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Faculty of Civil Engineering Institute of Construction Informatics

Structural analysis of half-frame structures on the example of a railway overpass

by

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Master of Science

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Dresden, December 2019

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Declaration of originality

I confirm that this assignment is my own work and that I have not sought or used the inadmissible help of third parties to produce this work. I have fully referenced and used inverted commas for all text directly quoted from a source. Any indirect quotations have been duly marked as such.

This work has not yet been submitted to another examination institution – neither in Germany nor outside Germany – neither in the same nor in a similar way and has not yet been published.

Dresden, 23 December 2019

Sina Soleimani

(Signature)

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Abstract

The performance of interoperability between BIM model and FEM simulation model is depending on the data-sharing conditions related to the project steps. These data-sharing requirements related to multiple features, like the available data quality, the expected input parameters, the methodology applied to prepare data, and the proposed results of data processing. Also, Building Information Modelling (BIM) leads to essential changes in the design method by offering building analysis in the first step of the design and how to reduce the gap between development, analysis and design. The principal benefit of BIM for modelling in the design stage is design optimisation based on design choices or design options. The decisions about when and how to simulate during analysis are essential to know the design consumption of buildings and its significance for designers to create clear design decisions [1].

Besides, the sustainable design of the building is carried out by beginning the iterative structural analysis from the early design stages of the project like Finite Element Methods (FEM) and Building Information Modeling (BIM) programs.

Concerning the matters mentioned above, in this research, the transformation of the models between BIM software (ALLPLAN 2019) and FEM software (InfoCad) is theoretically examined and described. The possibilities and opportunities between these two programs will be discussed.

Hence, a Half-Frame Bridge model will be created to explore the transformation practically. This issue will be studied for modelling a Half-Frame Concrete Bridge to explore the transformation practically. If possible, in the BIM-software, the potential transformation like the properties of the sections, materials, geometry, boundary conditions, loads and steel reinforcement will be considered.

Then, in the second part of this research, the FEM be considered. The intended structure to be designed in FEM part is the investigated Half-Frame Concrete Bridge in the BIM part. This bridge will be created and analysed with three various concrete class and three different Deck with **40** cm, **55** cm and **80** cm thicknesses.

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List of Abbreviations and Symbols

BIM	Building Information Modelling
FEM	Finite Element Method
LM 71	Load Models 71
SW/2	the static effect of heavy rail traffic
q_{vk}	characteristic value of the vertical load (uniformly distributed load).
Q_{vk}	characteristic value of the vertical load (concentrated load).
e	Eccentricity of vertical loads, eccentricity of resulting action (on reference plane)
Qvi	Wheel load
q_{v1}, q_{v2}	Vertical load (uniformly distributed load)
F_{Ed}	the design support reaction
δ	is the ratio of the redistributed moment to the elastic bending moment
x_u	is the depth of the neutral axis at the ultimate limit state after redistribution
d	is the effective depth of the section
ULS	Ultimate Limit States
SLS	Serviceability Limit States
f_{ck}	Characteristic compressive cylinder strength of concrete at 28 days
w_k	the crack width
d_g	Largest nominal maximum aggregate size
$L_{a,b}$	influence length for traction and braking effects

1 Introduction

1.1 Building Information Modelling (BIM)

Building Information Modelling (BIM) leads to essential changes in the design method by offering building analysis in the first step of the design and how to reduce the gap between development, analysis and design. The principal benefit of BIM for modelling in the design stage is design optimization based on design choices or design options. The decisions about when and how to simulate during analysis are essential to know the design consumption of buildings and its significance for designers to create clear design decisions [1].

The performance of interoperability between BIM model and FEM simulation model is depending on the data-sharing conditions related to the project steps. These data-sharing requirements related to multiple features, like the available data quality, the expected input parameters, the methodology applied to prepare data, and the proposed results of data processing.

This first part of this study aims to investigate the efficiency of transformation from BIM software to FEM software. With this knowledge, the designer can save time, because he or she knows which translations can be performed appropriately and which data failures will occur during each transformation when a BIM-model is transferred.

In the first section of the research, various occasions to transfer the models between BIM-software and FEM-software are theoretically examined and described. Next, a Half-frame Bridge model will be created to explore the transformation practically. That was done by modelling a Half-Frame Concrete Bridge where, if possible, in the BIM-software, boundary conditions were assigned to the nodes, loads were implemented, and for the concrete Half-Frame Concrete Bridge, reinforcement was designed. The possible transformation was studied, and the properties of the sections, materials, geometry, boundary conditions and loads were compared. To analyze the transformation of the nodes, their positions and the transformation of elements, an advanced model was created and transferred for links where excellent results were achieved in the simple model. The transformations are performed using by an IFC data format. Due to the IFC data format being developed as an exchange format that is adequate for a lot of software, it will be the focus of the authors to examine these transformations [2].

In some cases, the outcomes did not support the expectations that utilising an IFC file format is an excellent manner to exchange information between BIM-software and some FEM-software [2].

The considered program for BIM part in this study is ALLPLAN 2019 program. ALLPLAN 2019 is the open solution for Building Information Modeling (BIM) for architects and engineers in building and infrastructure is now available. The new version contains numerous functions that support the creation of buildings and structures with high geometric complexity. The selected IFC viewer software in this program is *Solibri*. The program *Solibri* is the leader in BIM Quality Assurance and Quality Control. It is providing out of the box tools for BIM validation, assent control, design review, design method coordination, analysis and code checking. The selected FEM programs are Ansys and InfoGraph 2019.

Ansys produces and vends finite element analysis software used to simulate engineering projects. The software creates simulated computer models of structures, electronics, or machine elements to simulate strength, toughness, electromagnetism, fluid flow and other properties.

The InfoCAD program system (also known as InfoGraph 2019) is a civil engineering software package for Finite Element System (FEM), 3D Frame (RSW), 2D Frame (ESW) and Axisymmetric Shell (ROS) structures. InfoCAD manages structures in project files. The information contained in a project file is organized in data sets which designer can access from the graphical view, the table view or the File Service command in the File menu. The structure type is defined for the project file and labelled by a file extension (FEM, etc.) in the Structure menu (InfoGraph®).

1.2 Finite Element Method (FEM)

In Finite Element Method (FEM), the software aim is to solve numerically physical equations, which is also called 'Finite Element Analysis' (FEA) and can be done by applying the finite element method (FEM). FEM exists since the initiation of the computer in the late '50s. Back in those times, the direct stiffness method was generalized and developed by M. Jonathan (Jon) Turner. Nowadays, several industries make use of FEM, such as the mechanical and AEC industry [3].

Thanks to FEM, a whole variety of problems can be answered by applying Partial Differential Equation (PDE) and Ordinary Differential Equations (ODE) in combination with the boundary conditions. The technique divides a geometrical model with boundary condition into finite elements, in other words: a mesh is produced, and presents a simulation on the model. Via this simulation, the engineer can detect where the crucial points in the design are found and if improvements should be made. It is possible to perform simulations of heat transfer, stress and strain [4].

1.3 Problem definition

The transformation from BIM to FEM is in developing; but, the advantages of applying BIM are clear, and it will continue to develop in the future. It is a timesaving method that will enhance the standard in the AEC industry. The primary model often had to be made from scratch, but in case of complex or massive models, if it be possible to import the geometry of the model and its data from the BIM model to the FEM software, the time and cost can be saved in the analysis and simulation step. Mostly the FEM software have a method to introduce data from a BIM model, which consequently the analyses of the construction can be fast and moderately simple.

The first part of the scope of this master's thesis is to investigate the visual, geometrical, computational and graphical properties of the created model in BIM software (ALLPLAN 2019) after importing in FEM software (InfoCad). Hence:

1. In BIM part of this research, first, a unique bridge model is created in ALLPLAN program based on predefined measurements.
2. Then, the required properties suitable for the bridge design will be activated and tested.
3. Then after, the modelled bridge will be exported in ICF format. This file later will be tested with an IFC-file viewer to be sure that the created IFC file is in the proper format.
4. Finally, the result will be imported to the FEM programs. In this step, two FEM programs will test the created IFC file.
5. The required properties of the model, such as measurements, coordinate system, volume element, solid element, visual features, mesh system and loading option will be checked in FEM programs.
6. Eventually, the model will be provided for FEM analysis if possible.

In the second part of this research, the FEM will be considered. The intended structure that would be investigated is a Half-Frame Concrete Bridge. The application of frame structures plays an essential role in the renewal of bridges for railway infrastructure systems. A wide variety of structures for smaller and medium span bridges are renewed with such types of constructions. Usually, reinforced concrete frames are designed as half-frame or full-frame structural systems.

Often, railway bridges are crossed by other traffic routes, such as roads, highways or waterways. The paths axes of these intersecting traffic routes are limiting factors for the replacement of such structures and especially the structural design. Often the dimensions of structural elements of the bridge result from these (external) parameters. Ultimate limit states are very often reached from a structural analysis point of view.

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Hence, the aims of this research on the subject, as mentioned above, are:

1. A short review of relevant standards and regulations such as *Eurocode EN 1992* is to be carried out.
2. The bridge must be designed based on predefined measurements and assumptions.
3. Various concrete deck models with Characteristic strength of **C30/37**, **C45/55** and **C60/75** must be modelled. Each concrete class must also be designed for three separate Deck thickness of **40** cm, **55** cm and **80** cm. But the part of the bridge like Abutments, Wing Walls and Foundation must be fixed in whole models with Concrete Characteristic strength of **C35/45**.
4. The bridge must be loaded based on relevant *load cases*.
5. After analysis of the bridge, based on the analysed data, the critical points such as the *Middle of Deck* and *Corners of Deck* must be investigated from **Maximum Vertical Deformation (U_z)**, **Maximum Bending Moments** and **Steel Reinforcement** in **X Direction** points of view.
6. Then after, in case of a technical problem, a solution must be considered.
7. Finally, the result must be explained and presented.

2 Eurocode Summary

It is clearly stated that this chapter is a summary of the Eurocode part Eurocode 1992-2 (Design of concrete structures, Part 2 Concrete bridge) and DIN EN 1991-2 (Traffic loads on bridges). And, the reference of calculations in this research are the Codes as mentioned above [5, 6].

In this chapter, a short review of Eurocode 1992-2 (Design of concrete structures, Part 2 Concrete bridge) and DIN EN 1991-2 (Traffic loads on bridges) is given.

2.1 Strength

The strength classes in Eurocode 1992-2-2 has been written by the characteristic cylinder strength f_{ck} determined at 28 days with a minimum value of C_{min} and a maximum value of C_{max} with a respectively recommended value of C30/37 and C70/85. For each country is recommended to see the National Annex.

2.2 Environmental conditions

Water penetration or leakage as well from carriageway into the joints and voids must be considered in the design part. Moreover, surface protection via waterproofs should be mentioned in expose classes. In this case, the exposer with type XC3 is recommended, while, the country National Annex should be considered for sure.

De-icing salt sometimes is used in carriageways. In all exposed concrete surfaces, as a recommendation, the classes XD3, Xf2 or XF4 are used in surfaces that directly are affected by de-icing salt.

2.3 Methods of verifications

2.3.1 Concrete cover

Where in-situ concrete is placed facing an existing concrete surface (precast or in-situ) the conditions for cover to the reinforcement from the interface may be adjusted:

- The existing concrete surface has not been subject to an outdoor environment for more than 28 days.
- The existing concrete surface is rough.
- The strength class of the existing concrete is at least C25/30.
- Bare concrete decks of road bridges, without waterproofing or surfacing, should be classified as Abrasion Class XM2.

- Where a concrete surface is subject to abrasion caused by ice or solid transportation in running water, the cover should be increased by at least 10 mm.

2.4 Structural analysis

2.4.1 General

In regarding the combinations of actions, the relevant load cases shall be considered to facilitate the critical design conditions to be established at all parts, within the structure or part of the structure considered.

2.4.2 Idealisation of the structure

Where a beam or slab is continuous over support which may be supposed to provide no restraint to the rotation, and the analysis assumes point support, the design support moment, calculated on the basis of a span equal to the centre-centre distance between supports, may be reduced by an amount MEd as follows:

$\Delta M_{Ed} = F_{Ed,sup}t/8$

2.4.3 Linear elastic analysis with limited redistribution

In continuous beams or slabs which:

a) are effectively subject to flexure and

b) have the ratio of the lengths of adjacent spans in the range of 0,5 to 2

redistribution of bending moments may be carried out without explicit check on the rotation capacity, presented that:

- $\delta \geq k_1 + k_2 x_u/d$ *for* $f_{ck} \leq 50$ MPa
- $\delta \geq k_3 + k_4 x_u/d$ *for* $f_{ck} > 50$ MPa

δ is the ratio of the redistributed moment to the elastic bending moment
x_u is the depth of the neutral axis at the ultimate limit state after redistribution
d is the effective depth of the section

2.4.4 Plastic analysis

Classifications based on plastic analysis should only be applied for the check at ***ULS*** *and only when authorized by National Authorities.*

The requisite ductility may be considered to be satisfied if all the following are fulfilled:

i. The area of tensile reinforcement is limited such that, at any section.

$x_u/d \leq 0{,}15$ for concrete strength classes ≤ C50/60

$x_u/d \leq 0{,}10$ for concrete strength classes ≥ C55/67

ii. Reinforcing steel is either Class B or C.
iii. The ratio of the moments at intermediate supports to the moments in the span is among 0,5 and 2.

It should be here noted that in regions of yield hinges, x_u/d should not exceed 0,30 for concrete strength classes less than or equal to C50/60, and 0,23 for concrete strength classes greater than or equal to C55/67.

2.4.5 Non-linear analysis

The non-linear analysis may be used provided that the model can appropriately cover all failure modes, and that the concrete tensile strength is not used as a primary load resisting mechanism. If one analysis is not adequate to prove all the failure mechanisms, separate further analyses should be carried out.

2.5 Ultimate Limit States (ULS)

2.5.1 Definition

The ultimate limit state is the design for the safety of construction and its users by restricting the stress that materials experience. To comply with engineering requirements for strength and stability under design loads, ULS must be achieved as a certain condition.

The ULS is a substantially elastic condition, usually located at the upper part of its elastic zone (nearly 15% lower than the elastic limit). This is in opposition to the ultimate state (US), which includes excessive deformations approaching structural collapse and is located intensely within the plastic zone.

If all factored bending, tensile, shear or compressive stresses are below the measured resistances, then a structure will satisfy the ULS criterion. Safety and reliability can be assumed as long as this criterion is fulfilled since the structure will behave in the same way under repeated loadings.

2.5.2 Assumption in ULS

During determining the ultimate moment resistance of reinforced or prestressed concrete cross-sections, the following assumptions are made:

- Plane sections remain plane.

- The strain in bonded reinforcement or bonded prestressing tendons, whether in tension or compression, is the same as that in the neighboring concrete.
- The tensile strength of the concrete is ignored.
- The stresses in the concrete in compression are derived from the design stress/strain relationship.
- The initial strain in prestressing tendons is taken into account when evaluating the stresses in the tendons.
- In case of bending, for external prestressing tendons, the strain in the prestressing steel among two continuous fixed paints is assumed to be constant.
- In case of bending, the strain in the prestressing steel is then similar to the remaining strain, after losses, increased by the strain resulting from the structural deformation between the fixed points considered.
- In the case of straight tendons, a huge level of prestress ($\sigma_{cp}/f_{cd} > 0,5$) and thin webs, if the tension and the compression chords are capable of carrying the entire prestressing force, and also blocks are given at the extremity of beams to disperse the prestressing force, it may be assumed that the prestressing force is distributed within the chords. In these situations, the compression field due to shear only should be considered in the web (α_{cw} = 1).

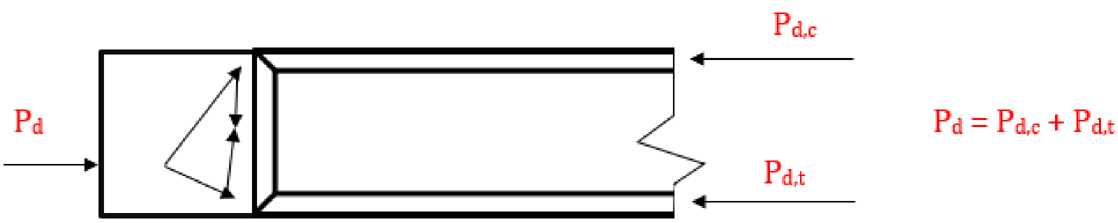


Figure 1. 1 Dispersion of prestressing by end blocks within the chords

- In the matter of segmental construction with precast elements and no bonded prestressing in the tension chord, the influence of the opening of the joint should be considered. In these circumstances, in the absence of a detailed analysis, the force in the tension chord should be assumed to remain unchanged after the joints have opened. In consequence, as the applied load increases and the joints open, the concrete stress field inclination within the web increases. The depth of the concrete section available for the flow of the web compression field decreases to a value of h_{red}. The shear capacity can be estimated by assuming a value of θ derived from the minimum value of residual depth h_{red}.

➢ $h_{red} = V_{Ed} / (b_w \nu f_{cd})(\cot\theta + \tan\theta)$

- The influences of ***torsion*** and ***shear*** for both hollow and solid elements may be superimposed, assuming the same amount for the strut inclination θ. The limits for θ are also entirely suitable for the case of combined shear and torsion. See Figure 1. 2.

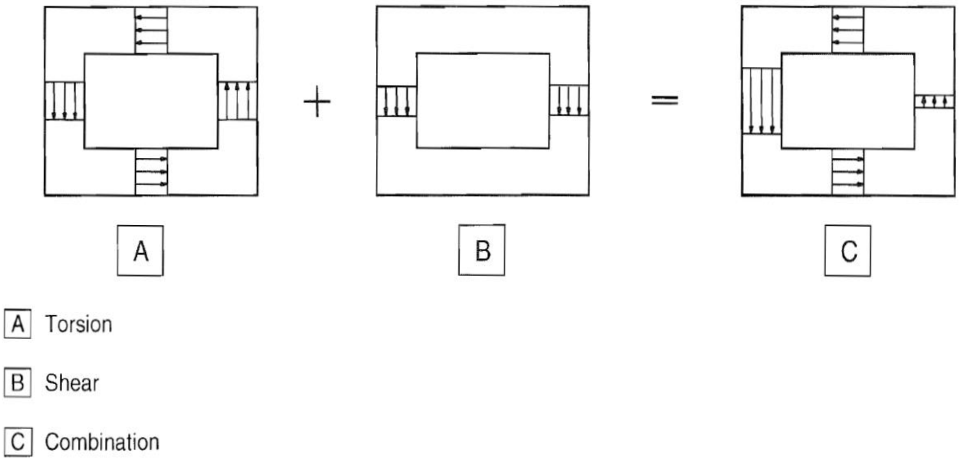


Figure 1. 2 Internal actions combination within the different walls of a box section

- In the case of segmental structure with precast box elements, and no internal bonded prestressing in the tension region, the opening of a joint to an extension greater than the thickness of the corresponding flange entails a substantial modification of the torsional resisting mechanism if the relevant shear keys are not able to transfer the local shear due to torsion. It changes from Bredt circulatory torsion to a combination of warping torsion and De Saint Venant torsion, with the first mechanism prevailing over the second (Figure 1. 3). As a result, the web shear due to torsion is functionally doubled, and a significant deformity of the section takes place. In these conditions, the capacity at the ultimate limit state should be checked in the most heavily stressed web according to the method in Annex MM taking into account the combination of bending, shear and torsion.

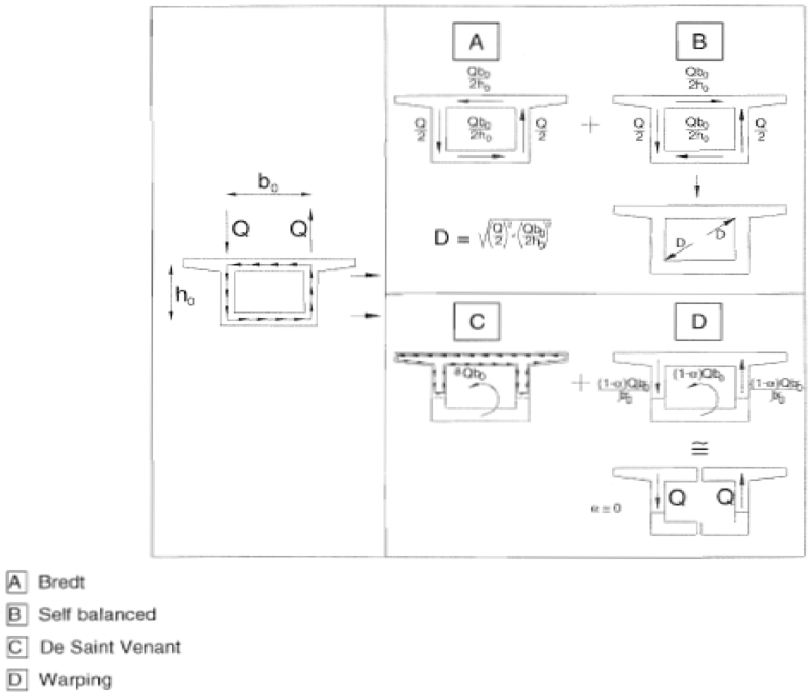


Figure 1. 3 Variation in torsional behaviour from closed to opened joint

- **Fatigue** verifying should be given for structures and structural elements which are subjected to regular load cycles.

In general, for the following structures and structural elements is not necessary to verify the fatigue:

- Footbridges, except structural components susceptible to wind action.
- Buried arch and frame structures with at least earth cover of 1.00 m and 1.50 m respectively for road and railway bridges.
- Foundations.
- Piers and columns which are not rigidly connected to superstructures.
- Retaining walls of embankments for roads and railways.
- Abutments of road and railway bridges which are not rigidly connected to superstructures; except the slabs of hollow abutments.
- Prestressing and reinforcing steel, in regions where, under the frequent combination of actions and P_k, only compressive stresses happen at the extreme concrete fibers.

- **Membrane elements** may be used for the design of two-dimensional concrete elements subject to a combination of internal forces evaluated through a linear finite element analysis. Membrane elements are subjected only to in-plane forces, namely σ_{Edx}, σ_{Edy}, τ_{Edxy} as shown in Figure 1. 4. And also, membrane elements may be created by the application of the theory of plasticity using a lower bound solution.

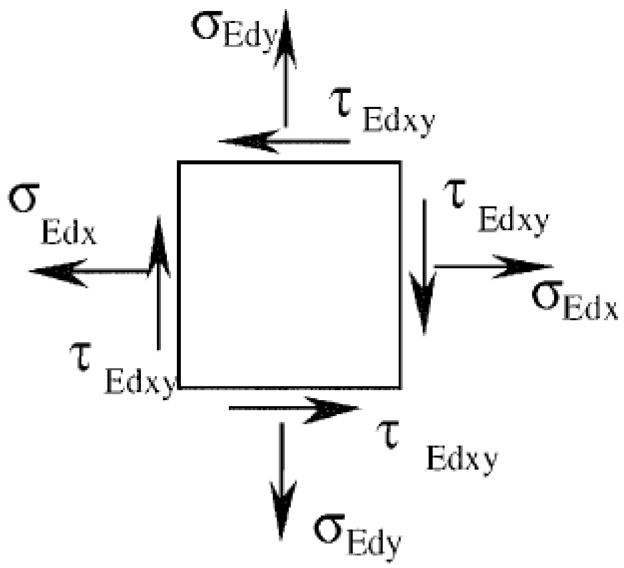


Figure 1. 4 Membrane element

2.6 Serviceability Limit States (SLS)

The **serviceability limit state** (SLS) is the design to ensure the construction is comfortable and useable. This includes vibrations, also deflections, as well as cracking and durability. These are the requirements that are not strength-based but still may provide the structure unsuitable for its intended use; for example, it may cause occupant discomfort under routine situations. For example, a skyscraper could oscillate severely and cause the inhabitants to feel annoying yet be perfectly sound structurally. This building is in no danger of collapsing, yet since it is no longer fit for human occupation, it is considered to have passed its serviceability limit state. It might also involve limits to non-structural issues such as acoustics and heat transmission.

To fill the serviceability limit state criteria, a construction must remain functional for its expected utility subject to usual loading, and as such, the structure must not begin resident discomfort under routine conditions. A structure is considered to satisfy the serviceability limit state when the constituent elements do not deflect by more than certain limits laid down in the building codes, the floors fall within calculated vibration criteria, besides, to other possible requirements as required by the applicable building code.

SLS requirements lead to be less rigid than strength-based limit states as the safety of the structure is not in question. A structure must remain functional for its intended use subject to routine loading to satisfy SLS standard. Also, serviceability refers to the requirements under which a structure is considered serviceable. It refers to conditions other than the building strength that render the buildings unusable.

Serviceability limit state design of structures involves factors such as durability, overall stability, fire resistance, deflection, cracking and excessive vibration.

2.6.1 Assumption in SLS

During determining the ultimate moment resistance of reinforced or prestressed concrete cross-sections, the following assumptions are made:

- The compressive stress in the concrete shall be restricted to avoid longitudinal cracks, micro-cracks or high levels of creep, where they could appear in unacceptable influences on the function of the structure.
- Longitudinal cracks may happen if the stress level under the characteristic compound of loads exceeds a critical value. Such cracking may lead to a reduction of durability. In the lack of other measures, such as an increase in the cover to reinforcement in the compressive zone or confinement by transverse reinforcement, it may be proper to limit the compressive stress to a value k1fck in regions exposed to environments of exposure classes XD, XF and XS.
- If the stress in the concrete under the quasi-permanent loads is less than k2fck linear creep may be considered. If the stress in concrete exceeds k2fck, non-linear creep should be regarded.
- In the reinforcement, tensile stresses shall be limited to avoid inelastic strain, unacceptable cracking or deformation.
- Cracking shall be restricted to an extent that will not damage the proper functioning or durability of the structure or cause its shape to be unacceptable.
- Cracking is typical in reinforced concrete structures subject to torsion, bending, shear or tension resulting from either direct loading or restraint or imposed deformations.
- Cracks may also occur from other causes such as plastic shrinkage or comprehensive chemical responses within the hardened concrete. Such cracks may be unacceptably large, but their avoidance and control lie outside the scope of this Section.
- Cracks may be allowed to form without any attempt to control their width, provided they do not damage the functioning of the construction.
- A limiting value, wmax, for the measured crack width, wk, considering the proposed purpose and nature of the structure and the costs of limiting cracking, should be authenticated.
- Specific criteria may be required for members subjected to exposure class XD3. The choice of suitable measures will depend upon the nature of the aggressive agent involved.

- When utilizing strut-and-tie models with the struts located based on the compressive stress trajectories in the uncracked state, it is possible to handle the forces in the ties to obtain the corresponding steel stresses to evaluate the crack width.
- Crack widths may be measured according to a simplified alternative like limit the bar size.
- If the crack check is needed, a minimum value of bonded reinforcement is necessary to control cracking in areas where tension is expected. The value may be estimated from the equilibrium within the tensile force in concrete just before cracking and the tensile force in reinforcement at yielding or at a lower stress if needed to restrict the crack width.
- In prestressed elements, no minimum reinforcement is needed in sections where, under the characteristic combination of loads and the characteristic value of prestress, the concrete is compressed, or the absolute value of the tensile stress in the concrete is below σcp,t.
- For reinforced or prestressed slabs in structures subjected to bending without notable axial tension, specific measures to control cracking are not necessary where the overall depth does not pass 200 mm.

Note that there are unique risks of large cracks occurring in sections where there are unexpected changes of stress like:

- *at changes of section;*
- *near concentrated loads;*
- *areas where bars are curtailed;*
- *regions of high bond stress, especially at the ends of laps.*

Caution should be taken at such states to minimize the stress changes wherever possible. Despite, the rules for crack control given above will usually ensure adequate control at these points contributed that the regulations for detailing reinforcement.

- The crack width, wk, calculated from expression:

$$\mathbf{w_k = s_{r,max} (\varepsilon_{sm} - \varepsilon_{cm})}$$

where:

$\mathbf{w_k}$ is the crack width.

$\mathbf{s_{r,max}}$ is the maximum crack spacing.

$\mathbf{\varepsilon_{sm}}$ is the mean strain in the reinforcement under the relevant combination of loads, including the impact of imposed deformations and considering the influences of tension stiffening.

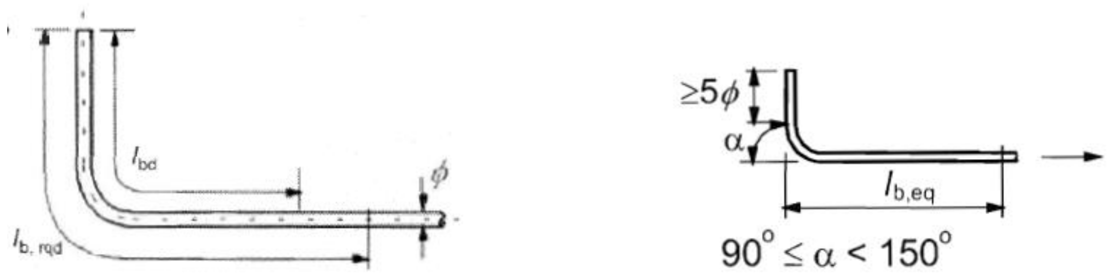
$\mathbf{\varepsilon_{cm}}$ is the mean strain in the concrete between cracks.

2.7 Detailing of reinforcement and prestressing tendons

2.7.1 General

The rules are given in this section apply to ribbed reinforcement, mesh and prestressing tendons subjected predominantly to static loading. They are appropriate for normal structures and bridges. They may not be sufficient for:

- Components subjected to dynamic loading induced by seismic effects or machine vibration, impact loading and.
- The items incorporating specially painted, epoxy or zinc-coated bars.
- The spacing of bars shall be such that the concrete can be set and compacted adequately for the development of adequate bond.
- The pure distance between single parallel bars or horizontal layers of parallel bars should be not less than the maximum of k1 bar diameter, (dg + k2 mm) or 20 mm where dg is the maximum size of aggregate.
- Lapped bars may be permitted to touch one another within the lap length.
- The minimum diameter to which a bar is bent shall be such as to avoid bending cracks in the bar, and to avoid failure of the concrete inside the bend of the bar.
- To avoid damage to the reinforcement, the diameter to which the bar is bent should not be less than φm,min.
- Reinforcing bars, wires or welded mesh fabrics shall be so anchored that the bond forces are safely transcribed to the concrete avoiding longitudinal cracking or spalling. Transverse reinforcement shall be provided if required.
- Some methods of anchorage are shown in Figure 1. 5 and Figure 1. 6:



(a) (b)

Figure 1. 5 Anchorage methodes

a) Basic tension anchorage length, $l_{b,rqd,}$ for any shape measured along the centerline
b) Equivalent anchorage length for standard bend

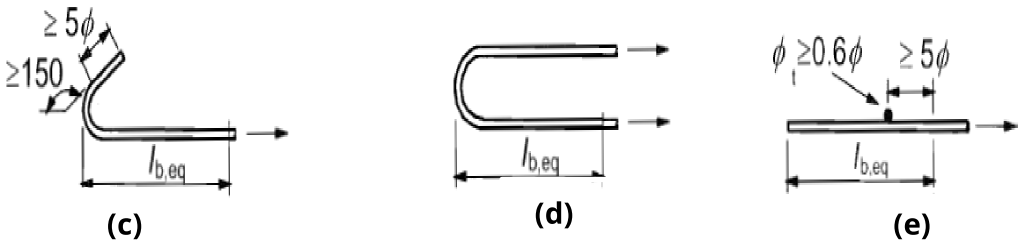


Figure 1. 6 anchorage methodes

c) Equivalent anchorage length for standard hook
d) Equivalent anchorage length for the standard loop
e) Equivalent anchorage length for welded transverse bar

- Where mechanical devices are applied, the test specifications should be in accordance with the proper product standard or European Technical Approval.
- Bends and hooks do not contribute to compression anchorages.
- The ultimate bond strength shall be enough to limit bond failure.
- The anchorage of links and shear reinforcement should usually be caused by means of bends and hooks, or by welded transverse reinforcement. A bar should be provided inside a hook or bend. The anchorages should comply with Figure 1. 7:

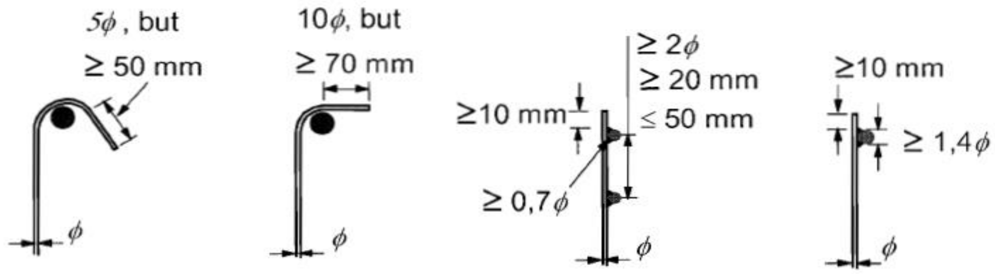


Figure 1. 7 Anchorage of links

2.8 Detailing of members and particular rules

The terms for safety, serviceability and durability are provided by following the rules given in this section in addition to the general rules given elsewhere:

- Minimum areas of reinforcement are given to prevent a brittle failure, wide cracks and also to resist forces arising from restrained actions.
- The area of longitudinal tension reinforcement should not be taken as less than $A_{s,min}$. The recommended value is given in the following:

  $A_{s,min} = 0.26\, f_{ctm}/f_{yk} * b_t d \geq 0.0013 b_t d$

- Sections containing less reinforcement than $A_{s,min}$ should be considered as unreinforced.
- The cross-sectional area of tension or compression reinforcement should not exceed $A_{s,max}$ outside lap locations.
- For members prestressed with permanently unbonded tendons or with external prestressing cables, it should be verified that the ultimate bending capacity is larger than the flexural cracking moment. A capacity of 1,15 times the cracking moment is sufficient.
- At intermediate supports of continuous beams, the total area of tension reinforcement A_s of a flanged cross-section should be spread over the effective width of the flange. Part of it may be concentrated over the web width (See Figure 1. 8).

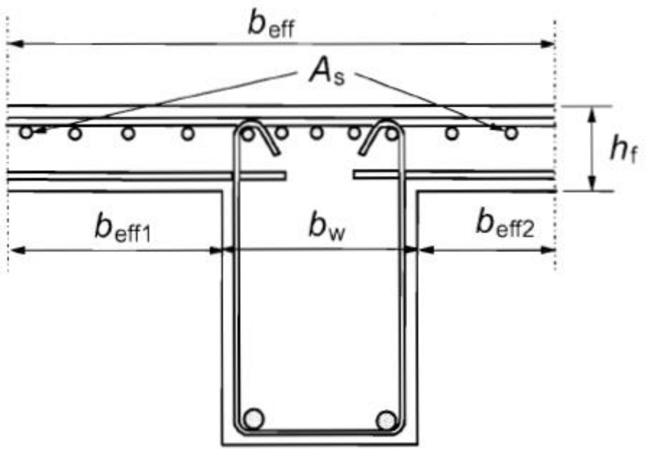
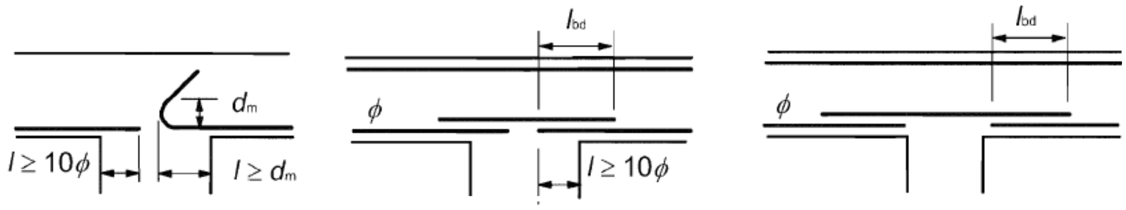


Figure 1. 8 Tension reinforcement in flanged cross-section.

- The anchorage length should not be less than 1O Φ (for straight bars) or not less than the diameter of the mandrel (in case of hooks and bends with bar diameters at least equal to 16 mm) or twice the diameter of the mandrel (see Figure 1. 9 (a)). The reinforcement expected to resist possible positive moments should be defined in contract documents. This reinforcement should be continuous, which may be obtained utilizing lapped bars (see Figure 1. 9 (b) or (c)).



(a) (b) (c)

Figure 1. 9 Anchorage at intermediate supports

- The diameter of the transverse reinforcement should not be smaller than 6 mm or one-quarter of the maximum diameter of the longitudinal bars, whichever is the larger. The diameter of the wires of welded mesh fabric for transverse reinforcement should not be less than 5 mm. The transverse reinforcement should be anchored enough.
- The distance from the outer edge of the pile to the edge of the pile cap should be such that the tie forces in the pile cap can be correctly anchored. The proposed deviation of the pile on site should be considered.
- Reinforcement in a pile cap should be calculated either by using strut-and-tie or flexural systems as relevant.
- The main tensile reinforcement to resist the action effects should be concentrated in the stress zones between the tops of the piles. A minimum bar diameter φ_{min} should be implemented. If the area of this reinforcement is at least equal to the minimum reinforcement, evenly distributed bars along the bottom surface of the member may be omitted. As well, the sides and the top surface of the member may be unreinforced if there is no risk of tension developing in these elements of the member.
- In case of *deep beams*, the distance between two adjacent bars of the mesh should not exceed s_{mesh}. The recommended value of s_{mcsh} is the lesser of the web thickness or 300 mm.
- For *pile caps*, the main tensile reinforcement to withstand the action effects should be concentrated in the stress zones between the tops of the piles. A minimum bar diameter d_{min} should be presented. If the state of this reinforcement is at least equal to the minimum reinforcement, equally divided bars along the bottom surface of the member may be eliminated. The suggested value for d_{min} is 12 mm.

2.9 Field of application

When the self-weight of none-structural elements are discussed, it means that the weight of all elements such as noise and safety barriers, ducts, cables and the others are considered.

During the design, the designer should consider temporary bridges due to the flexibility of some temporary structures. It means the loads and requirements should be correctly specified. Moreover, in this case, the National Annex should be used for each specific projects.

2.10 Representation of actions - nature of rail traffic loads

Some general rules are given for the consideration of the associated dynamic influences, braking forces, centrifugal forces, nosing force, traction and aerodynamic actions due to passing rail traffic.

As well, the railway operations include multiple actions that significantly affect the application of the railway. The considered effects are listed below:

- Vertical loads: Load Models 71, SW (SW/O and SW/2), "unloaded train" and HSLM,
- Vertical loading for earthworks (see 6.3.6.4),
- Dynamic effects (see 6.4),
- Centrifugal forces (see 6.5.1),
- Nosing force (see 6.5.2),
- Traction and braking forces (see 6.5.3),
- Aerodynamic actions from passing trains (see 6.6),
- Actions due to overhead line equipment and other railway infrastructure and equipment (see 6.7.3) (2).

2.11 Vertical loads - Characteristic values (static effects) and eccentricity and distribution of loading

2.11.1 General

The railway actions mainly specified based on load models. Here, five load models are sorted in this matter:

- Load Model 71 (and Load Model SW/0 for continuous bridges) to represent normal rail traffic on mainline railways,
- Load Model SW/2 to describes heavy loads,

- Load Model HSLM to represent the loading from passenger trains at speeds exceeding 200 km/h,
- Load Model "unloaded train" to describe the effect of an unloaded train.

It should be here mentioned that a factor ***α*** may increase the characteristic values for SW/0 and 71 Load Models for lines carrying rail traffic which is more substantial or smaller than the standard. In additions, some provisions also have provided to make differences between different railways in field of its nature, volume and maximum weight of rail traffic, as well, different qualities of track.

2.11.2 Load Model 71

Load Model 71 illustrates the static impact of vertical loading due to regular rail traffic.

The loading system and the characteristic values for vertical loads shall be taken as presented in Figure 1. 10.

The characteristic values are given in Figure 1. 11 shall be multiplied by a factor **α** (on lines carrying rail traffic), which is heavier or lighter than regular rail traffic. Consequently, when multiplied by the factor **α**, the loads are termed as "*classified vertical loads*". The factor shall be one of the following:

0,75 - 0,83 - 0,91 - 1,00 - 1,10 - 1,21 1,33 - 1.46.

*It should be noted that for international lines it is recommended to take **α** =1,00. The factor **α** may be specified in the National Annex or for the individual project. For example:*
*A coefficient **α** = 1.21 is to be used for bridge structures on lines of the Federal Railways for operating trains with 25 t axle loads. The SW/2 load model does not need to be added.*

For the geotechnical structures (earthworks, retaining structures and culverts with a clear width of < 2.0 m), **α** = 1.0 may also be used for classified load models. Influences of construction machinery may have to be considered separately.

A coefficient of **α** = 1.0 must be used for the static calculation. The coefficient α can be selected according to the principles of § 8 EBO.

For pure S-Bahn traffic, **α** = 0.8 can be used.

For construction stages, all actions listed under 6.3.2 (3) must be multiplied by **α** = 1.0 if heavy traffic is excluded during the construction phase.

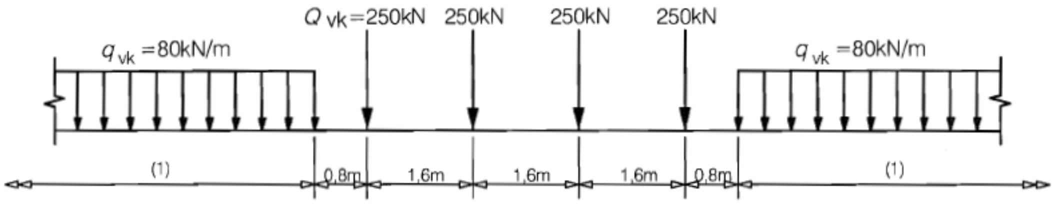


Figure 1. 10: Load Model 71 and characteristic values for vertical loads

The value of q_{vk} means the characteristic value of the vertical load (uniformly distributed load).

The value of Q_{vk} means the characteristic value of the vertical load (concentrated load).

2.11.3 Load Models SW/0 and SW/2 (6.3.3)

Load Model SW/0 represents the static effect of vertical loading for normal rail traffic on continuous beams.

Load Model SW/2 represents the static effect of vertical loading due to heavy rail traffic.

The loading system SW/0 and SW/2 shall be taken, as shown in Figure 1. 11, with the characteristic values of the vertical loads according to Table 1.1:

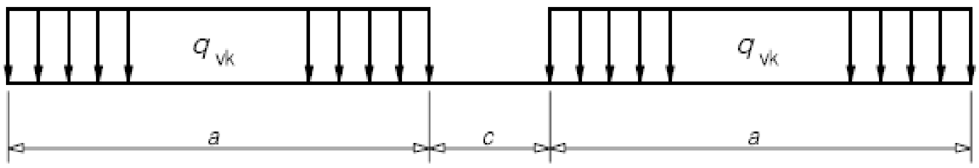


Figure 1. 11: Load Models SW/0 and SW/2

Table 1.1 Characteristic values for vertical loads for Load Models SW/0 and SW/2

Load Model	q_{vk} [kN/m]	a [m]	c [m]
SW/0	133	15.0	5.3
SW/2	150	25.0	7.0

2.11.4 Load Model "unloaded train"

For some special verifications (see EN 1990 A2, § 2.2.4(2)) a particular load model is applied, that called "unloaded train". The Load Model of "unloaded train" consists of a vertical uniformly distributed load with a characteristic value of 10,0 kN/m.

❖ EN 1990 A2, § 2.2.4 (2) describes:

"*Vertical and lateral rail traffic actions from the "unloaded train" without any dynamic factor together with wind forces for checking overall stability*."

2.11.5 The eccentricity of vertical loads (Load Models 71 and SW/0)

The impact of lateral displacement of vertical loads shall be considered by considering the ratio of wheel loads on all axles as up to 1,25:1,00 on any one track. The resulting eccentricity ***e*** is presented in Figure.

The eccentricity of vertical loads may be ignored when considering *fatigue*.

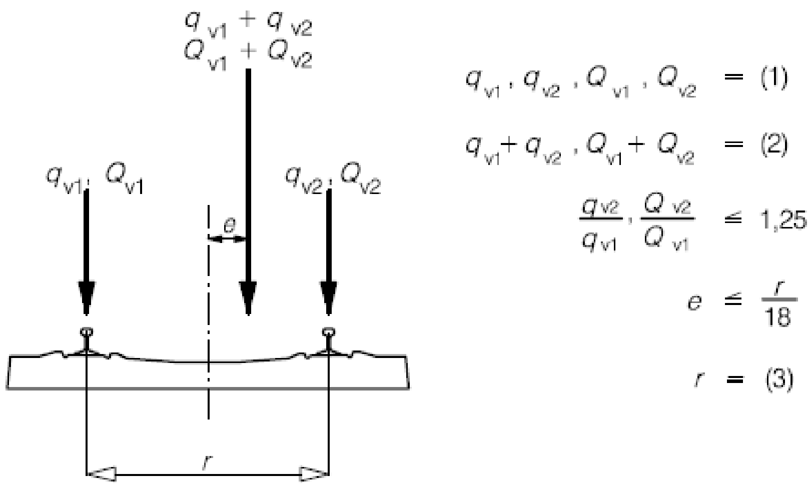


Figure 1. 12: Eccentricity of vertical loads

(1) Uniformly distributed load and point loads on each rail as appropriate
(2) LM 71 (and SW/0 where required)
(3) Transverse distance between wheel loads
qv1, qv2 Vertical load (uniformly distributed load)
Qvi Wheel load

2.11.6 Horizontal forces - characteristic values

Where the track on a bridge is curved over the whole or part of the length of the bridge, the centrifugal force and the track cannot shall be considered.

The centrifugal forces should be taken to act outwards in a horizontal direction at a height of 1,80 m above the running surface (see Figure 1. 12). For some traffic types, e.g. double-stacked containers, an increased value of h_t should be specified.

The centrifugal force shall always be combined with the vertical traffic load. The centrifugal force shall not be multiplied by the dynamic factor Φ2 or Φ3.

3 Building information modeling (BIM)

3.1 Benefits of BIM-Design and Analysis

The sustainable design of the building is carried out by beginning the iterative structural analysis from the early design stages of the project like Finite Element Methods (FEM) and Building Information Modeling (BIM) programs. The computational structural analysis helps all steps of the project and facilitates of the building to have limited dependency on the final phase of the finished structure, reducing the costs and required tests, faster in implementation, more precise in details, foresee the unexpected problems by adapting desirable design options. The assessed outcomes from the computational structural analysis (early-stage) help to recognize the optimal design and safety and thus reduce the operational cost and the construction period. These results support the design groups to select desirable plan options to decrease materials waste and final values of the building [3].

BIM models include a wide variety of information and options regarding engineering matters. But the significant benefit of the combination of FEM with BIM is to access the information required for design, planning, drawing and eliminates the need for creating the extra models in other programs [4]. There is an extensive range of advantages by using BIM-based over traditional modelling process. For example:

- BIM tools like ALLPLAN have inbuilt building orientation options to optimize the solar load and other option regarding the daylight.
- The time saving of the redrawing of the models, in the simulation and design steps [4].
- BIM-based energy analysis allows iterative simulations for a wide range of simulations to be performed within a short duration.
- The geometry and dimensions of models can be modified in the BIM easily and imported in the structural analysis programs by recreating or generating the model via the BIM program.

In BIM and structural analysis programs, the ability for data exchange between engineering programs and analysis tools will lead to more proper optimized design structures from the early stages of the design process. One of the crucial restrictions which prevent the wider adoption of BIM-based analysis is the issue with data exchange between the BIM model and the FEM programs. Therefore, finding a connection between these two, would help the structural design and also drawing matter. Besides, several aspects are influencing the seamless data exchange within the BIM model and FEM model. The quality and accuracy of the data is one significant aspect in the description of data exchange conditions; and also, reducing the additional modelling details in both software tools. Hence, the selection of correct data requirements for specific facility management and sharing of that selected data will simplify the data exchange process and eliminate redundancy in the model generation process [9].

The above-mentioned matter in data sharing is improved by the evolution of BIM metadata rules or data exchange schemas. Here, two favorite data exchange schemas will be expressed for the data exchange in the field of the AEC industry. The first one is green building eXtensible Markup Language (gbXML) and second is Industry Foundation Classes (IFC). The meta-data models IFC and gbXML have significantly improved the interoperability between BIM and BEM, by acting as a connection between BIM and BEM and FEM. These data exchange schemas have also increased the clarity of the data transfer process, due to the language is in both machine-readable and human-readable [10]. There are several mechanisms available to control, plan and confirm the data displayed in the interoperability file formats (.ifc, .xml). This reviewing and validation will help the user to improve manual data modelling errors.

3.1.1 gbXML

gbXML is supported by methods such as the U.S. Department of Energy, the National Renewable Energy Lab, Autodesk, Bentley System, and others. The primary structure of the gbXML schema is shown in Figure 3, and it describes the authority of the gbXML schema, which concludes Building, Location, Surface, Space and Opening. gbXML can work the object-based elements (for instance, Roof, Walls Floor, etc.) from the BIM means as a virtual element with 2D surfaces. This is the purpose for determining Roof, Ceilings, Floor and Walls as a surface element in the gbXML structure.

The position of surface elements is factored based on two-parent nodes. They are as follows:

- *Shell Geometry Node* that represents the inner surface of the wall, etc.
- *Space Boundary Node* that represents the centerline of the surface

Twice the difference among Space Boundary node and Shell Geometry node provides the depth of the element. And also, in the gbXML formation, Shading Surfaces and Room relate to the Building Space. As an example, in Revit Architecture program, Room is determined, and SPACES are represented in Revit MEP (for details such as Electrical, Mechanical and Plumbing). Meanwhile, the export method can be transported out based on either ROOM or SPACE; and bound volumes represent rooms. Room bounding sections define spaces. The room bounding elements include:

- Walls (Curtain, Standard)
- Roofs
- Floors,
- Ceilings
- Curtain System
- Room Separation Lines.

In all of the circumstances, surfaces which not relate to room bounding, are handled as shading components (D. Bell, 2014).

3.1.2 IFC

IFC is an open meta-data model used to control, collect, and transfer the building information linking software applications. An IFC file supports the STEP (Standard for the Exchange of Product Model Data - is also identified as ISO 10303) physical file XML exchange and file formats for sharing or transferring data. IFC defines a data schema to demonstrate about holding the data and connections between this data.

The buildingSMART purposes an open data scheme termed Industry Foundation Classes (IFC) to conduct building information. The main aim of buildingSMART is to enable the excellent feature of data sharing between multiple stakeholders during the life age of any built environment, regardless that they are used by which software [11]. That makes it possible to transfer the required data inside different software utilization. The Industry Foundation Classes (IFC) “represent an extensive (open), global and regulated term for Building Information Modelling (BIM) data, that, is replaced and shared among the several members in a building construction project [11].

Recently, the IFC4 – Addendum 2 (IFC4 ADD2) as the newest version of OpenBIM standards were announced in 2016, consists of 766 items, 185 types and 206 enumerator sorts. The data schema architecture of IFC describes four conceptual layers, and each individual schema is assigned to correctly one conceptual layer[12].

In the case of IFC schema architecture point of view, the following issues are defined:

- **Resource Layer:** which has the lowest layer in the IFC data schema architecture, which presents generally related to sources and can be utilised or referred by resources in the other layer.

- **Core Layer:** which is the next layer higher the resource layer, which involves kernel schema and the core extension schemas. The core layer presents the primary structure of the IFC and describes most abstract generic ideas that will be concentrated by higher layers of the IFC object model [13].
- **Interoperability Layer:** represents targets which more specialize the core layer objects and are shared by more than one domain model or application model.
- **Domain Layer:** provides model objects which help the applications utilised by domain specialists.

Figure 1. 13 stages the first five inherent levels of the IFC-inherent tree. The element (walls, slab, etc.) in a lower level inherits most of their properties and modelling ideas from the top levels in the IFC schema. All IFC components found from the IfcRoot entity. Elements defined in the IFC-resource layer do not found from the IfcRoot entity.

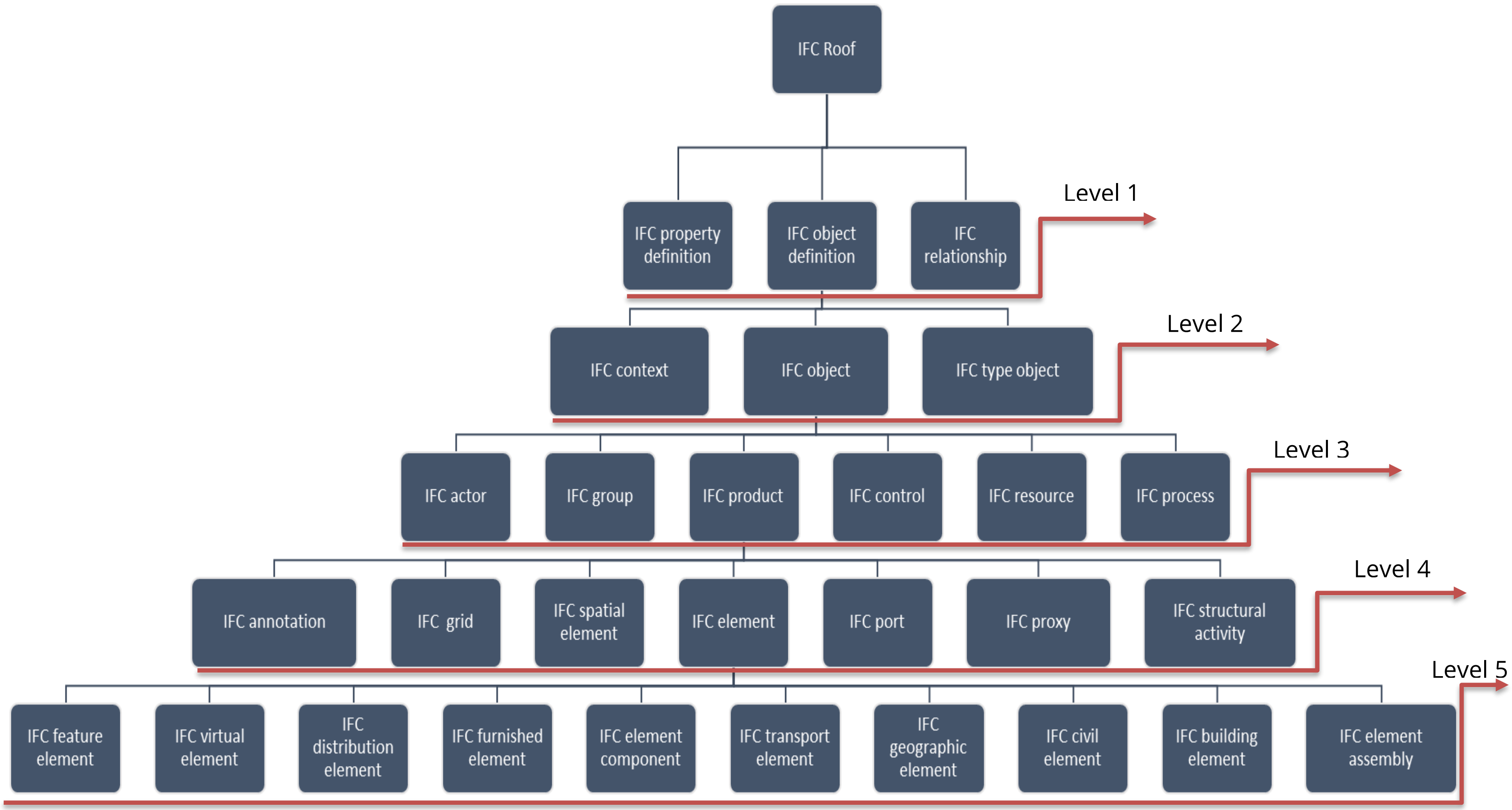


Figure 1. 13 First five inherent levels in IFC data schema architecture

Here should be reminded again that objects (e.g. wall) in IFC schema typically doesn't include complete information about for example geometry, attributes, material, etc. This sort of data saved with the various attributes presented in the IFC data schema. These attributes are connected to the object through objectified connections. It means connections are utilised to link the complete data regarding objects which are stored in the other attributes or objects. These relationships can be combined and detached over the life-cycle as per the specifications. There are six significant connections in the IFC data schema architecture as Figure 1. 14 shows:

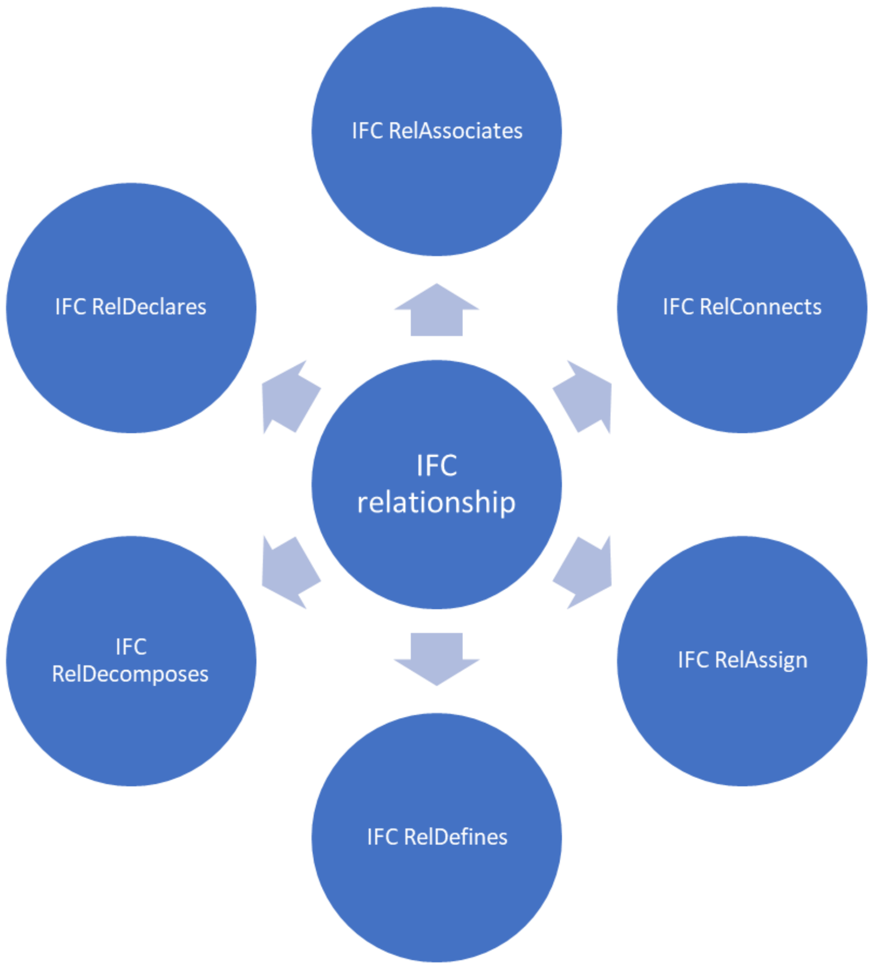


Figure 1. 14 Major Relationships in the IFC data Schema [11]

In this step, the purpose of each connections (Relationships) will be described one by one:

- IfcRelAssign: the connection that enables one object to navigate to different objects, or to model that a user object utilizes the services of a supplier object.
- IfcRelAssociates: the connection that leads to the source of information.

- IfcRelConnects: the relationship that relates objects under some rules. Constraints and semantics of determined by sub-types of this connection.
- IfcRelDecomposes: connection utilised to navigate from the all to the elements and vice versa; all elements depends on the description of the parts and the parts rely on the existence of the whole.
- IfcRelDeclares: Declaration of objects or features to the project or project library.
- IfcRelDefines: utilised for further description of object occurrences, object examples, property sets, and the connection between each other.

In this research, since more focus is on IFC format; more details will be presents as follows.

I. Spatial Structural Elements:

The decomposition of the spatial structural is the fundamental method to describe all subsets (components) in the project model based on the spatial arrangement. This method of decomposition of components is very usual to the many methods and design tasks for data exchange. This spatial structure (IfcSpatialStructureElelement) concludes of four subsets. They are, for example, Site, Building, Building Story, Space [12]. These subsets represent the levels of the structure. All these spatial structure components are connected by using objectified connection IfcRelAggregates. As related to the Site, Building, and Building Story, the building Space is expressed as IfcSpace in the IFC data schema, is utilised to present the area and volume of the room bounded by components. The IfcSpace defines the properties to provide additional information about the building spaces, such as follows:

• **Name:** holds the unique name to space.

• **LongName:** the full name of the space.

• **ObjectType:** holds the space type.

***Space boundary**: space boundary denotes the physical or virtual separators for space. It illustrates the relationship between space and its bounding elements such as floor, wall, roof, door and window. The objectified connection IfcRelSpaceBounding is utilised to connect the bounding components to space.*

II. Concept of Zone:

IFC schema describes the zone as IfcZone, and it is a subtype of IfcGroup. Zone describes the individual or set of spaces in the building considered together to provide engineering aids. For instance, lighting zones, HVAC zones, Heavy plug load zone, etc. The zone name is determined by applying the property called LongName (represents the objectified relationship IfcRelServicesBuilding to link the zone and space).

III. Building Element:

All the physical components (objects) of the building, such as a window, wall, door, roof, etc. are accepted as subsets of the IfcBuildingElement entity in the IFC data schema. IfcBuildingElementProxy is the unique subtype, which is not relating to the building objects list, but it presents the same functionality as subtypes of IfcBuildingElement, and it is utilised to determine the particular sort of the building component which is not considered in the subtypes list of the IfcBuildingElment. Each subtype further holds with different properties, and each of the subtypes inherits the attributes and relationships from the IfcElement entity. The building elements define spaces in the building. And also, the relationship is used to identify what are the building components linked to a particular space in the building [12].

IV.Opening and Filling:

IfcOpeningElement is a subset of IfcElement, which forms an opening or void within a different element. It means the opening is represented as an element in the IFC schema and which is arranged or related to various items such as a slab, floor, wall, roof, etc. in the building. The three characteristics of the opening element are,

- Determines the component which forms the voids.
- Describes the component which fills the void.
- Openings representation.

IfcRelVoidsElement shows the relationship between the opening component and the component, which is voiding. Now an opening component is located in the component which requires to be voided. This opening may be closed by another component, such as a window or door. In this way, the connection, IfcRelFillsElement is utilised to link the opening element and the element which is going to fill the opening [12].

V.Geometry and Placement:

The IFC data schema describes for the building object geometry, placement and their relationship as well. The geometric description and the object placement are determined at IfcProduct level, so the similar system is equally concerned for each subtypes. Accordingly, a unique idea of the geometric description and placement is utilised throughout the whole IFC schema and is the same for the IfcWall, IfcFurniture, IfcSpace, IfcFlowSegment, IfcProxy, IfcOpeningElement etc.

The placement of an object is referred to as IfcObjectPlacement in the IFC data scheme. This placement is carried out either by absolute or relative or constrained coordinate systems.

The object placement represents the object coordinate system as either two-dimensional axis placement (IfcAxis2Placement2D) or three-dimensional axis placement (IfcAxis2Placement3D). The fundamental way of describing the object placement based on relative coordinates systems is by utilising IfcLocalPlacement. The IfcGridPlacement illustrates the placement of an object corresponding to the grid intersections [12].

VI. Properties of Elements:

IFC schema represents most of the information regarding an object in terms of properties and the relationships between characteristics. There are many sorts of information that a user might need to exchange that are not currently involved within the entity. For this goal, IFC formed a mechanism called Property definition, which allows defining expandable properties with objects. Here are two different ways of defining features for an object are represented. One is according to the type of object, and another one is based on the object. But both by applying objectified connections. The typical method of determining the features is based on the object type, but for the object whose object types are not available, features are described based on relationships.

In this methodology, the objectified relationship (IfcRelDefinesByType) is used to link the object with its object type, which is defined by a set of features. The set of standard values defined in the IfcPropertySet is shared across multiple instances of that class. The IfcProperty subsets allow for the representation of different property values within the private copy of the IfcPropertySet for each example of the class. The IfcPropertySet class includes all sets of properties. It must contain at least one property and may consist of as several as required.

VII. Properties of Elements:

IFC schema describes the greatest of the information regarding an object in terms of properties and the connections between characteristics. There are many kinds of information that a user might require to exchange that are not currently involved within the entity. For this goal, IFC formed a mechanism called Property definition, which allows defining expandable properties with objects.

Here two various ways of determining features for an object are described. One is according to the type of object, and another one is based on the object. But both by applying objectified connections. The usual system of determining the characteristics is based on the object type, but for the object whose object types are not available, features are defined based on relationships.

3.2 Integration between BIM and FEM application

The famous market previously offers much software applications are providing the association of BIM and FEA. Many companies are experienced in interoperability of BIM and FEM by transferring .DWG and .DXF formats driving to transfer the geometrical objects only. The cooperation of BIM and FEA is achieved by various software that generally causes loss of some data, depending on geometric complexity. This method does not usually provide the exchange of material characteristics and boundary conditions. The value of full interoperability among BIM and FEM consists of smart data transfer, where specific simplifications are made, and the interaction can be delivered to both directions. That indicates that the BIM model transfers digital data toward the FEM, regarding all necessary parts demanded the study, and the BIM is updated at the same time as FEM is performed [14]. The method of integration of FEM and BIM can be shown as in Figure 1. 15.

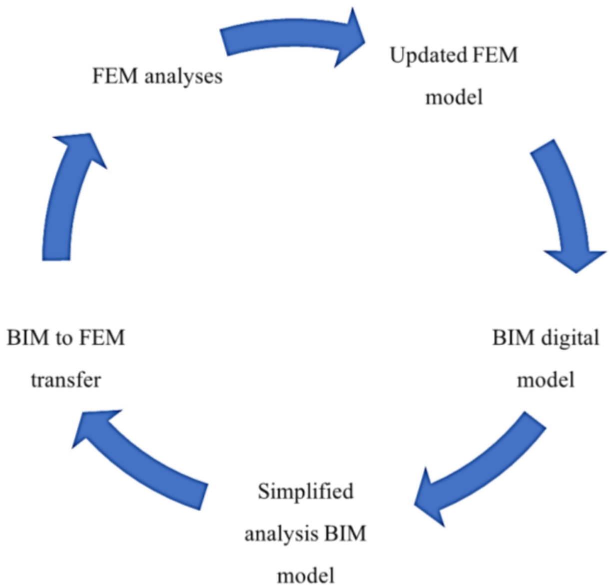


Figure 1. 15 BIM and FEM integration

The current phase of the integration of FEM and BIM consists of designing a model in BIM, transporting the model into the FEM, numerical simulation and evaluation of the acquired outcomes. In the case that the results are accepted, the model is not transferred back to the BIM, but the accepted as such. Unless the model is changed until the aim is reached and then transferred back to BIM. The challenges arise especially if detailed and special analysis, such as, seismic, thermodynamic, airflow, and other frequently dynamic analyses are required. The specified analyses are normally time-consuming. Therefore, certain simplifications need to be applied. Structural features, for instance, bridge facilities are eliminated from the FEM model to lessen its size as far as the removed components don't markedly influence the solution. From this point of view, the FEM should still be regarded as a unique method requiring detailed expert attention. In several cases, the BIM software enables certain finite element method that can be used in the BIM straightly [14, 15].

Besides, FEM experts prefer technoscientific multi-physics FEM software where more options of special simulations can be achieved. This reduces the BIM process considering challenges that may happen in transferring the digital model. If the simulation is presented within the BIM software, the integration is usually excellent and the possibility of happened errors is minimized. However, superstructures such as towers, bridges, skyscrapers, etc. considerably often represent constructions which are significantly influenced by dynamic forces, i.e. wind and earthquakes. These kinds of analyses are properly simulated in multi-physics software in feature, because, the exact numerical model definition, solution method and obtaining outcomes are likely more accurate and controlled [14, 15].

The FEM is commonly described by three methods: solution, preprocessing and post-processing. The FEM applications already allow parameterization of the model, which presents scope to mathematical techniques that require re-definition in the preprocessor based on outcomes achieved in the post-processing, like optimization. So, the optimization or other loop method used in the FEM perform their own part of the design. Then, the example in Figure 1 can be updated as follows Figure 1. 16.

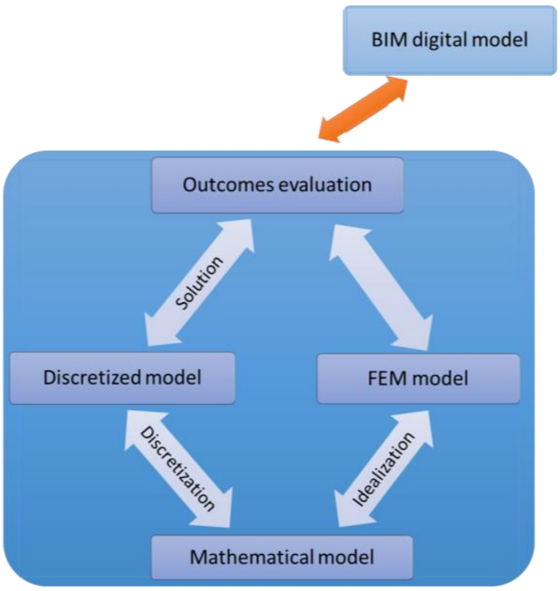


Figure 1. 16 loop computing procedure between BIM and FEM

Standards guide designers through parts of BIM that have developed by the Building_Smart and are continuously updated. Guidelines defining the conditions for BIM in new building designs, improvement and administration facility were published in Common BIM essentials 2012 [16]. COBIM 2012 cover, among others, the conditions for architectural design, structural purpose, administration of a BIM project, use of models in facility management, use of models in construction, etc. BuildingSMART has expected usage of BIM in building engineering sector about 20-30% in building plans. It advises 20% use in the public section, fewer than 10% in private section, 40-50% in huge construction firms, fewer than 10% in small construction companies and nearby 70% among AEC's. The lack of BIM application is mainly caused by private companies that have not found their profits yet. The discouragement is made by high primary costs where the adaptation of structural design needs further education of designers, constructors, financials and clients about BIM. The BIM in bridge purposes is also extensively discussed and investigated matter [15].

The most bridge designs still favour traditional techniques (Johansen J. BIM) [14], although pilot projects of BIM in bridge designs applications have confirmed that the advantages of the 3D visualization and close collaboration of all affected parties. Next to the continuously growing patterns and models for BIM application in building construction, a guide applicable to bridges has been recently produced .

BIM Guidelines Applicable to Bridges produces instructions on uniform methods for the BIM-based design, implementation and sustaining of bridges. The guideline covers the information of how to proceed through the whole method and lifecycle of BIM-based design of bridges.

3.2.1 BIM and FEM integration in example of bridge

The successful integration of BIM and FEM consist in the right connection connecting the applications to control the performance of the design and accurate data transfer. Particular design field explained in a focus of the BIM and FEM interoperability is a bridge design. The bridges are, in most examples, the super structure of individual shapes and characteristics, where the main roles play dynamic loading and infrastructural design. Interaction among BIM and FEM can be done by direct or indirect link allowing access to the information required to analyze the problem and promote all changes in the BIM method. The direct link is used within a software or from one to different via Application Programming Interface (API). The open API is an interface that allows interaction of two software applications to transfer data and communicate. The interoperability of BIM and FEM in commercially available applications has confirmed successful in both direct and indirect linking of simple structures, but in the case of complicated structures the outcomes are yet hardly satisfying. The direct link allows data transfer of loads, geometry, material properties and load groups. Indirect link (IFC) presents geometry and material properties data transfer only.

In different circumstances, the exterior data transfer through IFC is satisfying for a large-scale of designs, as the direct link requires a more improvement [14, 15]. Also, other techniques, such as hybrid method of BIM and FEM application has already been studied in scattering properties of a dielectric targets above a dielectric rough surface [17]. Difficulties in BIM and FEM interoperability in bridge design may occur in the case of complex geometry, where some geometry entities could be of irregular shapes. The accuracy of the FEM depends on a number of formed components. Therefore, if a proper amount of finite elements doesn't display the numerical model, the obtained results may affect practical design of the entire structures. Besides, the analyses done within the numerical model representing the real structure, some analyses need also a model of the surrounding space [14, 18].

3.3 Illustration of data sharing between ALL PLAN 2019 and InfoGraph

In this part, the data sharing between a BIM tool (ALL PLAN) and a Finite Element Method (FEM) program for analysis of structure (InfoGraph) is described that how to use IFC & gbXML as an interoperable file format. This demonstration intends to investigate the following:

- Level of data sharing, such as quality of data, is going to transfer between BIM and FEM.
- Confirmation or implementation of meta-model data schemas in a BIM tool (ALL PLAN) in case of data export.
- Ability or flexibility of the BIM tool to import the data from gbXML and IFC file formats, and also the ability of the FEM program to receive the correct data from that.
- Investigate and recognizing and checking modelling failures in the BIM model by using the FEM program (InfoGraph) features.

The BIM model is created utilising ALL PLAN 2019, and the similar data model is imported to InfoGraph tool by using gbXML or IFC data schemas.

The subsequent actions are carried out to discuss the researches mentioned above:

- A description of data transfer methodology.
- Guidelines for the successful implementation of data transfer to InfoGraph.

- A complete description of modelled project data in ALL PLAN 2019.
- Investigation of data in InfoGraph, which is imported from gbXML or IFC [12].

3.3.1 Analysis Methodology

The illustration of data sharing is carried out based on two interoperability file format IFC. Figure 1. 17, describes the methodology for data sharing between All PLAN 2019 and InfoGraph. For data sharing presentation, complex parts of the bridge simulation model are chosen, such as foundation, abutment, deck and wing wall.

At first, the BIM model is designed in All PLAN 2019 and also is combined with all required information to set-up a complex type of FEM simulation model. The complete BIM model is exported from All PLAN 2019 to the gbXML and IFC file formats (meta-data model schemas). These interoperability schemas operate as a neutral file format for the modelled BIM data and provide access for FEM software applications to import these data such as InfoGraph.

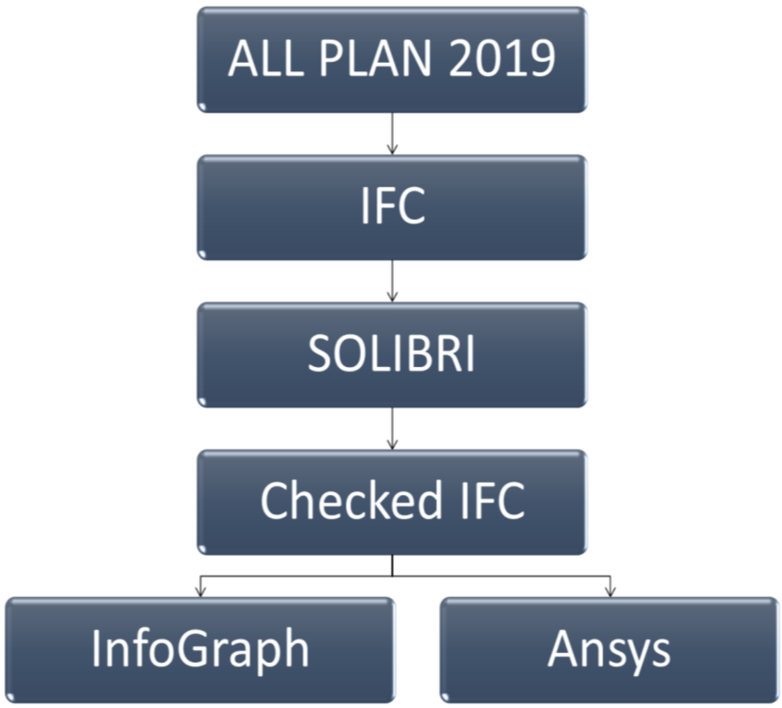


Figure 1. 17 Methodology for the data sharing linking BIM and FEM programs

The IFC and gbXML files are imported by the model view checker called Aragog gbXML viewer, and similarly, the IFC file is imported by Solibri Model Viewer. The idea of the model viewer is to know whether the model data has been exported to interoperability file formats accurately. These viewer mechanisms describe the BIM model as a virtual model or analytical model, which is the same design applied in the end software like InfoGraph. This assists the user to improve the modelling faults in the primary stage of the data sharing process.

These model viewer tools provide information regarding missing entities, issues and reports. This information is helpful to adjust the BIM model based on the model viewer representation to improve faults. Solibri enables the user to check and realize how the IFC schema reads the modelled data and how it is designed. This understanding will support the modeler to create the information in a convenient form for successful export. It should be noted that respective organizational bodies should authorize the model viewer tools.

After completing the confirmations and corrections, the confirmed IFC or gbXML file are imported by InfoGraph for FEM structural simulation. Additional confirmations about the data property are performed in InfoGraph.

3.3.2 View and check the IFC file by Solibri

In this part, the IFC file is imported to the Solibri model viewer program to check the IFC file properties, then after, the check file will be prepared and imported to the FEM programs.

The IFC file was imported in Solibri program correctly. In first glance, the 3D view of the model was presented in the program precisely. The whole elements of the bridge showed entirely. The elements like Slab, abutment and the others were defined in the Solibri program as a separate volume element individually. Please see Figure 1. 18.

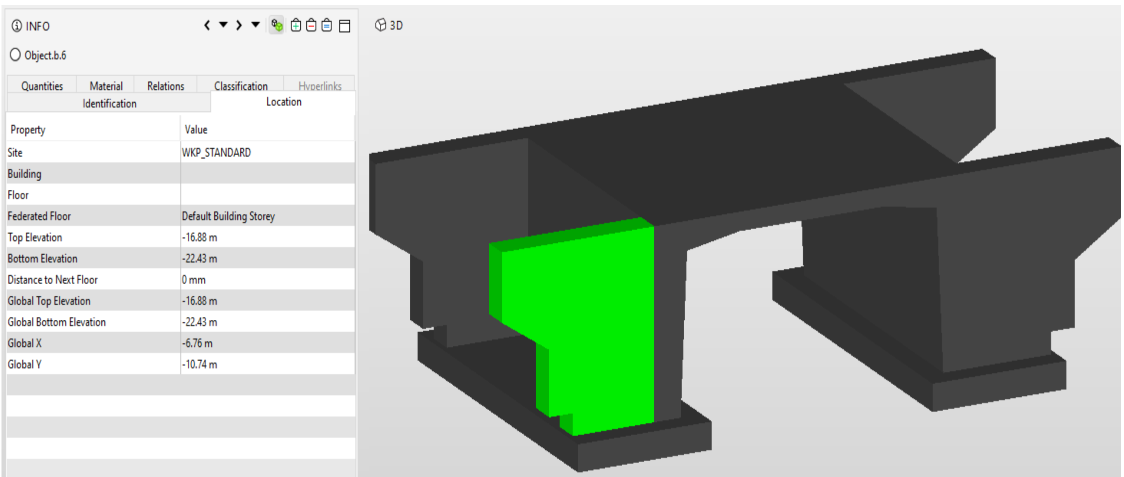


Figure 1. 18 3D model by Solibri model viewer

The Solibri program also presented and described some more details about the model, such as location and quantities of each element one by one. For instance, the wing wall parameters like area, volume and the other features were reported and presented in *Check* tab, part *INFO*. Please see Figure 1. 19. Via the given information by Solibri can be understood that the amount of details and information about the model depends on how the model was designed and introduced in ALL PLAN 2019. For example, there was no information about the materials in Solibri, since no data has been defined in ALL PLAN 2019 about the materials.

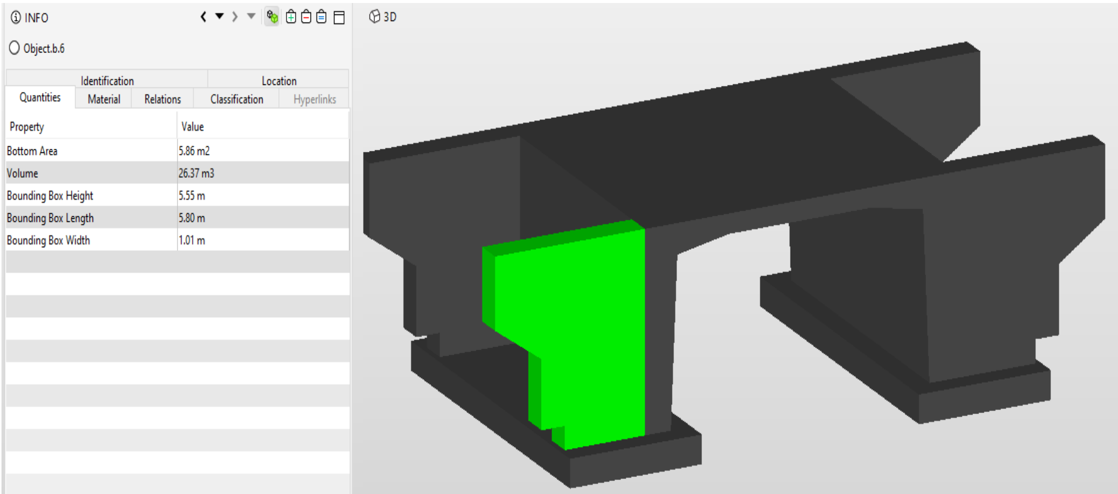


Figure 1. 19 Wing wall details via Solibri

In Solibri program also the relation between the elements was shown by the MODEL TREE. Please see Figure 1. 20.

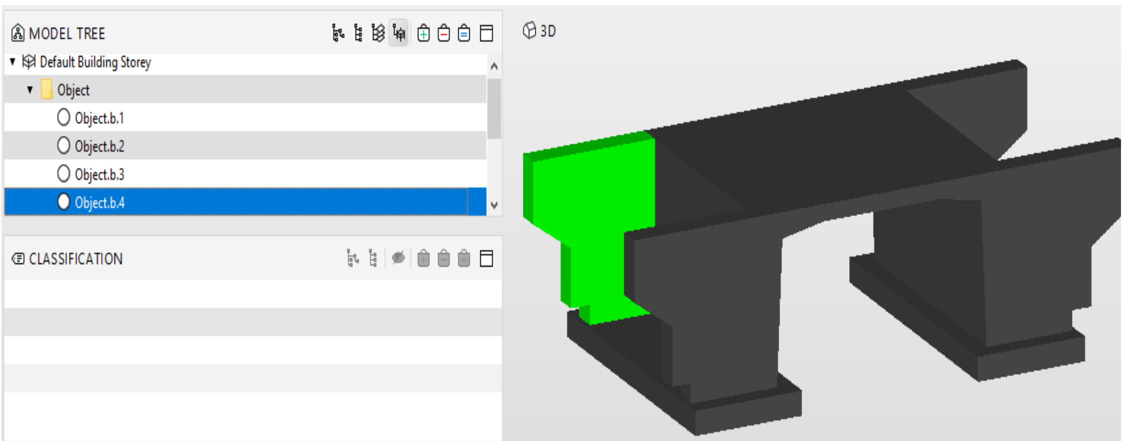


Figure 1. 20 Wing wall MODEL TREE

3.3.3 Ifc Structural Analysis Model

The Ifc Structural Analysis Model entity is used to represent the information needed in a structural model. General properties such as dimensionality or the position of the coordinate system in the attributes of the entity being declared. The specified values are valid for all elements within the Ifc Structural Analysis Model [19]. The specific characteristics of the class are listed in fig below:

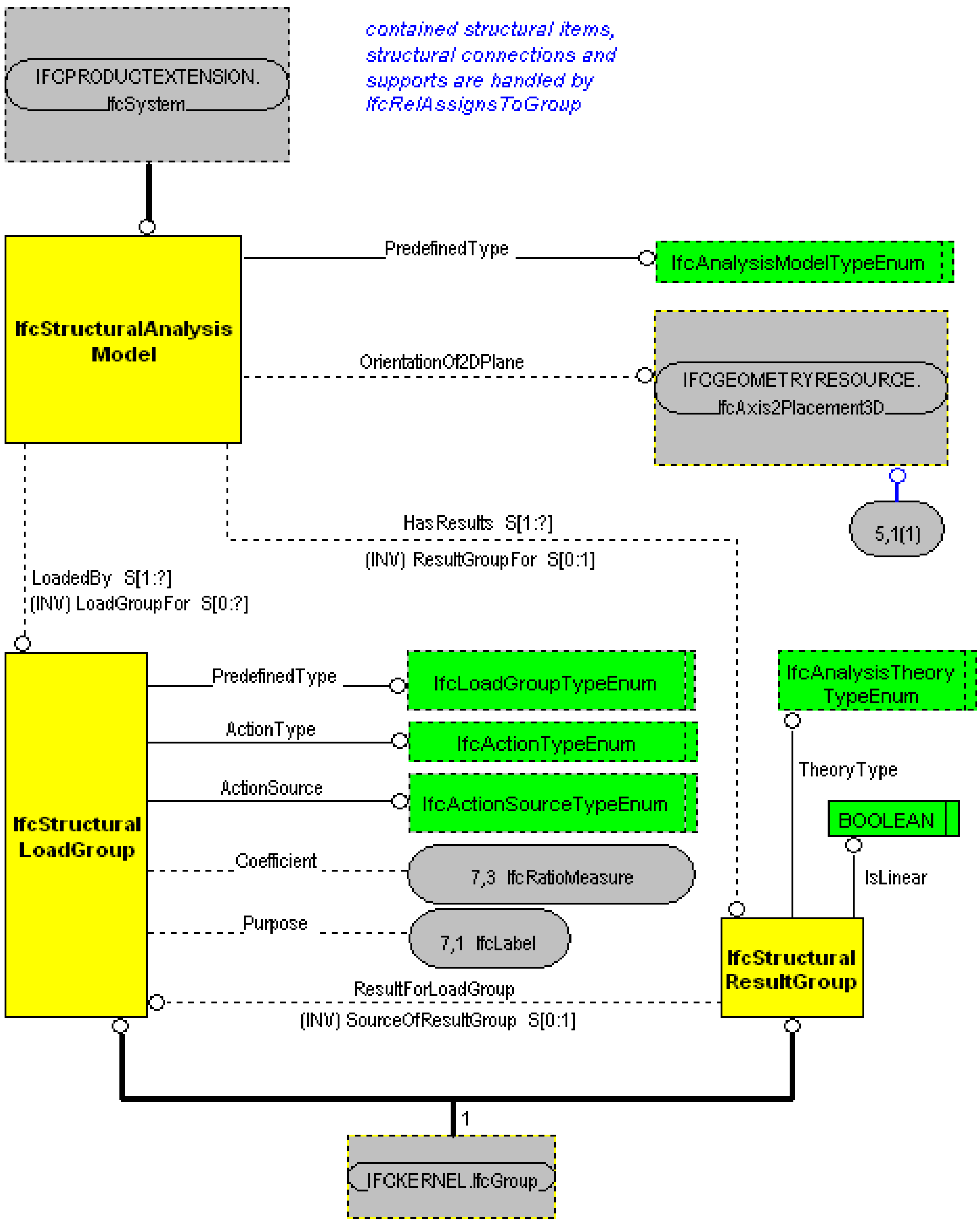


Figure 1. 21 Diagram of IfcStructuralAnalysisModel [19]

The attributes of Ifc Structural Analysis Model have the following functions:

- **PredefinedType:** Defines the dimensionality of the used structural model (2D or 3D model) via the IfcAnalysisModelTypeEnum entity.
- **HasResults:** Contains a list of entities of the class IfcStructuralResultGroup. Within this entity, resulting forces from structural calculations and support forces are combined [20].
- **OrientationOf2D Plane:** This attribute can be used to declare a coordinate system using IfcAxis2Placement3D that differs from the coordinate system of the IFC project. The assignment is optional, and in case of a zero value, the system of the IFC project is used.
- **LoadedBy:** Contains a list of entities of the class IfcStructuralLoadGroup. This entity combines physical actions in a group.

Via the inverse relation to IfcRelServicesBuildings several entities of IfcStructuralAnalysisModel can be assigned to a building model (IfcBuilding).

3.3.4 IfcStructuralItem

IfcStructuralItem is the superclass of IfcStructuralConnection and IfcStructuralMember and includes all structural elements, as well as the supports and connections (e.g. joints or clamping).

All static actions on the entity are declared by the inverse relationship AssignedStructuralActivity using IfcRelConnectsStructuralActivity [19].

The geometry of the instances of IfcStructuralItem is determined by the entity of the superclass IfcTopologyRepresentation, a subclass of IfcProductRepresentation. Besides, the position is also declared [20].

3.3.5 Ifc Structural Member

IfcStructuralMember is a subclass of IfcStructuralItem. Its instances represent the idealized structural behavior of building elements [19].

Entities of type IfcStructuralMember can have the following properties:

- A description of the material. This is defined by IfcStructuralMember (IfcRelAssociatesMaterial) IfcMaterial.
- A description of the profile. This is defined via IfcStructuralMember (IfcRelAssociatesProfileProperties)IfcProfileProperties.
- A Structural Analysis Model to which the entity is assigned. This is defined by IfcStructuralMember (IfcRelAssignsToGroup) IfcStructuralAnalysisModel.

The following Figure 1. 22 shows the corresponding links:

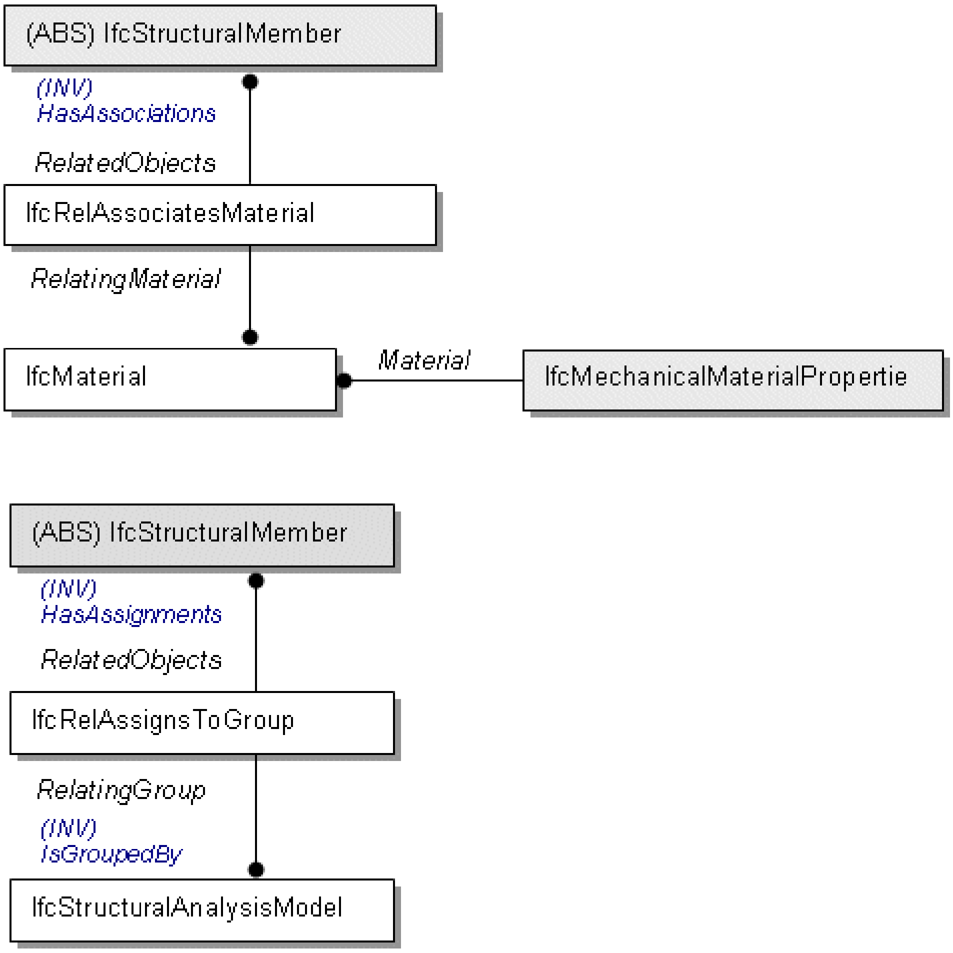


Figure 1. 22 Relations to IfcStructuralMember [19]

With the inverse attribute ConnectedBy, a list of entities of the class IfcRelConnectsStructuralMember can be used to enumerate all associated connecting elements of the component. In addition, the idealized component can be linked to the corresponding part of the IFC building model. This is done by the inverse attribute ReferencesElement, which also contains a list of IfcRelConnectsStructuralMembers [19].

IfcStructuralMember is an abstract superclass that is not applied in an IFC file. Instead, instances of the IfcStructuralCurveMember and IfcStructuralSurfaceMember subclasses are formed. These allow further differentiation with respect to the geometry of the section.

3.3.6 IfcStructuralAction and IfcStructuralReaction

IfcStructuralAction represents a mechanical activity (e.g. single force) that acts on an instance of the IfcStructuralItem entity (components or connections). IfcStructuralReaction, on the other hand, declares all reaction forces or deformations. The following figure shows the relationship between the two entity types [19].

The Figure 1. 23 shows two structural analysis models. The support forces generated in the upper model are defined as IfcStructuralReaction and are single forces acting in the lower model. These are therefore modelled as IfcStructuralAction. The relationship between the two types, which in principle represent the same force, is established using the CausedBy attribute [20].

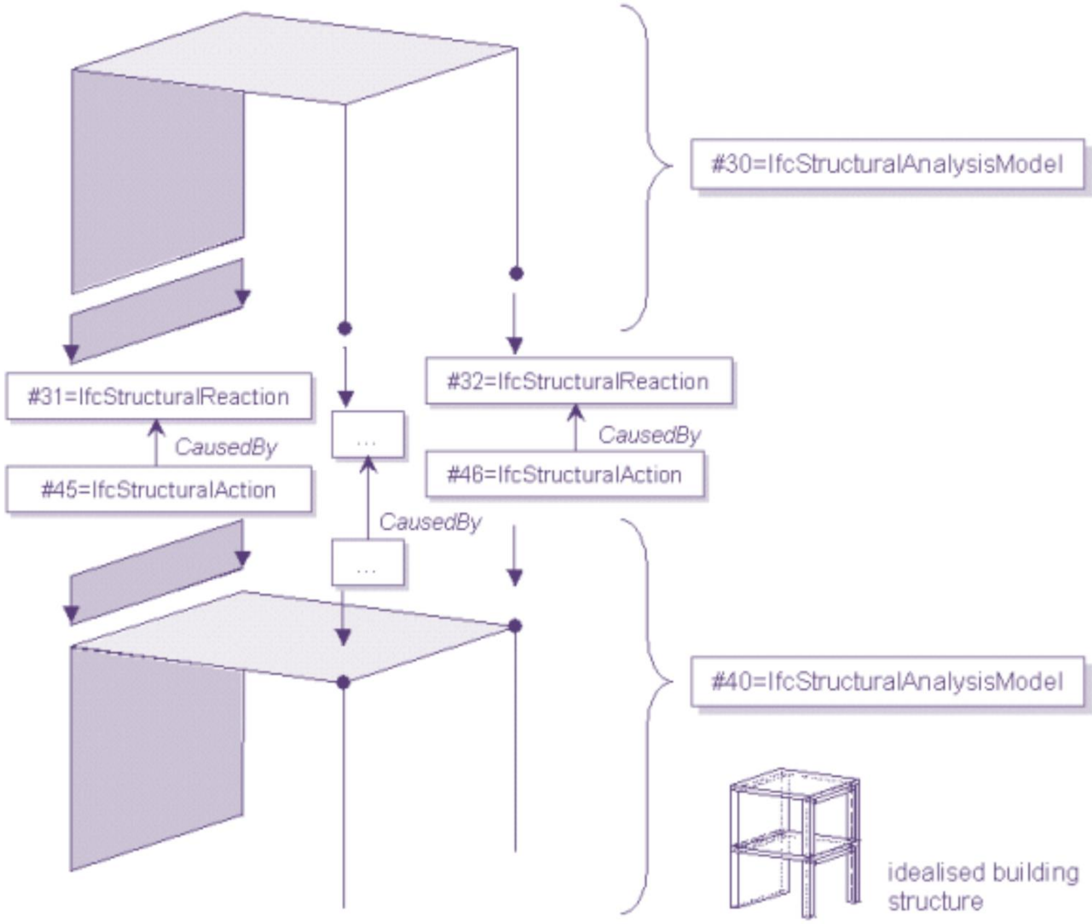


Figure 1. 23 IfcStructuralAction and IfcStructuralReaction interacting [19]

3.3.7 IfcStructuralLoadGroup and IfcStructuralResultGroup

The IfcStructuralLoadGroup entity is used to structure the mechanical actions within the Structural Analysis Model [19]. The attributes inherited from IfcGroup allow instances of IfcStructuralAction (or its subclasses) and IfcStructuralLoadGroup to represent groups containing load cases, load coefficients (such as safety factors), and load combinations. The following example (Figure 1. 24) shows a two-dimensional structural analysis model that is loaded by different force actions of the same type (dead load) and therefore, safety factor with the value of 1.35 [20].

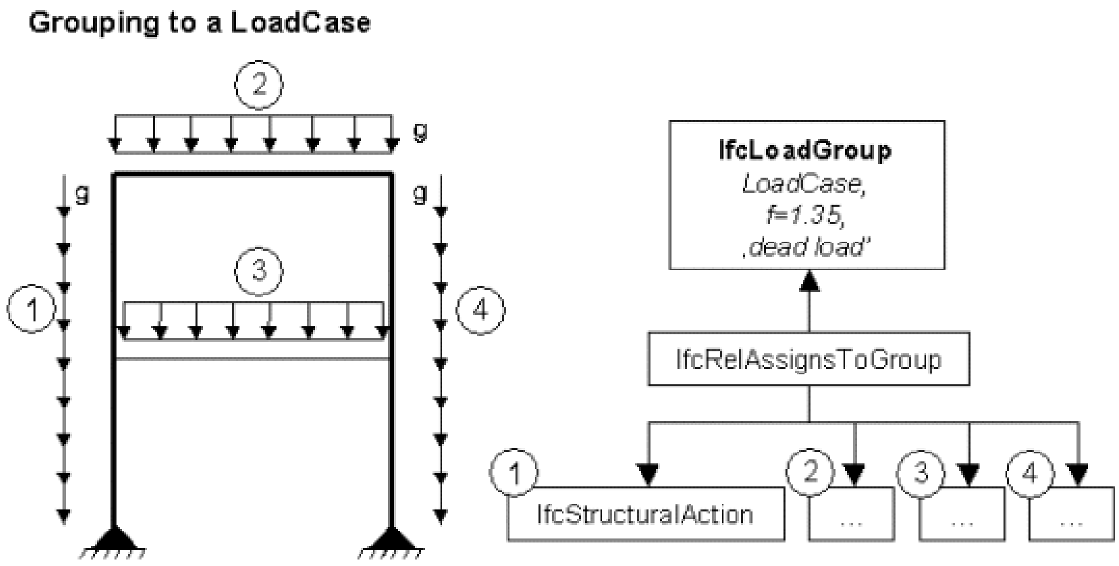


Figure 1. 24 The group in IfcStructuralLoadCase without load combination [19, 20]

The IfcRelAssignsToGroup entity can be used to assign mechanical activities to a group. This is especially useful when modelling load combinations, as shown in Figure 1. 25 [19, 20]. Also, like the IfcStructuralLoadGroup, IfcStructuralReaction entities can be grouped. This is done with the IfcStructuralResultGroup entity.

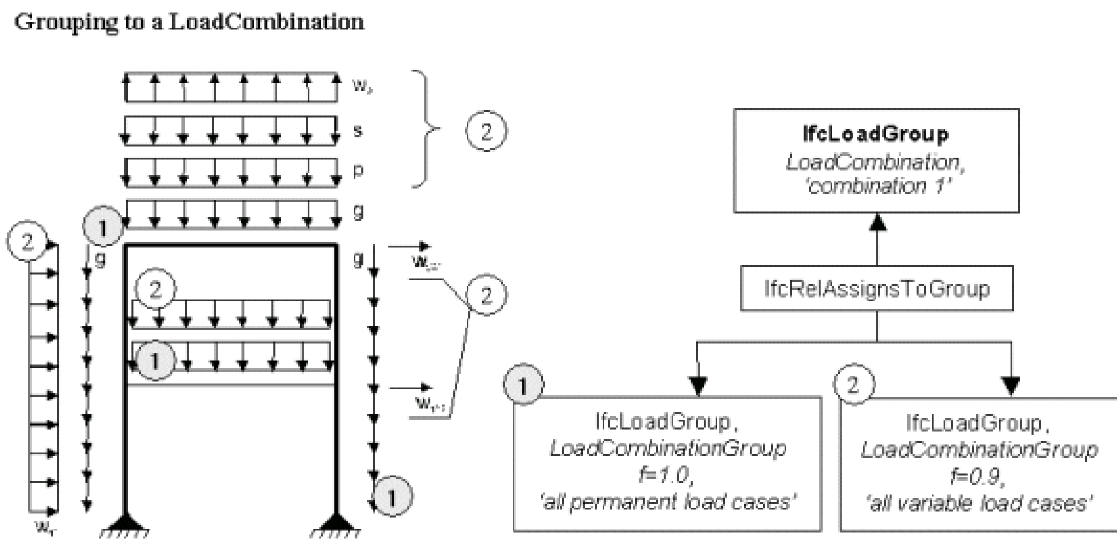


Figure 1. 25 Example of a group in IfcStructuralLoadCase without load combination [19]

3.4 Checklist of the target structure for data sharing between ALL-PLAN 2019 and InfoGraph

In this step, it should be noted that the project described and elaborated by detail:

The structure is a railway half-frame concrete bridge, located in Dresden, Germany. This bridge will be for trains uses only. The bridge has two lanes (round trip), and it will be constructed by reinforced concrete. In this research, the target bridge will be designed by three different concrete characteristics (C30/37, C45/55 and C60/75), and also will be analyzed by three different deck thickness (45 cm, 55 cm and 80 cm). Its dimensions of the bridge would be 12 m Longitudinal Length,10.74 m Width and 5.5 m Deck Height from the ground. The wing wall length is 5.8 m in the top, and 3.1 m in the foundation. The abutment has a length the same as the whole bridge length (10.74 m). More details can be seen by Figure 1. 27.

The methodology in this part of the research is to design the bridge and test and check the characteristics of the modelled bridge via ALL PLAN 2019 into InfoGraph program in fields of revision of visual, dimensional and measurements of the model. The bridge Slab, abutments, wing walls and foundation, must be defined for all areas of the structure in the BIM model to create a fully enclosed bridge. In the BIM model, the bridge will be designed in 3D form and elaborated with volume elements.

The target bridge calculations in ALL PLAN 2019 before exporting to InfoGraph should be completed, and the same should be verified with the InfoGraph model after export. This comparison will help to understand whether InfoGraph is designing the systems for accurate data or overwhelmed data.

3.4.1 Design procedure in ALL PLAN 2019:

The structure was defined in *Building Structure* part with a different section such as foundation, abutment and slab. Then, according to the available dimensions, the bridge was designed with the *3D line* to create a 3D dimension structure. After, by *Extrude* tab, the structure was volumized to have more functional properties in FEM program also.

Here should be noted that in this step, it was not possible to define the reinforcement or load case in ALL PLAN 2019, because there was no data about the amount of reinforcements. Figure 1. 26 illustrates more details about the model. In Figure 1. 27, the labels and the dimensions of the bridge were presented. As well, the bridge is *symmetrized*, then, the dimensions of Right side are as like as the Left side and also, they are based on the real planed bridge.

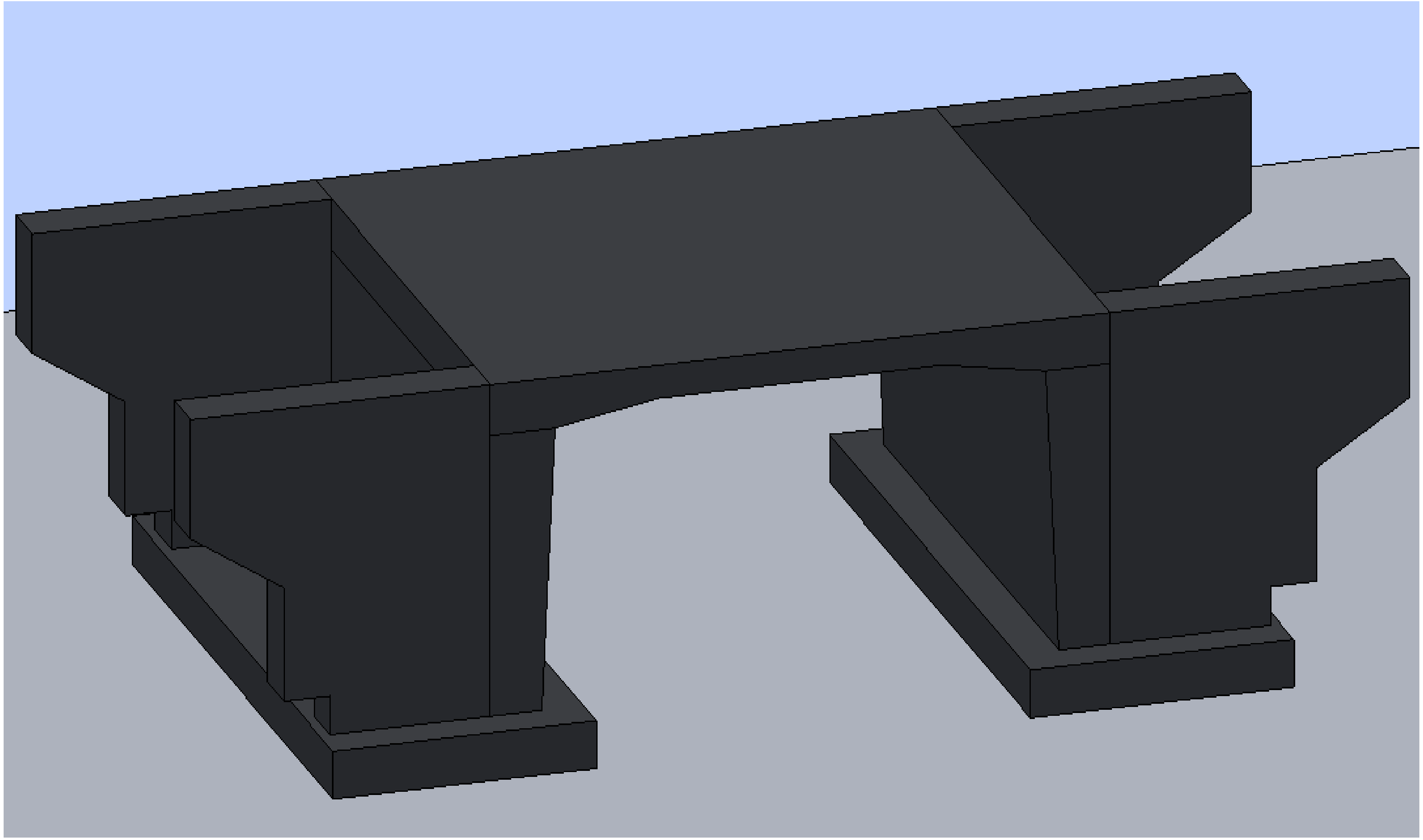


Figure 1. 26 3D view of bridge

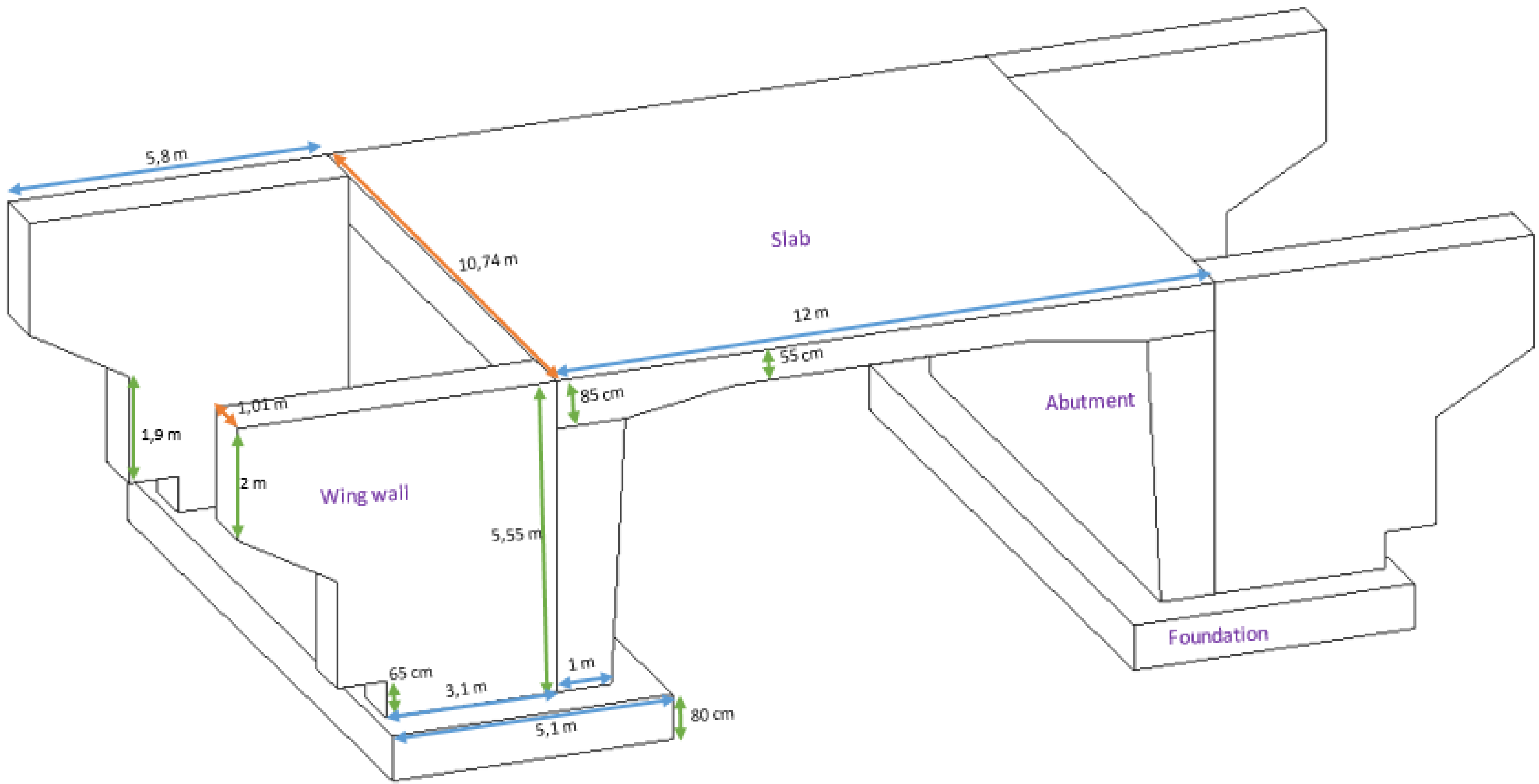


Figure 1. 27 Dimentions and labels

After completing the model, it was exported as IFC and format to be imported in FEM programs.

3.4.2 The imported model's characteristics in InfoGraph program

First, the IFC model was imported to the FEM program easily via *import command*. The model was displayed perfectly in InfoGraph, so that, the whole model was presented by details precisely. For example, lines, points, dimensions and other shapes of the bridge were exactly as like as the model in ALL PLAN 2019. For better comparison, please the Figure 1. 28.

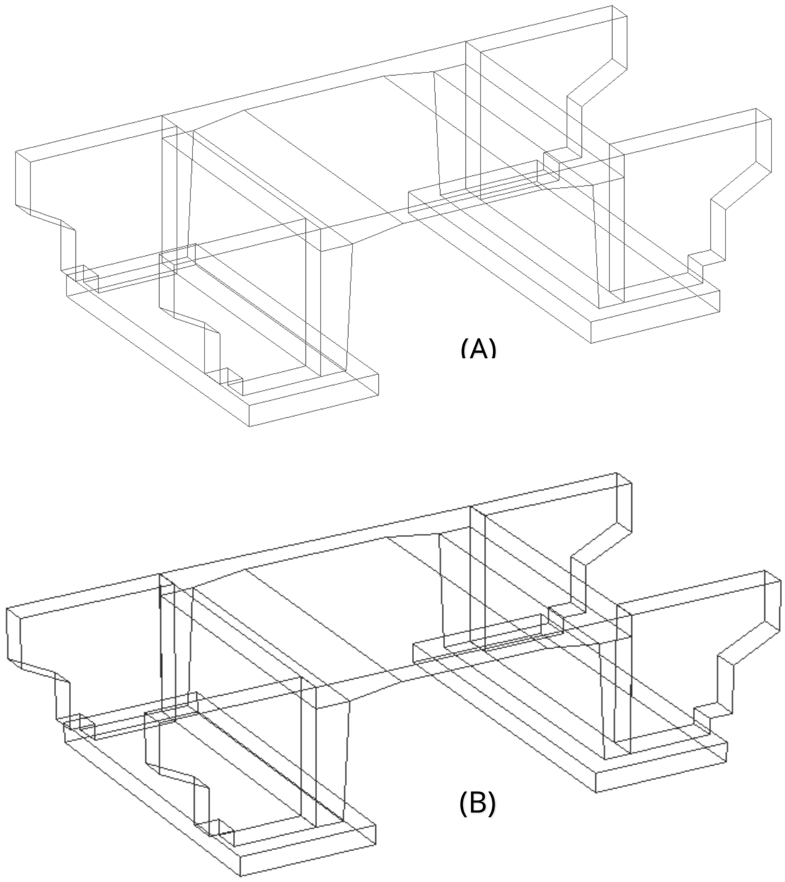


Figure 1. 28 comparison of model between ALL PLAN 2019 and InfoGraph
a) Imported in InfoGraph
b) Modeled in ALL PLAN 2019

After importing the model in FEM, the features, the commands and the parameters of the FEM program were examined. In this step, these parameters such as measurements, mesh system, local element coordinate system, volume element, load case are explained.

➢ **Measurements**:

As mentioned before and presented by Figure 1. 27, the bridge with predefined sizes, has the same measures in InfoGraph program also. It means both programs have the same definition in case of measurement. For instance, the wing wall size was 5.8 m in ALL PLAN 2019 and the in InfoGraph. Please see the fig. This function could be handy for the designers and also the bridge analyzer. Because there are no unnecessary functions for changing the units in programs.

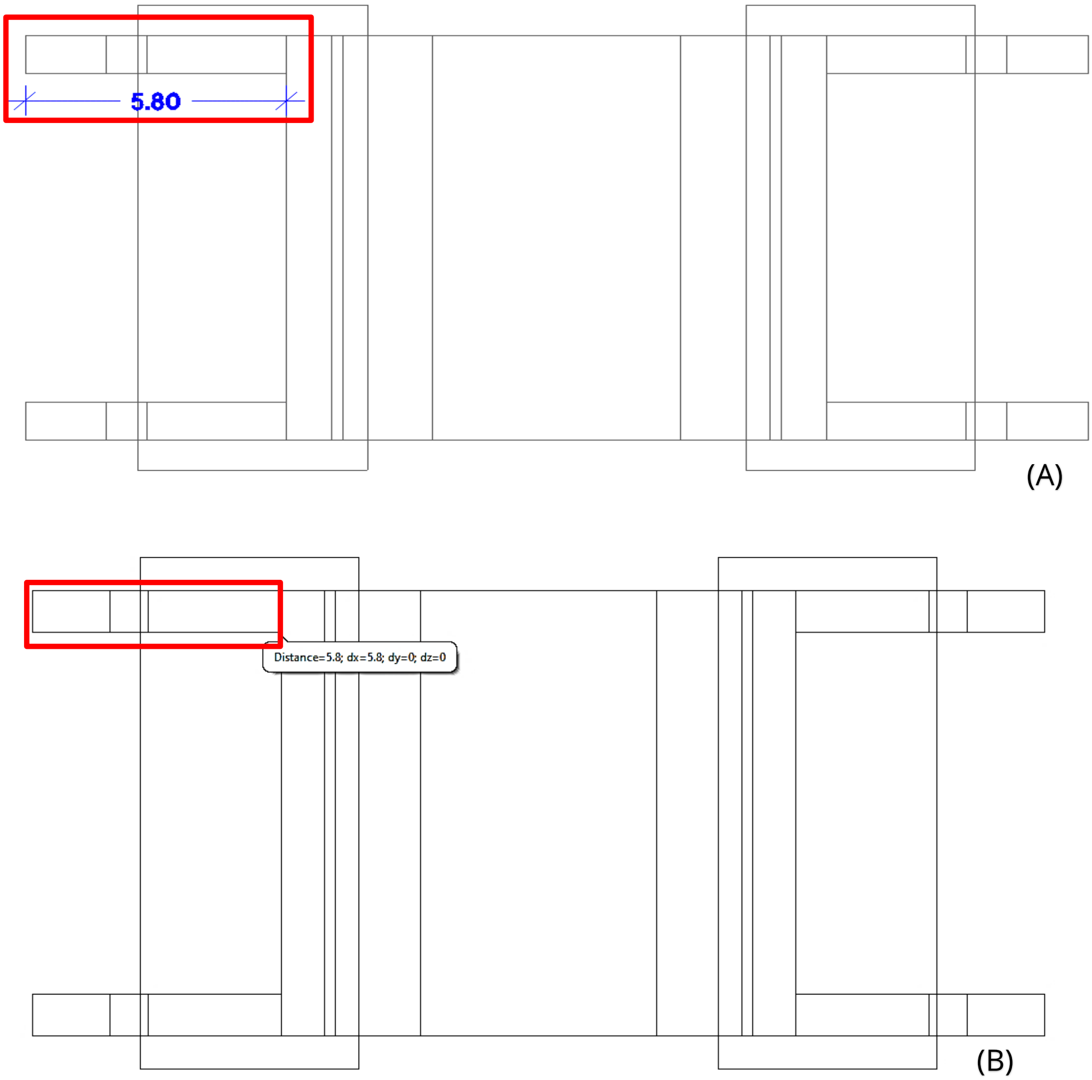


Figure 1. 29 comparison of measurements in both pragrams
The wing wall size in ALL PLAN 2019
The wing wall size in InfoGraph

- **Volume element:**

When the user imports the elements from a program to the other program, mostly the definition (characteristic) of the format is changed or disordered. But in this case, the definition of every single volume element remains stable after importing in the InfoGraph program hopefully. It means, the elements introduced as volume element to the InfoGraph individually (not as a *continuous* model). For more helpful comparison and conclusion, please see the Figure 1. 30.

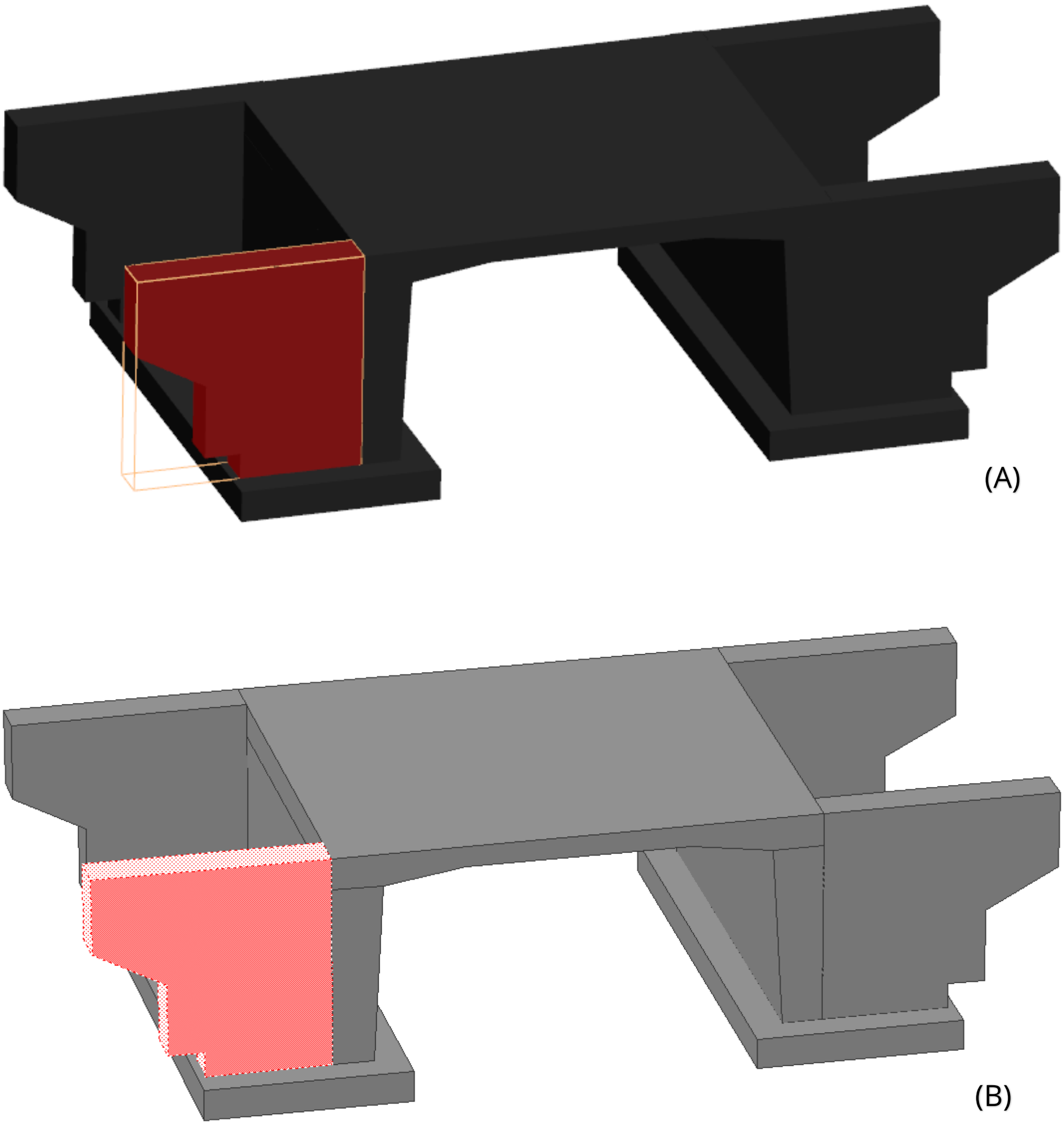


Figure 1. 30 Volume element comparison

A) Volume element in ALL PLAN 2019

B) Volume element in InfoGraph

- **Mesh system:**

The Mesh system in ALL PLAN 2019 was impossible directly, since it is not FEM program. But this matter is significant in FEM analysis. In this case, to save time and costs, the model should be capable to meshing and other FEM operations. Then, this feature was checked by InfoGraph, that the result was different in two commands *Mech Generation* and *Tetrahedrons from Solids*. The first function (Mech Generation) was not functional, because in InfoGraph to define the Mech Generation, a surface is needed. While the imported model in InfoGraph was identified as a *Volume Element*. Therefore, this function (Mech Generation) was not practical directly. When using the mesh generation function, it can be recognized that the volume element remains active even after meshing the item (Please see Figure 1. 31). From this point of view, this can be beneficial, provided that the volume element is removed after performing the mesh-system.

Besides, the model (the bridge) can be generated and consequently can be analyzed as FEM by *Tetrahedrons from Solids function*. Hence, this object was examined by InfoGraph that was practicable. In this case, each predefined element (for example, Slab. Wing wall, Abutment) was identified as a *volume element* (in the other word: Solid element) in InfoGraph and active for FEM generation. After selecting each specific element for the generation (or the whole model at the same time), the model was generated entirely. Please see the Figure 1. 32.

It here should be noted that after generating the model, the frame of the volume element remains however, that would be better to select it and delete it. And besides, it should be noted that in the case holding the frame volume element, nothing occurred; because the single volume element does not have structural meaning and has no influence on the structural analysis.

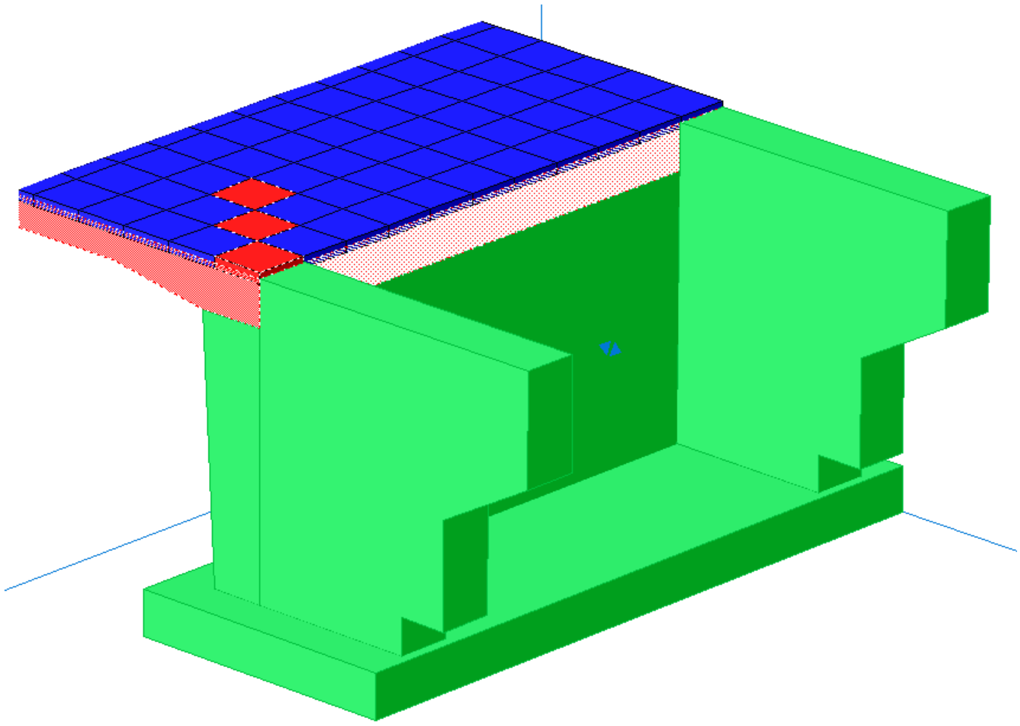


Figure 1. 31 FEM model, generated by Mesh generation (Blue: Meh generated element, Red: FEM units, Pink: selected volume element, Green: bridge elements)

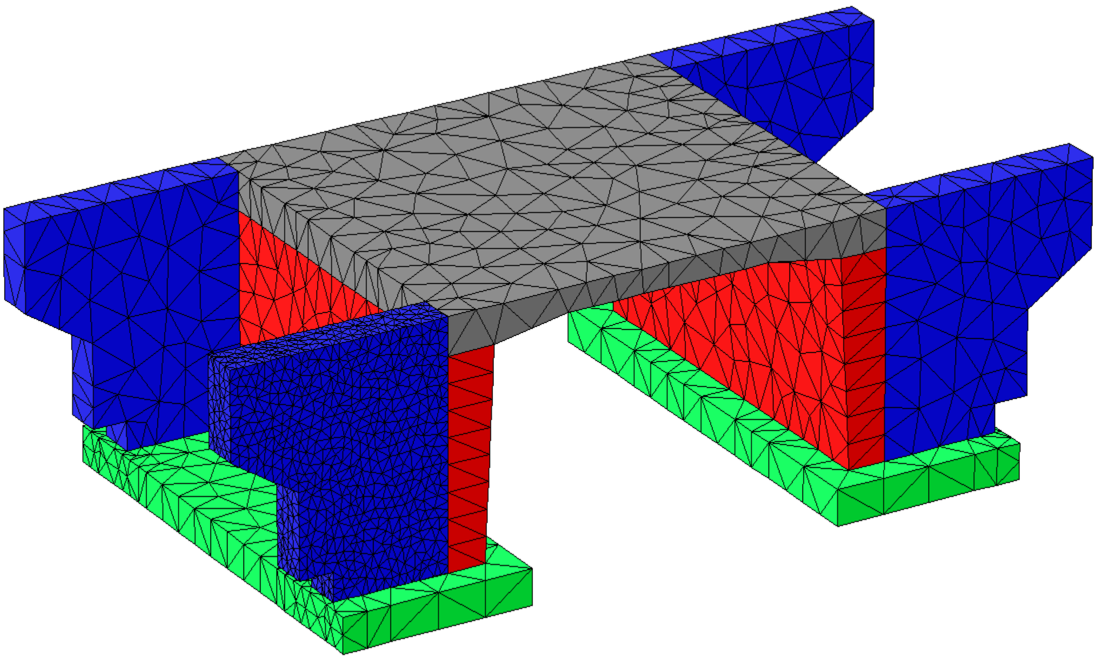


Figure 1. 32 FEM model, generated by Tetrahedrons from Solids function

➢ **Coordinate system:**

Another challenge that mostly appeared while importing a model to the other program is mainly the Coordinate system. In this research also a difference can be seen regarding to defining the coordinate system. In ALL PLAN 2019, the coordinate system was the global system (counterclockwise), while, in InfoGraph, the coordinate system change based on its definitions. It means the orientation was imported in reverse direction. For more expression, in ALLPLAN 2019, the three-dimensional system follows the right-handed rule. It means, in ALL PLAN 2019, the +Z direction defined perpendicular and outside to the plane, while, the direction of +Z would be vice versa in InfoGraph. For more helpful comparison and conclusion, please see Figure 1. 33.

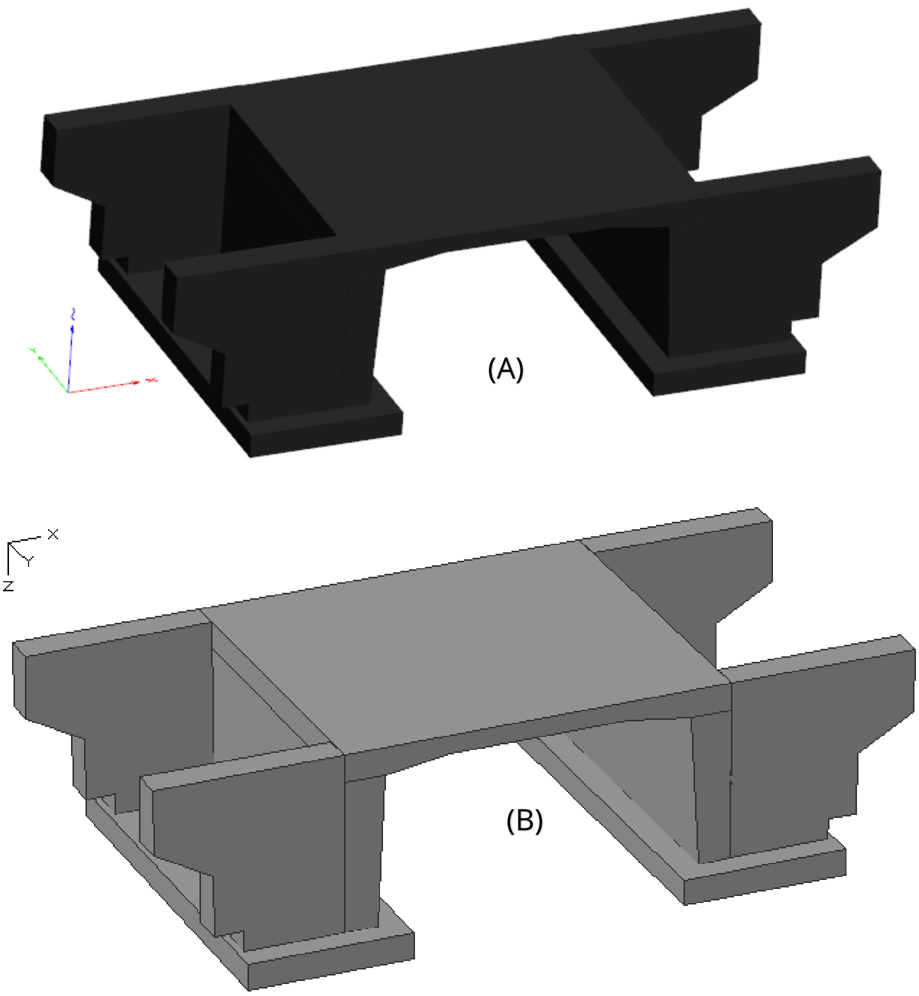


Figure 1. 33 Coordinate systems

A) Coordinate system in ALL PLAN 2019

B) Coordinate system in InfoGraph

➢ **Load case definition:**

As it is evident, a critical point of FEM analysis is regularly *load case* defining. As mentioned before, ALL PLAN 2019 is not a program to do a structural analysis finite element. Hence, it is expected to be done in InfoGraph. After importing the model in InfoGraph, it was tested that doing this matter was not possible directly or not! Hopefully, the elements even had only the volume elements characteristic and no structural definition, but they still were able to define as load cases. Therefore, the load case function was able to create this matter in InfoGraph after importing the bridge model. In Figure 1. 34 can loads like Load cases such as point load, line load, uniform load and moment load can be seen. The loads can be imported in different directions, for example X, -Y and Z and also in combine direction.

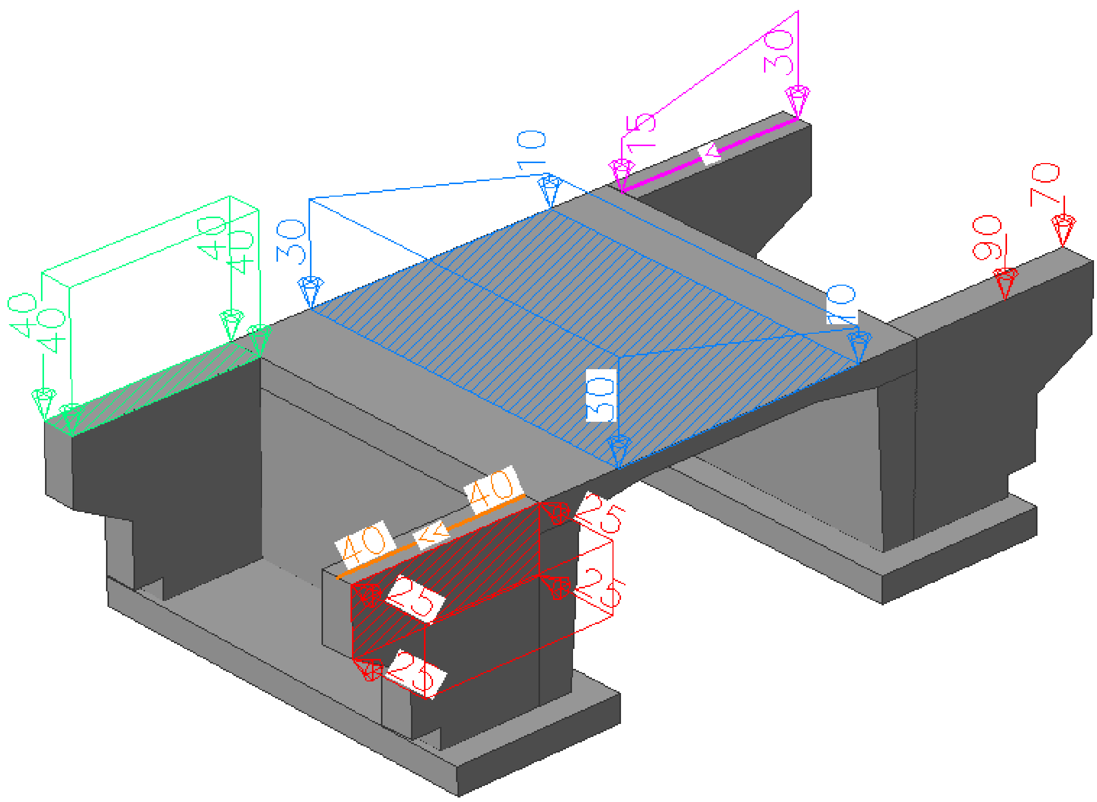


Figure 1. 34 Load: point load, line load, uniform load and moment load

4 Design Procedure by Finite Element Method (FEM)

4.1 Design Methodology

The previous chapter has covered some pre-knowledge that is necessary to perform the design scheme. In this chapter, the whole model by details will be genuinely. The items such as slab, foundation, abutment and wing walls would be described regarding the dimensions, load cases, type of elements, directions and other essential features.

4.1.1 Dimension and the bridge segment measurement

As discussed before, the half-frame bridge consists of four significant part. These parts are Slab, Wing Wall, Abutment and Foundation. Some part like the Foundation designed by experience in draft step, and finally designed precisely based on the result and other effects and the influences of different elements like the Soil strength and moment and so on. For example, the designer at first estimate an 80 cm thickness for Foundation, then, after analysis, the final thickness would be higher (**one** m) or less (**75** cm).

But in some cases, the designer must follow the accepted codes like Eurocodes or (particularly in this project) *German railway company* standards (Deutsche Bahn), the standard number *Ril 804.9040 Standardisierte*. For instance, in the case of Wing Wall design, depends on some parameters (like cement type, the slope of the Wing Wall, the distance between Wing Wall edge angle, abutment height) the dimensions and the thicknesses of it would be predefined. Hence, in this case, the thickness of the Wing wall would be 1.01 m. For better perception, please see Figure 1. 35.

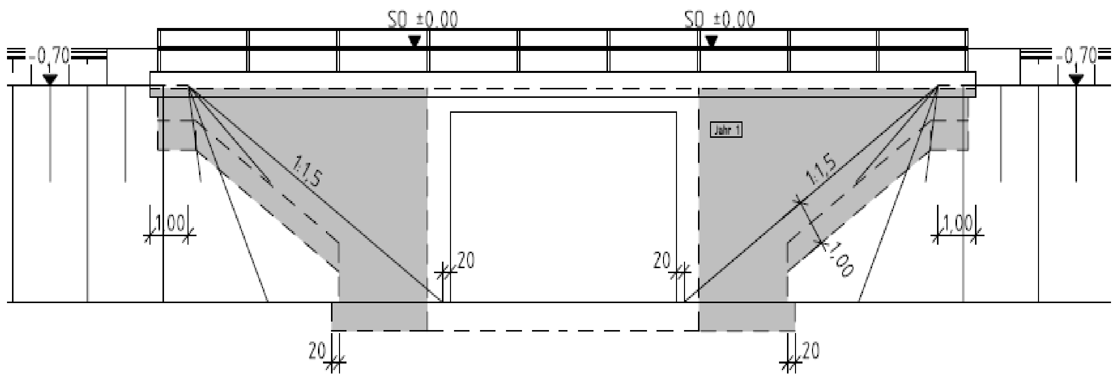


Figure 1. 35 an example of Wing Wall measurement criteria

The measurement of the other parts would be driven as follow:

Based on the project, the Slab has respectively **12** m and **10.74** m and Length and Width in the real plan, but in FEM model the Length was considered 11 m due to center by the center of the Abutments. And, it also **40** cm thickness in the middle and also **85** cm at the start, end and the corners. It should be here noted that in this project, three various Slab with a thickness of **40** cm, **55** cm and **80** cm (in the middle) has been modelled, that each one has been analyzed separately.

The *Abutment* has **5.35** m height (center by center of Slab and Foundation), and it has the width as the Slab with the **10.74** m. The Abutments also have **one** m thickness at the bottom and **1.25** m at the top side (connected to the Slab).

The *Wing Walls* have the Height as Abutment with **5.35** m, the Length with **5.8** m at top and **3.1** m at the bottom (the Foundation side), and the constant thickness of **1.01** m.

The *Foundation* has **5.1** m, **12.34** m and **80** cm respectively Length, Width and thickness. For better perception, please see Figure 1. 36.

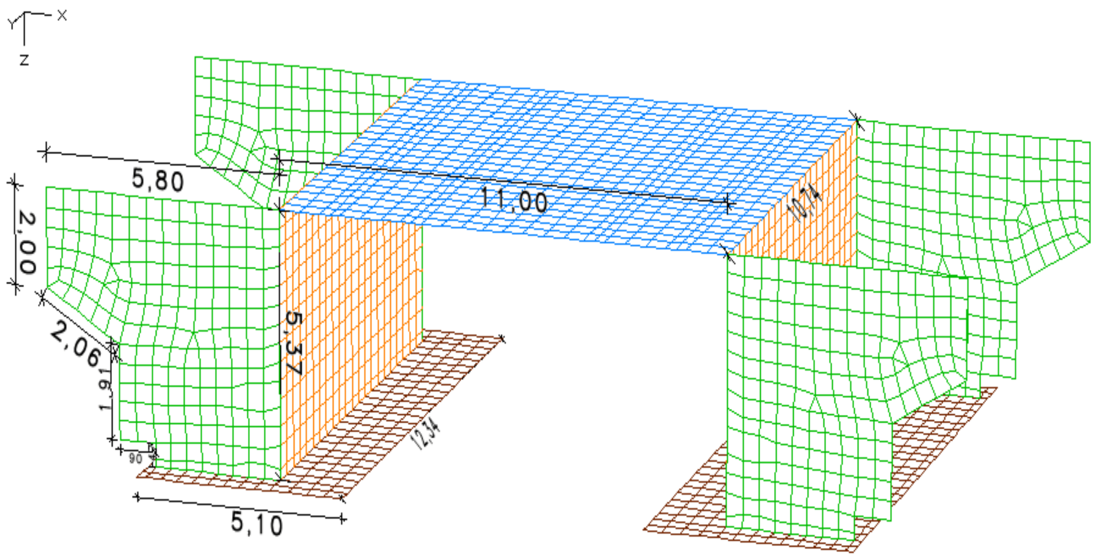


Figure 1. 36 the bridge section measurements

4.1.2 The coordinate reference system (global)

In the beginning, a global coordinate system was defined for the bridge to specify the direction of Forces (Positive or Negative), the location of each unique element, the direction of the Moments and also the Local Coordinate System. This system was defined based on below rules:

- **+X_Axis:** This element was determined based on the InfoGraph primary coordinate system. It is like the German driving system, *Right-Hand Traffic (RHT)*. It means the X values was considered as positive when the Train move from West to East side (from Plan view).
- **+Y_Axis:** This axis was deemed to be perpendicular to the X_Axis, and the value of this axis was considered positive when it runs from inside to outside of the 3D plane.
- **+Z_Axis:** In the IFC product model, the +Z_Axis of the global coordinate system typically operates from below to above. But in InfoGraph coordinate system it has 180-degree differences. Hence, the +Z_axis was considered from above to below direction. For better conclusion, please see Figure 1. 37.
- **Local Coordinate System:** Each item of the bridge has its specific Local Coordinate System. In this system, the Z_Axis is always perpendicular to the X_Y plane. For better conclusion, please see Figure 1. 38. Here also, each element has a specific number that was shown in Figure 1. 39.

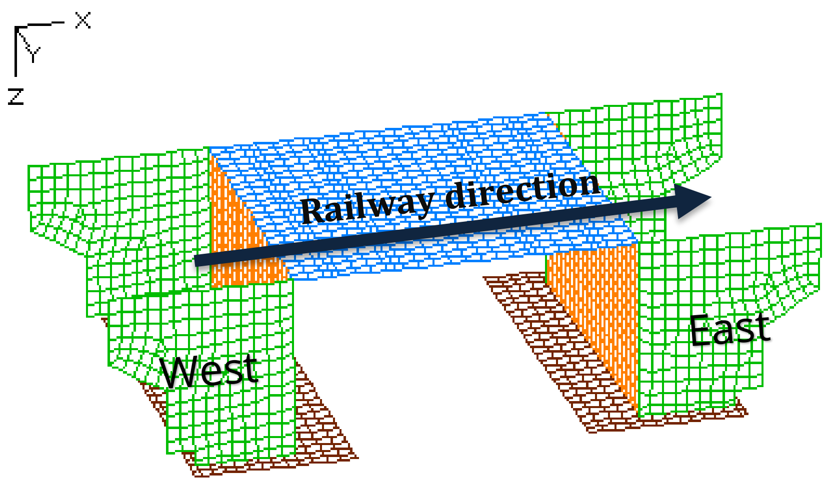


Figure 1. 37 Global coordinate system

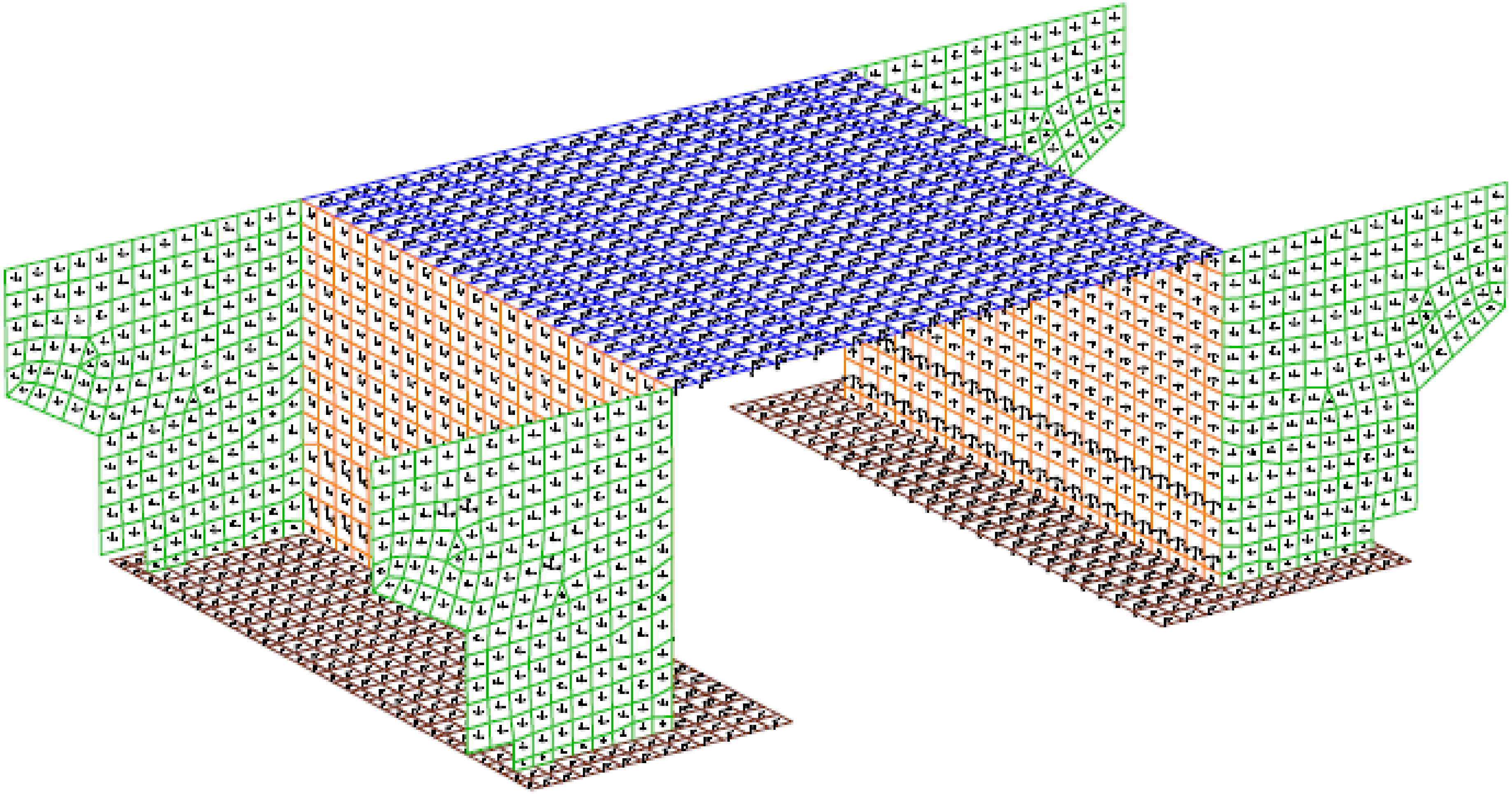


Figure 1. 38 Local coordinate system

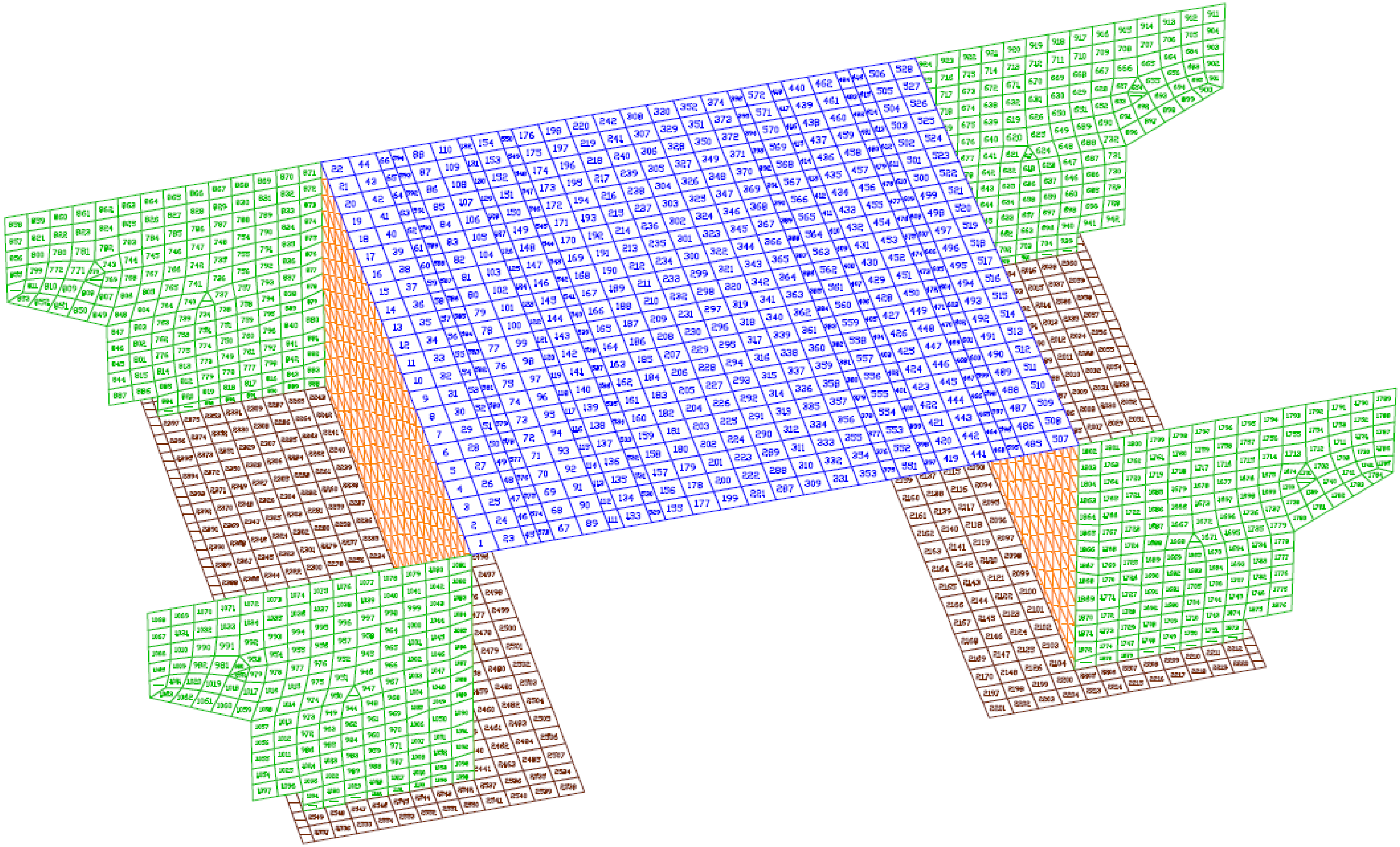


Figure 1. 39 Element Number

4.1.3 Creating the elements

To have a realistic model and perfect simulation, each particular element must be determined by the correct FEM system. In this case, in the begging, the model was defined as the finite element method in InfoGraph program via *Structure* tab (Figure 1. 40). Then, the components (for example, Slab) were identified with in Mesh Generation tab. In case of Slab, the item was defined by *Square - Grid on Four Edges*, because the Slab had the symmetric and rectangular form (Even creating the item was possible by the other modes). Please see Figure 1. 41.

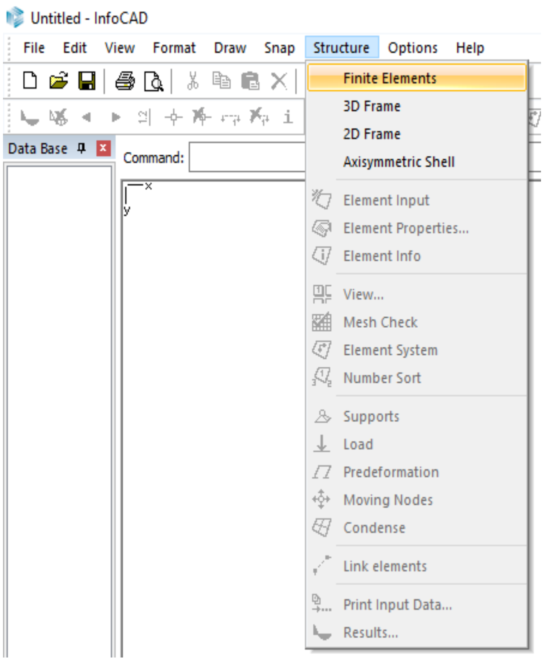


Figure 1. 40 Defining the system

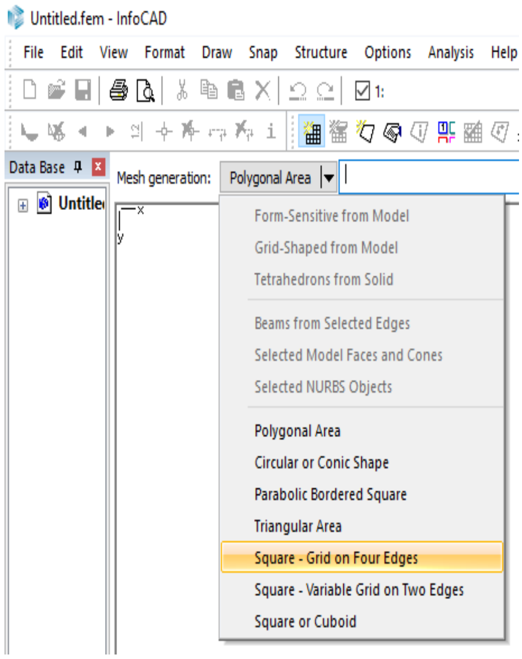


Figure 1. 41 Creating the item via a specific mode

Then after, the desired element type was selected as shell, and the Slab was created as shell element*. The created Slab can be seen in Figure 1. 42 and Figure 1. 43.

* ***The shell elements*** *are a combination of the element stiffness matrices of the plain stress elements and slab elements, meaning a curved shell area is approximated using facets. This does not mean, however, that there are any relevant limitations concerning the results that can be achieved.* (InfoGraph Help)

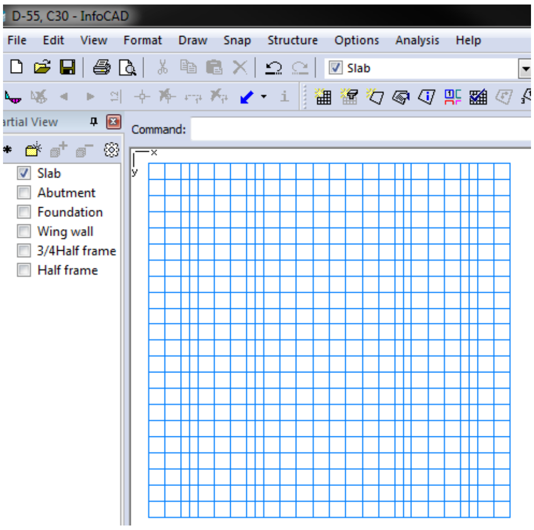


Figure 1. 42 Created Slab by Mesh Generation, 2D- view

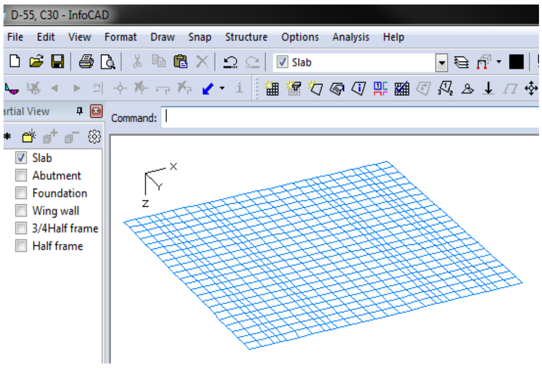


Figure 1. 43 Created Slab by Mesh Generation, 3D view

As it is clear, the model can be designed in 2D or 3D. In this project, since the FEM analysis is essential, the model was generated in 3D form. Besides, when the bridge parts such as Abutment or Wing Walls are created in 3D form, that would be more manageable and more helpful to follow and present the result later.

The selected distance between the Mesh element mostly was taken *50 cm*. This distance could be admittedly smaller, but more diminutive range not only made more element (that consequently is time-consuming and create more elements) but also had no significant modifications in the final result. But in some cases, to have a greater connection for example between the various components in corners or when the thickness of Slab was changed, the range between the elements was decreased to 30 cm or 20 cm to have a well-defined result and simulation.

4.1.4 Materials (Concrete type and characteristic)

To compare the result and also to have the best optimization, three different concrete type with **Characteristic strength of concrete C30/37, C45/55 and C60/75** were considered. And as mentioned before, each concrete must be analyzed with three different Slab thickness (only slab). The rest parts Characteristic strength of concrete was taken with the value of C45/55. Hence, It can be said that nine various bridges would be analyzed in this project. Please see the Table 1. 2.
The considered **Characteristic Yield Strength of Steel** for reinforcement was **B500B**.

Table 1. 2 Material characteristics

Title / Nr.	Concrete type for Slab (f_{ck})	Slab thickness (cm)	Concrete type for Other parts (f_{ck})	Reinforcement (f_{yk})
1	**C30/37**	40	**C35/45**	**B500B**
2		55		
3		80		
4	**C45/55**	40	**C35/45**	**B500B**
5		55		
6		80		
7	**C60/75**	40	**C35/45**	**B500B**
8		55		
9		80		

The Intended Bridge in this research consists of Concrete Deck, four Wing Walls, two Abutment and two-part Foundation. Please see Figure 1. 44. The main focus of this research is on the Bridge Deck. Because the rest part must be fixed (due to physical limitation, dimensions, regulation and other factors), hence, the Bridge would be analysed with three different Deck with 40 cm, 55 cm and 80 cm thickness. And also, each Bridge must be analysed with three various concrete type with Characteristic strength of concrete C35/45, C45/55 and C60/75. Hence, nine various Bridge will be investigated in this research.

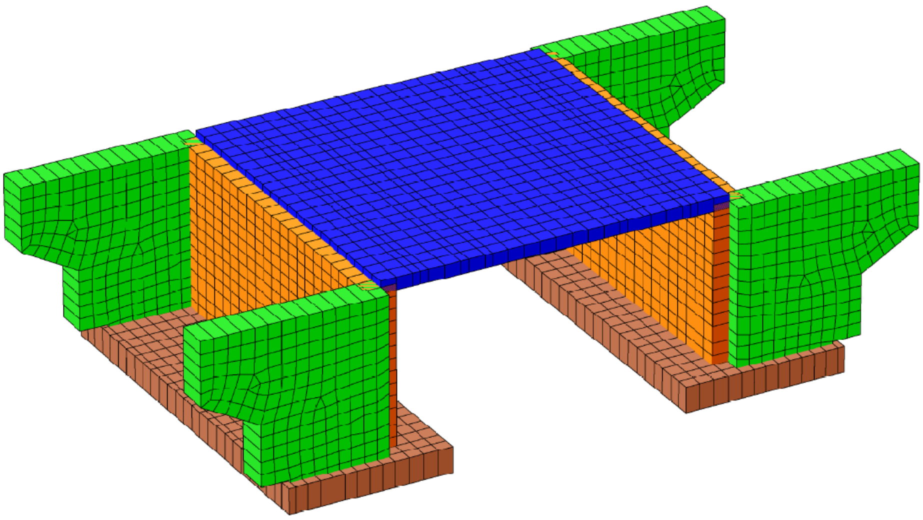


Figure 1. 44 The Bridge System

The created model includes **2657 Nodes**, **2520 Elements**, **60 Load Cases** and **15 Load Case Combination**. The **centroid** of the element considered the result point of each element.

In the first part of analyses, the Bridge Deck was defined continuously in both directions X and Y. It means, there was no Haunched part at the corner of Bridge Slab. Figure 1. 45 and Figure 1. 46 show the Deck thickness and appearance well.

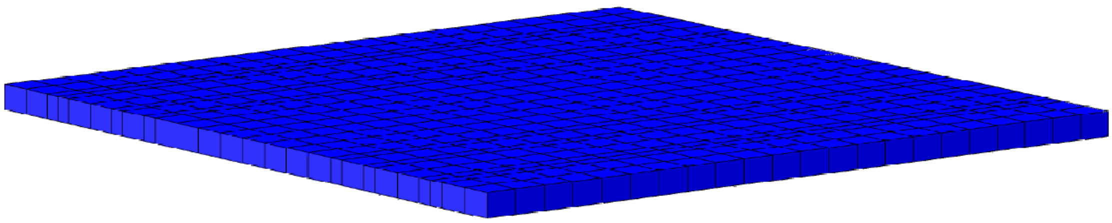


Figure 1. 45 the Constant Bridge Deck in 3D view

Figure 1. 46 the Constant Bridge Deck Cross Section

4.1.5 Assumptions

In this part, the supposed assumptions and are described.
From the geotechnical point of view, **the Bearing Capacity of the Soil** was assumed **10 MN/m^3** (as the Bedding Modulus) in the **Z-direction** and **1.5 MN/m^3** in **X** and **Y-direction**. Besides, the maximum **Speed** of the Train would be **160 km/h**. **Dynamic Analysis** and **Fatigue** are not desired in calculation also.

4.2 Load Groups

4.2.1 Load Groups and Partial Safety Factors

In this part, the Load Groups and the Criteria for each case will be discussed and calculated, and some of them will be presented based on the *EN 1991-2* (Actions on structures - Part 2: Traffic loads on bridges). Even though the Load-bearing capacity and other factors were discussed and summarized in previous chapters. Also, according to the EN 1991-2:2003 (E) (Table 6.11 - Assessment of Groups of Loads for rail traffic (characteristic values of the multicomponent actions)), the specified groups must be considered for bridge design. Besides, the safety factors and other parameters highlighted in below Tables must be regarded.

Table 1. 3 Assessment of Groups of Loads for rail traffic (characteristic values of the multicomponent actions)

number of tracks on structure			Groups of loads Reference EN 1991-2			Vertical forces 6.3.2/6.3.3	6.3.3	6.3.4	Horizontal forces 6.5.3	6.5.1	6.5.2	Comment
1	2	≥ 3	number of tracks loaded	Load Group[8]	Loaded track	LM 71[1] SW/0[1], [2] HSLM[6][7]	SW/2[1][3]	Unloaded train	Traction, Braking[1]	Centrifugal force[1]	Nosing force[1]	
			1	gr11	T_1	1			1 [5]	0,5 [5]	0,5 [5]	Max. vertical 1 with max. longitudinal
			1	gr 12	T_1	1			0,5 [5]	1 [5]	1 [5]	Max. vertical 2 with max. transverse
			1	gr 13	T_1	1 [4]			1	0,5 [5]	0,5 [5]	Max. longitudinal
			1	gr 14	T_1	1 [4]			0,5 [5]	1	1	Max. lateral
			1	gr 15	T_1			1		1 [5]	1 [5]	Lateral stability with "unloaded train"
			1	gr 16	T_1		1		1 [5]	0,5 [5]	0,5 [5]	SW/2 with max. longitudinal
			1	gr 17	T_1		1		0,5 [5]	1 [5]	1 [5]	SW/2 with max. transverse
			2	gr 21	T_1 T_2	1 1			1 [5] 1 [5]	0,5 [5] 0,5 [5]	0,5 [5] 0,5 [5]	Max. vertical 1 with max longitudinal
			2	gr 22	T_1 T_2	1 1			0,5 [5] 0,5 [5]	1 [5] 1 [5]	1 [5] 1 [5]	Max. vertical 2 with max. transverse
			2	gr 23	T_1 T_2	1 [4] 1 [4]			1 1	0,5 [5] 0,5 [5]	0,5 [5] 0,5 [5]	Max. longitudinal
			2	gr 24	T_1 T_2	1 [4] 1 [4]			0,5 [5] 0,5 [5]	1 1	1 1	Max. lateral
			2	gr 26	T_1 T_2	 1	1		1 [5] 1 [5]	0,5 [5] 0,5 [5]	0,5 [5] 0,5 [5]	SW/2 with max. longitudinal
			2	gr 27	T_1 T_2	 1	1		0,5 [5] 0,5 [5]	1 [5] 1 [5]	1 [5] 1 [5]	SW/2 with max. transverse
			≥3	gr 31	T_i	0.75			0.75 [5]	0.75 [5]	0.75 [5]	Additional load case

(1) All relevant factors (α, Φ, f, ...) shall be taken into account.
(2) SW/0 shall only be taken into account for continuous beam structures.
(3) SW/2 needs to be taken into account only if it is stipulated for the line.
(4) Factor may be reduced to 0,5 if favourable effect, it cannot be zero.
(5) In favourable cases these non-dominant values shall be taken equal to zero.
(6) HSLM and Real Trains where required in accordance with 6.4.4 and 6.4.6.1.1.
(7) If a dynamic analysis is required in accordance with 6.4.4 see also 6.4.6.5(3) and 6.4.6.1.2.
(8) See also Table A2.3 of EN 1990

Dominant component action as appropriate

to be considered in designing a structure supporting one track (Load Groups 11-17)

to be considered in designing a structure supporting two tracks (Load Groups 11-27 except 15). Each of the two tracks shall be considered as either T_1 (Track one) or T_2 (Track 2)

to be considered in designing a structure supporting three or more tracks; (Load Groups 11 to 31 except 15. Any one track shall be taken as T_1, any other track as T_2 with all other tracks unloaded. In addition the Load Group 31 has to be considered as an additional load case where all unfavourable lengths of track T_i are loaded.

Table 1. 4 Table NA.A2.1

Einwirkung	Bezeich-nung	γ-Werte für die Einwirkungen in den entsprechenden Bemessungssituationen nach			
		Tabelle A.2.4 (A) EQU		Tabelle A.2.4 (B) STR/GEO	Tabelle A.2.5 Außer-gewöhn-lich
		S/V	B	S/V	A
Ständige Einwirkungen					
Ungünstig	$\gamma_{G,sup}$	1,05	1,05	1,35[b]	1,0
Günstig	$\gamma_{G,inf}$	0,95[a]	0,95[a]	1,0	1,0
Vorspannung[h]					
Ungünstig	γ_{Psup}	1,0[i]/1,2[j]	1,0[i]/1,2[j]	1,0[i]/1,2[j]	1,0
Günstig	γ_{Pinf}	1,0[i]/0,8[j]	1,0[i]/0,8[j]	1,0[i]/0,8[j]	1,0
Setzungen[e]	γ_{Gset}	--	--	1,2[g]/1,35[h]	--
Einwirkungen aus Straßen- und Fußgängerverkehr					
Ungünstig	$\gamma_{Q,sup}$	1,35	--	1,35	1,0
Günstig	$\gamma_{Q,inf}$	0	--	0	0
Einwirkungen aus Schienenverkehr					
Ungünstig	$\gamma_{Q,sup}$	1,45	--	1,45[c]/1,2[d]	1,0
Günstig	$\gamma_{Q,inf}$	0	--	0	0
Lasten aus der Bauausführung					
Ungünstig	$\gamma_{Q,sup}$	--	1,35	--	1,0
Günstig	$\gamma_{Q,inf}$	--	0	--	0
Temperatur					
Ungünstig	$\gamma_{Q,sup}$	1,35	1,35	1,35	1,0
Günstig	$\gamma_{Q,inf}$	0	0	0	0

Table 1. 5 Table NA.A2.1

Einwirkung	Bezeich-nung	Bemessungssituation			
		Tabelle A.2.4 (A) EQU		Tabelle A.2.4 (B) STR/GEO	Tabelle A.2.5 Außer-gewöhn-lich
		S/V	B	S/V	A
Alle anderen veränderlichen Einwirkungen					
Ungünstig	$\gamma_{Q,sup}$	1,5	1,5	1,5	1,0
Günstig	$\gamma_{Q,inf}$	0	0	0	0
Außergewöhnliche Einwirkungen	γ_A	--	--	--	1,0

Table 1. 6 Recommendation for the numerical values of the ψ-factors for railway bridges

Einwirkungen			ψ_0	ψ_1	ψ_2[d]
Komponente der Verkehrseinwirkung[e]	LM 71		0,80	a	0
	SW/0		0,80	a	0
	SW/2		0	1,00	0
	Unbeladener Zug		1,00	–	–
	HSLM		1,00	1,00	0
	Anfahr- und Bremskräfte Zentrifugalkraft Interaktionskräfte infolge von Verformungen unter vertikalen Verkehrslasten		Für einzelne Komponenten der mehrkomponentigen Verkehrseinwirkung, die an Stelle von Lastgruppen als Leiteinwirkung verwendet werden, sollten die ψ-Faktoren verwendet werden, die für die zugehörigen vertikalen Lasten empfohlen werden.		
	Seitenstoß		1,00	0,80	0
	Lasten auf Dienstwege		0,80	0,50	0
	Betriebslastenzug		1,00	1,00	0
	Horizontaler Erddruck infolge Überschreitung der Verkehrslasten		0,80	a	0
	Aerodynamische Wirkungen		0,80	0,50	0
Einwirkung des Hauptverkehrs (Lastgruppen)	gr11 (LM71 + SW/0)	Max. vertikal 1 mit max. längs	0,80	0,80	0
	gr12 (LM71 + SW/0)	Max. vertikal 2 mit max. quer			
	gr13 (Bremsen/Anfahren)	Max. längs			
	gr14 (Zentrifugalkraft/Seitenstoß)	Max. seitlich			
	gr15 (unbeladener Zug)	Seitenstabilität mit „unbeladenem Zug"			
	gr16 (SW/2)	SW/2 mit max. längs			
	gr17 (SW/2)	SW/2 mit max. quer			
	gr21 (LM71 + SW/0)	Max. vertikal 1 mit max. längs	0,80	0,70	0
	gr22 (LM71 + SW/0)	Max. vertikal 2 mit max. quer			
	gr23 (Bremsen/Anfahren)	Max. längs			
	gr24 (Zentrifugalkraft/Seitenstoß)	Max. zur Seite			
	gr26 (SW/2)	SW/2 mit max. längs			
	gr27 (SW/2)	SW/2 mit max. quer			
	gr31 (LM71 + SW/0)	Zusätzliche Lastfälle	0,80	0,60	0
Andere Einwirkungen aus Betrieb	Aerodynamische Wirkung		0,80	0,50	0
	Allgemeine Lasten aus Instandhaltung für Dienstgehwege		0,80	0,50	0
Windkräfte[b]	F_{Wk}		0,75	0,50	0
	F_W^{**}		1,00	0	0

Table 1. 7 Table A2.3

Einwirkungen		ψ_0	ψ_1	ψ_2[d]
Temperatur[c]	T_k	0,60	0,60	0,50
Schneelasten	$Q_{Sn,k}$ (während der Bauausführung)	0,8	—	0
Lasten aus Bauausführung	Q_c	1,0		1,0

4.2.2 Permanent Loads (Dead Load of the bridge)

The Dead Load of the bridge, including the foundation, was calculated internally in the program using a weight of from γ = 25 kN/m³.

4.2.3 Permanent Loads (Dead Load of Railway)

The Weight of Railway (Lane weight) includes the Protective concrete, Ballast bed and Rail plus sleeper. These amounts regarding their weight and thicknesses were calculated below:

- **Dead Load of Railway:**

$g2.1 = 0.06\ m \cdot 25\ kN/m^3 = 1.50\ kN/m^2$	Protective concrete
$g2.2 = 0.60\ m \cdot 20\ kN/m^3 = 12.00\ kN/m^2$	Ballast bed
$g2,3 = 2 \cdot 1,2\ kN/m\ /\ 10,74\ m = 0,22\ kN/m^2$	Rail + sleeper
$\Sigma\ g2 =$ **13.72** kN/m^2	**Σ Railway**

Then the amount of **13.72** kN/m^2 as lane weight was imported to the model. Please see the Figure 1. 47.

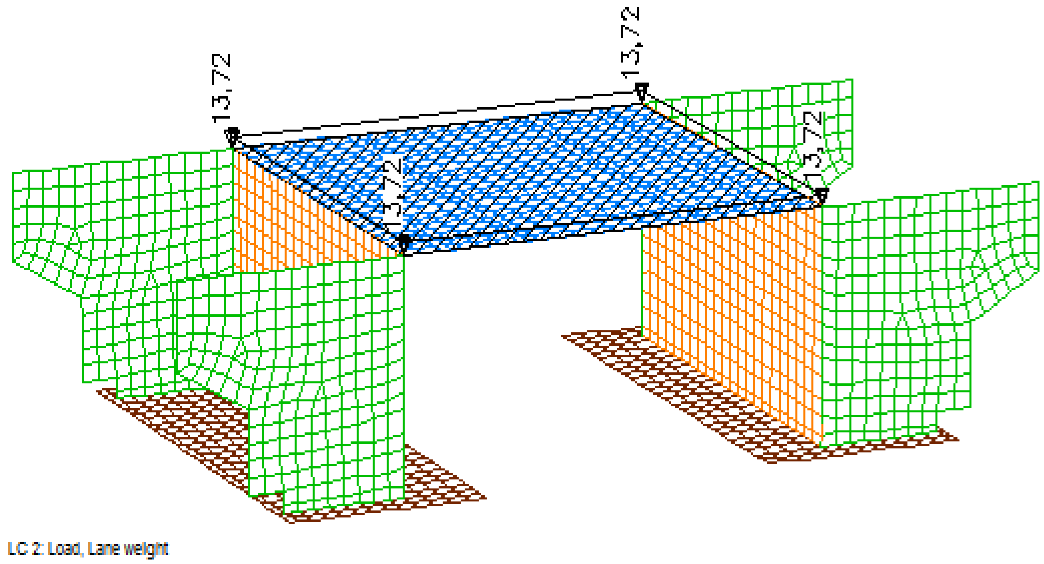


Figure 1. 47 Dead Load of Railway

4.2.4 Active earth pressure on the Abutment and Wing Wall

With assumption of **γ = 20 kN/m³** and **φ = 35°** and **δ =0.667 φ**, the load due to earth pressure on abutment wall was calculated in below, then the value of **46.23 kN/m²** was imported to the model. Please see Figure 1. 48.

γ = 20 kN/m³ $\qquad$ φ = 35.0 ° $\qquad$ δ = .667 φ

$\longrightarrow$ $\qquad$ K_{0gh} = 0.43

$$e_{0gh} = \gamma \cdot h \cdot K_{0gh} = 46.23 \text{ kN/m}^2 \qquad \text{mit: } h = 5.38 \text{ m}$$

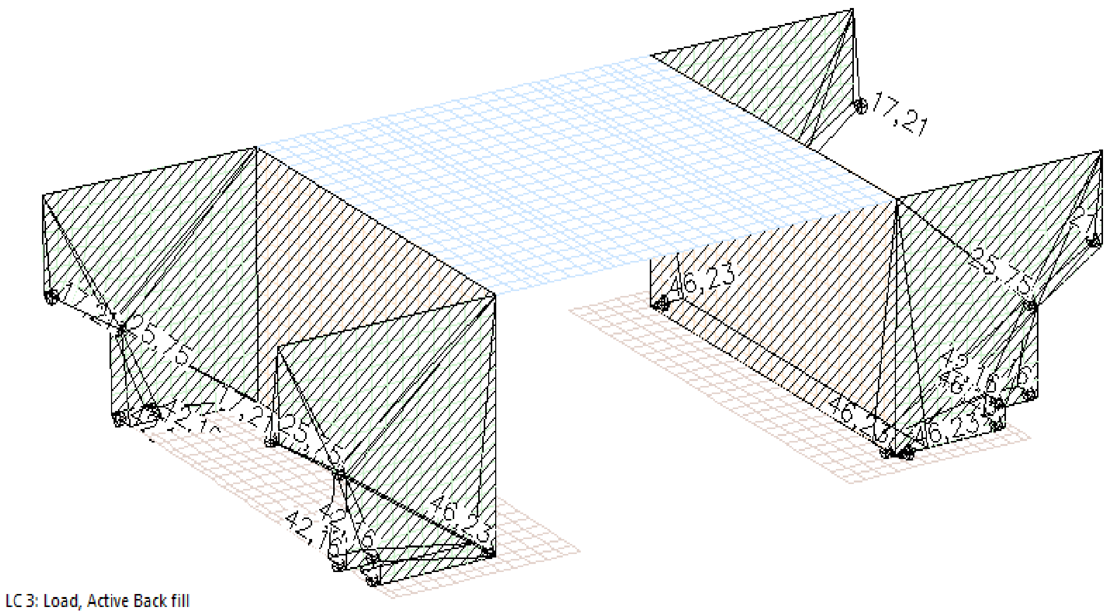


Figure 1. 48 earth pressure on the Abutment and Wing Wall

4.2.5 Lateral earth pressure (Compaction Force due to Earth Pressure)

Lateral earth pressure is the pressure that soil exerts in the horizontal orientation. The lateral earth pressure is essential because it affects the consolidation behaviour and strength of the soil and because it is held in the purpose of geotechnical engineering structures such as retaining walls, basements, deep foundations and braced excavations. This value around the ground surface is 25 kN/m³, then gradually is decreased to zero in height of 2.91 m (Figure 1. 49). This value was calculated via below formulas and imported to the model. Please see Figure 1. 50.

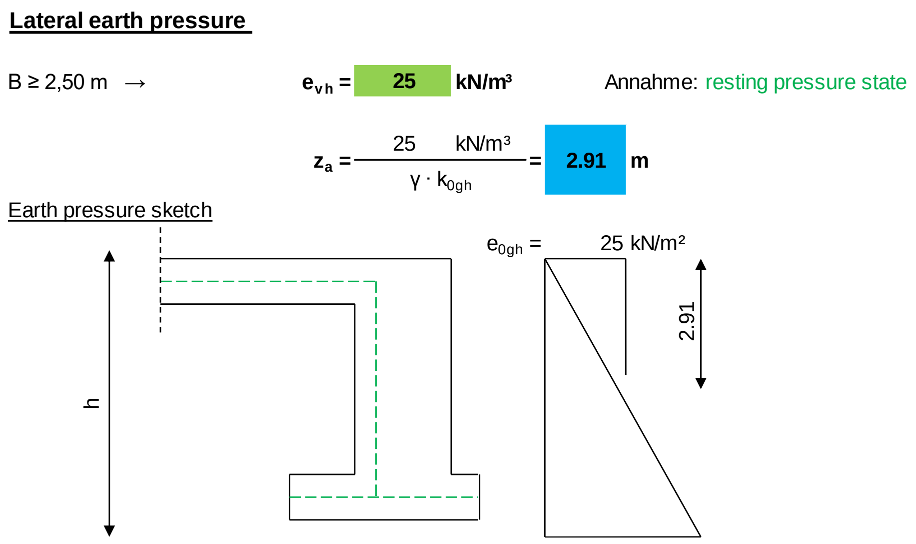


Figure 1. 49 schematic importing the Lateral earth pressure

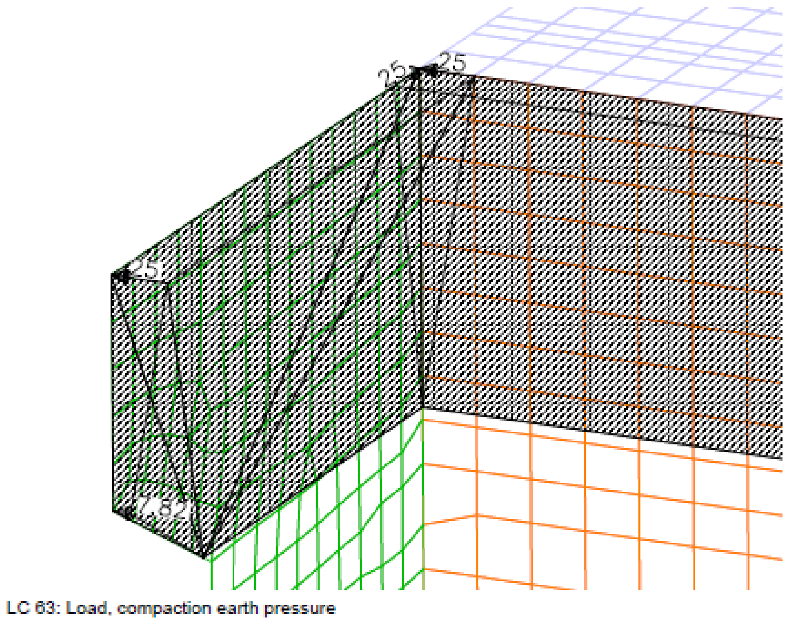


Figure 1. 50 4.1.10 Part of Lateral earth pressure (Compaction Force due to Earth Pressure)

4.2.6 Earth pressure and Overload earth pressure from LM 71

For the calculation of the earth pressure, the simplified load pattern of the uniformly distributed line load is assumed. The line load may be assumed to be evenly distributed over a width of 3.00 m at a depth of 0.70 m below the upper corner of the rail.

- γ = 20 kN/m^3
- φ = 35°
- δ = 2/3 φ
- α = 1.0

$\Rightarrow$ K_{0gh} = 0,43

- q_{vk} = 156 kN/m
- $e_{0ph} = \alpha \cdot K_{0gh} \cdot q_{vk}$ / 3,00 m = 1,0 · 0,43 · 156 kN/m / 3,00 m = **22,36** kN/m^2
- = 1,0 · 0,43 · 80 kN/m / 3,00 m = **11,5** kN/m^2

In program to have fewer load cases, the amount of **11,5** kN/m^2 was subtracted from **22,36** kN/m^2 and the result of **10,86** kN/m^2 was added to the model as overload earth pressure via *Load Case Combination* option in InfoGraph. Please see Figure 1. 51 and Figure 1. 52.

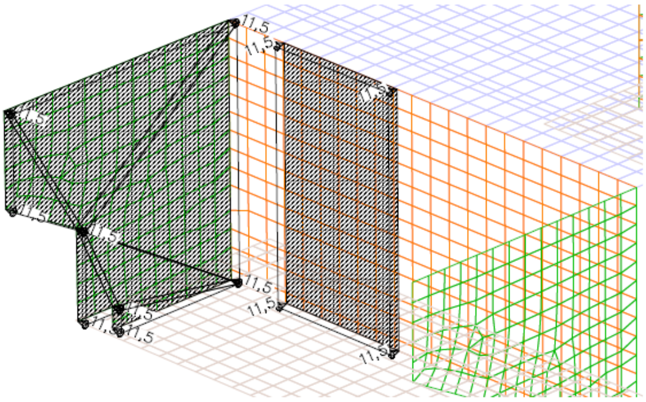


Figure 1. 51 Earth pressure from LM 71 on left track

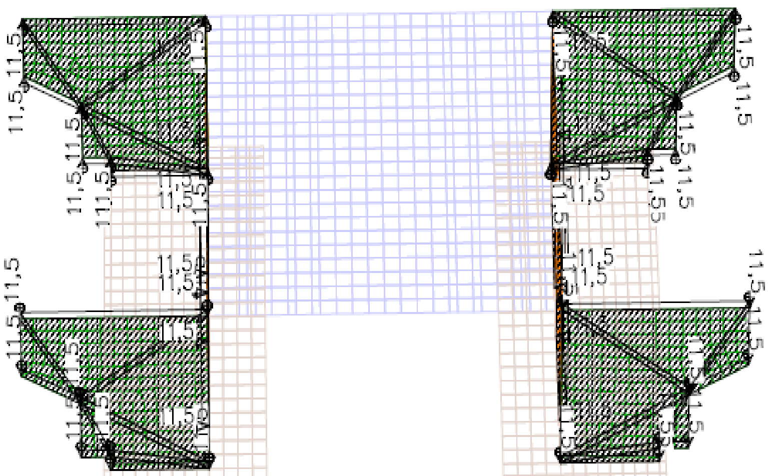


Figure 1. 52 Earth pressure from LM 71 on both sides of tracks

As Figure 1. 53 shows, for *Overload earth pressure from LM 71*, the value of **10,86** kN/m2 was imported to the model, because this amount would be added to **11,5** kN/m2 via *Load Case Combination* option (in the FEM program) and results the amount of = **22,36** kN/m^2 as mentioned before.

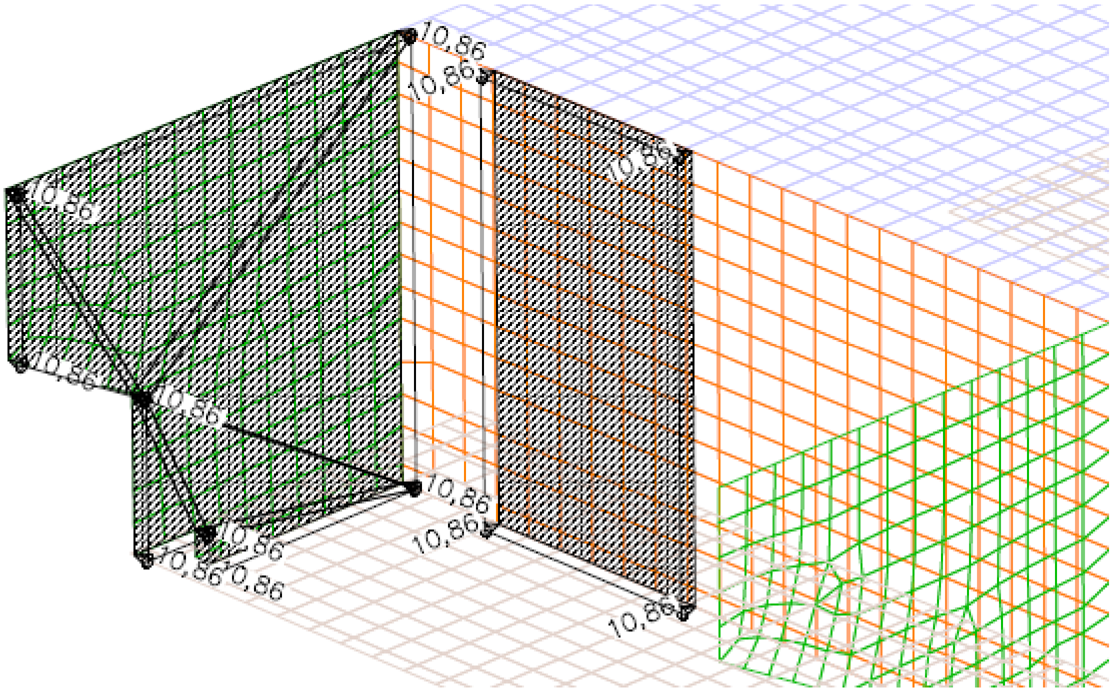


Figure 1. 53 Overload earth pressure from LM 71 on left track

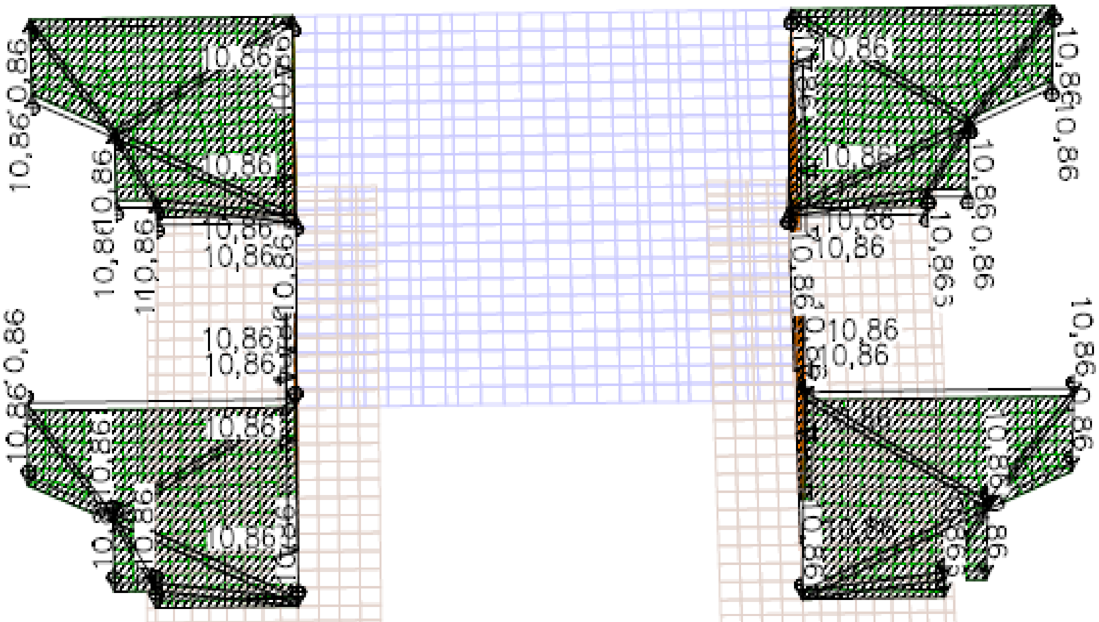


Figure 1. 54 Overload earth pressure from LM 71 on both sides of tracks

4.2.7 Rail traffic actions

This part refers to rail traffic on the standard track gauge and wide track gauge for European mainline network. The load models specified in this part, do not represent real loads. They have been selected so that their impacts, with dynamic increasements, taken into account separately, describe the effects of service traffic. Where traffic outside the scope of the load models defined in this part needs to be viewed, then alternative load models, with associated incorporation rules, should be defined. Please see Figure 1. 55.

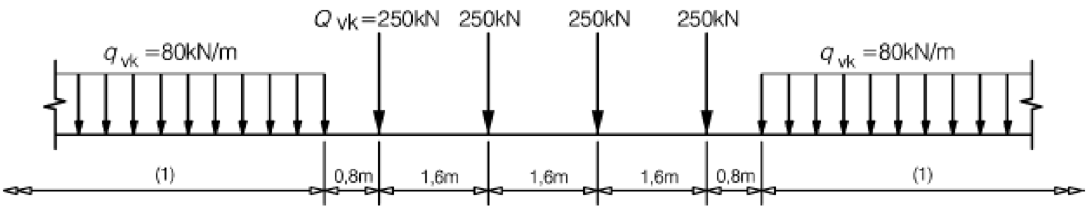


Figure 1. 55 the distribution of load LM 71

The load class coefficient **α** must be selected with **1.0** according to the task assumptions.

According to DIN EN 1991-2 6.3.6.2 (1), the individual loads of the LM71 can be assumed to be uniformly distributed, that, this results in a line load of:

q_{vk} = 4 x 250 kN / 6.40 m = 156 kN/m

Besides, based on the Euro code (DIN EN 1991-2, 6.3.6.1) in the longitudinal direction, the individual loads of the LM 71 that does not exceed three rail support points, the concentrated load Q_{vi} can be distributed to three parts (Q_{vi}/4, Q_{vi}/2 and Q_{vi}/4). Please see Figure 1. 56.

Q_{vi} / 2 = 250 kN / 2 = **125.0** kN

Q_{vi} / 4 = 250 kN / 4 = **62.50** kN

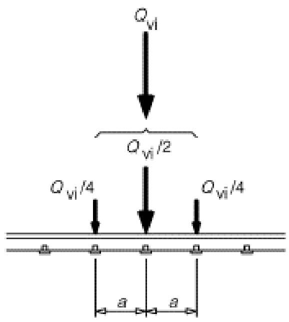


Figure 1. 56 Longitudinal distribution of a point force or wheel load by the rail

a) the distance of sleeper axe.

Additionally, when Q_{vi} is distributed to the sleepers and ballast, due to the thickness of sleeper and ballast, the load has a difference effect of the real position. Hence, the load is distributed up to the top of the frame based on DIN EN 1991-2, 6.3.6.2, and, the load Q_{vi} is divided to the value of 0.57 m. Please see Figure 1. 57.

b = 0,30 m + 2 (0,3 / 4 + 0,06) = **0,57** m

q_{vk} = 125.0 kN / 0.57 m = **219.30** kN/m

q_{vk} = 62.50 kN / 0.57 m = **109.65** kN/m

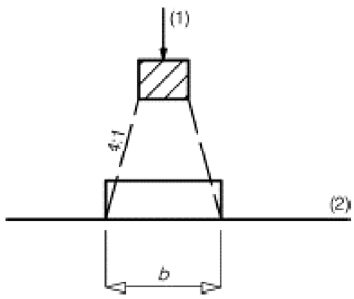


Figure 1. 57 Longitudinal distribution of load by a sleeper and ballast

The permissible lateral distribution of vertical loads by sleepers and Ballast is taken into account in the calculation model up to the middle plane of the carriageway slab. A load spread of 4:1 in the ballast and of 1:1 in the protected concrete is assumed. Therefore, the width **b** for load propagation under sleeper, ballast, protective concrete and half the roadway thickness was considered. Then:

Up-side of the sleeper to Bottom-side of ballast	**30** cm
Sleeper	**2.6** m
Protective concrete	**6** cm
Deck thickness	**40** cm
b = 2.6 + 2 × (0.3/4 + 0.06 + 0.4/2) = **3.47** m	Load propagation length

Here should be noted that in this research, three Deck thickness with 40 cm, 55 cm and 80 cm was modelled. In this part, the thickness of 40 cm was considered (as an example for description), and the other models were formed the same as this model.

By here, the Load LM71 was calculated. But to import this load to the model, the other factors like deck thickness, eccentricity, the combination of wind load and other factors must be considered.

4.2.8 Eccentricity

The lateral eccentricity of the vertical loads shall be considered by a ratio of the two-wheel loads of all axles on any track of 1,25:1,0. The eccentricity results from the ratio of **e ≤ r/18** (DIN EN 1991-2 6.3.5). The load can occur both sides to the left and the right of the runway. Please see Figure 1. 58.

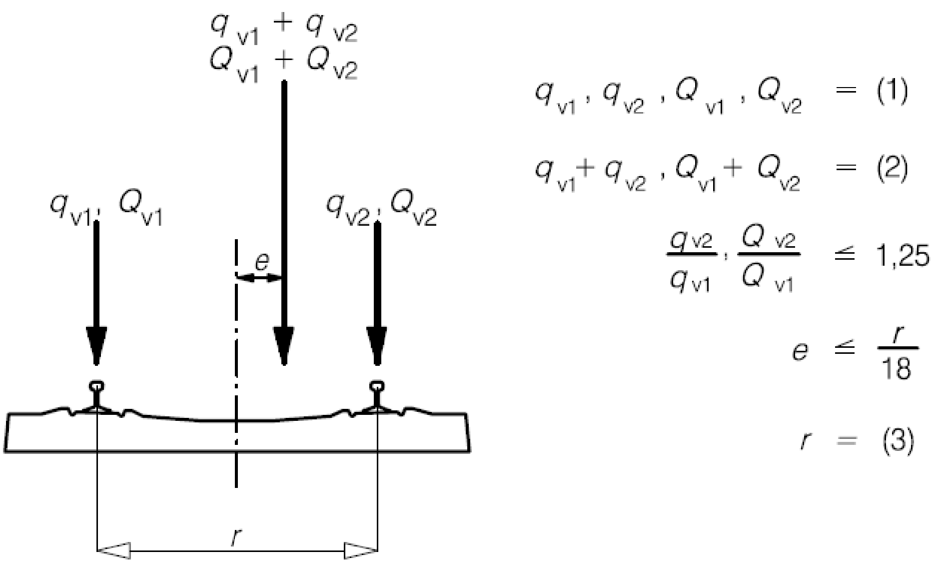


Figure 1. 58 Eccentricity of vertical loads

(1) Evenly distributed load and point loads on each rail as appropriate

(2) LM 71

(3) Transverse distance within wheel loads

The vertical load of the load model acting above the track axis at the level of the centrifugal force acts with an eccentricity to the center of the loaded surface when the track is elevated. This eccentricity must be taken into account in the lateral distribution. It follows from the geometry that the load is always directed towards the rail, which is not elevated.

4.2.9 Eccentricity due to centrifugal force:

In addition to the horizontal load, the centrifugal force also generates a moment around the center of the load surface. This must be considered by an eccentricity of the vertical load. Depending on the effect of the centrifugal force, the associated eccentricity is directed towards the excessive rail. In our case, since the location of the bridge was in a flat area that the route was straight, and consequently, the Radius was **∞**; accordingly, this force would be equal to zero.

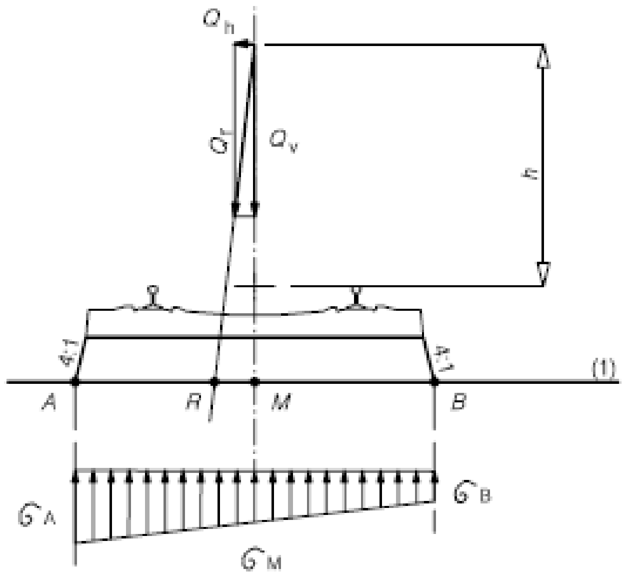
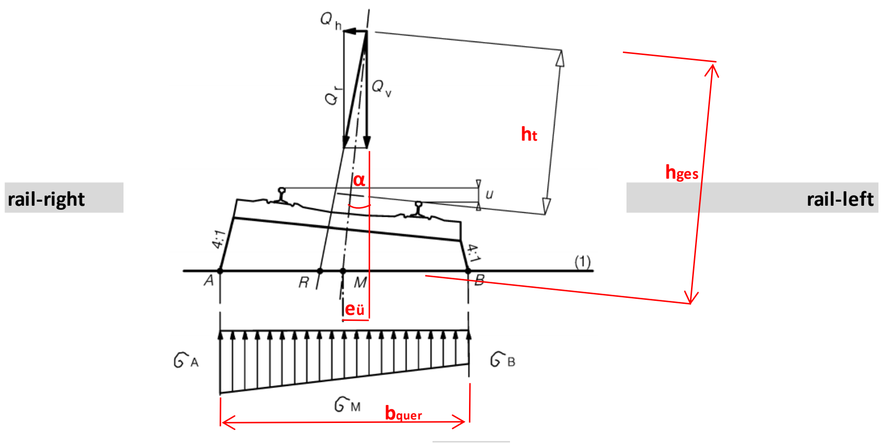


Figure 1. 59 Transverse distribution of performances by the sleepers and ballast without cant

4.2.10 Eccentricity due to wind with traffic:

Analogous to centrifugal force, the wind acting under traffic also generates a moment around the loaded surface. The eccentricity to be considered can be to the right or left of the runway, depending on the direction of the wind. Finally, the eccentricity effect and other factors was calculated in below:



α_{LKF}			= 1.00	-	load class factor
φ			= 1.22	-	Dynamic coefficient
q_{vk}	=	125 / 0,57	= 219.3	kN/m	Vertical load from LM without αLKF and φ
h_{wk}			= 14	kN/m	Line load from wind with traffic
q_{sk}			= 25	kN/m	Nosing force without αLKF
V_e			= 160	km/h	design speed
r_{curve}			= 1E+07	m	Radius of the track curve
f			= 1.0		Factor for centrifugal force determination
s			= 1435	mm	truck width
u			= 0.0	mm	superelevation
r			= 1.50	m	wheelbase
α	=	arcsin(u/s)	= 0.00	°	tilt angles
h_{BF}	=	0,06 + 0,4 / 2	= 0.26	m	vertical distance between Uppers-side ballast and load area

h_{OB}			= 0.80	m	vertical distance between Upper-side of ballast and bottom-side of rail
$b_{sleeper}$			= 2.60	m	sleeper width
$h_{ballast}$			= 0.30	m	average ballast thickness
b_{cross}	=	$b_{sleeper} + 2*(h_{ballast}/4 + h_{BF})$	= 3.27	m	divided width, approximate-determined wisely on a track that is not elevated Spread in gravel 4:1 Spread in concrete 1:1
h_{Deck}			= 0.40	m	Thickness of the Deck
d			= 5.22	m	Height of the wind intake area with traffic
h_t			= 1.80	m	Height of the point of application of the centrifugal force above Upper-side of Rail

DIN EN 1991-2 6.3.5

Eccentricity imprinted:

$e_{impressed}$	≤	r/18	= 0.083	m	imprinted eccentricity

Eccentricity due to superelevation:

h_{ges}	≈	$h_t + h_{BF} + h_{OB}$	= 2.86	m	it is simply assumed that h_{ges} is the distance of the vertical load to the loading plane, measured along the perpendicular to the upper edge of the rail (see picture in the description of the situation).
e	=	$h_{ges}*\sin\alpha$	= 0.000	m	Eccentricity due to superelevation

Eccentricity due to centrifugal force:

q_{tk}	=	$V_e^2*(f*q_{vk}*\alpha_{LKF})/(127*r_{Bogen})$	= 0.0044	kN/m	centrifugal force
m_{tk}	=	$q_{tk}*h_{ges}$	= 0.013	kNm/m	Moment due to centrifugal force, approximately determined with distance h_{ges}

e_t	=	$m_{tk}/(q_{vk}*\alpha_{LKF}*\varphi)$	= 0.000	m	Eccentricity due to centrifugal force

<u>Eccentricity due to wind with traffic:</u>

m_{wk}	=	$h_{wk}*(d/2 - h_{ü}/2)$	= 33.740	kNm/m	Moment due to wind with traffic
e_w	=	$m_{wk}/(q_{vk}*\alpha_{LKF}*\varphi)$	= 0.126	m	Eccentricity due to wind with traffic

<u>Eccentricity due to Nosing force:</u>

m_{sk}	=	$q_{sk}*\alpha_{LKF}*(h_{BF} + h_{OB})$	= 26.500	kNm/m	Moment due to Nosing force
e_s	=	$m_{sk}/(q_{vk}*\alpha_{LKF}*\varphi)$	= 0.099	m	Eccentricity due to Nosing force

maximum eccentricity on raised side, rail-right-

max_e_{br}	=	$e_{impressed} - e_{ü} + e_t + e_w + e_s$	= 0.308	m

maximum eccentricity on NOT excessive side, rail-left-

max_e_{bl}	=	$e_{impressed} + e_{ü} - e_t + e_w + e_s$	= 0.308	m

When calculating in InfoGraph, the load models were applied with the load distribution due to the maximum eccentricity:

<u>Linear Force due to eccentricities:</u>

maximum eccentricity for 125 kN on raised side, rail-right:

σ_A = $(q_{vk}*\alpha_{LKF}*\varphi/ b_{cross})*(1 + (6*max\_e/ b_{cross}))$ = **128.06** kN/m²

σ_B = $(q_{vk}*\alpha_{LKF}*\varphi/ b_{cross})*(1 - (6*max\_e/ b_{cross}))$ = **35.58** kN/m²

maximum eccentricity on NOT excessive side, rail-left:

σ_A = $(q_{vk}*\alpha_{LKF}*\varphi/ b_{cross})*(1 - (6*max\_e/ b_{cross}))$ = **35.58** kN/m²

σ_B = $(q_{vk}*\alpha_{LKF}*\varphi/ b_{cross})*(1 + (6*max\_e/ b_{cross}))$ = **128.06** kN/m²

maximum eccentricity for 62.5 kN on raised side, rail-right:

q_{vk} = 62,5 x 1,0 / 0,57 = **109.64** kN/m

maximum eccentricity for 62.5 kN on raised side, rail-right:

$\sigma_A = (q_{vk} * \alpha_{LKF} * \varphi / b_{cross}) * (1 + (6 * max\_e / b_{cross})) = \mathbf{64.02}$ kN/m²

$\sigma_B = (q_{vk} * \alpha_{LKF} * \varphi / b_{cross}) * (1 - (6 * max\_e / b_{cross})) = \mathbf{17.79}$ kN/m²

maximum eccentricity on NOT excessive side, rail-left:

$\sigma_A = (q_{vk} * \alpha_{LKF} * \varphi / b_{cross}) * (1 - (6 * max\_e / b_{cross})) = \mathbf{17.79}$ kN/m²

$\sigma_B = (q_{vk} * \alpha_{LKF} * \varphi / b_{cross}) * (1 + (6 * max\_e / b_{cross})) = \mathbf{64.02}$ kN/m²

These loads were imported to the model in two different ways. The first was when the eccentricity had been located on the Right Side of the Axe-track (In the program, the loads were named with Right(Out)(1), it means the load at the begging of the span, Right(Out)(3) in middle of span and Right(Out)(5) end of the span.); the second, when it was on the Left Side of the Axe-track (In the program, the loads were named with Right(in)(1), it means the load at the begging of the span, Right(in)(3) in middle of span and Right(in)(5) end of the span.). This loading method also was defined for Left track (Left line). See Figure 1. 60 and Figure 1. 61 for conclusion.

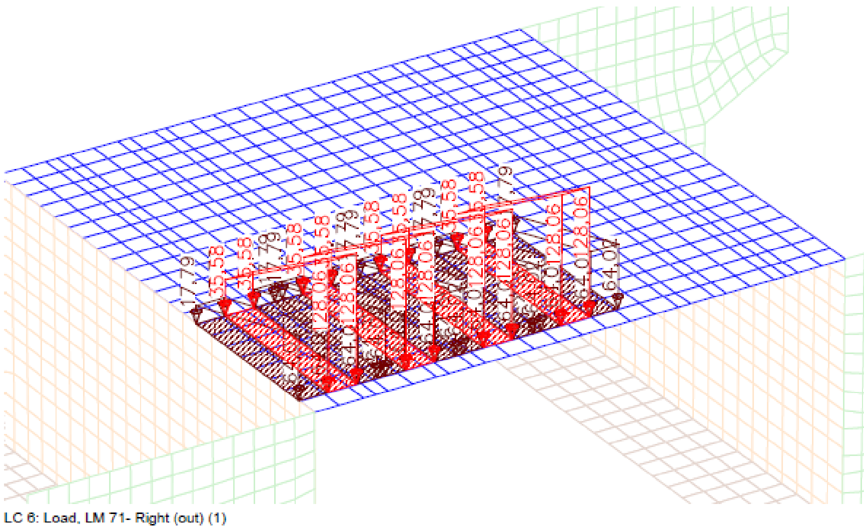


Figure 1. 60 LM 71, Right line, Right (Out)

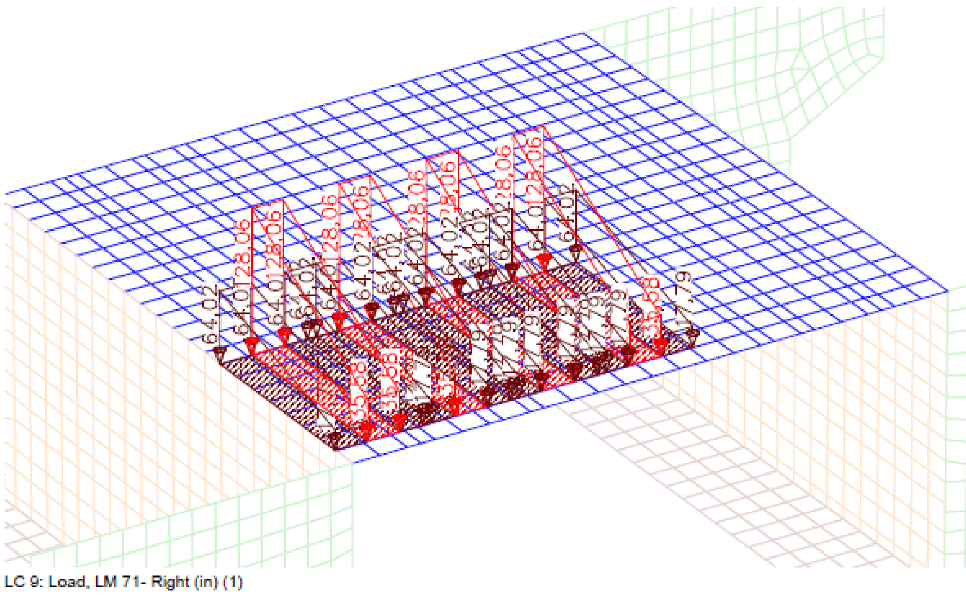


Figure 1. 61 LM 71, Right line, Right (in)

4.2.11 Derailment actions from rail traffic on a railway bridge

The derailment of rail traffic on a railway bridge should be counted as an Accidental case. For this matter, two Design Situations shall be recognized:

- **Design Situation I**: Derailment of railway vehicles, when the derailed vehicles remain in the track section on the bridge deck with vehicles kept by the adjacent rail or an upstanding wall.
- **Design Situation II**: Derailment of railway vehicles, with the derailed vehicles settled on the corner of the bridge.

Regarding the **Design Situation I**, the load can be distributed to an area of 0.45 m with 1.43 m distance by rail axis. Then, based on the below calculations, the amount of 242.67 kN/m²was implied to the model. It should be noted that this must be considered to both sides of each track. Please see Figure 1. 62.

Q_{a1d} = 1,00 x 0,70 x 80 kN/m = 56,00 kN/m

Q_{a1d} = 1,00 x 0,70 x 156 kN/m = 109,20 kN/m

(1) 1.5 times track width: 1.5 x 1.435 m = 2.15 m

(2) Track width: = 1.435 m

(4) $\alpha = 1$

(3) Distribution over the area of 0.45 m for ballasted

$q_{a,i}$ = 156 kN/m×α×0,7 / 0,45 m = 156 kN/m ×1,0 × 0,7 / 0,45 m **= 293,63** kN/m²

$q_{a,i}$ = 80 kN/m×α ×0,7 / 0,45 m = 80 kN/m × 1,0 × 0,7 / 0,45 m = **150,57** kN/m²

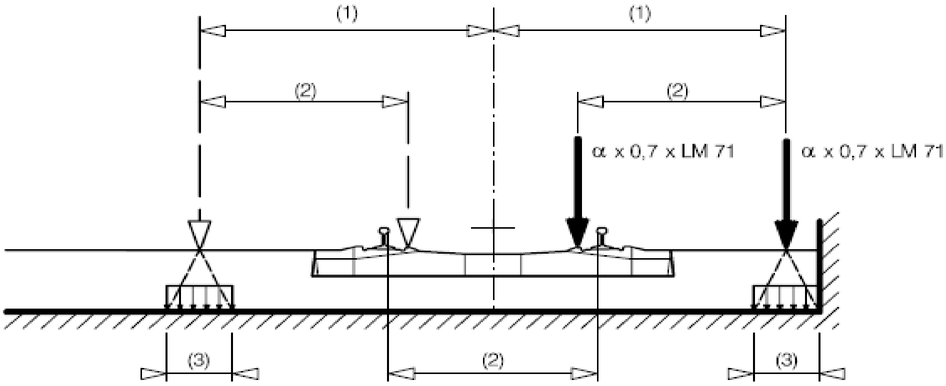


Figure 1. 62 Derailment, Design Situation I

(1) max. 1,5*s* or less if against wall

(2) Track gauge *s*

(3) For ballasted decks the point forces may be assumed to be distributed on a square of side 450mm at the top of the deck.

Then the maximum of loads was modelled to the bridge. Please see Figure 1. 63 and Figure 1. 64.

$$\max \begin{cases} q_{a,i} = \mathbf{293{,}63}\ \text{kN/m}^2 \quad \checkmark \\ \\ q_{a,i} = \mathbf{150{,}57}\ \text{kN/m}^2 \end{cases}$$

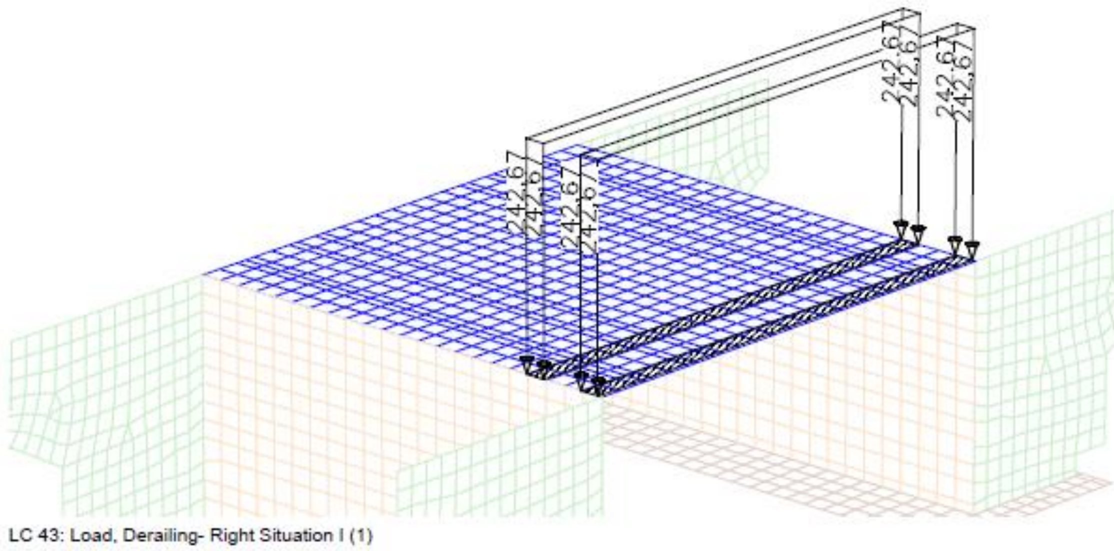


Figure 1. 63 Derailment actions, Design Situation I, Right side, corner location

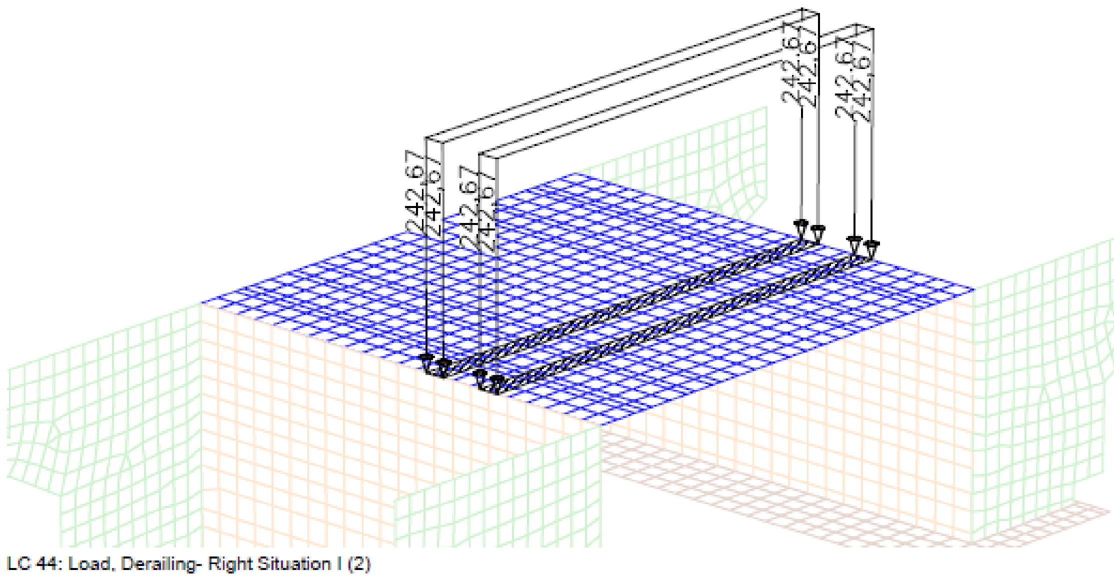


Figure 1. 64 Derailment actions, Design Situation I, Right side, middle location

Regarding the **Design Situation II**, the load can be distributed to an area of 0.**45** m with **1.43** m distance by rail axis. But in this case, the coefficient of **α** is **1.4** based on Euro code (*DIN EN 1991-2 6.7.1*). Then, based on the below calculations, the amount of **485.3** kN/m²was implied to the model. It should be noted that this must be considered to both sides of each track. As an example, please see Figure 1. 65

Q_{a1d} = 1,00 x 0,70 x 80 kN/m = 56,00 kN/m

Q_{a1d} = 1,00 x 0,70 x 156 kN/m = 109,20 kN/m

(1) 1.5 times track width: 1.5 x 1.435 m = 2.15 m

(2) Track width: = 1.435 m

(4) α = 1.4

(3) Distribution over the area of 0.45 m for ballasted

$q_{a,i}$ = 156 kN/m×α×0,7 / 0,45 m = 156 kN/m ×14 × 0,7 / 0,45 m **= 485.3** kN/m²

$q_{a,i}$ = 80 kN/m×α ×0,7 / 0,45 m = 80 kN/m × 1,4 × 0,7 / 0,45 m = **174.2** kN/m²

$$\max \begin{cases} q_{a,i} = \mathbf{485.3}\ \text{kN/m}^2 \quad \checkmark \\ q_{a,i} = \mathbf{174.2}\ \text{kN/m}^2 \end{cases}$$

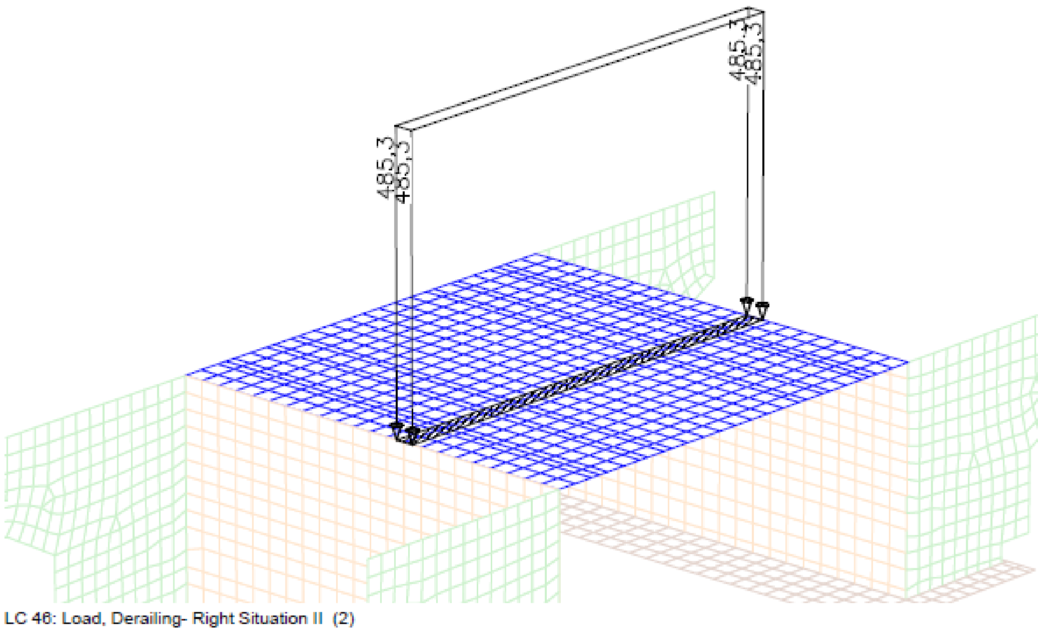


Figure 1. 65 Derailment actions, Design Situation II, Right side, middle location

4.2.12 Traction and Braking Loads

In overall, longitudinal forces are only recognized in track design in case of interactions with bridges, or in terms of the ability of the track to hold thermally Induced forces in continuous welded rail. On Heavy railways, traction and braking forces can be high sufficient to match a vital design case [21].

For this load case, both of Braking and Traction must be calculated, and the maximum must be defined to the model. Hence, based on the Euro code (*DIN EN 1991-2, 6.5.3*), below factors were considered, and **10.1** kN/m^2 was imported to the model. Please see. This load was defined to the model in both directions and also in both tracks separately for better simulation.

$L_{a,b} = 6{,}1$ m influence length for traction and braking effects

$b_{cross} = 2.6 + 2 \times (0.3/4 + 0.06 + 0.4/2) = 3.27$ m Load propagation length

- **Traction force:**

  $Q_{lak,LM71} = 33$ kN/m x 11 m = 363 kN < 1000 kN

  $q_{lak,LM71} = 33$ kN/m / 3,27 m = 10,1 kN/m²

- **Braking force:**

  $Q_{lbk,LM71} = 20$ kN/m x 6.1 m = 122.0 kN ≤ 6000 kN

  $q_{lbk,LM71} = 20$ kN/m / 3,27 m = 6,12 kN/m²

$$\max \begin{cases} q_{lak,LM71} = \mathbf{10.1} \text{ kN/m}^2 & \checkmark \\ q_{lbk,LM71} = \mathbf{6.12} \text{ kN/m}^2 \end{cases}$$

Here should be noted that in this research, three Deck thickness with 40 cm, 55 cm and 80 cm was modelled. In this part, the thickness of 40 cm was considered (as an example for description), and the other models were formed the same as this model.

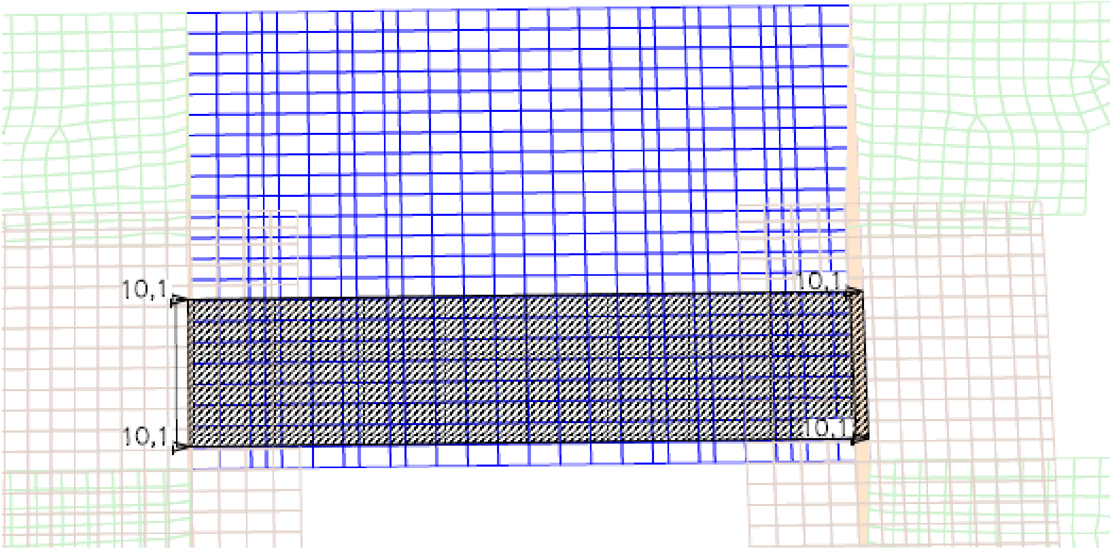


Figure 1. 66 4.1.16 Maximum load of Traction and Braking Loads

4.2.13 Nosing force

From the description of nosing force, it can be said that nosing force is a concentrated imaginative force which does not describe the real lateral force distribution on track. It intends to describe the entire peak lateral force magnitude produced by the whole vehicle.

Supposing long-span railway bridges, the real lateral force is axle forces distributed along the span. Related to concentrated nosing force, whose magnitude corresponds to the total sum of magnitude axle forces, the distributed axle force produces lower structural deformation. The nosing force is conservative opposed to axle forces in terms of structural mechanics [22].

For this load case, based on the Euro code (*DIN EN 1991-2, 6.5.3*), below factors were considered, and **25** kN/m was imported to the model. Please see Figure 1. 67. This load was defined to the model in both directions and in both tracks separately for better simulation.

Q_{sk} = 100 kN

α = 1.0

q_{sk} = 100 kN / 4 m × 1.0 = **25** kN/m   Distribution over 4.0 length (*DIN EN 1991-2/NA; 6.5.2*)

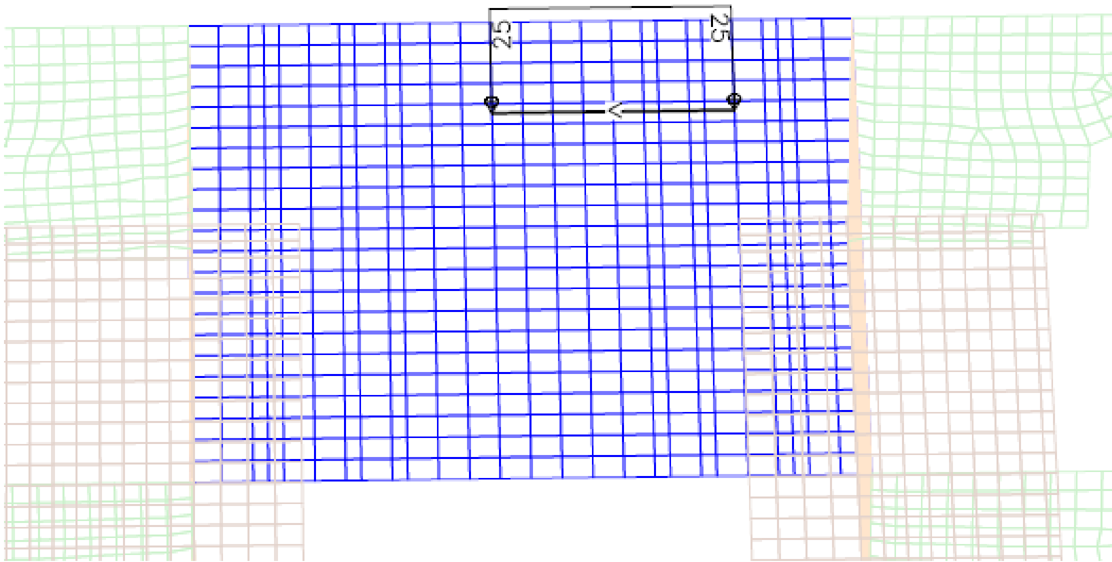


Figure 1. 67 Nosing force, Left track, in + direction

4.2.14 Wind Load

In this part, the Wind load was calculated based on some assumptions like the location of the Bridge, the altitude of the Bridge and with two different situations: Bridge with traffic and Bridge without traffic (*DIN EN 1991-1-4; DIN EN 1991-1-4 NA*).

Then, for first part, the bridge *without traffic* calculations are as follow:

- Wind-Zone: 2
- $z_e < 20.0$ m NA.N.5
- $h_d = 0.40$ m bridge deck thickness
- d = 40 cm + 6 cm + 76 cm = 1.22 m rail, lean concrete and ballast
- b = 12 m
- b/d = 9,8

⇨ w_{ov} = **0,95** kN/m2

- Line load: $h_{wk} = w_{ov} * d =$ **1.16** kN/m
- Moment: $m_{wk} = w_{ov} * d * (d/2 - h_d/2) =$ **0.48** kNm/m

Then after, for second part, the bridge *with traffic* calculations are as follow:

- Wind-Zone: 2
- $z_e < 20.0$ m NA.N.5
- $h_d = 0.40$ m bridge deck thickness
- d = 4 m + 40 cm + 6 cm + 76 cm = 5.22 m train, rail, lean concrete and ballast
- b = 12 m

➢ b/d = 2.29

$\Rightarrow$ w_{ov} = **1.12** kN/m2

➢ Line load: $h_{wk} = w_{ov} * d$ = **5.8** kN/m

➢ Moment: $m_{wk} = w_{ov} * d * (d/2 - h_d/2)$ = **14** kNm/m

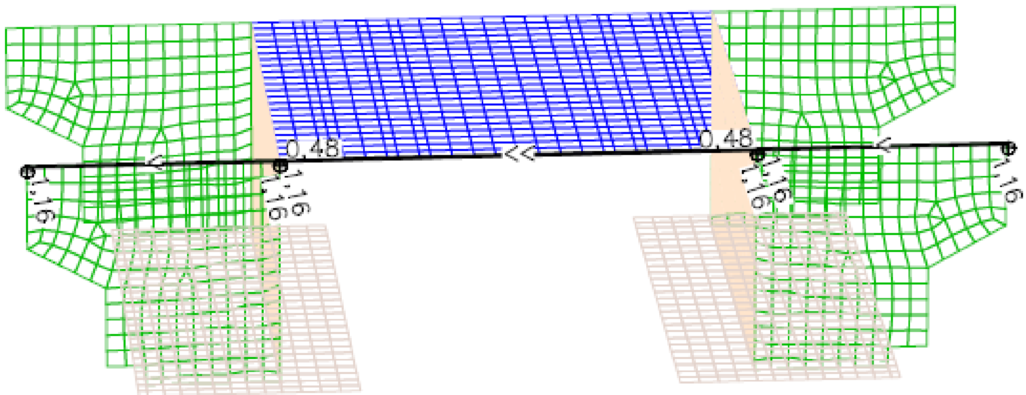


Figure 1. 68 wind load, without traffic

Here should be noted that in this research, three Deck thickness with 40 cm, 55 cm and 80 cm was modelled. In this part, the thickness of 40 cm was considered (as an example for description), and the other models were formed the same as this model.

4.2.15 Temperature Changes in Bridges

Thermal effects are studied in the design of bridges, as temperature changes can lead to restraint stresses and consequent cracking. However, load specifications are in some cases oversimplified or incompatible with new applications, and background studies are limited.

The uniform temperature component depends on the minimum and maximum temperature which a bridge will achieve. This results in a range of uniform temperature changes which, in an unrestrained structure would result in a change in element length.

The temperature actions were calculated based on some assumption via Euro code (*DIN EN 1991-1-5*). Then, four load cases were defined for $TN_{,con}$ (-26K), $TN_{,exp}$ (29K), Load, $TM_{,cool}$ (-8K) and $TM_{,heat}$ (+15K). Then after, these loads and other possible combinations were combined in InfoGraph by Load Case Combination option. To have better simulation, the temperature actions were defined to the FEM program gradually. It means, for example, for TN,con (-26K), the factor was not set to the program continuously. Instead, it was divided into five sections from *-26* to *-5*. The same procedure has been done for the rest actions. Please see Figure 1. 69.

T_0 [°C] = 10
T_{min} [°C] = -24
T_{max} [°C] = 37
$T_{e,max}$ = -16
$T_{e,min}$ = 39
$\Delta T_{M,heat}$ = 15
$\Delta T_{M,cool}$ =

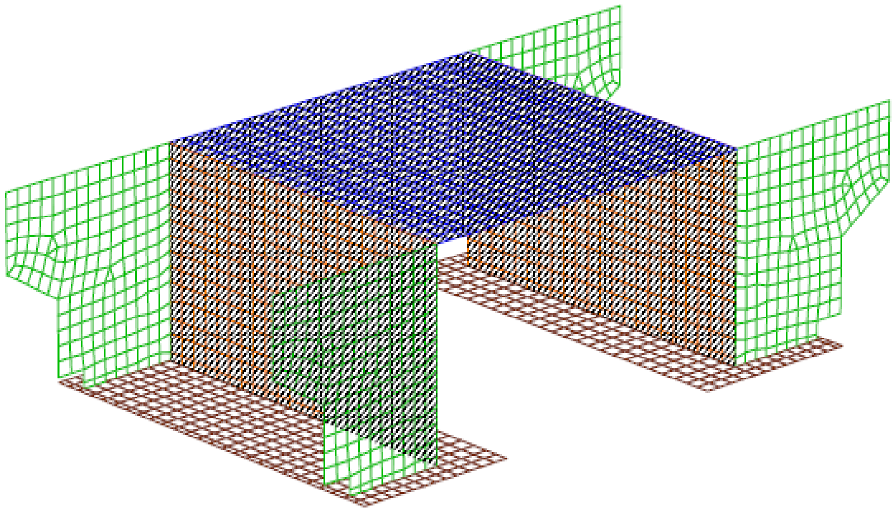
K_{sur} =1.0

$\Delta T_{M,heat}$ = 15
$\Delta T_{M,cool}$ = 8

$\Delta T_{N,exp}$ [k] = $T_{e,max}$ - T_0 = 29
$\Delta T_{N,con}$ [k] = T_0 - $T_{e,min}$ = 26

$\Delta T_M + \Delta_N \times \Delta T_N$ when **ω_N = 0.35**

$\Delta_N \times \Delta T_M + \Delta T_N$ when **ω_M = 0.75**



LC 51: Load, TN,con (-26K)

Figure 1. 69 Temperature Changes in Bridges, Load, TN,con (-26K)

4.3 Load Case Combination

As described before, the load groups were defined based on *EN 1991-2* (Actions on structures - Part 2: Traffic loads on bridges). Consequently, 15 Load groups set in program to cover the whole possible load cases. Besides, in the program via the *Actions DIN EN 1992-2 Bridge Checks* option, the safety factors, Permanent and temporary combination, combination coefficients, Accidental combination, Frequent combination, Quasi-continuous combination, Fatigue combination, Fatigue combination and other reduction factors were defined to the system. In below the Load Case Combinations can be seen:

- Load Case Combination 1, earth pressure
- Load Case Combination 2, LG 11 Right Track
- Load Case Combination 3, LG 11 Left Track
- Load Case Combination 4, LG 12 Right Track
- Load Case Combination 5, LG 12 Left Track
- Load Case Combination 6, LG 13 Right Track
- Load Case Combination 7, LG 13 Left Track
- Load Case Combination 8, LG 14 Right Track
- Load Case Combination 9, LG 14 Left Track
- Load Case Combination 10, LG 21- Left and Right
- Load Case Combination 11, LG 22- Left and Right
- Load Case Combination 12, LG 23- Left and Right
- Load Case Combination 13, LG 24- Left and Right
- Load Case Combination 14, Exceptional situation for Right Side (or Left)
- Load Case Combination 15, LM 71

Each Load Case Combination includes some individual Load Cases that can be combined inclusively or exclusively. This matter can be seen in the appendix with more details.

After combing the loads with an accurate definition based on the Euro Code *EN 1991-2*, the result was imported to the Action part, Permanent and temporary, Accidental, Fatigue, Frequent and Quasi-continuous combinations were defined to the model with the relevant safety factors and other coefficients. This matter can be seen in the appendix with more details as well.

- **Second-Order Theory**

The second-order theory program module is an addition of the finite element program and performs it possible to solve the following elastic problems:

- Beam buckling: In this matter, the beam forces can describe to either the deformed or undeformed beam chord.
- Slab and shell buckling.
- Computation of combined structures according to the second-order theory (equilibrium of the deformed system).

The equilibrium iteration is carried out according to the Lagrangian method based on the total tangential stiffness matrix according to the second-order theory. The iteration method normally converges after a few steps. Stability failures are displayed by the singularity of the global stiffness matrix.

The following requirements must be met:

- Elements are assumed to be straight.
- Section dimensions are small compared with the system dimensions.
- The section form of a component remains unchanged during the structure deformation, meaning that section warping is also not taken into account.
- Deformations are small compared with the other dimensions.
- The mathematical curvature is linearized.
- The material behaves linear-elastically.
- The load is slowly increased to its final value and does not undergo any deviation in direction as a result of the system deformation.

- **Formulation for area and solid elements**

The tangential stiffness matrix generated by formulating the virtual work and variation according to the nodal degrees of freedom.

The following applies:

$$K_T = \int_V dB_L{}^T \sigma dV + \int_V B_0{}^T D B_0 dV + \int_V (B_0{}^T D B_L + B_L{}^T D B_L + B_L{}^T D B_0)\, dV$$

$\boldsymbol{B_0}$ corresponding distortion matrix for small displacements

$\boldsymbol{B_L}$ corresponding distortion matrix for large displacements

$\boldsymbol{D}$ elasticity matrix

$\boldsymbol{\sigma}$ stress vector

written in a simpler form:

K_T	=	K_σ	+	K_0	+	K_L
		geometric matrix		*matrix for small deformations*		*matrix for large deformations*

When presenting an analysis, according to the second-order theory, the impact of the large deformations (third-order theory) are not taken into account.

The matrix K_σ is, in opposition to the linear matrix K_0, a function of the unknown node displacements as these are indirectly contained in the stress vector σ.

5 Data and Analyses

5.1 Data and Out-puts

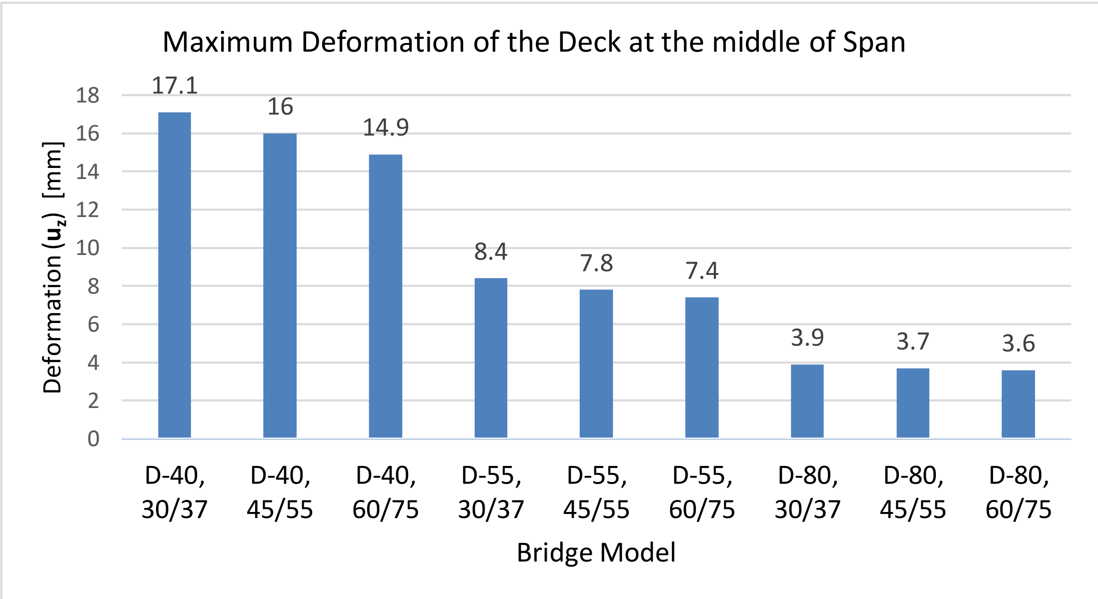
This section describes the results of analyses and simulation. As described in the previous section, the critical points are the Bridge Deck (especially for concrete bridges). For this item, the critical parts are the middle span of the bridge and the its corners. Therefore, the primary data and results are concentrated on these points. The maximum deformation of the structure and also the maximum Moment at the center of the span and maximum Moment and shear forces in supports are incredibly deciding.

As it is clear, the program has considerable information about the model, but the main consideration is on **vertical deformation** and of the Deck at the **centre of the span** and **Corners**. This parameter was named with $\mathbf{u_z}$ in the program. Hence, this item will be elaborated comprehensively. Besides, the **maximum moment** in **the centre of the span** and **the Corners** (named to $\mathbf{m_x}$ max) has been taken out. In addition, the maximum amount of **Steel Reinforcement** (in both direction **X** and **Y**) plays a critical rule in this research from a practical point of view. As a summary, in this section, the maximum deformation ($\mathbf{u_z}$), maximum moment ($\mathbf{m_x}$) and also the maximum amount of Steel Reinforcement (in both direction **X** and **Y**) will be discussed well.

5.1.1 Maximum Deformation (u_z)

The first data that can be achieved from the program is Maximum Deformation. This factor for the whole bridge models is calculated and summarized in the below Chart 1. As it was expected, by increasing the Deck thickness, the maximum Deformation was decreased. And besides, it can be seen that the type of Concrete characteristics had no significant effect on the Deformation. It means, by increasing the Concrete characteristics from C30/37 to 60/75 a forgivable value of the Deformation was decreased.

Chart 1, Maximum Deformation of mid-Deck



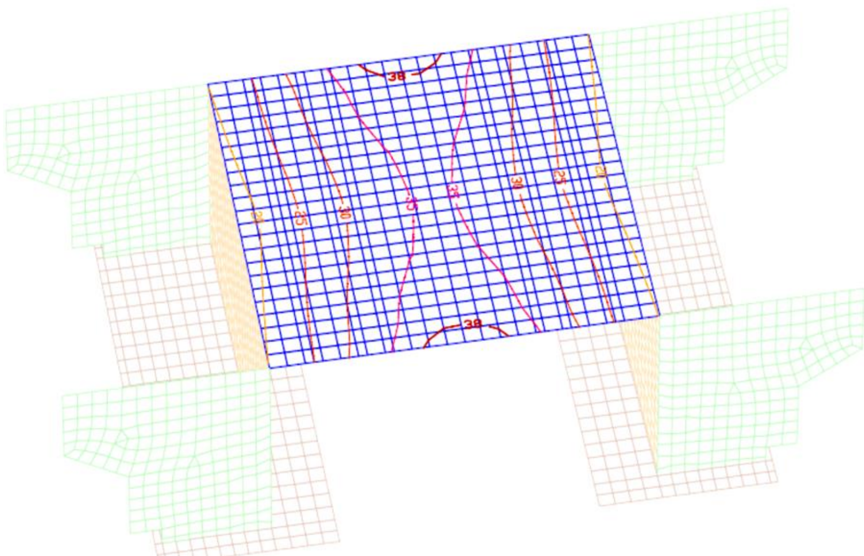
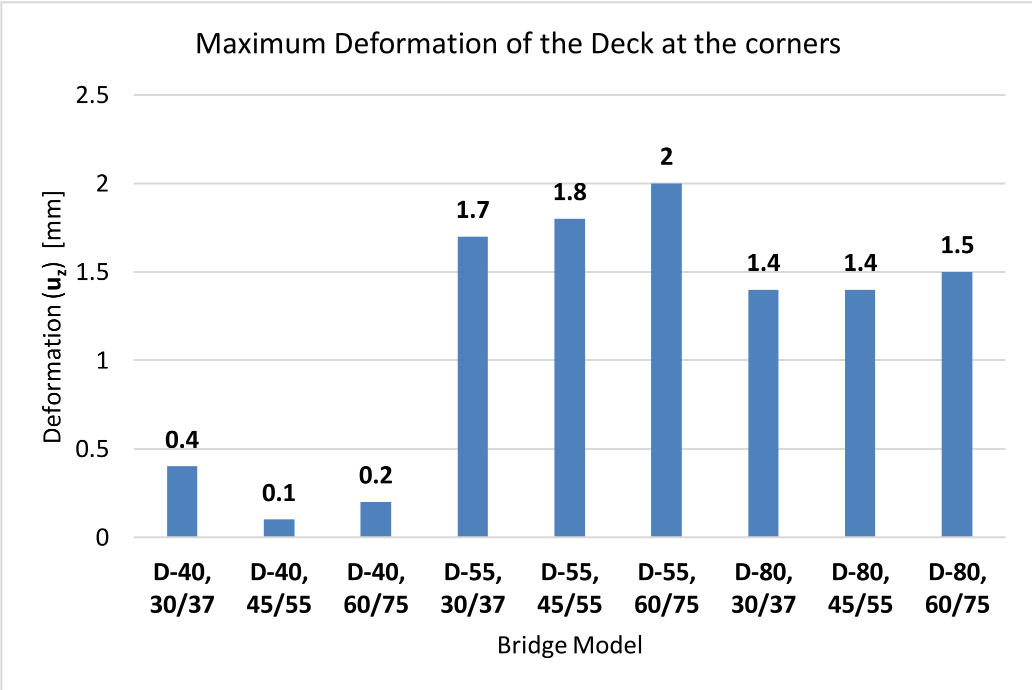


Figure 1. 70 Deformation Contour

The Deformation of Critical points in corners as it can be seen in the Chart 2 has not the same behaviour as the middle of Span. The Chart 2 also shows a fluctuation in both factors Deck thickness and Concrete type. From the thickness point of view, there was no certain pattern. For example, in models with 40 cm thickness, the deformation of the model with a lower strength (D-40, 30/37) was less than the model with higher strength (D-40, 60/75). This behaviour as well was repeated from thickness point on view. It means the D-40 models had deformation of 0.2 mm, while this value was 1.8 mm for D-55 models and 1.4 mm for D-80 models.

It should be added here that maximum deformation means the subtraction of the middle point of the Deck and deformation of Deck corners (Edge) at the same line.

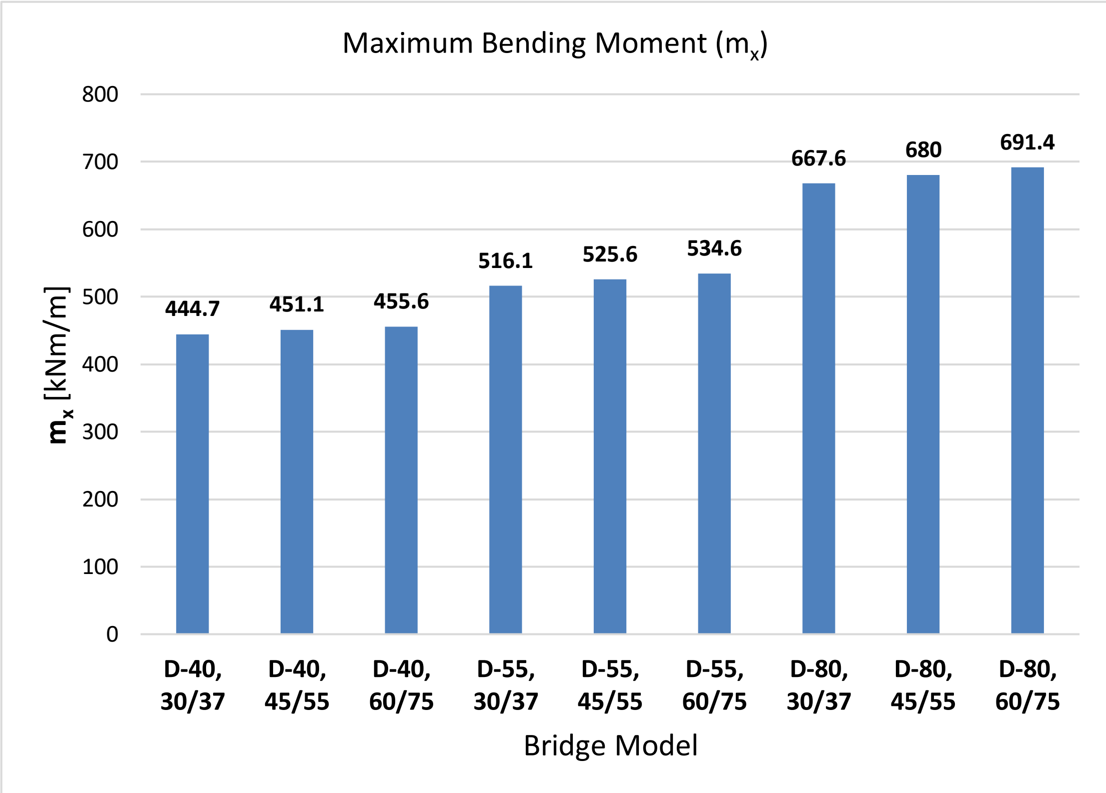
Chart 2 Maximum Deformation in the Corner of Deck



5.1.2 Maximum Bending Moment (m_x)

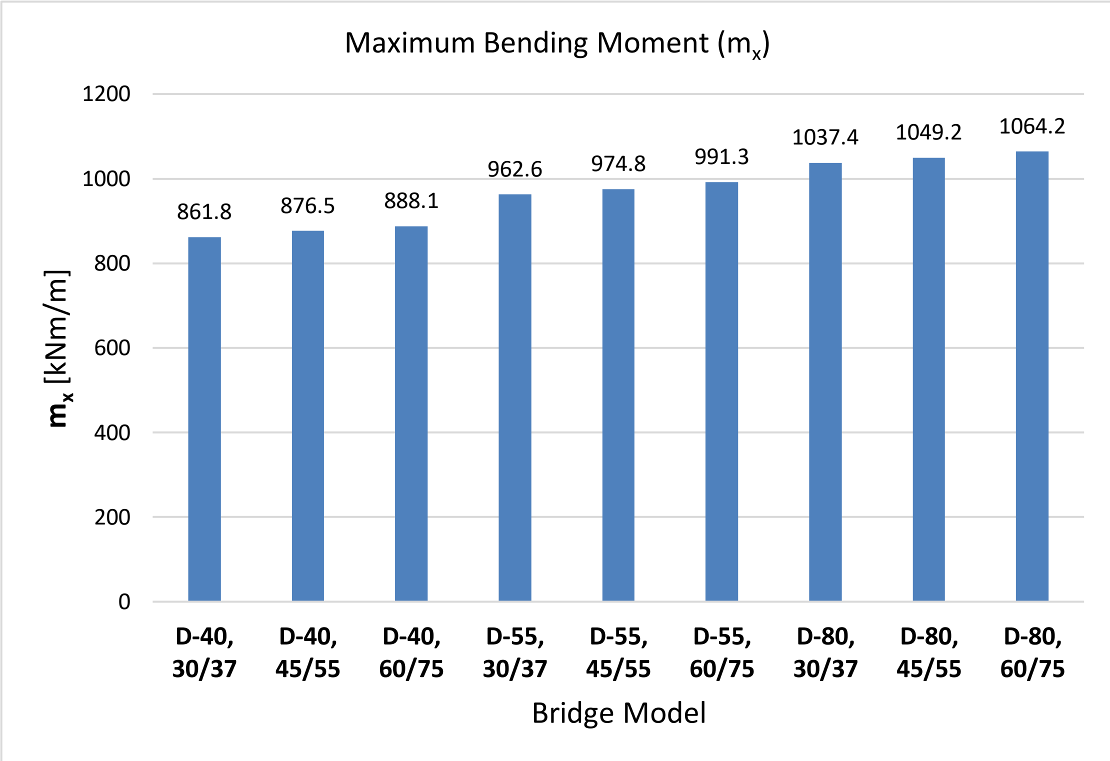
Depending on the selection, the internal forces are calculated in the *Nodes*, in the *Centroid* or the *Side middle*. This factor is significant because, by that, the maximum deformation and the amount of reinforcement can be calculated. After investigating the internal forces and moments, it was achieved that the mx was the critical item for deformation (Even though the other factor like $\mathbf{m_y}$, $\mathbf{m_{xy}}$ and **torsion** and others were calculated and analysed by the program). This item for all bridge models was calculated separately, and the summary summarized in below Chart 3. As it can be seen in the chart, a remarkable increment was happened by increasing the Deck thickness. For instance, in the same concrete C30/37, the amount of $\mathbf{m_x}$ had risen gradually from **444.7** kNm/m to **667.6** kNm/m respectively for the models with 40 cm and 80 cm. That means, by increasing the Deck thickness, the Bearing capacity of the bridge is increased. Regarding the concrete type, it can be mentioned that for the models with the same thickness (for example D-40 cm), the increase of mx is not remarkable. This increase was only around **15** kNm/m only.

Chart 3 Bending Moment (m_x) of mid-Deck



This factor also was checked for the corners. As the Chart 4 describes, this factor gradually increases by increasing the Deck thickness. It means for same concrete type (for example C30/37), the Normal force (n_x)was grown slightly from **861.8** kNm/m to **1037.4** kNm/m. To describe the **m_x** from the *concrete strength* point of view, it can be achieved that this factor remains almost stable in models with the same deck thickness. As can be seen in the Chart 4, around **15** kNm/m is the difference between models with various concrete type.

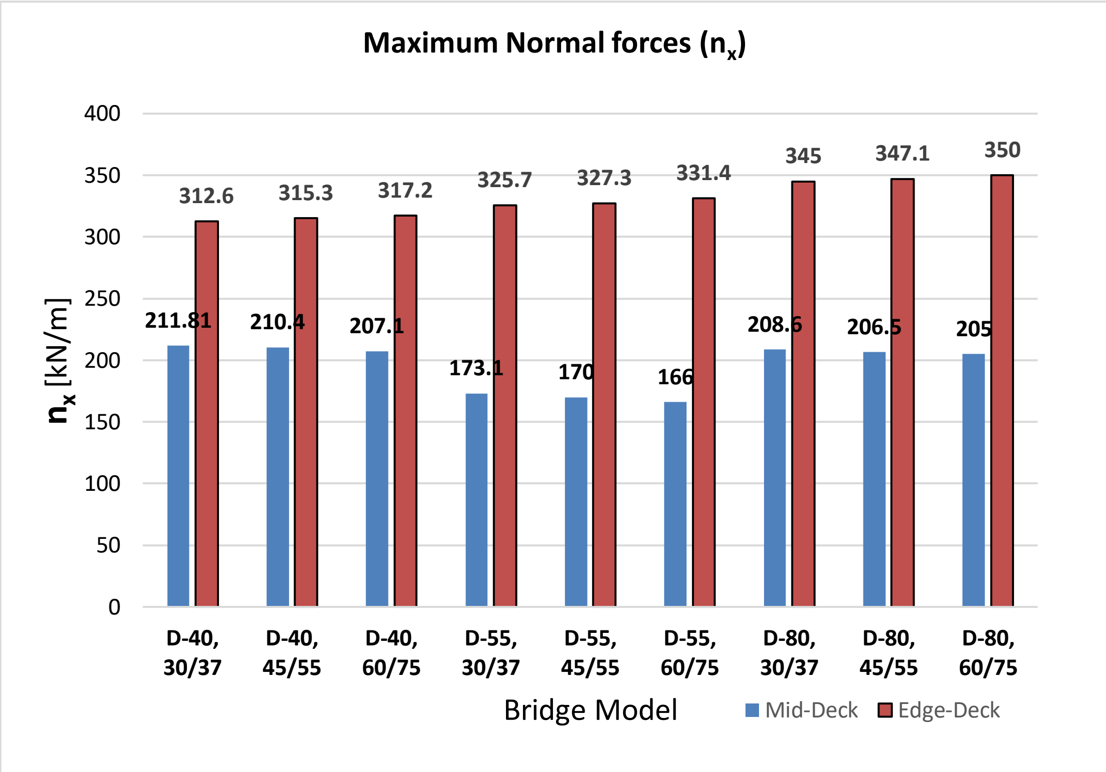
Chart 4 Bending Moment (m_x) of Deck- Corner



5.1.3 Maximum Normal forces (n_x)

Another factor that has a critical rule in the structure deformation and also reinforcement calculation is the *Normal force (n_x)*. This item is the imported force in the **X-direction**. The combination of this force and the relevant bending moment has summarized in the below Chart 5. The Chart 5 illustrates that Deck thickness has no direct effect on this factor. Because it does not follow a continuous process in the value of n_x. As chart shows, the models with lower Deck thickness, 40 cm, have higher Normal force as models with greater Deck thickness, 55cm; while this amount for models with 80 cm Deck thickness is higher than the models with 55 cm Deck thickness.

Chart 5 Normal forces (n_x)



This factor also was checked for the corners. As the Chart 5 describes, this factor gradually increases by increasing the Deck thickness. It means for same concrete type (for example C30/37), the Normal force (n_x)was grown slightly from **312.5** kN/m to **350** kN/m. To describe the Normal force from the *concrete strength* point of view, it can be achieved that this factor remains almost stable in models with the same deck thickness. As can be seen in the Chart 5, only around **3** kN/m is the difference between models with various concrete type.

5.1.4 Steel Reinforcement Quantity

Estimation of steel reinforcement quantity is a fundamental step in assessing the cost of Reinforced Concrete structure simultaneously with other building materials as per construction drawing. Precise calculation of reinforcement in the building plays a vital part in the overall costing of the project.

In this part with below consideration and assumption, the maximum Reinforcement will be calculated for the Bridge Deck for all models one by one. The factors are listed in:

- Concrete cover (C_{top}): **7** cm
- Concrete cover (C_{bottom}): **7** cm
- Yield Strength of Steel: **B500B** (**500** MN/m^2)

The amount of Reinforcements was calculated based on the Maximum Moment (m_x) and Maximum Normal force (n_x) related to the X-Direction *manually* according to the below formulas:

- $\mathbf{K_d} = \mathbf{d}[\text{cm}]/\sqrt{(\mathbf{M_{Eds}}[\text{kNm}]/\mathbf{b}[\text{m}])}$ **(5-1)**
- $\mathbf{A_s} = \mathbf{K_s} \cdot \mathbf{M_{Eds}}[\text{kNm}]/\mathbf{d}[\text{cm}] + \mathbf{N_{Ed}}[\text{kN}]/\mathbf{43{,}5}$ **(5-2)**
- $\boldsymbol{\mu} = \mathbf{M_{Eds}}/(\mathbf{b}.\mathbf{d^2}.\mathbf{fcd})$ **(5-3)**
- $\mathbf{A_s} = \mathbf{1/\sigma_{sd}}[\text{MPa}] \times (\boldsymbol{\omega}.\mathbf{b}.\mathbf{d}.\mathbf{f_{cd}} + \mathbf{N_{Ed}}[\text{kN}])$ **(5-4)**

Then after, the results from formulas was check with the outcome of the InfoGraph program.

Here, as an example for the Reinforcement Calculation, the model D-40, C30/37 and D-80, C60/75 is expressed, and for the rest of models, the results are presented by diagrams and tables. For more details, please see the Appendixes.

*It should be here noted that for concrete C30/37 and C45/55, the formula (*5-5*)has been used, while for concrete C60/75 the formula (*5-6*) has been used since the coefficient and factors were not available for concrete C60/75 in the table. Therefore, an approximation of this formula has been used in this case.*

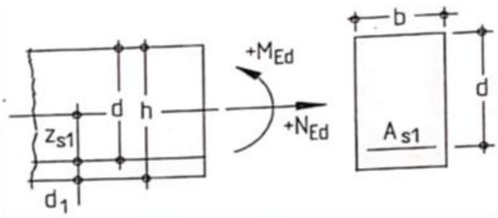
The calculation presents in below:

	h=	**40**	cm
	b=	**1**	m
	d_1=	**7**	cm
	zs_1=	**13**	cm
	d=	**33**	cm
$K_d = d[cm]/\sqrt{(M_{Eds}[kNm]/b[m])}$ $A_s = K_s \,.\, M_{Eds}[kNm]/d[cm] + N_{Ed}[kN]/43{,}5$			
Span	**M_{ed}=**	444.7	kNm
Edges	**M_{ed}=**	861.8	kNm
Span	**N_{ed}=**	211.81	kN
Edges	**N_{ed}=**	312.6	kN
Span	**M_{Eds}=**	417.1647	kNm
Edges	**M_{Eds}=**	821.162	kNm
Span	**K_d=**	1.615697913	
Edges	**K_d=**	1.15159433	
Span	**K_s=**	2.66	From Table 1. 8
Edges	**K_s=**	3.09	From Table 1. 8
Span	**A_s=**	38.49519849	cm^2
Edges	**A_s=**	84.07683053	cm^2

As it can be seen in the calculation , at the end, **A_s= 38.49** cm^2/m Steel reinforcement has been achieved for middle of Deck and also **A_s= 84** cm^2/m achieved for corners of the Deck. This amount of Steel will be sorted in Reinforcement diameter shape in the following parts. This formula (5.2) and calculation as mentioned before, was suitable for concrete with Characteristic strength of concrete C30/37 and C45/55.

Kd for Concrete characteristics

Table 1. 8 K_d for concrete characteristics [27]



k_d für Betonfestigkeitsklasse C									k_s
12/15	16/20	20/25	25/30	30/37	35/45	40/50	45/55	50/60	
14,37	12,44	11,13	9,95	9,09	8,41	7,87	7,42	7,04	2,32
7,90	6,84	6,12	5,47	5,00	4,63	4,33	4,08	3,87	2,34
5,87	5,08	4,55	4,07	3,71	3,44	3,22	3,03	2,88	2,36
4,94	4,27	3,82	3,42	3,12	2,89	2,70	2,55	2,42	2,38
4,38	3,80	3,40	3,04	2,77	2,57	2,40	2,26	2,15	2,40
4,00	3,47	3,10	2,78	2,53	2,35	2,20	2,07	1,96	2,42
3,63	3,14	2,81	2,51	2,29	2,12	1,99	1,87	1,78	2,45
3,35	2,90	2,60	2,32	2,12	1,96	1,84	1,73	1,64	2,48
3,14	2,72	2,43	2,18	1,99	1,84	1,72	1,62	1,54	2,51
2,97	2,57	2,30	2,06	1,88	1,74	1,63	1,53	1,46	2,54
2,85	2,47	2,21	1,97	1,80	1,67	1,56	1,47	1,40	2,57
2,72	2,36	2,11	1,89	1,72	1,59	1,49	1,41	1,33	2,60
2,62	2,27	2,03	1,82	1,66	1,54	1,44	1,36	1,29	2,63
2,54	2,20	1,97	1,76	1,61	1,49	1,39	1,31	1,24	2,66
2,47	2,14	1,91	1,71	1,56	1,44	1,35	1,27	1,21	2,69
2,41	2,08	1,86	1,67	1,52	1,41	1,32	1,24	1,18	2,72
2,35	2,03	1,82	1,63	1,49	1,38	1,29	1,21	1,15	2,75
2,28	1,98	1,77	1,58	1,44	1,34	1,25	1,18	1,12	2,79
2,23	1,93	1,73	1,54	1,41	1,30	1,22	1,15	1,09	2,83
2,18	1,89	1,69	1,51	1,38	1,28	1,19	1,13	1,07	2,87
2,14	1,85	1,65	1,48	1,35	1,25	1,17	1,10	1,05	2,91
2,10	1,82	1,62	1,45	1,33	1,23	1,15	1,08	1,03	2,95
2,06	1,79	1,60	1,43	1,30	1,21	1,13	1,07	1,01	2,99
2,03	1,75	1,57	1,40	1,28	1,19	1,11	1,05	0,99	3,04
1,99	1,72	1,54	1,38	1,26	1,17	1,09	1,03	0,98	3,09

*It should be here noted that the value of **K_d** for concrete C30/37 was **1.15** for the Edge of Deck, while the minimum available value in* Table 1. 8 *was **1.26**. Hence, as an assumption and for more simplification, the minimum amount of **1.26** was considered as **K_d** supposing corners of Deck, that, consequently the value of **3.09** was achieved for **Ks**.*

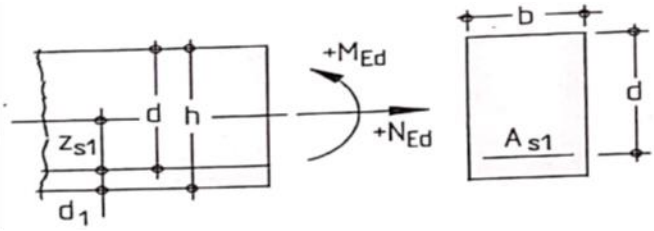
The calculation for model with concrete C60/75 presents in below:

h=	**40**	cm
b=	**1**	m
d_1=	**7**	cm
zs_1=	**13**	cm
d=	**33**	cm
α_{cc}=	**0.85**	
γ_c=	**1.5**	
f_{ck}=	**60**	N/mm²
$f_{cd}=\alpha_{cc}*f_{ck}/\gamma_c$	**34**	N/mm²

$\mu = M_{Eds}/(b.d^2.f_{cd})$ $A_s = 1/\sigma_{sd}[MPa]\times(\omega.b.d.f_{cd}+N_{Ed}[kN])$			
Span	**M_{Ed}=**	455.6	kNm
Edges	**M_{Ed}=**	888.1	kNm
Span	**N_{Ed}=**	207.1	kN
Edges	**N_{Ed}=**	317.2	kN
Span	**M_{Eds}=**	428.677	kNm
Edges	**M_{Eds}=**	846.864	kNm
Span	**μ=**	0.115777292	
Edges	**μ=**	0.228721439	
Span	**ω=**	0.1285	Table 1.9
Edges	**ω=**	0.2665	Table 1.9
Span	**σ_{sd}=**	435	MPa
	σ_{sd}=	435	MPa
Span	**A_s=**	37.90505747	cm²
Edges	**A_s=**	76.03057471	cm²

As can be seen in the above calculation , at the end, **A_s= 37.9** cm^2/m Steel Reinforcement has been achieved for middle of Deck and also **A_s= 76** cm^2/m achieved for corners of the Deck. This amount of Steel will be sorted in Reinforcement diameter shape in the following parts. This formula (5.4) and calculation as mentioned before, was suitable for concrete with Characteristic strength of concrete C60/75.

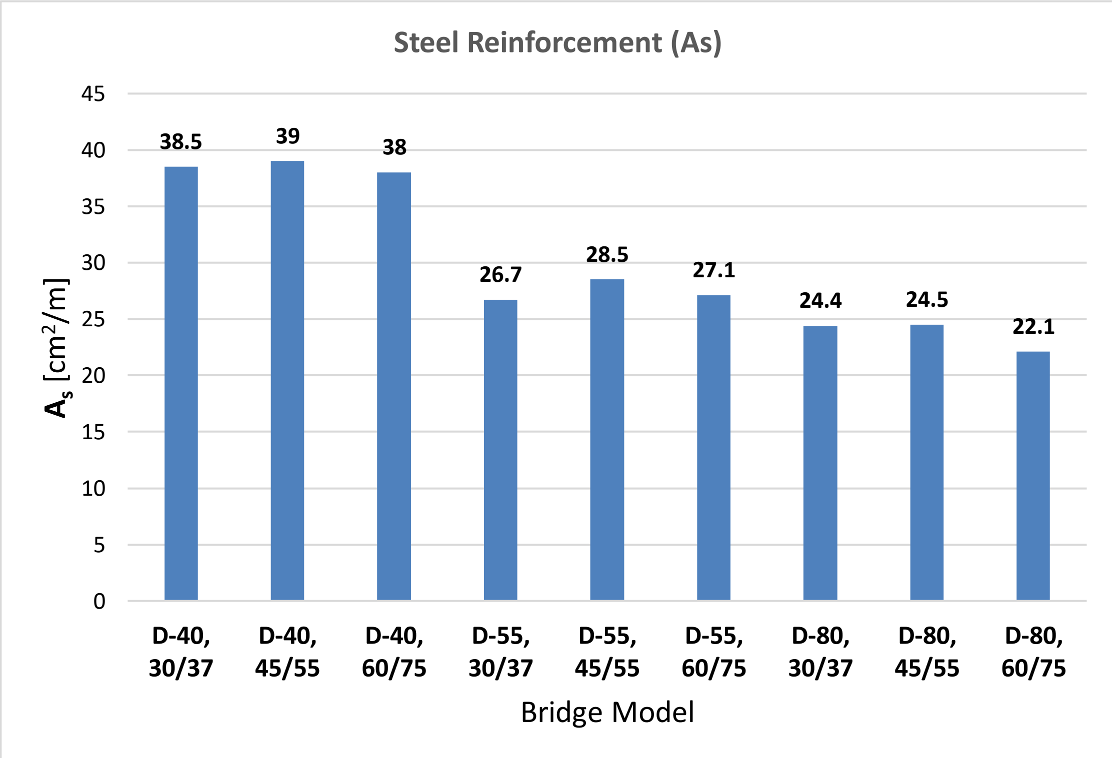
Table 1. 9 μ_{Eds} for concrete characteristics [27]



μ_{Eds}	ω	$\xi = \frac{x}{d}$	$\zeta = \frac{z}{d}$	ε_{c2} in ‰	ε_{s1} in ‰	σ_{sd} 1) in MPa BSt 500
0,01	0,0101	0,030	0,990	−0,77	25,00	435
0,02	0,0203	0,044	0,985	−1,15	25,00	435
0,03	0,0306	0,055	0,980	−1,46	25,00	435
0,04	0,0410	0,066	0,976	−1,76	25,00	435
0,05	0,0515	0,076	0,971	−2,06	25,00	435
0,06	0,0621	0,086	0,967	−2,37	25,00	435
0,07	0,0728	0,097	0,962	−2,68	25,00	435
0,08	0,0836	0,107	0,956	−3,01	25,00	435
0,09	0,0946	0,118	0,951	−3,35	25,00	435
0,10	0,1057	0,131	0,946	−3,50	23,29	435
0,11	0,1170	0,145	0,940	−3,50	20,71	435
0,12	0,1285	0,159	0,934	−3,50	18,55	435
0,13	0,1401	0,173	0,928	−3,50	16,73	435
0,14	0,1518	0,188	0,922	−3,50	15,16	435
0,15	0,1638	0,202	0,916	−3,50	13,80	435
0,16	0,1759	0,217	0,910	−3,50	12,61	435
0,17	0,1882	0,232	0,903	−3,50	11,56	435
0,18	0,2007	0,248	0,897	−3,50	10,62	435
0,19	0,2134	0,264	0,890	−3,50	9,78	435
0,20	0,2263	0,280	0,884	−3,50	9,02	435
0,21	0,2395	0,296	0,877	−3,50	8,33	435
0,22	0,2528	0,312	0,870	−3,50	7,71	435
0,23	0,2665	0,329	0,863	−3,50	7,13	435
0,24	0,2804	0,346	0,856	−3,50	6,60	435
0,25	0,2946	0,364	0,849	−3,50	6,12	435
0,26	0,3091	0,382	0,841	−3,50	5,67	435
0,27	0,3239	0,400	0,834	−3,50	5,25	435
0,28	0,3391	0,419	0,826	−3,50	4,86	435
0,29	0,3546	0,438	0,818	−3,50	4,49	435
0,30	0,3706	0,458	0,810	−3,50	4,15	435
0,31	0,3869	0,478	0,801	−3,50	3,82	435
0,32	0,4038	0,499	0,793	−3,50	3,52	435
0,33	0,4211	0,520	0,784	−3,50	3,23	435
0,34	0,4391	0,542	0,774	−3,50	2,95	435
0,35	0,4576	0,565	0,765	−3,50	2,69	435
0,36	0,4768	0,589	0,755	−3,50	2,44	435
0,37	0,4968	0,614	0,745	−3,50	2,20	435
0,38	0,5177	0,640	0,734	−3,50	1,97	395
0,39	0,5396	0,667	0,723	−3,50	1,75	350
0,40	0,5627	0,695	0,711	−3,50	1,54	307

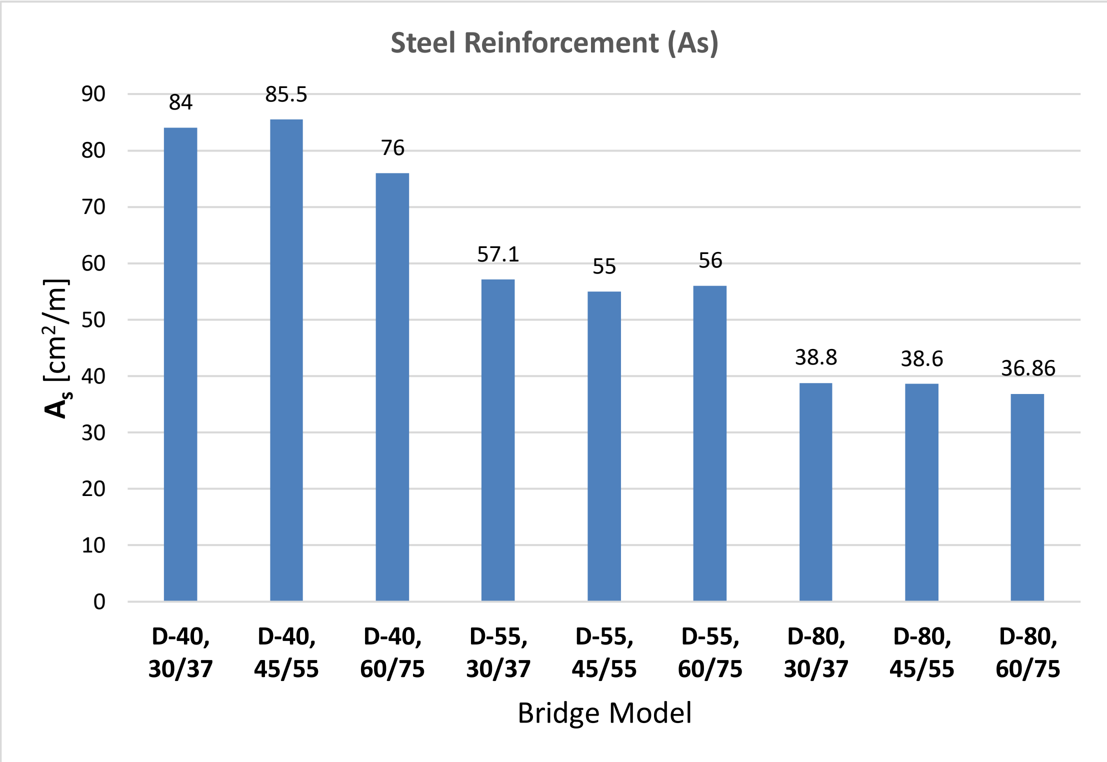
After analyzing the whole models, here, the outcomes can be compared. As Chart 6 illustrates, the amount of Steel Reinforcement significantly was reduced by increasing the Bridge Deck thickness. This issue was happened due to Moment Bearing Capacity. Since, as it scientifically accepted, by expanding the Cross-section, the required Steel Reinforcement is reduced. For this reason, the **A_S** was decreased from **38.5** cm^2/m to **24.4** cm^2/m respectively for models D-40, C30/37 and D-80, C60/75.

Chart 6 Steel Reinforcement (As), mid-Deck



This calculation was fulfilled for Bridge corners correctly. Here, the outcomes can be compared. As Chart 7 demonstrates, the quantity of Steel Reinforcement dramatically was reduced by increasing the Bridge Deck thickness. This issue was happened due to Moment Bearing Capacity. Since, as it scientifically accepted, by expanding the Cross-section, the required Steel Reinforcement is reduced. For this reason, the **A_S** was decreased from **84** cm2/m to **38.8** cm2/m respectively for models D-40, C30/37 and C60/75.

Chart 7 Steel Reinforcement (As), Deck- Corners



A standard and practical Reinforcement diameter that mainly used in constructions is the bar with **16** mm (**16 Φ**) or **20** mm (**20 Φ**) in distance of **10** cm. Therefore, the designer attempts to utilize this number in the design procedure. But as the table shows, the Reinforcements are over-size that are not practical in implementation. Or, if the designer tries to reduce the distance between the bars to cover the demanded Steel, the method will fail due to standards for distance, or, in other words, 5 cm or less distance between the bars would not be practical in implementation. For example, for model D-40, C30/37, 84 cm2/m was required, that, to cover this amount of Steel the bar-size **2×28Φ15** was selected. But as discussed, this amount of Steel is not worthwhile and practical.

Table 1. 10 Recommended Reinforcement Size for mid-Deck

Model	Rein. Size (Φ)
D-40, 30/37	20Φ10+10Φ10
D-40, 45/55	20Φ10+10Φ10
D-40, 60/75	20Φ10+10Φ10
D-55, 30/37	20Φ10
D-55, 45/55	20Φ10
D-55, 60/75	20Φ10
D-80, 30/37	20Φ12,5
D-80, 45/55	20Φ12,5
D-80, 60/75	20Φ12,5

Table 1. 11 Recommended Reinforcement Size for Deck Corners

Model	Rein. Size (Φ)
D-40, 30/37	2×28Φ15
D-40, 45/55	2×28Φ15
D-40, 60/75	2×28Φ15
D-55, 30/37	25Φ10+12Φ10
D-55, 45/55	25Φ10+12Φ10
D-55, 60/75	25Φ10+12Φ10
D-80, 30/37	20Φ10+10Φ10
D-80, 45/55	20Φ10+10Φ10
D-80, 60/75	20Φ10+10Φ10

From a practical point of view, this amount of Steel is enormous. As can be seen in the Table 1. 11, for construction and implementation as well, the contracture will face faced up with the problem in the implementation step. Because in this way the Optimum Distribution of Reinforcement is disordered. And besides, from the economy point of view, it is not profitable. Therefore, for this problem, another solution or design should be considered. The resolution will be discussed in succeeding chapter.

5.2 Solution Method

In the last part, it was seen that the amount of Reinforcement was dramatically high, and that caused some problem practically by the disordering the Optimum Distribution of the Reinforcement; also, that method was not advantageous. This issue mainly was happened on Deck corners. Therefore, it could be understood that the cross-section at the corners was not enough to bear the Bending moments. Hence, the new method was focused on Deck Edge cross-section. It should be added that changing the concrete was not a proper solution, because three different concretes were experimented before.

The considered decision matter was using and changing the Physical dimensions of Deck. It means, instead of a uniform Slab, a **Haunched Deck** was selected. By this way, the maximum Moment capacity of the Bridge at corners will be increased, that consequently the amount of Steel Reinforcement will be decreased. In this method, however, to avoid Stress Concentration, the Cross-section of the Deck at corners was changed gradually from **40** cm (started 2.75 m away from Deck) to **85** cm at the Corners (Please see Figure 1. 71 and Figure 1. 72.).

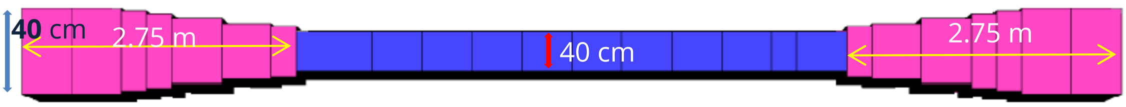


Figure 1. 71 Haunched Deck cross-section

The new method was applied to the whole models, and the Deck was changed to the innovative cross-section. Hence, **nine** models with Haunched Deck created again to analyses. The entire situation, like Load Cases, Concrete Characteristics, Critical points and others were precisely the same as the previous designs.

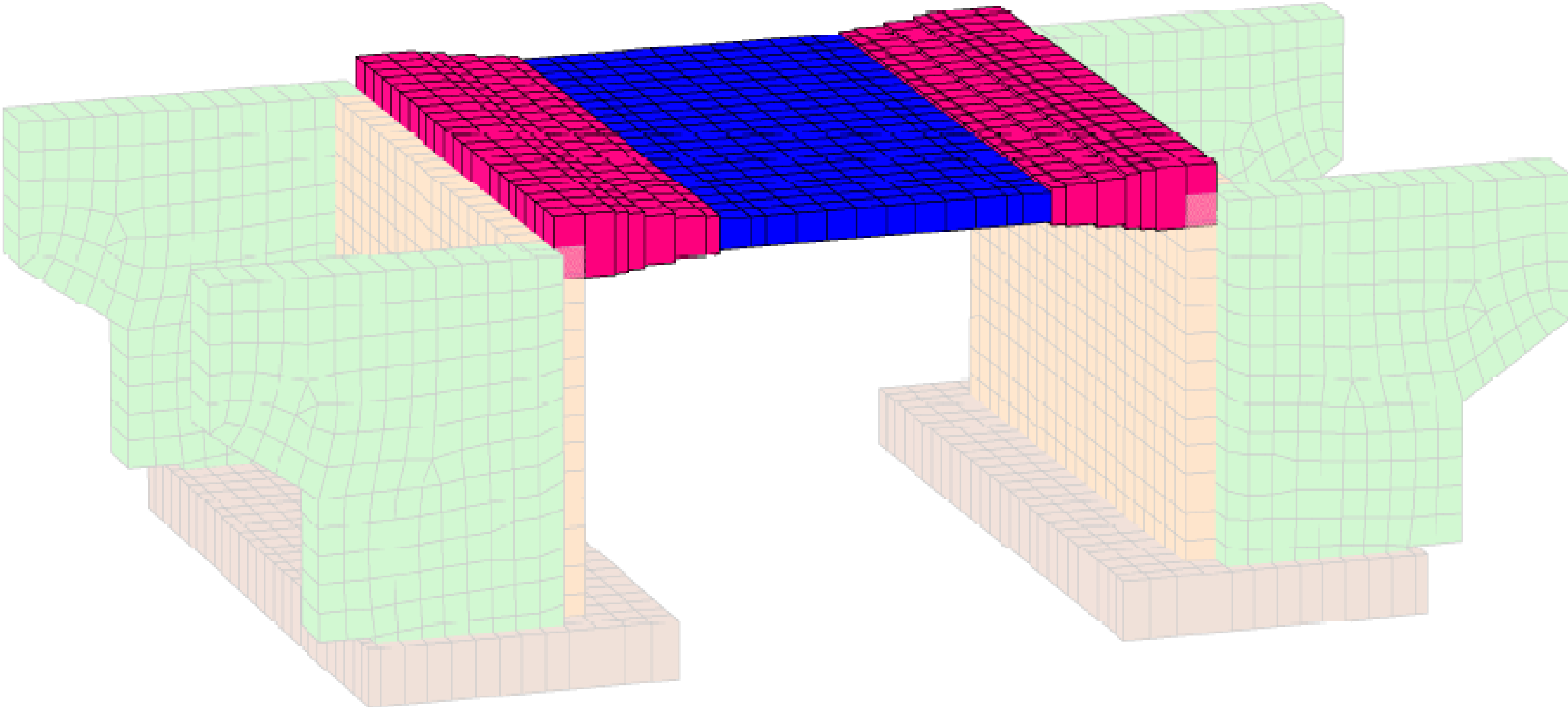


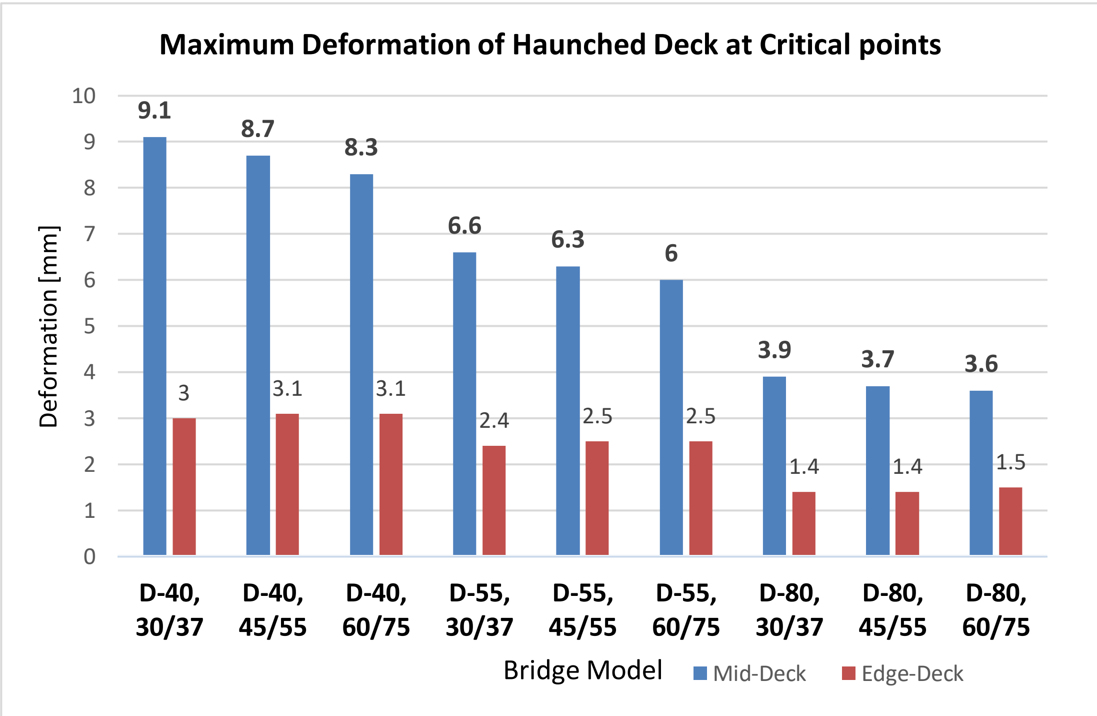
Figure 1. 72 Haunched Deck Bridge

The result of the new models was summarized below. Then after, two methods will be compared for the conclusion.

5.2.1 Maximum Deformation (u_z) in Haunched Bridge

In Chart 8, the maximum deformation of Deck is presented. The Chart 8 illustrates that the increase in Deck thickness gradually decreases the deformation. The results show a remarkable change in the deformation. For example, model **D-40, C30/37** in regular bridge had **17.1** mm deformation, while this value is **9.1** mm in the Haunched bridge model.

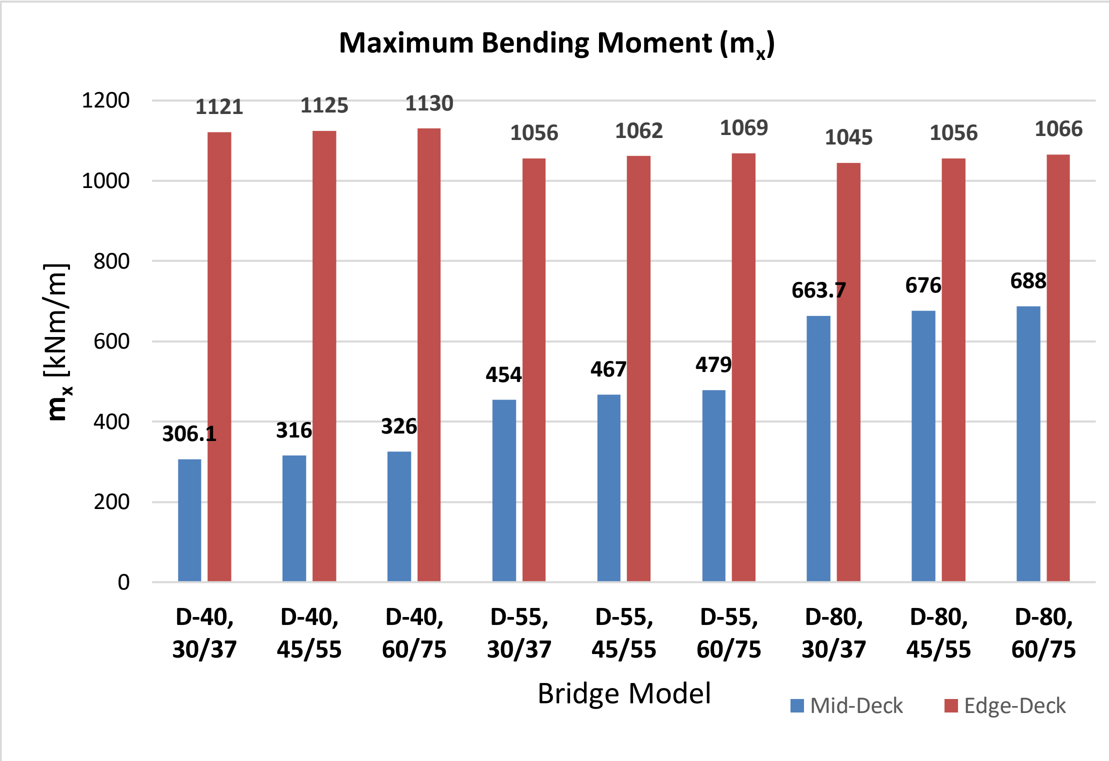
Chart 8 Maximum Deformation of Haunched Deck



5.2.2 Maximum Bending Moment (m_x) in Haunched Bridge

In Chart 9, the maximum Bending moment of Deck is presented. The Chart 9 illustrates that the rise in Deck thickness gradually increases the Bending moment capacity. For example, model **D-40, C30/37** in the Haunched bridge had **1121** kNm/m bending moment, while this value is **861.8** kNm/m in Bridge with Uniform Deck.

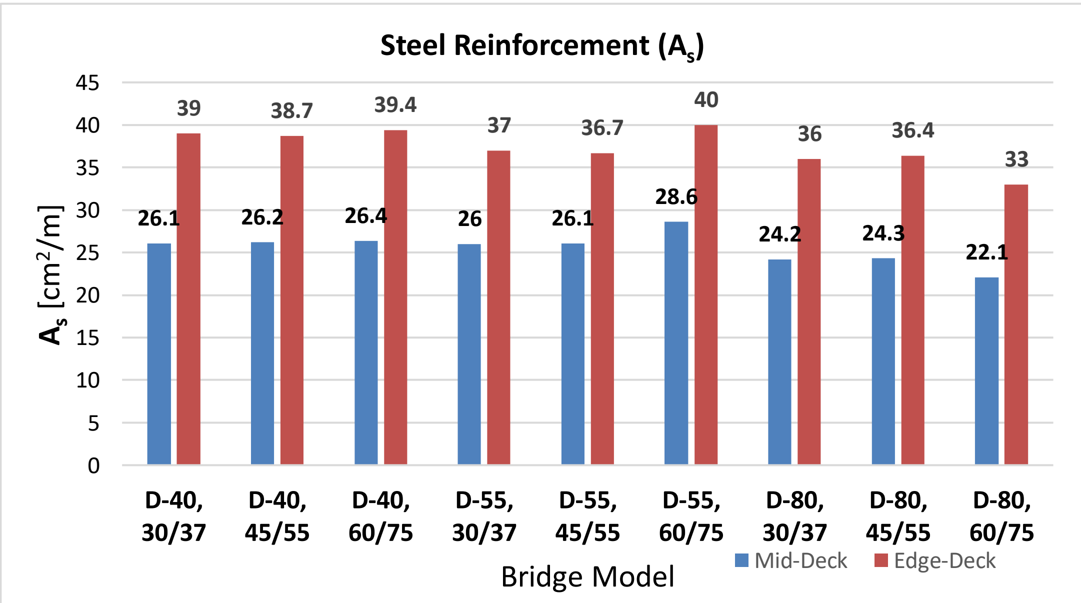
Chart 9 Bending Moment in Haunched Deck



5.2.3 Steel Reinforcement Quantity in Haunched Bridge

In Chart 10, the maximum Steel reinforcement of Deck is displayed. Chart 10 demonstrates that a rise in Deck thickness remarkably decreases the amount of Steel Reinforcement. For example, model D-40, C30/37 in the Haunched bridge has only $\mathbf{A_s}$= **39** cm^2/m, while this amount is $\mathbf{A_s}$= **84** cm^2/m in the Uniform Bridge.

Chart 10 5.2.3 Steel Reinforcement in Haunched Bridge



5.3 Analysis and Comparison of both Methods

In this part, the result of analysis in the first method and the second method are compared together to realize the differences in between.

5.3.1 Deformation (u_z)

From the Deformation point of view, it can be achieved that the increase in Deck thickness was a decent idea to decrease the deformation of Bridge Deck. As the Chart 11 illustrates, the deformation in Haunched Bridge is **47**% less than the Uniform Bridge. This influence can be due to the improvement in the Geometric Attributes of Bridge. It means, by an increase in the Corner thickness, the deformation was acceptably decreased.

Chart 11 Maximum Deformation comparison in Mid-Deck

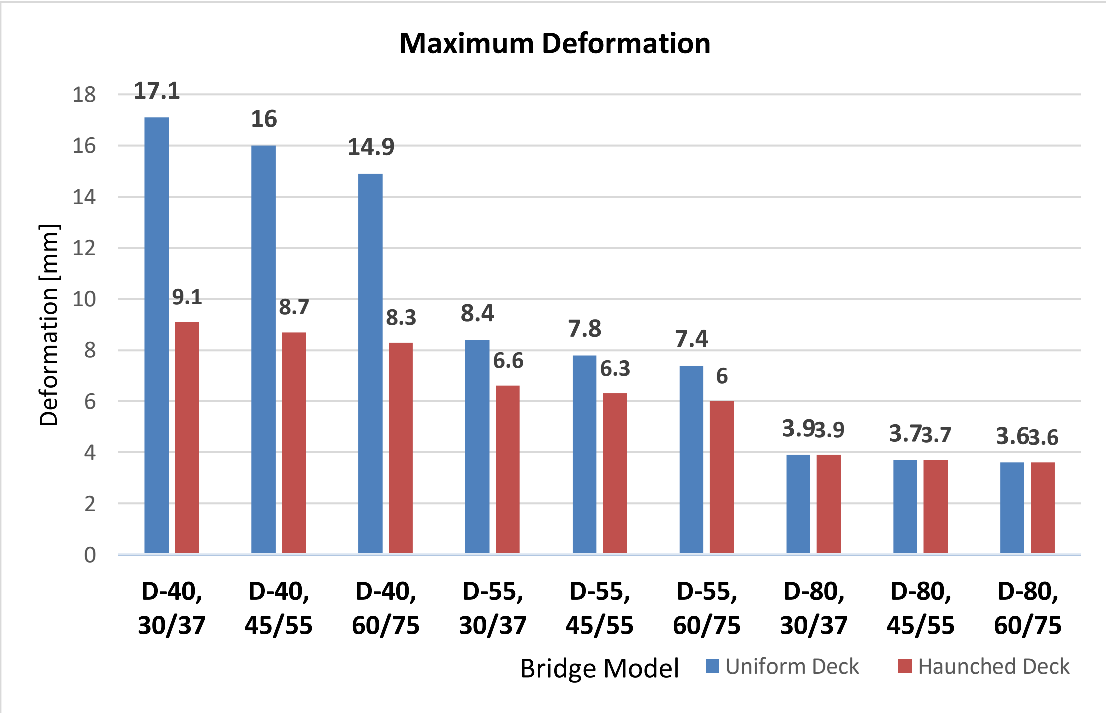
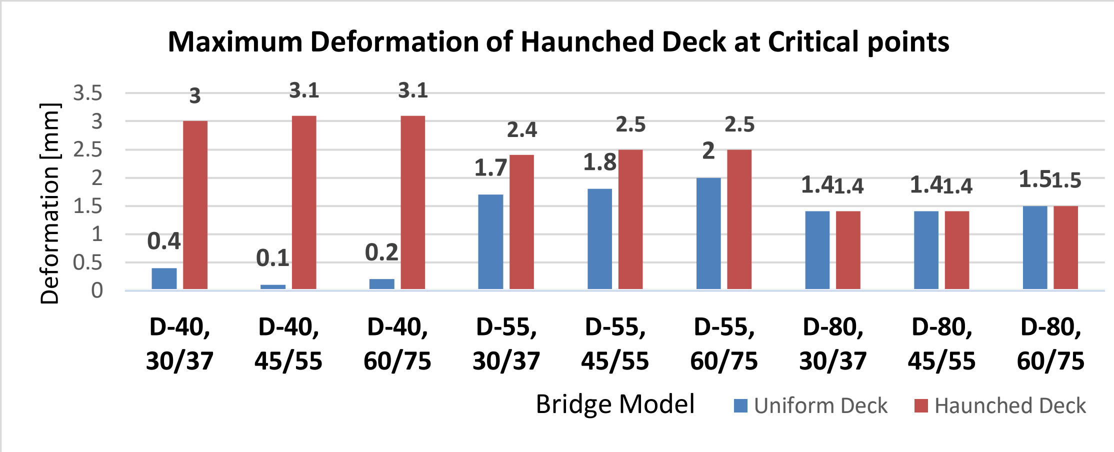


Chart 12 shows that the deformation in corners has the same behaviour. As can be seen, in models with **40** cm thickness, this factor is only **3** mm in Haunched Deck; that it is forgivable. For the rest models, it is mostly the same and ignorable.

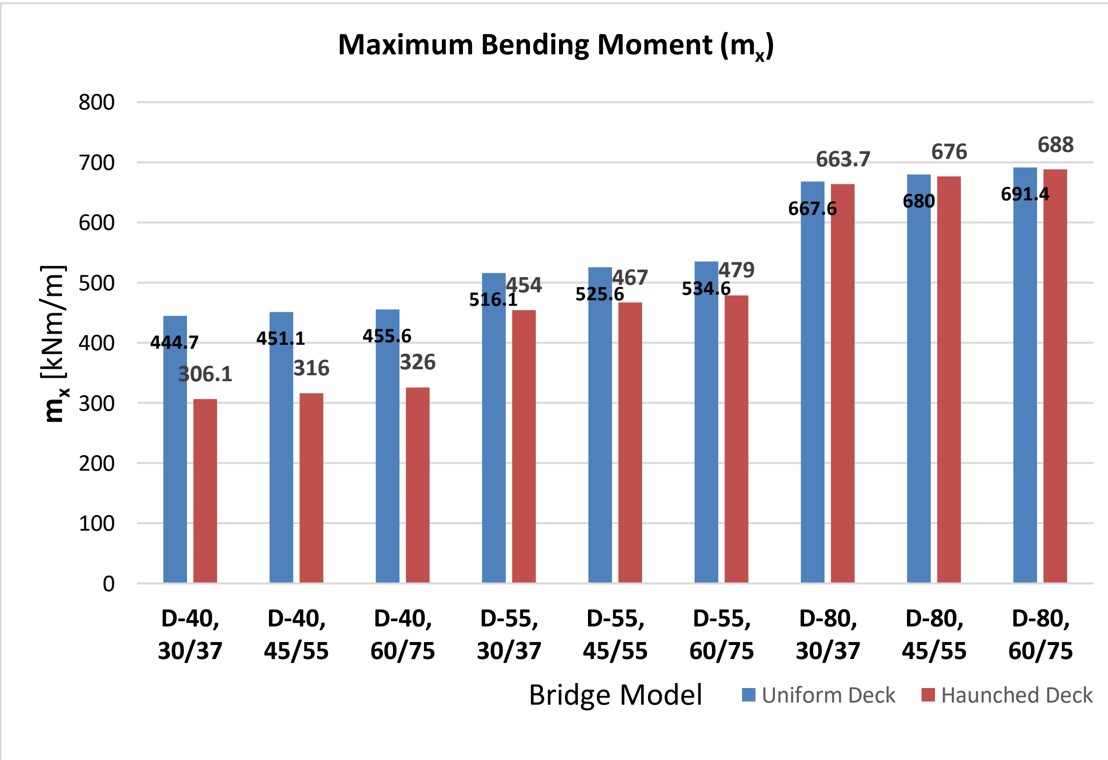
Chart 12 Maximum Deformation comparison in Deck Corner



5.3.2 Maximum Bending Moment (m_x)

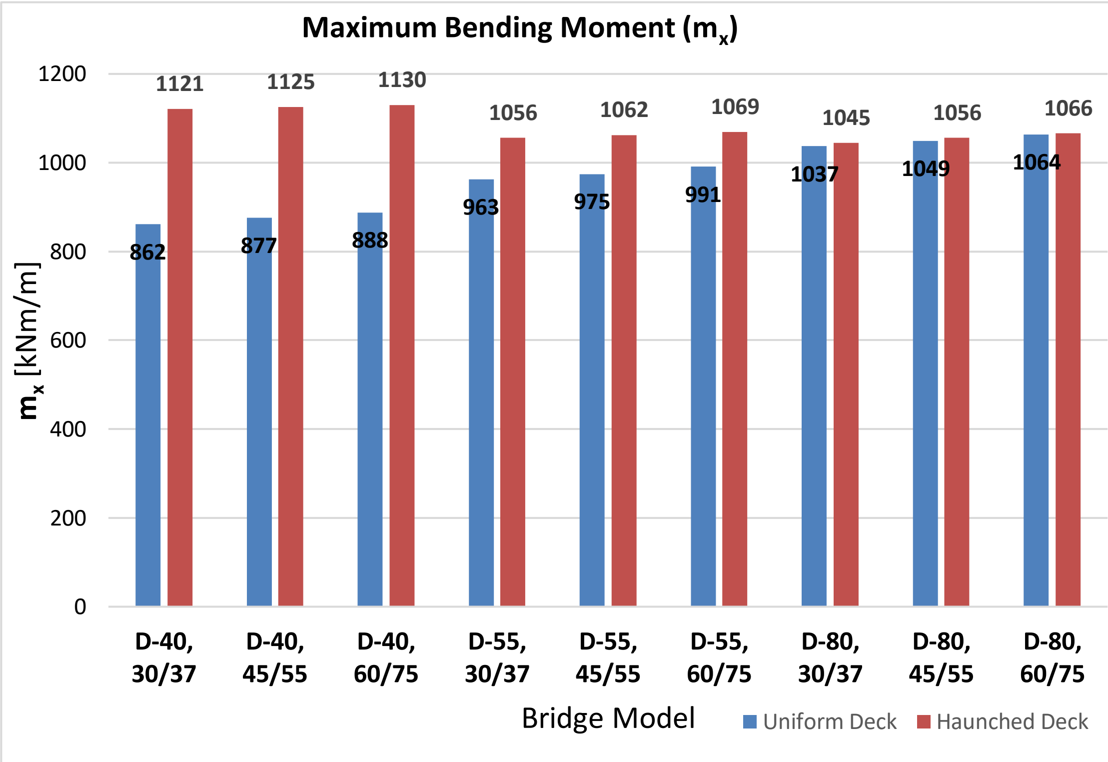
In Table 1. 12, the maximum Bending moment of Deck is presented. Table 1. 12 represents that the increase in Deck thickness gradually decreases the Bending moment. For example, model D-40, C30/37 in the Haunched Deck had **306.1** kNm/m bending moment, while this value is **444.7** kNm/m in the uniform Deck. The reason can be due to the well-distributed Bending Moment in the corners.

Table 1. 12 Bending Moment (mx) comparison in Mid-Deck



The maximum Bending moment of Deck is performed in Chart 13. The chart describes that the increase in Deck thickness in corners gradually increases the Bending Moment capacity. For example, model D-40, C30/37 in the Haunched Deck had **1121** kNm/m bending moment, while this value is **862** kNm/m in the Uniform Deck. This phenomenon can occur due to the well-distributed Bending Moment in the corners and shows that the bridge can bear more Bending only via editing in geometrical properties.

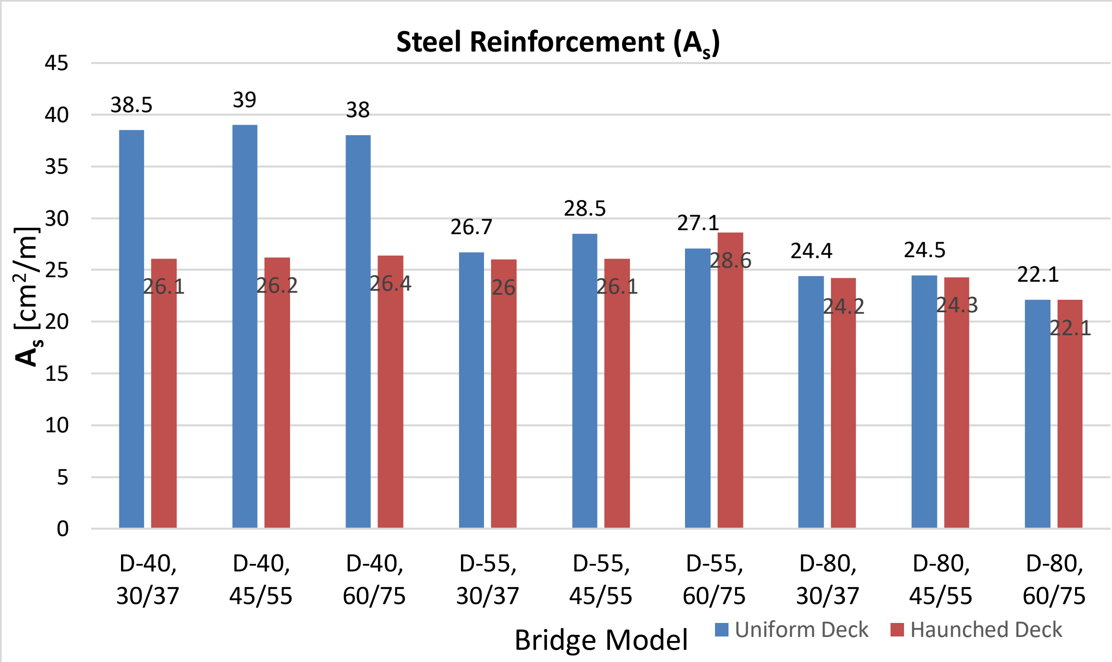
Chart 13 Bending Moment (m_x) comparison in Deck Corners



5.3.3 Steel Reinforcement Quantity comparison

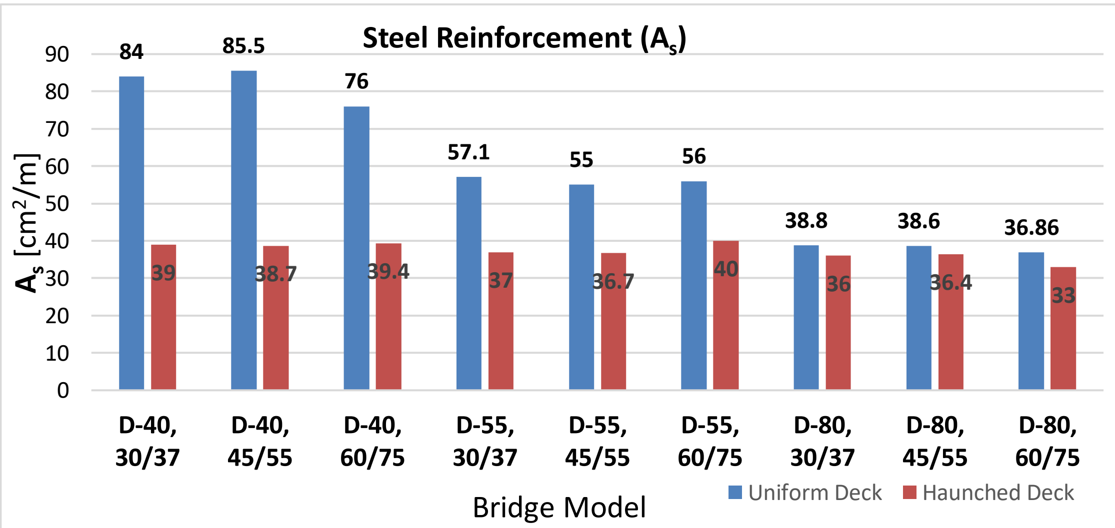
In Chart 14, the maximum Steel reinforcement of Deck is displayed. The chart demonstrates that a rise in Deck thickness considerably decreases the amount of Steel Reinforcement. For example, model D-40, C30/37 in the Haunched bridge has only $\mathbf{A_s}$= **26.1** cm^2/m, while this amount is $\mathbf{A_s}$= **38.5** cm^2/m in the Uniform Bridge. Therefore, by increasing the Bridge Corners thickness from 40 cm to 85 cm, **32**% Steel Reinforcement in Middle Span of Deck will be decreased.

Chart 14 Steel Reinforcement comparison in Mid-Deck



he improvement in the Deck Corners can be seen as well in Chart 15, that, the maximum Steel reinforcement of Deck was decreased. The chart shows that by an increase in Deck thickness, considerably the amount of Steel Reinforcement was reduced. This matter has mainly happened in models with **40** cm Deck thickness. For instance, model D-40, C30/37 in the Haunched bridge has only $\mathbf{A_s}$ = **39** cm^2/m, that this amount is around half of the same model in Uniform Deck with $\mathbf{A_s}$ = **38.5** cm^2/m.

Chart 15 Steel Reinforcement comparison in Deck Corners



From a practical point of view, this amount of Steel Reinforcement and its Size plays a critical role in implementation. As can be seen in Table 1. 13, these two factors have a high range in Uniform models. The Size of Bars are too large and also the distances for this kind Bars in not practically enough in concrete placement. This issue mostly improved in Haunched Deck Models. As discussed before, the amount of Steel Reinforcement in Haunched Deck is around **50** % less compared to Uniform Deck, and, the Bar Sizes are adequate with reasonable distance for concrete placement.

Table 1. 13 Recommended Reinforcement Size Comparison

	Deck Coroner		**Mid-Deck**	
	Uniform Deck	Haunched Deck	Uniform Deck	Haunched Deck
MODEL	REIN. SIZE (Φ)	REIN. SIZE (Φ)	REIN. SIZE (Φ)	REIN. SIZE (Φ)
D-40, 30/37	2×28Φ15	20Φ10 + 16Φ10	20Φ10 + 10Φ10	20Φ12
D-40, 45/55	2×28Φ15	20Φ10 + 16Φ10	20Φ10 + 10Φ10	20Φ12
D-40, 60/75	2×28Φ15	20Φ10 + 16Φ10	20Φ10 + 10Φ10	20Φ12
D-55, 30/37	25Φ10+12Φ10	20Φ10 + 16Φ10	20Φ10 + 10Φ10	20Φ12
D-55, 45/55	25Φ10+12Φ10	20Φ10 + 16Φ10	20Φ10 + 10Φ10	20Φ12
D-55, 60/75	25Φ10+12Φ10	20Φ10 + 16Φ10	20Φ10 + 10Φ10	20Φ12
D-80, 30/37	20Φ10+10Φ10	20Φ10 + 14Φ10	20Φ10 + 10Φ10	20Φ12,5
D-80, 45/55	20Φ10+10Φ10	20Φ10 + 14Φ10	20Φ10 + 10Φ10	20Φ12,5
D-80, 60/75	20Φ10+10Φ10	20Φ10 + 14Φ10	20Φ12.5 + 10Φ12.5	20Φ12,5

5.4 Variation parameters

To hold a quick comparison and also to have an idea between the models' differences, the data were classified based on a parameter. Via this parameter, the results such as deformation, bending moment and steel reinforcement can be analysed quickly. These parameters were considered based on the Deck thickness because as discussed before, the main variation was in this factor, and the difference from the concrete type of view was not remarkable. In below table de parameters are presented:

Table 1. 14 Variation parameter for Uniform Deck at Mid-Deck

Model	Deformation (U_z) = **Ω_U**	Bending Moment (m_x) = **β_U**	Reinforcement (A_s) = **λ_U**
D-80**/** D-55	**0.47**	**1.29**	**0.89**
D-80**/** D-40	**0.23**	**1.5**	**0.62**
D-55**/** D-40	**0.49**	**1.16**	**0.71**

Ω_U= is the Deformation (U_z) ratio

β_U = is the Bending Moment (m_x) ratio

λ_U = is the Reinforcement (A_s) ratio

Table 1. 15 Variation parameter for Uniform Deck at Deck Corners

Model	Deformation (U_z) = **Ω_U**	Bending Moment (m_x) = **β_U**	Reinforcement (A_s) = **λ_U**
D-80**/** D-55	**0.8**	**1.06**	**0.67**
D-80**/** D-40	**5.6**	**1.2**	**0.46**
D-55**/** D-40	**7**	**1.11**	**0.69**

Ω_U= is the Deformation (U_z) ratio

β_U = is the Bending Moment (m_x) ratio

λ_U = is the Reinforcement (A_s) ratio

These parameters were recorded for Haunched Deck models. Please see tables in below:

Table 1. 16 Variation parameter for Haunched Deck at Mid-Deck

Model	Deformation (U_z) = Ω_{Ha}	Bending Moment (m_x) = β_{Ha}	Reinforcement (A_s) = λ_{Ha}
D-80**/** D-55	**0.42**	**1.45**	**0.89**
D-80**/** D-40	**0.59**	**2.14**	**0.91**
D-55**/** D-40	**0.72**	**1.49**	**1.0**

Ω_{Ha} = is the Deformation (U_z) ratio

B_{Ha} = is the Bending Moment (m_x) ratio

Λ_{Ha} = is the Reinforcement (A_s) ratio

Table 1. 17 Variation parameter for Haunched Deck at Deck Corners

Model	Deformation (U_z) = Ω_{Ha}	Bending Moment (m_x) = β_{Ha}	Reinforcement (A_s) = λ_{Ha}
D-80**/** D-55	**0.47**	**0.99**	**0.97**
D-80**/** D-40	**0.56**	**0.93**	**0.92**
D-55**/** D-40	**0.8**	**0.94**	**0.95**

Ω_{Ha} = is the Deformation (U_z) ratio

β_{Ha} = is the Bending Moment (m_x) ratio

λ_{Ha} = is the Reinforcement (A_s) ratio

6 Conclusion and Future Work

6.1 BIM-Design and Analysis

Building Information Modelling (BIM) is an emerging technology in AEC production due to its ability to handle necessary building design and project data throughout a building's life-cycle. The seamless data sharing among various software applications will provide more effective software collaboration between all area of the AEC industry, but it is currently not in standard form. One of the main goals for this is the shortage of investigation on data transfer linking different software applications. Regular data exchange format like IFC, is the possible solution to develop the interoperability between various software applications. But this format doesn't run as expected, particularly in a detailed modelling. Therefore, continuous investigation and improvement concerning this should be conducted. One more main viewpoint is modelling errors, which influence the quality of data sharing between separate software applications. A set of controls must be specified when information is modelled. The modelers should be aware regarding why and what data they are modelling and where it should be applied.

Furthermore, in this research, various features of data sharing between a BIM model and a FEM model was investigated. The critical findings from this particular topic are listed in below:

- **Measurements**:

Both programs have the same definition in case of measurement.

- **Volume element**

The defining and importing of every single volume element remains stable after introducing in the InfoGraph program.

- **Mesh system**

This feature was checked by InfoGraph, that the result was different in two commands *Mech Generation* and *Tetrahedrons from Solids*. The first function (Mech Generation) was not functional, because in InfoGraph to define the Mech Generation, a surface is needed. While the imported model in InfoGraph was identified as a *Volume Element*. Therefore, this function (Mech Generation) was not practical directly.

- **Coordinate system**

In ALL PLAN 2019, the coordinate system was the global system (counterclockwise), while, in InfoGraph, the coordinate system change based on its definitions. It means the orientation was imported in reverse direction.

➢ **Load case definition**

The load case function was able to create and define directly in InfoGraph after importing the bridge model.

6.2 Finite Element Methods Design and Analysis

In this part of the research, the results of a study about *Half-Frame Bridge* considered for Railway with Reinforced Concrete slab is summarized. The finite element method was utilised to study the effect of different Characteristic strength of Concrete and slab thickness on a Half-Frame Bridge, one-span, reinforced concrete slab bridges. The concrete slab and other parts were modelled using *SHELL* elements. Deck thickness sizes of **40** cm, **55** cm and **80** cm were selected to model the slab bridges. The bridge width considered in this research is **12** m. One span length was considered in this study with **10.74** m. The models were loaded and designed based on loading conditions such as: Maximum DIN EN 1992-2, Ultimate Limit State (ULS), Euro code DIN EN 1991-2, Accidental combination, Permanent and temporary situation, DIN EN 1992-2. The investigation summary results are presented in two part: the Bridge with Uniform Deck and the Bridge with Haunched Deck.

The investigations were shown:

➢ **Bridge with Uniform Deck:**

1. Characteristic strength of Concrete has no important influence on deformation. For instance, models C30/37 and C60/75 have less than **one** mm of deformation variation.
2. Deck thickness has a greater influence than the Characteristic strength of Concrete in Slab Deformation. For example, the models D-40, 30/37 and D-80, 30/37 with the same Characteristic strength, have **13.2** mm deformation variation in the Uniform Deck model.
3. Characteristic strength of Concrete has no significant impact on the Bending Moment (m_x). For instance, models C30/37 and C45/55 have less than **ten** kNm/m in D-40 in Uniform Deck models.
4. Concerning the Bending Moment (m_x) and the Deck thickness relation, it can be stated based on the data that model (with the same Characteristic strength of Concrete) with 80 cm thickness has around 1.5 times and 1.3 times Bending Moment capacity than respectively the models with 40 cm 55 cm in Uniform Deck.

5. Bending Moment (m_x) difference in the Bridge Corners from both point of view (Deck thickness and Characteristic Strength of Concrete), is less than this factor in the Middle of Deck.
6. The amount of Steel reinforcement of Deck is approximately equal in models with the same concrete; but from thickness point of view, the models with 40 cm thickness has the amount of around **13** cm^2/m more than the models with 55 cm and 80 cm thickness.

➢ **Bridge with Haunched Deck**

1. Generally, when comparing the Haunched Deck models with each other, they have the same behaviour as the Uniform Deck models from Deformation Bending Moment (m_x) and Steel Reinforcement point of view.
2. Implementing the Haunched Deck in models with 40 cm thickness, was decreased the deformation in the Middle of Deck from **17.1** mm to **9.1** mm comparing to the Uniform Deck models. For the rest models with 55 cm and 80 cm thickness, this variation is not remarkable.
3. Deformation factor had reverse action at Corner of Deck in D-40 models with Haunched Deck (Despite, this value is ignorable). But for D-55 and D-80 models the deformation in corners is nearly the same.
4. The Bending Moment (m_x) at Middle of Deck, in D-40 and D-55 models with Haunched Deck, is smaller than the Uniform Deck models. However, in D-80 models, this factor is almost equal in both methods. But this factor has converse behaviour in *Corners* compare to the Uniform Deck models.
5. Steel Reinforcement in middle of Deck was considerably decreased in Haunched Deck Models with 40 cm thickness. The amount of Steel Reinforcement was **38.5** cm^2/m in D-40 with Uniform Deck, while this amount was reduced to **26.2** cm^2/m in Haunched Deck models. For the rest models with 55 cm and 80 cm, this factor remained mostly stable.
6. Steel Reinforcement in the Corner of Deck was significantly modified in Haunched Deck Models with 40 cm thickness. This amount of Steel Reinforcement in D-40 models with Uniform Deck was around **85** cm^2/m, while this number was reduced to **39** cm^2/m in Haunched Deck models. Also, this behaviour happened in D-55 models. It means, $\mathbf{A_s}$ was decreased from **56** cm^2/m to **37** cm^2/m in Uniform Deck models. For the rest models with 80 cm, this factor remained mostly stable.

6.3 Future Work

- The bridge was only loaded for the Railway Traffic Load cases. This kind of bridge can be loaded and analysed for Road Traffic.
- The thickness of Haunched part was **85** cm in maximum in this research. The thickness can be increased or decreased in the next study.
- The Shear Reinforcement of the bridge in the same example can be investigated.
- The Dynamic Analysis of this Bridge can be another item to be investigated.
- This bridge was designed and analysed for trains with a maximum of **160** km/h speed. Hence, a higher speed rate can be analysed.
- The Temperature, shrinkage and Fatigue are critical factors for Bridge design. Besides, this bridge has been analysed for Germany Temperature. Hence, it can be designed and analysed for other climates.

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8 Appendix and Attachments

Here the model **Haunched_Deck, Model D-40, C30/37** as an example, more detailed is presented. The details of other models can be found in the attached digital file (DVD).