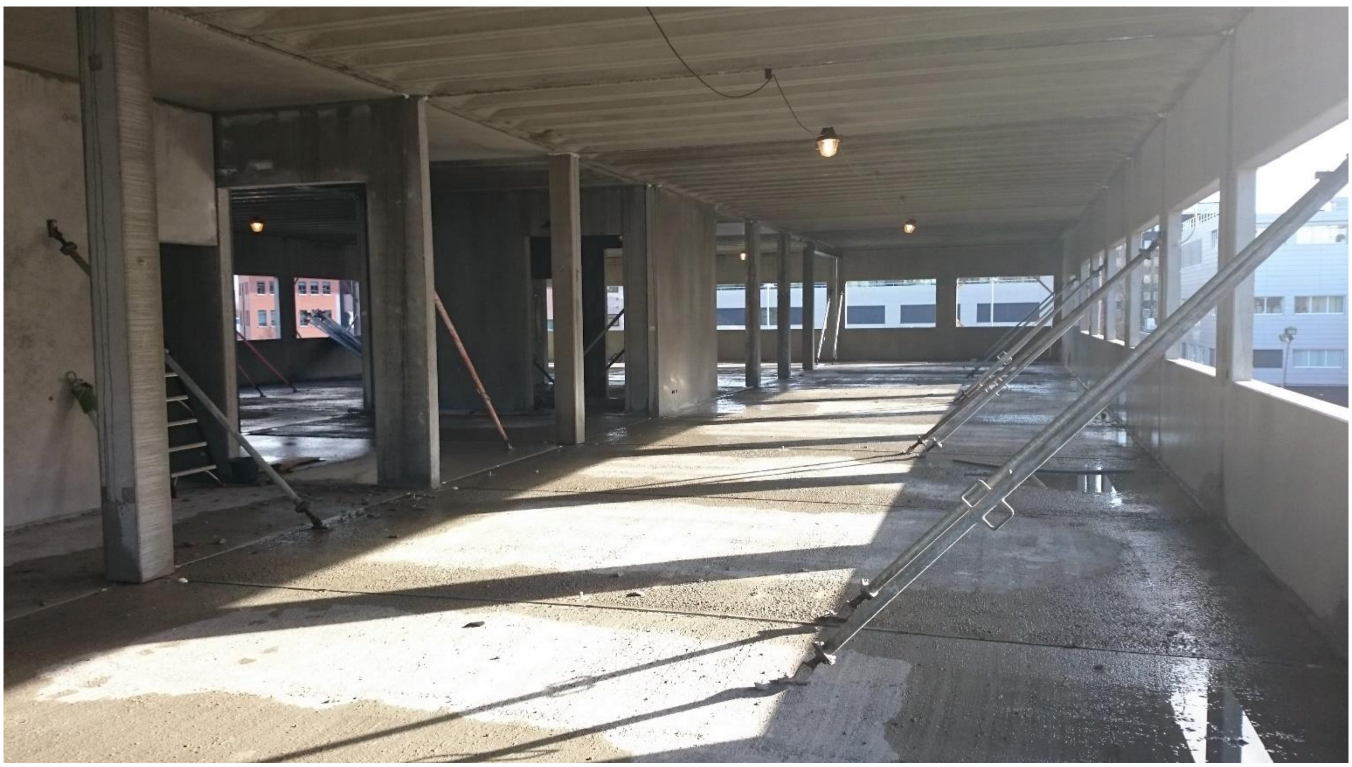
PREFAB CONCRETE BUILDING SYSTEM DURING AN EARTHQUAKE



3/31/2017

Earthquake resistance of the CD-20 building system

This thesis is the culmination of the Masters study of Arie Koster. It describes the testing of the CD-20 building system loaded by an earthquake.

Prefab concrete building system during an earthquake

EARTHQUAKE RESISTANCE OF THE CD-20 BUILDING SYSTEM

FOREWORD

The document that lies before you is the culmination of years long journey in Delft. With this document I will conclude my masters in building engineering. The subject of this master thesis is: the behavior of precast concrete structures during an earthquake. There are several parties I would like to thank for their part in helping me finish my thesis.

First I would like to thank my professors and teachers at the Technical University Delft for giving me the opportunity to write this dissertation. They have been critical in a supportive way and showed me where improvements needed to be made. Secondly I would like to thank Ir Reinier Ringers for all the time he has spent helping me model the connections and the building correctly. Without his help I would never have gotten this far. I am grateful for the opportunity RHDHV has given me to do my dissertation under their supervision. Also CD20 has been very helpful allowing me to take a look inside their factories.

I want to give special thanks to my company mentor Ir Jan Font Freide. He has guided me through this process from the beginning to the end. He has shown me where and what I needed to change. He has been supportive throughout the 2 years that I was doing my dissertation, especially during the tough times. Not only that, but he has always been honest with me about my work and I am a better man for it.

I want to thank my friends but especially my family for supporting me. It has been quite a struggle to be where I am right now, I am thankful to have had all of you by my side during this process. Last but not least I would like to give a special thanks to my Opa and Oma where I have been living during my study time. They have taught me valuable life lessons, that stretch way beyond the university. I am eternally grateful to you both!

Content

INTRODUCTION

Late in the 70's Engineer Johan A. Bonink of, Copreal engineering, assisted by, Corsmit Raadgevend Ingenieurs, now RHDHV, developed a system for constructing schools. The CD20 building system was designed as a Lego-Like system which enables for quick construction of buildings against reduced costs. In 1982 this system was awarded with the Building world award.

Since then this system has been used to construct several buildings in the Netherlands. The system as it was originally designed has been modified and adjusted when needed, so integration with other building methods is possible. The company's original idea was to provide a solution for the need for schools in areas where young families lived. When the families would get older and the school would no longer be needed, the school would be deconstructed and re-constructed at a new location.

The system used a Lego principal, so it is easy to assemble. Prefab construction, which means constructing elements of the construction in the factory and then transporting those to the construction site for construction is really being lifted to a new level. The idea of Lego stones is that you have one universal connection that you use to connect them. In the case of Lego it is the top and bottom configuration that is the same on all the elements allowing for just one form of connection being used for the entire building.

This Lego idea has been used to form a universal connection that could be used in real life construction. This connection allows for short build time and links columns and walls to floor panels. This means the skeleton of the construction can be rapidly assembled on site. The work shifts from the construction site to the factory so more preparation time before the actual construction starts, but far less time on the construction site.

The CD20-building system has been designed for the Dutch market, allowing it to be manufactured and tested under Dutch regulations. Earthquakes is a type of loading that the Netherlands normally don't have to anticipate and as such have to calculate with. However the desire to see if the system is applicable in earthquake environments is present. The wish to exploit the system in other parts of the world has risen and thus the need to see how the CD-20 system would perform under earthquake conditions.

The main question of this thesis is:

How can the prefab concrete building system CD20 be used in areas where earthquakes occur?

One specific system will be analyzed in order to answer the main question, the CD20 building system. The question arose from the companies involved with the CD-20 system, as they wanted to know if the system could be used in Oman. Since there are earthquake conditions in Oman, the system needs to be tested in a different way than it has been up to this point. If the system is not able to stand up to the earthquake, this report needs to show how and why it will fail and how to overcome these problems.

The CD-20 building system

The CD-20 building system, like other prefab systems can be compared with Lego. Lego as in the toy for children that allows to connect every element through one simple connection. The pins on top of the stones fit into the bottom of the stones, this allows for a simple and fast connection.

FIGURE 1 LEGO BRICKS

This idea of fast connecting elements exists in prefab concrete constructions. However, there are a number of different connections in regular prefab construction to compile all the elements into a single construction. The CD-20 building system approaches the Lego idea fairly close, as a single connection method is the starting point for the whole method.

The CD-20 building system uses a connection between columns and floor slabs as a primary way of connection. The connection between the column and the floor slab happens by using a specific corner shoe in the floor slab and a pin on the column. The images below give a first understanding of what the connection looks like. The pins on the column and the corner shoe are both made out of steel S235. They are anchored into the concrete of the column and the floor slab with steel anchors.

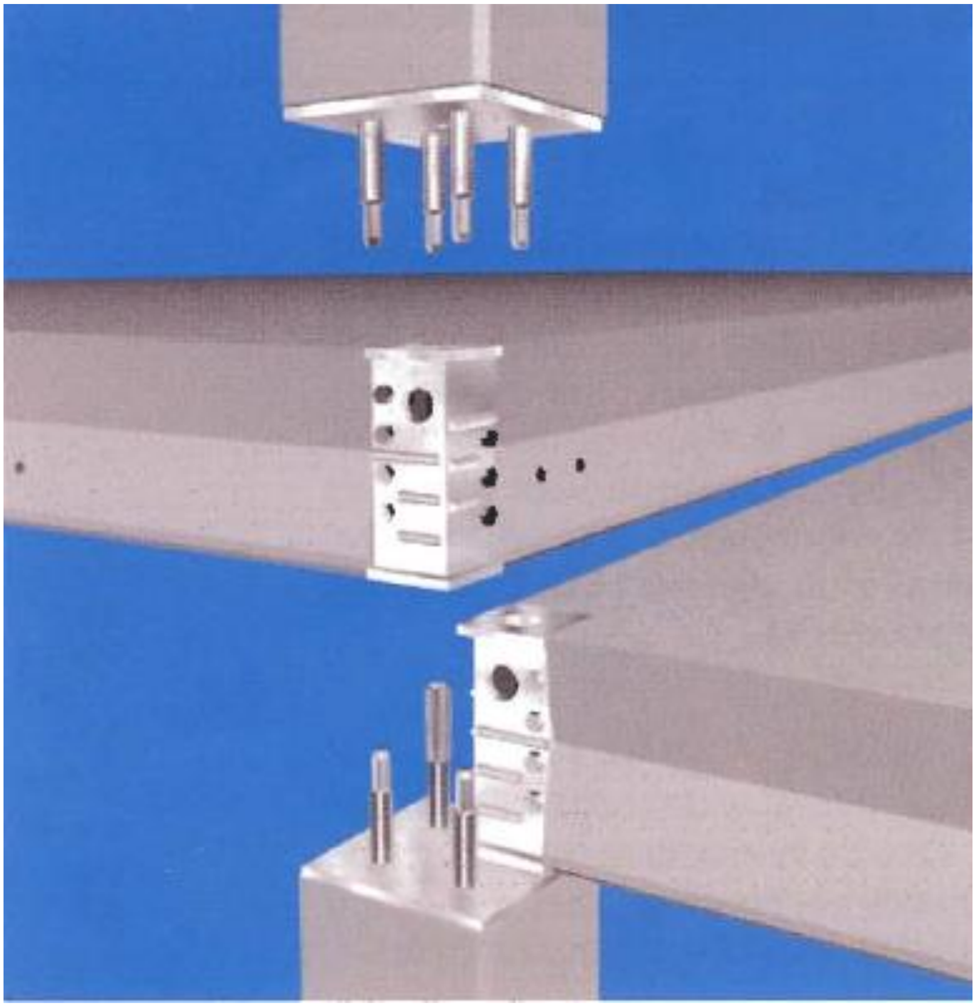


FIGURE 2 3D VIEUW OF THE CD-20 CONNECTION

As can be seen in figure 2, at the end of the column there are four pins. These four pins in turn allow for the connection of four floor slabs. The floor slabs also have four corners and can thus connect with up to four columns. Figure 3 shows a section of the connection between two floor slabs and two columns. There is a shaft per corner shoe where two pins fit into. One pin from the bottom column, and one pin from the top column.

In figure 3 a tube is visible that seemingly connects the two pins to each other. However this is not a connecting mechanism, but is merely there for alignment. This way the column is placed accurately above the one below. Now at this moment the connection can transfer horizontal loads and vertical pressure downwards. However, the connection is quite open and therefore not watertight. In order to solve this concrete is poured into the connection to make it watertight. This concrete is not necessary for the connection however, the connection can transfer the horizontal forces and the vertical forces without the poured in concrete. In order to pour in the concrete, there are canals in the sides of the floor slabs that allow for pouring concrete directly into the connection.

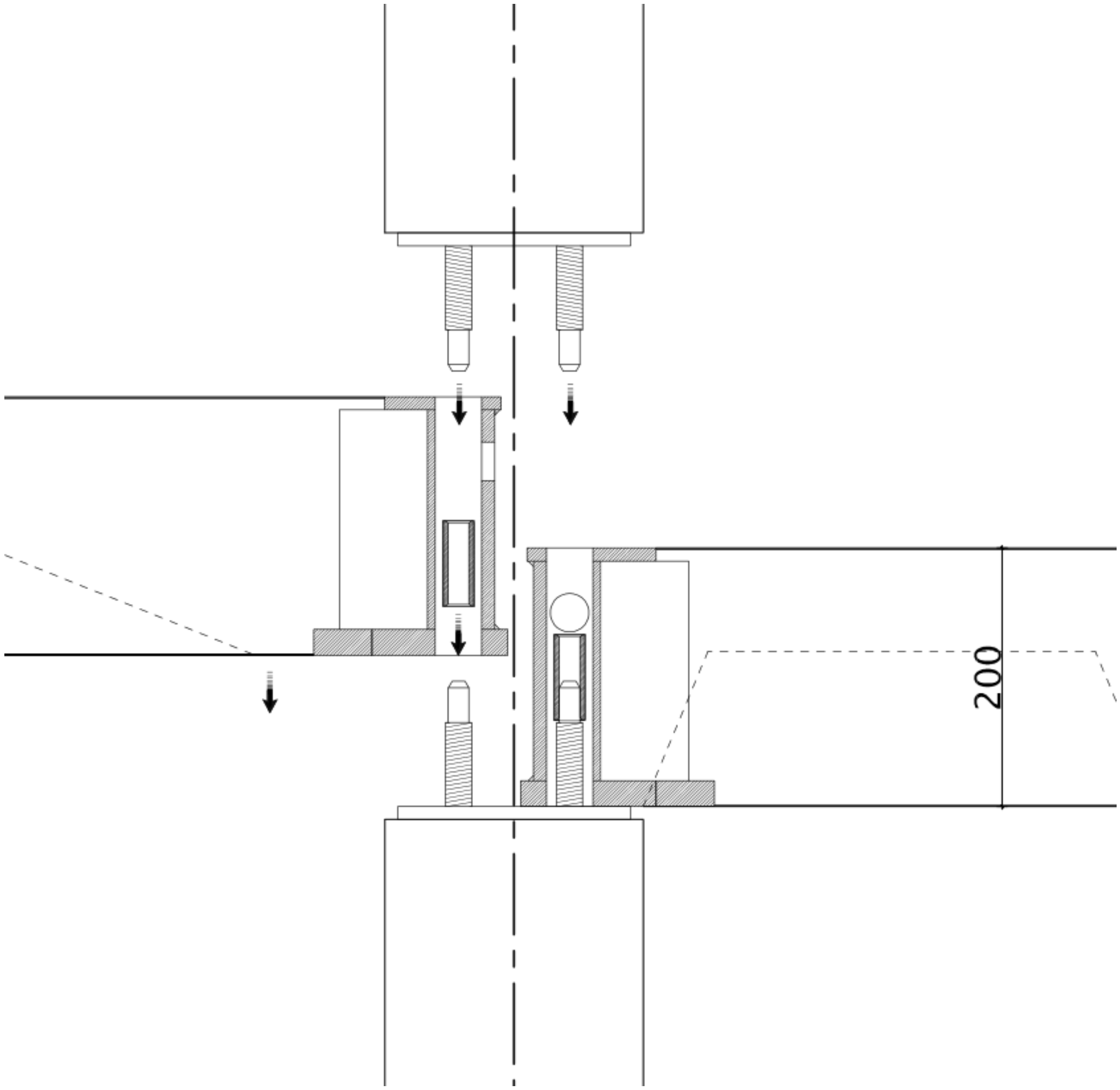


FIGURE 3 SECTION OF THE CONNECTION

The floor is a rib-casset floor element, as can be seen in figure 4. That means that there is a top layer of solid concrete with floor beams underneath. That is where the floor element gets its strength from. These floor beams are prestressed and this is where the strength of the panel comes from. These floor panels are usually finished off with a topping to create an equal floor surface. But they can also be extended with additional constructive toppings, this can further expand the capacity of the floor panels. The standard floor panel is calculated on 4 kN/m^2 of variable loading.

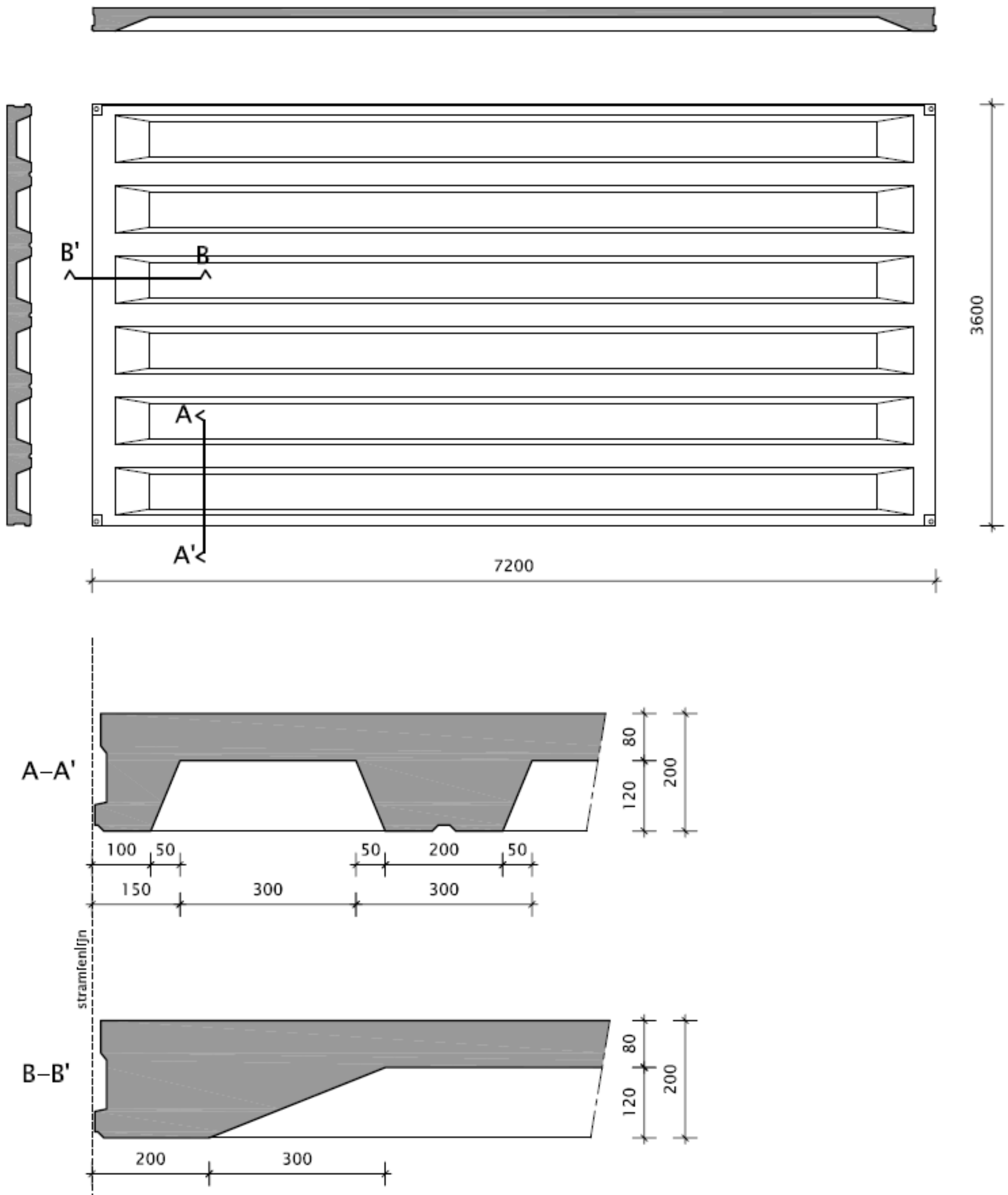


FIGURE 4: FLOOR PLATE

The floor elements are linked together at the connection points, being the points where they intersect with the columns. This interlinking of the floor elements creates the horizontal loadbearing system. The vertical loadbearing system transfers loads from the floor plates via the columns to the foundation. As has been mentioned before the system works when it is in place, filling the connection up with mortar is only needed to provide watertight connections. The system will have to cope with horizontal forces from the wind loading. The direct loading and the tension tie that will activate when the wind loading is active are shown in the picture below.

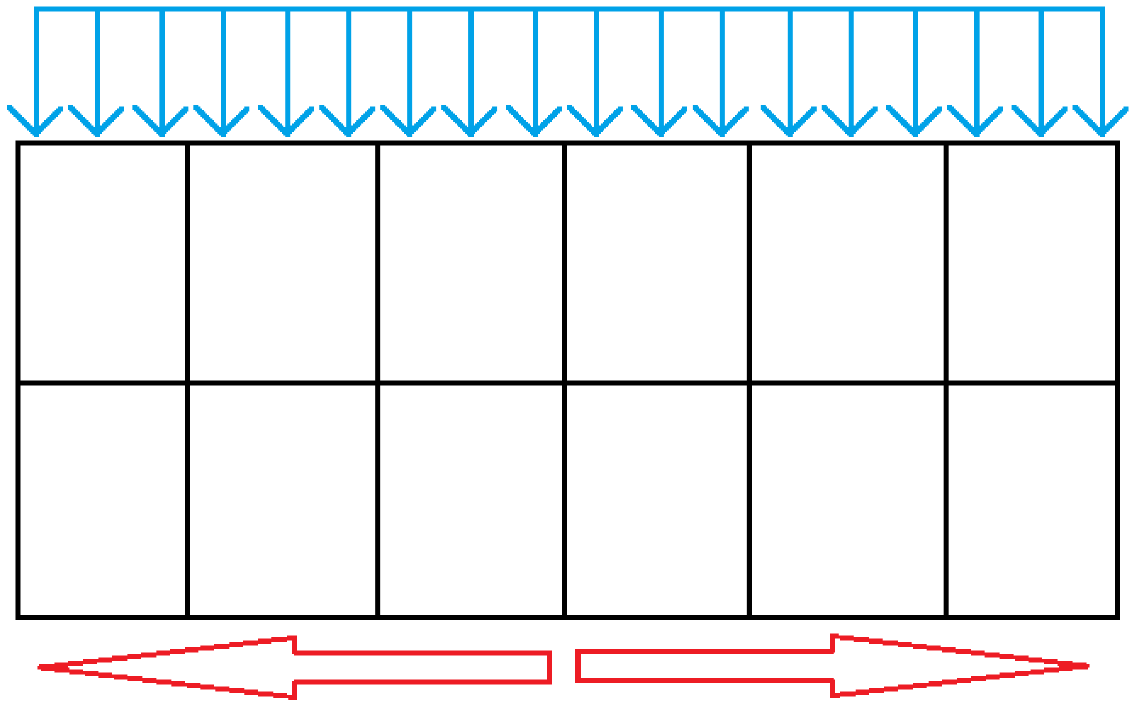


FIGURE 5: HORIZONTAL LOAD ON THE CONNECTION DUE TO WIND LOADING

Because this system has been optimized for a static construction the weight of the elements is rather light. This is an advantage when it comes to earthquake loading. The earthquake shakes the floor and thus the foundation the building is built on. That will cause the building to shake. This means the forces that will be applied on the building are in coherence with the mass of the building, so less mass means smaller forces. The downside is that the elements are optimized so there is not a lot of room to handle additional loading on the structure.

Another important factor for the earthquake is the ductility of the construction. That means the ability to handle the movement that is induced by the earthquake. This will have to be extracted for a large part from the steel connections, seen as that the rest of the building is made out of concrete elements. The question how these factors will interact is very interesting and important for the outcome of this thesis.

1. SETTING THE STAGE

This chapter describes the situation regarding CD20 building systems as it was at the start of the thesis: The CD-20 building system had been in use in the Netherlands for quite some time already. The loading in the Netherlands consists mainly out of self-weight, loading due to building usage and wind load. Under these circumstances the building system performs admirably. A new possible area to exploit this system is Oman and maybe later Saudi-Arabia. In this region earthquakes of a greater magnitude can occur than in the Netherlands, with the exception of the earthquake zone in Groningen. This needs to be accounted for in recalculating the system with this kind of loading.

A few months after this study had started an earthquake occurred in the province of Groningen in the Netherlands. This earthquake had nothing to do with a geographical fault line. In Groningen natural gas can be harvested. Harvesting the gas has made the ground unstable however and this has caused an earthquake to happen. The type of earthquake is different but the engineering world in the Netherlands had to step in to check and advise on adjusting buildings for the public safety. This means that the method they used for these checks can be used in this study most likely.

As has been discussed in the previous chapter, the system mainly exists out of 3 different parts: the column, the floor slab and the unique connection that makes the CD-20 building system what it is. In order to test the system the elements need to be properly assessed as well. Models will have to be setup to examine the behavior of the system. The floor slab and the concrete column should be modeled as concrete elements. The connection is a different story, since in most modeling programs the connection is just a point of interaction between two elements. This point of interaction needs to have the same properties as the connection in the CD-20 building systems has. The connection has to be examined and the result has to be implemented in order to give an accurate analysis of the system.

Given these points, some research needs to be done beforehand. In order to correctly analyze the way the connection functions, research that defines the boundary conditions, like the magnitude of the earthquake, the building code that will be used and details about the CD-20 building system need to be examined. The main question for the entire thesis is:

How can the prefab concrete building system CD20 be used in areas where earthquakes occur?

There are a number of sub questions that are answered throughout this document. In the section below the chapters are presented with the sub questions per chapter.

<u>1 Setting the stage:</u>

- What are the sub questions per chapter?
- How will the research be setup?
- What programs will be used?

<u>2 Literary research:</u>

- What are the boundary conditions, building codes, environmental influences?
- How will the earthquake load the building?
- Possibilities of construction at target location and/or transportation of the system?

<u>3 Connection model in Diana:</u>

- Why is the model built in Diana?
- How does the connection behave regarding stiffness and plasticity?

4 Translation from Diana to Scia Engineer:

- How can the characteristic CD20 connection be modeled in Scia Engineer?
- How to transfer the data from the Diana to the Scia Engineer model?

5 Building model in Scia Engineer:

- Why is the model built in Scia Engineer?
- How is the building loaded by the earthquake?
- Is the capacity of the system sufficient?

6 Conclusions:

- Can the CD20 building system be used in earthquake environment?
- If adaptations are needed what would they need to be?
- Are the adaptations a viable option?

First the situation needs to be examined and the boundary condition need to be set, that means a literature study will need to take place in order to answer the questions like, what building codes need to be used and what earthquake magnitude is appropriate for testing.

Then a model needs to be made in order to test the CD-20 system under the previously found circumstances. However the CD-20 system needs to be modeled correctly and approach the reality as close as possible. That means fist the special CD-20 connection will need to be examined and translated in some kind of way into the model.

The last part will consists out of the tests that have been conducted and what it means for the systems applicability. To see if changes are needed and if so what kind of changes need to be made. This results in the chart in figure 6, to bring this thesis to a proper end.

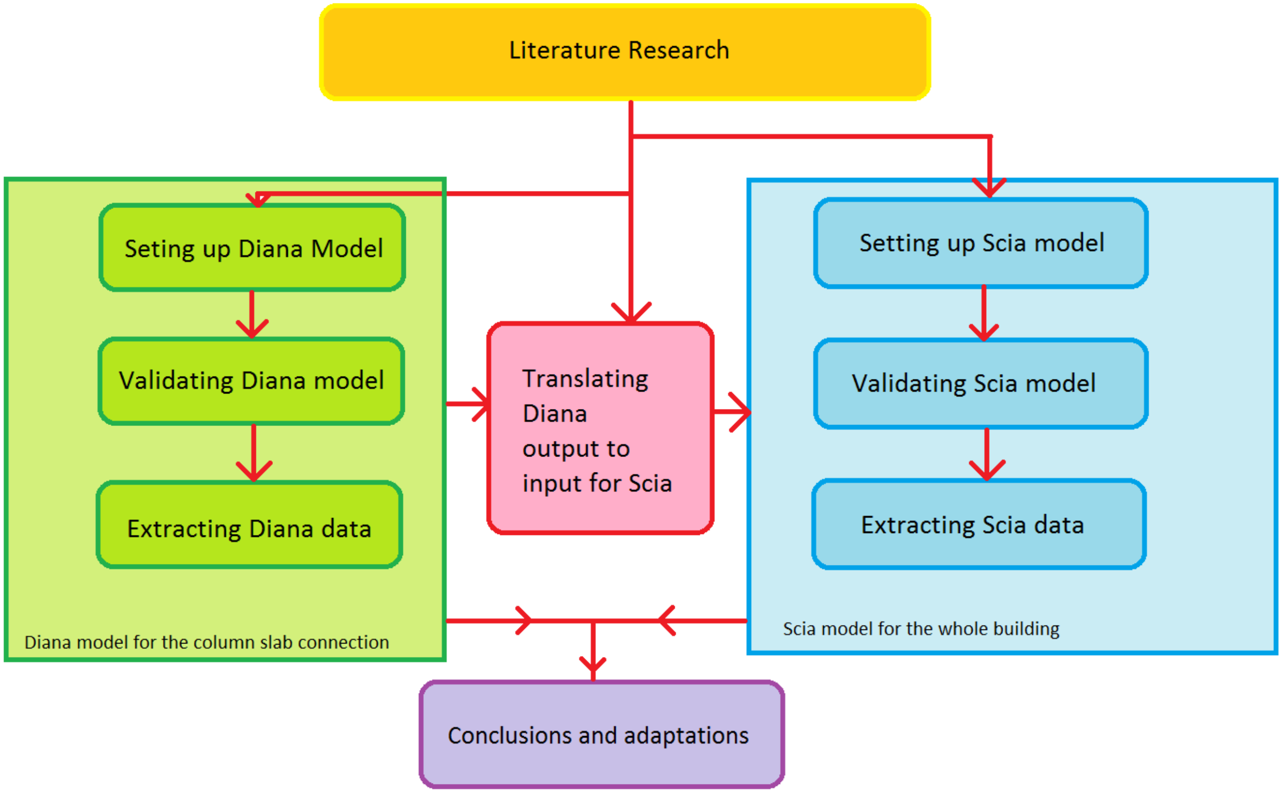


FIGURE 6 SCHEMATICS OF THE RESEARCH SETUP

2. LITERATURE RESEARCH

This chapter will answer questions stated in the previous chapter. The literature research is meant to look up the information that is already available about the subject and thus form the starting point from where this research will take off.

2.1. Location

The targeted location for building is in Oman, a country to the South-East of Saudi-Arabia.

FIGURE 7 LOCATION OF OMAN

FIGURE 8 ZOOMED IN PICTURE OF THE LOCATION

2.2. Building codes.

The application of the building system in other countries depends on the building decree of the country. Is it able to apply the system in the same manner as it has been applied in the Netherlands, or do regulations differ? A study has been carried out as part of another master thesis called: Feasibility of air traffic control towers around the globe (2014), by Joost Hartmann. In this thesis a number of building codes are compared for two different areas. Earthquakes and wind loading are the two subjects that are being compared. For this thesis only the part about the earthquakes is important. The conclusion is that the formulas consist out of different symbols but that they generally all mean the same thing. There are small differences but overall it is clear that the codes dictate the same all over the world. Different countries use the same principles to come to their respective safety codes. The regulations beyond the euro zone are not any stricter than the euro code and that means that every safety regulation that has been taken into account right now, has been sufficient. So the design of the elements does not have to be calculated for wind loading or floor loading with a different factor.

The earthquake loading is a different story, not because the regulations differ so much, but because the system has never been calculated on earthquake loading. So no loading has to be adjusted but the system has to be tested on earthquake loading and the euro code will be used in order to test the structure. The euro code is similar or stricter then the other building codes, so when applying the system worldwide the checks are valid. The Oman seismic design code, including specific design requirements for reinforced concrete, structural steel, composite and masonry buildings, are in compliance with the previous statement.

2.3. The earthquake

In Holland the phenomenon earthquakes is a relatively unknown occurrence. In 2014 Groningen has experienced a serious earthquake and all around the Dutch engineering world research is being done on the earthquake resistance of the buildings currently present. That makes the research on this field a very interesting one. The information in this chapter has been collected from two different sources, the Eurocode and the before mentioned thesis of Joost Hartmann, where he already made a summary of the loading of the Eurocode. The loading caused by the earthquake gives horizontal and vertical acceleration. This acceleration combined with the mass creates forces on the construction. The earthquake loading can be led back to Newton's second law:

$$F = m * a$$

The CD20-building system is a light precast concrete system and it is connected by the standard connections as has been mentioned before. The fact that the system is light has a positive effect on the forces working on the construction due to earthquakes. It is at the connections where the questions arise, it is unsure whether they will be strong enough to withstand earthquake loading.

The euro code is clear on how to approach a concrete structure when loaded by an earthquake. In the following part the calculations necessary to evaluate the connection will be summed up and the calculation method will be explained.

2.3.1. Scope

The scope of the euro code when addressing earthquakes are simple:

- Protect human lives
- Limit damage
- Structures important for civil protection must remain operational

There are two compliance requirements in the code, one concerning ultimate limit state and the other damage limitation state. For the ultimate limit state the following requirements are made:

1. It shall be verified that the structural system has the resistance and energy dissipation capacity specified in the relevant Parts of EN 1998.
2. The resistance and energy dissipation capacity to be assigned to the structure are related to the extent to which its non –linear response is to be exploited. Balance between resistance and energy-dissipation capacity is given by the behavior factor q. The value differs because there is a difference in dissipation between the buildings.
3. The structure as a whole shall be checked to ensure that it is stable under design seismic action. Both overturning and sliding stability shall be taken into account.
4. It shall be verified that both the foundation and the foundation soil need to be able to resist the action effects resulting the response of the superstructure without substantial permanent deformations.
5. Second order influences need te be taken into account.
6. Nonstructural elements can't be allowed to pose a threat to persons and can't have a detrimental effect on the response of the structural elements.

For damage limitation state:

1. An adequate degree of reliability against unacceptable damage shall be ensured by satisfying the deformation limits or other relevant limits defined in the relevant Parts of EN 1998.

2. In structures important for civil protection the structural system needs to be verified to ensure that it has sufficient resistance and stiffness maintain the function of the vital services in the facilities for a seismic event associated with an appropriate return period.

2.3.2. Ground conditions

There are five different ground types in the code, ranging from A to E and two special ground conditions S1 and S2, described by the stratigraphic profiles and parameters. Table 1, shows the corresponding parameters.

TABLE 1: GROUND TYPES

Table 3.1: Ground types

Ground type	Description of stratigraphic profile	Parameters		
		$v_{s,30}$ (m/s)	N_{SPT} (blows/30cm)	c_u (kPa)
A	Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface.	> 800	_	_
B	Deposits of very dense sand, gravel, or very stiff clay, at least several tens of metres in thickness, characterised by a gradual increase of mechanical properties with depth.	360 – 800	> 50	> 250
C	Deep deposits of dense or medium-dense sand, gravel or stiff clay with thickness from several tens to many hundreds of metres.	180 – 360	15 - 50	70 - 250
D	Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil.	< 180	< 15	< 70
E	A soil profile consisting of a surface alluvium layer with v_s values of type C or D and thickness varying between about 5 m and 20 m, underlain by stiffer material with v_s > 800 m/s.			
S_1	Deposits consisting, or containing a layer at least 10 m thick, of soft clays/silts with a high plasticity index (PI > 40) and high water content	< 100 (indicative)	_	10 - 20
S_2	Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types A – E or S_1			

2.3.3. Seismic action

Seismic action is subdivided by national authorities into seismic zones, depending on the local hazard. The hazard is described as peak ground acceleration agR, expressed in gravitational acceleration, unit [g], based on ground type A. This parameter corresponds with Tncr or Pncr, and an importance factor γI value 1.0 is taken in this consideration.

$$a_g = a_{gR} * \gamma_I$$

In case of low seismicity, reduced or simplified seismic design procedures for certain types or categories of structures may be used. This is allowed when agR is not bigger than 0.78 [m/s^2] or 0.08[g] where 1.0[g] is 9.81 [m/s^2]. When seismicity is 0.1[g] or lower it does not need to be observed. In figure 9 a seismic hazard map of Europe is shown, the values are expressed in [m/s^2].

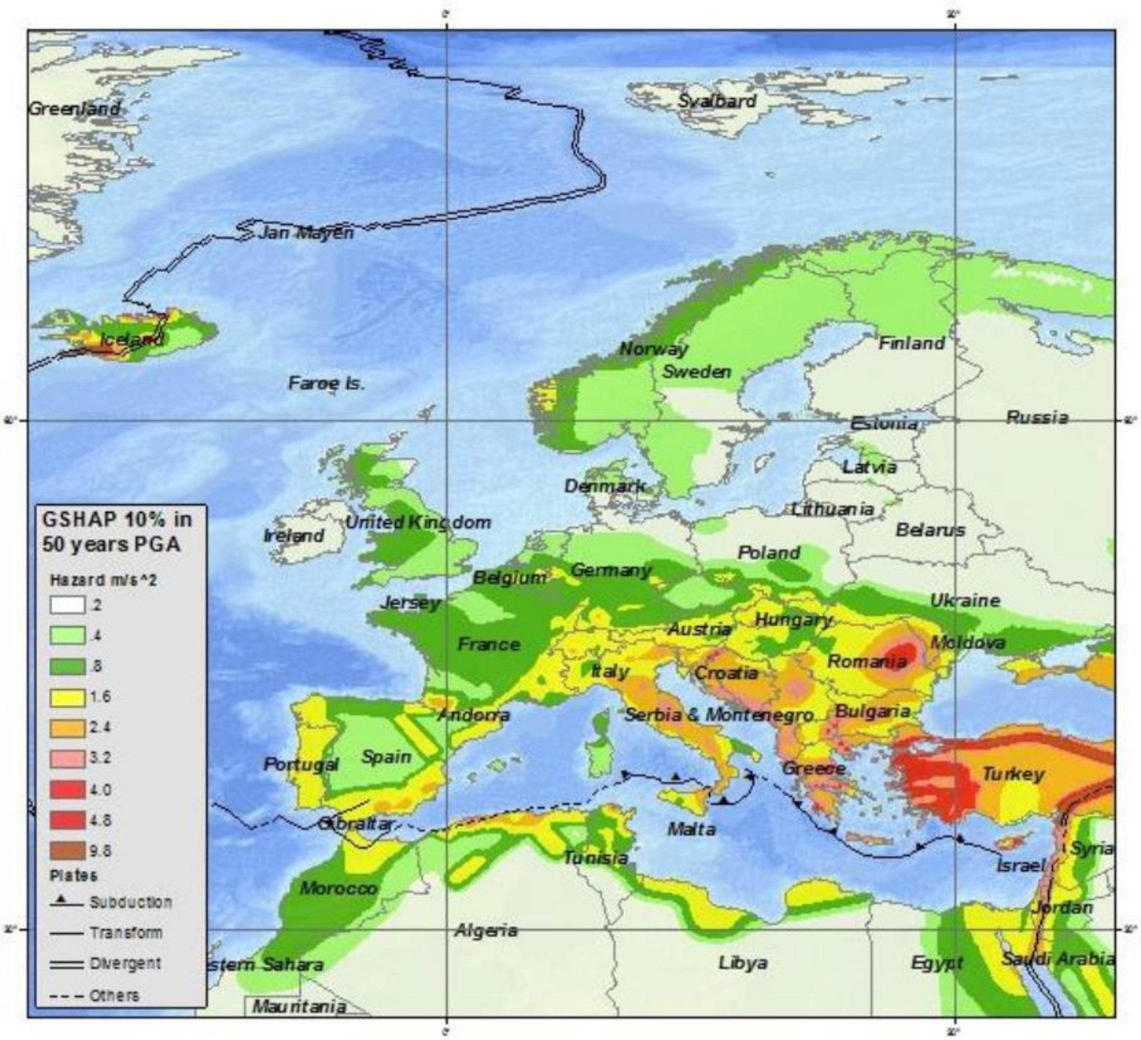


FIGURE 9: SEISMIC HAZARD MAP

At this point the peak ground acceleration is known and the earthquake motion at the given point on the surface can be calculated. This is presented by the ground acceleration response spectrum, which is called the elastic response spectrum. There are several aspects of the spectra; horizontal elastic, vertical elastic, horizontal design and vertical design.

Horizontal elastic response spectrum Se(T) (linear)

The horizontal elastic response spectrum can be defined by the following formulas:

$$0 \leq T \leq Tb: Se(T) = ag * S * \left[1 + \frac{T}{Tb} * (\eta * 2.5 - 1)\right]$$
$$Tb \leq T \leq Tc: Se(T) = ag * S * \eta * 2.5$$
$$Tc \leq T \leq Td: Se(T) = ag * S * \eta * \left[\frac{Tc}{T}\right]$$
$$Td \leq T \leq 4s: Se(T) = ag * S * \eta * \left[\frac{Tc * Td}{T^2}\right]$$
$$ag = \gamma I * agR$$
$$\eta = \sqrt{\left(\frac{10}{5 + \xi}\right)} \geq 0.55$$

Where

Se(T)	is the elastic response spectrum;
T	is the vibration period of a linear single degree-of-freedom system;
a_g	is the design ground acceleration on the type A ground $\left(a_g = \gamma 1 * a_{gR}\right)$;
Tb	is the lower limit of the period of the constant spectral acceleration branch;
Tc	is the upper limit of the period of the constant spectral acceleration branch;
Td	is the value defining the beginning of the constant displacement response range of the spectrum;
S	is the soil factor
η	is the damping correctioin factor with a reference value of η =1 for 5% viscous damping;
ξ	Is the viscous damping ration of the structure, recommended value of ξ 0.05

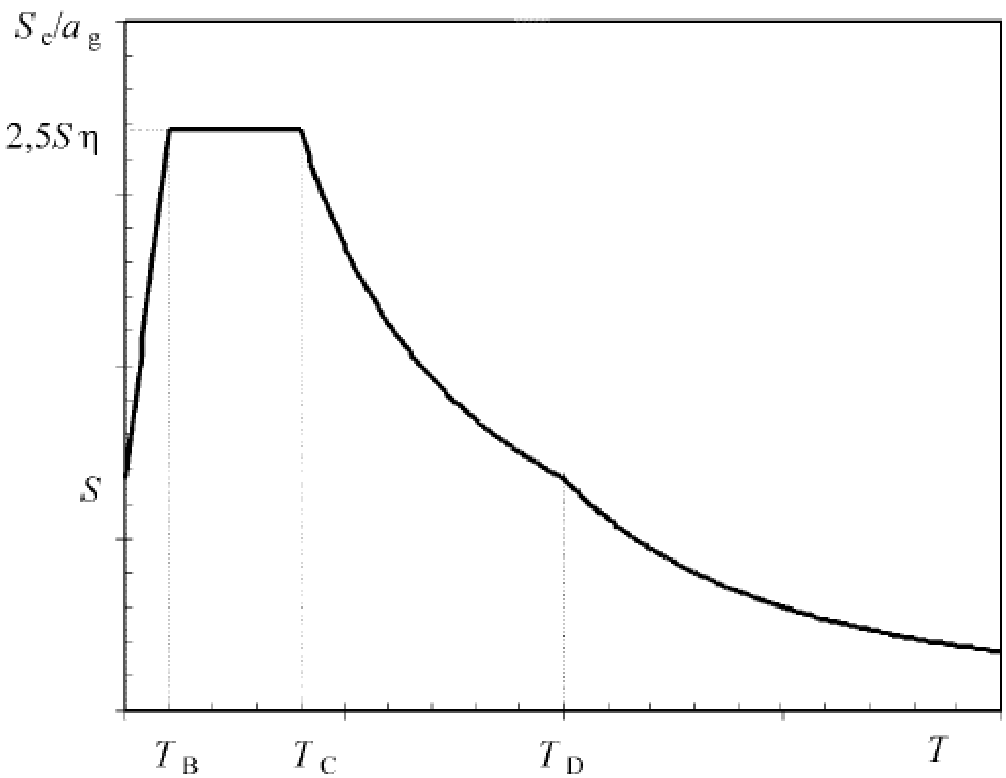


FIGURE 10: SHAPE OF THE ELASTIC RESPONSE SPECTRUM

If extended geological data is not accounted for, the recommended choice is the use of two types of spectra: Type 1 and Type 2. If the earthquakes that contribute most to the seismic hazard defined for the site for the purpose of probabilistic hazard assessment have a surface-wave magnitude, Ms lower than 5,5 it is recommended that the Type 2 (table 3) spectrum is adopted, otherwise use Type 1 (table 2).

Ground type	S	T_B (s)	T_C (s)	T_D (s)
A	1,0	0,15	0,4	2,0
B	1,2	0,15	0,5	2,0
C	1,15	0,20	0,6	2,0
D	1,35	0,20	0,8	2,0
E	1,4	0,15	0,5	2,0

TABLE 2: VALUE OF PARAMETER TYPE 1 [NEN-EN 1998-1:2004, TABLE 3.2]

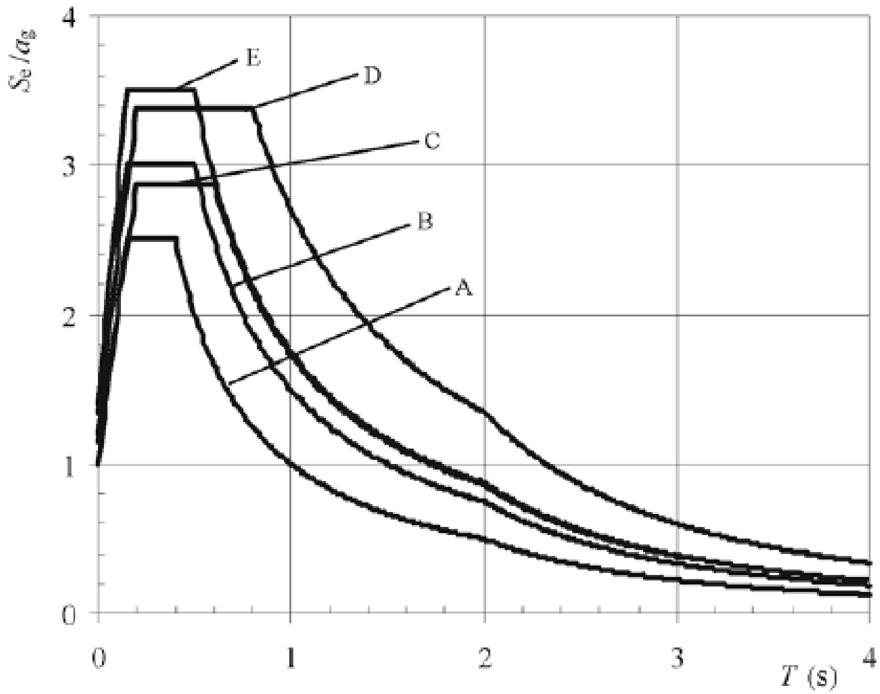


FIGURE 11: RECOMMENDED TYPE 1 ELASTIC RESPONSE SEPCTRA FOR GROUND TYPES A TO E (5% DAMPING)

Ground type	S	T_B (s)	T_C (s)	T_D (s)
A	1,0	0,05	0,25	1,2
B	1,35	0,05	0,25	1,2
C	1,5	0,10	0,25	1,2
D	1,8	0,10	0,30	1,2
E	1,6	0,05	0,25	1,2

TABLE 3: VALUE OF PARAMETER TYPE 2 [NEN-EN 1992-1:2005, TABLE 3.3]

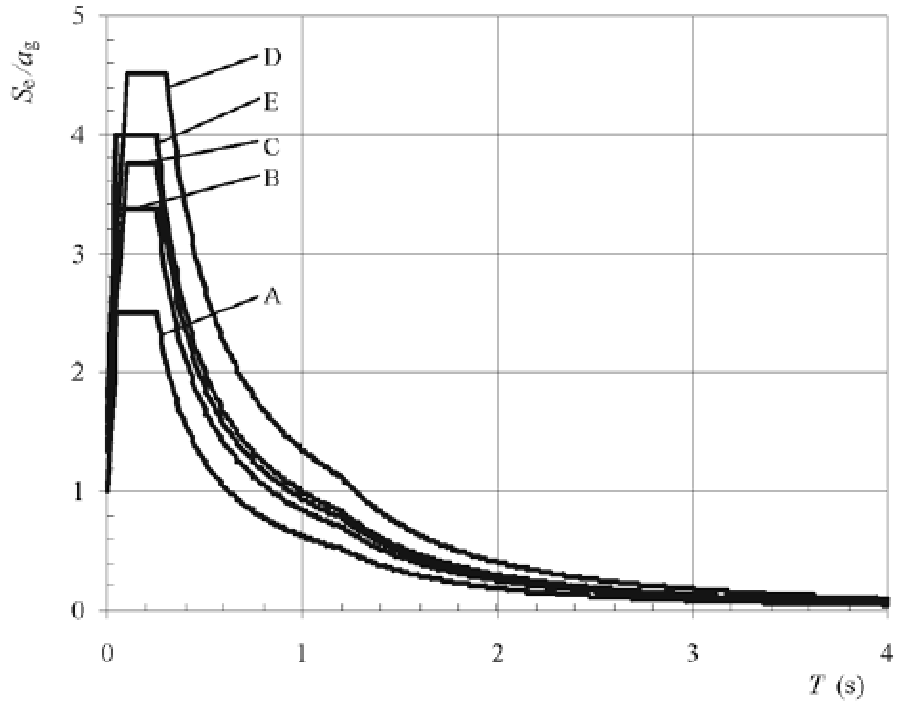


FIGURE 12: RECOMMENDED TYPE 2 ELASTIC RESPONSE SPECTRA FOR GROUND TYPES A TO E (5% DAMPING)

- dampening correction factor η may be determined by the expression: $\eta = \sqrt{\frac{10}{5+\xi}} \geq 0.55$
- Sde(T) shall be obtained by direct transformation of the elastic acceleration reponse spectrum, Se(T) using the expression: $S_{De}(T) = S_e(T)\left[\frac{T}{2\pi}\right]^2$

<u>Vertical elastic response spectrum</u>

The vertical elastic response spectrum can be defined by the following formulas:

$$0 \leq T \leq T_b : S_{ve}(T) = a_{vg} * \left[1 + \frac{T}{T_b} * (\eta * 3.0 - 1)\right]$$
$$T_b \leq T \leq Tc : S_{ve}(T) = a_{vg} * \eta * 3.0$$
$$T_c \leq T \leq T_d : S_{ve}(T) = a_{vg} * \eta * 3.0 \left[\frac{Tc}{T}\right]$$
$$T_d \leq T \leq 4s : S_{ve}(T) = a_{vg} * \eta * 3.0 \left[\frac{Tc * Td}{T^2}\right]$$

What applies to the horizontal elastic response spectrum, applies to the vertical elastic response spectrum : when Ms is lower than 5.5 adopt spectrum type 2, otherwise adopt spectrum type 1. Table 4 gives the recommended values of the parameters describing the vertical spectra, these do not apply for special ground types S1 and S2.

Spectrum	a_{vg}/a_g	T_B (s)	T_C (s)	T_D (s)
Type 1	0,90	0,05	0,15	1,0
Type 2	0,45	0,05	0,15	1,0

TABLE 4: RECOMMENDED VALUES OF PARAMETERS DESCRIBING THE VERTICAL RESPONSE SPECTRA

Design spectrum for elastic analysis

For the horizontal components of the seismic action the design spectrum, $S_d(T)$, shall be defined by the following expressions:

$$0 \leq T \leq T_b: S_d(T) = a_g * S * \left[\frac{2}{3} + \frac{T}{T_b} * \left(\frac{2.5}{q} - \frac{2}{3}\right)\right]$$

$$T_b \leq T \leq Tc: S_d(T) = a_g * S * \frac{2.5}{q}$$

$$T_c \leq T \leq T_d: S_d(T) \begin{cases} = a_g * S * \frac{2.5}{q} * \left[\frac{Tc}{T}\right] \\ \geq \beta * a_g \end{cases}$$

$$T_d \leq T \leq 4s: S_d(T) \begin{cases} = a_g * S * \frac{2.5}{q} * \left[\frac{TcTd}{T^2}\right] \\ \geq \beta * a_g \end{cases}$$

where

a_g, S, T and T_d	are as defined in 3.2.2.2;
$S_d(T)$	is the design spectrum;
q	is the behaviour factor;
β	is the lower bound factor for the horizontal design spectrum.

For the vertical component of the seismic action the design spectrum is given by expressions (3.13) to (3.16), with the design ground acceleration in the vertical direction, a_{gv} replacing a_g, S taken as being equal to 1.0 and the other parameters as defined in 3.2.2.3. For the vertical component q should generally be adopted for all materials and structural systems. Values greater than 1.5 should be justified through an appropriate analysis.

2.3.4. Design of buildings

This paragraph is about what loading to calculate on the building when calculating with earthquakes.

Combinations of loading for the seismic action with other actions

$$\sum G_{k,j} " + " \sum \Psi_{E,i} * Q_{k,i}$$

Where $\Psi_{E,i}$ is the combination coefficient for variable action i.
The combination coefficients Ψ_{2i} (for the quasi-permanent value of variable action q) for the design of buildings

$$\Psi_{E,i} = \varphi * \Psi_{2i}$$

Type of variable action	Storey	φ
Categories A-C*	Roof Storeys with correlated occupancies Independently occupied storeys	1,0 0,8 0,5
Categories D-F* and Archives		1,0

* Categories as defined in EN 1991-1-1:2002.

TABLE 5: VALUES OF φ FOR CALCULATING $\Psi_{E,i}$

Importance classes

Four importance classes exist, each having their own value for γI for importance classes I, II, III and IV are equal to 0.8, 1.0, 1.2 and 1.4, where the value of class II is equal to 1.0 by default. The description of each importance class is given in table 6. The importance factor $\gamma I = 1.0$ is associated with a seismic event having the reference exceeding period of once every 50 years.

Importance class	Buildings
I	Buildings of minor importance for public safety, e.g. agricultural buildings, etc.
II	Ordinary buildings, not belonging in the other categories.
III	Buildings whose seismic resistance is of importance in view of the consequences associated with a collapse, e.g. schools, assembly halls, cultural institutions etc.
IV	Buildings whose integrity during earthquakes is of vital importance for civil protection, e.g. hospitals, fire stations, power plants, etc.

TABLE 6: IMPORTANCE CLASSES

Different methods of analysis:

There are different methods that can be used to calculate the effect of an earthquake on a building. The lateral force analysis, the response spectrum analysis and then there are nonlinear analysis. Chapters 2.3.5 through 2.3.7 will describe the different methods of analysis.

2.3.5. Lateral force method of analysis

Lateral force method of analysis may be applied to buildings whose response is not significantly affected by contributions from modes of vibration higher than the fundamental mode in each principal direction. The fundamental periods of vibration T1 in the two main directions which are smaller than the following values:

$$T1 \leq \begin{cases} 4 * T_c \\ 2.0s \end{cases}$$

Base shear force

The seismic base shear force F_b for each horizontal direction in which the building is analyzed, shall be determined using the following expression:

$$F_b = S_d(T_1) * m * \lambda$$

Where

$S_d(T_1)$ is the ordinate of the design spectrum at period T_1;

T_1 is the fundamental period of vibration of the building for lateral motion in the direction considered;

m is the total mass of the building, above the foundation or above the top of a rigid basement;

λ is the correction factor, the value of which is equal to $\lambda = 0.85$ if $T_1 \leq 2T_c$ and the building has more than two stories , or $\lambda = 1.0$ otherwise.

For determining the fundamental period of vibration period T_1 of the building, expressions based on methods of structural dynamics may be used, for example the Rayleigh method. For buildings with heights of up to 40 m the value of T_1 (in s) may be approximated by the following expression:

$$T_1 = C_t * H^{3/4}$$

Where

C_t is 0.085 for moment resistant space steel frames, 0.075 for moment resistant space concrete frames and for eccentrically braced steel frames and 0.050 for all other structures;

H is the height of the building, in m, from the foundation or from the top of a rigid basement;

Alternatively, for structures with concrete or masonry shear walls the value C_t in the previous expression may be taken as being:

$$C_t = 0.075/\sqrt{A_c}$$

where

$$A_c = \sum \left[A_i * \left(0.2 + \left(\frac{l_{wi}}{H} \right) \right)^2 \right]$$

and

A_c is the total effective area of the shear walls in the first story of the building in m^2;

A_i is the effective cross-sectional area of shear wall I in the direction considered in the first story of the building, in m^2;

H is the height of the building, in m, from the foundation or from the top of a rigid basement;

l_{wi} is the length of the shear wall I in the first story in the direction parallel to the applied forces, in m, with the restriction that l_{wi}/H should not exceed 0.9

Alternatively the estimation of T_1 (in s) may be made by using the following expression:

$$T_1 = 2 * \sqrt{d}$$

where

d is the lateral elastic displacement of the top of the building, in m, due to the gravity loads applied in the horizontal direction.

<u>Distribution of the horizontal seismic forces</u>

The seismic action effect shall be determined by applying the two planar models horizontal forces F_i to all stories.

$$F_i = F_b * \frac{S_i * m_i}{\sum S_j * m_j}$$

Where

F_i is the horizontal force on story i;

F_b is the seismic base shear;

S_i, S_j are the displacements of masses m_i, m_j in the fundamental mode shape;

When the fundamental mode shape is approximated by horizontal displacements increasing linearly along he height the horizontal forces F_i should be taken as being given by:

$$F_i = F_b * \frac{z_i * m_i}{\sum z_j * m_j}$$

where

z_i, z_j are the heigts of the masses m, m_jabove the level of application of the seismic action (foundation or top of a rigid basement).

<u>Torsional effects</u>

The torsional effect may be accounted for by multiplying the action effects in the individual load resisting elements by a factor δ .

$$\delta = 1 + 0.6 * \frac{x}{L_e}$$

where

x is the distance of the element under consideration from the center of mass of the building in plan measured perpendicularly to the direction of the seismic action considered.

L_e is the distance between the two outermost lateral load resisting elements, measured perpendicularly to the direction of the seismic action considered.

2.3.6. Modal response spectrum analysis

When the lateral force method of analysis cannot be used, response spectrum analysis can be used. It takes the response of all modes of vibration, contributing significantly to the global response into account. torsional effects are also taken into account. This analysis can be performed using finite element programs, such as "SCIA engineer".

2.3.7. Non-linear methods

A pushover analysis is a non-linear static analysis carried out under conditions of constant gravity load and monotonically increasing horizontal loads. It may be applied to verify the structural performance of newly designed and existing buildings. The non-linear methods are complex and require finite element programs to solve.

The method chosen for this thesis is the Modal response spectrum analysis. This is the preferred method of testing by the engineering firms at this moment. The test will have to be performed by using SCIA engineer.

2.3.8. Safety verifications

This chapter lists the safety verifications for calculating with earthquakes.

<u>Ultimate limit state</u>

The no-collapse requirement (ultimate limit state) under the seismic design situation is considered to have been met if the following conditions regarding resistance, ductility, equilibrium, foundation, stability and seismic joints are met:

Resistance condition

$E_d \leq R_d$

This condition needs to be met for all structural elements including connections and the relevant non-structural elements, where

E_d is the design value of the action effect due to the seismic design situation.

R_d is the corresponding design resistance of the element, calculated in accordance with the rules specific to the material used and in accordance with the mechanical models which relate to the specific type of structural system.

Second order effects need to be taken into account if the following condition is fulfilled in all stories:

$$\theta = \frac{P_{tot} * d_r}{V_{tot} * h} \leq 0.10$$

where

θ is the interstory drift sensitivity coefficient;

P_{tot} is the total gravity load at and above the story considered in the seismic design situation;

d_r is the design interstory drift, evaluated as the difference of the average lateral displacements d_s at the top and bottom of the storie under consideration;

V_{tot} is the total seismic storey shear;

h is the interstory height

If $0.1 \leq \theta \leq 0.2$ the second-order effects may approximately be taken into account by multiplying the relevant seismic action effects by a factor equal to $\frac{1}{1-\theta}$, the value of θ shall not exeed 0.3

Global and local ductility condition

The structural elements and the structure as a whole need to possess adequate ductility, taken into account the expected exploitation of ductility depending on the selected system and the behavior factor.

$$\sum M_{Rc} \geq \sum M_{Rb}$$

where

$\sum M_{Rc}$ is the sum of the design values of the moments of resistance of the columns framing the joint. The minimum value of column moments of resistance within the range of column axial forces produced by the seismic design situation should be used in the above formula.

$\sum M_{Rb}$ is the sum of the design values of the moments of resistance of the beams framing the joint. When partial strength connections are used, the moments of resistance of these connections are taken into account in the calculation of $\sum M_{Rb}$.

Resistance of foundation

For the foundation of individual vertical elements (walls or columns), P of this subclause is considered to be satisfied if the design values of the action effects E_{Fd} on the foundations are derived as follows:

$$E_{Fd} = D_{F,G} + \gamma_{Rd} \Omega E_{F,E}$$

where

γ_{Rd} is the overstrength factor, taken as being equal to 1.0 for $q \leq 3$, or as being equal to 1.2 otherwise;

$E_{F,G}$ is the action effect due to the non-seismic actions included in the combination of actions for the seismic design situation (see EN 1990:2002, 6.4.3.4);

$E_{F,E}$ is the action effect from the analysis of the design seismic action;

Ω is the value of $\frac{R_{di}}{E_{di}} \leq q$ of the dissipative zone or element I of the structure which has the highest influence on the effect E_F under consideration;

R_{di} is the design resistance or the zone or element I;

E_{di} is the design value of the action effect on the zone or element I in the seismic design situation

The seismic joint condition states that a building must be protected from earthquake-induced pounding from adjacent structures or between structurally independent units of the same building. The damage limitation requirement is considered to have been satisfied, if under a seismic action having a larger probability of occurrence than the design seismic action corresponding to the “no-collapse requirement” and the interstorey drifts are limited.

For buildings that have non-structural elements of brittle materials attached to the structure:

$$d_r \nu \leq 0.005h$$

For buildings having ductile non-structural elements:

$$d_r \nu \leq 0.0075h$$

For buildings having non-structural elements fixed in a way so as not to interfere with structural deformations, or without non-structural elements:

$$d_r \nu \leq 0.010h$$

where

d_r is the design interstory drift

h is the story height

v is the reduction factor which takes into account the lower return period of the seismic action associated with the damage limitation requirement

2.3.9. Concrete buildings

the upper limit value of the behavior factor q for concrete structures is given with the following equations:

$$q = q_0 k_w \geq 1.5$$

Where

q_0 is the basic value of the behavior factor (table 7)

k_w is the factor reflecting the prevailing failure mode in structural systems with walls, value 1.0

STRUCTURAL TYPE	DCM	DCH
Frame system, dual system, coupled wall system	$3{,}0\alpha_u/\alpha_1$	$4{,}5\alpha_u/\alpha_1$
Uncoupled wall system	3,0	$4{,}0\alpha_u/\alpha_1$
Torsionally flexible system	2,0	3,0
Inverted pendulum system	1,5	2,0

TABLE 7: BASIC BEHAVIOR FACTORS Q0 CONCRETE STRUCTURES [NEN-EN 1998-1:2004, TABLE 5.1]

Where DCM stands for ductility class medium and DHM means ductility class high. For buildings with an unregular elevation, the value of q_0 should be reduced by 20%. The multiplication factor α_u/α_1 depending on the structural sytem is given in table 8.

Frames or frame-equivalent dual systems	α_u/α_1
One-storey building	1,1
Multi-storey, one bay frames	1,2
Multi-storey, multi-bay frames or dual structures	1,3
Uncoupled wall systems	α_u/α_1
Wall systems with only two uncoupled walls	1,0
Other uncoupled wall systems	1,1
Wall-equivalent dual, or coupled wall systems	1,2

TABLE 8: MULTIPLICATION FACTOR α_u/α_1 [NEN-EN 1998-1:2004, SECTION 5.2.2.2]

For precast concrete structures the same behavior may be used when in general provisions are taken regarding the connections of the precast elements. These provisions are connections located away from critical regions, overdesigned connections and energy dissipation connections.

2.3.10. Composite Steel-Concrete structures

The upper limit value of the behavior factor q for composite constructions is given in table 9 and the structural types correspond with those of table 10. The value α_u/α_1 is taken at 1.1.

STRUCTURAL TYPE	Ductility Class	
	DCM	DCH
a), b), c) and d)	See Table 6.2	
e) Composite structural systems Composite walls (Type 1 and Type 2)	$3\alpha_u/\alpha_1$	$4\alpha_u/\alpha_1$
Composite or concrete walls coupled by steel or composite beams (Type 3)	$3\alpha_u/\alpha_1$	$4,5\alpha_u/\alpha_1$
f) Composite steel plate shear walls	$3\alpha_u/\alpha_1$	$4\alpha_u/\alpha_1$

TABLE 9: BASIC BEHAVIOR FACTORS Q COMPOSITE STRUCTURES [NEN-EN 1998-1:2004, TABLE 7.2]

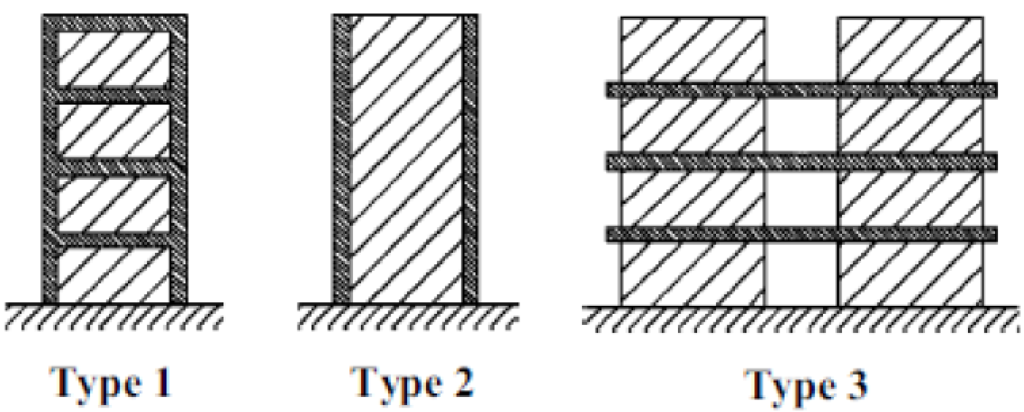


TABLE 10: TYPES OF COMPOSITE STRUCTURES [NEN-EN 1998-1:2004, SECTION 7.3.1]

2.4. Production

The subject of production and transportations will be briefly touched upon since it is not the focus of this thesis, but was included in the literature research in order to better understand the CD20 system. It has been a shallow research of the needs to produce the system and how the productions takes place. Also the possibility of producing the system around the world is shortly reviewed.

The elements that are used in the building system need to be produced first. This is done in two different factories in Germany. The elements are checked after production after which they are transported to the building site. At the building site the elements need to be hoisted into place and mounted onto the structure. This means that the production of the project consists out of four different aspects:

- Material costs
- Production of the elements
- Transport of the elements
- Assembly of the elements

If we take a look at these different aspects, it should give a clear view as to how the production of the elements is going to influence the use of the CD20-buildingsystem in earthquake regions.

2.4.1. Material costs

The CD20-buildingsystem consists out of two primary materials: concrete and steel. These materials are being used around the globe these days. There is one problem however to this scenario, not every location on earth has as the same accessibility to these recourses. It depends on the solution to whether or not that is a problem or not. The prefab system consists out of prefabricated elements that need to be mounted on site, which means that the elements can also be transported to the building site. However transporting it further than needed is a waste of money and resources, making it undesirable. The figures below present an overview of where Holcim and HeidelbergCement , worldwide companies, produces Cement and Concrete:

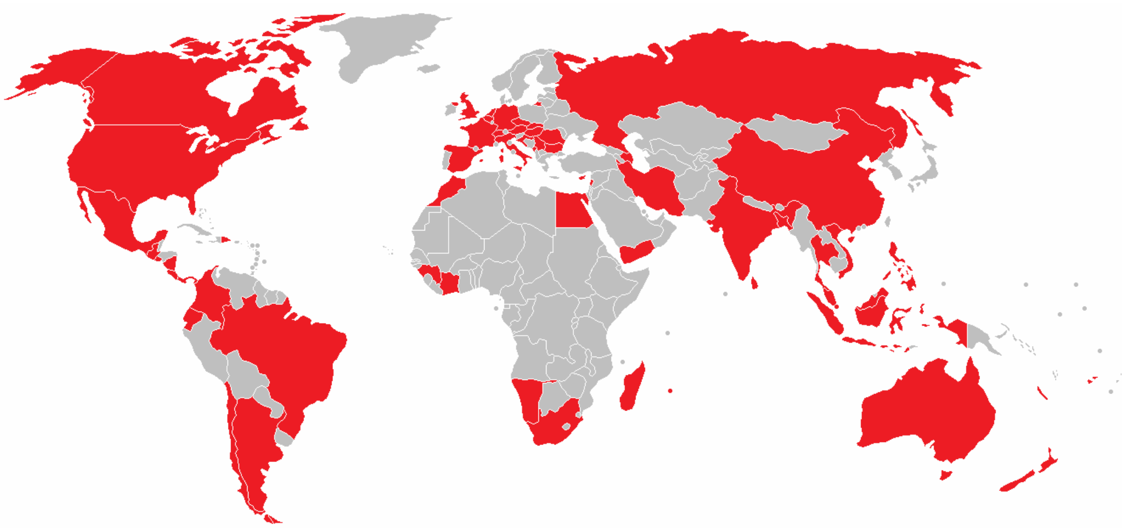


FIGURE 13: FACTORIES OF HOLCIM AROUND THE GLOBE, FROM WIKIPEDIA, ARTICLE NAMED HOLCIM.

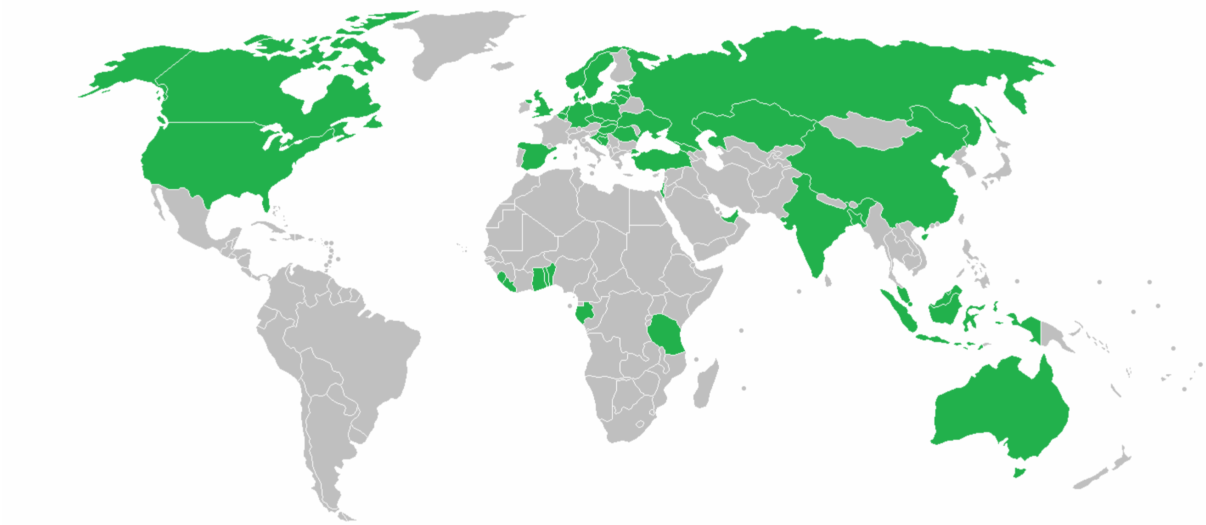


FIGURE 14: FACTORIES OF HEIDELBERGCEMENT WORLDWIDE, SOURCE: WIKIPEDIA, ARTICLE HEIDELBERGCEMENT

These two large production companies make cement all over the world and with the added "local" producers it is safe to say that the production of the precast concrete can be done all over the world. There is however one exception, the inland of Africa, where no such production facilities are found. This is easily explained by the fact that the countries here are far less inhabited and some parts are inhabitable. A large part of central Africa is dessert and a large part is jungle, which are not the most desirable building locations. The countries at the shore have more inhabitants and have factories that produce cement.

The steel parts can be transported to the building location if they are not able to manufacture them at the location of the concrete factory. Steel is widely used over the world in all kinds of building projects. Steel also plays a large part in the precast concrete industry, the precast concrete elements need rebar in the elements.

The different molds would need to be transported to the factories where the production has to take place. If this would be a single project, depending on the size of the project, it could be cheaper and faster to make the elements in Germany, where they are being produced now. If you are aiming for the future, to construct multiple buildings using the CD20-buildingsystem, than it would be useful to construct a factory at the location. So depending on the scope either transporting the building elements or the building process will be more suitable.

2.4.2. Production of the elements

Currently the elements of CD20-buildingsystems are being produced in two different factories in Germany. The wall and façade elements are being constructed at Fuchs Beton in Gladbach. At this facility a carrousel is present allowing Fuchs to build at a continuous rate. The carrousel consists out of a large number of motorized wheels and carts, transporting tables around the factory. On these tables are the molds that need to be filled with reinforcement and concrete. This carrousel has a capacity of 50 tables a day. In the figures below a part of this carrousel and the pouring process can be seen.

FIGURE 15: THE ROLLERS AND CARTS THAT MOVE THE TABLES AROUND

FIGURE 16: A MOLD ON ONE OF THE TABLES, READY FOR THE CONCRETE POURING

FIGURE 17: POURING OF THE CONCRETE INTO THE MOLD

This method is fast and clean and allows for a large production capacity. As can be seen on the pictures, the surroundings of the carrousel are very clean, allowing for a very safe and manageable workplace. The factory has a large field outside the halls where they can stack the elements in order to let them reach the required strength level.

The other factory where CD20-buildingsystems gets its products from is called Rekers-beton from Spelle, Germany. At first all of the products for CD20-buildingsystems were being produced here, but as has been told in the previous part, the walls and facades are now produced at Fuchs beton. The factory of Rekers-beton is a traditional factory, where everything is done stationary. That means a mold is made and concrete is poured into it and it needs to stay in that position until it is strong enough to be unmolded and transported outside. The factory is huge and employees use bikes to transport themselves around the premises. Rekers-beton produces the floors and columns for the current project of CD20-buildingsystems. The figures below give an impression of the vast scale of the facility and gives a look at the production of the floor slabs.

FIGURE 18: PRETENSIONING TABLE

FIGURE 19: A FLOOR SLAB IN THE PRETENSIONING TABLE

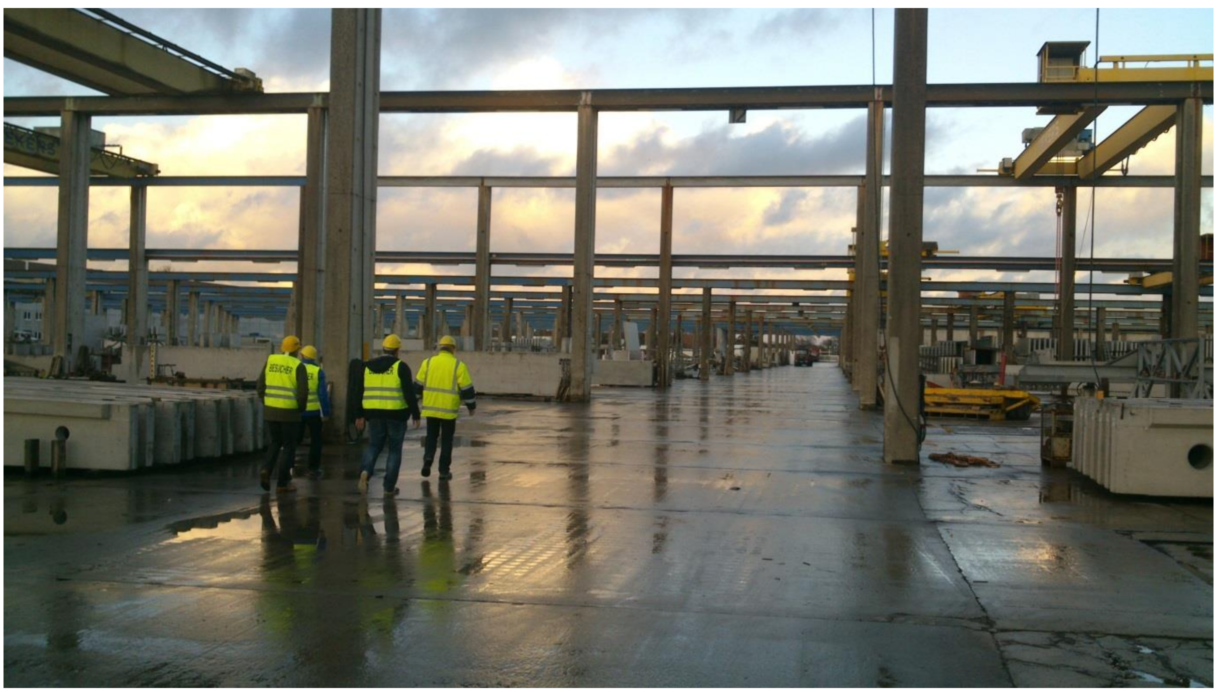


FIGURE 20: STORAGE FIELD OUTSIDE OF THE FACTORY

The fact that Rekers-beton has manufactured almost everything that was required for CD20 in the past decade, means that a stationary factory is suitable for production. This allows for more factories around the world to be a potential supplier of CD20 elements. The right molds and the right information in order to make the proper product should be provided. Therefore the production of the elements should not pose a problem, the only aspect that should be decisive should be the quality-price ratio.

2.4.3. Transport of the elements

The elements have varying lengths which these depend on the project that they are made for. The measurements of the common elements are 7,2x3,6 meter for the floor panels. These are the largest elements of the system as the columns and the wall panels all have smaller dimensions. The elements are still transportable by trucks, but large distances are preferred by ship. Elements with custom lengths are also being made, but this does not presents a problem for the transport since the maximum length is still within the transport length. Standard trucks have a transport floor length of 13.4 meters. Longer elements could be transported by using special issue trucks, but this will be an exception to the rule.

The transport in the wealthy parts of the world is no problem as roads and canals link al the sites together. A lot of areas where CD20-buildingsystems will be constructing are accessible by roads. That means a crane can be transported there and all the elements can be transported to the site. The ability to get a crane over to the site is essential for the building process. Without a crane the elements can not be lifted into place. In places that are tougher to reach, roads can be constructed and thus building in these locations is also a possibility.

2.4.4. Assembly of the elements

In order to assemble the different elements, one needs a crane to lift all of the elements into place. This crane is essential to the building process. Without the crane, it is not possible to assemble the building elements. The connections are simple and fast to make, so the construction time is short. When a certain level is finished, the finishing can be started for that level while the other levels are still under construction. By being able to start installing the systems while the rest of the load bearing structure is still under construction. The time before the building can be taken into commission is shorter in comparison to traditional building styles as well.

One building crew for the CD20-buildingsystem consists out of 7 men and a single crane. Usually a single crew is enough to build a structure, since the building process is fast. The assembly requires people that are familiar with the system and know what to do. This is not the difficult part of the system, however, and should be easily thaught to local construction workers. It may be useful to have an engineer that has experience with CD20 at the site to oversee the construction.

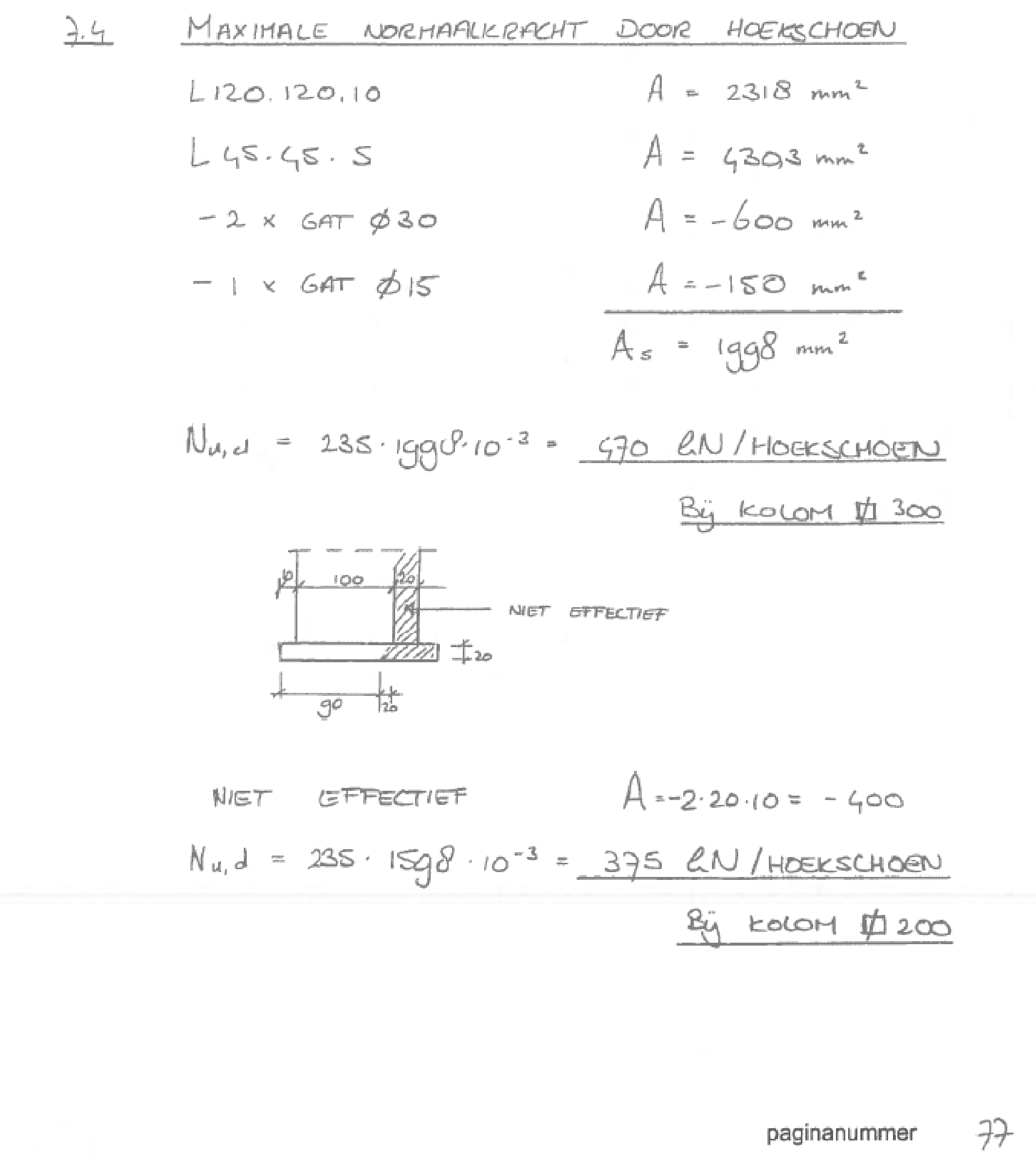
2.5 Details

The construction consists out of elements and these elements form a construction through the connections. These connections are of a different kind than usual structures, due to the fact that it is a concrete prefab structure. In this section three different connections will be presented:

- Column-Floor
- Floor-Wall
- Wall-Column

COLUMN TO FLOOR CONNECTION

The first connection is the Column to Floor connection. This is the cornerstone of the CD20 building system. The connection consists of a plate on top of the column with 4 studs and the floor elements have steel corner shoes. The steel corners have a cylindrical hole running through them. The studs of the column above and the column beneath will go into the hole and will be connected by a guiding ring. The load transfer does not go from one stud to another; it will go from the plate of the column, to the shoe, to the plate of the other column. The studs of the columns are meant to keep the connection in place and to make the montage easy. The vertical capacity of the connection is 470 kN for a 300x300mm and 375 for a 200x200mm. These calculations can be found in the standard calculations for the elements of CD20 at Royal Haskoning. They also transfer horizontal forces from the column to the floor slab and vice versa.



7.4 MAXIMALE NORMAALKRACHT DOOR HOEKSCHOEN

L120.120.10	$A = 2318\ mm^2$
L45.45.5	$A = 430\ mm^2$
−2 × GAT ⌀30	$A = -600\ mm^2$
−1 × GAT ⌀15	$A = -150\ mm^2$
	$A_s = 1998\ mm^2$

$N_{u,d} = 235 \cdot 1998 \cdot 10^{-3} = \underline{470\ kN / HOEKSCHOEN}$

BIJ KOLOM ⌀ 300

NIET EFFECTIEF $A = -2 \cdot 20 \cdot 10 = -400$

$N_{u,d} = 235 \cdot 1598 \cdot 10^{-3} = \underline{375\ kN / HOEKSCHOEN}$

BIJ KOLOM ⌀ 200

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FIGURE 21: STRENGTH OF THE CORONER SHOE IN THE PLATE

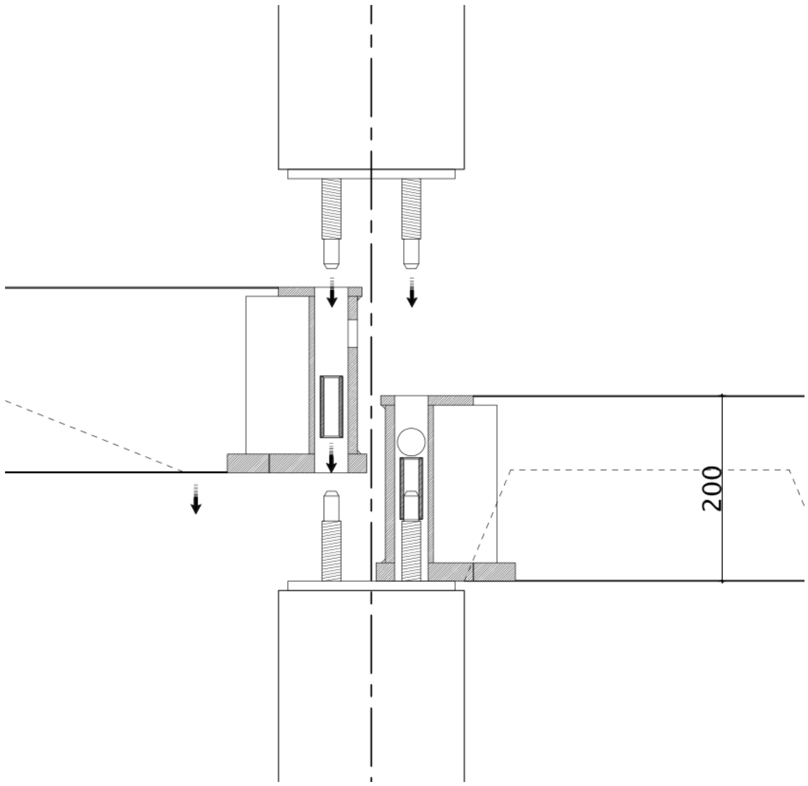


FIGURE 22: COLUMN TO FLOOR CONNECTION

FLOOR TO WALL

The second connection of importance is how to connect the floor to the wall elements. The columns have studs that make a connection with the wall. The gains in the walls can later be filled to finish the connection. Connection plates can be used if more horizontal force needs to be transferred between the wall and the floor plate.

FIGURE 23: PLACING OF A WALL ELEMENT

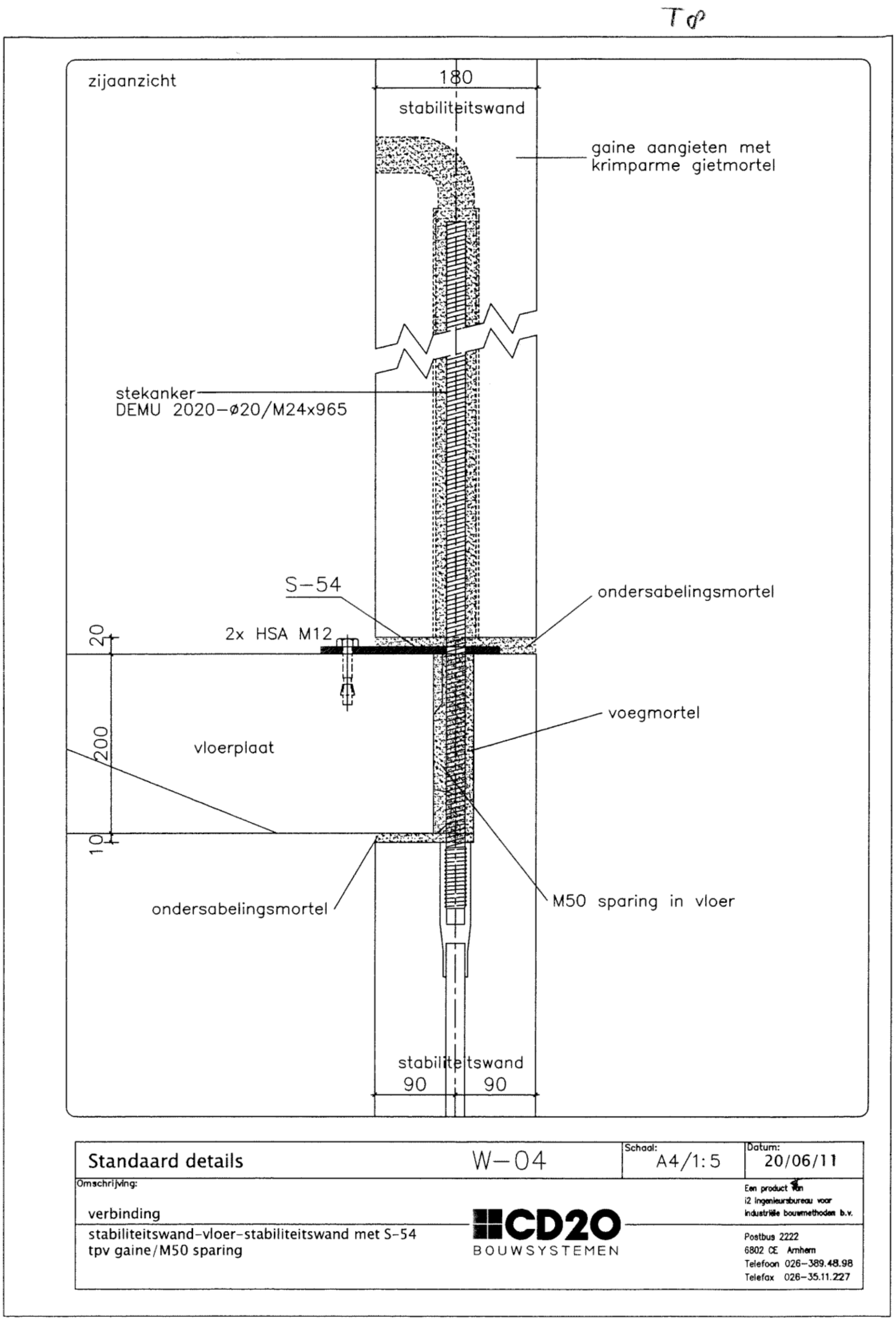


FIGURE 24: DETAIL OF THE CONNECTION BETWEEN THE WALL AND THE FLOOR ELEMENTS

FIGURE 25: CONNECTION BETWEEN WALL AND FLOOR ELEMENTS

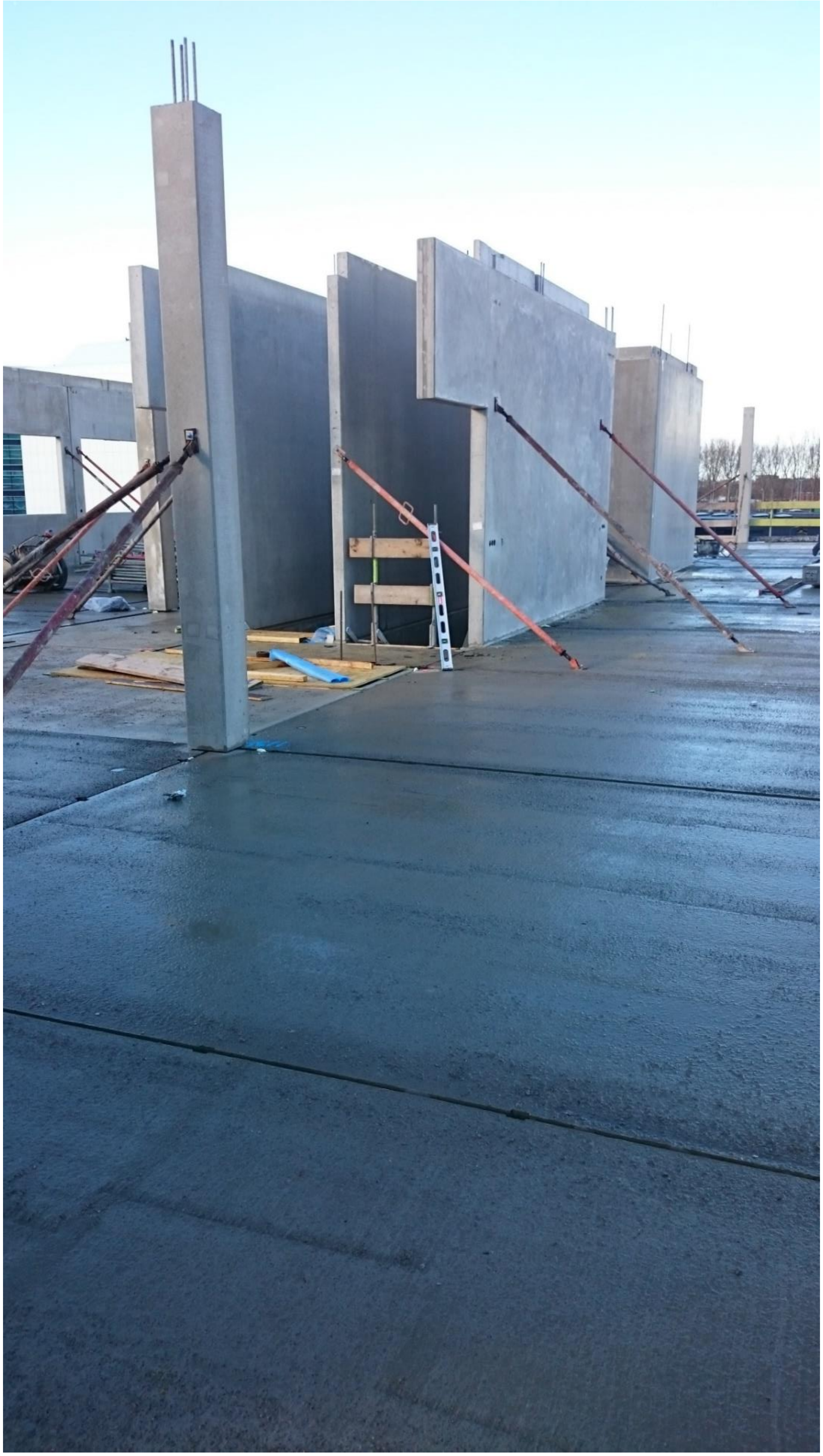


FIGURE 26: BUILDING UNDER CONSTRUCTION

FIGURE 27: COLUMN TO FLOOR CONNECTION, WITH MORTER

3. CONNECTION MODEL IN DIANA

Now that the structure of the study has been determined the methods to do so need to be further explained. The schematics, figure 1, show the first step after setting the boundaries is setting up the Diana model. In order to do this, an explanation is required of what kind of program Diana is and what it will be used for.

The program Diana was researched and constructed by TNO which stands for Toegepast Natuurwetenschappelijk Onderzoek in Dutch, in English that means Applied Scientific Research. TNO is the largest research institution in the Netherlands and does projects for the Government and other parties. They have created the earliest version of Diana in 1974, and have improved upon it ever since. Diana stands for DIsplacement ANAlyzer, because the code was based on the displacement method.

Diana is categorized as a FEA program or FEM. FEA stands for Finite Element Analysis, the overall term most commonly used for these kinds of programs is FEM Finite Element Method, but they mean the same thing. A FEM program is capable of cutting the object that needs to be calculated up in a large amount of slices. Then it calculates the stress for each of those slices using the boundary conditions such as the loading, the restraints, the material properties and the input from the surrounding slices. It then compiles the overview back to the engineer. Depending on how the model is setup and how good the software is more or less information is available.

The reasons Diana was used for this particular research are that Diana is a program that can calculate plastic, cracked situations and allows for programming in volume elements . The other FEM programs allow the engineer to program in 2D or 2,5D at best. In some cases that is sufficient to analyze the problem one is faced with. In this case a full understanding of the connection in 3 different directions is required. That calls for a model that can calculate behavior in 3 different directions. Diana can make use of volume elements which are full 3D and allow for better analysis of the problem.

This chapter will be divided into the following subjects.

1. Setup of the model
2. Testing of the model
3. The results of the model

3.1. Setup of the model

In order to build a working model, first the purpose of the model needs to be clear. The goal of this model is to find a way to incorporate the behavior of the connection into a Scia model. This must be done in order to best approach the behavior of the total system in earthquake conditions. The final output of the model will be a force-displacement relationship, which will be used to determine the stiffness of the connection

The connection consists of the columns with the pens and the floor slabs with the corner shoes. The connection has symmetry that allows reduction of the model size. The behavior of the elements on either side of the symmetry line is identical, which makes it function in the same way the other part is functioning. This allows for instance to only model half of the element, with added boundary conditions, instead of the entire model, thus saving memory space on the computer.

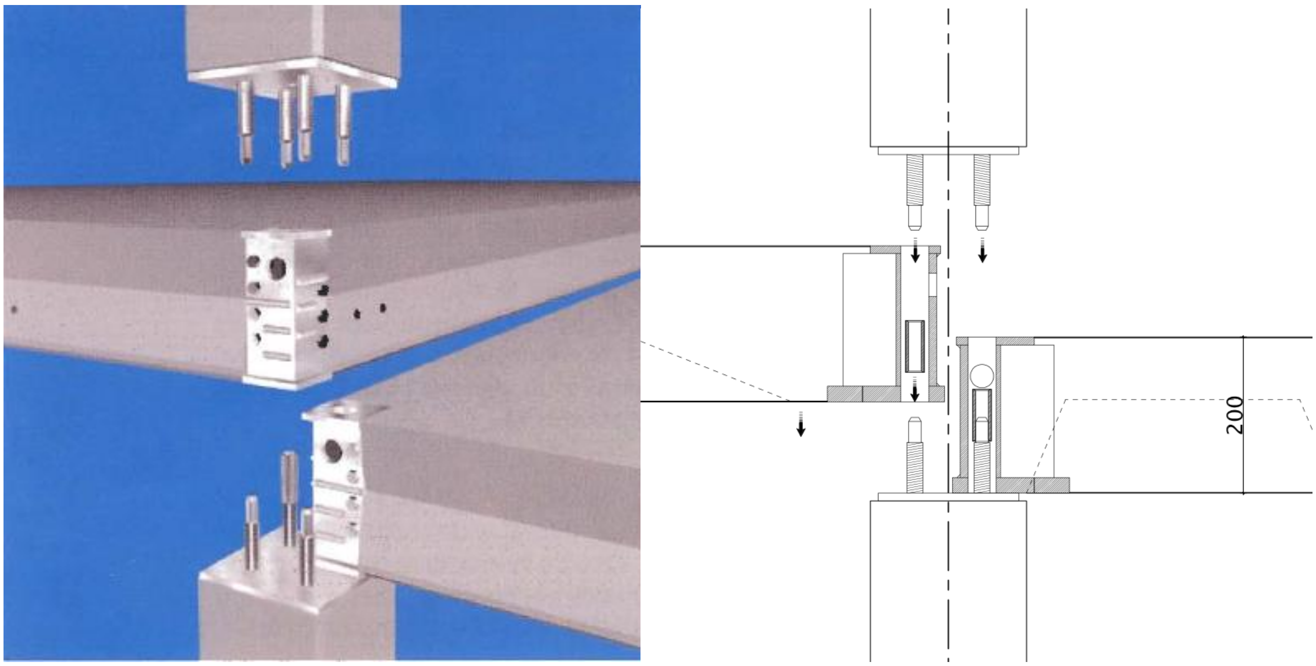


FIGURE 28 SYMETRY PRESENTED

In the case of this system it is most ideal to consider 2 lines of symmetry, a 3rd could be argued for but modeling this part will prove more useful in the end. When a top down view of the connection is shown, figure 27, simplified it comes down to 4 plates connected between 2 columns which is shown in figure 29. The three red squares and the yellow square represent the floor plates and the green outlined square represents the column. Since the connections to the column are all identical to each other this does not make a difference. That means there are four identical situations and they mirror each other from left to right and top to bottom.

In order to simplify the connection only the yellow part of the connection needs to be modeled. That means one gets the model based on half of what is shown in the right picture of figure 28. The following elements have to be built in the model:

- The corner shoe.
- Two quarters of a column plate with a pin, one on top and one on the bottom.
- The concrete that holds the anchors for both the corner shoe and the column plate.
- The concrete inside the corner shoe encasing the pins.

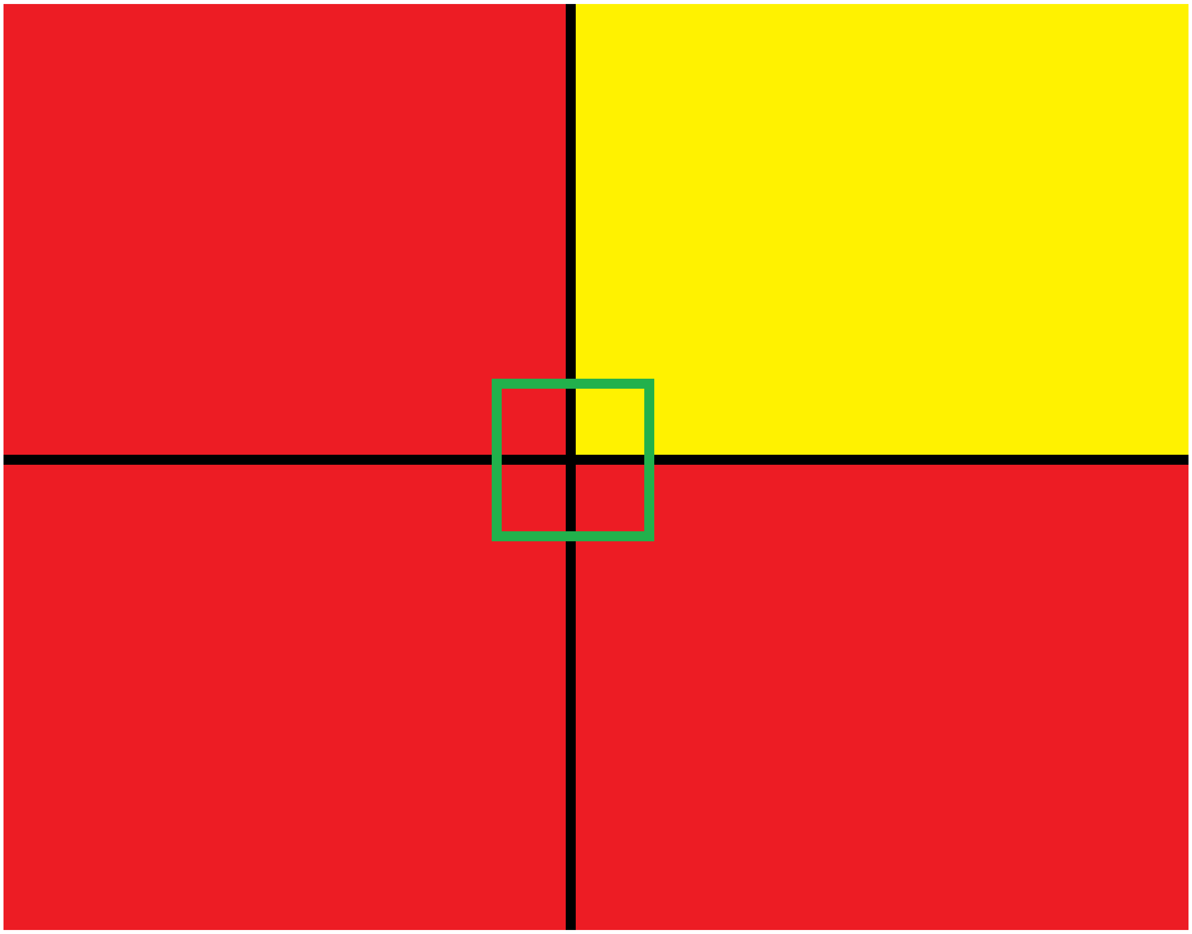


FIGURE 29 WHAT WILL BE MODELED

There is one last thing to consider, however, and that is how much concrete for the column and the floor need to be incorporated into the model. The concrete should be present to allow for a proper induction of the forces upon the model, however the elements themselves are being modeled in Scia engineer later on. Including too much of the elements in this model would result in getting certain areas double in the model. For example the column has a certain stiffness in the horizontal direction that is accounted for in the Scia model. If a meter of column is modeled in the Diana part, than part of this flexibility in horizontal direction is also coming from the column. Now this meter of column is again modeled in Scia, so instead of a column that is 3 meters in length, in horizontal sense, a column with a length of 4 meters is in fact what is being calculated with the values used. Therefore the behavior of the column will not be included in the Diana model and only the column plate will be modeled.

Concerning the floor plates, part of the concrete directly influences the deformation of the corner shoe. That is the concrete that is situated inside the corner shoe and the concrete that is in places that corner shoe could otherwise bend towards, but is now being opposed by the concrete to do so. That concludes the list of what needs to be modeled in order to successfully analyze the connection.

In order to create a model within Diana first a geometry model is needed, a 3D model that will be comprised of volume elements. In order to do that a geometry model will first be constructed using a 3D modeling program. For this task Rhinoceros has been used, because it allows for easy, but more important, accurate drawing. The whole model can be setup using 7 steps, these are presented below.

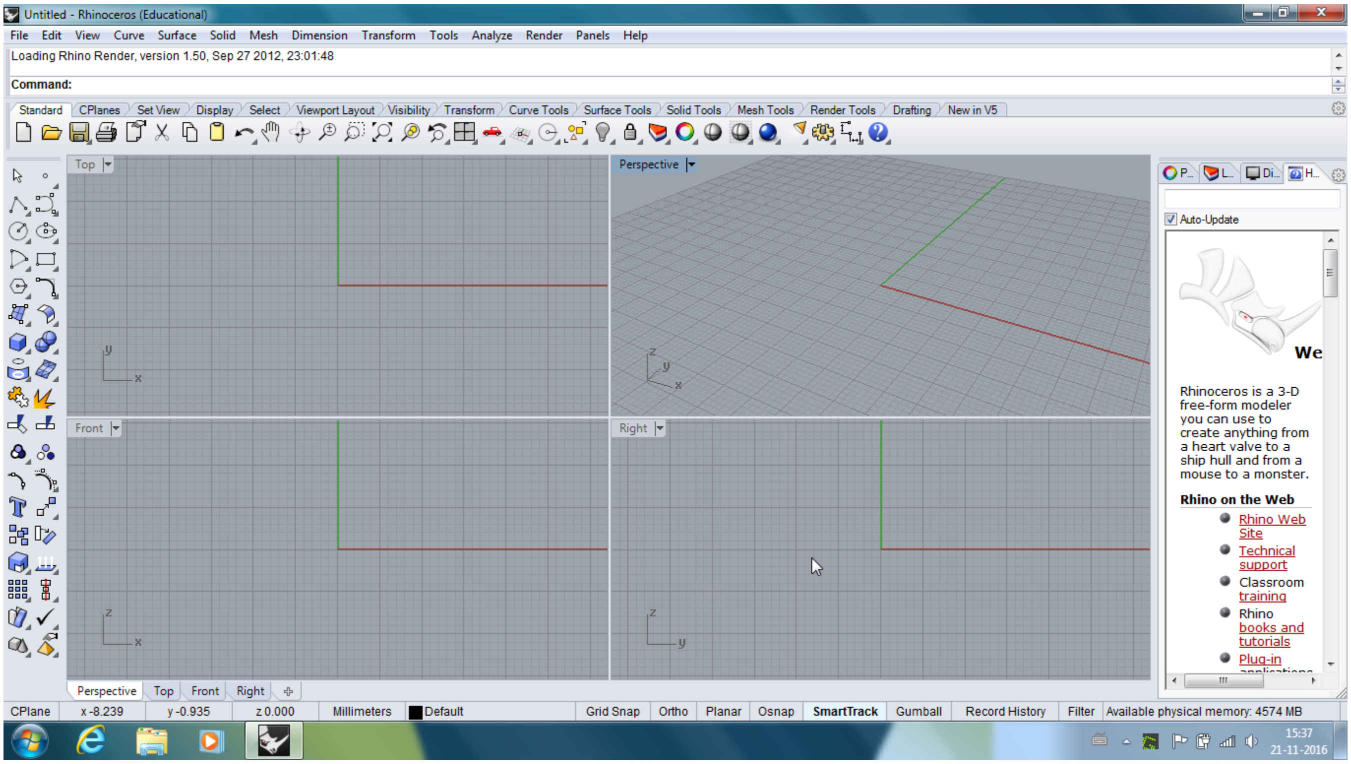


FIGURE 30 RHINOCEROS INTERFACE

1. Step 1 is to build the model in Rhinoceros.
2. Step 2 is to convert the model that was constructed in Rhino to FX+ for Diana.
3. Step 3 is assigning properties to the elements.
4. Step 4 is to build the relations that exist between the different parts by means of interface elements, in order to make every element respond to the next as it would in real life.
5. Step 5 is to apply the boundary conditions that apply on the part of the connection that was modeled in order to create the circumstances that best approach the reality.
6. Step 6 is to apply the loads to test the circumstances the connection is in.
7. Step 7 is to run the model in order to see if it works, if it does research can be performed and the stiffness of the connection can be determined

A full account of the way this Diana model has been developed can be found in appendix 9.2 Development of the Diana model.

3.2. Final model

The model has undergone a few drastic changes from start to finish. The final transformation will be illustrated by a number of figures.

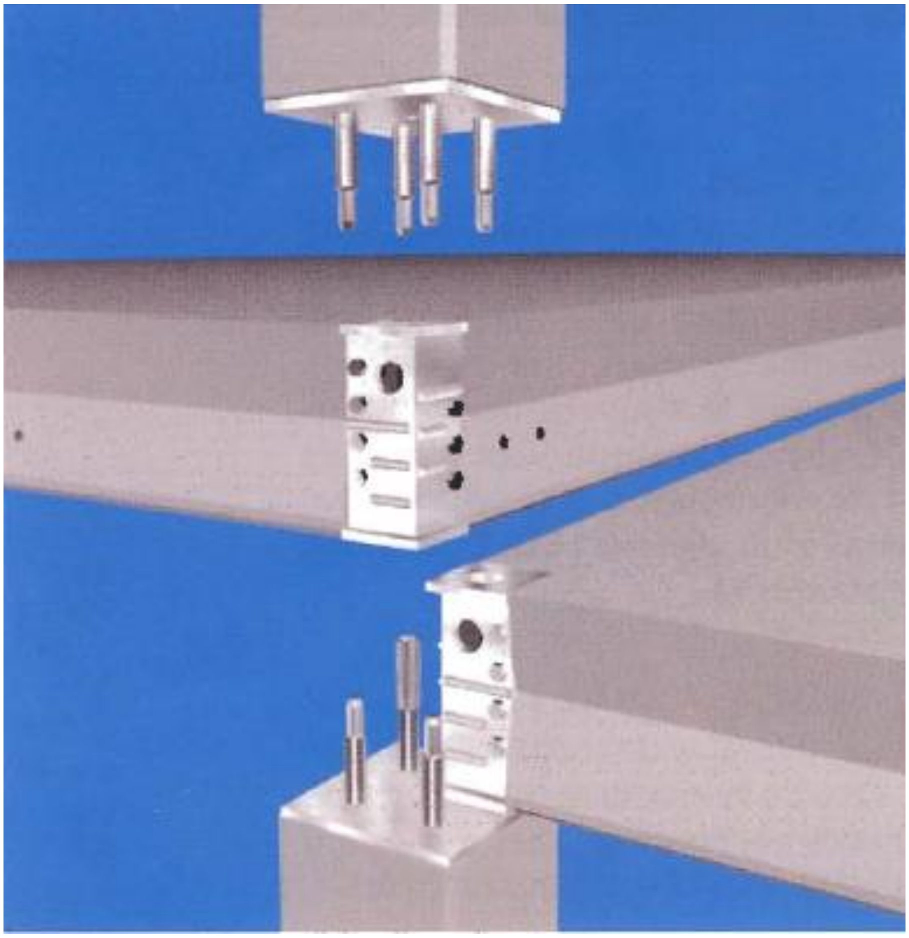


FIGURE 31 THE REAL SITUATION

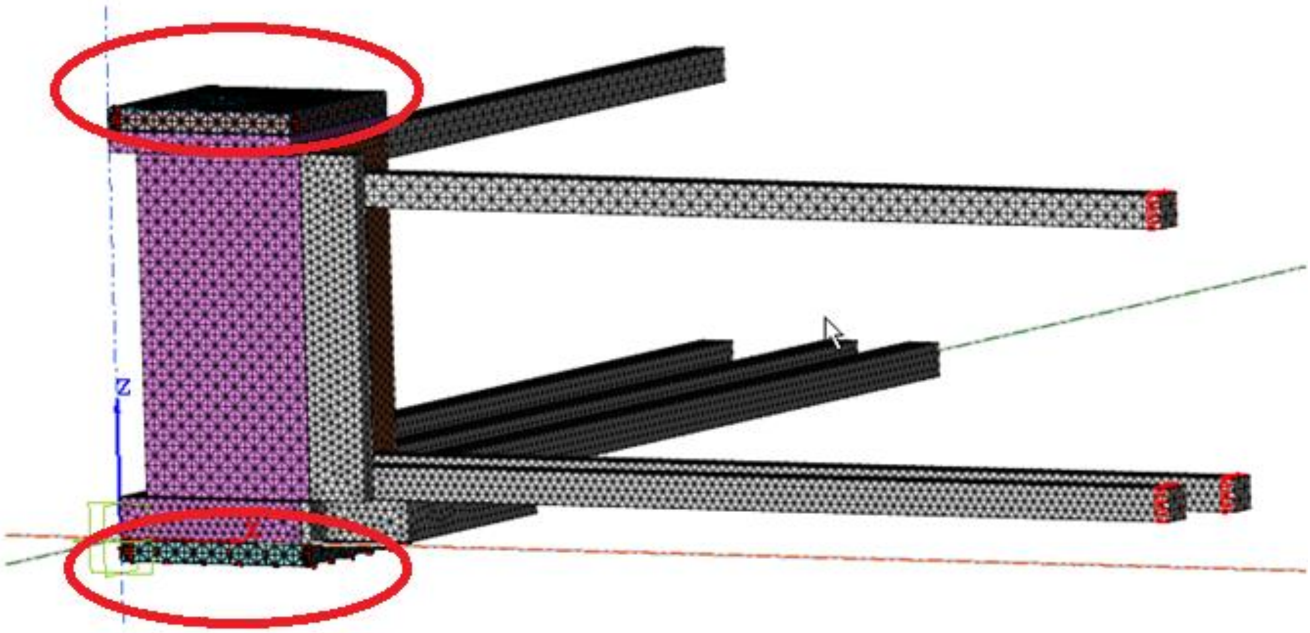


FIGURE 32 THE FINAL MODEL USED TO CALCULATE THE STIFFNESS OF THE CONNECTION

Figure 32 shows the final model that has been used to calculate the connection. The model consists of one-fourth of the steel plate of the upper column and one-fourth of the steel plate of the lower column. to these steel plates the pins are attached that go inside the corner shoe. The model furthermore consists of the corner shoe, and two pieces of concrete. The concrete inside the corner shoe around the pins, and the concrete filling up the empty space between the top and the bottom plate of the corner shoe itself. Figure 33 displays a corner shoe without any concrete yet, here the open space between the upper and lower plate can be seen.

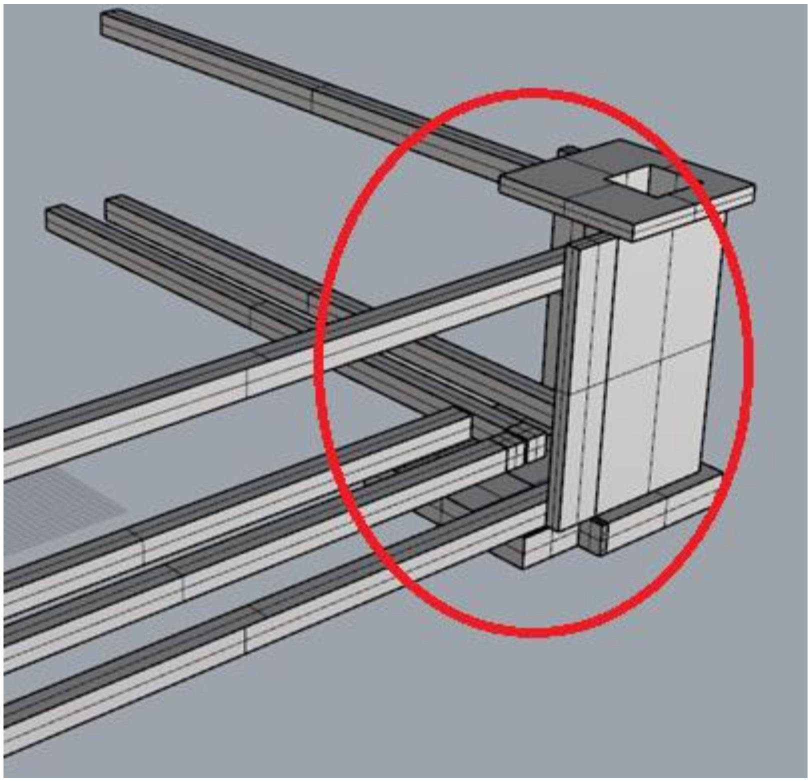


FIGURE 33 DISTANCE BETWEEN THE UPPER AND LOWER PLATE OF THE CORNER SHOE

The boundary conditions of this final model are shown in the figures 35 and 36 and are the results of the symmetry simplifications to the model. Figure 34 shows the entire connection in a top down view of 4 floor slabs and the position of the column by the green square. The yellow square is what will remain after the first simplifications.

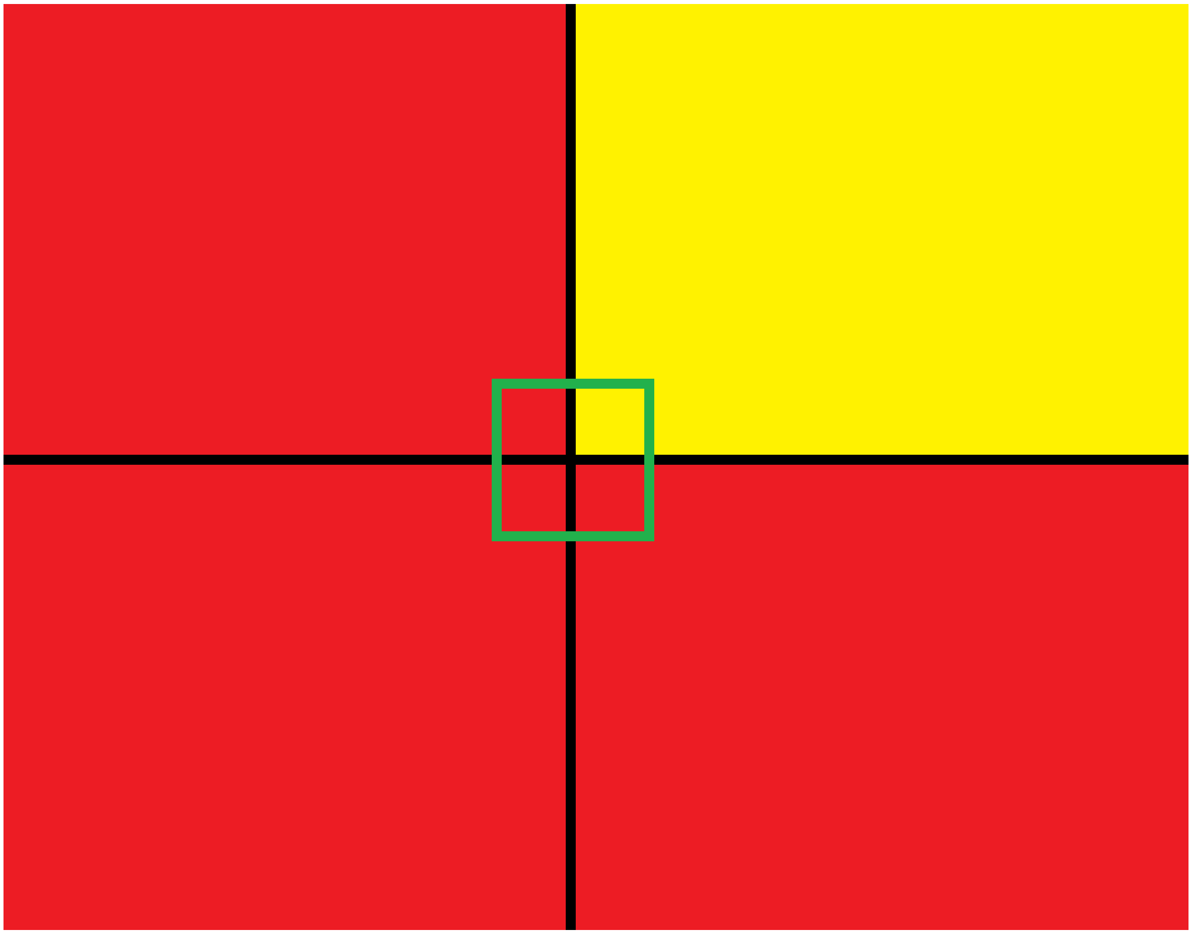


FIGURE 34 THE WHOLE MODEL

The first line of symmetry is from top to bottom, because the left of the figure mirrors the right of the figure. Removing this part of the connection does require a boundary condition for the remainder of the connection. The two plates on the left prevent the two plates on the right from moving towards the left, therefore the boundary condition will be a rolling support, limiting one degree of freedom, the movement from le ft to right, as seen in figure 35.

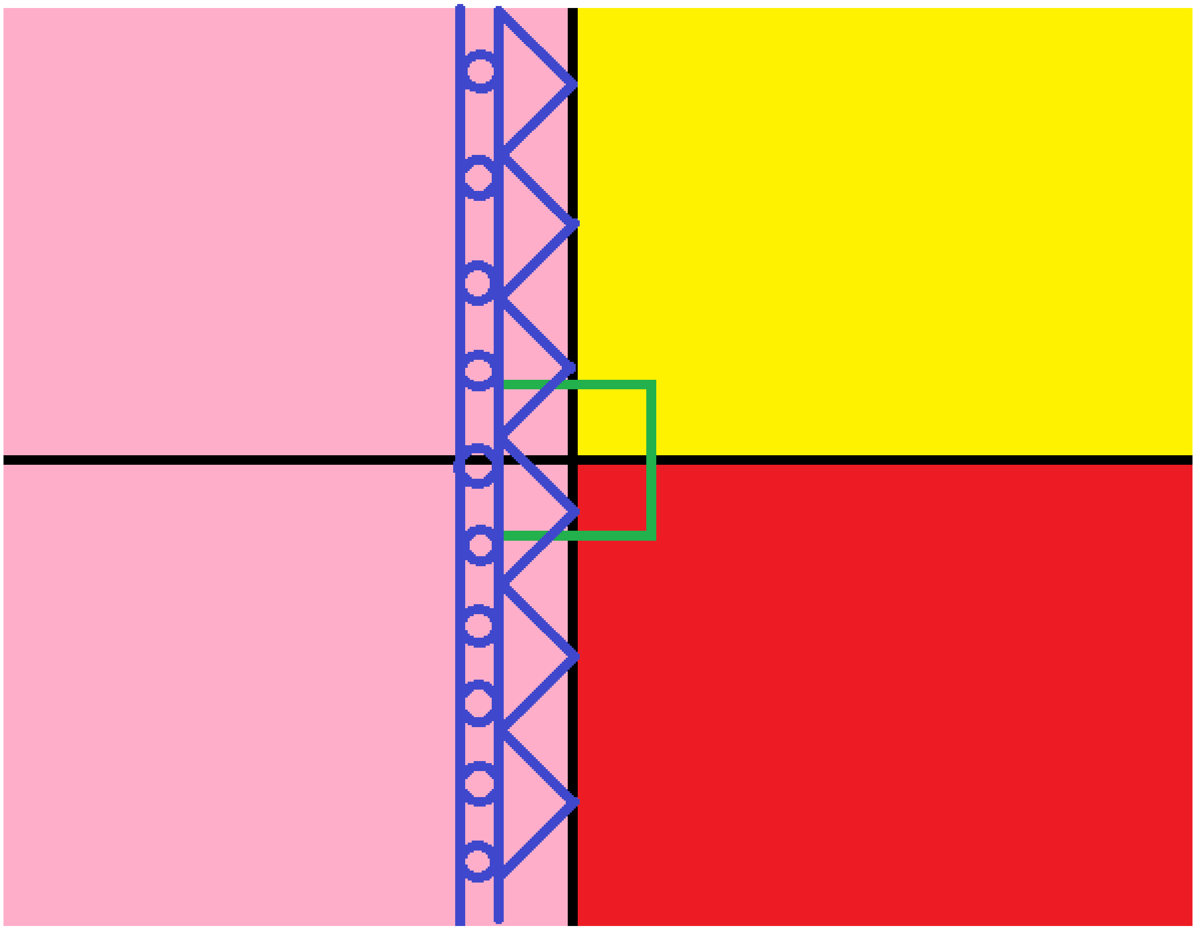


FIGURE 35 THE FIRST SYMETRY LINE, THE LEFT SIDE OF THE CONNECTION IS REMOVED AND REPLACED BY THE BOUNDARY CONDITION OF A ROLLING SUPPORT, LIMITING THE HORIZONTAL MOTION TO 0 IN THIS DIRECTION

The second line of symmetry is from left to right, allowing to cut the connection in half again. Leaving just one floor plate and a quarter of the column plates (one above and one below the corner shoe) as can be seen in figure 36.

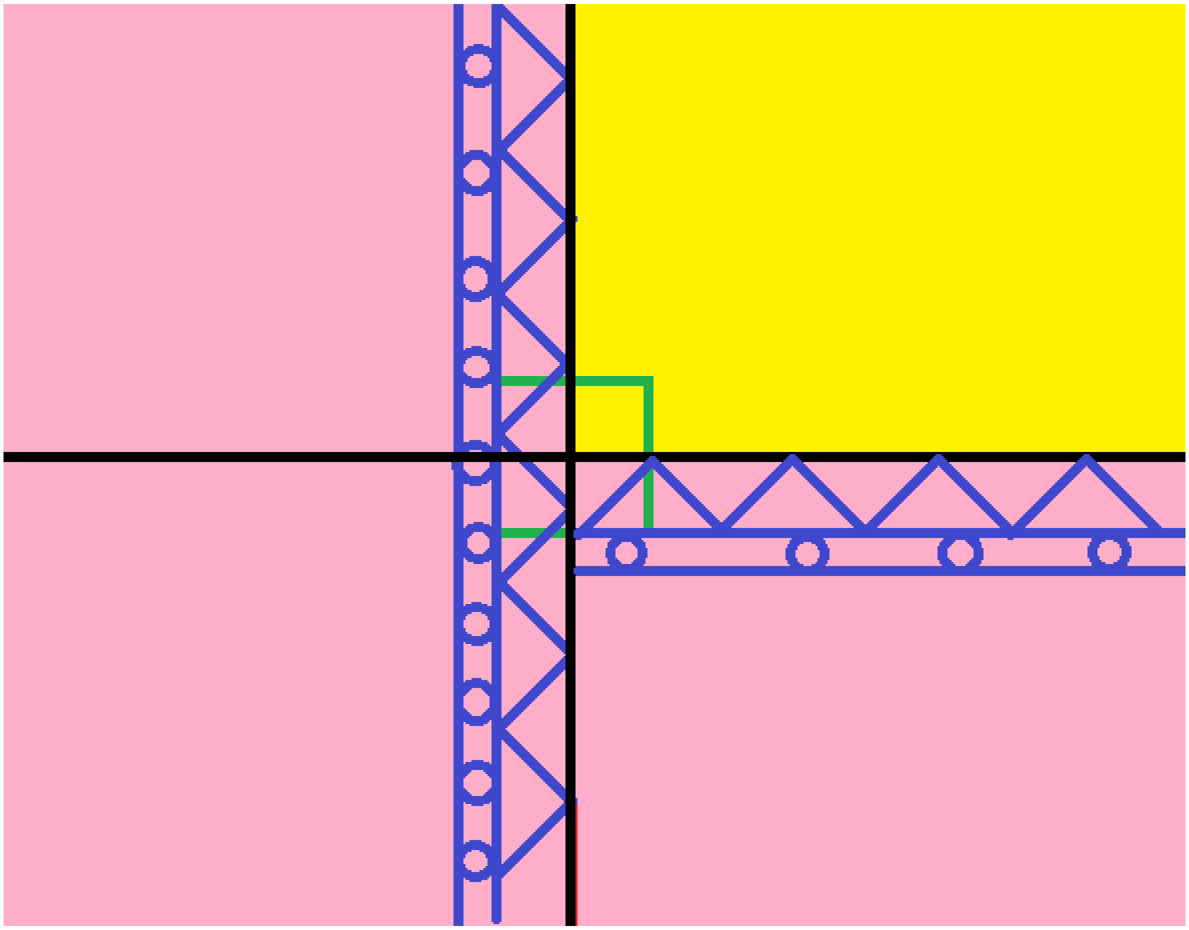


FIGURE 36 TWO LINES OF SYMETRY HAVE BEEN PROCESSED LEAVING ONE FLOOR PLATE WITH ONE CORNER SHOE AND THE CORRESPONDING PINS FROM THE COLUMNS

The final part of the adaptation concerns the removal of the plate and the column, as they will be modelled in SCIA engineer and therefore would otherwise have a double effect on the stiffness of the connection. That means the transfer of the forces from the plate to the corner shoe has not been taken into account. But seen as the anchors are a higher steel grade and have more capacity than the connection pin, it is fair to assume they will not be governing.

The final boundary conditions are the support at the bottom steel plate of the column. this will be restricted in movement because the column can not move and the plate is attached to that. So the steel plate will be supported along the bottom surface. The boundary conditions retrieved from the previous steps are also only applied to the steel plate of the columns. because this steel plate is the element connecting this model to the rest of the connection, that has been simplified due to symmetry. These steel plates are seen in figure 37. Test will be run by applying a force on the anchors and reading the displacements in the connection.

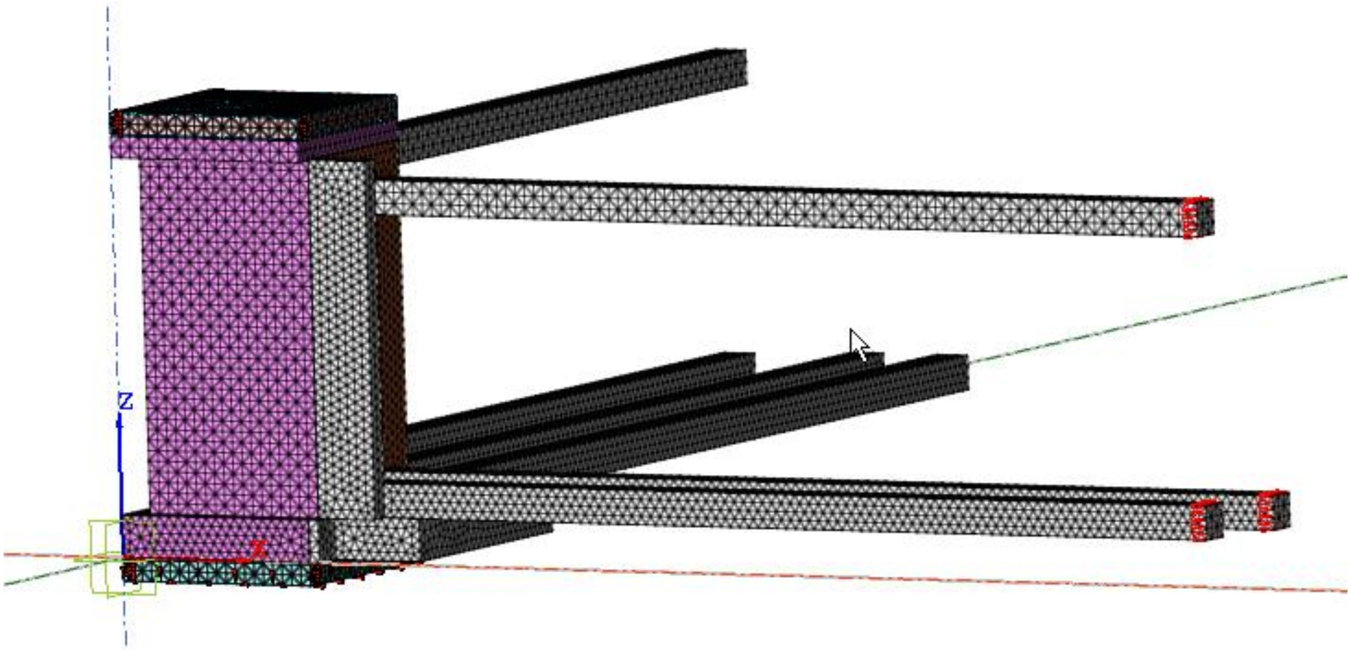


FIGURE 37 FINAL DIANA MODEL

Next the model will be tested on different load cases, in such a way that several conclusions can be made. The model is tested in two directions, X and Y. These Directions have been chosen in such a way that the rebar anchors coincide with the directions. The loads will be put on the rebar anchors on the ends as a tensile force. At the same time a normal load will be applied on the corner shoe representing the force of the stories that are above this floor loading the corner shoe.

3.3. The displacements

The goal of the Diana model is to find out what the displacement is by a corresponding force. Note, the final values that have been used are given in chapter 4.5 Improved model.

The force-displacement relation is what will be used to determine the stiffness of the connection as a whole. The output of the Diana model will be the displacements measured when the forces are applied. These values are presented in figures 41 and 42. The reason for having two measuring points for the displacements is because there are several points where the forces are pulling simultaneously during the tests.

The anchors of the corner shoes are being pulled on, in both directions separate from each other. The displacements are measured at the back of the corner shoe as is illustrated in figure 38. The corner shoe is connected at the top to the column above and at the bottom to the column below. Because there is a difference in the number of anchors and the corner shoe is not symmetrical top to bottom. There could be differences in the displacements of the corner shoe at the top and at the bottom. That is why the displacements are measured in a bottom point and a top point.

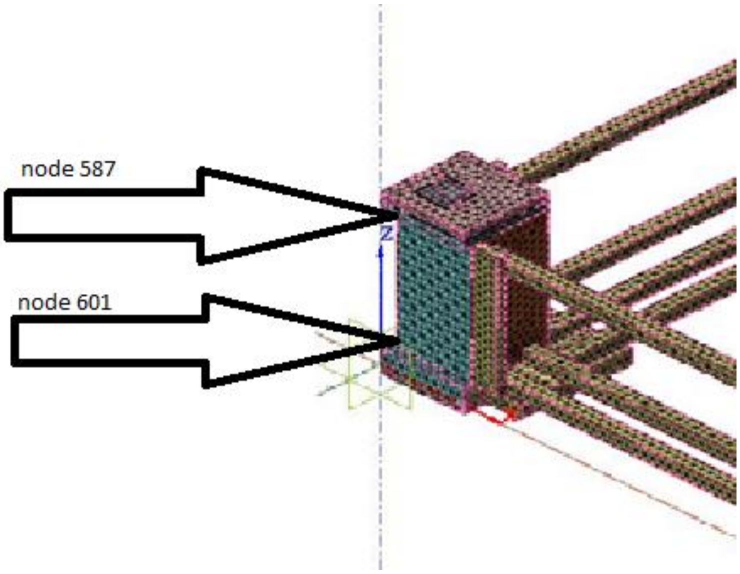


FIGURE 38: POSITION OF NODES

Lastly applying a moment on the corner shoe and measure the corresponding displacements does not server any purpose. The pulling forces used now on all anchors would become a pulling force on one side and a pushing force on the other side. This creates the same force-displacement relation but now with different values, but the stiffness of the connection does not change, so the relation between the two will not change. Therefore it is essentially the same test that is performed and gives no additional information. Figures 39 and 40 illustrate this.

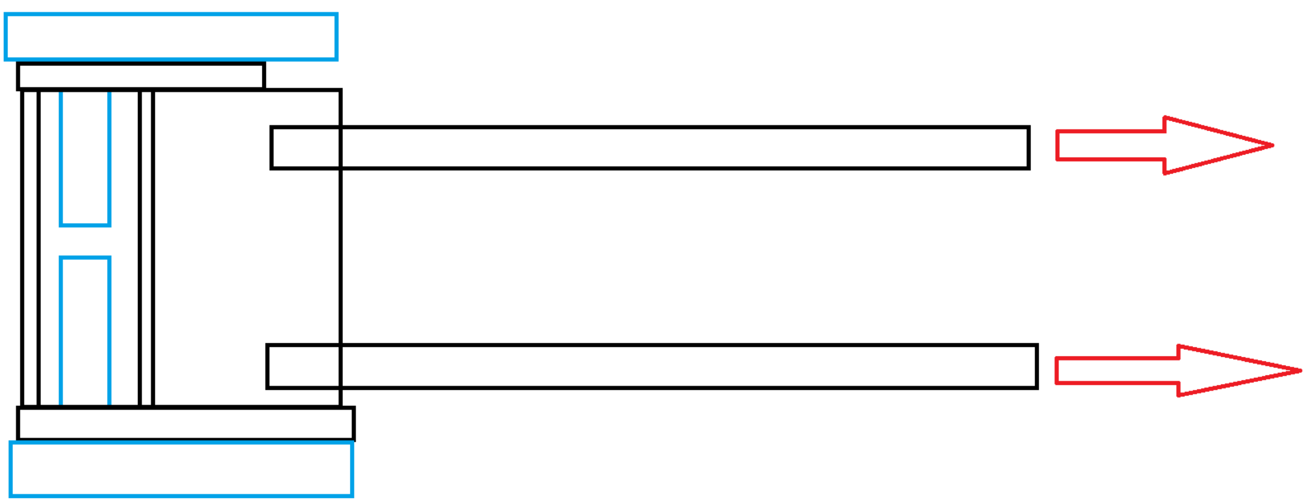


FIGURE 39 PULLING FORCE TEST, LEADING TO A FORCE DISPLACEMENT RELATION

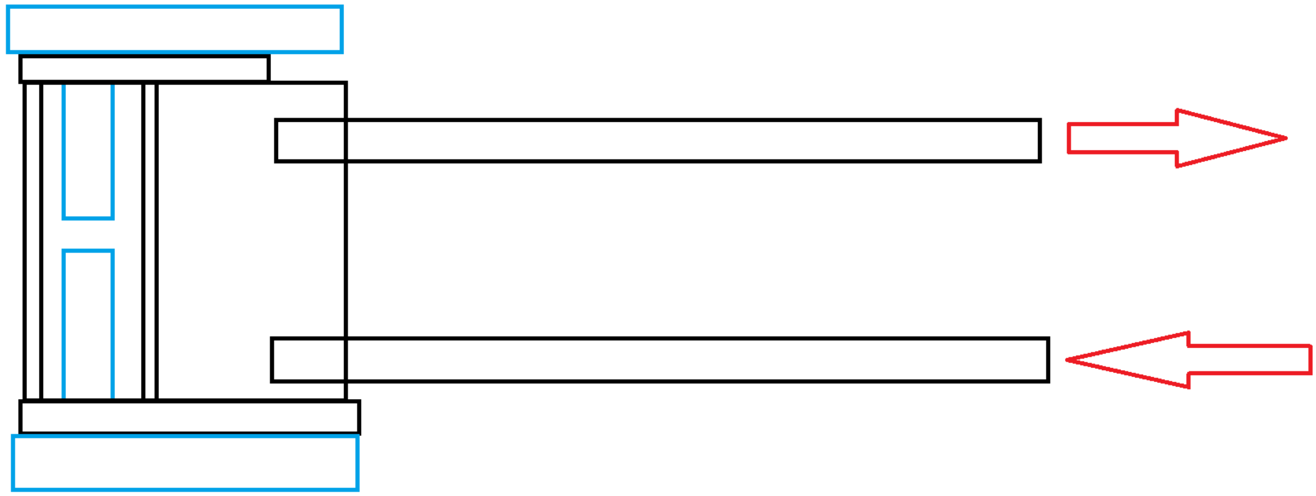


FIGURE 40 THE WAY A MOMENT WOULD BE INTRODUCED ON THE CORNER SHOE, THIS WOULD HOWEVER GIVE THE SAME FORCE DISPLACEMENT RELATION, BECAUSE IT IS THE SAME CONNECTION, ONLY THE DIRECTION OF THE FORCES DIFFER

Figure 41 and 42 show the displacements in all three principle directions per load case. The load cases are in the two main directions, in the directions the anchors stretch, they are both being researched because there are 3 anchors in one direction and 2 in the other what could lead to different displacements.

Test case point 587 Horizontal force	DtX [mm]	DtY [mm]	DtZ [mm]
Load case 1			
0V-50H	5.856E-03	-8.221E-04	-2.949E-03
0V-100H	1.171E-02	-1.644E-03	-5.899E-03
0V-200H	2.343E-02	-3.289E-03	-1.180E-02
100V-50H	-7.877E-02	-8.403E-02	-9.875E+00
100V-100H	-7.291E-02	-8.485E-02	-9.878E+00
100V-200H	-6.120E-02	-8.649E-02	-9.884E+00
200V-50H	-1.634E-01	-1.672E-01	-1.975E+01
200V-100H	-1.575E-01	-1.681E-01	-1.975E+01
200V-200H	-1.458E-01	-1.697E-01	-1.976E+01
Load case 2			
0V-50H	-1.758E-04	4.531E-03	-2.288E-03
0V-100H	-3.515E-04	9.063E-03	-4.577E-03

0V-200H	-7.031E-04	1.813E-02	-9.154E-03
100V-50H	-8.480E-02	-7.867E-02	-9.874E+00
100V-100H	-8.497E-02	-7.414E-02	-9.876E+00
100V-200H	-8.532E-02	-6.508E-02	-9.881E+00
200V-50H	-1.694E-01	-1.619E-01	-1.975E+01
200V-100H	-1.696E-01	-1.573E-01	-1.975E+01
200V-200H	-1.699E-01	-1.483E-01	-1.975E+01

FIGURE 41: DISPLACEMENTS AT GIVEN FORCES AT POINT 587

Test case point 601 Horizontal force	DtX [mm]	DtY [mm]	DtZ [mm]
Load case 1			
0V-50H	5.539E-03	-2.041E-04	2.622E-03
0V-100H	1.108E-02	-4.083E-04	5.244E-03
0V-200H	2.215E-02	-8.165E-04	1.049E-02
100V-50H	-6.845E-02	-6.199E-02	-8.895E+00
100V-100H	-6.291E-02	-6.220E-02	-8.893E+00
100V-200H	-5.183E-02	-6.260E-02	-8.887E+00
200V-50H	-1.424E-01	-1.238E-01	-1.779E+01
200V-100H	-1.369E-01	-1.240E-01	-1.779E+01
200V-200H	-1.258E-01	-1.244E-01	-1.779E+01
Load case 2			
0V-50H	1.581E-03	3.861E-03	2.657E-03
0V-100H	3.163E-03	7.721E-03	5.314E-03
0V-200H	6.326E-03	1.544E-02	1.063E-02
100V-50H	-7.240E-02	-5.793E-02	-8.895E+00
100V-100H	-7.082E-02	-5.407E-02	-8.893E+00
100V-200H	-6.766E-02	-4.634E-02	-8.887E+00
200V-50H	-1.464E-01	-1.197E-01	-1.779E+01
200V-100H	-1.448E-01	-1.159E-01	-1.779E+01
200V-200H	-1.416E-01	-1.081E-01	-1.779E+01

FIGURE 42: DISPLACEMENTS GIVEN AT FORCES IN POINT 601

4. TRANSLATION FROM DIANA TO SCIA ENGINEER

From the Diana model calculations a force-displacement curve can be setup, which will be used to determine the stiffness of the springs in Scia Engineer. The Diana model used to determine the stiffness of the connection consists out of the corner shoe, the concrete inside the corner shoe and the concrete on the outside transferring part of the vertical load, the pins and the plate that is attached to the column.

The rest of the column is irrelevant concerning the stiffness of the connection because the column will be modeled in the Scia model, so the additional displacements it gives are accounted for. If the column would be modeled into the Diana model than the effect of the column would be taken into account twice, which would be wrong.

A series of tests has taken place with varying vertical and horizontal loads. Three different vertical loads have been applied: 0 kN, 100 kN and 200 kN. Three different Horizontal loads have been applied as well, being: 50 kN, 100 kN and 200 kN. That means a total of 9 different tests that can be performed. This leads to the following table of displacements:

Force	DtX [mm]	DtY [mm]	DtZ [mm]
0V 50H:	4.29E-03	-1.07E-03	-2.21E-03
0V 100H:	8.58E-03	-2.13E-03	-4.42E-03
0V 200H:	1.72E-02	-4.27E-03	-8.84E-03
100V 50H:	-6.55E-02	-6.11E-02	-1.95E+00
100V 100H:	-6.13E-02	-6.21E-02	-1.95E+00
100V 200H:	-5.27E-02	-6.43E-02	-1.96E+00
Verschil 100V-0V	-6.98E-02	-6.00E-02	-1.95E+00
"	-6.98E-02	-6.00E-02	-1.95E+00
"	-6.98E-02	-6.00E-02	-1.95E+00
200V 50H:	-1.35E-01	-1.21E-01	-3.90E+03
200V 100H:	-1.31E-01	-1.22E-01	-3.90E+03
200V 200H:	-1.23E-01	-1.24E-01	-3.91E+03
Verschil 200V-0V	-1.40E-01	-1.20E-01	-3.90E+03
"	-1.40E-01	-1.20E-01	-3.90E+03
"	-1.40E-01	-1.20E-01	-3.91E+03

TABLE 11: DISPLACEMENTS OF CORNER SHOE IN X-DIRECTION

The values seen in the table are not what one would initially expect. The most logic thing would be that the displacements would become smaller when the vertical force increases due to a clamping effect increasing the friction on the corner shoe. However, the displacements are larger and become relatively smaller when the horizontal force increases.

The reason behind this is that the vertical force makes the corner shoe bend and thus there is an initial displacement. This can be seen in the values if we subtract the effect of the horizontal forces from the displacements. This can be done by using the value with no vertical force as base value and subtracting this value from the corresponding other values at that vertical force. All that remains is the effect from the vertical force and, as can be seen in the table above, these values are all the same.

That means that the vertical force has no influence on the horizontal spring stiffness of the connection. So in determining the spring stiffness only the highlighted values will be used. This will be done for the other direction as well as for the lower point in order to generate the correct spring stiffness. From these numbers a force-displacement diagram can be formed.

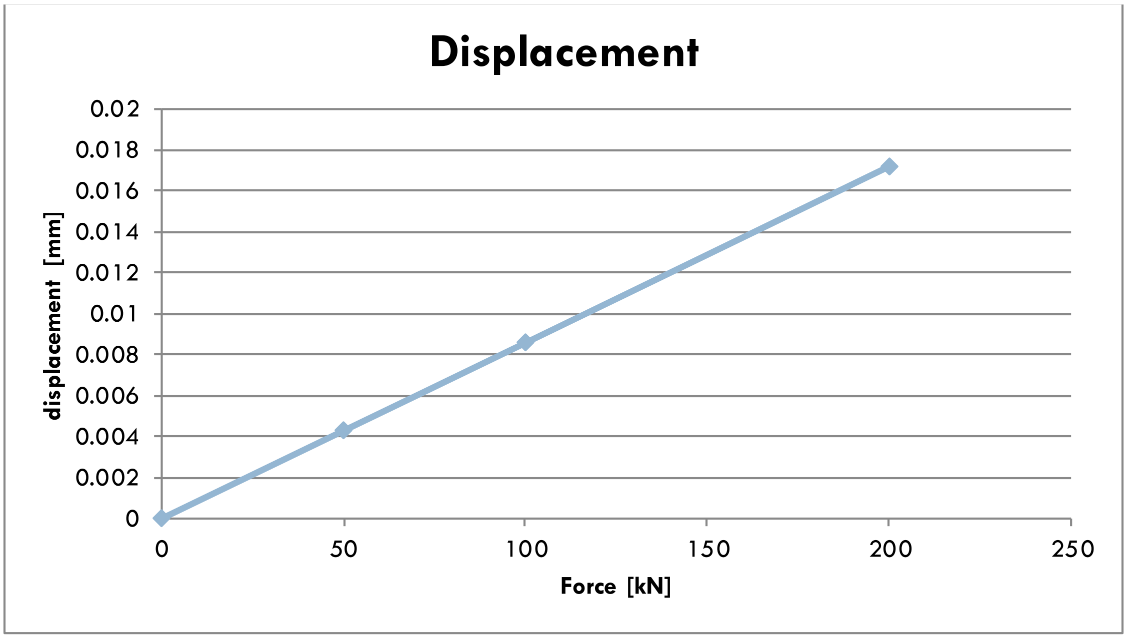


FIGURE 43: FORCE-DISPLACEMENT GRAPH

The characteristic spring stiffness is defined by the force divided by the displacement caused by this force. That gives the following value for the stiffness:

$$\frac{50{,}000}{0.00429} = 11{,}600{,}000 \ [N/mm]$$

$$\frac{100{,}000}{0.00858} = 11{,}600{,}000 \ [N/mm]$$

$$\frac{200{,}000}{0.0172} = 11{,}600{,}000 \ [N/mm]$$

The spring stiffness becomes $11{,}600{,}000$ N/mm, that makes for a very stiff spring. When the connection is modelled a reference is available to test the correctness of the model. In the section below all of the spring stiffnesses are worked out:

	Displacements				Stiffnesses			
kN	Upper Dx	Upper Dy	Lower Dx	Lower Dy	Upper Kx	Upper Ky	Lower Kx	Lower Ky
50	4.29E-03	3.22E-03	1.90E-03	1.28E-03	1.16E+07	1.55E+07	2.63E+07	3.92E+07
100	8.58E-03	6.44E-03	3.80E-03	2.55E-03	1.16E+07	1.55E+07	2.63E+07	3.92E+07
200	1.72E-02	1.29E-02	7.60E-03	5.11E-03	1.16E+07	1.55E+07	2.63E+07	3.92E+07

TABLE 12: SPRING STIFFNESS'S

4.1. The framework for the Connection

The connection made in Scia has a couple of boundary conditions it needs to comply to. The connection between the column and the plates needs to be established as well as the interaction between the plates. The Diana model has given the spring stiffness of a connection between the corner of a plate and a column. That value has to be translated to Scia engineer in order model the connection right.

If all the previous mentioned connections would be made by springs, that would amount to a total of 8 springs in the connection. They would all be in the same horizontal plane however, so a transfer of moments would not be possible this way. If the knot is built up out of two layers of springs, one on top and one on the bottom, it would be possible to transfer a moment via the connection. The series of figures below illustrates how the model in SCIA engineer came to be.

FIGURE 44 CONNECTION BETWEEN FOUR FLOOR SLABS AND THE COLUMNS

Figure 45 shows a schematic of this connection where the column has been removed, leaving only the steel plate of the column, which is important for the connection.

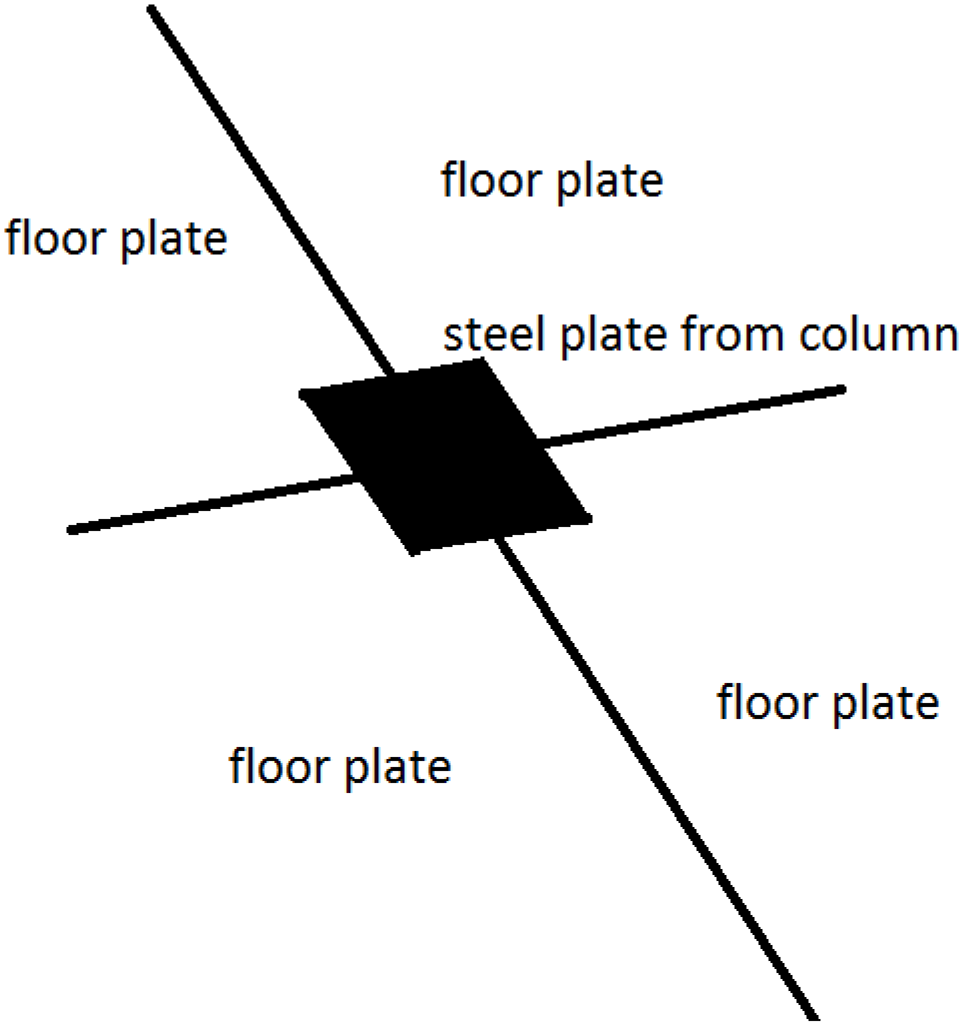


FIGURE 45 WITH THE COLUMN REMOVED, THE FLOORPLATES WITH THE STEELPLATE

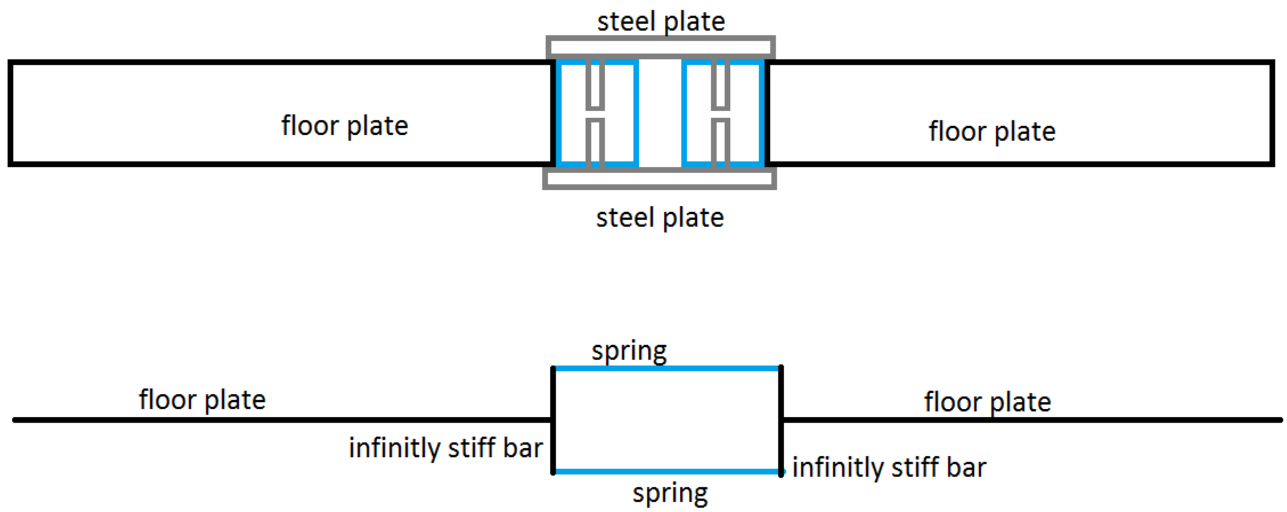


FIGURE 46 THE CONNECTION VIEWED FROM THE SIDE, WITH THE TRANSLATION STEP TO THE MODEL BELOW

Figure 46 describes the step from reality to model. The floorplate will be modeled as a plate. The two floorplates in the picture above are connected via the corner shoe, the pins and the steel plate. The stiffness of the corner shoe to the steel plate connection has been calculated and that is what needs to be captured in the model. This is achieved by replacing the corner shoe, the pins and the steel plate by a single spring. Seen as there are two steel plates there should also be two springs. This way the connection can also transfer rotations. The distance between the two springs is the thickness of the floor plate, this is achieved by adding a infinitly stiff bar, that has no influence on the connection itself. Figure 47 shows the connection between all the floor plates seen in a 3D perspective with four floorplates.

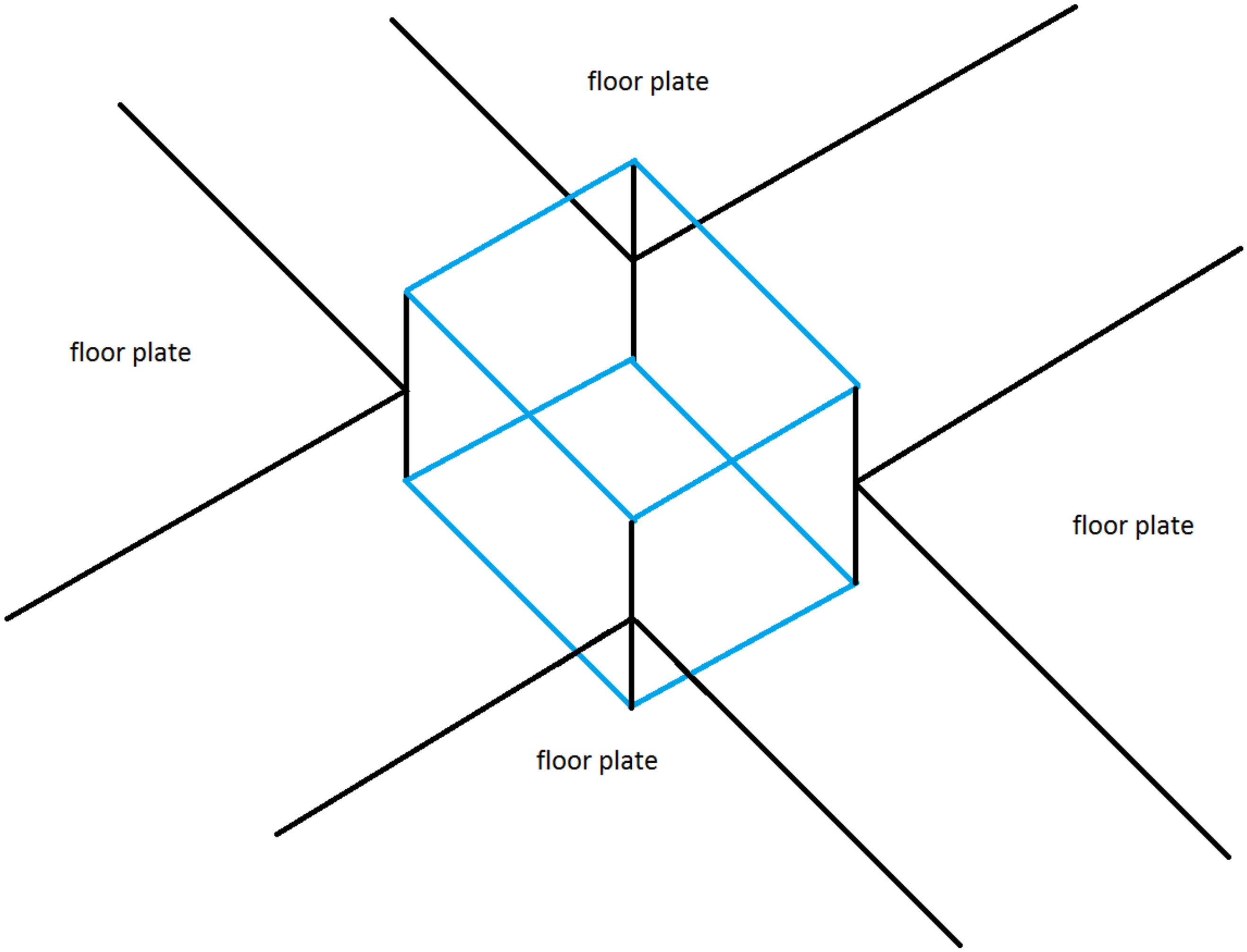


FIGURE 47 3D PLAN OF THE FLOORPLATES WITH THE SPRINGS CONNECTING THE PLATES

Now the last missing item is the connection between the column and the floorplates, in order to do this the column is drawn in the middle and via diagonal springs the connection between the infinitly stiff bars and the column is made.

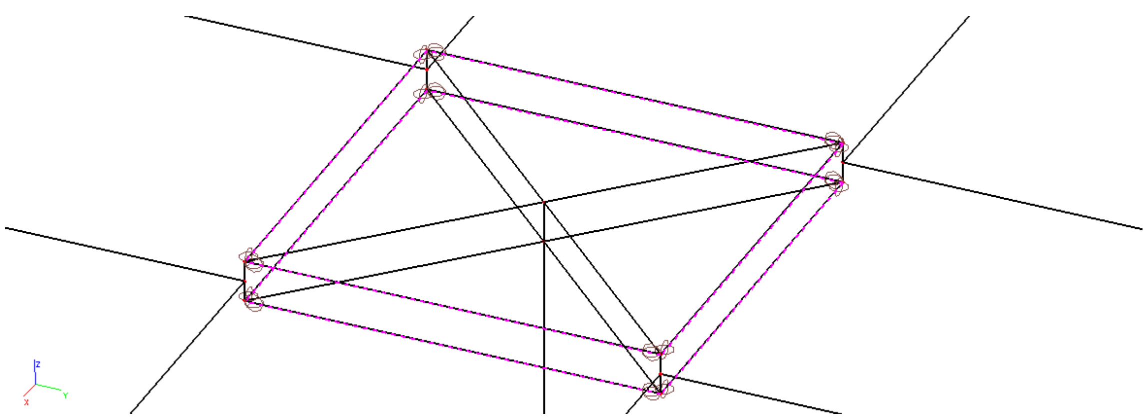


FIGURE 48: SETUP OF THE SPRINGS

Figure 48 presents a view of the final design of the connection between the columns and the floorplates. The purple springs are the springs between the plates and have hinging connections. The black lines inside the purple ones running to the center are the 8 springs that simulate the connection from the floor plate to the column. They are clamped connections in order to transfer the vertical load. This allowes the plates to move separate from each other but still transfer the horrizontal forces and deformations.

4.2. Setup of the calculation

The spring stiffness calculated in the previous paragraph will have to be implemented in Scia engineer. In order to do this a small beam is applied where the dimensions with corresponding EA create the spring properties. The E is the Young's modulus of the material that is used for the spring and A the surface area of the beam element. When multiplied they will have the same stiffness that is calculated, thus forming the spring.

The springs can be made in Scia Engineer using a numerical cross section. When the material is set to steel, giving an E of 210000 N/mm^2, dividing the stiffness by E leads to the right value for A.

spring type	N/mm	A [mm2]	D [mm]
Upper Kx	1.16E+07	55.47	8.404281
Upper Ky	1.55E+07	73.99	9.705935
Lower Kx	2.63E+07	125.38	12.63479
Lower Ky	3.92E+07	186.52	15.41062
E	2.10E+05		

TABLE 13: CALCULATION OF THE AREA REQUIRED FOR THE SPRING

The Y direction is the short side and the X direction is the long side of the floorplate. The short side of the plate has 3 bars and the long side has 2 bars. The first step has been made for the connections, but this is only half of the needed information. The thickness of the spring that is going to run to the middle has yet to be determined.

The values stated above are for the connections between one single corner shoe and two pens. The connection between the shoe and the column will always be double because there are two corner shoes and 4 pens between the two plates. That means that the spring mechanisms are in a series so the extension becomes twice as much as the values in table 13.

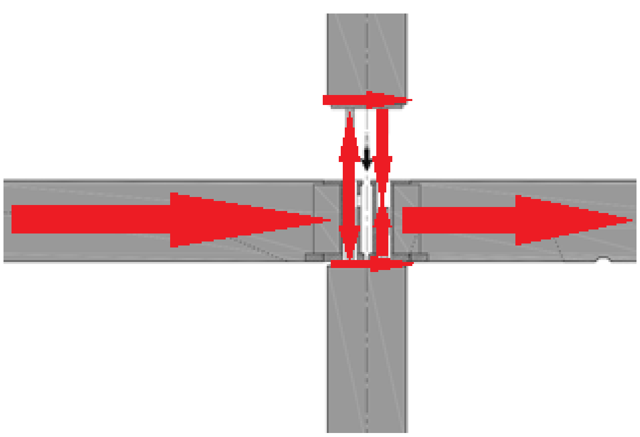


FIGURE 49 PLATES AND THE COLUMN

4.3. Translating the Diana values to spring stiffnesses.

In order to make the translation from Diana to Scia succeed, it has been chosen to make the two different stiffnesses, the one in X direction and the one in Y direction, to one that is equal for both directions. Since the stiffness depends on the pin that is placed upon the column plate this is an assumption that can be presumed correct.

The two spring stiffnesses topside were 5.800.000 N/mm and 7.750.000 N/mm, those were the S/2 values that resulted from the earlier analysis. These values need to be added up and divided by 2, giving the following value as a spring stiffness in horizontal direction:

$$\frac{5.800.00 + 7.750.000}{2} = 6.775.000\ N/mm$$

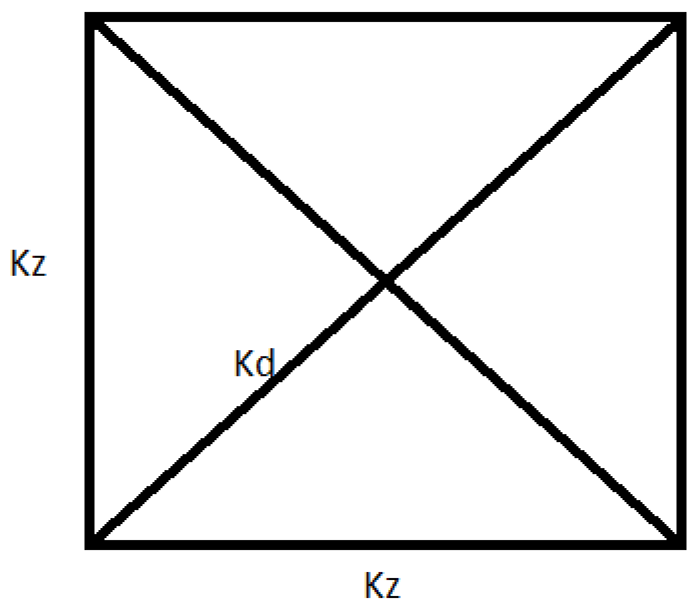


FIGURE 50: SCHEMATICS OF THE SPRING SYSTEM TOPSIDE

In order to start testing, a starting position is required. The first assumption is that half of the force is transferred from one plate to the other via the direct spring and that the other half of the force is transferred via the column with the diagonal springs. In order to make an estimate we assume that the diagonal spring is √2 times Kz.

That means that the starting values for Kz and Kd are:

Kz: 3.387.500 N/mm
Kd: 4.790.648 N/mm

In order to see if the spring set is correct two checks are performed:

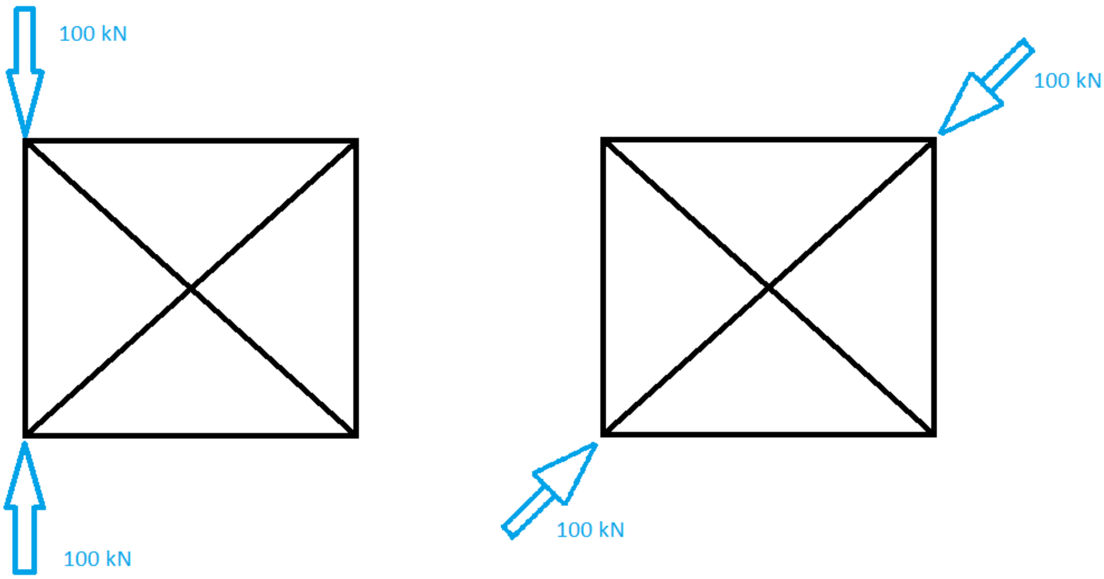


FIGURE 51: THE TWO TEST TO CHECK THE SPRING STIFFNESSSE

According to the earlier assumption that the main determining factor is the pin on the column, the displacement should be the same horizontal and diagonal.

The displacement should be

$$\frac{200.000\ N}{6.775.000\ N/mm} * 20\ mm = 0.59\ mm$$

The calculations are being made in scia engineer where a model with the schematics of figure 51 is being implemented. The displacement of the spring can be measured by looking at the displacement of the points relative to each other. The extension of the spring is presented below:

Kz: 0.4995 mm
Kd: 0.4446 mm

This means that both springs are too stiff right now and need to be adjusted in order to reach the right stiffnesses. Using an Iterative process is the best way to achieve the right values. By dividing 0.590 by the values that were presented above, a step in the right direction could be made. The spring stiffness will then be divided by the obtained number thus reaching the new spring stiffness. This process has been repeated several times until the displacement was accurate up to 2 decimals.

The achieved stiffnesses are the following:

Kz: 2.926.363 N/mm
Kd: 3.541.778 N/mm

That means the relation between the springs can be achieved and for the lower springs all that needs to be done is checking if the displacement values are correct.

4.4. Setup of the second spring set

The relation between the measured stiffness S and the Kz and Kd values can be seen in the table below. The first column of values is explained by the previous column. The second column of values represents the ratio to which the original acquired stiffness needs to be multiplied with, in order to achieve the correct spring values.

Upper spring set:						
	Stiffness:	6775000				
	Kz:	2926363	0.431935			
	Kd:	3541778	0.522772			
Lower Spring set:						
	Stiffness:	16375000				
	Kz:	7072944				
	Kd:	8560386				

TABLE 14: CALCULATION SECOND SET OF SPRINGS

This assumption can not stay an assumption and needs to be tested in order to check the validity of this assumption.

The extension of the spring should be

$$\frac{200.000\ N}{16.375.000\ N/mm} * 20\ mm = 0.24\ mm$$

The real displacements are once again calculated with the use of Scia Engineer.

Kz: 0.11919*2(adjustment for amount of springs) = 0.23838
Kd: 0.08577*2(adjustment for amount of springs)*√2(adjustment for length) = 0.24

The calculated values are within the safety margin of two decimals accuracy, so the ratio was correct and the entire spring system has now been set.

4.5. Improved model

The first checks were promising, but a more accurate Diana model can be designed if an interface were to be used between the steel pin and the concrete inside the corner shoe. The interface separates the corner shoe from the plate on the column. This means that the element primarily holding the corner shoe in place is the connection of the two pins with the corner shoe. The displacements will get higher and become more realistic than it was in the previous model.

The new values are presented in the table below where DtX, DtY and DtZ are the displacements in set directions. The Kx, Ky and Kz values are the corresponding stiffnesses.:

Load	Vert(kN	Hor(kN)	DtX [mm]	DtY [mm]	DtZ [mm]		Kx	Ky	Ez
Load case X	0	50	1.58E-02	-1.45E-03	-3.88E-03		3.16E+06	-3.44E+07	-1.29E+07
	"	100	3.16E-02	-2.91E-03	-7.76E-03		3.16E+06	-3.44E+07	-1.29E+07
	"	200	6.32E-02	-5.81E-03	-1.55E-02		3.16E+06	-3.44E+07	-1.29E+07
	100	50	-9.78E-02	-9.22E-02	-2.04E+00		-5.11E+05	-5.42E+05	-2.45E+04
	"	100	-8.30E-02	-9.36E-02	-2.05E+00		-1.21E+06	-1.07E+06	-4.89E+04
	"	200	-5.14E-02	-9.65E-02	-2.05E+00		-3.89E+06	-2.07E+06	-9.75E+04
Load case y	0	50	4.99E-02	-3.76E-02	-4.32E-03		1.00E+06	-1.33E+06	-1.16E+07
	"	100	9.97E-02	-7.52E-02	-8.64E-03		1.00E+06	-1.33E+06	-1.16E+07
	"	200	1.99E-01	-1.50E-01	-1.73E-02		1.00E+06	-1.33E+06	-1.16E+07
	100	50	-6.47E-02	-1.28E-01	-2.04E+00		-7.72E+05	-3.90E+05	-2.45E+04
	"	100	-1.49E-02	-1.66E-01	-2.05E+00		-6.72E+06	-6.03E+05	-4.89E+04
	"	200	8.48E-02	-2.41E-01	-2.05E+00		2.36E+06	-8.30E+05	-9.74E+04
Load case X	0	50	1.43E-02	2.51E-03	4.32E-04		3.49E+06	1.99E+07	1.16E+08
	"	100	2.86E-02	5.02E-03	8.63E-04		3.49E+06	1.99E+07	1.16E+08
	"	200	5.73E-02	1.00E-02	1.73E-03		3.49E+06	1.99E+07	1.16E+08
	100	50	-1.79E-01	-1.62E-01	-3.23E-01		-2.79E+05	-3.08E+05	-1.55E+05
	"	100	-1.66E-01	-1.60E-01	-3.22E-01		-6.04E+05	-6.25E+05	-3.10E+05
	"	200	-1.37E-01	-1.55E-01	-3.22E-01		-1.46E+06	-1.29E+06	-6.22E+05
Load case Y	0	50	5.94E-02	-4.22E-02	1.14E-04		8.41E+05	-1.19E+06	4.41E+08
	"	100	1.19E-01	-8.43E-02	2.27E-04		8.42E+05	-1.19E+06	4.40E+08
	"	200	2.38E-01	-1.69E-01	4.54E-04		8.41E+05	-1.19E+06	4.40E+08
	100	50	-1.35E-01	-2.07E-01	-3.23E-01		-3.70E+05	-2.41E+05	-1.55E+05
	"	100	-7.55E-02	-2.49E-01	-3.23E-01		-1.32E+06	-4.01E+05	-3.10E+05
	"	200	4.33E-02	-3.34E-01	-3.23E-01		4.62E+06	-6.00E+05	-6.20E+05

TABLE 15: DISPLACEMENTS AND CORRESPONDING STIFFNESS

The values from the model with the interface plane, result in stiffnesses with a factor 10 smaller than the factor without the interface plane. The translation to Scia engineer will be made using the new values. That means the factor that has been achieved in the previous section can be applied to get the right data for the new spring values.

S/2 was the required value, and that has to be the average for the X and Y spring direction:

$$Upper\ value\colon \frac{3.160.000 + 1.330.000}{4} = 1.122.500\ N/mm$$

$$Lower\ value\colon \frac{3.490.000 + 1.190.000}{4} = 1.170.000\ N/mm$$

	Stiffness:	1122500	Factor:		A:	D
	Kz:	484847.6	0.431935		2.308798	1.714542
	Kd:	586811.2	0.522772		2.794339	1.88623
Lower Spring set:						
	Stiffness:	1170000				
	Kz:	505364.5			2.406498	1.750442
	Kd:	611642.8			2.912585	1.925725
			E:	210000		

TABLE 16: CALCULATION OF THE SPRING VALUES

The stiffnessess and the corresponding thicknesses for the spring units are the resulting values that will be used. The plasticity of the system will be incorporated into the system via a plasticity factor in the earthquake model in scia engineer. This factor is the ratio between the point at which the connection will become plastic and the point where the construction reaches its yield strain.

The factor, in this case, depends on the connection between the column and the floorplate. The model used to calculate the spring stiffness of the connection, is used again to calculate this factor. When looking at the model used to calculate the spring stiffness changes can be made in order to attain the needed information. However, the information was mixed up and the stresses exceeded the yield strength. Clearly the model had given the wrong results which means something is wrong with the model.

After adapting and re-meshing the model the changes did not make the model converge. When looking at the results of the model there is a clear indicator of why the model failed. The analysis was force induced so a force was applied to the anchors of the corner shoe. This force increases over time and the displacements are measured per step. However, if there should be plasticity the model can completely deform in a single step, that is what the results indicate.

In order to counter this and to visualize the plastic phase a deformation induced analysis will be performed. This analysis deforms the ends of the anchor which creates stresses and strains. This way the model will not yield instantly when the moment of plasticity is reached. But the strain will increase and this will give the plasticity part of the stress strain model. Several types of analysis, mesh types and other options were used and via trial and error the final result is value based on the results from the latest analysis using the von misses stresses and strains.

In figure 52 the behavior of the entire connection is shown where there are two lines both showing behavior of the connection. The red line displays the behavior of the connection if everything would display elastic behavior. The blue line shows the plastic behavior of the connection the way it would react in reality with one exception, there is no tear strain. This means that the material can stretch to infinity. It is however sufficient for the study and to gain the information needed from the model. Appendix 9.3 Force-displacement table shows how this graph has been constructed. The analysis is displacement induced, which means the forces follow as a reaction to this displacement. The displacement is given at the anchors and the corner shoe is being held in place. The graph is created by reading the stress inside the anchors and multiplying this with the surface area of the anchors, thus creating three forces. Adding these forces leads to finding the total reaction force in het model. The same thing has been done for the elastic representation but since the forces are far larger this had to be interpolated. This model also shows the maximum force that can be transferred via the connection: 108 kN.

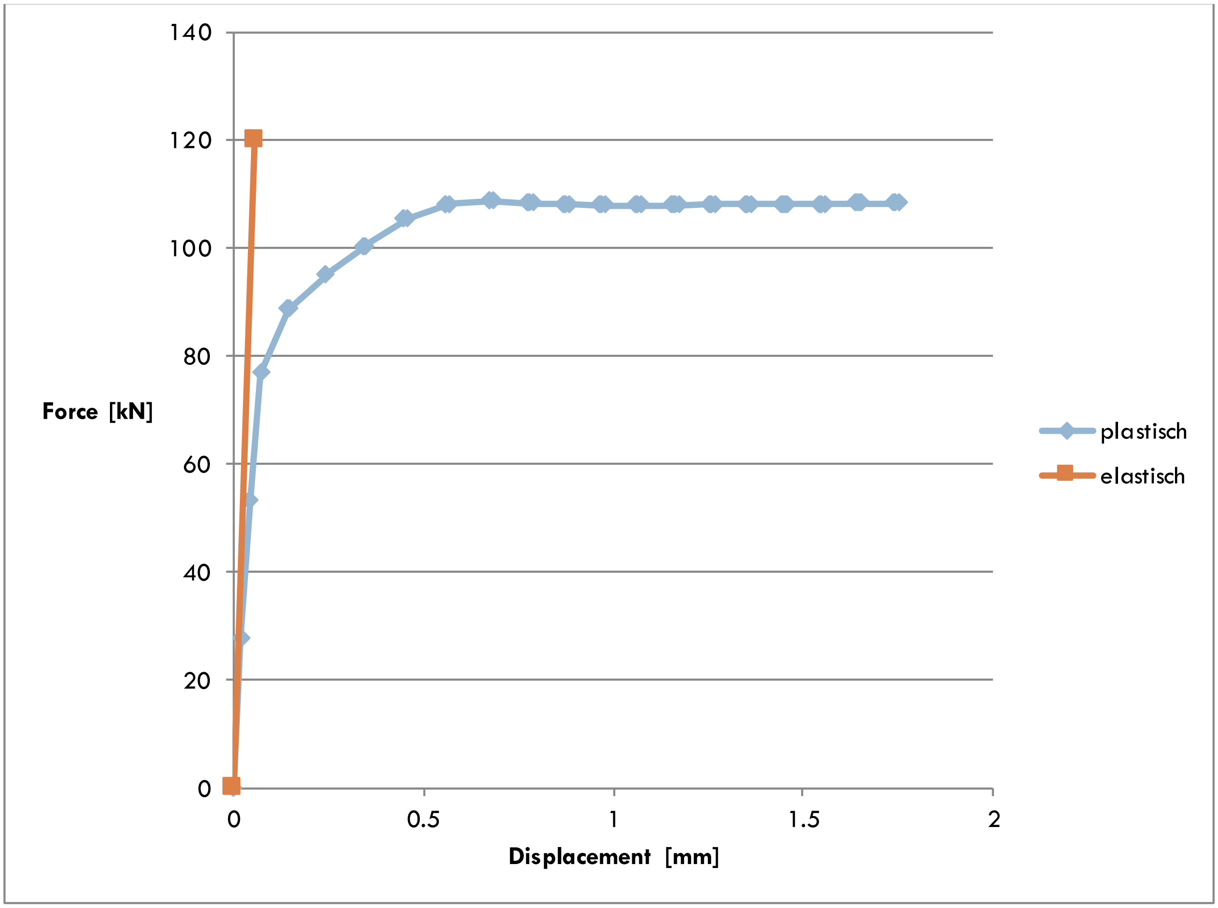


FIGURE 52 ELASTIC AND PLASTIC DEFORMATION OF THE CONNECTION

In order to check the values of the model, it is compared to a pair of bolts. A M20 grade 4.6 has a maximum tensile capacity of 47 kN per bolt. Two bolts brings that total to 94 kN. That means the answers found in the model are plausible and the model functions correctly.

The plastic phases can be recognized by looking at the model per time step. Three of those are lined up underneath showing the different phases of the loading, deforming and finally failing of the connection. As has been addressed before, the model will not break because the materials can stretch to infinity.

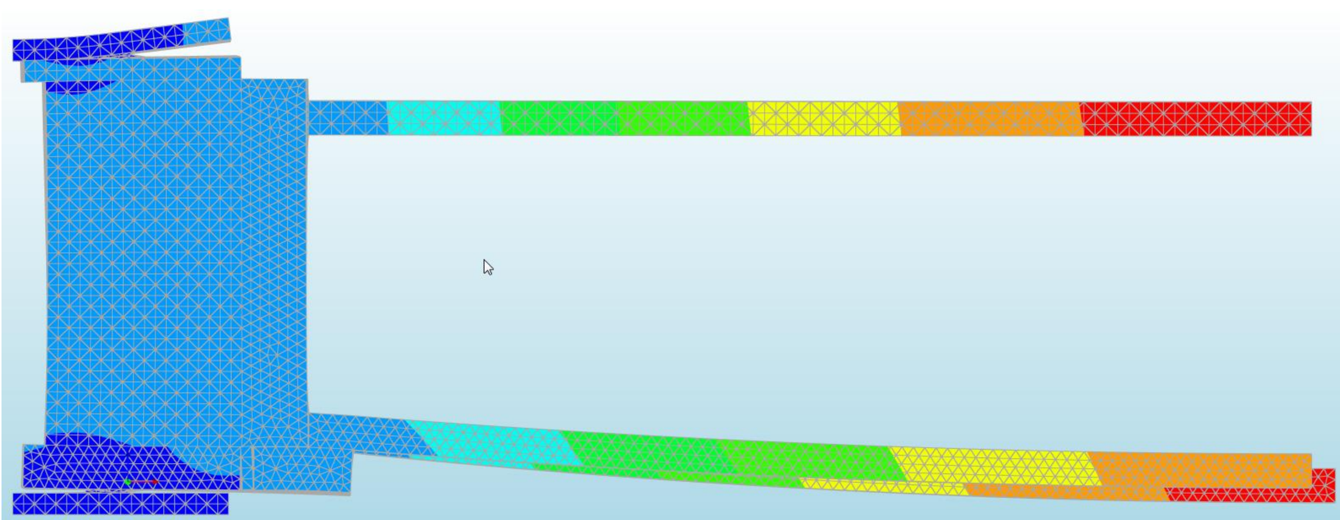


FIGURE 53 FIRST PART A SMALL DEFORMATION IS FOUND NO PLASTICITY YET, THE DEFORMATION LOOKS LARGE BUT IS SMALL, IT IS ENLARGED IN THE PICTURE TO MAKE IT VISIBLE

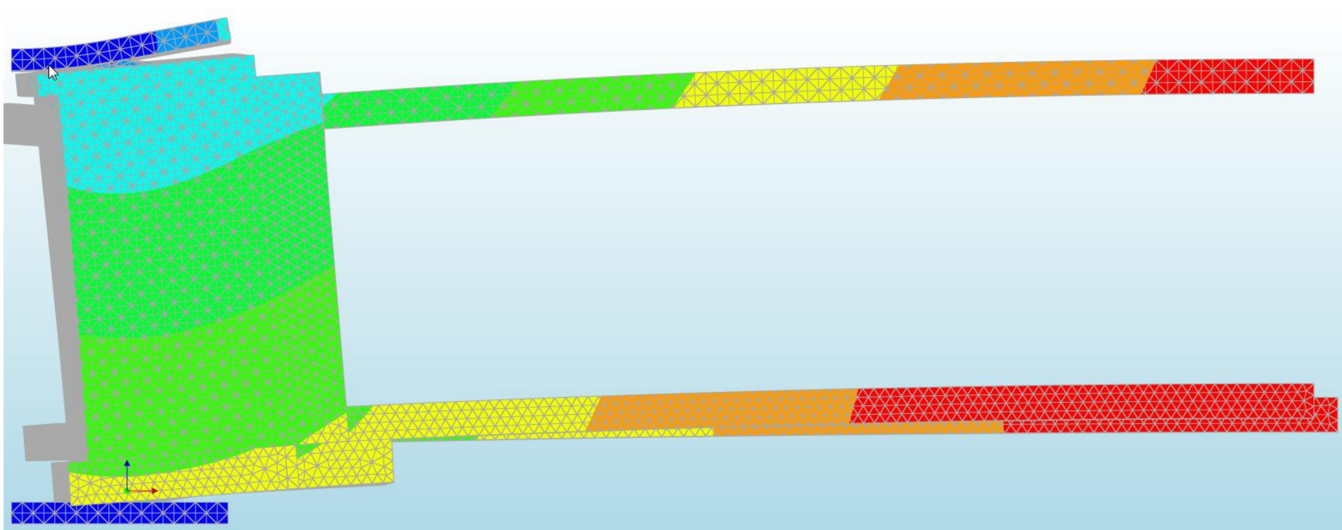


FIGURE 54 THE BOTTOM PIN IS STARTING TO DEFORM AND SINCE THERE ARE TWO ANCHORS THERE THE DEFORMATION SHOULD BE LOWER AT THE BOTTOM

FIGURE 55 THERE PINS HAVE BOTH BECOME COMPLETELY PLASTIC AND THE PINS HAVE FAILED

Based on the ductility of the connection, a behavior factor q can be deducted. This factor allows that the ductility of the structure can be accounted for in the elastic calculation. The formulas to calculate the behavior factor from the ductility are presented below.

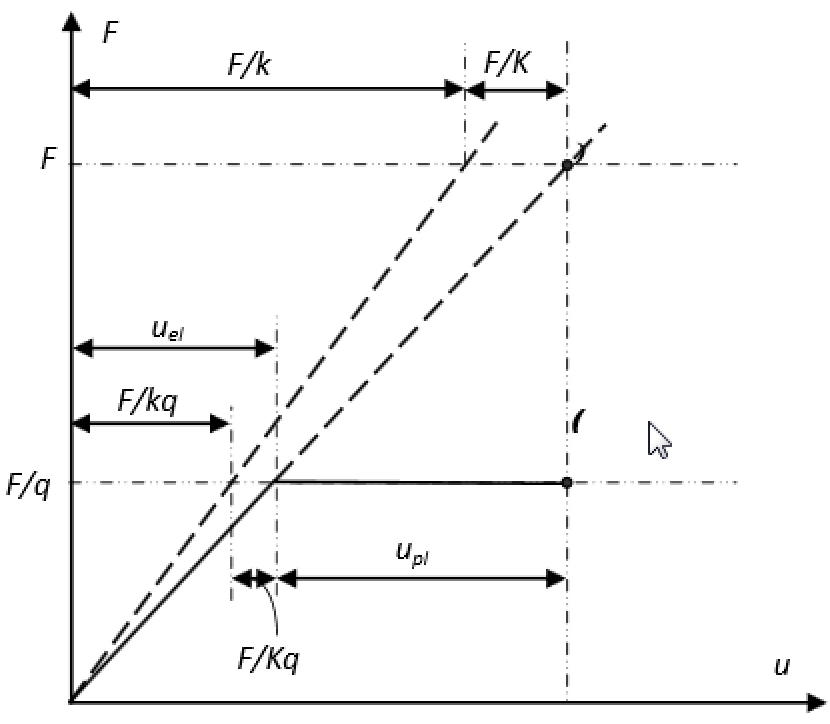


FIGURE 56: EXPLANATION OF FACTORS IN THE FORMULA

$$\mu = \frac{u_{pl}}{F/Kq}$$

$$q = \sqrt{(2\mu - 1)}$$

The formulas come from the NPR 9998 paragraph 5.14. In order to calculate the behavior factor the other values need to be deduced from the graph in figure 58. The points A and B need no further explanation, however point C is not clear. Point C states where the plasticity of the entire connection has reached its limit.

This limit is reached when both pens reach a material strain of 19%, which is the strain limit for S235 steel. The lower pen reaches this limit before the upper pen does, which makes the upper pen the indicator for the limit on the plastic strain.

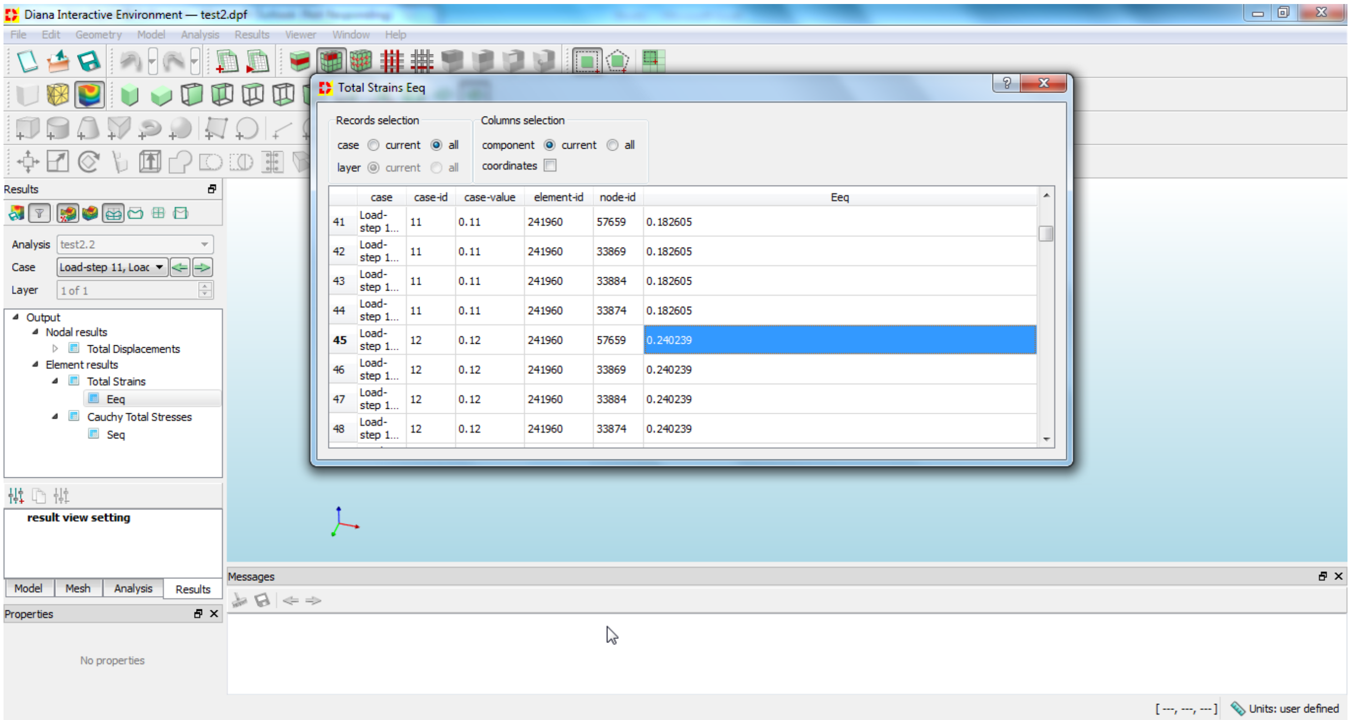


FIGURE 57 STRAIN OF THE UPPER PEN

Figure 57 shows that from step 11 to step 12 of the plastic calculation of the Diana model the strain increases from 18% to 24 %. The maximum is in between those two values, Interpolation gives the value of 19% at a deformation of 0.919 mm.

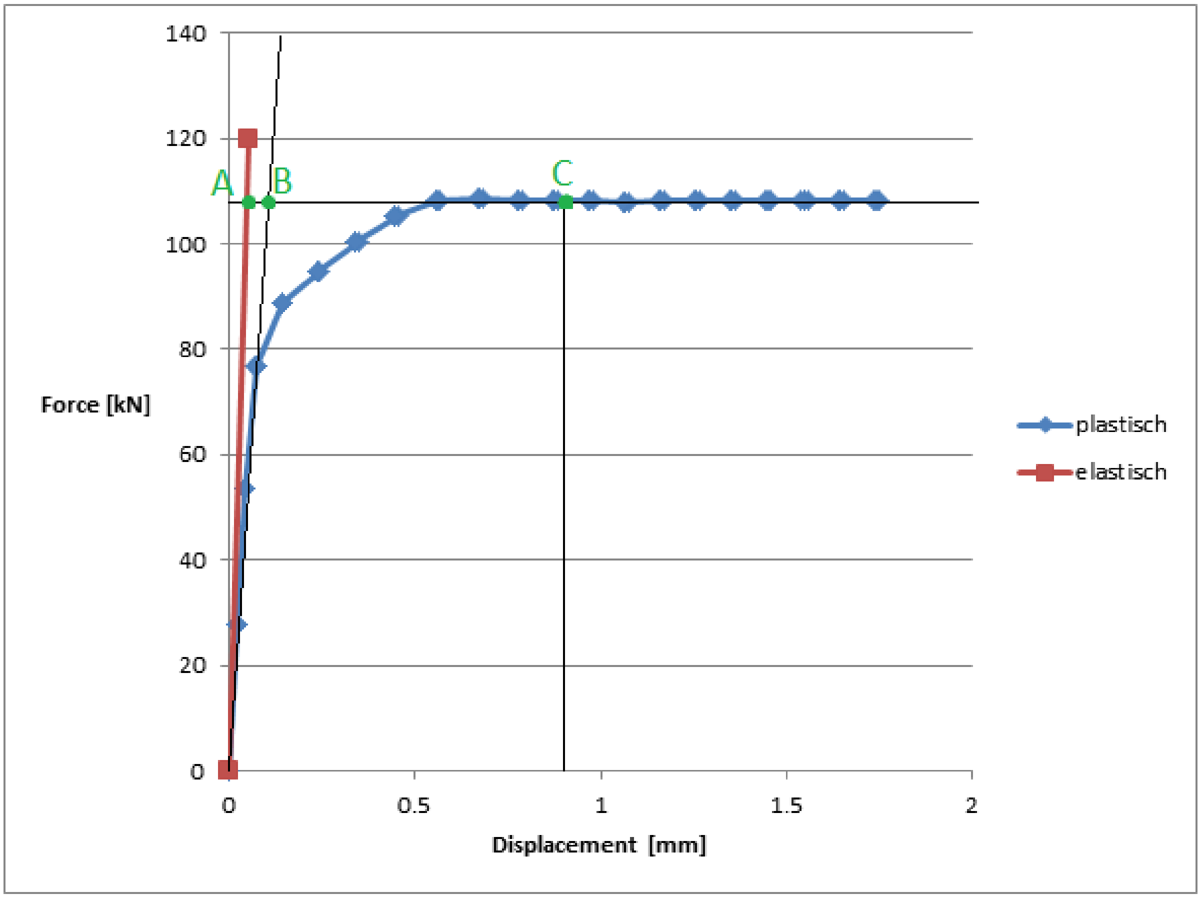


FIGURE 58: THE FIGURE BELONGING TO THIS CASE

$$\mu = \frac{u_{pl}}{\frac{F}{Kq}} = \frac{C - B}{B - A} = \frac{0.919 - 0.101}{0.101 - 0.055} = \frac{0.818}{0.046} = 17.8$$

$$q = \sqrt{(2\mu - 1)} = \sqrt{(2 * 17.8 - 1)} = \sqrt{34.6} = 5.88$$

There are a couple of remarks that need to be made regarding this q factor. The first one is that the behavior factor is a factor for the entire system and all that has been reviewed so far is the characteristic connection between the columns and the floor slabs. The sway mechanism of the connections is the swaying of the pin. This is a metal pin and therefore a value near six is correct The only way to justify this is by assuming that the rest of the system will not show plastic behavior. Since this is all concrete this assumption is quite accurate.

The second remark is concerning the so called: “low cycle” fatigue. This concept means that the earthquake loading is a dynamic loading and that means a difference in loading. The loading caused by the earthquake comes in the form of a spectrum, which means the structure is pushed into its plastic spectrum more than once. The low cycle fatigue deals with the fact that when this happens repeatedly the construction loses strength and will fail even though the maximum strain has not been exceeded at all.

5. BUILDING MODEL IN SCIA ENGINEER

For the second model another program is required to test the entire system on earthquake loading. Scia engineer is the program used by RHDHV to model buildings that need to be tested on earthquake loading. This program allows for quick modeling of buildings and elements. The model will be made in 3D but the elements will not be in true 3D. This means that the elements are modeled along a plane or line and the line is given properties like a thickness, shape and material properties. The model will look like just a line but this merely indicates the position of the center of the element and where it connects to.

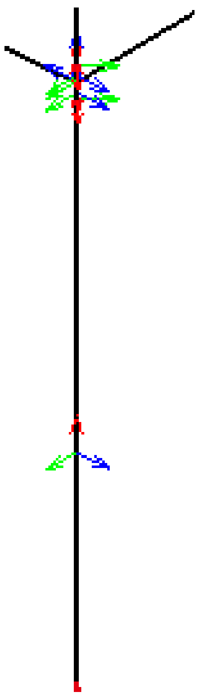


FIGURE 59 EXAMPLE OF HOW A COLUMN LOOKS LIKE IN SCIA ENGINEER

It is important to understand how this program applies the earthquake load, because the type of earthquake and the soil conditions will be specific to this region. These parameters need to be adjusted in order to calculate the right kind of response from the building.

5.1. The model

As has been previously discussed, a model of the entire building needs to be setup. This model will be constructed in the program Scia engineer, because it is used by Royal Haskoning DHV for similar calculations and therefore the needed expertise is present with the structural engineers of RHDHV.

For the model a suitable project had to be chosen to serve as a starting point of a real CD20 building. The then latest built building with the traditional CD20 Building system has been chosen; the Stork Building in Hengelo. This is a three story high building consisting of traditional 20x20 cm thick columns and the standard 20 cm thick floor plates.

In the images below the real building and the model as it looks like in Scia Engineer are shown. As is clearly visible, only the elements that have a constructive importance are modeled and thus the model looks like a stripped version of the real building. However, in order to make the model calculable this is exactly what was needed.

FIGURE 60: PICTURE OF THE REAL BUILDING

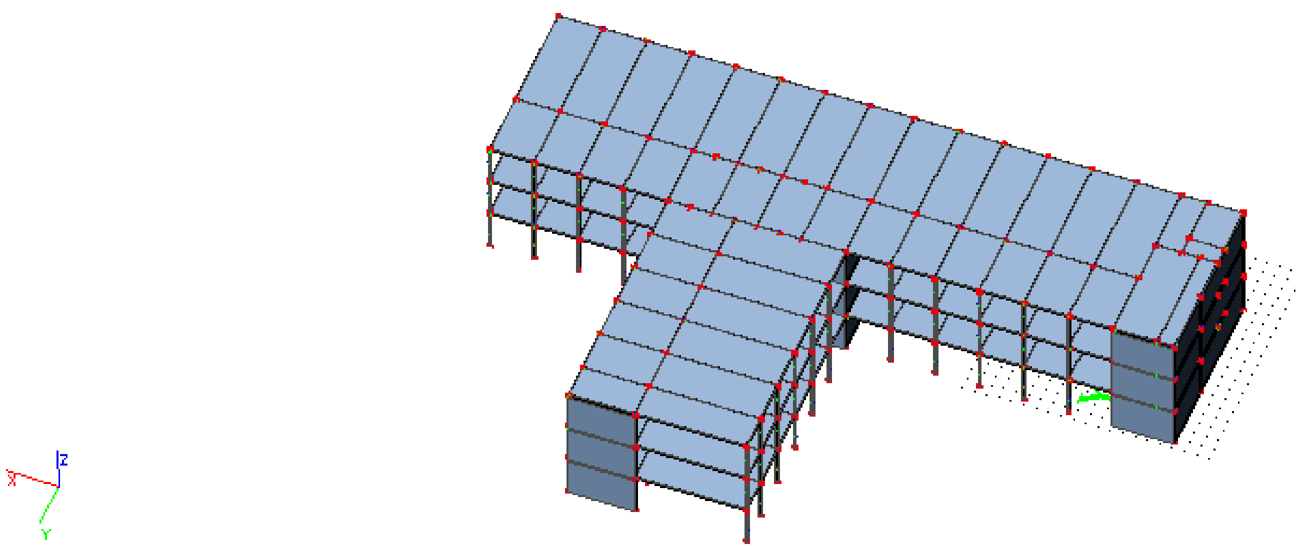


FIGURE 61: MODEL OF THE BUILDING

As can be seen in the Scia Model presented where, only those elements that are of structural importance are present. The outer walls of the structure are not structurally important and their weight is represented by a load. This makes the model smaller, because the entire outer façade can be replaced by loads which take up less memory to calculate or visualize.

The standard floor loading as described by the Eurocode will be applied on the first and second floor levels. The load of the ground floor is directed to the foundation and has no effect on the rest of the building. The loading on the roof will be just a maintenance load and can be left out of the earthquake calculation.

5.2. The earthquake

In order to run checks on the building regarding earthquake loading, first it has to be determined what kind of earthquake will serve as a reference point. The project area that is to be tested is Oman, so it would only be logical to obtain a value suitable in this area. The peak value of the GPA, which stands for Ground Peak Acceleration, reaches a maximum value of 4.0 meters per second in Oman. However, this occurs only in a very small part of the country, so modeling the system after these requirements would be over-engineering. In order to prevent this, a GPA is taken that covers 90% of the country. This means that the earthquake loading on the building will be similar to an earthquake with a GPA of 2.4 meters per second.

The second important aspect is the type of building and the related safety factor for this type of building. This building is designed as an office building and as such it needs to be tested. The loading in the finite element method program Scia Engineer is the first thing that needs to be determined.

The loading is divided into 5 different load cases and they are the following:

1. Own weight
2. Permanent loads
3. Live loads
4. Earthquake X-direction
5. Earthquake Y-direction

These different forms of loading are combined into the loading that is required to test the structures capability of withstanding an earthquake. The self-weight, the resting and the variable load are self-explanatory, but the earthquake loading requires some additional information. Scia Engineer will vibrate the structure in different Eigen frequencies and the response of the structure is depending on the elastic response spectrum. The next chapter will address this part of the calculation.

The final step is to combine these loadings in the correct way, according to the code. That means that not all of the loads have to be accounted for 100% all the time. The Eurocode determines the kind of load combinations for different types of buildings. This building is characterized as an office building and will be tested as such, according to the specifications in the Eurocode.

Seismic load combination:

$$G + \gamma_2 * Q + Exy$$

Where E is the combined seismic action for both directions.

5.2.1. The response spectrum

In order to test the system on earthquake loading, a couple of choices have to be made. There are different kind of checks and different kind of earthquakes. In order to properly test the building, first a method needs to be selected and the earthquake defined. Also, different surfaces react different to an earthquake. So a couple of parameters need to be set before applying the right loading is possible

The analysis that will be performed is the response spectrum analysis, this method is used by RHDHV in their earthquake calculations. Basically the building can be seen as a single degree of freedom system. This means that there is a natural frequency at which the building can resonate. This spectrum is the response plot at the free surface of the earth. The damage that may occur if the building will resonate can be substantial. The reason this analysis is chosen is because most building codes describe this test when dealing with quakes.

Step one in this analysis method is creating the right spectrum from the circumstances that are in the target area. There are four areas to the response spectrum of the nature of the building. These four areas can all be described by different formulas. These four areas are shown and described by figure 63, where the formulas and the corresponding graph is shown. The basic values are input data that have been gathered from the literature research. The values are presented by the Boğaziçi University of Istanbul, Turkey (Deif[1] et al.2013).

Table 1.2 – Soil classification parameters

Soil Class	$\bar{v}_s$ (m/s)	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$ (kPa)
A. Hard rock	> 1500	NA	NA
B. Rock	760 – 1500	NA	NA
C. Very dense soil and soft rock	360 – 760	> 50	100
D. Stiff soil	180 – 360	15 – 50	50 – 100
E. Soft clay soil	< 180	< 15	< 50
	or any profile with more than 3 m of soil with plasticity index: $PI > 20$, moisture content: $w \geq 50\%$, undrained shear strength: $\bar{s}_u < 25$ kPa.		
F. Soils requiring site response analysis	1. Soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils 2. Peat and/or highly organic clays with more than 3 m. 3. Very high plasticity clays with more than 7.5 m and $PI > 75$ 5. Very thick, soft/medium stiff clays with more than 35 m and $s_u < 50$ kPa		

FIGURE 62: SOIL CLASSIFICATION PARAMETERS

[1] Deif,A. &I. El-Hussain & K.Al-Jabri & N. Toksoz & EL-Hady & S. Al-Hashmi & K. Al-Toubi & YAl-Shijbi & M.Al-Safi. 2013. 'Deterministic seismic hazard assessment for Sultanate of Oman' *Arabian Journal of Geosciences* 6:4947-4960

ELASTIC RESPONSE SPECTRUM

$$S_{AE}(T) = 0.4\,S_{SD} + 0.6\frac{S_{SD}}{T_o}T \qquad (T_o \le T)$$

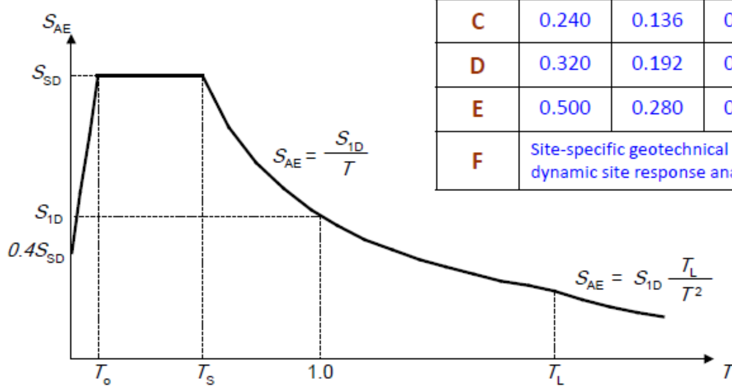
$$S_{AE}(T) = S_{SD} \qquad (T_o \le T \le T_S)$$

$$S_{AE}(T) = \frac{S_{1D}}{T} \qquad (T_S \le T \le T_L)$$

$$S_{AE}(T) = \frac{S_{1D}\,T_L}{T^2} \qquad (T_L \le T)$$

$$T_S = \frac{S_{1D}}{S_{SD}}$$

$$T_o = 0.2T_S$$



Short period and 1.0 second elastic spectral accelerations

Local Soil Class	Seismic Zone			
	Zone 1 Muscat, Sohar, Diba, Khasab		Zone 2 Nizwa, Sur, Salalah	
	S_{SD} / g	S_{1D} / g	S_{SD} / g	S_{1D} /g
A	0.160	0.064	0.080	0.032
B	0.200	0.080	0.100	0.040
C	0.240	0.136	0.120	0.068
D	0.320	0.192	0.160	0.096
E	0.500	0.280	0.250	0.140
F	Site-specific geotechnical investigation and dynamic site response analysis required			

$$T_L = 8\ \text{s}$$

FIGURE 63: ELASTIC RESPONSE SPECTRUM FOR OMAN

There are four parts to the spectra:

Part 1:	$S_{AE}(T) = 0.4 * S_{SD} + 0.6 * \frac{S_{SD}}{T_0}T$	$T_0 \le T$	$T_S = \frac{S_{1D}}{S_{SD}}$	g: 9.81 m/s
Part 2:	$S_{AE}(T) = S_{SD}$	$T_0 \le T \le T_S$	$T_0 = 0.2T_S$	
Part 3:	$S_{AE}(T) = \frac{S_{1D}}{T}$	$T_S \le T \le T_L$		
Part 4:	$S_{AE}(T) = \frac{S_{1D}T_L}{T^2}$	$T_L \le T$		

Calculating the Spectrum

			seconds
S_SD:	3.14	T_S:	0.60
S_1D:	1.88	T_0:	0.12
		T_L:	8

Local Soil Class	Seismic zone			
	Zone 1 Muscat, Sohar, Diba, Khasab		Zone 2 Nizwa, Sur, Salalah	
	S_{SD}/g	S_{1D}/g	S_{SD}/g	S_{1D}/g
A	0.160	0.064	0.080	0.032
B	0.200	0.080	0.100	0.040
C	0.240	0.136	0.120	0.068
D	0.320	0.192	0.160	0.096
E	0.500	0.280	0.250	0.140
F	Site specific tests and analysis required			

FIGURE 64: FORMULAS FOR THE CALCULATIONS OF THE ELASTIC RESPONSE SPECTRUM

Figure 64 shows the situation that will be used in this research. The values are from Zone 1 with local soil class: D. This presents the S and the T values that are needed to calculate the different zones and values that comprise the spectrum. Filling out these formulas provides a list of coordinates as the result. These values are set into Scia Engineer where they form the elastic response spectrum.

The second part of the calculation is that Scia Engineer calculates the Eigen frequency of the building, in this case the Stork building. This is the natural frequency at which the building vibrates. After this is done the total frequencies are combined by taking the square root of the squares of both the values, thus receiving an acceleration. This acceleration is multiplied by the weight to gain representative loads on the building. These loads are placed on the building and from this point on it is a normal calculation.

The calculation needs to include a plasticity factor. This is a factor that accounts for the plasticity in the joints in the earthquake calculation. The earthquake calculation is a linear elastic calculation and thus the plasticity will not be taken into account via the calculation mechanism. The plasticity factor is a factor that influences the earthquake loading on the building. When we take a look at the Eurocode concerning earthquake loading we find the following formulas:

<u>Design spectrum for elastic analysis</u>

For the horizontal components of the seismic action the design spectrum, $S_d(T)$, shall be defined by the following expressions:

$$0 \leq T \leq T_b : S_d(T) = a_g * S * \left[\frac{2}{3} + \frac{T}{T_b} * \left(\frac{2.5}{q} - \frac{2}{3}\right)\right]$$

$$T_b \leq T \leq Tc : S_d(T) = a_g * S * \frac{2.5}{q}$$

$$T_c \leq T \leq T_d : S_d(T) \begin{cases} = a_g * S * \frac{2.5}{q} * \left[\frac{Tc}{T}\right] \\ \geq \beta * a_g \end{cases}$$

$$T_d \leq T \leq 4s : S_d(T) \begin{cases} = a_g * S * \frac{2.5}{q} * \left[\frac{TcTd}{T^2}\right] \\ \geq \beta * a_g \end{cases}$$

where:

a_g	is the design ground acceleration on the type A ground $\left(a_g = \gamma 1 * a_{gR}\right)$;
T	is the vibration period of a linear single degree-of-freedom system;
S	is the soil factor
Td	is the value defining the beginning of the constant displacement response range of the spectrum;
$S_d(T)$	is the design spectrum;
q	is the behavior factor;
β	is the lower bound factor for the horizontal design spectrum.

These formulas result in a response spectrum that will be used in order to calculate the buildings reaction on an earthquake. In order to calculate the spectrum an excel file has been setup to make adjustments to the values easier for future use. Note that the values in the excel file correspond with the final form of the spectrum, the reduction factor q has been changed from 6 to 3, an explanation follows in paragraph 5.4.

1						
2	0 < T < Tb			1/T	T	Sd
3				#DEEL/0!	0	2.88
4			$Sd = ag*S*\left[\frac{2}{3}+\frac{T}{Tb}*\left(\frac{2,5}{q}-\frac{2}{3}\right)\right]$	50	0.02	3.024
5				25	0.04	3.168
6	Tb<T<Tc			16.66667	0.06	3.312
7				12.5	0.08	3.456
8			$Sd = ag*S*\frac{2,5}{q}$	10	0.1	3.6
9				4	0.25	3.6
10	Tc<T<Td			3.846154	0.26	3.461538
11			$Sd = ag*S*\frac{2,5}{q}*$[Tc/T]	3.703704	0.27	3.333333
12				3.571429	0.28	3.214286
13				3.448276	0.29	3.103448
14	Td<T			3.333333	0.3	3
15			$Sd = ag*S*\frac{2,5}{q}*$[Tc*Td/T^2]	2.5	0.4	1.6875
16				2	0.5	1.08
17				1.666667	0.6	0.75
18				1.428571	0.7	0.55102
19	ag	2.4		1.25	0.8	0.421875
20	S	1.8		1.111111	0.9	0.333333
21	T			1	1	0.27
22	Tb	0.1		0.909091	1.1	0.22314
23	Tc	0.25		0.833333	1.2	0.1875
24	Td	0.3		0.769231	1.3	0.159763
25	q	3		0.714286	1.4	0.137755
26				0.666667	1.5	0.12
27				0.625	1.6	0.105469
28				0.588235	1.7	0.093426
29				0.555556	1.8	0.083333
30				0.526316	1.9	0.074792
31				0.5	2	0.0675
32						

FIGURE 65 EXCEL OF FORMULAS THAT CREATE THE RESPONSE SPECTRUM

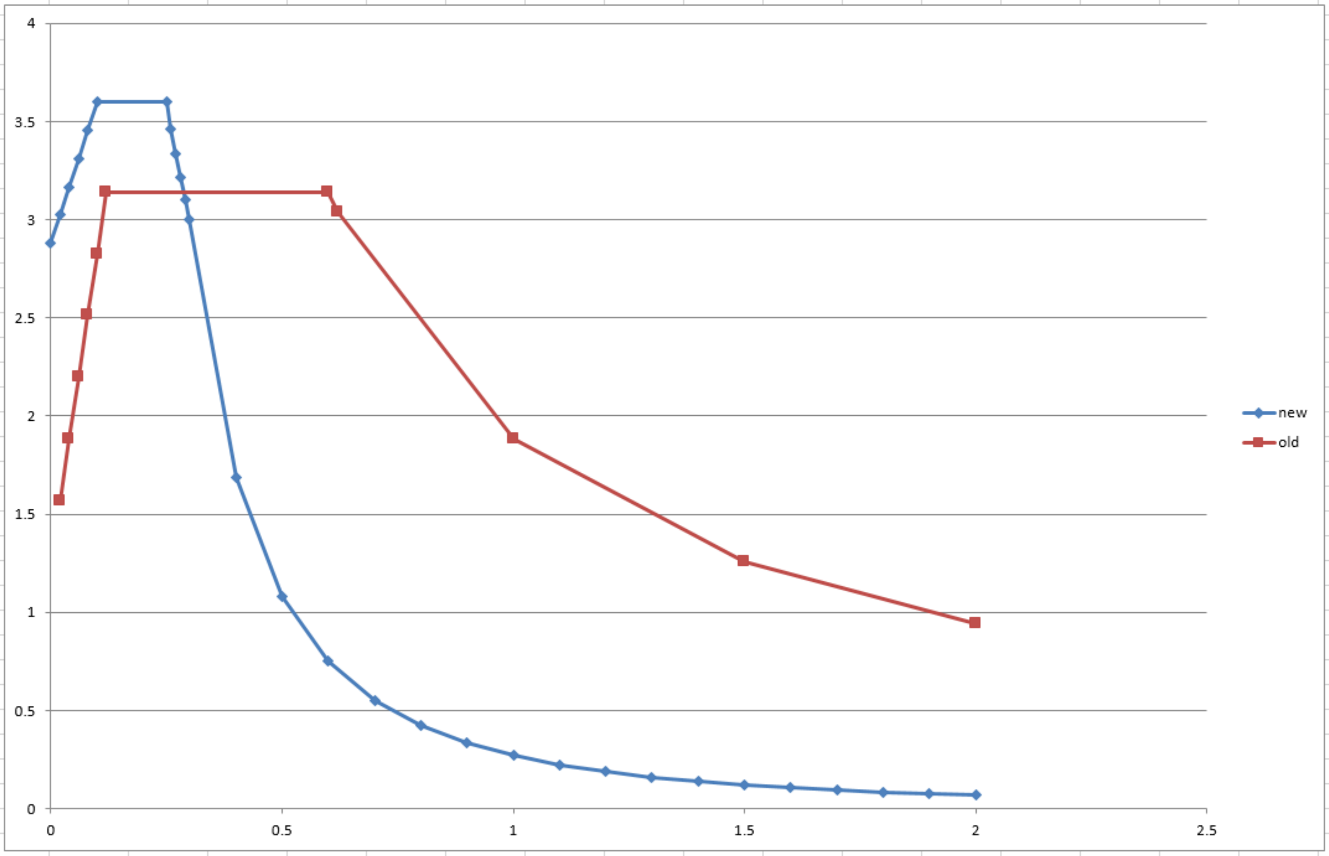


FIGURE 66 RESPONSE SEPECTRA, THE BLUE ONE CORRESPONDS WITH THE GIVE NUMBERS IN THE PREVIOUS IMAGE

The spectrum according to the recent values is presented in figure 67. This spectrum looks nothing like the spectra from the theory or the two lines indicated above. This is due to the fact that a very large plasticity factor has been used which created a very large dampening effect in the response factor. At a later stage this factor will be reviewed.

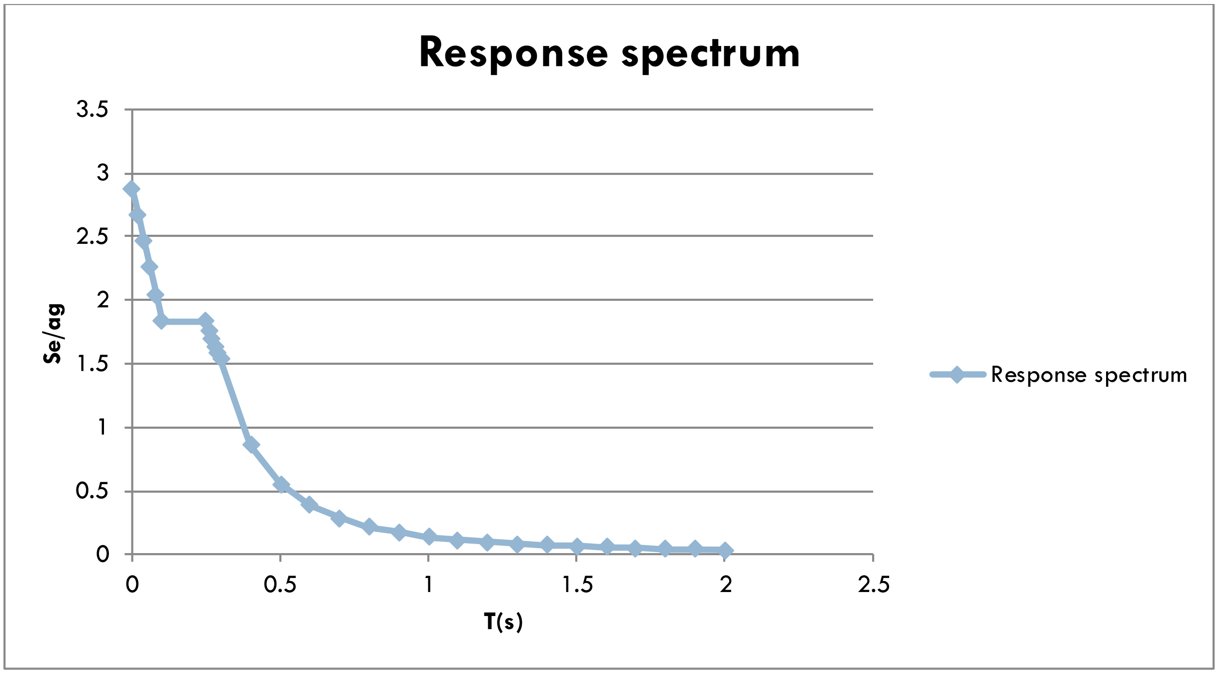


FIGURE 67: RESPONS SPECTRUM

The behavior factor has been derived at an earlier stage and has influenced the response spectrum. The factor, in this case of almost six, makes for a significant decrease of the forces that will load the building due to the earthquake.

5.3. The analysis

The method used to test the building on an earthquake loading is the response spectrum analysis. This means that a spectrum of accelerations is applied on the construction at ground level. Scia Engineer will then calculate the forces at the Eigen frequencies of the building with the spectrum. The Forces are calculated by multiplying the acceleration from the spectrum with the masses from the building.

5.3.1. Results

The stork building has been calculated under earthquake loading as has been set in the previous paragraphs. This leads to a number of loads that can be used to check the individual members of the construction. These loads can be divided into two different categories, the reaction forces and the lateral forces in the elements.

Of course the one is a result of the other and therefore they are bound. However the paths the loads follow to the supports can differ and thus the loading of different parts can differ. The connections, the elements and the support connections all have a maximum amount of force they can withstand before failing. It must be checked what element or elements fail and in which direction. Therefore these three aspects need to be assessed separately.

The connections:

The largest horizontal forces can be found in the connections, the maximum force reaches -394 to 329 Kn/m in the general Y-direction and from -280 to 280 Kn/m in the general X-direction. The forces are given as a line load over the pens of the connection. The line loads need to be multiplied with the length of the pin (0,1m) to find the loads. In this case that means 39.4 kN as a maximum in one direction and 28 kN in another direction. The results in Scia Engineer are presented in such a way that separate values are given for the horizontal directions. Reasonably, a resulting force needs to be calculated from these different values in order to present a total force on the connection element.

However, when examining the file after the calculations and assessing the way the forces are being transferred to the soil, a problem arises and the results have to be dismissed. The earlier assumption was that the system would function as a dual system. The walled system would be supported by the frame system. However, when taking a look at the results the frame system is hardly participating in the transferring of the forces to the foundation and almost all of the forces are being transferred by the walled system. An adaptation is needed in order to simulate the way the system functions and such is presented in the next chapter.

5.4. Adaptation

After careful examination of the force flow the current results had to be corrected. The behavior factor in the previous calculation is used under the assumption that the rest of the system is a frame system that stays elastic. However, the rest of the system is not just a frame system, stabilizing parts of the construction are in large part the stabilization walls. This means the frame system, the mechanism that could develop and would allow for the high behavior factor, is not the leading stabilization method.

This means the q factor is too large and needs to be adjusted, a new value has to be set. The value that was calculated in the previous chapter correlates with a steel structure. In this case a closer look at concrete systems is what is needed. In order to do this first the kind of structural system that is actually in place needs to be evaluated. When taking a look at the Eurocode 8 about earth quakes 4 different categories come up which cover most of the systems.

Table 5.1: Basic value of the behaviour factor, q_o, for systems regular in elevation

STRUCTURAL TYPE	DCM	DCH
Frame system, dual system, coupled wall system	$3,0\alpha_u/\alpha_1$	$4,5\alpha_u/\alpha_1$
Uncoupled wall system	3,0	$4,0\alpha_u/\alpha_1$
Torsionally flexible system	2,0	3,0
Inverted pendulum system	1,5	2,0

FIGURE 68: TABLE FROM THE EUROCODE

When evaluating Stork Hengelo it clearly gives not one but two different systems. The main CD20 system that consists of columns and floor slabs can be defined as a frame system where plastic hinges can form. The second system is the wall system that actually is the key system in transferring the horizontal loads to the foundation. Since there are two systems it is fair to assume this building falls in dual-system category. When looking up the definition of the dual system in the Eurocode it reads:

dual system

Structural system in which support for the vertical loads is mainly provided by a spatialframe and resistance to lateral loads is contributed to in part by the frame system and in part by structural walls, coupled or uncoupled.

However, this scheme covers DCM(Ductility Class Medium) and DCH (Ductility class High) structures. Seen as this system is primarily made out of precast concrete elements, it is wise to take a look at what the Eurocode says about concrete systems and precast concrete systems.

When looking at the concrete section in the Eurocode one point that stands out considering this system is the following:

Systems of large lightly reinforced walls cannot rely on energy dissipation in plastic hinges and so should be designed as DCM structures.

Seen as that an inverted pendulum system has a q factor of 1.5 and the same holds for masonry buildings, a factor of 3.0, so not taking any further cutbacks from the joints into account, is a valid assumption and that is the behavior factor that will be used to calculate the entire construction again. Moreover the building basically is a wall system, when considering the stability, since over 90% of the horizontal forces are transferred to the foundation via the wall system. Therefore the behavior factor should be that of a wall system, which holds true for the value of 3.0 that will be used from now on.

The spectrum changes due to adaptation of the behavior factor, leading to the following diagram:

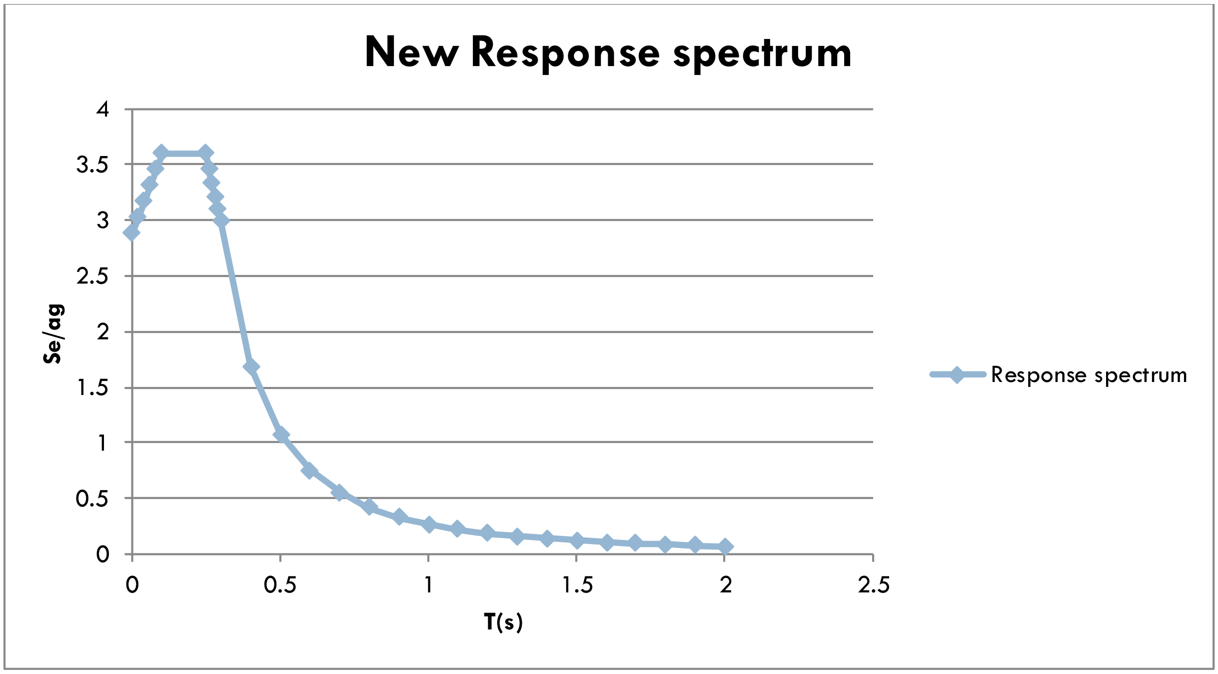


FIGURE 69: THE NEW ADAPTED RESPONSE SPECTRUM

The new response spectrum obviously gives far larger values, since the spectrum dampens a lot less. At the same time the observation made in the previous chapter is very important and a different way of attaining the maximum force needs to be used. The testing of the system will need to take place in the same manner as before, namely by checking the connections, the elements and the supports. The figures below show input of the spectrum in Scia Engineer and the resulting strains over the floor loading.

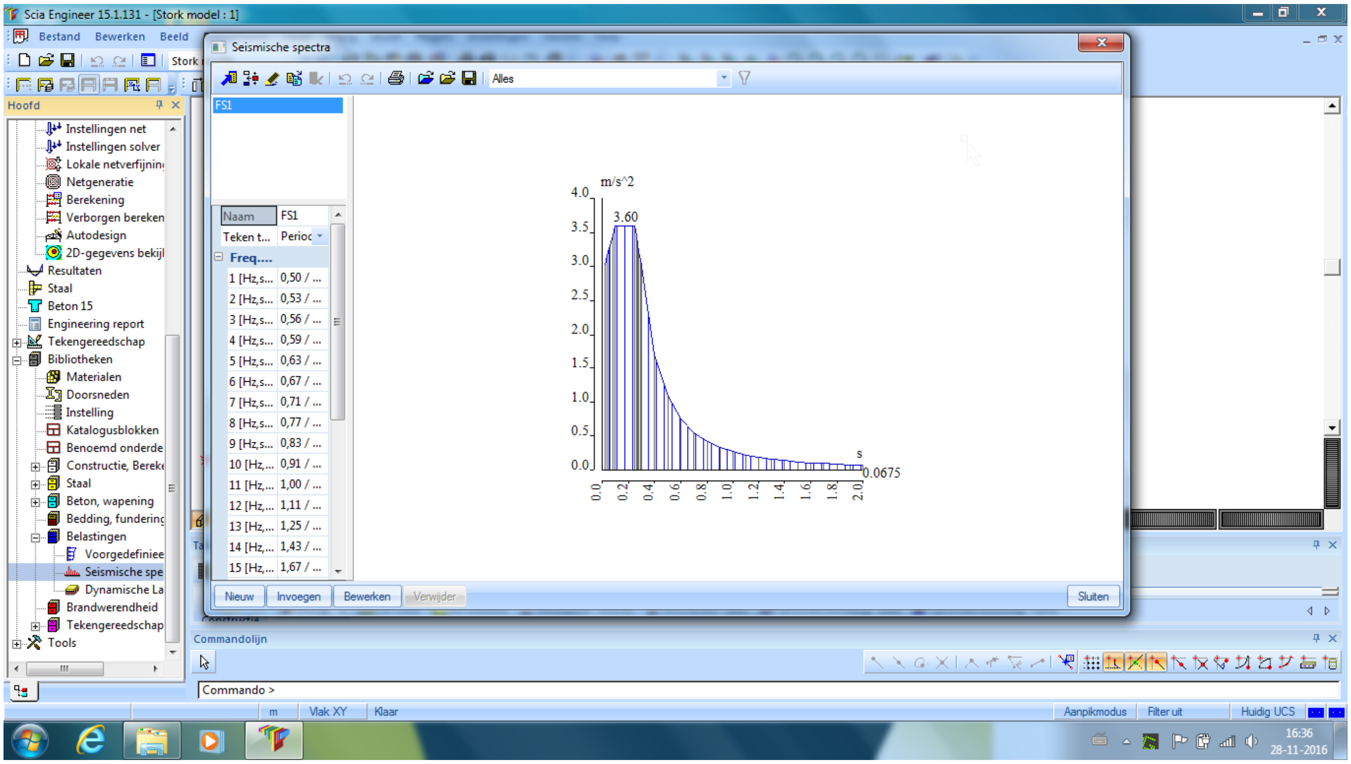


FIGURE 70 SCIA EARTHQUAKE RESPONS SPECTRUM

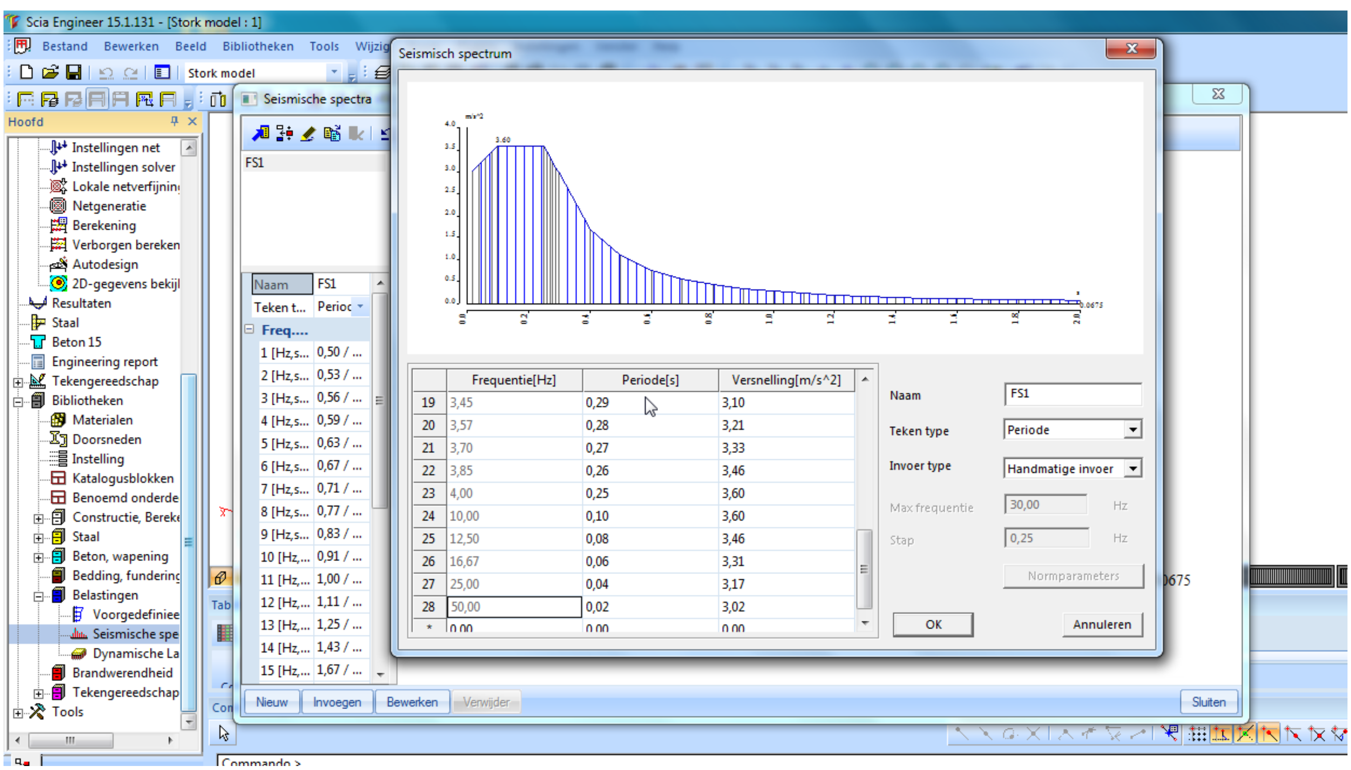


FIGURE 71 INPUT OF THE SPECTRUM IN SCIA

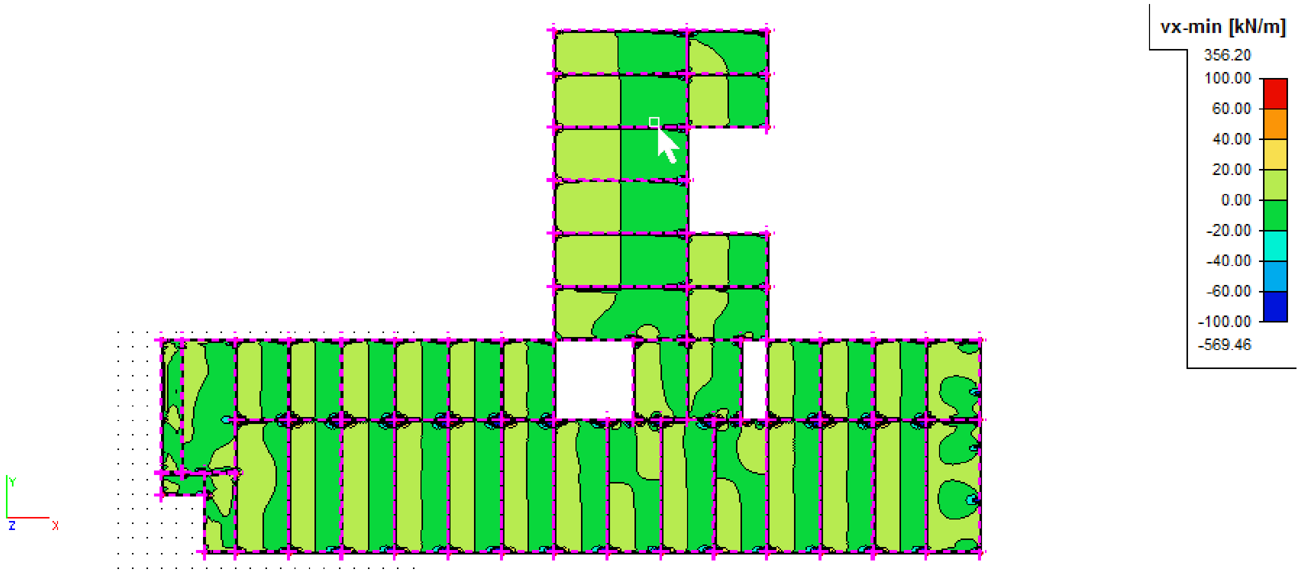


FIGURE 72 EARTHQUAKE FORCES ON FLOOR IN X DIRECTION

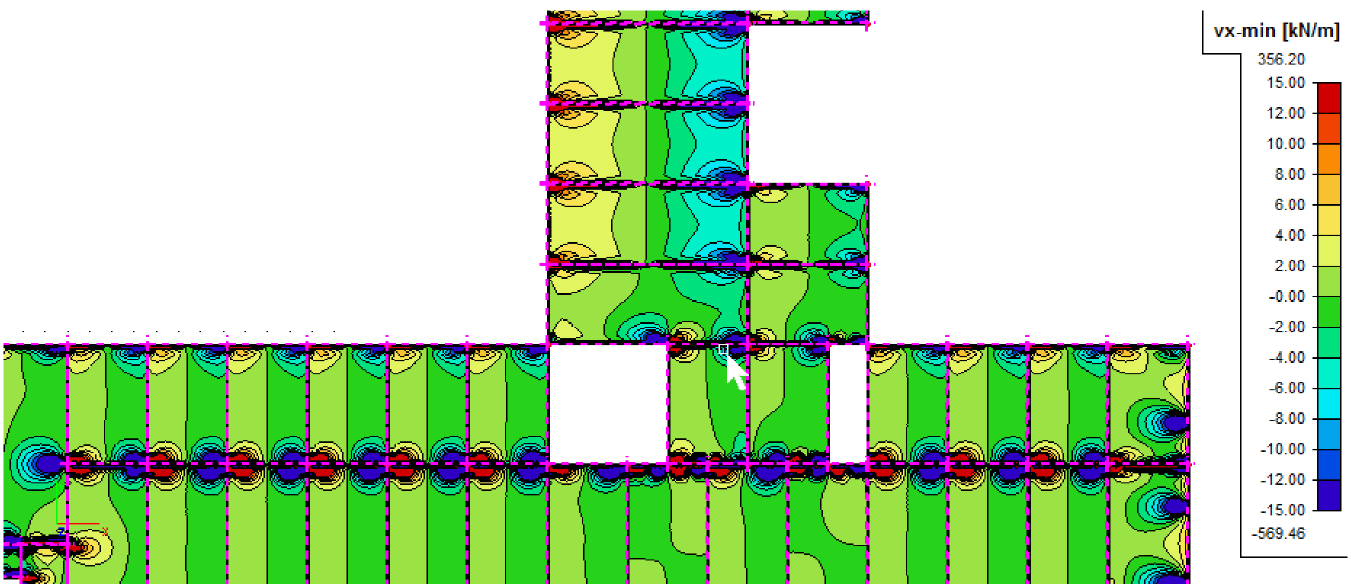


FIGURE 73 EARTHQUAKE FORCES IN X DIRECTION ZOOMED IN

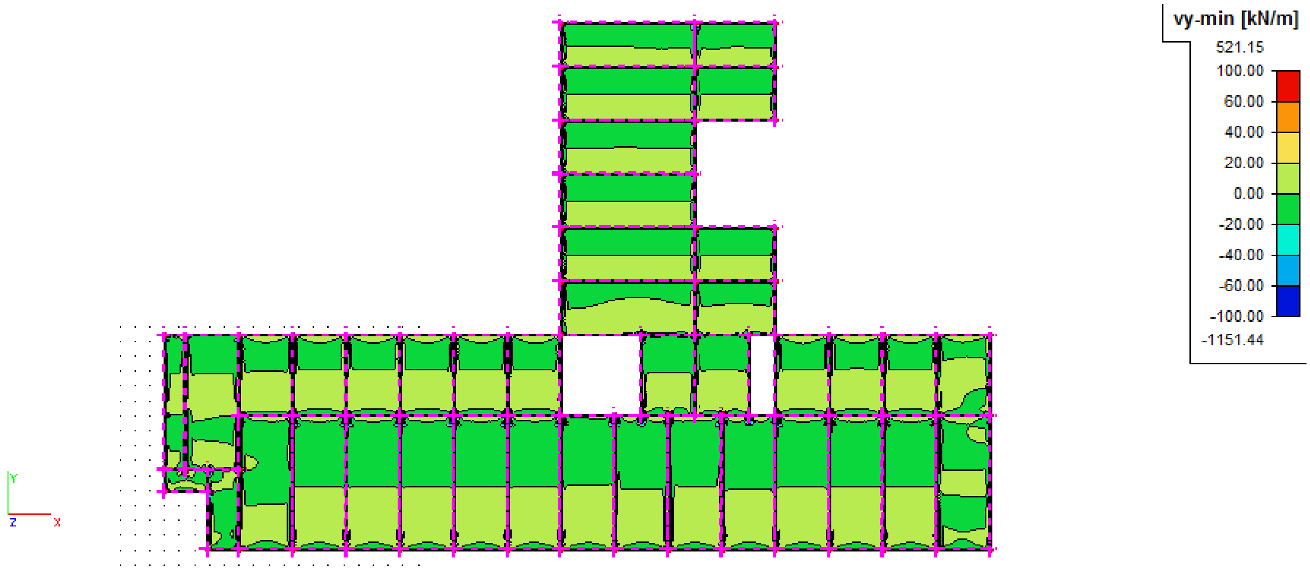


FIGURE 74 EARTHQUAKE FORCES ON FLOOR IN Y DIRECTION

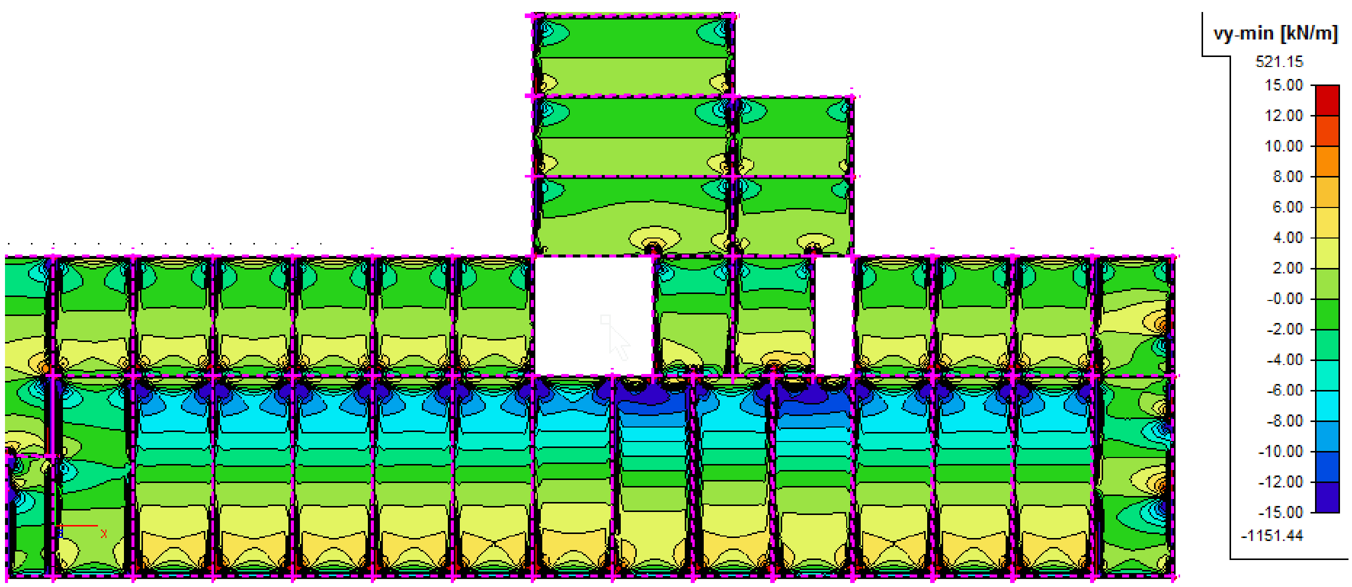


FIGURE 75 EARTHQUAKE FORCES ON FLOOR IN Y DIRECTION ZOOMED IN

In order to check the elements first a table output is made that gives the forces on the pins in the connections as a line load. These are then transferred to loads and the final step is transferring them to the resulting forces. The forces are in X and Y directions, so a resulting load has to be compiled first. When the final resulting forces have been calculated, they can be compared to one another and the maximum force can be determined. The maximum forces on a connection pin amounts to: 66,9 kN, which exceeds the maximum force the pin can handle by (66.9 - 108/2 =) 12.9 kN. That is a 24 % exceedance of the maximum force per pin. Appendix 9.3 Force displacement table shows the capacity of the connection.

This indicates that the system, as it is right now, will fail in this kind of earthquake environment, but a further assessment could prove useful so the other aspects will still be evaluated. If all the other elements and the supports do suffice than the governing aspect is clearly the connections. The different elements need to be checked in order to confirm that they have sufficient capacity.

5.4.1. The column:

The columns of the original building have been calculated by using a table with design strengths of the different columns, figure 76. There are four different scenarios for the column, ranging from 1 corner loaded to all four of the corners loaded by floor slabs. The relevant column lengths have been highlighted in figure 76. The maximum normal force found in the columns reads 236 kN. Only the eccentrically loaded column, loaded on 1 point, with the lowest rebar configuration can not withstand this. Thus it is important to know what rebar is present in the columns. The scia calculation shows that there are a number of columns reaching a normal force higher than 220 kN. However none of these are corner columns, eccentrically loaded on one pin. The capacity of the columns loaded on two pins is higher than the maximum load found in the scia calculation. The moment capacity of the columns has been calculated as well and amounts to 36.2 kNm the column capacity is twice that of the loading on the column, so it is safe to say that the columns are not a problem with the current earthquake.

Overzicht draagkracht kolommen 200x200:

A. Excentrisch belaste kolommen **1-zijdig opgelegd**

Normale omstandigheden: UGT

Kolomlengte	4ϕ12	8ϕ12	4ϕ16 + 4ϕ12	8ϕ16
l = 2800 mm	257	381	450	497
l = 3000 mm	250	368	436	483
l = 3200 mm	238	357	422	472
l = 3500 mm	222	335	405	453
l = 3900 mm	201	306	376	427

Brandsituatie: Quasi-blijvende combinatie

60 minuten brandwerendheid ; kniklengte = 0,5L				
Kolomlengte	4ϕ12	8ϕ12	4ϕ16 + 4ϕ12	8ϕ16
l = 2800 mm	113	195	249	290
l = 3000 mm	109	192	244	284
l = 3200 mm	106	185	237	278
l = 3500 mm	103	178	228	267
l = 3900 mm	96	167	216	252

Beton C53/65
Dekking 25 mm

ey = 50 mm
ez = 50 mm

B. Excentrisch belaste kolommen **2-zijdig opgelegd**

Normale omstandigheden: UGT

Kolomlengte	4ϕ12	8ϕ12	4ϕ16 + 4ϕ12	8ϕ16
l = 2800 mm	563	665	741	797
l = 3000 mm	529	642	713	771
l = 3200 mm	506	618	687	746
l = 3500 mm	467	584	653	707
l = 3900 mm	405	542	609	663

Brandsituatie: Quasi-blijvende combinatie

60 minuten brandwerendheid ; kniklengte = 0,5L				
Kolomlengte	4ϕ12	8ϕ12	4ϕ16 + 4ϕ12	8ϕ16
l = 2800 mm	298	401	458	504
l = 3000 mm	282	390	448	492
l = 3200 mm	270	379	435	480
l = 3500 mm	251	356	419	462
l = 3900 mm	226	327	394	441

Beton C53/65
Dekking 25 mm

ey = 50 mm

ez = 0 mm

Overzicht draagkracht kolommen 200x200:

C. Excentrisch belaste kolommen **3-zijdig opgelegd**

Normale omstandigheden: UGT

Kolomlengte	4ϕ12	8ϕ12	4ϕ16 + 4ϕ12	8ϕ16
l = 2800 mm	812	917	997	1072
l = 3000 mm	775	890	972	1037
l = 3200 mm	732	844	924	993
l = 3500 mm	673	782	860	929
l = 3900 mm	595	712	790	855

Brandsituatie: Quasi-blijvende combinatie

60 minuten brandwerendheid ; kniklengte = 0,5L				
Kolomlengte	4ϕ12	8ϕ12	4ϕ16 + 4ϕ12	8ϕ16
l = 2800 mm	490	574	634	684
l = 3000 mm	469	556	615	663
l = 3200 mm	446	538	595	644
l = 3500 mm	405	513	569	615
l = 3900 mm	359	467	536	581

Beton C53/65
Dekking 25 mm

ey = 17 mm

ez = 17 mm

D. Centrisch belaste kolommen **4-zijdig opgelegd (et ≥ 20 mm)**

Normale omstandigheden: UGT

Kolomlengte	4ϕ12	8ϕ12	4ϕ16 + 4ϕ12	8ϕ16
l = 2800 mm	1054	1180	1280	1378
l = 3000 mm	1031	1157	1254	1342
l = 3200 mm	1005	1129	1217	1315
l = 3500 mm	958	1082	1175	1261
l = 3900 mm	904	1019	1104	1186

Brandsituatie: Quasi-blijvende combinatie

60 minuten brandwerendheid ; kniklengte = 0,5L				
Kolomlengte	4ϕ12	8ϕ12	4ϕ16 + 4ϕ12	8ϕ16
l = 2800 mm	708	794	856	914
l = 3000 mm	694	780	843	900
l = 3200 mm	682	766	827	885
l = 3500 mm	662	744	806	863
l = 3900 mm	630	714	774	829

Beton C53/65
Staal B500B

ey = 0 mm
ez = 0 mm

FIGURE 76 COLUMN DESIGN STRENGHT

5.4.2. The support wall:

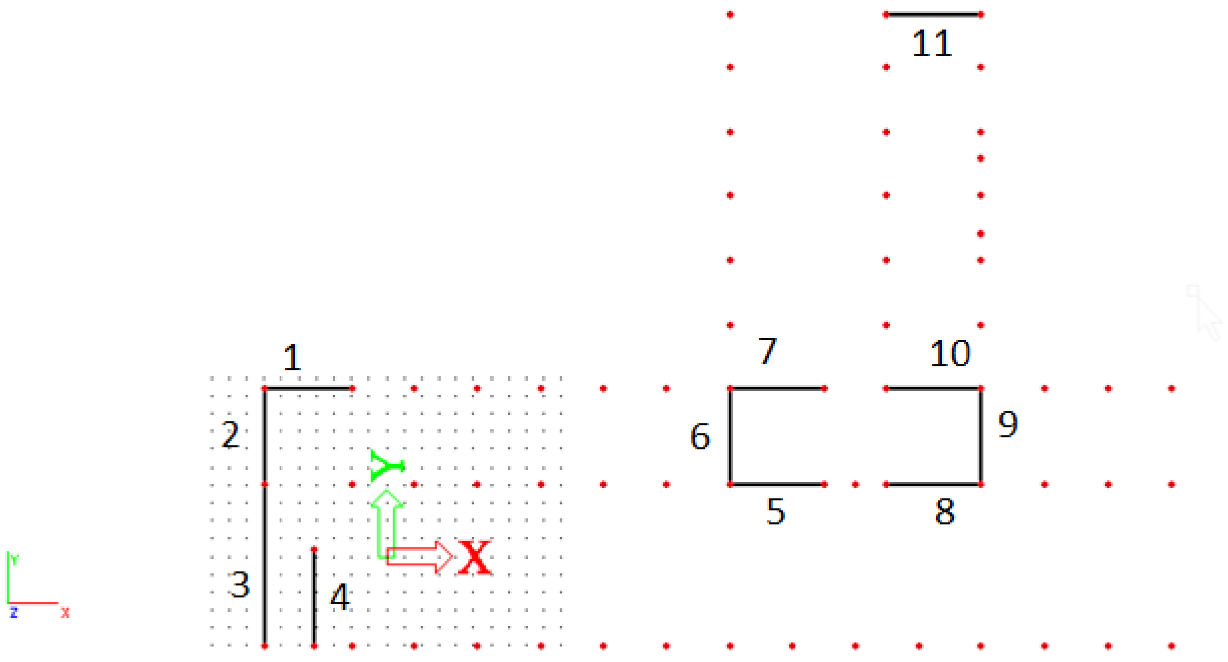


FIGURE 77 WALL NUMBERS

There are 11 walls in the building that transfer the loads to the foundation. Of these walls only the highest loaded walls need to be checked. The highest loading can be found in wall number 5

Vertical loading:

The largest vertical loading is 1084 kN, this is to be transferred by the support wall to the foundation. The standard reinforcement of the walls is a net of 150-ϕ6 and 2 ϕ12 along the sides. Just like the original checking of the walls all the forces will be calculated through 0,6 meter of the wall. In order to check the cross section the program Construct from Technosoft is used. appendix 9.5, technosoft calculation, show the cross sectional properties of this part of the wall. The figures show that the wall is capable of withstanding these forces.

Horizontal loading:

The horizontal loading of the walls is substantially higher than due to the wind load. Additional measures for the walls have to be taken. The standard reinforcement in the wall has a capacity of 147 kN from the rebar already present. The maximum horizontal load is 335 kN. Adding 2 ϕ18 gives enough capacity for the wall to transfer to horizontal loading to the foundation.

5.5. Conclusions

After all the calculations it is clear how the CD20 system structure functions and what the problematic area is for the building. The Scia model showed how the forces are being transferred to the foundation. This happens mainly via the wall elements, which are proficient. The system is a dual system consisting of a coupled wall system and a framework system at first glance. However, the frame action is limited and therefore this system needs to be examined as a wall system. This allows for earthquakes to use a behavior factor of 3, that allows a cutback on the forces that are occurring due to the earthquake.

When analyzing the construction, the main problem occurs around the connections. The forces here are 24 % higher than they can be and thus the structure will fail in a non-adapted form. The problem is that the forces in the connection are too great for the connection to transfer. This means that the solution lies in one of two options: the first would be to lower the forces that would have to be transferred per connection. The second is upgrading the resistance of the connection so that the forces that can pass through can be larger.

A number of solutions can be thought of and the ones worthwhile examining are the following:

1. Lowering the forces by adding more wall elements. This will allow for more points where the horizontal force can be transferred through, thus allowing for less force on the connections.
2. Using a higher steel grade. Right now S235 is used, upgrading this would also enhance the forces the connection could transfer. The ductility would become lower but this is not a factor that is deciding for the building since the walls are too rigid for the ductility to have effect.
3. Welding the corner shoes in place. This would allow a larger surface of the corner shoe to transfer forces to the next plate and the column as well.

When looking at the three options, welding the corner shoes in place seems like the easiest solution. That way the construction process does not have to be altered. The construction can be put together the same as it has been now, other than that the corner shoes will have to be welded into place when they are placed, this will only give a slight increase of construction time. The corner shoes can be welded into place after mounting the structure. With this modification to the CD20 building system the building can hold its own and survive an earthquake. The columns and wall elements are not tested to their limits and have enough capacity to transfer all the loads to the foundation.

6. CONCLUSIONS

Finally after all the research and calculations it is time to reflect on the answers that have been found. The main question that was asked in the startup of this research was:

How can the prefab concrete building system CD20 be used in areas where earthquakes occur?

At the start of this document a number of sub questions or topics were given per Chapter. A couple of conclusions and findings will be repeated in order to support the final conclusion.

The building was put in Oman for this thesis, that resulted in a maximum seismic load of 66.9 kN on a sigle connection pin. The maximum this pin can withstand is 54 kN. At this point the maximum forces in other connections is not yet reached, but the computation has a built in factor that will account for plastic behavior. Therefore it is not acceptable to assume redistribution due to plasticity in the connections. Also the ductility the connections provide in the system is limited. Therefore this value must be regarded as the governing force.

The connections are not strong enough in their current form, which means that the connections either need to be altered or the loading on the connections needs to be altered. A number of solutions can be thought of, worthwhile examining are the following:

1. Lowering the forces by adding more wall elements, this will create less of the buildings mass per wall and thus the forces per connection should be lower.
2. Using a higher steel grade, right now S235 is used, upgrading this would also enhance the forces the connection could transfer. The ductility would become lower but this is not a factor that is deciding for the building since the walls are too rigid for the ductility to have effect.
3. welding the corner shoes in place. This would allow a much larger surface of the corner shoe too transfer forces to the next plate. The pins would still for the connection to the column but the horizontal forces can for a large part be transferred through the plate to the wall system.

The durability remains, because the construction is still prefabricated, and that means more effective production and higher quality elements than when cast in situ. The construction stays durable and sustainable. The idea to potentially take it apart is not used in the Netherlands either, but when using a higher steel grade as solution it is possible. That would make the entire building very sustainable and durable as well.

The building asks for additional costs to build it in an area with earthquake hazard, but so do the other building options, because the loading a construction needs to process is larger. Larger loading causes for additional structural elements. The concept does not need to be greatly altered, and the building speed can be maintained. The system has restrictions because of the limited capacity of the joints. When adapting the system, li could potentially withstand a greater force allowing it to be built in areas with significant earthquake hazard like Saudi Arabia and Oman.

The final conclusion of this report is the answer to the main research question:

How can the prefab concrete building system CD20 be used in areas where earthquakes occur?

This thesis has shown that the light weight construction of the CD20 building system does not generate forces that are too large for the system to cope with. This allows the slender construction to take up the forces and direct them towards the foundation. However, the connections can only deal with light earthquakes, for more severe ones like for instance in some parts of Saudi Arabia, the horizontal forces on the connections are higher than the current strength, but this is a manageable problem. There are several options available to improve the connection. Welding the corner shoes in place, thus creating more capacity in the joints would be the best solution in my opinion. This would allow the system to be applied in area's with more severe earthquake hazard, but not change the production of the system. The adaptation to the current system would be in the construction phase, without lengthening the building process much.

7. TABLE OF FIGURES

8. TABLE OF TABLES

9. APPENDIX

9.1. Diana images

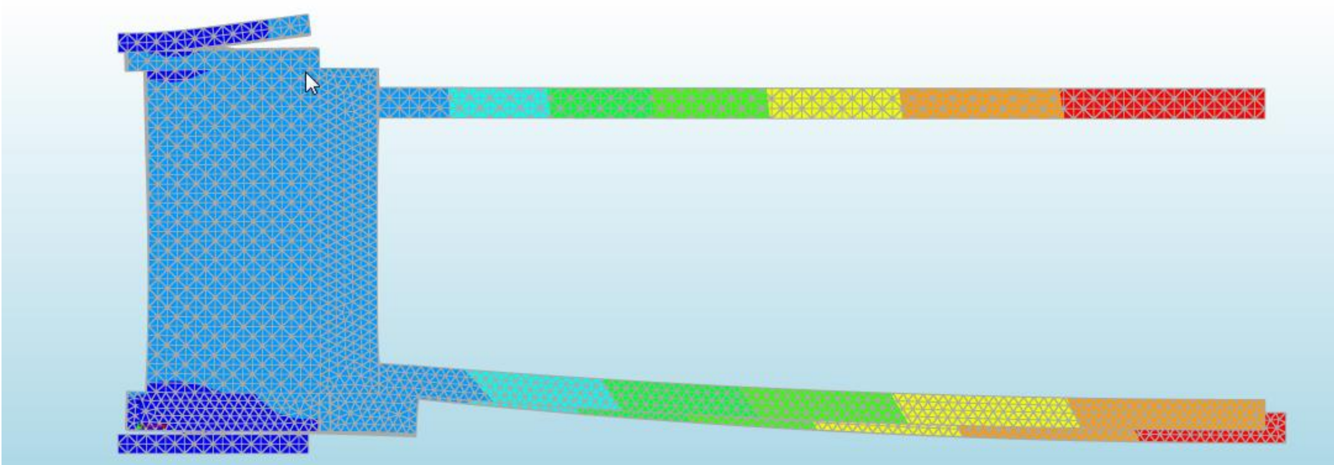


FIGURE 78 DIANA MODEL OF CONNECTION, DISPLACEMENT 0.1 MM

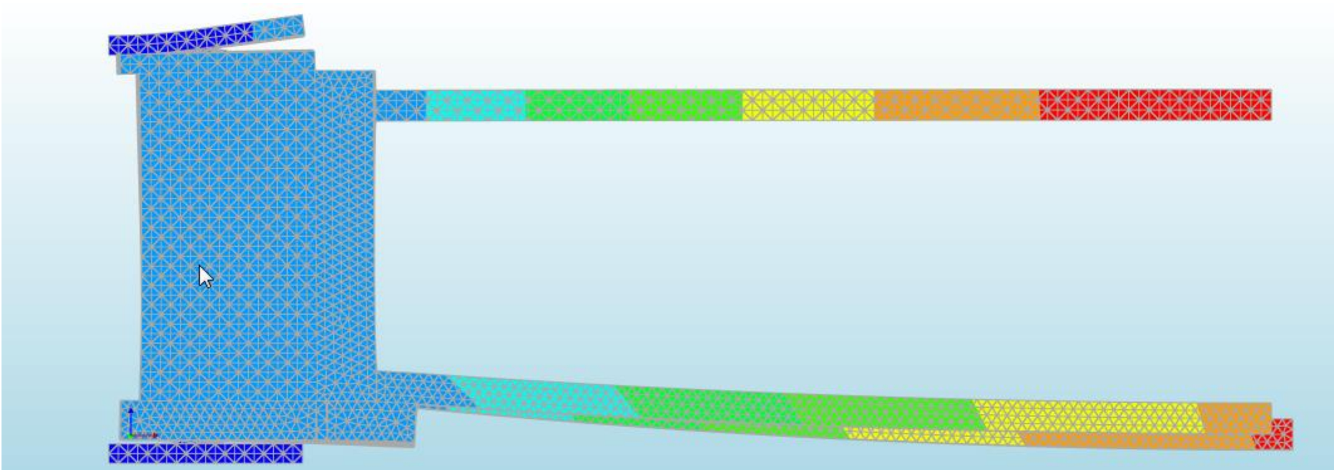


FIGURE 79 DIANA MODEL OF CONNECTION, DISPLACEMENT 0.3 MM

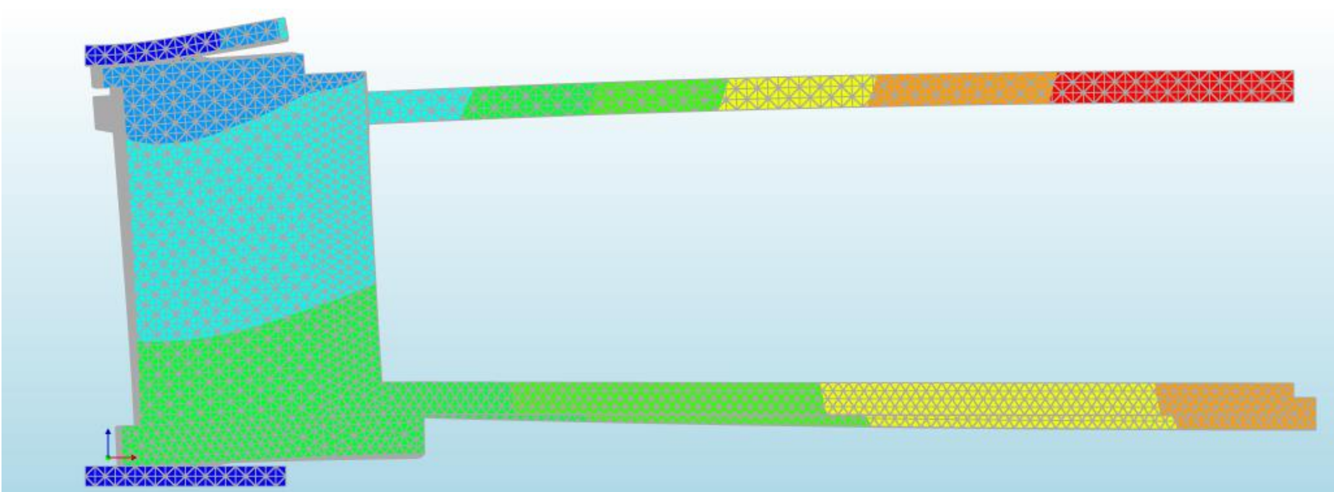


FIGURE 80 DIANA MODEL OF CONNECTION, DISPLACEMENT 0.5 MM

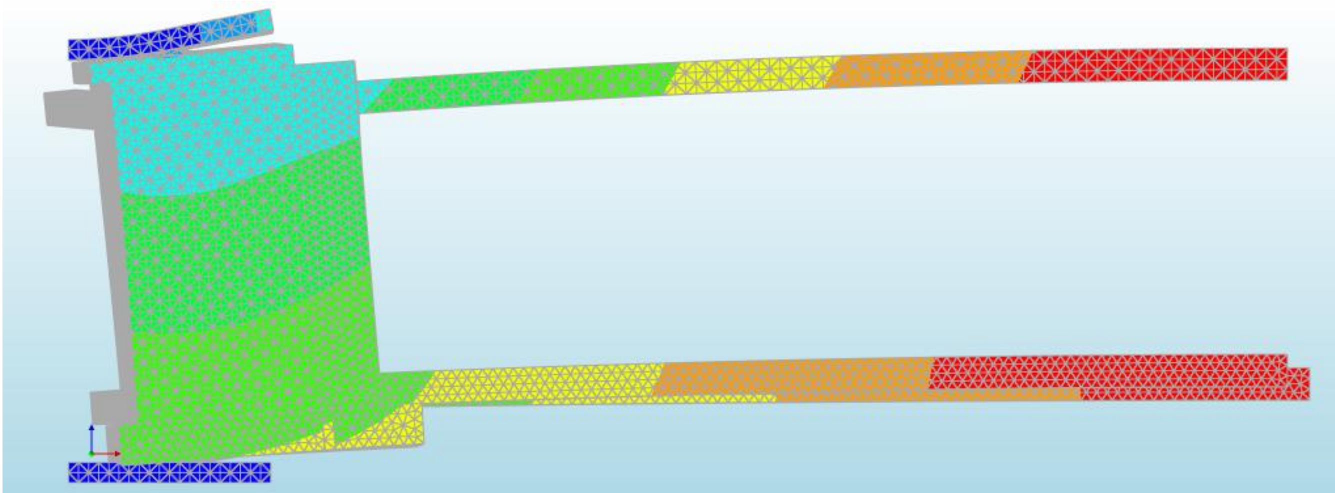


FIGURE 81 DIANA MODEL OF CONNECTION, DISPLACEMENT 0.7 MM

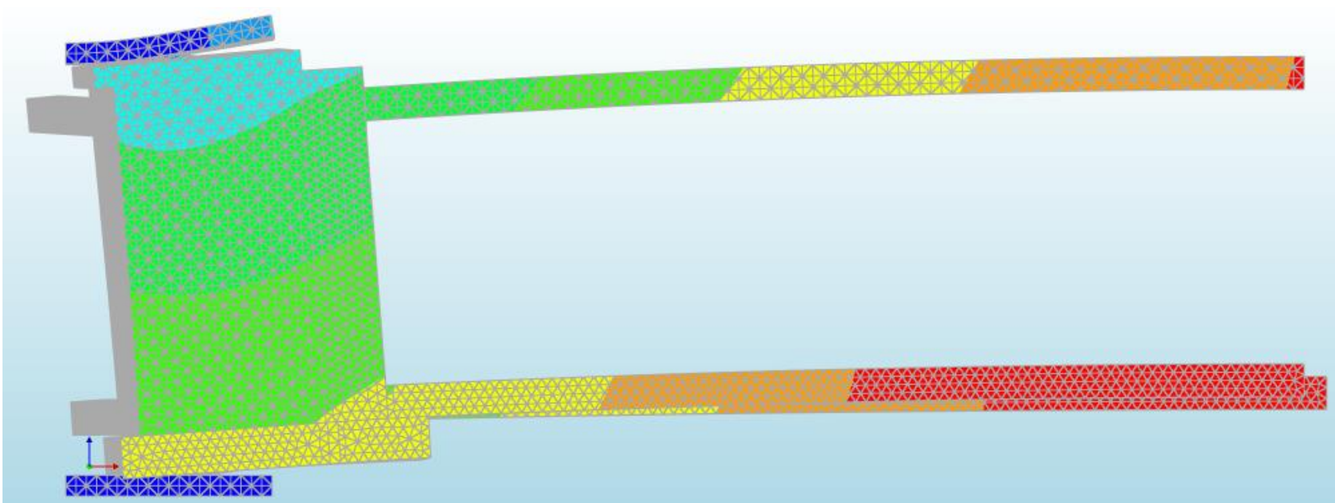


FIGURE 82 DIANA MODEL OF CONNECTION, DISPLACEMENT 0.9 MM

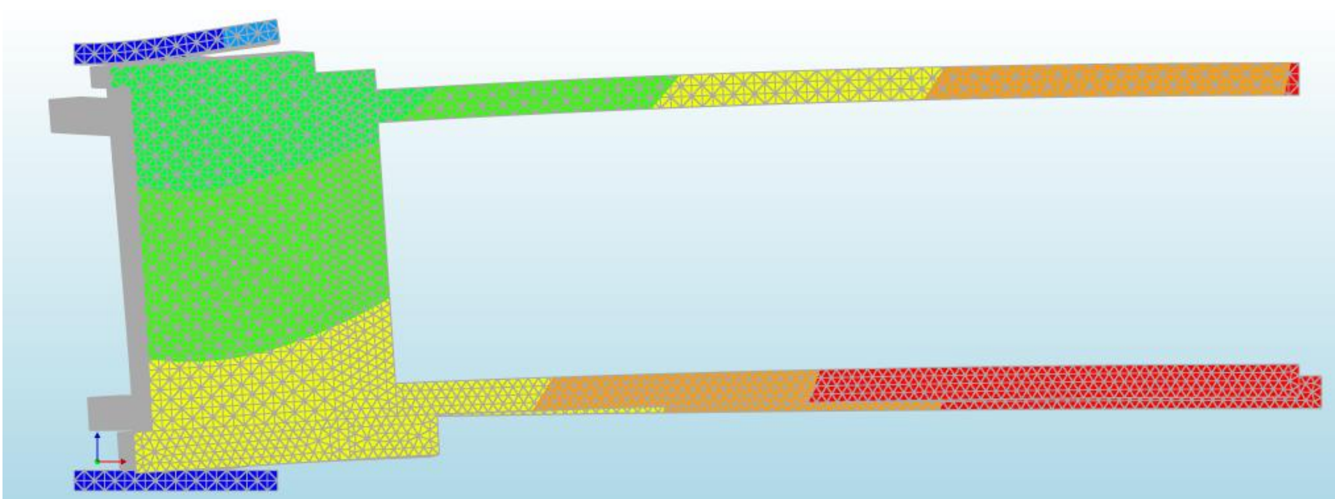


FIGURE 83 DIANA MODEL OF CONNECTION, DISPLACEMENT 1.1 MM

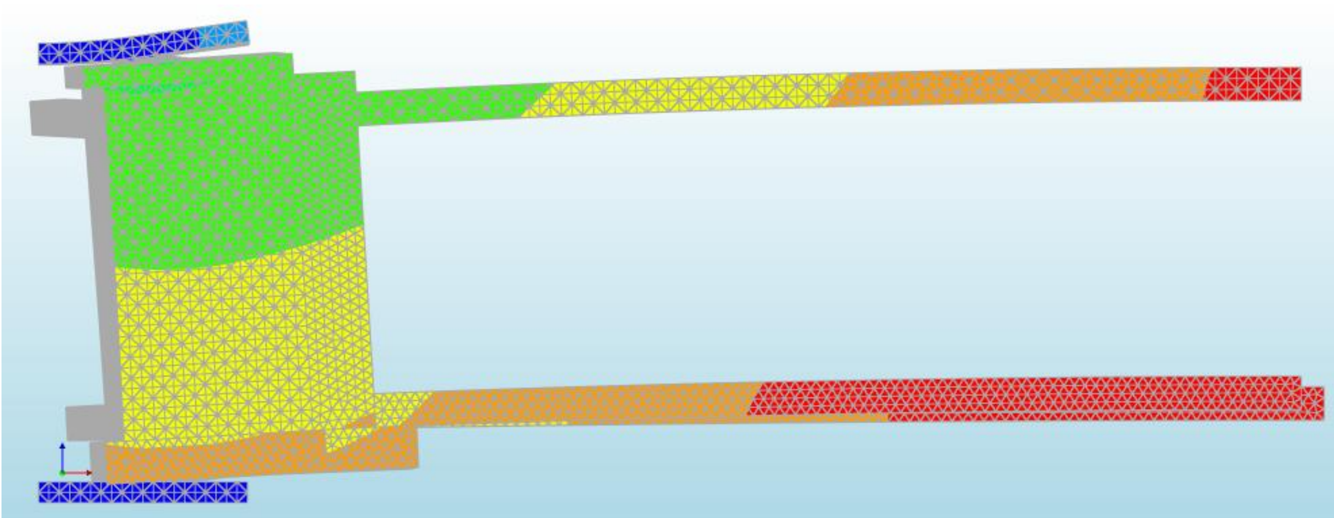


FIGURE 84 DIANA MODEL OF CONNECTION, DISPLACEMENT 1.3 MM

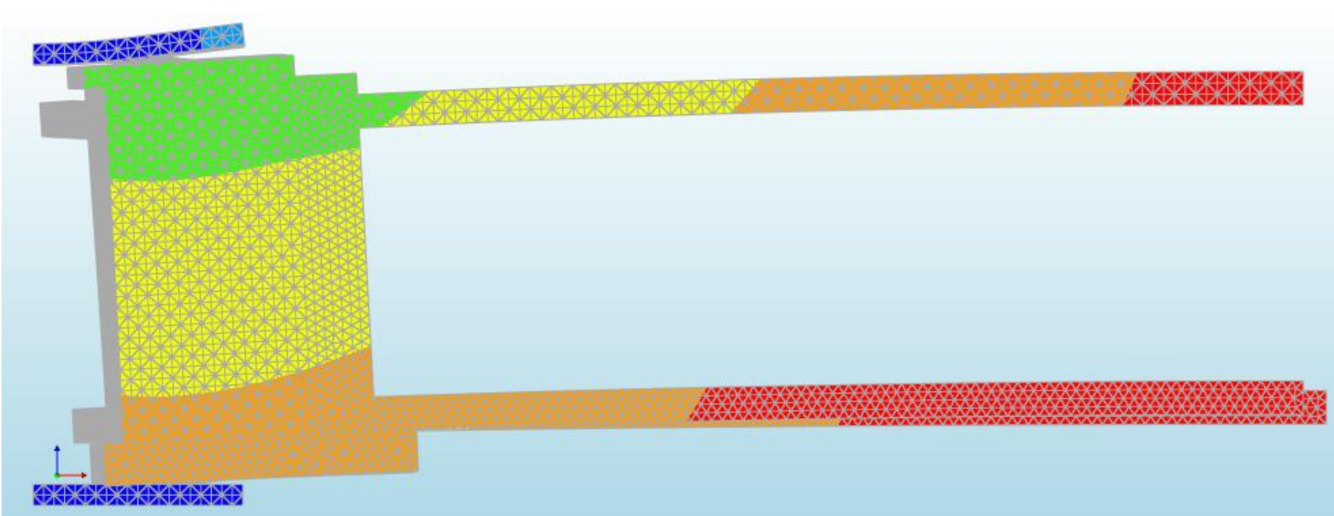


FIGURE 85 DIANA MODEL OF CONNECTION, DISPLACEMENT 1.5 MM

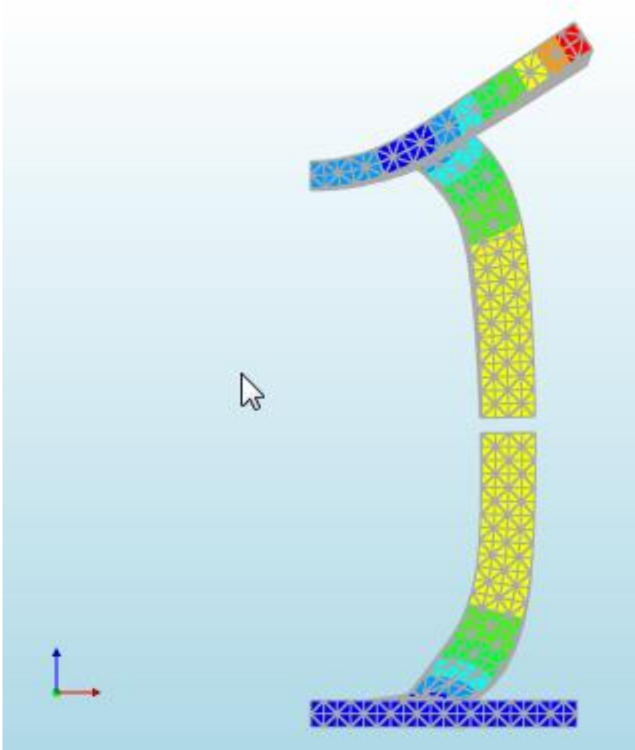


FIGURE 86 DIANA MODEL OF PINS, DISPLACEMENT 0.1 MM

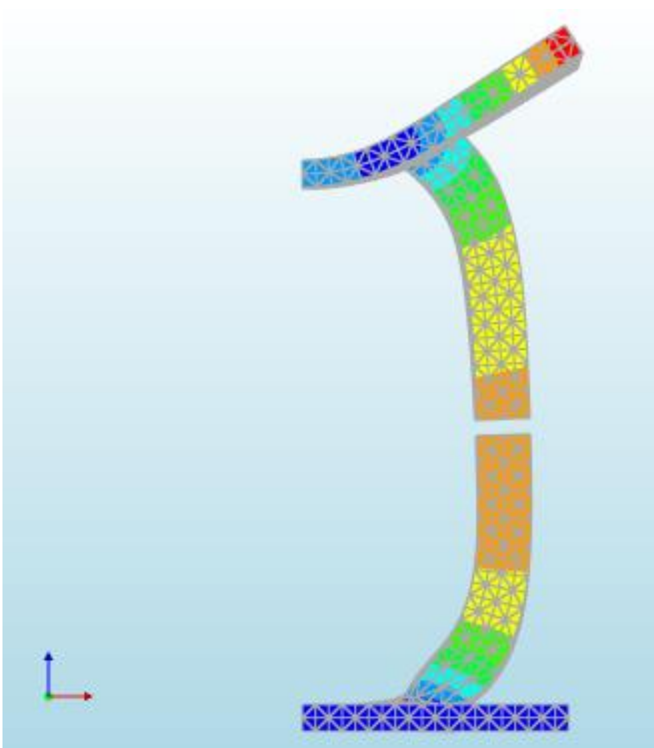


FIGURE 87 DIANA MODEL OF PINS, DISPLACEMENT 0.3 MM

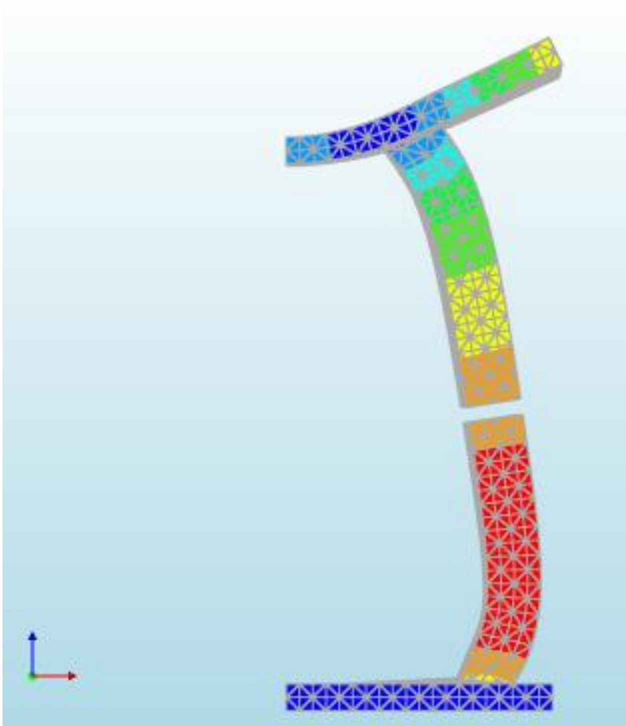


FIGURE 88 DIANA MODEL OF PINS, DISPLACEMENT 0.5 MM

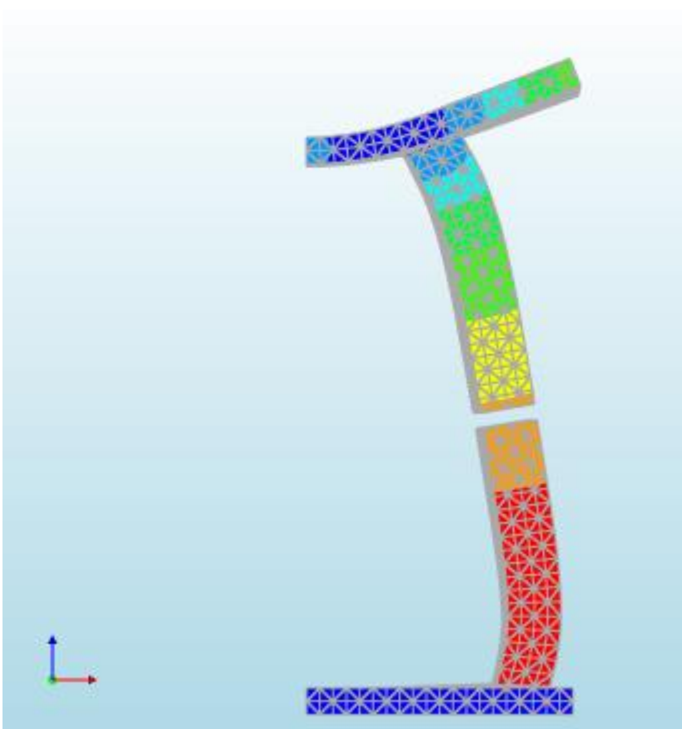


FIGURE 89 DIANA MODEL OF PINS, DISPLACEMENT 0.7 MM

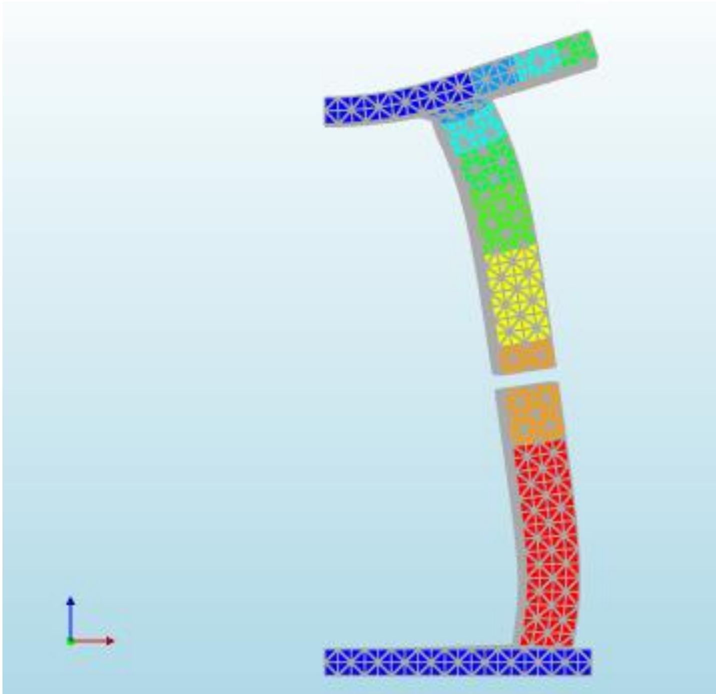


FIGURE 90 DIANA MODEL OF PINS, DISPLACEMENT 0.9 MM

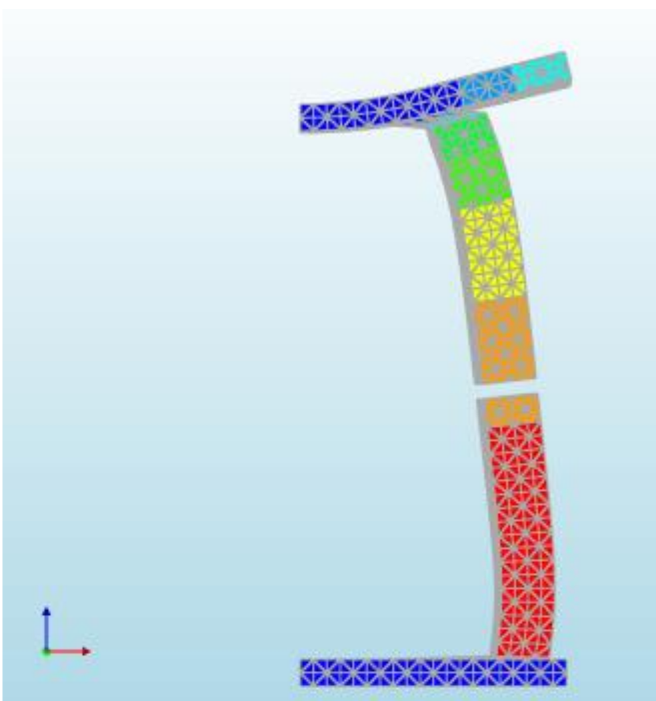


FIGURE 91 DIANA MODEL OF PINS, DISPLACEMENT 1.1 MM

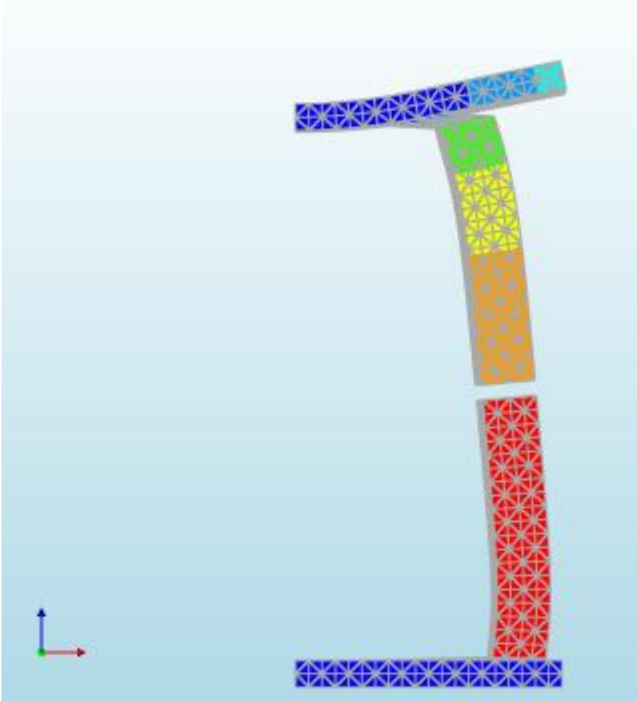


FIGURE 92 DIANA MODEL OF PINS, DISPLACEMENT 1.3 MM

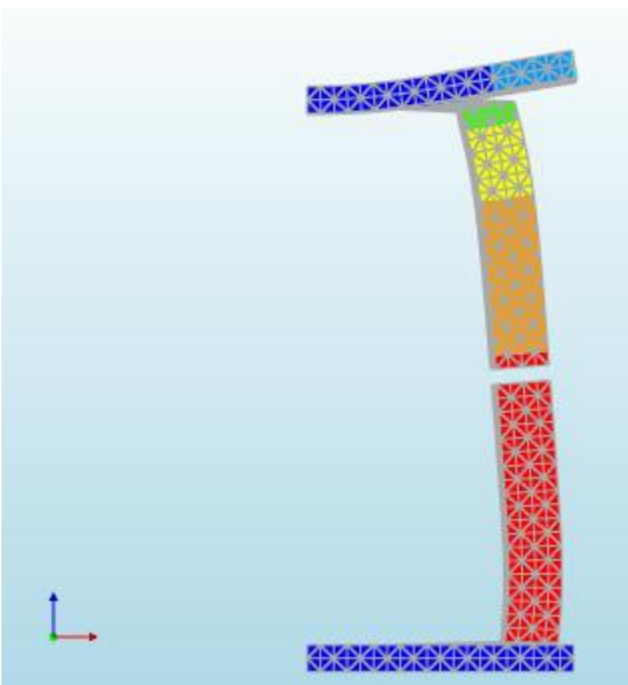


FIGURE 93 DIANA MODEL OF PINS, DISPLACEMENT 1.5 MM

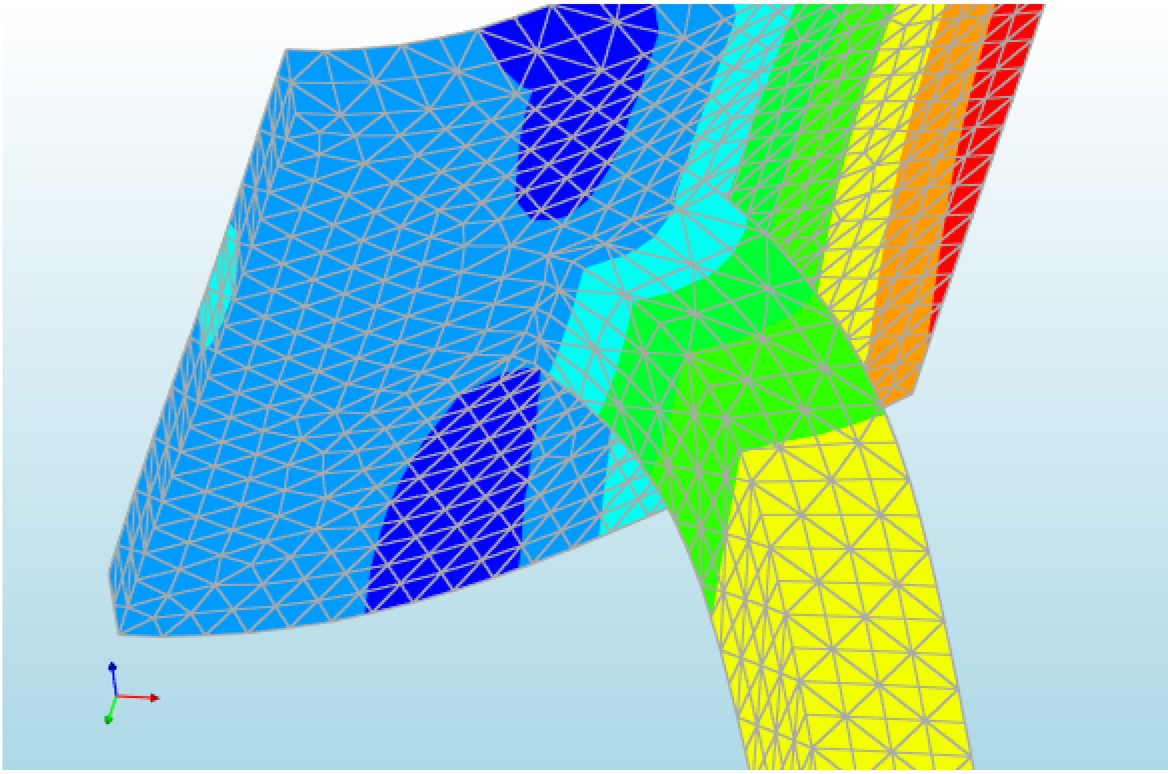


FIGURE 94 DIANA MODEL OF DETAIL, DISPLACEMENT 0.1 MM

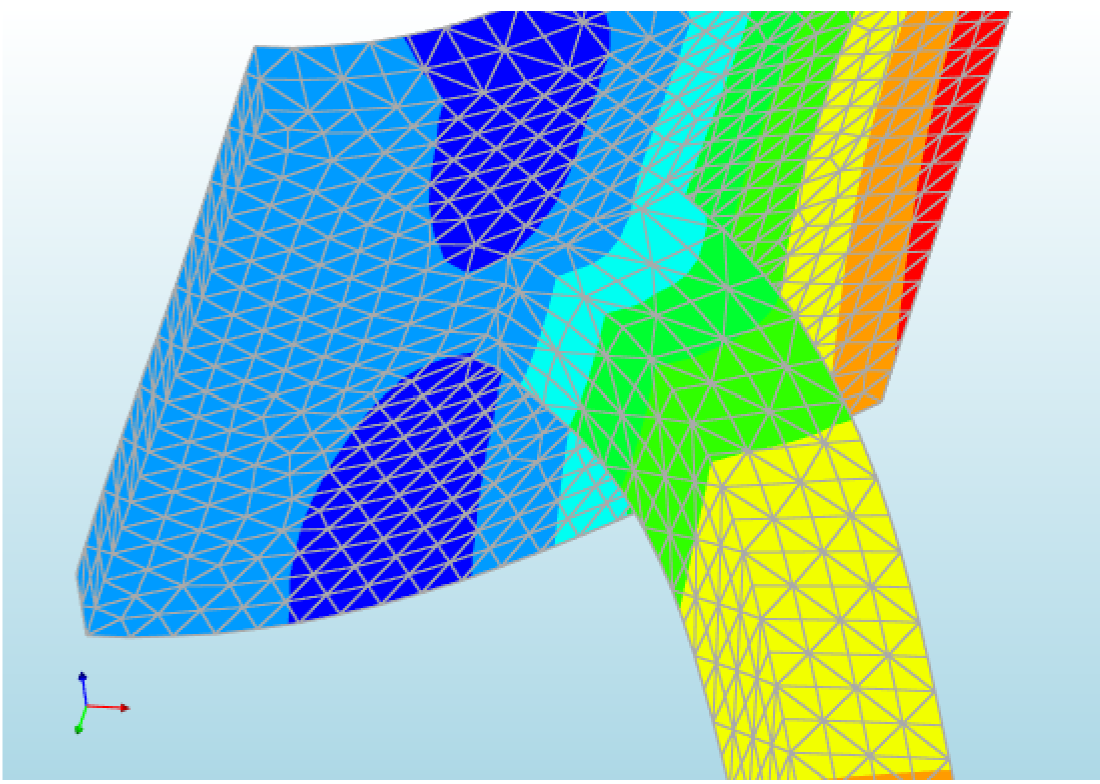


FIGURE 95 DIANA MODEL OF DETAIL, DISPLACEMENT 0.3 MM

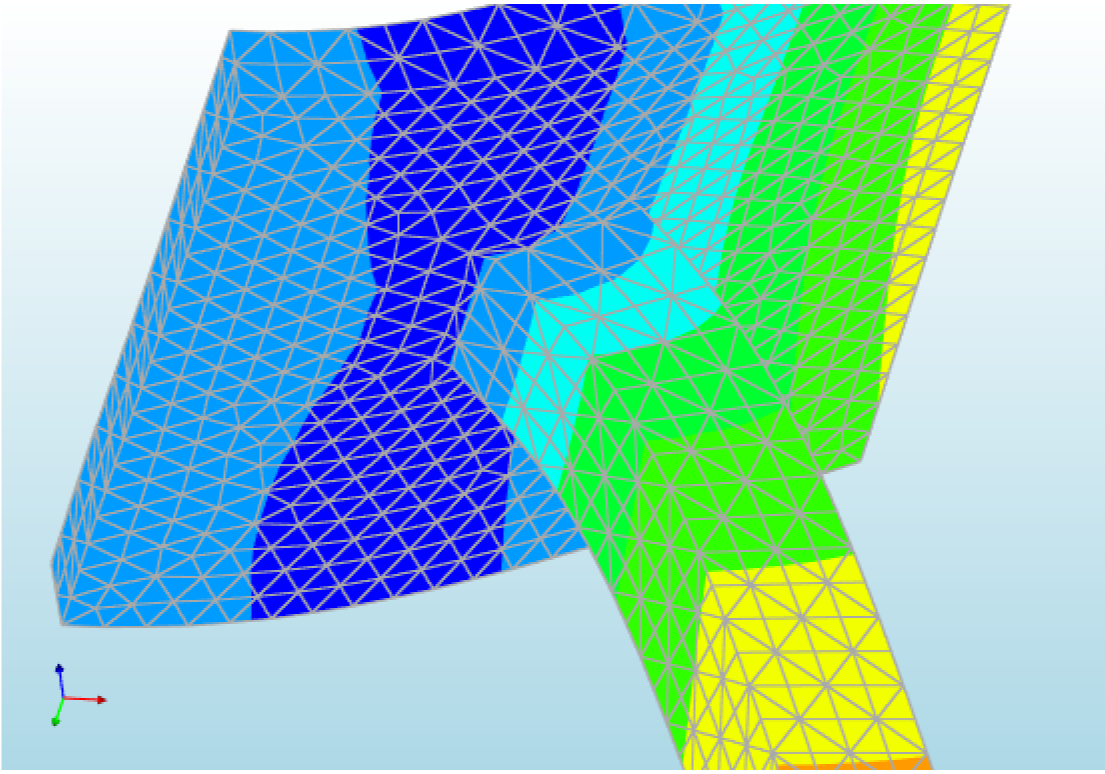


FIGURE 96 DIANA MODEL OF DETAIL, DISPLACEMENT 0.5 MM

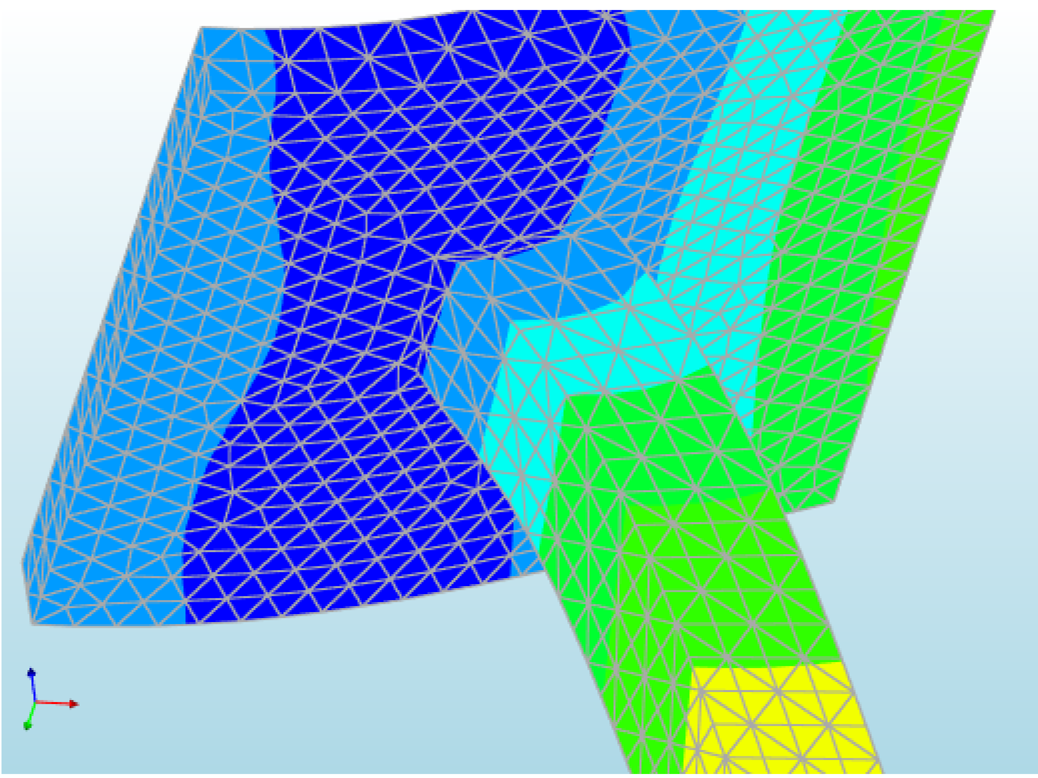


FIGURE 97 DIANA MODEL OF DETAIL, DISPLACEMENT 0.7 MM

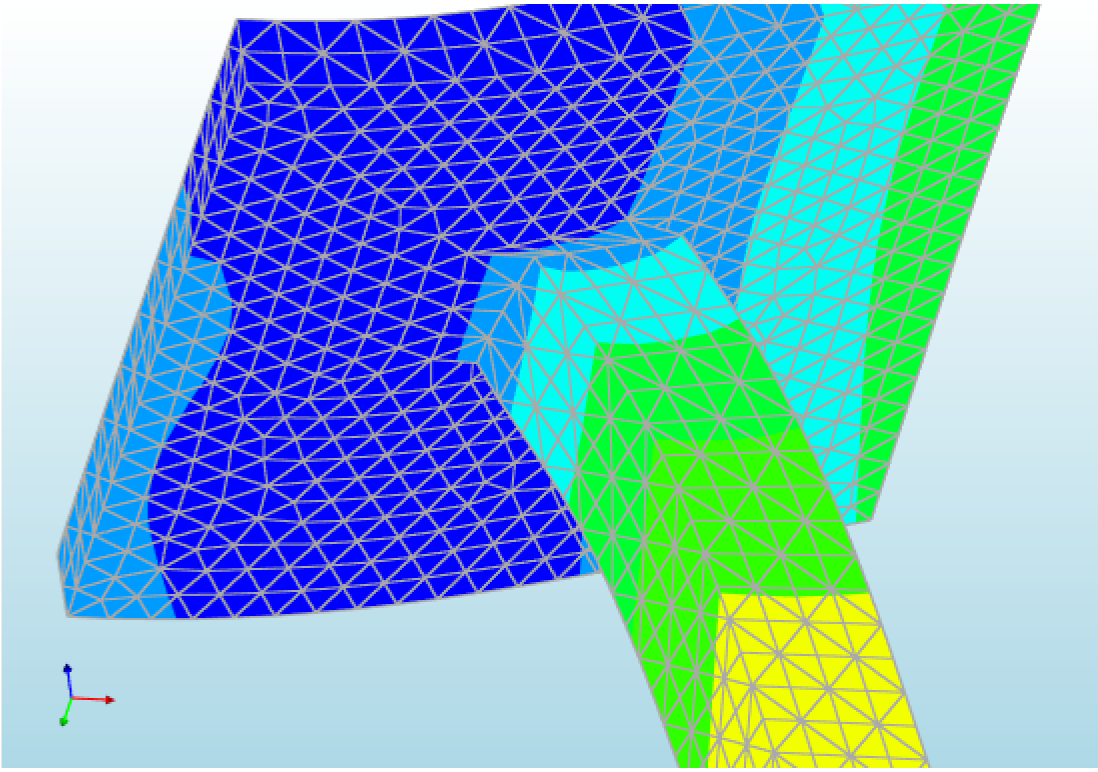


FIGURE 98 DIANA MODEL OF DETAIL, DISPLACEMENT 0.9 MM

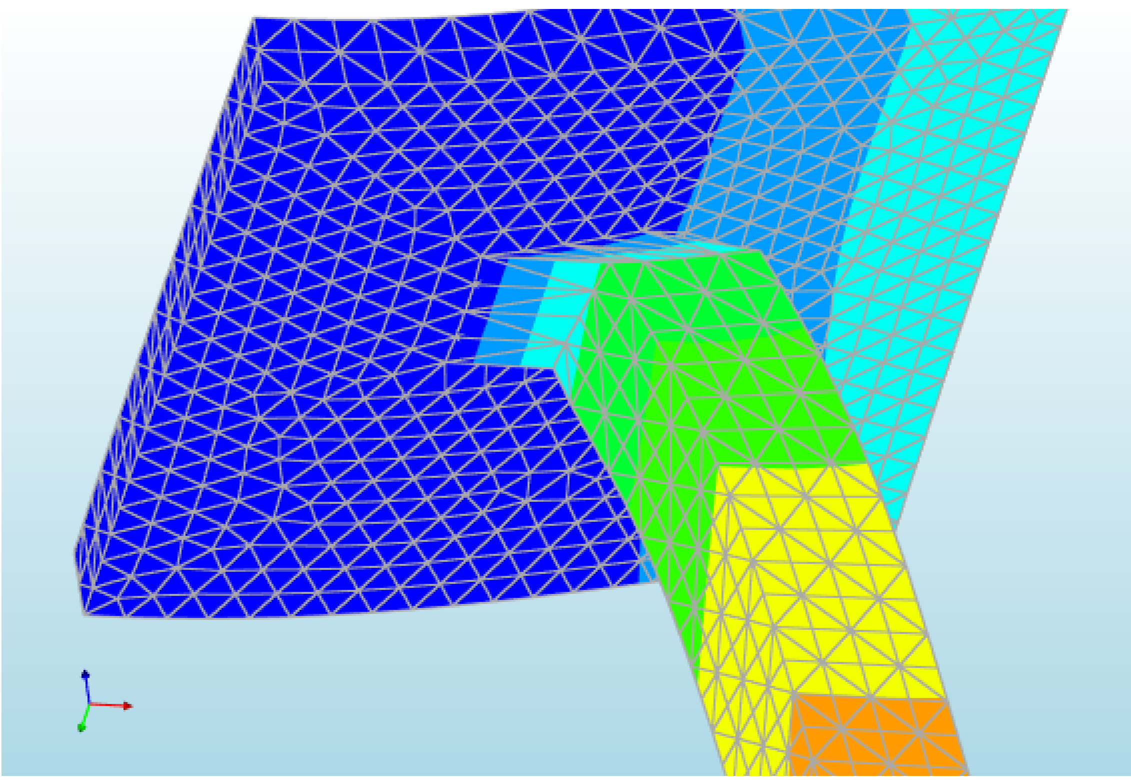


FIGURE 99 DIANA MODEL OF DETAIL, DISPLACEMENT 1.1 MM

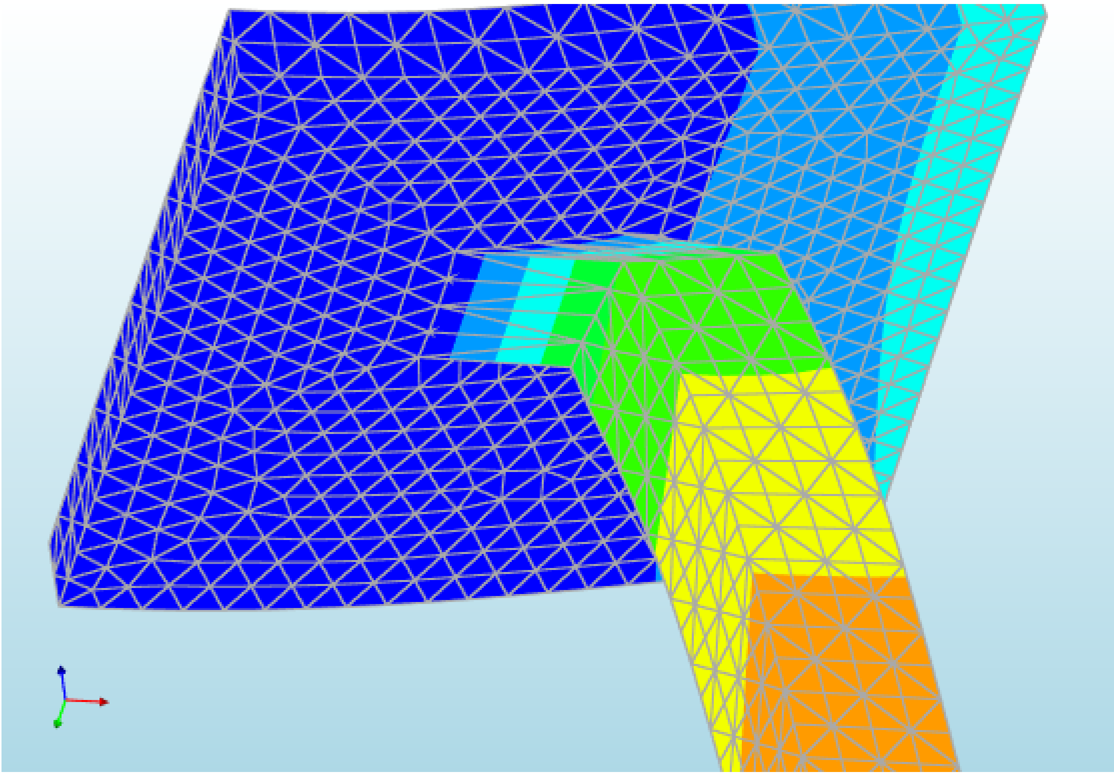


FIGURE 100 DIANA MODEL OF DETAIL, DISPLACEMENT 1.3 MM

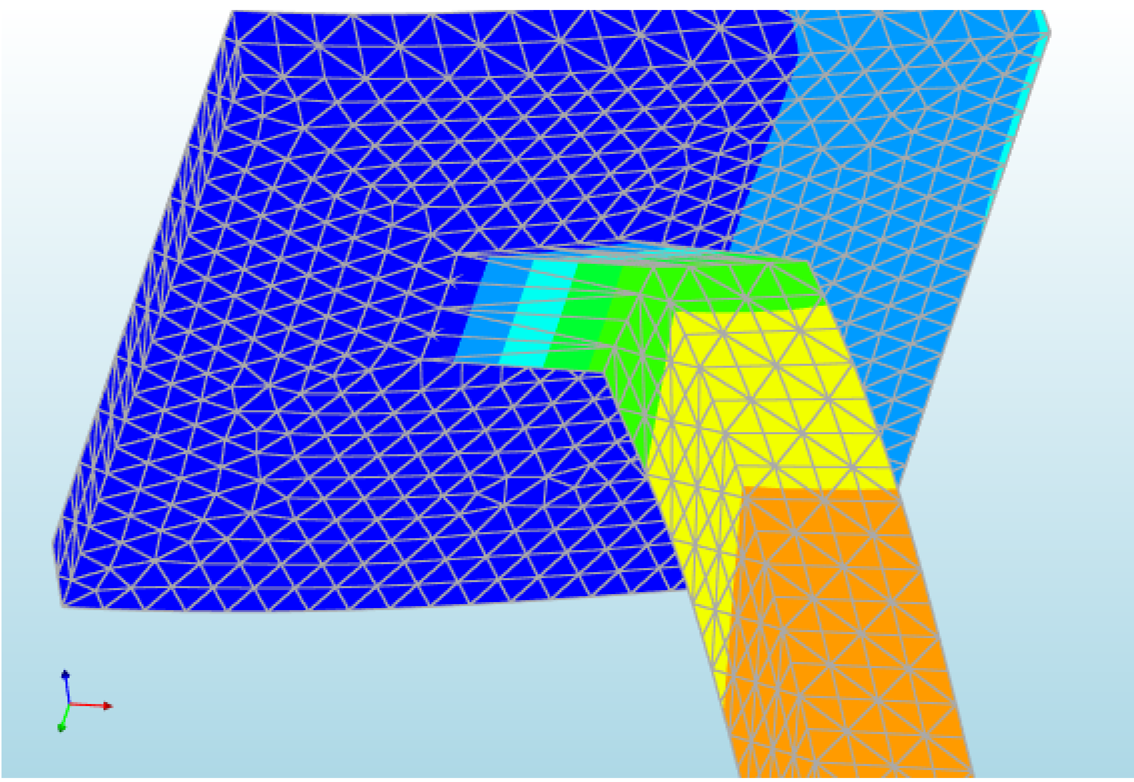


FIGURE 101 DIANA MODEL OF DETAIL, DISPLACEMENT 1.5 MM

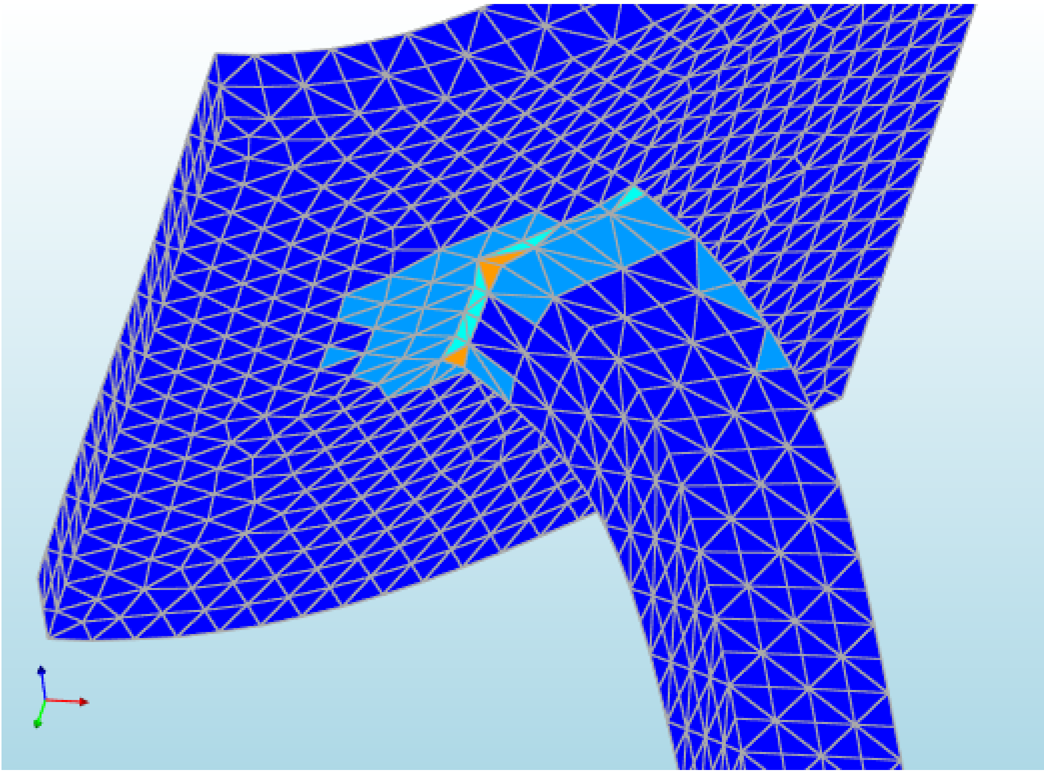


FIGURE 102 DIANA MODEL OF DETAIL, STRESSES AT 0.1 MM DISPLACEMENT

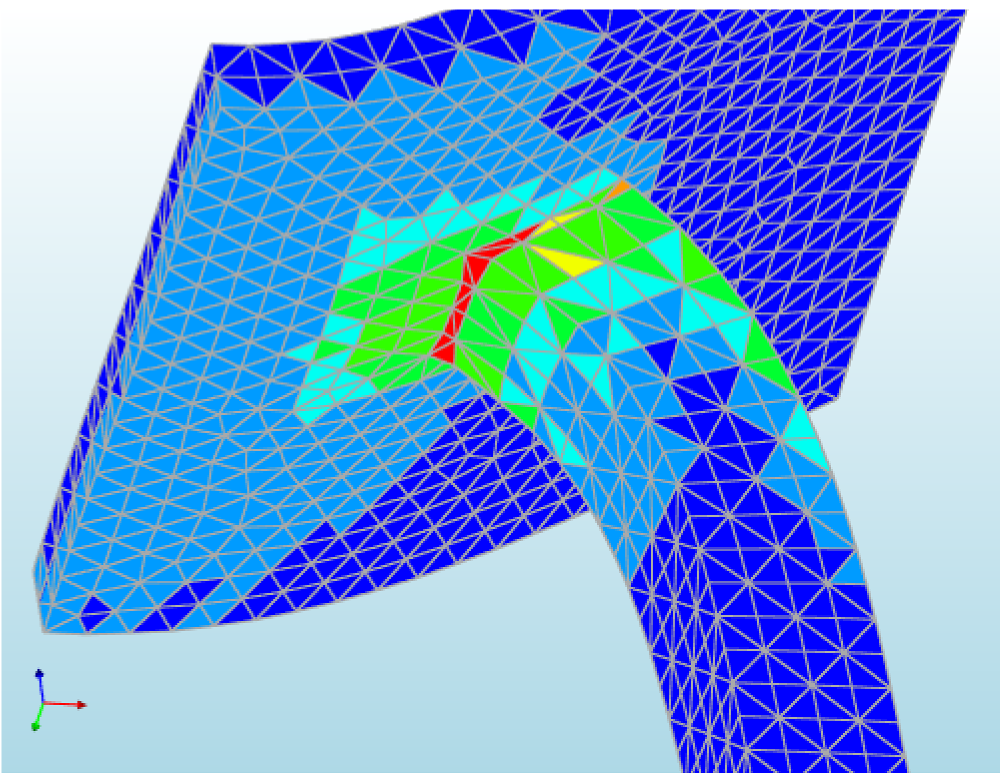


FIGURE 103 DIANA MODEL OF DETAIL, STRESSES AT 0.3 MM DISPLACEMENT

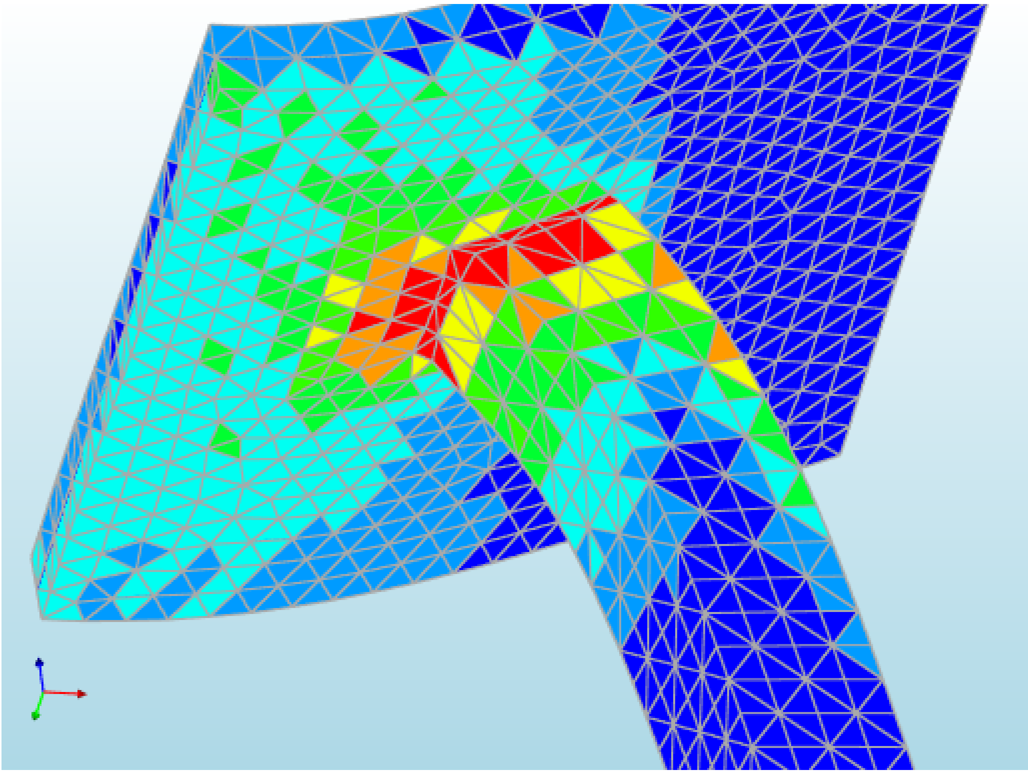


FIGURE 104 DIANA MODEL OF DETAIL, STRESSES AT 0.5 MM DISPLACEMENT

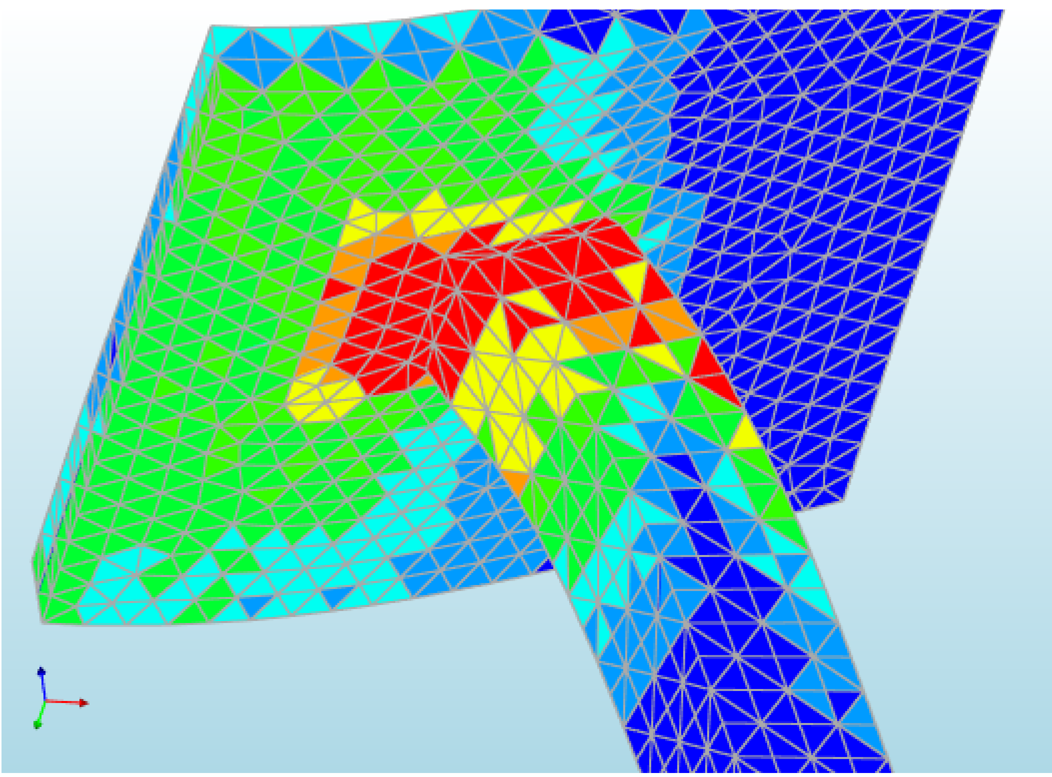


FIGURE 105 DIANA MODEL OF DETAIL, STRESSES AT 0.7 MM DISPLACEMENT

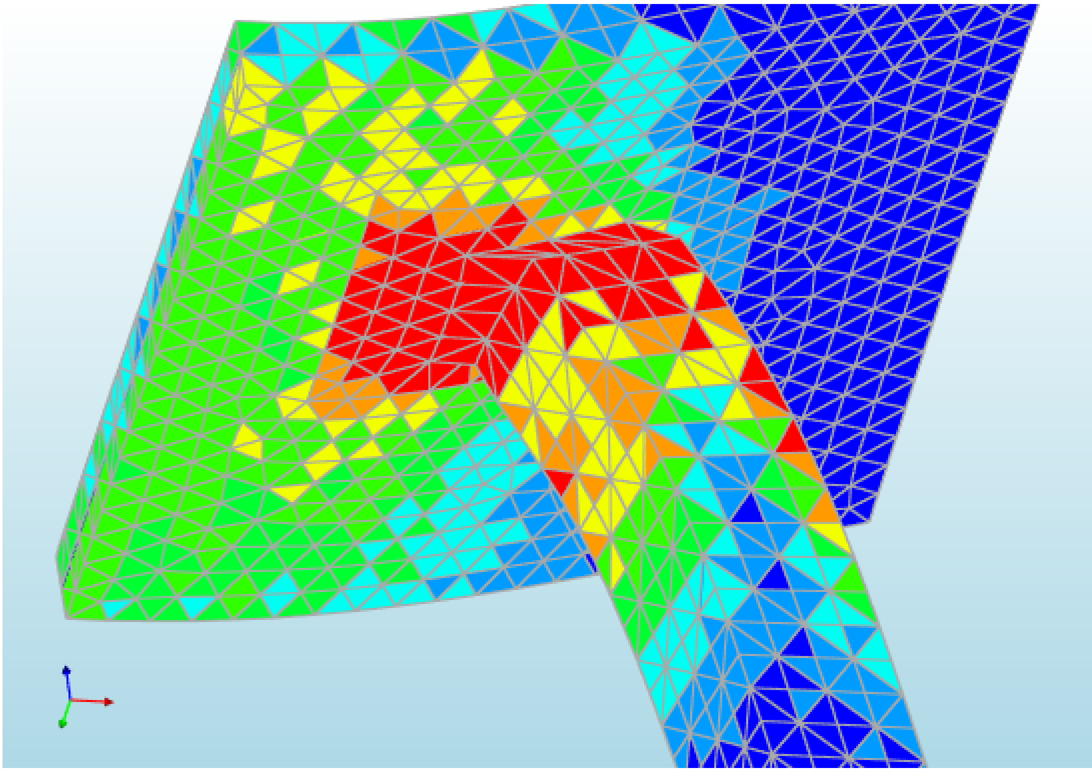


FIGURE 106 DIANA MODEL OF DETAIL, STRESSES AT 0.9 MM DISPLACEMENT

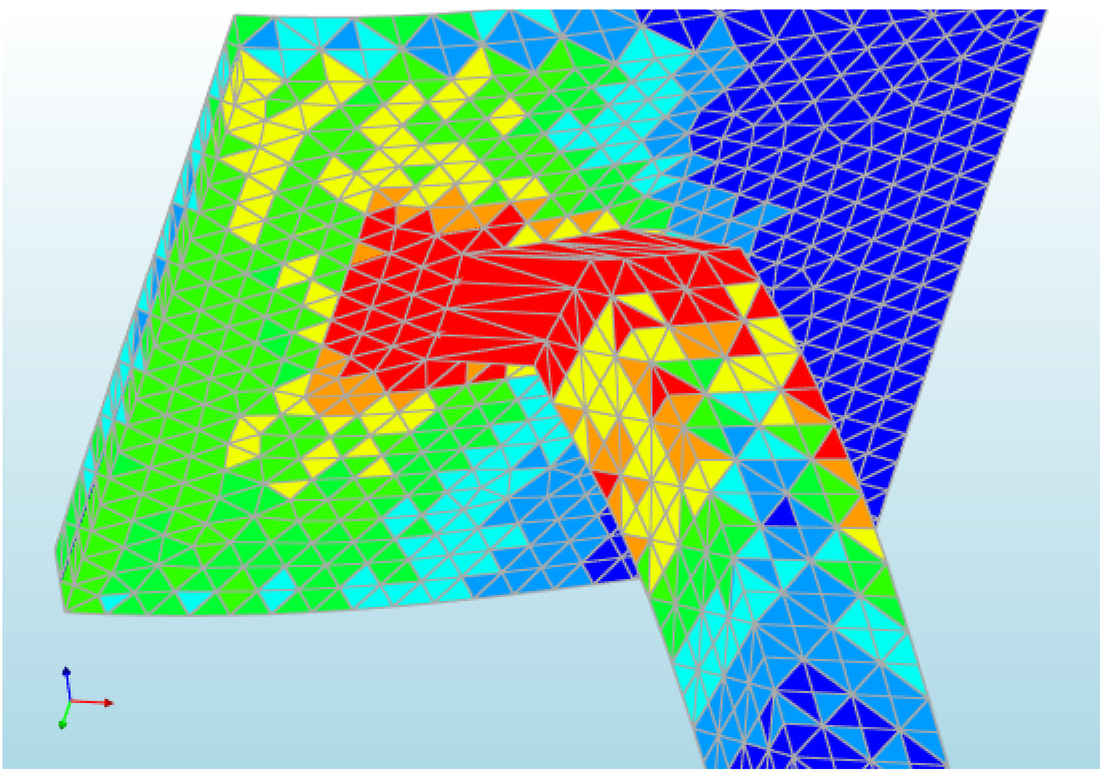


FIGURE 107 DIANA MODEL OF DETAIL, STRESSES AT 1.1 MM DISPLACEMENT

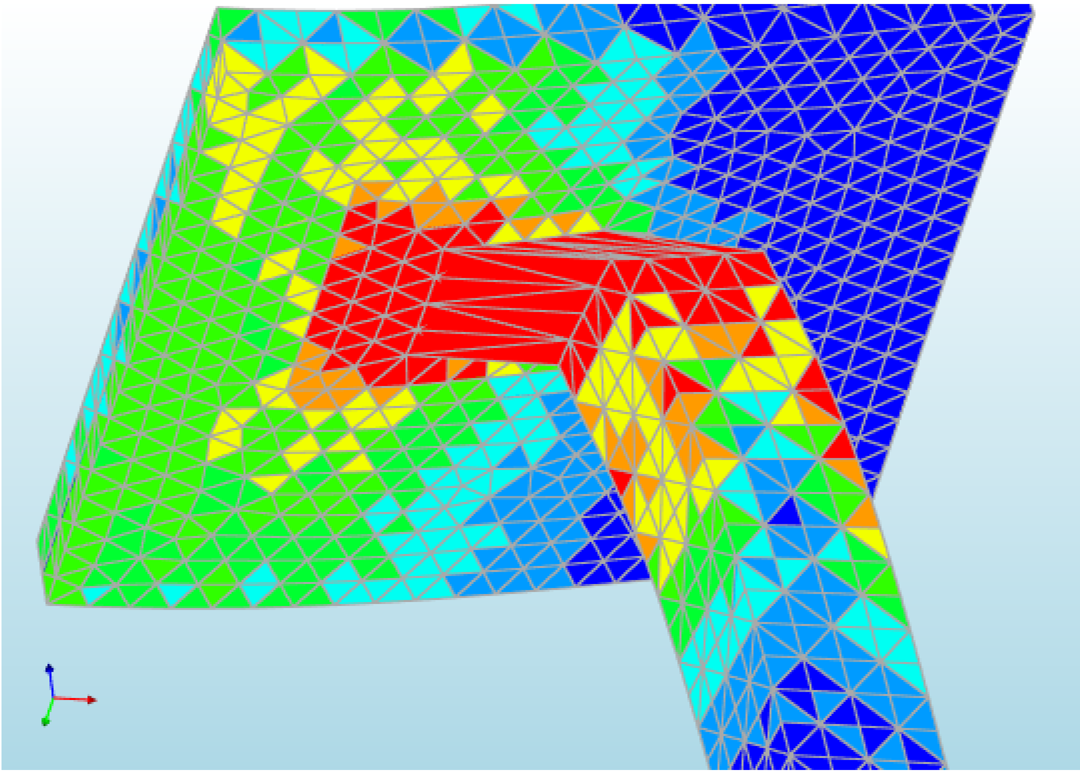


FIGURE 108 DIANA MODEL OF DETAIL, STRESSES AT 1.3 MM DISPLACEMENT

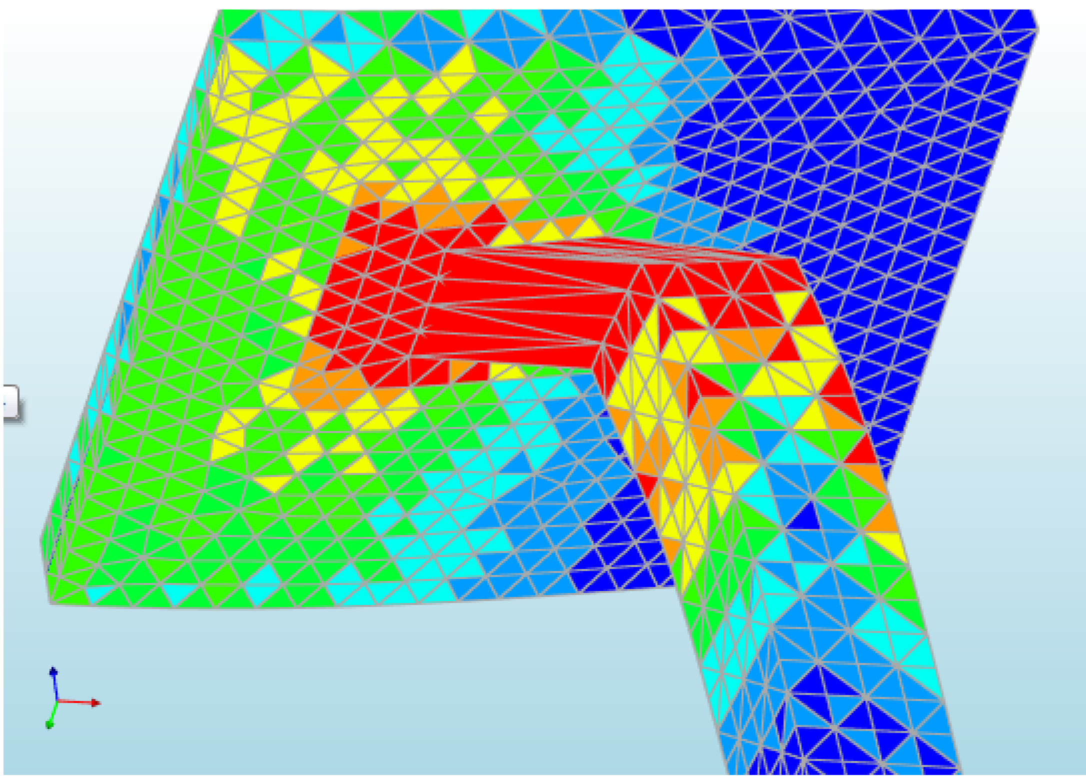


FIGURE 109 DIANA MODEL OF DETAIL, STRESSES AT 1.5 MM DISPLACEMENT

9.2. Development of the Diana model

Construction of the model

Appendix 9.4 describes the development of the Diana model. Where the mistakes were made and the learning curve in the new

Step 1

The first step is to build a geometric model in Rhinoceros in order to have the right input for the geometry in FX+ for Diana. This is a new interface for the use of Diana, meaning that it is still calculated by Diana, but the interaction with the program runs via FX+. In order to get this geometric model information is required about the properties of the elements of the connection. This information was acquired by studying the drawings and the digital model that was provided by CD20 building systems. Only a quarter of the floor slab is taken in order to cut back on the modeling and calculation time.

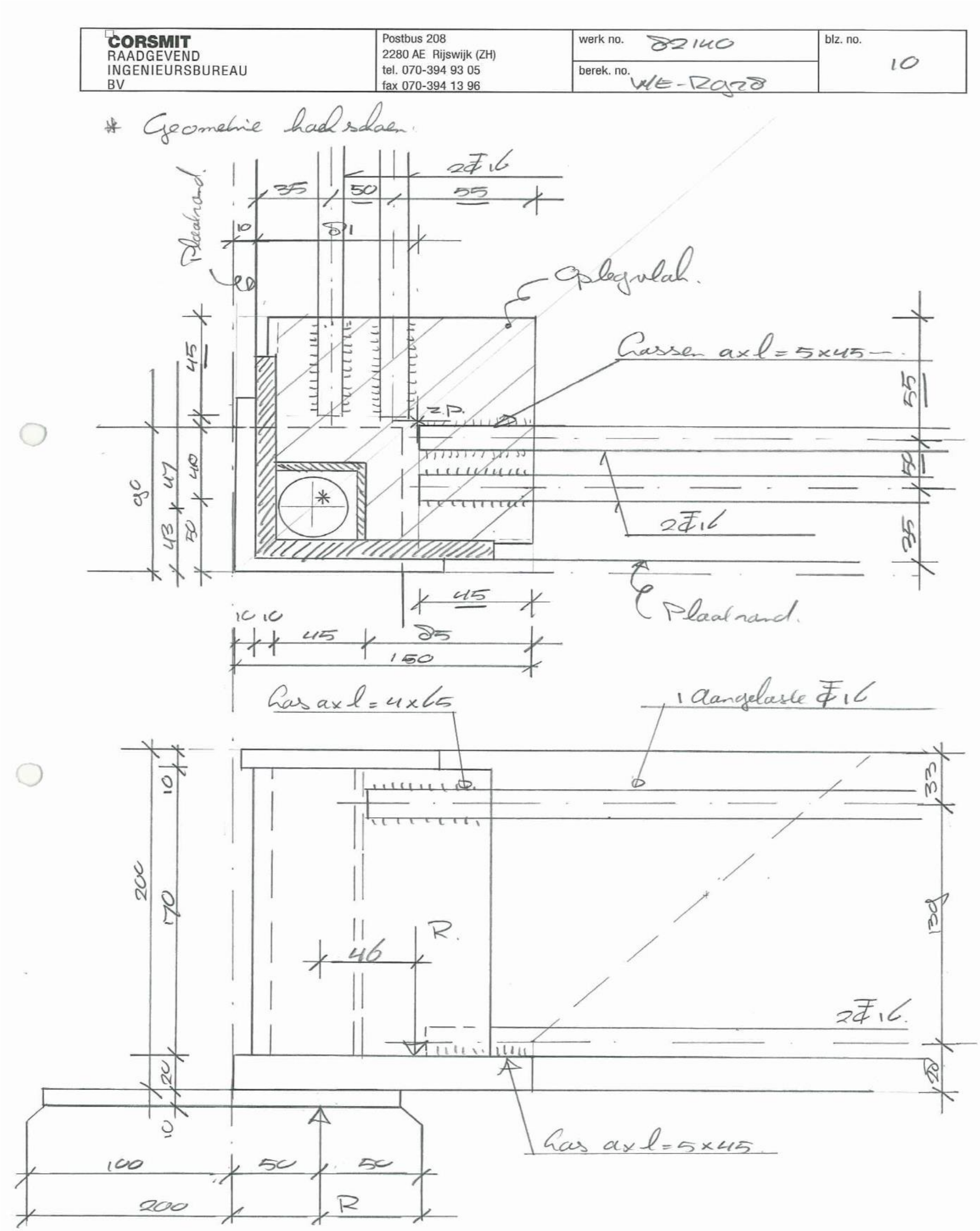


FIGURE 110MEASUREMENTS OF THE CORNER SHOE

Step 2

This step should have been relatively simple, but it was at this point that the first problems occurred. The model could not be exported from Rhinoceros to FX+ for Diana all at once, so the knot had to be divided into 8 parts. It consists out of two column elements each built up out of a concrete section and a steel top plate with a dowel. This dowel is cast into the concrete, thus creating the means to transfer the forces from one element to the next. One element is the corner shoe of the floor plate and another is the concrete that is used to fill this corner shoe up from the inside. The final two elements are the different parts of the floor that are used in order to model the floor. A 3D model is made of the part engulfing the rods of the reinforcement from the shoe, the rest of the plate is made in a 2D plate in order to cut back on the calculating time. That makes for 8 elements, all put together they form the joint at which this entire thesis revolves around.

There were a couple of problems concerning the initial import of the elements. The first one concerned the way the model was built. The model needed to exist completely out of solid elements, because they can be imported into FX+ for Diana. The first problem was that certain shapes would not be filled up to become a solid. After this was solved by rebuilding the concerning elements in FX+, another problem arose: the connecting of the different solids into the before mentioned elements.

Linking the rebars to the corner shoe to become one solid element. The rebars in real life only just touch the surface of the steel plate and are welded into place. however if the two elements only share a line, a problem arises when a volume needs to be cut out of the floor slab. In order to solve this problem the bars are placed half a millimeter inside the plate, thus the connecting surface is larger, but a change this minimal does not affect the force flow. With all of these elements in place now, importing the geometry into Diana FX+ is possible, meaning the modeling can now continue in FX+ for Diana.

Step 3

After the geometry had been loaded in, assigning the properties was next. However that does not go without creating the mesh in which that needs to happen. First the different materials have to be created, these two elements are steel and concrete. The first time the model is being constructed it is more important to have a working model rather than to have all the details worked out.
After creating the different elements, the element size needs to be determined in order to form the mesh. The element size can be determined in two different ways, in absolute measurements and in divisions per edge. The last option makes for a very remarkable division around the holes, but more important the meshes do not correspond with the mesh on the neighboring element, allowing no interaction between the different elements. In order to solve this the different contact surfaces need to be adjusted by lowering the contact part by 0.1 mm. That way the surfaces and meshes are an exact match and the forces can be transferred. This does however requires to go back to the drawing board and make adjustments. With the adjusted elements the meshes do align and there is interaction between the different parts of the model.

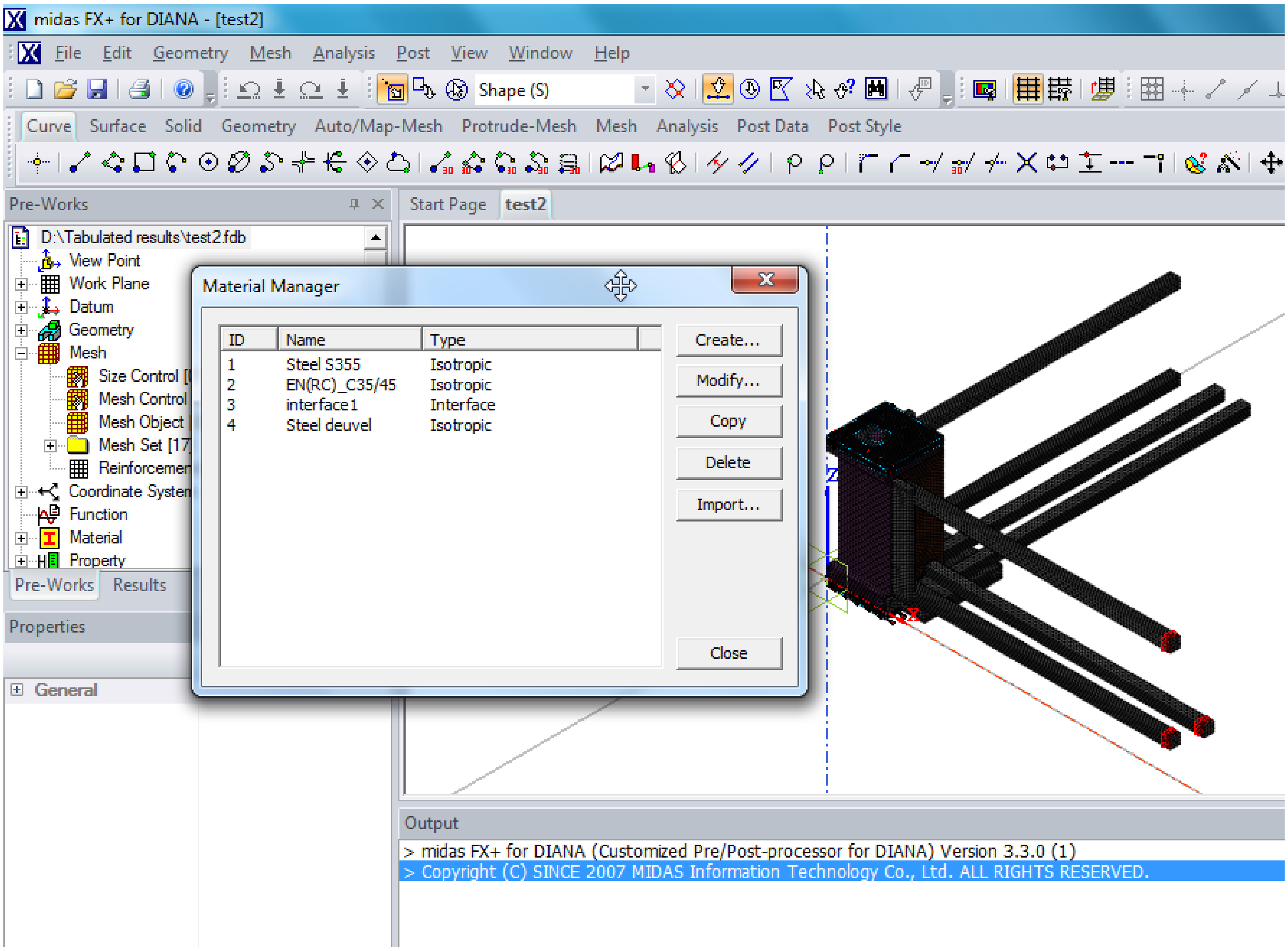


FIGURE 111 MATERIAL INPUT

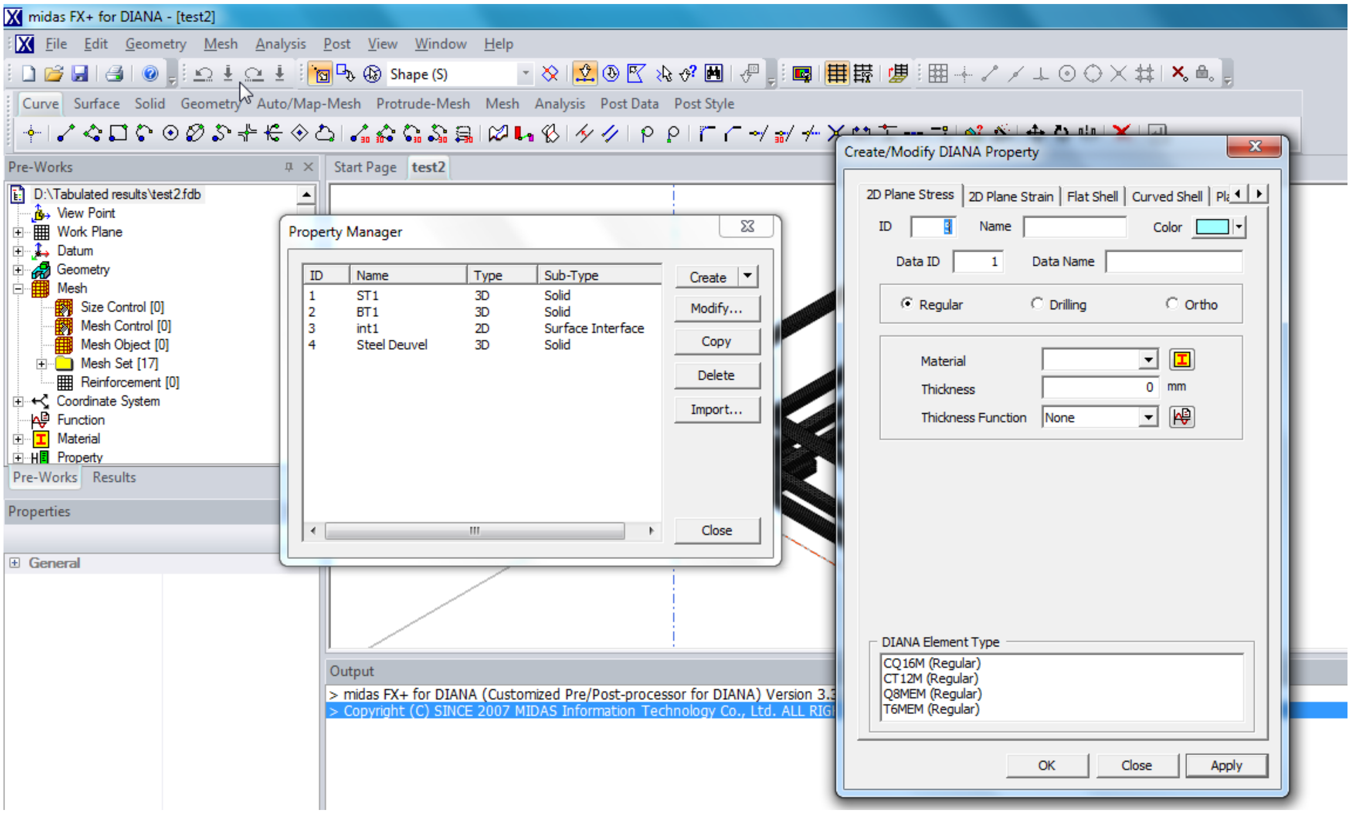


FIGURE 112 PROPPERTIES INPUT

Step 4

Before the interface elements are constructed, it is imperative that the model functions the right way. This is checked first by connecting everything 100%. The model will not present any usable values yet but the setup of the model can be observed and evaluated.

When the first test should have taken place the first problem arose. This problem has to do with the capacity of the computer calculating the model. It is clear that the hardware available was not suited to make this many calculations and would seize functioning and the calculation had to be aborted. Switching to a larger storage drive solved this problem, but keeping the model as simple as possible was important to get results.

After the first checks, it was clear that the connection between some of the elements was still not as desired, as there was no physical link between the elements yet. This caused the model to move elements through one another instead of being influenced by each other. Re-meshing parts caused more alignment and a better interaction between the different elements. After the first real analysis was created, the following images show the initial results:

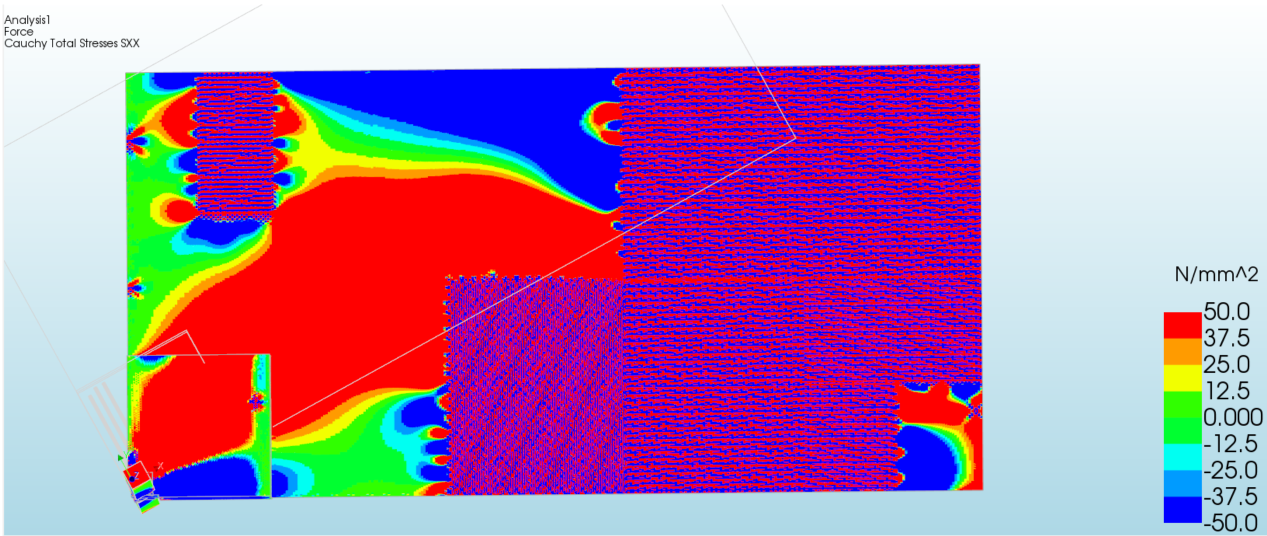


FIGURE 113 DISPLAY PROBLEMS WITH DIANA

The first analysis shows a large part of the floor slab wrongfully calculated and also presenting strange readings. This is cause to believe that the current model is still incomplete. The next step in controlling the model was examining the individual parts of the model in order to see if there was a force transfer between the different parts of the model. This looked good for some of the elements but the reinforcement rods of the corner shoe showed a complete blank, .

After carefully examining and consulting several Diana experts on the matter, the decision has been made to alter the model to a simplified version. The current version is too complicated to figure out what the problem exactly is. The new version will consist out of square and rectangular elements and will be granting matching surfaces in order to get a better success of aligning the nodes.

The rebuilding of the model will also allow space for reevaluating the current model and changes that need to be applied to the model in order to get the answers that are needed. Different groups are needed for the rebar and the steel corner shoe in order to make the interface elements all attach in the right way to the concrete. The interaction between the two steel parts is of a different kind than the previous one was.

The New Model

The new Diana model needed to contain less challenging shapes and surfaces. The problem with the previous model was that the nodes of the different elements did not align. The biggest cause for this was that not every surface was matched with the exact same surface on the other element which led to the meshes not matching up. Forces were not being transferred properly, leading to peak tensions and distorted elements. For the new model only rectangular surfaces have been used and every surface has been matched. This led to more surfaces but no complicated ones. It is possible to recreate the step plan from the previous model in order to construct a good model.

Step 1 (squared)

Step one is altering the geometry of the model from all the different shapes, to only square shapes. Some shapes are made straight in order to prevent problems. That means that from the model that was presented by CD20, the current model has been created and will be analyzed.

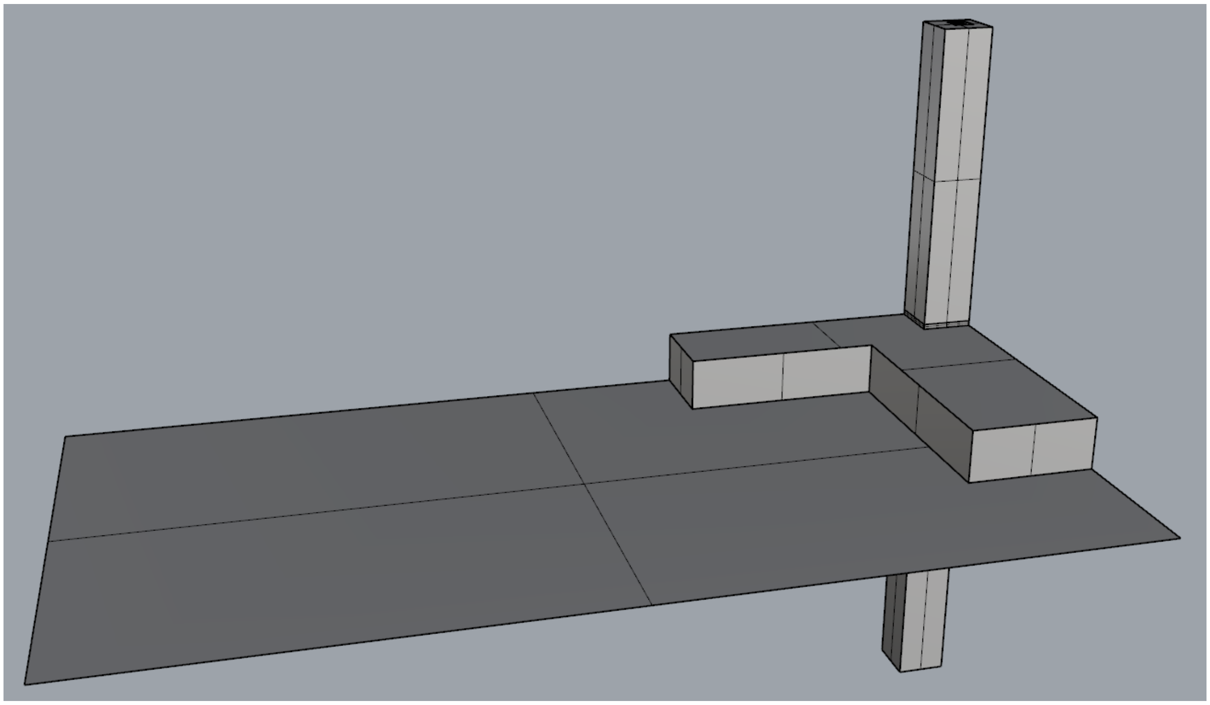


FIGURE 114 OVERVIEUW OF SQUARED DIANA VIEW

This model does not appear to be any different from the previous one, however when we remove the concrete and take a look at the corner shoe it will become clear what has changed.

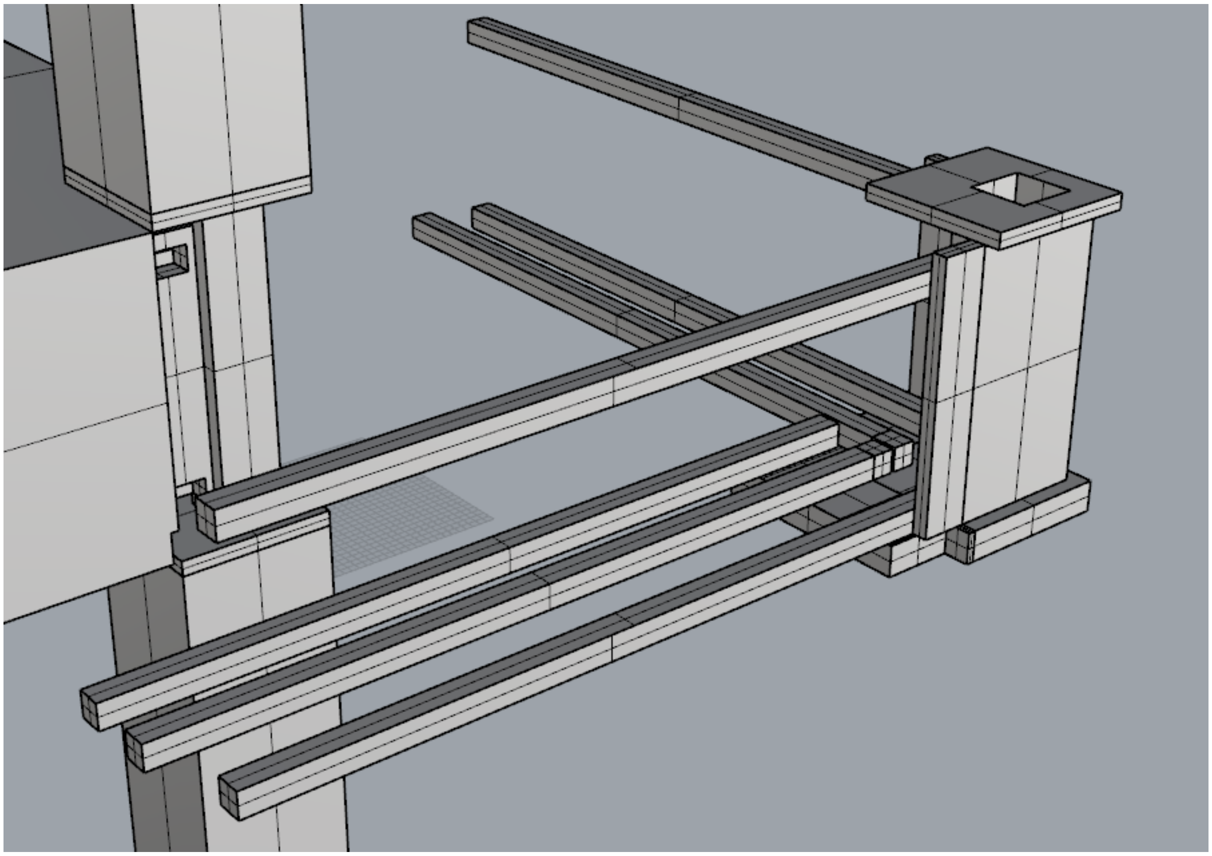


FIGURE 115 CLOSE-UP OF THE CORNER SHOE

The rebars do not look like this in real life, they have a cylindrical shape. By modeling them as extended cubical shapes, the meshing later on in the process is simplified. As can be seen in the first picture, there are two different shapes used to represent the same floor area. This has been done in order to save calculation time and to make the calculations use less memory. The computer that has to do the calculations is not as good as is required to make the model larger or more complicated.

Step 2(squared)

The second point is to translate the current Rhinoceros file to a working Diana model. At this point the problems arose last time. The Rhinoceros file has been severely altered and simplified in order to increase the chance of success. The different surface areas are free of overlaps and only matching surfaces are in existence right now. This should make sure that matching the different meshes should give no trouble this time.

However, there was a problem this time as well as the metal and the concrete parts were meshed at the same time. This should not pose a problem but at the connection between the concrete and the corner shoe there was something amiss, the mesh had errors and could not be constructed.

In order to solve this, one or two additional divisions were made in the geometry, area's that still posed a problem and were missed the first time. The second solution and maybe the most important difference, is the change of meshing order. The meshing has started from the inside out to first match the mesh of the concrete surrounding the rebar to the rebar. Once this mesh connected the other meshes could be matched to the already existing mesh. Meshing could be done from the inside out, this way most meshes only had to match one other mesh.

Finally this resulted in a meshed model of easier shapes than the previous one and matching meshes due to the orders given during the meshing. Every mesh created had to match the aligning meshes. This should result in a model where the connections between the different mesh sets are complete and the forces can be transferred completely.

Step 3(squared)

The third step is assigning the right materials to the elements, which possessed no problems or challenges. The right values can be implemented into the database and Diana will use these values to calculate the right solution. The use of Mesh edit has however given the opportunity to use a large database of materials that is stored within the program. This saves time and makes the modeled material more accurate.

Step 4(squared)

The point of this step is to make sure the forces are transferred in the correct way from the concrete to the steel shoe and onwards to the column. The problem with the normal system is that there is a connection between everything that has been meshed next to each other. That basically means that there is 100% connection between the concrete and the steel of the corner shoe. This leads to the majority of the force being transferred via tension of the concrete. That is of course a ludicrous idea and has to be corrected for the model to work properly. This can be achieved using interface elements which are elements that have no physical property but determine the way the mesh elements interact with each other. The problem with this was however that the interface elements did not present the desired behavior for the model.

Step 5(squared)

Applying the boundary conditions was not a difficult part of the modelling phase at first. Applying boundary conditions on its own is not difficult. However, applying the right boundary conditions for the right reasons is a different story. One has to go over the complete model and rethink why it looks like it does and what thet boundary conditions should be. The figure below shows the boundary conditions that are used in the model.

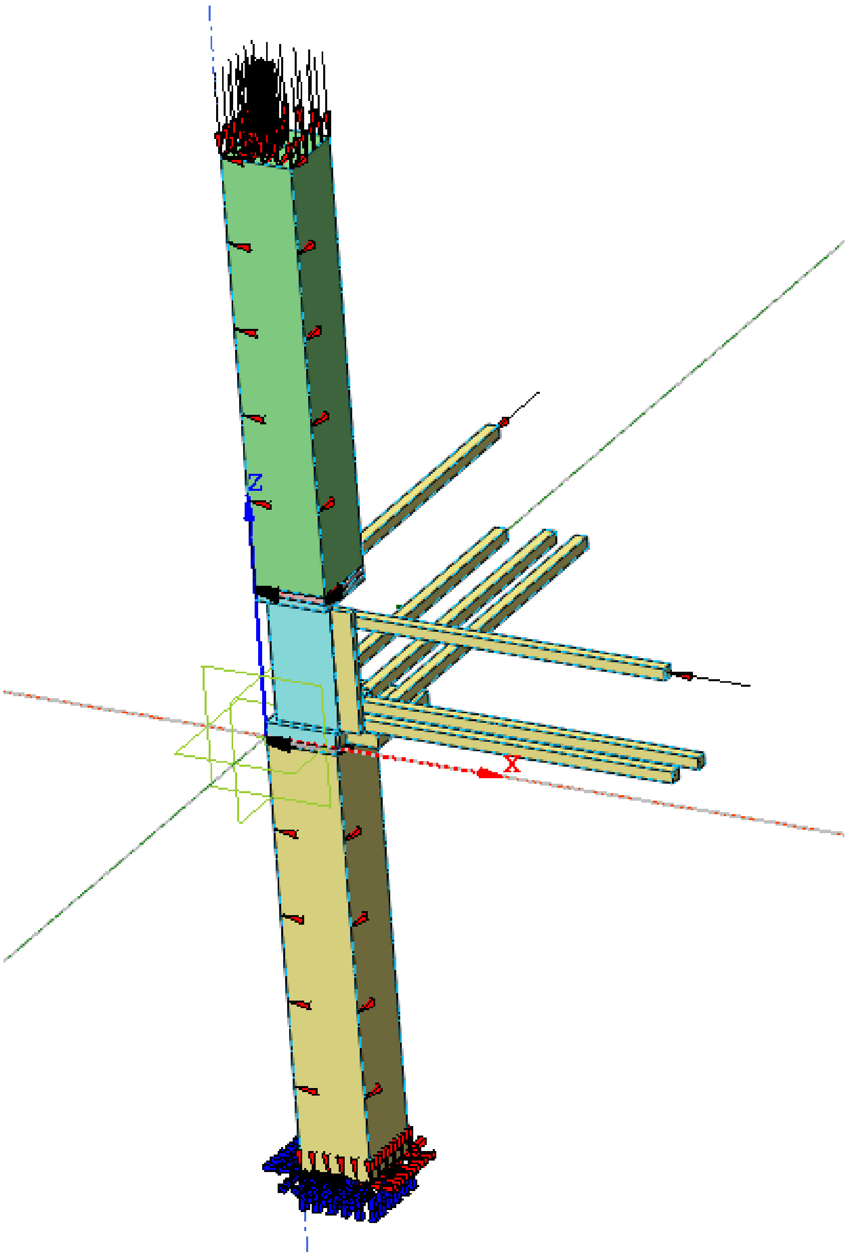


FIGURE 116 BOUNDARY CONDITIONS OF THE MODEL

Step 6(squared)

Applying the loads is really close to applying the restraints and is in fact a boundary condition. But the loads can change over time in order to do the checks which is the difference regarding these boundary conditions. There will be a vertical load on the column and a horizontal load or a moment on the floor slab. The normal force will be subject to change as well in order to see if that makes any difference.

Step 7(squared)

The test of the model presented the following results.

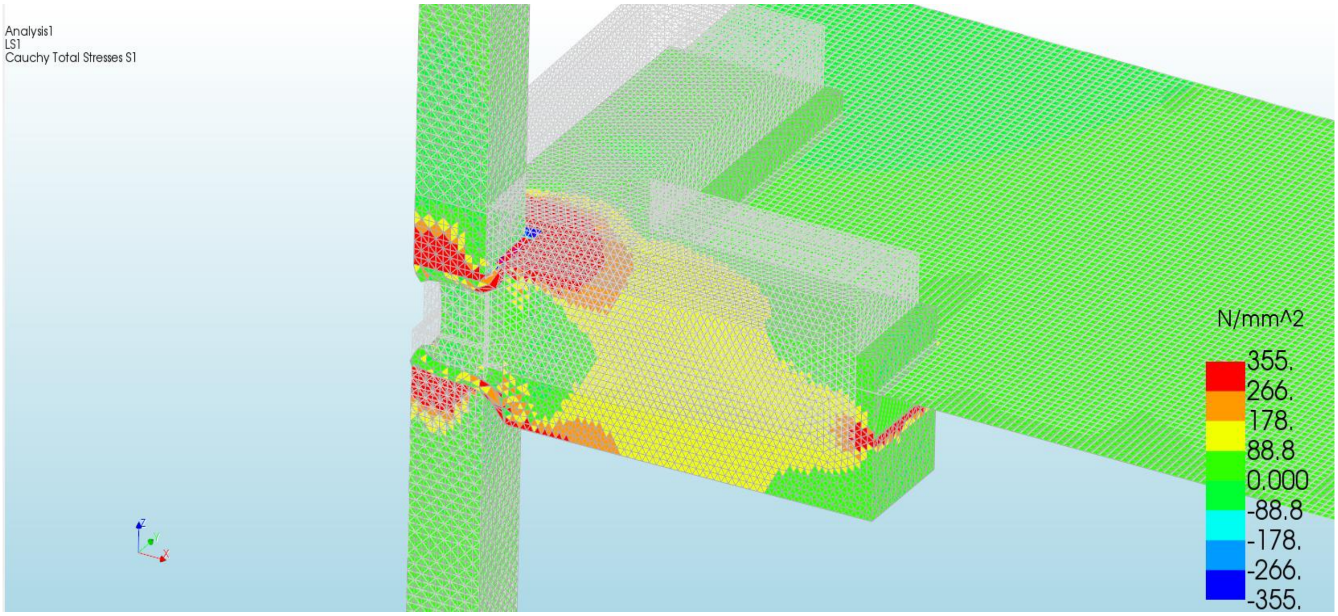


FIGURE 117 RESULTS OF FIRST TEST

Figure 117 presents both good and bad values. The force is nicely distributed from the plate to the block part of the plate. The distribution of the forces between the two column sections is good as well. However, when the model is dissected one can clearly see the problems that occur.

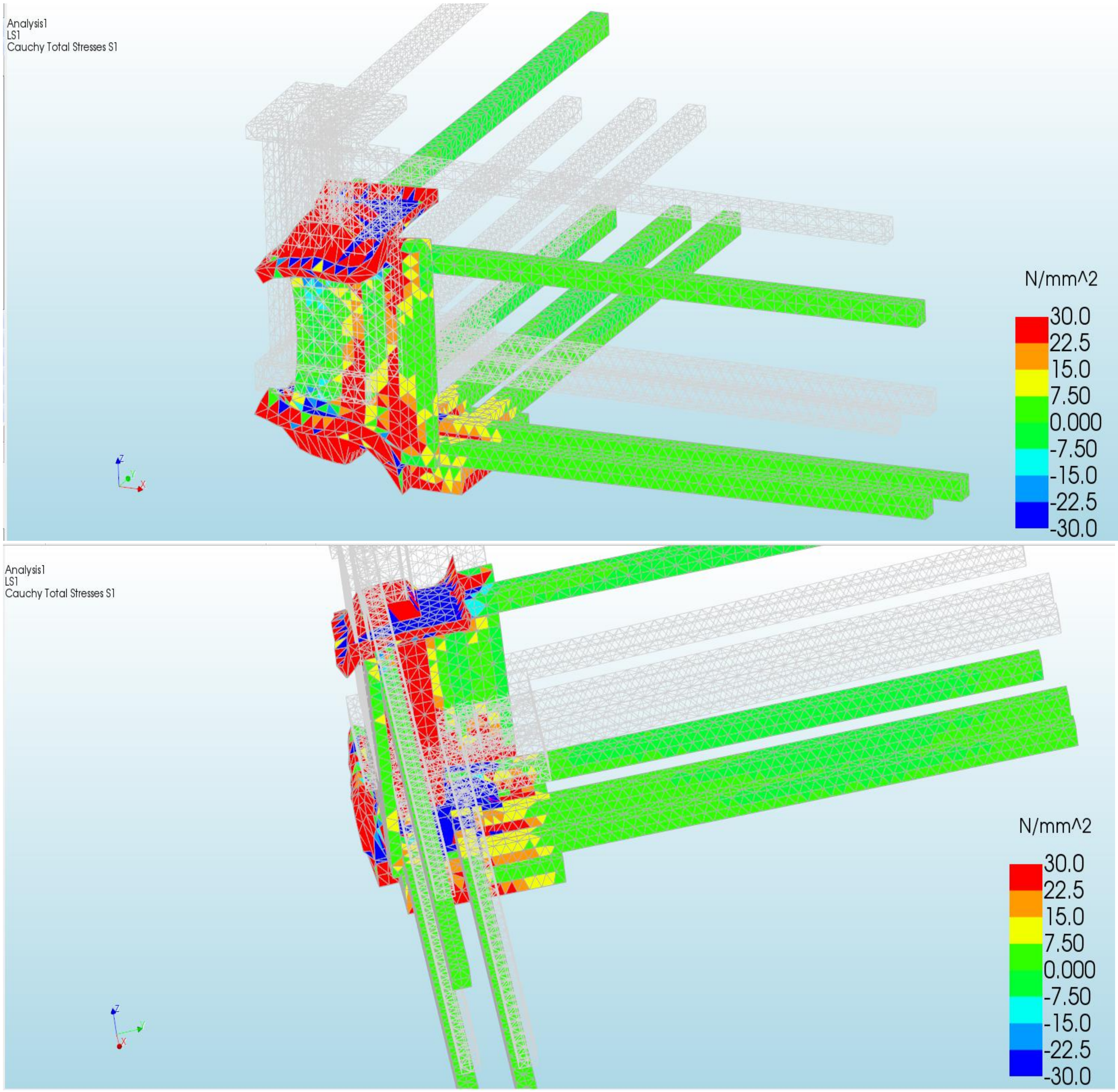


FIGURE 118 RESULTS OF THE TENSIONS IN THE CORNER SHOE

The reinforcement gives a value of zero, that means that the forces are being transferred completely wrong. The forces are now being transferred via tension on the steel body of the corner shoe, which is a problem because this is not the way the forces are transferred normally. Also disturbing is the shape of the deformed model as can be seen in figure 119.

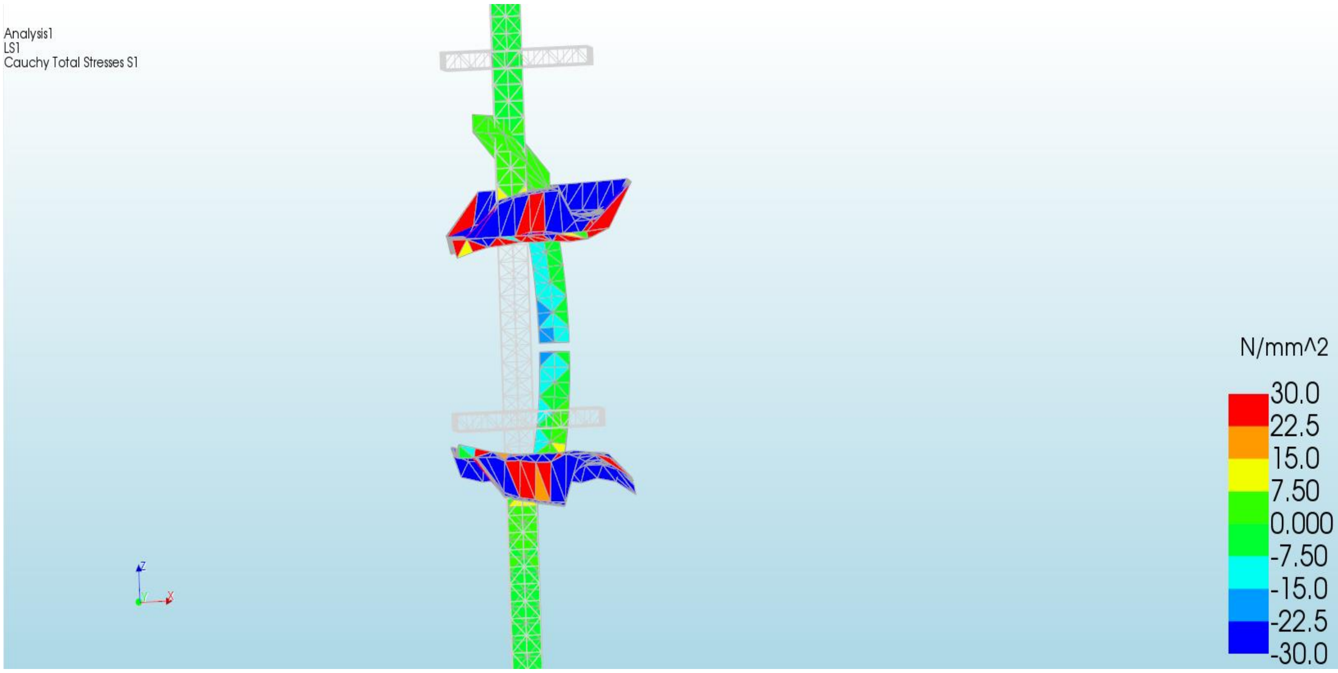


FIGURE 119 PROBLEM WITH THE DISPLACEMENTS

It is visible that the shape of the plate is strange, but upon a closer look one can see that a link is missing between parts of the steel plate. There is a piece of plate that is attached to the anchor that is floating above the rest of the plate. On top of that, the part is without any stress, so the model is failing and it cannot be used in order to make a good displacement-force diagram in order to create a spring stiffness for the Scia model of the entire building.

After all the work that has been put in and the disappointing results after reviewing those results, it is clear that this model does not suffice and changes have to be made. The main problem occurs when translating the forces from the concrete to the steel section, from the concrete in the floor slab to the steel of the corner shoe. In order to solve this problem the concrete around the anchors has been removed and the forces are applied directly on the steel of the corner shoe. This way the forces are working the right way when reaching the connection.

The concrete around the anchors was there to mimic the reality as much as possible. In order to lead the forces into the anchor the way they would in reality. The analysis that will be carried out is a displacement induced analysis. That means that the anchors will be given a displacement and the forces that are caused due to the displacement will be the output. Removing the concrete around the anchors does leave the concrete that influences the connection in place which is the concrete inside the corner shoe and the concrete between the bottom and top side of the corner shoe. This makes sure the outcome is not different due to a bad setup of the model. The final thing is the traction between the steel plate of the column and the steel plate of the corner shoe. Normally there would be resistance, thus creating more capacity for the connection. However, the connection will be tested in an earthquake environment and therefore traction between these two elements is not a reliable occurrence. Due to the shaking of the building the tremors could and most likely will cause the two plates to have some space in between them, allowing no transference of forces.

9.3. Force displacement table

1							tension inside the three anchors												combined	0	displacement of the corner shoe				0
2	Load-step	1	0.01	183441	37285	37.01155	Load-step	1	0.01	153218	47471	36.09462	Load-step	1	0.01	178992	49428	34.63951		27.58289	Load-step	1	0.01	44954	0.019082
3	Load-step	1	0.01	183441	37384	37.01155	Load-step	1	0.01	153218	36456	36.09462	Load-step	1	0.01	178992	44897	34.63951		27.58289	Load-step	1	0.01	44955	0.019029
4	Load-step	1	0.01	183441	38088	37.01155	Load-step	1	0.01	153218	36101	36.09462	Load-step	1	0.01	178992	44708	34.63951		27.58289	Load-step	1	0.01	45033	0.019057
5	Load-step	1	0.01	183441	37284	37.01155	Load-step	1	0.01	153218	36277	36.09462	Load-step	1	0.01	178992	44709	34.63951		27.58289	Load-step	1	0.01	45221	0.019057
6	Load-step	2	0.02	183441	37285	71.85715	Load-step	2	0.02	153218	47471	70.06116	Load-step	2	0.02	178992	49428	66.70891		53.40857	Load-step	2	0.02	44954	0.042462
7	Load-step	2	0.02	183441	37384	71.85715	Load-step	2	0.02	153218	36456	70.06116	Load-step	2	0.02	178992	44897	66.70891		53.40857	Load-step	2	0.02	44955	0.042281
8	Load-step	2	0.02	183441	38088	71.85715	Load-step	2	0.02	153218	36101	70.06116	Load-step	2	0.02	178992	44708	66.70891		53.40857	Load-step	2	0.02	45033	0.042363
9	Load-step	2	0.02	183441	37284	71.85715	Load-step	2	0.02	153218	36277	70.06116	Load-step	2	0.02	178992	44709	66.70891		53.40857	Load-step	2	0.02	45221	0.042432
10	Load-step	3	0.03	183441	37285	105.1528	Load-step	3	0.03	153218	47471	100.2372	Load-step	3	0.03	178992	49428	94.68119		76.81823	Load-step	3	0.03	44954	0.072166
11	Load-step	3	0.03	183441	37384	105.1528	Load-step	3	0.03	153218	36456	100.2372	Load-step	3	0.03	178992	44897	94.68119		76.81823	Load-step	3	0.03	44955	0.071668
12	Load-step	3	0.03	183441	38088	105.1528	Load-step	3	0.03	153218	36101	100.2372	Load-step	3	0.03	178992	44708	94.68119		76.81823	Load-step	3	0.03	45033	0.071866
13	Load-step	3	0.03	183441	37284	105.1528	Load-step	3	0.03	153218	36277	100.2372	Load-step	3	0.03	178992	44709	94.68119		76.81823	Load-step	3	0.03	45221	0.072178
14	Load-step	4	0.04	183441	37285	133.3184	Load-step	4	0.04	153218	47471	108.4343	Load-step	4	0.04	178992	49428	105.2187		88.82466	Load-step	4	0.04	44954	0.144258
15	Load-step	4	0.04	183441	37384	133.3184	Load-step	4	0.04	153218	36456	108.4343	Load-step	4	0.04	178992	44897	105.2187		88.82466	Load-step	4	0.04	44955	0.142176
16	Load-step	4	0.04	183441	38088	133.3184	Load-step	4	0.04	153218	36101	108.4343	Load-step	4	0.04	178992	44708	105.2187		88.82466	Load-step	4	0.04	45033	0.142628
17	Load-step	4	0.04	183441	37284	133.3184	Load-step	4	0.04	153218	36277	108.4343	Load-step	4	0.04	178992	44709	105.2187		88.82466	Load-step	4	0.04	45221	0.142997
18	Load-step	5	0.05	183441	37285	159.2854	Load-step	5	0.05	153218	47471	104.0724	Load-step	5	0.05	178992	49428	107.5842		94.96116	Load-step	5	0.05	44954	0.243804
19	Load-step	5	0.05	183441	37384	159.2854	Load-step	5	0.05	153218	36456	104.0724	Load-step	5	0.05	178992	44897	107.5842		94.96116	Load-step	5	0.05	44955	0.239385
20	Load-step	5	0.05	183441	38088	159.2854	Load-step	5	0.05	153218	36101	104.0724	Load-step	5	0.05	178992	44708	107.5842		94.96116	Load-step	5	0.05	45033	0.239829
21	Load-step	5	0.05	183441	37284	159.2854	Load-step	5	0.05	153218	36277	104.0724	Load-step	5	0.05	178992	44709	107.5842		94.96116	Load-step	5	0.05	45221	0.240171
22	Load-step	6	0.06	183441	37285	183.1981	Load-step	6	0.06	153218	47471	98.92365	Load-step	6	0.06	178992	49428	110.3441		100.4712	Load-step	6	0.06	44954	0.347781
23	Load-step	6	0.06	183441	37384	183.1981	Load-step	6	0.06	153218	36456	98.92365	Load-step	6	0.06	178992	44897	110.3441		100.4712	Load-step	6	0.06	44955	0.341145
24	Load-step	6	0.06	183441	38088	183.1981	Load-step	6	0.06	153218	36101	98.92365	Load-step	6	0.06	178992	44708	110.3441		100.4712	Load-step	6	0.06	45033	0.341422
25	Load-step	6	0.06	183441	37284	183.1981	Load-step	6	0.06	153218	36277	98.92365	Load-step	6	0.06	178992	44709	110.3441		100.4712	Load-step	6	0.06	45221	0.341616
26	Load-step	7	0.07	183441	37285	205.1061	Load-step	7	0.07	153218	47471	93.10218	Load-step	7	0.07	178992	49428	113.6275		105.43	Load-step	7	0.07	44954	0.455722
27	Load-step	7	0.07	183441	37384	205.1061	Load-step	7	0.07	153218	36456	93.10218	Load-step	7	0.07	178992	44897	113.6275		105.43	Load-step	7	0.07	44955	0.446955
28	Load-step	7	0.07	183441	38088	205.1061	Load-step	7	0.07	153218	36101	93.10218	Load-step	7	0.07	178992	44708	113.6275		105.43	Load-step	7	0.07	45033	0.44696
29	Load-step	7	0.07	183441	37284	205.1061	Load-step	7	0.07	153218	36277	93.10218	Load-step	7	0.07	178992	44709	113.6275		105.43	Load-step	7	0.07	45221	0.446817
30	Load-step	8	0.08	183441	37285	218.6775	Load-step	8	0.08	153218	47471	86.64448	Load-step	8	0.08	178992	49428	117.494		108.2409	Load-step	8	0.08	44954	0.569977
31	Load-step	8	0.08	183441	37384	218.6775	Load-step	8	0.08	153218	36456	86.64448	Load-step	8	0.08	178992	44897	117.494		108.2409	Load-step	8	0.08	44955	0.559611
32	Load-step	8	0.08	183441	38088	218.6775	Load-step	8	0.08	153218	36101	86.64448	Load-step	8	0.08	178992	44708	117.494		108.2409	Load-step	8	0.08	45033	0.559295
33	Load-step	8	0.08	183441	37284	218.6775	Load-step	8	0.08	153218	36277	86.64448	Load-step	8	0.08	178992	44709	117.494		108.2409	Load-step	8	0.08	45221	0.558084
34	Load-step	9	0.09	183441	37285	222.9638	Load-step	9	0.09	153218	47471	80.9127	Load-step	9	0.09	178992	49428	121.1698		108.8119	Load-step	9	0.09	44954	0.685291
35	Load-step	9	0.09	183441	37384	222.9638	Load-step	9	0.09	153218	36456	80.9127	Load-step	9	0.09	178992	44897	121.1698		108.8119	Load-step	9	0.09	44955	0.674077
36	Load-step	9	0.09	183441	38088	222.9638	Load-step	9	0.09	153218	36101	80.9127	Load-step	9	0.09	178992	44708	121.1698		108.8119	Load-step	9	0.09	45033	0.673557
37	Load-step	9	0.09	183441	37284	222.9638	Load-step	9	0.09	153218	36277	80.9127	Load-step	9	0.09	178992	44709	121.1698		108.8119	Load-step	9	0.09	45221	0.6707
38	Load-step	10	0.1	183441	37285	222.1016	Load-step	10	0.1	153218	47471	78.41242	Load-step	10	0.1	178992	49428	122.8443		108.3797	Load-step	10	0.1	44954	0.788998
39	Load-step	10	0.1	183441	37384	222.1016	Load-step	10	0.1	153218	36456	78.41242	Load-step	10	0.1	178992	44897	122.8443		108.3797	Load-step	10	0.1	44955	0.777559

40	Load-step	10	0.1	183441	38088	222.1016	Load-step	10	0.1	153218	36101	78.41242	Load-step	10	0.1	178992	44708	122.8443	108.3797	Load-step	10	0.1	45033	0.777319
41	Load-step	10	0.1	183441	37284	222.1016	Load-step	10	0.1	153218	36277	78.41242	Load-step	10	0.1	178992	44709	122.8443	108.3797	Load-step	10	0.1	45221	0.773018
42	Load-step	11	0.11	183441	37285	221.4359	Load-step	11	0.11	153218	47471	77.81014	Load-step	11	0.11	178992	49428	123.1778	108.1405	Load-step	11	0.11	44954	0.885016
43	Load-step	11	0.11	183441	37384	221.4359	Load-step	11	0.11	153218	36456	77.81014	Load-step	11	0.11	178992	44897	123.1778	108.1405	Load-step	11	0.11	44955	0.873495
44	Load-step	11	0.11	183441	38088	221.4359	Load-step	11	0.11	153218	36101	77.81014	Load-step	11	0.11	178992	44708	123.1778	108.1405	Load-step	11	0.11	45033	0.873759
45	Load-step	11	0.11	183441	37284	221.4359	Load-step	11	0.11	153218	36277	77.81014	Load-step	11	0.11	178992	44709	123.1778	108.1405	Load-step	11	0.11	45221	0.868508
46	Load-step	12	0.12	183441	37285	221.3127	Load-step	12	0.12	153218	47471	77.62044	Load-step	12	0.12	178992	49428	123.2754	108.0854	Load-step	12	0.12	44954	0.980226
47	Load-step	12	0.12	183441	37384	221.3127	Load-step	12	0.12	153218	36456	77.62044	Load-step	12	0.12	178992	44897	123.2754	108.0854	Load-step	12	0.12	44955	0.96864
48	Load-step	12	0.12	183441	38088	221.3127	Load-step	12	0.12	153218	36101	77.62044	Load-step	12	0.12	178992	44708	123.2754	108.0854	Load-step	12	0.12	45033	0.969365
49	Load-step	12	0.12	183441	37284	221.3127	Load-step	12	0.12	153218	36277	77.62044	Load-step	12	0.12	178992	44709	123.2754	108.0854	Load-step	12	0.12	45221	0.963419
50	Load-step	13	0.13	183441	37285	221.2918	Load-step	13	0.13	153218	47471	77.59439	Load-step	13	0.13	178992	49428	123.2849	108.0758	Load-step	13	0.13	44954	1.075703
51	Load-step	13	0.13	183441	37384	221.2918	Load-step	13	0.13	153218	36456	77.59439	Load-step	13	0.13	178992	44897	123.2849	108.0758	Load-step	13	0.13	44955	1.064078
52	Load-step	13	0.13	183441	38088	221.2918	Load-step	13	0.13	153218	36101	77.59439	Load-step	13	0.13	178992	44708	123.2849	108.0758	Load-step	13	0.13	45033	1.0652
53	Load-step	13	0.13	183441	37284	221.2918	Load-step	13	0.13	153218	36277	77.59439	Load-step	13	0.13	178992	44709	123.2849	108.0758	Load-step	13	0.13	45221	1.058714
54	Load-step	14	0.14	183441	37285	221.3171	Load-step	14	0.14	153218	47471	77.67539	Load-step	14	0.14	178992	49428	123.2304	108.0891	Load-step	14	0.14	44954	1.171511
55	Load-step	14	0.14	183441	37384	221.3171	Load-step	14	0.14	153218	36456	77.67539	Load-step	14	0.14	178992	44897	123.2304	108.0891	Load-step	14	0.14	44955	1.159865
56	Load-step	14	0.14	183441	38088	221.3171	Load-step	14	0.14	153218	36101	77.67539	Load-step	14	0.14	178992	44708	123.2304	108.0891	Load-step	14	0.14	45033	1.161329
57	Load-step	14	0.14	183441	37284	221.3171	Load-step	14	0.14	153218	36277	77.67539	Load-step	14	0.14	178992	44709	123.2304	108.0891	Load-step	14	0.14	45221	1.154423
58	Load-step	15	0.15	183441	37285	221.3757	Load-step	15	0.15	153218	47471	77.84507	Load-step	15	0.15	178992	49428	123.1222	108.1198	Load-step	15	0.15	44954	1.267583
59	Load-step	15	0.15	183441	37384	221.3757	Load-step	15	0.15	153218	36456	77.84507	Load-step	15	0.15	178992	44897	123.1222	108.1198	Load-step	15	0.15	44955	1.25593
60	Load-step	15	0.15	183441	38088	221.3757	Load-step	15	0.15	153218	36101	77.84507	Load-step	15	0.15	178992	44708	123.1222	108.1198	Load-step	15	0.15	45033	1.257692
61	Load-step	15	0.15	183441	37284	221.3757	Load-step	15	0.15	153218	36277	77.84507	Load-step	15	0.15	178992	44709	123.1222	108.1198	Load-step	15	0.15	45221	1.250462
62	Load-step	16	0.16	183441	37285	221.457	Load-step	16	0.16	153218	47471	78.08038	Load-step	16	0.16	178992	49428	122.9764	108.1635	Load-step	16	0.16	44954	1.363908
63	Load-step	16	0.16	183441	37384	221.457	Load-step	16	0.16	153218	36456	78.08038	Load-step	16	0.16	178992	44897	122.9764	108.1635	Load-step	16	0.16	44955	1.352258
64	Load-step	16	0.16	183441	38088	221.457	Load-step	16	0.16	153218	36101	78.08038	Load-step	16	0.16	178992	44708	122.9764	108.1635	Load-step	16	0.16	45033	1.354281
65	Load-step	16	0.16	183441	37284	221.457	Load-step	16	0.16	153218	36277	78.08038	Load-step	16	0.16	178992	44709	122.9764	108.1635	Load-step	16	0.16	45221	1.346804
66	Load-step	17	0.17	183441	37285	221.5525	Load-step	17	0.17	153218	47471	78.36063	Load-step	17	0.17	178992	49428	122.8066	108.2163	Load-step	17	0.17	44954	1.460483
67	Load-step	17	0.17	183441	37384	221.5525	Load-step	17	0.17	153218	36456	78.36063	Load-step	17	0.17	178992	44897	122.8066	108.2163	Load-step	17	0.17	44955	1.448844
68	Load-step	17	0.17	183441	38088	221.5525	Load-step	17	0.17	153218	36101	78.36063	Load-step	17	0.17	178992	44708	122.8066	108.2163	Load-step	17	0.17	45033	1.451094
69	Load-step	17	0.17	183441	37284	221.5525	Load-step	17	0.17	153218	36277	78.36063	Load-step	17	0.17	178992	44709	122.8066	108.2163	Load-step	17	0.17	45221	1.44343
70	Load-step	18	0.18	183441	37285	221.66	Load-step	18	0.18	153218	47471	78.67199	Load-step	18	0.18	178992	49428	122.6201	108.2757	Load-step	18	0.18	44954	1.557293
71	Load-step	18	0.18	183441	37384	221.66	Load-step	18	0.18	153218	36456	78.67199	Load-step	18	0.18	178992	44897	122.6201	108.2757	Load-step	18	0.18	44955	1.545669
72	Load-step	18	0.18	183441	38088	221.66	Load-step	18	0.18	153218	36101	78.67199	Load-step	18	0.18	178992	44708	122.6201	108.2757	Load-step	18	0.18	45033	1.548118
73	Load-step	18	0.18	183441	37284	221.66	Load-step	18	0.18	153218	36277	78.67199	Load-step	18	0.18	178992	44709	122.6201	108.2757	Load-step	18	0.18	45221	1.540317
74	Load-step	19	0.19	183441	37285	221.7607	Load-step	19	0.19	153218	47471	78.97339	Load-step	19	0.19	178992	49428	122.4415	108.333	Load-step	19	0.19	44954	1.654428
75	Load-step	19	0.19	183441	37384	221.7607	Load-step	19	0.19	153218	36456	78.97339	Load-step	19	0.19	178992	44897	122.4415	108.333	Load-step	19	0.19	44955	1.642822
76	Load-step	19	0.19	183441	38088	221.7607	Load-step	19	0.19	153218	36101	78.97339	Load-step	19	0.19	178992	44708	122.4415	108.333	Load-step	19	0.19	45033	1.645444
77	Load-step	19	0.19	183441	37284	221.7607	Load-step	19	0.19	153218	36277	78.97339	Load-step	19	0.19	178992	44709	122.4415	108.333	Load-step	19	0.19	45221	1.637535
78	Load-step	20	0.2	183441	37285	221.8539	Load-step	20	0.2	153218	47471	79.25999	Load-step	20	0.2	178992	49428	122.2732	108.3871	Load-step	20	0.2	44954	1.751857

The remainder of the pages leads to no additional forces and give the same

9.4. Scia images

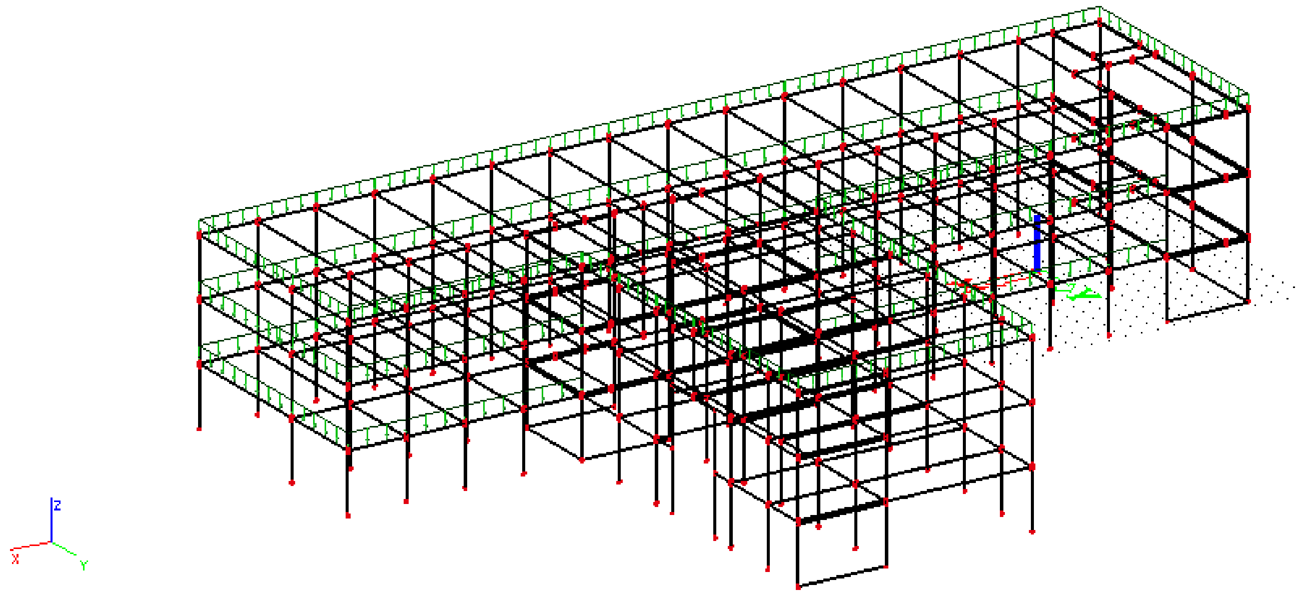


FIGURE 120 LINE MODEL IN SCIA WITH LOADS

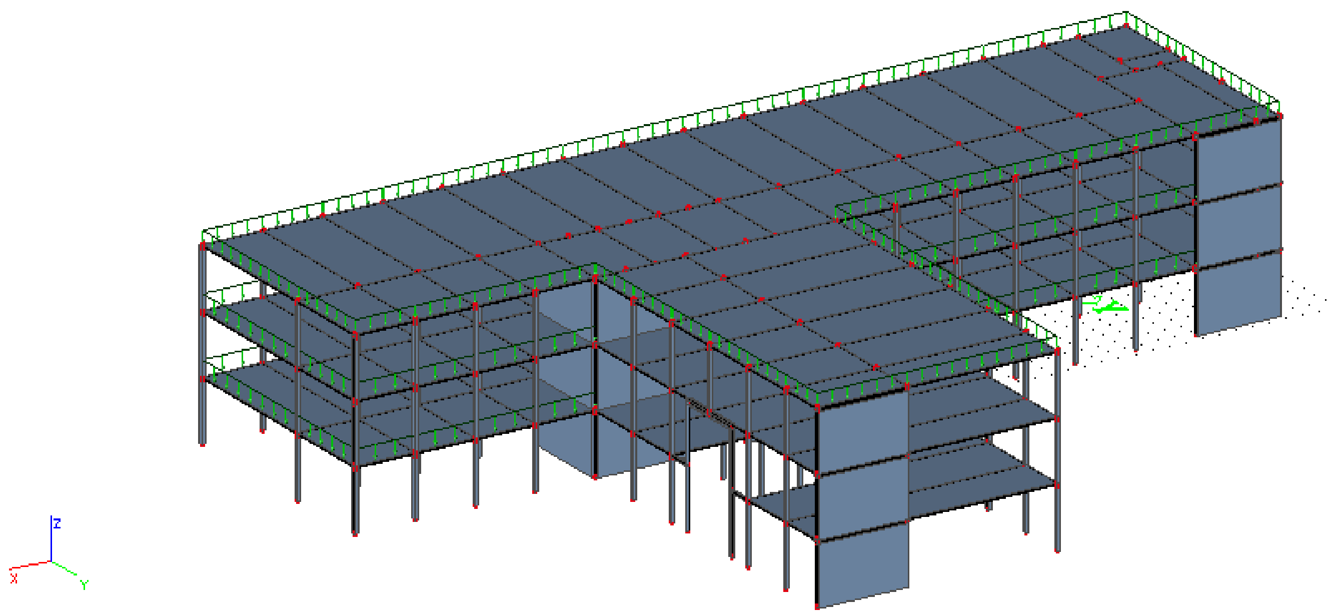


FIGURE 121 FILLED OUT MODEL OF THE CONSTRUCTION WITH LOADS

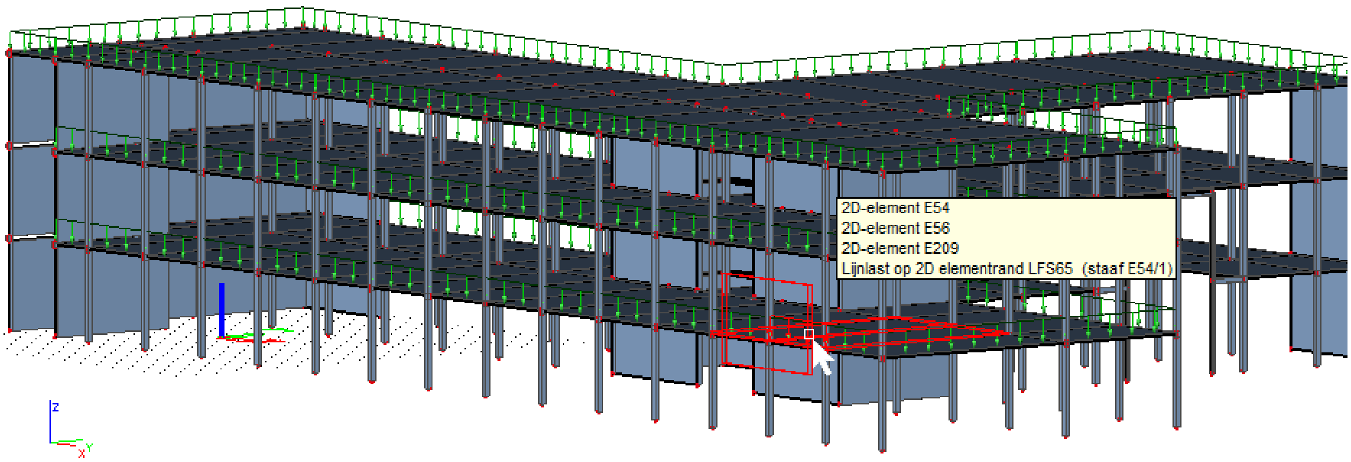


FIGURE 122 ALTERNATIVE VIEW OF BUILDING

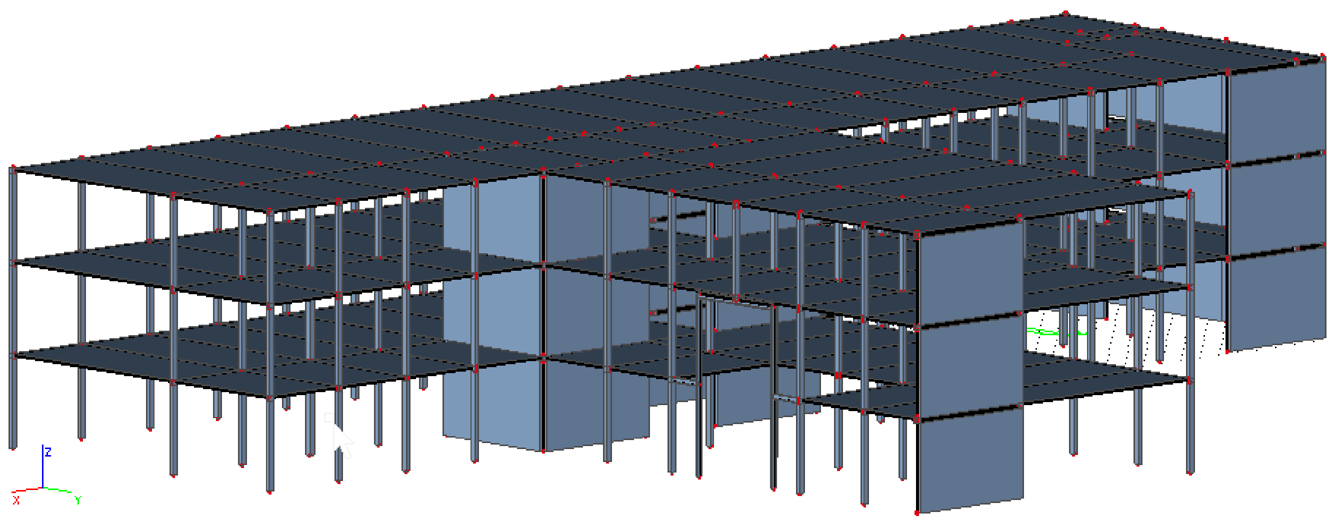


FIGURE 123 BUILDING WITHOUTH LOADING

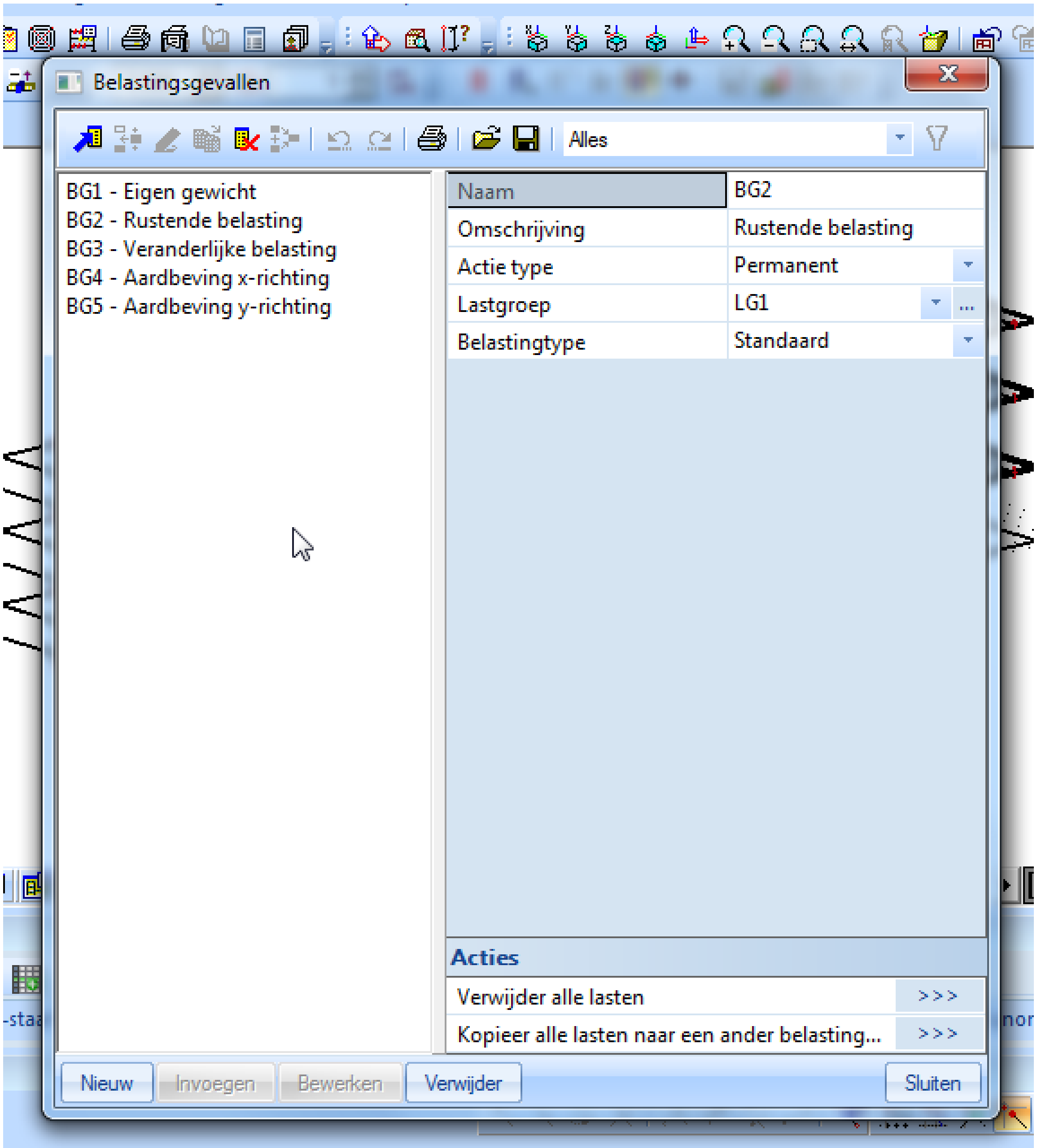


FIGURE 124 LOAD CASES

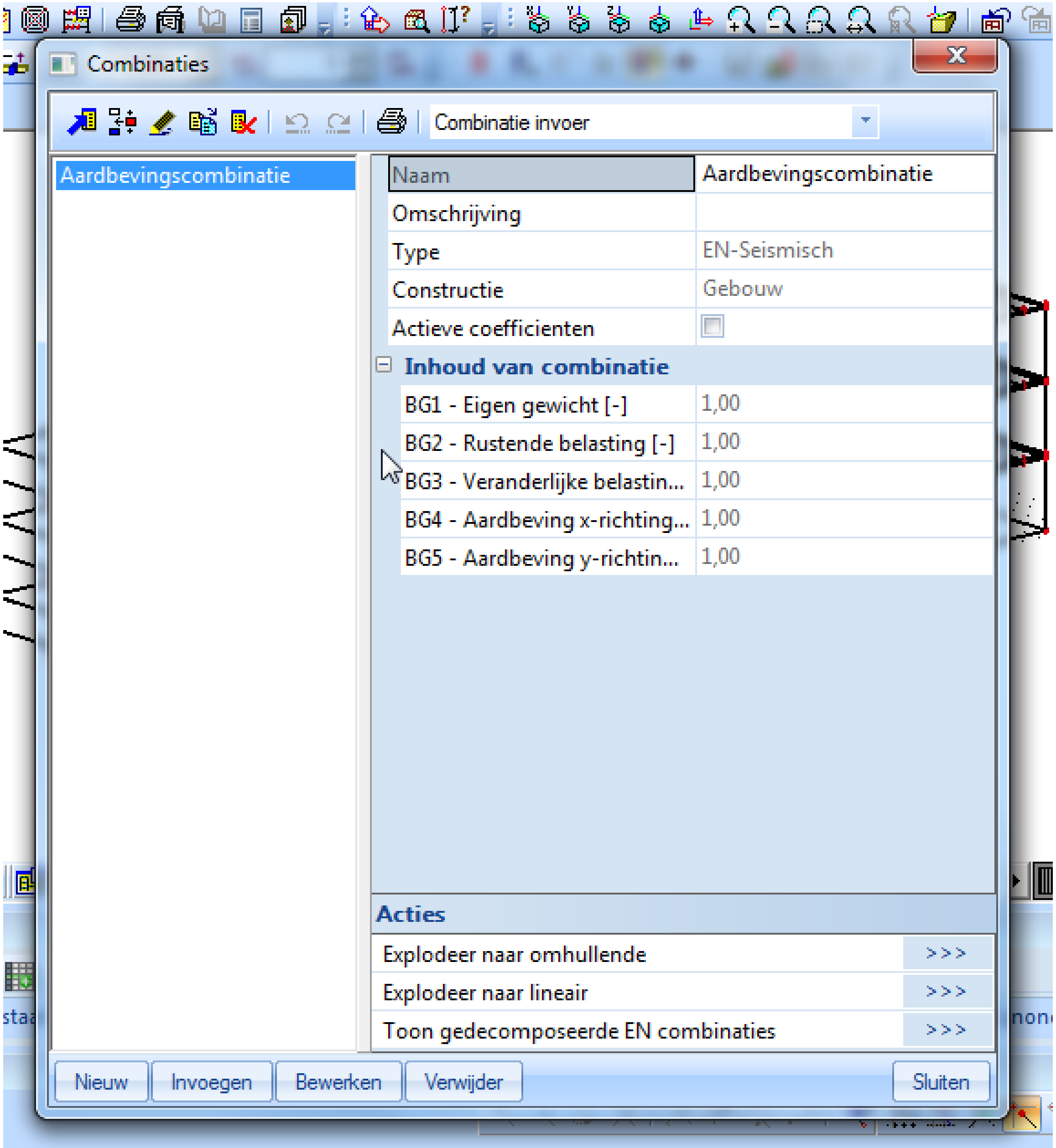


FIGURE 125 LOAD COMBINATIONS

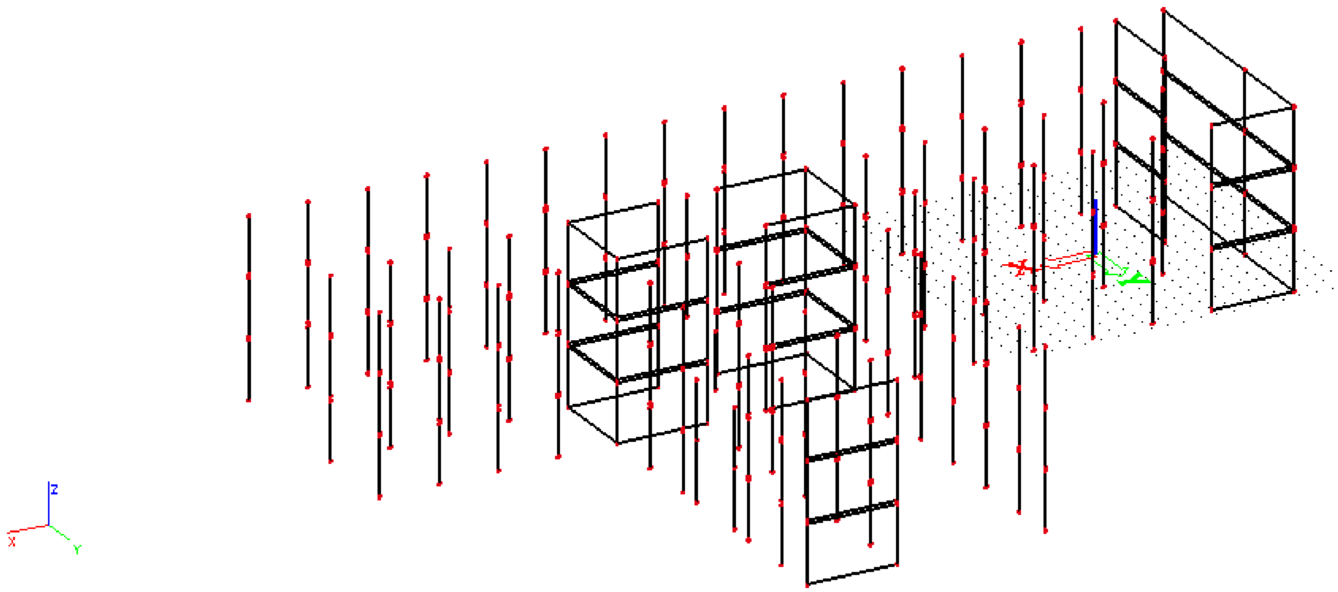


FIGURE 126 COLUMNS AND STABILITY WALLS

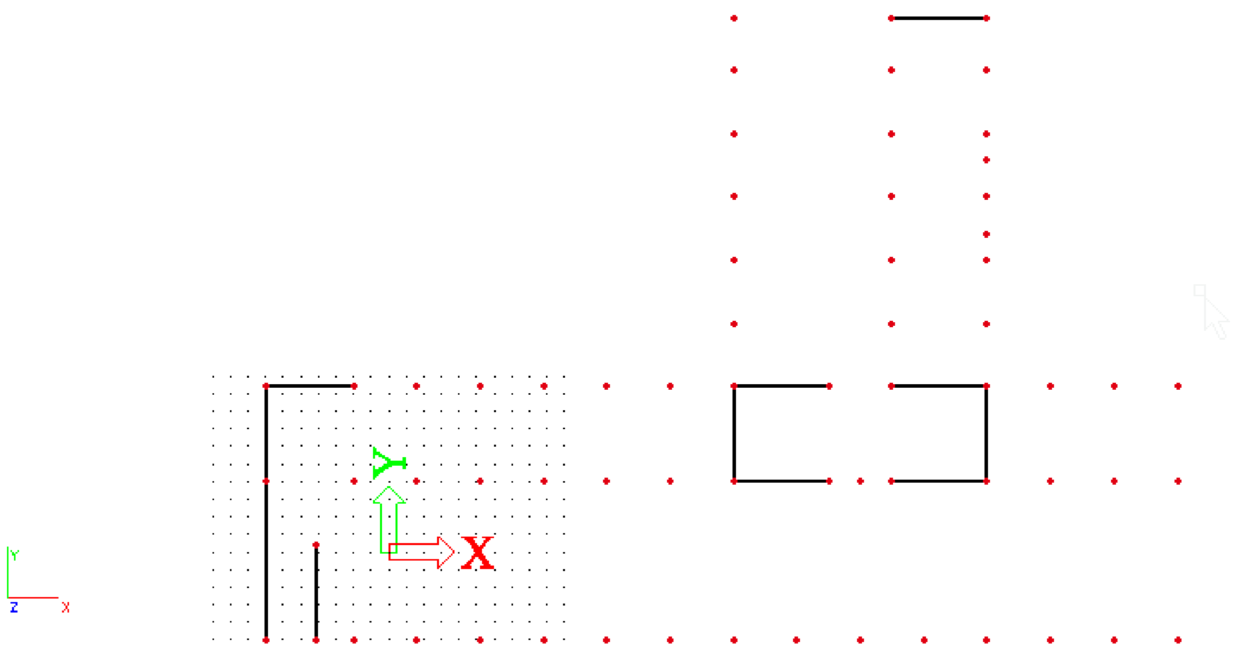


FIGURE 127 TOP VIEW OF COLUMNS AND STABILITY WALLS

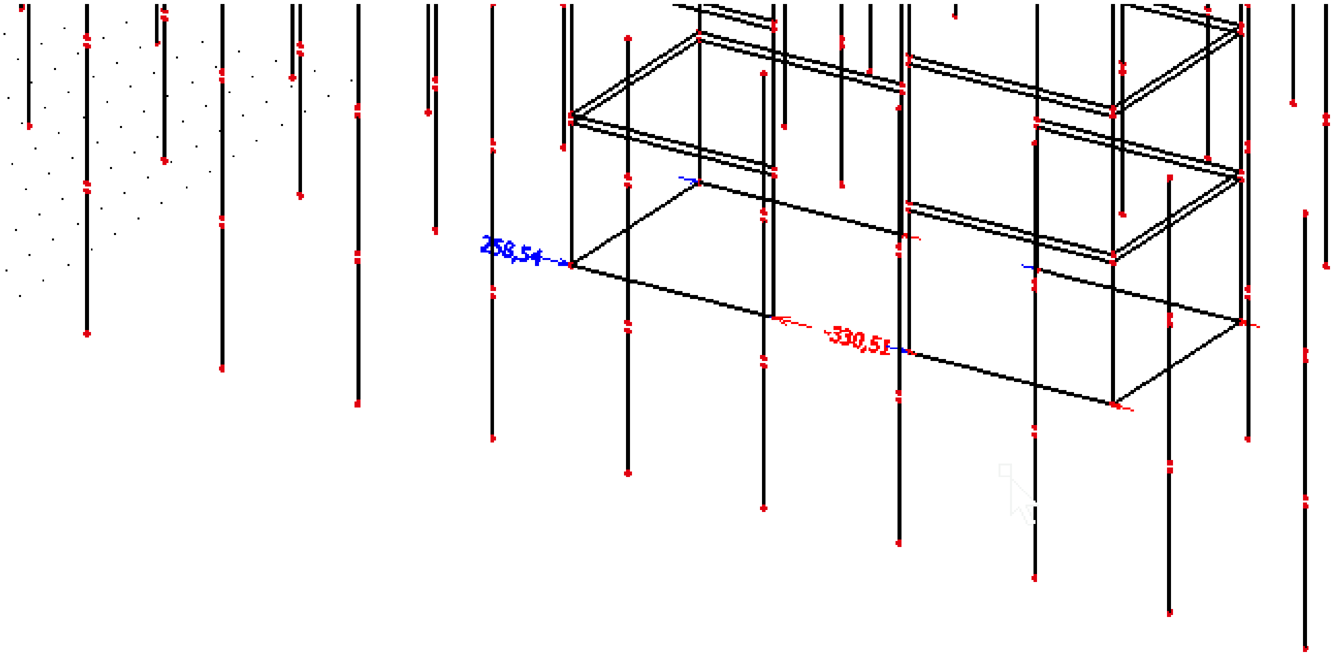


FIGURE 128 MAXIMUM HORIZONTAL REACTION FORCES ON THE WALLS

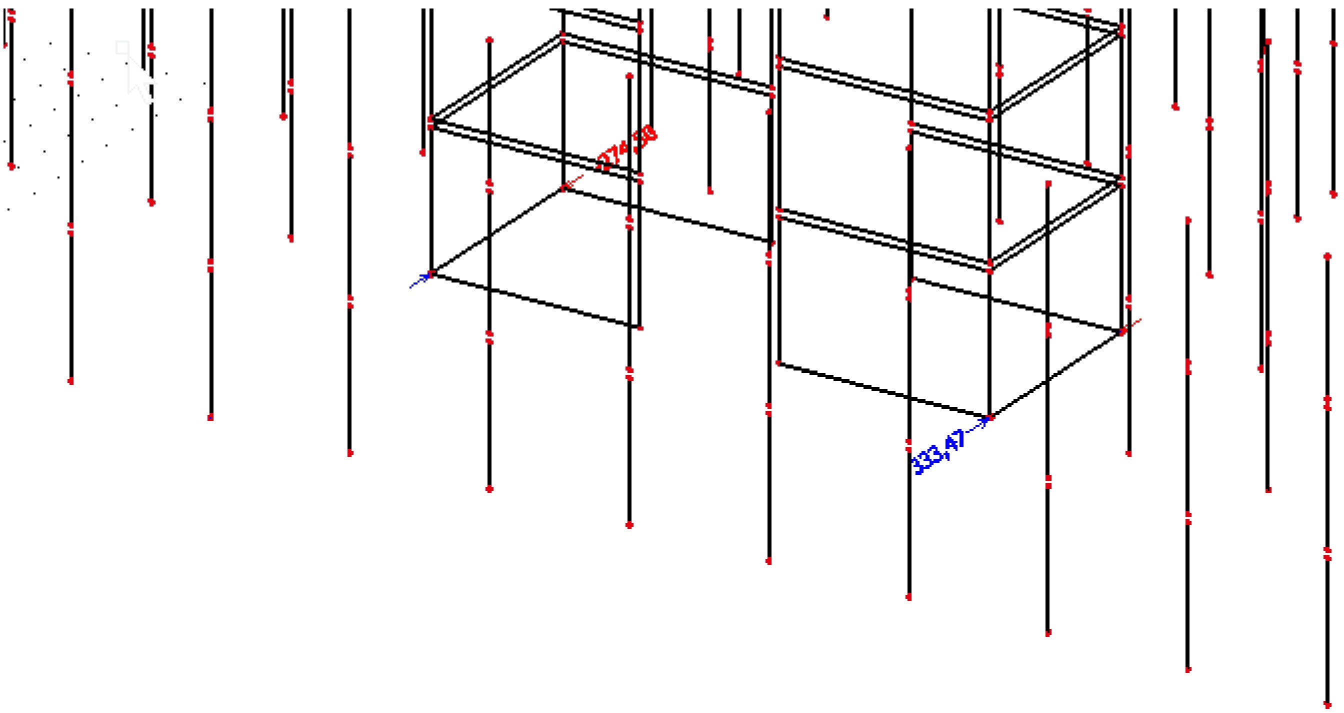


FIGURE 129 MAXIMUM HORRIZONTAL REACTION FORCES ON THE WALLS IN THE SECOND GENERAL DIRECTION

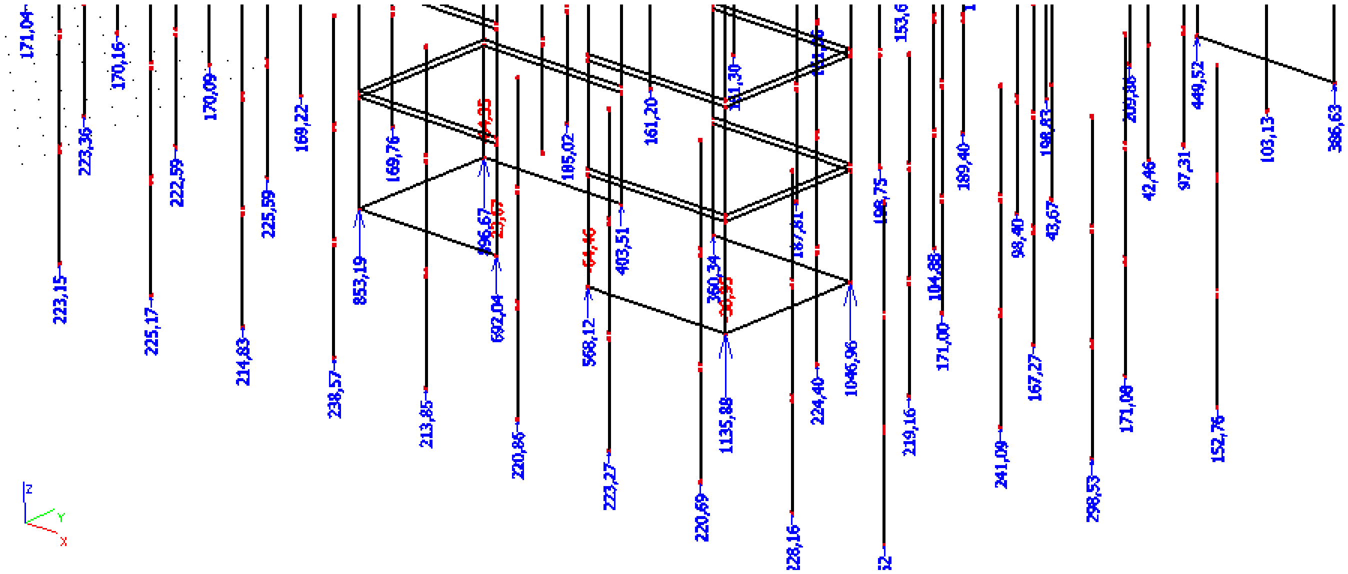


FIGURE 130 OVERVIEW OF REACTION FORCES IN Z DIRECTION

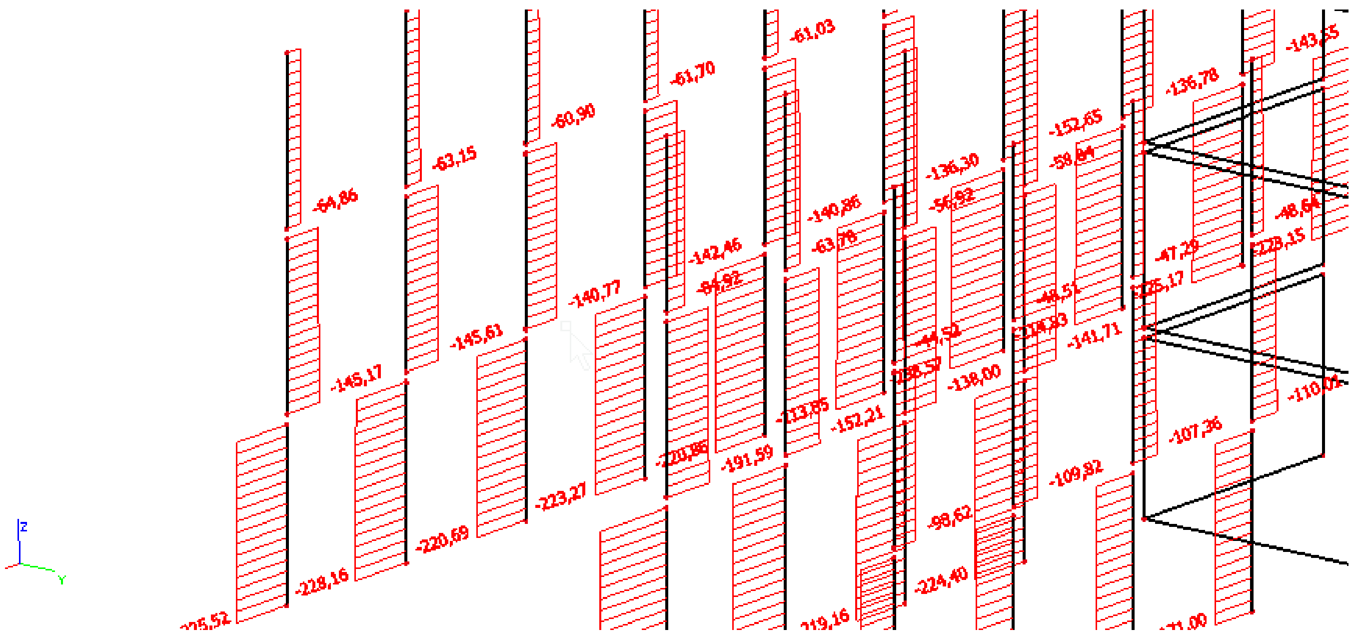


FIGURE 131 NORMAL FORCES ON THE COLUMNS

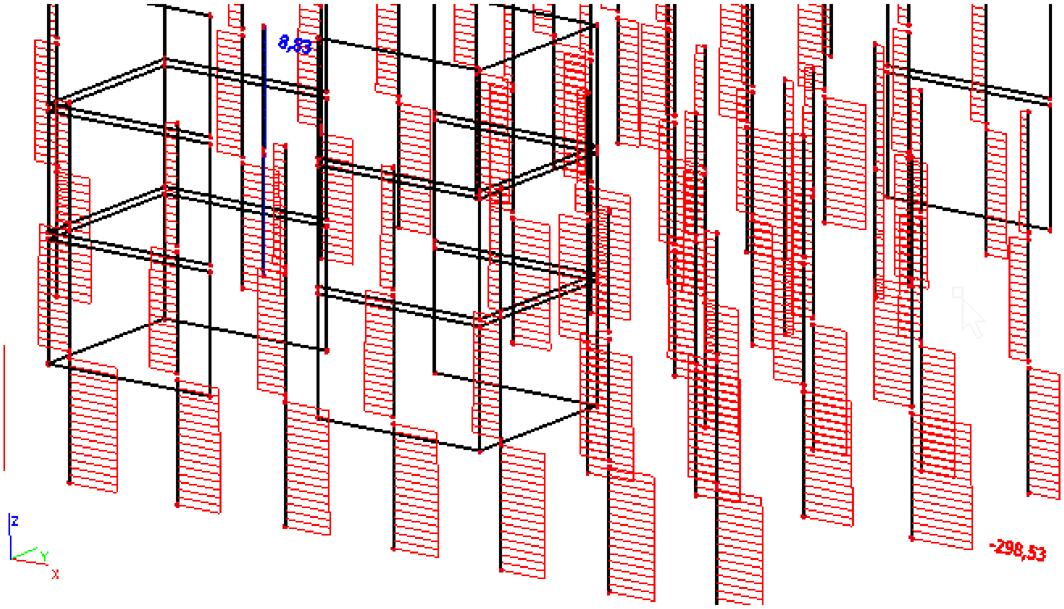


FIGURE 132 MAXIMUM NORMAL FORCES ON THE COLUMNS

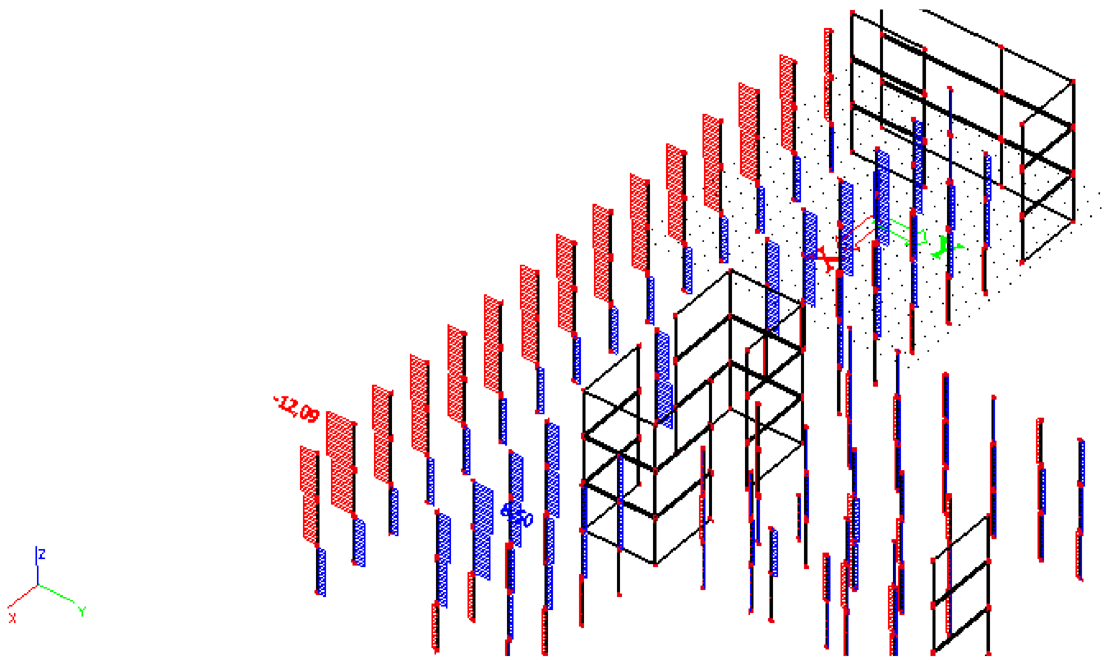


FIGURE 133 MAXIMUM HORIZONTAL FORCES

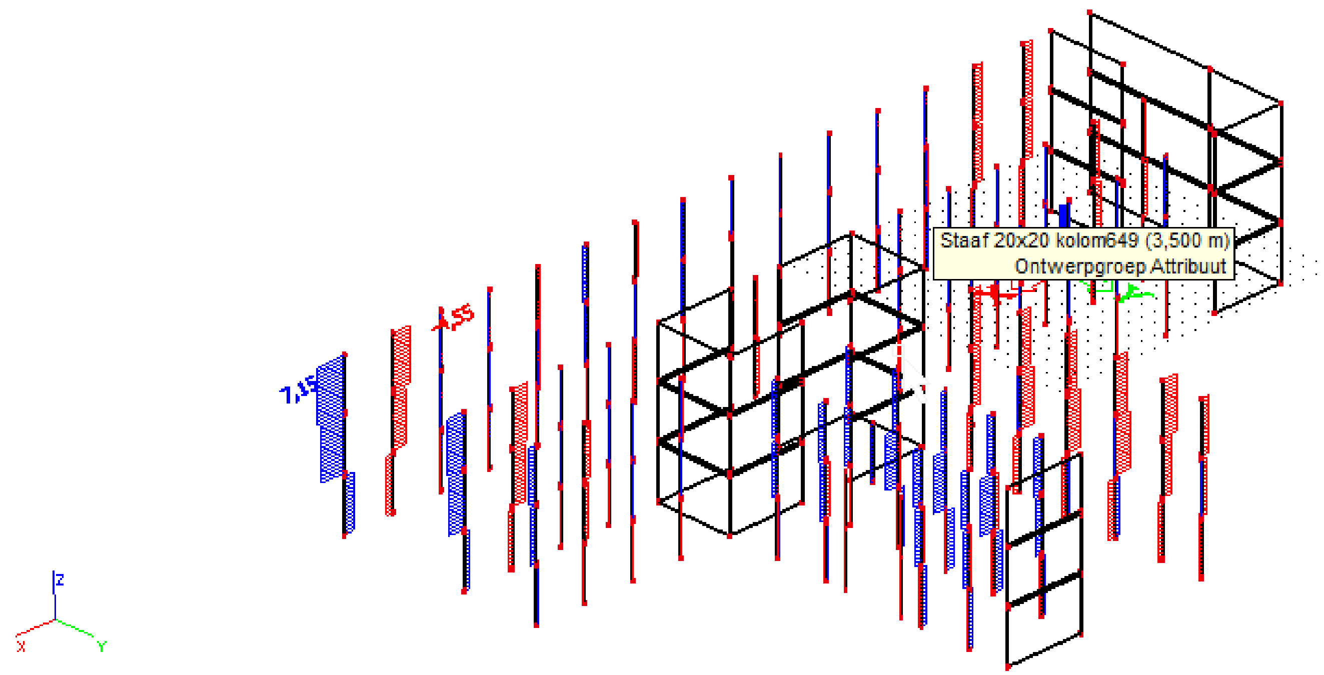


FIGURE 134 MAXIMUM HORIZONTAL FORCES SECOND DIRECTION

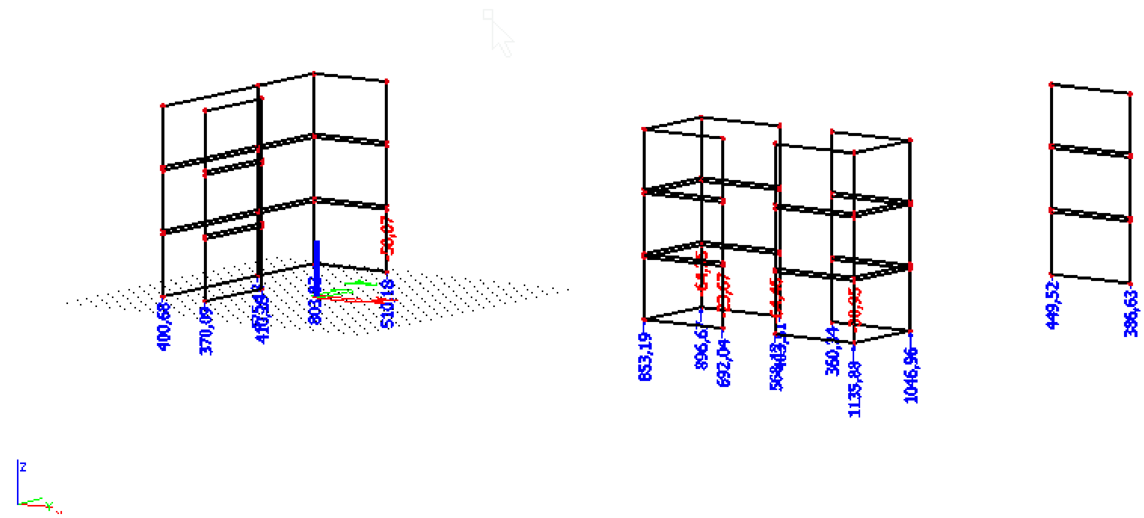


FIGURE 135 REACTION FORCES IN Z DIRECTION OF STABILITY WALLS

9.5. Technosoft calculation

Royal HaskoningDHV Blad: 1

TS/Kolomwapening Rel: 6.00 25 jan 2017

Project	:
Onderdeel	:
Dimensies	: kN;m;rad (tenzij anders aangegeven)
Datum	: 25/01/2017
Referentieperiode	: 50

Toegepaste normen volgens Eurocode (CEN)

Beton	EN 1992-1-1:2005	C2:2010

Geometrie

Type constructie		:	Wand
Wandbreedte	[mm]	:	600
Wanddikte in buigingsricht.	[mm]	:	180
Wandhoogte (L)	[mm]	:	3400
Belastingschema		:	Geschoord
Kniklengtefactor X		:	1.00
Pendelkolom		:	Nee

Belasting

			BG1	BG2	BG3	Maatgevend BC
Omschrijving belastinggeval		:				
Normaalkracht N Ek	[kN]	:	1084.00	0.00	0.00	1084.00
MEk,X boven	[kNm]	:	0.00	0.00	0.00	0.00
MEk,X onder	[kNm]	:	0.00	0.00	0.00	0.00
Belastingfactoren						
BC1	Fundamenteel	:	1.00	0.00	0.00	Maatgevend X

Beton en Wapening

Betonkwaliteit		:	C55/67	Prefab	:	Nee
Soort spanningsrekdiagram		:	Parabolisch - rechthoekig diagram			
Staalsoort		:	B500A	Symm.wapening:		2-zijdig
f_{yk}	[N/mm²]	:	500	ε_{uk} [%]	:	2.5
Soort spanningsrekdiagram		:	Bi-lineair diagram met klimmende tak			
Basiswapening	[mm]	:	ø6.0 hoh 150	Bijlegw.[mm]	:	ø12.0,12.0
Hoofdwapening in laag		:	2	Verdeelw.[mm]:		ø 8.0

Betondekking

Milieu	:			XC1
Gestort tegen bestaand beton	:			Nee
Element met plaatgeometrie	:			Nee
Specifieke kwaliteitsbeheersing	:			Nee
Oneffen beton oppervlak	:			Nee
Ondergrond	:			Glad / N.v.t.
Constructieklasse	:			S3
Grootste korrel	:			31.5
Hoofdwapening	:			2de laag
Nominale dekking	:			22
Toegepaste dekking	:			28
Gelijkwaardige diameter	:			12
$c_{min,b}$ $c_{min,dur}$ Δc_{dur}	:	12	10	0
c_{min} Δc_{dev} c_{nom}	:	12	10	22
Beugel / Verdeelwapening	:			1ste laag
Nominale dekking	:			20
Toegepaste dekking	:			20
Gelijkwaardige diameter	:			8
$c_{min,b}$ $c_{min,dur}$ Δc_{dur}	:	8	10	0
c_{min} Δc_{dev} c_{nom}	:	10	10	20

FIGURE 136 TECHNOSOFT CONSTRUCT CALCULATION

Royal HaskoningDHV Blad: 2

TS/Kolomwapening Rel: 6.00 25 jan 2017

Project :
Onderdeel :

Belastingcombinatie 1: (Fundamenteel)

Tussenresultaten		X-as	BC1
Traagheidsmoment I	[mm^4]	29160e4	
Kniklengte l_0	[mm]	3400	

Art. 5.8.4 (2)

kruipfactor ($\varphi_{ef}(on,t_0)$)		1.67

Art. 5.2 (7)

Basis imperfectie (θ_0)		0.005000
Factor (α_h)		1.000
Aantal elementen (m)	[st]	1
Factor (α_m)		1.000
Imperfectie (θ_i)		0.005000
Excentriciteit e_i	[mm]	8.500000

Art. 5.8.3.1 (1)

Lambda (λ)		65.43
Wapeningsoppervlak (A_s)	[mm^2]	226
Betonoppervlak (A_c)	[mm^2]	108000
Betondruksterkte (f_{cd})	[N/mm^2]	36.67
Moment (M_{01})	[kNm]	9.21
Moment (M_{02})	[kNm]	9.21
Moment ratio (r_m)		1.000
Factor A		0.749
Factor B		1.025
Factor C		0.700
Grensslankheid (λ_{lim})		20.54
Volstaat 1e orde toetsing?		Nee

Art. 5.8.8.3

Nuttige hoogte (d)		149
Vloeigrens (f_{yd})		434.8
Elasticiteitsmodulus (E_s)		200000
Factor (ω)		0.049
Factor (n_u)		1.0486
Factor (n_{bal})		0.4000
Factor (n)		0.2737
Coëfficiënt K_r		1.0000
Factor (β)		0.1888
Coëfficiënt $K\varphi$		1.3160
Kromming ($1/r_0$)		3.2422e-5
Glob. kromming (1/r)		4.2669e-5

Art. 5.8.8.2

Excentriciteit e_2	[mm]	49.3
M_{0e}	[kNm]	0.00
M_{0Ed}	[kNm]	9.21
M_2	[kNm]	53.47
M_{Ed}	[kNm]	62.68
N_{Ed}	[kN]	1084.00

Art. 6.1 (4)

Minimale excentriciteit e_0	[mm]	20.00
$M_{Ed,min}$	[kNm]	21.68

FIGURE 137 TECHNOSOFT CONSTRUCT CALCULATION

Royal HaskoningDHV Blad: 3

TS/Kolomwapening Rel: 6.00 25 jan 2017

Project :
Onderdeel :

Berekende gegevens X-as BC1

Beginexcentriciteit e_{02}	[mm] :	0.0			
Beginexcentriciteit e_{01}	[mm] :	0.0			
Excentriciteit e_i	[mm] :	8.5			
Excentriciteit e_2	[mm] :	49.3			
Totale excentriciteit e_t	[mm] :	57.8			
Min. wapening art. 9.6.2(1)	[mm2] :	216.0	(= 360.0 [mm2/m])		
Min. wap. art. 9.6.2(1)&(3)	[mm2] :	0.0	=2x(ø0.0 hoh 400)	(=	0.0 [mm2/m])
Min. wap. art. 7.3.2	[mm2] :	0.0		(=	0.0 [mm2/m])
Totaal ber. wap. 1e/2e orde	[mm2] :	0.0		(=	0.0 [mm2/m])
Maatgevende wapening	[mm2] :	216.0		(=	360.0 [mm2/m])

Tussenresultaten doorsnede X-as BC1

Voorwaarde Eps;c=Eps;cu2 op de vezel y= -90.0 mm

y [mm]	Wapening	Perc. [o/o]	A_s/A_p [mm2]	$\Delta\varepsilon$ [o/oo]	σb [N/mm2]	$\Delta\sigma s$ [N/mm2]
-90.0				-3.125	-36.67	-
-59.0	4.000Ø6	100	113.1	-1.677	-	-335.45
59.0	4.000Ø6	100	113.1	3.834	-	434.83
			226.2			

Inwendige krachten

y [mm]	N_b [kN]	$N_s/\Delta N_p$ [kN]	Δy [mm]	N [kN]	$N*\Delta y$ [kNm]
-63.7	-1095.239		-63.7	-1095.239	69.795
-59.0		-37.939	-59.0	-37.939	2.238
59.0		49.178	59.0	49.178	2.901
		totaal inwendig		-1084.000	74.935

FIGURE 138 TECHNOSOFT CONSTRUCT CALCULATION

Royal HaskoningDHV Blad: 4

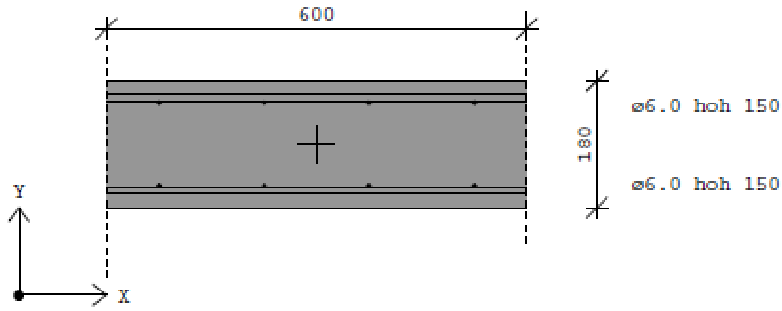
TS/Kolomwapening Rel: 6.00 25 jan 2017

Project :
Onderdeel :

Maatgevende belastingcombinatie 1: (Fundamenteel)

Gevonden wapening		basiswapening	extra staven
Bijlegcombinatie 1	226 [mm2]	: 2x(ø6.0 hoh 150	)

Grafische uitvoer bijlegcombinatie 1



Opmerkingen
[101] De berekende wapening is de totale wapening in de doorsnede.
[123] De lengte/dikteverhouding is kleiner dan 4.0, zie (art. 9.6.1(1))
[113] Twee-zijdige wapening

FIGURE 139 TECHNOSOFT CONSTRUCT CALCULATION