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# ABOUT THE HISTORY OF ORTHOTROPIC BRIDGE DECKS

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***Keywords:*** *orthotropic steel plate*

## ABSTRACT

The development of steel bridges after the WWII is characterized by new constructive forms like: steel roadway closed sections, new static systems considering the roadway as a plate rather than a beam grid, new computational methods covering rod statics and statics of continuum, and determining the internal forces of the roadway as internal forces of an orthotropic plate. Moreover, the main girders, the roadway and the horizontal links connect statically and constructively in a whole monolithic structure. Thus, the construction follows better the real distribution of internal forces in steel structure. Loading possibility of high-quality steel material is well used, by which the economic effect of the construction increases significantly compared to the riveted old bridges. The application of this theory in Bulgaria in the period 1976 – 1992 allowed the Bulgarian bridge construction to build several unique by their size steel road bridges of steel orthotropic decks.

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## 1. Introduction

In the second half of 1945, after the end of World War II, the entire Central Europe and the European part of the USSR laid in ruins. Only in West Germany, out of 24 380 railway bridges 3149 were destroyed or damaged, i.e. 13%. 575 road bridges, (12%) being part of all 4827 bridges owned by the Ministry of Railways were destroyed. From 22 railway bridges and 20 road bridges on the Rhine, along the entire length between the Swiss and the Dutch border, at the end of the war none was left. All the 11 bridges on the river Weser were destroyed. Of the 24 bridges on the River Main 23 were destroyed, and out of 34 bridges on the Danube River – 22 were destroyed. Thousands more road bridges and most of the spectacular bridges on the roads laid in debris. Similar was the situation in other countries over which the storm of the destructive war passed [5]. Of vital interest to these countries was the fast construction of road and railway network. In Fig. 1, the process of reconstruction of railway bridges in West Germany during the period 1945 – 1955 is shown [7].

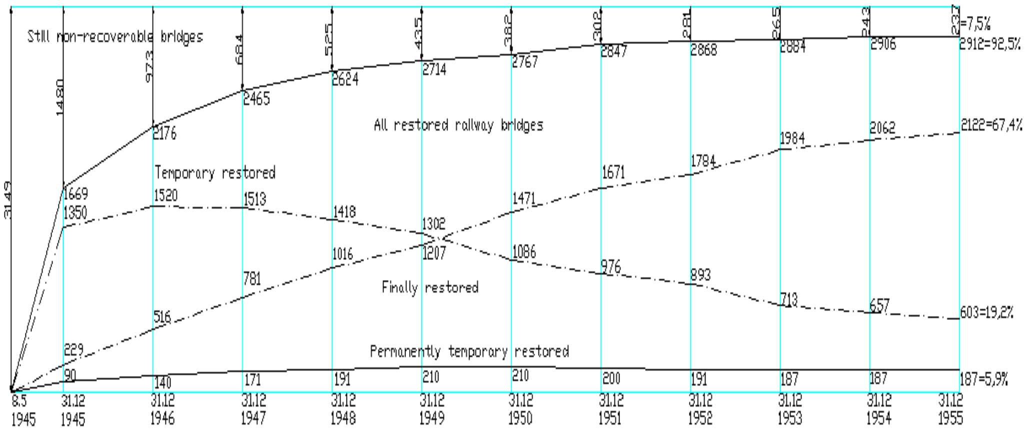



**Figure 1. Diagram of remainig railway bridges in West Germany for period 1945-1955, (Damage or destroyed railway bridges on 8 May 1945 equalled near 3149)**

The dire economic situation after the War and the lack of the building materials necessitated to seek the most effective ways of extending the life of the existing steel bridges, beyond their life-cycle, whose condition due to inadequate maintenance during the war years was worsened. For steel bridges, age limit is 60 and for massive is 90 years. Hopeless economic situation demanded decisive savings in steel and other building materials. This gave new impetus to the theoretical, constructive and economic development in all fields of engineering. New directions in the development of steel bridges were expressed; a) in amending the existing rules for loading and the calculation of steel railway bridges and adjusting them to the actual work of construction; b) in finding new forms, static structural systems and new computational methods that cover even more, like better distribution of forces which allow for increased safety factors to use the existing reserves in the bearing capacity of the structure and material; c) in improving the quality of construction materials and creating new kinds of steel; d) perfecting the art of welding and its wide use in the construction of road and railway steel bridges; e) in designing and execution of new connections.

## 2. New Computational Methods and Constructive Forms

### 2.1. Railway Bridges with Open Roadway

In order to fully use the load-bearing capacity of the steel and thereby to increase the economical effect of the structure, it is necessary to know well the real distribution of the forces in parts of the construction and their deformation. Only then can a proper and appropriate distribution of the material in the construction occur. To simplify solving statically an indeterminate and repeated space bridge system, it is usually seen as composed of separate planar systems which do not interfere with each other. This approach is completely arbitrary and not consistent with the requirement of knowledge of the real behaviour operation of the bridge construction. Measurements on existing bridges showed that the loading of the longitudinal beams depends on the deflection of the main beam and vice versa [15, 25]. In fact, there is a significant interaction between the main support system and road grillage. To reduce the overhead road grillage railway bridge structure of an open road-lane, it is usually recommended for the roadway to be assembled by segments. The striving today, however, is with appropriate measures to ensure full cooperation between the longitudinal and main beams, as the main beams are reduced considerably. This requires longitudinal beams to construct a continuous beam of vertically protruding props, not partially stiff connected as before. The vertical deflection of the supports of the continuous longitudinal beam is a result, on the one hand, of the elastic bending of the crossbars on which they are based, and on the other hand of the elastic displacement (deflection) of the main beams, to which they have crossbars attached. The exact study of the longitudinal beams as continuous beams on elastic feeding props, taking into account the interaction between the main and longitudinal beams is relatively heavy. C. Popp [33] developed for this purpose an approximate method that is used in the German codes for calculation of steel railway bridges. The influence of the elasticity of the crossbars at the bending moments of the longitudinal beams is so much greater, that the distance between the crossbars is less, in comparison with the supporting distance between the main beams, and the resistance of the longitudinal beams against bending is so much larger, in comparison to that of the crossbars. The influence of the displacements of the main beam at the bending moments of the longitudinal beams decreases with increasing the distance of the support of the main beams. For large supporting distances it can be ignored. In Fig. 2 influence lines of the moment of the longitudinal beam in point 2 of the present railway bridge, which spans 10 meters, are compared. The longitudinal beam was once viewed as a continuous beam on unmovable supports, then on the elastic supports, ignoring displacements of the main beams, and then under consideration of displacements the cross and the main beams. From the comparison of the three lines of influence seen, the size of the impact of displacement on the retaining moment in point 2 of the longitudinal beams is shown. The longitudinal beams can be regarded as continuous, and if indeed their continuity is established. This requires their upper and lower flange to contact with tensile and compressive flange plate and also to lie upon a console that would take the vertical load of the longitudinal beam and transmit it to the cross beam.

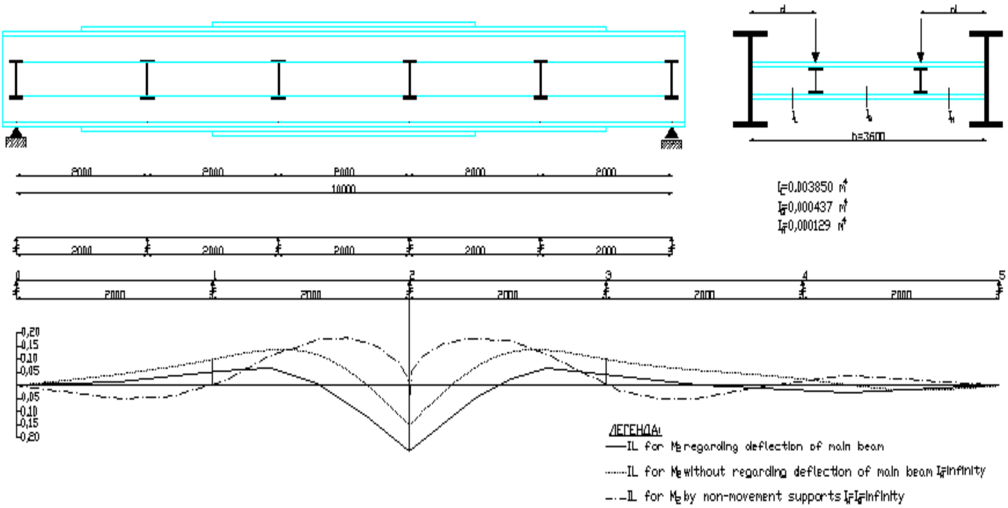



**Figure 2. Railway bridge of 10-meter span. Comparing influence line for a bending moment in longitudinal beam in point 2**

## 3. Road Bridges and Railway Bridges with Closed Roadway

### 3.1. Orthotropic Plate

In road bridges when roadway consists of road grid (longitudinal and transverse beams) and lying on her concrete slab, longitudinal and transverse beams are calculated as separate beams, independent of each other. The plate was seen also as a stand-alone bearing element independent of each other. Every part of the roadway performed only one function, without taking into account the interaction between the individual parts. Some progress was made, when a later reinforced concrete slab was joined by dowels to its supporting road grill; thus, ensuring collaboration between the steel beams and reinforced concrete slab which with its effective width increases significantly the moment of inertia of the beam. Longitudinal and cross placed beams have been considered independent of each other. However, the plate already performed two functions – it took acting upon *immediately*, and its load *gave way* at its supporting beams, and also worked as an integral part of the road grid (like upper belt of longitudinal and transverse beams). The next decisive step taken was: a more accurate coverage of the forces exerted upon the roadway, and to approach the actual work of the construction as the road began to be seen as a beam grid. Again, the plate was used as the upper belt of the beams. The calculation was conducted by using "rod statics" i.e. beam grid was seen as a system composed of individual bars (beams), without taking into account the spatial effect of the plate. *In fact, the roadway consisting of orthogonally intersecting longitudinal and transverse beams, whose upper belts are connected to the deck plate, is forced to follow the deformations of the main girder and thus to work as an integral part of the* *main supporting system.* The most decisive step in bringing and adapting the bridge construction to the actual distribution of internal effort was made when determining the internal forces of the roadway was reduced to determining the internal forces of an orthogonal anisotropic plate i.e. when they were calculated by applying *statics of rod to continuum method* [4, 5, 22, 23]. *Resistance* against bending of the roadway transverse plane to the axis of the bridge is much greater than resistance on its longitudinal axis. Upon this particular different elasticity of the roadway, in both cross and longitudinal direction, the explanation of the anisotropy of the plate followed. A classic example of transition from static rods to static of the continuum is the problem of investigation of compression of upper belts of open lattice bridges (with no upper stability link). F. Engesser resolved this problem *with unique elegance*, accepting the bending resistance of free frames (two verticals tied to a crossbeam) on which compression belt rests horizontally and its load is distributed evenly over the entire length of the corresponding field. Thus, Engesser reduced the problem of elastic support at specific points of compression of the rod to resolve the elastic foundation continuum. The study of F. Bleich on the same problem proved later that at considerable distances between the transverse frames, examining belt beam of elastic foundation, leads to practical accurate results. In 1942 research on multi-sectional grid-systems with continuous belts (nodes are joints) was also reduced to solving a continuum, as filling rods were imaginary replaced with a continuous wall of constant density [2, 5, 26]. The calculation results were confirmed by model tests. All the results from the study of a system as a continuum – imitating a real situation being replaced by densely distributed continuum elements, i.e. the higher the statics indeterminations. Knowing that, the more elements a structure is made of, the higher its security. Removal or destruction of an element in any case means destruction of the entire structure; keep in mind that the functions of this element will be borne by neighboring elements. For example, the security of the chain which consists of separate vertebrae is much less than that of the wire-rope, which forms a plurality of separate fibers. The new development of statics will result in secure and economical structures, each part of the bridge construction dimensioned at a different safety factor depending on the importance and the way it is engaged. When the roadway is configured as an orthogonal – anisotropic plate, abbreviated as "*orthotropic"* plate, it carries and at the same time applies in the calculation of the bridge cross-section the cross-section of the *main beam*, but also the cross-section of the *longitudinal secondary beams* and the congruent deck plate. *The orthotropic plate forms up the upper belt of the main girders.* Thus all parts of the structure, except for the implementation of their immediate destination, are included in the joints (ties) execution of the functions of the main structure. For example, the deck plate works as a part of the main supporting system, as an integral part of orthotropic plate (flange of the longitudinal and transverse beams), and then as separate bearing element which takes direct current load and carries it to the road-deck. For that reason, the loading resistance of the construction and the material used are the appropriate. The constant load (as of composite steel-concrete beams) is vital to the economy of the bridge, especially in loadbearing and supporting large distances. These disadvantages of reinforced concrete roadbed drops in steel roadway, which is made of 12 – 16 mm thick steel road plate tied to its supporting orthogonal grill of longitudinal and transverse ribs. On the steel plate a layer of 5 cm thick asphalt is applied, as its weight is only 1,25 $kN/m^2$. Naturally, in the steel roadway, due to the nature of orthotropic plate, longitudinal and transverse ribs form a thick beam grid. With the construction of the steel roadway as orthotropic plate, the steel-road-bridge structure has entered a new stage of development. Dead load gets reduced many-fold. Connection of the orthotropic plate and the main beams into a solid main supporting structure allows for facilities to be build, which by its lightness and elegance have far surpassed borders unknown till now. For example, for an entire wall of beams it was thought that a span of 100 meters is the maximal and more economical. Today, these are easily transferred supporting stretches of over

260 meters. Fig. 3*a* is the cross section of the suspension bridge Köln-Mülheim, over the Rhine river in Köln, where the structure and the steel orthotropic deck plate can be seen. In this model, the construction height is considerably reduced in comparison to the old type of structure, as it further allowed for greater saving of steel material.

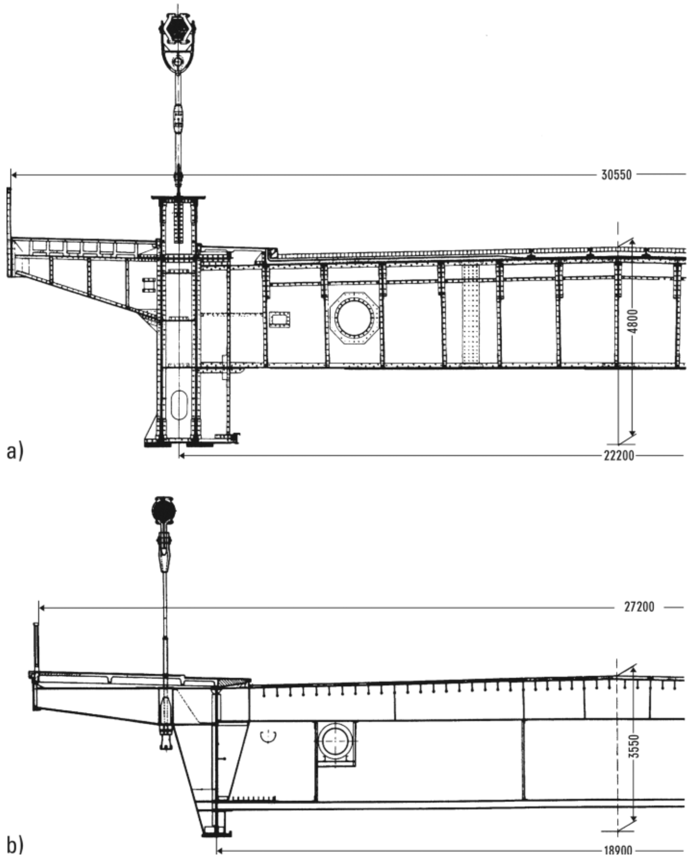



**Figure 3*a,b*. Mülheim suspension road bridge over the river Rhine, of supporting distances 85,0 +315,0 + 85,0 m; Sections: old-*a*) and new bridge-*b*)**

## 4. Generally on Orthotropic Plates

The development of steel bridges, as we have seen, is characterized by new constructive forms – steel roadway, closed cross-section, new static systems – considering the roadway as a plate rather than a beam grill – by employing new computational methods – going from rod statics to statics of a continuum and determining the internal forces of the roadway as internal forces of an orthotropic plate. Moreover, the main girders, the roadway and the horizontal links connect statically and constructively in an entire monolithic structure. Thus, the structure follows better the real distribution of the internal forces, whereupon the possibility of the higher quality steel material becomes better utilized, whereby the economy effect of the construction increases significantly compared to old bridges. Fig. 3 shows the old roadway built in 1929, a suspension bridge Köln-Mülheim and the new suspension bridge built in the same place in 1951. Comparing the two roadways, as shown, a great reduction of dead load has been achieved in the last twenty years. The pavement, cement-bed insulation and concrete filler of the old bridge weigh 4,0 kN/m$^2$ – on average, versus 1,25 kN/m$^2$-weight of 5-cm thick asphalt layer for the new bridge. The thickness of the steel plate varies from 12 to 16 mm. Because of the specific plate thickness, due to structural reasons, the design of the roadway requires the most adapted approach to be found for the distribution of the ribs and for their best dimensioning. These ribs correspond to the longitudinal and transverse beams of the normal road grill, as is shown in Fig. 3 for the old and new Mülheim bridge; however, they will have to adapt to their functions in an orthotropic plate. When designing the roadway as a real-size plate there *are two principles* to be applied that lead to an entirely new system of reinforced ribs where less material may be used. *The first of these principles is*: the path of the load from the moment it is launched onto a driveway to its placement onto the main structure should be the shortest. On Fig. 3*a* the load, marked as dead load and live load – through the longitudinal beam it is transferred onto the cross-beams and from there on to the main beams. Due to the distance between the cross-joists here which is very large, the path to transfer the load onto the main beam is long, therefore the road grillage becomes quite heavy. Moreover, the use of roadway material will be fully utilized when the entire plate turns into flanges of longitudinal and transverse ribs. Therefore, the distance between the ribs has to be determined so as to be equal to the effective width of the plate (Fig. 3*b*). *The second principle* is that the concentrated loads must be distributed, if possible as much, onto the roadway. This can be achieved only if longitudinal and transverse ribs laid onto their road slab are continuous. The computational and the constructive inset of hinge joints will restrict the load distribution. For transporting a load like: dead load, people, road-vehicles, etc., which is located parallel to a propping line (i.e. longitudinal ribs), it is necessary for the road structure to have resistance against bending only in the transverse direction. For concentrated loads, however, a resistance across bending in all directions is required, as in an isotropic plate. In reality, the bridge loads are distributed and concentrated, however, due to the large width of the bridge the load is mainly distributed. The optimum road grillage must have resistance against bending in all directions, but in the transverse direction it should be much greater than in the longitudinal. At the bridge Köln-Mülheim the ratio of resistance in the longitudinal and transverse directions is 1:20, while the old version – 1:8 [5]. To find the most suitable distribution of the ribs and the most efficient cross-section, it is necessary to draw diagrams of the weight of steel embedded in the roadway. Usually the width of the bridge, the load and thickness of the plate are specified, but all these vary upon the distance between the longitudinal and transverse ribs, as well as their cross-sections. After these diagrams are calculated, it is then easy to determine the most economical structure of the grillage ribs. A similar study is generally only possible if there is a calculation method where the real distribution of forces and the complete *depletion* of static reserves of the structure is connected with practically affordable computing. One advantage of the theory of orthotropic plate is that it enables engineers to utilize an additional load resistance of a construction, which by the known methods of calculating a beam grillage has not been taken into account. This reserve lying in the resistance of the road deck against torsion, which as a result of the eccentric arrangement of the plate relative to a parallel plane passing through the center of gravity of the overall cross-section, is significant. The effective resistance to the roadway torsion can be determined only through test models. For the Köln-Mülheim Bridge, laboratory tests were conducted on one element of the actual roadway in a manner closer to the actual load of the bridge [22]. Measured were extensions and deflections of the structure, upon which conclusions about the stress and load bearing capacity of the plate were made. While isotropic plate resistance against the bending and torsion resistance are equal, the model resistance against the torsional forces accounted for approximately: $0,3\sqrt{D_x D_y}$ where $D_x$ and

$D_y$ are resistance forces against bending along the axes of $x$ and $y$. The model was fabricated of steel S235 (St 37) in thickness of the bridge deck = 1 cm, distance between the longitudinal ribs 30 cm and 100 cm between the transverse ribs. The plate was loaded in two adjacent areas along their entire width. The load was being transferred onto the plate by using pads of hard tire (size 16/30 cm) corresponding to the contact surface of the wheels of a motor car, whose pressure according to the norms of load is 0,0833 kN/cm$^2$. In testing the plate through exerting a load of 0,312 kN/cm$^2$ – the largest vertical displacement of the plate equal to 0,24 cm was measured, while it remained at complete elasticity. Upon a load of 0,782 kN/cm$^2$, the largest vertical plastic displacement of 0,8 cm was measured, and also an elastic movement of 0,32 cm. In this metric test, the ultimate load was not reached. On assumption that the limit load $P_{ult}$ corresponds to a measured pressure of 0,312 kN/cm$^2$, the estimate strength of a plate would be 0,312:0,0833 = 3,75. It shows how great the load resistance of the plate is. Based upon the results of the laboratory study, prof. K. Klöppel [22] came to the conclusion that there is hardly another support system that better suits the character and qualities of steel, like a steel plate anchored on four edges. As of the specific advantages and competitiveness of the orthotropic plate compared to other roadway-systems, what attests for its properties is its mass application in recent years. For the first time an orthotropic plate structure was employed upon the calculations of W. Cornelius by M.A.N. after the year of 1945 for building several mobile road bridges, and then for a large street bridge in Mannheim over the river Neckar. As announced in 1948 competition for the bridge over the river Rhine, on the site of the destroyed bridge Köln-Mülheim, 39 projects were presented of which 20 hanging bridges. Only the M.A.N. company proposed suspension bridge with a steel roadway with orthotropic deck developed by W. Cornelius. This novelty to be implemented for such a large facility (of supporting distances 85 + 315 m + 85 m) was met with uncertainty by both Commission and other participating companies in the competition. After a vital voting, choice fell on the orthotropic plate. Building such structures far after the construction was ended in 1948, stresses the fact that the majority of steel bridges built after 1950 were designed as a roadway made of orthotropic deck. Of all the bids tendered in 1955 – a project-contest for a fourth bridge over the river Rhine in Köln, 38 of them suggested steel bridges made of a steel roadway as an orthotropic plate.

## 5. Theory of the Orthotropic Plate

### 5.1. The Theory of Maksymilian Tytus Huber

Orthogonal – anisotropic plate is characterized by the fact that its resistance against bending in two mutually perpendicular directions is different and its resistance against torsion can be arbitrarily large. We can look at it as being composed of an isotropic plate (road, steel or concrete) and reinforced longitudinal and transverse ribs (beams) (Fig. 3). Orthotropic plate theory was developed by prof. M. T. Huber from the University of Lvov concerning the solving of a problem of a cross-reinforced concrete slab [16 – 20]. It was built on the same assumptions on which the theory of isotropic plate was built, namely: a) the thickness of the plate is small compared to the length and width, as a result of which, the superimposed points from the normal line to the middle plane of the plate remain after the bending the plate on a straight line which is perpendicular to the deformed middle surface; b) movements perpendicular to the plane of the plate are small compared to the thickness, due to which the appearing in the middle surface of the plate angular and linear deformations can be ignored.

This leads to the assumption that the average surface of the plate is non-deformed, i.e. it is non-stretchable (movements there are equal to zero); c) the linear deformation of the plate thickness is equal to zero. XY coordinate plane right-oriented coordinate system coincides with the center of non-deformed plane of the plate, which is at the same time a plane of symmetry.

In the bridge deck, the longitudinal and transverse ribs are situated on the underside of the plate, and its middle plane is no longer plane of symmetry (Fig. 4). The average plane of the isotropic plate (deck plate) does not match the parallel plane passing through the center of gravity of the orthotropic plate (deck plate and ribs), and the stresses in the middle plane of the isotropic plate are equal to zero. The premise that the average plane of the isotropic plate is not stretchable, i.e. the linear and angular deformations are zero, is not executed [8, 36].

However, the theory of Hubert used to determine the internal forces and orthotropic plates, according to Fig. 4, as they are common in bridge construction. For a complete explanation of the theory of Hubert on the orthotropic plate, it will be described briefly. We know that in the second decade of the 20th century, according to [24], the calculation of reinforced concrete slabs was based on a simple structural model, essentially based on the beam theory. In the method attributed to F. Grashof [13], for example, a rectangular slab is divided into two orthogonal strips and the respective deformations and internal forces in the slab strips in the in *x* and *y* directions calculated at the points of intersection based on the condition of the equality of the deflections. The torsion in the slab is neglected in this method. On the other hand, tests on reinforced concrete slabs with the same amount of reinforcement in the *x* and *y* directions confirmed the validity of Kirchhoff's plate theory for homogeneous and isotropic slabs [21]. However, it could not be applied directly to reinforced concrete slabs purely for the reason that the bending stiffness of a reinforced concrete slab, depending on the reinforcement, can assume very different values in different directions [16]. This is why Huber, in 1914, developed the general theory of reinforced concrete slabs reinforced in both directions and derived the differential equation for their deflection $w(x,y)$ [16]. In February 1929 he held a number of lectures at the Swiss Federal Institute of Technology in Zurich and in that same year these appeared in the form of a monograph in German, published in Warsaw [17]. Witold Nowacki drew attention to the origins of the theory of the orthotropic plate as early as 1951 [30]. After Huber has talked about fundamental but also critical points in the theoretical foundation of tests in reinforced concrete construction, he derives the differential equation of deflection $w(x,y)$:

$$D_x \frac{d^4 w}{dx^4} + 2H^4 \frac{d^4 w}{dx^2 dy^2} + D_y \frac{d^4 w}{dy^4} = p(x, y)\,, \tag{1}$$

due to load $p(x,y)$ with the help of the energy principle [18]. Applied to orthotropic road decks on steel bridges, Huber's differential equation contains the plate bending stiffness transverse to the axis of the bridge (bending stiffness of the road deck plate) $D_x$ , the plate bending stiffness in the direction of the bridge axis (bending stiffness of the longitudinal stiffeners) $D_y$ and the effective torsional stiffness:

$$H = 0,5(4C + \mu_y D_x + \mu_x D_y)\,, \tag{2}$$

for thin, homogeneous-elastic but orthogonal-anisotropic plates. Of course, Huber's theory applies to all thin, homogeneous-elastic and orthogonal-anisotropic plates such as steel or reinforced concrete. In equation (2): • $2C$ is the pure torsional stiffness, • $\mu_x$ is the lateral strain due to normal stress in the *x* direction; • $\mu_y$ is the lateral strain due to normal stress in the *y* direction. In the isotropic case the plate bending stiffnesses or lateral strains in the two directions are equal, i.e. $D_x = D_y = D$ and $\mu_x = \mu_y = \mu$. The pure torsional stiffness in this special case is: $2C = D(1-\mu)$; and entered into equation (2) this results in the value $H = D$, which means that Huber's differential equation (1) is converted into Kirchhoff's differential equation for plates [21].

$$\frac{d^4w}{dx^4} + 2\frac{d^4w}{dx^2dy^2} + \frac{d^4w}{dy^4} = \frac{p(x,y)}{D}, \tag{3}$$

The differential equation (1), which Huber derived in the Journal *Der Bauingenieur* [19], was used by W. Cornelius in his version of orthotropic plate theory [4, 5]. A steelwork structure which adopted reinforced concrete, as during the 1950s and 1960s, encouraged a far-reaching development of the theory of the orthotropic plate, driven by the technical progress in steel bridgebuilding (Fig. 8*a*) and aircraft construction (Fig. 4*a*, 4*b*).

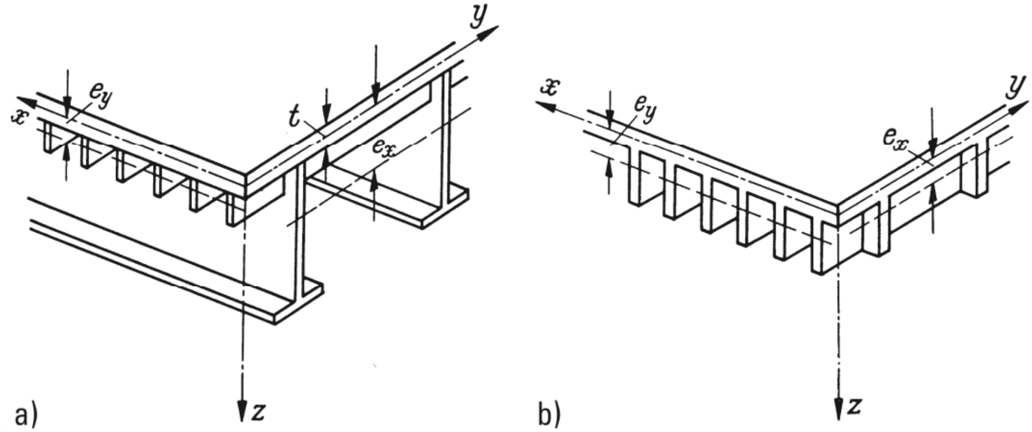



**Figure 4*a,b*. Plate cross-sections around 1960; flat steel plate (bridge-building)-*a*), integral plate (aircraft construction)-*b*)**

## 5.2. The Theory of Wilhelm Cornelius

It was in 1945 when Wilhelm Cornelius suggested the theory of orthotropic plate to determine the internal forces and displacements of practically occurring carriageways for linear and distributed loads giving solutions to the form of Fourier rows. The path for structural and economic development of such steel bridge structure was cleared. The study of the steel roadway as orthotropic plate minimized the use of ready-made formulas, allowing for new study-cases to determine the most economic and favorable distribution of longitudinal and transverse ribs. According to [24], six months after the new Köln-Mülheim suspension bridge was opened, the Klöppel's student, Wilhelm Cornelius, revealed the theoretical basis behind his recipe for success [4, 5]. Cornelius consciously completed the transition from a single member to continuum analysis. It was important that Cornelius recognised the genesis of the loadbearing systems as an organic development from discontinuum to continuum, so to speak, which he also observed in reinforced concrete construction. Such a change in the modelling of loadbearing structures, from member to continuum analysis, was not new because "even the progress in reinforced concrete constructions" replaced, for example, the previous lattice structural design by the structural design of shells and folded plates [4, 5]. Cornelius's work was based on the plate theory developed by Maksymilian Tytus Huber [16 – 20] for reinforced concrete construction. He solved Huber's differential equation for orthotropic plates for various types of plates such as a steel plate with a group of rolled sections and a grillage in conjunction with a concrete slab, i.e. he specified integral functions for the deformations and internal forces and tabulated the constants for the integral functions for common types of loading.

### 5.3. The Theory of Guyon & Massonnet

It was in 1946 that Yves Guyon presented his theory of a zero-torsion grillage based on Huber's orthotropic plate theory [13]. Charles Massonnet (1996) generalised Guyon's [28], i.e. at the same time as the grillage theories of Leonhardt/Andrä and Homberg. Massonnet devised graphs for simply supported beams, constant moment of inertia and identical moment of inertia for all main girders. He therefore created another method for the simple analysis of grillages. Over the years 1955 – 1960 Konrad Sattler [34], extended the graphs to cover main girders with a varying moment of inertia, perimeter and inner beams with different moments of inertia of/for any structural system [21]. Finally, in 1966, the monograph of Bareš and Massonnet appeared [3], which embraced all the findings based on the Guyon/Massonnet method and delivered a series of new ideas and experience.

### 5.4. The Theory of Pelikan & Esslinger

This method [31] differs from the others with its exceptional engineering approach to solving the problem. The approach is based on the actual parameters of the orthotropic bridges, as they were found in the period between years 1955 and 1975. For example, thickness of the orthotropic plate – dimension equal to 12 mm was adopted, the distance between the stems of the longitudinal ribs was assumed to be 300 mm, the distance between the transverse beams was accepted to be 1,0 – 1,80 m with open cross-sections of the longitudinal ribs, and between 2,0 – 3,0 m by closed cross-sections of the longitudinal ribs. The results of the studies have been developed in the form of tables and graphs, resulting in the shortening and simplification of the calculation of the orthotropic bridge compared to the W. Cornelius method.

### 5.5. The Slope Deflection Method for Orthotropic Plated Bridge Decks

The composition of an orthotropic bridge deck is highly suited for modeling by applying the 'Slope-Deflection Method' [1]. The structure with several trapezoidal longitudinal stiffeners allows for the swift solution of the problem for a simplified base module, consisting of part of the deck plate with only one stiffener. Afterwards, several of these base modules can be linked together to form an entire deck plate. In [1] analytical calculation method is developed and refined, which is based on the application of the above mentioned method. Although it only models a two-dimensional cross-section of the orthotropic plated bridge deck, it also incorporates the influence of the functioning in two orthogonal directions of an orthotropic plated deck as a whole. A number of spring restraints are designed representing the longitudinal and transversal stiffness of the bridge deck as well as the torsion stiffness of the longitudinal stiffeners. This calculation method can be used to study the influence of the different geometric properties of the orthotropic plated bridge deck on its overall structural behaviour.

### 5.6. Numerical Methods for Designing Steel Bridges with Orthotropic Decks

According to [24] the publications of Guyon, Massonnet, and Cornelius were followed by numerous further contributions to the theory of orthotropic plates – the papers of Trenks, Mader, Giencke, Klöppel and Schardt to name but a few. Whereas in Huber's continuum the stiffeners must be positioned symmetrically about the central plane of the isotropic deck plate, Alf Pflüger [32] grouped together the isotropic deck plate and the "distributed" stiffeners eccentric to this into a continuum that has been named after him. Ernst Giencke also used the same theoretical basis for his work on the fundamental equations for orthotropic plates with eccentric stiffeners, which was published in 1955 [8]. Mader [27] and Giencke [9] dealt with the discontinuity of the cross girders and considered the orthotropic steel bridge deck as a composite system consisting of Huber's continuum and the discontinuous cross girders below. According to [24] in a later paper, Giencke analysed the hollow-rib plate, a variation of the orthotropic plate, whose success first came in the mid-1960s as the Krupp company took on a series of large bridges simultaneously and was forced to rely on large-scale production with maximum standardisation. At the same time, the steel industry switched the production of lightweight sheet piling sections from hot- to cold-rolling, which rendered possible the standardisation of deep trapezoidal profiles with transverse beams at spacings of up to 5 m. This technical progress led to the orthotropic bridge deck so typical these days: "Automatic welding and assembly plants for welding hollow ribs to deck plates rendered possible good-quality weld seams with good penetration for the typical solution. And the arrangement of the close-tolerance longitudinal rib penetrations through cut-outs in the cross girder webs, with adequate room for compensating for the tolerances of the trapezoidal profiles, plus the design of the longitudinal rib splices ensured details not susceptible to fatigue" [29]. The forerunner of the hollow-rib plate, perfected in the 1960s, had been produced many years before. Klöppel and Schardt achieved a graphic synthesis of the Huber [19] and Pflüger [32] continuum theory for anisotropic shell structures with the help of matrix calculations [23]. In 1960 Hans Schumann published his dissertation on the analysis of orthotropic rectangular plates [34], supervised by Pflüger. Schumann's theory, formulated in the language of matrix calculations, takes into account both the eccentricity and the discontinuous arrangement of the longitudinal and transverse stiffeners. In his summary he notes that the matrix formulation of his theory would simplify the programming of calculations for program-controlled automatic electronic calculators [35]. The consequential matrix formulation can be regarded as equally important because it considerably simplified the transformation into algorithms for computer programs. Giencke was the driving force behind this development. In 1967 he managed to formulate a finite method for calculating orthotropic plates and slabs [11]. Three years later, Giencke and J. Petersen published his finite method for calculating shear-flexible orthotropic plates [12], which at that time were being used mostly for building sandwich constructions.

## 6. Description of the Design Decision of Some Steel Bridges with Orthotropic Deck in Bulgaria

Bulgarian bridge construction enters a new phase of development as a result of a decision made by the expert council of the Ministry of Transport on 22 May 1968, which gives the green light for the use of steel structures in bridge construction in the country. This reasonable act gives the opportunity for several competitions to be announced for bridging (spanning) large obstacles, which in that period of development of reinforced concrete technology could not be realized. Some of these steel bridges were built as an orthotropic steel deck, whose design, scope and development will be the subject of this paper. The paper will cover seven steel bridges of orthotropic steel deck, arranged in chronological order, depending on the year of their construction and opening for traffic.

### 6.1. The “Elin Pelin” Bridge

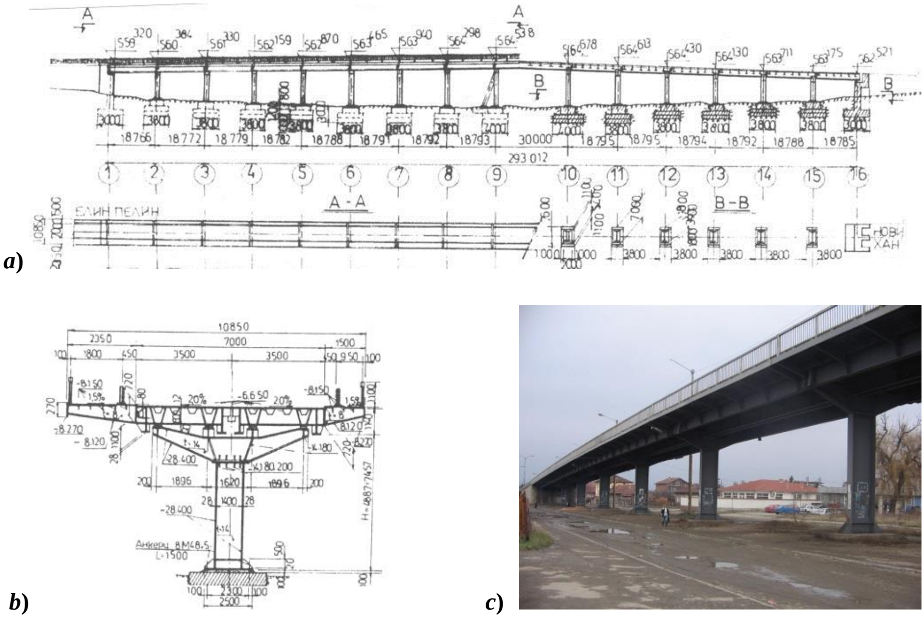
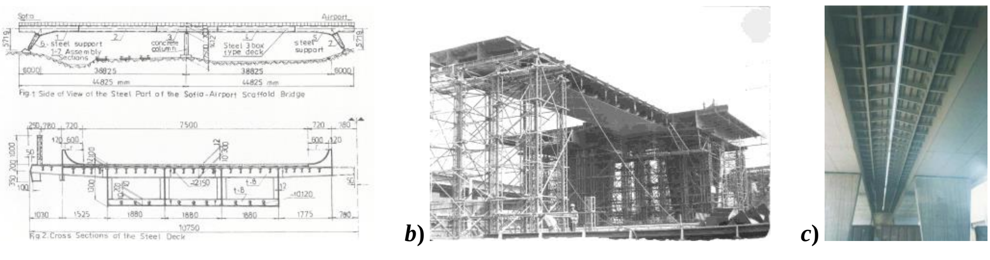



**Figure 5*a,b,c*. “ElinPelin” bridge: Longitudinal section-*a*), Cross-section-*b*), View on the orthotropic bridge-*c*)**

The “Elin Pelin” bridge is an overpass structure located in Elin Pelin – shown in Fig. 5*a,b,c*. It carries the traffic of a second class road 165 from Yordankino to Novi Han and crosses over the railway of the local railway station. The bridge was designed and built by “Gosha” company and supervised by B. Bankov. The bridge owner is the Municipality of Elin Pelin. The overpass was opened to traffic in 1981. The overpass is a steel orthotropic plate girder structure, Gerber system, with a total length of 300 m and includes 15 spans of different lengths. The largest span has a length of 30,00 m; the rest of the spans vary from 18,766 to 18,793 m. The superstructure is 7,00 m wide and consists of four main girders. The deck plate is 12 mm thick and stiffened with a longitudinal ribs and transversal beams. The longitudinal stiffeners are cold formed trapezoidal sections fabricated by a 6 mm thick metal sheet. The main girders of the central span are 1,10 m deep. The bottom flange has dimensions of 28×350 mm and is reinforced at the middle of the span by an additional 28×320 mm cover plate. The girders web is 10 mm thick. The rest of the spans have girders with same dimensions as above, except of the bottom flange whose dimensions are 20×350 mm. At 400 mm away from the bridge piers there are 100 mm diameter openings for the hinges of the Gerber system. The transversal beams with overhangs are 2,85 m long, 600 mm deep and are spaced at 3,00 m. The flanges have dimensions of 10×100 mm. The overhangs have a varying depth. The road part of the deck in an each span consists of two symmetrical assembly units each 3,00 m wide. They are connected to each other by 24 mm diameter high strength bolts. Two pedestrian lanes (2,35 and 1,50 m wide) are attached to the road parts. Each pedestrian lane is designed as an orthotropic slab of 10 mm thick top plate, and stiffeners with cross-section of 6×150 mm were used – spaced at 435 mm. The pedestrian units are connected to the road units by bolts. The piers are steel single-column hammerhead bents. The columns for the part of the bridge with spans of 18,766 m are designed as steel “I”-sections. The columns for the span of 30,00 m are designed as steel box-sections. The steel base plates of the columns have dimensions of 20×1600×2600 mm for the box-section columns and 20×740×2500 mm for the “I”-section columns. 18 anchor bolts BM 36 are used to connect the base plates to the footings. The cap beam is a double-armed cantilever with a varying depth which is connected to the column by bolts. The transversal beams and the stiffeners of the orthotropic deck are welded to the steel plate. The orthotropic deck units and secondary beams are made from steel S235-JR(St37-2) and the main girders and piers are made from steel S355J2G3 (St52-3).

### 6.2. The “Brussels boulevard” Steel Bridge

A steel bridge structure is a part of the “Brussels boulevard” overpass and serves as a connection between the Sofia Airport and the entrance highway of the city (Fig. 6*a*,*b*,*c*). The bridge is placed over the railway Sofia-Istanbul. The designer is B. Bankov, and the Municipality of Sofia is the owner. The overpass was opened to traffic in 1983. The overpass consists of two parallel structures separated with a gap of 1,56 m. The superstructure is a steel orthotropic 3-cell 90,00 m long box girder and includes two spans of 45,00 m. The bridge is placed in a horizontal curve with a radius of 1000,00 m. The box girder is 1,30 m deep and the total width is 21,75 m. These dimensions were required and limited by aesthetic considerations because the steel part had to fully follow the silhouette of the concrete part with the same depth. In longitudinal direction, the box walls are connected to each other by upper and lower transversal steel beams at intervals of 1,493 m. The upper transversal steel beams are connected at the top by a 12 mm thick steel plate. The lower transversal beams are connected at the bottom by a 10 mm thick steel plate. The steel orthotropic deck plate is stiffened with longitudinal flat ribs: 12×150 mm with a distance between them of 313 mm. The upper transversal beams have “T” cross-sections with web dimensions of 8×300 mm and flange dimensions of 10×120 mm. The lower beams have “T” cross-sections with web dimensions of 8×200 mm and flange dimensions of 10×100 mm. The box walls are 12 mm thick and stiffened by vertical ribs of 10×100 mm spaced at 1,493 m. The lateral bracings are placed at intervals of 7,465 m and at supports regions. The bridge end piers are “V”-shaped bents with inclined columns. The columns are triple-box cross-sections with wall thickness of 12 mm. The columns are rigidly connected to the deck and have hinged connections to the footings. The central pier is a concrete hammerhead bent.

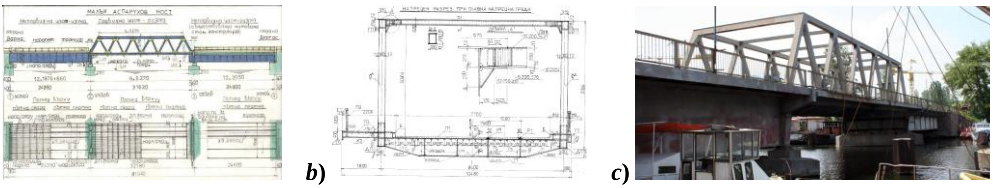



*a*) *b*) *c*)

**Figure 6*a*,*b*,*c*. “Brussels“ Blvd. steel bridge: Longitudinal and cross-section-*a*), While erecting the bridge-*b*), View of the orthotropic bridge-*c*)**

### 6.3. The “Varna Lake” Movable Bridge

The “Varna Lake” movable bridge is a steel structure near Varna (Fig. 7*a,b,c*). The bridge carries the railway and road traffic of the industrial part of Varna over the ship canal which connects Lake Varna to the Black Sea. The bridge was designed and built by the “MAN” company. The bridge owner is the Municipality of Varna. It was opened to traffic in 1939. The bridge is a 3-span riveted steel structure with a total length of 80,34 m and consists of two stationary parts with spans of 24,36 m and a movable central span of 31,62 m. In 1975 the movable part of the bridge was completely destroyed by a ship accident and the stationary parts were seriously damaged. The new “Varna Lake” movable bridge was designed by B. Bankov. At Varna side, the stationary part was designed as a simply supported bridge with an orthotropic deck. The superstructure consists of two main girders which are 2,80 m deep with a distance between them of 8,00 m. The deck slab is 12 mm thick and 13,86 m wide. The girder has bottom flanges with dimensions of 12×400 mm and a 12 mm thick web. The floor beams are 1,55 m deep with 1,975 m distance between them, and there are bottom flanges with dimensions of 20×300 mm and an 8-mm thick web (lattice). The centreline of the railroad has an offset from the centreline of the bridge by 1,55 m, and it is supported by two longitudinal girders spaced at 1,50 m, with a span of 1,975 m and depth equal to the depth of the secondary beams. Their web (lattice) has dimension of 8×540 mm. The longitudinal girders also serve as deck stiffeners, and are designed as continuous beams supported by the secondary beams (at 1,975 m centers). The lateral bracings of the deck are “K”-braces placed at bottom flanges of the main girders. The central movable part is a trough truss structure which consists of two identical Warren trusses with spans of 31,62 m which include 6 panels of 3,162 m long. The members of the upper, the lower chords and compression diagonals have box-like cross-sections; the tension diagonals have “I”-shaped cross-sections. The truss deck is also orthotropic plate structure and consists of bottom chord members and a 12 mm thick steel plate. The deck is stiffened by longitudinal ribs of 10×160 mm spaced at 300 mm. The floor beams are placed at the lower joints of the trusses and have a span of 8,40 m. The distances between the stringers are 1,757 m. The portal bracing of the trusses is formed by box-sections members placed at upper joints of the trusses. At Asparuhovo, the stationary part remains as the original riveted structure was. The old bridge piers are used for a new movable part. The bearings are elastomeric bearing pads. Two type steel grades were used for steel bridge: steel grade M16C and steel grade 10G2CF.



***a*)** ***b*)** ***c*)**

**Figure 7*a,b,c*. “Varna Lake” movable bridge: Longitudinal section-*a*), Cross-section-*b*), View of the orthotropic bridge-*c*)**

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