Connection between concrete columns and continuous floors with integrated steel beams

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CONNECTION BETWEEN CONCRETE COLUMNS AND CONTINUOUS FLOORS WITH INTEGRATED STEEL BEAMS

By

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Abstract

In the ongoing building industry, there is a constant demand for the design of more effective and cheaper building designs. A common building technique for office buildings is the application of continuous floors consisting of integrated steel beams and hollow core slabs. These floors are supported by steel columns. As concrete is a cheaper material than steel, this research replaces the steel columns for concrete columns to obtain a cheaper design. Such design has not been explored by researchers yet and is hardly used in practice. The research focusses primarily on the structural aspect of this new building design.

This research studies a simplified connection between one storey high concrete columns and the continuous floors with an integrated steel beam, of the THQ type, carrying hollow core slabs and covered by a reinforced structural screed. The final goal is to determine the strength of the connection. This is done with a calculation method, which may be used outside of this research by designers for an estimation of the design strength of their connection.

A simplified design for the connection is set up as a reference design. It is loaded by two load combinations of the use stage of the building, a symmetric and an asymmetric load combination. A behaviour prediction is set up for the reference design under these load combinations. Based on that prediction, additions to the design are studied, which should improve the connection strength. Two adjusted designs are chosen to analyse alongside the reference design: a design with the integrated steel beam supported by additional steel webs at the connection and a design with the integrated steel beam being filled with mortar at the connection. The three designs are numerically modelled and analysed with the Finite Element Analysis (FEA) in DIANA FEA. With the results of the numerical analysis, the design strength of the connection is calculated.

The numerical results show that the structural screed and the bottom column are the governing connection parts. The structural screed in the reference model is influenced most by the steel webs of the beam underneath and a limited load transfer area, which leads to higher stresses in the concrete. In the adjusted models this situation is improved, with the model with a mortar filling being the best adjustment, as it provides the maximum load transfer area. The bottom column is most affected by the bending of the steel beam flanges due to the load of the floor, causing a large load to be transferred over a small area at the column edges. The adjusted models do not show a strong improvement here, as the adjustments are limited in the stiffening of the beam flanges.

The calculation of the design strength of the connection shows the adjustment of the mortar filling to be the best solution. However, the final design strength is low compared to the design strength of the concrete in the structural screed or the columns. This is the result of the horizontal tension in the structural screed, which reduces the concrete strength. A better result may be obtained by the removal of the structural screed at the connection and the placement of the concrete column directly on top of the steel beam. This adjustment needs a further research.

The conclusion is that the researched design in its current form is not effective enough to be applied in practice, but its drawbacks can be solved with a few adjustments to the design, making it an effective design solution.

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§1 – Introduction

This thesis is about the research on a system of integrated floors, in particular, the connection of a continuous floor to concrete columns. Currently, many office buildings are built using a floor consisting of hollow core slabs resting on integrated steel beams, as shown in Figure 1. The hollow core slabs have hollow cores, which make them lighter, removing the less effective positioned material. The use of hollow core slabs provides the possibility to increase the height of the



floor, without increasing the load. By doing so the resistance of the floor to bending increases, which allows for larger spans and thus more column-free space in the building. The integrated steel beams are integrated into the floor and do not stick out underneath it, as was common in the previous decennia. This makes the total floor height smaller, since the beam is inside the floor, creating more vertical space in the building. Also, there is more room and freedom for installations under the floor, as the beams do not stick out of the floor. Furthermore, due to the steel beams being surrounded by concrete, they have a high fire resistance[2]. All this leads to the integrated beams being a desirable solution for new office buildings (but also buildings like hospitals and libraries). The integrated beams can be performed in concrete and steel. The latter one has a much smaller size and weight and is therefore favoured over the former.

These floors can have a variety of connections with the columns. The preference lies towards the fully constrained connections, for it allows for larger spans and thus larger open spaces in buildings. The columns are continuous and the beams are attached to them using a variety of connection methods.



Figure 2. Graphical representation of the column-floor connection

In this thesis, the floors are made continuous and the concrete columns are one storey high. A graphical representation is presented in Figure 2. This means that the columns compress the floor and all load from the upper parts of the building goes through each floor below. This makes the research interesting, for there is a large load put on the lower floors of the building.

The connections of one storey columns with continuous floors is

not a new concept, but the connections consisted of steel columns in combination with steel beams[3], which is a totally different situation from a structural point of view. This research focusses on concrete columns instead of steel columns, as they are cheaper and may lead to a cheaper design.

The design of the continuous floors and one storey columns, especially with concrete columns, has not been explored by researchers yet and is hardly used in practice. This lack of knowledge is a problem and

prevent the design from being widely used. This thesis shall thus be a pioneering research in this area, making it a first step in the discussion on the effective design of the continuous floor.

This research aims to provide insight in the connection behaviour of a simple connection design with concrete columns and continuous floors. This insight should allow engineers to better understand the reaction of the connection to a load combination similar to the load combination in practice and lead to effective designs of these connections in practice. This research also provides an analytical method to calculate the vertical design load on the connection. The motivation behind this research is to obtain a cheap and simple building design.

This research seeks to answer the following research questions:

- How does the designed connection behave under loading?
 - > What problems arise in the design of this connection?
 - > What can be adjusted in the design to solve these problems? Is it an effective measure?
 - > What are the recommendations for the application of or further research on this connection?

The research starts with a literature study to analyse available information on the desired topic. With the use of this information, a reference connection design is set up, from which other designs will follow. For the reference connection, a behaviour prediction is made, which serves as a check for the numerical model later on in the research. After, the possible variations to the design and their effect on the connection behaviour are analysed, based on the behaviour predictions made. From this analysis two variations are chosen and two additional numerical models are created. These are loaded in the same way as the reference model and the results are compared to understand the general behaviour of the reference connection and the effectiveness of the designed variations. First, the numerical models are linearly analysed in 3D using DIANA, which brings forth the stress distributions in the linear elastic stage. In order to make the analysis of the behaviour more accurate and account for cracking and stress redistribution, a non-linear analysis is performed on the governing cross-section in 2D. Using the numerical results, a maximum vertical design load for the three connections is set up and compared. The results are then discussed and recommendations for future design and research are given.

§2 – Literature study

In the literature study a search is set up for connections similar to the desired idea of the design used or researched in the past and all other information regarding this floor type. The latter one includes the loads on the floors, the common material types, common assumption etc. These are not covered in this chapter, but will be introduced or referred to in the course of the thesis. The sources used are articles, scientific books and guidelines.

During the search for literature it appeared that no equivalent of the desired connection is used anywhere and has been researched. Nevertheless, similar connections are discovered, which will be discussed in this chapter.

Peikko made a design of the Deltabeam^[4], a perforated trapezoidal integrated steel beam. The hollow core slabs rest on the flanges of the beam. The beam and the connection to the hollow core slabs are filled with concrete, making it a monolith concrete floor. The



Figure 3. Deltabeam connection [4]

suggested connection has columns with steel endplates, which are one storey high and rest directly on the beam, connected to the column below by external rebars, as depicted in Figure 3. Their base design does not involve a structural screed, but leaves the possibility to use it open as a variation.

For this thesis, the desire is to analyse the simplest set-up. With the understanding of the connection behaviour in its simplest state, effective measures may be taken to refine the connection into a more effective design, which is still simple and therefore easy to understand, design and build. Due to its simplicity it may prove to be economically beneficial when compared to the current connection designs. The Deltabeam is too advanced to work with in this research due to its monolith concrete structure, trapezoidal shape of the beam and steel to steel connection between columns and beam. Still, the information found in this solution gave a global understanding of the way a design can be made.

The other document of interest is a report on a contest published by Bouwen met Staal, a Dutch scientific journal about steelworks[5]. In the contest, engineers looked at the preferred design of a connection involving two concrete columns with a THQ beam in between. In the problem statement, the connection of Figure 4 was assumed. It was assumed the beam would be perforated by column reinforcement, causing holes in the THQ. These holes were defined as a weak spot and thus something to improve on in the proposed designs. The holes are significantly wider than the rebars to ensure the reinforcement will fit through, since account must be made of the inaccuracy of the production and build-up of the floor. The second stated problem of the connection is the introduction of the vertical load from the top column to the beam and from the beam to the bottom column. It is stated that the THQ webs can buckle when designed to be 4-6mm thick. Then, prior to the displayed results, a number of solutions are given to strengthen the beam, which include local thickening of the beam, applying additional webs and applying steel endplates to the concrete columns.

Seven groups presented their design drawings. In all drawings the top columns are connected directly to the beam and there is no mention of the structural screed, as it was not involved in the problem statement. One drawing, shown in Figure 5, suggests the use of endplates for the concrete columns and the use of additional steel webs to fill the THQ between the columns. The thickness of the webs is taken as 25mm, approximately four times larger than the advised thickness of the regular webs. Another group (Figure 6) proposed to fill the part of the beam between the columns with mortar. And another group (Figure 7) proposed the local thickening of the beam at the location of the connection in combination with column endplates and additional webs. These designs are simpler than the Deltabeam and come close to the desired design. Unfortunately, no explanation on the design choices was presented, nor were the problems of the design of Figure 4 explained. Still, the improvements of the connection are a valuable addition to this research and will be considered in this thesis.



Figure 4. Problem statement contest[5]



The most important information missing for this connection type is the information on the stress introduction from the top column to the floor and forth to the bottom column. This is the reason why this research focusses on the most basic and fundamental design of the connection. With the knowledge of the behaviour of the simple design effective designs can be researched and made in the future, but for that a base research is needed, which is presented in this thesis. Furthermore, there is no method for designing connections of this type. This thesis will provide the reader with a design method for designing the strength of the connection.

§3 – Reference model description

In this chapter the reference model, or base model, is introduced. This model in the starting point of the research and from this model all numerical models will be derived.

The connection is located in a fictional office building. The introduction of this office building makes the connection design more practical in the sense that all loads applied on the connection are defined by the building it is located in, as specified in the Eurocode[6] and a design guide on multi-storey buildings[3]. The design of the connection is made for this particular office building. The applied loads on the building and the connection will be presented later in this chapter.



The plan of a part of the floor in the building is shown in Figure 8. In the bottom left corner, the axis are defined. They are used over the length of the thesis, with the exception of the 2D numerical model. The Xdirection is the horizontal direction in which the beams are laid out and the Y-direction is the horizontal direction in which the hollow core slabs are laid out. The Z-direction is the vertical direction in the design. The columns, each 4m high and 6m apart in the X-direction, carry the integrated steel beams (THQ). The beams carry hollow core slabs (HCS) of each 1.2m in the width and 8m in the length, spanning from one beam to the other. The hollow core slabs are laid out next to each other and

connected to the beam by a concrete filling (CF). This is then covered with a reinforced structural screed (SS). For this research, only a small area is relevant in terms of load calculation. It is the area of the floor, which is carried by one column, in Figure 8 defined as Column floor area and marked in red. Furthermore, the number of floors is important, as the designed connection is located at the intersection between the ground floor and the first floor, making it the most loaded connection in the building. The office is 3 storeys high, meaning the designed connection is subjected to the load from its own floor, the floor above and the roof. The arrows in Figure 8 show the load transfer. The HCS transfers the load to the beam, which transfers the load to the column.



Figure 9. Graphical front view

Figure 10. Graphical side view

The overview of the designed connection, graphically presented in 3D in Figure 2, is presented from the front view in Figure 9 and from the side view in Figure 10. The THQ rests on the bottom column. The hollow core slabs (HCS) rest on the flanges of the THQ and connect to its body through the concrete filling and the hollow core slab connection rebars. This is topped with the reinforced structural screed. On top of it, the top column rests. The column reinforcement runs from the bottom column through the THQ and the structural screed into the top column. All parts of the connection are explained in the order of their parameter importance.

The dimensions in this thesis are for the larger part based on design recommendations[3]. Therefore, the validation checks will be limited to the connection only. For the thickness of the HCS the recommendations are summed up in Figure 11. The figure shows the span of the HCS on the horizontal axis and the span of the THQ on the vertical axis. With the combination of 8m long slabs and 6m long beams, the thickness of the HCS is recommended to be 200mm or a safer 260mm. The safer option of a 260mm thick HCS is chosen for this design. The decision for the length ratio THQ:HCS to be 3:4 has been made based on the more recent design recommendation[2], which stated that this ratio is most



Figure 11. Design graph for thickness HCS[3]

effective in the economic sense, which makes this design ratio more attractive to use. As this research prefers to stay close to the practical use of these connections, these recommendations are adopted in the design.

The HCS is thus 260mm thick and 8m long. It is made out of concrete class C50/60. The HCS is prestressed in its lower part for moment resistance in the middle of the slab span. The HCS has its cores filled at the location of the connection (see Figure 10) to prevent top column standing partly on a hollow core. For this research, the HCS is used for the definition of the floor height and the applied load on the connection. In the researched connection design, this part is simplified to a monolith concrete block.

The schematic front view with all the connection parts is shown in Figure 12. At the connection the HCS is connected to the webs of the beam by the



concrete filling (CF), which is 60mm thick and is made of concrete class C35/45. Furthermore, to keep the HCS in place, they are connected to each other by means of Ø16mm rebars running through the THQ with a spacing of 250mm. This is not a key part of the connection in itself, but does create holes in the THQ, which lower its resistance and thus must be accounted for. The rebars are of steel class B500 and are

located at the bottom of the HCS to prevent immediate collapse of the floor in case of the failure of the bottom column.

The HCS rest on the flanges of the integrated steel beam. For this research the THQ has been chosen, as it is a frequently used beam. The thickness of the HCS usually defines the height of the THQ, as it makes for the optimal combination of the capacities of both units[3]. Therefore, the beam is designed to be at the top on the same level as the top of the HCS. The THQ consists out of two horizontal and two vertical plates. The vertical plates are called THQ webs. The beam is of class S355. The parameters are displayed in Figure 13 and Table 1.



THQ	Webs	t _w	8	mm
		h	260	mm
	Тор	t _o	40	mm
		b _o	234	mm
	Bottom	t _u	20	mm
		b _u	500	mm
		ŭ		

Figure 13. THQ parameters[6]

Table 1. THQ parameter definition

As previously stated, the THQ has reinforcement running through it. This means that there are holes in the steel plates, which reduce its total resistance capacity. Due to the imperfection factor in fabrication and installation, these holes have to be sufficiently wide to ensure the reinforcement will pass through. For the HCS reinforcement of Ø16mm, the holes have a diameter of 22mm. For the Ø25mm column reinforcement the holes have a diameter of 34mm. In the validation of the choice of material, parameters and loads in the Appendix A, the resistance of steel plates will be calculated at the cross-section with the holes.

The rectangular prefabricated columns are 4m high and designed with a width and depth of 400mm. It is important that the width is larger than the width of the top plate and smaller than that of the bottom plate. This allows part of the concrete on the side of the beam to be involved in the load transfer from the top column to the bottom column. The columns have a reinforcement of four rebars of Ø25mm. These rebars stick out of the bottom column, go through the holes in the THQ, through the SS and enter the prefabricated tunnels in the top column, which are then filled with concrete. This method is shown in Figure 14, perpendicular to the beam direction. The HCS and SS are not depicted here. This system is modelled as a continuous reinforcement. By designing this way, the reinforcement of the column may transfer a part of the load from the top column to the bottom, this way relieving the floor slightly. The prefabricated columns, just like the prefabricated HCS are of concrete class C50/60. They are designed without endplates and are connected to the beam and SS directly. In reality there



would be connected by a layer of mortar to ensure the connection between the columns and the floor, but this has been simplified in the design, as it does not influence the connection behaviour.

The last part of the designed connection is the structural screed (SS). This is a reinforced layer of concrete on top of the HCS and THQ. Over the whole floor it serves as a connecter of the individual independent HCS, combining them into one floor. The reinforcement in the SS ensures that the sagging of neighbouring slabs is averaged out over more slabs to prevent large cracks in the floor. If cracks arise, the rebars ensure a wider spread of smaller cracks instead of a randomly located large crack. Furthermore, the net of rebars near the THQ serves as a constrain for the HCS. The floor can take up bending moments at the THQ location in the slab (Y-) direction. This leads to a smaller sagging and extreme bending moment in the middle of the beam. The SS is included in the designed connection to ensure this constrain is valid for the HCS at the connection and would not lead to that slab having an initial larger sagging and extreme bending moment than the neighbouring slabs. Furthermore, this decision makes the design of the connection simpler, making it a favourable design for engineers due to simplicity.

The SS is 60mm thick and is made of concrete class C35/45 with reinforcement of steel class B500. In both directions, the reinforcement bars have a diameter of 12mm and a spacing of 100mm. The concrete cover is 18mm, which satisfies the minimum requirement defined by the Eurocode 2[6] and the Dutch National Annex[7], with the concrete being of environmental class XC1 and construction class S1 due to its location inside the building.

The build-up of the building is important for the loading of the connection and needs a short description. Upon the bottom column, the THQ is positioned. The HCS are laid down on one side first, with the THQ being supported by studs on the loaded side, and later on the other side. Then, the reinforcement for the HCS is laid in position and the void between the THQ and HCS is filled with concrete, along with the hollow cores of the HCS at the location of the connection. After, the SS reinforcement net is put in position and the concrete of the SS is cast. After drying, the top column is positioned on the SS. The same process happens for the floor above. After the building is constructed, the loading can differ depending on the way the building is used. This leads to a large variety of load combinations during the building stage and the use stage afterwards the connection is subjected to.

To the connection, the load is introduced from three directions: through the top column the load from the building above is introduced, through the HCS the load of the floor over the width of the connection is introduced and through the THQ the load from the rest of the floor is introduced. The magnitudes of these loads are calculated based on the building parameters introduced in this chapter.

For this research, the variety in load combinations is limited to two situations in an already build and loaded building: a symmetrical load on the floor of a design load including the maximum fixed (0.3kN/m²) and variable load (4kN/m²) and an asymmetrical load on the floor, where the variable load on one side of the connection is 6kN/m² and on the other side 2kN/m². The same load situation holds for the floor above, meaning the column on top of the connection introduces a vertical load and a bending moment on the connection for the asymmetric case. While the Eurocode[8] states that designs should be tested for the maximum variable load of 4kN/m² on one side and an absent variable load on the other side, the (according to the Eurocode[8] overdesigned) load combination in this research is performed to ensure an equal total load with the symmetrical load combination. This will provide for a good comparison between symmetric and asymmetric loading of this connection. This load combination is applied because it is the largest total load on the floor according to the design recommendations of Eurocode[8].

To get an understanding of which load leads to what reaction of the connection, a behaviour prediction is performed for each of these three load introductions separately. This way, the results in FEM are better interpreted. Furthermore, this prediction serves as a check for the correctness of the numerical model.

§4 – Behaviour prediction

In the previous chapter, the connection design (Figure 12) and the load combinations applied on the connection have been defined. In order to get a better understanding of what load leads to what reaction in the connection, each load combination is split up into three parts, each covering one load introduction. The first introduction covered is the load introduced by the concrete floor directly to the connection. Second, is the introduction of the load from the floor through the beam to the connection. Third, is the load introduction from the top column to the connection. For all three introductions the predicted reaction of the connection is discussed. Accumulated, these discussions lead to conclusions on the behaviour

prediction of the connection. This chapter is not a behaviour analysis in itself, but serves as a rough prediction of the reaction of the connection to the applied load, which is used as a check for the numerical model for correctness. The predictions in this chapter are compared with the results of the numerical model further in the thesis.

Depending on the load introduction, a side view in the X-Z plane (Figure 15) or a front view in the Y-Z plane (Figure 12) is shown. For the side view in this chapter, the HCS and CF are not included to show the THQ top and bottom plate, which are important for the load introduction through the beam, for which the side view is used. The loads on the connection are introduced through shear forces *V*, distributed loads *q* and bending moments *M*. The exaggerated deformations are shown by dashed lines, since thinking in deformations makes it easier to understand the stress distributions. For each load introduction the predicted stresses in relevant cross-sections are shown, as well as the location of compressed material and material under tension.

The column floor area, defined by the red square in Figure 8, is shown in Figure 16. Since the HCS transfer their load to the beam, which transfers the load to the column, two load transitions may be distinguished in Figure 16: the HCS to the column through the beam (blue area) and the HCS directly to the column. The load introduction from the









beam ($V_x - M_x$) is significantly larger in magnitude than the load introduction from the concrete floor directly ($V_y - M_y$).

4.1 Load introduction through the floor

For the symmetrical load combination, the load introduction from the concrete floor directly to the connection is shown in Figure 17, in the front view of the connection. On the left and right V_{y1} and V_{y2} represent the shear forces and M_{y1} and M_{y2} represent the bending moments. These are boundary conditions that account for the load on the concrete floor over the width of the column. The shear force and bending moment on the left is of equal magnitude as on the right.



Due to this load the flanges of the THQ curl downward, lifting the inner part of the bottom plate. This causes

vertical and horizontal compression at the corners. It is important to understand that due to this deformation of the bottom plate of the THQ, the contact area between the beam and the column

decreased, leading to larger stresses at the contact area, the edges of the bottom column. The predicted stress distribution in the column is shown in Figure 18. In case of a large load in the concrete slabs, the stresses in the middle of the column can become zero and the stresses at the edges very large. This is an undesired situation, since the available area is not used well and since the stresses at the edges lead to failure sooner than in the middle of the column (due to breaking off). This behaviour is caused by the empty volume in the beam.



Figure 18. Stress distribution at A-A

In the horizontal direction the bending moment causes tension in the structural screed and compression at the bottom of the hollow core slab. In the design, the reinforced structural screed above the beam must be able to carry the whole tensile load created by the bending moment.

The asymmetric load combination is shown in Figure 19, with the load on the right being larger than the load on the left. The deformation of the bottom plate should be larger on the right side, and so should the stresses in the corner of the column, the structural screed and the bottom of the HCS. The predicted stress distribution in the bottom column is shown in Figure 20.



Figure 19. Front view: asymmetric loading through floor



Figure 20. Stress distribution at A-A

4.2 Load introduction through THQ

Figure 21 shows the side view of the connection with the introduction of the load on the connection through the THQ. The system is loaded by the load of the beams and by the concrete that rests on them. The load is represented as a set of identical shear forces (V_{x1} and V_{x2}) and bending moments (M_{x1} and M_{x2}). As previously stated, V_x and M_x are significantly larger than V_y and M_y respectively. The bending moments put the bottom of the floor in horizontal compression and the top of the floor in horizontal tension. The floor consists out of the steel beam and the concrete around it. For a flat



and smooth surface like in this connection, the materials have a weak cooperation based on cohesion. This means that when a certain stress is reached the cooperation stops and the two materials act loose from each other. The Eurocode prescribes this maximum stress to be 1 MPa[6]. Since this value is very low and will surely be surpassed in the procedure of loading, it may be assumed that steel and concrete act loose from each other, unless compressed to each other. Therefore, the steel beam is assumed to resist the shear load and bending moment alone. The THQ top plate is thus loaded in tension and the THQ bottom plate in compression due to the bending moment. The webs also contribute to the bending resistance, but for the selection and validation of parameters the conservative approach is used, stating that the webs only carry shear and the top and bottom plate only the moment. For all plates holds that the critical locations are at the holes, since there the effective area is decreased.

In the model there is one stress distribution of interest, the one in the bottom column due to the beam load. The vertical stress distribution is depicted in Figure 22. The beam is expected to bend slightly more downwards at the edges of the column, causing a larger stress in the corners of the column. This distribution is, however, flatter than the distribution due to the THQ bottom plate bending in the other direction, as shown in Figure 18. While the loads introduced by the beam

are much larger than the loads introduced by the concrete floor directly, due to the different stiffness of the floor in the two directions, the peak stress $\sigma_{x,max}$ in Figure 22 may be similar in magnitude to the peak stress in $\sigma_{y,max}$ Figure 18.

Examining the asymmetric loading in Figure 23, with a larger load on the right, it may be considered that the tension and compression locations and all distributions in the model are similar to the symmetric load combination, but with higher stresses at the right side.



Figure 22. Stress distribution B-B



Figure 23. Side view: asymmetric loading through THQ

4.3 Load introduction through top column

In Figure 24 shows the front view of the connection for the symmetrical load introduction through the top column. In terms of total vertical loads, the vertical load introduced by the top column is the largest load on the connection, as it accounts for the load from the floor, roof and columns above. It is assumed that a part of this vertical load is transferred by the column reinforcement. However, it is unknown how the column reinforcement affects the stress distribution in the connection and how local this effect is. Therefore, the column reinforcement is not included in the behaviour prediction, making the prediction more conservative.

Since the THQ body is empty, the distributed load from the top column must accumulate towards the THQ webs and the concrete next to it. Assuming that the vertical load from the



Figure 24. Front view: symmetric loading through column

top column is transferred by the THQ webs and only the concrete underneath the column, the active transfer area is 166mm in width, with the concrete column being 400mm wide and the empty THQ body 234mm. This limitation in transfer width leads to higher stresses in the concrete and steel, than in a case without the empty THQ body.

The top plate of the THQ bends in, having a low stiffness as a structural element. The load from the column accumulates towards the THQ webs and the concrete next to them.

There are three cross-sections of interest in this case. The first, A-A, is located in the bottom of the top column. The structural screed underneath the column has a similar stress distribution, but with higher stress peaks, since it is closer to the steel beam. Accompanied by the lower concrete class of the structural screed, this makes the structural screed a more critical part than the top column. Still, the width of the cross-section of interest in the structural screed is the width of the column. Therefore, to prevent a chaotic picture, the cross-section in the structural screed is marked in the top column from here on. Due to the low stiffness of the sagging top plate of the THQ large amount of stress is transferred to the webs and the concrete filling on both sides. The predicted stress distribution is presented in Figure 25.



The next cross-section of interest is located on the level of the beam, B-B. The stress distribution is shown in Figure 26. The main prediction is that the concrete filling also transfers the vertical load. The stresses in the steel will be higher due to a higher material stiffness of steel compared to concrete, but when taking

into account that the concrete directly in between the columns is 9 times thicker than the beam webs, it is reasonable so say that the concrete next to the THQ web plays a very important role in this connection. A calculation is performed in Appendix B to determine stresses in the steel and concrete for the case of a unit distributed stress from the column. This is important, as the SS directly above the THQ web and CF (for this case and the 2D model, the CF reaches from THQ webs till between the edges of the columns) will have the same stresses and it is here that the stress peak will be largest in the SS due to the load transition to the steel web. This spreads out over a larger width along the height of the SS. Based on that calculation the stress relation of concrete:steel may be estimated as 1:6. Assuming a distributed stress of 1N/mm² in the column, the CF has a vertical stress of 1.6N/mm² and the THQ web a stress of 9.9N/mm². This calculation assumes that only the concrete and steel directly beneath the column transfer the vertical load. This will be compared with the linear and non-linear results later on in the thesis.

Because the integrated steel beam is hollow inside, the webs have a possibility to buckle inwards. However, this movement is resisted by the sagging top plate which forces the webs to bend in the direction of the concrete and by the bottom plate bending upwards, which also forces the webs to bend in the direction of the concrete. This way, it can be said with confidence that the steel webs do not buckle inwards and are therefore not a point of attention in this design.

The last distribution of interest is located in the bottom column. For the current loading, the distribution there is assumed to be the same as in Figure 25.



Figure 27. Front view: asymmetric loading through column



Figure 28. Stress distribution A-A



Figure 29. Stress distribution B-B

In Figure 27, an exaggerated asymmetrical compressive load is presented to account for the asymmetric load combination of the floor above. The bending moment due to the overload to the right of the floor above creates a bending moment, which is combined with the vertical compressive load into this variable distributed load q_{cy} . The stress distributions are predicted to be similar to the symmetric load combination,

but with a larger stress magnitude to the right. The stress distributions for cross-sections A-A and C-C are depicted in Figure 28 and the stress distribution for cross-section B-B is depicted in Figure 29.

The asymmetric load in the floor above the designed connection can be asymmetric in both horizontal directions. Therefore, in Figure 30 the variable distributed load q_{cx} can be larger than q_{cy} , depending on the asymmetry of the load in the two different directions.

The beam stiffness in this direction (along the X-axis) is unchanged, thus leaving the stress distribution in the bottom column the same as the applied load q_{cx} in the top column.



Figure 30. Side view: asymmetric loading through column

4.4 Predictions combined

The three load introductions are combined for a full picture of the predicted connection behaviour. The side view for the asymmetric load combination is shown in Figure 31. The load on the column is presented as a combination of a constant stress distribution (with a resulting normal force N_{c1}) and a bending moment M_{x3} . The examined connection is located between the ground floor and the first floor, which means the top column transfers the load from the floor above, the columns and the roof. The total load from the top column is thus larger than the total load from the floor the connection is in. This does not mean the load from the column is dominating as much that the other loads can be neglected, but it is something that needs to be kept in mind during the research.

 M_{a} M_{a}

Figure 31. Side view: combined introductions

In the floor, there will be tension at the top and compression at the bottom due to the bending moments M_{x1} and M_{x2} . Due to the shear forces the stress distribution in the bottom column will be as in Figure 25, but with a larger stress magnitude at the right due to the larger moment M_{x2} and the bending moment M_{x3} from the top column.

The front view for the asymmetric load combination is presented in Figure 32. Due to the bending of the floor, the bottom of the floor is in horizontal compression and the structural screed is in horizontal tension. Due to the column load, the structural screed is in vertical compression, which means the structural screed is stretched in one direction and compressed in the other. That is an unfavourable condition for concrete. An element of concrete vertically compressed and under horizontal tension has a lower compression strength than a concrete piece only compressed in one direction. A number of scientists researched this topic, of which the Japan Society of Civil Engineers (JSCE)[9] found that the concrete strength can be reduced to a minimum of 60% of its nominal strength. The stress distributions in the



Figure 32. Front view: combined introductions

cross-sections A-A and B-B are the same as in Figure 28 and Figure 29 respectively. The stress distribution in cross-section C-C is a combination of the stress distribution in Figure 20 and Figure 28. Which load from which load introduction (floor or top column) is governing is hard to predict. The answer to that is found in the numerical calculation. Combined with the load in the beam (X-) direction, it may be stated that the largest stress peaks will occur in the corners of the bottom column or at the edges of the column underneath the THQ webs.

For the symmetrical load combination, the same idea holds as for the asymmetric load combination with the difference of symmetrical stress distributions. Assuming the same total load on the connection for the two load combinations, the asymmetrical loading will lead to the largest maximum stresses in the connection.

4.5 Conclusions and discussion

The prediction leads to a number of conclusions presented below. These predictions serve as a first check for the numerical model. They will be compared with the numerical results.

- Bending of the THQ in both directions simultaneously assumingly causes a large stress in the corners of the of the bottom column or in the edges underneath the THQ webs.
- The small active area of vertical load transition due to the empty body of the THQ leads to higher stresses in the concrete and steel.
- The combination of concrete and the steel web is expected to cause large differences in stress at the bottom of the structural screed, which may lead to local failure and stress redistributions.
- The vertical resistance of the structural screed may be weakened by the horizontal tension.
- The failure of the structural screed is governing over the failure of the top column, since it is of a lower concrete class and it is closer to the THQ webs, which are assumed to cause large stresses in the concrete above it.

The governing connection parts seem to be the SS and the bottom column. The first is loaded in horizontal tension and vertical compression, reducing the concrete strength. The latter is subjected to the largest vertical load, which may be concentrated in the column edges due to the bending of the THQ bottom plate. This, however, must be proven by the numerical analysis.

This chapter did not cover the possibilities of other parameters or additional parts on the connection, while that is of great importance to the connection behaviour. Therefore, the next chapter will examine the different effects the parameter adjustments may have, which may lead to the design of a more effective connection. The predictions made in this chapter serve as a first check for the correctness of the numerical model. The numerical results will be compared with the predictions further in the thesis.

§5 – Connection variations

In the previous chapter the reference model is analysed for its behaviour under different load. The results showed the active width for vertical force transition is small compared to the total width (166mm out of 400mm), which leads to large stresses especially in the concrete above the steel webs. Also, the bending of the beam in two directions simultaneously should lead to large stresses in the corners of the bottom column. In this chapter the parameters, which can solve these problems, will be discussed along with parameters that otherwise might improve the design.

There are many possibilities to change the connection to improve its resistance. This can be done by adjusting the parameters of the base connection or applying an additional part to it. The material can also be improved, but that will not drastically improve the stress distribution and is therefore left out of the research. The starting point is always the reference model, which is described earlier in the thesis. When a part of the design is adjusted, the rest of the design remains unchanged.

5.1 Width of the column

Widening of the column leads to the increase of the resistance of the column. Apart from that, it also has an effect on the floor beneath it, which will be covered in this paragraph. The widening can be split into widening in the X-direction and in the Y-direction, because they have a different effect on the connection.

5.1.1 Y-direction

The column widening in the Y-direction is shown in Figure 33. In this direction, the widening of the column leads to a larger coverage of the floor, which means the active area for load transfer is increased. This means that the stress peaks in the vertical stress distribution will be lower everywhere, not only the structural screed, but also the concrete filling, the parts of hollow core slabs involved, the THQ and the columns. Here the assumption of filled HCS cores is held up. Otherwise this method is ineffective as it would mean the column would partly rest on the hollow cores of the HCS. The widening is limited by the width of the THQ bottom plate, since that plate connects the bottom column with the rest of the structure.



5.1.2 X-direction

Figure 34 shows the widening of the column in the X-direction to 500mm. The widening causes the increase of the load transfer area, decreasing the vertical stresses in the connection. Furthermore, it allows for the increase of the amount of rebars by another row. This may lead to a decrease of stress peaks in the vertical stress distribution, since the column rebars transfer part of the load introduced through the top column. However, the effect of the vertical rebars on the stress distributions in the connection is hard to predict and should be studied separately and is not included in the further analysis of this connection. Still, the addition of a new row of rebars and thus a new row of holes, as displayed in Figure 34, does not influence the resistance to bending in the beam direction, since the active transfer area in the Y-Z cross-section is unchanged.



The disadvantage of the widening of the column is the addition of weight. However, the additional weight is very small compared to the total load of the structure. It is sufficient to mention that the floor of 8000mm x 6000mm x 320mm is carried by a column of 400mm x 400mm x 4000mm. The weight of the floor is at least 15 times larger than the weight of the column. This comparison does not consider the fixed and variable loads on the floor. Thus, it can be concluded that the addition of weight is neglectable. Widening the column will add a bit more cost and will look aesthetically unpleasant.

5.2 Width of the THQ

The widening of the THQ is displayed in Figure 35. There are no reinforcements displayed and the only

vertical lines in the grey area show the edges of the hollow core slabs. The top connection is the reference model and the bottom is the wider connection. Above the connections the stress distributions are shown for the structural screed.

A wider THQ diminishes the amount of concrete available for load transfer. This causes the stress peaks to rise. The difference in stress distribution is shown in Figure 36.



column width --- wider THQ stress --- normal THQ stress Figure 36. Stress distributions comparison

Widening the THQ is in fact the opposite of widening the column if all other elements remain unchanged.

Making the THQ wider does increase the beam resistance to the beam bending moment. That can also be achieved by making the horizontal plates thicker. Furthermore, the beam will have more room for reinforcement. It must be reminded that every new rebar needs a



Figure 35. THQ with comparison

sufficiently larger hole for it in the THQ. This makes it more attractive to

increase the thickness of the rebars, instead of the amount of rebars. In short, this method is ineffective without the widening of the column too.

5.3 Thickness of the THQ components

Increase in thickness of THQ components can be seen as an increase over the whole length of the beam or locally, at the position of the connection, the most critical part in the beam. The latter will be the better design, for it will save material and with that weight and cost.

Thickening of the top and bottom slab will increase the resistance to the bending moment in the beam direction and it will increase the stiffness of the plates, which leads to a better stress distribution in the neighbouring concrete. Furthermore, a thicker plate will allow for more stress dispersion. This is shown in Figure 37. Here, the corner between the web and the top plate is depicted. The black area shows the area of THQ top plate over which the load is accumulated towards the web, the load transfer area. The grey area is the initial thickness of the top plate and the



Figure 37. Stress dispersion over height

red area is the additional thickness. This shows that a thicker plate will provide a larger area over which the stresses in the SS accumulate towards the THQ web, meaning the stresses in the SS above the THQ will be lower for a thicker THQ top plate, assuming the vertical load stays unchanged. This is important as concrete can take up much less stress than steel and will fail near the web if the stresses are not spread well enough. Thus, the magnitude of the stresses in the SS atop of the THQ is assumed to be linked to the thickness of the horizontal plates. The same principle holds for the THQ bottom plate.

For the webs, the same idea holds. Thicker webs will lead to smaller stresses in the concrete above and below it. It will also decrease the stresses within the web. However, due to the relatively small size of the webs and their increase, increasing their thickness is ineffective as it will probably add more selfload than really disperse stresses.

5.4 Additional options

Apart from the parameters discussed in the previous paragraph, there are additions to the base connection, which may seriously adjust its behaviour. These will be discussed in this paragraph. Again, the base connections parameters stay unchanged unless said otherwise.

5.4.1 Column – THQ connection

There are two interesting ways to prevent the structural screed from being a governing connection part. This is either by removing it or strengthening it. The option to remove the structural screed requires something in its place to ensure dispersion of stresses towards the top column. A possibility is the direct positioning of the concrete column on top of the THQ. The prefabricated column is of a higher concrete class and is not weakened by the tensile loading in the horizontal direction. This may be done with the application of an endplate to the column to ensure a better dispersion of stresses. This method means there will be no structural screed between the column and the beam, so the reinforcement will have to be led around it. However, this could mean the HCS at the connection becomes a simply supported slab, which could lead to a larger sagging and maximum bending moment compared to the neighbouring slabs, causing shear stresses in the concrete along the length of the slab. It is hard to predict how this will influence the floor and a separate analysis would be needed to describe the consequences of this solution.

The solution with an endplate can also be performed for the bottom column, which will aid to the dispersion of stresses and prevent the column from failing in its corners due to the two-way bending of the beam or at the edges of the beam underneath the THQ webs.

The other solution is artificially confining the concrete in the structural screed between the beam and column. This will lead to a stronger resistance of the concrete. One way is to set steel plates around the column into the structural screed. This way the SS directly underneath the top column is confined and not connected to the rest of the SS, thus not put under tension, thus strengthened. The reinforcement will also have to be led around this area.

Other options may involve changing the concrete underneath the top column for another material, which has a larger strength for a combination of horizontal tension and vertical compression, is a better stress disperser than concrete and allows the SS reinforcement to pass through it. For this, an additional material study is necessary.

5.4.2 Filling option for the THQ

In the base model the THQ is empty inside. This leads to the sagging of the THQ top plate and to a stress concentration in the structural screed near the THQ webs. The beam can be filled with mortar. This will prevent the sagging of the top plate and the filling will be active in the load transfer. This must significantly reduce the peak stresses in the structural screed and deliver a more flattened out stress distribution. The addition of mortar makes the beam more rigid, reducing the deformation due to double bending and this way increasing the contact area of the beam with the bottom column, which must lead to lower stress peaks in the corners of the beam. This mortar needs to have a 100% contact with the top plate of the THQ in order to prevent a too large sagging of the top plate. Thus, the mortar must not set or shrink in the process. This can be achieved by applying low shrinkage mortar. This mortar consists out of very fine aggregate, making it less stiff than the usual concrete. To have the mortar locally in the beam, two

additional webs must be placed in the THQ to enclose the mortar. They must be sufficiently far away from the column to create a stiffer beam section, which is larger than the width of the column. This is graphically shown in Figure 38. The mortar must be cast in through holes in the top plate at the sides of section, this since directly underneath the column the THQ top plate is loaded the most and is already perforated by reinforcement. Mortar is to be poured in on site.



Figure 38. Mortar filling in the THQ

The other option is filling the THQ with additional steel webs at the location of the columns. These can be positioned differently. Since there is a double bending of the beam, it seems reasonable to apply two webs above the edges of the bottom column to include a larger portion of the edge to be active in the load transition. Parallel to them two extra webs must be applied, as shown in Figure 39. The last extra web must be placed parallel to the original webs, in the middle of the beam. The extra webs must increase the vertical load transition area. They are assumed to cause extra peak stresses in the concrete above and below, but as transfer area increases, the peak stresses should be smaller than in the original design. The installation of



Figure 39. Additional webs in the THQ

the additional webs can be performed in the factory and is easier than the casting of the mortar in situ.

5.4.3 Hollow core slab connection

In the Literature Study a number of examples have been found for the fixation of the hollow core slabs on the beam. In the current design the webs of the beam are perforated by the rebars and form a weak spot around the holes in the steel. The two options found in the information sources propose ways to avoid the perforation of the webs, leaving the web intact.

The first option is presented in Figure 40. The webs have demu's welded to them, in which the rebars are installed that go into the hollow core slabs. This way no holes are made in the web. This is a more expensive solution as welding needs to be done on site. It may be considered to do this not over the whole beam, but only at the location of the highest stresses in the THQ webs, which is around the column.

The next option is depicted in Figure 41. Here a rebar connects the two slabs by going over the THQ. This is cheaper than the previous solution due to the absence of welding, but the thickness of the structural screed must allow for such a method.



These solutions work to avoid holes in the THQ and keep the active area unchanged. The other option to tackle the diminishing of the active area of the web is to increase the thickness of the THQ webs. This can also be done locally, as mentioned before. Increasing THQ web thickness is easier and cheaper than connecting the rebars to or leading them around the THQ. However, they were worth mentioning.

5.5 Conclusions and choice for additional models

The adjustments discussed in this chapter are summed up below with their advantages and shortcomings.

- Wider column
 - $\circ\,$ Increases the vertical load transition area, which is assumed to flatten the stress distribution.
 - Allows for another row or column reinforcement, which can transfer a larger part of the load.
- Thicker THQ components
 - Thickness of THQ top and bottom slab increase the horizontal bending resistance, but also the vertical stress dispersion, which should cause smaller stresses in the concrete above the THQ.
 - To avoid large increase in weight, the thickening may be applied locally.
- Structural screed
 - It can be removed for steel endplates on column, but that might be costly.
 - It can be strengthened by separating the concrete underneath the columns from the rest of the structural screed and confine it.
- THQ can be filled
 - Filled with low shrink mortar to stiffen the beam.
 - Filled with extra webs to stiffen the beam.
- Hollow core slab connection
 - Connection through the THQ.
 - Connection over the THQ.
 - Connection with the THQ.

With the predictions of the connection behaviour performed in Chapter 4 and the possible adjustments for the model discussed in this chapter, an addition to the reference design may be chosen. The reason for this is the possibility to evaluate the improvements on the connection behaviour an adjustment to the design may have, using the numerical calculation.

The predictions in Chapter 4 predict possible large stresses in the SS and the bottom column. Both are influenced by the empty core of the THQ, as it leads to a limited area of transition for the vertical load introduced by the top column, which is the largest load on the connection. This chapter sought to enlarge this transition area by column widening, thickening of the THQ horizontal plates and filling of the THQ body. The column widening is limited to the THQ bottom plate, which in this case is a 100mm wider than the column. Along with that, widening the column too much may lead to a serious overdesign and thus a waste of money. A thickening of the THQ top and bottom plate will also not lead to a much larger dispersion, as the addition of steel comes with an addition of its large material load. The empty body of the THQ is 234mm in width, compared to the 400mm width of the column. By filling the THQ body this 234mm, or a part of it, may be included in the vertical load transition area. Therefore, out of these three models, the filling of the THQ body seems the most effective measure. The other option discussed in this chapter is the replacement of the governing part or its strengthening. For the SS, the option stands to artificially confine it or remove it and place the top column directly on the THQ, with or without a steel endplate attached to the former. Both options may cause a different behaviour in the concrete floor away from the connection and need additional studying. For the bottom column, the option of an endplate is

also considered to ensure the resistance to the double bending of the beam and the concentrated stresses under the THQ webs at the edge of the beam.

The addition chosen for numerical modelling is the filling of the THQ body to transfer the largest load, the vertical load introduced by the top column, through the floor to the bottom column. It is chosen because the addition affects the SS and the bottom column. It should spread the stresses due to the vertical load introduced by the top column more evenly and ensure a stiffer beam in both bending directions of the beam, which should reduce the potential stress peaks in the bottom column, spreading the stresses due to the load from the floor more evenly. Furthermore, this addition is compatible with the removal of the SS and the placement of the concrete column directly upon the THQ. As a last argument in favour of the filling of the THQ, the numerical modelling must be considered. Due to limitation in hardware and time, a simpler design adjustment is welcomed, as it will be modelled closer to reality with fewer simplifications and will therefore provide a more accurate result.

Two additional designs are created for the numerical analysis. The first design is the design in which the THQ is filled with mortar, which differs from the reference design by having the THQ body fully filled with mortar at the location of the connection, as shown in Figure 38. As the assumption is that the two additional webs in the THQ, which enclose the mortar, are sufficiently far away from the connection not to influence it, they are not modelled in the numerical model. The second design is the design in which the THQ is filled with additional steel webs, as shown in Figure 39.

With the behaviour predictions completed and the adjustments for the reference design chosen, the models are set up and numerically analysed to validate the predictions. The numerical analysis of the 3D models is performed in the next chapter.

§6 – 3D numerical model

6.1 Introduction

In this chapter, the numerical 3D models are set up and analysed. The design undergoes simplifications to optimise it for the numerical calculation. The calculations are performed in the program DIANA. The input is reported in detail to allow future research to recreate the model. Apart from the input, the type of analysis and output are reported too. As the goal is not to describe the workings of the finite element model, but the behaviour of the connection, the choices for different aspects of the numerical analysis are only shortly explained without going into detail. The relevant results are presented for all three models and compared to each other and the behaviour predictions of Chapter 4. Then all is concluded and discussed.

6.2 Input

6.2.1 Model dimensions

The size of the model is determined by the limitations of hardware to process a lot of data and the desire for the boundary conditions not to influence the important parts of the structure directly. Every discontinuity in the numerical model causes a local reaction. This includes geometry and the boundary conditions, which account for the rest of the structure around the connection. The model is presented in Figure 42. Here, the colours represent the type of material used. Dark grey is C50/60, light grey is C35/45 and blue is S355. Not all lines and elements have a physical meaning, since they were created to mould the numerical calculation in the desired shape. Furthermore, DIANA has the option to view the results of a specific element, which is constantly used in this research. Splitting the connection into numerous elements creates a convenient way to analyse the results. The floor is 2000mm by 2000mm with the steel beam sticking out on both sides for another 500mm. The columns are each 2000mm in height, which is half their real length.



Figure 42. 3D model

6.2.2 Connection parts

The description of the model elements will proceed from top till bottom. Along with the geometry and material description, the various assumptions of the numerical model are presented. An overview all material properties and design dimensions are listed in Appendix A.

The model starts at the top with the reinforced concrete columns. They are 400mm by 400mm and are made up out of two blocks of 1000mm in height. DIANA has the possibility of entering the predefined concrete code according to the Eurocode, which this thesis uses. Therefore, the material is defined as C50/60 in the program. As the 3D model is only linearly analysed, only the stiffness of the material is important. The stiffness of the different concrete is slightly different and the using this predefined material is convenient and clear. The finite elements used for the columns, and all other concrete and mortar, are the Solid Elements, called Structural Solids in DIANA.

The columns are reinforced by four 25mm rebars. These run as one Truss Element from the top till the bottom of the connection. Their material is B500 and is manually defined, like for all reinforcement. The material is chosen to be non-hardening. The elements are defined as Embedded Reinforcement, which fully integrates the trusses into Solid Elements around it. A perfect connection between the truss elements and the surrounding elements is assumed, which saves computational time. This is a much-needed simplification due to time limitation. The bottom column is in terms of geometry and material an exact copy of the top column.

The top column stands on the structural screed. This is a Solid Element of 60mm high and 2000mm wide in both horizontal directions. Its material is defined to be C35/45 in the predefined Eurocode option in DIANA. The structural screed consists out of 5 elements, which are made for the sake of convenience in the result analysis. It has a perfect connection with the top column, neglecting the mortar and its effect that would be there in reality. The structural screed has two levels of reinforcement, each in one direction. The rebars are 12mm in diameter, have a spacing of 100mm and are made of B500.

The HCS on each side of the THQ is 260mm high, 800mm wide and 2000mm long. The material is defined as C50/60 and the slab is defined as monolith inside the boundaries of the model. The elements have reinforcement running through it. These are B500, 16mm in diameter and 250mm in spacing. The rebars run through the whole connection.

Next to the HCS the concrete filling is located. This is one Solid Element on each side with a height of 260mm, a width of 60mm and a length of 2000mm. Its material is programmed as C35/45.

The THQ consists out of four 2D elements, Regular Curved Shells. On the sides of the model it has dummy elongations of four more plates on each side. This is done to ensure a more realistic stress distribution in the THQ after applying the boundary conditions to it. Modelling this in 2D elements had three main reasons; this element type is well suited for steel plates, a 2D element saves computation time and the webs are 8mm thick, which is way below the mesh size applied, creating a very irregular mesh near the webs which leads to bad results. The webs are 8mm thick, the bottom plate is 20mm thick and the top plate 40mm. All are made out of the predefined S355. The connection of the beam with the concrete around is perfect, which is unpleasant, but it was impossible to create working interface conditions defining the cooperation between the two materials. Therefore, this must be accounted for in the results.

Returning to the integration of the trusses in the Solid Elements; the 2D elements of the beam do not interact with the trusses. Thus, the THQ is modelled without holes, as these cause unreasonably high peak stresses around them. This is valid, as the trusses do not interact with the 2D elements. A further notice on the reinforcement is that it needs an embedding element over the whole length to register the rebar. Therefore, the inside of the THQ is filled with a dummy Solid Element. This allows DIANA to register the rebars as continuous.

6.2.3 Boundary conditions

This model is subjected to a symmetric load combination only. As the linear analysis shows no cracking and stress distributions, the asymmetric load combination is left for the 2D non-linear analysis. The load combination combines all loads discussed in Chapter 4. The three load introductions to the connection are translated into the boundary conditions.
The load combination applied on the numerical models is the symmetrical combination discussed in Chapter 3. In the building, the connection is loaded by the selfload of all parts of the floor and the fixed and variable load applied on the floor, each with their corresponding safety factors. Furthermore, the connection is loaded through the top column by the combined load of the columns, another floor and a roof above this connection.

The total load on the numerical model is split into the boundary conditions, which account for the loads on the model by the building outside the model boundaries, and the loads on the connection parts within the model. Figure 43 schematically shows the boundary conditions. These are calculated with the loads and safety factors for the fictional building from Appendix Table 4 in Appendix A.

The boundary conditions at the concrete floors (V_y and M_y) are translated into a stress distribution over the height and width of the floor edge. The vertical load at the beam edges (V_x) is distributed over the area of the THQ webs and the bending moment on the beam (M_x) is introduced as a set of two horizontal forces, which are spread out over the area of the THQ top and bottom plate. To reach a more realistic stress distribution in the THQ in the concrete and not cause the concrete to be loaded by the load introduction of the beam directly, the beam is elongated by dummy elements for 500mm out of the model on each side. These four boundary conditions

account for the load of the floor outside the numerical model, as discussed in Chapter 3 and Chapter 4.

The vertical load introduced by the top column (N_{c1}) is applied as a distributed stress over the area of the top column. This load accounts for the combined load of the columns, floor and roof in the building above this model. The selfload of the elements in the model is applied as a load per volume. The fixed and variable load on the floor (q_{ss}) are applied as a distributed stress on the top of the SS. The total loads applied on the numerical model are shown in Table 2. The distributed stresses these boundary conditions are converted to in the numerical model are listed in Appendix Table 5 in Appendix A.

The sides of the top plane of the top column are fixed in the horizontal directions, shown by the red pyramids in Figure 43. At the bottom of the bottom column, the plane is fixed in the vertical direction. This way the connection is pinned in place, but remotely. Boundary conditions preventing movement have a large effect on the connection behaviour, even over longer distances. The floor is left without any restrictions on movement or rotation.



Figure 43. Boundary conditions on the 3D model

Total loads in 3D				
V _x	253	kN		
M _x	106	kNm		
Vy	88	kN		
My	68	kNm		
N _{c1}	943	kN		
q _{ss}	6.4	kN/m²		

Table 2. Total loads on the boundary conditions

6.2.4 Meshing

The mesh is quadratic and hexagonal. It is used for its great overview. The mesh size is 50mm for the whole model. Various attempts to reduce the mesh size at the centre of the model and to increase the mesh size towards the outer edges led to a misfunctioning of the model. For the 3D analysis this is enough. In the 2D analysis the mesh will be smaller and the focus will be on the centre of the model. The mesh of the 3D model is shown in Figure 44.

6.2.5 Output

In the 3D model the interest lies with the stresses and the displacements. The stresses are more familiar than strains and thus will be easier to understand when shown on a result. The displacements are another check of the validity of the model. Only the stresses of the parts of interest will be shown in the results. It is also possible to show the



Figure 44. Meshed model

results in a form of principal stresses, but since the loads on the system are strictly in the X-, Y- and Zdirection, it is better to view the results that way for a better overview of what load leads to which stresses.

6.2.6 Additions for Filled THQ

The second numerical model will have a filled THQ. The filling is simplified from mortar into concrete class C35/45. The application is rather simple. The dummy element inside the THQ is now given the predefined material C35/45. To avoid discontinuities, the webs to contain the mortar are not modelled. They are assumed to be outside the model boundaries.

6.2.7 Additions for Webbed THQ

The third numerical model includes additional webs inside the THQ at the location of the column. They have the same thickness as the original webs. Four of them are perpendicular to the beam direction. Two are located under the edges of the column and two are located in between them, under the middle of the column. In the direction of the beam, one web is located in the middle of the beam, spanning from one additional web on the edge to the other web on the edge. These webs are shown in yellow in Figure 45. In blue is the THQ and in grey the column.



Figure 45. Additional webs in yellow

6.3 Results 3D linear analysis

The results of the numerical calculations are best analysed for each part of the design separately, as this will give a better overview. Each relevant part is analysed for all three models simultaneously and compared to each other. The parts to be analysed are the structural screed (SS), concrete filling (CF) and bottom column. While the CF is not considered to be governing part of the connection, it is directly subjected to the load introduction form the top column and therefore good to include in the analysis. In the process of the results analysis, references will be made to the estimations made earlier on in the thesis.



Figure 46. Vertical stress in Y-Z cross-section



Figure 47. Vertical stresses in THQ in Y-Z cross-section



Figure 48. Horizontal stresses in THQ in Y-Z cross-section

Prior to this, the cross-section Y-Z in the middle of the Base model is shown to compare it with the predictions made in Chapter 4. Figure 46 shows the vertical stresses for the concrete parts of the connection. In Chapter 4 (Figure 25) it was assumed that the part of the concrete above the empty body of the THQ would transfer a small load. From the numerical analysis it seems that the concrete there does not transfer vertical load, as the stresses are around zero. The same is valid for the concrete underneath the beam. The bottom column has a small tensile stress in its middle, which is due to the upward curving of the THQ bottom plate. This is possible, since the concrete has a perfect bond with steel in this model. In reality, the concrete would let go of the steel when the cohesion strength is reached, which is assumed to be around 1MPa[6]. In the SS, the stresses above the THQ webs are higher than above the CF. This is due to the steel webs underneath the SS concrete having larger stresses than the CF concrete next to it, due to the stiffness difference of the materials, as described in the Paragraph 4.3 and shown in Figure 29. The stresses in the beam webs, displayed in Figure 47, are much higher than in the concrete above it. This happens in the model because of the way the finite element method works with two different finite elements connected to each other, which are not fully compatible with each other. Furthermore, Figure 46 shows that the SS is governing over the top column due to the larger stresses in the former, while also made of a weaker material. In the same figure it can be seen that the vertical load is transferred over a larger width than predicted in Chapter 4, where the active load transition area is located under the top column directly.

For the connection under design load, the vertical stresses in the THQ webs are shown in Figure 47 and the stresses in the X-direction in Figure 48. As can be seen in the legend, the stresses are very small compared to steel capacity. This shows that the steel beam is not a failure mode.

6.3.1 Structural screed

The structural screed (SS) is analysed from the top and the bottom, as the top is connected to the top column, which transfers a large vertical load to it and the bottom is connected to the THQ and CF, to which the SS transfers the vertical load.



Figure 49. S_{xx} in SS from top in Base model



Figure 50. S_{xx} in SS from top in Filled THQ



Figure 51. S_{xx} in SS from top in Webbed THQ

The first stress analysed is Sxx, in the X-direction, shown in Figure 49 for the Base model, in Figure 50 for the Filled THQ and in Figure 51 for the Webbed THQ. The three models show similar results, which is logical, as the filling of the THQ has no impact on the connection being constrained and having a large negative bending moment at the location of the connection. Due to this bending moment the SS is under tension in the X-direction and will have cracks over an area near the connection. It is important that the reinforcement of the SS will take up the tension or that the beam fully takes up the tension from the bending moment. In this thesis the latter is designed for. The red peak stresses near the top column, which is located in between the red lines, are due to the top column pushing into the SS and thus creating local peak tensions at its edge. As this is a linear elastic analysis, the concrete does not crack when reaching its tensile strength. However, the peak stresses in this analysis give a prediction on the location of the first cracks in the design.

Figure 52 till Figure 57 show the stresses in the Ydirection of the SS from the top and bottom. Figure 52 (Base model), Figure 54 (Filled THQ) and Figure 56 (Webbed THQ) show approximately the same situation as with the stresses in the X-direction. The concrete is in tension due to the semi-constrained connection, as defined in Appendix B. At the edges of the column the concrete has its local tension peaks due to the top column pushing into the SS.

Figure 53 (Base model), Figure 55 (Filled THQ) and Figure 57 (Webbed THQ) show the bottom part of

the SS. Here, it is important to notice the lines of stress concentrations along the X-axis. These are the stress concentrations at the corner of the THQ. These concentrations happen due to the introduction of the tensile stresses in Y-direction, due to the bending moment of the slabs, from the HCS and CF to the SS above the beam. Graphically this is shown in Figure 58. The peak results differ over the three models as the perfect bond comes into play, which allows the tensile stresses to be taken up by the steel and THQ filling in the Filled THQ (Figure 55). The rather low stresses above the beam are due to the THQ top plate

taking up the stresses from the concrete due to the perfect bond. In reality, this interaction stops at the cohesion capacity. Again, it may be assumed that at the location of the tensile stress peaks, the concrete cracks first. For a better understanding of the cracking of the SS, a non-linear analysis must be performed.



Figure 52. S_{yy} in SS from top in Base model



Figure 54. S_{yy} in SS from top in Filled THQ



Figure 56. S_{yy} in SS from top in Webbed THQ



Figure 53. Syy in SS from bottom in Base model



Figure 55. S_{yy} in SS from bottom in Filled THQ



Figure 57. S_{yy} in SS from bottom in Webbed THQ



Figure 58. Tensile force introduction from HCS and CF into SS



Figure 59. Szz in SS from top in Base model



Figure 61. Szz in SS from top in Filled THQ



Figure 63. Szz in SS from top in Webbed THQ



Figure 60. Szz in SS from bottom in Base model



Figure 62. S_{zz} in SS from bottom in Filled THQ



Figure 64. Szz in SS from bottom in Webbed THQ

Figure 59 till Figure 64 show the vertical stresses, Szz, of the structural screed from its top and bottom. As can be seen from the comparison of Figure 59 (Base model – top SS) and Figure 60 (Base model – bottom SS) the stress concentrations are highest at the edge with the THQ webs and is more spread out over the area on the top of the SS. This shows that the stresses in the SS are larger than in the top column, proving the prediction from Chapter 4. Looking at Figure 60, the predictions of a limited vertical load transfer area and the large stress difference in SS, due to the combination of concrete and steel underneath it, prove to be correct. The same comparison for the two other models (Figure 61 for Filled THQ top with Figure 62 the bottom and Figure 63 top and Figure 64 bottom of the Webbed THQ) show that the governing part of the SS is on the side of the connection with the THQ. The difference between the Base model and the adjusted models is best seen by comparing the bottoms of the SS. The Filled THQ (Figure 62) has an active area between the webs, which is missing in the Base model, where the stresses in the middle are around zero. The Filled THQ has stress peaks of -11.48N/mm² at the THQ webs compared to the -16.49N/mm² of the Base model. The Webbed THQ (Figure 64) has a stress peak of -11.35N/mm² at the THQ webs. The grid of the webs is visible, but it may be noticed that at the location of the two additional webs under the edges of the top column no peak stresses appear. It seems these additional webs fall beyond the main load transfer area and are thus of less value for the vertical load transition. It may be advised to relocate these two webs closer to the centre of the connection. However, that may affect the stiffness of the beam at the edges of the column, resulting in a stronger bending of the beam at the edge and a smaller effective area with the bottom column. As mentioned, the stress peaks of the two additions are of similar magnitude according to this model. Upon further vertical loading, the concrete at the location of the stress peaks reaches the compressive stress first. Then, the stress redistribution occurs and the vertical strength of the connection is dependent upon how the stress redistributes and over what area. This must be researched in the non-linear analysis, as it allows for stress redistribution.

It can be concluded that this model shows high horizontal tensile stresses for the SS in the area around the connection. The adjustments have no effect on these stresses. Nor do they have an effect on the stress peak at the THQ edge where the tensile stresses from the HCS and CF are introduced to the SS on top of the THQ. As this is a linear elastic analysis, the result of these stress peaks is probably the location of the first cracks appearing in the concrete, but a better behaviour description is given with the non-linear analysis, which will show the crack pattern and crack location. The adjustments do have a positive effect on the vertical stresses in the SS between the top column and the THQ. Which one of the adjustments is better for the stresses in the SS, is unclear from the results of the 3D numerical model.

6.3.2 Concrete filling

The concrete filling (CF) is analysed due to being in the vertical transition area of the load from the top column to the bottom column.

Figure 65, Figure 67 and Figure 69 show the horizontal stress in the Y-direction in the CF for the Base model, Filled THQ and Webbed THQ respectively. The floor in bending causes the bottom of the CF to be in compression, while the top is in concentrated tension near the connection with the SS, which is loaded in concentrated tension there, as previously shown. The adjustments make no difference in this local stress peak, as previously stated. Figure 66, Figure 68 and Figure 70 show the vertical stress in the Z-direction in the CF for the Base model, Filled THQ and Webbed THQ respectively. Here the effect of the adjustments is clearly visible with the adjustments having lower vertical stresses in the CF.



Figure 65. S_{yy} in SS in Base model



Figure 67. S_{yy} in SS in Filled THQ



Figure 69. S_{yy} in SS in Webbed THQ



Figure 66. S_{zz} in SS in Base model



Figure 68. Szz in SS in THQ Filling



Figure 70. Szz in SS in Webbed THQ

6.3.3 Bottom column

The results of the vertical stresses of the bottom column, shown in Figure 71, Figure 72 and Figure 73 for the Base model, Filled THQ and Webbed THQ respectively, show that the adjustments to the model have a serious effect on the stress distribution.

In the Base model (Figure 71) the middle of the column is in vertical tension, which is due to the THQ bottom plate bending upwards in the middle due to the loading of its flanges by the concrete floor. The steel plate pulls the concrete with it and since this is a linear analysis with perfect concrete-steel connection, this tension in the concrete is possible. This can also be seen in Figure 73 for the Webbed THQ. The Filled THQ (Figure 72) shows the best stress distribution, having the largest active area for vertical load transfer.

The edges of the column are loaded the most due to the double bending predicted in Chapter 4. In Figure 74 the stress distribution along the edge of the bottom column parallel to the Y-direction is shown. This is the edge of the column where the effect of the THQ filling is observed best and it is the edge with the largest stress peaks according the 3D model. These stress distributions show the largest local stresses for the column and should not be taken as a base for designing a connection, as in reality the stresses will redistribute over a larger area. Furthermore, the bottom column is loaded in compression from all side, which leads to a confinement, increasing the concrete strength. The figure clearly shows that the Filled THQ is the best model. This is due to the larger contact area with the bottom column, which allows more load to be transferred through the middle of the beam and reduces the local peaks at Y=125mm and Y=375mm. The adjustments to the reference design are applied in the THQ body, which is located between Y=125 and Y=375. These adjustments lead to the stiffening of the THQ bottom plate. The effect on the stresses in the bottom column edges is a stress peak reduction of 14% for the Webbed THQ and 24% for the Filled THQ.



Figure 71. S_{zz} in bottom column in Base model



Figure 72. Szz in bottom column in Filled THQ



Figure 73. S_{zz} in bottom column in Webbed THQ



Figure 74. Stress distribution 3D model at column edge



Figure 75. Stress distribution 3D model in column middle

However, this holds for the largest stress distribution in the bottom column. This stress distribution should be compared to the vertical stress distribution in the middle of the column, which is shown in Figure 75. The difference in stress level is clear. This difference is due to the bending of the beam in the X-direction (beam direction). Due to this bending, a larger part of the vertical load is transferred in the corners and a

smaller part of the load in the centre. When designing the connection, the stress distribution to account for in the bottom column should be somewhere in between the two stress distributions discussed. In the cross-section in the middle of the column, the stress peak in the column edge is reduced by 21% for the Webbed THQ and by 33% for the Filled THQ.

Depending on the stress redistribution of the peaks stresses in the column edges upon reaching the concrete strength, an additional solution may be thought of to relieve the bottom column from the high stresses in the corners. This could be done by a local thickening of the THQ bottom plate, which would increase its stiffness, diminish the bending and thus increase the contact area with the bottom column, or by the application of steel endplates for the bottom column, which spread the local stress peaks over a larger area towards the concrete.

6.4 Conclusions and discussion

In the analysis of the 3D numerical model a number of conclusions can be made, which are lined up below. For key problems, it includes whether the additions to the model, Filled THQ and Webbed THQ, have improved the stress distribution compared with the Base model.

- The THQ is not the governing element in the connection
- The SS is mostly loaded near the THQ:
 - o In the vertical direction with stress peaks above the THQ webs
 - This is improved by the mortar filling the most
 - $\circ~$ In the Y-direction at the edge of the THQ due to the introduction of vertical tensile stresses from the HCS and CF to the SS
 - This is not improved by the additions
- The SS is loaded in horizontal tension over a large area in its top and will crack for this design load, with the first cracks appearing at the edge of the column
- The CF has lower vertical stresses with the additions, but the stresses in the Y-direction at the top stay unchanged, just like in the SS
- The bottom column is loaded most in its corners with the additions reducing the peak stresses slightly

For most of the stress distributions, the Filled THQ proved to be the better option as it reduced the stress peaks the most and spread the stresses more evenly over a larger effective area. However, a few problems the additions did not solve, with in its head the stress introduction in the Y-direction from the HCS and CF into the SS above the THQ. This is a problem that needs to be looked at in a non-linear analysis to understand what this local failure results in for the whole connection. It is assumed that the result is the formation of the first crack in the concrete, which leads to a redistribution of stresses.

The peak stresses in the edges of the bottom column (Figure 74) were reduced by the adjusted designs, but not entirely solved. As the stress peaks show the location where the stresses first reach the concrete strength, the first locations in the bottom column to reach the concrete strength are the edges and corners. This leads to a stress redistribution. Depending on how the stresses redistribute and over what area, another solution may be applied to spread the stresses in the column edges better and ensure a larger load can be applied on the column. Two possibilities for a solution are the local thickening of the THQ bottom plate at the connection and the application of a steel endplate to the top of the bottom column. Both solutions must lead to a better distribution of stresses over the area of the bottom column. It is necessary to numerically analyse this in 3D, as the two-way bending of the beam is the cause of these accumulating stress peaks, preferably with a non-linear analysis.

This linear analysis did not include concrete cracking. Doing a non-linear analysis, in which cracking is possible, is important as concrete cracking may change the stress distributions. The horizontal tension in the concrete floor may lead to cracking, which could diminish the active area for the vertical load transition and lead to larger stresses in the remaining area. This analysis must be performed in a 2D model, as the 3D model is too large to perform a non-linear analysis on.

For the 2D non-linear model, the relevant cross-section must be selected. As it is the goal to analyse the connection behaviour of the reference model and the adjustments made to it, a cross-section in the Y-Z plane must be chosen, as it accounts for the THQ body filling. Modelling the additions in the X-Z plane is

hard in a 2D model as the floor is not continuous in the Y-direction. Furthermore, the load introduction from the top column to the SS and THQ and further to the bottom column is best analysed in a Y-Z plane, as it will show the stress concentrations due to the limited load transfer area. Therefore, the 2D model will be set up in the Y-Z cross-section, as displayed in Figure 12.

The location of the cross-section is chosen to be in the middle of the connection, in the middle of the columns. This is the cross-section from which the results in Figure 46 and Figure 75 are taken. Here, the asymmetric load combination in X-direction has the same effect as the symmetric load combination set up for this design. Furthermore, at this location there is no additional web perpendicular to the beam direction, which is important for the modelling.

Analysing a 2D model will also allow the redefining of the THQ in the same finite element as the concrete, which will lead to the full compatibility of the two parts in the numerical model, which was not the case for the 3D model. Modelling the THQ with a real thickness is assumed to lead to the dispersion of stresses over the height of the THQ top plate, as shown in Figure 37, which should result in the stresses being dispersed over a larger area, leading to a more realistic stress distribution. Due to mainly this factor, the comparison of the vertical stresses in the SS is not covered for the 3D model, but left to the 2D model, which will provide a more accurate comparison of the stress distributions in the three models.

§7 – 2D model

7.1 Introduction

The 2D model is set up for the non-linear analysis, which must add the concrete cracking and stress redistribution to the design, which was unavailable for the 3D model, as it was too large to perform the non-linear analysis on. Furthermore, due to a different modelling, the THQ can now have a real thickness in the model, which will aid to a more precise stress distribution in the SS. The cross-section to be modelled is shown in Figure 76. It is the cross-section located at the middle of the connection in the Y-Z plane. Graphically, this is shown in Figure 77. This plane is chosen due to the good ability to analyse the vertical load transition from the top column, through the floor to the bottom and because this plane allows to model and analyse the connection adjustments most effective. This does mean that the bending of the floor in the beam (X-) direction and the bending of the column in X-direction for the asymmetric load combination are not accounted for in this model. The load introduced to the connection by the THQ is accounted for in a vertical load in the THQ webs. The reference model and the two adjustment models are loaded in a symmetric and asymmetric way. The connection is loaded in an asymmetric way to analyse the difference in connection behaviour with the symmetric loading.



Figure 77. The graphical cross-section for the 2D model

7.2 Input

The 2D model fully consists out Plane Stress elements, as these are better suited for the design than the Plane Strain elements, which are used for continuous models in the third dimension and which this model is not due to the local presence of the column. The sizes of the elements have been left unchanged apart from the CF, which has been increased by 15mm in its thickness to allow for a better mesh. This does not influence the model, as the HCS is also modelled as a solid concrete plate at the location of the connection. Furthermore, the floor width is shortened till 1370mm and the columns are shortened till 500mm each. All model parts are 100mm thick to allow a good modelling of the 16mm SS reinforcement, which has a

spacing of 100mm. The column reinforcement is not modelled, as it is not present over the whole area of the column and it is unclear how the load transition influences the whole transition area. In this way, the designed model holds the safer approach. The model with the THQ body filled by mortar is modelled with an additional element to account for the mortar inside the THQ, which is still simplified as concrete of class C35/45. The model with additional webs is modelled with one extra web in the middle, the web parallel to the beam direction. The four webs perpendicular to the beam direction could not be modelled in this 2D model. Therefore, the stress distribution of this model will be slightly larger in magnitude than in reality, making a design based on these values a more conservative and safe design.



Figure 78. Stress-strain curve Eurocode

All material is the same as in the 3D model. For all materials predefined in DIANA according to the Eurocode, the Eurocode material properties apply. For the reinforcement steel class B500, applied for all reinforcement, the yield stress is set to 500N/mm² and no hardening is defined, as it is assumed the yielding of the reinforcement is not reached. The stress-strain curve for concrete according to the Eurocode, implemented in DIANA, is depicted in Figure 78. This is a conservative approach, as in reality the concrete does have a resistance to tension directly after

reaching its tensile strength. The cracking behaviour in the numerical model is defined by the Total Strain Rotating crack model. During the examination of stresses, the strains are always checked to determine the location on the stress-strain curve to know if the material is past its failure point or not. The intersections between various parts of the connection have a perfect bond, which must be accounted for in the results.

The model has different axis than the 3D model. In the 2D model the Y-direction is the vertical direction and the X-direction is the direction perpendicular to the beam. The model is meshed with a quadratic, rectangular mesh of 10mm.

For the non-linear analysis, the iteration method used is the Regular Newton-Raphson method with a maximum of 30 iterations. The convergence norm is set to displacement only, with a tolerance of 0.07 and the option to continue the analysis in case of no convergence.

The model is loaded according to the same loading situation described in Chapter 3. This is shown in Figure 79. Due to the two dimensions this model is made in, the introduction of the load to the connection by the beam is translated into a distributed vertical load in the THQ webs. The load introduction by the column (N_1 and M_{y3}) is applied on the top edge of the top column. For the symmetric load combination, this is a constant vertical distributed load over the column width and for the asymmetric load it is a variable vertical distributed load, accounting for the bending moment introduced by the column. The load introduction through the concrete floor (V_{y1} and V_{y2}) is translated into a vertical distributed load over the height of the edges of the HCS and SS. The



Figure 79. Loading situation of the 2D model

bending moment in the HCS direction (M_{y1} and M_{y2}) is introduced through a horizontal variable load distribution over the height of the edges of HCS and SS. As previously stated in Chapter 3, the asymmetric loading assumes a difference of variable load between the left part of the connection and the right part of the connection. The left part has a variable load of $2kN/m^2$ and the right part has a variable load of $6kN/m^2$, which in total add up to the same total load on the connection as for the symmetrical load situation.

An overview of the total loads applied on the 100mm thick model and shown in Figure 79 is given in Table 3 for the symmetric and asymmetric load combination.

2D Symmetric			2D A	symm	etric
V_{y1}	4.79	kN	V _{y1}	3.74	kN
М _{у1}	4.74	kNm	M _{y1}	3.70	kNm
V_{y2}	4.79	kN	V_{y2}	5.84	kN
М _{у2}	4.74	kNm	М _{у2}	5.77	kNm
N _{c1}	234	kN	N _{c1}	234	kN
М _{у3}	0	kNm	М _{у3}	2.08	kNm
q _{ss1}	0.64	kN/m	q _{ss1}	0.34	kN/m
q _{ss2}	0.64	kN/m	q ss2	0.94	kN/m
Table 2 Boundary conditions in 2D model					

Table 3. Boundary conditions in 2D model

All parts of the model are loaded by their selfweight and the

fixed and variable load on the floor is applied as a line load on the SS (q_{ss1} and q_{ss2}). The connection is restricted from displacement in the X- and Y-direction at the bottom of the bottom column. The boundary conditions and model loads are shown in Appendix Table 6 for the symmetric case and in Appendix Table 7 for the asymmetric case. The load on the model is applied in 20 steps of 5% of the total load.

7.3 Results

In this model the interest lies predominantly with the location of the cracks in the concrete and the vertical stresses in the concrete. First, the linear results for the vertical stresses in the concrete parts are shown in Figure 80. It can be noted that a broad part of the concrete transfers the vertical load through the floor. The figure is very similar to Figure 46, in which the same cross-section in the 3D model is shown. Figure 81 shows the non-linear results for the vertical stresses in the Base model. The model shows distortions in the stresses in the concrete floor, which are due to the cracks appearing there. To make the cracking pattern clearer, the horizontal strain is displayed in Figure 82. It shows the positions of the cracks and the deformation of the floor clearly. As this cracking pattern is similar for the other two models and not depending on the THQ body filling, the results for just this model are shown. Cracks appear in the top column too, but this may not be the case in reality, as there would be a mortar layer present between the top column and SS, which would ease the bond between SS and top column and may not lead to the cracking of the latter. The column would also have helical reinforcement, which would prevent it from failure due to lateral tension. For the cracking of the concrete, it is important to assure that the reinforcement does not yield under the design load. The stresses in the SS reinforcement are displayed in Figure 83. The stresses in the rebars are half of their capacity, so the reinforcement does not yield. Figure 84 and Figure 85 show the vertical stresses for the Filled THQ and the Webbed THQ. They show a similar cracking pattern in the floor.



Figure 80. Vertical stresses in Base model (linear)



Figure 82. Horizontal strains Base model (non-linear)



Figure 84. Vertical stresses Filled THQ (non-linear)



Figure 81. Vertical stresses Base model (non-linear)



Figure 83. Stresses in the SS reinforcement (non-linear)



Figure 85. Vertical stresses Webbed THQ (non-linear)

Figure 81, Figure 84 and Figure 85 show that the vertical stresses in the SS are limited to the width of the top column. This is due to the cracking of the concrete, preventing the stress dispersion as in the linear analysis in Figure 80. Table 4 shows the percentages of the vertical load transferred through the SS over the width of the top column compared to the total load introduced by the top column. As the percentages

for the non-linear analysis come close to a 100%, it may be assumed that the SS and the CF and THQ directly beneath it transfer the full load introduced by the top column. This confirms the prediction made in Chapter 4. This allows for a convenient hand calculation of the design strength of the SS as a connection part.

A comparison is made by comparing the vertical stresses in the SS just above its connection with the THQ and CF. The result are shown in Figure 86. Compared to the prediction of the stresses in the SS in Figure 25, it may be stated that the presence of the THQ webs under the SS leads to significantly higher stress above the THQ web compared to the concrete above the neighbouring CF.

Not a single model reaches failure for the design load. The "noisy" data in Figure 86 above the CF in the SS shows that the concrete cracks there. For each model, the average vertical stress in the SS concrete above the CF is shown in Table 5. The concrete in the SS does not crack above the THQ top plate in this model due to the perfect connection between the SS and THQ top plate. In reality, the

concrete will crack there too. According to the Japan Society of Civil Engineers, the strength of the concrete reduces due to lateral cracking to a minimum of 60%[9]. Reducing the capacity of the SS concrete in the model (f_{cm} =43N/mm² in Appendix Table 2 in Appendix A) by 60%, gives a capacity of 25.8N/mm², which is still larger than the results show in the concrete of the SS.

Figure 86 shows that the filling of the THQ body does affect the stress distribution in the SS, with the Filled THQ reducing the peak stress above the THQ web by almost 40%. This is important, as further vertical loading will lead to the local failure of concrete at that location and a redistribution of stresses. Assuming that the mortar in the THQ body is stiff enough to prevent a too large sagging of the THQ top plate, which would make it less active in the vertical load transition and would lead to a stress concentration above the THQ webs, the Filled THQ will have a stress redistribution over the whole width of the THQ as all the area is active in the load transfer. For the Base connection this is not the case, as the THQ body is empty and the area of stress redistribution is therefore assumed to be limited. As the Webbed THQ has no filling between the additional webs, as seen in Figure 45, it may be assumed that the THQ top plate will have a stagging in between the steel webs, which reduces the stress redistribution area. However, due to the additional webs, the total area of stress redistributions is assumed to be higher for the Webbed THQ than for the Base model.

Furthermore, for the Webbed THQ, the stress peak above the THQ web is lower by almost 20%, which means that the Webbed THQ will transfer a larger vertical total load at the start of the local failure of the concrete above the THQ web. Due to the presence of the four perpendicular webs, which are not included in this numerical analysis, the height of this stress peak may be assumed lower in reality, which would mean that the vertical total load transferred prior to failure is larger than according to this model. When it comes to the resistance in the SS to the vertical load from the top column, the Webbed THQ has a larger

Base model	87.74	97.10	%	
Filled THQ	91.51	98.36	%	
Webbed THQ	89.66	98.35	%	
Table 4. Percentages of load from the column				

Linear

Non-linear

Table 4. Percentages of load from the column transferred through the SS over column width

Average stress in CF				
Base	-6.85	N/mm ²		
Filled	-4.45	N/mm ²		
Webbed -5.63 N/mm ²				
Table 5 August stars in CE				

Table 5. Average stresses in CF

resistance than the Base Model, but the Filled THQ has the largest resistance due to the whole area under the column being active in the vertical load transfer.



Figure 86. Vertical stress comparison in SS (non-linear)



Figure 87. Stress distribution in bottom column (non-linear)

Figure 87 shows the stress distributions in the bottom column. Here, the stress peaks at the edges of the column show that for the bottom column the load from the concrete floor is governing over the load from the top column. This answers the question of Paragraph 4.4 which of the load introductions is governing for the bottom column. For the stresses at the edge, it may be noted that the Filled THQ has a peak stress of 5N/mm², approximately 17%, lower than the Base model. This is due to the stiffening of the THQ bottom plate by the mortar filling. Another attention point is that the stress peaks under the THQ webs are lower than for the SS, meaning the bottom column is less affected by the load concentration due to the presence of the THQ in the floor than the SS. The load from the floor leads to a bending of the THQ bottom plate, which, very locally, transfers a large part of the load to the column, at the column edge. Increasing the load from the floor would lead to the local failure of concrete at the edge of the column. This should lead to a certain stress redistribution, but it must be researched how much of the column area may be assumed for the stress redistribution. A good solution for this problem is the application of a steel endplate to the bottom column to ensure a larger area of load transfer, which would allow the bottom column to take up a larger load from the floor. This could mean that the span of the floor may be increased. A good solution for the stress dispersion may lead to a lower concrete class for the column, since the next stress peak in order of magnitude reaches a maximum stress of 10N/mm² in the bottom column. However, it must be noted that this stress distribution is on the unsafe side, due to the bending in the beam direction being absent in this model.



Figure 88. 2D asymmetric Base model (non-linear)



Figure 89. 2D asymmetric Filled THQ (non-linear)



Figure 90. 2D asymmetric Webbed THQ (non-linear)

As the loading is seldom fully symmetric, an exaggerated asymmetric load combination is numerically analysed to compare with the connection response to a symmetric load combination. The vertical stresses in the concrete for the asymmetrical load combination are presented in Figure 88, Figure 89 and Figure 90 for the Base model, Filled THQ and Webbed THQ respectively. The displayed deformation of the models is multiplied by a factor 100 to make them visible to the reader. The results for asymmetric loading show that the stresses concentrate on the more heavily loaded side.

As these figures do not show the difference in stresses very well, the stresses in the SS are plotted out in Figure 91. The numerical results come close to the prediction in Figure 28. For the designed asymmetric load, none of the models reach the concrete strength. The stress distribution in Figure 91 is similar to the symmetric stress distribution in Figure 86, only with larger stresses on the right. Regarding the stress redistribution in the SS for an asymmetric load combination, the same idea holds as for the symmetric load combination: the Filled THQ may have a redistribution of stresses over the whole area under the column, while for the Webbed THQ and the Base model this redistribution area is limited.

Figure 92 shows the vertical stress distribution in the bottom column. It shows that the Filled THQ reduces the stresses by almost 10N/mm², which is approximately by 25%, but the stress peak remains large compared to the rest of the stresses in the bottom column. The bottom column edge becomes more important for the case of asymmetric loading. This can be seen by comparing the stress distribution in Figure 92 with the stress distribution in Figure 87. Otherwise, the stress distributions for the two load cases are rather similar. For this cross-section under this design load, none of the models reach concrete strength in the bottom column of concrete class C50/60, but this model does not include the beam bending and load in the direction of the beam. Just like for the symmetrical loading, a further research on stress redistribution in the bottom column is advised along with a research on a solution for the stress dispersion in the column edges.



Figure 91. Comparison of stress distribution in SS for asymmetric loading (non-linear)



Figure 92. Comparison of stress distribution in bottom column for asymmetric loading (non-linear)

7.4 Conclusions and discussion

The non-linear analysis of the numerical models provided with results, out of which the following conclusions are made:

- The concrete in the connection cracks and the cracking pattern in similar in all three models
- The Filled THQ is the best solution for the resistance to the load introduction from the top column
- The bottom column is most affected by the load introduced by the floor

The concrete in the connection cracks in a way shown in Figure 82. Due to the modelling of the connection parts with a perfect bond, the cracking of the SS above the THQ is not shown but should be present in reality. This cracking affects the strength of the SS. Regarding the cracking, it must be pointed out that the prestressed HCS are modelled as monolith concrete plates without prestressing. The effect of prestressing might change the crack pattern of the HCS, but the CF and SS under the top column are unaffected directly.

The introduction of horizontal tensile stresses from the HCS and CF into the SS above the THQ top plate, found in the 3D linear analysis, caused the first crack in the concrete. The cracking of the concrete evolved from there into the situation depicted in Figure 82. The local failure did not turn into the connection failure at the designed load in the non-linear analysis.

Due to the cracking, the vertical load introduced by the top column is transferred in the SS over the same width as the width of the column. As can be seen in Figure 81, there is no dispersion of stresses as was present in the linear analysis shown in Figure 80. Table 4 underlines this by the percentages of the total vertical load transferred through the area directly underneath the top column at the bottom of the SS. This validates the assumption made in Chapter 4 about the load transfer area being strictly under the top column.

The SS is loaded horizontally in tension due to the bending moment in the floor and vertically due to the load introduction by the top column. The latter causes a stress distribution in the SS as depicted in Figure 86. The stress peaks above the THQ webs show that upon further vertical loading the local failure of the concrete will take place at that location and a stress redistribution will follow. In case the Filled THQ has perfect mortar filling, the stresses will redistribute over the whole area underneath the top column. For the other two models, it should be researched over what width the redistribution of stresses will take place. The Webbed THQ shows a reduction in stress peak of almost 20% compared to the Base model, meaning it will be able to transfer a larger load prior to reaching the concrete strength in the SS.

The bottom column is less affected by the load introduction through the top column, as seen in the comparison of stress peaks above and below the THQ webs in Figure 86 and Figure 87. The bottom column is more affected by the load introduced through the floor, which causes a bending of the THQ bottom plate, which leads to relatively large stress peaks at the bottom column edge. The Filled THQ causes a lower stress peak by approximately 17% due to the stiffening of the THQ bottom plate. However, this improvement is not enough and a separate solution must be applied to the bottom column to facilitate the dispersion of the concentrated stresses in the column edge. In this 2D model, the bending of the floor in the beam direction is not included, which makes the model on the unsafe side regarding the bottom column.

Regarding the additions to the reference model, the Filled THQ gives the best result as it ensures the full area underneath the top column for vertical load transfer and it stiffens the THQ bottom plate, causing a

smaller peak stress at the column edge. This is valid if the mortar fills the THQ body for a 100% and if it will prevent the THQ top plate from sagging. This must be researched. If making a good mortar filling for the THQ appears to be too hard or too expensive, the Webbed THQ may be used instead of the reference model.

For the Webbed THQ and the Base model, a research must be done on the stress redistribution in the SS to determine the area over which the stress redistributes and the total vertical load at failure of both models. Regarding the bottom column, a research must be done on the stress redistribution at the column edge. This should be enlarged with range of solutions for the dispersion of the stresses in the bottom column edges.

The results of this numerical analysis do not show the difference between the designs in a practical way. With the current results of the non-linear analysis, it is possible to perform a calculation of the maximum design load which can be introduced to the connection by the top column. This gives an understanding of how much the connection can be loaded by the load of the building above the connection and shows the difference in design load between the three models.

§8 – Vertical design load

In this chapter, a calculation is performed of the maximum design load that can be introduced to the connection through the top column (from here on: vertical design load). This vertical design load makes a clear comparison for the effectiveness of the adjustments compared to the reference design. Furthermore, this calculation procedure may be used by engineers as a first estimation for the vertical design load of their designs of this connection type, provided they do not differ too much in parameters.

As concluded in Chapter 7, the SS is influenced by the load introduced by the top column, the load from the floors above. A calculation of the design maximum of this load defines the maximum number of floors above this connection (given a fixed total load from each floor). The bottom column is most affected by the load introduced from the floor. The calculation of the design maximum of this load defines the maximum span between columns in the floor. In the 2D model, no account could be made for the bending of the beam in the beam direction, which makes the calculation of the maximum floor span not possible in the current state. However, it is possible to calculate the vertical design load introduced by the top column for each of the models. It is valid to say that the resistance is calculated for the whole connection, as Figure 86 and Figure 87 show that the SS is more affected by the loading through the top column than the bottom column. The procedure is shortly explained below and elaborated upon further in this chapter.

As the stresses accumulate towards the THQ web, assuming the dispersion of stresses over THQ top plate height (discussed in Paragraph 5.3), it may be assumed that over a certain width at the intersection between the THQ top plate and the SS there is a uniformly distributed load, which reaches the design strength of the concrete. This load leads to a vertical stress in the THQ web. Based on the average stress relations between the connection parts obtained from the numerical results, the stresses in the other connection parts (CF and the extra web in the Webbed THQ) are calculated. Summed up and multiplied by their correspondent areas, they form the total vertical design load, which is then divided by the column width to obtain the uniformly distributed stress in the top column, that is the vertical design stress.

As stated in the previous chapter, for the Filled THQ, it is assumed that after the concrete reaches its strength under further loading, the stresses in the Filled THQ will redistribute over the whole area underneath the top column. Therefore, the vertical design stress for this model is equal to the design strength of the concrete in the SS.

The Webbed THQ and the Base model have the situation of the vertical stresses from the top column accumulating towards the THQ webs. However, as can be seen in Figure 86, the accumulation is parabolic for the Base model over the THQ top plate width. This is due to the stress dispersion of the THQ top plate. For the calculation of the vertical design load, use is made of the "stress block"-design principle[12]. This design principle assumes a linear strain distribution in concrete over a certain area, which is translated into a constant distributed stress over a smaller area to ease the design calculation.

For this calculation a few assumptions must be made:

- 1. The vertical strain distribution in the SS is similar to the vertical stress distribution in the SS. This is acceptable, as the vertical stresses have not reached the reduced characteristic strength of the concrete.
- 2. A linear stress distribution may be assumed, which is equivalent to the parabolical stress distribution, which allows the determination of an average width *X*, over which the vertical load towards the THQ web is spread.

As the strain distribution is assumed to be similar to the stress distribution, the calculations are performed using the earlier found stress distribution. Figure 93 shows the stress distribution in the SS with two blue areas of equal area. The parabolical stress distribution on the left is translated into a linear stress distribution on the right. The area (the vertical load on one THQ web) is known and so is the peak stress, leaving the only unknown to be the average width *X*, over which the stresses towards the THQ web are dispersed in the SS at the interaction with the THQ. This average width is *X*= 75mm. It is assumed that over this width there is a linear stress distribution.



Figure 93. Stress distribution in SS with areas marking the total vertical load on the THQ web

For the vertical design load, the situation is regarded in which the vertical peak strain in the SS reaches the failure strain, $\varepsilon_c = 3.5\%$. According to the "stress block"-design principle[12], the stress block spans over a width of D = 0.75*X and has a stress value of f_{cd} . The design stress must be reduced by 40% to account for the weakening due to horizontal tension in the concrete. For concrete class C35/45, this becomes 14N/mm². The width of the stress block becomes D = 56mm. Thus, the maximum stresses accumulated in the SS at the intersection with the THQ top plate may be 14N/mm² over a width of 56mm. Due to the similar geometry of the models and similar stress distribution in the SS near the THQ web, it may be assumed that the this stress block holds for the Base model and for the Webbed THQ.

In Appendix B, a hand calculation is performed to calculate the stresses in the THQ web and CF for a vertical load of 1N/mm² from the top column in the reference model, based on the material stiffness of the two connection parts. With the load from the top column being 5.89N/mm² (see Appendix C), the stresses in the CF and THQ web are calculated and shown in Table 6. The relation between the two stresses is 6.14, which is also the relation between the stiffnesses of the materials. In the linear and non-linear calculation, these stresses are analysed in the 2D numerical model at the location of the border of the CF and THQ web with the SS. This stress data is taken from the concrete and steel directly under the column, thus over a

width of 400mm. Table 6 shows the average stresses recorded in the Base model and the Webbed THQ and the relation between the stresses. According to the results of the numerical models, the hand calculation based on the material stiffness is not valid. As mentioned in the previous chapter, for the non-linear

Syy in N/mm2	Hand	Base model	Webbed THQ
	calculation	Non-linear	Non-linear
CF (N/mm2)	-9.49	-6.99	-5.80
Web (N/mm2)	-58.29	-76.20	-58.25
Web mid (N/mm2)			-61.91
Relation CF:Web (-)	6.14	10.90	10.04
Relation Web:Web mid (-)			1.06

Table 6. Stresses in the models

analysis, the percentage of the vertical load transferred over the width directly under the top column is very close to 100%. Thus, it may be assumed that in reality the vertical load from the top column is transferred by the concrete and steel in the area directly underneath the top column. Therefore, the relations between the stresses in the steel webs and the concrete of CF are used instead of the stress relations based on material stiffness. These stress relations are used to calculate the average stresses in the CF and additional web for the Webbed THQ, knowing the stresses in the THQ webs, which are calculated based on the concrete design strength f_{cd} in the SS over the width D of the stress block. The assumption is made that these stress relations hold until the strength of the concrete in the SS is reached and a stress redistribution occurs.

With all assumptions and stress relations at hand, the maximum vertical design load for the Base model is calculated. Hereby, the load of the continuous floor remains unchanged. The calculation is done according to the following steps.

- 1. Definition of design strength concrete
- 2. Calculation of the stresses in THQ webs due to this stress capacity
- 3. Calculation of the stresses in the CF (and THQ filling and THQ extra web) using the stress relations
- 4. Calculation of the maximum vertical design stress on the SS

The design strength of class C35/45 concrete is f_{cd} = 23.3N/mm². This is the maximum stress allowed in the SS and thus the THQ top over the active width D = 56mm for the Base model. A reduction for lateral cracking is applied as the concrete is under horizontal tension and cracks above the top of the THQ. This reduction brings the vertical design strength to 60% of its original strength, f_{cd} = 14N/mm². The stress in the steel is calculated by:

$$\sigma_{steel} = 14 * \frac{56}{8} = 98N/mm^2$$

The stress relation between the CF and the THQ web is taken from Table 6 as 1:10.9. This makes the stress in the concrete of the CF become:

$$\sigma_{concrete} = \frac{98}{10.9} = 9.0N/mm^2$$

The vertical design stress to be applied is calculated by summing up all stresses multiplied by the areas they work in and dividing that by the width of the top column. The THQ webs are 8mm thick and the CF is 75mm wide.

$$\sigma_{design} = \frac{A_{steel} * \sigma_{steel} + A_{concrete} * \sigma_{concrete}}{A_{column}} = \frac{2 * 8 * 98 + 2 * 75 * 9.0}{400} = 7.30 \text{ N/mm}^2$$

The same calculation can be performed for the Webbed THQ. As the Filled THQ is assumed to redistribute its stresses freely over the whole width underneath the top column, its vertical design stress is equal to the reduced strength of the concrete in the SS. The result is shown in Table 7. The addition of an extra web to the reference model allows for an increase in the vertical design stress of over 30%. Filling the THQ body with

Vertical design stress					
Base model	7.3	N/mm ²			
Filled THQ	14.0	N/mm ²			
Webbed THQ	9.7	N/mm ²			

Table 7. Vertical design stress for three models

mortar allows almost the double of the vertical design stress in the reference model and is therefore the best solution. In terms of addition of extra floors, the weight of one extra floor on one column is calculated to be 691kN in Appendix C. Spread over the area of the column (0.16m²), it is an additional stress of 4.32N/mm². The current load from the top column on the SS, 5.89N/mm² (see Appendix C), accounts for one floor and one roof above the modelled connection. Looking at the values in Table 7, it may be stated that with this floor load for the Filled THQ model, another floor may be added to the building.

This calculation is also valid for the calculation of the combination of a vertical force with a bending moment. The calculated stress in Table 7 is for that case the average stress of the heavier loaded side. Depending on the total vertical load, the maximum bending moment in the top column can be determined.

For the current design, in which the concrete columns are of concrete class C50/60, the vertical design load is rather small. This is due to the SS being of a lower concrete class C35/45 and the SS being loaded horizontally in tension. In Paragraph 5.4.1, an adjustment to the reference design was considered, in which the SS is removed at the connection and the top column is placed directly on the THQ. It is assumed that in this situation, the SS reinforcement is led around the column and the column is not loaded in horizontal tension. Assuming this design is possible, a calculation can be performed to calculate the vertical design stress for the additional application of the top column directly on the THQ without an endplate. The calculation is performed according to the same steps as for the SS. The strength of concrete class C50/60 is $f_{cd} = 33.3$ N/mm². The calculation results are shown in Table 8. The design stresses are much higher than

for the designed models with the SS. In terms of additional floors, the Base model without the SS allows for 2 more floors to be added to the fictional building, the Filled THQ allows for 6 more floors and the Webbed THQ for 3 more floors. This calculation shows that a future research on this adjustment is necessary, as it may lead to a large improvement of the design. However, the vertical design stress calculation for the design option of a column resting directly on the THQ is not

Vertical design stress without SS				
Base model	17.4	N/mm ²		
Filled THQ	33.3	N/mm ²		
Webbed THQ 23.0 N/mm ²				
Table 8. Vertical design stress for the three models without the SS				

valid for the whole connection, since the bottom column is of the same size and strength as the top column, but is additionally loaded by the concrete floor. This means that for this adjustment the connection strengths must be lower than shown in Table 8, but it shows the large difference between the design option to remove the SS at the design option to leave the SS as in the reference design.

§9 – Discussion

In this research, the behaviour of the connection under two load combinations is studied. The load combinations were chosen according to the Eurocode[6] and a design guide for multi-storey buildings[3]. In the construction and use stage, these load combinations have the largest total load acting on the connection. The numerical results showed the reaction of the connection to an asymmetric load to be similar to the reaction of a symmetric load, but with higher stresses on the larger loaded side.

In the designed connection under the load combinations, the governing parts are the structural screed (SS) and the bottom column. The stress distributions in the concrete parts found for these loadings are applicable for a wider range of concrete classes, as they have a similar stiffness and the applied load combination does not cause the concrete to reach its compressive strength. For the steel beam, it has been found that it is not a governing part of the connection. This is shown in Figure 47 and Figure 48, where the stresses in the steel beam are a factor four smaller than its design strength.

The structural screed is loaded horizontally in tension and vertically in compression, which reduces its strength. While this does urge for a solution, the vertical stress distribution found for the SS under the designed load is valid for any structural concrete that would be applied on top of the THQ, independent of the presence of horizontal loading. This is valid because the designed load did not lead to the vertical stress reaching the compressive strength of the concrete of the SS. As shown in the stress distributions in Figure 86 and the vertical design stress introduced by the top column in Table 7, the filling of the THQ body has a large effect of the connection strength, especially for the Filled THQ, which in fact turns the connection into a monolith concrete block. This is valid under the assumption that the THQ body can be filled with mortar for 100% and this mortar has similar properties as concrete of concrete class C35/45, which it was simplified to in this model. It has been assumed that after the stress in the concrete above the THQ web reaches the concrete strength, the stresses will redistribute over the whole area under the top column. For the Base model and the Webbed THQ, this is not the case and a research should be performed on the vertical stress redistribution after reaching the concrete compressive strength.

The stress distribution of the bottom column is dominated by the load introduced from the floor, which leads to the relatively high stress peaks in Figure 87. This stress distribution shows that for an increase in load from the floor, the bottom column will start failing at its edge. In future research, it must be determined how the concrete of the bottom column will redistribute the stresses and what adjustment will disperse the stresses in the edges the best. A good adjustment may allow for a reduction of the concrete class of the columns or an increase in floor span. However, the concrete class of the column is also dependent on the stress distribution in Figure 86, if the decision is made to remove the SS and place the top column directly on the THQ. Regarding the filling of the THQ body, it may be stated that this has a small influence on the stress distribution in the bottom column, with Filled THQ showing the best stress distribution.

From an engineering point of view, the final connection design has a number of shortcomings as a design. Firstly, due to the SS being loaded in tension horizontally and in compression vertically, its design strength must be reduced by 40%. This is a serious bottleneck of the connection and must be solved. One possibility is the removal of the SS and the application of the top column directly on the THQ. The column is of a higher concrete class and thus the vertical design load will increase compared to the current design. If the column is not loaded in horizontal tension, the vertical design load becomes even higher. This case is discussed in Chapter 8 and Table 8 shows the possible vertical design stress for this adjustment (not accounted for the bottom column), which is much higher than for the current designs. Another possibility is the substitution of the concrete in the SS at the connection with another material, which is less sensitive to horizontal tension. It is assumed that the continuity of the SS reinforcement is necessary for a semi-rigid connection, which will avoid the HCS at the connection to sag lower than the neighbouring HCS and have a larger bending moment, which can cause undesired cracking along the length of the HCS. Thus, keeping the reinforcement on each side of the connection connected to each other is desired.

The columns in this design are overdesigned. They can be reduced in width and concrete class. In the case of the column application directly on the THQ, the concrete class should not be reduced. Narrowing down the column in both horizontal directions leads to an increase in vertical stresses in the column and the connection, as described in Paragraph 5.1. If the THQ is filled with mortar, the narrowing of the column will lead to a small increase of stresses due to a smaller load transition area. However, if the Filled THQ is not available and the Webbed THQ is designed, then the narrowing of the column should be combined with the narrowing of the THQ in order to preserve a part the load transition area. As the THQ is also overdesigned, the reduction of the THQ top and bottom plate width is acceptable. However, it should be noted that a smaller column is only aesthetically beneficial, as the reduction of weight and material cost is very small compared the whole building.

Regarding the mortar filling for the THQ, it must be researched if a filling of 100% is achievable. If due to small shrinking or settling the mortar will not fully support the THQ top plate, then the stresses from the top column will concentrate more above the THQ webs and the monolith concrete system will not work. Instead of the THQ, other integrated beams may be considered for this connection type. An open beam like the I-beam may be easier to fully surround by concrete and ensure a good vertical load transition.

If the Filled THQ proves to be hard to design or expensive, the Webbed THQ must be further researched. Here, it is good to look into the number of extra webs, their position and thickness. In the Literature study, additional webs were encountered with a thickness of 20mm, while in this research the webs were designed to be 8mm thick.

The column reinforcement has not been included in the analysis but could be an effective element in the vertical load transition as it could relieve the floor of a part of the vertical load. It should be researched how column reinforcement will affect the stress distributions in the connection and whether its effect is very local or may be assumed to be active over the whole column area.

Regarding the numerical modelling, it must be stated that this connection is complex. Due to the three loading directions and a large area with numerous elements, performing a non-linear analysis in 3D proved to be impossible. For the future analyses, it is advised to simplify the analysis by taking a small part of the connection and only the relevant load for that connection part. However, for the bottom column, a non-linear analysis should be performed in 3D, since the column is dependent on the load of the floor in two directions.

In the numerical modelling, the application of interface conditions failed in DIANA, which led to the perfect bonds between connection parts. While it has been accounted for, a numerical calculation with interface conditions would be more accurate. Regarding the finite element program, it is advised to use DIANA only in case of experience with the program, as the information support for the program is limited.

In this thesis, only the structural side of the connection was researched. No account has been made of the cost of the connection. In a future research, a cost analysis of this connection type should be performed.

§10 – Conclusion

In this thesis, a research is performed on the connection between one storey high concrete columns and a continuous floor, which consist of hollow core slabs (HCS), carried by an integrated steel beam (THQ) and covered with a reinforced structural screed (SS). The goal is to determine the strength of the connection and provide the reader with an insight in the behaviour of the connection under the designed load combinations and an analytical method for strength calculation of similar designs. The motivation behind the research is to obtain an effective construction method for office buildings, which is cheaper than the design of continuous floors with steel columns.

A reference model is designed (Figure 9), for which a behaviour prediction is made based on engineering thinking. The model is loaded by a symmetric and an asymmetric load combination. The load combinations consist out of a load introduced by the column on top of the connection, a load introduced by the concrete floor directly to the connection and a load introduced by the THQ. The prediction states that the bottom column and the structural screed are the governing connection parts. Upon this prediction, possible adjustments to the models are analysed in order to find an adjustment, which improves the strength of the connection. The best predicted solution is the filling of the empty THQ body with mortar or additional steel webs. Three numerical models are set up for the reference model (Base model), the model with the THQ body filled with mortar (Filled THQ) and the model with the THQ body filled with additional webs (Webbed THQ). All three are analysed in the finite element program DIANA in a 3D linear analysis and a 2D non-linear analysis. Then, using the numerical results, the vertical design strength is calculated for all three models.

The results of the numerical calculations show that the prediction regarding the SS and bottom column being governing connection parts are proved right. For the bottom column, the load introduced by the floor is governing over the load introduced by the top column to the connection. The bottom column is loaded most in its edges due to the double bending of the floor and the beam not being stiff enough to ensure the load is transferred from the floor to the bottom column over a larger area. The adjustments to the reference model make the bottom plate of the THQ stiffer, which slightly flattens the stress distribution, with the Filled THQ showing the best improvement (Figure 87). The SS is influenced by the load introduction from the top column and by the bending moment in the floor, which leads to a horizontal tension in the concrete. The horizontal tension in the SS leads to cracking of the concrete, which reduces the strength of the concrete and limits the vertical load transition area in the SS to the area directly under the column. The SS is strongly affected by a limited load transition as all the area under the column is active in the load transition (Figure 86). The models under the asymmetric load combination show similar results as under the symmetric load combination, with only an increase in magnitude of the stresses on the heavier loaded side (Figure 91 & Figure 92).

For all three models, a maximum design load introduced by the top column is analytically calculated based on the numerical results. This calculation shows that the addition of an extra web in the THQ body improves the vertical design load by 30% and the addition of mortar in the THQ body improves the design load by almost 100% compared to the reference model (Table 7). The calculation procedure may be applied as an estimation of the maximum design load introduced by the top column for connections similar to the designed connection in this thesis. This calculation procedure is not limited by the concrete class of the concrete used in the design. From the results of the numerical and analytical calculations, a number of conclusions can be drawn. The three connection designs for this research do not fail under the symmetric load combination. This cannot be concluded for the asymmetric load combination, since it was not analysed in 3D and the influence of the bending of the beam in two directions is not accounted for in the numerical results. It can be concluded that the asymmetric load combination has a larger influence on the bottom column (Figure 87 & Figure 92) than on the SS (Figure 86 & Figure 91).

In their current form, the designed connections are ineffective and not suited for use in practice, as they are not balanced enough. For these designs, the THQ and the columns are overdesigned. The THQ should be reduced in width, which allows for a larger load transfer area and an increase in the design strength for the Base model and Webbed THQ.

The top column is of a concrete class C50/60 with a design strength of 33.3N/mm², while the highest vertical design load on the connection introduced by the column is limited to 14N/mm² (Table 7). The SS, which is under horizontal tension, forms a large bottleneck in the design. With the removal of the SS and the positioning of the concrete column directly on the THQ, the concrete columns can be used to their full capacity and a vertical design stress closer to the values in Table 8 may be obtained, which is a very large difference from the current design (Table 7).

In its current state, the Filled THQ is the best option out three designs, followed by the Webbed THQ. Leaving the THQ body empty leads to the largest bottlenecking in terms of connection resistance. For further improvement of the connection design, it is advised to take the Filled THQ as the reference model.

While the designed connection with its variations is not effective for use in practice yet, its drawbacks can be solved with a few adjustments, making it an attractive design for engineers. For this, a number of researches should be conducted.

The first adjustment to research is the replacement of the SS at the connection for another material or connection part with a larger strength. Hereby, the reinforcement of the SS should remain continuous, but not cause a strength reduction of the material above the THQ at the connection.

A further research must be performed on the redistribution of stresses in the bottom column to determine over what area the stresses redistribute prior to connection failure. This should be combined with a research on the effect of adjustments to the bottom column, which lead to a dispersion of stresses in its edges. This could be a steel endplate or another material able to disperse stresses well.

Regarding the filling of the THQ body with mortar, it must be researched if the mortar can ensure the prevention of sagging of the THQ top plate under vertical load. If the THQ top plate sags too much due to a low stiffness of the mortar or the inability to fully fill the THQ body, then the assumption of the stress redistribution over the whole area under the column no longer holds for the Filled THQ. In that case, a research must be performed on the stress redistribution of the concrete above the THQ for the design with additional webs in the beam. This will allow for the calculation of the collapse load of the connection and the definition of the area of the stress redistribution, which can lead towards a more effective positioning of additional steel webs.

Since the connection is complex from a numerical point of view, for future research, it is advised to split up the desired areas of research in more simplified models of parts of the connection with their governing load.

Appendix A: Material, loads, geometry and validation

In this appendix a swift overview is presented of:

- The structure geometry
- The material properties
- The loads on the structure
- The boundary conditions for the numerical calculation

Appendix Table 1 presents the relevant geometry of the fictional structure and the designed connection.

THQ	Webs	tw	8	mm	HCS	Heigh	t 260	mm
		h	260	mm		Width	1200	mm
	Тор	to	40	mm		Column to Colur	nn 8000	mm
		bo	234	mm		Rebar diameter	16	mm
	Bottom	tu	20	mm		Rebar spacing	250	mm
		bu	500	mm				
	Holes	plates	34	mm	CF	Heigh	t 260	mm
		webs	22	mm		Width	60	mm
	Column to Colu	umn	6000	mm		Column to Colur	nn 6000	mm
	Extra webs	tw	8	mm	Columns	Heigh	t 4000	mm
		h	260	mm		Width	400	mm
		length	400	mm		Depth	400	mm
	Extra webs +	tw	8	mm		Rebar diameter	25	mm
		h	260	mm		Rebar spacing	200	mm
		length	250	mm				
					SS	Heigh	t 60	mm
	THQ filling	h	260	mm		Column to Colur	nn X 6000	mm
		d	250	mm		Column to Colur	nn Y 8000	mm
		length	2000	mm		Rebar diameter	12	mm
						Rebar spacing	100	mm

Appendix Table 1. Geometry of connection parts

The materials used in the connection are concrete and steel. Both have two classes implemented in the construction. The prefabricated concrete parts (HCS and columns) are made of the concrete class C50/60 and the concrete parts cast in-situ are made of concrete class C35/45. The construction steel parts (THQ) used is made of steel class S355 and the reinforcement is made of steel class B500. Their material properties are shown in Appendix Table 2 for concrete and Appendix Table 3 for steel.

Concrete	C35/45	C50/60
f _{ck} (MPa – N/mm²)	35	50
Material factor γm	1.5	1.5
f _{cd} (MPa – N/mm²)	23.3	33.3
f _{cm} (MPa – N/mm²)	43	58
f _{ctm} (MPa -N/mm ²)	3.2	4.1
E _{cm} (MPa)	34000	37000
ε _{c1} (‰)	2.25	2.45
ε _{cu1} (‰)	3.5	3.5
v-uncracked (Poisson)	0.2	0.2

Steel	S355	B500
fy (MPa – N/mm²)	355	500
Material factor ym	1	1.15
fy _d (MPa – N/mm ²)	355.0	434.8
E _{cm} (MPa)	210000	210000
v (Poisson)	0.3	0.3

Appendix Table 3. Material properties steel

Appendix Table 2. Material properties concrete

In Appendix Table 4 the general loads on the construction are presented. The loads are based on the Eurocode[6] and the design guide[3].

	General loads			
Floor load				
	Selfweight			
		Hollow core slabs	3.9	kN/m2
		Structural screed	1.5	kN/m2
		THQ	1.9	kN/m
	Fixed load		0.3	kN/m2
	Variable load		4	kN/m2
Column load				
	Selfweight			
		Column	100	kN/m2
		Connection details	0	kN/m2
	Load from top			
Roof load				
	Selfweight		1	kN/m2
	Variable load		2	kN/m2
ULS				
	part factor variable	loads	1.5	
	part factor fixed lo	ads (unfavourable)	1.35	
	part factor fixed lo	ads (favourable)	0.9	
Weight				
	Reinforced concret	te	25	kN/m3
	Steel		80	kN/m3

Appendix Table 4. General loads and load factors

Out of these loads all boundary conditions are calculated. The spreadsheet with the calculations is provided in Appendix C. The calculations are done based on construction mechanics and the geometry of the numerical model. For the calculations, it is important to know that the beam is considered to be fully constrained in the connection. Hollow core slabs on integrated beams are usually modelled as a hinged

connection. Due to the presence of the reinforced structural screed, this is not realistic as a negative moment will appear in the floor in the hollow core slab direction. Thus, the hollow core slab and the structural screed at the beam are considered to be halfway between a fully constrained and a hinged connection. For the calculation of the boundary conditions the hollow cores of the connection are assumed hollow, but in the model the slabs have the full self-weight of reinforced concrete as the hollow cores are assumed to be filled at the location of the connection. The boundary conditions applied are shown in Appendix Table 5 for the 3D model, Appendix Table 6 for the symmetrically loaded 2D model and Appendix Table 7 for the asymmetrically loaded 2D model.

Boundary conditions	3D	
Column stress	-5.84	N/mm ²
Floor shear	-0.137	N/mm ²
Floor moment	1.97	N/mm ²
Beam shear	-484	N/mm
Beam moment:		
top stress	1684	N/mm
bottom stress	842	N/mm
Model loads	3D	
Fixed & variable load	-0.006405	N/mm ²
Concrete selfload	-3.38E-05	N/mm ³
Steel selfload	-1.08E-04	N/mm ³

Appendix Table 5. Boundary conditions 3D model

Boundary conditions	2D - symmetric	
Column stress	-584	N/mm
Floor shear	-15	N/mm
Floor moment	278	N/mm
Beam shear	-0.4124	N/mm ³
Model loads	2D - symmetric	
Fixed & variable load	-0.00641	N/mm
Concrete selfload	-3.4E-05	N/mm ³
Steel selfload	-1.08E-04	N/mm ³

Appendix Table 6. Boundary conditions 2D symmetric loading

Boundary conditions	2D - asymmetric	
Column stress	-584	N/mm
Column moment	120	N/mm
Floor shear L	-11.7	N/mm
Floor shear R	-18.3	N/mm
Floor moment L	217	N/mm
Floor moment R	338	N/mm
Beam shear	-0.4124	N/mm ³
Model loads	2D - asymmetric	
Fixed & variable load L	-0.34	N/mm
Fixed & variable load R	-0.94	N/mm
Concrete selfload	-3.4E-05	N/mm ³
Steel selfload	-1.08E-04	N/mm ³

Appendix Table 7. Boundary conditions 2D asymmetric loading



Appendix B: Parameter validation and hand calculation
Load Jrom	roof: (1,3	5 · 1,0 + 1,5 « N/m²	·2)·8 uN/m²	m·6m	= 20 9 4
Johal load	on column:	1610 WW			
Resistance	: 5328 u		Load :	1610	ĸN
		LOKI			



vali	ation	r wo	b. ar	d CF		Cf	? : C 2	35/45 \$
C.35/4	5 :	Jcd =	23	3 N/2	nm ²	web	: S:	355
\$335	- :	fyd =	355	- N/m	m ²			
area	CF	: 60)mm x	400 mm	n = Act	- = 240	000 mn	,2
area	Web	: 8	mm ×	(4 <i>0</i> 0 mm	, - 2×	22 mm)	- 284	8 mm = A
Assein	np Hon	: onl	y area	betweer	r colum	ns tocal	gees l	oad.
(Coh.	serval lu	re)						
70 tal 2 ×	0000 V	es/stance 0×23	, 3 +	.clfy : 2848 .	355) =	3140,	5 uN	
This	is	larger	than	tootto	tofa	l load	d on	the
bot	tom clo	colu n fl	un .	and t	hus l	arger	Hhan	total





Resistance under cohemn of SS in vertical direction. Re NRd = 400mm + 400 mm + 23,3 N/mm² = 3728 LN/ This is larger than total load on the bottom column and thus larger than total load on SS. (OK) Resistance at top of SS to tension due to bending moment Ot $\frac{N}{120}$ firm = 3,2 N/mm² N = 171, 25 $N/mm = 0_{\pm} \cdot \frac{320}{2} \cdot \frac{1}{2}$ $\sigma_t = 2, 34 \, N/mm^2$ OK





Appendix C: Boundary condition calculations

The boundary conditions are calculated based on the loads on the loads on the structure, which are based on the Eurocode[6] and design guide[3]. The calculation procedure is as follow:

- 1. Definition of all material loads and fixed and variable loads in the fictional building
- 2. Definition of load transfer in the building
- 3. Calculation of the loads for each load introduction separately based on the geometrical location of the boundary in the model
- 4. Calculation of the applied stress distributions at the boundaries

Step 3 explains the slight difference between the boundary conditions of the 3D model and the 2D model, since the two models have a different (geometrical) size. For instance, if a model includes a larger area of the floor, it will have a smaller load at its boundary than a smaller model. The difference between the loads of the boundary conditions is present in the model itself in the form of material load and fixed and variable load on the floor. These calculations were performed in an Excel spreadsheet of which a simplified version is shown below.

			1.97 N/mm2	II	1.97 N/mm2	r moment	Edge floo					
			-0.14 N/mm2	II	-0.14 N/mm2	r load vert.	Edge floo					
			-3.4E-05 N/mm3	II	-33.75 kN/m3	elfload	Column se					
			-0.00527 N/mm2	П	-5.265 kN/m2	S	Load on H					
			-0.00641 N/mm2	П	-6.405 kN/m2	S	Load on S					
			-5.8919 N/mm2	Ш	-943 kN	op of SS fro	Load on to					
							MODEL					
									8 N/mm3	0.00010	elfload	Steel THQ s
0.13695 N/mm2		Face distr. Shear					0.006405 N/mm2	II	5 kN/m2	6.40	able load:	Fixed + vari
43.824 kN/m	8m)	Shear at edge (x=0.					0.005265 N/mm2	II	5 kN/m2	5.26		HCS load:
54.78 kN/m		Vmax (x=0m)					3.38E-05 N/mm3	11	5 kN/m3	33.7	fload:	Column sel
		Floor: shear load	81 KN	13	ate due to moment	Load on pla	115 N/mm2	II	5 kN/m	11	h webs:	Load in bot
			96 kN/m	2 2	inforcement struc.s	Load on re	5.8 N/mm2	II	2 kN/m2	584	ď	Column loa
1.97 N/mm2	distr.	Stress top bottom of	43 kN	9	Q web and side con	Load on TH	0.017205 N/mm2	11	2 kN/m2	: 17.	ad slabs and screed:	Variable lo
0.213 m	s distr.	Arm in classic stres	33 kN	16	west column	Load on lov	0.00843 N/mm2	II	4 kN/m2	œ	ad screed:	Variable lo
33.60 kNm/m	x=3.2m)	Moment on edge ()	09 kN	2		Load roof	0.008775 N/mm2	11	8 kN/m2	.00	ad slab:	Variable lo
36.52 kNm/m	middle beam)	Moment on x=0m (22 kN		olumn	Load one c						
	0	Floor: moment load	91 kN	6	oor	Load one f		80 kN/m3			Steel	
			15 kN/m	1		q.b		25 kN/m3		ete	Reinforced concr	Weight
487 N/mm	i one web	Distributed load on	3.7 kN/m2	11		q.fl						
253 kN		Shear on x=0.8m										
345 kN		Shear on x=0m	32 m	0.	It	Floor heigh		0.9		oads (favourable)	part factor fixed I	
	on webs	THQ: Vertical load of	2		columns above	Number of						
					roofs	Number of		35	<u>1</u>	oads (unfavourable)	part factor fixed I	
847 N/mm	tom	Distr. Line load bot			floors above	Number of		1.5		le loads	part factor variab	ULS
1695 N/mm		Distr. Line load top	ω		storeys	Number of						
424 kN	(top, bottom)	Force in each plate	37 m	0.	etween slabs	Distance b						
106 kNm	3	Moment at position	06 m	0.	ictural screed	Height stru		2 kN/m2			Variable load	
0.8 m	edge model	Distance column to	16 m2	0.	n	Area colum		1 kN/m2			Selfweight	
nt	ad due to momen	THQ: Horizontal loa	4 m		nmr	Height colu						Roof load
			6 m		B	Length bea					Load from top	
442555 N	•	Capacity in 1 meter	48 m2		per column	Area floor		0 kN/m2		Connection details		
1018 mm2	er	Area bars in 1 mete	8 M		low core slabs	Length hol		.00 kN/m2		Column		
113 mm2		Area bar	L9 kN/m		veight	THQ total v					Selfweight	
9	1 meter	Number of bars in 2									đ	Column loa
100 mm		Spacing	00 mm	л	bu			4 kN/m2			Variable load	
12 mm		Diameter bar	20 mm		Bottom tu			0.3 kN/m2			Fixed load	
	SS	Reinforcement of S	34 mm	2	ьо			1.9 kN/m		THQ		
			40 mm		Top to			1.5 kN/m2		Structural screed		
0.25 m		Arm in THQ	60 mm	2	Ъ			3.9 kN/m2		Hollow core slabs		
0.2467 m	sqt	Arm in concrete sla	8 mm		Walls tw						Selfweight	
0.06 m	ckness	Concrete filling this				THQ						Floor load
2 m	nodel	Height column in m	34 mm		holes							
0.26 m	eight	Hollow Core Slab h								on calculation	: boundary conditic	Appendix (

			Asymmet	ric loading	; 2D				
					<u> </u>				
For variab	le load = 2	kN/m2				For variab	le load = 6	kN/m2	
	Var. Load	Z	kN/m2				Var. Load	р	kN/mz
		00	1.51/					120	
	a.p	55 10 7	kN/m				d.p	159	kN/m
	q.11	10.7	KIN/IIIZ				q.11	10.7	KIN/IIIZ
THO: Horizontal load	due to mo	oment				ontal load	due to ma	oment	
Distance column to 6		0.8	m		Distance (olumn to e		0.8	m
Moment at position		82	kNm		Moment a	t nosition	uge mous	128	kNm
Force in each plate (ton. bottor	328	kN		Force in e	ach plate (ton, hottor	513	kN
Distr. Line load top		1314	N/mm		Distr. Line	load top		2053	N/mm
Distr. Line load botto	om	657	N/mm		Distr. Line	load botto	om	1027	N/mm
THQ: Vertical load or	n webs				THQ: Vert	ical load or	n webs		
Shear on x=0m		268	kN		Shear on y	(=0m		418	kN
Shear on x=0.8m		196	kN		Shear on >	(=0.8m		307	kN
Distributed load on o	one web	378	N/mm		Distribute	d load on d	one web	590	N/mm
Floor: moment load					Floor: mo	ment load			
Moment at column		57.04	kNm/m		Moment a	nt column		89.04	kNm/m
Moment on edge (x=	=3.2m)	26.24	kNm/m		Moment o	on edge (x=	-3.2m)	40.96	kNm/m
Arm in classic stress	distr.	0.213	m		Arm in cla	ssic stress	distr.	0.213	m
Stress top bottom di	str.	1.54	N/mm2		Stress top	bottom di	str.	2.40	N/mm2
Moment on edge (x=	=3.5m)	36.99	kNm/m		Moment	on edge (x=	=3.5m)	57.74	kNm/m
Stress distr. (x=3.5)		2.17	N/mm2		Stress dist	r. (x=3.5)		3.38	N/mm2
Elecry cheer lead					Floorisho	arlaad			
Floor: snear load		12 70	kNI/m		Floor: sne	arioad		66 70	kNI /m
Shear at edge (y=0.8	m)	42.70	kN/m		Shoar at o	////) dgo (y=0.8	m)	52 /2/	kN/m
Face distr Shear (x-	0.8m)	0 10695	N/mm2		Face distr	Shoar (v-l	111) 1 8m)	0 16695	N/mm2
Face distr. Snear $(x=0.8m)$ Shear at edge $(x=0.5m)$		37 4325	kN/m		Shear at e	Shear at edge (x=0.5m)		58 4325	kN/m
Face distr. Shear (x=	0.5m)	0.11697656	N/mm2		Face distr	Shear (x=	0.5m)	0.182602	N/mm2
		0.2200					,	0.121	
Fixed and variable lo	oad (left)	0.003405	N/mm2		Fixed and	variable lo	ad (right)	0.009405	N/mm2
Column max stress d	lue to mon	nent 2D	1	N/mm2		For 100mr	n:	120	N/mm
Shear webs 2D		0.4124	N/mm3						
						2D BC Asy	mmetric		
				Γ	T	V _{y1}	3.74	kN	
						M _{v1}	3.70	kNm	
					1	V	5 84	μN	
						v yz	5.01	L NLm	
						IVI _{y2}	5.77		
						N _{c1}	234	kN	
						M _{y3}	2.08	kNm	
						q _{ss1}	0.34	kN/m	
						q _{ss2}	0.94	kN/m	

				Symmetri	ic loading 2	D and 3D				
				_						
	For variab	le load = 4	kN/m2				For variab	le load = 4	kN/m2	
		Var. Load	4	kN/m2				Var. Load	4	kN/m2
		q.b	114.3682	kN/m				q.b	114.3682	kN/m
		q.fl	13.695	kN/m2				q.fl	13.695	kN/m2
THQ: Hori	zontal load	I due to mo	oment			THQ: Horiz	zontal load	I due to mo	oment	
Distance of	column to e	edge mode	0.8	m		Distance c	olumn to e	edge mode	0.8	m
Moment a	at position		105.2188	kNm		Moment a	t position		105.2188	kNm
Force in e	ach plate (top, bottoi	420.875	kN		Force in e	ach plate (top, botto	420.875	kN
Distr. Line	e load top		1683.5	N/mm		Distr. Line	load top		1683.5	N/mm
Distr. Line	e load botto	om	841.75	N/mm		Distr. Line	load botto	om	841.75	N/mm
THQ: Vert	ical load o	n webs				THQ: Vert	ical load o	n webs		
Shear on Shear	x=0m		343.1046	KN		Shear on x	(=0m		343.1046	KN
Shear on x	x=0.8m		251.6101	KN		Shear on x	(=0.8m		251.6101	KN
Distribute	ed load on (one web	483.8655	N/mm		Distribute	d load on (one web	483.8655	N/mm
-1						-1				
Floor: mo	ment load		72.04	1.51 /		Floor: moi	ment load		70.04	1.51 /
Noment	at column	22.	/3.04	KNM/M		Nomenta	it column	22.	73.04	KNM/M
Noment	on edge (x=	=3.2m)	33.5984	KNM/M		Noment	on edge (x:	=3.2m)	33.5984	KNM/M
Arm in cia	ISSIC STRESS	aistr.	0.213333	m Ni /as as 2		Arm in cia	ssic stress	aistr.	0.213333	m
Stress top	bottom di	Str.	1.968656	N/mm2		Stress top	bottom di	Str.	1.968656	N/mm2
ivioment o	on eage (x=	=3.5m)	47.36188	KINM/M		Noment of	on eage (x=	=3.5M)	47.36188	KNM/M
Stress dis	tr. (x=3.5)		2.77511	N/mm2		Stress dist	r. (x=3.5)		2.77511	N/mm2
Floorisho	arlaad					Floorisho	arlaad			
FIGOR: She			F 4 70	L(NL/ma		Floor: sne			F 4 70	LAL/ma
Choor of o	UIII) dae (v=0.9		54.78			Vinax (x=u	/11) dae (v=0.9		54.78	KIN/III
Enco distr	Shoor (v-	(11) 0.8m)	43.824	KIN/III		Snear at e	uge (x=0.8	(11) 0.8m)	43.824	KIN/III
Shoor at o	dao (x = 0.5)	m)	47 0225	kNI/m		Shoar at o		m)	47 0225	kN/m
Face distr. Shear (x=0.5m)		47.9323	N/mm2		Eaco distr	Shoar (v-	0.5m)	47.9323	NI/mm2	
			0.145785	11/11/12		race uisti.	Sileal (x-		0.145785	11/11/12
Edge floo	r load vert		-0 13695	N/mm2		Edge floor	- load vert		-0 13695	N/mm2
Edge floo	r moment		1 968656	N/mm2		Edge floor	moment		1 968656	N/mm2
Distr. Line	load top		1683.5	N/mm		Distr. Line	load top		1683.5	N/mm
Distr. Line	load botto) Dm	841.75	N/mm		Distr. Line	load botto) Dm	841.75	N/mm
Distribute	d load on i	nne weh	483 8655	N/mm		Distribute	d load on	nne weh	483 8655	N/mm
			403.0033						405.0055	
Fixed and	variable lo	oad (left)	0.006405	N/mm2		Fixed and	variable lo	bad (right)	0.006405	N/mm2
Shear wel	bs 2D		0.412385	N/mm3						
	2D BC Sym	metric					3D BC			
	V	4 70225	LNI	0.40 = 100			v	252	LNI	
	v _{y1}	4.79525	KIN	0061 1001			v _x	255	KIN	
	M _{y1}	4.736188	kNm	Over 100r	mm		M _x	106	kNm	
	V _{y2}	4.79325	kN	Over 100r	mm		Vy	44	kN/m	Over 2 m
	M _{v2}	4.736188	kNm	Over 100r	mm		M _v	34	kNm/m	Over 2 m
	N-1	222 76	kN	Over 100	mm		, N.1	0/12	kN	
	•••••	255.70		0.001			· •c1	543	KIN 2	
	ıvı _{y3}	0	ĸnm	Over 100r	nm		Чss	6.4	км/m ⁻	
	q _{ss}	0.64	kN/m	Over 100r	mm					

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