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HEKURUDHA SHQIPTARE SH.A

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DETAILED DESIGN OF THE SECTION
TIRANA PUBLIC TRANSPORT TERMINAL - TIRANA CENTRAL CITY STATION
(EBRD TECHNICAL COOPERATION FUNDS)

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FINAL DESIGN
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TECHNICAL REPORT RAILWAY BRIDGE CH.34+425
RAPORT TEKNIK I URES HEKURUDHORE KM.34+425

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Table of Contents

Page

List of Figures

List of Tables

1. Introduction

This technical report has been drafted in the context of the **design** of the Railway overpass in intersection with Tirana Ring Road spanning between Ch.34+389.90 and Ch. 34+460.90 of the Railway Line, connecting PTT Station with Tirana New Boulevard.

1.1 Available Data

- "Study of Engineering Seismic Hazard Potential - Technical Report", ALTEA & Geostudio 2000, Tirana, February 2022.
- Plan layout and cross sections from "Studim Projektim Rishikim i Projektit per Segmentin Rrugor te Vazhdimit te Unazes se Madhe te Tiranes nga Sheshi Shqiponja - Bulevardi I Ri", ATELIER4 shpk & SEED CONSULTING shpk, Tirana, July 2021.
- Superstructure Design (Hor & Vert Alignment), Tirana, September 2022.
- Railway Design (Open Line Plan Layout & Long Profile), Tirana, September 2022.
- Railway Design (Progressive Cross Sections), Tirana, September 2022.
- Report for Geological and Geotechnical conditions of the area where is located Bridge at Ch.34+425, ALTEA & Geostudio 2000, Tirana, February 2022.

1.2 General

Railway Overpass bridge is located at Ch. 34+425 km. At that area the horizontal alignment crosses Tirana Ring Road. The bridge is spanning over a total length of 71.00m (measured between abutments' axis) and the longitudinal inclination is complying to the alignment to reach the minimum ballast thickness 40cm while the horizontal alignment is straight.

2. Bridge description

2.1 Typical Cross Section

The typical cross section is complying with the TSI and it has been applied in other railways sections in Albanian.

The main advantages of the proposed configuration are that footpaths with adequate dimensions are foreseen at both sides of the track, while there are provisions for system space (e.g. cable ducts) under the pathway. Moreover, for safety reasons, railings will be attached at both sides of the vertical edge beams without adding –in this way- extra width to the deck with their anchorage system.

The total width of the deck is 6.00m minimum, comprising of 4.50m ballasted track and two footpaths 0.75m, each. Footpath/cable ducts may be cast together or in a later phase than the deck slab, but in any case, it is proposed that edge beams 25cm wide are formed at both sides, covering the deck slab profile. In this way the standard structure width adopted in the railway line is 5.5m, giving a rather compact cross-section without compromising in safety or functionality. Consequently, the bridge dimensions are reduced and as a result, the total weight and cost of the structure is kept low. See figure below:

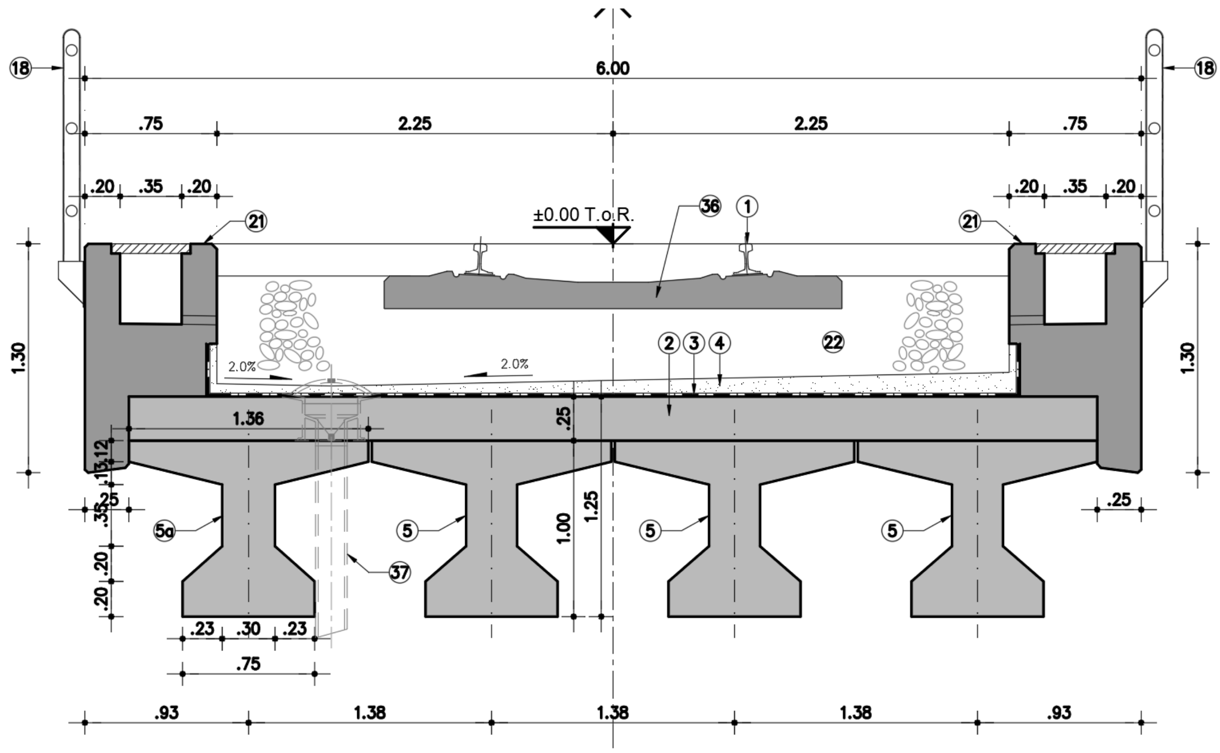


Figure 2- 1 Typical cross section of Superstructure

2.2 Superstructure

The statical system of the bridge is that of four consecutives, simply supported decks on piers and on abutments. The length of each span, measured between support axes is 15.95m, while the spans' lengths between expansion joints at piers and at abutments are 17.15m – 18.35m – 18.35m – 17.15m, with a total bridge length including abutments back walls/transition slab equal to 83.20m. the edge of the superstructure is skewed with 10degrees along the normal bridge axis.

Each deck is supported by four precast - prestressed concrete girders 17.55m long. The section is classic T with 1.00m height. The upper flange width is 1.36m and the bottom rib width is enlaged from 30cm up to 75cm. The spacing between the girders is 1.38m.

The girders are connected to each other, via a rectangular cross-beam at both ends, along the support/bearing axis. The dimensions of the cross beam are 1.15m height (including the slab) and 0.60m width.

The thickness of the deck slab is constant and equal to 0.25m, while over the cross beams, where a 10cm recess is foreseen at the top of the girders, the thickness is increased to 0.35m. The total deck slab width is 5.50m spanning over the four beam flanges.

2.3 Bearings' scheme

Precast girders are seated on pot fully fixed bearings at one end of each span and on pot, longitudinal sliding bearings at the other end. The later are restrained along the transversal direction.

The arrangement of the bearings is presented in the respective drawing page "951-01-08-DR-DW-BRG-05".

2.4 Piers

The piers are single-column and are made of reinforced concrete. The three piers are the same. The cross-section of the column is circular with 2.20m diameter. The total height including the head beam, together with the dimensions of the pile caps, the number and the length of piles are listed in the following table.

Table 2- 1 Dimensions of Pier geometry

Pier	Total height (m)	Footings dimensions wxbxh (m)	Number, diameter(cm) and length of piles(m)
P1/ P2/ P3	6.15	6.00 x 8.00 x 1.80	9Ø100 x 15m

Over the pier head are foreseen to use the 30cm wide seismic walls that are placed between girders.

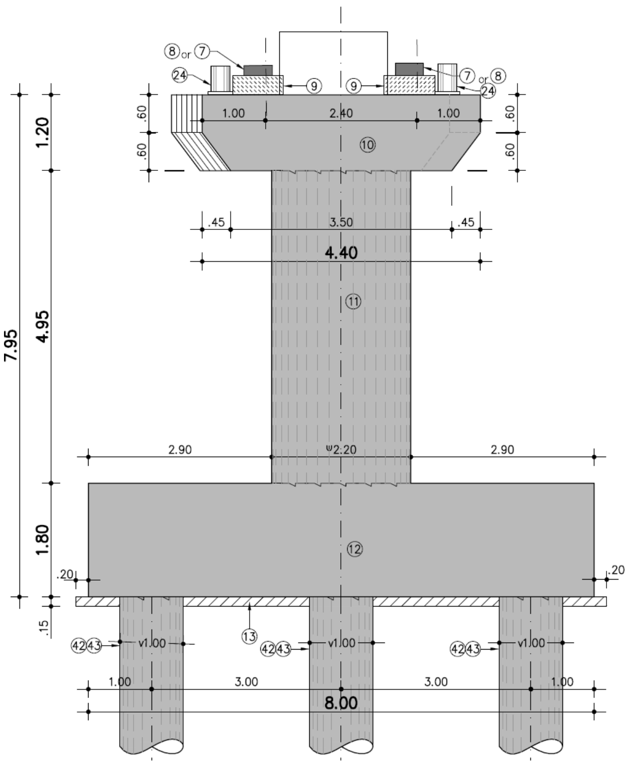


Figure 2- 2 Pier geometry

2.5 Abutments

The two abutments A1 and A2 are the same. The support beam (under the bearings) is about 1.20m tall, 5.50m wide and 5.40m thick measured along the longitudinal direction. Between the back wall and the girders, an abutment gallery 0.80m wide is foreseen serving for the inspection of the bearings and the expansion joints. At the top of the back wall a corbel is formed with thickness ranging from 0.70m to 0.50m at the edge, along the expansion joint gap. The back wall hand the wing walls have the same thickness that varies from 0.50m at the top up to 0.70m at the bottom of wing walls, while at the joint between them, a hunch is formed providing additional stiffness and increased resistance. In the side of the backfill of the abutments is foreseen to use transition slab with 0.40m thickness and 4.00m long referring to the bridge axis direction. Transition slab is 4.40m wide and has 5cm gap between the sidewalls. The connection of the transition slab and abutment is made by chromed steel bars to prevent no-covered steel corodation from the possible moisture.

The pile caps are rectangular 8.00m long and wide and also 1.50m thick, accommodating nine piles Ø1.00 at 3.00m distances between them. The length of the piles is 15.00m for abutments A1 and A2, following foundation design.

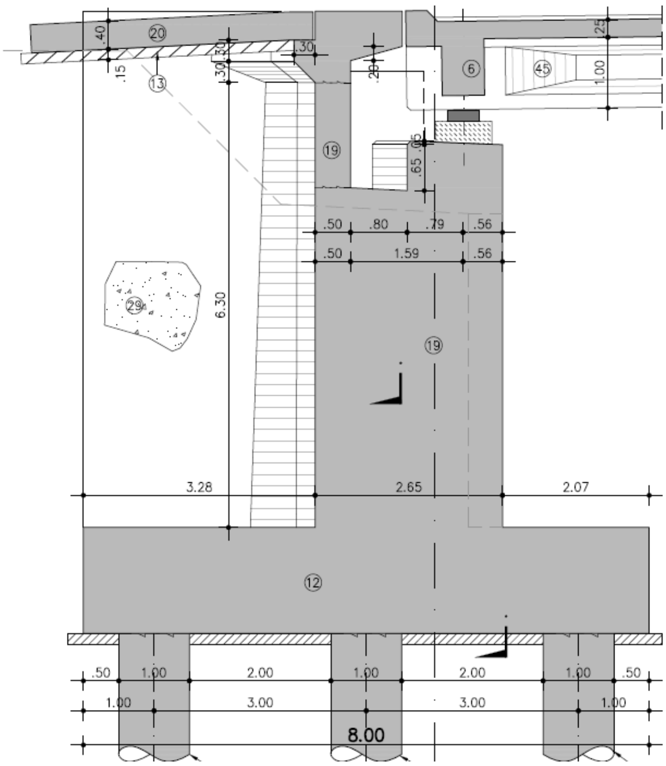


Figure 2- 3 Abutment geometry

3. Material properties

3.1 Concrete characteristics

3.1.1 Exposure class

All components exposed to ambient air are considered to be at risk of corrosion by airborne chlorides and therefore, according to EN 1992-1-1 table. 4.1, the general exposure class **XD1** is considered. In particular, the vertical surfaces of piers, abutments, and retaining walls, which are exposed to rain and freezing, are also categorized as **XF1**.

For piles, wet, rarely dry conditions are assumed and therefore **XC2** class is adopted.

For the cast in situ deck slab, seated on the girders' flanges and protected by a waterproofing membrane, it may be assumed that it is adequately protected from and airborne chlorides, and for this, **XC3** class is chosen.

3.1.2 Structural Class

According to table 4.3 of EN1992-1-1, for a design life of 100 years, the Structural Class generally is S6. Assuming quality assurance in concrete production (as stipulated in tender documents), the construction category is reduced to **S5**.

3.1.3 *Concrete cover*

Taking into account all the above, and the minimum cover for the reinforcement $c_{min, dur}$ set out on table 4.4 of EN1992-1-1, the following table is derived regarding the minimum concrete grade (as presented in annex E of EN1992-1-1) and the nominal concrete cover c_{nom}. A minimum deviation equal to Δ_{cdev} = 10mm has generally been considered.

Table 3- 1 Concrete characteristics

Structural element		Exposure class	Stru-ctural Class	Minimum concrete grade	Max water/ cement Min cement ratio [kg/m³]	Minimum concrete cover $c_{min,dur}$ [mm]	Nominal concrete cover c_{nom} [mm]
	Piles	XC2	S5	C25/30	0,60 300	40	75
General		XD1/XF1/XC4	S5	C30/37	0,50 330	40	50
	Near coast	XS1/XF1/XC4			0,50 330	45	55
Deck slab		XC3	S4	C35/45	0,55 300	35	45

3.2 Reinforcing Steel

For all structural elements Steel grade B500C will be used following the provisions of EN1992-1-1 table C.1, EN1421-3 and EN10080.

3.3 Prestressing Steel

3.3.1 Assumptions

The assumptions about the prestressing are based on the proposed values stated in EN 10138 and EN 1992-1-1.

- The characteristic tensile strength fpk of the prestressing steel used is 1860Mpa.
- The 0,1% proof stress (fp0,1k) is equal to 1600Mpa.
- Coefficient of friction μ between the tendon and its duct is assumed equal to 0.20. This value is in accordance with EN 1992-1-1 table 5.1 and tendons made of strands.
- Unintentional angular displacement (wobble coefficient) is assumed 0.573deg/m (or 0.01rad/m). See also EN 1992-1-1 §5.10.5.2(3).
- Wedge drawn-in is taken equal to 6.0mm.
- The max stress of the tendons is equal to $\sigma_{pm0,max}$ =min{0.75· fpk ; 0.85 fp0,1k } = min{1,395 ; 1,360} =1,360Mpa (EC2-1-1 §5.10.3(2)).
- The internal and external sheath diameters are 72/75mm
- The minimum radius of curvature is 6.50m and the minimum straight line is 0.90m.

3.3.2 Prestressing scheme

In each main girder 2 tendons 9T15 will be used. Each tendon consists of strands with nominal diameter 15.7mm and nominal area of 150mm^2 (A_p=9x150=1,350mm^2).

The applied tendon force is $\mathbf{F_P}=\boldsymbol{\sigma_{pm0,max}}*A_p$ =1,360*1,350= 1,836kN.

4. Construction sequence

The bridge will be constructed using conventional methods, while the sequence of construction works will be as follows:

- All the participation of materials should be removed and if it is possible to use for construction.
- The level of excavation should be finally decided in the presence of the Tirana Ring Road and Railway Alignment Projects to ensure the minimum of 5.00m height from the bottom of Rail Structure to the upper of the pavement Ring Road.
- Excavation works for the two abutments.
- For the all piles must to be drilling, casting and testing after in-situ casting of the concrete.
- Reinforcement placement and concreting of the pile-caps, of the columns and of the head-beam of the piers using conventional formwork.
- Construction of the pile-caps and the walls of the abutments.
- Fabrication and prestressing of the precast girders and transportation on site. The upper plate of the pot bearings should be incorporated in the bottom flange of the beams.
- Placement on the girders on the bearings' plinths. The exact position of the bearings with respect to the plinths will be defined after measuring the position of the respective top plates at the bottom of the girders. The holes for the bearing's dowels on the plinths will be filled with non-shrink epoxy mortar.
- Construction of the deck slab over the prefabricated beams' flanges, which serve as a formwork. Drainage gullies should have been placed in advanced together with the dedicated reinforcement around them as presented on the drawing "951-01-08-DR-DW-BRG-05".
- Construction of the footpaths/cable ducts along the edges of the deck slab and over the wing walls using starter bars protruding from the later.
- Application of the waterproofing membrane over the deck slab and of the protective concrete layer (or asphalt protection course). Slope formation on the protective layer will serve for the water runoff towards the gullies, see drawing "951-01-08-DR-DW-BRG-05".
- Construction of the transition embankment fill behind the abutments.
- Construction of the transition slab over the backfill.
- Completion of the appropriate road layers for the Tirana Ring Road.

5. Details - Finishes

5.1 Drainage of the deck

The drainage of the deck is realized using gullies, arranged at a spacing of 6.00m, along the interface between the first and second beam flanges. Slope configuration of 2% over the levelling concrete ensures water runoff towards the gullies, while free drainage to the river through vertical pipes Ø150 is foreseen.

5.2 Waterproofing of the deck

For the waterproofing of the deck, a monolayer elastomer bitumen membrane is foreseen. The membrane will be weldable, formed by a sheet of elastomer bitumen 4mm thick, six kilograms per square metre of nominal mass (6kg/m2). This membrane is proposed to be bent up over the sides of the cable ducts. For the protection of the membrane, a concrete layer with minimum thickness of 6cm, lightly reinforced with T139 wire mesh may be used or alternatively an asphalt protection course with minimum thickness of 3cm. In any case, the waterproofing membrane should be compatible with the protective layer.

5.3 Cable ducts

The internal effective dimensions of the cable ducts will be defined by the signalling and telecommunication study. In the absence of other information, 0.30mx0.30m can be considered as indicative minimum dimensions. The cable duct may be covered with a lid ensuring access from above all along its length over the bridge or formed as a concrete box without a cover, where access is provided only through manholes.

Their construction is anticipated by in-situ C20/25 concrete, in combination with the lateral fascia.

5.4 Transition embankment

Transition embankment behind abutments aims at ensuring smooth transition from the open line embankment to the bridge deck. The embankment geometry is presented in drawing “951-01-08-DR-DW-BRG-03”., while the requirements regarding the material and the construction method are given in “construction requirements” (T.S.) documents.

5.5 Waterproofing of concrete surfaces in contact with soil

On all the concrete surfaces, which are foreseen to be in contact with soil (pile caps, piers' columns, abutments), double bituminous coating will be applied. In addition to that, all over the inner/soil side of the abutments' walls, draining sheets of MS-drain type will be attached.

5.6 Expansion joints

Expansion joints are foreseen between adjacent decks and between deck and abutments back-wall. The anticipated movement of the joints over sliding bearings is about 50mm, and smaller at abutments, where only fixed bearings are arranged. Nevertheless, all the joints are chosen to be of the same type i.e. TW5/100 or equivalent, which is watertight and appropriate for ballasted track bridges.

5.7 Formwork

For all the concrete surfaces, which will be visible (i.e. precast girders, pier columns, head-beam, etc), special fare faced formwork will be used either for curved or plane surfaces.

6. Design assumptions

6.1 Analysis Model

For the static and dynamic analysis of the structure SAP2000 software is used. The structure is simulated with a grid of shell and beam elements, while the soil – structure interaction is explicitly taken into account with springs. Specifically, the soil-structure interaction is modelled using Winkler springs along the pile length, as well as on the pile base. For the geotechnical calculations, design and verifications is used GEO5 software. The value of the constants of the elastic supports follows the respective geotechnical, foundation design.

Two models are used for the design of the whole bridge. The first one is 3d model of superstructure and piers for static and dynamic results at bearings and pier's foundation, and the second one is 3d model of abutments taking into account the loads comes from the previous analysis.

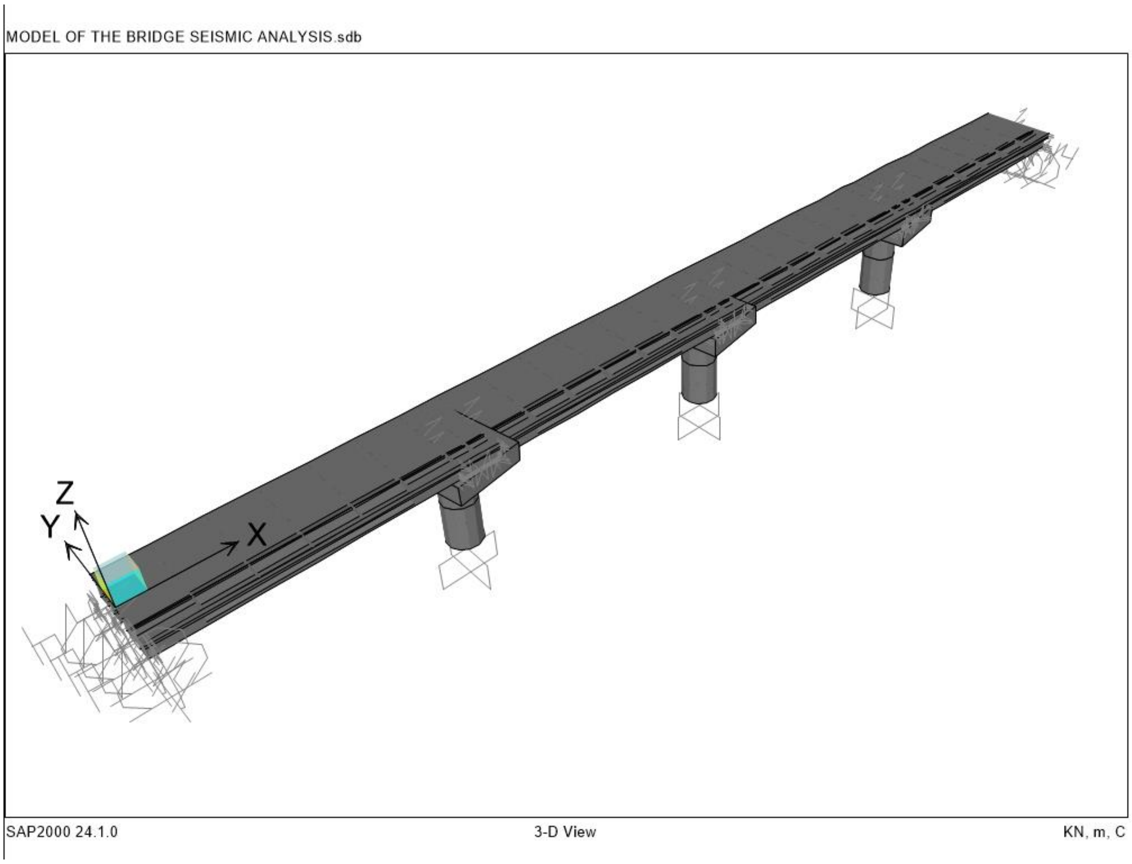


Figure 6- 1 *3D Model –Superstructure and Piers*

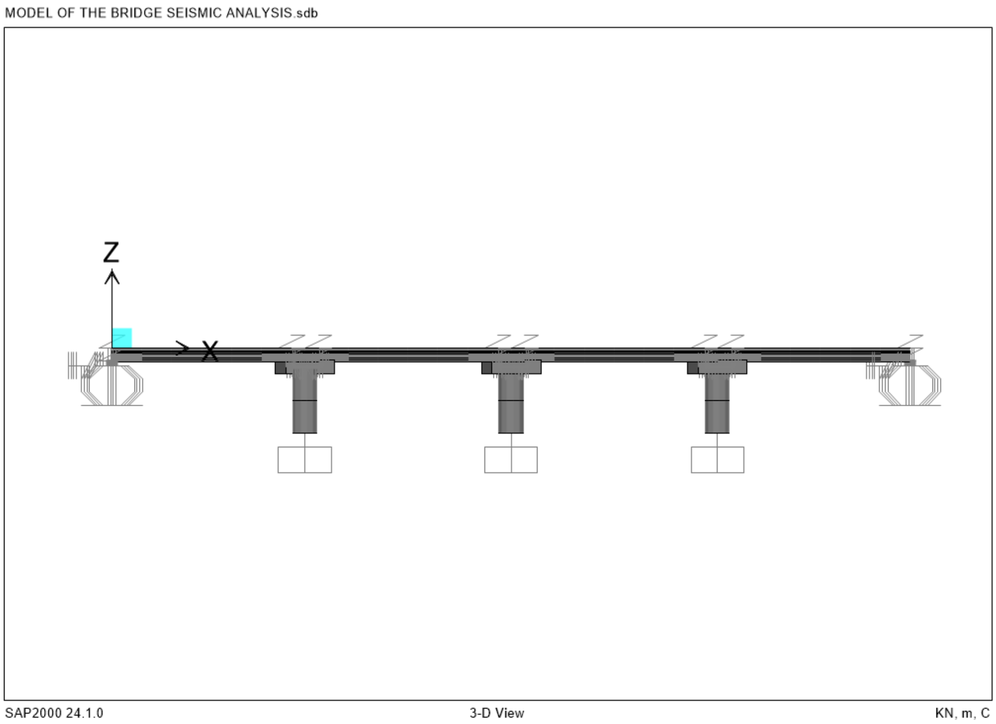


Figure 6- 2 Longitudinal Model - Superstructure and Piers

*Figure 6- 3 Plan-view Model-*Superstructure and Piers

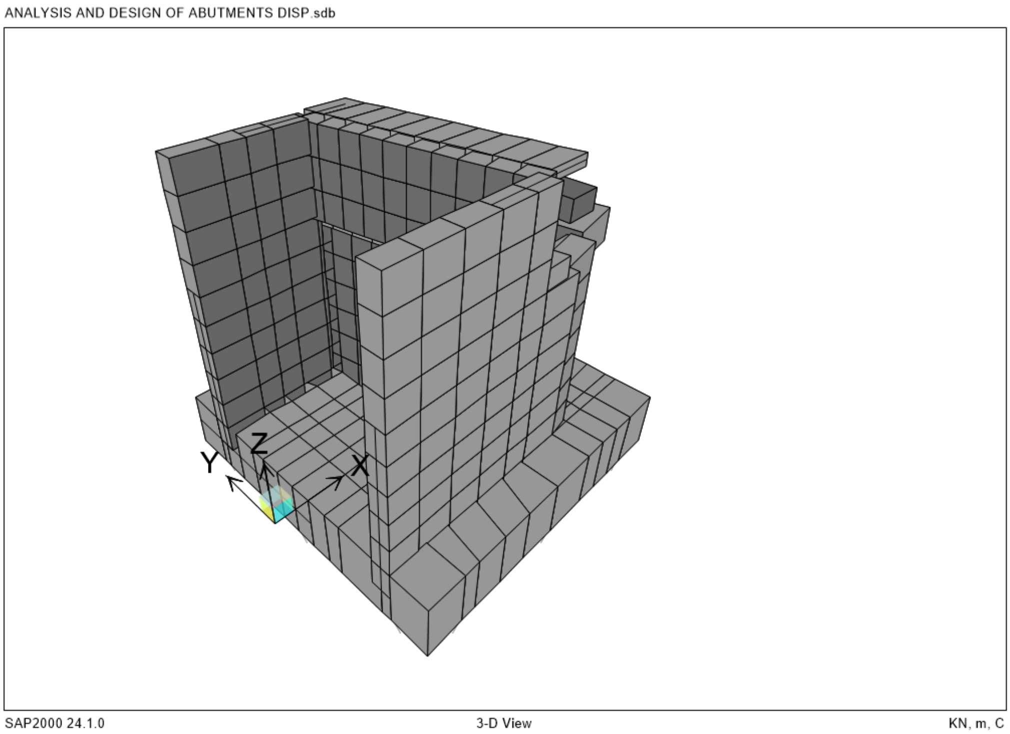


Figure 6- 4 3D Model Abutment Back Side

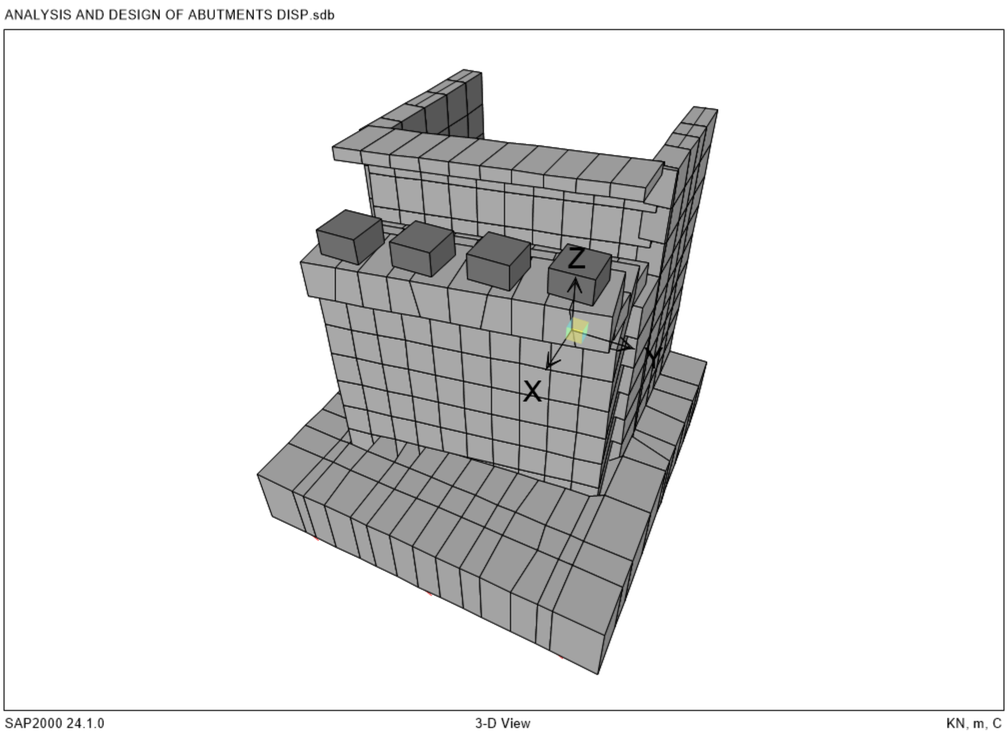


Figure 6- 5 3D Model Abutment – Front Side

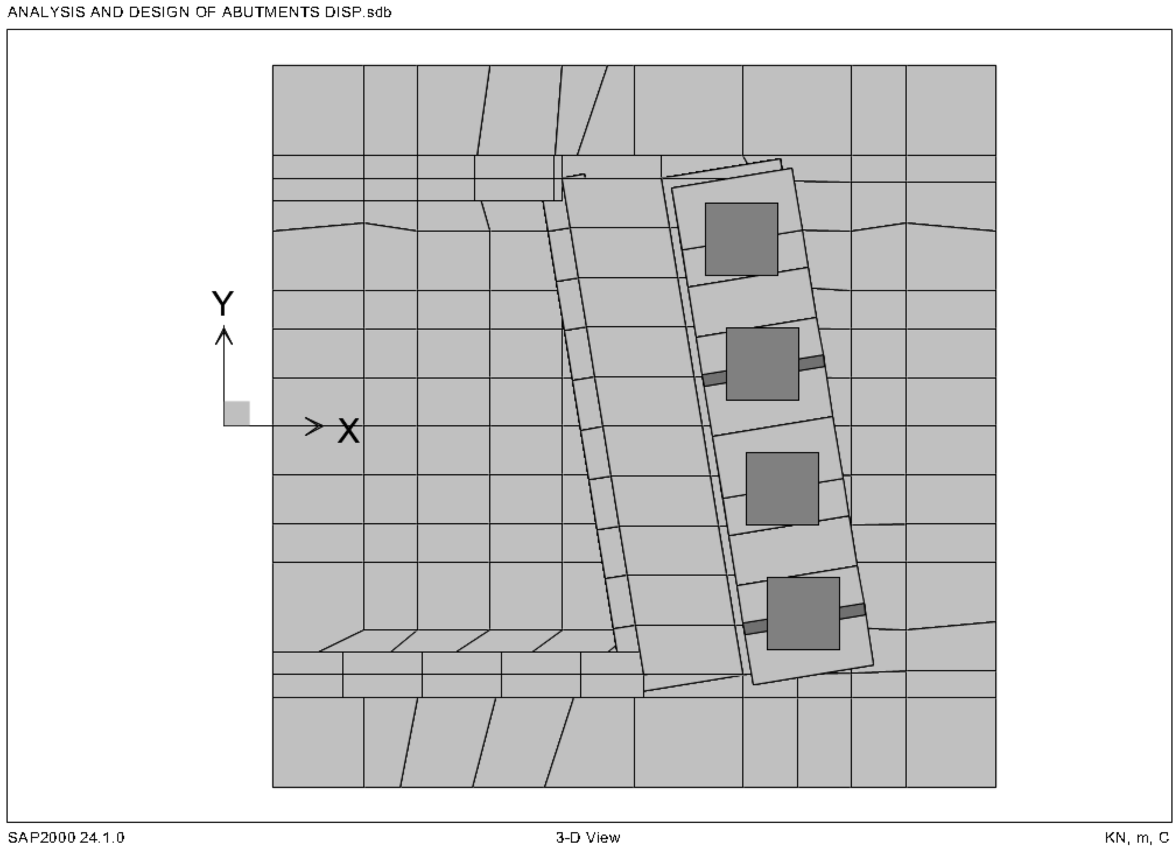


Figure 6- 6 Abutment Plan-view

6.2 Loads

6.2.1 Permanent Loads

In accordance to and EN1991-1-1 the following permanent loads are considered:

- Reinforced concrete density: $25kN/m^3$
- Two rails UIC 60: 1.2kN/m
- Railings at each footpath 0.3kN/m
- Normal ballast track (non basaltic ballast): $20kN/m^3$
- Prestressed concrete sleeper with track fastenings 4.8kN/m
- Infill for abutments $20\ kN/m^3$

6.2.2 Earth Pressure

The earth pressure at back walls is estimated for friction angle ϕ= 30º and the coefficient of earth pressure at rest, Ko is estimated as follows:

$$Ko = 1 - \sin\phi = 0.50$$

A compaction load equal to 25kPa is also considered together with pressure at rest.

In load case combination, where the anticipated displacements are comparatively large, active earth pressure is considered.

$$K_a = \frac{\sin^2(\psi + \varphi_d^{'})}{\cos\theta \cdot \sin^2\psi \cdot \sin(\psi - \delta_d) \cdot \left[1 + \sqrt{\frac{\sin(\varphi_d^{'} + \delta_d) \cdot \sin(\varphi_d - \beta)}{\sin(\psi - \delta_d) \cdot \sin(\psi + \beta)}}\right]^2}$$

For δ=0, β=0 (no slope inclination) and ψ=90 (vertical wall) we take Ka = 0.27

The dynamic force due to earth pressure increment is taken according to EN1998-5 E.9 as being equal to

$$\Delta P_d = \alpha \cdot S \cdot \gamma \cdot H^2$$

Where ***H*** is the total abutment height including ballast thickness, ***γ*** is the embankment soil nominal weight the and the rest parameters are given in § 4.1.3.3 of EN1998-5.

6.2.3 Thermal Actions

General Assumptions

The coefficient of linear expansion for concrete and reinforcing steel is taken $\alpha_T=10^{-5}$ $(^{O}C)^{-1}$, while for structural steel is taken equal to $\alpha_T=1.2*10^{-5}$ $^{O}C^{-1}$ in accordance with Table C.1 of EN 1991-1-5.

In accordance §6.1.1. of EN 1991-1-5, concrete deck ισ classified as Type 3.

Uniform Temperature Component

Since no national annex for EN1991-1-5 is available in Albania, following local practice, the minimum T_{min} and maximum T_{max} shade air temperature is taken as

T_{min}= -20^{O}C and

T_{max}= +40^{O}C.

The minimum and maximum uniform bridge temperature components $T_{e.max}$ and $T_{e.min}$ are derived from the Figure 6.1 of EN 1991-1-5. For bridge deck Type 3 (concrete deck):

$T_{e,min} = T_{min} + 8^{O}C = -20^{O}C + 8^{O}C = -12^{O}C$ and

$T_{e,max} = T_{max} + 2^{O}C = 40^{O}C + 2^{O}C = 42^{O}C$

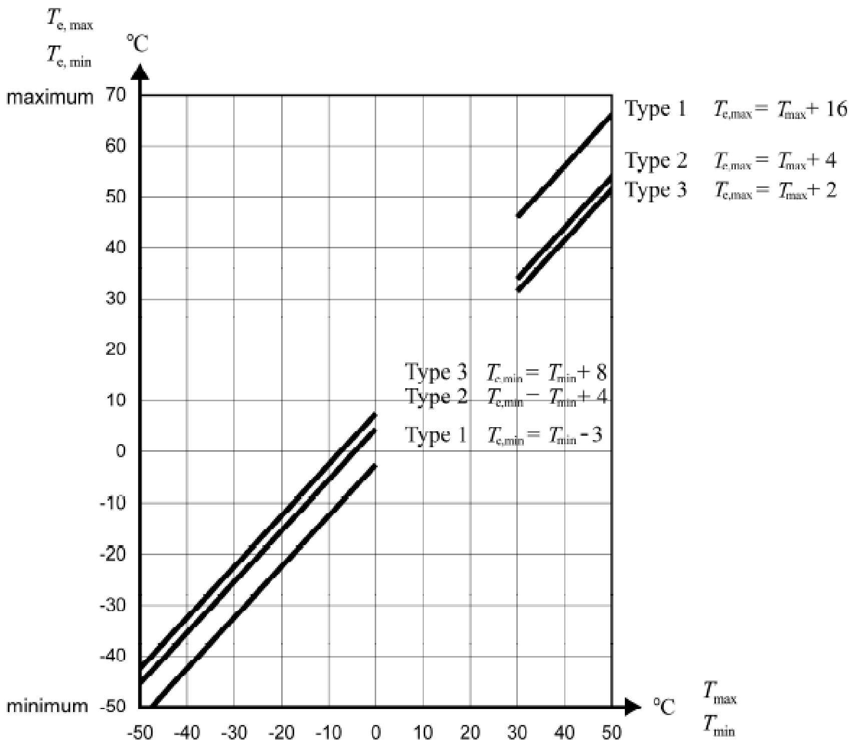


Figure 6- 7 Values of $T_{e.max}$ and $T_{e.min}$ (Figure 6.1 of EN 1991-1-5)

The initial temperature when structural element is restrained is taken equal to $T_0=15^OC$ (default value in EN 1991-1-5). Characteristic values of maximum contraction $\Delta T_{N,con}$ range and maximum expansion $\Delta T_{N,exp}$ range of uniform structure temperature components are derived from expressions (6.1) & (6.2) of EN 1991-1-5.

$\Delta T_{N,con} = T_0 - T_{e,min} = 15^OC - (-12^OC) = 27^OC$

$\Delta T_{N,exp} = T_{e,max} - T_0 = 42^OC - 15^OC = 27^OC > T_0$

Vertical Linear Temperature Component

In accordance to §.6.1.2(2) of EN 1991-1-5:2004, the linearly varying temperature difference component is derived according to Approach 1. According to Tables 6.1 & 6.2 of EN 1991–1–5, for ballasted concrete deck:

Maximum heating (top surface warmer): $k_{sur} \cdot \Delta T_{M,heat} = 0.6 \cdot 15^OC = +9.0^OC$

Maximum cooling (bottom surface warmer): $k_{sur} \cdot \Delta T_{M,cool} = -1.0 \cdot 8^OC = -8.0^OC$

Simultaneity of uniform and temperature difference components

The following load combinations as per §6.1.5 of EN 1991-1-5, are considered.

$\Delta T_{M,heat} + \omega_N \cdot \Delta T_{N,exp}$	$\omega_M \cdot \Delta T_{M,heat} + \Delta T_{N,exp}$
$\Delta T_{M,heat} + \omega_N \cdot \Delta T_{N,con}$	$\omega_M \cdot \Delta T_{M,heat} + \Delta T_{N,con}$
$\Delta T_{M,cool} + \omega_N \cdot \Delta T_{N,exp}$	$\omega_M \cdot \Delta T_{M,cool} + \Delta T_{N,exp}$
$\Delta T_{M,cool} + \omega_N \cdot \Delta T_{N,con}$	$\omega_M \cdot \Delta T_{M,cool} + \Delta T_{N,con}$

Where ω_N and ω_M are taken equal to 0.35 and 0.75 respectively.

6.2.4 Vertical Railway Loads - Load Model 71

The load arrangement of Load Model 71 is presented in the following figure. The axle loads, Q_{vk}, of LM71 are equal to 250kN and the distributed load, q_{vk}, on both rails is equal to 80kN/m.

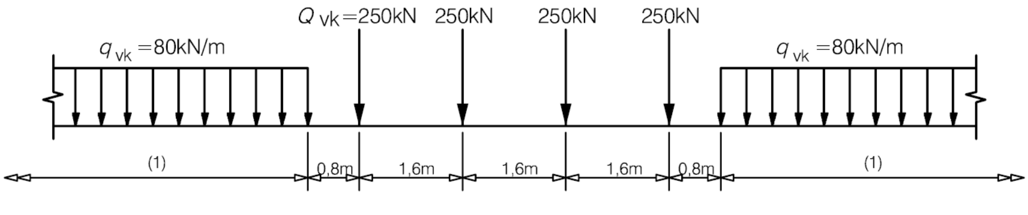
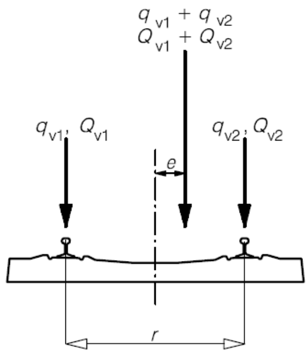


Figure 6- 8 Load model 71 (Figure 6.1 of EN 1991-2)

The classification factor, α, is considered equal to 1.00.

Horizontal Eccentricity of Vertical Traffic Loads

Considering the effect of lateral displacement of vertical loads, the ratio of wheel loads for LM71 on all axles is taken as up to 1.25:1.00 on any one track, the resulting eccentricity is presented in the following figure. This eccentricity is taken into account for the fatigue traffic loads as well.



$(q_{v2}/q_{v1}), (Q_{v2}/Q_{v1}) = 1.25$

Resulting eccentricity $e = 0.083m \leq r/18$

q_{v1}, q_{v2} : Uniformly distributed load on each rail

Q_{v2}, Q_{v2} : Point loads on each rail

r: Transverse distance between wheel loads

Figure 6- 9 Eccentricity of Vertical Loads (Figure 6.3 of EN 1991-2)

Distribution of axle loads by the rails, sleepers and ballast

The distribution in the transverse direction in case of eccentricity of vertical load Qv combined with nosing or centrifugal forces Q_h is as follows. See also EN 1991-2 §6.3.6.3.

When the effects of eccentricity, centrifugal and nosing forces are considered, the loads Qv are displaced to the direction of Q_h or opposite whichever produces the worst effect.

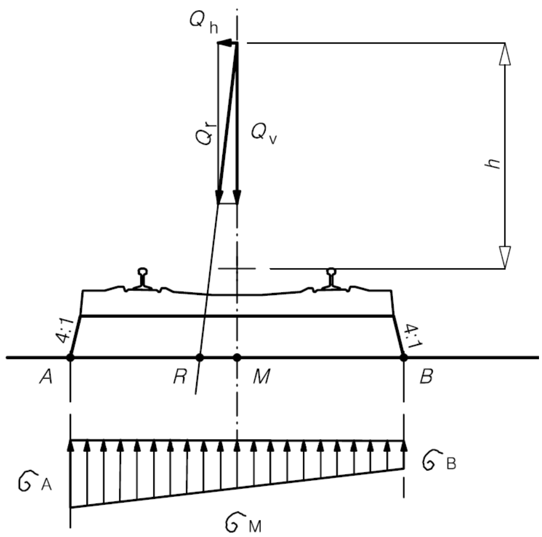


Figure 6- 10 Transverse distribution of actions (EN1991-2 Figure 6.6 & 6.8)

According to the design of superstructure, concrete sleepers will be used with width of 2.60m and min ballast depth of about 0.40cm beneath the sleepers. Considering these, the resulting minimum loading width at the concrete deck level is about 2.80m.

Dynamic Effects

Determining whether a static or a dynamic analysis is required

Since the design Line speed is less than 200km/h, the first natural bending frequency $\boldsymbol{n_0}$ of the span loaded by permanent actions is examined. See EC1-2 §6.4.4(1) Note 6.

The first bending eigenform of the deck is calculated as follows:

$\delta_0 = 5/384 \cdot W \cdot L^4/EI$ = 9.6mm, where W=228kN/m represents the permanent loads (G), E=34GPa for concrete C35/45 and I=0.5915m^4 moment of inertia for deck cross section.

$n_0 = 17.75/\sqrt{\delta_0}$ = 5.75 Hz

The upper and lower frequency limits are given by figure 6.10 of EN1991-2:

L =15.95m: $23.58 \cdot L^{-0.592}$ = 5.02 Hz < $\boldsymbol{n_0}$ **= 5.75Hz** < $94.76 \cdot L^{-0.748}$ = 11.94Hz

Thus, dynamic analysis is not required.

Dynamic factor

The influence of the effects of moving traffic on the static stresses and deformations, are considered by the application of dynamic factor Φ.

As per §3.3.1.2 of document, the track is considered to have standard maintenance. Thus, dynamic factor Φ_3 is applicable, equal to:

$$1.00 \leq \Phi_3 = \frac{2.16}{\sqrt{L_\varphi} - 0.2} + 0.73 \leq 2.00$$

Where, L_Φ is the determinant length, estimated according to the table 6.2 of EN1991-2.

For L_Φ=15.95m we get Φ_3=1.30

According to EN1991-1 §6.4.5.4(2), the effects of rail traffic actions on abutments, foundations and ground pressure are calculated without taking into account dynamic effects.

6.2.5 Horizontal Railway Forces

Nosing Forces

The nosing force is a concentrated force equal to $\alpha \cdot Q_{sk} = \alpha \cdot 100$kN acting horizontally, perpendicular to the center-line of track, at the top of the rails. The acting force is applied in various positions along the rail track and the result envelope is combined with the vertical loads. Since nosing force acts at the top of the rail, additional torsional moments at the deck are also considered.

Traction & Braking Forces

The characteristic values of traction and braking forces are considered as uniformly distributed over the corresponding influence length $L_{a,b}$ and are taken as follows:

Traction Force: $Q_{lak} = \alpha * 33$ kN/m * $L_{a,b}$ [m] = 33*18.25 = 602.25 kN ≤ 1000 [kN]

Braking load: $Q_{lbk} = \alpha * 20$ kN/m * $L_{a,b}$ [m] = 20*18.25m = 365 kN ≤ 6000 [kN]

Considering the results above, it is apparent that only the traction force needs to be considered in the design. In any case, it is checked in both directions.

6.2.6 Actions for non-public Footpaths

Pedestrian or general maintenance loads are considered, by a uniformly distributed load, as per §5.3.2.1 of EN 1991-2, with a characteristic value q_{fk} = 5kPa. The assumption that is made is that in the transverse direction this load acts at two separate lanes at the edges of the bridge deck with width equal to that of the footpath (0.75m).

6.2.7 Earth pressure due to rail traffic

The equivalent characteristic vertical loading due to rail traffic actions for earth pressure effects at abutments and wings walls is taken as the load model 71 uniformly distributed over a width of 3.00 m at a level 0.70 m below the running surface of the track. In the aforementioned loading, no dynamic factor is applied.

6.3 Load Combinations

Combinations are defined as par EN 1990 – Annex A2 (not modified by NA for the railway bridges).

6.3.1 Combination Rules

The following combination rules for traffic loads are derived according to EN1991-2 §6.8, table 6.11.

Table 6- 1 combination rules for railway traffic loads

group	Vertical traffic load	Horizontal traffic load (added only if unfavorable)		
gr11	α·Φ·LM71	α·Qlbk	0.5·α·Qtk	0.5·α·Qsk
gr12		0.5·α·Qlbk	α·Qtk	α·Qsk
gr13	0.5·α·Φ·LM71	α·Qlbk	0.5·α·Qtk	0.5·α·Qsk
gr14		0.5·α·Qlbk	α·Qtk	α·Qsk

6.3.2 Partial and combination factors

The values of the combination coefficients γ_Q and ψ are derived according EN1990 §A2.2.4, table A2.3 & A2.4 (B).

Table 6- 2 Partial factors for actions in ULS

Action	Contribution	Factor	Persistent / Transient
Permanent actions	unfavourable	γ_{Gsup}	1.35
	favourable	γ_{Ginf}	1.00
Traffic loads LM71	unfavourable	γ_Q	1.45
	favourable	γ_Q	0.00
Non-public footpath load	unfavourable	γ_Q	1.50
	favourable	γ_Q	0.00
Wind	unfavourable	γ_Q	1.50
	favourable	γ_Q	0.00
Thermal actions	unfavourable	γ_Q	1.50
	favourable	γ_Q	0.00
Earth pressure	unfavourable	γ_Q	1.35
	favourable	γ_Q	0.00

Table 6- 3 Combination Factors ψ for railway bridges

Actions	ψ_0	ψ_1	ψ_2
Traffic loads LM71	0.80	0.80	0.00
Horizontal earth pressure due to surcharge	0.80	0.80	0.00
Non-public footpath load	0.80	0.50	0.00
Wind forces	0.75	0.50	0.00
Thermal actions	0.60	0.60	0.50
Uniform live load for seismic analysis (derived from Traffic loads LM71)	-	-	0.30

6.3.3 Ultimate Limit State

For Ultimate Limit State (STR/GEO) the following combination as per EN1990 §6.4.3.2 & table A.2.4 (B) is considered:

$$\Sigma\gamma_G G_{k,j}\text{"+"}\gamma_{Q,1}Q_{k,1}\text{"+"}\Sigma\gamma_{Q,i}\psi_{0,i}Q_{k,i}$$

where the partial factors γ_G and γ_Q and factors ψ_0 are listed in the tables above as per EN 1990-A2, table A.2.3 (B).

6.3.4 Design under the seismic load situation

(EN1990 §6.4.3.4 & EN1998-2 §5.5(1)A):

$$E_d = \sum G_k \text{"+"} \quad A_{Ed} \quad \text{"+"} \quad \sum_{i\geq 1}\psi_{2,i}\cdot \quad Q_{1k} \quad \text{"+"} \quad Q_2$$

Q_{1k} a uniform load equivalent to the characteristic value of load model 71 on full length of the deck.

ψ_2 = 0.3 (the bridge is assumed that serves intense traffic).

6.3.5 Fatigue Verifications

The design and verification for Fatigue Load isn’t presented in this stage, it will be performed during the Detailed Design Stage

6.3.6 Serviceability Limit State (S.L.S.)

Serviceability Limit State combinations are determined as per Table A2.6 from EN 1990/A1. Symbols used for the partial factor are from Table A1.4 of EN 1990.

Table 6- 4 General description of SLS Combinations (EN1990 A2.6)

Combination	Permanent actions G_d		Variable actions Q_d	
	Unfavourable	Favourable	Leading	Others
Characteristic	$G_{kj,sup}$	$G_{kj,inf}$	$Q_{k,1}$	$\psi_{0,i}Q_{k,i}$
Frequent	$G_{kj,sup}$	$G_{kj,inf}$	$\psi_{1,1}Q_{k,1}$	$\psi_{2,i}Q_{k,i}$
Quasi permanent	$G_{kj,sup}$	$G_{kj,inf}$	$\psi_{2,1}Q_{k,1}$	$\psi_{2,i}Q_{k,i}$

General assumptions

All ***γ*** factors are taken equal to 1.00, see EN1990 §A2.4.1

Creep, shrinkage and temperature actions are taken into account unless otherwise specified.

S.L.S. characteristic combinations

Under the characteristic combination of loads the tensile stress in the reinforcement and the concrete compressive stress are checked so as not to exceed the limiting values $0.8 \cdot f_{yk}$ and $0.6 \cdot f$ck respectively. See also EN1992-1-1 §7.2(5) and EN1992-2 §7.2(102).

$$\Sigma G_{k,j} \text{"+"} r \cdot P \text{"+"} Q_{k,1} \text{"+"} \Sigma \psi_{0,i} Q_{k,i}$$ EN1990 (6.14b)

S.L.S. quasi-permanent combinations

The crack width is checked under the quasi-permanent combination of actions. See EN1992-2 table 7.101.

$$\sum_{j \geq 1} \mathrm{G}_{k,j} \text{"+"}\ r \cdot \mathrm{P} \text{"+"} \sum_{i>1} \psi_{2,i} \mathrm{Q}_{k,i}$$ EN1990 (6.16b)

According to table 7.101 of EN1992-2 the limiting crack width is 0.3mm for all the exposure classes, which are valid for the specific structure.

6.4 Earthquake analysis

6.4.1 Seismic design parameters

$\alpha_g = \gamma_I \cdot \alpha_{gR}$ with reference peak ground acceleration $\alpha_{gR} = 0.293 \cdot g$ and importance factor equal to $\gamma_I = 1.3$, since the specific bridges may be classified as a major one.

Soil category is B according to "*Study of Engineering Seismic Hazard Potential - Technical Report*". The respective soil factor is S=1.20.

$q = 1.5$ is the behavior factor for limited ductility

$S_e(T) = 2.5 \cdot S \cdot \alpha_g = 1.43 \cdot g$ for $T_B < T < T_C$ and

$S_d(T) = 2.5 \cdot S \cdot \alpha_g / q = 0.762g$ when behavior factor $q = 1.5$ applies

Vertical component of seismic action is considered for the design of the bearings:

$\alpha_{vg} = 0.90 \cdot \alpha_g$, $q_V = 1.00$

The first bending eigenform of the deck is calculated as follows:

$T_v = 1/n_0 = \sqrt{\delta_0}/17.75$

$\delta_0 = 5/384 \cdot W \cdot L^4/EI$ = 9.6mm, where W are the quasi permanent loads ($G + \psi_2 \cdot Q$)

$Tv = 0.174sec$

$T_C < T_v < T_D$, $S_{ve}(T) = a_{vg} \cdot \eta \cdot 3.0 \cdot [T_C/T_v] = 0.9 \cdot 1.30 \cdot 0.293 \cdot g \cdot 1.00 \cdot 3.00 \cdot 0.15/0.174 = 0.88g$

6.4.2 Seismic analysis methodology

According to EN1998-2 §5.6.2(2), the bending resistance of the piers can be verified using behavior factor q = 1.5. Nevertheless, verification of shear resistance of all structural members is carried out using seismic action derived from elastic response spectrum and, moreover, the shear resistance values are divided by an additional safety factor γ_{bd1} = 1.25, see EN1998-2 §5.6.2(2). Similarly, the demand of elastic behavior is ruling the design of foundation (EN1998-2 §5.8.2(2)), the design of bearings (EN1998-2 §6.5.2), and also of the abutments (EN1998-5 §7.3.2.2) leading to the use of elastic response spectrum (or effectively to q = 1.0)

Regarding the design of abutments, the following actions are taken into account:

- Inertia forces acting on the mass of the structure
- Static earth pressures
- Additional seismic earth pressures as per EN1998-5 E.9 ΔPd = αg·S·γ·H2

6.4.3 Traffic load under the seismic design situations

A uniform load equivalent to the characteristic value of LM71 acting on top of the bridge multiplied with partial factor ψ_2 = 0.30 is considered.

6.5 ACCIDENTAL

Derailment 1 and 2 also for any accidental caused by road vehicles during operation is not presented for this stage, these effects will be considered in the Detailed Design Stage.

7. Applicable codes

Eurocode EN1990	Basis of Structural Design
Eurocode EN1990/A2	Basis of Structural Design, Application for Bridges
Eurocode EN1991-1-1	General Actions - Densities, Self-Weight, Imposed Loads
Eurocode EN1991-1-4	General Actions - Wind Actions
Eurocode EN1991-1-5	General Actions - Thermal Actions
Eurocode EN1991-1-6	General Actions - Actions During Execution
Eurocode EN1991-2	Traffic Loads on Bridges
Eurocode EN1992-1-1	Design of Concrete Structures - General Rules
Eurocode EN1992-2	Concrete Bridges Design and Detailing Rules
Eurocode EN1997-1	Geotechnical Design - General Rules
Eurocode EN1998-1	Design of Structures for Earthquake Resistance - General Rules - Seismic Actions
Eurocode EN1998-2	Design of Structures for Earthquake Resistance - Bridges
Eurocode EN1998-5	Design of Structures for Earthquake Resistance Foundations, Retaining Structures and Geotechnical Aspects
EN1337	Structural bearings
EN 206	Concrete - Specification, performance, production and conformity