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A NEW LIGHTWEIGHT STEEL BRIDGE FOUNDED IN PEAT
Optimal Design and Soil Improvement

Chrysanthos Maraveas[1,2]

[1] C. Maraveas & Associates P.C. – Consulting Engineers, Greece
[2] University of Liege, Structural Engineering Section, ArGenCo Dept., Belgium
e-mail: c.maraveas@maraveas.gr

ABSTRACT: This paper presents the planned design for a new steel road bridge in the Tenagi valley, Kavala, Greece. The plans are for a single span steel truss trough bridge with a span of 67 m over an irrigation channel. The new bridge will replace an existing reinforced concrete one that is no longer in service due to excessive rotation of its single pier and deck failure. The major challenge associated with the design of the new bridge is the poor soil characteristics in the region. The soil is composed of peat to a depth of over 200 m. Optimization of the type, shape, and size of the bridge superstructure is critical in order to minimize soil intervention. In this article, the effects of several types of deck (a reinforced concrete deck, a fiber reinforced polymer deck, and a steel deck are considered) on the weight of the steel truss are examined. Shape optimization of the truss is conducted with the truss height as a variable. Beyond minimizing the weight of the bridge, soil improvement techniques such as deep soil mixing and the preloading of embankments must also be implemented to minimize settlement and increase the bearing capacity of the soil.

KEYWORDS: Steel Truss-Through Bridge, Peat, Sand Drains, Soil Improvement, Soil Deep Mixing.

1 INTRODUCTION
This study began with a thorough examination of the available data on conditions in the region, to help with the development of an optimal bridge design. The bridge will be in the Tenagi valley in Kavala, northern Greece. In the past, the valley was occupied by the Pravious lake, which was manually dried during the decade 1930-1940. It was then that the main irrigation channel was constructed. The channel has a length of 29 km and a width of around 30-50 m. The depth of the channel is almost 2 m. This channel divides the region into two parts. Many bridges have been constructed over the years to facilitate the crossing of vehicles over the channel. The new bridge will replace the

existing one, which is no longer in use. According to general geology maps, the soil of the region is composed of peat for a depth of over 200 m. These poor foundation conditions, combined with an inappropriate type of bridge, were the main reasons for the failure of the existing bridge. A thorough study of possible lightweight superstructure designs and soil improvement techniques was therefore required.

2 PRELIMINARY – OPTIMAL DESIGN

2.1 Site investigations and geotechnical profile

A geotechnical investigation was conducted to determine the detailed soil profile of the area of the foundation of each abutment. Two boreholes were drilled up to a depth of 23 m and standard penetration tests (SPTs) were carried out every 2 m, with blow counts varying from 0-5. Additionally, four cone-penetrometer tests (CPTs) were carried out from depths of 23 m to 30 m. The measured values of the cone resistance, q_c, were less than 0.5 MPa. Samples of soil were also taken for laboratory testing.

According to the site investigation and laboratory test results, the soil was identified as peat with very poor geotechnical characteristics, which are summarized in Table 1.

Table 1. Estimated geotechnical parameters for the existing soil profile [1].

Depth	γ (kN/m^3)	Su (kPa)	c’ (kPa)	φ’(^{o})	Es (MPa)
0-10 m	11	4	0.5	10	0.25
>10 m	11	10	2.5	10	2.0

2.2 General layout

As the soil in the region of the bridge is peat with poor foundational characteristics, the deck of the existing bridge, which was simply supported, failed due to excessive rotation of the pier (Figure 1a,b). The site investigation confirmed that soil conditions were poor, and it was therefore concluded that a single span lightweight superstructure should be designed. The bridge should be simply supported through special bearings on the piers to accommodate settlement due to the poor foundation characteristics.

The appropriate general layout of the new bridge was determined to be a half-through truss steel bridge with a parabolic top chord. More specifically, the design consists of two Pratt trusses connected transversally via cross girders and horizontal bracing (Figures 2-3). The deck type and the sizing of the truss members were determined from an optimization process.

(a) *(b)*

Figure 1. (a) The existing bridge, and (b) the location of the project

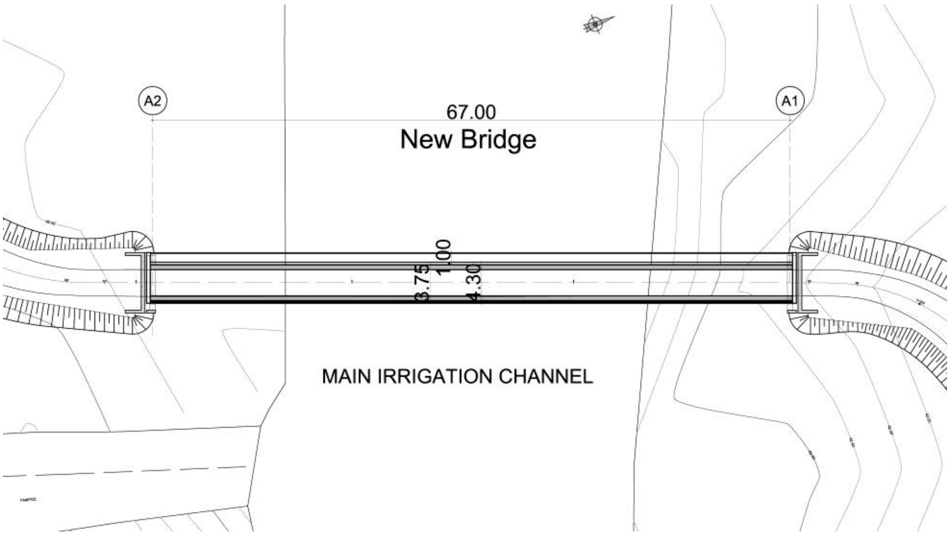


Figure 2. A plan view of the new bridge

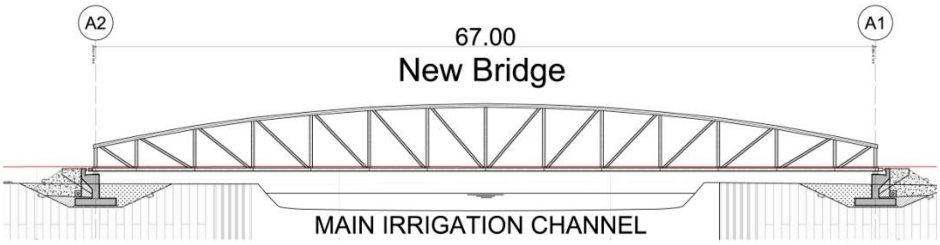
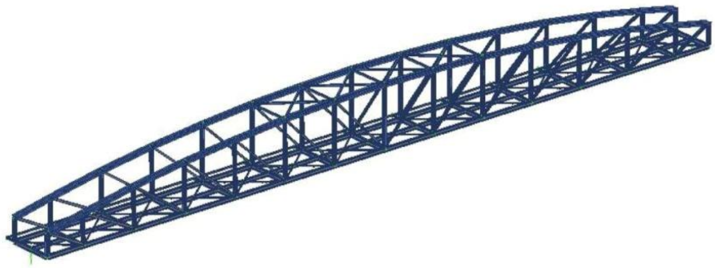


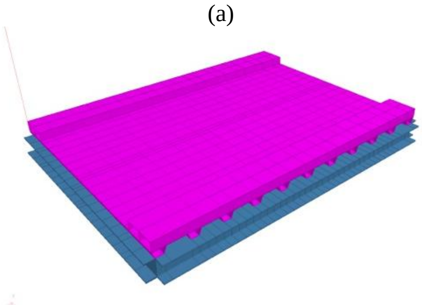
Figure 3. The elevation profile of the new bridge

2.3 Design assumptions

An exhaustive series of analyses was performed during the design process. The bridge was simulated as a spatial frame using the commercial program RM Bridge 2000 (Bentley) [2] which performs 4D time-dependent analysis of the entire construction progress (Figure 4a) and commercial software STAAD.Pro V8i [3] for the deck (Figure 4b), which implements the Finite Element Method (FEM) using surface elements for optimization.



(a)



(b)

Figure 4. (a) A 3D RM Model of the truss-through structure, and (b) an STAAD model of the metal deck

The design followed Eurocode regulations and the necessary checks were performed. During the working life of the bridge, actions arising from superimposed permanent loads (asphalt layers, sidewalks, etc.), thermal actions per EN1991-1-5 [4], and wind actions per EN1991-1-4 [5] were taken into account in addition to the traffic loads. A relevant summary is given in Table 2.
The design of the steel members, which complies with the requirements of EN1993-1-1 [6] and EN1993-2 [7] for steel bridges, was also obtained using the same software [2,3]. The selected quality for the steel was S460. Additional routines were implemented in Matlab to cover the checks specified in EN1993-2 [7] regarding buckling of compressed members of the top chord. These routines calculate the top chord resistance according to the geometry, configuration of the semi-frame, member sections, etc., and help to perform the necessary adequacy checks for the axial force of the design.

Table 2. Design loads of the bridge

Permanent loads	
Self-Weight Steel	78.5 kN/m^3
Self-Weight Concrete	25 kN/m^3
Asphalt layers	24 kN/m^3
Live loads	
Traffic Load	LM1 (aQ_{1k} Q_{1k}=150 kN, aq_{1k} q_{1k}=5.0 kN/m^2.)
Thermal Action	T_{max} = +45 °C, T_{min} = -20 °C
Wind	V_{bo}=27 km/h, Terrain I
Earthquake	
Ground Acceleration	0.16 g
Importance Factor	1.00
Soil Category	D

2.4 Optimal design variables

The aim of the optimization process was to minimize the objective function, which in this case is the weight of the superstructure. The effects of three different deck types were considered. The selected deck would lie above the main bottom transversal beams that connect the two trusses at the bottom chord. They are simply supported through these beams and thus the deck has the general form of a two way simply supported continuous slab (Figure 5).

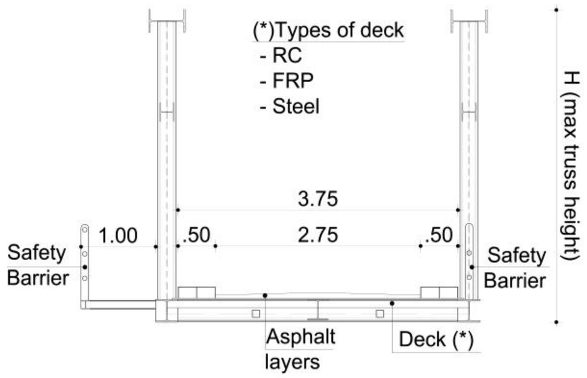


Figure 5. A typical section of the bridge indicating the design variables

The first possibility studied was a reinforced concrete (RC) deck with a 5 kN/m^2 self-weight, the second a fiber reinforced polymer (FRP) deck weighing 1.5 kN/m^2, and the third a steel deck with a weight of 2.5 kN/m^2. Shape optimization of the truss was conducted by selecting its height at midspan as the design variable. The cross-sectional areas of the members constituted the sizing optimization design variables. European standard hot-rolled sections were selected for the steel members in the following configuration: a) top chord members – HEA or HEB, b) bottom chord members – HEA, c) diagonal members – HEA, d) vertical members – HEB, e) cross girders – HEB, f) stringers – HEA, g) bottom lateral bracing members – IPE. For each cycle of the shape optimization process, the member sections were iteratively selected to obtain the lowest possible weight for the truss, with the structure still conforming to all design requirements [8].

2.5 Optimal design results

The results of the optimization analyses are summarized in the following charts. In the first (Figure 6), we can see the relationship between a single truss weight and its height at the center of the span. Each point in the diagram is obtained from the sizing optimization process for the specific height depicted. Similarly, the results for the relationship of the total bridge weight and the truss height at the center are presented in Figure 7.

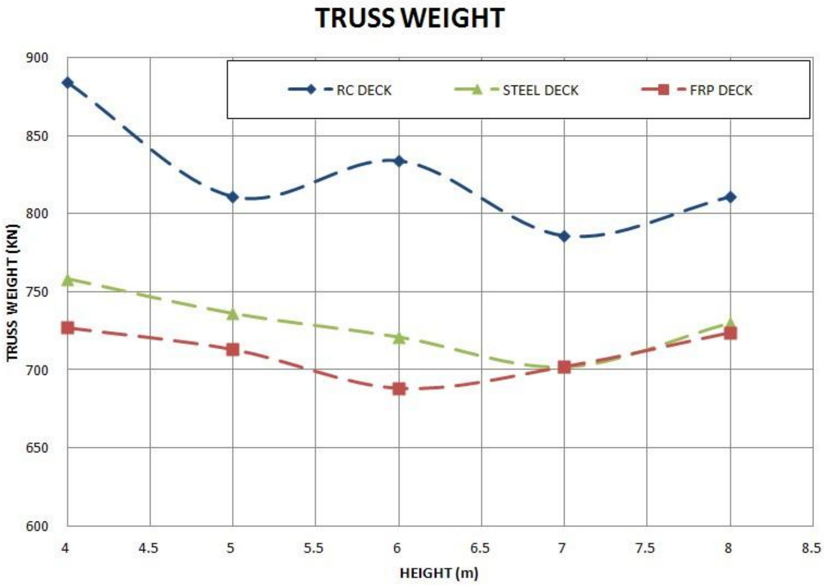


Figure 6. The single steel truss weight associated with each height at the center of the span.

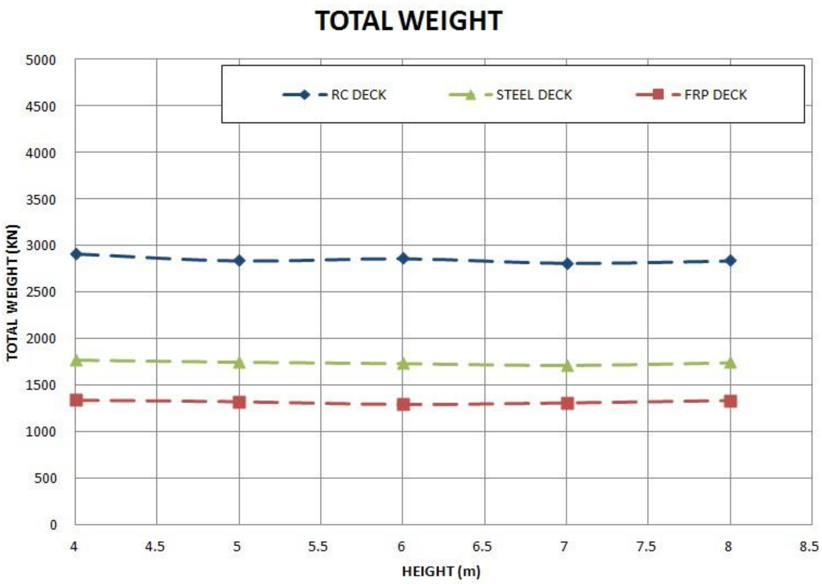


Figure 7. The total bridge weight for each truss height.

From the above plots we can see clearly that the lightest and most economical solution for the truss height is in the range of 5 to 6 meters at the center of the span.

Following the optimization process, the next step was to examine how the different decking affects the economy of the structure. Thus, assuming that the height of the truss at the center of the bridge is 5.50 m, the weight of a single truss and the weight of the whole bridge was calculated in each case. Then the normalized values of the weight with respect to that of the reinforced concrete deck were calculated (Figure 8). From the figure we can see that using the fiber reinforced polymer deck results in the lightest structure, using the steel deck results in a slightly heavier structure, with about a 3 % difference in the single truss weight and a 16% difference in the total weight. The use of a reinforced concrete deck results in a heavier solution with a 55 % difference over the FRP deck.

The last step of the optimization process was to evaluate the cost of each type of decking. For this cost analysis of the bridge, applicable current rates from suppliers and agents were used, including the cost of manufacturing, delivery, installation, and overlay.

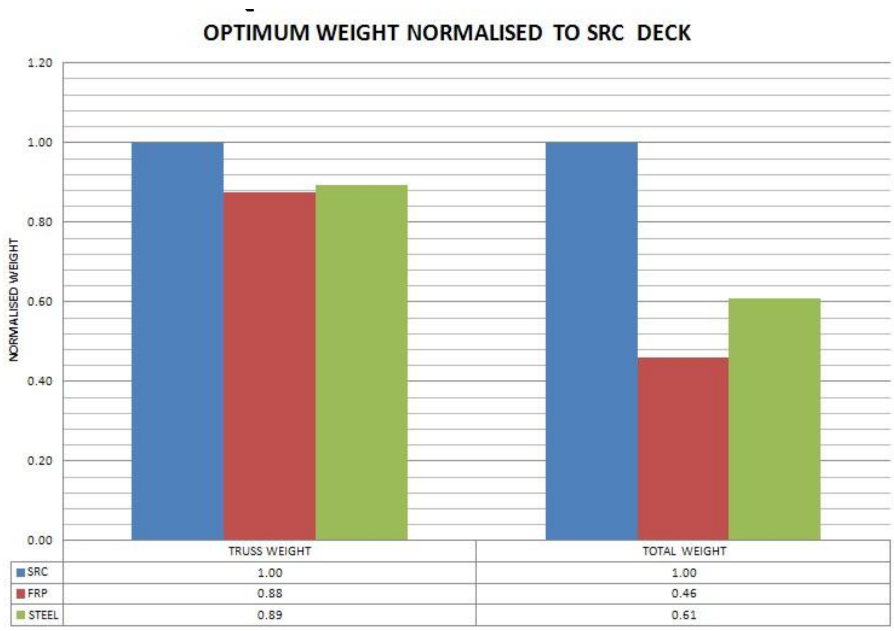


Figure 8. The effect of the type of deck on the weight of the truss and the bridge, normalized to the values for the RC deck.

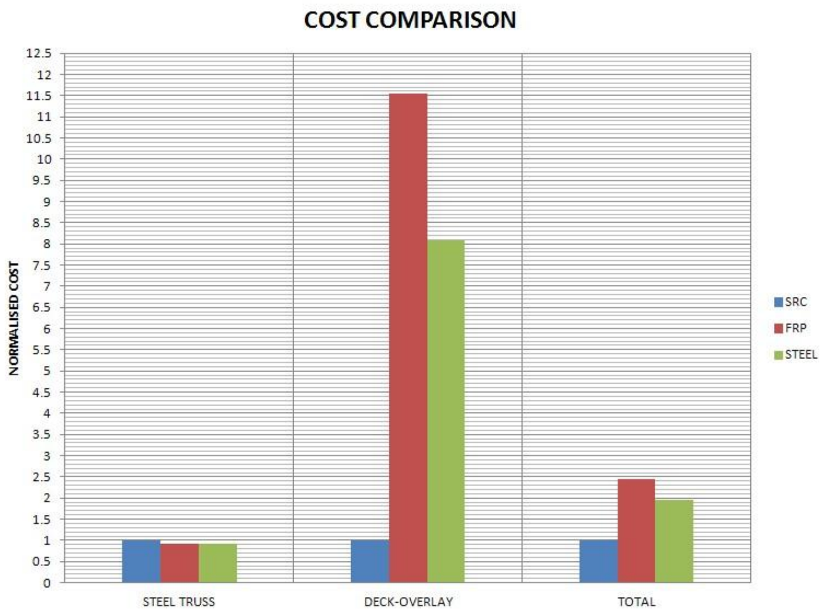


Figure 9. The cost of the steel truss, deck, and the total bridge for each type of deck, normalized to the values for the RC deck.

The results of the economic analysis are presented in the three panels of Figure 9, normalized to the cost of the reinforced concrete deck. The first panel shows the construction costs of the steel truss, the second the construction costs of the decking, and the third shows the total cost for the construction of the bridge. According to this analysis, a bridge with a 5.5 m maximum truss height and a steel deck plate with ribs proved to be the best solution techno-economically.

3 SOIL IMPROVEMENT - FOUNDATION DESIGN

3.1 Improvement techniques adopted

Although the superstructure was designed to have an optimal weight, the preliminary foundation study showed that a shallow foundation could not be built on the existing soil. The bearing capacity of the soil was exceeded and excessive settlement would therefore be expected. The possibility of a foundation on piles was rejected because soil with similar pour characteristics extends to depths grater that 200 m according to the geological study of the region. Therefore, a soil improvement plan was necessary. More specifically, the following techniques were adopted:

- Vibratory soil replacement
- Preloading of embankments
- Soil deep mixing

The main objective of these measures is to increase the bearing capacity of the soil by increasing the theoretical internal friction angle φ′, the unit weight γ, and the constrained elastic modulus Es of the soil. In the first stage, sand drain grids were constructed in two phases. Initially, 20 m deep sand drains with diameters of 80 cm were constructed in a triangular grid of 1.40 m spacing over an area of 20.60 m x 20.40 m. Secondly, 3.5 m sand drains with diameters of 80 cm were constructed within the first stage drains, overlapping them for the first 3.5 m (Figure 10). The filling material of the drains was well graded according to Greek Technical Specification 1501-11-03-03-00:2009 [10]. The improved soil parameters are summarized in Table 3.

Table 3. Geotechnical parameters for the improved soil profile [1].

Depth	γ (kN/m^3)	c’ (kPa)	φ’(°)	Es (MPa)
<3.5 m	19	0	36.5	17
3.5-20 m	13.5	0	19	6

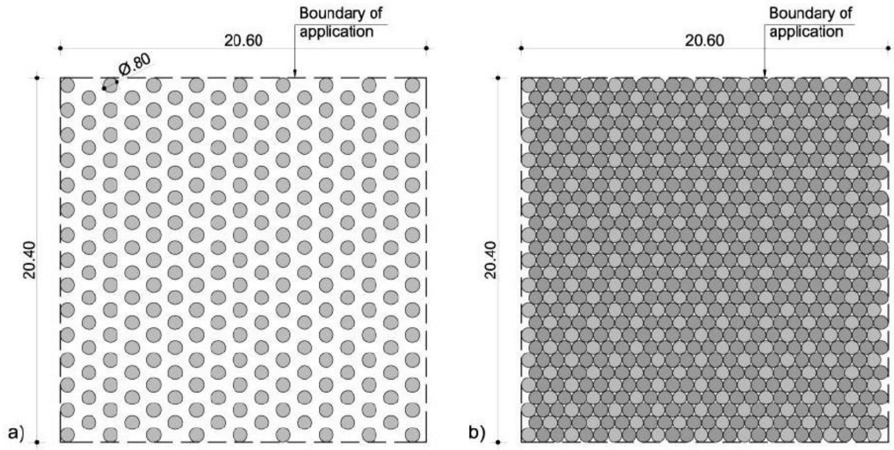


Figure 10. The sand drain arrangement: (a) 1st phase, (b) 2nd phase.

Despite these measures, the calculated settlement of the improved soil was still excessive at almost 7 cm. To eliminate such settlement during the working life of the bridge, preloading the embankment at each pier location was proposed. Each embankment will have a height of 2.2 m and top plan dimensions 10 m x 3.5 m. They will consist of a 0.80 m bottom sand layer for drainage purposes and a compacted top clay layer. The embankments are estimated to remain in place for at least six months.

3.2 Foundation of the bridge – Soil deep mixing

After all the above foundation bearing capacity improvements, it was decided that the bridge will be founded on two spread footings with dimensions of 10 m x 3.5 m in plan view and depths of 2.0 m according to the regulations of EN1997-1 [11].

Besides the aforementioned checks, the slope stability of the spread footings was also checked with the help of the geotechnical engineering program LARIX-5 [12]. This analysis was performed for both the preloading embankment phase and for the working life of the bridge. According to the analyses, failure was possible both in the preloading phase and during the working life of the bridge. It was necessary to increase the shear strength in regions surrounding the sand drains because of possible failures due to the proximity of the foundation to the irrigation channel. The layout of the region of soil for which improvement techniques were adopted defined the final span of the bridge, a total of 67 m.

A soil deep mixing technique was adopted. More specifically, cement columns of depth 30 m and diameter 80 cm were arranged symmetrically in sets of four 3.20 m wide regions surrounding the sand drains (Figure 11). The final critical sliding surface of the foundation soil during the working life of the bridge is presented in Figure 12. The minimum calculated safety factor was approximately 1.20.

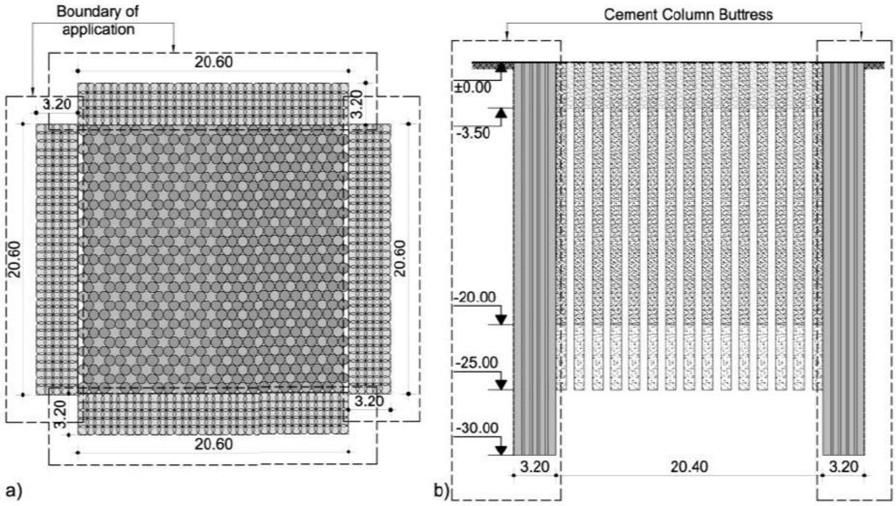


Figure 11. Soil deep mixing piles: (a) plan view, and (b) elevation

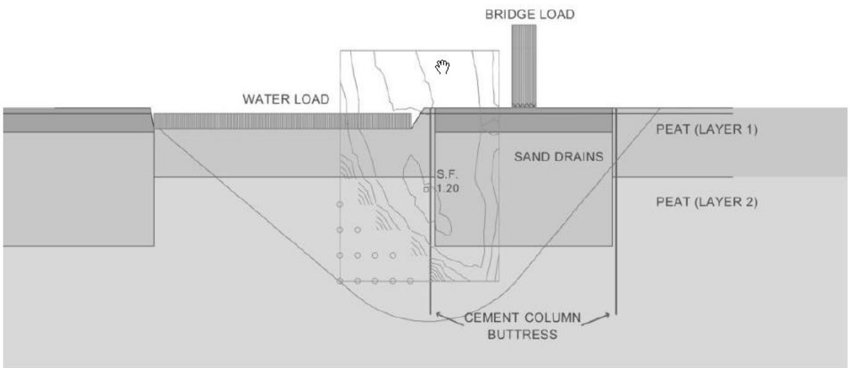


Figure 12. The critical sliding surface for slope stability during working life of the bridge

4 DESIGN OF THE SUPERSTRUCTURE

4.1 Bridge general arrangement final design

Following the aforementioned analysis, the bridge was designed as a 67 m single span half-through Pratt truss. The bridge is simply supported with elastomeric bearings in each abutment. Each abutment has a rectangular spread footing in plan view of 10 m x 3.5 m according to the geotechnical design (Figure 13).

The main trusses are formed with HEB, HEA, and SHS steel cross sections and the top chord of the main truss is parabolic with a height of 2.0-5.50 m

(Figures 14-15).
The total width of the bridge is 5.30 m with 2.75 m width lanes, a 1 m kerb, and a 1 m pedestrian path (Figures 16-17).

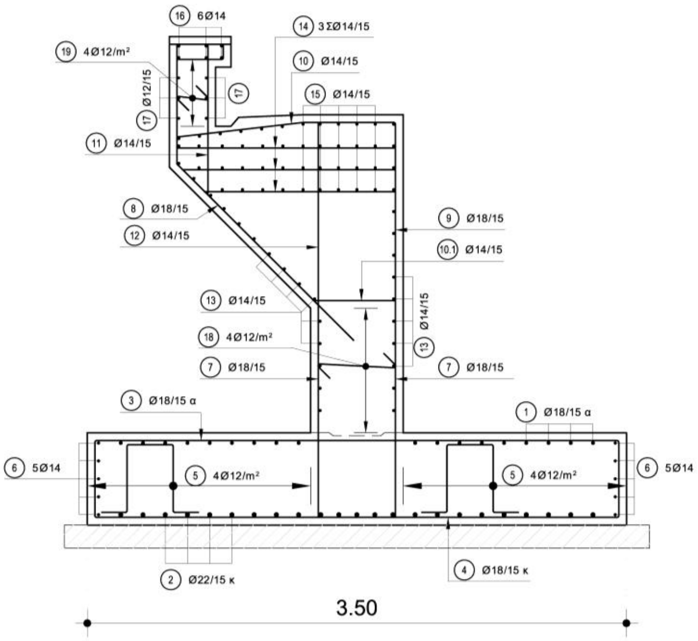


Figure 13. A typical section of the abutment

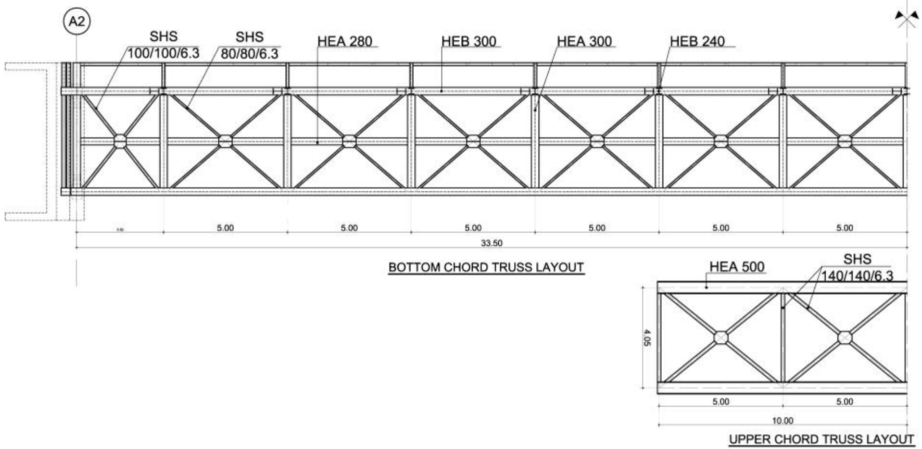


Figure 14. The general arrangement of the bridge (plan view)

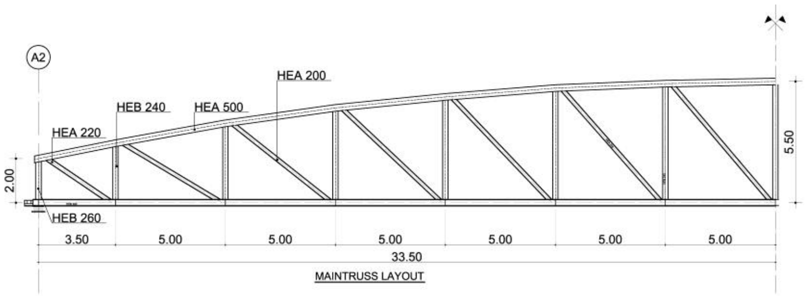


Figure 15. The general arrangement of the bridge main truss (elevation)

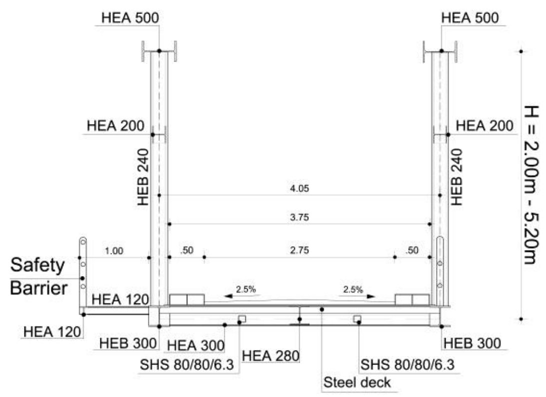


Figure 16. A typical cross section of the bridge

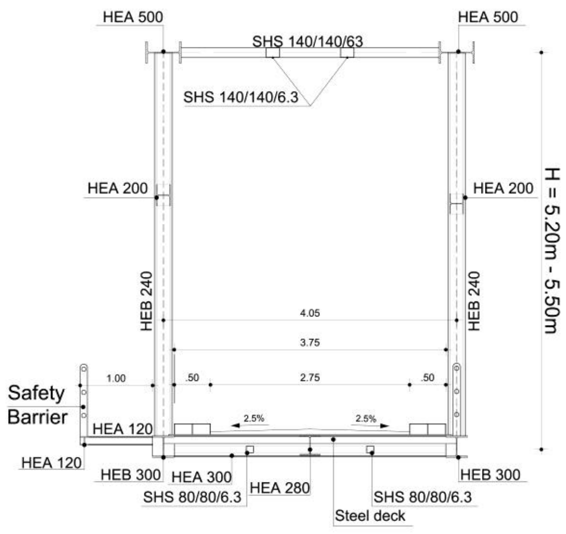


Figure 17. A cross section of the bridge at midspan

In the transverse direction, the main trusses are connected with semi-frames every 5.0 m at the bottom chord of the trusses. The semi-frames are designed to be stiff enough to resist the buckling of the top chord. In the bottom and part of the top chord of the bridge, the diaphragm is formed with SHS steel bracings.
All the connections of the bridge are prestressed with a grade of 10.9. The major connection of the semi-frame is moment-resisting and is presented in Figure 18.

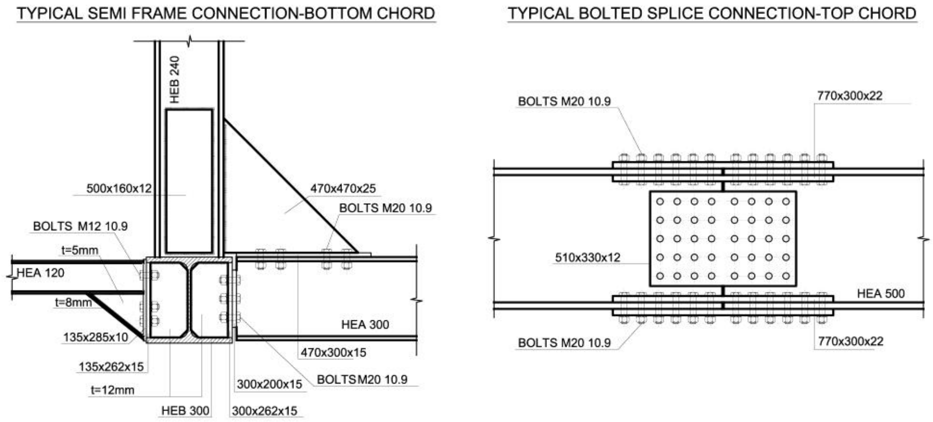


Figure 18. The typical connections of the semi-frames and the top chord splice

The deck of the bridge (Figure 19) was selected to be a steel deck plate of 16 mm with ribs because this was the best solution techno-economically. The steel deck is supported on the main truss bottom chord beams and a longitudinal bottom chord internal beam via welding.

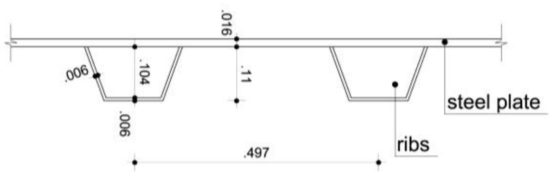


Figure 19. The cross-section of the steel deck (dimensions in m)

4.2 Analyses and design checks

The results of the optimization analysis and the final design were extracted using the Eurocodes group of codes. Static and dynamic analysis were performed, and as already mentioned, all member checks were performed using commercial programs [2,3].

All the checks were performed in terms of forces, stresses, and deformations. Members were also designed to resist the maximum variation of stresses possible due to fatigue traffic loads.

The axial forces on the top and bottom chords of the main truss members are presented in Figures 20-21 for each design load case and combination.

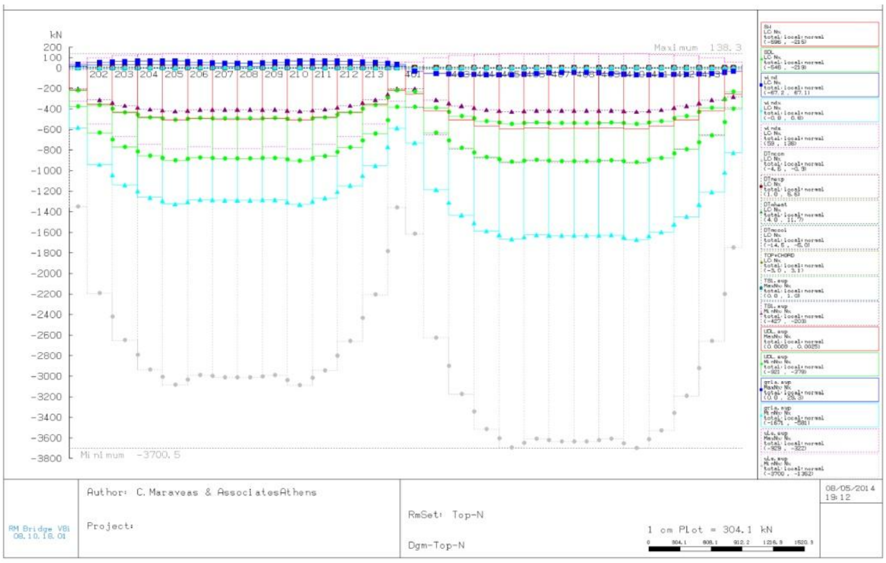


Figure 20. Axial forces on the top chord of the main truss

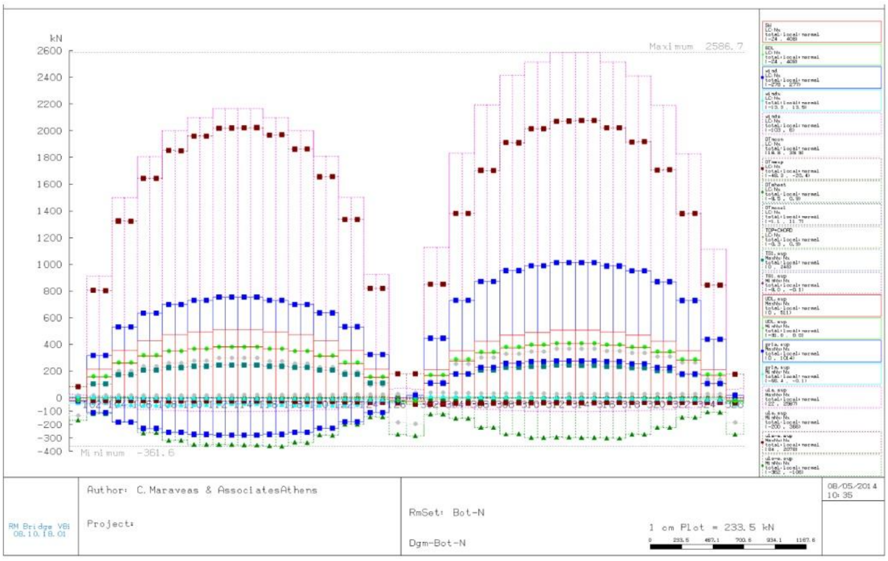


Figure 21. Axial forces on the bottom chord of the main truss

Additional checks with in-house spreadsheets and Matlab were performed for the buckling of the top chord and for the steel deck.

More specifically, the lateral stiffness of the semi-frames of the bridge was calculated according to EN1993-2 [7] Annex D case 1a as in Eq. (1). The buckling check of the top chord was then performed taking into consideration the elastic lateral stiffness as per Eq. (2), which affects the buckling resistance of the top chord.

$$C = \frac{EI_v}{\frac{h_v^3}{3} + \frac{h^2 b_q I_v}{2I_q}} \quad (1)$$

$$c = C_d / I \quad (2)$$

The design checks of the steel deck plate with ribs was performed in terms of stresses according to EN1993-1-5 [9] as in Eqs. (3) and (4).

$$\left(\frac{\sigma_{x,Ed}}{f_y/\gamma_{M1}}\right)^2 + \left(\frac{\sigma_{z,Ed}}{f_y/\gamma_{M1}}\right)^2 - \left(\frac{\sigma_{x,Ed}}{f_y/\gamma_{M1}}\right)\cdot\left(\frac{\sigma_{z,Ed}}{f_y/\gamma_{M1}}\right) + 3\left(\frac{\tau_{Ed}}{f_y/\gamma_{M1}}\right)^2 \leq \rho^2 \quad (3)$$

$$\frac{\rho \cdot \alpha_{ult,k}}{\gamma_{M1}} \geq 1 \quad (4)$$

Were $\rho = \min\{\rho_x, \rho_z, \chi_w\}$

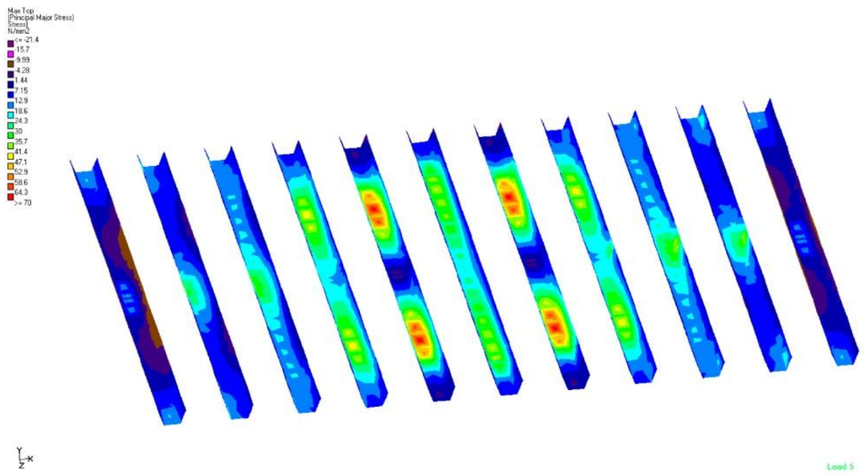


Figure 22. Stresses at the ribs of the steel deck for traffic loads

4.3 Construction Sequence

The bridge construction sequence was defined as the following steps:

1. Soil improvement according to the geotechnical design.
2. Preloading of soil and monitoring of settlement.

3. Industrial construction of the parts of the main trusses. Each truss will be constructed in five parts with lengths of 15 m, 12 m, 13 m, 12 m, and 15 m, respectively.
4. Transportation of the main truss parts on site.
5. Erection and splicing of the main truss parts.
6. Connection of the main trusses transversally with a semi-frames and bracings arrangement.
7. Construction and placement of the steel decking.
8. Construction of asphalt layers and the pedestrian sidewalk.

5 COST DATA

The estimated construction cost for the bridge is €12,608,936.50 at current contractor rates. A summary of the basic quantities of the materials is shown in Table 4.

Table 4. Basic Quantities of the Bridge and Foundation Materials

Sand drain piles Ø80cm	16,300 m
Cement piles Ø80cm	41,550 m
Preloading embankment	302 m^3
Concrete C30/37	75 m^3
Reinforcement Steel B500C	10,000 kg
Elastomeric bearings	4
Structural Steel S460	126,500 kg
Anticorrosion Paintings (each layer)	65,000 m^2

6 CONCLUSIONS

The new bridge presented in this paper will be located in the Tenagi valley in Kavala, northern Greece. In the past, the valley was occupied by the lake of Pravious, which was manually dried during the decade 1930-1940, and a system of irrigation channels was then constructed. The main irrigation channel separates the region into two parts. The new bridge crosses the main irrigation channel of the valley and will replace the existing bridge, which has failed due to excessive deformation of the ground. The soil of the region is composed of peat for a depth of more than 200 m. These poor foundational characteristics mandated the thorough study provided in this paper of all possibilities for a lightweight superstructure combined with soil improvement techniques. More specifically, according to the optimal design analysis that was conducted, the most appropriate type of bridge techno-economically proved to be a steel half-through truss bridge with a parabolic top chord and a steel deck plate with ribs. The length of the bridge is 67 m with a maximum height of the truss at mid-span of 5.5 m. Additional soil improvement techniques such as vibratory soil replacement, soil deep mixing, and the preloading of embankments were also adopted in order to increase the soil bearing capacity and minimize settlement.

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