – Coriolis' coefficient (adopted 1.2) [-],
g – the acceleration of gravity [m/s^2],

l_1 – minimal width of a culvert or small bridge [m].

Then it is possible to calculate the necessary width of an object:

$$l_1 = \frac{Q}{v_{\max}\mu\left(E - \frac{\alpha v_{\max}^{\ 2}}{2g}\right)}, \tag{1.10}$$

where:

E – the amount of the water energy in an undeveloped trough [m].

The resulting value shall be rounded to tenths of a metre and designated by l_0:

$$h_0 = \frac{Q}{v_{\max}\mu l_0}, \tag{1.11}$$

where:

h_0 – water depth within the bridge structure [m],

l_0 – minimal width of the bridge structure [m].

The theoretical value of the critical depth is used in the formula:

$$h_{kr} = \frac{2}{3}\left(h + \frac{\alpha v^2}{2g}\right), \tag{1.12}$$

where:

h_{kr} – critical depth [m^3/s],

h – watercourse depth in an undeveloped section [m],

α – Coriolis' coefficient [-],

v – the average velocity of the water in an undeveloped watercourse [m/s],

g – the acceleration of gravity [m/s^2].

If the initial assumption is correct, the condition $h_0 > h_{kr}$ must be fulfilled. Then it is possible to calculate the actual speed of water within the bridge structure:

$$v_0 = \frac{Q}{\left(h_0 l_0\right)}, \tag{1.13}$$

where:

v_0 – the actual speed of water in an undeveloped watercourse [m/s],

Q – design flow [m^3/s],

l_0 – adopted width of the bridge structure [m],

h_0 – water depth within the bridge structure [m].

Then the height of accumulated water could be determined as follow:

$$H = h_0 + \frac{\alpha v_0^{\ 2}}{2g} - \frac{v^2}{2g}, \tag{1.14}$$

where:
H – height of accumulated water [m],
v – the average velocity of water in an undeveloped watercourse [m/s],
v_0 – the actual speed of the water in a culvert [m/s],
g – the acceleration of gravity [m/s^2].
The minimum overall height of a culvert should be greater than $4/3h_0$.

Turbulent movement. When $h_0 < h_{kr}$ there is the case of a false initial assumption. Damming is followed by water accumulation. The water table rises until the stored energy is sufficient to cause a uniform water flow. The formula for the critical water flow is:

$$Q = \mu \alpha l_1 h_{kr} v_{kr}, \tag{1.15}$$

where:
Q – design flow [m^3/s],
v_{kr} – critical water velocity [m/s],
h_{kr} – critical depth [m/s],
μ – coefficient of contraction [-],
α – Coriolis' coefficient (adopted 1.2) [-],
l_1 – minimal width of a culvert [m].

The dimensions of the inner cross-section of a culvert should be adjusted in such a way that the flood created by accumulated high water does not result in any damage to the adjacent area.

The critical depth can be expressed by the critical water velocity:

$$h_{kr} = \frac{v_{kr}^{\ 2}}{g}. \tag{1.16}$$

Substituting eqs. (1.16) to eqs. (1.15) and transforming with the assumption that $v_{kr} = v_{max}$, we obtain the formula for the minimal width of an object:

$$l_1 = \frac{gQ}{\alpha v_{max}^{\ 3} \mu}. \tag{1.17}$$

The resulting value shall be rounded to tenths of a metre and designated as l_0 – the adopted width of a culvert [m]. The critical velocity is determined by means of the following formula:

$$\mathrm{v}_{kr} = \left(\frac{gQ}{\alpha \mu l_0} \right)^{1/3}. \tag{1.18}$$

We calculate the final depth of water within an object as follows:

$$\mathrm{h}_0 = h_{kr}. \tag{1.19}$$

Then the height of accumulated water is determined:

$$H = h_{\mathrm{kr}} + \frac{\alpha v_{\mathrm{kr}}^{2}}{2g} - \frac{v^{2}}{2g}. \tag{1.20}$$

The minimum overall height of a culvert should be greater than $4/3h_0$ and greater than the given depth of accumulated water at the inlet H.

Dimensioning of a circular culvert – laminar peaceful movement

The value of the laminar flow at the critical depth h_{kr} less than the depth of the watercourse in front of object a is expressed by the following formula:

$$Q = \mu f_1 v, \tag{1.21}$$

where:

Q – design flow [m^3/s],
μ – coefficient of contraction [-],
v – water velocity in the culvert [m/s],
f_1 – active cross-sectional area of the circular culvert [m^2].

When designing a culvert with a circular cross-section, the internal cross-sectional diameter must be determined. The water level in an unsunk culvert of a circular cross-section shall not exceed 80% of the culvert. According to this rule, and assuming the maximum allowable flow rate, the cross-sectional area can be determined by means of the formula:

$$f_1 = \frac{Q}{\mu v_{\max}}. \tag{1.22}$$

Determine the minimum diameter circular culvert D_1. At the height of water in the culvert $h_0 = 0.8 \cdot D_1$, the active cross-sectional area of the circular culvert is $f_1 = 0.6736 \cdot D_1^2$. After the transformation the formula is replaced by:

$$D_1 = \sqrt{\frac{Q}{0.6736 v_{\max} \mu}}. \tag{1.23}$$

Adopt the minimum diameter circular culvert $D_0 > D_1$. We assume that the depth of water in the culvert h_0 is not less than the depth of the watercourse before the culvert a. We set the depth of the water in the culvert [m] as follows:

$$h_0 = \left(\frac{Q}{3\mu\sqrt{D_0}} \right)^{1/2}. \tag{1.24}$$

After determining h_0 check whether $h_0 \geq a$ and $0.2D_0 \leq h_0 \leq 0.8D_0$. Then calculate the velocity of water in the culvert:

$$v_{\mathrm{h}} = \frac{Q}{\mu f_{\mathrm{h}}}, \tag{1.25}$$

where:

v_{h} – flow rate when the culvert is filled to the height h_0 [m/s],

f_{h} – flow area in the culvert with the depth h_0 [m^2],

$$f_{\mathrm{h}} = 0.393 D_0 + c D_0 \left[1 - 0.7 \left(\frac{c}{D_0} \right)^2 \right], \tag{1.26}$$

where:

c – the distance between the water level in the pipe and the plane parallel thereto and passing through the axis of the pipe [m], $c = 0.3D$.

Then we calculate the wetted culvert perimeter:

$$U = 1.57 D_0 + 2c \left[1 + 0.8 \left(\frac{c}{D_0} \right)^2 \right], \tag{1.27}$$

and the hydraulic radius:

$$R_{\mathrm{h}} = \frac{f_{\mathrm{h}}}{U}. \tag{1.28}$$

In the end determine the hydraulic gradient in the culvert:

$$i = \frac{v_w^{\ 2}}{C^2 R_{\mathrm{h}}}, \tag{1.29}$$

where:

v_{w} – velocity of water in a ditch determined on the basis:

$$v_{\mathrm{w}} = \frac{Q}{f_{\mathrm{w}}}, \tag{1.30}$$

where:

f_{w} – sectional area of the trench to a depth coefficient,

C – coefficient of watercourse bed resistance eqn.(1.2).

Dimensioning of a circular culvert – turbulent movement

The value of the supercritical flow at the critical depth h_{kr}, greater than the depth of the watercourse in front of the culvert a is determined by the equation:

$$Q = \mu f_{\mathrm{kr}} v_{\mathrm{kr}}, \tag{1.31}$$

where:

f_{kr} – the active area of the circular cross-section of the culvert at the critical depth [m^2],

v_{kr} – critical velocity expressed by the formula:

$$v_{kr} = \left(\frac{f_{kr} g}{s_{kr}} \right)^2, \tag{1.32}$$

where:

g – the acceleration of gravity [m/s^2],

s_{kr} – the width of the stream in a culvert on the surface of water at the critical depth [m].

For the accumulation of water in front of the culvert the following formula is used:

$$H_{sp} = h_{kr} + \frac{\alpha v_{kr}^2}{2g} - \frac{v_0^2}{2g}, \tag{1.33}$$

where:

g – the acceleration of gravity [m/s^2],

α – Coriolis' coefficient; adopted $\alpha = 1.2$,

v_0 – the average velocity of water in an undeveloped watercourse.

Then we check if the accumulated water height is less than 80% of the culvert. In the following steps we shall calculate the wetted culvert perimeter:

$$U' = 1.55 D_0 + 2c \left[1 + 0.8 \left(\frac{c}{D_0} \right)^2 \right], \tag{1.34}$$

and the hydraulic radius:

$$R_h = \frac{f_h}{U'}. \tag{1.35}$$

We calculate the required longitudinal inclination of the culvert bottom:

$$i_p \geq \left(\frac{v_{kr}}{\frac{1}{n} R_h^{2/3}} \right)^2. \tag{1.36}$$

At the end we calculate the velocity of water just behind the outlet of the culvert:

$$v_w = \sqrt{2g\left(h_{kr} - a\right) + v_{kr}}. \tag{1.37}$$

Culverts with animal transitions are calculated analogously to culverts carrying only water (temporarily or permanently) increasing, however, the required cross-section of a culvert, making shelves for animals. The shelves may be made in the form of steel plates covered with soil, suspended above the floor in such a way that the water flowing through the passageway is carried below the upper surface thereof. In the case of shelf gabions filled with aggregate covered with soil, the cross section of a calculated passage comprises a space between the gabions. This causes a corresponding increase in the whole passage section.

1.6.2. Types of culverts due to the water flow

With regard to the height of an opening passage for high water there are two types of culverts:

- unsunk, wherein the upper edge of an opening is above this level, the water fills the cross-section and partially flows freely (Fig. 1.16a),
- sunk, wherein the inlet opening is situated completely below high water (Fig. 1.16b).

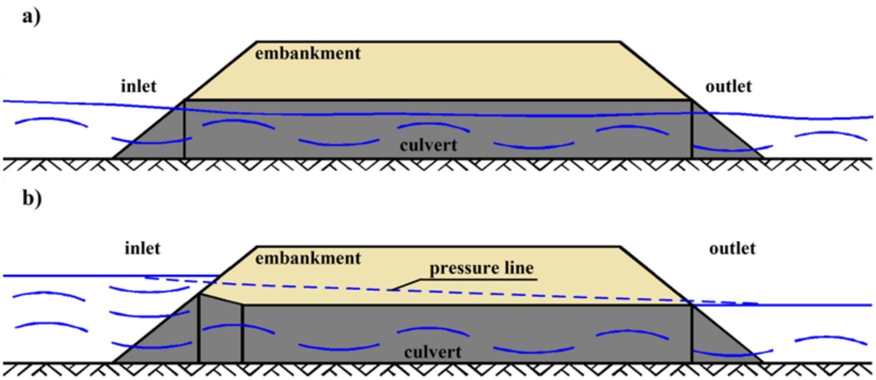

Fig. 1.16. Types of culverts: a) with unsunk inlet, b) with sunk inlet

According to [28], in the case of lowland culverts on watercourses with the bed inclination < 0.02, it is recommended to apply the following basic and most common hydraulic diagrams:

- culvert with an unsunk inlet and outlet satisfying the conditions: the inlet is unsunk $H \leq 1.2 \cdot h_p$ and the outlet is unsunk $h_p \leq 1.25 \cdot h_{kr}$,
- culvert with a sunk inlet and unsunk outlet, leading water by a part of the cross-section (with a free water level in the pipe), in which: the inlet is sunk $H > 1.2 \cdot h_p$ and the outlet is unsunk $h_p \leq 1.25 \cdot h_{kr}$,
- culvert with a sunk inlet and outlet, in which water flows by culverts full cross-section, which requires simultaneous use of a streamlined inlet, and depth before the culvert $H > 1.4 \cdot h_p$, inclination $i_p < i_t$, and depth at the sunk outlet $h_d < 1.1 \cdot h_p$,

- culvert with a sunk inlet and outlet conducting water by culverts full cross-section, in which the sunk of the inlet $H > 1.2 \cdot h_p$ and the outlet is sunk $h_d \geq 1.1 \cdot h_p$.

H is the height of water at the inlet, h_p is the height of the culvert, h_{kr} is the critical height of water in the culvert, i_p is the inclination of the bottom of the culvert, i_t is the inclination of the bottom of the watercourse before the culvert, h_d is the height of water behind the outlet of the culvert.

1.6.3. Solid culverts

Rigid culverts include stone, brick, concrete and reinforced concrete ones. Rigid culverts alone carry the burden of covering ground and payloads located on the embankment. Depending on the construction, there are three types of rigid culverts: pipe, plate and frame.

Pipes are used for small volumes of flowing water. They are often circular in the cross-section. They can be made of precast concrete or reinforced concrete, or on-site. Also passages from artificial materials (HDPE) are used more and more often. When a culvert is based on poor soils, a stone or concrete bench along the entire length of the culvert is used as foundation. On good permeable soils, the foundation is based directly on the ground keeping in mind the benches at the inlet and outlet. Concrete surfaces in contact with the ground should be insulated. Generally, a smooth pipes of a cross section of diameter 0.6 m, 0.8 m, 1.0 m, 1.2 m and 1.5 m are usually applied. It is recommended that prefabricated elements be connected to each other by a reinforced concrete slab of the length of the culvert and a thickness of about 12 cm.

A plate culvert is usually made with reinforced concrete slabs. The strength of a plate culvert is calculated as single-supported scheme. If a larger span is needed, then culvert with two spans and a continuous plate supported on three pillars is constructed.

Reinforced concrete frame culverts are made of reinforced concrete prefabricated elements or monolithically. Prefabricated rectangular cross-sections have a wide range of height and width dimensions. Box elements (closed) have internal dimensions [cm] of 100×100, 120×120, 150×150, 200×200, 250×250, 300×300, 250×150 and 300×200. Elements of a bipartite box-section (opened, C-shaped) have internal dimensions [cm] of 300×100, 300×150, 350×100, 350×150, 400×100, 400×150, 450×100, 450×150 and 450×200. Elements are connected by locks or contacts. Precast edge elements are adapted for connection on the construction site by monolithic inlet/outlet, with a minimum length of 0.9 m. Internal elements are usually made of C35/45 concrete. Teaming plates, monolithic inlets and outlets are made mostly of C25/30 concrete [40].

Basically, culvert foundations are directly on the ground. Their design depends on the type and condition of soil. There are three common types of foundations, i.e. a concrete bench (C8/10), soil stabilized with cement and natural or broken, mechanically compacted aggregate. A prefabricated culvert framework cannot be based directly on rock. It is necessary to apply a separating layer of gravel or gravel sand about 30 cm thick. In the case of dusty ground it is necessary to exchange the ground to a depth of 20 cm below the depth of the frost penetration.

Fig. 1.17. Reinforced concrete precast culvert a) assembling of elements, b) finishing works[8]

Culverts must be adequately insulated. If on the aggregating plate a road pavement is planned or the plate surface has a width greater than 1.5 m, it must be covered with insulation. All the mounting contacts must be protected against water. It is therefore recommended to secure mounting contacts by adhered membrane strips of a width of 30 cm or any other material meeting the requirements. Moreover, all the concrete surfaces in direct contact with the ground should be protected with a thin insulation layer.

A culvert should also be properly connected to the embankment. To protect the embankment before settling on of the culvert and the road surface cracking, between the culvert and the embankment there must be backfill done with a compaction ratio of at least $I_s = 1.0$. Depending on the technical grade of the road and traffic over the culvert, one must consider use of transition plates or other solutions to strengthen the embankment at the junction with the duct preventing deformation and cracking of the surface of the duct.

Reinforced concrete structures can be made of precast elements or monolithically. Culverts of other materials are made only at the place of installation. The inlet and outlet of a rigid culvert is usually topped by a head or the head wall. The recess foundation for the head is always higher compared to the through portion, due to the greater depth of the frost line and for the blur prevention at the inlet and outlet.

[8] Photos by M.Kowal

Monolithic reinforced concrete culverts should be dilated at length by the use of joints because of the possibility of concrete shrinkage cracking and subsidence on the ground.

In rigid culverts with diameters/cross-section larger than 60 cm heads with specially selected shapes allowing to water to enter without choking are used. The heads are also used to support the embankment slope. There are the following forms of heads:

- with head walls (to a height of 3 m),
- with wings, perpendicular or diagonal to the axis of the road (preferably up to 30°) – at a high flow and speed of water,
- flange (in the pipe passes at small depths and small flows of water),
- extended,
- flowing with expanded inlet.

1.6.4. Shell tubes integrated with sand filling material

Shell structures are engineering structures built in the form of a shell surrounded by compacted soil. The effect of the cooperation between soil and ground is achieved through the phenomenon of a trumpet arch in the ground, a relief shell. The backfill of a ground-shell construction is an essential carrying component [16].

Ground-shell objects can be of a small, medium or large span. They are attractive architecturally and technologically due to a short period of erection. They can be economically attractive in reference to a rigid structure costs due to lower maintenance needs.

The intensity of the impact of soil on the carrying structure depends on the stiffness of the shell relative to the surrounding backfill. For this reason, ground-shell structures are divided into rigid and flexible ones. Rigid structures include objects made of low tensile strength materials (concrete, stone, brick). These are vaulted objects. Flexible structures are carrying structures (shell) made of corrugated metal sheets, mainly steel, but also aluminium, plastic or thin precast concrete.

Backfill around the shell is a construction material, so it must be properly designed and produced. At the stage of construction, backfill constitutes a significant burden on the shell structure, but in the end, thanks to the phenomenon of a trumpet arch, it increases the load capacity of the coating on the external load.

An efficient cooperation of a shell with the surrounding ground depends on the use of aggregates of a sufficient quality and a proper compaction of backfill around the shell. It is recommended to use non-cohesive grounds, sand, gravel, river gravel-sand mixtures that meet the congestion requirements. The aggregate should have a fraction 0÷32 mm, the varying granularity rate $Cu \geq 4$, the rate of curvature $Cc \leq 1 \leq 3$ and permeability $k10 > 6$ m/day. The material used for the foundation and backfill should not be chemically aggressive or contain organic compounds. Backfill should be placed in layers with a maximum thickness of 30 cm. Backfilling must be performed symmetrically. The height of backfill should be the same on both sides of a steel tube, with a difference equal to the height of one layer. Before laying another backfill layer, one should make sure that the previous one is properly compacted. The aggregate backfill compaction index, according to [36], should be min. 0.98, in the immediate vicinity of the structure up to 0.95 is permitted.

Ground-shell structures can be designed as closed (tubular, elliptical) or open (arch, frame). Closed structures are founded on aggregate foundation benches. Open structures are founded on reinforced concrete (rigid) plates or on aggregate foundation benches (flexible).

Flexible shells. In Poland, as a metal shell structure material mainly used are galvanized steel corrugated spiral pipes, called HelCor® in catalogues (smaller). For larger cross-sections, multi-dimensional structures made of galvanized corrugated steel called MultiPlate and SuperCor in catalogues are used. There are also shell structures of plastic corrugated pipes with double wall. The main advantages of corrugated sheet shells are:

- design simplicity because of a small number of parts,
- an existing database of drawings, profiles and a database of strength calculations of standard applications,
- simple and quick installation,
- usability in freezing temperatures (no seasonality),
- possibility to build objects without traffic disturbances,
- due to the relatively low weight, there is a possibility of assembling a chosen section or a whole structure near the target site, without interference with other works,
- often lower costs of investments compared to traditional solutions,
- the possibility of works phasing.

HelCor® shells are made as steel pipes with diameters of 0.3÷3.6 m, with galvanized corrugated steel, spirally wound, thickness of 1.5÷3.5 mm. Arch-wheel shell HelCor PA has about 65%–100% more flow at the same level as a round pipe of about the same height. HelCor PA shell are made with the following dimensions: width x height [m] 1.05÷1.34×3.67×2.61.

A metal sheet is typically coated with a layer of zinc, and can also be protected against corrosion by a coating of polyamide, depending on the estimated service life of the structure. The dimensions of the wave depend on the diameter of a pipe and amount to 68×13 mm, 125×26 mm.

The HelCor® corrugated shell and tube-shaped and circular-arc cross-section HelCor PA can be used as road and railway culverts, underpasses, hydraulic structures, housing conveyor belts and pipes, as well as for strengthening and reconstruction of existing engineering structures. Objects can be designed as refracted in the plan and profile.

HelCor tubes are manufactured in standard lengths of 6, 7 and 8 m, but the manufacturing process allows for the manufacture of tubes of any length. Pipes with a circular-arc cross-section HelCor PA are manufactured in lengths up to 10 m (standard 6 m). Sections are constructed of pipes of a total length in accordance with the planned length of the culvert. The final elements, i.e. the inlet and outlet are cut to the appropriate length and in accordance with the inclination of the slope of the embankment. The pipe sections are connected using band connectors. The couplings are made of stainless smooth or corrugated steel [41].

Fig. 1.18. Culvert of HelCor® type a) during mounting, b) finished[9]

The production technology of HelCor® or HelCor PA tubes allows to adjust the inlet and outlet to local conditions in terms of the slope inclination. It allows to measure the inlet/outlet angle at which the passage axis intersects the edge of the slope of the embankment in the plan. A bevel according to the inclination of the slope of an embankment can be made for the entire height of the pipe or end with a vertical path. Strengthening the slopes of an embankment around the culvert inlets can be performed by means of vertical walls of reinforced soil, reinforced concrete, gabions or prefabricated elements.

When using inlets with a traverse bevel tailored to the slope inclination, inlets can be finished with the appropriate slope with concrete or stone on a sand-cement mix, with perforated concrete panels, riprap or by using a reinforced concrete ring and a sown grass slope.

Bevelling culvert inlets for the plan ≠ 90° can be made both at the end of a vertical pipe and at cuttings according to the inclination of the slope. It is not recommended to construct the culvert inlets bevel angle in the plan < 55°.

Another type of ground-shell constructions is MultiPlate [42]. It is used in civil engineering in the construction of culverts, bridges, overpasses, tunnels,

[9] Photos by M.Kowal

subways, transitions for service, green transitions. MultiPlate is a multi-jacket steel shell structure with corrugated sheet of a thickness of 2.5÷8 mm. Depending on the function of the object, MultiPlate structures can be equipped with lighting, ventilation, skylights, niches, technological holes, shelves for animals or other elements.

A steel sheet is typically coated with a layer of zinc, and also can be protected against corrosion by a coating polyamide, depending on the estimated useful life of the structure. The dimensions of the wave are mainly 200x55 mm or 150x50 mm. Spans of these coatings range from about 1.4 metres to over 12.0 metres. The cross-sections of MultiPlate shells are in the shape of wheel, circular-arc, elliptical horizontal and vertical or curved frame. The sheet standard width of 1.2 m are coupled by screws. For construction one uses MultiPlate steel in accordance with the standards EN 10025 and EN 10149, in grades S235JR, S355J2 or S355MC.

MultiPlate structures can also be used for multi-hole sections (Fig. 1.19).

Fig. 1.19. Multi-Plate[10]

MultiPlate structures are used for all load classes of road and rail in accordance with standards [37], [32] and the special NATO vehicle load according to the standardization agreement (STANAG 2021). The MultiPlate construction dimensioning method is Sundquist-Pettersson method. It is also called the Swedish method. Calculations can be performed by other methods, for example CHBDC or, in complex cases, the finite element method (FEM).

MultiPlate structures are also used to strengthen existing objects using the so-called relining method. A MultiPlate structure is introduced in the width of an existing object. Then the space between the inner shell and the strengthened object is filled with concrete mix C16/20. This method allows to strengthen the

[10] Photo by S.Karaś

existing facility without a traffic disturbance and eliminates the need for demolition of the old structure.

The foundation of these structures depends on the type of cross-section. Structures with a closed section (circular, elliptical, pipe-arch) are founded on a foundation of gravel with a minimum thickness of 30 cm. On the ground of aggregate one should put a layer of sand and gravel ballast with a thickness of approx. 5 cm so that the notches of a construction could sink in. The upper surface of a foundation should be contoured to the shape of the bottom and carefully concentrated in the groin area.

Structures with an open cross-section are based on rigid foundations of concrete or flexible sheets (corrugated sheets). In backfill and foundation the gravel aggregate can be used, a mixture of gravel and sand, crushed aggregate, key aggregate. Requirements for aggregates are consistent with the requirements of the construction of HelCor® structures.

In order to secure a structure against rainwater, one needs to put over the backfill key a geotextile and geomembrane screen with a thickness of 10 to 15 cm. The membrane material should not only be waterproofed, but also be resistant to any puncture during the compaction of backfill and during transport. Water from the surface of the membrane may be led to drainage pipes arranged parallel to the structure.

a)

b)

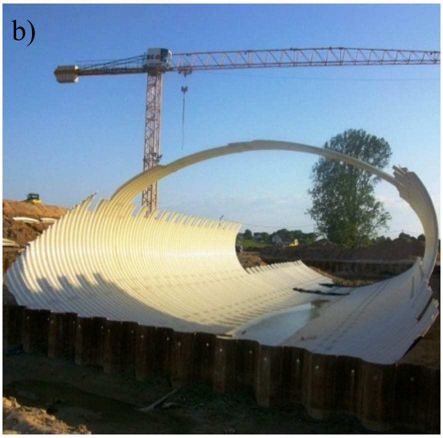

Fig. 1.20. SuperCor® a) opened type – arch, b) closed type – ellipse[11]

Shells with a high profile, SuperCor®, are made of sheet with a thickness of 3.5 to 7 mm and a wave size 380x140 mm [43]. They have an open or closed section. Sections encountered in this type are: arches, box sections and circular and elliptical cross-sections. The object destination and corrosion protection is analogous to the MultiPlate structure.

[11] Photos by M.Kowal

SuperCor® structures are a new generation of flexible structures of corrugated metal sheets of very high rigidity. Their carrying capacity is much greater than the capacity of traditional structures of corrugated metal sheets. These structures are used in the construction of engineering structures over and under roads and railway lines. Their range reaches 25 m. In production SuperCor® steel grade S315MC is used.

SuperCor® structures are used for all load classes of road and rail in accordance with standards [37], [32]. For the purposes of design one must define: function (culvert, viaduct, ecological transition, etc.), shape, flow and width, foundation type, height, depth of cover, type and size of load, backfill with the parameters and methods of installation. Then static-strength calculations must be performed, including the expected deformation during backfilling, the durability object selection and editing techniques to design appropriate finish and equipment, taking into account the aesthetics of the object. SuperCor® structures are dimensioned by means of analogous techniques to MultiPlate's.

In order to stiffen the inlets cut according to the inclination of the slope, a wreath is used. The wreath is a part of an embankment finishing trim and structures located on a slant to the axis of the road when the angle between road and culvert ≤ 65°, span > 3.5 m or culvert span exceeding 6.0 m. Smaller objects can also be stiffened by a wreath or band.

Erection of SuperCor® with galvanized corrugated steel sheets requires adherence to a strict technological regime. It is important that before installation all assembly guidelines are read and the assembly drawings supplied. One must adhere to the element assembly sequence.

Regarding profiles with a closed section, erected on the foundation of aggregate, the assembly work begins after laying and compaction of aggregate and checking the density indicator. With an open profile section, erected on concrete benches, the assembly work begins after mounting the anchors serving to attach the mounting of channel elements for design.

During the first phase of the installation of SuperCor® remember to do not tighten the bolts too tight, ie. not to the required torque value, because it allows for an easier fitting of plates and holes taking into account the dimensional tolerance and element flexibility. The final tightening of screws should be performed only after the whole structure is assembled. Exceptions are screws, which will not be available when assembled whole structure. During backfilling, one should carry out a check of the torque on the screws just before backfilling. The recommended tightening torque depends on the design span and has a minimum of 300 Nm in the structure with a span ≤ 7.0 m and a minimum of 360 Nm in the construction span > 7.0 m.

During backfilling SuperCor® structures are subject to deformation. This phenomenon is desirable, and helps to incorporate soil to cooperate in the process of carrying loads (due to the structure compression). The process of the deformation of a structure should be controlled. After the complete assembly of a structure and before backfilling, one should measure the span and height in order to check whether the dimensions are within the tolerance. The deformation of the cross-section after charging should not exceed 1% of the span structure measured after the assembling.

Recently, the pallet of ground-shell structures have included UltraCor® [44]. According to the manufacturer, the most durable wave profile in the world of flexible construction. It can carry very heavy loads, spans over 30 m. Available sections of shells of this type are box and arched ones. The wave height of 240 mm, a wavelength of 500 mm. The thickness of the plates offered is 7 mm, 8 mm, 9.5 mm and 12.5 mm.

Rigid shells include arched culverts and shell objects of a reinforced concrete coating. Arched culverts can be made of stone, brick, concrete and reinforced concrete, and the dimensions of the holes are standardized and can vary from 0.6 m to 4.0 m with a height of 0.6 m to 4.7 m. In arched culverts of spans up to 3.0 m, a vault forms the whole structure with foundations. In culverts with a span of more than 3 m, foundations are usually separated and diagonal wings on the inlets are applied. Foundations under wings and walls are laid 25 cm below the freezing zone. The depth of foundations ranges from 1.0 m to 2.5 m depending on the culvert span. Glands divided into sections of 2÷2.5 meters. Between the walls within the culvert there is a stone layer of a thickness of 50÷75 cm.

Shells of reinforced concrete can be divided into monolithic and prefabricated ones. Monolithic shells are characterized by considerable spans. Prefabricated structures are cross-sections of pipes, open and closed (type Matiere), parabolic (type Prefac), box (type Box-culvert and Opti-quadro), and frame (type CON/SPAN®). Due to the considerable weight of prefabricated elements, heavy construction equipment is necessary during mounting. This is a drawback in terms of the design.

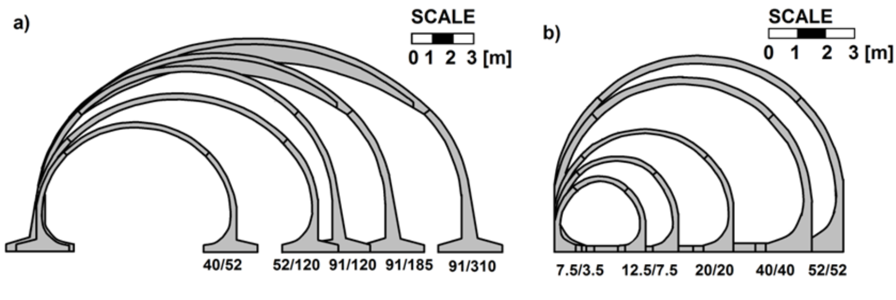

Fig. 1.21. Matiere arc type structure cross-section: a) opened, b) closed [25]

The Matiere-type structures are formed from prefabricated units of any number of segments with a width of 1 to 5 m. The composition of each segment includes two lower elements (walls) and one upper element (vault). A wide range of height and width of these structures allows to obtain a span of 22 m and a height of 10 m. Matiere structures exist in two cross-sectional forms – arc (Fig. 1.21), and rectangular [27].

Taking into account backfill as stiffener of a structure at the design stage, it is possible to reduce the thickness of the elements. The object flexibility is achieved by a unique combination of upper and lower elements. It is a joint, where is no rigid connection between these elements.

Additionally, the system allows to use prefabricated cornices and retaining walls. This minimizes the amount of construction works and reduces the total time of the construction of a facility.

It is important to ensure a proper isolation of bridge structures. As the objects are designed for at least 100 years, the same service life is required of the materials used for the structure insulation. The basic premise when designing and making objects is an entirely maintenance-free facility throughout its use. The choice of insulation materials depends on the cross-sectional shape. In the case of a rectangular cross-section, the outer surface of an object is secured by a bitumen-epoxy insulation coating. In the case of an arc cross-section a bitumen-epoxy insulation is replaced by a smooth, non-reinforced membrane insulation based on plasticized polyvinyl chloride. Other methods of isolation for both sections are essentially the same, and the outer surface of the foil is secured by the perforated foil. External joints are secured using bulging putty, a permanently elastic polyurethane sealant and an additional cover by the system tape. The internal combination of elements is filled with bulging putty and permanently elastic polyurethane putty. The use of high-grade concrete also affects the tightness property.

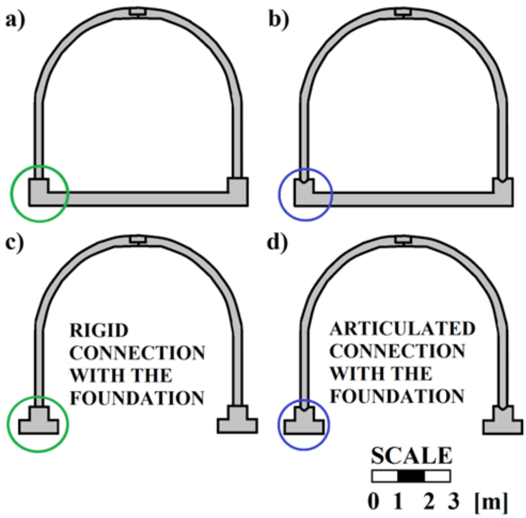

Fig. 1.22. Prefac-type structure cross-section: a), b) closed, c), d) opened [25]

The above described engineering structures can be used for the purposes of the construction of roads, railways, a pedestrian or pedestrian and bicycle trail, wildlife or other communication and production devices (e.g. conveyor belt, pipeline) by or under a terrain obstacle.

Prefac-type structures can be open or closed. The structure consists of two prefabricated units connected pivotally or rigidly mounted on a single or two separate benches (Fig. 1.22). They are produced in spans of from 6 to 10 m [25].

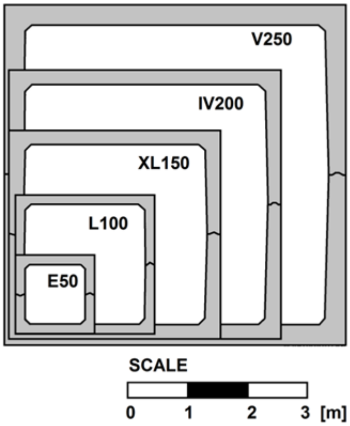

Fig. 1.23. Box-culvert structure cross-section [25]

The box-culvert type structures are frame systems of a closed section formed with two superposed prefabricated U-shape units. Their dimensions amount normally to 1.0×1.0 m 5.0×5.0 m (Fig. 1.23) with a plate thickness of 15 cm to 30 cm. A prefabricated unit plate has a constant thickness. The thickness of vertical walls decreases towards the joint. Prefabricated structures of this type are similar in shape and dimensions to box prefabricated reinforced concrete culverts.

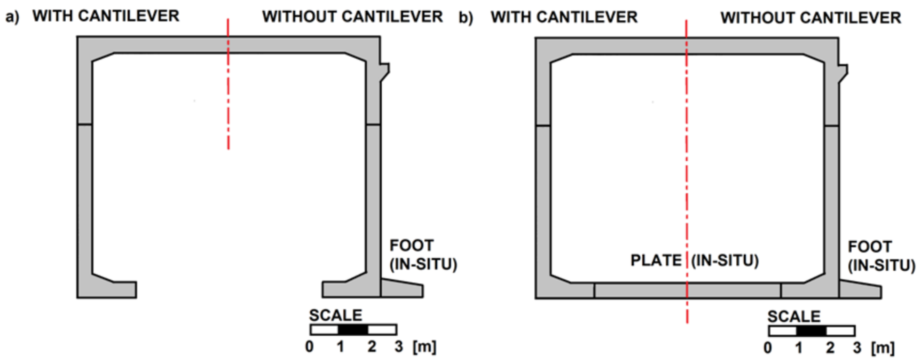

Fig. 1.24. Opti-quadro structure cross-section a) open, b) closed [25]

An Opti-quadro structure is an extension of box-culvert structures to span from 7.0 m to 9.5 m. The upper element, prefabricated and U-shaped, rests on two elements of the T-track section or the L-shaped. Opti-quadro structure can have an open cross-section or may be closed by the bottom plate (Fig. 1.24).

CON/SPAN® structures used under or over an obstacle on a road, rail trail, a pedestrian or walking-cycling trail, wildlife passing and activity or production routes [45].

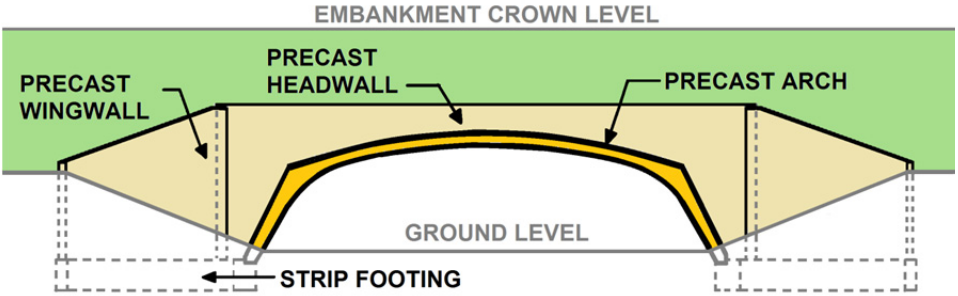

Fig. 1.25. CON/SPAN® type structure

The components of such objects are prefabricated reinforced concrete foundations – precast express foundations, prefabricated carrying structures, prefabricated walls and prefabricated front wings, Fig. 1.25. The advantages of the system CON/SPAN® are span dimensions from 3.96 m to 19.80 m at heights from 0.98 m to 4.13 m, the possibility to apply to all classes of road and rail loads in accordance with standards [37], [32] and special vehicle loads (STANAG 2021).

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Chapter 2.

Hydro-hydraulic analysis

Krzysztof Śledziewski

2.1. Introduction

The subject of this chapter are the basic hydrologic and hydraulic calculations to assist in determining the minimum bridge clearance taking into account the safety of buildings in the transition of flood waters. The purpose of the calculations of bridge clearance is to determine the horizontal dimension of undeveloped watercourse guaranteeing a secure and reliable flow of great water. The method for calculating the bridge clearance is different for large and small bridges (with spans up to 10 m). The method of calculating small bridges is similar to the calculation of the internal dimensions.

Bridge clearance is determined by tests consisting in defining the light minimum, the assumed position of abutments, supports and their dimensions, calculating the expected washout and accumulation, and then comparing them to the requirements which must be met with regard to a bridge, speed limits of the flow of water, washout at the bottom, the accumulation of water or the minimal rise of a structure above the level of flood waters. Sections of a bridge are designed to allow for a flow with a certain probability of exceedance, as constituting the structural safety.

The scope of calculating the bridge clearance includes the hydrologic and hydraulic analysis. Hydrological calculations determine the level, velocity and flow volume of flood waters [2]. Hydraulic calculations determine the amount that allows for the design of a bridge of a suitable size that would guarantee an efficient flow of water, non-threatening objects or adjacent sites, giving at the same time the possibility of efficient use of the space under the bridge. Therefore, hydraulic calculations of bridges include: defining the minimum bridge clearance, determining the expected deepening of the trough in the case of cross bridges, determine the local washout next to pillars and determining the amount of accumulation before the bridge.

2.2. Hydrological calculations

2.2.1. River basin

The main part of water resources is the water flowing in the river beds, which comes mostly from precipitation. Every area, regardless of its size, from which water flows into a specific section (the section closing a part of the basin, e.g. the mouth of an inlet, a bridge over a river, etc.) is called the basin.

In hydrology, arranging the water balance in an area, relates generally to an interesting us river basin. Therefore, it is necessary to establish its boundaries which are determined based on topographical maps – determining the so-called watersheds. Surface watersheds (drainage basin topography) is determined on the map by a contour line, leading lines of partition by ridges and tops according to the instructions in Fig. 2.1.

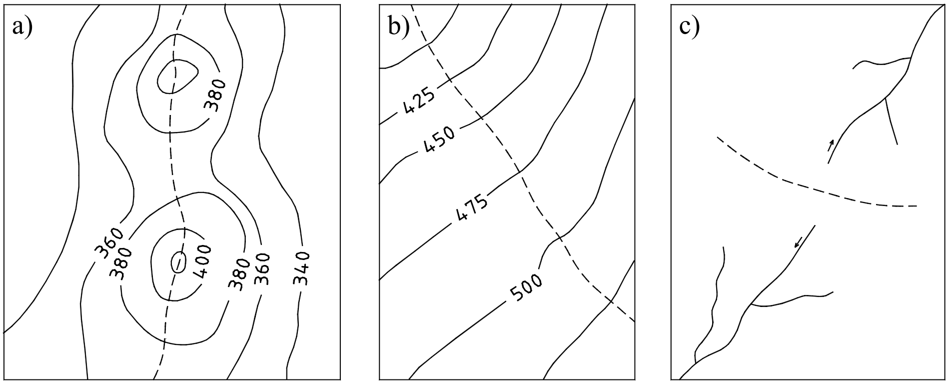


Fig. 2.1. Delineation of a river basin: a) according to the highest points of a field, b) perpendicular to the contour lines, c) between adjacent tributaries of rivers [5]

In hydrological terms the drainage basin can be described by the varying characteristics of the terrain, such as:

- lot inclination of the land and the drainage basin area,
- the manner of managing the drainage basin area,
- soil permeability,
- the presence of wooded areas, rocky wasteland, gravel or sand, all kinds of vegetation,
- the bogginess of the land surface and underground water retention levels.

Described features have a significant impact on the ability to retain water from the rain, or the so-called retentive capacity of a drainage basin.

Physical-geographic parameters of watercourses and drainage basins used in calculating the maximum flow are as follows [3]:

- field drainage basin area A defined in km^2 or ha.

The drainage basin area should be determined with high accuracy, because errors significantly affect the flow rate calculation, which affects substantially the dimensions of designed objects:

- length L defined as the length of the main stream with a dry valley or the maximum size of a catchment in a straight line along the main stream valley,

- length l determined in km as the length of the dry valley measured along the axis of the valley from the beginning of the stream up to the intersection of the valleys of the watershed,
- river network density ρ calculated as the ratio of the total length of all watercourses with their dry valleys and the catchment area:

$$\rho = \frac{\Sigma(L+l)}{A}, \tag{2.1}$$

- a decrease of stream I_r calculated as multiple elevations of the watershed at the intersection with the axis of the dry valley (W_g in m ASL) and the cross-sectional elevation calculation (W_d in m ASL) and the length of the ditch with a dry valley:

$$I_r = \frac{W_g - W_d}{L+l} \text{ [m, km}^{-1}\text{,‰]}, \tag{2.2}$$

- an average decrease of watercourse I_{rl} calculated on the basis of the longitudinal profile of the watercourse with a dry valley:

$$I_{rl} = \frac{W_{gl} - W_d}{L+l} \text{ [m, km}^{-1}\text{,‰]}, \tag{2.3}$$

The average decline of a stream can be taken as indicative of:

$$I_{rl} \approx 0{,}6 \cdot I_r, \tag{2.4}$$

- a contractual rate of decline in the watercourse I_{ru} calculated by dividing the difference in the elevation of the uppermost sources ($W_ż$ in km) in the drainage basin area and the hill cross section calculation (W_d w km) and a maximum length of watercourse in the drainage basin area (L_{max} w km):

$$I_{ru} = \frac{W_{ż\max} - W_d}{L_{max}} \text{ [km, km}^{-1}\text{]}, \tag{2.5}$$

- computing the roughness factor of a streambed m section,
- average slope catchment ψ calculated as the quotient of the difference of the highest elevation (W_{max} in m ASL.) points in the drainage basin area and the hill cross section design (W_d in m ASL) and the square root of the drainage basin area (A in km^2):

$$\psi = \frac{W_{max} - W_d}{\sqrt{A}} \text{ [m, km}^{-1}\text{,‰]}, \tag{2.6}$$

- average length of slopes l_s calculated as:

$$l_s = \frac{1}{1{,}8 \cdot \rho} \text{ [km]}, \tag{2.7}$$

- average fall slopes I_s calculated according to the formula:

$$I_s = \frac{\Delta h \sum k}{A} \text{ [m, km}^{-1}\text{,‰]}, \tag{2.8}$$

  where: Δh is the height difference of adjacent contour lines, Σk the total length of the contour lines in a drainage basin,
- slope roughness factor m_s,
- runoff factor φ,
- impermeability of soil indicator N,
- drainage basin indicator JEZ calculated according to:

$$JEZ = \frac{A_{j1} + A_{j2} + \ldots + A_{jk}}{A} = \frac{\sum_{1}^{k} A_{ji}}{A}, \tag{2.9}$$

  where: A_{ji} is the drainage basin area of a lake, the surface of which (s_i) is equal to or greater than 1% of its drainage basin area,
- drainage basin bogginess indicator B calculated according to:

$$B = \frac{A_{b1} + A_{b2} + \ldots + A_{bk}}{A} = \frac{\sum_{1}^{k} A_{bi}}{A}, \tag{2.10}$$

  where: A_{bi} is a single peatlands area,
- the amount of average annual atmospheric precipitation P,
- the maximum daily precipitation with the emergence probability of 1% $H_{1,}$
- the maximum amount of outflow h_1 with the probability of appearance 1%.

2.2.2. Probability flow

In the design practice it is necessary to adopt the probability superiority the great flow of water, which is the basis for determining the design flow and determine the dimensions of the building engineering. This involves on the one hand the security of the proposed buildings, on the other hand economic factors.

Tab. 2.1. Probability of design flows *p* [%] [6]

Construction type	Probability value *p*		
	Roads class		
	A, S, GP (%)	G, Z (%)	L, D (%)
Bridge	0.3	0.5	1
Temporary bridge	2	3	3

The probability value is assumed depending on the road category including a bridge (Tab. 2.1) or the validity of a railway line. The more important the route in terms of communication, the lower probability p is.

The lower the percentage of the probability superiority of the maximum flow is taken to be authoritative for the calculation, the greater the safety of the proposed buildings.

2.2.3. Design flow calculation method

Design flow for the calculation of engineering structures is such great water the flow, with the probability of superiority determined appropriately to the object class percentage. The maximum probability flow for a particular section of a watercourse can be determined by the following methods:

Direct method, which is used when a given section has complete hydrometric data from a multi-year period (at least 15 years, although a longer period is desired, e.g. 40 years or more). This method can be applied only for the river gauge section. Its essence is based on assumption that maximum flows are arranged according to a given probability distribution.

Transporting flow method (moving) It is used when hydrometric data is incomplete or absent with regard to the section under consideration. Transporting involves transfer size calculations for drainage basins to the calculation cross-section.

Indirect method is applicable when there is no hydrometric data available. This applies in particular to small uncontrolled drainage basins. Indirect method includes:

- regression equation of the area,
- thawing formula,
- and rainfall formula.

Projects for the construction of engineering structures usually relate to small drainage basins, and even very small ones: below A=1 km^2. A small drainage basin area does not exceed 50 km^2 while streams do not exceed 2 km. Therefore, the following section looks into the calculation of the maximum flow of a certain occurrence probability by means of the rainfall formula only, which is described by:

$$Q_{\mathrm{p}} = f \cdot F_1 \cdot \varphi \cdot H_1 \cdot A \cdot \lambda_{\mathrm{p}} \cdot \delta_{\mathrm{J}}, \tag{2.11}$$

where:

F_1 – the maximum elementary outflow module expressed as a fraction:

$$F_1 = \frac{q_1}{\varphi \cdot H_1}, \tag{2.12}$$

q_1 – the maximum elementary outflow probability of 1%;

φ – the outflow factor;
H_1 – maximum daily precipitation of the emergence probability of 1% [mm];
f – dimensionless ratio equal to 0.45 waveform in lake districts and 0.6 in the remaining areas;
λ_p – quintiles of variable distribution for a given probability p;
δ_J – the lake reduction factor, read depending on the lake density indicator *JEZ* (2.9).

The maximum outflow module unit F_1 is determined depending on hydro-morphological characteristics of a riverbed ϕ_r and the runoff time of the slopes t_s. Hydro-morphological characteristics of a river are calculated as follows:

$$\Phi_r = \frac{1000 \cdot (L + l)}{m I_{rt}^{1/3} \cdot A^{1/4} \cdot (\varphi H_1)^{1/4}}, \tag{2.13}$$

where:
$L+l$ – is the length of a watercourse along with the dry valley [km];
m – the river-bed roughness measure;
I_{rl} – an average decrease of watercourse calculated by (2.4).

The runoff time on the slopes is determined depending on hydro-morphological characteristics of the slopes:

$$\Phi_s = \frac{(1000 l_s)^{1/2}}{m_s I_s^{1/4} \cdot (\varphi H_1)^{1/2}}, \tag{2.14}$$

where:
l_s – the average length of slopes calculated by (2.7) [km];
m_s – the slope roughness measure;
I_s – the average slope fall calculated by (2.8) [$m \cdot km^{-1}$] or [‰].

For a drainage basin area of more than 10 km2 the slope run-off is determined in a simplified manner depending on the location of the drainage basin in one of the five macro-regions [3].

2.3. Bridge clearance

2.3.1. Calculation of the bridge clearance

Before calculating the bridge clearance one should carry out a number of preliminary tests, such as:
a research study (map, rainfall measurements, etc.),
measurements and field trials (the current cross-section of a watercourse on the site of the proposed bridge to establish a local drop in the water table at an

authoritative flow, measuring the water velocity on the site of the proposed crossing),
analytical studies (the impact of the proposed crossing on the environment, water flow conditions, etc.).

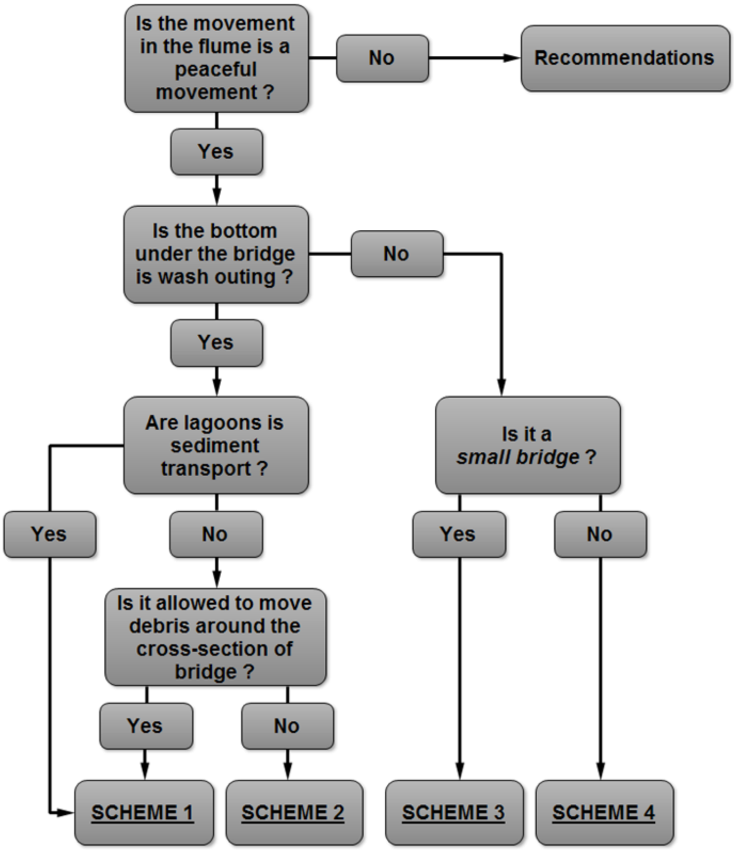


Fig. 2.2. The algorithm for selecting the computational bridge clearance

Four diagrams used in the bridge clearance calculation [7]:

Scheme 1 – a cross-sectional bridge with a washout bottom and the movement of sediment in the entire cross-section.

Scheme 2 – a cross-sectional bridge with a washout bottom and the movement of sediment in a portion of a section.

Scheme 3 – a cross-section of a bridge with without bottom washout.

Scheme 4 – a section of a bridge with without bottom washout: a small bridge.

The algorithm selection is shown in Fig. 2.2.

For each of the selected schemes there are templates and procedures for calculating the minimum light of a bridge and washout (deepening cross-section) for complex dimensions of a bridge.

In cases where there is washout of the bottom and possible movement of the rubble in the entire cross section (Scheme 1), the minimum bridge clearance L calculated depending on the intended degree of dilution P:

$$L = B_{\mathrm{og}} \left(\frac{Q_{\mathrm{m}}}{Q_{\mathrm{og}}} \right)^{4/3} \cdot P^{-3/2}, \tag{2.15}$$

and the average speed in cross-section of bridge after washout is calculated as follows:

$$v = \frac{Q_{\mathrm{m}}}{P \cdot F}, \tag{2.16}$$

where:
B_{og} – the width of the body of water in the main trough,
Q_{m} – the design flow,
Q_{og} – the flow in the main trough,
P – the acceptable washout,
F – the cross-sectional area of a bridge.

The amount of washout at the assumed light L to be calculated from the formula:

$$P = \left(\frac{L}{B_{\mathrm{og}}} \right)^{-2/3} \cdot \left(\frac{Q_{\mathrm{m}}}{Q_{\mathrm{og}}} \right)^{8/9}. \tag{2.17}$$

If it is possible to move the rubble only in part of the cross-section (Scheme 2), then cross-section of the bridge is divided into a main part about the clearance L_{g}, in which the moves rubble and part of the sides L_{z}. For the main part of the section is calculated washout from condition of continuity of rubble, and washout in the side parts is determined by comparing the resulting velocity speeds. The minimum bridge clearance is calculated on the basis of the flow in the main part of the cross section of the clearance L_{g} by the washout degree P, then the choice of clearance L_{z}. Clearance L_{g} is taken as part of the main channel B_{og} reduced by expected width of pillars. In this cross-section rate of flow is determined from the formula:

$$Q_{\mathrm{g}} = Q_{\mathrm{og}} \left(\frac{L_{\mathrm{g}}}{B_{\mathrm{og}}} \right)^{3/4} \cdot P^{9/8}, \tag{2.18}$$

and then the flow through the remainder of the cross-section of the bridge L_{z} is calculated as follows:

$$Q_{\mathrm{z}} = Q_{\mathrm{m}} - Q_{\mathrm{q}}. \tag{2.19}$$

After determining the necessary clearance value L_z, such that the flow Q_z can take place, fulfilling the set pre-assumptions, should calculate the range of washout in both parts of the riverbed L_g and L_z.

In the case where no bottom washout is expected (Scheme 3), which implies that the average flow velocity does not exceed the speed specified in Tab. 2.4, Tab. 2.5 and Tab. 2.6 the minimum bridge clearance is determined by the formula:

$$L = \frac{Q_m}{u \cdot h \cdot v}, \tag{2.20}$$

where:

h – average depth in bridge cross-section,

v – an established standard flow rate not greater than:

– critical speed $v_{kr} = \sqrt{g \cdot h}$,

– the smallest cross-sectional speed v_{nr} or acceptable v_d,

μ – the factor that should be applied in the case of single-span bridges based on Tab. 2.2:

– for the pillars of the rounded side water inflow $\mu = 0.78{+}0.021\sqrt{L}$,

– for the pillars of sharpened side water inflow $\mu = 0.85{+}0.014\sqrt{L}$,

– if the bridge clearance is greater than 100 m $\mu = 0.99$,

– for the bridge clearance of less than 30 m when the design flow is accompanied by the flow of ice, it is recommended to decrease the calculated value μ by 0.05;

here, the average flow velocity is calculated as follows:

$$v = \frac{Q_m}{\mu \cdot L \cdot h}. \tag{2.21}$$

2.3.2. Calculation of the clearance of small bridges

Small bridges are subject to a different calculation procedure allowing accumulation of the water table in front of the bridge and the occurrence of a movement rushing on the section crossing. The method of calculation depends on the hydraulic conditions prevailing under a bridge. The given rules are designed on the assumption that a small bridge acts as a spillway with a broad crown, whether sunk or not.

If the condition is met:

$$h_d = N \cdot H, \tag{2.22}$$

where:

N – the border sinking coefficient whose value depends on the flow rate transfer m (Tab. 2.2),

h_d – depth in the trough after the bridge,
H – the dammed water depth before the bridge.

Tab. 2.2. The values of factors for small bridges

Abutments type	μ	m	N	k
with curved wings	0.93	0.36	0.78	0.54
a body embedded in the embankment	0.91	0.35	0.80	0.52
with oblique wings	0.88	0.34	0.81	0.49
with wings parallel to the axis of the road	0.86	0.33	0.83	0.47
with wings perpendicular to the axis of the road	0.83	0.32	0.84	0.45

If the condition (2.22) is not satisfied, then the bridge acts as overfall sunk and the flow cross-section bridge does not change; the movement remains peaceful.

In the case of calculations according to the unsinkable overfall scheme the clearance bridge is calculated as follows:

$$L = \frac{Q_m}{m\sqrt{2g}H_o^{3/2}}, \tag{2.23}$$

where:
Q_m – the design flow,
m – the flow rate depending on the geometry of the inlet (Tab. 2.2),
$H_o = H + \frac{v_s^2}{2g}$ – amount of accumulated water energy before bridge,
v_s – the average velocity of the water dammed before the bridge,
g – acceleration due to gravity.

The cross-section flow of bridges is calculated as follows:

$$Q = m \cdot L\sqrt{2g}H_o^{3/2}, \tag{2.24}$$

and the speed of:

$$v_m = \frac{Q}{k \cdot L \cdot H}, \tag{2.25}$$

where:
k – a coefficient selected on the basis of Tab. 2.2.

If the case of the sunken overfall scheme the clearance bridge is calculated by the formula:

$$L = \frac{Q_m}{\mu \cdot h_d\sqrt{2g(H_o - h_d)}}. \tag{2.26}$$

In various cases different procedures may apply:

- the bridge clearance is determined for the assumed accumulation before a bridge,
- the bridge clearance is calculated for the assumed maximum speed of in section of bridge selected depending on the resistance of the substrate to washout or reinforcements used in the bottom.

For the bridge with clearance *L*, the depth of water dammed before the bridge is determined as:

$$H = \left(\frac{Q}{m \cdot L \cdot \sqrt{2g}} \right) - \frac{v_s^2}{2g}. \tag{2.27}$$

Because of the relationship v_s and the depth *H*, the calculation is performed by iteration, assuming as the first approximation $v_s = v_o$.

2.3.3. Bottom washout in bridge section

Bridge clearance should be large enough to power the natural flow so that it can overcome obstacles is a narrowing section. At the same time washout the bottom of the bridge cross-section must not exceed the limits set for the method of foundation supports (Fig. 2.3).

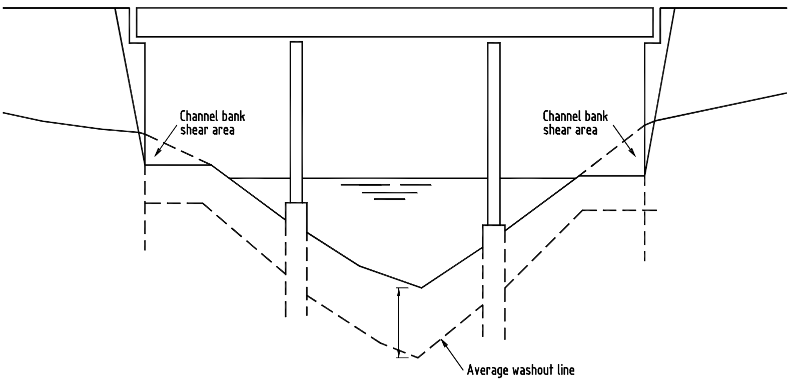


Fig. 2.3. The cross-section under the bridge after washout the flume

In an extreme case, it can be assumed that washout is unacceptable. If you do not breach of the bottom of the riverbed, the speed in a sectional bridge cannot exceed a speed of not causing washout (bottom natural) or speed limit (bottom fortified) – called the critical speed of flow, depending on the type of soil in the ground). In any case, calculations require the adoption of span dimensions and location of supports. Next, flow rates under the bridge and the expected deepening of a possible cross-section of the bridge can be calculated. It is also required to calculate the depth of local washout at pillars, accumulation before the bridge and the elevation of the minimum elevation spans of the watercourse.

The fundamental relationship, which applies when determining the light of the bridge, binding the two parameters of two neighbouring sections (1 and 2) of the same river is as follows [6]:

$$\frac{B_1}{B_2} = \left(\frac{Q_1}{Q_2}\right)^{4/3} \cdot \left(\frac{h_1}{h_2}\right)^{-3/2}, \tag{2.28}$$

where:
B – the width of a river-bed,
H – an average depth of a river bed,
Q – the flow causing of sediment transport.

Deepening the bottom section of a bridge results from an increase of the water flow rate, which is the direct cause of the reduction in the cross-section of the stream (if it takes place). The process continues after aligning the transport capacity of the stream in the main section of the bridge and the river bed under the bridge. The permissible amounts of washout are given in Tab. 2.3.

It is assumed that the washout in proportion to the primary section depth is:

$$h_1 = P \cdot h, \tag{2.29}$$

where:
h_1 – the depth of a stream after washout,
h – the depth of a stream before washout,
P – the amount of washout.

Tab. 2.3. Permissible amount of washout *P* [6]

Foundation suport type	Not streamline foundation	Half streamline foundation
Massive foundations deep, large diameter on piles and foundation directly on the rocks	1.3	1.4
Foundations on stilts in the wall sealed	1.1	1.25
Foundations on piles without piling	1.0	1.1
Foundation work directly on the ground	1.0	1.0

The presented assumption is strongly simplified. Since the washout process is dynamic and may vary from time to time, it applies to both the depth and location of the largest washout. Sometimes it may happen that at a given time it may lead to reduction of the maximum washout in the bridge section.

The local washout at supports is, however, the result of disturbances caused by the appearance of the watercourse obstacle in this section. For this reason it is very important that pillars be correctly positioned in the watercourse section and have a proper shape to disturbance of flow was low as possible. The above reasoning leads to the following recommendations regarding the location of supports:

- the plane of the side pillars and abutments should be tailored to the intended direction of flow and should not form an angle greater than 20° with respect to the direction of flow on a normal level;
- on navigable rivers the permitted deviation of the planes of pillars and abutments of the water run-off navigable by an angle no greater than 10°,
- the axes of supports of bridges located next to each other should be on the same line.

The limit values of the speed at which there is no more washout of the ground are shown in Tab. 2.4 and Tab. 2.5.

Tab. 2.4. Speeds not causing washout v_{nr} for cohesive soils at a depth equal to 1 m [6]

Type of soil	The cohesiveness of soil		
	Average cohesive	Cohesive	Very cohesive
Lessy	0.7	1.0	1.3
Clays	0.8	1.2	1.7

In the case of a depth different from 1 meter speed read from Tab. 2.4 and Tab. 2.5 must be multiplied by $h^{1/5}$, where h is the depth of rivers expressed in meters. For cohesive soils, with water depth greater than 3 m, speed that does not cause washout is assumed as for a depth equal to 3.0 m.

Tab. 2.5. Speeds not causing washout v_{nr} for soils with a depth equal stream 1 m [6]

Type of soil	Average grain diameter (mm)	Speed (m/s)
Dusty sand	0.005÷0.05	0.20÷0.30
Fine sand	0.05÷0.25	0.30÷0.45
Medium sand	0.25÷1.00	0.45÷0.60
Coarse sand	1.0÷2.0	0.60÷0.70
Fine gravel	2.0÷5.0	0.70÷0.85
Medium gravel	5.0÷10.0	0.85÷1.05
Pebble stones	10.0÷15.0	1.05÷1.20
Average boulders	25.0÷40.0	1.40÷1.80
Thick boulders	40.0÷75.0	1.80÷2.40
Weak rocks	–	2.50÷3.50
Hard rocks	–	3.50÷5.00

For non-homogeneous soils to the determine the speed of the non washout is the value calculated by the formula:

$$d_w = \frac{\sum d_i p_i}{100}, \tag{2.30}$$

where:

d_i – the average fraction i,

p_i – the percentage distribution of fractions i.

For land with a large particle size uniformity for a reliable speed not causing washout, one should take the speed corresponding to diameter $d_{80\%}$ ($d_{80\%}$ – the diameter of the land, that come along with the smaller represent 80% of his weight).

In order to protect a streambed from blur, if the expected flow velocity exceeds the critical speed, should be used consolidation of the ground. The maximum allowable flow rate ensuring the lack of blur varies depending on the strengthening of the ground (Tab. 2.6).

Tab. 2.6. Speed limit in fortified troughs v_d [6]

Bank protection type	Speed (m/s)
Turfing	
on flat	1.2
turf in wicker hurdles	1.8
Overhead stone without hurdles	
stone thickness of 7.5 cm	2.4
stone thickness of 10 cm	2.7
stone thickness of 15 cm	3.3
stone thickness of 20 cm	3.9
Pavements	
single thick (15–25) cm layer of moss	2.5÷3.0
single thick (15–25) cm in wicker hurdles	3.0÷3.5
single rubble thick (20–25) cm layer of crushed stone 10 cm	3.5÷4.0
Hexagonal paving slabs on a layer of gravel	3.5
Fascine mattresses of 50 cm	3.0
Flume lining	
crushed stone on mortar	5.0÷6.0
of concrete	6.0÷8.0
Temporary reinforcements	
fascine lining of (15–25) cm	1.2
fascine lining of (25–30) cm	2.2
stone lining fascines	3.3

Technical ways to strengthen the ground to prevent washout can be found in literature, e.g. [1], [4].

2.3.4. Water swelling before bridge

Building the streambed of a river usually leads to the narrowing of its cross section and changes in the flow conditions. In effect the speed of the water flow increases and there occurs damming before the bridge. The accumulation depends primarily on the geometry of the stream and the bridge section, the degree of stenosis and the flow velocity, and, to some extent, the dimensions and shape of supports.

As a rule, to washout the trough, to a lesser or greater extent, there is always a meaningful degree of accumulation usually calculated for the fuzzy channel. Washout riverbed occurs only after a certain period of operation of the bridge. Damming up of water is always accompanied by an increase in the speed of its flow. An important requirement affecting the acceptable level of stagnation, is to prevent excessive blur the bottom and sides of the watercourse. This is related to the permitted speed limit for the type of residual water in the soil substrate and the method of foundation support.

Damming of water depends on the speed of the flow, and as this velocity depends inter alia on accumulation, its calculation is performed by iteration, assuming, as the first approximation, that the speed of water in the damming section is the same as the cross-sectional undeveloped of bridge, which means that $v_s=v_o$. Under this assumption, the expected size is calculated using the formula of accumulation [7]:

$$\Delta_z = K\frac{\alpha v^2}{2g} + \frac{\alpha_o\left(v_o^2 - v_s^2\right)}{2g}, \tag{2.31}$$

where:

K – the loss factor,

α_o, α – Saint-Venant's factor, which amounts to before a bridge:

– for cross-compact α_o=1,2,

– for cross-contingency:

$$\alpha_o = 1{,}1\cdot\frac{v_{og}^2\cdot Q_{og} + v_{oz}^2\cdot Q_{oz}}{v_o^2\cdot Q_m}, \tag{2.32}$$

– under the bridge:

$$\alpha = 1 + M\left(\alpha_o - 1\right), \tag{2.33}$$

g – acceleration due to gravity,

v – the average speed of water under a bridge cross-section,

v_o – the average speed of the water under a bridge in an undeveloped section:

$$v_o = \frac{Q_m}{F_o}, \tag{2.34}$$

v_s – the average velocity of the water above the bridge, after swell:

$$v_s = \frac{Q_m}{\left(F_o + B_o\Delta_z\right)}, \tag{2.35}$$

F_o – the sectional area of the watercourse flume,

B_o – the width of the body of water,

$$M_s = \frac{Q_s}{Q_m}, \tag{2.36}$$

Q_m – the design flow,
Q_{og} – the flow in the main trough,
Q_{oz} – the flow on floodplains,
Q_s – the flow in the undeveloped part of a trough corresponding to the cross-sectional area of the bridge.

If the calculation of the accumulation of the cross-sectional area of the stream before a bridge is not different from the original surface by more than 5%, the calculated value requires no adjustment. Otherwise, one should calculate the speed v_o and v_s, and then include them in the formula. The calculations must be repeated to achieve the compliance area.

If, as a result of building, a section will washout the bottom, it should be taken into account in the calculation of accumulation. The value of stagnation after blurring is determined based on input accumulation on the assumption that there is no washout:

$$\Delta z_c = \left(\frac{F}{F_r}\right)^{8/5} \cdot \Delta z, \tag{2.37}$$

where:
Δz_c – the accumulation after washout,
F – the sectional area of a stream before washout,
F_r – the sectional area of a stream after washout.

When washout covers the whole cross-section of a bridge, ratio F/F_r is equal to the inverse degree of dilution P.

The loss factor K is calculated as follows:

$$K = K_o + \Delta K_f + \Delta K_e + \Delta K_\phi, \tag{2.38}$$

where:
K_o – the basic loss coefficient which depends on the shapes of bridgeheads and the degree of the narrowing of a watercourse by the bridgeheads (Fig. 2.4); curves for the cases where: 1 – the distance between bridgeheads is greater than 60 m (regardless of their shapes) or when the distance between bridgeheads is less than 60 m and a bridgehead ends in pile cones or there are vertical diagonal wings with an angle of deflection from the flow direction (30÷45°); 2 – the distance between bridgeheads is less than 60 m and there are vertical diagonal wings with an angle of deflection from the flow direction amounting to 60°; 3 – the dis-

tance between bridgeheads is less than 60 m and there are vertical wings positioned parallelly to the flow direction,

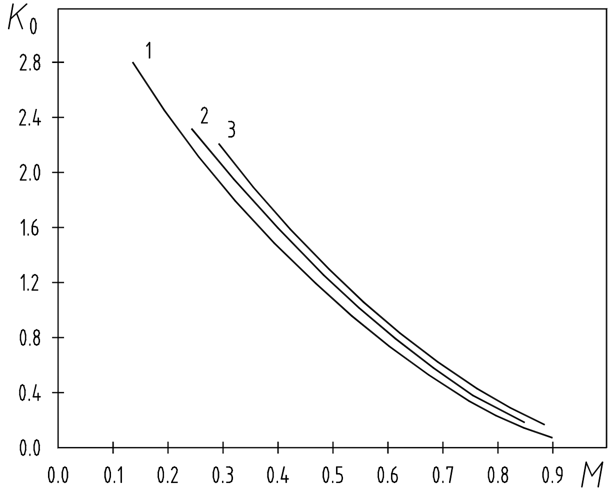


Fig. 2.4. Nomogram for establishing coefficient K_o

ΔK_f – amendment taking into account the influence of pillars which equals $m\Delta K_f'$ (Fig. 2.5),

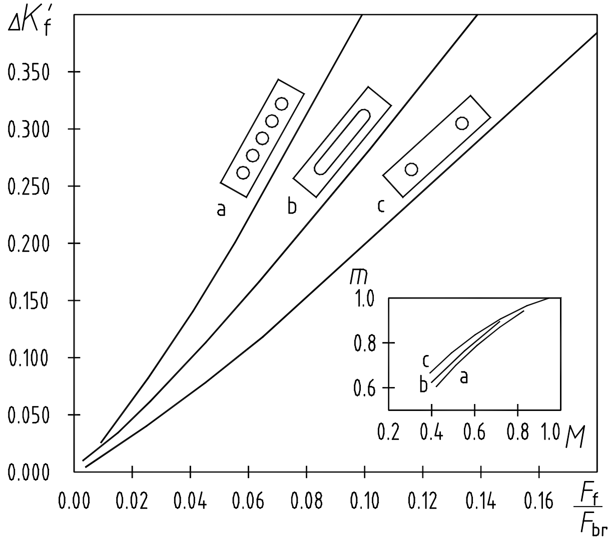


Fig. 2.5. The values of coefficients *m* and K_f'; F_f – the area taken by pillars, F_{br} – the area of the cross-section limited by bridgeheads; curves for: *a* – a row of poles, *b* – rounded solid pillar, *c* – two-pole pillar

ΔK_e – amendment taking into account the influence of the asymmetry of the watercourse narrowing (Fig. 2.6),

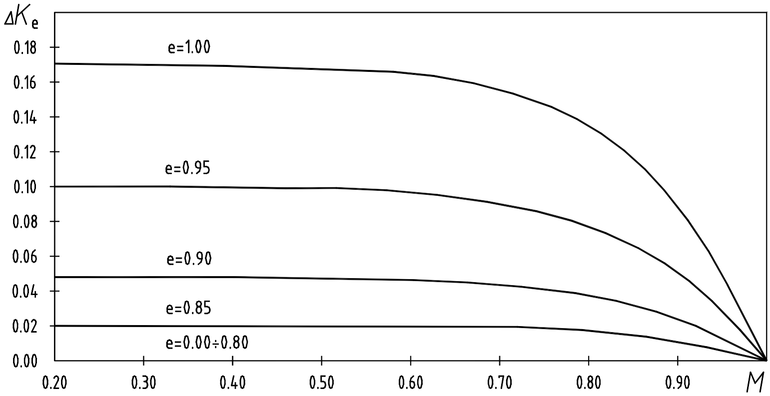


Fig. 2.6. The values of correction coefficients ΔK_e; e = 1-Q_p/Q_l if $Q_l>Q_p$, if $Q_p>Q_l$ than Q_l - the flow in the right part, Q_p – the flow in the left part of an undeveloped trough with closed access embankments

ΔK_ϕ – amendment taking into account the influence of a diagonal location of a bridge as compared to the axis of the watercourse (Fig. 2.7),

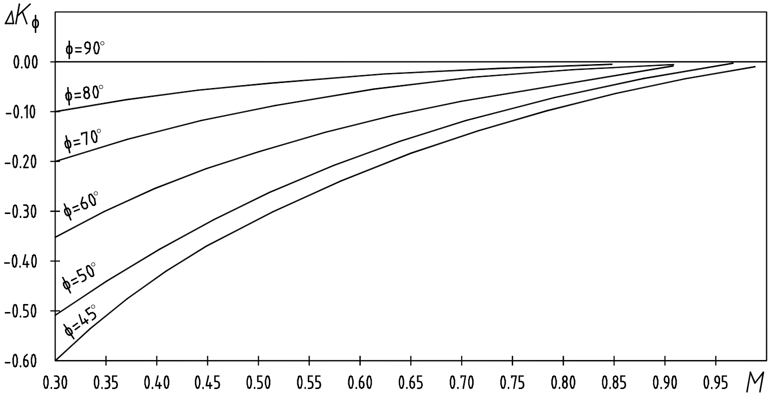


Fig. 2.7. The values of correction coefficients ΔK_φ; ϕ – the angle of the intersection of the bridge axis and the watercourse axis

Q_{oz} – flow on floodplains,

v_o – the average speed in a constant cross-section or a complete compound cross-section,

Q_m – flow in a constant cross-section or a complete compound cross-section.

In the case of washout of the bottom, the accumulation shall equal:

$$\Delta z_r \left(F / F_r \right)^{8/5} \Delta z. \tag{2.39}$$

If washout relates to the whole bridge cross-section, the proportion F/F_r equals the reciprocal of washout P.

2.4. Example

2.4.1. Hydrological calculations of a permanent bridge

An example analysis has been performed to determine the hydrological conditions in the catchment area. The determination of the size of large water bodies suitable for the planned bridge construction was made on the basis of the analysis referring to the whole catchment area.

Hydrological data of the structure were determined by empirical methods using the rainwater rule. The physical and geographical parameters of the catchment area were determined on the basis of a map (Fig. 2.8) and basic assumptions set out in Section 2.2:

- maximum flow of the probability of the occurrence $p = 0.3\%$,
- length of watercourse along the dry valley of a watershed $L+l = 6.48$ km,
- measure of the roughness of a streambed $m = 9$,
- elevations of the watershed at the intersection with the axis of the dry valley $W_g = 300.30$ m,
- elevations of the cross-section calculation $W_d = 206.45$ m,
- river basin area $A = 11.75$ km^2,
- maximum daily precipitation of the occurrence probability of 1% $H_1 = 100.00$ mm,
- waveform factor $f = 0.60$,
- runoff coefficient $\varphi = 0.55$,
- quantile of a variable distribution for a given probability $\lambda_p = 1.31$,
- drainage basin indicator $JEZ = 0.00$ calculated according to (2.9),
- lake reduction factor $\delta_J = 1.00$,
- a decrease of stream $I_r = 14.5$ ‰ calculated according to (2.2),
- an average decrease of watercourse $I_{rl} = 8.7$ ‰ calculated according to (2.4),
- height difference of adjacent contour lines $\Delta h = 10$ m,
- total length of the contour lines in a drainage basin $\Sigma k = 10.18$ km,
- average fall of slopes $I_s = 8.7$ ‰ calculated according to (2.8),
- total length of all watercourses with their dry valleys $\Sigma(L+l) = 6.48$ km,
- river network density $\rho = 0.55$ km^{-1} calculated according to (2.1),
- measure of the slope roughness $m_s = 0.20$,
- average length of slopes $l_s = 1.01$ ‰ calculated according to (2.7),
- runoff time on slopes $t_s = 205.3$ min,
- maximum outflow module unit $F_1 = 0.025$ calculated according to (2.7).

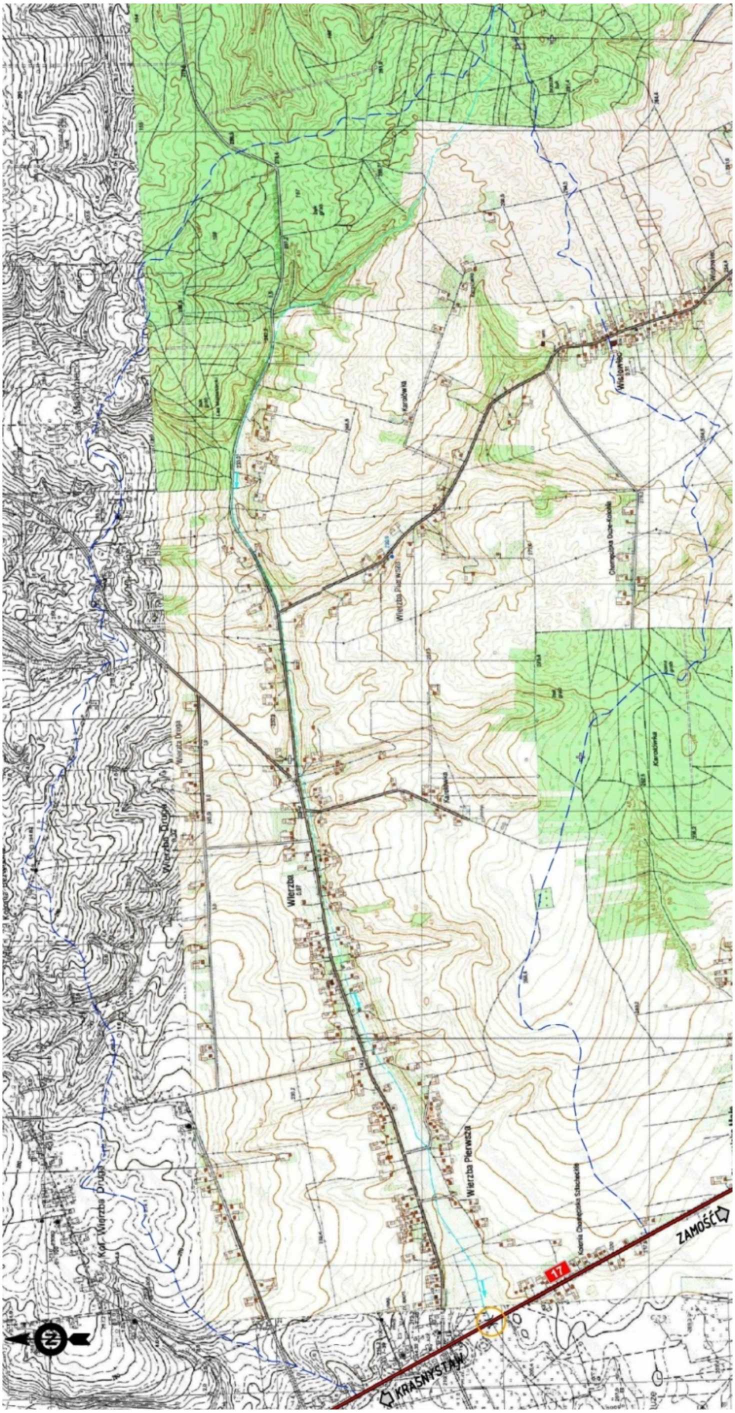


Fig. 2.8. Map of the river basin area

Calculation of the hydro-morphological characteristics of the river according to (2.13):

$$\Phi_r = \frac{1000 \cdot 6.48}{9 \cdot 8.7^{1/3} \cdot 11.75^{1/4} \cdot (0.55 \cdot 100)^{1/4}} = 69.5 .$$

Calculation of the hydro-morphological characteristics of the slope according to (2.14):

$$\Phi_s = \frac{(1000 \cdot 1.01)^{1/2}}{0.20 \cdot 8.7^{1/4} \cdot (0.55 \cdot 100)^{1/2}} = 12.47 .$$

Calculation of the maximum flow rate of the occurrence probability $p = 0.3\%$ according to (2.11):

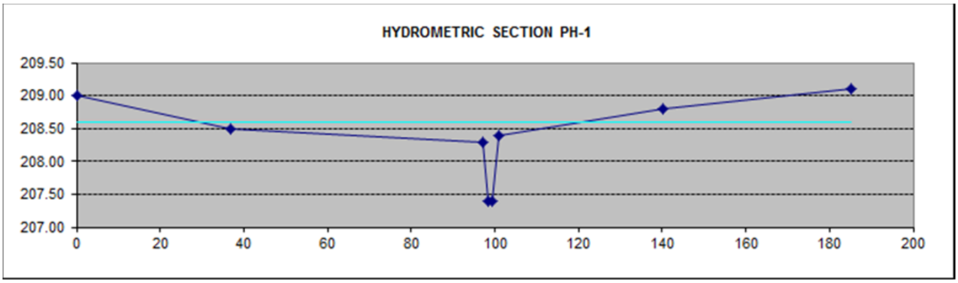
$$Q_p = 0.60 \cdot 0.025 \cdot 0.55 \cdot 100 \cdot 11.75 \cdot 1.31 \cdot 1.00 = 12.70\,\mathrm{m}^3/\mathrm{s}.$$

2.4.2. Permanent bridge hydraulic calculations

In order to determine the free surface of great water in the analysis of the cross section of a bridge, the calculations of the flow in hydrometric section. The average drop in the water table is accepted at $i = 0.004$.

Hydrometric section PH-1:

- 175 m above the axis of the analyzed bridge,
- high water level 208.59 m,
- undeveloped section,
- water depth 1.19 m.



Trough ordinates	209.00	208.50	208.30	207.40	207.40	208.40	208.80	209.10	Σ
Distance	–	37.00	60.00	1.30	1.00	1.80	39.00	45.00	–
Water level L	–	6.66	60.00	1.30	1.00	1.80	18.52	–	**89.28**
Bottom L (O_{zi})	–	6.66	60.00	1.58	1.00	2.06	18.53	–	**89.83**
Area (F_i)	–	0.30	11.40	0.96	1.19	1.24	1.76	–	**16.85**

	Lagoon left	Flume	La-goon right	Σ	
m	2	3	2	–	– the number of sections in different parts of the trough
n	0.035	0.03	0.035	–	– roughness coefficients in different parts of the trough
F	11.70	3.39	1.76	16.85	– cross-sectional area
O_z	66.66	4.64	18.53	–	– wetted perimeter
R_h	0.176	0.731	0.095	–	– hydraulic radius
v	0.566	1.711	0.376	0.78	– speed in different parts of the trough (and average speed)
Q	6.63	5.81	0.66	**13.10**	– flow in different parts of the trough

$$\Delta Q_{0.3\%} = \frac{12.70 - 13.10}{12.70} = -0.03 < 0.05.$$

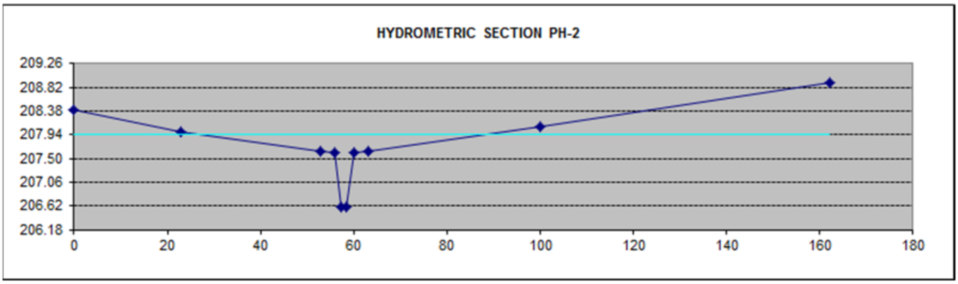
Level high water established properly.

<u>Hydrometric section PH-2:</u>

- 13 m above the axis of the analyzed bridge,
- high water level:

$$208.59 - \left[(175 - 13) \cdot 0.004\right] = 207.94 \text{ m},$$

- undeveloped section,
- water depth 1.34 m.



Trough ordinates	208.40	208.00	207.64	207.60	206.60	206.60	207.60	207.64	208.10	208.90	Σ
Distance	–	23.00	29.80	3.20	1.30	1.00	1.80	3.00	37.00	62.00	–
Water level L	–	–	25.08	3.20	1.30	1.00	1.80	3.00	24.37	–	**59.75**
Bottom L (O_{zi})	–	–	25.08	3.20	1.64	1.00	2.06	3.00	24.37	–	**60.36**
Area (F_i)	–	–	3.80	1.03	1.10	1.34	1.52	0.97	3.69	–	**13.45**

	Lagoon left	Flume	Lagoon right	Σ	
m	3	3	3	–	– the number of sections in different parts of the trough
n	0.035	0.03	0.035	–	– roughness coefficients in different parts of the trough
F	4.83	3.96	4.66	13.45	– cross-sectional area
O_z	28.28	4.70	27.37	–	– wetted perimeter
R_h	0.171	0.842	0.170	–	– hydraulic radius
v	0.556	1.880	0.555	0.945	– speed in different parts of the trough (and average speed)
Q	2.69	7.44	2.59	**12.71**	– flow in different parts of the trough

$$\Delta Q_{0.3\%} = \frac{12.71 - 12.70}{12.70} = 0.001 < 0.05.$$

High water level it was set properly. High water ordinate in the axis of a bridge:

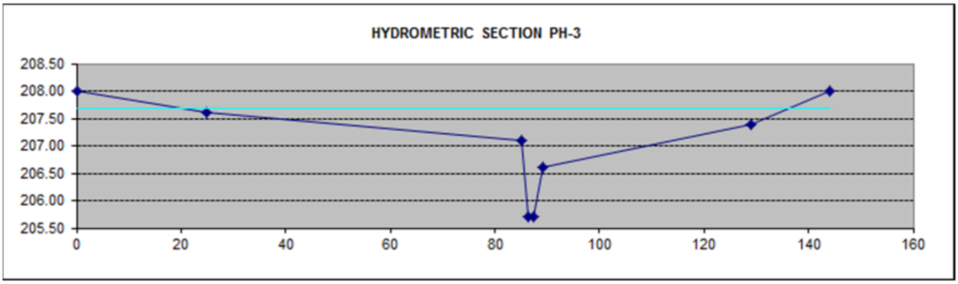
$$207.94 - 13 \cdot 0.004 = 207.89 \text{ m.}$$

Hydrometric section PH-3:

- 48.6 m below the axis of a analyzed bridge,
- high water level:

$$207.94 - \left[(13 + 48.6) \cdot 0.004\right] = 207.69 \text{ m,}$$

- undeveloped section,
- water depth 1.99 m.



Trough ordinates	208.00	207.60	207.10	205.70	205.70	206.60	207.40	208.00	Σ
Distance	–	25.00	60.00	1.30	1.00	1.80	40.00	15.00	–
Water level L	–	5.63	60.00	1.30	1.00	1.80	40.00	7.25	**116.98**
Bottom L (O_{zi})	–	5.63	60.00	1.91	1.00	2.01	40.01	7.26	**117.81**
Area (F_i)	–	0.25	20.40	1.68	1.99	2.77	27.60	1.05	**55.74**

	Lagoon left	Flume	Lagoon right	Σ	
m	2	3	2	–	– the number of sections in different parts of the trough
n	0.035	0.03	0.035	–	– roughness coefficients in different parts of the trough
F	20.65	6.44	28.65	55.74	– cross-sectional area
O_z	65.63	4.92	47.26	–	– wetted perimeter
R_h	0.315	1.308	0.606	–	– hydraulic radius
v	0.836	2.521	1.294	1.27	– speed in different parts of the trough (and average speed)
Q	17.27	16.24	37.08	**70.59**	– flow in different parts of the trough

$$\Delta Q_{0.3\%} = \frac{70.59 - 12.70}{12.70} = 4.55 > 0.05.$$

Section is not reliable due to the level of water – lowering the bottom of the watercourse about 50 cm. With the filling hydrometric section PH-3 has not been achieved clear high water level resulting from the filling the adjacent section – all sections characterized by different configurations of the land (most similar sections include PH-1 and PH-2).

2.4.3. Calculations of permanent bridge clearance

Bridge clearance bridge designated for a permanent bridge on the structure carrying the prefabricated beams of type $L_t = 11.30$ m connected supports to framework agreement (Fig. 2.9). The main technically usable parameters of the structure are:

- span of 11.30 m of the theoretical bridge,
- bridge clearance of 10.60 m,
- load-carrying structure height of 0.82 m,
- ordinate of the water level of 207.94 m.

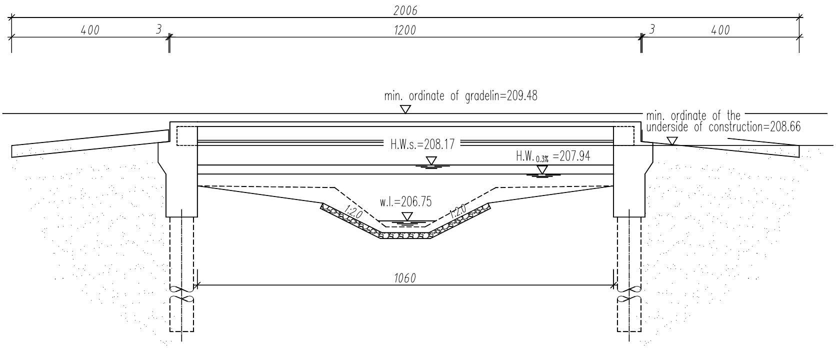


Fig. 2.9. View of the analyzed permanent bridge

Calculation of the flow rate under the bridge

The surface flow in the riverbed:

$$F_{\mathrm{M}} = 8.36\,\mathrm{m}^2 .$$

Average speed of movement:

$$v = \frac{Q_{\mathrm{m}}}{\mu \cdot F} = \frac{12.70}{0.91 \cdot 8.36} = 1.669\,\mathrm{m/s} .$$

Accumulation with no blur at the bottom of the bridge section

High water level in cross bridge is 207.94 m and decline in local watercourse $i = 0.004$. Results for the undeveloped section of the bridge summarized in the table.

Flood	**F**	**B$_o$**	**R$_h$**	**n**	**v**	**Q**
	[m^2]	[m]	[m]		[m/s]	[m^3/s]
Flood left	3.80 + 1.03	25.08	0.17	0.035	0.556	2.69
Trough main	3.96	10.30	0.84	0.030	1.880	7.44
Flood right	3.69 + 0.97	24.37	0.17	0.035	0.555	2.59
AMOUNT:	13.45	59.75		-	**0.945**	**12.71**

The flow in the undeveloped part of the trough corresponding to the cross-sectional area of the bridge:

$$Q_{\mathrm{s}} = 1.03 \cdot 0.556 + 3.96 \cdot 1.880 + 0.97 \cdot 0.555 = 8.55\,\mathrm{m}^3/\mathrm{s} .$$

Assumed strengthening of the bottom of the streambed riprap:

$$\mathrm{M} = \frac{8.55}{12.70} = 0.67 ,$$

$K_0 = 0.55$ – basic loss factor depends on the degree of the narrowing of the watercourse by abutments and their shape,

$\Delta K_{\mathrm{f}} = 0$ – amendment takes into account the impact of pillars,

$$e = 1 - \left(\frac{Q_{\mathrm{l}}}{Q_{\mathrm{p}}}\right) = 1 - \left(\frac{1.03 \cdot 0.556}{0.97 \cdot 0.555}\right) \approx 0 ,$$

$\Delta K\mathrm{e} = 0$ – amendment takes into account the impact of the asymmetrical narrowing of the watercourse $\varphi = 90°$,

$\Delta K\varphi = 0$ – amendment takes into account the impact of the oblique position of the bridge relative to the watercourse.

Saint-Venant's factor in the cross-section before the bridge:

$$\alpha_{\mathrm{o}} = 1.1 \cdot \frac{\left(v_{\mathrm{og}}^2 \cdot Q_{\mathrm{og}} + v_{\mathrm{oz}}^2 \cdot Q_{\mathrm{oz}}\right)}{v_{\mathrm{o}}^2 \cdot Q_{\mathrm{m}}} ,$$

$$Q_{og} = 7.44\,\mathrm{m^3/s},\ Q_{ozl} = 2.69\,\mathrm{m^3/s},\ Q_{ozp} = 2.59\,\mathrm{m^3/s},$$

$$v_{og} = 1.88\,\mathrm{m/s},\ v_{ozl} = 0.556\,\mathrm{m/s},\ v_{ozp} = 0.555\,\mathrm{m/s},$$

$$v_o = \frac{12.71}{4.83 + 3.96 + 4.66} = 0.945\,\mathrm{m/s},$$

$$\alpha_o = 1.1 \cdot \frac{1.88^2 \cdot 7.44 + 0.556^2 \cdot 2.69 + 0.555^2 \cdot 2.59}{0.945^2 \cdot 12.70} = 2.25.$$

Saint-Venant's factor in cross-section under the bridge:

$$\alpha = 1 + M \cdot (\alpha_o - 1) = 1 + 0.67 \cdot (2.25 - 1) = 1.84,$$

$$K = K_0 + \Delta K_f + \Delta K_e + \Delta K_\varphi = 0.55 + 0 + 0 + 0 = 0.55,$$

$$v = 1.766\,\mathrm{m/s},$$

$$B_0 = 59.75\,\mathrm{m}.$$

At first approximation, without taking into account differences in speed ν_s i ν_o:

$$\Delta z = \frac{0.55 \cdot (1.84 \cdot 1.669^2)}{2 \cdot 9.81} = 0.15\,\mathrm{m}.$$

This increases the surface before the bridge to:

$$F_s = 13.45 + (59.75 \cdot 0.15) = 22.41\,\mathrm{m^2},$$

$$v_s = \frac{12.70}{22.41} = 0.57\,\mathrm{m/s}.$$

Full height impoundments will be equal to:

$$\Delta z = 0.15 + \frac{2.25 \cdot (0.945^2 - 0.57^2)}{2 \cdot 9.81} = 0.22\,\mathrm{m}.$$

The level of the water table for high-dammed water:

$$207.94 + 0.22 = 208.16\,\mathrm{m}.$$

Minimum ordinate of the underside of the bridge structure:

$$208.16 + 0.50 = 208.66\,\mathrm{m}.$$

Construction depth with a pavement:

$$0.71 + 0.11 = 0.82\,\mathrm{m}.$$

Minimum ordinate of the vertical alignment on the bridge:

$$208.66 + 0.82 = 209.48\,\mathrm{m}.$$

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Chapter 3.

Basics of design

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3.1. Eurocodes and ULS, SLS

3.1.1. European standards

Designing structures in Poland should be based on:

- national Polish standards which, from 1 April 2010, have the status of withdrawn standards,
- Eurocodes which is a European system of rules and compatible with this principles of rules, because they were made in the Polish language version, including annexes national [11].

The use of Eurocodes is not mandatory, unless it results from separate regulations or a contract [10]. If one wants to apply the Eurocodes in the design process, one should apply the principles marked (P) denoting the general principles, definitions, requirements and computational models for which there is no alternative. Derogations are permitted from those listed in the Eurocodes rules, which are marked with numbers in parentheses, for example. (2) and comply with the following rules. Derogation from the rule is acceptable, provided the documentation of compliance with the applicable rules of replacement rules is available.

There are five basic groups of documents in the basic system of the Eurocodes:

- GROUP 1: EN 1990 with appendices giving common rules and principles of structural design (including limit states, the values of safety factors, rules and methods for determining the combination of actions, etc.) [4].

The Eurocodes generally do not distinguish between conceptually different types of bridges, i.e. overpasses, bridges and overpasses, simply calling them all bridges. The Eurocodes establish principles for the design of road bridges, footbridges and railway bridges. There are no established rules and principles of design, e.g. for road and railway bridges, channel or airport, as well as road and tram, and road and railway bridges. General principles and rules provided in the Eurocodes may be, however, (after an appropriate adaptation) used in the design of these types of unusual bridges.

- GROUP 2: EN 1991 standards describing models and giving the affect on structures and the principles and rules for their use [5].

The concept of „impact" broadly describes the effect of force to which a bridge structure is subject. „Affect" also refers to an impact directly and commonly referred to as „load” (e.g. a crowd of pedestrians or dead weight) and intermediate termed an „affect” (e.g. an impact of the thermal environment on the structure of an object which gives rise to distortion which in turn causes or invokes internal forces in the structure of the object).

- GROUP 3: EN 1992.3 ... 6.9 contains the rules and principles of structural design, taking into account types of construction materials (i.e. concrete, steel, composite, massive and/or aluminium) [6], [7], [8], [9].
- GROUP 4: EN 1997 which contains principles and rules concerning the design of geotechnical structures or parts thereof (foundation) [3] and
- GROUP 5: EN 1998 which contains principles and rules of the design of structures exposed to seismic activity [2].

The fundamental system standards of Eurocodes accompanied by a lot of standards regarding materials and workmanship from the point of view of the principles and objectives of the system are equally important because of the need to ensure the quality required by the above mentioned fundamental standards.

In terms of impacts on bridges the important standards groups are EN 1990, EN 1991and EN 1997 and with regard to locations exposed to seismic activity also EN 1998 [1], [2], [3].

3.1.2. Limit states (ULS and SLS)

Eurocodes recommend the use of safety factors methods, but also allow the use of probabilistic methods. A check requires two limit states: ultimate (ULS) and serviceability (SLS). Limit states should be related to design situations which are divided into:

- persistent design situations, which refer to the conditions of normal use;
- transient design situations, which refer to temporary conditions applicable to a structure, e.g. during execution or repair;
- accidental design situations which refer to exceptional conditions applicable to a structure or to its exposure, e.g. to fire, explosion, impact or the consequences of a localized failure;
- seismic design situations which refer to the conditions applicable to the structure when subjected to seismic events.

Serviceability limit states (SLS) are features in the structure or any element thereof under the normal conditions of use, user comfort and appearance of a building. One distinguishes between reversible and irreversible serviceability limit states. The criteria to meet in the case of serviceability limit state usually refer to deflection (affecting the appearance, comfort of users and / or function of structures), vibration (causing discomfort to people or restricting the usefulness of a structure) and damage (affecting the appearance, durability or the functioning of structures).

The general condition for serviceability limit state is formulated as [12]:

$$E_{\mathrm{d}} \le C_{\mathrm{d}}, \tag{3.1}$$

where E_d is the value of the effect of influences such as internal force, and C_d is the appropriate limit value criterion adopted to serviceability.

The ultimate limit states (ULS) relate to the structural safety and/or people, being treated as a state of disaster or state immediately preceding the disaster, e.g.:

- destruction/loss of the load capacity manifested by excessive strain (STR/GEO) or fatigue (FAT),
- loss of structural stability as manifested by the loss of equilibrium structure or any part thereof regarded as a rigid body (EQU), or the loss of stability of a structure or a part of it, including supports and foundations (UPL and HYD – according to EN 1997).

Eurocodes do not define the exact scope of verification in each case of state limits, but only describe the main models of structural damage:

a) EQU: the loss of static equilibrium by a structure or a portion thereof taken as a rigid body when a small change in the value or distribution of impact coming from a single source has significant consequences for the structure –in this case the load/strength of the material or substrate is not analysed. In the case of geotechnical structures it is usually a quite complex failure mode.
b) STR: exceeding the load/strength of materials or an excessive deformation of a structure or any element thereof (also applies to footings, piles);
c) GEO: a failure or excessive deformation of the ground which plays a decisive role in ensuring bearing capacity (exceeding geotechnical carrying capacity);
d) FAT: the fatigue failure of a structure or any element thereof.

Eurocodes includes two additional models of structural damage associated with EN 1997. These are:

- UPL associated with loss of balance design, an element or the ground due to a displacement and/or vertical external forces and
- HYD associated with hydraulic tilted structures immersed in water (groundwater), erosion or puncture caused by hydraulic mirrors difference in the water levels.

The general condition of the ultimate limit state has the following form:

- when checking the load capacity (STR/GEO):

$$E_d \leq R_d, \tag{3.2}$$

where E_d is the value of calculating the effect of influences such as internal force, and R_d is the design value of the corresponding load capacity,

- when checking the static balance (EQU):

$$E_{d,dst} \leq E_{d,stb}, \tag{3.3}$$

where $E_{d,dst}$ is the design value of the effect of destabilizing interactions, and $E_{d,stb}$ is the design value of the effect of stabilizing interactions.

E_d interaction effect is a function of computing F_d interaction, possibly a computational geometric size of the a_d and the computational model:

$$E_d = \gamma_{Sd} \{F_d; a_d\} \text{ i} \geq 1, \tag{3.4}$$

where γ_{Sd} is a partial factor taking into account the uncertainty effects of the model of interactions and, in some cases, modelling interactions (factor model).

An interaction is determined according to:

$$F_d = \gamma_f \cdot F_{rep}, \tag{3.5}$$

where γ_f is a partial factor for the interaction taking into account the possibility of unfavourable deviations from the impact of a representative value F_{rep}.

A representative value of impact can be determined:

$$F_{rep} = \psi \cdot F_k, \tag{3.6}$$

where F_k is the characteristic impact, and ψ is the coefficient of the combination of a host of 1.0, ψ_0, ψ_1 or ψ_2.

3.1.3. Combination of actions

The effects of interactions in the form of displacements, deformations or internal forces in the structure are determined for the combination of actions, appropriate to the state border and the situation of calculation. In any case, one should only use these interactions that may occur at the same time. The effects of impacts that cannot occur simultaneously for physical or functional reasons should not be taken into account simultaneously in combinations of interactions (Fig. 3.1).

The symbols used in Fig. 3.1 and the formulas 3.7÷3.14 denote:

$G_{k,j}$ – characteristic value for permanent action j,
$\gamma_{G,j}$ – partial factor for permanent action j,
$Q_{k,1}$ – characteristic value of the leading variable action,
$\gamma_{Q,i}$ – partial factor for variable action,
P – relevant representative value of a prestressing action,
γ_p – partial safty factor for prestressing actions,
ξ – reduction factor for unfavourable permanent actions (it is recommended to take value 0.85),
A_d – design value of an accidental action,
ψ – factors for the combination value of a representative action,
”+” – implies „to be combined with”,
Σ – implies „the combined effect of”.

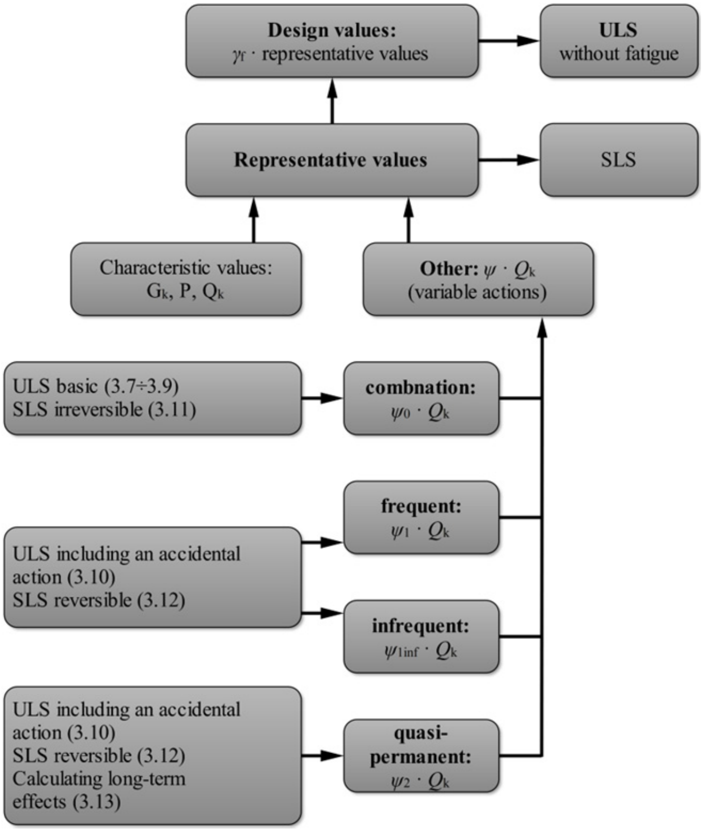

Fig. 3.1. Diagram used to determine the value of the interactions in combinations

In all the analyzed ultimate limit states (STR, GEO and EQU) a combination of general primary measures (needed for consideration for permanent and temporary design situations) was applied in the following form:

$$\sum_{j\geq 1} \gamma_{G,j} \cdot G_{k,j} \text{"+"} \gamma_p \cdot P + \gamma_{Q,1} \cdot Q_{k,1} \text{"+"} \sum_{i>1} \gamma_{Q,i} \cdot \Psi_{0,i} \cdot Q_{k,i}. \tag{3.7}$$

Alternatively, but only in the case of ultimate limit states STR and GEO, it is necessary to check the less favourable of the two expressions shown below (also applicable in the case of permanent and temporary design situations):

$$\sum_{j\geq 1} \gamma_{G,j} \cdot G_{k,j} \text{"+"} \gamma_p \cdot P + \gamma_{Q,1} \cdot \Psi_{0,1} \cdot Q_{k,1} \text{"+"} \sum_{i>1} \gamma_{Q,i} \cdot \Psi_{0,i} \cdot Q_{k,i}, \tag{3.8}$$

$$\sum_{j\geq 1} \xi_j \cdot \gamma_{G,j} \cdot G_{k,j} \text{"+"} \gamma_p \cdot P + \gamma_{Q,1} \cdot Q_{k,1} \text{"+"} \sum_{i>1} \gamma_{Q,i} \cdot \Psi_{0,i} \cdot Q_{k,i}. \tag{3.9}$$

The combination of impacts in the event of exceptional design situations can be written as:

$$\sum_{j\geq 1} G_{k,j} "+" P + (\Psi_{1,1} \text{ lub } \Psi_{2,1}) \cdot Q_{k,1} "+" \sum_{i>1} \Psi_{2,i} \cdot Q_{k,i}. \tag{3.10}$$

When calculating each combination the leading variable action or the impact of exceptional action should be determined. As a leading variable action, index 1 should be adopted for the impact causing the most adverse effect. In the case of moving loads they will be called "group moving loads" (discussed in Section 3.2.3). One determines thus which of the variable loads can be considered simultaneously in the static analysis (Tab. 3.1).

Tab. 3.1. Groups of traffic loads on roads bridges [14]

Groups of loads	Leading variable action	Accompanying variable actions
1a	LM 1 (characteristic values)	q_{fk} (characteristic values, recommended 3 kN/m^2)
1b	LM 2 (characteristic values)	–
2	Horizontal forces: Q_{1k} and Q_{tk} (characteristic values)	LM 1 (frequent value)
3	q_{fk} (characteristic values)	–
4	LM 4 (characteristic values)	q_{fk} (characteristic values)
5	LM 3 (characteristic values)	LM 1 (frequent value)

When the group moving loads occur as influence of covariates, one must apply one value ψ to the whole group, taking the value ψ as used for the main component of the group.

In turn, the combinations of actions in serviceability limit state have the following forms:

a) characteristic combination – usually for irreversible SLS

$$\sum_{j\geq 1} G_{k,j} "+" P "+" Q_{k,1} "+" \sum_{i>1} \Psi_{0,i} \cdot Q_{k,i}, \tag{3.11}$$

b) frequent combination – usually for reversible SLS

$$\sum_{j\geq 1} G_{k,j} "+" P "+" \Psi_{1,1} \cdot Q_{k,1} "+" \sum_{i>1} \Psi_{2,i} \cdot Q_{k,i}, \tag{3.12}$$

c) quasi-permanent combination – usually for evaluating the long-term effects and appearance of structures (e.g. heavy scratches, deflection)

$$\sum_{j\geq 1} G_{k,j} "+" P "+" \sum_{i>1} \Psi_{2,i} \cdot Q_{k,i}. \tag{3.13}$$

This method of determining the load combination does not apply to the calculation of the effects of fatigue. Load combinations included in the calculation of fatigue give marketing standards for dimensioning a specific type of construction, for example of concrete (EN 1992) or steel (EN 1993). For example, in the EN 1992 in order to calculate the stress range, relevant for calculating fatigue, one should divide the load into acyclic and cyclic, so those that cause fatigue. The cyclical impact should be considered in connection with the unfavourable impact of a combination of fundamental and cyclical impact, and can be expressed as:

$$\left(\sum_{j\geq 1} G_{k,j} "+" P "+" \Psi_{1,1} \cdot Q_{k,1} "+" \sum_{i>1} \Psi_{2,i} \cdot Q_{k,i} \right) "+" Q_{fat}, \tag{3.14}$$

where:
Q_k – non cyclical impact of variables,
Q_{fat} – fatigue loading of vehicles set out in EN 1991.

3.1.4. Representative values and design values of actions

The starting point for determining the value of the effects of actions in various combinations are representative values of actions. Representative values are used directly in combinations of interactions to check the serviceability limit state and the limit state of load capacity in emergency situations.

The main value is the representative characteristic value (G_k, P, Q_k). In the case of variable actions there are other representative values (combination, frequent, infrequent, quasi-permanent), which are expressed by the product of the characteristic value Q_k a suitable factor $\psi \leq 1.0$ (Fig. 3.1). In the case of bridges separate collections of ψ factors for road bridges, footbridges and railway bridges have been set up.

In the calculations of fatigue and the dynamic analysis of structures additional representative values are used. The fatigue calculations, different load models than in static analyses (both in calculations of road bridges and rail) and fatigue assessment are based on checking the stress range in accordance with EN 1992, EN-1993 and EN-1994.

The characteristic value of any impact given in EN can be determined as:

- average value,
- the top value (or bottom) of an assumed probability (return period) that it is not exceeded (or that does not appear to have a lesser value)
- the nominal value which is not determined statistically but arbitrarily, based on the tradition or other experience.

In the case of permanent actions (G) shall be:

- for permanent actions of low volatility – one characteristic value G_k in the form of an average,
- for permanent actions of greater volatility – two characteristic values determined statistically or nominally: lower G_{kinf} (as quantile 5% of statistical distribution size G) and a top G_{ksup} (quantile 95%).

In the case of bridges, two characteristic values are recommended for the following permanent loads:

- breakstone ballast on railway bridges,
- waterproofing, pavement and other layers of bridge roofing,
- cables, pipelines and pass inspection.

In determining the effects of the standard representative values of variables an important role plays the reference period. It is a fixed length of time taken as a basis for determining random variable actions.

Q_k characteristic value of the variable affect can be top, bottom or nominal value. For example, the characteristic values in road load model LM1 were calibrated for a 1000-year return period (or the probability of exceeding 5% in 50 years), traffic on the main roads of Europe. In turn, the crowd assumed load nominal values.

Combining value of the variable affect takes account of the reduced probability of simultaneous occurrence of the most adverse of several measures. It is determined statistically, as far as possible, so that the probability that the effect of the combination (loaded $\psi_o \cdot \gamma_F \cdot Q_k$) is exceeded, is approximately the same as in the case of effects of single $\gamma_F \cdot Q_k$. ψ_o factors which are used only for checking the fundamental ultimate limit state (ULS) and the so-called irreversible serviceability limit states (SLS).

Tab. 3.2. Factors ψ relating to the loads of road bridges [12]

Groups of loads		Combination value	Frequent value	Quasi-permanent value
		ψ_0	ψ_1	ψ_2
gr1a	TS	0.75	0.75	0
	UDL	0.40	0.40	0
	Pedestrian+cycle-track loads	0.40	0.40	0
gr1b		0	0.75	0
gr2		0	0	0
gr3		0	0	0
gr4		0	0.75	0
gr5		0	0	0

Frequent value ψ_1 Q_k of variable effects is determined – if possible statistically – in such a way that:

- the period during which this value is exceeded, represent a rather large part of the period of reference, for example 0.05,
- the frequency of exceeding the value for the reference period was limited to a specified value.

Factor ψ_1 is used for checking the ULS taking into account exceptional burdens and so-called reversible SLS.

Quasi-permanent value $\psi_2 \cdot Q_k$ affects variable is calculated that the period in which it is exceeded constitutes a substantial part of the reference period, for example 0.5. Factors ψ_2 are used for checking the ULS taking into account the exceptional and reversible load SLS and in determining long-term effects.

Recommended factors ψ, relating to the loads of road bridges, are summarized in Tab. 3.2.

Tab. 3.3. Calculation factors γ permanent and variable loads on road bridges, in combinations of basic [12]

Equation	Permanent actions[5]		Prestress	Leading variable action	Accompanying variable actions	
	unfavourable	favourable			main	others
	$\gamma_{Gj,sup}$	$\gamma_{Gj,inf}$	γ_P	$\gamma_{Q,1}$	$\gamma_{Q,i}$	$\gamma_{Q,i}$
			Set B			
3.7	1.35	1.00	1.00 1.20 [3] 0.80	1.35[1] 1.50[2]		1.35[1] 1.50[2]
3.8	1.35	1.00	1.00 1.20 [3] 0.80		1.35[1] 1.50[2]	1.35[1] 1.50[2]
3.9	1.35[4]	1.00	1.00 1.20 [3] 0.80	1.35[1] 1.50[2]		1.35[1] 1.50[2]
			Set C			
3.7	1.00	1.00	1.00 1.20 [3] 0.80	1.15[1] 1.50[2]		1.15[1] 1.50[2]

[1] applies to vehicles and pedestrians (cyclists),
[2] applies to other movable interactions or other variable actions,
[3] 1.00 during prestressing; 0.80 and 1.20 during used [15],
[4] to the loads of this group applies reduction factor ξ,
[5] characteristic values of all permanent actions from one source are multiplied by $\gamma_{Gj,sup}$ if the total resulting action effect is unfavourable and $\gamma_{Gj,inf}$ if the total resulting action effect is favourable.

In ULS (except fatigue design values of impacts) are used. They are obtained by multiplying a value representative of the partial safety factor γ. In general, this ratio only takes into account the possibility of adverse deviation the value of a affects on the representative value. Then it is called a load factor and stands for γ_f. Sometimes, a partial safety factor also takes into account the uncertainty of the calculation model and the dimensional change – designation γ_F. According to the EC, the same type of interactions can take various design values depending on the type of ULS and depending on how the impact on the relevant size of the static. For bridges three sets (A, B, C) of partial factors γ have been identified. Set A is used to check the state of static equilibrium (EQU), set B to test for these states of STR of those parts of the structure which are not affected by geotechnical factors. STR states of footings, piles, pillars, walls and front wings abutments etc. which require that a geotechnical interactions and limit states on the bearing capacity (GEO) be checked using sets B and C.

Examples of load factors γ recommended for bridges are given in Tab. 3.3. Empty fields in the table correspond to the absence of relevant components of load combinations in the tables in Eurocode 0.

Every interaction should be constant throughout the structure represented by a single computing producing unfavourable effect – top or bottom. So, for example, a continuous beam of the same design value as the weight of the structure can be applied to all spans independently of the influence lines. In practice, this means that the entire structure can be ordered once the weight of G_{inf} and once the value G_{sup} and to analyses load capacity accept more unfavourable value. Exceptions to this rule apply when calculation results can be strongly influenced by changes in the size of the permanent load with a change in its place of action. This applies especially when checking a static equilibrium structure treated as a rigid body. In this case, the destabilizing load (unfavourable) should be represented by its top design values and load stabilizers (favourable) – by their lower design values (e.g. specific sections of continuous beam respectively burden G_{inf} value or G_{sup}).

3.2. Bridge loads

3.2.1. Introduction

Bridges are influenced by dead loads and variable loads. Dead loads include: structure weight, earth pressure and equipment weight. Variable loads include: wind pressure, the ice flow pressure, temperature changes, rheological agents, friction and service loads (traffic, tram, rail, pedestrian crowd). Dead loads are relatively easy to define, by means of geometry, the knowledge of materials and assumed constancy over time. Variable loads are much more difficult to calculate, because different vehicles can move over the bridge, with a time varying pattern of movement, weight, and strength.

The loads acting on bridges are classified according to three criteria:

- the occurrence of a function of time and place (dead and variable),
- mode of operation (static, dynamic),
- parts of a bridge directly affected by these loads (supports, span, concentrations, equipment etc.).

Another criterion for load classification might be the impact of a load on the calculated element, safety, and durability of an object or its component. According to [16], loads can be divided into basic, additional and unique ones. A direct consequence of this division are the values of assumed load factors assumed when checking the ultimate limit state, depending on the combination admitted to calculate loads.

Basic loads determine the sustainability efforts of an object or its parts exerting the greatest impact on operational safety. Basic loads include all dead loads and the variables loads.

Additional loads include variable load that acts simultaneously with dead loads and whose transfer is not the aim of the object or its part. Unique loads are variable loads exceeding the values specified by the standards or used under conditions different from the normal operation of a facility or its part.

The same weight may be basic, additional or unique, depending on the situation and the importance of the load of a calculated structure element.

The bridge deck dominant load is service load (traffic or a crowd of pedestrians). Weight has a negligible share in the internal forces of a deck. In addition to its own weight, load consists of the weight of equipment (insulation, surfacing, barriers, railings, paving covers etc.).

The basic load of main girders is weight derived primarily from its own weight, the weight of a deck and the weight of equipment and variable load (traffic, pedestrian, railway or tramway), depending on the destination object. Variable loads which affect a structure can also be thermal loads or the wind load.

Loads acting on abutments are their own weight, reactions from the span, the ground load, variable load acting on the wings, dead and variable load on an embankment located on the wedge.

Pillars (intermediate supports) are loaded by their weight, load reactions from the span, ground load, the wind load. In the case of river pillars one also should take into account the ice floe pressure and buoyancy of water, lateral hitting by ships. In the case of the pillars of overpasses and flyovers one must take into account the vehicle lateral impact.

In [14], there are two notions of moving loads. The variable loads and unique loads. There are models for strength calculations and to check for fatigue. Load values in different models are divided into characteristic, infrequent, frequent and quasi-static. The last three values are computable, i.e. load factor $\gamma \neq 1$. Unique load should be considered in the absence of a safety hence for in entering the vehicle on the pavement or hitting support. Unique loads are calculated with a coefficient $\gamma = 1$.

3.2.2. Permanent loads

Permanent loads of bridges and any other kinds of civil engineering structures should be determined in accordance with the standard [13].

The dead load includes structural and non-structural elements, together with the devices and the weight of the earth and ballast. Following the recommendations of the standard, load combinations must take into account permanent loads as a single interaction.

The dead loads of the structure should be presented with a single characteristic value (g_k), calculated on the basis of the nominal dimensions and the characteristic weight densities of materials. If the dead load may vary over time, it is recommended to take it into account as the upper and lower characteristic values, respectively: $g_{k,sup}$ and $g_{k,inf}$ (these values can be provided in an appendix in the national annex). In the case of road bridges, this applies to dead loads of the layers of waterproofing, pavement and other layers of bridge covering where their thickness variation may be significant. In the absence of the national annex, it should be assumed that the deviation of the total thickness from the nominal value (or other specified value) may be equal to ± 20% where the nominal value includes the as-built coverage and +40% and -20% where such a coverage is not included. Additionally, the deviation of ±20% from the nominal value should also be taken into account when considering the dead loads of cables, pipelines and inspection passages. For dead loads of non-structural elements, such as railings, barriers, kerbs and secondary elements, it is recommended to assume them as equal to the nominal values (in the absence of the national annex).

Tab. 3.4. Nominal density of construction materials [13]

Materials	Density [kN/m^3]
Concrete	24.0 + 1.0$^{(1)}$ + 1.0$^{(2)}$
Granite elements	27.0÷30.0
Seel	77.0÷78.5
Gussasphalt and asphaltic concrete	24.0÷25.0
Mastic asphalt	18.0÷22.0
Hot rolled asphalt	23.0

$^{(1)}$ for normal percentage of reinforcing and prestressing steel,
$^{(2)}$ for unhardened concrete.

The nominal weight densities of materials most commonly used for the construction of road bridges are presented in Tab. 3.4.

3.2.3. Road bridge load scheme

Load models discussed in [14] relate to road bridges with a span length of less than 200 m, and a carriageway no wider than 42 m.

A road bridge carriageway should be divided into conventional lanes (Fig. 3.2), their width and number are assumed on the basis of Tab. 3.5.

Tab. 3.5. Number and width of notional lane [14]

Carriageway width w	Number of notional lanes n_l	Width of a notional lane w_l	Width of the remaining area
$w < 5.4$ m	$n_l = 1$	3 m	$w - 3$ m
$5.4\text{ m} \leq w < 6$ m	$n_l = 2$	w/2	0
$6\text{ m} \leq w$	$n_l = Int(w/3)$	3 m	$w - 3 \cdot n_l$

If the carriageway on a bridge is physically divided into two parts in a sustainable manner, the number of lanes is determined separately for each part, while in the case of a separable road barriers treated as a whole, emergency lanes and a paved shoulder are included. Notional lanes are assigned a load scheme and load value. The lane number does not have to match their order on a bridge. If two carriageways are located on one bridge deck, even if they are permanently separated, the lane number can appear only once. If platforms are based on separate supports numbering is conducted independently. If independent platforms are based on the same support, the support is used in calculating numbered lanes as with two carriageways for a single platform.

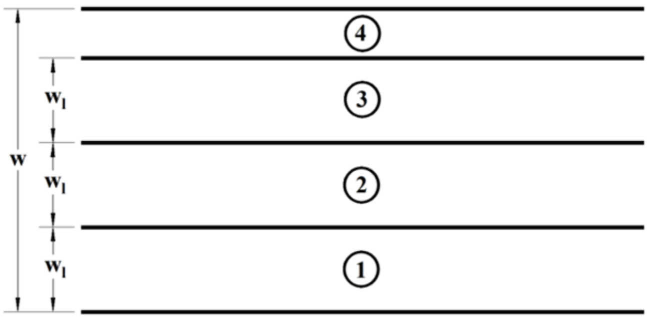

Fig. 3.2. Example of the lane numbering in the most general case: w – carriageway, wl – notional lane width, 1 – notional lane Lane Nr. 1, 2 – notional lane Lane Nr. 2, 3 – notional lane Lane Nr. 3, 4 – remaining area

Load model 1 is the main load. It consists of concentrated loads of weight per axle $\alpha_i Q_{ik}$ and evenly distributed on the $\alpha_i q_{ik}$ (Fig. 3.3). The load value of notional lanes and the remaining area q_{ik}, Q_{ik} are given in Tab. 3.5. The correction coefficients may take different values depending on the road class or expected traffic. In the absence of specification, the factors to be taken equal to unity. Load model 1 includes a dynamic factor.

UDL system is a uniformly distributed load. TS system is a tandem system in which there are 4 concentrated forces, the two forces on the axle. Forces are spaced at a distance of 2 m in the transverse direction, and axes at a distance of 1.2 m in the longitudinal direction. Forces focused on the lanes adjacent to each other contract terms should not be placed closer than 0.50 m. The tandem system TS should have their axes of symmetry in accordance with notional lanes.

Tab. 3.6. Load model 1: characteristic values [14]

Location	Tandem system TS	UDL system
	Axle loads Q_{ik} [kN]	q_{ik} (or q_{rk}) [kN/m^2]
Lane number 1	300	9.0
Lane number 2	200	2.5
Lane number 3	100	2.5
Other lanes	0	2.5
Remaining area (q_{rk})	0	2.5

In the case of bridge spans greater than 10 m, each tandem system is being replaced on each lane contractual focused uniaxial load with a value equal to the total load of the two axes, ie.: 600 kN Lane No.1, 400 kN Lane No.2 and 200 kN Lane No.3.

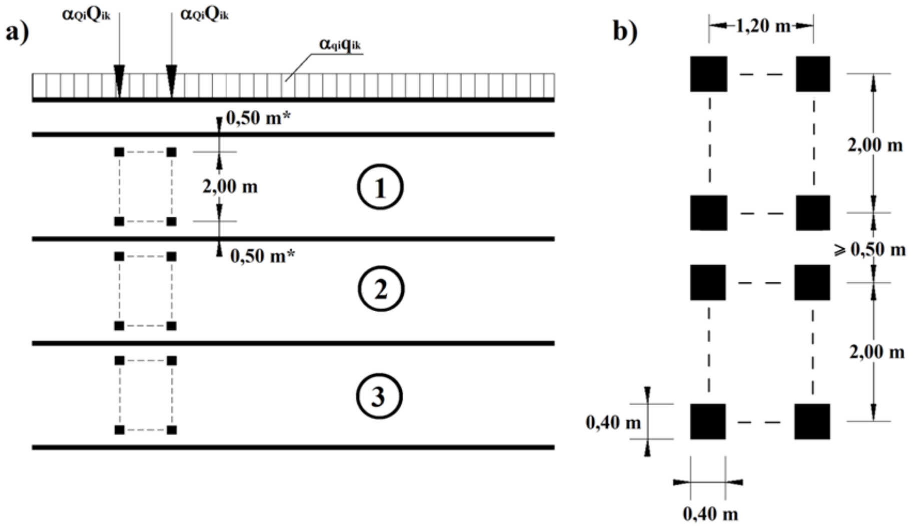

Fig. 3.3. a) Application of load Model 1, b) application of tandem systems for local verifications: (1) Lane Nr. 1: Q_{1k} = 300 kN, q_{1k} = 9 kN/m^2, (2) Lane Nr. 2: Q_{2k} = 300 kN, q_{2k} = 2.5 kN/m^2, (3) Lane Nr. 3: Q_{3k} = 300 kN, q_{3k} = 2.5 kN/m^2, * For w_l = 3.00 m

Load model 2 is a single-axle applied to the contact surfaces special tyres, used to calculate very short structural elements (Fig. 3.4). This model is used regardless of the Load model 1 and should be used for local checks. It consists of a single axis $\beta_Q Q_{ak}$ in which Q_{ak} load value is 400 kN and, similarly to Load model 1, includes a dynamic factor. The value of the β_Q shall be equal to the values α_{Q1}. The load is adjusted to anywhere in the carriageway, so as to obtain the extreme value of the internal forces. If it is more unfavourable in the calculations can be given to a single wheel with pressure $200\beta_Q$.

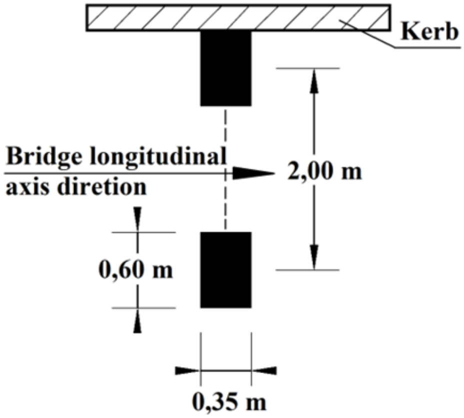

Fig. 3.4. Load Model 2

Load model 3 is a collection of axles showing special vehicles that can be incorporated into the traffic on the roads under road administrator authority. It is used only on request. There are eight classes of special vehicle weights of 600÷3600 kN, and in some cases there are "subclasses" varying in the axis number and layout. Detailed descriptions of all types of vehicles are given in the annex to [14]. The number of models under consideration may vary and depends only on the customer. Vehicles are set on a single lane (1) load (model pressures of 150 kN and 200 kN per axis) or in two adjacent lanes (lane 1 and 2), the load (models with higher weight axis). The load included without a dynamic factor when travelling at a speed no greater than 5 km/h. If a lane or lanes occupied by a special vehicle, the Load model 1 is set at a distance of no less than 25 m from the extreme axis of the vehicle. Load model 3 is taken into account in the case of a temporary design situation.

Load model 4 refers to a crowd of pedestrians. The model is applied on request. It consists of a uniformly distributed load with the value of 5 kPa. Load model 4 is taken into account in the case of a temporary design situation.

Load models 1 and 2 are generally used in design and Load models 3 and 4 only in certain design situations.

Braking forces Q_{lk} (acceleration), limited to 900 kN for the total width of a bridge, should be calculated as a fraction of the total maximum vertical loads corresponding to the Load Model 1 and likely to be applied on Lane Number 1 as follows:

$$Q_{\mathrm{lk}} = 0.6 \cdot \alpha_{\mathrm{Q1}} \left(2 \cdot Q_{\mathrm{1k}}\right) + 0.1 \cdot \alpha_{\mathrm{Q1}} \cdot q_1 \cdot w_1 \cdot L,$$
$$180 \cdot \alpha_{\mathrm{Q1}} \mathrm{kN} \le Q_{\mathrm{lk}} \le 900\mathrm{kN}, \tag{3.15}$$

where L is the length of the deck or of the part of it under consideration.

The centrifugal force Q_{tk} should be taken as a transverse force acting at the finished carriageway level and radially to the axis of the carriageway, it should be taken from Tab. 3.7.

Tab. 3.7. Characteristic values of centrifugal forces [14]

$Q_{lk} = 0.2Q_v[kN]$	if r < 200 m
$Q_{lk} = 40Q_v/r[kN]$	if 200 ≤ r ≤ 1500 m
$Q_{lk} = 0$	if r ≥ 1500 m

r – is the horizontal radius of the carriageway centerline [m],
Q_{v} – is total maximum weight of vertical concentrated loads of the tandem system of LM1, ie. $\Sigma\alpha_{Qi}(2Q_{ik})$.

Load groups. Simultaneity of the occurrence of specified system loads is taken into account when considering groups of loads. Provides for 5 groups loads that are a combination of different model loads and additional forces of braking and centrifugal forces. Each of these groups of loads, mutually exclusive, should be regarded as a defining characteristic of the impact of the combination of stationary loads.

In addition to the models used to determine the internal forces and deformation of the structure, also special fatigue models have been introduced. There is are five models consisting of vertical forces.

Additionally, bridges must be checked with regard to a special vehicle load according to the NATO standardization agreement (STANAG 2021).

3.2.4. Footbridge and sidewalk load

[14] introduces a model for the calculation of footbridges. The basic normative load is an evenly distributed load on the q_{tk} = 5 kN/m^2, but in the case of footbridge spans, various spans of over 10 m, one should consider the following values:

$$2.5\,\mathrm{kN/m^2} \le q_{\mathrm{tk}} = 2.0 + \frac{120}{L_{\mathrm{v}} + 30} \le 5.0\,\mathrm{kN/m^2}, \tag{3.16}$$

where L_v is the span length [m].

In the case of a road with sidewalks or bicycle paths only the value 5 kN/m^2 should be taken into account. The load including a crowd of pedestrians in combination with other loads is taken as the 2.5 kN/m^2.

Footbridges may be checked to special vehicles load.

3.2.5. Railway bridge load schemes

[14] shows the five models load. The first one shows normal traffic on main lines (Load Model 71 – MO71 – Fig. 3.5a), another two represent an unusual heavy burden (Load Models SW – Fig. 3.5b), the fourth one shows the load of passenger trains at speeds exceeding 200 km/h (Load Models HSLM). The last load model refers to "the train without cargo" showing the effect of an unloaded train used in some cases. The scheme for this train is uniformly distributed load of 12.5 kN/m.

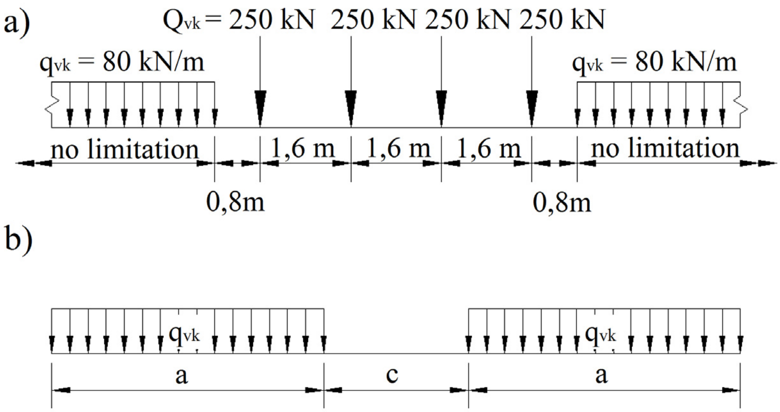

Fig. 3.5. a) Load Model 71 and characteristic values for vertical loads, b) Load Models SW/0 and SW/2

The load values given in MO71 are multiplied by factor α, depending on the category of a railway line. Load multiplied by coefficient α are called "vertical loads classified". The values of α may be of 0.75 – 0.83 – 0.91 – 1.00 – 1.10 – 1.21 – 1.33 – 1.46. The load models SW are described in Tab. 3.8.

Tab. 3.8. Characteristic values for vertical loads for Load Models SW/0 and SW/2 [14]

Load model	q_{vk}	a	c
	[kN/m]	[m]	[m]
SW/0	133	15.0	5.3
SW/2	150	25.0	7.0

Railway objects should be also checked for a side impact of trains on rails, acceleration and deceleration forces and the centrifugal forces if the object is curved in the plan.

Side impact strength is taken as a concentrated force acting on the rail head, perpendicularly to the axis of the track, with value of 100 kN (Q_{sk}). The strength of acceleration is equal to:

$$Q_{\text{lak}} = 33\,[\text{kN/m}] \cdot L\,[\text{m}] \leq 1000\,\text{kN}, \tag{3.17}$$

for MO71, SW/0, SW/2 and HSLM. The braking force is equal to:

$$Q_{\text{lbk}} = 20\,[\text{kN/m}] \cdot L\,[\text{m}] \leq 600\,\text{kN}, \tag{3.18}$$

for MO71, SW/0, SW/2 and HSLM and:

$$Q_{\text{lbk}} = 35\,[\text{kN/m}] \cdot L\,[\text{m}]. \tag{3.19}$$

in the case of SW/2. $L_{a,b}$ is the length of the impact line.

3.2.6. Other loads

Other loads of bridges that need to be considered when designing a bridge are loads caused by temperature changes, the wind load, the ice floe pressure, the load bearing resistance, the impact of ships and vehicles on supports.

Under the influence of temperature changes a structure is deformed (elongation, shortening, bowing). In systems with freedom of deformation, effects of temperature changes do not cause the formation of additional internal forces. In systems that do not have freedom of deformation, thermal effects cause additional internal forces in the body. Thermal effects may result in additional internal forces in structures made of materials with different coefficients of thermal expansion, e.g. in the composite steel-concrete constructions. Effect of temperature should be taken into account when the force of these changes represents more than 5% of the forces from basic load.

Deformations of a structure caused by the internal forces caused by thermal effects used for calculations relate to the processing temperature (conventionally 10ºC). If the structure is mounted at a different temperature, then calculations need the appropriate adjustments. The calculations assume an average value of the construction period, wherein should take into account installation of the elements affecting the irreversible effects (introduction lock free deformation). In calculations extreme changes in temperature should be taken into account according to Tab. 3.9.

Tab. 3.9. The values of extreme temperature changes in the calculation of bridges

Concrete	Steel	Composite steel-concrete
-15ºC to +30ºC	-25ºC to +55ºC	concrete part as in concrete bridges, steel part as in steel bridges

In the case of concrete or stone elements of the smallest dimension of at least 60 cm, shown in Tab. 3.9, temperature deviations should be reduced by 5°C.

In addition to the warm weather influences, one must also take into account possible thermal influences from local sources of heat (welding, thermal straightening, heating pipelines) on the structure. In the case of composite structures one should also take into account both the differences in coefficients of thermal expansion and the differences in extreme temperature changes.

The impact of the wind load has a particular impact on cable-stayed and suspended structures. In the case of other objects one can refer to the general construction standards. With regard to concrete bridges on massive supports of heights up to 10 meters one does not need to check the wind load. Concrete bridges on pole-mounted supports or on supports in excess of 10 meters should be always checked. Openwork designs should be adopted for the actual projected area of the first beam on the plane perpendicular to the direction of wind, including a carriageway and sidewalks, and 50% of the corresponding surface of the girders sheltered from the wind by the first girder. This area cannot be larger than the area defined by the outline of the structure.

The influence of the wind pressure should be taken into account both when checking the stability with regard to shifting and capsizing, and – especially in double-girder railway bridges or an independent tram bridges – overload extreme girder.

The ice floe pressure should be taken into account when calculating supports and starlings located in the waters of rivers or flooded areas. This also applies to scaffolds, if the construction period covers the winter period.

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Chapter 4.

Classical and numerical methods in bridge design

Michał Jukowski, Krzysztof Śledziewski, Sławomir Karaś

4.1. Bridge design

The modern bridge design procedure is based on principles developed over decades. According to these principles, we can distinguish the following elements of the design process:

- conceptual work,
- main structural calculations EN 1992-1 [15],
- main strength calculations,
- structural design,
- static and strength calculations taking into account the characteristic dimensions, ULS, SLS,
- the final structural design.

Separation of the static analysis and strength is deeply rooted in the tradition of design. The breakthrough in the development of the theory and design techniques have become the method of numerical and virtual simulations of real processes. In particular, the Finite Element Method (FEM) has become a new quality in the theory of structures. Thanks to computers, it is possible to solve a number of issues which until now could only be known by experimental studies. Within 40 years of the development and use of FEA, it has become an essential research tool for engineers creating innovative structures. FEA is also used to automate the design process. Commercial software, thanks to friendly pre and post processors, has become a commonly used tool and it makes it possible to meet or examine new concepts or previously unknown areas.

This chapter discusses the possibilities of the numerical modelling of bridge structures using specialized software. Numerical tools enable the formulation of the process of static analysis and dimensioning into one coherent whole. Possibilities of methods enable resignation of the models of beam and quasi-real engineering design models and even volumetric surface.

In addition, one of simplified methods is presented. This is J. Courbon's method which was formulated in 1940 [1]. Actually the method is adjusted to the needs of lectures on bridges similar to the concept which was given in the article [6]. Nowadays the method seems to be an archaic one however the method has still a great didactic potential. Especially, the method shows clearly the bridge carrying-deck behaviour under the traffic action, is both enough simple to do the calculations by proverbial pencil and is still in use in cases of temporary bridges design. Simply, the method is intelligent, whatever it means.

4.2. J. Courbon method

4.2.1. The basic variant

The simplified methods were in use up to the 1980s i.e. to the beginnings of the computer era [12]. One of them is the Courbon's method. Its advantages are simplicity and reliability which contribute to the estimations on the safe side. The error value is of 5% to 20% in comparison to more advanced methods including FEM. Alternatively, this concept is known as the *rigid crossbeam method*. In the mathematical sense, we can also say the *infinitely rigid crossbeam method*.

It is the case where the assumption of symmetry of the structure cross-section takes place i.e. the beam spacing in the cross-section fulfils the condition of the mirror reflection along the vertical symmetry axis. In the following part we shall depart from these assumptions.

Now, let us focus on a simple and regular bridge, the side view of which is shown in Fig. 4.1. By arbitrary perpendicular cutting A-A the symmetrical cross-section is displayed in the axonometric sketch in Fig. 4.2.

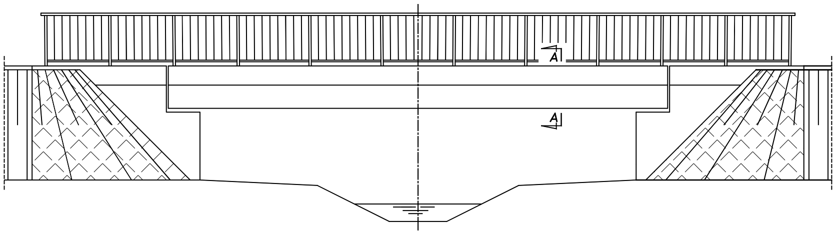

Fig. 4.1. The simple bridge under consideration

In Fig. 4.2 a rigid cross-beam in the form of a truss is visible as well as a concentrated force placed on the cross-section at an arbitrary point on the deck.

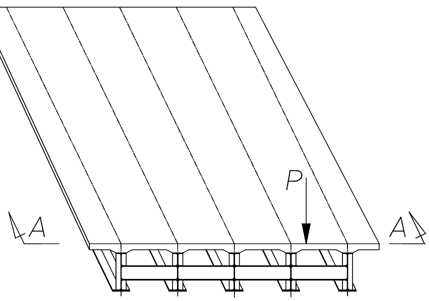

Fig. 4.2. The bridge cross-section, axonometric view

The equilibrium conditions are of the greatest importance in tasks of mechanics. We are going to apply them, however, some additional assumptions are necessary. They are as follows:

1) the cross-section of the carrying structure has a vertical symmetry axis, Fig. 4.3, the horizontal axis of the structure is the principal axis of inertia, the bending stiffness of the beams are equal,

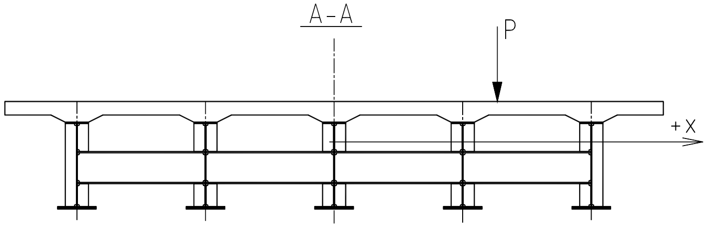

Fig. 4.3. The symmetrical cross section

2) the problem is static and flat, two-dimensional, linearly elastic and the Hooke's law is valid, with the stiffness principles of rigor (the current configuration converges with the initial configuration), linearity allows to apply the principle of superposition,
3) in the analysed cross-section of the load-carrying structure of the bridge there is an infinitely rigid cross member – Courbon's assumption,
4) the displacement of any point of the cross-section is limited by the reaction caused by the resistance of bending girders, therefore, we can assume that in the cross-section the girders are supported on elastic springs/bearings of Winkler's type, Fig. 4.4, where for any of the n-th girders the constitutive relation has the following form:

$$\vec{\eta}_{(n)} \sim \vec{u}_{(n)} \,, \tag{4.1}$$

where:

$\vec{\eta}_{(n)}$ – impact (vector reaction) of the n-th girder on the platform,
$\vec{u}_{(n)}$ – displacement vector in the place of the girder.

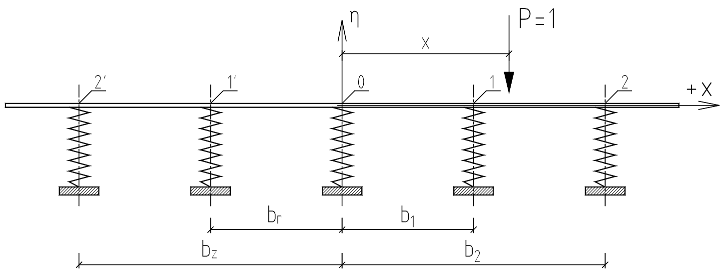

Fig. 4.4. Flat model cross-section of the load-carrying structure of a bridge, the initial configuration

By virtue of assumption 2), after the static load is applied, the infinitely stiff cross-section will appear deformed as shown in Fig. 4.5.

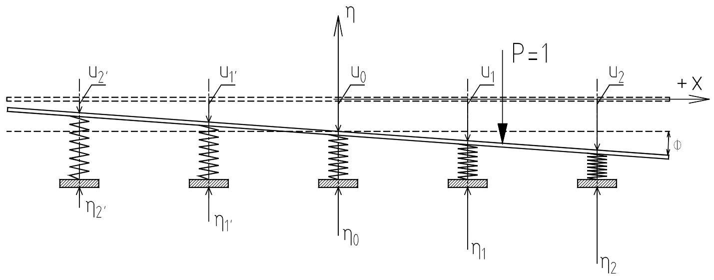

Fig. 4.5. The current configuration – after loading

On the basis of assumption 1), it is possible to break the deformation shown in the Fig. 4.5 into two additive states/parts – symmetric (Fig. 4.6) and asymmetric (Fig. 4.7). In both drawings the symmetric and asymmetric states are referenced to the initial configuration section marked symbolically by a dash line.

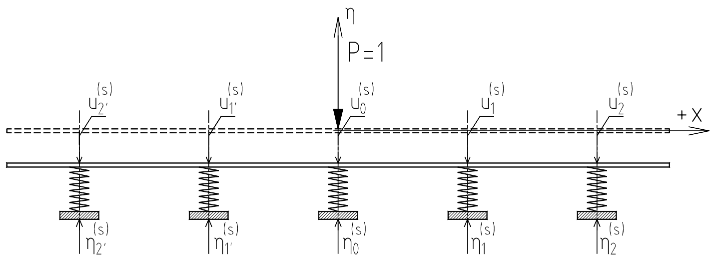

Fig. 4.6. Symmetrical part of deformation

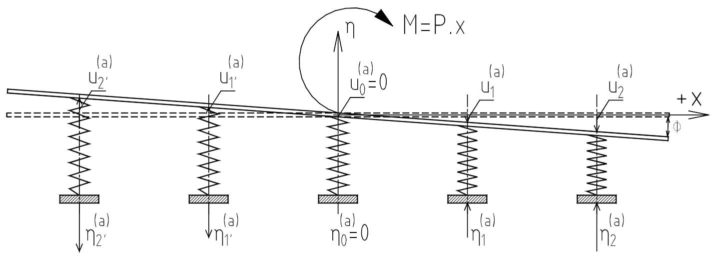

Fig. 4.7. Antisymmetric component of the deformation

The drawings above show 5 girders symbolizing, in general, any number that is marked by 'k'. By virtue of the superposition rule, we can analyse the flat equilibrium problems independently in both cases, i.e. symmetric and antisymmetric.

Antisymmetric state

Using assumptions 1) and 4), the cross-section of the structure displaces evenly down (uniform translation), as a rigid body, of vector $\vec{u}^{(s)}$, which will, in accordance with (1), produce the same reactions in all contractual elastic supports $|\vec{\eta}^{(s)}| = \eta^{(s)}$. In the analysis of the equilibrium state a variant of the equilibrium equations (2) is used, which consists of the sum of projections in the vertical and horizontal directions and the sum of the rotational moments relative to any point on the plane. Finally:

$$\begin{cases} \sum V = 0 \rightarrow \eta^{(s)} = \dfrac{P}{k}, \\ \sum H \equiv 0, \\ \sum M_0 = 0. \end{cases} \tag{4.2}$$

Asymmetric state

As a result of the existence of an infinitely rigid transom, the cross-section will rotate at the angle φ around the origin point 'O' (turnover). Therefore, by virtue of assumption 4), the only essential equation in the equilibrium set is the rotational moment around the origin 'O'. In such circumstances, the balance sum of moments gives reaction vector products as it is shown in Fig. 4.7.

The only essential condition in equations (4.3) contains two unknowns (in the general case we have: Int(k/2) = [k/2]), one must therefore formulate an additional condition that will solve the problem. As usual, the condition stems from the deformation compatibility requirement. For this purpose, again Courbon's assumption is used and, in the present case, the compatibility condition has the form of Thales' rule as follows:

$$\begin{cases} \sum V = 0, \\ \sum H \equiv 0, \\ \sum M_0 = 0 \rightarrow 2\left(\eta_2^{(a)} b_2 + \eta_1^{(a)} b_1\right) = Px. \end{cases} \tag{4.3}$$

In the analysis of the current configuration shown in Fig. 4.5, it was found that the maximal impact is attributable to the outer girder numbered 2. That is why the system of equations (4.3) was solved by determining the value of $\eta_2^{(a)}$ reaction and as a consequence:

$$tg\phi = \frac{\left|\vec{u}_2^{(a)}\right|}{b_2} = \frac{\left|\vec{u}_1^{(a)}\right|}{b_1} \xrightarrow{(1)} \frac{\left|\eta_2^{(a)}\right|}{b_2} = \frac{\left|\eta_1^{(a)}\right|}{b_1} \tag{4.4}$$

hence:

$$\eta_2^{(a)} = \frac{Pxb_2}{2\left[(b_1)^2 + (b_2)^2\right]}, \tag{4.5}$$

which can be generalized as the case 'k' of girders in the cross-section of a bridge by the relationship:

$$\eta_{\mathrm{s}}^{(\mathrm{a})} = \frac{Pxb_{\mathrm{s}}}{2\sum\limits_{m=1,2,...}^{[k/2]} (b_{\mathrm{m}})^2}, \tag{4.6}$$

wherein the index 's' is the number of the outer girder.

After some elementary algebraic operations, the analogous reaction value can be found in any n-th girder by means of the relationship:

$$\eta_{\mathrm{n}}^{(\mathrm{a})} = \frac{Pxb_{\mathrm{n}}}{2\sum\limits_{m=1,2,...}^{[k/2]} (b_{\mathrm{m}})^2}, \tag{4.7}$$

Applying the superposition principle, we are able to determine the total reaction of any girder, but for the reasons described above, i.e. the expected maximum effort in the outer girder, we focus on this one. Now, assuming that the force P = 1, the following linear relationship is obtained:

$$\eta_{\mathrm{s}} = \eta_{\mathrm{s}}^{(\mathrm{s})} + \eta_{\mathrm{s}}^{(\mathrm{a})} = \frac{1}{k} + \frac{xb_{\mathrm{s}}}{2\sum\limits_{m=1,2,...}^{[k/2]} (b_{\mathrm{m}})^2} = a_0 + xa_1 = \eta_{\mathrm{s}}(x), \tag{4.8}$$

where:

$$a_1 = tg\phi = b_{\mathrm{s}} \left\{ 2\sum\limits_{m=1,2,...}^{[k/2]} (b_{\mathrm{m}})^2 \right\}^{-1}, \tag{4.9}$$

whereby the argument "x" is the abscissa locating the position of the force P = 1 (4.9) in the cross-section.

In the expression (4.8), within the range of taken generalization, and assuming the girders continuous distribution which is proper for a case of plate carrying-deck, we can localise an arbitrary area, but defined, as a girder. Then let us introduce the argument ξ (instead of used previously used b_{n}):

$$b_{\mathrm{n}} \to \xi. \tag{4.10}$$

We receive:

$$\eta = \frac{1}{k} + \frac{x\xi}{2\sum\limits_{m=1,2,...}^{[k/2]} (b_{\mathrm{m}})^2} = a_0 + x\xi A_1 = \eta(x,\xi) = \eta(\mathrm{y},\xi), \tag{4.11}$$

where:

$$A_1 = \left\{ 2 \sum_{m=1,2,...}^{[k/2]} (b_{\mathrm{m}})^2 \right\}^{-1}. \tag{4.12}$$

Assuming that the response from the displacement in the cross section of the unit forces will be deposited as the ordinates in locations occupied by this force P = 1, and having visible in the relationship (4.11) bilinearity function η relative to its two arguments x, ξ, with simultaneous symmetry of these arguments, it can be seen that $\eta(x, \xi)$ is the influence line of $P(x) = 1$ forces of material area reaction (in particular of girder) in the cross section of the bridge localized by ξ value, also this mechanical magnitude is called as the influence line of the lateral load distribution in the cross-section of a bridge.

The two points of (4.8) determine the examined influence line. We can use $x = 0$ and $x = b_{\mathrm{s}}$, for instance. Therefore, we have:

$$x = 0 \rightarrow \eta_{\mathrm{s}}(0) = \eta_{\mathrm{s}}^{(\mathrm{s})} = \frac{1}{k}, \tag{4.13}$$

$$x = b_{\mathrm{s}} \rightarrow \eta_{\mathrm{s}}(b_{\mathrm{s}}) = \frac{1}{k} + \frac{(b_{\mathrm{s}})^2}{2 \sum_{m=1,2,...}^{[k/2]} (b_{\mathrm{m}})^2}. \tag{4.14}$$

Let us find the position of $P = 1$ for which the influence line has a root:

$$\eta_{\mathrm{s}}(x_0) = 0, \tag{4.15}$$

hence were obtained:

$$x_0 = -\frac{2 \sum_{m=1,2,...}^{[k/2]} (b_{\mathrm{m}})^2}{k b_{\mathrm{s}}}. \tag{4.16}$$

The x_0 divides the influence line into two branches (Fig. 4.8). It determines ranges called the positive and negative branch of an influence line. Knowing them allows to reduce overloads resulting from the values of partial load coefficients γ_{f} of the ultimate limit state (ULS) respectively in the cases of unloading and overload when looking for a reaction of the girder load.

By the positive influence line branch interval we can understand the areas where ordinates are positive. When searching for the maximum design value $\eta_{s\,max}^{(d)}$ in the range of the positive branch, the ordinates were multiplied by the partial safety factors $\gamma_{\mathrm{f\ max}}$, while the negative branch implies the use of $\gamma_{\mathrm{f\ min}}$ multipliers. In determining $\eta_{s\,min}^{(d)}$ we proceed inversely.

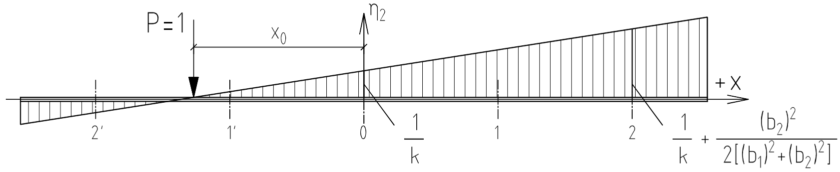

Fig. 4.8. Influence line

The previously used expression “reaction of the girder” can be replaced with a different one, more adequate for applications of η_s, namely, we can talk about the participation of the girder in carrying loads occurring in the cross-section, which is equivalent to the interpretation of the appointment of the load falling on the selected girder.

At this point we arrive at the greatest achievement of the discussed method. Now, it is possible to separate/remove a single girder from the construction of the load-carrying structure of a bridge, which is the intention of the whole method. It means that we can calculate a simple/continuous girder instead of a whole superstructure. Mechanics is nice, isn't it.

4.2.2. General variant

In the above analysis, a variant taking into account the asymmetry of the distribution of the material and its density in the cross-section is also considered. Symbolically, this state is shown in Fig. 4.9, where girders are irregularly spaced and their stiffness varies. Descriptively, the simplification is to be understood as follows:

$$E_{(m)}J_{(m)} \to EJ_{(m)}. \tag{4.17}$$

By changing the assumptions 1) and 4), the following counterparts should be adopted:

1’) the cross-section of a load-carrying structure does not have a geometrical vertical axis of symmetry, the spacing of girders is different in addition to the bending stiffness of individual girders which also varies,

4’) the displacement of the n-th girder is proportional to the impact of the transverse beam and inversely proportional to its bending stiffness $EJ_{(n)}$:

$$\vec{u}_{(n)} \sim \frac{\vec{\eta}_{(n)}}{EJ_{(n)}}, \tag{4.18}$$

we take the form of Courbon’s assumptions also taking into account the earlier states of symmetry and antisymmetry in the form:

$$\eta_{(n)} = EJ_{(n)}(c_0 + c_1 y_{(n)}), \tag{4.19}$$

where $y_{(n)}$ is an abscissa of the n-th girder relative to the arbitrarily adopted early cut-off. In Fig. 4.9 the left edge in the cross-section is assumed as the origin of the now used coordinate system, denoted by 'O'.

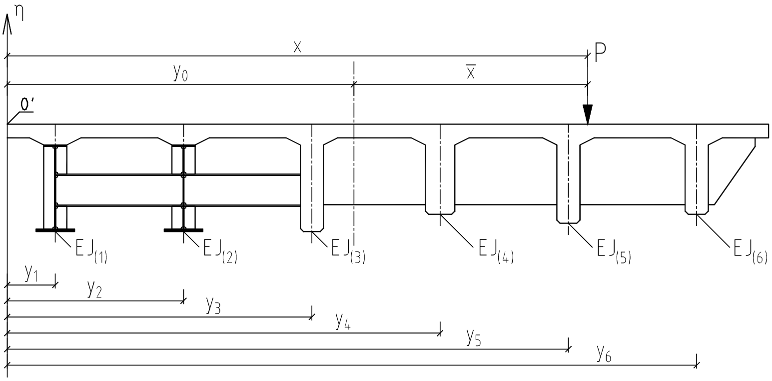

Fig. 4.9. A fictitious cross-section of a load-carrying structure illustrating the variation in the bending stiffness of girders and their irregular spacing

Taking into account (1'), and from the equilibrium equations the following is obtained:

$$\begin{cases} \sum V = 0 \quad \rightarrow P = c_0 \sum\limits_m^k EJ_{(m)} + c_1 \sum\limits_m^k y_{(m)} EJ_{(m)}, \\ \sum H = 0, \\ \sum M_0 = 0 \rightarrow P\,x = c_0 \sum\limits_m^k y_{(m)} EJ_{(m)} + c_1 \sum\limits_m^k \left(y_{(m)}\right)^2 EJ_{(m)}. \end{cases} \tag{4.20}$$

which, in the matrix notation is:

$$\begin{bmatrix} \sum\limits_m^k EJ_{(m)} & \sum\limits_m^k y_{(m)} EJ_{(m)} \\ \sum\limits_m^k y_{(m)} EJ_{(m)} & \sum\limits_m^k \left(y_{(m)}\right)^2 EJ_{(m)} \end{bmatrix} \begin{bmatrix} c_0 \\ c_1 \end{bmatrix} = P \begin{bmatrix} 1 \\ x \end{bmatrix}. \tag{4.21}$$

The equation (4.21) can be simplified by changing the coordinate system so that the coefficient matrix has a diagonal form. The condition:

$$\sum_m^k y_{(m)} EJ_{(m)} = 0, \tag{4.22}$$

corresponds to the determination of the abscissa of girders' stiffness centroid:

$$y_0 = \frac{\sum_m^k y_{(\mathrm{m})} EJ_{(\mathrm{m})}}{\sum_m^k EJ_{(\mathrm{m})}}. \tag{4.23}$$

As well as the transformation of other abscissas:

$$\overline{y}_{(\mathrm{m})} = y_{(\mathrm{m})} - y_0; \overline{x} = x - y_0. \tag{4.24}$$

Then:

$$\eta_{(\mathrm{n})} = EJ_{(\mathrm{n})}(\tilde{c}_0 + \tilde{c}_1 \overline{y}_{(\mathrm{n})}), \tag{4.25}$$

$$\begin{bmatrix} \tilde{c}_0 \\ \tilde{c}_1 \end{bmatrix} = \frac{P}{\sum_m^k EJ_{(\mathrm{m})} \cdot \sum_m^k \left(\overline{y}_{(\mathrm{m})}\right)^2 EJ_{(\mathrm{m})}} \begin{bmatrix} \sum_m^k \left(\overline{y}_{(\mathrm{m})}\right)^2 EJ_{(\mathrm{m})} & 0 \\ 0 & \sum_m^k EJ_{(\mathrm{m})} \end{bmatrix} \begin{bmatrix} 1 \\ \overline{x} \end{bmatrix}, \tag{4.26}$$

or:

$$\tilde{c}_0 = \frac{P}{\sum_m^k EJ_{(\mathrm{m})}}; \tilde{c}_1 = \frac{P\,\overline{x}}{\sum_m^k \left(\overline{y}_{(\mathrm{m})}\right)^2 EJ_{(\mathrm{m})}}. \tag{4.27}$$

On the basis of (4.19) and (4.27) is obtained a reaction of the n-th girder on the stiff cross-bar as follows:

$$\eta_{(\mathrm{n})} = EJ_{(\mathrm{n})} P \left(\frac{1}{\sum_m^k EJ_{(\mathrm{m})}} + \frac{\overline{x}\, \overline{y}_{(\mathrm{n})}}{\sum_m^k (\overline{y}_{(\mathrm{m})})^2 EJ_{(\mathrm{m})}} \right). \tag{4.28}$$

The formula (4.28) with assumptions 1) and 4) leads to relations (4.8). The methods shown above are further developed in the Guyon [3] and Massonnet methods [7], in which the authors – in addition to the flexural rigidity – also take into account the torsional rigidity of the bridge superstructure.

The presented version of Courbon's method is relevant to simple and typical small bridges. The error of the method, relating to more precise methods, is known. It can be said that in terms of the proportion of the width of the load-carrying structure to its span $B/L_t \le 0.5$ we get good results. There are many measures, the value of one of which [12] is determined by the following formula:

$$\alpha = \frac{b^3}{6EJ'\Delta_g}, \tag{4.29}$$

where:

b – girder spacing,

EJ' – bending rigidity of a girder, the spacing of cross-bars (a):

$$EJ' = \frac{EJ}{a}, \tag{4.30}$$

Δ_g – deflection of a single girder due to its dead weight.

If the value $\alpha \leq 0.005$, then the method can be successfully applied.

Courbon method is still of concern to engineering and even research. This is evidenced by a number of ongoing studies, for example [5], [8], [9].

4.3. Numerical modeling

The twenty first century is the time of rapid technological development. During this period a lot of programs for designing different types of structures have come into being. Tedious, long-term, manual calculations have been replaced by programs that create numerical models of structures. These models can be simplified or accurate. Simplified ones are used in a less detailed analysis. A simplified model must be formulated so that the results are reasonable. Simplification involves less effort at the stage of creating a model.

The most common method in the numerical calculations is the Finite Element Method (FEM). This method involves an attempt to find a solution to a complex problem (usually described with a differential equation) by replacing it with simpler one [2], [10], [11].

Thanks to numerical methods, designers can analyse the behaviour of a structure under the influence of various kinds of loads. Structures are most often subject to the following types of analyses:

- static (linear, nonlinear),
- dynamic (linear, nonlinear),
- modal.

Describing a structural model, we must take into account two parameters:

- the dimension of an element,
- the dimension of the space where elements are located.

In respect to the size of an element, we can distinguish: one-dimensional elements (e^1) – rod, two-dimensional elements (e^2) – plate or disc and three-dimensional elements (e^3) – solid. Analysing space we have: one-dimensional space (p^1), two-dimensional space (p^2) and three-dimensional space (p^3).

Created models can consist of a combination of the above mentioned parameters, for example (e^1+e^2, p^3). An example of a simple bridge model is shown in

Fig. 4.10 and Fig. 4.11. The deck was modelled as a two-dimensional element, and girders as a part of the rod, a one-dimensional element.

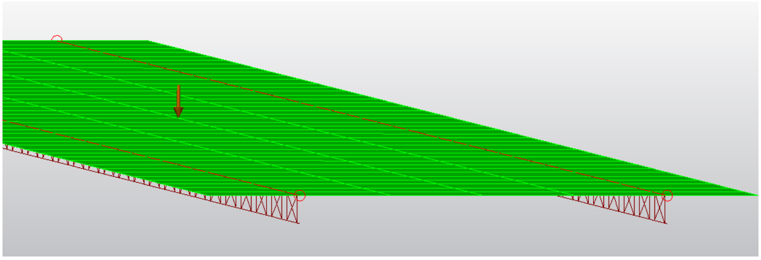

Fig. 4.10. Example of a model bridge: e^2 – element plate (green), e^1 – beams (red), p^3 – space

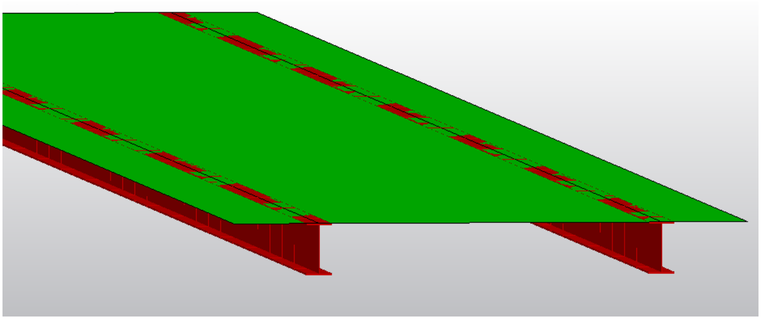

Fig. 4.11. 3D visualization of model elements

While creating a one-dimensional model of an element it is necessary to provide its geometric characteristics. Fig. 4.12 shows an example of a selection of the geometrical characteristics of a beam for the simplified model of a bridge described above (the model was designed in Autodesk Simulation Multiphysics).

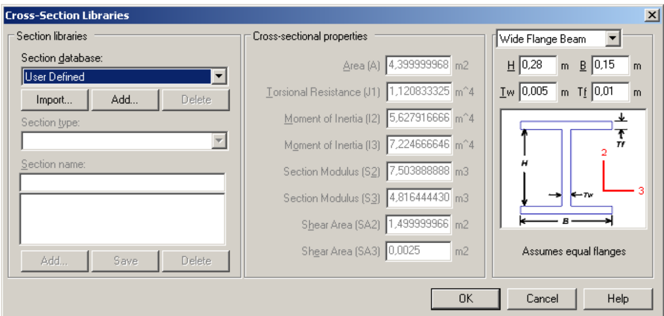

Fig. 4.12. The geometrical characteristics of the girder modeled as part of the rod

When creating a model, the author must have the necessary knowledge with regard to which element will be suitable to be used in the model. Each of the above-mentioned elements has a separate application and different method of burdeing. In simpler construction a model may consist of only one type of a component, for example, an industrial building construction model can be created only with rod elements. Complex models consist of at least 2 or 3 kinds of elements.

At the modelling stage of construction with one-dimensional elements, bars can be divided into [11]:

- flat frame elements,
- spatial frame elements,
- flat lattice elements,
- spatial lattice elements.

In modelling of a structure of rod elements their size should be taken into consideration. The truss structure is used when the ratio of the cross section to the longitudinal section is less than 0.1. Additionally, bar elements are used when the width of an element is relatively small and establishment of a MES gird at a given point would significantly reduce the size of the net.

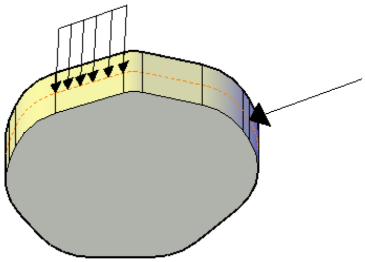

Fig. 4.13. Scheme of the shield load

A disc body is a chunk whose one dimension (thickness) is much smaller than the other ones (Fig. 4.13). The central surface, which is equidistant from the outer edges, is the plane. Shield can be loaded only on its plane. These elements can work in the following states [11]: PSS (plane state of stress), PSD (plane state of deformation) and axisymmetric state of stress.

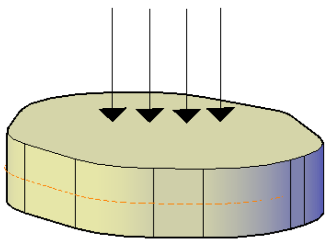

Fig. 4.14. Scheme of the plate load

The plate is an element whose thickness is much smaller than the other dimensions, and the middle surface is equidistant from the outer edges. The sheets are loaded in the direction perpendicular to their planes (Fig. 4.14).

The coating is an element the thickness of which is much smaller than the other two dimensions as in the case of the shield and the plate, but the median area, which is equidistant from the outer edges, need not to be a plane. A load can be applied at any angle (Fig. 4.15).

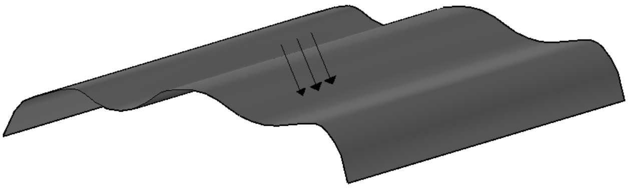

Fig. 4.15. Scheme of shell load

A solid is a three-dimensional element (Fig. 4.16). It is element, which all dimensions are of the same order. The load and its shape is arbitrary. The solid may consist of the following spatial elements: four-node (tetrahedron), six-node and eight-node.

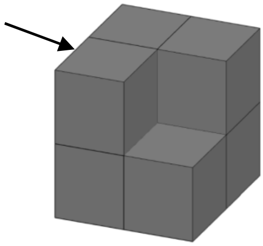

Fig. 4.16. Scheme of brick load

When modelling a structure using the disc, plate or coating elements, one must pay special attention to the shape and size of the MES grid. These are the main principles of the FEA meshing for these elements [11]:

1) FEA mesh shape must reproduce the shape of the structure,
2) grid nodes must be placed at their support points,
3) where the job is symmetrical, grid also needs to be symmetrical,
4) in the presence of holes, grid nodes must be located in such a way that it is impossible to create a FEA grid,
5) concentrated forces must be applied at the nodes, and therefore the grid must include the points of the force application,
6) grid limits must take into account the locations of the changes of thickness of the structure elements or components made of different materials.

Creating a good computer model is not an easy task. Here is an example of a numerical model of a bridge, which was created with different elements. The model will replicate a multi-girder concrete bridge structure. Object parameters:

- deck slab thickness of 0.20 m,

- concrete beams with a section of 0.40 m x 1.50 m,
- theoretical bridge span of 30.00 m.

The superstructure is supported on six bearings one constant located under the central beam and five of sliding. All the elements are made of the same material – concrete (Fairly Hight Strength). The model was loaded with a uniformly distributed load of 10 kN/m^2, applied over the entire top surface of the deck.

There were made five computational models:

Model 1 – deck and girders were modelled using plate elements,

Model 2 – deck modelled as a plate and girders as rod elements giving them the appropriate geometric characteristics. In addition, there was the option of an offset of 0.75 m used to the actual mapping position of girders with respect to plate,

Model 3 – the bridge plate was modelled as a plate element, and beams as parts of the rod elements pushing them to the value of 0.85 m to take into account the mutual overlap of each plate and beam (Fig. 3.17),

Model 4 – the whole section was modelled using solid elements,

Model 5 – the deck modelled as a plate, and girders as part of the solid bodies.

The calculations were made in Autodesk Simulation Multiphysics 2012.

Model 1

The deck plate with beams were modelled as an element of the type – plate (e^2). The MES grid was concentrated in the places of supporting girders on the bearings and at the point where the girders contact the bridge plate. Fig. 4.17 and Fig. 4.18 show a 3D visualization of the model.

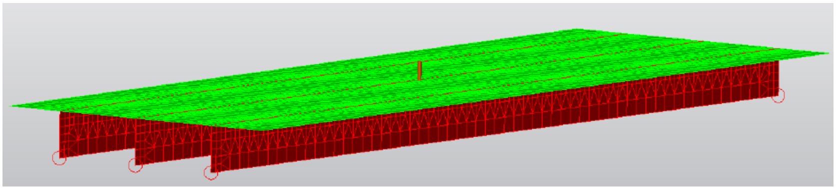

Fig. 4.17. 3D model view

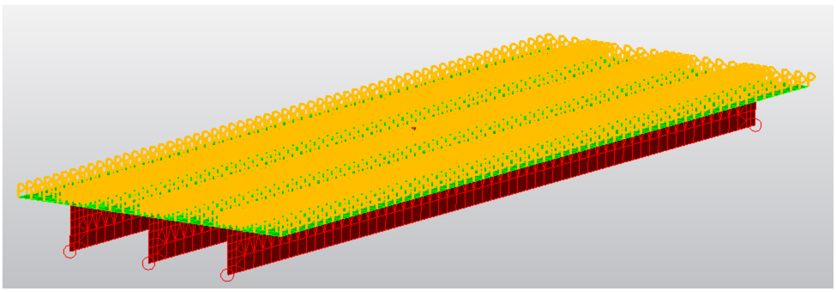

Fig. 4.18. The view on the loaded plate

Below the density and shape of the FE mesh for the plates and girders are shown (Fig. 4.19 and Fig. 4.20).

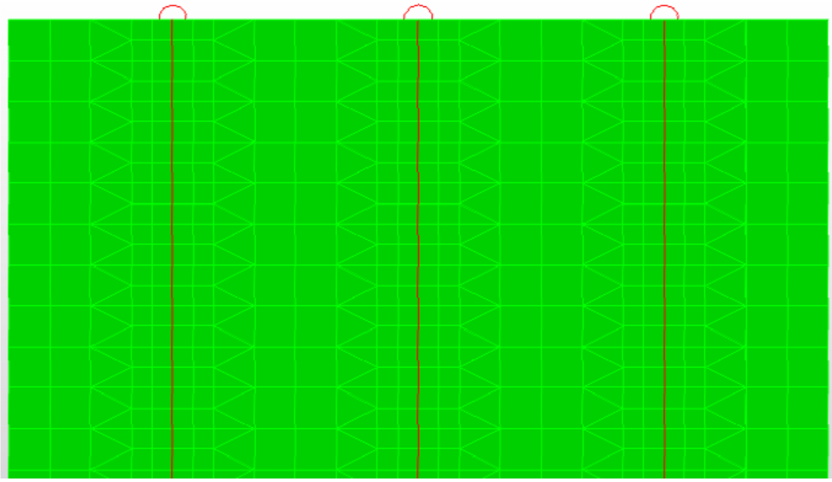

Fig. 4.19. FEM mesh on the deck

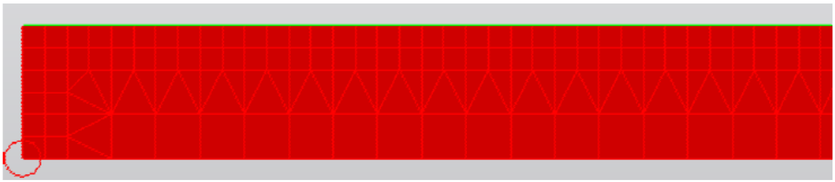

Fig. 4.20. FEM mesh girder

The results for each model of a bridge were read at the same point, i.e. in the middle of the span and the width of the deck. The results of the displacement in the middle of the plate span and the value of stresses in the most unfavourable combination is shown in Fig. 4.21 and Fig. 4.22.

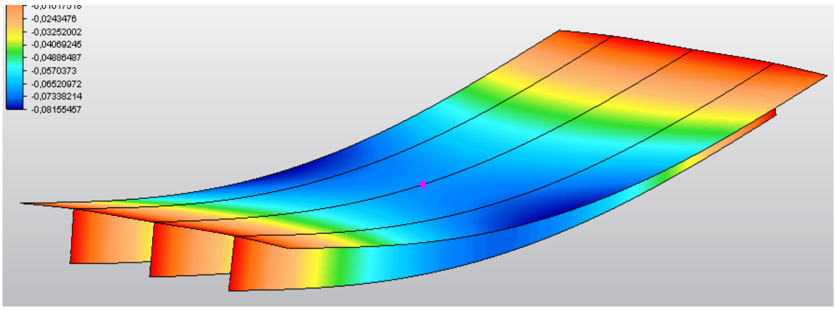

Fig. 4.21. Structural displacements in the middle of the span length *u* = 0.0707 m

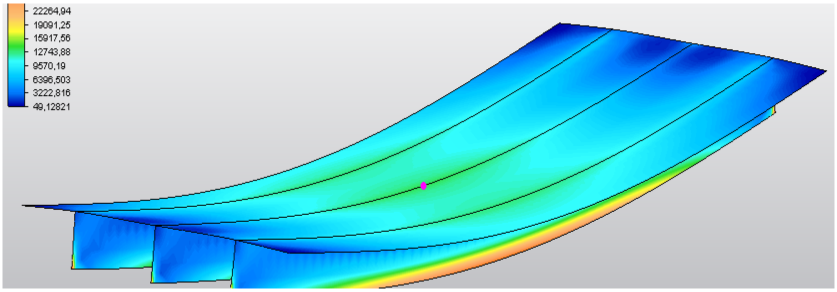

Fig. 4.22. Stresses in their most unfavourable combination in the middle of the span length σ = 13 234 kN/m^2

Model 2

The bridge plate was modelled as an element of the type – plate (e^2) while the girders as elements of the type – beam (e^1). The model was loaded with a load to take into account the actual position of the beam was used, and the offset option evenly distributed to 10 kN/m^2, over the entire length and width of the plate. In shifted the centre of gravity beams of value – 0.75 m in the direction of the Z-axis. The bridge model (e^1+e^2, p^3) is shown in Fig. 4.23.

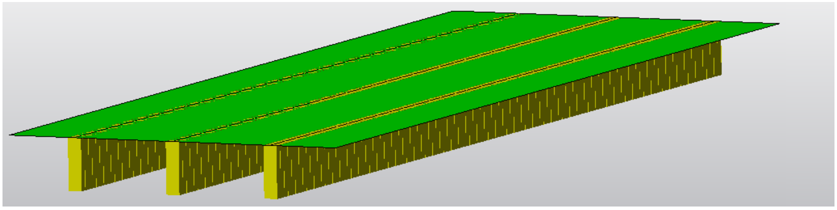

Fig. 4.23. The model is made of plate and beams elements: top – e^1+e^2, p^3

The results of displacements in the middle span of the plate and the value of stress in the most unfavourable combination are shown in Fig. 4.24 and Fig. 4.25.

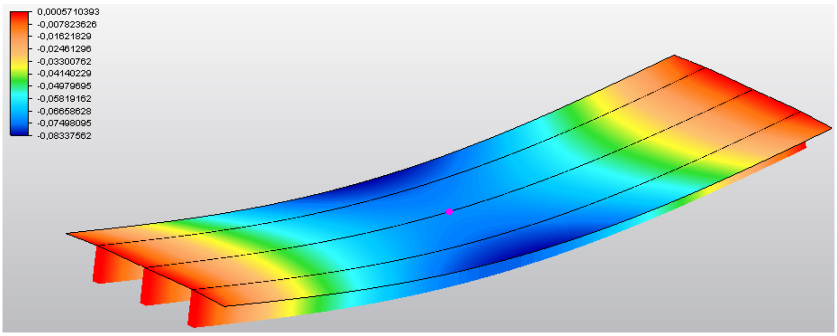

Fig. 4.24. Structural displacements in the middle of the span length *u* = 0.0722 m

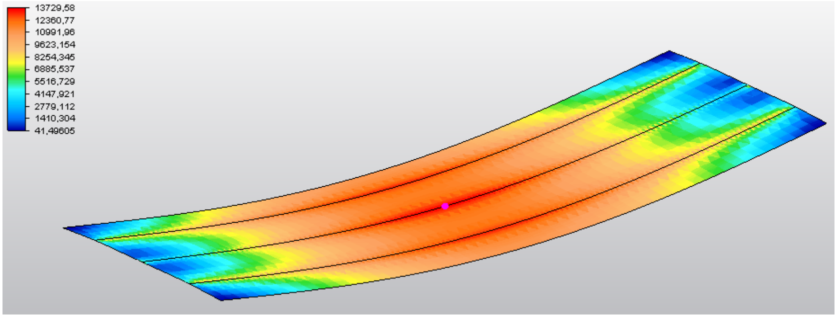

Fig. 4.25. Stresses in their most unfavourable combination in the middle of the span length $\sigma = 13\ 640\ kN/m^2$

Model 3

The bridge plate was modelled as an element of the type – plate (e^2) while the girders as elements of the type – beam (e^1). In order to take into account the actual position of the beam and overlaps of the surface of the girder and deck plates, the offset option was used, moving the centre of the beam gravity of the value 0.85 m in the direction of the Z axis.

The results of the displacement in the middle of the plate span and the value of stresses in the most unfavourable combination are shown in Fig. 4.26 and Fig. 4.27.

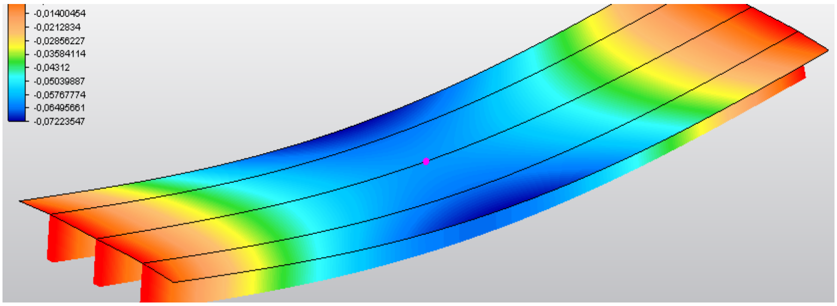

Fig. 4.26. Structural displacements in the middle of the span length $u = 0.0615$ m

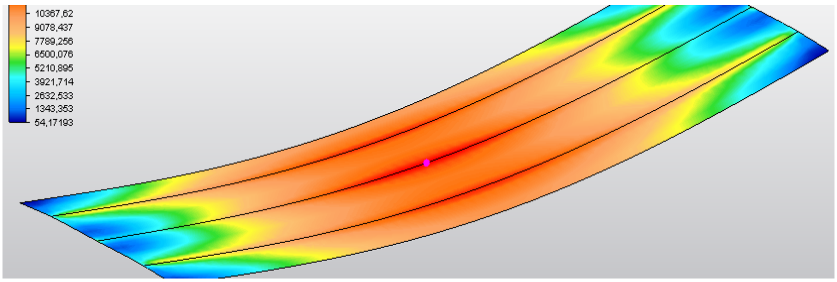

Fig. 4.27. Stresses in their most unfavourable combination in the middle of the span length $\sigma = 12\ 946\ kN/m^2$

Model 4

The deck disc and girders were modelled as elements of the type – brick (e^3). The model is shown in Fig. 4.28.

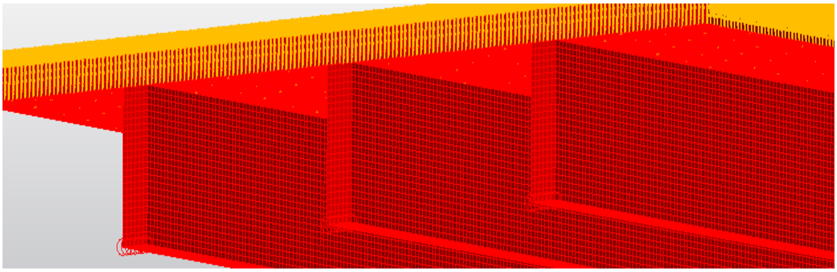

Fig. 4.28. A model made of solid elements

The results of the displacement in the middle of the plate span and the value of stresses in the most unfavourable combination are shown in Fig. 4.29 and Fig. 4.30.

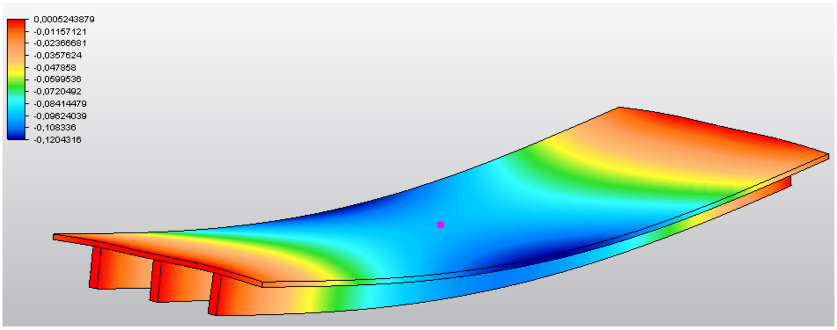

Fig. 4.29. Structural displacements in the middle of the span length *u* = 0.0967 m

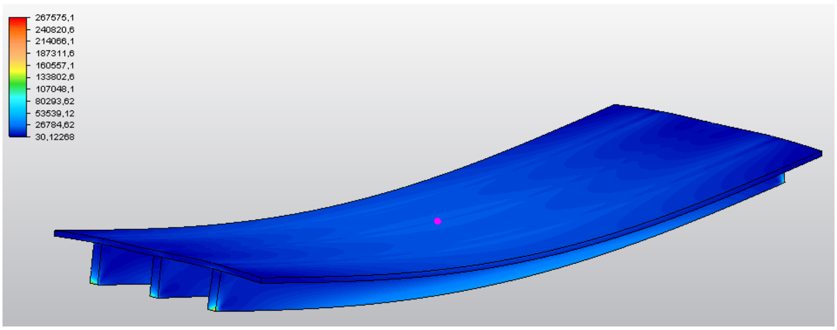

Fig. 4.30. Stresses in their most unfavourable combination in the middle of the span length σ = 18 724 kN/m^2

Model 5

The bridge plate was modelled as element of the type – plate (e^2) while the girders as elements of the type – brick (e^3).

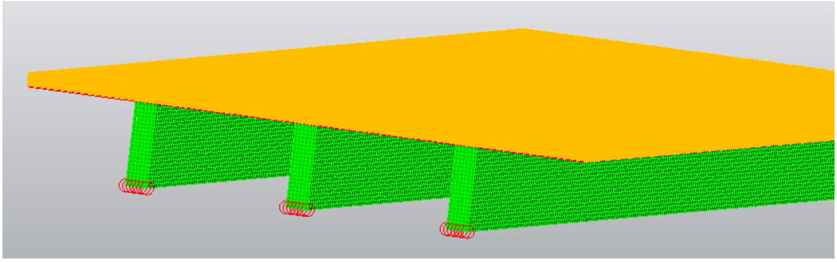

Fig. 4.31. The model made of solid and plate elements

The results of the displacement in the middle of the span plate and the value of stresses in the most unfavourable combination are shown in Fig. 4.32 and Fig. 4.33.

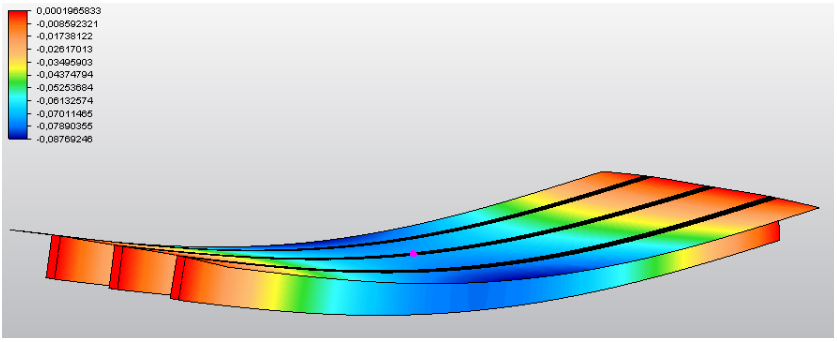

Fig. 4.32. Structural displacements in the middle of the span length *u* = 0.0739 m

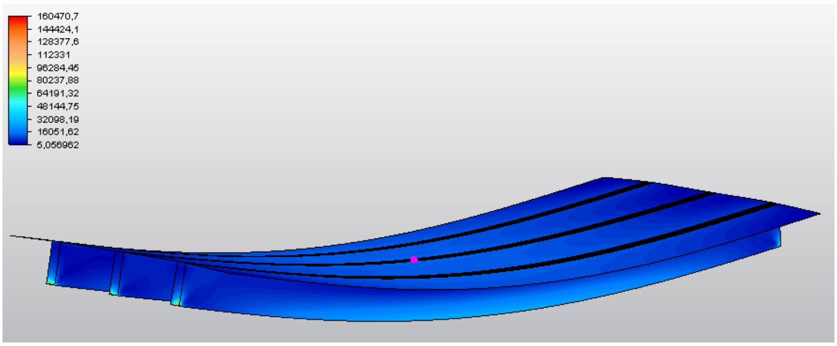

Fig. 4.33. Stresses in their most unfavourable combination in the middle of the span length σ = 12 750 kN/m^2

Comparison of results

At the stage of the comparison of results it was necessary to select a model of comparison. The reference system is model no. 2.

The summary of results of displacements and stresses is shown below in Fig. 4.34 and Fig. 4.35.

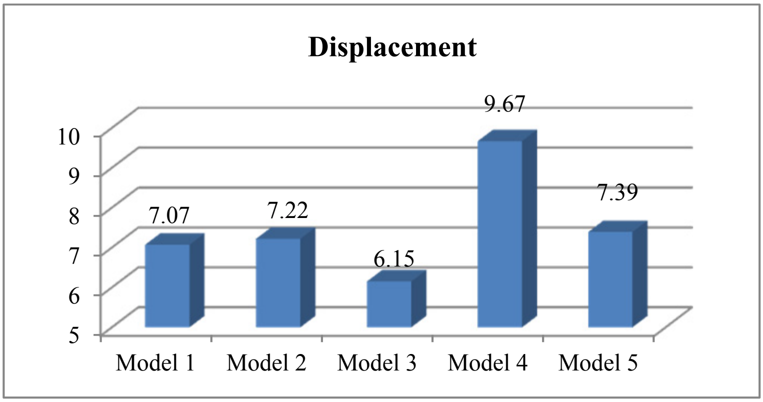

Fig. 4.34. The results of displacements [cm] for different numerical models

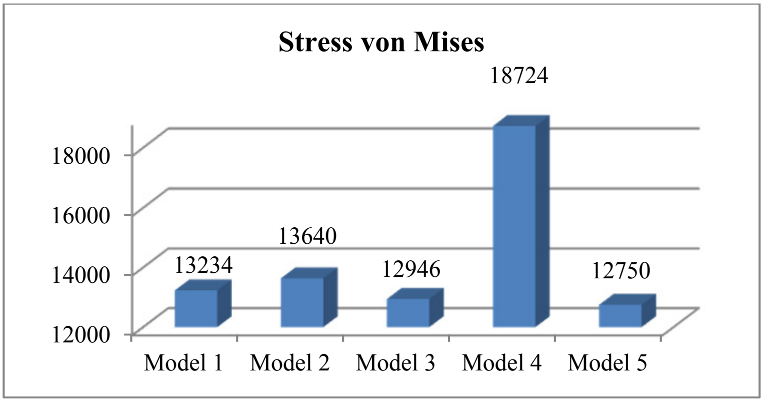

Fig. 4.35. The results of stress von Mises [kN/m^2] for different numerical models

The comparison of results is presented in the form of a percentage deviation with respect to the analysis of model 2 (Fig. 4.36 and Fig. 4.37).

When analysing the results of displacements it was found that the model created with pavers under an evenly distributed load behaved very much like model 2. The difference of displacements in the middle of the plate span is only 2.08%, i.e. 0.015 m.

When analysing the results of stresses it was noted that the model made entirely of pavers reflected the plate and rod model in the highest degree. Particular attention should also be paid to model 5 which, in terms of displacement, does not significantly differ from the reference system.

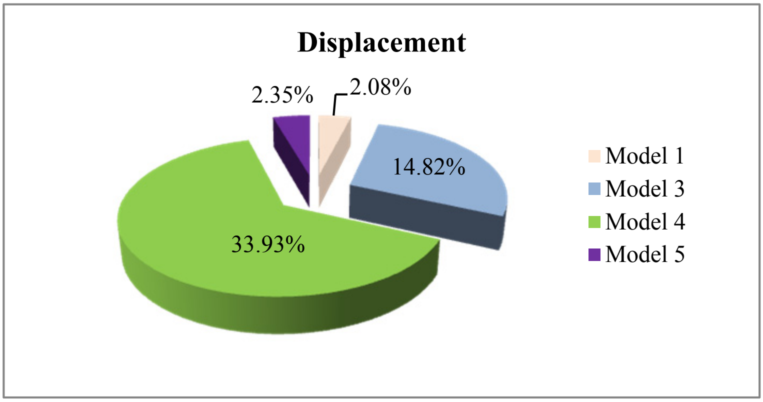

Fig. 4.36. A comparison the results of displacements with respect to model 2

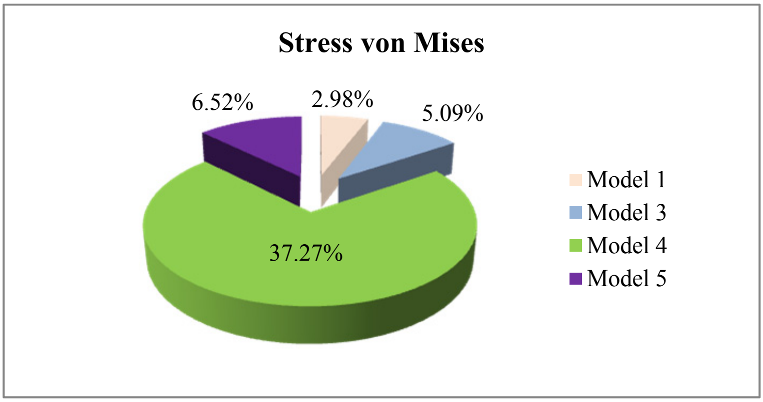

Fig. 4.37. A comparison the results of stress von Mises with respect to model 2

Based on the results it can be concluded that the model made of plate elements under load behaves virtually the same as the model of a plate and a rod. However, when we look at the definition of the plate, which defines it as an element whose thickness is much smaller than the other dimensions, model 1 becomes not entirely correct. The beam width is approximately 27% of its height, and so it is not "definitely" less. In addition, we should take into account the impact of the mutual overlap of the beam and the deck surfaces (Fig. 4.38).

Because of large differences between the results model 4 is not an appropriate model. Solid components are used in the analysis of the local analysis on the object, e.g. an analysis of a support or foothold. The model created with solid elements requires a very high computing power. For comparison, the weight of the model 1's file is 10.8 MB and model 4's – 618 MB. It should be added that working on such a file is highly difficult, and therefore buildings are not entirely modelled with solid elements. It ought to be kept in mind that in the case of narrow elements, a model must consist of at least four lanes of items thanks to which it will be possible to describe the change of stresses in the section [11].

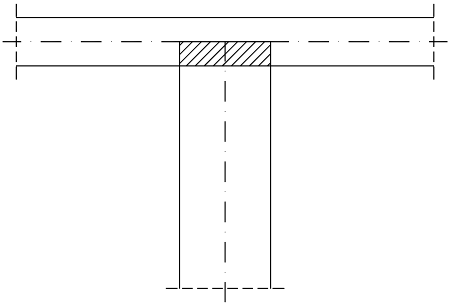

Fig. 4.38. Overlapping area of the plates and the girder

One of the basic principles when creating a model of a structure is the duration of calculations. A model should be developed with a view to producing correct results within the shortest possible calculating time [11]. Present day computers have a high computing power, so designers can create models that reflect the actual conditions of a structure to a high degree. Fig. 4.39÷Fig. 4.43 show the FEM model of the Solidarity Bridge in Plock [4].

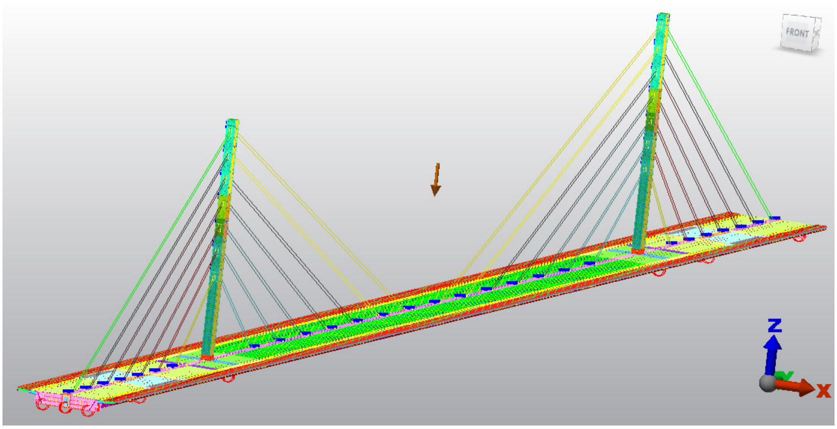

Fig. 4.39. An isometric view of the calculation model [4]

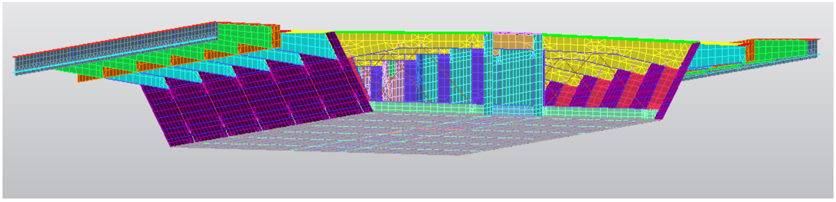

Fig. 4.40. One of the installation units span suspended [4]

The created model of a bridge was classified a class e^1+e^2; p^3. It was modelled in Autodesk Simualtion Multiphysics 2012. The model was created basing on the following elements:

- shell/plate,
- beam and truss.

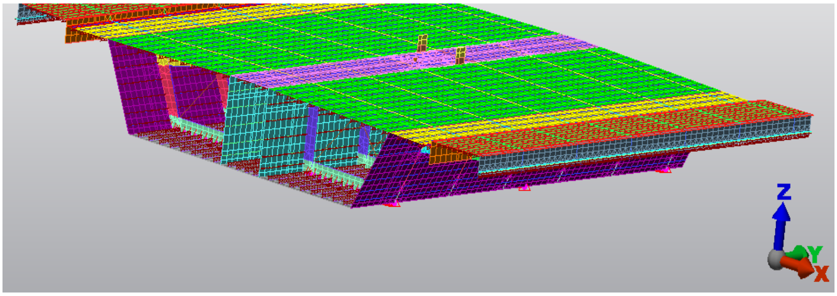

Fig. 4.41. One of the installation units span suspended [4]

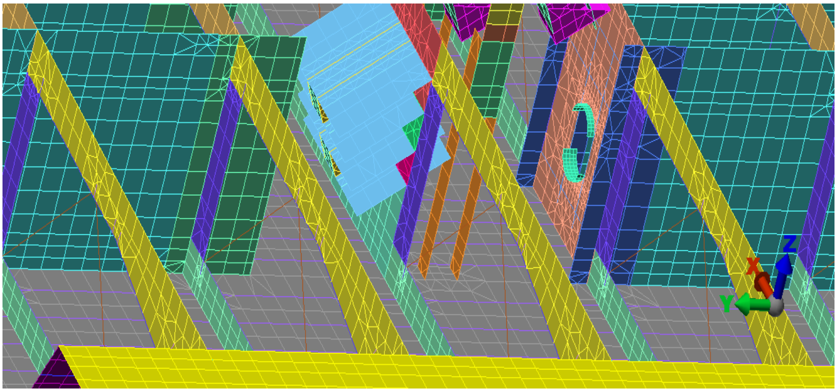

Fig. 4.42. View anchor blocks of tendon in the span [4]

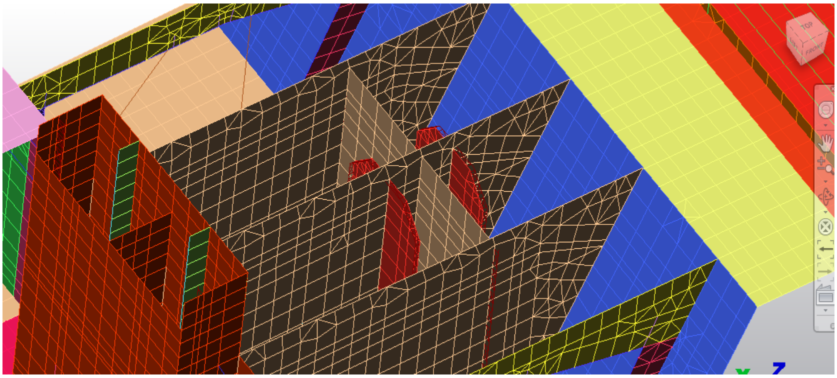

Fig. 4.43. The cross-section by the mounting JM 9, wherein the invisible upper plate and the longitudinal ribs [4]

The model consists of 893.694 elements, 643.257 of which are plate elements, 250.381 are the frame members and 56 – lattice elements. The model maps the existing structure to a high degree.

The merits of a modelled structure are the results obtained thanks to it. The designer is obliged to examine them. The final stage is to check whether the limit states comply with the current regulations. With regard to the bridge construction the following should checked in particular [13]:

- ultimate limit state (ULS),
- serviceability limit state (SLS).

In order to verify the status of the ULS, the designer must have the results of stress or internal forces. Very often the programs automatically create a bitmap of results for a particular item or for the whole structure. An example of a bitmap of the stresses in the structure of a bridge is shown in Fig. 4.44. Different shades of colours represent the stress values at a given point of the structure. On the basis of such data the designer can establish whether stresses do not exceed the limit.

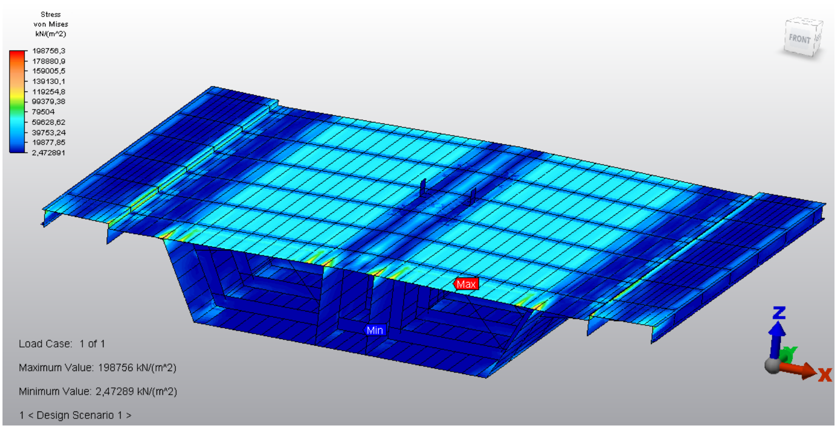

Fig. 4.44. The results of the stress of a continuous load

In order to verify the status of the SLS, the designer must have the results of structure displacements. A decisive influence on the limit state of the bridge usefulness is set to the vertical displacement value in the middle of the span length. Fig. 4.45 shows a vertical displacement on the Z axis, the main span (suspended) of the Bridge of Solidarity of the LM1 variable load, according to the PN-EN 1991-2 [14]. Fig. 4.46 show horizontal displacements of the pylon in the direction of the axis X of the wind load.

The wind load plays an important role in the construction of suspended and hanging bridges. In adverse weather conditions (strong, gusty wind) it may contribute to large displacements of the top of the pylon.

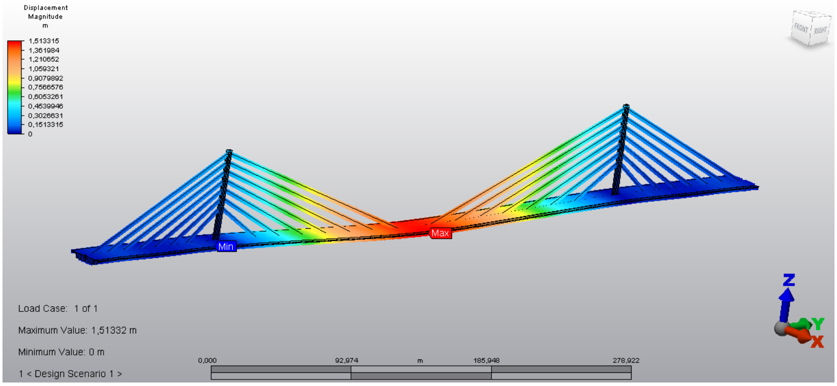

Fig. 4.45. The results of vertical displacement of the load variable

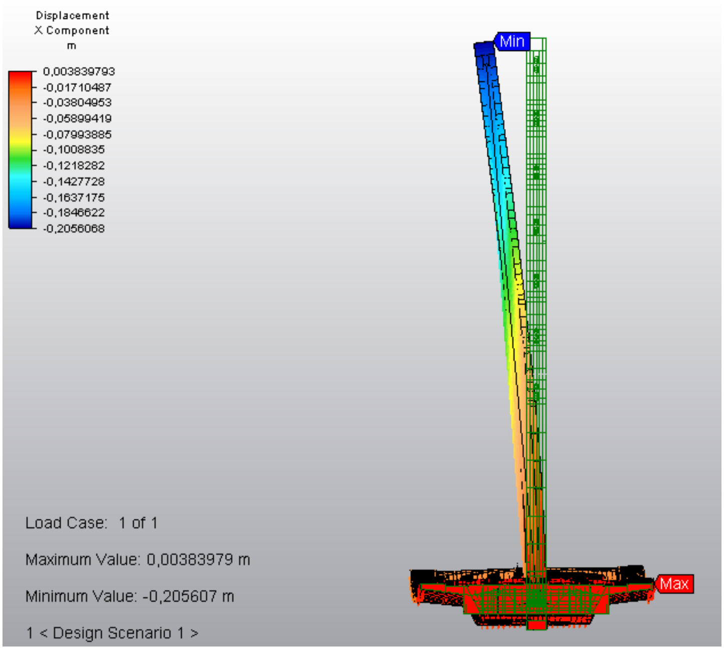

Fig. 4.46. The displacements of the pylon in the direction of the axis X of the wind load

Large-span bridges, in particular hanging and suspended ones, are subject to a detailed endurance analysis. Such objects are very susceptible to the effects of wind. At the design stage it is necessary to execute a dynamic and modal structure analyses. On the basis of their results, the constructor can establish whether the structure does not fall into so-called resonance under the influence of time-varying wind actions. Special attention ought to be paid to the fact that manually performing such complex calculations for very complex structures would be quite a challenge even for the best designers or engineers. Examples of the results of the vibration of the Solidarity Bridge are shown below in Fig. 4.47 to Fig. 4.49.

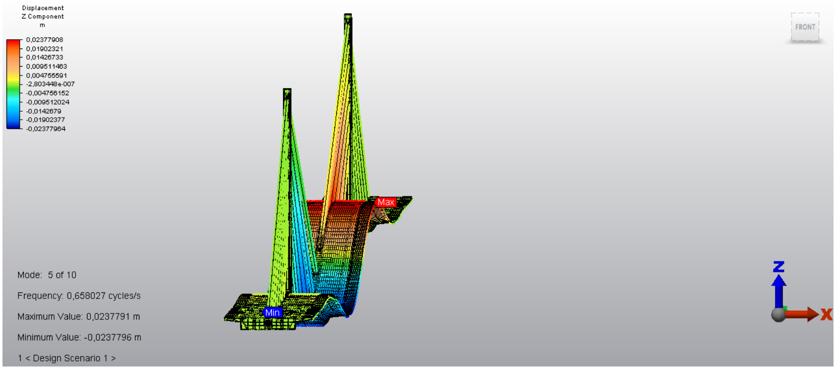

Fig. 4.47. The fifth form of the vibrations of the Bridge of Solidarity – the flexural vertical span, the frequency equals 0.658 Hz [4]

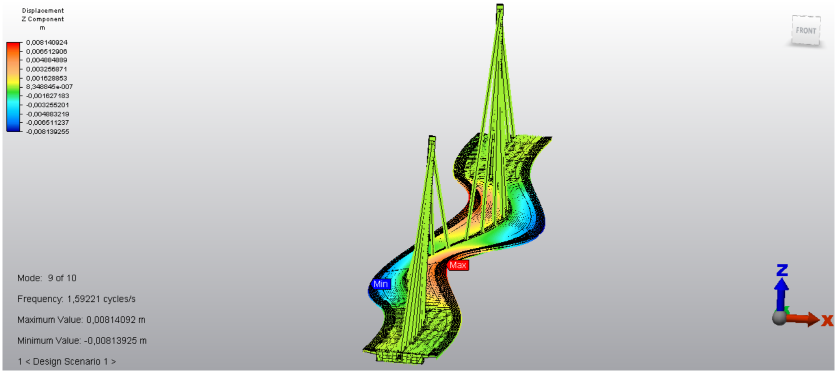

Fig. 4.48. The ninth form of the natural frequency mode in the span plane of the Bridge of Solidarity – torsional horizontal spans, equal to 1.592 Hz frequency [4]

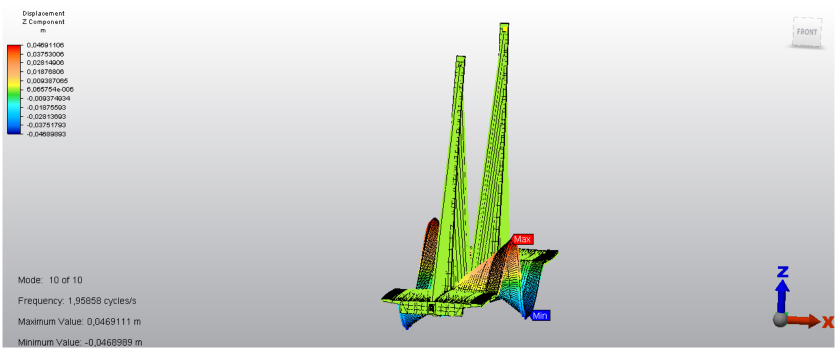

Fig. 4.49. Tenth form of the vibrations of the Bridge of Solidarity – torsional horizontal span, the frequency equals 1.959 Hz [4]

When creating a complex structural model, one must remember about the limitations of hardware. A model bridge consisting of a large number of elements will be calculated for a long time. To carry out a more sophisticated analysis, e.g. a dynamic analysis in the non-linear field, it may be impossible to do. In such situations, one needs to create a simplified model consisting of fewer components. However, such a model should possess realistic geometric parameters, e.g. the torsional rigidity of a bridge in a simplified model should correspond to the accurate model.

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[14] EN 1991-2, *Eurocode 1: Actions on structures*, *Part 2: Traffic loads on bridges*.

[15] EN 1992-1-1, *Eurocode 2: Design of concrete structures, Part 1-1: General rules and rules for buildings.*

Chapter 5.

Monitoring of environmental structures and facilities

Krzysztof Śledziewski, Wioleta Czarnecka,Sławomir Karaś
Maciej Kowal, Michał Jukowski

5.1. Environment and road-bridge engineering

The aim of the consideration is to characterize the monitoring of environmental systems in the neighbouring areas of existing bridges or objects to be built in the future. Actually, the need and functionality of transitions for different sizes animals are unrecognisable. The constant monitoring is the only recognition method because of its reliability and possibility of creating the quantitative basis for statistical analyses. There have been some attempts taken to do such a registration. However, the approaches were local and result availability is problematic. Here comes the suggestion of using the same system which is in used with highways, known as the road traffic safety system (SAFE-ROAD). Those solutions are proved by practice. It is crucial to ensure the accessibility to gained results for anybody who just needs to know how the transitions serve for the environment. Texts present general issues relating to sustainable development and the problems associated with them. There are examples of the Polish and European road investment in which the problems arising from the lack of a general approach to environmental principles and consequently sustainable construction occurred. The examples of technical projects of culverts and bridges adopted to environmental issues, some current problems are shown. The existing problems are of dual nature, the first group being very general aspects i.e. concerning the concept of ecology, while the other one involves detailed tasks, e.g. shaping the image of a bridge. Several questions of great significance have been formulated and addressed to ecologists. The answers are indispensable for bridge engineers to solve technical aspects of the proper design of environment-friendly bridges. Last but not least, the suggestion to use bridges as places to monitor the environment in their surroundings is presented. This research work might be crucial for further good cooperation of bridge engineers and environmental ones.

5.2. Management of the environment and ecology

5.2.1. Introduction

In order to understand the essence of the management of the environment and ecology it is necessary to take a look at some important terms which are described briefly below:

The environment is the surroundings or conditions in which a person, animal, or plant lives or operates, including air, water, land, natural resources, flora, fauna, people and their interrelationships [50].

Ecology is the study of a nature structure and its functioning, dealing with the study of interactions between organisms and their environment.

The management can be explained as an act of making effective use of human, capital and material resources to achieve the goal of the managed entity.

Environmental management is the domain of public authority, it includes planning, organizing, motivating and controlling of the measures taken to reduce the negative environmental impact.

Environmental protection means proper use and restoration of natural resources and components, particularly of wild plants, animals, natural complexes and ecosystems but also keeping the environment clean, minimizing pollution and consumption of media such as water and heat.

The ways in which we can protect our environment are:

- the creation of national parks, nature reserves, landscape parks, etc.,
- taking species of plants and animals under legal protection,
- establishment of natural monuments.

5.2.2. Methods of environmental analysis

Today, environmental analysis methods can be divided into two main groups. The first covers analytical procedures developed for routine measurement (monitoring) of common types of contamination of water, wastewater, soil and atmosphere to determine the eligibility of individual elements of the environment for certain classes or categories of cleanliness, on the basis of appropriate legislation. These are both simple colorimetric methods or even organoleptic ones, giving only a roughly estimated level of the measured parameter in a given environmental component as well as complex instrumental methods, requiring the use of measuring equipment, often sophisticated and very expensive [33].

The second group includes analytical methods which have been developed for the measurement of selected environmental pollutants, not specified in the legislation, but with important implications for understanding the condition and quality of the environment. In such cases, we usually have to deal with complex analytical procedures, often aiming for the determination of trace or ultra-trace amounts of substances [33].

Both groups of methods that often intermingle in terms of applied equipment or procedures used for sample preparation, should be considered inseparable as broadly defined environmental analytics, used (but not exclusively) in environmental monitoring [33].

To ensure the credibility of immission and emission measurements, standardization of methods for obtaining, processing and transmission of information on the state of the environment, the State Environmental Monitoring (SEM) was established on the basis of the Act of 20 July 1991 on inspection of environmental protection.

5.2.3. State Environmental Monitoring

Information from the State Environmental Monitoring is used by the authorities of central and local government for (Fig. 5.1) [33]:

- environmental management through legal instruments (eg. the authorization to introduce substances into the environment, environment protection plan, spatial planning etc.),
- monitoring the effectiveness of environmental protection, planning sustainable development of a region taking into account the state of pollution of and protection against it, implementation of international agreements signed and ratified by Poland,
- developing negotiating positions in the field of environmental protection within the European Union.

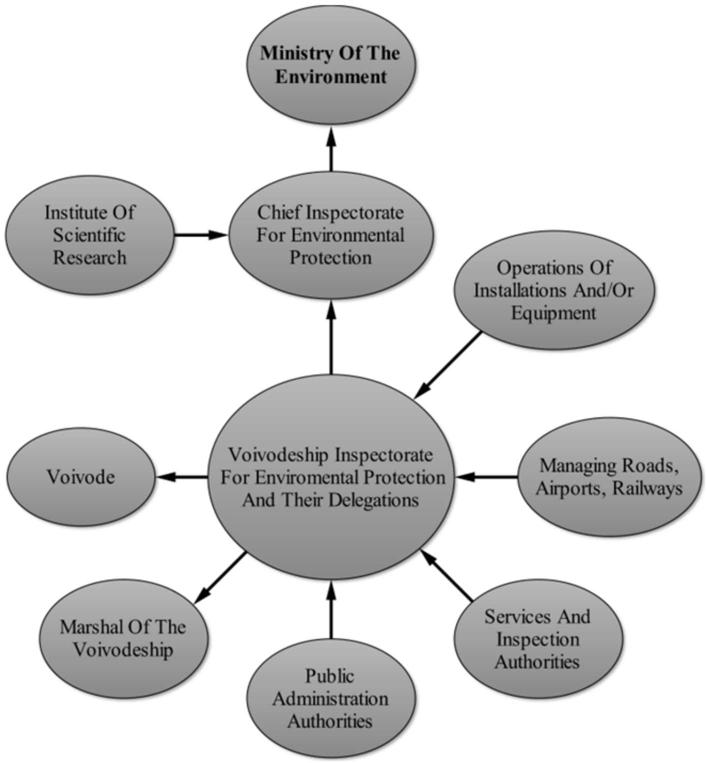

Fig. 5.1. Scheme of the State Environmental Monitoring

The main objective of the SEM is to implement new programs and monitoring techniques, including accreditation of testing and measuring labs, controlling the accuracy obtained in analytical methods, also through participation in national and international comparative studies and cooperation with the European Commission and the European Environment Agency.

5.2.4. Environmental management tools

Management is an art of achieving intended results but through other people's actions, it is also an act of allocation of resources. Management is a set of activities (planning, organizing, motivating, controlling) that by using the resources of an organization (human, financial, material, informational) achieve the objectives of an organization.

Environmental management tools include [33]:

- management tools,
- management measures,
- ecological procedures and recommendations.

Management tools include political and legal institutions, management measures, instruments and procedures applied for the organization of the management system and to ensure its inner workings and to control the impact on the subject of management.

Management measures include organizational and informational tools. They organize the system and ensure the flow of information so that the systems function.

Management instruments are the tools that directly or indirectly affect the subject of management.

Management procedures are formalized procedures to help achieve the desired goals.

5.2.5. Environmental Information

Information in terms of engineering, is closely related to the theoretical concept of "communication system", in which there are the following elements: a source of news, encoder, channel, decoder, recipient of the message and noise.

Environmental information is a set of messages about the status of components and their changes, the processes and couplings occurring in the environmental management system, and about the relationship between this system and the environment. Information needed in environmental management can be divided into [33]:

- information about the level of anthropogenic impact on the environment, in other words any form of direct or indirect human influence on the environment,
- information about the status of ecosystems and their responses to various influences,

- information about the functioning of each tool (the means and instruments) of undertaken environmental policy,
- information about expenditures (costs) and the effects of the operation of the system of control of environment protection,
- information about the state of environmental awareness of a society and reactions of different groups to management instruments.

Sources of environmental information [33]:

- official information,
- statistical reporting,
- environmental monitoring,
- the results of scientific research,
- non-institutionalized information.

Not only the government but each of us should have, process and react to necessary information. Personally, we are responsible for what is happening in our daily lives, what air we breathe, what water we drink and in what condition the animals that share the environment with us are.

5.2.6. Example ecological environment monitoring

A 10 km stretch of a road most of it in non-built-up, was analyzed. A large part of the route goes through forests belonging to the Forest District Pulawy. The road consists of one carriageway and two lanes, which encourages high speed. At some intersections there is traffic segregation in the form of separating left-turn or right-turn. There is heavy traffic on that road and trucks are a very big part of that traffic.

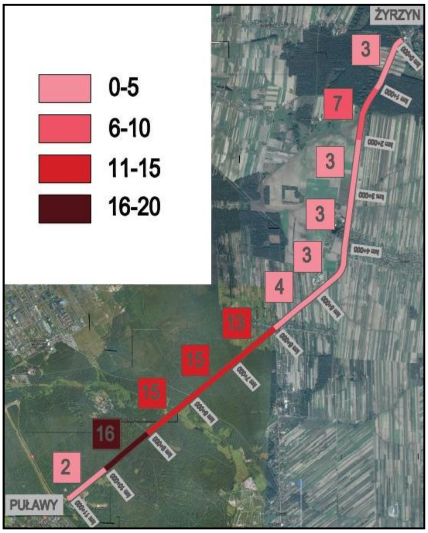

Fig. 5.2. The number of collisions involving animals at the DW 824 in Pulawy between 2007 and 2015 (July)

The monitoring of events [58] shows that the concentration of collisions occur in the forest complex and on its outskirts (Fig. 5.2). The section of the road from km 6+500 to km 10+500 has the highest risk of collision, and on this stretch at km 9+400 on 09.07.2015 a photocell detected a small deer near the road, in the month in which there was the greatest number of road accidents involving animals – Fig. 5.3.

Fig. 5.3. The picture was taken by a photocell Bushnell – a little deer near the road[1]

Fig. 5.4. A view of the bridge along the DW 821 taken by a photocell hanging on a tree[1]

Undeveloped space under the bridge along the regional road is ideally suited as an underpass for animals (Fig. 5.4 and Fig. 5.5). The study of the monitoring in the area shows that most traces were left by boars and animals of the deer family.

[1] Photo by W.Czarnecka

Fig. 5.5. The space under the bridge used by animals as an underpass[2]

Fig. 5.6. A Bushnell photocell hung on a tree[2]

Fig. 5.7. A photo taken by a Bushnell photocell – forest animals[2]

[2] Photo by W.Czarnecka

The images that confirm the site visit were obtained from a photocell hanging on a tree, highlighted in the Fig. 5.6 and Fig. 5.7.

Unfortunately, there is only one safe passage for animals on that 10-km-long stretch of road. Apart from that passage, animals pass in an uncontrolled manner and often end up dead.

5.3. Struggle with unclear perspective and environmental proposals

Let us start the analysis from the so-called Rospuda River event which exemplifies the problems and processes in the social field at the intesection of road engineering and the environment protection.

The spectacular action was conducted by young ecologists when the construction of a detour around the small town of Augustow was commenced. In 1992 the decision on the design work of the circuit road was taken. Among dozens of bridges, overpasses and culverts, there were also objects designed for small and big animals. All this, i.e. roads and bridges as project elements, was performed strictly according the Polish and European standards on environmental conservation. On the road run was the nature reserve on the Rospuda River which was intended to be exceeded by means of high road embankments and overpasses as well as bridges in the valley of the river.

It is necessary to emphasize that the life of citizens in Augustow reached the limit of uncomfortable conditions due to heavy transport traffic, high noises, difficulties of pedestrian and bicycle traffic and even dangerous accidents.

Investment Project	Informal action
Optional conceptual design	
Financing	
EIA - Environmental Impact Assessment made by authorised administration	
Decision - basis technical specification for design	
Tender for the design	
design, design documents	
Acceptance of the design documents	
Tender for construction works	
Construction	
Commissioning	
Warranty period	
Completion	

Fig. 5.8. The scheme of an investment project, young ecologists protest

The road construction works started in 2007 but soon they were blocked due to ecological action which was supported by some TV channels. In 2008 the investment was stopped. Recently, after significant modifications of the project the construction works were started again. By now the Rospuda slogan has been readable anywhere and anytime in Poland when one thinks of designing roads.

Here, the opinion explaining why such an event has occurred is depicted. The whole investment process was conducted in accordance with applicable the Polish and European provisions. The necessary environmental risk assessment has been made, which showed that the planned investment will not have a vital adverse effect on the environment. The diagram of management steps, shown in Fig. 5.8, localise the moment of young ecologists' impact.

While seeking reasons for such an unprecedented in Poland and violent protest against the conduct of construction of a bypass one can enumerate many of them. Out of this the simplest interpretation could be selected. It is the image of bridge fundaments and rising up heavy abutment walls. Always do road works cause transition aggression stages (6 to 12 months) against the environment in places of building roads, and bridges are always the first to build. In Fig. 5.9a the road construction works are shown. Disharmony or even aggressive chaos are perceptible, while in Fig. 5.9b the relaxation and harmony are in contrast to the photograph on left.

Summing up the above considerations, it is clear that road construction in environmentally sensitive areas should be protected by screens in order to protect the environment and avoid creating images that may cause social protests.

The second problem which occurred during road construction in the Rospuda River valley is associated with full access, without limitation, to formal documents and technical information, in general i.e. for anyone interested in [43], [56]. In particular, unlimited access is necessary also for informal groups, which in the case of the Rospuda had a dominant role.

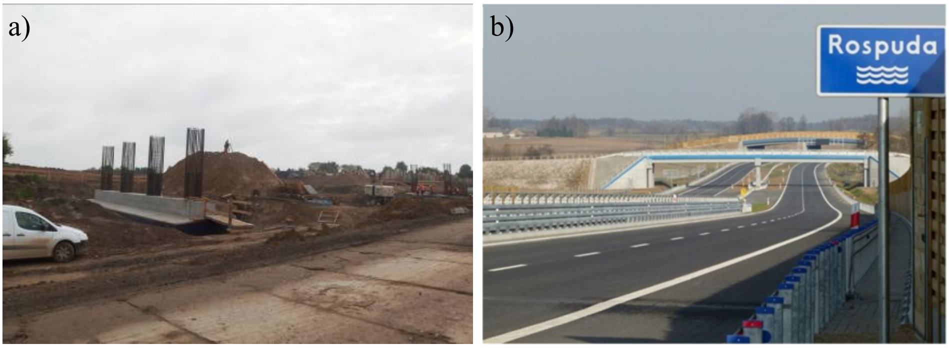

Fig. 5.9. a) typical view in a place of bridge building b) motorway view before a commissioning[3]

It is not clear how to carry out the discussion with informal groups, in particular, how the administration authorities and informal groups can organise the

[3] Photos by S.Karaś

discussion platform, the informal group not always having the leadership or staff of people engaged in the action. Certainly meeting the criteria for a directive of the access to information is helpful or essential.

The above example shows how the problems associated with various aspects of sustainable development: environmental, social, economic ones as well as political and legal ones interpenetrate [39]. It seems that due to certain provisions of the EU issues [38], [40], [41], [42], [43], mainly areas and species specially protected, have been harmonized within the EU member states. Unfortunately, there are still differences in the approach to the issues concerning the preparation of a project – for example, the approach to skills investment from the point of view of their impact on the environment even greater disparities between EU countries are at a technical level.

Assuming that the correct procedure is to put forward proposals rather than merely formulate demands, further below one can find an extensive argumentation of a technical proposal for a green bridge. The main advantage of the proposed solution is that it can be used right away. Other advantages concern both ecology and bridge engineering.

5.4. Environmental protection facilities

5.4.1. Introduction

Environmental protection facilities are designed to reduce or completely eliminate the negative influence of adverse impacts caused by various means of transport on the life of people and animals. In the recent years transport has been expanding very fast. The increase in the number of wheeled, aerial and water vehicles has been causing environmental degradation, often irreversibly. One of the main pollutants which enter our environment are gases as well as high noise levels. Gases are more dangerous because some of them can cause different kinds of disorders, sometimes incurable. The main types of gases penetrating the environment are:

- sulphur dioxide,
- carbon dioxide,
- nitrogen oxide,
- Carbon monoxide,
- polycyclic aromatic hydrocarbons,
- benzene,
- various kinds of particulates,
- lead,
- platinum,
- ozone, etc.

There are several means of transport. The most important are: road transport, railway transport, air transport and water transport.

Looking statistically at the above-mentioned modes of transport, road transport causes the greatest harm to the environment. It is because of the number of cars on the roads compared to other types of vehicles. Due to this fact, the following systems and environmental protection facilities will be discussed proceeding directly from road transport.

The total emission of carbon dioxide, which is a product of transport, contributes largely to the climate change around the world (the greenhouse effect) within just 14 years (1990–2004) grew by almost 50%. It is a considerable problem for institutions involved in reducing the negative effects of the human impact on ecosystems. Below there is a graph of changes in the quantity of the CO_2 emission in millions of tons from road transport in 25 countries of the European Union, Bulgaria, India, Norway, Romania and Turkey between 1990–2004.

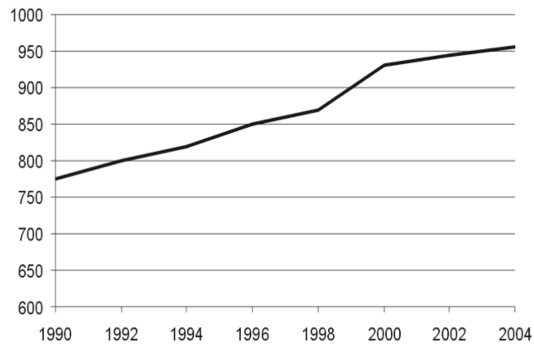

Fig. 5.10. Carbon dioxide emission from road transport in millions of tons in 25 countries of the European Union, Bulgaria, India, Norway, Romania and Turkey between 1990–2004[4]

The working of any internal combustion engine would be impossible without oxygen. In order for a reaction of the combustion gasoline engine to occur, oxygen is necessary in large quantities. The car's engine during one hour of working requires about 6000 litres of oxygen, whereas an average person consumes at the same time about 30 litres. In comparison to this, a deciduous tree is able to produce within 60 minutes about 1200 litres of oxygen. Having analysed these numbers, it is clear that, in order to balance the demand for oxygen, as many as 5 trees are needed for one car consuming as much oxygen as about 200 people.

Definitely, the traffic noise, which is directly related to the movement of motor vehicles, has a negative impact on the environment. The acoustic climate in urban areas and in the immediate vicinity of dual carriageways, is particularly endangered. Noise can cause many diseases, e.g.:
- fatigue,
- difficulty in remembering, learning, orientation,

[4] http://lanckoronska.zm.org.pl/?a=koalicja.broszuras_03

- aggression.

In Poland it is estimated that about 13 million people suffer from noise affecting the comfort of their living. There are many ways in which its level or intensity can be significantly reduced. After an analysis of an area adjacent to a source of noise, the following three zones can be distinguished [1]:

- emission zone,
- zone of protective measures,
- zone of collection (receiver).

Chapters 5.4.1÷5.4.5 describes in detail the above mentioned zones and provides examples of solutions applied in order to reduce the adverse effects of noise on the quality of people's life.

5.4.2. Atmosphere protection against pollution caused by transport

Continuous development of transport associated with an increase in the number of vehicles on the road causes a release large quantities of pollution into the atmosphere. The way of spreading harmful substances in the air depends on terrain and land use around the road [2].

Planting trees or shrubs next to a lane plays a key role in moving air masses. At greenfield sites there are better conditions where there is no greenery along the road. In built-up areas, greenery is a kind of barrier protecting buildings from pollution. Planted trees and shrubs are resistant to the harmful effects of exhaust fumes.

The main factor reducing the amount of pollutants discharged into the atmosphere is the implementation of the European Commission's more stringent emission regulations. They force manufacturers to constantly improve the motor engine design, for example, by installing catalysts [2]. The Commission is implementing even higher quality standards. In 2014 on the basis of Regulation 2007/715/EC a legal principle EURO 6 applying only to heavy goods vehicles was introduced. The maximal permissible amount of nitrogen oxides is 400 mg/kWh. Compared to the older EURO 5, it is about 80% less. Another way to reduce pollution inputs to the atmosphere is using engines with alternative energy sources (electricity, hydrogen). Vehicles equipped with such engines are environmentally friendly. The main way to restore the natural composition of the air in big cities is to replace passenger cars by public transport. The bus can carry approximately 20 times more people than a single car. Considering the amount of oxygen used by one car, the conclusion is obvious. The most favoured mode of transport is a bicycle, because it does not intoxicate the environment and makes us move, which is very important for the proper body functioning. Another solution could be designing safe crossroads. Around the crossroads fitted with the traffic lights during rush hours, the value of emission fumes at the junction is many times higher than the acceptable values. The use of grade separation completely solves this problem because it does not create congestion and cars are moving smoothly.

5.4.3. Noise protection

The traditional approach to the question of noise protection is based on the division of land affected by the impact of noise into three zones (Fig. 5.11) [2]:
- emission zone (source) (a),
- zone of preventive solutions (b),
- intensity zone (the recipient) (c).

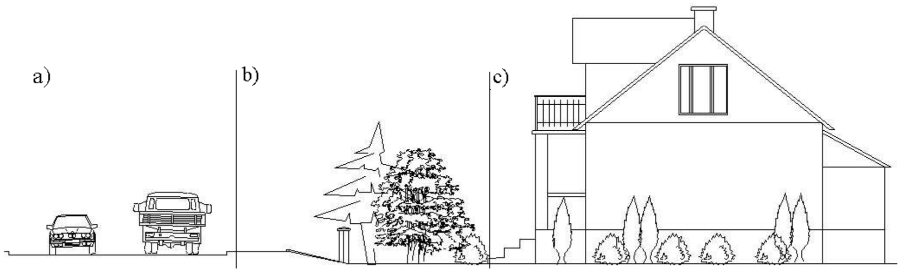

Fig. 5.11. Traditional approach to protection against noise [2]

This division unfortunately implies that the area that can be protected against the noise pollution is only the one in the middle of it. This results, of course, in applying safeguard measures only in those areas and mainly noise barriers, for that matter, which cannot always be used. This is due to technical and economic reasons (geometric parameters, location). On the basis of only two zones of emission and intensity can be distinguished and both can qualify as the area secured by protective solutions at the same time. This is illustrated in Fig. 5.12.

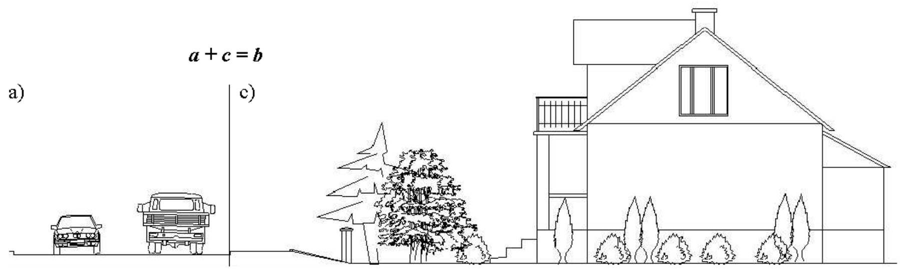

Fig. 5.12. Zone emissions and noise intensity and the area of security measures in the universal approach is a kind of protection against traffic noise [2]

This solution enables the use of safeguard measures also in the area of emission and concentration. A combination of the methods and means used in both zones contributes to the so-called cumulative effect of noise protection. In some cases, such an arrangement also protects inhabitants from air pollutants.

The emission zone is a potential source of noise. Actions in this area concern taking protective measures which can be applied on a section of the road. The noise is primarily generated by vehicles moving on the road.

The intensity zone is an area potentially threatened by the negative effects of noise. Protective measures should therefore enable the noise reduction to the acceptable level close to the plots.

Protective measures against the negative effects of noise can be divided according to the zone in which they are [2]:

1) Emission Zone:
   a. Vehicle and driver:
      – The construction of a vehicle, engine design, the type of tires,
      – driver's behaviour.
   b. Determining the geometrical axis of the road section and selecting its features:
      – the point of the road elevation (longitudinal),
      – the point of the road situation (location, surroundings),
      – the intersection of the road,
      – the type of pavement,
      – the intersecting and overlapping of roads and tunnels.
   c. Traffic management:
      – regulation of high traffic,
      – control of the vehicle structure,
      – regulation of the traffic flow,
      – traffic moderating.
2) Intensity zone:
   a. The devices placed on the way of the sound wave propagation between the source and the noise receiver:
      – noise barriers (in the form of a wall),
      – embankments (screens),
      – a merger of embankments and noise barriers,
      – central reservation,
      – Non-residential structure which is a form of residential security.
   b. The location and suitable configuration of a building along with its protective insulation from the effects of excessive noise:
      – designing building facades with built-in noise barriers,
      – keeping an appropriate distance between a building and the communication path,
      – changing the function of a building,
      – window replacement and wall insulation,
      – closing the gable situated perpendicularly towards the axis of the road.

5.4.4. Aquatic environment protection

Road transport has a negative impact both on surface and subsurface waters. This influence can be direct and indirect. Direct actions are related to the exploitation of vehicles and roads. Indirect actions concern air and land pollution. The most commonly used devices whose task is to drain rainwater out of the lane are trenches. When well shaped and made, they not only meet the above-mentioned requirements, but also reduce the groundwater and land pollution. The most commonly used types of ditches are [2]:

- grassy ditches (without security, and with geotextile filters),
- ditches with compartments (wooden palisades with riprap or concrete baffles with flow regulation).

Also, often used are tanks whose main task is to collect water, purify it and channel to the receivers.

There are several types of containers of the following functions:

- retention: collecting water runoff which is then channelled in a controlled way to consumers,
- retention and infiltration: fulfilling the same task as reservoirs and, in addition, enabling rainwater purification. Water infiltrates through the wall slopes and bottom, which generates a self-cleaning effect,
- evaporation (outflow): rainwater is evaporated,
- infiltration basins: similarly to retention and infiltration tanks they purify water, but cram with pollution. The bottom of a trough is impermeable;
- plant arcades, areas of impermeable grounds under cultivation: purification occurs at several stages. The purification occurs at several substrates, ie. By purification by means of degradation processes of aerobic and anaerobic, mechanical filtration through contamination in the ground and the chemical and physical bonding of harmful substances on particles of soil.

There are three basic devices reducing the amount of harmful substances in wastewater:

- separators of petroleum products – their task is to separate light liquids of density lesser than water, for example oil, gasoline,
- mud drums purifying rainwater or snowmelt and retaining solid particles such as gravel or sand,
- hermetic ditches preventing rainwater from soaking into the ground.

5.4.5. Soil protection

Soil is contaminated in two ways: aerial – pollution is present in the air and penetrates soil through precipitation. The most important and simultaneously the easiest ways to protect soils are [2]:

- planting crops that are immune to some chemical elements in the areas affected by pollution
- compounds,

- proper soil maintenance, for example: using green fertilizers, supplying with organic substances, calcium soil, which results in raising the pH value of the ground.

Under current law (Art. 141, Par. 1 EPL), operation of an installation or equipment should not exceed emission standards. At the same time, observing the emission standards, referred to in Art. 145 POŚ and other regulations, do not exempt from the obligation to maintain the environmental quality standards (Art. 144 paragraph. 4 POŚ)[5].

5.4.6. Emission standards and limits

The emission standards, more or less, refer to the emission limit values. The emission is defined as a direct of indirect introduction of substances or energy into the air, water or soil as a result of human activity and can take different forms: heat, noise, vibration and electromagnetic field.

A side effect of the continuous development of transportation is increasing penetration of harmful substances from vehicle exhaust into the atmosphere. On the basis of a series of European Directives, the European Union has introduced the so-called European Emission Standards in order to prevent degradation of the environment. In other words, it is a set of norms which contains the limit values for emissions of new vehicles that are sold throughout the European Union. If a vehicle does not meet the standards mentioned above, it cannot be sold in the EU. This provision does not apply to vehicles already in use. So far, six standards have been introduced. Every one contains a stricter limit. The following outlines the historical standards of EURO:

- EURO 1 – for cars and light trucks, from 1993 r.,
- EURO 2 – only for passenger cars, from 1996 r.,
- EURO 3 – for all types of vehicles, from 2000 r.,
- EURO 4 – for all types of vehicles, from 2005 r.,
- EURO 5 – for passenger cars and light commercial vehicles, from 2009 r.,
- EURO 6 (current) – for heavy vehicles, from 2014 r.

Tab. 5.1. The limit values for emissions generated by gasoline engines

Emission [g/km]	EURO 1	EURO 2	EURO 3	EURO 4	EURO 5	EURO 6
CO	2.720	2.200	2.300	1.000	1.000	1.000
HC	-	-	0.200	0.100	0.100	0.100
NOx	-	-	0.150	0.080	0.060	0.060
HC + NOx	0.970	0.500	-	-	-	-
PM	-	-	-	-	0.005	0.005

[5] Environment Protection Law Act of 27 April 2001 (POŚ) – Dz. U. No. 25/2008 r., Item 150, as amended – Art. 3, 76, 141, 144, 145, 147, 149, 150, 152, 169, 171, 186, 195, 201, 220, 221, 222, 224, 339, 365

Tab. 5.1 shows differences in the emission value limits for vehicles with petrol engines according to the above-mentioned standards.

5.5. Measuring, monitoring and identification of threats

5.5.1. Continuous and periodic measurements of the extent and distribution of harmful emissions

The essential matter for the environment protection are all the actions that help reduce and limit the contamination from large industrial plants, starting with the energetic combustion plants and steel factories through landfills to even poultry farms.

It is absolutely necessary to prevent environment pollution or reduce it by undertaking technical projects and introducing some systemic and organizational solutions in economic activities. Only in this way can we achieve a high level in protection of both human health and natural environment.

Continuous or periodic measurements of the emissions to the air are required [44]:

- for fuel combustion systems,
- for waste incineration and co-incineration plants,
- for production and processing of items containing asbestos,
- volatile organic compounds, where the regulations of the Environment Minister on emission standards are applied.

Fuel combustion systems

For fuel combustion systems, measurements of dust and sulphur dioxide are taken (depending on the type of material used for burning) – Fig. 5.13. The materials that can be used are the following: lignite, coal, peat (turf), fuel oil and biomass. We define biomass as a product composed entirely or partly of vegetal components coming from farming or forests. It can be used as fuel in order to recover its energetic content. The following biomasses are employed as fuels:

- Vegetable waste from food industry,
- Vegetable waste from original pulp and paper pulp production,
- Cork waste,
- timber, except for timber waste which can contain halogen organic compounds or heavy metal compounds as a result of preservation treatment or coating, especially wood waste from construction and demolition sites.

Continuous air emission measurements are taken for fuel combustion systems of combined nominal heating power (output) not less than 100 MW. Periodic air emission measurements are taken for fuel combustion systems of combined nominal heating power (output) less than 100 MW.

Periodic measurements of air emissions are made twice a year, once in winter (October – March) and once in summer (April – September).

Fig. 5.13. Dusts and sulphur dioxides spreading into the atmosphere[6]

For the systems that work temporarily for less than six months, air emissions are measured once a year during operation time [44].

Waste incineration and co-incineration plants

Although waste is a source of the environment pollution it is also a secondary raw material that can be used by some branches of industry (Fig. 5.14).

Fig. 5.14. Processed waste[7]

For incineration and co-incineration plants, the levels of hydrochloride, hydrofluoride and sulphur dioxide are measured.

Continuous and periodic air emission measurements are taken for all incineration and co-incineration plants.

Air emissions are measured periodically at least once in six months, and in the first year of the operation, at least once every three months.

Emission measurements are not made for the plants where only the following waste is incinerated or co-incinerated:

- Vegetable waste from farming and forestry,
- Vegetable waste from food industry if the generated heat energy is recovered,

[6] http://www.euroinfrastructure.eu/wp-content/uploads/2012/10/dym.jpg
[7] http://blog.gsenergia.pl/energia-z-odpadow-czyli-paliwo-alternatywne

- Vegetal fiber waste from original cellulose pulp and paper pulp production if the waste is incinerated where it is generated, and the energy that is produced during this process is recovered,
- Chipboard, unless it is hazardous waste,
- cork,
- timber, with the exception of the timber contaminated with impregnants and protective coatings which can contain halogenated organic compounds or heavy metals, and wood waste from construction or demolition sites,
- radioactive,
- waste from search and mining of oil and natural gas on platforms and incinerated on the platforms,
- animal remains.

Systems for production and processing of items containing asbestos

Asbestos is a highly hazardous material but until it is left untouched there is no danger. If we begin to move it or drill in it, the asbestos fibers can be extremely dangerous for our health (Fig. 5.15).

Fig. 5.15. Asbestos cement board roof covering[8]

Levels of air emissions are taken periodically for the production and processing of asbestos containing items which have been granted an authorization to be produced, marketed and imported, according to the regulation about the ban on products containing asbestos, if the amount of the raw asbestos used in these processes exceeds 100 kg/year.

These are taken twice a year, or once a year if the results of ten consecutive measurements do not differ by more than 5%.

Volatile organic compounds

Volatile organic compounds (VOC) – Fig. 5.16 – is a large group of organic substances which, when in the air, bring about a number of negative effects both

[8] http://blog.gsenergia.pl/energia-z-odpadow-czyli-paliwo-alternatywne

on the environment and human health. VOC largely affect the levels of ozone concentration in the atmospheric air [37].

Fig. 5.16. Volatile organic compounds[9]

Continuous or periodic VOC emission measurements are taken for the systems where organic solvents are employed.

Continuous measurements are made if the amount of VOC from one emitter is not less than 10kg/hour on average, on total organic carbon emission (TOC) basis.

Periodic (once a year) measurements of VOC air emission are made if the amount of VOC from one emitter is not more than 10kg/hour on average, on total organic carbon emission (TOC) basis.

Sewage

The measurements of the quantity and characteristics of the sewage that is conveyed into the water or the ground are made when the sewage comes from special use of waters, that is, the use that goes beyond the common or ordinary one like intake and runoff of surface or underground waters, draining sewage into the water or the ground, using water for energy, extracting stones, gravel, sand and other materials from the water [45].

Surface water

The measurements of surface or underground water consumption are made when the nominal demand of all the installations located on the site is higher than 100 m^3 per 24 hours.

Noise

Periodic measurements of the noise levels in the environment are illustrated by noise indicators, which are applied to establish and monitor the conditions of

[9] http://www.posventa.com/es/notices/2015/06/proteccion-respiratoria-necesaria-para-la-salud-del-pintor-52393.php#.WGts-FXhC70

operating in the environment (LAeq D and LAeq N). The measurements are made for the plants where there are installations generating noise, if an authorization for noise emission or a decision on the acceptable noise levels or an integrated authorization have been issued.

Periodic noise level measurements in the environment, including impulse noise, are made once a year, taking into account the operational characteristics of the noise sources [44].

5.5.2. Environment monitoring in the surrounding of bridges

Currently, there is a tendency to monitor the strength of the bridge construction using the wireless methods, which allows collecting the data of structure deflections and derivatives thereof, seeing [8], [34] for instance. These techniques themselves are substantially complementary to in situ studies. This is the result of significant construction costs of a transmission cable system. On the other hand, especially on highways there are optical devices and sensors mounted that monitor air temperature and humidity as well as the road surface state. Outside the scope of sensory techniques, it is important to storage the collected data, in particular the software quality for data systematization is essential. In the environmental monitoring the method used is similar but on a less advanced scale [57].

Current observations are conducted in an environment of selected objects recognized as interesting and potentially important in terms of environmental sustainability. There are many studies whose aim is mainly to confirm the applied solutions in the case of animal transitions that are transitions for the animals. From the point of view of a road engineer who systematically studies numerous publications in this field, it results that there are no clear general conclusions to be seen. The identified partial positive or negative cases are generally consistent with intuitive feelings [3].

The events in time are recognized and recorded, mostly as photographs. The information emerging on the basis of written reports is not available within the field. Partial images resulting from scientific publications do not give rise to practical use in engineering. Those still have been studied group of animals with the presumption that the population separation occurs due to obstacles in the form of roads. So far, no publication has been found in which the presumption of this state has been negated. In this respect the researcher rank is rather convenient. It is not necessary to formulate en engineering proposals, which always have their financial consequences. On the other hand, time runs fast. There are new bridges built and the existing ones rebuilt in accordance with environmental legislation which, in fact. is only seemingly friendly to nature.

Across animal routes bridges can perform the gateways or orifices. Here functioning of an extended look at monitoring the environment in their neighborhood is proposed. The aim is to identify scientifically and in an engineering way at the same time which will indicate quantitatively measurable phenomenon. It is assumed that the quantity of identification will allow the use of statistical evaluation leading to technical proposals, which will be used to modify the applied construction solutions. To implement such a program the following objectives should be considered:

- to standardize the method of recording the results of the environment measurements,
- to provide everybody with the accessibility to the all environmental document archives,
- at the same time to discuss and produce simple and reasonable number of some basic criteria for the analysis of environmental materials,
- to use the digital recording of measurements and publish them along with reports,
- to apply hitherto proven methods for measurement, including, in particular, those basing on photo registration,
- to extend the research on the issue of sounds within the frequency range potentially important and those used by animals,
- to identify the range of sounds generated by bridge structures with their identification in terms of friendly/unfriendly to animals,
- to lead identification of odors at junctions to define the states of friendly/unfriendly,
- to define acoustic and olfactory stimuli potentially allowing to control animal behavior,
- to formulate conclusions appropriate to engineering applications.

To enter the program described above, the first step is to enter a new bridge project element i.e. to design a monitoring system of the bridge ambient.

Bridge monitoring means periodical inspections i.e. monthly, quarterly, annual or special – every five years. Each inspection is focused on the technical state of a bride. Inspections, in detail, cover:

- the technical condition of the road surface on a bridge and its access as well as the pavement,
- the condition of the traffic safety markings, vertical and horizontal,
- the technical condition of a superstructure and bearing elements,
- the technical condition of the riverbed and road embankments in the vicinity of a bridge.

As can be seen from the above description of the diagnosis scope of the bridge condition, conducted by a bridge inspector, may be extended to ecological issues. In this case, additional tasks are:

- identification of animal footprints and the migration of amphibians,
- identification of the river fish species and their volume,
- taking water samples to determine its purity and test it for the presence of harmful substances,

- identification of the wealth of flora, in particular protected species,
- measurement of noise and the emission of pollutants into the atmosphere.

The observed and measured results may be collected into a special eco-report or into an extended bridge protocol. In such circumstances, bridge inspectors must be additionally trained in the field of environmental issues and automatically become bridge-environmental inspectors. Just as in the bridge reports, containing a so-called insight sheet and a recommendation sheet, reports of the results of environmental inspection could contain similar pages. The strength of this solution is that it is easier to expand a bridge inspector's knowledge by environmental tasks than vice versa, i.e. to train an environmental inspector in bridge issues.

Reviewing numerous scripts and manuals, e.g. [9], [18], [27], designed for bridge inspectors, one may think that there is nothing simpler than evaluating the technical condition of bridges. However, it is not the case. An inspection shows some levels of different examination. Every inspection should be recorded in a number of reports. Reports contain the bridge data which become the history of the bridge in question. In Poland, an elementary check of the bridge condition is known as annual review which is connected with so called 'current overview' conducted occasionally, when a road patrol crosses a bridge. On the basis of the conclusions of elementary inspection reports, a decision about performing an extended report may be taken. Optionally, every 5 years a detailed inspection may or must be carried out. In this case, a bridge is subject to a thorough diagnosis in terms of its carrying capacity as a whole and its individual elements, material components, conditions and operation of the bridge equipment, the state of corrosion, cracks, spalling, etc. An opinion on the current state as well as the conclusion on the form of maintenance, repairs and a date of the next inspection as well as its type must be presented in a report form. Actually, these systematic and reliable assessments constitute a generally efficient procedure helping to maintain good bridge conditions. The maintenance processes are heavily dependent on the financial availability of the bridge administration responsible for starting bridge corrective actions. This is the weakest link in the combined processes of the bridge condition diagnosis and its possible repair.

The determination of bridge durability on the elementary and extended level of inspection based on the non-destructive testing (NDT) techniques, mostly by hammer testing of the surface hardness, Pull-Out and Pull-Off. The major advantage of NDT methods has been recognized as their capability to test in situ. A great deal of expertise is required for an interpretation of NDT field observations and test results. According to EN 13791 [46], NDT methods must be referred to a more reliable assessment i.e. material core testing for compressive strength. In the case of concrete, the standard EN 13791 regulated the concrete assessment providing clear references with regard to determining the class of concrete compressive strength.

Among NDT one is of a major significance. It is the proof load test that should be conducted using design loads. Only in this way it is possible to diagnose the bridge toughness, material and design quality. A spectacular example took place during the proof test of the cable-stayed Novi Most in Bratislava in 1972. There occurred cracks between the box carrying deck and the pedestrian gangway plate, which were really a secondary problem. Probably, due to its minor importance it was omitted during the design stage. Since the corrections the whole bridge has been serving reliably till now.

In Poland the corrosion of RC and structural steel is the main cause of bridge material defects. Due to this, a well-functioning hydro-isolation is a necessary condition for limiting corrosion of any type. As a consequence, reporting on any water leakage symptoms are strongly required.

The more advanced testing methods are depicted in many papers. Here, for instance, the [22] is suggested.

5.6. Modern measurement capabilities

5.6.1. Introduction

Measurement automation of bridges is introduced to provide constant monitoring of the technical and service condition.

Because of the introduction of new materials, the construction of innovative bridges, and the employment of new design codes or practices, there is a need to search for additional bridge structural behaviour and safety information. Often, maintenance plans for progressively aged bridges are inadequate. A consistent safety evaluation methodology is much required. Furthermore, bridges demand an integrated surveillance strategy during their service life. Structural health monitoring (SHM) is a proper tool to obtain useful information aimed at the safety and maintenance of structures. The most widespread monitoring scheme of bridges is to undertake visual inspections.

Bridge monitoring is mainly focused on the technical condition of individual elements of an object, the condition of the entire structure and its behaviour during service under external or internal loads. The impact of wind, temperature, live loads (including the deflection and dynamics), scour of foundations, subsidence and cracks could be monitored.

The monitoring system could be designed to provide data from dynamic and static loading parameters. Response parameters refer to the tri-axial state of strains in a concrete deck, strains and bending moments as well as the shearing stresses and inclinations of steel girders, forces at the diaphragm members, expansion and contraction of bridge ends, strains and stresses in rebars, the crack formation in a concrete deck and inclinations of abutments and piers. Based on strain measurements, stresses in a concrete deck, bending moments and shearing stresses in steel girders, axial forces in the diaphragm members, and stresses in the reinforcing rebars can be calculated. Loading parameters include long-term effects such as seasonal and diurnal climatic changes, and dynamic effects such as traffic loads [31].

The scour of foundations is one of the main causes of the bridge collapse. During the last 30 years, 600 bridges have failed due to scour problems causing major operating disruption and financial losses [24]. Scour can be defined as the excavation and removal of material from the beds and banks of streams as a result of the erosive action of flowing water. Scour occurs in three main forms [24]:

- general scour, occurs naturally in river channels,
- contraction scour, occurs as a result of the reduction in the channel's cross-sectional area that arises due to the construction of structures such as bridge piers and abutments,
- local scour, occurs around individual bridge piers and abutments.

A generated scour hole has the effect of reducing the stiffness of foundation systems and can cause bridge piers to collapse without warning. Notable bridge failures due to scour in Europe include the failure of the Sava bridge in Zagreb and the collapse of the Malahide viaduct in Dublin [24].

Under the influence of wind an object can be set in motion. The structure can be subjected to additional stress. With a complex state of stress or improper dynamic behaviour, the object may be damaged.

Under the influence of temperature changes, a structure suffers strains. Uneven heating or cooling can affect the high difference strain level of structural elements, causing stresses or even the structure damage.

The problem of the general overload of vehicles causing premature failure of bridges and roads is becoming more and more obvious. Structural behaviours of bridges and roads need a real-time monitoring and diagnosis, timely damage detection, safety evaluation and necessary precautions in order to prevent accidents such as the cracking or collapsing of bridges and roads [36].

The basic method of the structure health monitoring is an in-situ inspection. Visual inspections rely on a non-invasive statement of object faults on the basis of a local vision. On the basis of apparent defects described in the revision protocol, the technical state of an object is assessed. If it is necessary, a further diagnosis of an object with invasive methods is performed. Modern methods of monitoring the technical condition of objects consist in a permanent analysis of the results obtained from sensors placed within the object and in its area. Some new structural health monitoring methods regarding bridges are described below.

Close range photogrammetry (CRP) – an alternative for traditional deformation monitoring techniques.

The fiber-optic monitoring system. Standard structural health monitoring (SHM) systems are based on the use of point sensors (e.g., strain gauges, crack-meters, tiltmeters, etc.). The fiber-optic monitoring system is based on the use of distributed optical fiber sensors, in which the whole structure is monitored by the use of a single optical fiber. In particular, distributed optical fiber sensors based on the stimulated Brillouin scattering (SBS) permit detection of strain in a fully distributed manner with a spatial resolution in the meter or submeter range, and a sensing length that can reach tens of km. When the sensors are opportunely installed on the most significant structural members, this system can lead to the comprehension of the real static behaviour of the structure rather than merely measuring the punctual strain level on one of its members [17]. Its small size, high precision and stability, the provision of a linear response to strain and temperature in a wide range, as well as the electrically passive operation, the electromagnetic field immunity and the possibility of being inserted in large sensor networks adopting multiplexing configurations are the most significant advantages associated with the fiber optic sensing technology [25].

The wireless monitoring system architecture deploys accelerometers and tactile sensors on the bridge deck to measure deck accelerations and the time when the vehicle crosses fixed points, respectively. Simultaneously, wireless sensors installed in a vehicle are used to measure the vehicle's horizontal acceleration, vertical vibratory acceleration, and gyroscopic pitching motion [11].

The Acoustic Emission (AE) technique uses the dynamic response of a structure to the traffic load. As a result, vehicles become an excitation signal. The AE equipment monitors the response of the bridge excitation. Acoustic energy is emitted from the materials inside a bridge when there is a movement resulting from load changes. A number of acoustic sensors is used to record the responses and facilitate the location of the noise sources in the structure. The AE technique is widely used in Japan for the in-situ monitoring of reinforced concrete bridges. AE has the potential to provide a cost effective condition monitoring of bridges because it does not disrupt the flow of traffic or require the closure of the bridge, it is relatively quick and easy to install, it is not labour-intensive, provides continuous monitoring, and it can be installed in existing structures as well as adapted to the existing topologies [30].

5.6.2. Sensing elements

Scour monitoring using depth-measuring instrumentation. A choice of instrumentation has been developed to monitor the scour hole development. These instruments can be broadly categorized as follows [24]:

- Single-use devices consist of float-out devices and tethered buried switches that can detect scour at the locations of their installation.
- Pulse or radar devices utilize radar signals or electromagnetic pulses to determine changes in the material properties that occur when a signal is propagated through a changing physical medium.
- Fiber-Bragg grating sensors are a form of the piezoelectric device. These types of sensors operate based on the concept of measuring strain along

embedded cantilever rods to generate electrical signals, which can indicate the progression of scour along the rod.

- Buried or driven rod systems include such systems as the magnetic sliding collar, the "Scubamouse", the Wallingford "Tell-Tail" device and mercury tip switches.
- Sound wave devices such as:
  - Sonic fathometers,
  - Reflection seismic profilers,
  - Echo sounders,
  - Electrical conductivity devices.

Scour monitoring using changes in structural dynamic properties. To date, in scour monitoring underwater instrumentation to measure the progression of the scour depth in time has mostly used. A limited research has been undertaken to consider the effect of scour on the response of the bridge structure itself. Some of the instruments developed to measure the response of a bridge structure to scour include tiltmeters and accelerometers.

Tiltmeters, also known as inclinometers, measure the relative rotation of a structural element and as such can be used to detect differential settlement, which can occur as a result of the scour process. The only major disadvantage of the device is that it does not give a direct indication of scour depth.

Accelerometers enable the measurement of the structural response, particularly to a change in the boundary conditions. The soil structure interaction process is complex during scour, however. The removal of the material under (or around) a foundation during scour will cause increased stress and consequently reduced stiffness of the remaining soil. Since the frequency of the vibration of a structure depends on the system stiffness, observing changes in vibration frequencies is a potential method for damage identification and health monitoring.

To obtain information on the **performance and condition** of a cable-stayed or suspended bridge sensing elements can be used. Sensors monitor cable loads, structural and environmental temperatures as well as wind speed and profile. Such elements include [6]:

- anemometers to measure wind speed and profile,
- a fluid pressure level – based sensing system to measure deck vertical displacement;
- temperature sensors for the main cable, deck steelwork and air temperature;
- extensometers and resistance strain gauges to determine loads in additional cables.

Piezoelectric sensors are used in measuring the traffic spectrum. The weight-in-motion (WIM) system is usually installed at the entrance of a bridge and provides continuous time histories of axle loads moving on the roadway. The WIM system is designed to provide data, including the weight of the wheels, current speed, number of axles for every vehicle, and the distance between the axles as vehicles travel at highway speeds [31].

The **close range photogrammetry** technique utilizes a digital single lens reflex camera. Continuous data recording by the GPS satellites, ground-based receivers and robust telemetry can be used for monitoring the health of engineered structures [31].

A **ground penetrating radar** transmits high-frequency electro-magnetic waves out through specialized antennas. Waves reach and often penetrate into the surfaces of solids. A part of the waves reflects back to the receiving antenna. The arrival time, the shape and amplitude of the back wave relates to the location and the characteristics of the dielectric material [31].

Fiber-optic sensors. Fiber-optic sensors (FOSs) have the ability to modulate some properties of the light that is emitted by a source into the core of a fiber. This modulation can be caused by changes in strain, temperature and pressure experienced by the sensor through which the light travels. In consequence, an optical signal is generated and reflected towards a demodulation device to be translated into a measurement of the gauged quantity [5].

The fiber-optic sensors could be employed in the measurement of several parameters of bridge structures, such as strain, displacement, pressure, load, acceleration, rotation, temperature, concrete cracking and reinforcement corrosion monitoring. The fiber-optic strain sensors are particularly useful in the strain measurement because they can be short-gauge, long-gauge or distributed solutions depending on the sensing principle, sensor configuration and installation procedure [25].

The **Fiber Bragg Gratings** sensors operate on the basis of the information conveyed by the change of the wavelength of the light reflected by the FBG grating when illuminated by a broadband light source. Strain or temperature variations have an impact on the fiber refractive index, as well as on the grating period, thus inducing variations in the wavelength reflected by the sensor [26].

The **Fabry–Perot (FP)** sensor is an interferometer composed of two semi-reflective mirrors positioned at a certain distance from each other, termed the FP cavity. A major advantage of the FP sensor is that it can be easily produced to be temperature self-compensated taking into account the coefficient of the thermal expansion of the host material, and therefore strain readings are stress-induced only [5].

The **SOFO** (Surveillance d'Ouvrages par Fibre Optiques) sensor is a long gage device which can be defined as a double Michelson fiber-optic interferometer arrangement. One of the fibers is prestrained and acts as a sensing arm attached to the host structure, whereas the other is used as a reference being of a well-known length. Since both fibers are located side by side, they experience the same temperature variation, thus making the sensor temperature insensitive. Contrary to the Michelson interferometer, the SOFO system uses a broadband source as the light source, typically a LED or a SLED [5].

Wireless sensor nodes (WSN). A WSN consists of several to thousands of wireless sensor nodes joined into a wireless communication network. The principle is to set up several sensors in a specific area to collect the environmental parameters, and, by means of a multiple-hopping relay, the parameter data is transmitted to the control centre so that remote monitoring staffs can access the relevant data by way of the control centre. Therefore, in a WSN nodes can be added and removed conveniently whenever it is necessary to achieve the arrangement of a low cost and the flexibility of the layout. A WSN mainly utilizes adjacent wireless sensor nodes to accomplish data transmission. Each node has its own intensity and coverage of radio wave [13].

The **Polyvinylidene Fluoride (PVDF)** sensors can be used in surface monitoring with high sensitivity and fast response. Because of the advantages of their good compatibility with substrate material, little influence on the structural properties, and the anti-interference ability, the PVDF sensors present excellent properties in strain, acceleration, force and other physical quantities. Therefore, the PVDF traffic sensors are very suitable for a dynamic weighing system [36].

The **LVDT sensors.** Linear Variable Differential Transformer is a linear displacement sensor, a transformer of a differential circuit with a sliding core. Such sensors can measure displacements in the range from several micrometres to several centimetres. The parameters of these sensors are strongly influenced by their construction. The LVDT sensor supply frequency falls within the range of tens of Hz to several kHz.

The **electromagnetic (EM) sensors** are based on the principle of the electromagnetic surface wave propagation and consist of a thin layer of the wave-guiding media attached to a metal base (e.g., steel bridge elements) and an EM wave emitter and receiver. The fundamental concept of the sensor is similar to that of the fiber-optic sensor, apart from the fact that the fiber-optic sensors usually detect the phase change of an optical wave [20].

5.6.3. GPS stations

Constantly increasing traffic results in the deficiency of bridges. With the growing complexity and costs of large-scale bridges, the construction and maintenance have become more important. The aging of bridges and bigger average loads than predicted during the design stage have significantly increased the need to monitor bridge performance. An early identification of the possible damages of existing structures has become necessary. It enables to maximize the lifespans of these bridges thanks to the maintenance and repair work at the initial damage phase, at minimum costs. Conventional monitoring methods i.e. accelerometer, tilt meters, strain gauges, optical devices and survey equipment mentioned above have been employed.

Unlike other sensors, Global Positioning System (GPS) is readily able to provide three-dimensional absolute position information at the rate of 20 Hz and higher, if necessary. It provides an opportunity to monitor the dynamic characteristics of a structure to which a GPS antenna is attached. The rapid advancement of the GPS device and algorithms have also enabled the monitoring of bridges in continuous real-time, that is commonly referred to as Real Time Kinematic (RTK).

RTK-GPS is now actively applied to measure static, quasi-static and dynamic displacement responses of a large civil engineering structure under different loads due to its global coverage and continuous operation under all meteorological conditions. This enables the analysis of the frequency response and (by using multiple receivers in strategic locations) the dynamics of the vertical profile. Attempts have also been made to use RTK-GPS alone to monitor very long span suspension bridges due to the shortcomings of an accelerometer used to measure the slow structural movement with a vibration frequency lower than 0.2 Hz.

The Global Positioning System (GPS) technology exhibits several advantages such as an easiness to obtain absolute displacement measurements, weather independence, autonomous operation, and not requiring a line-of-sight between target points. This provides a great opportunity to monitor, in real-time, the displacement or deflection behaviour of bridges under different loading conditions through an automated change detection and alarm notification procedures [35].

The GPS technology offers real-time solutions with relatively high precision. There is an obvious potential for GPS to provide high-frequency information. With the capability of measuring low frequencies making it a suitable technology for bridge health monitoring applications.

5.6.4. Video monitoring

A video surveillance system (VA) can be used to constantly monitor the movement and weather conditions occurring on bridges. Such a system consists among others of speed dome cameras. Video images are transmitted directly from a video surveillance point to the monitoring centre. Such systems are used on dual carriageways, motorways and in big city centres in order to improve traffic surveillance, as well as to observe the weather conditions and to identify collisions and congestion [12].

[7] presented the vibration and displacement monitoring of civil engineering structures using Image Assisted Total Stations (IATS) and passive target markings. By means of the telescope camera of a total station, it is possible to capture video streams in real time. Due to the high angular resolution resulting from the 30×optical magnification of the telescope, monitoring can cover large areas. The laser distance measurement unit integrated in a total station allows to precisely set the camera's focus position and to relate the angular quantities gained from the image processing to the units of length. To accurately measure the vibrations and displacements of civil engineering structures, circular target markings rigidly attached to an object were used by the researchers.

5.7. Load tests used in bridge monitoring

5.7.1. The essence of the acceptance tests

Since bridges were built custom carrying load test was accompanied by the completion of construction of every major object. In the past, the load test was carried out without additional measurements [21]. This test did not check the operational suitability and safety of a bridge in full, and existing reserve of security. For a more specific assessment of the actual characteristics of currently constructed bridges during such load measurements, among other things, a determination of displacements and deformations, a possible crack width, frequency and the amplitude of vibration should be performed as well identification of the actual properties of a material. The obtained results compared to such values determined analytically give a complete picture of the actual behaviour of the structure.

Except for the ability to compare the theoretical results with actual ones, a load test should include:

- assessment of the construction of the whole structure,
- the calculation model check,
- evaluation of the condition of the object,
- fitting the structure to the operational requirements,
- disclosure of the hidden defects of the design,
- verification of the performance requirements (deflection, vibration damping, etc.),
- determination of the critical speeds,
- preparation of a report which will serve as the basis for the comparisons of object at any service testing.

It should be remembered that tensile test results, obtained during the construction of bridges, change during lifetime (this is particularly true of concrete structures). These changes can be quite easily discerned in the course of checks carried out by comparing values obtained from research carried out during a trial load. The acceptance testing of bridge superstructures allows to measure the corresponding quantities for the actual construction, which are useful for evaluating the safety of the bridge during use.

Additionally, using the same procedures for make an load tests, can be accumulate and systematize information on the behaviour of different types of load-carrying structure. Formed in this way, the database contains valuable information on the actual construction work. In the future it may be used by bridge inspectors, as well as other persons performing an expertise.

Test loads are also used in assessing the carrying capacity of existing bridges. The verification study is carried out by gradually increasing the load to a predetermined value. When an object does not exhibit unwanted behaviours (e.g. scratches) such a load is regarded as safe. The value of this load, divided by an appropriate safety factor, becomes the transportation permissible load for a given bridge.

Another purpose of carrying out the load test is to verify the established way to strengthen a rebuilt bridge. After completing a repair or reconstruction of an object, a test load should be performed. The results of such an assessment will confirm whether or not the structural reinforcement is efficient, which often for technical reasons it is unusual.

A wide range of the applications of the load test allows not only to verify the computational model of a structure and confirm the proposed security stocks but also to better understand its actual behaviour. The test load is especially valuable in the case of unusual innovative design and material solutions. It can then provide information confirming or improving the theoretical analysis [19].

5.7.2. Types of load testing

The basic application of the load test is a test of new objects before approving them for use. Another one concerns the diagnosis of existing bridges [14]. Therefore, depending on the subject of a study, three different types can be distinguished [16]:

- acceptance tests of new objects,
- supplementary load tests of existing objects in use (old),
- load tests of rebuilt structures.

Whereas, with regard to the way of the implementation of a test load, the test can be conducted as follows:

- under static load,
- under varying loads, called dynamic testing or load utility.

During testing, you can use static and dynamic loads. During acceptance testing of new bridges should always be use static and dynamic loads.

The static load test can help determine the behaviour of a structure without the influence of load changes. It is the basis for the determination of the nature of the operating range of a structure (linear elastic, etc.) and the transverse and longitudinal rigidity, as well as the degree of effort.

The dynamic load test gives the opportunity to evaluate the properties of a structure which are difficult to determine by means of a theoretical analysis. Testing by dynamic load varies over time it is used in evaluating the dynamic performance of the structure. It can be used in all types of research using the load test. The results, which are a measure of the stiffness, are useful to confirm the design calculations, or alternatively, may provide a basis for the comparison of the results of a research carried out at a later date. Lowering of the stiffness over time is a sign of deterioration or a serious structural damage. At the same time, the dynamic load test may be the only study to determine the safe speed load (critical speed). The dynamic load test level is much lower than useful load.

It depends on the "sensivity of a dynamic" and the sensitivity of the measuring equipment used.

Tab. 5.2. Criterion differences in formal documents

	PN-89 S-10050 steel bridges	PN-99 S-10040 concrete bridges	Regulation GDDKiA road bridges	Instruction Id-16/2005 railway bridges	Instruction Id-16/2014 railway bridges	Standards PKP railway bridges	Polish Centre for Accreditation
Static tests							
(1)	L>21m	L>20m	L≥20m	not applicable	not applicable	not applicable	L≥20m
(2)	L>21m	all	not applicable	all	not applicable	all	all
(3)	absence	Not tested	L≥20m	not applicable	not applicable	not applicable	L≥20m
Dynamic tests							
(1)	L>21m	Not tested	L≥20m	not applicable	not applicable	not applicable	L≥20m
(2)	L>21m	L>15m	not applicable	st. L>21m, con. L>10m	not applicable	L>21m	L>21m
(3)	absence	not tested	L≥20m	not applicable	not applicable	not applicable	L≥20m

(1) road bridges,
(2) railway bridges,
(3) footbridges.

The formal framework of the load test in Poland is regulated by Decree No. 47 [55] in terms of road bridges and Technical Standards [54] in terms of railway bridges. These documents refer to the old standards of approval [52] and [53], which do not have their counterparts in the Eurocodes [28]. In the case of railway objects, Instruction Id-16 has been introduced [48]. However, after the introduction of its amendment in 2014 [49], under the current wording it does not refer to the load test, but only to the maintenance of bridges. That is why the binding documents regarding the load testing of railway bridges should be narrowed to [54]. Currently, in order to harmonize the requirements for the accreditation of laboratories performing the test load, a new document PCA has been developed which can significantly affect the standardization of the initial inspection of bridges [4], [29]. Still the main problem is to decide which objects (from the point of view of the formal requirements), should be subject to a trial load. Although all of the documents use a simple criterion span, it is very diverse – Tab. 5.2.

According to the referenced document, extortion used in static studies should trigger from 75% to 100% of the effects characteristic of a particular class of the useful load [51] (typically, the bending moments). It is also permissible load levels below 75% for object, which overload to the required level is impossible due to the available load, which often happens in the case of railway bridges. This research is always accompanied by ambient-temperature measurements and design elements whose change significantly affects the behaviour of an object during the test.

The approach to the use of test loads in European countries varies to a high degree. In Tab. 5.3 presents the scope of load tests in 11 European countries [14].

Tab. 5.3. Areas of the application of the load test in various European countries

Country	Testing of bridges before putting into service			Research work	Evaluation of transport capacity
	New bridges	Prototype construc-tions	Modernized or reinforced bridges		
Austria	–	–	–	+	–
Czech Republic	+/–	+	+/–	+	+/–
Denmark	–	–	–	+	–
Germany	–	+/–	–	+	+
Italy	+	+	+/–	+	+
Netherlands	–	–	–	+	–
Norway	–	+/–	–	+	–
Slovenia	+	+	+	+	+
Spain	+	+	+/–	+	+
Switzerland	+/–	+/–	–	+	+
Great Britain	–	–	–	+	+

Whereby the duty load test regards the following:

- in Slovenia, railway bridges with spans in excess of 10 m spans and more than 15 meters for road bridges,
- in Spain, the construction of spans over 10 m,
- and in Switzerland (until 1999), railway bridges with spans over 10 m and for road bridges with spans over 10 m.

5.7.3. Load test project

Both the design and execution of a load test should be carried out in accordance with uniform testing procedures. As part of their includes, among others, the selection of vehicles and the load scheme, determination of the type of data points, an analysis ofthe real stiffness of a span and the identification of the basic parameters of the dynamic construction [23].

Usually, in projects load test is used flat rod-slab model or a three-dimensional shells models and rod-shell (Fig. 5.17). More complex models can also be used, e.g. a solid shell (Fig. 5.18). Depending on the type of the load-carrying structure models are selected to reflect the geometry of not only the structure but also the flexural and torsional stiffness of the entire system, which later are verified by measuring the deflection (spans).

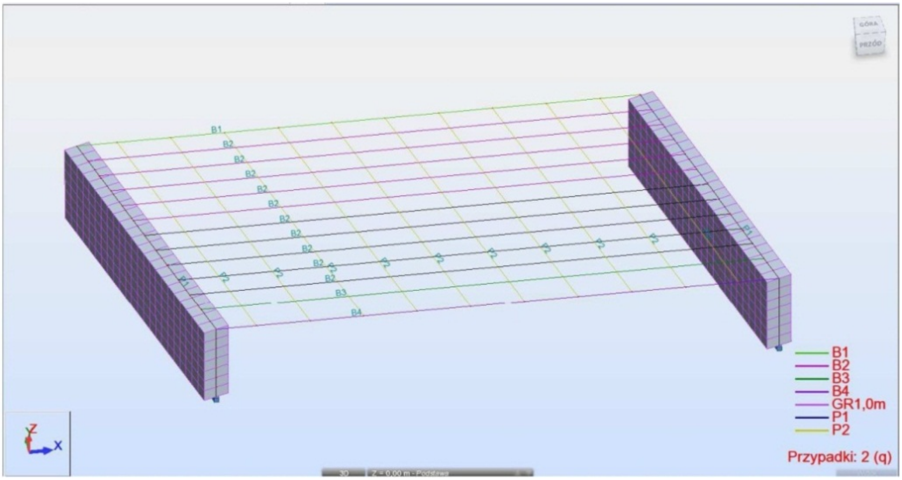

Fig. 5.17. Visualization of the rod-shell computational model

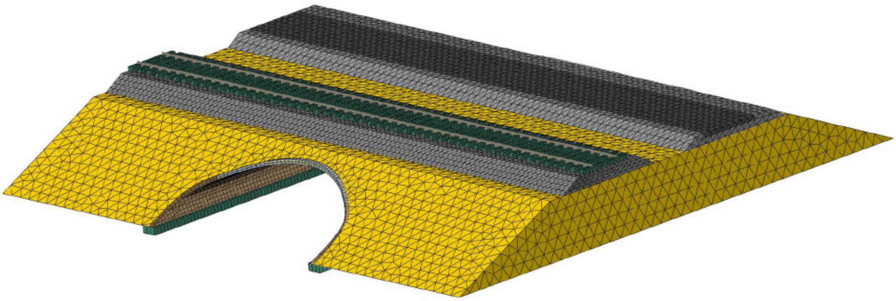

Fig. 5.18. Visualization of the solid-shell computational model [32]

At the design stage there is a computational model the two parts of which (the load model and the geometrical model) are known in sufficient detail to eliminate substantial differences during tests. Whereas, the material model, in particular in the case of concrete structures, is dependent on many variables, e.g. time and the type of aggregate [15]. In addition, every of this kind project before the test is subject to approval by the designer. That is why the load test project, as indicated in [15], is called the Preliminary Test Load Model of standard characteristics of the material. It makes it possible to eliminate the possible errors of calculation by verifying the results of the model adopted by the designer and the research team.

In the report of the test load, obtained in concrete differences relationship between the measured deflection and design deflection are reviewed based on the so-called. Verified Model Test Load. In this model the actual value of the characteristics of concrete, obtained during laboratory strength tests, is taken into account. Fig. 5.19 shows a diagram of modelling procedures in the study of real objects under the test load of bridges.

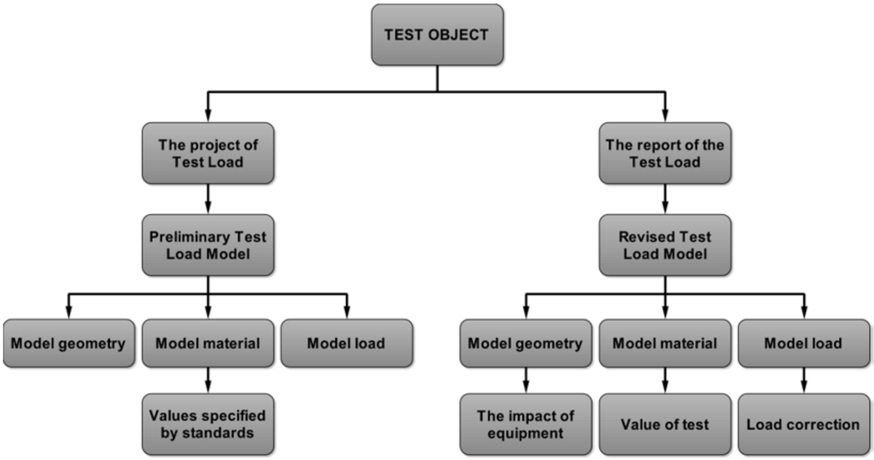

Fig. 5.19. Procedure for verifying the calculation model in the load tests of bridges

In preparing a dynamic test program is conducted modal analysis model, which previously was used in the static analysis. However, it is supplemented by the mass distribution. The results of a modal analysis (frequencies and mode shapes) are the basis for the determination of measurement methods and the location of sensors (Fig. 5.20 and Fig. 5.21).

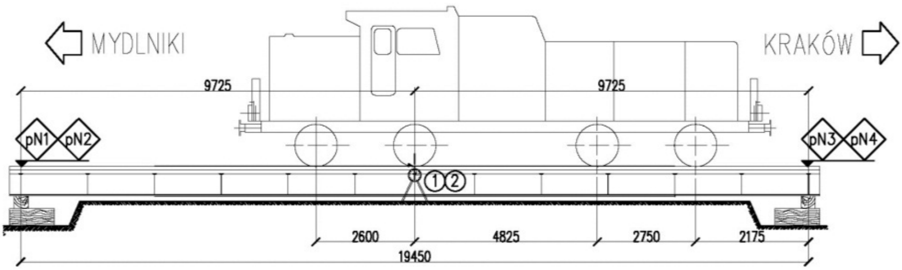

Fig. 5.20. Scheme of the measurement point arrangement in the longitudinal section

It takes into account the specificities of a structure, availability of the space under the object and the traffic organization both on and below it. In the case of railway objects on lines with high speeds of above 200 km/h, the additional time analysis is performed aiming to determine the acceleration of structural elements. In this type of analysis actual train and HSML models are used according to [47].

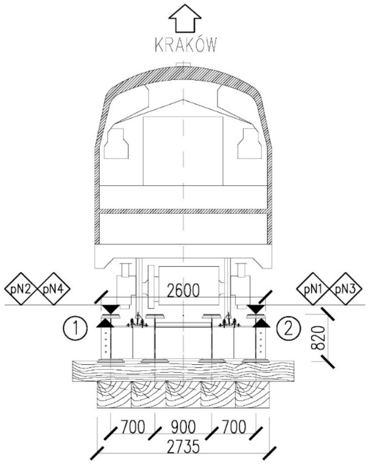

Fig. 5.21. Scheme of the measurement point arrangement in the cross-section

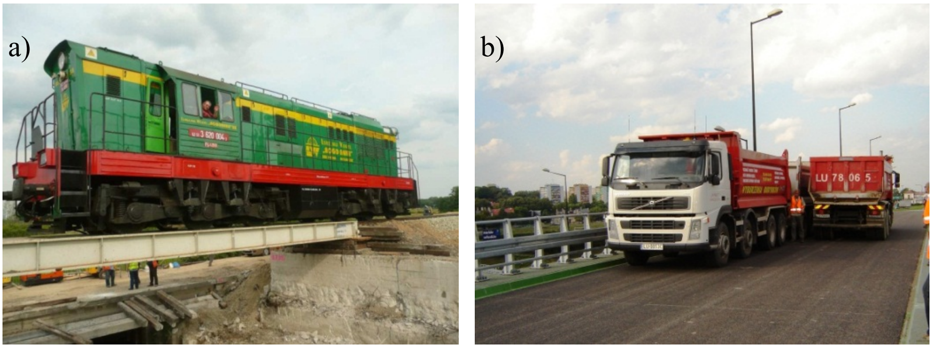

Fig. 5.22. Examples of the use of different vehicles to load bridges: a) locomotive, b) trucks[10]

The most frequently used type of extortion in the static tests is load in the form of a locomotive with loaded wagons or loaded trucks (Fig. 5.22). Their location and number are determined at the project design stage of the load test. Typically, with regard to continuous load-carrying structures three types of static loads are applied (Fig. 5.23):

- basic pattern span, selected from the condition of the maximum allowable effort of span,
- additional supporting scheme, selected from the condition of biggest moment at the support,
- additional asymmetric scheme, selected to assess the cooperation of the transverse beam structure or to determine the torsional stiffness of slab and box spans.

[10] Photo by B.Skulski/S.Karaś

Setting the load in the case of more complex objects might look similarly, but most frequently are used span schemes. This is due to the fact that in the setting of the maximum moment at supporting more vehicles are needed, whereby is difficult to study in terms of logistics and economics. There is no a full possibility to assess the structural behaviour of spans. The evaluation involves only a comparison of deflections, and these are usually lower than in the schemes. Besides, it is not possible to observe the formation and width of possible cracks in a support – they would be covered with layers of insulation and pavement.

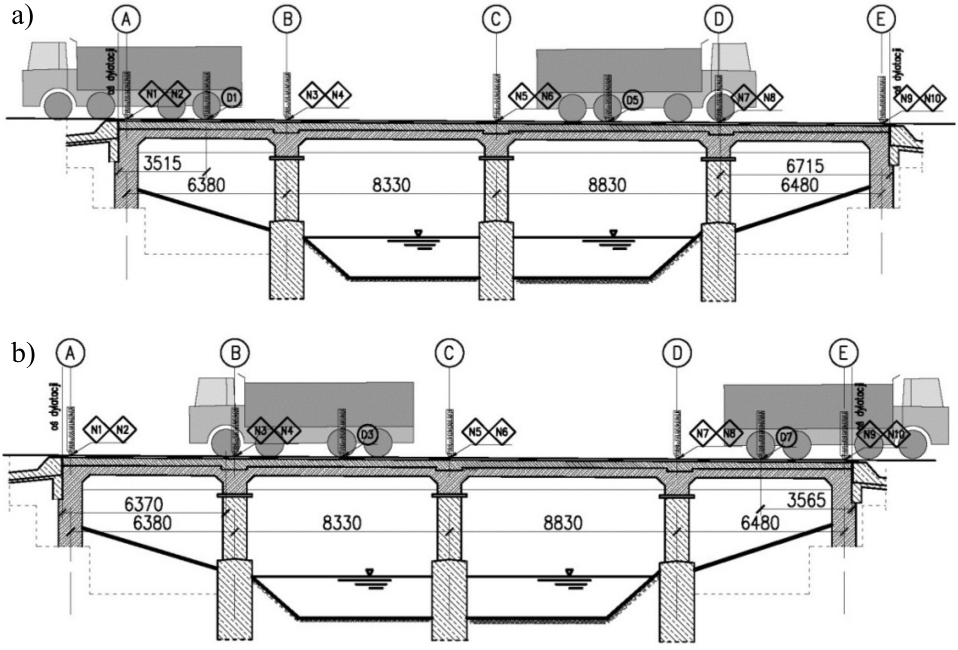

Fig. 5.23. Location of a vehicle in the longitudinal section, static scheme: a) I, b) II

A loads in the dynamic test are usually forcing are usually double-sided passage test vehicle (locomotive, train or truck) with growing every 20 km/h speeds from 10 to the maximum permissible speed on the route. Additionally, carried out various additional dynamic tests, for example passage through artificial inequality in the form of a threshold about height of 5 cm, which is designed to simulate uneven road surface. In addition, it allows the realization of quasi-impulsive extortion. In the case of railway bridges, it is often used attempt with the rapid deceleration of the vehicle on the span [10]. Vibration damping devices, braked braces and the behaviour of bearings are checked in this way. At the footbridges applied load in the form of a group of pedestrians. The gait modes include free and synchronous walking, free and synchronous running and jumping.

In static tests to measure the values (displacements and deformations of the spans, subsidence and deflections supports – Fig. 5.24) the following are used (Fig. 5.25):

- levelling instruments: optical and digital,
- theodolites and electronic tachymeters,
- sensor strings,
- mechanical and electronic sensors,
- electric resistance wire strain gauges.

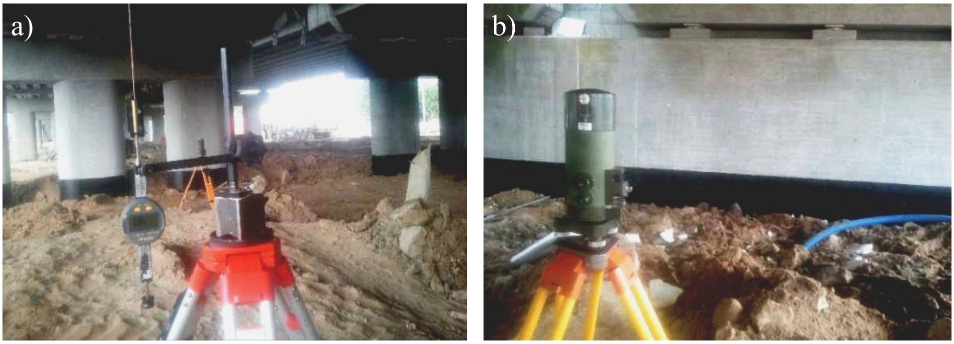

Fig. 5.24. Measuring equipment used in static tests: a) sensor, b) precise leveller[11]

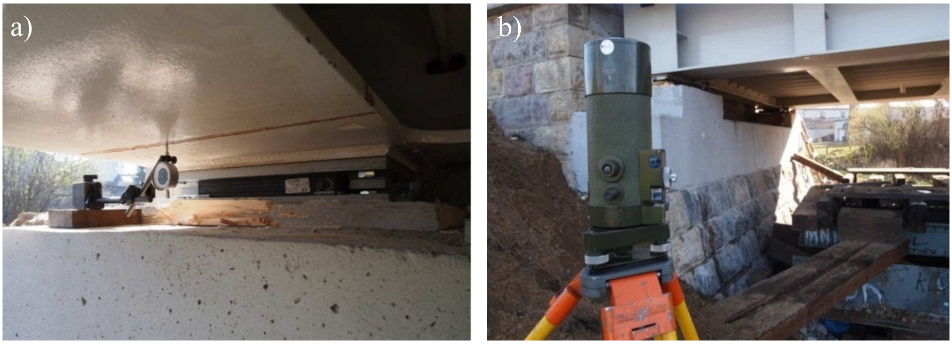

Fig. 5.25. Measurements during the static test: a) crumple bearings, b) subsidence support[11]

In the dynamic tests for measuring the vibration of structural elements and the acceleration the following are used (Fig. 5.26):

- inductive sensors,
- interferometric radar,
- accelerometers,
- laser devices for the measurement of displacements and speed.

[11] Photos by B.Skulski

Fig. 5.26. Measuring equipment used in the dynamic test: a) accelerometers, b) measuring set[12]

5.7.4. Assessment of structure with a load testing

The main purpose of the assessment of a structure is to determine the load which can be carried by it with a safety margin. Most often it applies to the determination of the class of loads according to the general standards. Such an estimate is possible only when the load test will not lead in any way to the deterioration of any part of the bridge.

An additional safety margin should refer to the impossibility to predict the structure of traffic during the use of a bridge and the lack of capacity to assess the pace of deterioration of the technical state of constructions. Therefore, the diagnosis of structures by means of the load test requires knowledge and experience. The assessment of a structure by means of the load test can be divided into 14 stages [14], [59]:

Stage 1 – analysis of technical documentation. If a structure was inspected before the test load has been subject to inspection, verify that the condition of the structure is actual (to ensure that the available description and drawings accurately reflect the current structure).

Stage 2 – simplified evaluation. The theoretical estimation of the structure in order to determine its safe load.

Stage 3 – the designation of the assessment results of simplified elements that are crucial for the design load capacity. Identify the characteristics of the material and the stress levels of the elements which determine the load capacity of the bridge by means of an on-site study. Making a decision about necessity a load test or conduct a more complex theoretical analysis.

Stage 4 – identification purpose of load test and predicting its consequences. Confirm that the load test results are helpful in determining the maximum load that the structure carries.

[12] Photos by B.Skulski

Stage 5 – determine the volume of loads and their placement. Determination of values to be measured and the place of their measurements.

Stage 6 – feasibility study of the load test. Definition of technical tests.

Stage 7 – definition of measuring methods. Select and develop a plan for the deployment of sensors and measurement equipment.

Stage 8 – calculation of measured deformations and displacements. Check the accuracy and range of the selected sensors and measuring equipment.

Stage 9 – risk analysis of the project. Assess the likelihood of structural damage during the trial and the consequences of such a damage.

Stage 10 – assessment of the cost of the load test and making a decision to carry out the study or not.

Stage 11 – develop a detailed implementation plan of the load test.

Stage 12 – make a test load and measure determinate values.

Stage 13 – interpretation of results and using them to determine the load capacity of the bridge. The results can also be used to modify the calculation model used in the analysis of the structure and to determine the dynamic parameters and the scope serviceability and durability of the object.

Stage 14 – making recommendations on further operation of the structure.

It is important to remember that not all types of construction and construction materials are suitable to assess the load capacity using a test load. The reason may be e.g. the rigid-plastic nature of a material and the consequent destruction without measurable deformation values or displacements. Conducting the test load is associated with significant costs and it is necessary to analyse it in order to determine whether the benefits of such attempts compensate for expenses.

The results of load tests should be treated very carefully. It is difficult to separate the real construction from random elements affecting the stiffness of the object. This may be e.g. its surface the stiffness of which is different in winter and in summer; they may be crash barriers, railings, cornices, etc. It is necessary to ensure that various factors which influence to the measured values are fully identified and understood.

A proper placement and selection of suitable types of sensors are of great importance. The placement of sensors in the vicinity of damage (cracks, delamination etc.) can significantly or even completely distort the image of the work the structure.

Load tests are most effective in the cases of complex designs for which it is difficult to create an adequate computational model. They constitute the a special assistance in the process of determining the actual capacity of a design.

The current analytical capacity of the evaluation procedure is similar to procedures used in the design of new structures. It is a treatment for real construction, as if it were made perfectly, without heterogeneity of material or imperfection executive. A theoretical analysis is subject to a high error rate.

The older the bridge, the greater may be the difference between the real and the computational load capacity. Modern analytical methods may help calculate the test results, and especially the actual distributions of transverse and longitudinal stiffness.

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Chapter 6.

Maintenance of bridges

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6.1. Introduction

The maintenance of bridges is an argument of the durability function and in consequence it is also an argument of the reliability function.

It is possible to find some definitions of the bridge durability, however, they are formulated in a descriptive form more or less proper for the bridge standards. For instance:

- durability is the measure of structure's performance with respect to a specified time period [17],
- durability is the capability of maintaining the serviceability of a structure over a specified time, or a characteristic of the structure to function for a certain time with required safety and corresponding characteristics, which provide serviceability [43],
- durability – the safe performance of a structure or a portion of a structure for the designed life expectancy [21],
- durability – the capability of maintaining the serviceability of a product, component, assembly or construction over a specified time [36],
- durability of a structure or a part of it in its environment is such that it remains fit for use during the designed working life given appropriate maintenance [41].

On this basis it is possible to introduce a more general definition as an optional concept, i.e. the bridge durability stands for maintaining the structural load capacity, at least for the time period assumed in the design.

Trying to make a survey on the bridge reliability definition one can observe two directions i.e. mathematics, where reliability is established by means of statistics and which is expressed by probability values, generally in percentage; [8], [43]. The other direction is a form of a physical estimation based on load capacity, real loads action, a diagnosis of corrosion and consumptions in the technical sense – here called the maintenance type.

Reviewing numerous scripts and manuals, e.g. [9], [16], [24], intended for bridge inspectors, one may think that there is not a simpler operation than an evaluation of the technical condition of bridges. However, it is not the case. Inspections show some levels of an examination. Every inspection should be recorded in a number of reports. On the basis of the conclusions included in elementary inspection reports, the decision about drawing up an extended report may be taken. Optionally, every 5 years a detailed inspection may or must be carried out. In this case, the bridge is subject to a thorough examination in terms of its carrying capacity as a whole and its individual elements, material components, conditions and the operation of the bridge equipment, identifying the state of corrosion, cracks, spalling, etc. An opinion on the current state as well as a conclusion regarding the form of maintenance, repairs and the date of the next inspection and its type must presented in the report form. Actually, these systematic and reliable assessments constitute a generally efficient procedure helping to sustain good bridge conditions. Maintenance processes are heavily dependent on the financial means of the bridge administration responsible for starting the bridge corrective actions.

The recognition of bridge durability on the elementary and extended levels of an inspection is based on non-destructive testing techniques (NDT), hammer testing of the surface hardness, Pull-Out and Pull-Off. The major advantage of NDT methods has been recognized as their capability to test in situ. A great deal of expertise is required to interpret NDT field observations and test results. According to EN 13791 [41] NDT methods have to be related to a more reliable assessment i.e. material core testing for compressive strength.

Among NDT methods, one is of major significance. It is the *proof load test* that should be conducted using design loads. Only in this case, it is possible to diagnose the bridge toughness, material and design quality.

It works, however hundreds of reports' pages could be replaced by one graphic presentation which is known as the *degradation curve*.

The *degradation curve method* is a form of the Sommerville's concept [7], [28]. The method could be used as a theoretical one, created heuristically or built on the basis of recorded events during the life of a bridge. In each case a calibration is necessary by means of the obtained examination results of real bridge structures [30].

Alternatively, the *degradation curve* [32] can be used instead of reports. The degradation curve method looks very practical, intuitive and explicit. However, it is not commonly used. Probably, it ought to be more appropriately adjusted to practical needs and should be put into inspection manuals.

The reliability of a bridge should be treated as the reliability of the bridge service as a part of a road net. Reliability as an estimation of hazardous risks is extensively used in scaling of the partial safety coefficients.

6.2. Maintenance and reliability of bridge objects

6.2.1. Passive maintenance

The idea of passive maintenance of a bridge refers to the minimization of interfering in maintaining the bridge structure, but as a result without having negative effects on the quality and durability. Designing an object for the passive maintenance consists in selecting a form, materials and a manufacturing technology where the maintenance needs would be minimal while the required durability of the specific elements of an object would be met.

Nowadays technology does not allow to produce materials and products for building bridges of the same longevity and corresponding to the assumed period of service.

According to the relevant regulations [37] when using available materials and providing a basic level of maintenance, periods of service for different elements of the bridge structure should not be less than:

- 200 years – bridge supports in stagnant waters of a stable level, 150 years in the depths of rivers and 100 years in the floodplain,
- 100 years – solid abutments, retaining structures, massive arch and plate structures and tunnels,
- 80 years – beam or box spans with massive piers,
- 60 years – viaducts supports, light abutments, beam or box spans with light decks, prestressed spans,
- 50 years – tangential and roller bearings,
- 40 years – culverts, massive decks,
- 30 years – light decks, massive waterproof deck insulation, balustrades,
- 25 years – drainage,
- 20 years – elastomeric bearings, light waterproof deck insulation, paving decks, expansion joints, barriers,
- 15 years – new protective coatings of steel structures,
- 10 years – road surface,
- 5 years – repainted protective coatings of steel structures.

The minimum durability described as 100 years stimulate imagination. Durability of 5 or even 10 years compared to the previously mentioned 100 years enables to realize that within the minimum time of use, one element will be replaced 10 or 20 times for another element. Furthermore, the lack of proper maintenance of individual structural elements or equipment in proper working conditions can lead to the failure of the whole structure. Therefore, we should strive for optimal solutions that will help achieve longer service periods of specific elements.

Finding the optimal solution

When designing the structure and object forms, it is necessary to take into account not only the present function of the object, but also its possible future. The bridge span range depends on obstacle dimensions. A proper selection of the static scheme, for example the use of fewer spans, may allow to reduce or completely avoid the need of maintenance that will be reduced or omitted. A theoretical object span and the cross section of a deck results in further design solutions.

Today, bridges are designed for moving load "LM1" [43] and oversized vehicle STANAG 2021 class 150. The load LM1 is in fact higher than the standard load for the normal bridge use. This keeps a certain reserve carrying capacity that over the years may decrease with an increase in service load or a reduction in mechanical properties of an object due to external factors. Moreover, the use of appropriate safety factors allows to maintain a reserve capacity of the so-called "human error". It can occur both during the design and construction of an object. Bridges cannot be designed with maximum effort only to reduce the cost of the object elevation. Later, maintenance costs can significantly exceed the temporary savings achieved at the drawing board stage.

An appropriate recognition of the ground in the area of the designed object is crucial for the proper design of works. Savings on the level of geological exploration, an insufficient number of wells and probes or too little depth can cause serious problems during ground-works. A wrong diagnosis of geotechnical parameters, soil layers or the level of groundwater can cause a sub-optimal positioning of the object, which in the long-term could cause problems in the use and maintenance of the entire object.

Abutments and pillars today are mostly made of reinforced concrete, less frequently of plain concrete or steel. The main aim of supports is to transfer to the ground their own weight, service load and other loads acting on the structure. Abutments must also ensure the stability of the embankment in contact with the object. Therefore, abutments should be designed for the required load with a correct isolation from ground waters. Water cannot persist in the areas adjacent to support, because it can have a negative impact on the concrete and reinforcement. In order to improve the aesthetic perception of a support and the protection against chemical agents dissolved in water, concrete surfaces are covered with colourful anticorrosion or hydrophobic coatings. In the case of massive abutments with significant lengths over 20.0 m, and when more than one bridge deck is based on the support, and between the wings and the leading abutment walls, it is recommended to use vertical dilatations. Dilatation should be sealed from the embankment side.

Abutment wings also serve the embankment stability. Wings may be arranged in parallel, perpendicularly or at an angle to the longitudinal object axis. The wings can be suspended to the abutment, standing on a footing or mixed construction. The wings must be embedded in an embankment by at least 1.0 m. If between the abutment and the wings there occurs the angle of 90°, it is recommended to use axils on the side of the embankment with dimensions of 0.50 m × 0.50 m.

The bearings bench, transfer loads from the bearings by ashlars to the support. The bench should be topped with a cornice which protects the support against dripping water.

A transition plate ensures the continuity of variation in the stiffness in the transition from the road to the object. It prevents the formation of faults on the interface between the embankment and the object. A transition plate has a length depending on the height of the embankment, but not less than 4.0 m. It is made with a decrease of 10% on the outside of the object. The plate must have sufficient stiffness and strength, so the thickness should not be less than 30 cm, and the concrete class not lower than C25/30.

Bearings have two main functions. They transmit the load from the span to the support and enable the freedom of movement of the structure. The bearing type depends on the pressure exerted by the span and the expected direction of displacement. Bearings, depending on the type and size, should have security elements with sliding surfaces and protection from contamination. If they are made of steel, they should be appropriately protected against corrosion. The bearings which receive tensile forces should be equipped with a security anchor. The proper use of a structure and abutments depends on the appropriate selection of bearings.

The water insulation of a bridge can be thin or thick. Thin insulation is used to protect concrete which is in permanent contact with the soil (footings, abutments from the side of the embankment, the part of pillars sunk into the ground). The thickness of the thin insulation layer may vary. Nowadays, the required layer thickness is above 2 mm. Thick insulation, often with heat-sealable tar, has thickness of 5 mm. It is laid directly on the bridge deck (over the entire surface) and the horizontal parts of the deck, wings and the gravel wall. Its is to protect the structure from rainwater and harmful substances that can penetrate the pavement or sidewalks. Insulation covered with aggregate, e.g. the ballast troughs of railway bridges must also be covered with a layer of cement concrete (5 cm thickness) preventing punctures. Proper insulation of concrete elements protects an object from ground water or rainwater, increasing its life and reducing maintenance costs.

The carrying structure should be selected according to the strength requirements, utility, surrounding environment, the technological possibilities of execution and subsequent maintenance needs.

The equipment of a road bridge deck normally refers to sidewalks between which a road surface is arranged. A properly designed and constructed road surface placed on the insulated surface of a rigid plate deck can serve longer than the required 10 years. A properly wide sidewalk allows the removal of a steel barrier from the curb. This reduces the possibility of vehicle impacts on the barrier, which increases the chances of the sidewalk longevity.

The use of a wider roadway on the bridge than on the preceding road, with keeping the width of lanes, enables placing the elements of the surface drainage and (sewage, drains) outside lanes, which:

- increases the driving comfort of road users,
- does not cause additional dynamic effects on the bridge deck,
- reduces the likelihood of splashing water at pedestrians on the sidewalks, the sidewalk pavement and other pieces of equipment,

- reduces the probability of a vehicle hitting the curb and the barrier.

The narrower road surface and sidewalks, the less it is likely that water penetrates into the bridge deck, which increases the longevity of the carrying deck structure. Sidewalks decline toward the curbs, where the drainage inlets and the elevation of cornice boards prevent a run-off of rainwater on the lateral edges of the cornices. De-icing agents (mainly NaCl) should not be used on bridges. They accelerate the corrosion of concrete and steel. In practice, the roadway maintenance services – during snow or freezing temperatures and the possibility of black ice – do not run the risk of a traffic accident due to the slipperiness of a road. For this reason, curbs on a bridge should be made of materials resistant to de-icing agents. The basic material is stone (granite) and curbs entirely made of solid stone blocks. The joints between the curbs and the road surface must be absolutely sealed. For this purpose permanently plastic (bitumen and polyurethane) materials should be used. In addition, it is proposed to cover a few centimetres of curbs by the sidewalk pavement. It can provide a seal between the sidewalk and the curb. Under the curbs, depending on the solution, there should be placed drains over the insulation, along the object. Drains discharge the water from the surface of the insulation. They protect the roadway surface against flaking. This greatly extends the life of the road surface on a bridge.

The bridge road surface must meet functional requirements with regard to paving on the road. Nowadays, bridge decks are covered mainly by bituminous pavements. They consist of two layers. The bottom layer – an equalization layer (tack, binding) directly covers insulation. The upper layer – a wear layer – on which the traffic moves. Bitumen layers (the bottom layer or both) can be made of asphalt concrete or modified cast asphalt. A wearing course may also be made of concrete asphalt or mixtures of SMA. Road surface on the bridge deck should be made of a sealing material, at least the bottom layer.

Sidewalks are often covered by insulation surfaces. They are made from synthetic resins (polyurethane, epoxy), or a modified bitumen emulsion with the thickness of 3–10 mm. They are both used for waterproofing and paving. It is recommended to use partially flexible layers of pavement on the sidewalks. Flexible pavement on a sidewalk could carry both minus and plus temperatures and does not break during the deflection of an object.

Expansion joints must be chosen depending on the expected displacement of the span. It is recommended to use seal expansion joints to ensure that no rainwater from the surface of the roadway and sidewalks enters into the support and end surfaces of the superstructure. At present, the block and modular expansion joints systems are mainly used. At facilities where the longitudinal movements are small (less than 1 cm), in order to protect the road surface from the dilatation from cracking, bituminous covering expansion joints could be used.

The complete drainage system of a bridge is made up of inlets, drainage, sewerage drains and collectors which leads water out of the object to receivers (settling tanks, separators, storm water drainage). Drains used for the acquisition of surface water from the carriageway should have decanters. Water from the object is led outside. Therefore, it is essential that decanters are cleaned of contaminants to ensure the efficiency of drainage. Drainage inlets are placed in the points where the deck inclination changes, in the area of dilatation, and in the places with an anticipated stagnant water isolation level under the road surface and the sidewalks. They should be protected from damages. A drainage system prevents a dangerous accumulation of water beneath the road surface. Drains are tubes of steel or plastic, completed by the cup, placed on the axis of drainage. The main role is to lead the water out of the drainage. A drain despite its small size is a very important element of draining system. Errors in their execution may affect other parts of bridges causing corrosion which could be expensive to eliminate and contribute to increased costs of the object maintenance [13], [14].

Materials used in various parts of an object in addition to the design solution, influence the durability of the object, and thus the frequency of maintenance works. Currently, the most popular construction materials are concrete and steel. When designing bridges, in addition to the grade of concrete one must specify additional physical, mechanical and structural materials. Assuming the environmental classification of types introduced in the European Union, a concrete bridge may be exposed to different classes of exposure [40]. Different classes of concrete exposure cause different effects. These include:

- corrosion due to carbonation, which is dangerous if the concrete which is exposed to air and moisture has reinforcements,
- corrosion due to chlorides contained in water or air that is not originating or derived from sea water,
- corrosion of wet concrete caused by alternating freezing and defrosting,
- corrosion of the soil and groundwater contaminated with chemicals.

According to the requirements [40] to ensure at least a 50-year durability of the object, different strength classes of concrete, cement content and air, depending on the class of the exposure (from a minimum of C20/25 to C35/45) must be used. Moreover, in chemically aggressive environments there must be used sulphate resistant cement (HSR). To build massive pillars, cement with low heat of hydration (LH) is necessary. In the case of the alkali aggregate reactivity there must be used low-alkali cement (NA). It is recommended to use Portland cement (CEM I) in carrying decks, especially for prestressed structures. For the construction of supports and foundations, it is advisable to use metallurgical cement (CEM III), because of the low heat of hydration, slow binding and reduced shrinkage, allowing a reduction or even elimination of shrinkage cracks.

Apart from selecting concrete of the appropriate class of exposure, concrete which water absorption is not greater than 4% should be used; the water permeability of at least W8 and frost resistance of at least the F150 [48].

The durability of steel depends primarily on its resistance to corrosion. The corrosion of reinforcing steel is prevented by a suitable concrete cover, which, depending on the item is 30 to 55 mm (main reinforcement). Prestressing steel rods are secured by filling injection (cement grout or wax) into cables or casing pipes. In justified cases an additional security in the form of galvanic coatings or plastic adjacent to the bars is applied. To ensure the sustainability of prestressing steel and reinforcing steel the minimum diameters of 4 mm for wire compression, 6 mm for rebars, and 15 mm for tension rod are introduced.

The durability of structural steel primarily depends on protective painting coatings and/or metallization coatings. Protective coatings may be avoided only in the case where stainless steel is used. However, steel of this kind cannot be used in highly industrialized regions (chemically aggressive environment) and with high humidity, because in such conditions it does not produce a natural anti-corrosion layer.

Bridge construction steels are characterized by the yield strength in the range of 235–460 MPa [44]. Due to the influence of dynamic loads and fatigue loads, the resistance of steel to fracture is required where the structure is exposed to low temperatures. Reinforcing steel can be smooth or ribbed. It is recommended to use weldable steel. It should be reassured and have a yield in the range of 240÷490 MPa. The prestressing steel should have a yield point of not less than 85% of the tensile and yield strength at a strain of 0.1%. The minimum elongation at break of 3.5%, have low relaxation, provide an adequate fatigue strength (from 2×10^6 of stress cycles), provide a minimum tensile strength of strands in a complex state of stress and provide resistance to stress corrosion. In the case of structural elements made of steel it is possible to ensure less maintenance work or no maintenance. In the case of a construction of corrugated sheet, one may use an appropriately thicker than required steel sheet. In the case of steel culverts, using thicker steel and through the establishment of an appropriate corrosion protection, it is possible to completely eliminate the necessity to maintain the culvert. Similarly, in the case of structures of stainless steel, corrosion will develop, but slowly and will not threaten the facility's structure.

Composite materials can be used both to build new bridges and to repair or strengthen existing structures (discussed in Section 1.3.4). Products from composite materials can be characterised by enhanced durability. Among fibre reinforced polymers, which once were widely used, there are carbon composites (CFRP), glass (GFRP) and aramid (AFRP). FRP composites are corrosion resistant and if properly protected they are also fire-resistant.

A reduction in the maintenance requirements of the object results also from a proper implementation of taluses at the abutments. Taluses, at the ends of the embankments, should be strengthened by grassed geogrid or prefabricated elements. Talus strengthening has a significant influence on its subsequent persistence. A geogrid reinforced and grassed talus requires several mowings a year. The strengthening of a talus by means of prefabricated elements enables reducing its maintenance to the minimum.

For abutment filling and taluses the best sandy soil (sand medium or coarse or a mixture) that allows to gain density index above $I_s = 1.00$ should be used.

The result is a full densification of the embankment in the area of the bridge. This prevents the surface subsidence in the bridge area.

The bridge maintenance system consists of a set of actions to ensure the stability of the bridge and its functionality, including bridge cleanliness, strengthening and even reconstruction. The bridge maintenance in a wider sense refers also to elimination of the sources of potential corrosion, small repairs, painting, periodic inspections and tests. All these factors greatly influence the stability of a bridge [31].

An appropriate design, careful workmanship and the use of good quality materials contribute to passive maintenance, i.e. maintenance limited to the minimum. Unfortunately, administrators often do not grasp the idea of passive maintenance but have a passive interest in maintenance, minimizing operations. It happens that an administrator reduces maintenance of an object to a sweep in the summer and snow removal in the winter. This is not the idea of passive maintenance.

6.2.2. Reliability assessment and critical case assessment

In the case of bridges the most important characteristic is the durability of the bridge carrying elements i.e. the carrying deck, pillars and abutments. They consist of several substructures. They, both as a group or/and individually, ought to be designed and made of materials meeting the highest quality requirements. The mathematical estimation of reliability is commonly used in various sub-disciplines concerned with bridges, especially with regard to durability (element of reliability) there are attempts to characterise it in relation to bridge materials and loads used [18].

There is another concept related to durability and reliability i.e. serviceability [25] which could be defined as the capability of a building product, component, assembly or construction to perform the function(s) for which it is designed and constructed.

It is possible to find some definitions of bridge durability, however, they are formulated in a descriptive form more or less proper for the bridge standards. The quotation from EN 1990 [42] displays difficulties with defining bridge reliability. There, were used items depicting the definition, or rather additional explanation e.g.:

- *reliability* – an ability of a structure or a structural member to fulfill the specified requirements, including the design working life, for which it has been designed. Reliability is usually expressed in probabilistic terms.
- *reliability differentiation* intended for the socio-economic optimisation of the resources to be used to build a structure, taking into account all the expected consequences of failure and the cost of the construction works.

It is easier to define the opposite term i.e. *the 100 percent unreliability* of a bridge – simply because it may mean that the bridge has not been examined, primarily in terms of mechanics, material properties and its design parameters. Being not examined also means that there does not exist any data allowing to apply any statistical estimation method. This shows that bridge reliability has to be treated as a maintenance problem which is primary when compared to statistical ones.

Theoretically, the full reliability of a bridge (i.e. when the probability of failure tends to zero) could be understood as the case where the bridge in question serves by carrying the road traffic during its whole life-time, excluding any maintenance or failure.

The statistical reliability regarding a bridge durability assessment displays its attributes in scaling of partial safety coefficients. The standard approach comes from [34], [36] which provides load combination equations for both *Load and Resistance Factor Design* (LRFD) and *Allowable Strength Design* (ASD). This method is discussed in many papers, e.g. [32], where the load and resistance factors are presented. The LRFD concept is shown in Fig. 6.1.

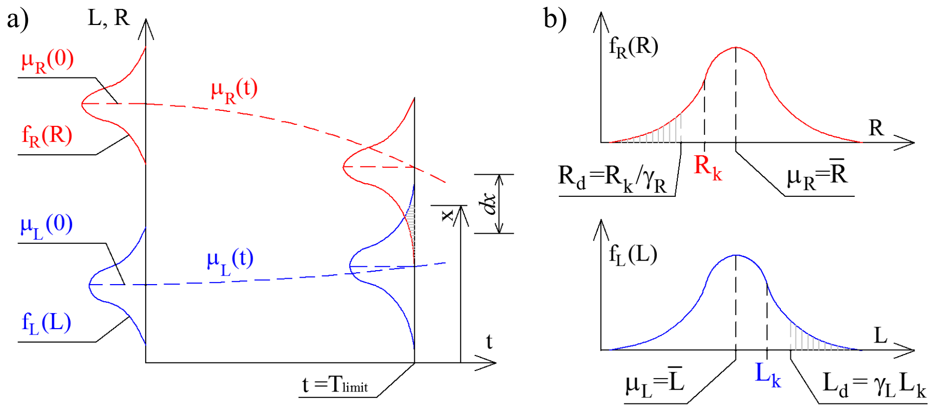

Fig. 6.1. Normal distributions functions a) Presentation of load effects and resistance as a stochastic processes b) partially safety margins, L – load effects – overcharge, R - resistance – undervaluation

A durable design is understood as a process in which the loads and load effects are taken with an overcharge margin and at the same time resistance measures comply with an undervaluation margin. They create, so called, *the conservative design method* which is fundamental in designing the durability of a future structure. The procedure of the LRFD calibration complete and extended to details is carried out in [15].

There are various reliability analysis methods in use to obtain reliability indices. Certainly, there may occur quantitative differences. The conservative approach means that it is necessary to select an appropriate group of methods and assume the final result as minimum reliability indices [27]. At that moment there arouses the question of the economics of conservative solutions.

The second approach, reliability in the maintenance sense, is of greater usefulness, however, it is more complex and complicated. Focusing only on the bridge arrangement over the Vistula River in the area of Lublin Province we can deduct what follows. The river is on the west border of Lublin Province, Fig. 6.2. The bridges are located on or near to 3 important national roadways.

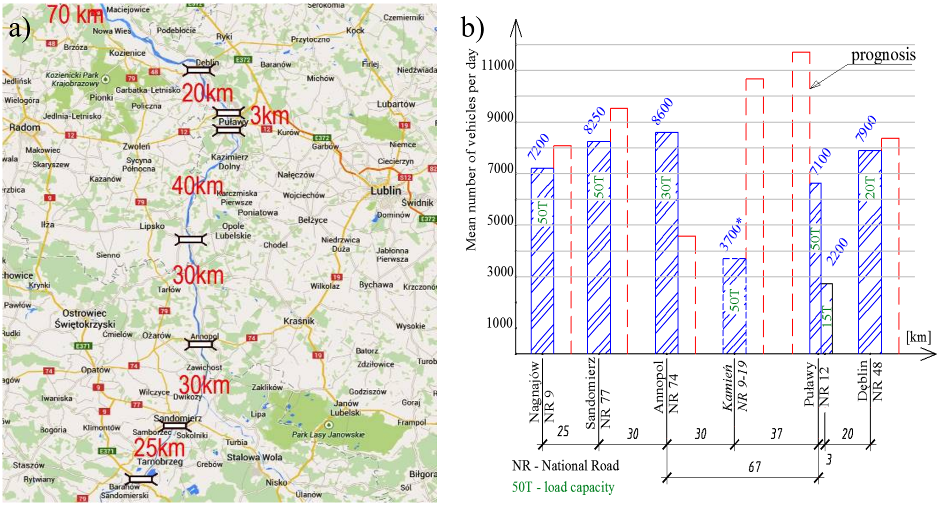

Fig. 6.2. Bridges over the Vistula River in Lublin Province a) map with distances and localizations b) traffic load intensity[1]

Out of 7 different bridges, one is not fully included into the road system (*Kamień*), only 4 of all the bridges are able to carry heavy vehicles of 50 T, while the others only 15/30T. Assuming that the two bridges in the city of Puławy are one entity (only 3 km distance between the two) the average distance between the bridges is almost 30km with the variation coefficient of 0.22. While dividing the whole distance of about 150 km by the number of bridges (7), we obtain 21km per the statistical bridge. Bearing in mind that covering the distance of 50 km for a car/lorry in motion takes ca. 1h, it leads to quite a good assessment of the existing bridge distribution, because in the case of any movement or obstacles on any bridge, the driving time to the next bridge will cause delays for about 30 minutes. Having passed the bridge in *Kamień*, the mean vehicle number is ca. 8250 vehicles per bridge with the variation coefficient of 0,095 (~10%). The new bridge in *Kamień* is certain to influence the traffic net. Firstly, when one looks at the diagram of the distances between the bridges, the lack of one bridge may be noticed, just where the bridge in *Kamień* has been erected. Secondly, its location forces the traffic distribution resulting in reducing the

[1]http://www.gddkia.gov.pl/userfiles/articles/g/GENERALNY_POMIAR_RUCHU_2013/Mapa_SDR2010.pdf

traffic intensity on the neighbouring bridges. Here, it might be worth mentioning that the idea of building the bridge in *Kamień* was born before World War II.

Reliability in the maintenance sense is a steady process; periodical estimating of the structure technical condition creates the basis for the decisions regarding specific or general repair works. Due to this the necessary limitation of traffic is signalized. Information of potential difficulties is published on road management sites as well as in mass media. All this creates the possibility to distribute the traffic intensity among other bridges and the caused impediments are moderated.

6.2.3. Standard schemes of bridge diagnosing

Damage resulting from a variety of degradation processes is a natural phenomenon for any bridge structure [31]. The knowledge of degradation causes and mechanisms, the ways to reflect the impact of failures on the object condition are important for the design reliability, safety and users' comfort [10]. That is why bridge engineering uses a series of actions – diagnostic tests –identification and classification of defects, and an assessment of their condition.

The purpose of diagnostic tests is usually to obtain a causative diagnosis including the causes, mechanisms and degradation processes contributing to the damage to the facility. Identification of the degradation phenomena is of fundamental importance not only with regard to the safety of the facility and its users, but also to rational planning and carrying out maintenance activities [1].

Developing a reliable assessment of the condition of a bridge structure is usually a complex process that requires the fullest possible and precise knowledge of the diagnosed construction. In this process there are used both various sources of information, as well as a variety of research methods tailored to the individual needs of diagnostic procedures. The scope of the necessary information mainly includes: the technical characteristics of a structure (design solutions, geometrical data, material characteristics, etc.), operational characteristics of an object (the working conditions of an object, the history of its operation, completed maintenance procedures, etc.), information about the damage that occurred during its erection and use.

Primary sources of information used in the diagnosis of the bridge are:

- technical documentation of the object: design, documentation of the construction process and as-built documentation,
- documentation covering the history of the use of the bridge with special emphasis on the results of the initial inspection, the story of changes in live loads and environmental conditions, and information about the course of maintenance activities, as well as special event supplies,
- the results of surveys conducted under current technical inspections and usability of objects, which are also the basic source of information on the damage suffered,

- the results of research conducted under load or carried out in the form of load tests or in the form of research during use.

The essential source of information about the damage to bridge structures is obtained is their inspection. In Poland, as in most European countries, a strategy of systematic monitoring of the condition of infrastructure is implemented, based on the results of bridge inspections whose main purpose is to detect and identify damage. Relevant information on the condition of bridges is also obtained in the course of testing under load. One should also mention here static and dynamic load tests performed as an initial inspection of new buildings and research facilities to verify existing installations.

Diagnostic tests used with regard to bridges are based on two basic groups of methods:

- methods used regardless of the load acting on the object, divided into:
  - non-invasive methods, also known as non-destructive methods, the use of which does not cause damage to the structure of an object [26],
  - slightly invasive method requiring a limited interference with the structure of a bridge, consisting usually in taking a sample of the material [11],
- methods used during the bridge testing under loads, with distinction:
  - static and dynamic load tests carried out using designed and controlled loads [23],
  - using tests field tests conducted in the form of short-term studies, in the form of immediate or long-term studies for monitoring, by randomly changing useful loads.

The objectives of the diagnostic tests of any bridge structure result from the legislation which constitutes the basis of a bridge maintenance system [46], [49], and are tailored to individual needs of every structure. Typical areas of diagnostic measures include:

- definition of the real geometrical characteristics of structural elements and their spatial shape,
- recognition of the types of material used for the construction of an object, and the determination of the properties of these materials,
- detection and identification of damage during the manufacture of materials and components, during the construction as well as during the operation of a facility,
- determination of the behaviour of a structure under the influence of the load acting on it.

In the diagnosis of bridges a variety of research techniques is used, some of which can be formed into three basic groups:

- physical techniques – using the physical phenomena,
- chemical techniques – which use chemical processes,
- biological techniques – associated with biological processes.

Fig. 6.3 shows the general classification of basic diagnostic methods associated with these technologies [3].

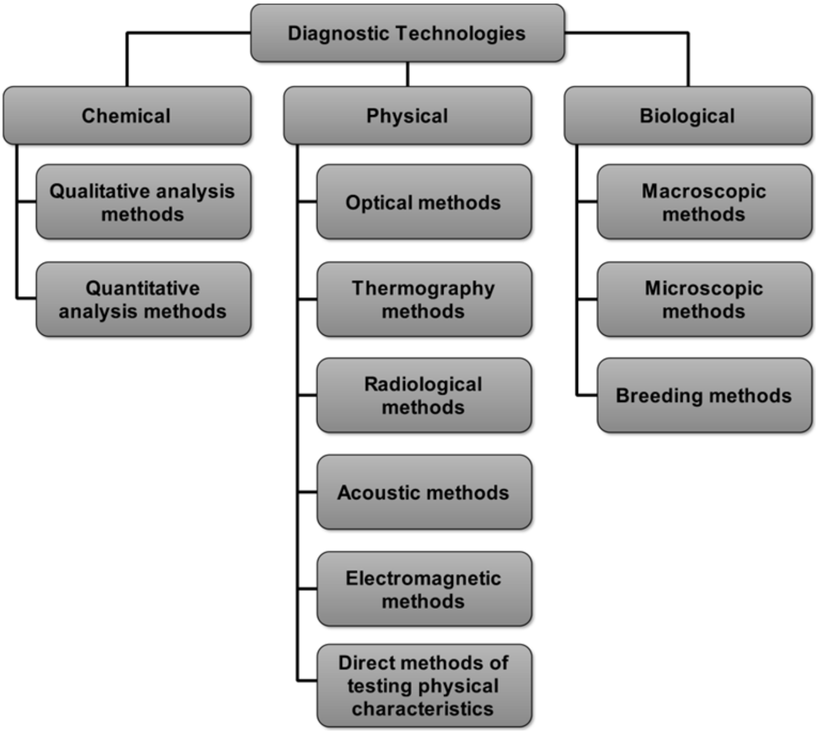

Fig. 6.3. Bridge basic technologies and diagnostic methods

The physical phenomena offer the greatest potential for technologies used. A large group of methods uses the wave phenomena involving the formation of variables in time and space and energy transfer disorders [5]. Among the physical diagnostic technologies, there are also very important a direct methods of testing physical characteristics used in the field or in the laboratory – using core samples taken from the structure.

Chemical technologies used in the diagnosis of bridges are primarily qualitative and quantitative chemical analyses [5]. By means of the chemical qualitative analysis the chemical composition of the tested mixtures could be determined. These analyses allow to establish (or exclude) the presence of a component in the tested material. In studies chemical reactions characteristic for individual compounds are typically used. The chemical quantitative analysis is a set of techniques that aim at quantifying the chemical composition of the chemical compounds mixtures tested or determining the amount of a selected component in the tested substance. In the studies are used characteristic/specific methods for the quantitative analysis of selected depending on the properties of the test substances.

Biological methods are used the least frequently [33]. The conducted studies are generally qualitative in nature and are directed towards identifying microorganisms, plants or animals which adversely affect the condition of the bridge construction. To basic groups of methods used in this regard include:

- macroscopic methods – involving a visual examination of an object,
- microscopic methods – including various research techniques by means of a magnifying device,
- breeding methods – involving in vitro cultures of microorganisms, as well as plant or animal cell cultures.

Considering the nature and location of the defects, the methods used in the diagnosis of bridges can be divide as follows

- geometry test methods – used to determine the spatial configuration of a structure and its components, most often using optical methods in the form of visual research and surveying,
- surface test methods – focused on the phenomena localized in the surface layers of bridges (included here are a number of methods of optical, thermal, electromagnetic nature and most direct methods of testing physical characteristics, as well as methods associated with biological technologies),
- volume test methods – enabling a diagnosis of the entire volume of the material of an object (this group includes radiological methods, radar, acoustics, as well as studies of samples taken from different areas of a structure in order to determine the physical or chemical characteristics of the material).

Individual research methods and techniques are very varied in diagnosing of bridges. Their usefulness depends primarily on the type of material, design solutions and the type of a problem which is the subject of diagnostic tests. As far as the suitability of each method is concerned, the right selection also requires a certain level of preparation and experience with their application, costs of research, the desired type and presentation of results, the availability of research facilities, etc.

6.3. Durability of bridge objects

6.3.1. Bridge load increase

The indicative working life of a project, an element of durability, for non-temporary bridges is 100 years [42]. The practice shows that this condition is too difficult to be fulfilled in the case of typical average bridges. Certainly, a structure occasionally serves even longer but in general it is rather for the period of about 70 years. There are many reasons for this situation. Among them one of the most significant is the load capacity decreases.

The current bridge load standards are analysed when one think about design. However, the changes in bridge loads are important when the toughness as well as durability of a bridge is considered. Here it will turn out that changes of bridge loads are visible and should be taken into account in the case of bridge life. The history of bridge loads will be discussed here on the basis of changes in the Polish standards [12]. The proper load standards are listed in [47]. Tab. 6.1 includes the bridge load characteristics which were used during the 20th c.

Tab. 6.1. Polish bridge load standards during the XX c. period

Issue	Title	Lane [m]	Vehicle per lane [kN]	Resultant per lane [kN]	Vehicle length [m]	UDL [kN/sm]	Crowd [kN/sm]	Comments
1920	1920 norm	2.5	3×(40+80)	360	13.5		5	
1926	1926 norm	2.5	80+2×60	200	6.0	5.0	5	
1931	DIN 1072[1]	2.5	2×(40+80) +(10+2×7)	264	6.0	5.0	5	3 lanes
1945	1945 norm	3.0	80+120	200	6.0	6.0	6	
1952	1952 norm	3.0	40×3	120	0.0	4.0	3	
1956	1956 norm	3.0 (3.5)	2×(80×0.6)	96	0.0	2×(8×0.6)/ 3=3.2	4	In case of 3 lanes – 0.8 factor
1966	PN-66/B 02015	3.0 (3.5)	2×(80×0.6)	96	0.0	2×(8×0.6)/ 3=3.2	4	
1985	PN-85/S 10030	3.5	8×100	800	5.0	4.0	2.5	Only 1 lane
2010	EN 1991-2[2]	3.0	600 400 200	600 400 200	0.0 0.0 0.0	9.0 2.5 2.5 2.5	5	1. lane 2. line 3. line Remaining

[1] Occasionally, this standard was in use, however, as a complement to the official standards,
[2] This standard is obligatory since 2010.

The loads can be treated as design values or characteristic values. Here, they are assumed as characteristic values. The design values depend on the system of safety or partial safety coefficients. In the last century, many changes were implemented by different schools of design. Briefly, one could recall the concept of the total safety coefficient which now is replaced by partial safety coefficients according to loads and materials. Also, the dynamic coefficient concepts and their values have changed in time. Up till 1970s the computer techniques were basically non-existent in bridge design which attracted the use of simplified methods bringing the static scheme to beam or frame. It is necessary to mention that the idea of load models was different in different standards.

For instance, Fig. 6.4 shows two models. The load model of the 1920 Norm refers to a real set of vehicles on the bridge while in the case of PN-EN 1991-2 model, known as LM1 (discussed in Section 3.2), it shows the statistically equivalent model of real loads. In the case of the 1920 model, the DAF (Dynamic Amplifying Factor) was assumed at 1.3, while in LM1 it amounts to 1.0 because the dynamic overcharge is included into load weights. Also, the LM1 model has optional values of adjustment factors, here they are taken as $\alpha_Q = \alpha_q = 1.0$.

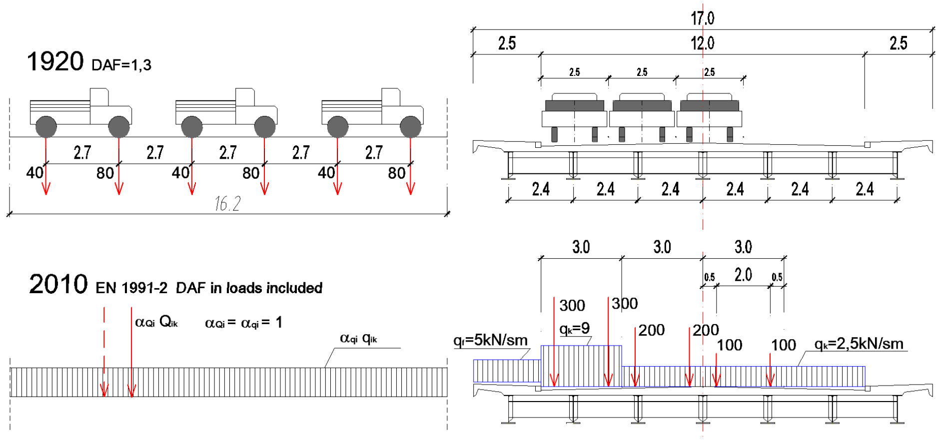

Fig. 6.4. Bridge load models according to 1920 Polish Norm and EN 1991-2 one, concentrated forces in [kN]

It is possible to extend above divagations and another which will show the problem of the bridge load standards comparison displaying the case as not easy. As a result, here two approaches are referred to:

- using the beam (grid) model solved by means of Courbon's method [7] to obtain the bending moment values (discussed in Section 4.2),
- when the carrying-deck FEM model is applied to get necessary magnitudes (discussed in Section 4.3).

An average length of the bridge span in Poland is about 20 m. Therefore, there is also a group of bridges of the greatest significance. For this reason, a simple structure of 20 m long (19.6 m between supports) is chosen to be calculated. For the sake of the carried out comparison, it is assumed that the cross-section consisted of a 11 m wide and two sidewalks of 2.5 m each (actually, deducting the railings and barriers widths, the side walk is 1.5 m wide i.e. equal to the width of the two-persons clearance gauge). Regarding the standards the DAF has increased from 1.25 to 1.3 and in the LM1 the dynamic effects are included into load values. Due to this, here all the mechanical effects of LM1 are divided by 1.25 to equalize them to others. The results of the first approach are shown in Fig. 6.5.

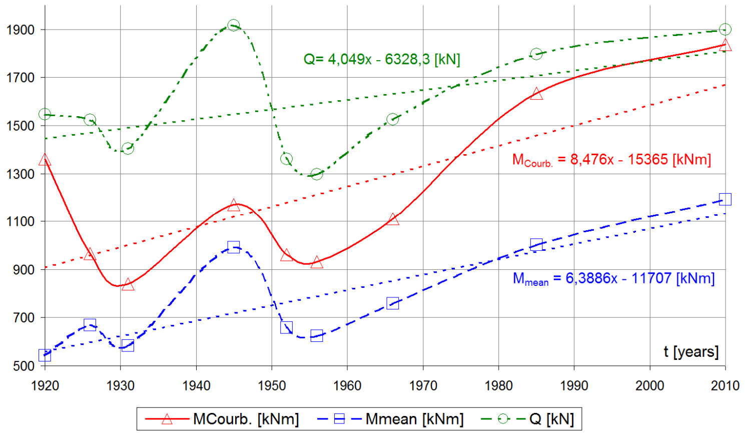

Fig. 6.5. The results of Courbon method, max bending moments M_{side}, M_{mean} of side and mean girders and the resultant of dead and life weights Q versus time in years

The Courbon's method allows to determine the loads acting on a selected girder, and as a result the max. bending moments are found for the side and mean girders. In this method the side girder is relatively overloaded. The trend line formulae shows the increase of 850 kNm and 640 kNm per centaury for the side and mean girder respectively. The related assessment gives the rise of ca. 100%.

In the case of the resultant weight the increase is approximately 400 kN per centaury and 30% relatively which shows the tendency to impulse-like distribution of loads.

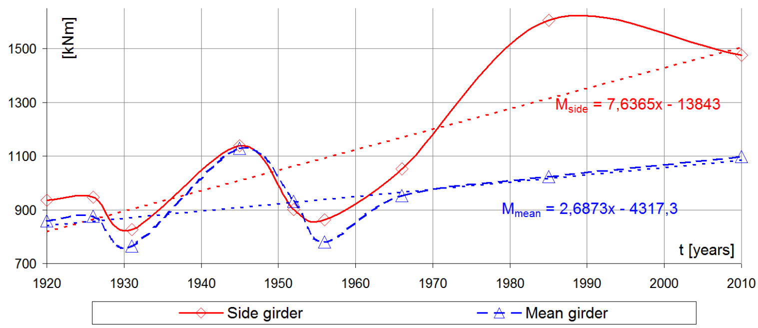

Fig. 6.6. Bending moments for side girder – $M_{Courb.}$ and mean girder M_{mean} obtained from static FEM analysis

The FEM model for the static and dynamic analyses was designed by ABAQUS software. The bridge concrete deck plate of the height of 26 cm was created by shell elements, while the grid of girders and cross-beams was built by means of beam elements. The integration between the grid and plate fulfils the Newmark-Rhzanitzyn's [19], [22] assumption where both composite members i.e. girders and the plate have the same curvatures in deformed configuration, here introduced by internal constrains known as ties.

Similarly, first, the bending moments are found. Their distribution and values are shown in Fig. 6.7. The comparison with the results of Courbon's method shows that their line courses are placed between appropriate lines in the case of Courbon's analysis. This also allowed to repeat the conclusions on the loads increase formulated earlier.

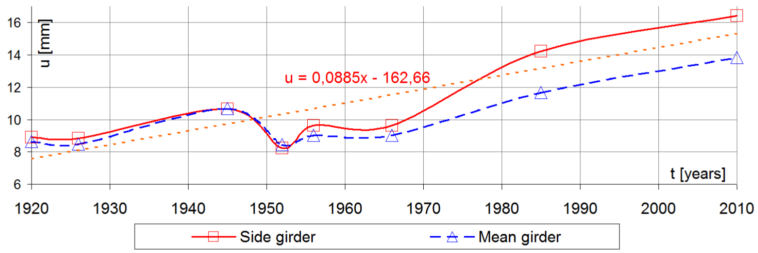

Fig. 6.7. The max displacement values at a mid point for side and mean girders

The graph of static displacements, Fig. 6.7, corresponds to the bending courses in Fig. 6.6. The relative increase of the displacement value is approximately 50%.

All above indicates that the raise of loads used in the 20th century was significant and will probably still occur in a similar manner in the years to come. The different measures used give assessments which are different but at least not smaller than 30% per century.

6.3.2. Design allowing the structure adjustment to future roles

This is a problem which is commonly known and it is formulated by the following question – *who knows what the future holds*?

In terms of technology, or, more general, of civil engineering, it is possible to anticipate changes in the service goals of objects. However, the prospect is not unlimited. In fact, the anticipating period amounts to about 30 years. It is the length of the active operation of one civil engineers' generation. Forecasting accuracy is closely associated with the knowledge of trends involving the development of the construction materials industry, social needs, and, finally, the concepts of development.

The development of materials engineering, particularly building materials, can be easily shown by an example of the usually used materials, steel and concrete, which are still fundamental in bridge engineering. In the fifties of the last century it was considered that concrete used in bridge construction should be of no smaller class than C24/30, which in those days was regarded as a high standard. Currently, in general, the bridge concrete class should be C45/50. Reinforcing steel bars were characterised by the grade B235 and nowadays the most popular is the grade B500. The case of steel bridges is very similar – where S235 was accepted in the past, now it is S500. Here, it is necessary to mention that the best results are obtained when high quality steel is combined high quality concrete. This allows to reduce the dead weights of used materials and, in a sense, enlarge the load capacity range for live loads.

Fig. 6.8. The bridge in Menorca, Balearic Islands, made with stainless steel[2]

It is difficult to guess the future, but numerous symptoms indicate that stainless steel (SS) can change the RC bridges. The idea of using non-corrosive steel as concrete reinforcement is not new and started with the production of stainless steel (or inox steel). The corrosion resistance of iron-chromium alloys was discovered in 1821 by Pierre Berthier[3]. In general, the SS is a steel alloy with a minimum of 10.5% chromium content by mass. Chromium contained in SS forms a passive film of chromium oxide[4], which prevents further surface corrosion. Passivation occurs only if the share of chromium is high enough and oxygen is present. The most popular SS is X5CrNi18-10 (EN 1.4301) in which the chromium content is 18% and nickel 8% of mass. Also, there is commonly used Duplex SS which is a hardenable alloy. Its microstructure has both austenite (chromium-nickel) and ferrite (chromium only) phases, which results in similarities between austenitic and ferritic SS. The corrosion resistance is greater than that in the most commonly used grades of stainless steel (1.4301).

[2] See: http://www.euro-inox.org/pdf/case/CalaGaldana/CalaGaldana_EN.pdf

[3] Pierre Berthier (1782 - 1861) French geologist and mining engineer; chief of École des Mines's laboratory, the inventor of the 'into blast furnaces' method

[4] CrO, Cr_2O_3, CrO_3 and others

The SS could be doubly attractive [2]. Firstly, it could be architecturally important and interesting when one wants to create an image necessary or appropriate with regard to the location of a bridge, e.g. *Cala Galdana Bridge* in Menorca.

The total length of the mentioned bridge is 55 m, max. span 45 m. The carrying frame structure was made of Duplex steel (EV 1.4462),while the concrete deck was connected in a typical manner to the composite steel-concrete.

The second option is when SS bars are used as concrete reinforcement. A spectacular example was depicted in [50], where the advantage of the SS application is illustrated by two, built in one place, massive marine piers in the port of Progress, Mexico. The first pier, completed in 1941, was reinforced by SS bars and now is still in a quite good technical state. The other one, finished in 1960, was made as ordinary i.e. of carbon steel (CS) RC. Now, only remains of the massive pillars protrude from the water. Repairing RC, where CS was used, by adding SS bars rises a question of the impact of the electrical corrosion potential.

On the basis of the performed experiments as well as the experiments discussed in [20], it was concluded that *the use of stainless and carbon steel reinforcing bars in the same concrete pore solution will not increase the corrosion risk on carbon steel, even when these bars are in direct (electrical) contact*.

SS is more and more recognised in the scientific research field and possible applications are being developed in different fields including construction. In [21] the corrosion risk of SS (EN 1.4301) is investigated by subjecting the samples to concentrated salt solutions. A procedure as well as a model of diagnosing to estimate the pitting corrosion risk are proposed. Similar tests and results are discussed in [29].

Speaking of RC with SS bars, in [9] the cases of evaluation, repair and restoration of structures are discussed. The experiment dealt with the corrosion tests on concrete slabs (different concrete classes) with different steel bars subjected to Cl^- aggression. It was noticed that none of the tested stainless steels displayed the beginnings of pitting corrosion, also in the slabs with 2% or 4% Cl^- aggression.

Now an interesting project in Northern Virginia is in progress [16]. The concrete deck of a highway bridge was reinforced by epoxy-coated reinforcing steel (ECR) and corrosion-resistant reinforcing steel (CRR) according to ASTM A1035. The deck was made as two independent parts connected longitudinally. The bridge was open for traffic in 2007. The costs were estimated in the project and verified by the real costs of construction works. In the coming years the bridge decks will be observed and the conclusions drawn will be very helpful in estimating the used steel adequateness.

To sum up this part of the chapter: SS seems to be a good proposal to elongate bridges and especially the service life of concrete bridge decks. An important barrier is a higher cost of SS but in a longer period of bridge exploitation it is unquestionably the right choice. A separate issue is the development of composite materials used in bridges. This is a group that can be called *Fibre Reinforced Plastics* (FRP). Initially, the FRP tapes were used to reinforce concrete structures, then steel elements and, in the last decade, bridges have been made entirely by means of the FRP technology. Many spectacular examples can be found on the Internet. One variant is of special significance. This is the case of *Recycled Structural Composites* (RSCs or RSC) based on technologies developed at Rutgers University. Such bridges are colloquially called Trash Bridges[5]. This technology was developed on the basis of the sustainable development concept in which the recycling of waists is crucial. Simply, this action cleans the environment. Used bottles of juices, cola-like beverages are recycled into elements similar to wooden boards, logs, balls etc. The similarity of RSC elements to wooden elements is not accidental. After many years the nineteenth-century technology of bridges made of solid wood is coming back.

Social determinants are as significant as new material applications. Here again, the concept of sustainable development should be recalled, which, among other things, foresees a reduction of irrational development to protect the environment. Uncontrolled growth of industrial production has been and still is connected to excessive poisoning of the environment and leads to irreversible changes in nature. In this sense, restricting the present day development results result in sustainable development, which is currently considered to be appropriate.

In terms of traffic engineering, you can consider the following problem as an example – for economic reasons transportation of large loads is preferable, but if we allow such thinking than we incur additional costs.

It will be necessary to rebuild existing highways and other types of roads. Bridges as an integral part of roads also will need to be rebuilt or build from scratch. New pavement on the roads will be required, too. Reconstruction of the road infrastructure is one of the main sources of emitted carbon monoxide and dioxide, and other substances in the gaseous form which are harmful to humans and nature. Also the rail transport of heavy loads, freight, requires additional investment in the modernization of the existing railway lines or the construction of new lines.

Limited growth does not mean ceasing to develop. Instead of unimpeded development there arise technologies contributing to controlled development, which also help achieve the same objectives, however, they are not aggressive to the environment. An example of success in this field is the operation of large-scale high-speed electric trains: Japanese Shinkansen[6], French TGV[7] or

[5] https://www.asme.org/engineering-topics/articles/construction-and-building/trash-bridges
[6] https://www.youtube.com/watch?v=VdFD2hy7kFM
[7] TGV – de train à grande vitesse, https://fr.wikipedia.org/wiki/TGV

Spanish AVE[8]. It is true that high-speed trains consume large amounts of electricity, but this energy is produced as so-called clean energy and comes mostly from nuclear power plants.

6.3.3. Designing bridges to last for 70 to 100 years

A general knowledge of the development trends in technical and social areas allows us to formulate the following non-exhaustive list of necessary preconditions. Having in mind that the bridge loads increase is measured in percentage as 30% to 60% per centaury, a designed bridge should be designed in ULS and SLS with a small reserve of safety rather than just according to the standard criteria. Any other design should be allowed only in the case of temporary bridges. How much should be the reserve magnitude in load capacity? The engineering practice shows that just about 20% is acceptable.

Here we get the chance to present the solution adopted by Czech engineers. It regards the railway viaduct in Prague, which was given the name of President Masaryk, 2007. For precast, RC and prestressed structure, at the design stage extra space was provided for prestressing cables and their anchor blocks. This was seen as a progressive design optionally to load increase or devastation caused by an explosion, for instance, Fig. 6.9.

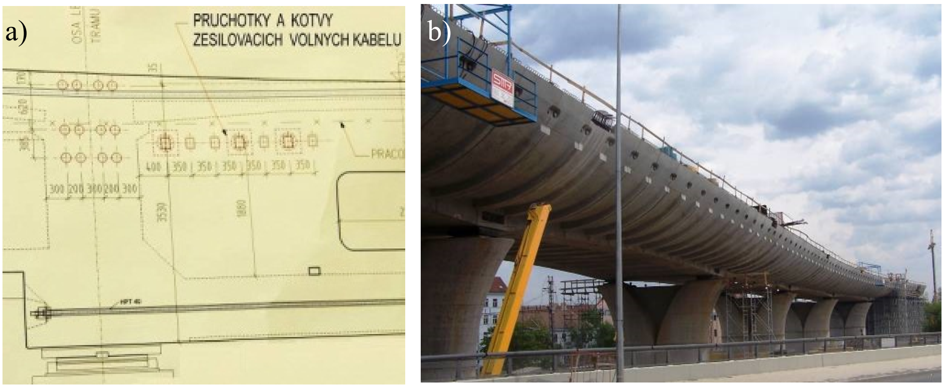

Fig. 6.9. President Masaryk rail viaduct in Prague, 2007: a) drawing of prestressing cable transitions b) side view during the construction works[9]

In most countries bridges are subject to degradation as a result of environmental impact and not overloading. Especially, a different form of corrosion is caused by ionised water penetration or leaking through a structure which results in the necessity of repairs and refurbishing. To avoid these adverse effects, it is necessary to use a very durable and efficient hydro-isolation as well as an uncomplicated slope/drainage system, easy to maintain. The alternative solution

[8] AVE – Alta Velocidad Española, https://es.wikipedia.org/wiki/Alta_Velocidad_Española
[9] Photos by S.Karaś

may be the use of an appropriate SS steel type and concretes of high strength classes. It is not difficult to predict which parts of the bridge will be quickly degraded. Therefore, one can design an easy access facilitating possible replacement. Such elements are, for instance, bearings, beams cornice, railings, curbs and other parts of the road on a bridge.

Another aspect of the design is organisation of the traffic on the bridge deck. Just like on the road, the space is also used for emergencies and reservoirs, so the same must be applied to a bridge in the traffic organization project.

Fig. 6.10. Footbridge in Biłgoraj over Czarna Łada River a) longitudinal view b) side view[10]

With the automotive development there has occurred a reduction in the number of pedestrians on the sidewalks. For this reason, there is a tendency to reduce the clearance gauge for pedestrian traffic. On the other hand, the development of cycling tourism is observed. Existing older bridge walkways for pedestrians are not wide enough and be can hardly be adapted to the mixed traffic of pedestrians and cyclists.

The last solution to this problem consists in building independent footbridges located in the vicinity of existing bridges, Fig. 6.10. This is a good solution that completely separates the pedestrian and bicycle traffic from vehicle traffic, so it is compatible with the general objectives in the field of traffic safety. The same can happen in the future, which is why in the spatial design of a bridge a land reserve for this purpose should be take into account.

According to Fig. 6.10a, the prestressed concrete bridge, visible in the illustration, was commissioned in 1950s and is typical for the bridge construction at that time. The 6.0 m wide road and two sidewalks of 1.5 m constituted a traffic area. In 2006 the bridge was rebuilt to accommodate vehicle traffic only and 1.5 m before the existing bridge a new footbridge was added.

In conclusion, a reasonable balance between progressive and conservative design constitutes a necessary design condition with regard to the long service life of a bridge.

[10] Photos by P.Jamińska

6.4. Periodic inspection of the technical state of a bridge

6.4.1. Provisions and methodology

The technical condition of a bridge and its suitability for use must be regularly inspected. Such an obligation results from the law [39]. According to Art. 62, when used objects should be subject to control:

1) periodically, at least once a year by inspection of the technical condition,
2) periodically, at least once every 5 years, by checking the technical condition and suitability for use, the aesthetics of a object and its surroundings;
3) in the event of external factors affecting the object, which results in damage or a direct threat of such damage that could endanger human life, health, safety, property or the environment.

In the study [35] are shown the rules of applying technical condition scale (pointwise) and suitability for use in the case of a basic and extended inspection are described.

Methods of conducting inspections are discussed in the Instructions [49]. The Instructions introduce a system of inspections consisting of a current inspection, basic inspection, extended inspection, detailed inspection and an expertise.

[35] describes the scale of technical condition and suitability for use in the case of a basic inspection and extended inspection.

A basic inspection checks the technical condition of an object and its surroundings, the condition of wiring and appliances for environmental protection. An expanded inspection further assesses the usefulness of an object, the aesthetics of the object and the condition of electrical equipment, lightning protection and ventilation systems. The result of an inspection is a protocol, in which the inspector describes control results. To prepare the protocol, the inspector refers to the catalogue of failures set out in [49] and the numerical rating scales regarding technical condition and suitability for use published in [38] and [49]. Subsequently, the inspection results are used to plan routine maintenance of road structures, renovations or reconstructions. Considering the total number of road structures in the whole country, it is particularly important that inspectors assess the condition and suitability for use in a uniform manner. Consequently, the largest manager of roads in the country, i.e. GDDKiA, introduced the use of position [35], which is intended in the first place for use in survey engineering facilities managed by GDDKiA. It can and even should be used in reviews of objects that are the responsibility of other managers (provincial, district, municipal and private).

Inspections of objects located on national roads have been carried out since from 1991. In 1991 a 6-point technical condition assessment was introduced [35]. The main idea was to preserve the continuity of the survey, which allows to use the data collected in the analysis of changes in the technical condition of road structures over several years. This enables further improvements in the development of systems, maintenance of premises and verification of their proper design and execution.

For the proper assessment of premises a knowledge of the causes and effects of damage essential. It is recommended that every inspector evaluating road objects should complete an appropriate training. There exists also important information about the technology of an object and the course of its construction, which can greatly facilitate identification of the causes of possible faults and their removal [13], [14].

When evaluating an element of a road object, the inspector examines the individual failure of this component, i.e. contaminants, corrosion, scratching, sagging, deformation. The final assessment grade should be the lowest grade of partial assessments. For the purposes of the assessment of the technical condition of the facility and suitability for use a bridge inspector should be guided by the generally accepted rules and, above all, their own knowledge and experience. If necessary, they should turn to an expert or reduce the capacity of the object even when such necessity does not result directly from the generally accepted rules and is justified in view of the state of the object. An inspector who has an independent technical function must be aware of the importance of the work and its consequences. The inspector must be an experienced person, familiar with the task and well-informed, who is not influenced by third parties when drawing conclusions which suggest more far reaching measures. Reduction of the capacity of a facility or a complete closure can have a profound economic and social impact. If it involves the safety of road users above and below the object, the inspector must be persistent in their efforts. The inspector evaluating the usefulness of an object to use, signing construction qualifications, is responsible under the law for any damage caused an object which has not met the technical requirement but was released for use.

Point grading scales of technical condition and suitability for use and a catalogue of failures

Object assessment must be based on the scale of assessments of the technical condition, described in [38] according to the following criteria:

- grade 5 – appropriate condition – without damage and contamination that can be found during the review,
- grade 4 – satisfactory condition – shows the first signs of contamination or damage to the deteriorating aesthetic appearance,
- grade 3 – condition of concern – shows damage which, if not repaired, will shorten the period of safe service,
- grade 2 – unsatisfactory condition – damaged, reduced usefulness, but can be repaired,
- grade 1 – condition before failure – shows irreversible damage disqualifying usefulness,
- grade 0 – the state of emergency – has been destroyed or ceased to exist.

An overall assessment of the technical condition of an entire object is determined by measuring the smallest value of:

- the arithmetic average assessment of all the elements assessed in the basic inspection
- assessment of the bridge construction,
- main girders assessment,
- the arithmetic average assessment of abutments and piers.

An assessment of technical elements is expressed in integers. The arithmetic mean to be quoted to two decimal places [38].

Tab. 6.2. Catalog of failures [49]

	Designation and type of damage	Damaged material										
						Steel						
		Concrete	Wood	Brick	Stone	Structural	Prestressing	Reinforcing	Rubber	Asphalt	Soil	Plastic
		B	**D**	**C**	**K**	**S**	**P**	**Z**	**G**	**A**	**T**	**M**
N	Pollution	NB	ND	NC	NK	NS	NP	-	NG	NA	NT	NM
W	Plants vegetation	WB	WD	WC	WK	WS	WP	-	WG	WA	WT	WM
C	Water leaks	CB	CD	CC	CK	CS	CP	-	CG	CA	CT	CM
O	Deposits or efflorescence	OB	OD	OC	OK	OS	OP	-	OG	-	-	OM
A	Destruction of anti-corrosion protection	AB	AD	AC	AK	AS	AP	AZ	-	-	-	-
K	Corrosion, decay, aging	KB	KD	KC	KK	KS	KP	KZ	KG	KA	-	KM
R	Scratches and cracks	RB	RD	RC	RK	RS	RP	RZ	RG	RA	-	RM
L	Connectors damage	LB	LD	LC	LK	LS	LP	LZ	LG	-	-	LM
D	Deformations	LD	DD	-	-	DS	DP	DZ	DG	DA	-	DM
P	Displacement, subsidence	PB	PD	PC	PK	PS	PP	PZ	PG	PA	PT	PM
B	Blockage, restriction of movement	BB	BD	-	-	BS	BP	-	BG	-	-	BM
U	Defects, deficiencies or material erosion	UB	UD	UC	UK	US	UP	UZ	UG	UA	UT	UM
Z	Material structure destruction	ZB	ZD	ZC	ZK	ZS	ZP	ZZ	ZG	ZA	-	ZM

In [49] an additional insulation rating scale can be found. The scale and criteria for assessing suitability for use and a catalogue of failures, as shown in

Tab. 6.2. According to [49], the scale and criteria for evaluation of the insulation shall be as follows:

- grade 5 – appropriate condition – there are no signs indicating a leak in isolation,
- grade 2 – unsatisfactory condition – there are a few small stains; local service may stop the process of the destruction of the element,
- grade 0 – the state of emergency – extensive leaks will reduce the sustainability element.

According to [49], the scale and criteria for assessing suitability for use shall be as follows:

- grade 5 – fitness for use is right – parameters meets or exceeds the requirements of users,
- grade 2 – fitness for use is limited – a parameter does not meet the legitimate expectations of users or fulfills them only in part – does not require immediate refurbishment or remodelling,
- grade 0 – fitness for use is inadequate – a parameter does not meet the legitimate expectations of the users – there is an urgent necessity to carry out intervention work, urgent performance of repair or rebuilding.

Pollution and plants vegetation. In case of dirt or vegetation one should consider the surface on which they appear. Qualifying pollution as damage should be guided by its negative impact on the aesthetics and/or durability of components. Pollution hampers rapid drainage of a surface and promotes plant growth. Plant growth accelerates degradation of the structure. Some contaminants can obstruct the flow of water, others may cause fire. Plant growth should be considered only if it has a negative impact on the aesthetics or stability of an element. Lush vegetation can cause harm to the sustainability of taluses. On the other hand, abutments can also hinder the flow of ice and great waters. It should be noted that properly maintained vegetation enhances the aesthetics of the environment and consolidates bridge embankments and slopes. In the case of objects designated for the use of animals, properly designed and maintained vegetation is an essential and integral part of the transition.

Impurities and/or plant vegetation on the elements of a deck – graffiti, petrol, engine railway sediments at the bottom of the plate object, moss, lichens, etc., may affect the stability of a structure. Pollution on a wooden deck and moisture accelerate corrosion.

Water leaks. Water leaks should be assessed depending on the surface on which they occur. Leaks and spills deteriorate the aesthetics and also reduce durability. Water leaks through the bridge deck indicate its corrosion or the occurrence of scratches or cracks. In the case of this type of damage, it is necessary to perform a detailed inspection or an expertise. Water leaks should not be confused with deck splashes created as a result of surface stains.

Deposits or efflorescence. Deposits or efflorescence mainly provide for reduced component durability. Advanced deposits or efflorescence may also indicate a reduction in the strength of concrete.

Destruction of anti-corrosion protection. Destruction of corrosion protection should be assessed taking into account the damaged surface paint coating. Destruction of an anti-corrosion layer affects the durability of a structure.

Corrosion, decay, aging. In the assessment of corrosion the corroded surface and the intensity of corrosion processes must be taken into account. Corrosion of reinforcement should be assessed taking into account the size of the deficits in a rebar and the impact of these losses on the carrying capacity of the support. Corrosion of steel elements is assessed according to a type of corrosion, the depth of corrosion, defects and the surface on which they appear. In case of the suspicion of fatigue, stress or intergranular beam corrosion, a detailed inspection or expertise must be strictly recommended.

In the case of aging or excessive deformation of an elastomer, an assessment should be adopted according to the estimates of the degree of loss of the function of load carrying - by a bearing, for example. In any case of a rupture of a bearing, a detailed inspection or expertise and bearings replacement are required.

Scratches and cracks. Mesh cracks on the approach to an object can be caused by material aging, a poor quality of pavement, a subsidence of a transition plate or a move of an abutment. Single surface cracks occur most frequently in the points of variation in the stiffness of the road pavement base near an edge of a transition plate or near expansion joints.

Road pavement mesh cracks on the object can be caused by material aging, a bad surface quality or a slender bridge deck. Single surface cracks on the object occur most often in the place of change of the substrate stiffness.

The rating of pavement mesh cracks should depend on their surface. The rating of pavement individual cracks should depend on their length.

An assessment of bridge cracking must determine the probable cause of cracks. In the case of surface cracks, a shrinkage assessment should depend mainly on the surface on which they occur.

Scratches, which might have emerged due to overload, should be evaluated depending on the estimated probability of failure. Scratches caused by overloading are primarily vertical and diagonal:

- on the abutment body,
- under bearing blocks on the edge of the abutment body,
- on the wings combined with the abutment body,
- on the gravel walls.

During the evaluation of a scratch it is important to determine changes in relation to the previous detailed inspection, i.e. answer the question whether the width or length figure has increased or created new features? In the case of an increasing aperture, length or the emergence of new cracks, a detailed inspection or expertise should be advised. In the case of cracks in the area affecting the capacity of support, expertise should be advised. Such cracks generally occur on a part or the whole height of the bridge body and on foundations under edge bearing blocks.

Vertical cracks found on the cornice of the pillars of cantilever bridges and bridges with the continuous beam scheme should evoke special attention as resulting in possible damage to the bridge, girders and/or isolation of these pillars.

In the scratch assessment of a bridge deck one should take into account the cause of scratches, the crack width and the crack surface. If cracks are caused by physical factors, one should consider the width of cracks as well as the surface on which they occur. In the case of elements made of reinforced concrete, cracks in concrete from 0.2 to 0.5 mm primarily affect the element durability and cracks of the aperture above 0.5 mm may pose a security threat – a detailed inspection or expertise is necessary in such a case.

Identification of scratches or cracks in a steel deck should result in a detailed inspection and/or expertise. Identification of scratches or cracks in a steel girder should lead to recommendations of a detailed inspection or expertise. Particularly prone to scratching are welded structures. When any scratches on the coating are noticed, it is necessary to remove the coating in this area to see if scratching has reached the steel construction, or it is just on the surface.

Every crack in a compressed girder is evidence of the irregularity of the girder work, and therefore it is strictly advised to carry out an expertise.

Connectors damage. Loosening of items can be caused by the lack of screws, nails, concrete loss or stone elements joints loss. Fasteners damage can result in falling elements which could jeopardize the traffic safety.

Deformations. Deformations, depressions, ruts, surface irregularities causing an increased dynamic impact on the bridge reduce safety and the driving comfort. The lack of adequate declines in the surface water drainage may cause hollows. If not removed effectively (quickly), water from the road surface can penetrate deep into the structure, accelerating the destruction of a bridge. Ponds of water pose a threat to traffic safety. Depressions can be caused for example by a poor compaction of backfill during the construction, the lack of or improper implementation of transitional plates, soil erosion and/or a displacement of abutments. Rating should depend on the road surface, on which there is no proper decline. Deformations of pavement surfaces reduce the aesthetics and can also be the cause of pedestrian accidents. The lack of a proper pavement decline can cause water ponding and freezing. Incorrect declines, for example. Pipes, manifolds and waste water may hamper the water flow, and if the decline is in the wrong direction it may withhold the draining process. This causes traffic hazard.

Mechanical damage of bridge elements are mainly caused by vehicles impacts. Normally, mechanical damages mainly concerns the bottom plate of girder, crosshead or hanger. Designers should be aware that the deformation resulting from the impact of a vehicle may be the beginning of element cracking or buckling. In order to accurately identify defects and their effects there should be performed an expertise.

Deformation of an expansion joint reduces the driving comfort and can endanger safety. The effect of the deformation of the expansion joint is also increased noise in the object environment.

Displacement, subsidence. Depending on the displacement of a structure one must take into account the height of the element and the amount of displacement of the structure relative to the original position – e.g. to the position of neighbouring elements.

In the case of a displacement of prefabricated girders, a detailed inspection should be carried out or an expertise. One should try to clarify whether the displacement took place during the construction of the object or during its operation.

Identification of a beam or support displacement (position change) should result in an expertise.

Deflections should be evaluated based on an observation of beams, cornices and railings. One should assess the position of beams in the middle of the span with respect to the original position or to the position in the last measurement. If there is a suspicion that the central span point changed, it is recommended to conduct a detailed inspection or expertise.

Blockage, restriction of movement. A restriction of the freedom of a beam extension as temperature increases may be due to: moving of the span, moving of the abutment or due to a contamination of an expansion joint. In the case of this type of damage a detailed inspection or expertise should be advised.

In the case of a block or a limitation of movement of a bearing, if the blockage results from contamination it is advised to clean it in safe mode. If the cause of the limitation of movement is a failure of the bearing, a detailed inspection and/or expertise should be performed.

Defects, deficiencies or material erosion. In the assessment of a construction material loss one should take into account the place of the occurrence of the defect (strenuous section) and its depth. The rating depends on the estimation of the percentage of the weakening of the structure.

Small landslides and a loss of ground can only worsen aesthetics. A large landslide or blur may threaten the stability of both embankments and abutments. The ground lowering can cause damage to the wings and a deformation of the surface of the roadway and sidewalks on the access roads to the facility. The destruction of taluses may threaten the stability of slopes or embankments.

Surface defects such as scratches and cracks can be caused by aging, a poor quality of material or workmanship. Dropout joints in paving promote the penetration of water into the structure, the formation of surface defects and deck corrosion. Dropouts reduce aesthetics, the pedestrians/cyclists' comfort, but also the durability of the pavement. Large losses can threaten the safety of users.

If parts of a bridge, supports, railings beams or a cornice fall, it may threaten the safety of users of the traffic under the object. An action should be taken immediately to bring the elements to a state that does not cause safety hazards under the facility.

Material structure destruction. Spalling, structural destruction of the material causes not only a reduction in the stability, but also a weakening of the structure.

Elements subject to assessment of the technical condition

In accordance to the principles set out in [35], depending on the type of a road structure (bridge, tunnel, tunnel, culvert, retaining wall), the technical condition of the following components of the object and its environment can be assessed:

- Embankments and slopes in the vicinity of a bridge, i.e. in the area, where their possible damage affects the condition of the facility and the safety of users. This evaluation should take into account the state of stairs (steps, railings, landings) located on slopes or embankments.
- Roadway in the area between the wings. In particular the impact of the technical condition of the access road to the bridge. A similar assessment is conducted with regard to the condition of the pavement and shoulders.
- Carriageway, hard shoulders and bands located between curbs. For objects with no curbs, in the assessment of the road surface condition shoulders should also be considered.
- Sidewalks pavement and curbs on a road bridge and on access roads, between the wings. The assessment should also include pavement of footbridges, bicycle paths, elevated technical shoulders and the central reservation without ties.
- Railings and barriers found on bridges and access roads within the wings. The technical condition of anti-electrical shock guards, anti-blinding glare shields and acoustic screens should also be assessed.
- Railing beams and cornices (including cornice boards) located on the bridge deck and abutment wings.
- Drains, filters, sewage curbs, gutters on slopes and rainwater pipes. For objects with no surface drainage inlets, the efficiency of the water discharge from the facility shall be assessed.
- Bridge insulation, which is assessed in an indirect way, i.e. on the basis of the state of the bridge deck and sidewalk cantilevers.
- Bridge deck, sidewalks cantilevers and ribs. Main girders bracing elements (crossbars) should be evaluated together with the girders. In the case of half-deck bridges, in evaluating the deck part of the structure between the girders and sidewalk cantilevers should be considered. The deck in plate bridges is not isolated and is not graded – sidewalk cantilevers should be evaluated together with beams. In the case of arched bridges, the deck

should be evaluated in an indirect way, i.e. by assessing the condition of the surface and the side walls.

- Concrete, reinforced concrete, prestressed concrete and steel girders. When assessing girders, elements bracing them should also be included. Cross-beams, crossbars and struttings in upper arch bridges and crossbars and wind bracing in steel bridges. In the case of plate bridges, along with an assessment of girders, sidewalk cantilevers should be assessed.
- Vaults of brick and stone bridges.
- Bearings on supports and spans (in the joints). Bearing ashlars should be evaluated together with pillars and abutment bodies.
- Expansion joints should be assessed over the entire width of the bridge. In the case of an expansion gap, over which there is no expansion joint, a piece of pavement on this slot is assessed. In arched and integrated bridges there are no expansion joints and therefore they are not evaluated.
- Abutment body and its foundation. Assessing an abutment body, one should pay attention to the front wall and wings monolithically connected to the body, the gravel wall, an under bearings bench and bearing ashlars. The foundation is assessed in most cases in an indirect way, i.e. by assessing the state of the body of an abutment.
- A pillar body and its foundation in the extent possible to assess. An assessment of a pillar body should take into account the status of the top plate, under bearing benches and bearing ashlars. As in the case of abutments, most pillar foundations are assessed indirectly, i.e. by an evaluation of the body.
- The river bed and the space under the bridge should be assessed based on the area where any damage or irregularities may have a negative effect on the stability or safety of the bridge.
- Retaining walls adjacent to the abutments, not included in the register as separate road engineering structures, massive abutment wing dilatation separated from the body, structures holding in the stability of embankments near the cantilever bridges if they are not connected to the span.
- Environmental protection devices, i.e. noise barriers, wells and rainwater separators, guards at crossings for wild animals.
- Tendon anchorages and anchorage zones (cables, ropes) in post-tensioned bridges, suspended, hanging and arched bridges and reinforced by external compression. An assessment should also include deviators.
- Cables, ropes and hangers in suspended and arch bridges and tendons in bridges reinforced by external compression.
- Foreign devices (e.g. lighting, gas, telecommunications, energy, water supply, heating), checking the status of the guards and attachments of these devices. In case of damage, the facility manager should notify the owner of the foreign devices, in order to remove the irregularities. The owner specifi-

cally evaluates the technical condition of a device and makes decisions regarding repairs.

- Stairs and ramps that are parts of footbridges and staircases with bridges intended for public traffic. An evaluation should include the structure (supports, girders, steps, landings), as well as equipment (decking, railings). Assessment of the stairs (intended to facility servicing), located on embankments or slopes should be taken into account in the assessment of embankments and slopes. A detailed control of elevators should be conducted under the existing law by persons entitled to assess this type of equipment.
- Accessories aimed at facilitating an access to a construction, i.e. bridges, galleries, trolleys and inspection ladders.
- Front walls on both sides of a tunnel/underpass.
- Walls of a housing interior, the vaulted ceiling of a tunnel.
- Bottom plate. Rating the biggest damage to the bottom plate is performed indirectly on the basis of the surface defects in the tunnel/underpass. A detailed assessment of the bottom plate is possible after removing the pavement.
- Supports in a tunnel or underground passage, and in particular supports in tunnels and underpasses for bridge construction.
- Retaining walls at the inlet and/or outlet of a tunnel, which do not appear in the records as separate road engineering structures.
- The upper plate in frame monolithic culverts and stone, brick or concrete culverts and vaults.
- Walls of frame culverts and high arched culverts or small bridge abutments listed in the records as a conduit.
- Bottom plates and foundations of culverts. The bottom plate is a horizontal element.
- Prefabricated elements of pipe culverts.
- Prefabricated frame elements in box culverts.
- Head inlet and outlet of culverts.
- Underflow riverbed trough upstream and downstream from culvert sections, where damage or abnormalities affect the functioning of the culvert.

Other items, such as joints, chamber ventilation, lighting, ventilation, the area above a tunnel, traffic, a terrain/road over and in front of the structure, a housing retaining structure, anchorage grounds, anchors.

Suitability for use

An assessment of suitability for use of a bridge structure should be carried out with regard to:

- Safety of public traffic. The suitability for use of a bridge in terms of traffic safety must be assessed on the basis of the roadway, sidewalks and curbs, the bridge access within the wings, railings, protective barriers and anti-electric shock guards, railing beams, cornices (only when road transport runs under the bridge), expansion joints, drainage facilities, foreign devices, labelling of the object, the motor section parameters. The analysis should also include the possibility of an occurrence and potential effects of fire, flood, flow of ice, mining damage, ship and vehicle strokes.

- The current carrying capacity of an object.
- The speed limit for vehicular traffic. Suitability for use of a bridge in terms of the speed limit of vehicles to be assessed on the basis of the speed limit traffic on the access roads to the bridge structure and the maximum permissible speed of movement of vehicles on the road bridge.
- Gauge width on a bridge. Suitability for use of a bridge in terms of the gauge width should be assessed on the basis of the width of the road with security bands depending on the technical grade of the road, the gauge width for tram routes, the width of the pavement, the bike path width, the wild animal migration track width, i.e. the useful width of the viaduct designed for the movement of wildlife.
- Gauge height on an object. Suitability for use of a bridge in terms of the gauge height should be assessed on the basis of the vertical gauge on the road, depending on the technical grade of the road, the gauge vertical structures for tram routes, a vertical gauge on the sidewalk, a vertical gauge on the bike path.
- Gauge under the object. Suitability for use of a bridge in terms of a gauge under the object should be assessed on the basis of the dimensions of the space, i.e. the elevation of the bottom edge of the bridge span and bearings more than the highest level of the dammed water flow, a navigable gauge depending on the class of inland waters, beacons depending on its technical grade, gauge railway lines, a gauge for tram lines, a gauge the sidewalk, a bike path gauge.

The final evaluation of individual parameters characterizing the suitability for use of a bridge, is the lowest rating from all analysed parameters.

An assessment of suitability for use of a tunnel/underpass analyses and evaluates the following parameters:

- public traffic safety,
- the current carrying capacity of an object loaded with traffic and rail,
- speed limit in a tunnel,
- gauge width of an object,
- gauge height of an object,
- traffic speed limit on an object,
- gauge width on an object, the efficiency of ventilation.

The final evaluation of individual parameters characterizing the suitability for use of the tunnel/underpass, is the lowest of the ratings of the elements analysed for each of these parameters.

Suitability for use of a tunnel/underpass in terms of the gauge width should be assessed on the basis of the following values: the width of the road with safety bands depending on the technical grade of the road, the width of traffic in the underground passage depending on its length, the width of an object in the light of an adaptation to the size of the animals.

Tab. 6.3. Suitability for use of wildlife crossings in terms of width and height gauge in the facility [35]

Parameter			Grade
Horizontal gauge adapted to the size of the animals	Little	Diameter ≥ 1.00 m	5
		Diameter < 1.00 m	0
	Medium	Horizontal gauge ≥ 3.50 m	5
		Horizontal gauge < 3.50 m	0
	Big	Coefficient of relative tightness ≥ 1.5	5
		Coefficient of relative tightness < 1.5	0
Vertical gauge adapted to the size of the animals	Little	Diameter ≥ 1.00 m	5
		Diameter < 1.00 m	0
	Medium	Vertical gauge ≥ 1.50 m	5
		Vertical gauge < 1.50 m	0
	Big	Vertical gauge ≥ 4.50 m	5
		Vertical gauge < 4.50 m	0

Suitability for use of a tunnel/underpass in terms of a gauge in an object should be assessed on the basis of:

- the gauge vertical roadway in the tunnel according to the technical grade of the road,
- the gauge vertical for tram routes in the tunnel,
- the gauge vertical upright on the sidewalk or bicycle lane in a tunnel or underpass,
- the gauge vertical of underpass which could be used by emergency vehicles with a total weight up to 2.5 t,
- height of the object adapted to the size of the animals.

Suitability for use of wildlife crossings in terms of the width and height of a gauge in a building is shown in Tab. 6.3.

6.4.2. Road bridge inspection

While conducting bridge maintenance works, an actual condition of the structure in terms of safe use should be particularly emphasized. In such a case, a thorough analysis of the collected data is necessary and it is obtained on the basis of:

- carried out overhauls of bridge objects (including surveys),
- analyses of technical and computable documents,
- tentative load tests.

In Poland five types of overhauls are distinguished:

- current overhaul,
- basic overhaul (periodic annual inspection),
- extended overhaul (periodic inspection every 5 years),

- detailed overhaul,
- special overhaul, also called a survey.

All they aim at obtaining vital information about the state of an object, but they are also limited with respect to responsibility. They differ form each other in terms of e.g. the frequency of performance, the method of performance, person performing the review or used equipment. The current review, as well as the periodic one, supply precious data used to make plans about the running maintenance of a bridge. The detailed review comprises an in-depth analysis of the grade of utilisation of an object and it may result in serious decisions such as the closure of an object, reduction of vehicle speed in/on an object, decreasing its current carrying capacity. All these restrictions are strictly linked to indispensability of planning a project of restoration or modernization of a structure.

Fig. 6.11 shows an organization scheme of bridge overhauls [4]. Information contained in it illustrates links between types of inspections and steps taken when any abnormalities occur [4], [6], [49].

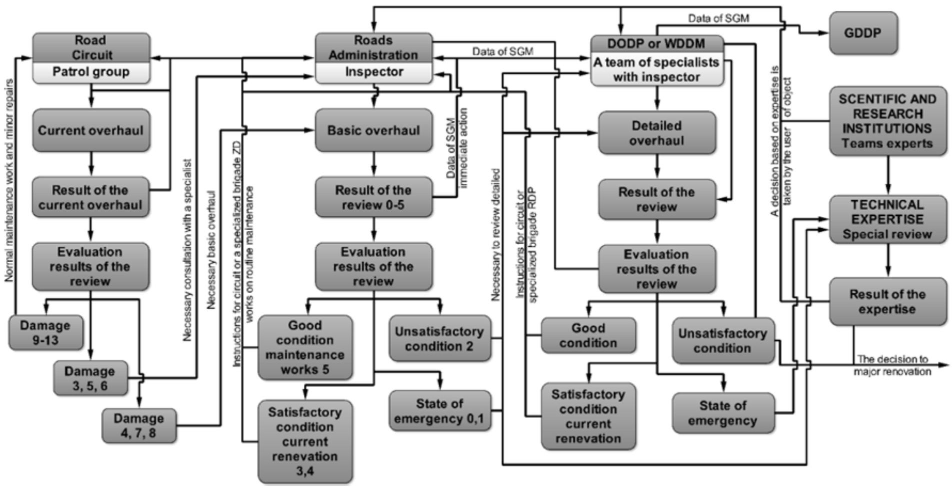

Fig. 6.11. Organizing scheme of bridge inspections

Current overhaul, short characterization

Its main purpose is to find as many defects affecting negatively the lifespan of a structure as possible. The examples are:

- road, access route and pavement damage,
- the lack of or wrong signposting of an object,
- water stagnation showing irregularities in functioning of the draining systems,
- water eruption as evidence of a malfunction of the insulation of the object,
- contaminations in an object,

- breakdowns of structural elements of an object (deformations, decrements in material, cracks, clefts and dislocations),
- landslides.

An incorrect construction work may result in defects, such as deficiency of rectilinearity of elements (curbs, rails). The effect of it is e.g. a settlement of pillars or too big a deflection of a structure [6].

Reviewing contractor is a person who should be trained by an inspector of the bridge, designated by the representative of the road administrator.

Method of carrying out an overhaul. An examination of an object is made from the level of the road and in special cases (e.g. when a threat of condition for structure is stated) at least once every six months from the level of the land under the object and next to the object.

Frequency of overhauls. The regulations [49] determine carrying out such inspections during every road control of the road where an object is situated and after the occurrence of:

- cataclysms (landslides, fires, floods, seismic waves, ice flow),
- road accidents,
- passage of oversize cars.

Documentation of inspections. Information about conducted inspections must be confirmed by an entry into the report at least twice a year in March and October and any time an irregularity is detected. Inspectors are obliged to store the reports for three years since inspections [4], [49].

Basic and extended inspections

The basic inspection, also called the periodic annual control, is an examination which has to be performed at least once a year in the second or third quarter in the order compatible with the year-long schedule. Obviously, in the case of sudden, unpredictable situations such as natural disasters or road accidents, the inspection must be conducted immediately, on the basis of an analysis of the current overhaul findings. It needs to be underlined that carrying out the basic review is not obligatory in the year the extended one is done.

The aim of an annual periodic control is to present the factual technical state of a bridge along with essential remarks resulting from safe moving around the object or the range of maintenance works performed and defects appearing during exploitation. The inspection examines among others [4], [6], [49]:

- damage to objects, property or environment connected with the safety of people using it,
- damage having a negative influence on the state of a structure which may lead to a disaster,
- regulations providing for the secure use of an object,
- damage identified in current or basic inspections concerning present maintenance or urgent elimination of impairments irregularities,
- damage to installations and equipment used for environmental protection,

- damage to the elements of accoutrement,
- damage to fastenings or cover foreign devices threatening road users' or an engineering object's safety to call owners for making inspections and remove damage,
- the necessity of making an extended or detailed inspection despite the inspection schedule admitted previously,
- the necessity of making a special overhaul (survey),
- an implementation of recommendations resulting from the former inspection.

Inspection basics meet the requirements of a periodic control set out in [39], [49].

The extended overhaul, also called a periodic five-year control, should be performed at least once every five years, in the second or third quarter, in the order of problems identified in the current or basic inspection; such a control ought to be executed immediately. Additionally, the frequency of such overhauls should be boosted (eg. once every three years) in the case where average traffic is bigger than twenty thousand vehicles per twenty-four hours as well as when:

- archs are made of prebuilt balks of the following types: CZDP, Płońsk, Gromnik, Korytkowego, Strzegom, T (old type),
- bridge abutments are of prebuilt elements.

The purpose of such a review is identification of irregularities relating to the use of an object, the aesthetics of a structure and its surroundings, its present technical state, necessary routine maintenance works, as well as installations and equipment for environmental protection. In addition, there should be performed electrical wiring, lightning and ventilation tests to allow the structure to be used. In addition to the found irregularities, on the basis of a fundamental review, the following should be shown [49]:

- damage to the equipment necessary for the proper use of an object (e.g. the electrical system, lightning protection, ventilation equipment),
- The usefulness of an object to the use,
- changes in visual aspects of a facility and its surroundings.

Control protocols of external equipment and installations and equipment enabling the use of an engineering object should be attached to the inspection report of such an object [49].

Extended inspections meet the requirements of a periodic control set out in [39], [49].

The controller of a review. The person performing the basic and extended review should have building qualifications in the relevant specialisation, be a member of the Chamber of Civil Engineers, have an appropriate certificate issued by the Chamber and have trained in conducting such inspections. Such a specialist is called an inspector. They also must be equipped with a written authorization to carry out inspections resulting from a positive assessment of their qualifications and professional ethical attitude by the head of local roads [4], [49].

Both checks should be carried out in the presence of the road area manager or a person representing him [49], in the case of bridges:

- objects recognized as monuments,
- bridges with a length exceeding 50 m, with a theoretical span of more than 30 m or moving bridges.

As far as the five-year periodic inspection is concerned, GDDKiA road workers should obligatorily participate in it with regard to the following areas:

- performance, maintenance and operation on scaffolds and mobile devices that enable a direct access to structures and equipment of buildings under examination,
- sharing concealed structural elements and perform other work recommended by the team leader making the inspection.

In case of inability to perform a research, for example due to the lack of proper equipment, specialized companies may be commissioned to put up scaffolding or rent equipment necessary to conduct the inspection.

Technical checks of electrical installations, lightning and ventilation devices, enabling the use of facilities and foreign equipment should be performed by persons having the qualifications required for the supervision over the exploitation of these installations and equipment. A contractor here can only be a member of the Chamber of Civil Engineers in the period of the inspection and the holder of a relevant certificate issued by this chamber [49].

Method of performing a review. The typical beginning of a review is to familiarizing with appropriate evidential and technical documentation of the object.

Both inspections, the annual and five-year one, should examine the technical condition of an object, its environment, and include research and measurement (in the primary). The above-mentioned measurements and examination of a facility are carried out:

- in the case a basic review from the level of the road and the land, under the object with binoculars, ladder or scaffolding,
- in the case of an extended review – from the level of the road and the land, under the object and approximately 1 m from it, allowing an analysis of the state of damage with „ the naked eye" [49]. If needed, one should use devices that allow an access to the examined element [4], [49].

The extended level range also includes preparing photographic documentation of an object and its damage.

Collected results of surveys should be included in appropriate protocols. In the case of defects having a large impact on the safety of the structure, people moving across it, property or the environment, it is necessary to inform the supervisor immediately and sent them a copy of the inspection report to take adequate decisions [4], [49].

An annual inspection schedule regarding engineering facilities should be prepared first, while schedules of the surveys of facilities located on national roads must be approved by the head of the road area of GDDKiA and put to the GDDKiA, by the end of the first quarter of the year. The confirmation of a review is a periodic inspection protocol.

Detailed overhaul

A detailed overhaul is a thorough analysis of the actual state of all structural elements, containing measurements and tests. It is conducted in order to determine the technical condition of an object together with the conditions of safe use, the nature and scope of its renovation or remodelling.

A detailed inspection shall be carried out within the warranty period and with regard to the following types of bridges:

- longer than 50 m,
- containing spans of over 30 m,
- containing mobile spans,
- selected following a basic or extended review.

The review does not take into account all electrical, electronic and mechanical equipment installed in road engineering objects, and does not also apply to enclosed structures associated with them, e.g. lifts, ventilators, air intake, air con or a monitoring centre. These devices and objects should be controlled in accordance with separately prepared operating instructions. An overview does not also include an examination of the technical condition and suitability for use of foreign devices located on the object or common pillars [49].

According to the regulations [49], a detailed inspection must be performed at least once every five years under the current plan of maintenance of bridges, from April to the end of September, in the order of the annual schedule of detailed inspections, as well as in emergency mode in the case of a decision made on the basis of the results of a basic or extended review. The first detailed review of newly built facilities should be held before the end of the contractor's warranty period [4].

Purpose of a review. The main task of a detailed review is to provide information on [4], [49]:

- the technical and functional conditions of a facility,
- damage to a facility, which should be removed by performing works under the renovation plan,
- the necessity of improving the functional (operational) characteristics of a facility under the rebuilding plan,
- The need to conduct an expertise (special review).

The contractor of a review. In the case of the detailed review, contrary to the current, basic and extended ones, the contractor is a team of experts appointed by the head of GDDKiA, under the supervision of a divisional bridge inspector. According to the regulations, contractor performing the review may be a team of specialists under the direction of an engineer specializing in bridges, who has been trained and has the authority to carry out detailed inspections, as well as have building qualifications in the relevant specialisation and is a member of the Chamber of Civil Engineers during the review period and holds a relevant certificate issued by this chamber. This manager should also have a written authorization to carry out detailed inspections, which is only issued by the head of GDDKiA on the basis of qualifications and ethical-professional attitudes of the manager [4], [49].

It is recommended for the inspector who has conducted a basic or extended review to participate in the detailed one.

During the five-year periodic inspection GDDKiA road employees should participate in:

- putting up, maintenance and use of scaffolding and mobile devices that enable a direct access to structure elements and equipment of objects under examination,
- measurement and inventory works,
- sharing concealed structural elements,
- performing other works recommended by the head of the team conducting the review.

In case of inability to perform a research due to the lack of proper equipment, specialized companies may be commissioned to put up scaffolding or rent equipment by means of which the research could be carried out [49].

Method of conducting an inspection. The standard beginning of a review is to look at the relevant technical and accountancy documents of an object i.e. a list of objects, book of object, as well as cards, records and reports from previous inspections conducted in the minimum of five past years. The purpose of the detailed review is:

- a visual inspection (examination of an object and its surroundings),
- control mechanical research,
- examinations using instruments and apparatus,
- measurement works.

In the case of the above-mentioned object inspection, measurements and tests are performed:

- from the level of the road and the land, under the object and at a distance of 1 m from the element, allowing an analysis of the state of damage with „the naked eye" [49]. It may be necessary to use devices that allow access to inspected elements.

A group of experts carrying out the review shall be responsible for:

- an analysis of the conformity of the object parameters with the data in the register and technical documentation,
- photographic documentation of the facility and its damage,

- drawings (sketches) of the object [4], [49].

The confirmation of the performance of a review is a "*detailed review report*", which includes:

- the title page,
- the table of contents,
- a protocol of the detailed review,
- attachments:
  - an overall drawing (side view, top view, of the base of structure's spans, characteristic cross sections),
  - a drawing of an inventory of damage with clearly marked: a kind of damage, the location of damage, the dimensions of damage and the intensity of damage.
  - a protocol of levelling measurements with a comparative analysis of previous results and information about the date, scope and contractors of these works,
  - other protocols of measurements or examinations,
  - copies of orders and/or decisions relating to the object
  - other documents, if necessary [4], [49].

The report must contain a description of all the perceived failures and irregularities in the facility and its surroundings. When performing a failure analysis, it is necessary to specify its size, intensity of its occurrence and its range, relying on photographs and pictures documenting the damage. If necessary, report templates need to be expanded by additional pages or boxes [49].

The team preparing a detailed inspection must prepare the report in duplicate. The holder of the original is the departmental bridge inspector, while the holder of the copy is the regional bridge inspector who encloses it in the book object – if required. It is permitted to keep records in an electronic form, provided that it is adequately protected against data loss and the electronic signature is used.

Special review (survey)

The special overview also called "survey" is carried out in exceptional cases:

- when the inspection methods used in the review are not detailed enough to determine the state of an object, its parts or determine the causes of any damage,
- if it is necessary to assess whether an oversized car can cross the bridge,
- to assess the capacity of an object after repairs or after its modernization,
- to determine the mechanical properties of materials
- to determine technical and economic conditions to take a decision on further exploitation of a facility.

An expertise is carried out by specialized institutions or teams of experts trained in performing specific overhauls, possessing necessary equipment [4], [6] [49].

6.4.3. Rail bridge inspection

The primary purpose of an inspection and diagnostic tests of railway bridges is to evaluate the technical condition of these objects. This generally involves doubts as to the safe exploitation of objects while incur costs of the current operation or the decision to strengthen or rebuild the object. The scope of the fundamental tasks linked to maintaining railway engineering facilities should include:

- maintenance management,
- facilities conducting diagnostics,
- planning maintenance works,
- implementation and acceptance of maintenance works,
- keeping records of construction and maintenance.

Fig. 6.12. Narrow-gauge railway viaduct a) broken and reconstructed b) reconstructed[11]

Railway engineering object maintenance includes procedures of the system maintenance, in particular:

a) the implementation of settled strategy of maintaining engineering objects,
b) diagnosis of engineering structures,
c) determining needs in the field of maintenance of engineering structures and planning maintenance works,
d) preparing technical documentation for maintenance works,
e) organization of tenders, contracts, construction supervision and acceptance of maintenance works carried out by foreign contractors,
f) revision of documentation and technical equipment to carry out foreign civil engineering structures,
g) keeping records of construction and maintainability of railway engineering structures and their archiving,
h) cooperation with territorial units of the state administration, local governments, road and water managing agencies with regard to maintenance of railway engineering objects.

[11] Photos by K.Śledziewski

All components of the system maintenance of engineering structures located on railway lines should be carried out in such a way as to ensure a long service life of these objects, i.e. to limit the possibility of their degradation. All damage of engineering objects, both resulting from normal use (aging or fatigue), as well as from random events (transport of oversized loads, accidents or natural disasters), accelerate their degradation. From this point of view, adequate maintenance of facilities and early detection of defects as well as their removal are very important elements of the system (Fig. 6.12).

Often due to insufficient financial resources, maintenance or upgrading of objects must be planned in the long term, therefore, it is important to allow for an appropriate assessment of objects, which helps to plan maintenance works in order to prevent their premature destruction.

Inspection system

The aim of a survey is to determine the technical condition of engineering objects and their fitness for use [6]. Maintenance of railway engineering facilities includes:

1) a visual inspection,
2) periodic inspections at least once a year (annual checks),
3) periodic inspections at least once every five years (five-year checks),
4) special inspections (initiated by the chief engineer).

Tab. 6.4÷Tab. 6.7 summarize the purposes, frequency, contractors and conclusions, which should be completed each of these inspection [45], [46].

Tab. 6.4. The main parameters – Visual inspection

Visual inspection	
Purpose	check if the object does not pose a risk to the safe using find damaged parts of objects visible from the track
Frequency	on mainline tracks, main essential and major additional lines, main and first class lines – twice a week on mainline tracks, main essential and major additional secondary and additional lines of local importance – once a week remaining tracks of all categories of lines – once a week a closed track line or out of operation (regardless of the categories of lines) – every 6 months
Contractor	visual inspection carried by a lineman
Conclusions	all defects and irregularities of tracks, bridges, tunnels, viaducts, culverts and possibly other railway equipment noticed by a person performing a visual inspection should be stored in the book of inspection and on their basis the specified: the need to perform an additional non-scheduled review in the scope of the annual review within a specified period introduction of restrictions in the operation of an object

Tab. 6.5. The main parameters – Annual checks

Annual checks	
Purpose	evaluation of the technical condition of individual elements of an object identification of potential damage to individual elements of an object determine the type and the take-off the necessary current maintenance of work usability evaluation of an object
Frequency	once a year off-schedule, based on the conclusions from the analysis of a visual inspection
Contractor	diagnostic team comprised of employees, with the possible involvement of a bridge engineer or another employee
Conclusions	annual audit records are included in "Protocol of periodic inspection of a railway engineering object at least once a year" a check or five-year review of a special object within a specified period the type and take-off the current maintenance determination of the conditions of the operation of an object

Tab. 6.6. The main parameters – Five-year check

Five-year check	
Purpose	clarifying the technical condition assessment of elements subject to an annual inspection determination of the type, starting date and the cost of maintenance works in two strategies: maximum – work necessary to restore the original object and its technical parameters and utility minimum – works necessary to maintain minimum technical parameters and functionality to ensure its safe operation possible qualification of an object for a special inspection determination of the suitability of use of an object
Frequency	determined by the chief engineer
Contractor	diagnostic team comprised of employees, with the possible participation of other persons designated by the chief engineer
Conclusions	five-year check records are included in "Protocol of periodic inspection of a railway engineering object at least once every five years" subsequent five-year checks of the object within a specified period conducting a special inspection of an object or its components, stating the scope and timing of such a review the type and extent of maintenance works, an estimate of their cost determination of the conditions of the operation of an object

Tab. 6.7. The main parameters – Special inspections

Special inspections	
Purpose	the purpose and scope of special inspections is determined individually by the chief engineer and approved by the director of the railway lines
Frequency	the chief engineer determines dates special inspections of objects should be conducted when the results of other types of surveys do not provide sufficient grounds for any decision relating to determination of operational parameters and follow-up maintenance a special inspection with regard to the safe use of an object must be carried out whenever there are external factors affecting the object: the performance of the man or the forces of nature such as lightning, earthquakes, strong winds, heavy rain, landslides, ice on rivers, lakes, reservoirs, fires or floods, after every derailment of the rolling stock on the object or in its immediate vicinity which result in damage to the object or a direct threat of such damage that could endanger human life or health, safety, property or the environment
Contractor	performed by specialist units outside the company in the course of special inspections a section worker is constantly present on site, coordinating the safety of rail traffic and people, and observing conducted tests and measurements the chief engineer should be present at the facility during the performance of activities under special inspections
Conclusions	documentation of a special inspection includes: „The protocol of special inspection", which should be stored in the railway company's files „The protocol of special inspection", which should be stored in the headquarters of a railway company until the object is demolished (liquidated) an additional special inspection in the revised range special inspection repeated by another contractor identification of the type, starting date and approximate costs of maintenance works determination of the conditions of the operation of an object

On the basis of the table the following conditions can be noted:

- for all maintenance activities performed by employees of the railways in each case it should be determined whether the object's condition allows its safe operation, i.e. that there is no danger to the safety of use,
- in any inspection addition to visual inspection, the proposals should specify the conditions for the operation of a facility, i.e. the requirements relating to the weight or the speed of rolling stock travelling on the object, wherein even as a result of the visual inspection of the spot one can restrict the movement.

The methodology of a technical survey

Every case of maintenance work is intended to answer specific issues relating to the maintenance and operation of an object. These issues arise from regulations and instructions, but also can be defined in detail by the user of the object (the head of railways). When the scope of issues is known, the whole process of performing an inspection can be divided into four fundamental stages.

Stage I – before an inspection:

- analysis of documentation maintenance:
  - inspection documentation,
  - documentation of maintenance works,
  - operational documentation, in particular information about special events,
  - Book of the Construction Object;
- carrying out an on-site verification of the object,
- terms assumed for the purposes of an inspection and the scope of research necessary to perform the inspection, which will result in a draft inspection containing:
  - test program,
  - method of conducting an inspection,
  - equipment used for testing,
- if, on the basis of the above subdivisions and developed criteria such as price and time, a contractor is selected to perform an inspection, a safety supervisor gives instructions regarding the use of the facility to meet security conditions.

Stage II – conducting an inspection:

- coordination of the safety of rail traffic and people on the object,
- inventory sections of individual elements – if it is required,
- assessment of the individual elements of the object, in accordance with the assumptions of the inspection:
  - main girders assessment: inventory damage including its location, for example; measurement of the thickness of items, measurement of the depth of corrosion of concrete, measuring the diameter of a rebar etc.,
  - assessment of support: the location and extent of damage and irregularities occurring, such as: material defects, scratches, corrosion status, i.e.: leaks, deposits, efflorescence, measuring the correct location, i.e.: deformity, displacement,
  - assessment of the state of bearings: the correct location, i.e.: deformities, abnormal position, blocking or restricting traffic, the state of corrosion, i.e.: cavities, aging material,

  - assessment of pavement on the object and estate road: loss of ballast, the state fenders, the occurrence of sheet metal to allow proper rolling derailed of the rolling stock, rail wear,
  - assessment of the object: the correct location and height handrails, pavement condition of the working platform,
- possibly testing on site or sampling built-in structural materials for laboratory tests,
- condition of corrosion, i.e.: cavities, aging of materials, paint coating.

Stage III – analysis of the state of the object:
- an assessment of individual components of the object and an assessment of the whole object,
- determination of the geometric parameters and the strength of individual structural elements,
- determining the actual load capacity of the object,
- collecting all existing permanent loads, useful load, climatic load, determine the load from the lateral impact of the rolling stock, loads of acceleration and brake,
- setting-up of loads by combinations specified by standards [42], where the maximum forces should be set internally,
- calculation of the effort of individual structural elements including their current technical condition, fatigue or weakness on connectors (sections stretching),
- checking the conditions of the ultimate limit state (ULS) and serviceability (SLS).

Stage IV – drawing conclusions and making recommendations concerning the future operation of the object; at this stage an inspector should address all the issues that were identified in the assumptions for inspection, e.g.:
- present an overall assessment of the entire object,
- determine the scope, method and mode of urgency (runtime) of the necessary repair work,
- define restrictions (permanent or temporary) on the object if they are required to be implemented,
- recommend the frequency of inspections, increase the frequency of current inspections or determine the timing and extent of the implementation of the next special inspection,
- develop detailed recommendations for the operation of the object (e.g. exclude damaged platforms).

In developing the results of the inspection performed, one can use a diagram defining the four basic conditions of the evaluation criteria regarding an object, shown in Fig. 6.13.

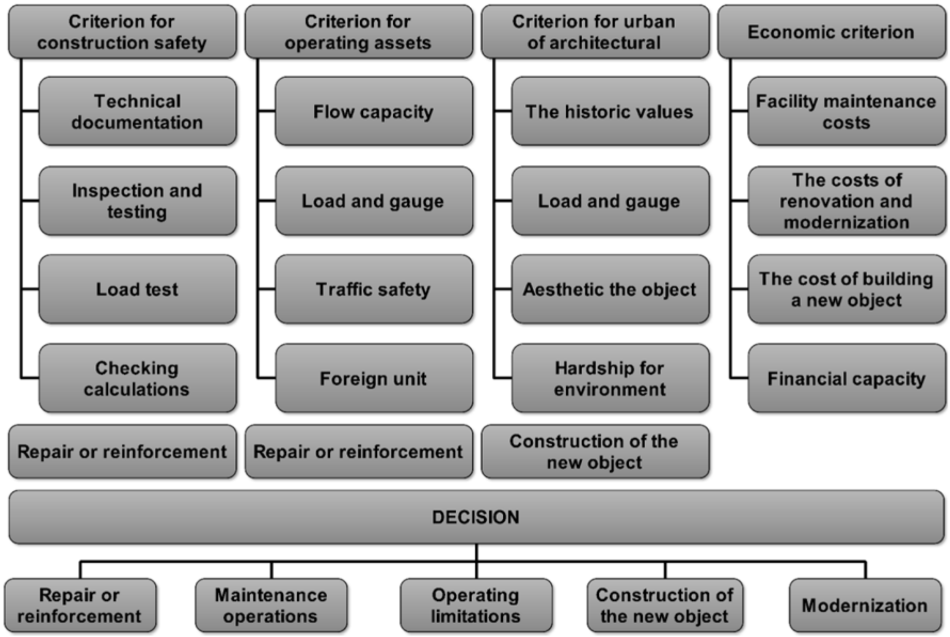

Fig. 6.13. Scheme analysis of the object railway

Before attempting to perform an inspection it is recommended to prepare a working draft of the object on which one can mark the location of damage. This sketch enables the analysis of changes in the structure in relation to the previous survey (if available). It should also be applied to the scheme design points where the shots for the photographic documentation of damage to the facility were taken, which enables the assessment of their development during subsequent reviews, and greatly facilitates an analysis of the state of the object.

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[45] Id-7 (D-10), *Instrukcja o dozorowaniu linii kolejowych*, PKP Polskie Linie Kolejowe S.A.,Warszawa 2005.

[46] Id-16, *Instrukcja utrzymania kolejowych obiektów inżynieryjnych na liniach kolejowych do prędkości 200/250 km/h*, PKP Polskie Linie Kolejowe S.A.,Warszawa 2014.

[47] PN-85/S-10030, *Obiekty mostowe*, *Obciążenia*.

[48] PN-99/S-10040, *Obiekty mostowe, Konstrukcje betonowe, żelbetowe i sprężone, Wymagania i badania*.

[49] Załącznik do Zarządzenia nr 14 Generalnego Dyrektora Dróg Krajowych I Autostrad z dnia 7 lipca 2005 r., *Instrukcje przeprowadzania przeglądów drogowych obiektów inżynierskich*.

Other

[50] *Inspection Report by Arminox*, Pier in Progreso, Mexico 2007.