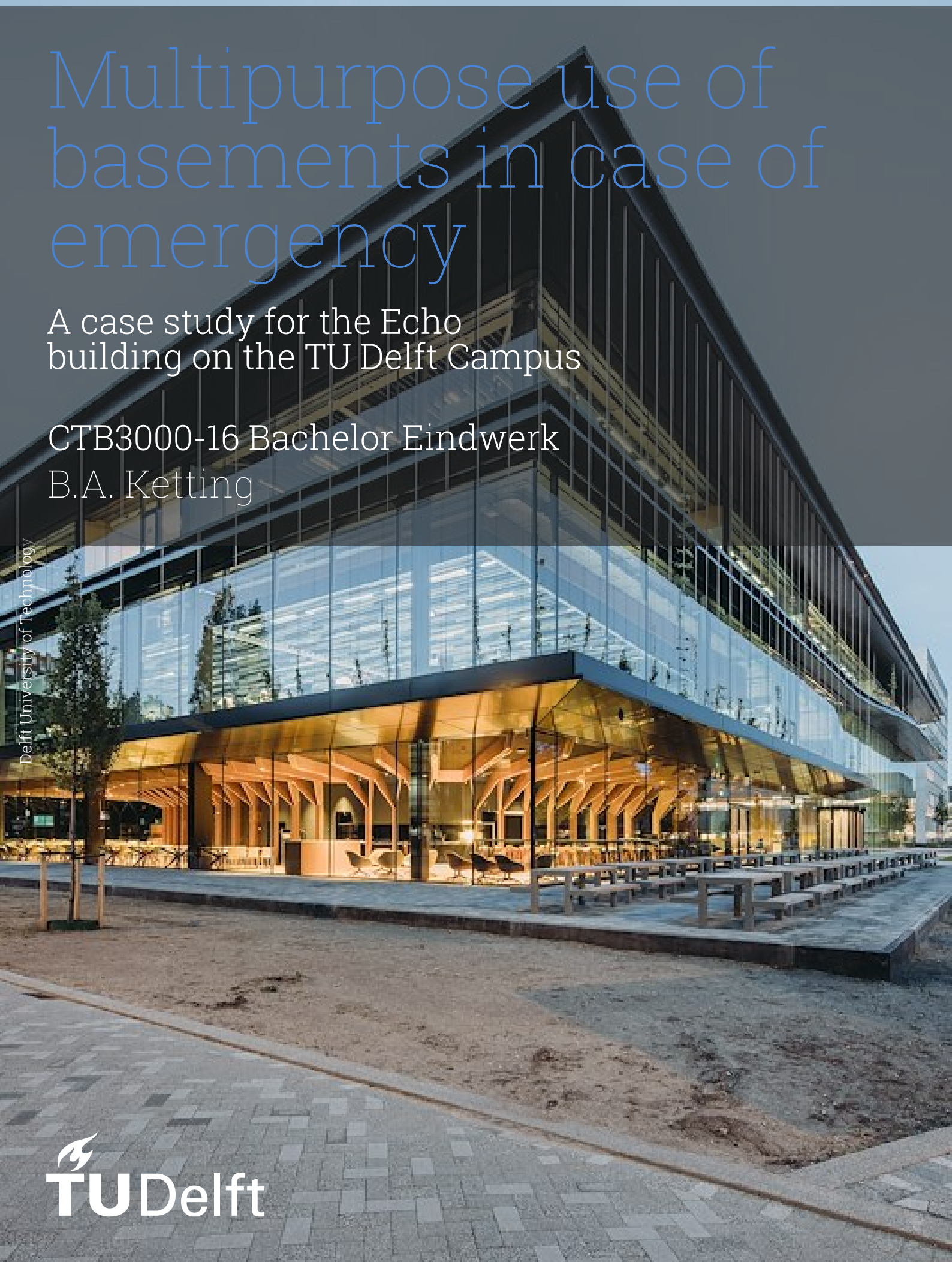
Multipurpose use of basements in case of emergency

A case study for the Echo building on the TU Delft Campus

CTB3000-16 Bachelor Eindwerk

B.A. Ketting



Delft University of Technology

TU Delft

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A case study for the Echo building on the TU Delft Campus

by

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to obtain the degree of Bachelor of Science
at the Delft University of Technology,

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Cover: Echo building, located at TU Delft campus (Muis, 2022)
Style: TU Delft Report Style, with modifications by Daan Zwaneveld

TU Delft

Preface

Disclaimer: The results produced in this report are not recommended to be used in real-world applications, due to the use of conservative calculation methods and due to assumptions and approximations made for parameters which were unknown

This Report is written for students of the BSc Civiele Techniek. Prior knowledge of structural mechanics and dynamics is recommended. I would like to thank my supervisors Pierre Hoogenboom and Roel Schipper for supporting me during my research. I also would like to thank Rik Kwakman and Roy Crielaard from Arup and George Abrahamzen from TU Delft Campus Real Estate & Facility Management for providing blueprints of the Echo building.

Google Gemini has been used for the following purposes while writing the report:

- Writing code for implementing figures, tables, and other auxiliary components of the report.
- Finding specific information from sources. The sources were manually checked afterwards.
- Translation of the introduction chapter to English. This text was originally written in Dutch.
- Writing code for the APA style references for the bibliography.

B.A. Ketting
Delft, June 2026

Abstract

Following the end of the Cold War, most public shelters in the Netherlands were abandoned, repurposed, or removed from public records. Recent geopolitical developments have renewed interest in civil protection infrastructure, while the scarcity of available construction space limits the feasibility of constructing new shelters. This study investigates whether the basement of the Echo building on the TU Delft campus can function as an emergency shelter during wartime.

A structural assessment was performed for five representative attack scenarios: a kamikaze drone impact, an air-burst cruise missile explosion, ground shock caused by a missile strike, collapse of the surface structure, and a direct hit from a ballistic missile. Simplified analytical models were used to determine the loads imposed by each scenario and to compared against the resistance of the relevant structural elements. The structural capacity of the floor slabs was assessed using energy-based methods, while the stability walls were evaluated using Eurocode-based shear resistance calculations. Structural drawings and manufacturer data were used to determine the geometry and material properties of the building.

The results show that the basement is not suitable as an emergency shelter in its current configuration. A drone impact generates sufficient energy to initiate progressive collapse of the upper floors, leading to failure of the slabs covering the basement. The air-burst scenario causes shear forces in the stability walls that exceed their calculated capacity, resulting in collapse of the surface structure. Although ground shock alone does not cause stability wall failure, the combined effects of blast loading and ground motion are expected to lead to structural collapse. The collapse of the surface structure imposes approximately 166 times more energy on the basement roof than it can absorb before failure. A direct hit from a ballistic missile exceeds the combined resistance of all floor systems by more than five orders of magnitude and results in complete penetration of the structure.

Several emergency reinforcement measures were investigated, including carbon-fibre reinforcement strips, ultra-high-performance concrete toppings, and structural struts. While these measures may improve resistance against collapse-related scenarios, they are not a cost-effective measurement. It is therefore concluded that substantial structural strengthening and further investigation would be required before the basement of the Echo building could be considered a viable emergency shelter.

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1

Introduction

After the Cold War, the urgency to construct and maintain bunkers and fallout shelters rapidly diminished in the Netherlands. The shelters built during that era have since fallen into disrepair or have been repurposed. Vos and Rijken (2026) state that many bunkers have vanished from the archives of government agencies: ”Only a small fraction of municipalities still know the locations of the old shelters.” However, with the increasing geopolitical tensions on the European continent, this urgency is resurfacing. Nevertheless, the Netherlands faces a different challenge: a lack of construction space. Given the high housing demand and limited territory, there is little room left in zoning plans for emergency facilities such as bunkers and shelters. As far as is known, Delft currently possesses no operational public shelters. This research investigates whether the basement of the Echo building on the TU Delft campus can function as a shelter in case of emergency.

1.1. Main goal and subquestions

The main goal of this thesis is to check if the basement of the Echo building is suitable as a shelter in case of attacks during wartime. This goal is achieved by answering the following sub questions:

- *Which attack scenarios are considered when determining the suitability of the basement as a shelter?*
- *Is the current basement structure suitable as a shelter?*
- *How should the basement structure be reinforced if the current structure is not suitable?*
- *Is reinforcing the basement structure cost-effective?*

The first sub question is answered in chapter 2. The second sub question is answered in chapter 4. The third and fourth sub questions are answered in chapter 5.

1.2. Scope

This thesis aims to check whether the occupants of Echo and nearby buildings could use the basement as a shelter in case of conflicts. Due to Echo’s location and current Dutch diplomatic relations with neighboring countries, only long range attacks are considered. This thesis focuses specifically on the structural resistance of the shelter. Therefore, other requirements for shelter, such as ventilation etc., will not be thoroughly assessed in this thesis. Furthermore, the focus lies especially on the structural integrity of the basement. The surface structure will be checked for structural failure in order to determine the load imposed by the collapsing debris on the basement. The basement of Echo will be deemed suitable if the floor covering the basement will not fail.

2

Assessment of impact scenarios

In this chapter, five attack scenarios are assessed and the methods to check for failure are described . For this research, precision guided munitions, such as guided bombs, ballistic missiles and kamikaze drones, are the only scenarios considered. This is due to the geopolitical location of the Netherlands and conflict with adjacent countries being highly unlikely. Hence, other scenarios, such as artillery barrages, are not considered.

2.1. Scenarios

In this section, five scenarios are listed and described. The scenarios drone impact, air-burst and ground shock could all lead to complete or partial collapse of the surface structure. The collapse of the surface structure is the key scenario in concluding the suitability of the basement as a shelter.

2.1.1. Drone impact

The first scenario is an FPV drone or more advanced kamikaze drones impacting Echo from the side or top of the building. InsideFPV (2026) states that three types of drone can be distinguished:

- *Small FPV drones:* Small drones controlled by army personnel using a headset, carrying around 300-500 grams of explosives. These drones are capable of disabling or destroying vehicles and injuring or killing people.
- *Medium sized drones:* Medium sized drones carry around 2-5 kilograms of explosives. These drones are big enough to damage or destroy tanks, bunkers, and large installations
- *Heavy duty drones:* These Heavy Duty drones are capable of carrying payloads of 10-30 kg, with some carrying even larger warheads.

The governing case of this scenario is a heavy duty kamikaze drone directly impacting the building, causing partial or complete collapse structural elements. The collapsing debris imposes kinetic loading on the top slab of the shelter. For this scenario, it is checked if a kamikaze drone exploding on the second floor will cause the floor slabs to collapse. If that is the case, the debris will fall on the floor slabs of the first floor. It is then checked whether the falling debris breaks the first floor. If this floor will also break, this will be considered collapse of the surface structure and the results of the scenario 'Collapse of surface structure' will be governing for determining the suitability of the basement as a shelter.

For the calculations in section 4.2, an Iranian HESA Shahed 136 drone is used. It has a warhead of 50 kg, with the explosive compound consisting mostly of the type OLA, a mix of octogen (HMX) and aluminum powder. (Kohanets & Frolova, 2025). The TNT equivalent mass is calculated using the following equation, adapted from Karlos and Solomos (2013):

$$M_{TNT,eq} = M_{exp} \cdot \frac{Q_{exp}}{Q_{TNT}} \tag{2.1}$$

Where $M_{TNT,eq}$ is the TNT equivalent mass (kg), M_{exp} is the mass of the warhead(kg), Q_{exp} is the detonation heat of the explosive and Q_{TNT} the detonation heat of TNT, both in $\mathrm{kJ/kg}$. Q_{exp} for Octogen is found to be 6.192 $\mathrm{MJ/kg}$ (Yugoimport–SDPR, 2024). For TNT, the detonation heat is 4.184 $\mathrm{MJ/kg}$ (Wikipedia contributors, 2026). The TNT equivalent mass of the warhead is $50 \cdot (6.192/4.184) = 74\ \mathrm{kg}$.

2.1.2. Air-burst shockwave

A distant airburst explosion will cause a pressure wave on the surface structure of Echo. For this scenario, the structure will be checked for total collapse caused by the shockwave of the explosion. For the calculations in section 4.3, a Russian 3M14 Kalibr cruise missile with an explosive payload of 450 kg (Oleniak, 2026). It will explode in the air at a distance of 50 m from the building. The explosive compounds of its warhead are not known. Thus a TNT equivalent explosive mass of 450 kg is assumed.

2.1.3. Ground shock

In this scenario the same Russian 3M14 Kalibr missile is considered, this time striking the ground and then exploding. This causes ground shock: an earthquake-like agitation of the ground, which imposes a load on the building in the form of displacement of the ground and thus displacement of certain structural elements. In this scenario, it is checked if the displacement causes shear failure in the stability elements. If this failure happens, the building will collapse (section 2.1.4).

2.1.4. Collapse of surface structure

Failure in scenarios 2.1.1 to 2.1.3 will cause partial or complete structural collapse. The collapsing debris will land on the ground floor, covering the basement. For this scenario, it is checked whether the collapsing debris will break the ground floor. This leads to collapse of the basement, causing potential injury or death to its occupants. In case of failure, it can be concluded that the basement is not suitable as a shelter.

2.1.5. Direct hit from a ballistic missile

Intuitively, it could be assumed that a direct hit from a ballistic missile is likely not survivable, because of it's supersonic speeds and large explosive payload. These assumptions still have to be verified, which is why this scenario is considered in this report. In this scenario, it is checked whether this ballistic missile will penetrate through the roof and all floors of the building, reaching the basement. If the missile penetrates through the floor covering the basement, it can be concluded that the basement is not suitable for resisting penetrating munitions.

The Zolfaghar ballistic missile is used in the calculations. It is produced in Iran and still in operational use. The missile has a range of 700-1400 km, depending on the type, and has a vertical dive speed of mach 3.3 (1132 $\mathrm{m/s}$) (Deagel, 2026). It's 10.3 meters long, has a diameter of 0.68 m and can carries a warhead of 590 kg. It's total launch weight is 4615 kg (Wikipedia, 2026).

2.2. Calculation Methods

2.2.1. Break Energy of floors

For failure of the floor, the elastic and plastic bending strain energy is calculated using the following equations:

The elastic strain energy is calculated using 2.2, taken from Goodno and Gere (2018).

$$U = \int_0^L \frac{M(x)^2}{2EI} dx \tag{2.2}$$

For this case, $M(x)$ is derived from a simply supported beam subjected to a distributed load q.

$$M(x) = \frac{qLx}{2} - \frac{qx^2}{2} = \frac{q}{2}\left(Lx - x^2\right) \tag{2.3}$$

Substituting equation 2.3 into equation 2.2 and solving the integral gives the equation for the elastic strain energy absorbed by the floor.

$$U = \frac{1}{2EI}\frac{q^2}{4}\int_0^L \left(Lx - x^2\right)^2 dx = \frac{q^2}{8EI}\int_0^L \left(L^2x^2 - 2Lx^3 + x^4\right) dx =$$

$$\frac{q^2}{8EI}\left[L^2\frac{x^3}{3} - 2L\frac{x^4}{4} + \frac{x^5}{5}\right]_0^L = \frac{q^2}{8EI}\left(\frac{L^5}{3} - \frac{L^5}{2} + \frac{L^5}{5}\right) = \frac{q^2L^5}{240EI}$$

$$U = \frac{q^2L^5}{240EI} \tag{2.4}$$

Where:

q = distributed load acquired by solving M_{Rd} for q
L = span of the floor (m)
EI = bending stifness of the floor (Nmm^2)

The energy per unit length absorbed in the plastic rotation zone is acquired by calculating the area under the $M-\kappa$ curve between the yielding moment (M_y) and the ultimate moment (M_u). M_y and M_u are both equal to M_{Rd}.

$$u_{pl} = \int_{\kappa_y}^{\kappa_u} M_{Rd}d\kappa \qquad \kappa_{pl} = \kappa_u - \kappa_y$$

$$u_{pl} = \int_0^{\kappa_{pl}} M_{Rd}d\kappa = M_{Rd}\cdot\kappa_{pl} \tag{2.5}$$

The total energy absorbed in the plastic rotation zone is acquired by integrating u_p.

$$E_{pl} = \int_0^{\Delta L} u_{pl}dx \tag{2.6}$$

$$\kappa = \frac{d\theta}{dx} \Rightarrow \theta_{pl} = \int_0^{\Delta L} \kappa_{pl}(x)dx$$

For simplicity reasons, it is assumed κ_{pl} is constant over the plasticity zone.

$$\theta_{pl} \approx \kappa_{pl}\cdot\Delta L \tag{2.7}$$

Substitution of equations 2.7 & 2.5 in equation 2.6 gives the total energy absorbed by the floor during plastic yielding until it breaks.

$$E_{pl} = M_{Rd}\cdot\theta_{pl} \tag{2.8}$$

$$M_{Rd} = A_p\cdot f_p\cdot 0.9\cdot d \qquad \kappa_{pl} = \frac{1}{d} \qquad \theta_{pl} = \frac{\Delta L}{d} \tag{2.9}$$

An empirical formula for ΔL is used, adapted from Hendriks (2025).

$$\Delta L = (2\cdot c_{nom} + 10\cdot\phi_p)\cdot\epsilon_u \tag{2.10}$$

The total energy a floor can absorb is the sum of the elastic strain energy and the plastic strain energy.

$$E_{Rd,floor} = U + E_{pl} \tag{2.11}$$

2.2.2. Drone impact

To check for failure of the floor elements, the energy per surface area produced by the explosion is compared to the amount of strain energy per surface area that the floor slab can bear before it breaks. It is assumed that the floor will plastically deform to reach it's breaking point. The explosion energy per surface area ($\mathrm{J/m^2}$) is calculated using equation 2.12.

$$E_{explosion} = \frac{1}{2}\frac{P_{so}^2}{\rho \cdot c} t_o \tag{2.12}$$

Where:

P_{so} = Peak overpressure caused by the explosion determined from figure 2.1 ($\mathrm{N/m^2}$)
ρ = Air density = 1.225 ($\mathrm{kg/m^3}$) (ISO, 1975)
c = Phase velocity shockwave (speed of sound) = 340.294 ($\mathrm{m/s}$) (ISO, 1975)
t_o = Negative phase duration, determined from figure 2.1 (s)

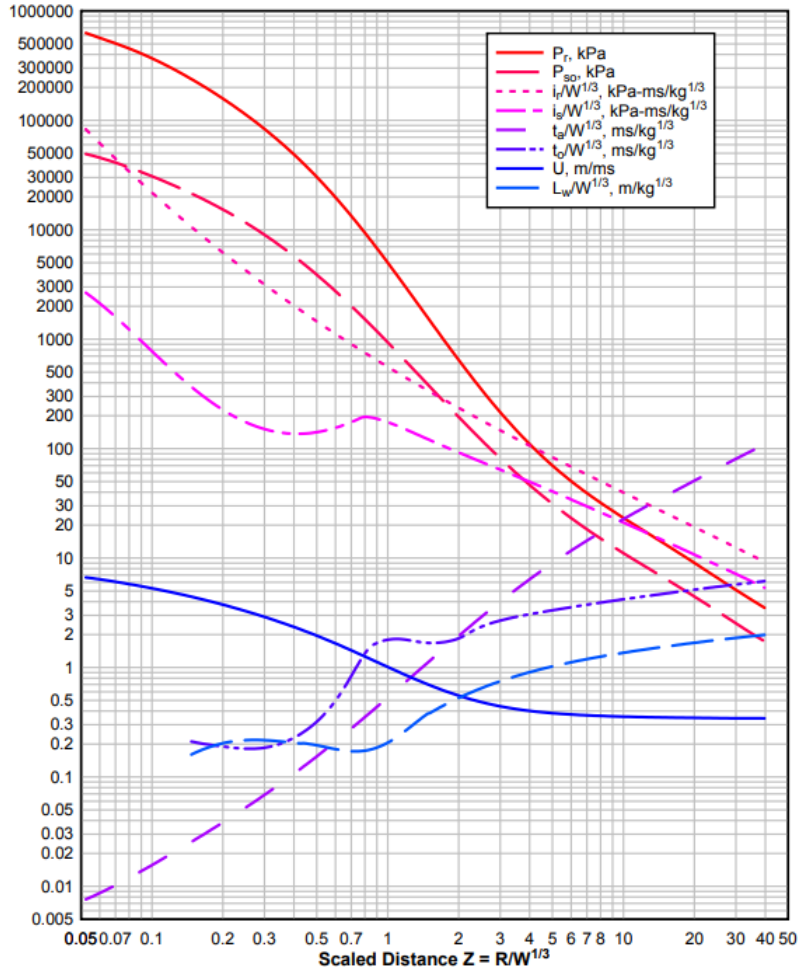


Figure 2.1: Scaled distance against multiple shockwave parameters (Karlos & Solomos, 2013).

It now has to be checked whether the slabs on the second floor will collapse. This is done by calculating the maximum energy the floor can absorb before it breaks

2.2.3. Air burst shockwave

For the shockwave energy caused by an airburst missile, equation 2.12 can also be used. The parameters are different however. The explosive mass is 450 kg and the radius from the explosion to the building is 50 m. Instead of checking for failure on local construction elements, the entire building is checked for failure in this case. Using a mass-springsystem, the maximum force occuring in the shear

walls is calculated and checked for failure, shown in figure 2.2.

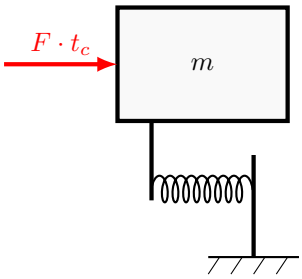


Figure 2.2: Mass-spring system used to calculated the maximum shear force in the wall.

Where $F \cdot t_c$ is the impulse created by the shockwave, m is the total mass of the surface structure of the building, and the spring stifness k is determined using formula 2.21.

Next, the maximum shear force in the shear walls is determined, using the corresponding SDOF system:

$$u(t) = u_{max} \cos(\omega t + \phi)$$
$$u_{max} = \frac{v_0}{\omega}$$
$$\omega = \sqrt{\frac{k}{m}}$$
$$v_0 = \frac{F_{so} \cdot t_c}{m}$$
$$F_{so} = A_{facade} \cdot P_{so}$$

Substitution gives:

$$u_{max} = \frac{F_{so} \cdot t_c}{\sqrt{k \cdot m}}$$

This results in the following equation, which will be used in the calculations:

$$F_{max} = k \cdot u_{max} = F_{so} \cdot t_c \cdot \sqrt{\frac{k}{m}} \tag{2.13}$$

Where:

t_c	=	Negative phase duration, determined from figure 2.1 (s)
F_{so}	=	Total shockwave force on the facade of Echo (N)
k	=	Stiffness of the shear wall $(\mathrm{N/m})$
m	=	Mass of the structure above the shear wall (kg)

Using the properties of the shear wall determined in section 3.2, the wall can be checked for failure. The equation for the maximum shear force of the wall, limited by the failure of the compression struts, is used.

$$V_{Rk,max} = bzv \cdot f_{ck} \cdot \sin(\theta) \cdot \cos(\theta) \tag{2.14}$$

With:

$$v = 0.6 \cdot (1 - \frac{f_{ck}}{250}) \tag{2.15}$$
$$z \approx 0.9 \cdot d \tag{2.16}$$

Where:

b	=	width of the wall (mm)
d	=	effective height of the cross-section (mm)
f_{ck}	=	characteristic compression strength of the concrete $(\mathrm{N/mm^2})$
θ	=	angle of the compressive struts $(45°)$

2.2.4. Ground shock

In order to determine the load imposed by the ground shock, the following formulas are used. First, the maximum horizontal displacement (u_H) of the ground is determined using the following formulas, adapted from U.S. Department of Defense, 2008:

First the maximum horizontal displacement of the ground, where the ground consists of either dry or saturated soil is calculated:

$$u_H = 0.17\, R_G^{1/3} \cdot \frac{W^{1/3}}{Z_G^{2.3}} \tag{2.17}$$

$$Z_G = \frac{R_G}{W^{1/3}} \tag{2.18}$$

Where:

R_G = ground distance from the explosion (ft)
W = weight of TNT charge (lbs)
Z_G = scaled distance from the explosion ($\mathrm{ft/lb^{1/3}}$)
u_H = horizontal displacement of the ground (inch)

After calculation, the value for u_H is converted to mm so it can be used for further calculations.

Using this horizontal displacement, the force in the stability walls can be calculated with

$$F = k \cdot u_H \tag{2.19}$$

Figure 2.3 shows the mechanical system used to determine the wall stiffness k. Using the forget-me-nots from Welleman (2026) the wall stiffness is acquired:

$$V_{Ek} = \frac{12EI}{L^3} u \tag{2.20}$$

$$k = \frac{12EI}{L^3} \tag{2.21}$$

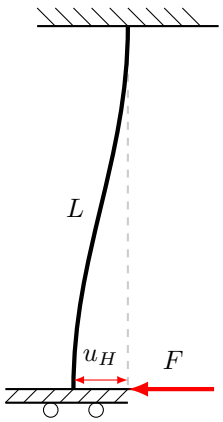


Figure 2.3: Schematische weergave van een eenzijdig ingeklemde ligger met een horizontaal verplaatsbare ingeklemde roloplegging onderaan.

Using equation 2.14 again, the wall can be checked for shear failure:

$$V_{Rk,max} = bzv \cdot f_{ck} \cdot \sin(\theta) \cdot \cos(\theta)$$

2.2.5. Collapse of surface structure

To determine if the basement roof will hold when the surface structure collapses. The potential energy of the falling debris is compared to the energy that can absorbed by the floor before it breaks:

$$E_G = E_{Rd,A200} \tag{2.22}$$

Using the weight en height of each floor, calculated in chapter 3, the total potential energy can be calculated.

$$E_g = m_{roof} \cdot h_{roof} + m_{floor_2} \cdot h_{floor_2} + m_{floor1} \cdot h_{floor1} \tag{2.23}$$

The maximum energy the floor can absorb before breaking is calculated as $E_{Rd,A200}$ in section 4.1.1. Using a unit check, the ground floor can be checked foor failure in this scenario.

$$U.C. = \frac{E_g}{E_{Rd,A200}} \leq 1.0 \tag{2.24}$$

2.2.6. Direct hit from Ballistic missile

In order to check if a ballistic missile will penetrate into the basement, punching shear resistance is used:

$$v_{Rd,c} = \left(C_{Rd,c} k (100 \rho_l f_{ck})^{\frac{1}{3}} + k_p \sigma_{cp}\right) \tag{2.25}$$

$$V_{Rd,c} = v_{Rd,c} \cdot u_1 \cdot d \tag{2.26}$$

$$k = \min\left\{1 + \sqrt{\frac{200}{d}}; 2\right\} \tag{2.27}$$

Where:

$C_{Rd,c}$	=	material factor for slabs (0.12)
k	=	Size effect factor
ρ_l	=	Reinforcement ratio in both directions
f_{ck}	=	Characteristic compression strength of concrete ($\mathrm{N/mm^2}$
u_1	=	Perimeter of loaded area
d	=	Effective slab height
σ_{cp}	=	Mean normal stress in the concrete (pretensioning stress)
k_p	=	Empirical coëfficient

$$v_{ed1} = \frac{V_{Ed}}{u_1 d_H} \tag{2.28}$$

σ_{cp} is calculated using the following formulas taken from NEN-EN 1992-1-1:2005 (2015):

$$\sigma_{cp} = \frac{P_{eff}}{A_c} \tag{2.29}$$

$$P_{eff} \approx 0.8 \cdot P_{max} \tag{2.30}$$

$$P_{max} = A_p \cdot \sigma_{p,max} \tag{2.31}$$

With:

$$\sigma_{p,max} = \min(k_1 \cdot \mathrm{f}_{pk}; k_2 \cdot \mathrm{f}_{p0.1k}) \tag{2.32}$$

Where:

f_{pk}	=	tension strength of the prestressing steel
$\mathrm{f}_{p0.1k}$	=	0.1 % yield strength of the prestressing steel
k_1	=	0.8 (-)
k_2	=	0.9 (-)

The missile will penetrate if $V_{Ed} \geq V_{Rd,c}$ To check this, the kinetic energy of the missile is calculated:

$$E_k = \frac{1}{2} m_{missile} v_{missile}^2 \tag{2.33}$$

The maximum moment caused by the punching shear force is plugged into equation 2.2 and by solving the integral, the energy required to punch through is acquired. Note that it should be checked if the punching shear resistance is less than the equivalent maximum concentrated load of the moment capacity M_{Rd}. If this is not the case, the maximum absorbed energy before breaking should be used (section 4.1).

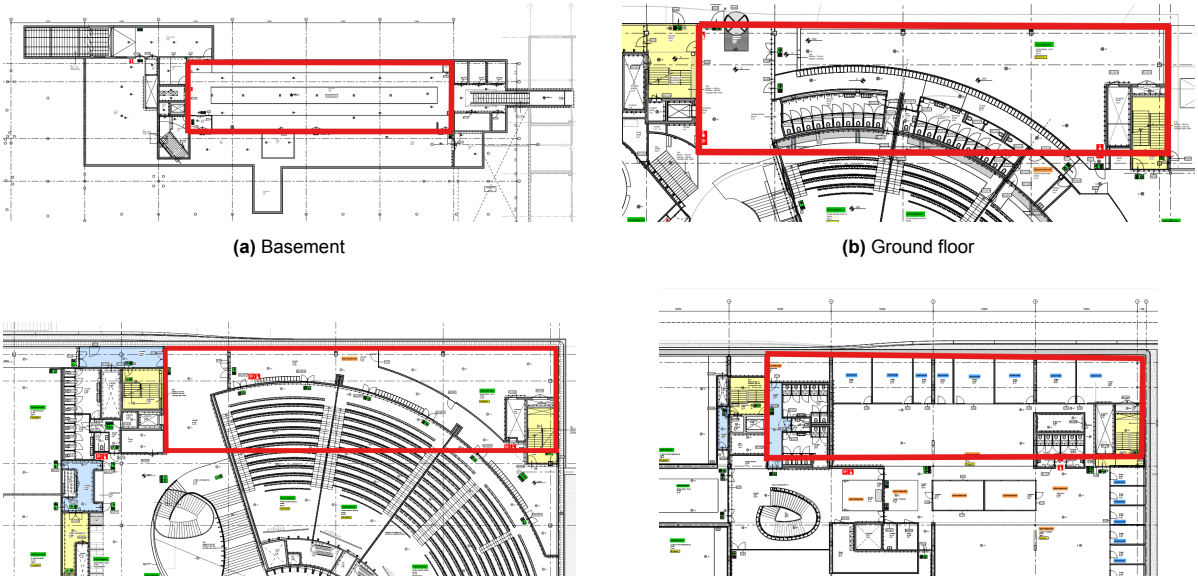
Calculating the strain energy required to punch through each floor gives the total energy required to penetrate to the basement. The missile will not penetrate if:

$$E_k \leq \sum_{i=0}^{2} E_{Rd,i} \tag{2.34}$$

3

Structural Properties of Echo

In this chapter the technical properties of Echo are discussed. The relevant structural elements with their respective dimensions are determined. The blueprints are provided through private communication by R. Kwakman of Arup and Ir. G.R. Abrahamzen of TU Delft Campus Real Estate & Facility Management. Echo can be seen as two parts; part A and part B. Part A is the part containing the lecture hall on the southern side of the building. The lecture hall reaches to the second floor with a first floor surrounding the lecture hall. Part B contains a full first and second floor which has mixed use, such as classrooms and study rooms. The shelter location considered is the main bicycle storage in the basement marked in the figures below. All floor plans and blueprints can be found in Appendix A.



(a) Basement

(b) Ground floor

(c) First floor

(d) Second floor

Figure 3.1: Overview of the designated shelter location in the basement of Echo, with it's location shown on the other floor plans as well.

3.1. Weight of floors on part A

Part A of Echo is the southern part of the building. This part has two floors: The ground floor consists of a large lecture hall, which is surrounded by smaller study rooms. The first floor contains mostly of study rooms as well. This floor is continuous with part B, where it serves as the second level. The floor plan of the ground floor is shown in appendix A.7

3.1.1. Roof
The debris load of the roof is approximated based on the roof detail A.6. The roof consists of the following elements:

- 3 mm Wedeflex Roofing. Assuming bitumen density of 1050 $\mathrm{kg/m^3}$, taken from Bouwkosten.nl (2026), gives a weight of 0.03 $\mathrm{kN/m^2}$
- 160 mm isolation. Assuming PIR isolation with a density of 40 $\mathrm{kg/m^3}$, taken from ISO-Direct (2026), gives a weight of 0.06 $\mathrm{kN/m^2}$
- 153 mm corrugated steel sheet, assuming thickness of 1 mm, gives a weight of 0.14 $\mathrm{kN/m^2}$, calculated and adapted from Industriebouwen.be (2026)
- IPE600 spaced c-to-c 6500 mm has a weight of 1,24 $\mathrm{kN/m}$. (Limtrade.nl, 2026). Dividing by the centre-to-centre distance gives a surface weight of 0.19 $\mathrm{kN/m^2}$

This sums up to a total weight of 0.42 $\mathrm{kN/m^2}$ for the roof.

3.1.2. Ground floor
The hollow core slabs are subjected to their self-weight and permanent loads. It's self weight is 3.08 $\mathrm{kN/m^2}$. Taken from the blueprints, the dead load of the floor finish is 3.00 $\mathrm{kN/m^2}$ and the dead load of the separation walls is 0.80 $\mathrm{kN/m^2}$. This totals to a static load of 6.88 $\mathrm{kN/m^2}$.

3.1.3. First floor
The exact floor type and dimensions of the first floor are unknown. Using appendix A.8, the height is approximated to be 375 mm. For the calculations, it is assumed these floors are composed of A400 hollow core slabs. The self-weight of these floors is 4.9 $\mathrm{kN/m^2}$ (VBI, 2026b). It is assumed these floors have the same weight of the floor finish and separation walls as the ground floor. This gives a total weight of 8.7 $\mathrm{kN/m^2}$.

3.1.4. Second Floor
From NEN-EN 1991-1-1 (2019) is the permanent load for the studyroom determined. The class is C2. The load is 3 to 4 $\mathrm{kN/m^2}$ The floor type is unknown, using the cross-sectional view from appendix A.8 the height is approximated to be 467 mm. It is assumed the floor consists of A500 hollow core slabs, spanning 6.5 m. The weight of these floors are 6.01 $\mathrm{kN/m^2}$.(Consolis VBI, 2026). Assuming the same dead loads of the floor finish and walls, 0,80 $\mathrm{kN/m^2}$ and 3.00 $\mathrm{kN/m^2}$, the load of the first floor sums up to 9.81 $\mathrm{kN/m^2}$

3.2. Shear walls
The overview of the stability elements are shown in figure 3.2. The shears walls W3 and W4 are inside the area of part A of the building. The properties of shear wall W3 are shown below, approximated using the blueprint of the ground floor, shown in Appendix A.2. Measuring the wall on the blueprint using the scale the following dimensions are approximated.

- $h = 5000\ \mathrm{mm}$
- $L = 8625\ \mathrm{mm}$
- $b = 200\ \mathrm{mm}$

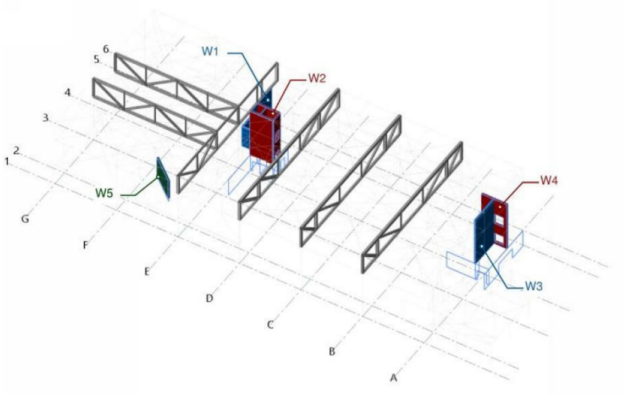


Figure 3.2: Overview of stability elements on the ground floor. Acquired through private communication with Arup

Using NEN-EN 1991-1-1 (2019), the concrete cover and the strength class are approximated. The nominal concrete cover is determined using table 4.4N and section 4.4.1.3 of NEN-EN 1991-1-1 (2019). with recommended construction class S1 and environment class XC1, $c_{min,dur}$ is found to be 10 mm. The recommended value for Δc_{dev} is 10 mm. This gives a concrete cover c_{nom} of 20 mm. In appendix E of the Eurocode the recommended strength class is C20/25 for environment class XC1. The E-modulus for this strength class is 22000 MPa. The details of the wall reinforcement are unknown. For calculations in the air burst (Section 4.3) and ground shock (Section 4.4) scenarios, maximum shear reinforcement is assumed. With this assumption, equation 2.14 can be used. This equation gives the maximum shear force the wall can bear, limited by failure of the compressive struts. To calculate the effective height of the cross-section, stirrups with a diameter of 8 mm are assumed.

$$d = h - c_{nom} - \frac{1}{2}\phi = 8625 - 20 - \frac{1}{2} \cdot 8 = 8601\ mm$$

3.3. Floor slab details

3.3.1. Ground floor slab

The basement is covered by A200 hollow core slabs with a height of 200 mm. The concrete class is C45/55. The slab is reinforced with Fep1860 prestressing steel. It's maximum strain is $\epsilon_{pu} = 3.5\%$ (NEN 3868, 2001). The slab used in the calculation has a prestress reinforcement with 6 x 9.3 mm & 2 x 12.5 mm. These parameters are found in the table of Appendix A.7. The strands have an initial prestress of 1100 MPa (Gemeente Heemskerk, 2026). The properties A_c, I, b and h, shown in table 3.1, are taken from VBI (2026a).

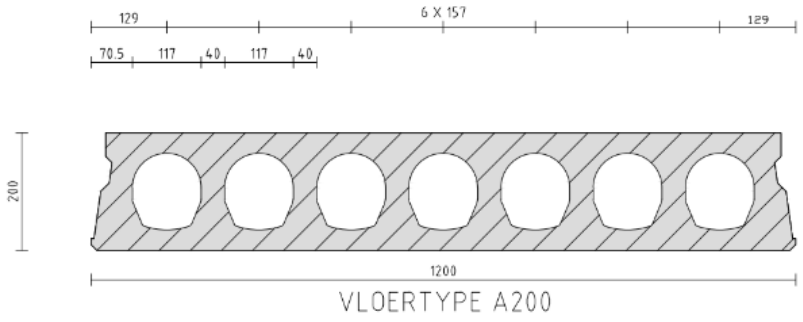


Figure 3.3: A200 hollow core slab. Taken from blueprint *1908667-002-1TF BG deel B.* By Arup

Table 3.1: A200 hollow core slab properties

Parameter	Unit	Parameter	Unit
h	200 mm	d	173.75 mm
b	1200 mm	b_w	381 mm
L	6210 mm	$\varnothing$	6$\times$9.3 & 2$\times$ 12.5 mm
A_c	143823 mm^2	ϵ_{pu}	3.5 %
c_{nom}	20 mm	$Class$	C45/55
I	677.4 mm^4	σ_{cp}	1100 MPa

3.3.2. First floor slab

The first floor is assumed to consist of A400 hollow core slabs. A document by Gemeente Heemskerk (2026) shows a slab configuration with 8 x 12.5 mm prestress reinforcement, 1100 MPa initial prestress and a concrete strength class of C45/55. It is assumed that the concrete cover for this slab is 20 mm as well. For the calculations, a span of 6210 mm is used. The properties A_c, I, b and h, shown in table 3.2, are taken from VBI (2026b).

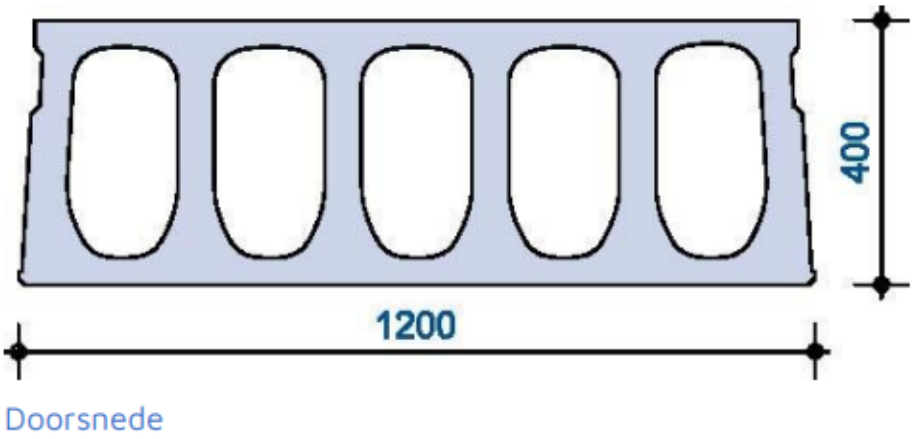


Figure 3.4: Cross-section of an A400 hollow core slab. (VBI, 2026b)

Table 3.2: A400 hollow core slab properties

Parameter	Unit	Parameter	Unit
h	400 mm	d	373.75 mm
b	1200 mm	b_w	381 mm
L	6210 mm	$\varnothing$	8$\times$12.5 mm
A_c	225911 mm^2	ϵ_{pu}	3.5 %
c_{nom}	20 mm	$Class$	C45/55
I	$4421 \cdot 10^6$ mm^4	σ_{cp}	1100 MPa

3.3.3. Second floor slab

The second floor is assumed to consist of A500 hollow core slabs. The document from Gemeente Heemskerk (2026) does not provide information regarding A500 slabs. As with the first floor slabs, it is assumed this slab type also has strength class C45/55, 8 x 12.5 mm prestress reinforcement and a concrete cover of 20 mm. For the calculations a span of 6210 mm is used. The properties A_c, I, b and h, shown in table 3.3, are taken from Consolis VBI (2026).

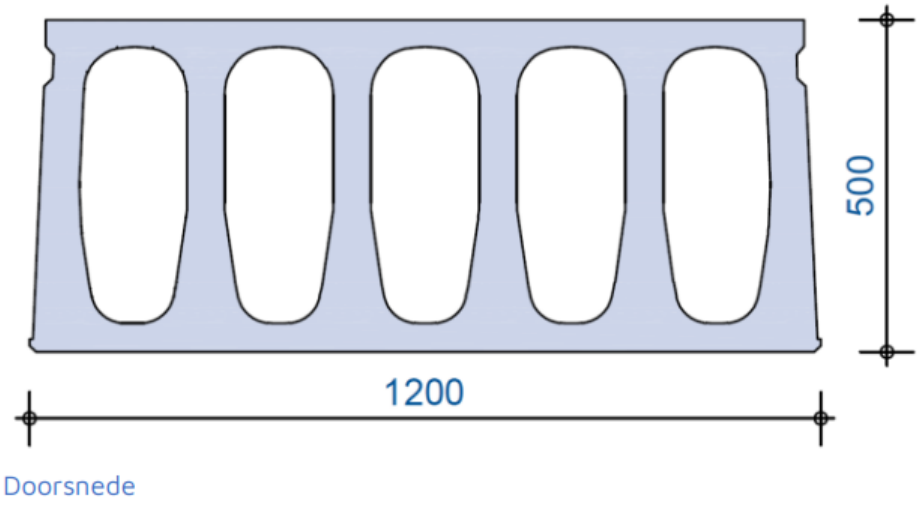


Figure 3.5: Cross-section of an A500 hollow core slab (Consolis VBI, 2026).

Table 3.3: A500 hollow core slab properties

Parameter	Unit	Parameter	Unit
h	500 mm	d	473.75 mm
b	1200 mm	b_w	381 mm
L	6210 mm	$\varnothing$	8×12.5 mm
A_c	278359 mm^2	ϵ_{pu}	3.5 %
c_{nom}	20 mm	$Class$	C45/55
I	$8229 \cdot 10^6$ mm^4	σ_{cp}	1100 MPa

3.3.4. Prestress Strands

Using table 5 & 6 from NEN 3868 (2001), the 0.1% yield strength of the 12.5 & 9.3 mm strands can be calculated.

- **12.5 mm:** $F_{p,0.1} = 149\ kN$, $A_{nom} = 93\ mm^2$. $f_{p,0.1} = \frac{149000}{93} = 1602.15\ MPa$
- **9.3 mm:** $F_{p,0.1} = 83\ kN$, $A_{nom} = 52\ mm^2$. $f_{p,0.1} = \frac{149000}{93} = 1596.15\ MPa$

For the ground floor slab. The average 0.1% yield strength is used:

$$f_{p,0.1} = \frac{2}{8} \cdot 1602.15 + \frac{6}{8} \cdot 1596.15 = 1597.65\ MPa$$

4

Calculations

In this chapter, the calculations for each of the scenarios are done using the equations found in section 2.2. Note that the results of the calculations shown in this chapter may differ from the results shown in Appendix B. This is due the results being rounded up in this chapter, for the purpose of keeping the shown calculations clear and allowing simple reproduction for the readers of this report.

4.1. Break Energy of floors

In this section, the energy required to break each floor is calculated. The equations from section 2.2.1 are used.

4.1.1. Ground floor

First the moment capacity is calculated:

$$d = 200 - 20 - \frac{12.5}{2} = 173.75 \text{ mm} \qquad z \approx 0.9 \cdot 173.75 = 156.38 \text{ mm}$$

$$M_{\text{Rd}} = 652.99 \cdot 1597.65 \cdot 156.38 \cdot 10^{-6} = 163.14 \text{ kNm}$$

The equivalent line load of the moment capacity can now be calculated.

$$q = \frac{8 \cdot 163.14}{6.21^2} = 33.84 \text{ kN/m}$$

Next, the elastic strain energy is calculated:

$$E_{\text{el}} = \frac{33.84^2 \cdot 6210^5}{240 \cdot 36000 \cdot (677.4 \cdot 10^6)} \cdot 10^{-6} = 1.81 \text{ kJ}$$

In order to calculate the plastic strain energy, the plastic elongation of the prestress strands and the plastic rotation angle are determined.

$$\Delta L = (2 \cdot 20 + 10 \cdot 12.5) \cdot 0.035 = 5.775 \text{ mm} \qquad \theta_{\text{pl}} = \frac{5.775}{173.75} = 0.0332 \text{ rad}$$

The plastic strain energy can now be calculated.

$$E_{\text{pl}} = 163.14 \cdot 0.0332 = 5.42 \text{ kJ} \qquad E_{\text{Rd,A200}} = 5.42 + 1.81 = 7.23 \text{ kJ}$$

A single slab on the ground floor can absorb 7.23 kJ of energy before breaking.

4.1.2. First floor

First the moment capacity is calculated:

$$d = 400 - 20 - \frac{12.5}{2} = 373.75 \text{ mm} \qquad z \approx 0.9 \cdot 373.75 = 336.38 \text{ mm}$$

$$M_{\text{Rd}} = 981.72 \cdot 1602.15 \cdot 336.38 \cdot 10^{-6} = 529.07 \text{ kNm}$$

The equivalent line load of the moment capacity can now be calculated.

$$q = \frac{8 \cdot 529.07}{6.21^2} = 109.75 \text{ kN/m}$$

Next, the elastic strain energy is calculated:

$$E_{\text{el}} = \frac{109.75^2 \cdot 6210^5}{240 \cdot 36000 \cdot (4421 \cdot 10^6)} \cdot 10^{-6} = 2.91 \text{ kJ}$$

Using the plastic elongation of the prestress strands calculated in 4.1.1, the plastic rotation angle can be calculated.

$$\theta_{\text{pl}} = \frac{5.775}{373.75} = 0.0155 \text{ rad}$$

The plastic strain energy can now be calculated.

$$E_{\text{pl}} = 529.07 \cdot 0.0155 = 8.17 \text{ kJ} \qquad E_{\text{Rd,A400}} = 8.17 + 2.91 = 11.09 \text{ kJ}$$

An A400 slab on the first floor can absorb 11.09 kJ of energy before breaking.

4.1.3. Second floor

First the moment capacity is calculated:

$$d = 500 - 20 - \frac{12.5}{2} = 473.75 \text{ mm} \qquad z \approx 0.9 \cdot 473.75 = 426.38 \text{ mm}$$

$$M_{\text{Rd}} = 981.72 \cdot 1602.15 \cdot 426.38 \cdot 10^{-6} = 670.64 \text{ kNm}$$

The equivalent line load of the moment capacity can now be calculated.

$$q = \frac{8 \cdot 670.64}{6.21^2} = 139.12 \text{ kN/m}$$

Next, the elastic strain energy is calculated:

$$E_{\text{el}} = \frac{139.12^2 \cdot 6210^5}{240 \cdot 36000 \cdot (8229 \cdot 10^6)} \cdot 10^{-6} = 2.51 \text{ kJ}$$

Using the plastic elongation of the prestress strands calculated in 4.1.1 again, the plastic rotation angle can be calculated.

$$\theta_{\text{pl}} = \frac{5.775}{473.75} = 0.0122 \text{ rad}$$

The plastic strain energy can now be calculated.

$$E_{\text{pl}} = 670.64 \cdot 0.0122 = 8.18 \text{ kJ} \qquad E_{\text{Rd,A500}} = 8.18 + 2.51 = 10.69 \text{ kJ}$$

A A500 slab on the second floor can absorb 10.69 kJ of energy before it breaks.

4.2. Drone Impact

4.2.1. Explosion Energy

For this scenario, an Iranian HESA Shahed 136 drone with an explosive mass of 74 kg TNT equivalent hits the second floor 20 m from the floor used in calculations. The parameters for these calculations are shown in table 4.1. With these parameters the scaled distance is calculated:

$$Z_g = \frac{20}{74^{\frac{1}{3}}} = 4.76 \frac{\mathrm{m}}{\mathrm{kg}^{\frac{1}{3}}}$$

Using the graph in figure 2.1 the peak overpressure and negative phase duration are determined. $P_{so} = 35\ kPa$, $t_0/W^{1/3} = 3.2$ ms/kg$^{1/3}$. The negative phase duration is given in $\mathrm{ms/kg}^{1/3}$, which has to be converted to ms.

$$t_0 = 3.2 \cdot 74^{1/3} = 13.43\ ms$$

These values are shown in table 4.1, along with the other parameters required for the calculations.

$\mathrm{kJ/m}^2$. The calculations are shown in appendix B.4

Table 4.1: Drone explosion parameters

Parameter	Unit
R_g	20 m
W_{TNT}	74 kg
Z_g	4.76 $\mathrm{m/kg}^{1/3}$
P_{so}	35 kPa
t_c	13.43 ms

The surface explosion energy can now be calculated:

$$e_{explosion} = \frac{1}{2} \frac{35000^2}{1225 \cdot 340.294} \cdot 0.01343 = 19.73\ \frac{kJ}{m^2}$$

The total Energy imposed on the floor is:

$$E_{explosion} = 19.73 \cdot 1.2 \cdot 6.21 = 147.1\ kJ$$

4.2.2. Failure check

Next, the explosion energy is compared to the maximum absorbed energy by the floor slab on the second level to check for failure. The absorbed energy is calculated in section 4.1.3 and is found to be 10.69 kJ.

$$U.C. = \frac{E_{expl}}{E_{Rd,A500}} = \frac{147.1}{10.69} = 13.76$$

This shows that the second floor slab will fail and fall down to the first floor. The first floor can absorb a maximum of 11.09 kJ. After calculation of the potential energy of the second floor slab, the first floor slab can be checked for failure.

$$M_{A500} = m_{A500} \cdot L \cdot b = 9.81 \cdot 1.2 \cdot 6.21 = 73.1\ kN$$

$$E_{g,A500} = M_{A500} \cdot h = 73.1 \cdot 6.25 = 456.9\ kJ$$

$$U.C. = \frac{E_{g,A500}}{E_{tot,A400}} = 41.2$$

This means the first floor will also fail if the second floor falls on it with it's full weight. The total energy imposed on the ground floor slab can now be calculated using the resulting energy of the falling floors. The energy absorbed by the ground floor slab (A200) is 7.23 kJ.

$$M_{A400} = m_{A400} \cdot L \cdot b = 8.7 \cdot 1.2 \cdot 6.21 = 64.83\ kN$$

$$\begin{aligned} E_{net} &= E_{g,A500} + E_{g,A400} - E_{Rd,A500} - E_{Rd,A400} - E_{Rd,A200} \\ &= (73.1 \cdot 11.25) + (64.83 \cdot 5) - 10.69 - 11.09 - 7.23 \\ &= 1117.515\ kJ \end{aligned}$$

$E_{net} > 0$. This means the ground floor slab will also break and for this scenario, the basement is not suitable as a shelter.

4.3. Air-burst shockwave

In this section, calculations are performed for the air-burst scenario, using the Russian 3M14 Kalibr missile. The calculations done in maple are shown in Appendix B.5. First the total mass of the first floor, second floor and roof are approximated. Next, the surface area of the facade, for which the impulse load will be absorbed by the shear wall, is determined. Lastly, the maximum shear force in the wall is calculated and checked for failure.

4.3.1. Mass calculation

Using the blueprints of the first and second floor, the mass of the first and second floor can be approximated. The mass of the first floor surrounding the lecture hall, the second floor and the roof are used. For the floor finish and separation walls, the loads from the table in appendix A.7 are used on the first and second floor. It is assumed the mass of the lecture hall is connected to the ground floor, so it will not be used in the calculations. These surface loads are multiplied with their respective area in the confines of part A of the Echo building.

- **Roof:** Using appendix A.5 the roof area is calculated. The area is confined by grid lines E-A' and 6-1. Grid line 1 does not reach the end of the roof. Using the scale, an extra 963 mm is approximated. The weight of the roof is 0.42 $\mathrm{kN/m^2}$ (Section 3.1.1).

$$A_{roof} = 53.125 \cdot (33.920 + 0.963) = 1853.159\ m^2$$

$$m_{roof} = A_{roof} \frac{W_{roof}}{g} = 1853.159 \cdot \frac{0.49 \cdot 10^3}{10} = 77833\ kg$$

- **Second Floor:** Using appendix A.4, the area of the second floor is calculated. The area is confined by grid lines E-A' and 6-1. Grid line 1 and 6 do not reach the end of the building. Using scaling, and extra 1700 mm is approximated on both sides. The total weight of the floor is 9.81 $\mathrm{kN/m^2}$ (Section 3.1.4)

$$A_{floor,2} = 53.125 \cdot (33.920 + 2 \cdot 1700) = 1982.625\ m^2$$

$$m_{floor,2} = 1982.625 \cdot \frac{9.81 \cdot 10^3}{10} = 1944955\ kg$$

- **First floor:** Defining the surface area of the first floor is more difficult, therefore the surface measurement tool from Oppervlaktemeten.nl (2026) was used. The results from the tool are shown in figure 4.1. The total area is found to be $686.47 + 68.72 = 755.19\ m^2$. The surface weight of this floor is 8.7 $\mathrm{kN/m^2}$ (Section 3.1.3).

$$m_{floor,1} = 755.19 \cdot \frac{8.7 \cdot 10^3}{10} = 657015\ kg$$

The total mass used in the calculation sums up to $2.679803 \cdot 10^6$ kg or 2680 tons.

4.3.2. Blast surface

For the calculation, the missile is detonated on the west facade of the building. The blast surface is determined using the blueprint in Appendix A.9. The horizontal distance is measured from grid E-A'. The vertical distance is measured from the +2500 to +15300.

$$A_{facade} = 53.000 \cdot (15.300 - 2.500) = 678.4\ m^2$$

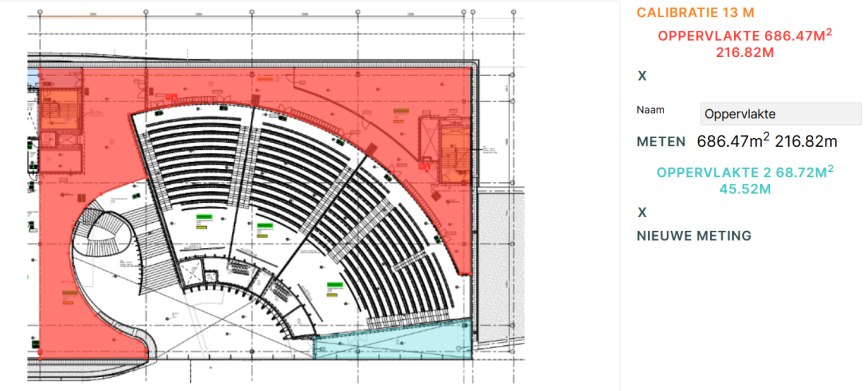


Figure 4.1: Surface area of the first floor measured using Oppervlaktemeten.nl (2026)

4.3.3. Shear Force calculation

Using the equations from section 4.3, the maximum shear force is calculated. First, the scaled distance of the explosion is determined:

$$Z_G = \frac{50}{450^{1/3}} = 6.525\ m/kg^{1/3}$$

The peak overpressure and negative phase duration can now be determined from figure 2.1.$P_{so} = 20\ kPa$ and $\frac{t_o}{W^{1/3}} = 3.5\ \frac{ms}{kg^{1/3}} \rightarrow t_o = 3.5 \cdot 450^{1/3} = 26.82\ ms$. Next, the force on the facade caused by the explosion is calculated:

$$F_{shockwave} = 678.4 \cdot 20 = 13568\ kN$$

$$P_{shockwave} = 13568 \cdot 0.02682 = 363.905\ kNs$$

Next, the wall stifness is determined.

$$I = \frac{1}{12} \cdot 0.200 \cdot 8.625^3 = 10.96\ m^4$$

$$k = \frac{12 \cdot 22000 \cdot 10.96 \cdot 10^{12}}{5000^3} = 2.258499374 \cdot 10^7\ N/mm$$

The resulting shear force can now be calculated.

$$V_{Ek} = 363.905 \cdot 10^3 \sqrt{\frac{2.26 \cdot 10^7}{2679803}} = 3.340770202 \cdot 10^7\ N \approx 33408\ kN$$

Assuming the wall has maximum shear reinforcement, the shear resistance is calculated:

$$v = 0.6 \cdot \left(1 - \frac{20}{250}\right) = 0.552\ (-)$$

$$V_{Rk,max} = 200 \cdot (0.9 \cdot 8601) \cdot 0.552 \cdot 20 \cdot \sin(45) \cdot \cos(45) = 8.545953604 \cdot 10^6\ N \approx 8546\ kN$$

$$U.C. = \frac{V_{Ek}}{V_{Rk,max}} \leq 1 \Rightarrow \frac{33408}{8546} = 3.91$$

The applied shear force is approximately four times larger than the maximum shear force the wall can bear. It can be concluded the wall will fail and the surface structure will collapse. The collapse of the surface structure is further calculated in section 4.5.

4.4. Ground shock

Using the same Russian 3M14 Kalibr missile, this time exploding on the ground, the ground shock calculations can be done. The missile explodes 50 m from the building. The shear wall inside is located 20.3 m from the facade. The total distance from the explosion R_g is 70.3 m. The explosive mass is 450 kg. In order to use these parameters in equation 2.17, they have to be converted into imperial units.

$$70.3\ m \cdot 3.28084 = 230.64\ ft$$

$$450\ kg \cdot 2.20462 = 992.08\ lbs$$

The scaled distance can now be calculated

$$Z_g = \frac{230.64}{992.08^{1/3}} = 23.12551632\ \frac{ft}{lbs^{1/3}}$$

Using the weight, distance and *scaled* distance, the maximum horizontal displacement is calculated.

$$u_H = \frac{0.17 \cdot 230.64^{1/3} \cdot 992.08^{1/3}}{23.12551632^{2.3}} = 0.007578\ inch \cdot 25.4 = 0.1925\ mm$$

The maximum shear force in the wall can now be calculated using the wall stiffness k calculated in section 4.3.3.

$$V_{Ek} = 0.1925 \cdot 2.258499374 \cdot 10^7 = 4346983.755\ N \approx 4347\ kN$$

Using the shear resistance of the wall calculated in 4.3.3, the wall can be checked for failure.

$$V_{Rk,max} \approx 8546\ kN$$

$$U.C. = \frac{V_{Ek}}{V_{Rk,max}} = \frac{4347}{8546} = 0.51$$

The unity checks shows that the value is < 1.0. It can be concluded that the shear wall will not fail if it is subjected to pure ground shock. However, this calculation does not account for the shockwave from the explosion on the ground. Looking at the results from the airburst scenario (4.3), combining ground shock and a corresponding shockwave will cause failure of the shear wall. This leads to the collapse of the surface (4.5). From those results can be concluded that for this scenario, the basement is not suitable as a shelter.

4.5. Collapse of surface structure

For the collapse of the surface structure, the potential energy of the debris is compared to the maximum energy the ground floor can absorb before breaking. First the potential energy is calculated using the weight of the floors, determined in chapter 3. The maximum absorbed energy of the ground floor is calculated in section 4.1.1. The parameters used in the calculations are shown in table 4.2.

Table 4.2: Parameters used in the collapse calculations

Parameter	Unit	Parameter	Unit
$E_{Rd,A200}$	7.23 kJ	$L \cdot b$	7.452 m^2
m_{roof}	0.42 $\mathrm{kN/m}^2$	h_{roof}	15.3 m
m_{floor2}	9.81 $\mathrm{kN/m}^2$	h_{floor2}	11.25 m
m_{floor1}	8.7 $\mathrm{kN/m}^2$	h_{floor1}	5 m

The potential energy of the debris can now be calculated.

$$E_g = (0.42 \cdot 15.3 + 9.81 \cdot 11.25 + 8.7 \cdot 5) \cdot 7.452 = 1194.47\ kJ$$

$$U.C. = \frac{E_g}{E_{Rd,A200}} = \frac{1194.47}{7.23} = 165.2$$

This unity check shows that the potential energy is almost 166 times larger than the maximum energy absorbed by the ground floor. For this scenario, the basement is not suitable as a shelter.

4.6. Direct hit from a ballistic missile

Using the Iranian Zolfaghar ballistic missile, the kinetic energy of the missile is calculated.

$$E_k = \frac{1}{2} \cdot 4615 \cdot (3.3 \cdot 340.294)^2 \approx 2.910 \cdot 10^6 \ kJ$$

Next, the punching shear resistance and the total energy absorbed by the floors are calculated.

4.6.1. Ground Floor

In order to calculate the punching shear resistance, the reinforcement ratio, the k-factor and the punching shear perimeter are calculated using the properties of the A200 slab (Table 3.1).

$$A_p = \frac{1}{4} \cdot \pi \left(6 \cdot 9.3^2 + 2 \cdot 12.5^2\right) = 652.99 \ mm^2 \qquad \rho_l = \frac{652.99}{143823} = 0.454\%$$

$$k = \min\left(2, 1 + \sqrt{\frac{200}{173.75}}\right) = 2 \qquad u_1 = 2 \cdot \pi \left(\frac{680}{2} + 2 \cdot 173.75\right) = 4320 \ mm$$

The punching shear resistance can now be calculated, using the effective width of the hollow core slab and the punching shear perimeter.

$$V_{Rd} = \left(0.12 \cdot 2\left(100 \cdot 0.00454 \cdot 45\right)^{\frac{1}{3}} + 0.5 \cdot 1100\right) \cdot 381 \cdot 4320 = 906336 \ kN$$

The moment capacity and it's equivalent concentrated load are calculated, to check whether punching shear failure or bending failure is governing for this case.

$$M_{Rd} = 652.99 \cdot 1597.65 \cdot (0.9 \cdot 173.75) = 163.14 \ kNm$$

$$F = \frac{4 \cdot 163.14 \cdot 10^6}{6210} = 105.08 \ kN$$

It can now be concluded that bending failure is governing for this floor. Hence, the maximum bending energy absorbed by the plate, calculated in section 4.1.1 is used.

$$E_{Rd,GroundFloor} = 7.23 \ kJ$$

4.6.2. First floor

The parameters required for the punching shear resistance are again calculated, using the properties of the A400 slab (Table 3.2

$$A_p = 8 \cdot \frac{1}{4} \cdot \pi \cdot 12.5^2 = 981.72 \ mm^2 \qquad \rho_l = \frac{981.72}{225911} = 0.435\%$$

$$k = \min\left(2, 1 + \sqrt{\frac{200}{373.75}}\right) = 1.73 \qquad u_1 = 2 \cdot \pi \left(\frac{680}{2} + 2 \cdot 373.75\right) = 6833 \ mm$$

The punching shear resistance can now be calculated.

$$V_{Rd} = \left(0.12 \cdot 1.73\left(100 \cdot 0.00435 \cdot 45\right)^{\frac{1}{3}} + 0.5 \cdot 1100\right) \cdot 381 \cdot 6833 = 1433313 \ kN$$

The moment capacity and it's equivalent concentrated load are calculated again.

$$M_{Rd} = 981.72 \cdot 1602.15 \cdot (0.9 \cdot 373.75) = 529.09 \ kNm$$

$$F = \frac{4 \cdot 529.07 \cdot 10^6}{6210} = 340.79 \ kN$$

It can now be concluded that bending failure is governing for this floor. Hence, the maximum bending energy absorbed by the plate, calculated in section 4.1.2 is used.

$$E_{Rd,FirstFloor} = 11.09 \ kJ$$

4.6.3. Second Floor

The parameters required for the punching shear resistance are again calculated, using the properties of the A400 slab (Table 3.3

$$A_p = 8 \cdot \frac{1}{4} \cdot \pi \cdot 12.5^2 = 981.72\ mm^2 \qquad \rho_l = \frac{981.72}{278359} = 0.353\%$$

$$k = \min\left(2, 1 + \sqrt{\frac{200}{473.75}}\right) = 1.65 \qquad u_1 = 2 \cdot \pi \left(\frac{680}{2} + 2 \cdot 473.75\right) = 8090\ mm$$

The punching shear resistance can now be calculated.

$$V_{Rd} = \left(0.12 \cdot 1.65\left(100 \cdot 0.00353 \cdot 45\right)^{\frac{1}{3}} + 0.5 \cdot 1100\right) \cdot 381 \cdot 8090 = 1696794\ kN$$

The moment capacity and it's equivalent concentrated load are calculated again.

$$M_{Rd} = 981.72 \cdot 1602.15 \cdot (0.9 \cdot 473.75) = 670.63\ kNm$$

$$F = \frac{4 \cdot 670.63 \cdot 10^6}{6210} = 431.97\ kN$$

It can now be concluded that bending failure is governing for this floor. Hence, the maximum bending energy absorbed by the plate, calculated in section 4.1.3 is used.

$$E_{Rd,SecondFloor} = 10.69\ kJ$$

4.6.4. Failure Check

Using the sum of the energy absorbed by the floors, it can now be checked whether the missile will penetrate into the basement.

$$U.C. = \frac{2.910 \cdot 10^6}{7.23 + 11.09 + 10.69} = 100310$$

The missile will penetrate into the basement. For this scenario, the basement is not suitable as a shelter.

5

Emergency Reinforcements

In this chapter, simple calculations are made to check whether emergency reinforcements are possible during risk of wartime. The following reinforcements are considered: reinforcement of the ground floor slabs with carbon fiber strips to increase the moment capacity; Extra structural topping using Ultra High Performance Concrete (UHPC) to prevent brittle failure of the concrete compression zone and constructive struts to decrease the span of the slabs.

5.1. Ground floor slabs reinforcement

If the ground slabs are able to be reinforced to the required capacity, the basement would be suitable for the following scenarios: a kamikaze drone exploding on the second floor, failure of the shear walls in the air-burst and ground shock scenarios and thus also the complete collapse of the surface structure. The scenario where a ballistic missile strikes the building is not considered, because the floor would be required to be 10000 times stronger than it is now. This would not be financially viable. To achieve this, the ground floor slabs have to be able to absorb the potential energy of all debris falling on them. This potential energy, calculated in section 4.5, is 1194.47 kJ.

5.1.1. Carbon Fibre reinforcement strips

Carbon fiber reinforcement strips are used to reinforce a concrete slab or beam, due to their high tensile strength. The maximum tensile strength of a grade I performance strip is 2400 Mpa and it's maximum strain before breaking is 1.6 %. The maximum thickness available for these strips is 1.4 mm. (ZRETE, 2026). Because carbon is a brittle material, it will not yield. Therefore, only the energy for elastic bending is calculated.



Figure 5.1: Carbon Fiber reinforcement slabs applied to a concrete slab (Sika, 2026)

5.1.2. Strut reinforcement

Placing reinforcement struts is another method to increase the moment capacity by decreasing the span. For the calculations, a configuration of struts is used where the span of each floor is halved. This gives the following new parameters for the calculation:

$$E_g = \frac{1194.47}{2} = 597.235\ kJ \qquad L = \frac{6210}{2} = 3105\ mm$$

5.1.3. Ultra High Performance Concrete

To ensure brittle failure of the concrete compression zone does not happen before failure of the carbon reinforcement, an extra layer of Ultra High Performance Concrete (UHPC) could be used. This concrete has a compression strength of 150 MPa and an E-modulus of 60000 MPa (ScienceDirect, 2026). The material factor for extraordinary design situations is 1.2 (NEN-EN 1991-1-1, 2019). This gives a design compressive strength of 125 MPa. According to paragraph 5.6.3 of NEN-EN 1991-1-1 (2019), $\frac{x_u}{d} \leq 0.35$ for concrete classes C55/67 or higher. For the internal lever arm, equation 5.1 is used, adapted from paragraph 3.1.7. and figure 3.5 from NEN-EN 1991-1-1 (2019). The Eurocode does not provide guidelines for λ and η for UHPC strength classes, therefore these factors are calculated using the upper limit of the equations: $f_{ck} = 90\ MPa$.

$$z = d - \lambda \cdot \frac{x}{2} \qquad \lambda = 0.8 - \frac{f_{ck} - 50}{400} \text{ for } 50 < f_{ck} \leq 90\ MPa \tag{5.1}$$

$$\lambda = 0.8 - \frac{90 - 50}{400} = 0.7 \qquad z = d - 0.35 \cdot x \tag{5.2}$$

$$\eta = 1.0 - \frac{f_{ck} - 50}{200} \text{for } 50 < f_{ck} \leq 90\ MPa \qquad \eta = 1.0 - \frac{90 - 50}{200} = 0.8 \tag{5.3}$$

Now that the height of the internal lever arm is known, the height of the of the compression zone can be calculated using equation 5.4.

$$N_{carbon} = N_c = \eta \cdot b \cdot f_{cd} \cdot x \tag{5.4}$$

5.2. Calculation of required reinforcement

To calculate the required required reinforcement, the moment capacity is firstly calculated inversely using the equations for the bending energy. The parameters used are taken from table 3.1. VBI (2026a) states the center of gravity (z_1) for the A200 slab is 99.4 mm from the bottom. In order to determine the required concrete layer and carbon reinforcement, the required concrete layer is iteratively determined until the conditions for $\frac{x_u}{d}$ are met and until the height of the compressive zone is equal or smaller than the added concrete layer. The iterations are done using maple (Appendix B.8).

$$U = \frac{q^2 L^5}{240 \cdot EI} \Rightarrow q = \sqrt{\frac{240 \cdot EI \cdot U}{L^5}} \tag{5.5}$$

$$M_{Rd,req} = \frac{1}{8} \cdot q \cdot L^2 \tag{5.6}$$

The extra layer causes a shift in the bending stiffness of the floor EI. The new bending stiffness therefore has to be calculated. First the new center of gravity and the transformation factor (Equation 5.7) are calculated, using formulas adapted from (Goodno & Gere, 2018). Using Steiners theory, the new bending stifness can be calculated.

$$A_2 = b \cdot h_2 \qquad I_2 = \frac{1}{12} b h_2^3 \qquad z_2 = z_1 + \frac{h_2}{2} \qquad n = \frac{E_2}{E_1} \qquad \tilde{z} = \frac{A_1 \cdot z_1 + n \cdot A_2 \cdot z_2}{A_1 + n \cdot A_2} \tag{5.7}$$

$$I_{new} = \left(I_1 + A_1 \cdot (\tilde{z} - z_1)^2\right) + n \cdot \left(I_2 + A_2 \cdot (\tilde{z} - z_2)^2\right) \qquad EI = E_1 I_{new} \tag{5.8}$$

Where:
A_1 = Cross-sectional area of the A200 slab (mm^2)
A_2 = Cross-sectional area of the new concrete layer (mm^2)
h_2 = Thickness of the new layer (mm)
I_2 = Area moment of inertia of the new layer (mm^4)
z_2 = Center of gravity of the new layer (mm)
E_1 = E-modulus of the A200 slab ($\mathrm{N/mm}^2$)
E_2 = E-modulus of the UHPC ($\mathrm{N/mm}^2$)
n = transformation factor (-)
$\tilde{z}$ = Center of gravity of the new cross-section (mm)

Using the moment capacity, the required carbon reinforcement area can now be calculated. Because the carbon strips are placed on the bottom of the slab, the effective depth d is now equal to the height of the entire cross-section.

$$N_{carbon} = N_c = \frac{M_{Rd,req}}{z_n} \qquad z_0 = 0.9 \cdot d \qquad d = h_1 + h_2 \tag{5.9}$$

$$x = \frac{N_c}{b \cdot f_{cd} \cdot \eta} = \frac{1.25 \cdot N_c}{b \cdot f_{cd}} \tag{5.10}$$

$$\text{a) } A_{carbon} = \frac{M_{Rd}}{z \cdot f_{u,carbon}} \qquad \text{b) } z = d - 0.35 \cdot x \tag{5.11}$$

The Iterations can now be performed in Maple. The iteration starts with the initial internal lever arm z_0. Running the worksheet outputs an initial height for the compressive zone. The new lever arm is calculated using equation (5.11b). The variable of the new lever arm is saved and the worksheet is executed again after changing z_0 in the equations to z, until the value for the compression zone height converges. The height of the new concrete layer is increased with increments of 100 mm and iterated until the value for the compression zone height is converged. The iteration is completed once the conditions $\frac{x}{d} \leq 0.35$ and $x \leq h_1$ are met. The results are shown in Appendix B.8. The required reinforcements are found to be as follows: The height of the concrete layer should be between 2500 and 2600 mm. The required carbon strip reinforcement is 48906 mm^2. This means that the height of the floor has to be increased with by a factor with UHPC. Dividing the required carbon strip reinforcement with the width of the floor gives a required thickness of almost 41 mm. Note that in these calculations the increased self-weight of the floor slab has not been taken into account. The self-weight will already account for a part of the energy absorbed, this means an even higher moment capacity would be required when designing the reinforcement.

5.3. Costs of carbon fiber reinforcement

An approximation of the costs of carbon fiber reinforcement is now made using a pressed carbon strip from Carbon-winkel (2026). This strip costs € 76,59 with an cross-section of 50 x 10 mm with a length of 1000 mm. Using the costs and dimensions, the price per m^3 can be approximated.

$$V_{carbon} = 10 \cdot 50 \cdot 1000 \cdot 10^{-9} = 0.0005\ m^3 \qquad \text{Price of reinforcement} = \frac{76.59}{0.0005} = 153180 \frac{€}{m^3}$$

Using Appendix A.1 the area of the basement which needs to be reinforced is approximated to be 550.56 m^2. Using the thickness of the carbon strip reinforcement of 8.09 mm, the total volume of the reinforcement can be calculated. $V_{reinforcement} = 0.041 \times 550.56 = 22.57\ \mathrm{m}^3$. This gives a total cost of $22.57 \times 153180 =$ €3 457 273. The volume of the concrete layer is $2.6 \times 550.56 = 1431.46\ \mathrm{m}^3$. Using ConcreteMath (2026) the price of industrial grade concrete is found to be approximately €178 per m^3. The total cost of the layer is approximately $1431.46 \times 178 =$ € 254 800. This gives a total cost of € 3 712 073. Upcoming (2026) states that an basic private shelter costs approximately €5000 per m^2. For a shelter this size, this would cost $5000 \times 550.56 =$ €2 750 000. It can be concluded reinforcing the basement of Echo is not cost effective.

6

Discussion

This report investigated whether the basement of the Echo building can function as an emergency shelter during wartime by assessing its structural resistance against several realistic attack scenarios. The results indicate that the basement is not suitable in its current state. However, several limitations and assumptions influence the reliability and applicability of these findings.

A major limitation is the availability of structural information of the surface structure. Detailed reinforcement drawings and material specifications for several structural were unavailable. As a result, assumptions had to be made regarding floor types, reinforcement layouts, concrete cover, material properties, and wall reinforcement. In particular, the first and second floor slabs were approximated as A400 and A500 hollow-core slabs respectively. Although these assumptions were based on available architectural drawings and engineering judgement, the actual structural capacity may differ.

The calculation methods used in this research were intentionally conservative. Simplified energy-based approaches were adopted to evaluate floor failure and collapse mechanisms. The actual response of reinforced and prestressed concrete structures subjected to blast loading is significantly more complex and would require more advanced calculation methods. These phenomena were not explicitly modelled. Therefore, the calculated failure probabilities should be interpreted as conservative estimates rather than exact predictions of structural behaviour.

For the drone impact scenario, the entire explosion energy was assumed to act on a single floor slab area. In reality, part of the blast energy would dissipate through the surrounding structure and atmosphere. Similarly, the collapse calculations assume that the full potential energy of falling debris is transferred to the floor below. Actual energy transfer would be reduced due to different forms of dissipation. Nevertheless, the calculated unity checks exceed the failure limits by large margins, suggesting that the overall conclusions are unlikely to change even when more advanced calculation techniques are applied.

The air-burst and ground-shock scenarios were assessed using simplified single-degree-of-freedom models and approximate wall properties. The dynamic behavior of the entire building was not considered. Furthermore, maximum shear reinforcement was assumed for the stability walls, which likely overestimates their actual capacity. Despite this favorable assumption, failure of the shear walls still occurred in the air-burst scenario, indicating that structural collapse remains a realistic outcome under the considered loading conditions.

The direct-hit ballistic missile scenario produced extremely high unity checks. Although the simplified penetration model cannot accurately represent high-velocity impact mechanics, the difference between the missile kinetic energy and the calculated floor resistance spans several orders of magnitude. Therefore, the conclusion that the basement cannot withstand a direct ballistic missile impact is considered robust.

Finally, this research only examined structural integrity. Other critical shelter requirements, including ventilation, radiation protection, emergency power supply, water availability, sanitation, accessibility, fire

safety, and occupant capacity, were outside the scope of this study. Even if the structural deficiencies identified in this report were mitigated, further assessment would be required before the basement could be classified as a functional emergency shelter.

Future research should focus on obtaining detailed structural drawings, performing finite-element analyses of blast and impact loading, investigating progressive collapse mechanisms, and evaluating the feasibility of permanent strengthening measures. In addition, a broader assessment of shelter functionality should be carried out to determine whether the basement can meet modern civil-protection requirements.

7

Conclusion

The main question of this report was whether the basement of the Echo building on the TU Delft campus could function as an emergency shelter during wartime. To answer this question, five realistic attack scenarios were analyzed: drone impact, air-burst shockwave, ground shock, collapse of the surface structure, and a direct hit from a ballistic missile.

The calculations show that the current basement structure is not suitable as a shelter for the considered scenarios. In the drone impact scenario, the explosion energy generated by a Shahed-136 drone exceeds the energy absorption capacity of the upper floor slabs, leading to progressive collapse of the floors above the basement. The resulting debris load causes failure of the ground floor slab covering the basement.

For the air-burst scenario, the shockwave generated by a Kalibr cruise missile produces shear forces in the stability walls that exceed their calculated resistance by a factor of approximately four. This leads to collapse of the surface structure. Although the ground-shock loading alone does not cause wall failure, the combined effects of ground shock and blast loading are expected to result in structural collapse as well.

The collapse-of-surface-structure scenario was found to be particularly critical. The potential energy of the falling debris is approximately 166 times greater than the energy absorption capacity of the ground floor slab. Consequently, the basement roof fails under the imposed loading.

The direct-hit ballistic missile scenario produces the most severe results. The kinetic energy of the missile exceeds the combined resistance of all floor systems by more than five orders of magnitude. Therefore, the missile would penetrate through the building and reach the basement.

Based on these findings, it can be concluded that the basement of the Echo building does not provide adequate structural protection against the considered wartime attack scenarios and cannot be regarded as a suitable emergency shelter in its current configuration.

Emergency reinforcement measures may improve resistance against collapse-related scenarios, such as drone impacts or collapse of the surface structure. However, such measures are unlikely to provide meaningful protection against direct ballistic missile impacts. Therefore, if the basement is to be considered for emergency shelter use, significant structural reinforcement and further detailed research will be required.

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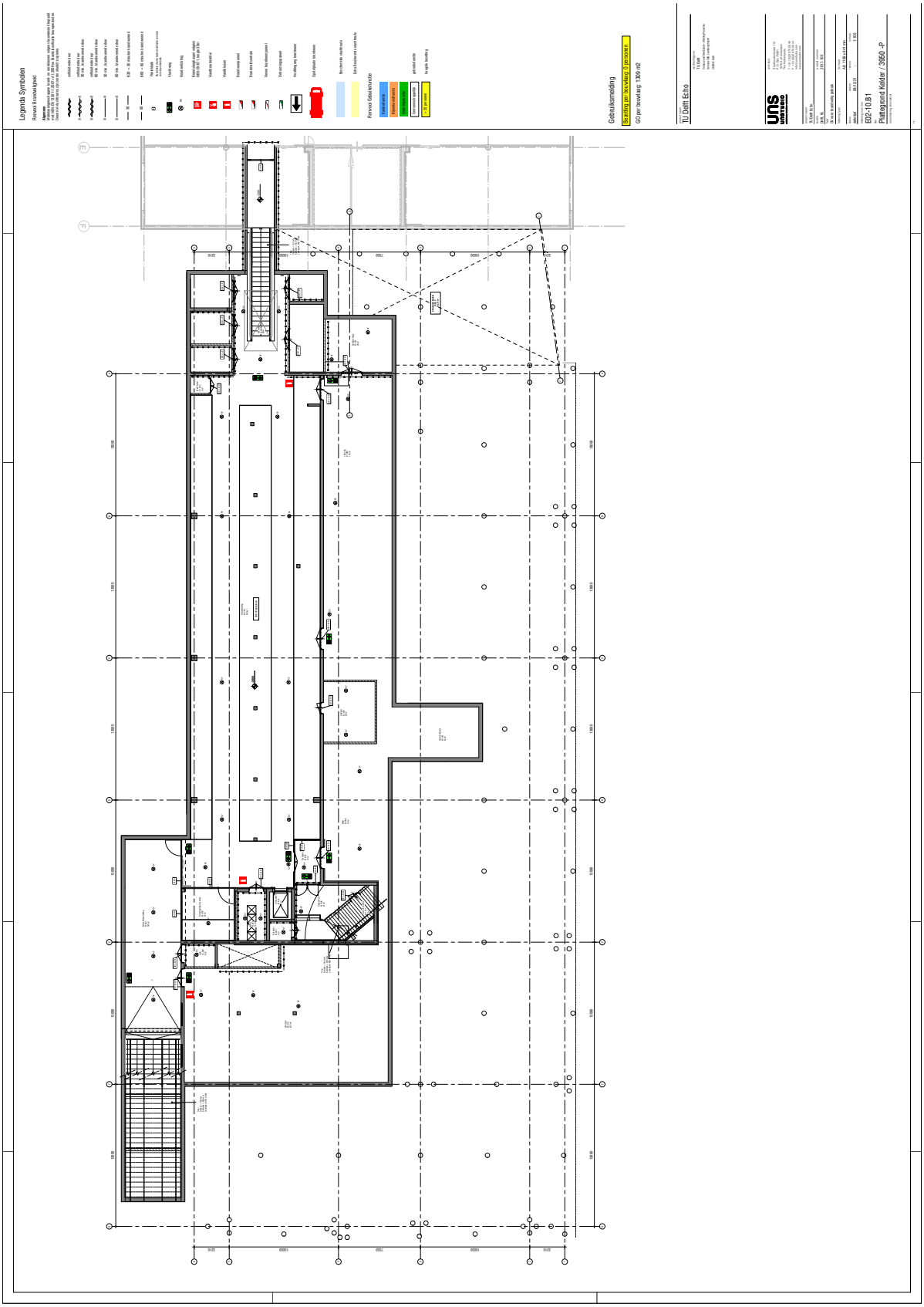
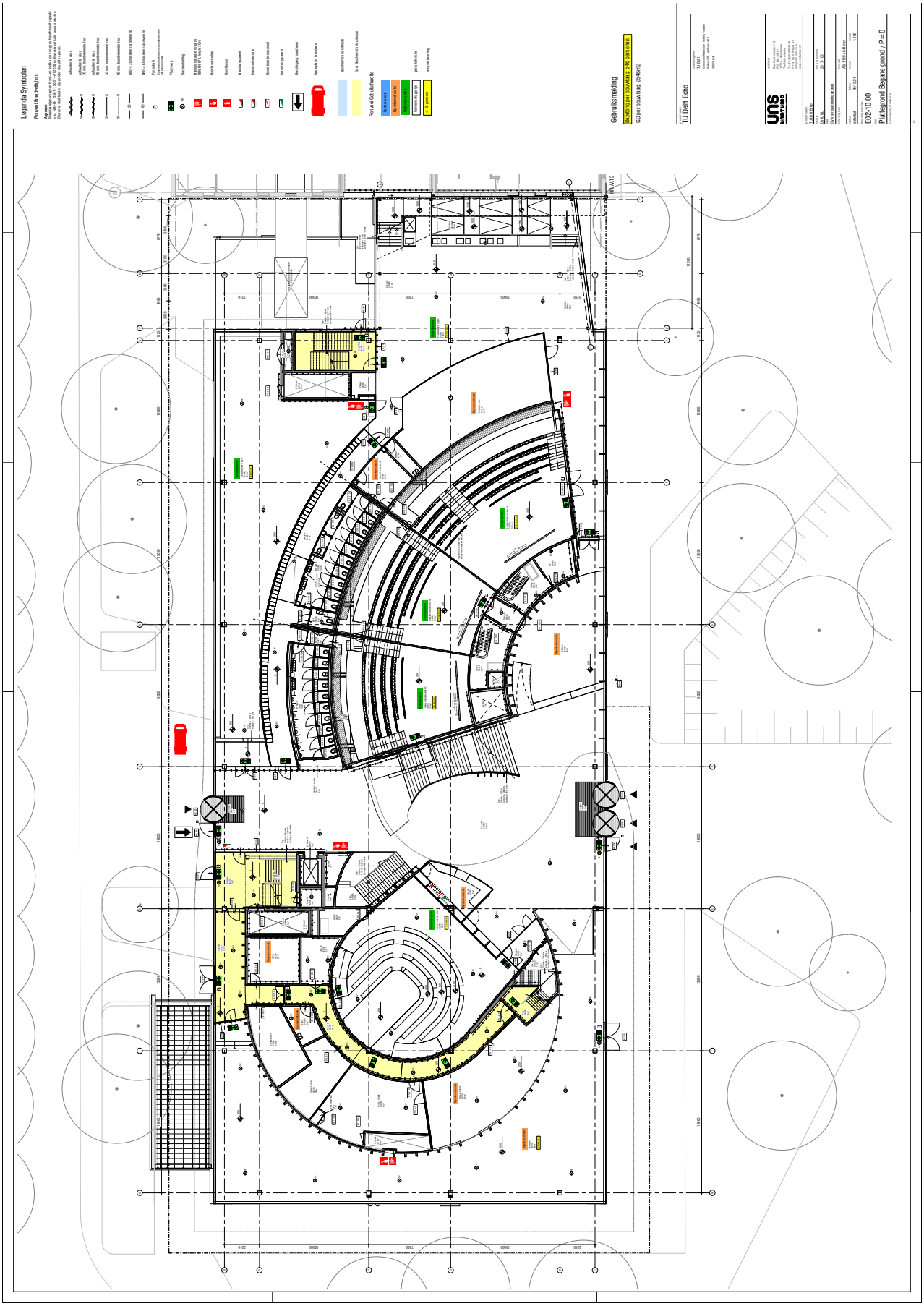
A

Blueprints Echo

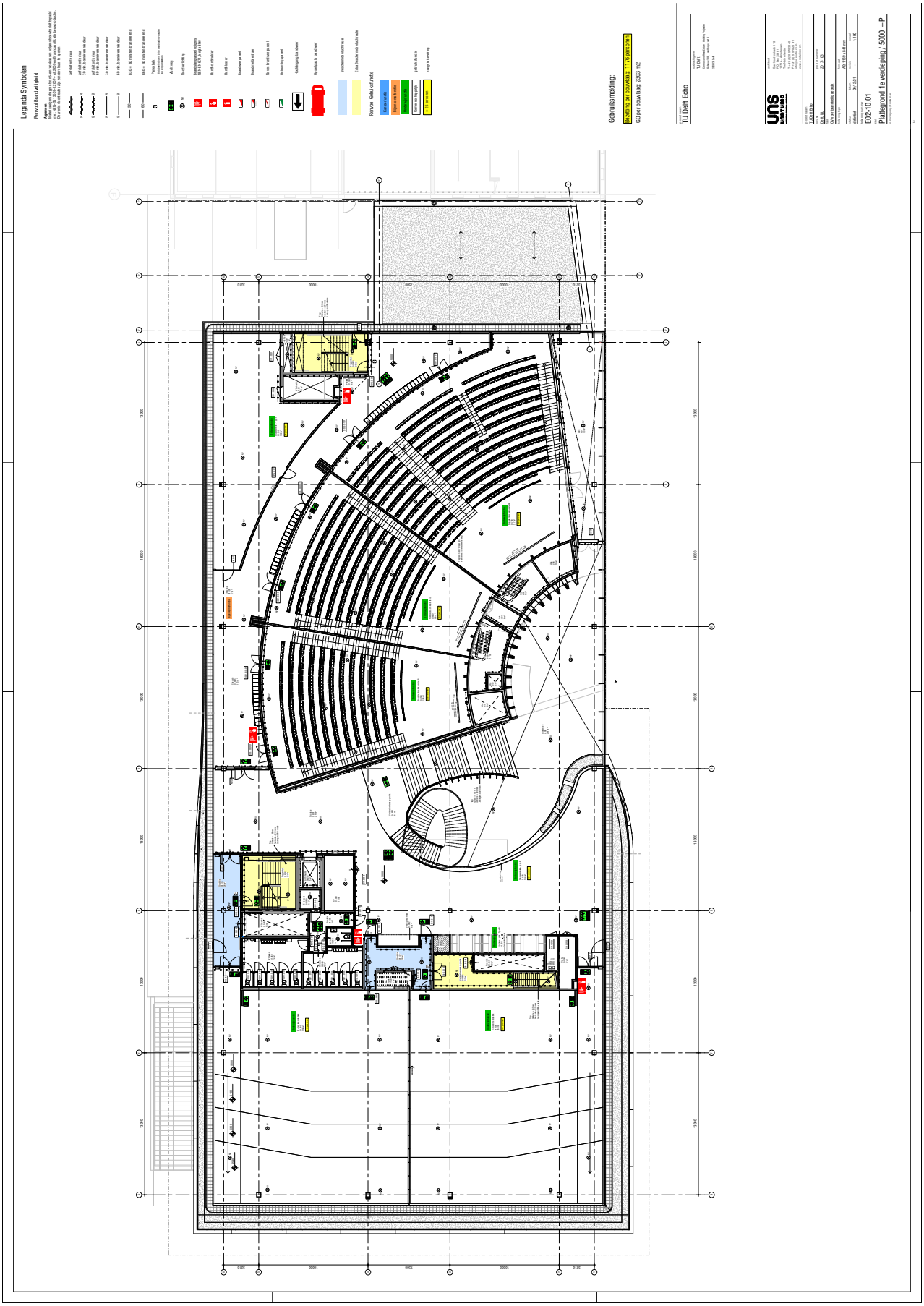
A.1. Basement Floor



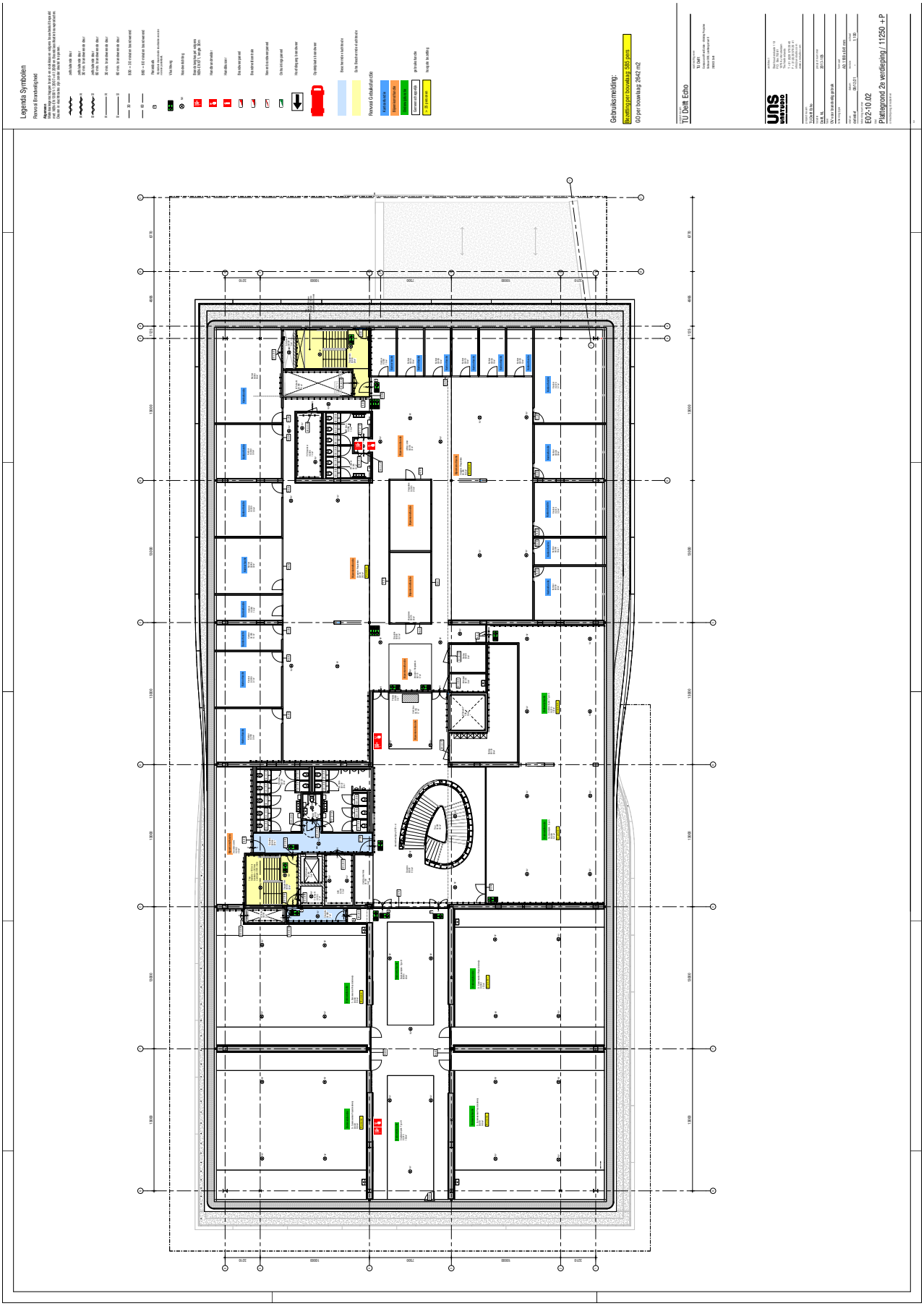
A.2. Ground Floor



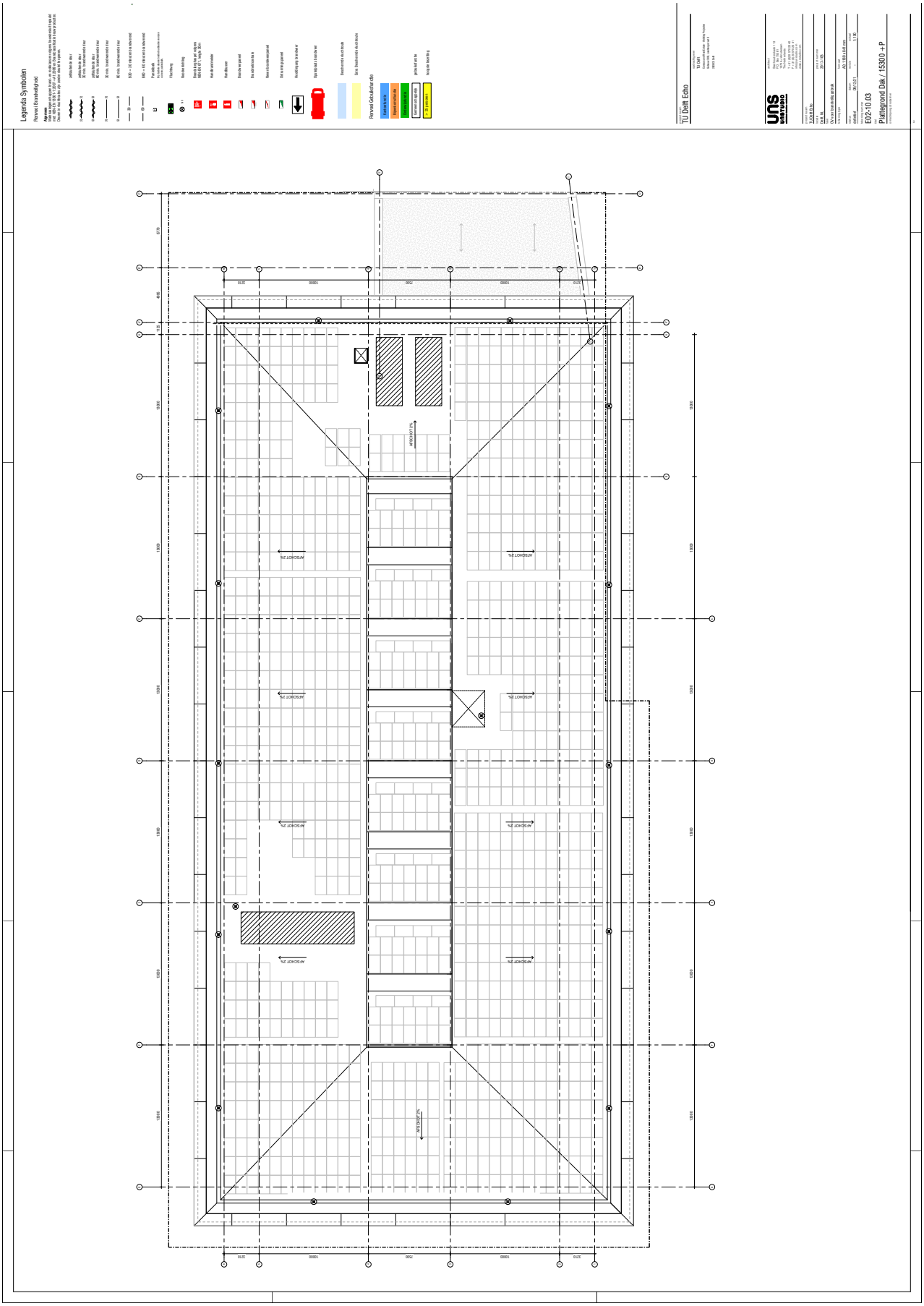
A.3. First floor



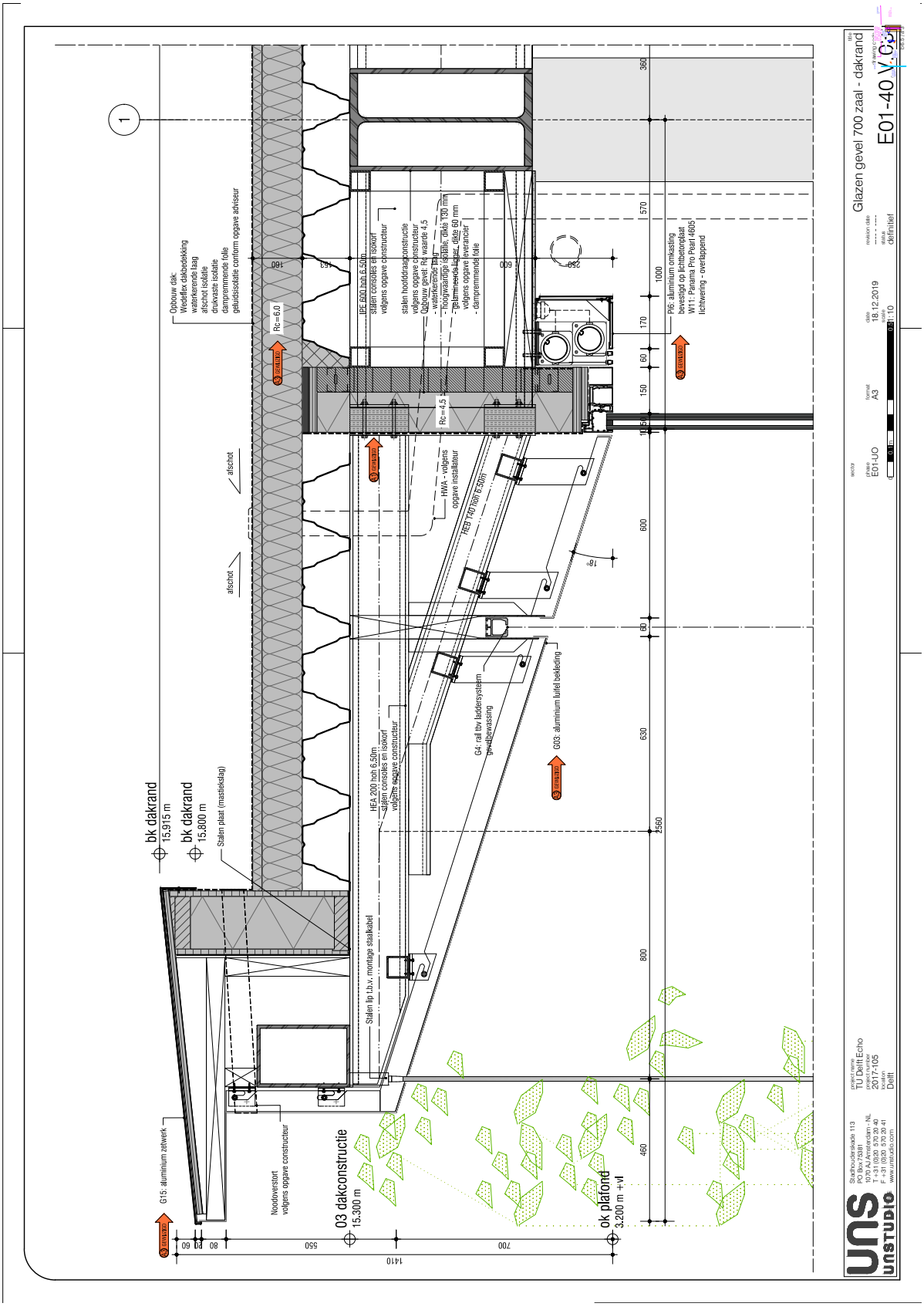
A.4. Second Floor



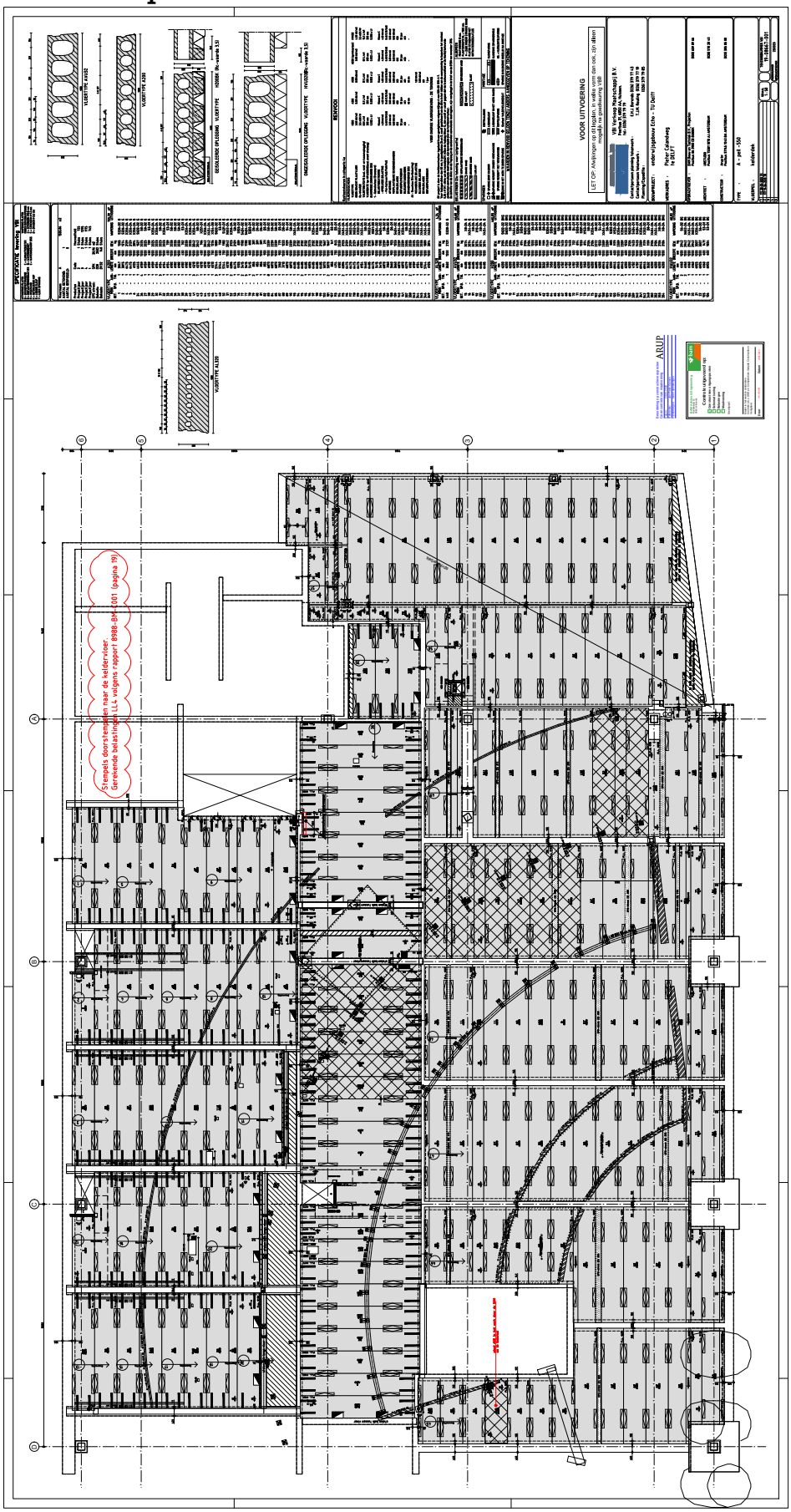
A.5. Roof


Legenda Symbolen
Renovo Brandveiligheid
TU Delft Echo
UNS
UNSTUDIO
E02-10.03
Plattegrond Dak: / 15800 + P
AFSCHOT 2%

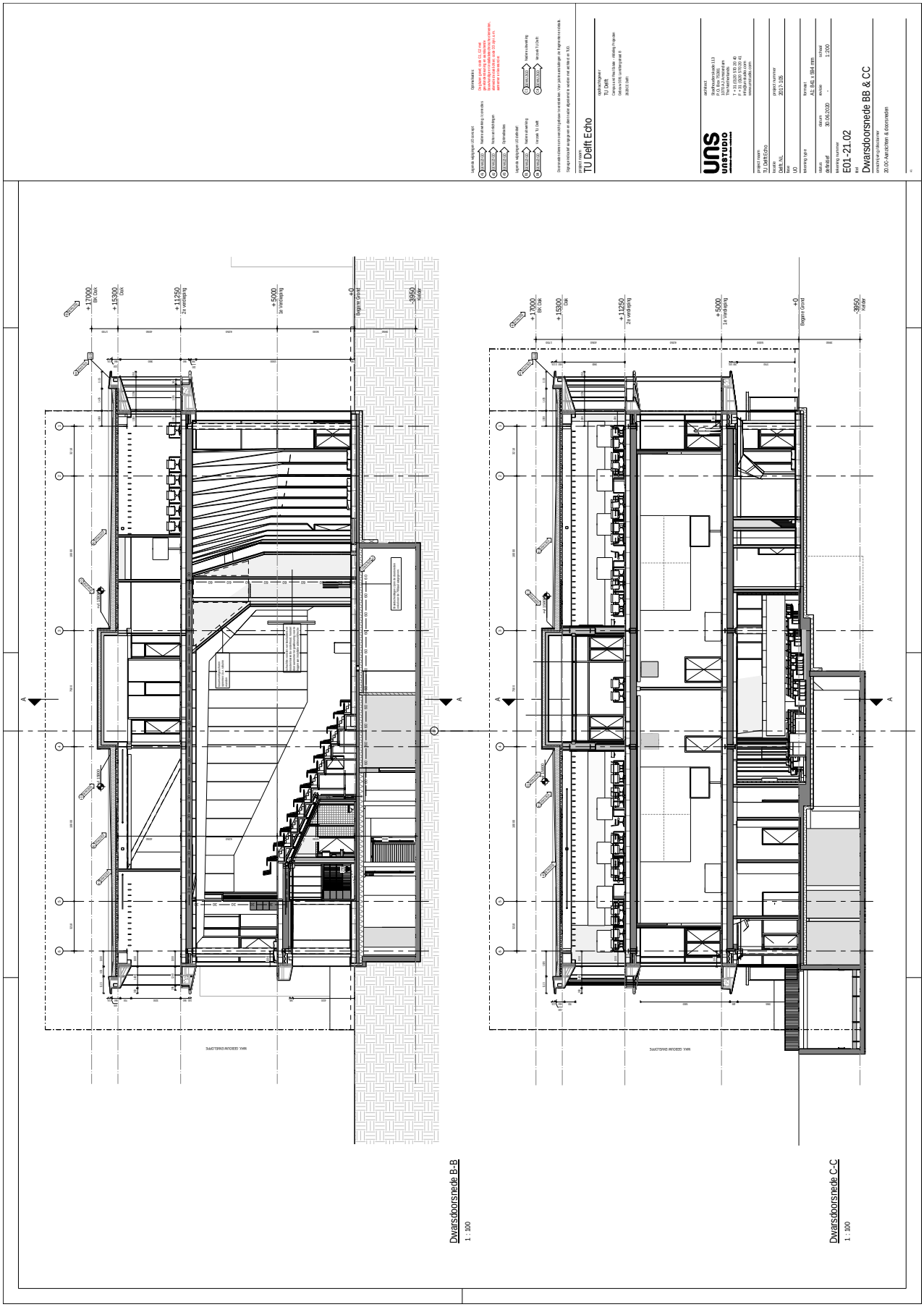
A.6. Detail Roof



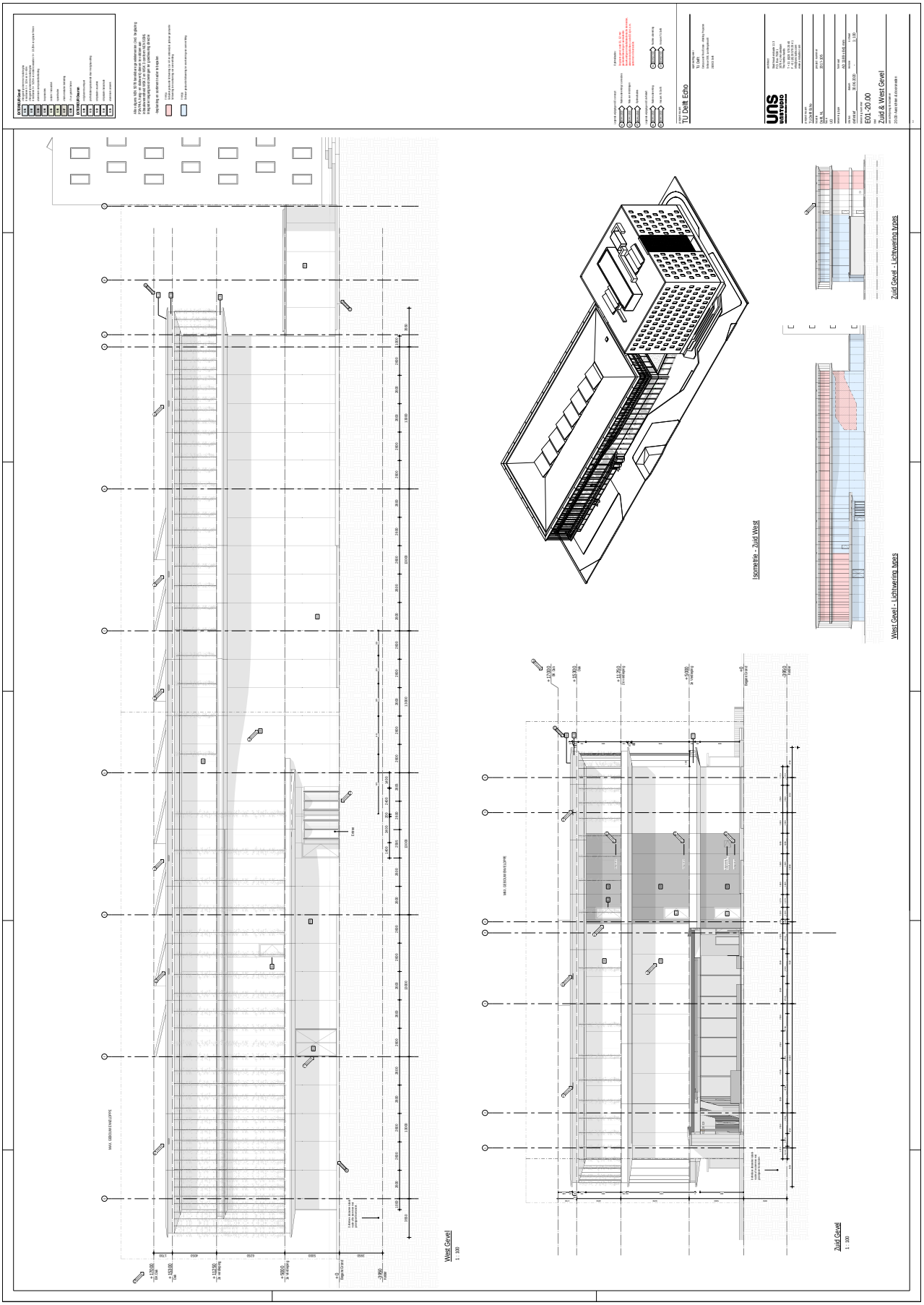
A.7. Floor detail part A



A.8. Cross-Section Part A



A.9. South & West facade



B

Maple Code

B.1. A200 break energy calculation

```
> restart :
> #A200 Parameters
> h := 200.·mm :
> c := 20.·mm :
> phi_l := 12.5·mm :
> l := 6210.·mm :
> E := 36000.·N/mm^2 :
> I_c := 677.4 ·10^6·mm^4 :
> f_y := 1597.65·N/mm^2 :
> ep := 0.035 :
> d := h − c − phi_l/2;
```

$$d := 173.7500000\ mm \tag{1}$$

```
> z := 0.9·d;
```

$$z := 156.3750000\ mm \tag{2}$$

```
> A_p := 1/4·3.1415·(12.5·mm)^2·2 + 1/4·3.1415·(9.3·mm)^2·6;
```

$$A_p := 652.9921900\ mm^2 \tag{3}$$

```
> mm := m/1000 : N := kN/1000 :
> M_Rd := A_p·f_y·z;
```

$$M_Rd := 163.1386835\ m\ kN \tag{4}$$

```
> unassign('mm') : x_u := A_p·f_y/(0.85·45·N/mm^2·381·mm); mm := m/1000 :
```

$$x_u := 71.58684382\ mm \tag{5}$$

```
> #Elastic Strain Energy
> q := 8·M_Rd/l^2;
```

$$q := \frac{33.84260149\ kN}{m} \tag{6}$$

```
> kN := kJ/m :
> E_el := q^2·l^5/(240·E·I_c);
```

$$E_el := 1.807284593\ kJ \tag{7}$$

```
> unassign('kN')
> #Plastic Strain Energy
> L_pl := 2·c + 10·12.5·mm;
```

$$L_pl := 0.1650000000\ m \quad \textbf{(8)}$$

> $theta_pl := \frac{L_pl}{d} \cdot ep;$

$$theta_pl := 0.03323741007 \quad \textbf{(9)}$$

> $kN := \frac{kJ}{m}$:

> $E_pl := M_Rd \cdot theta_pl;$

$$E_pl := 5.422307322\ kJ \quad \textbf{(10)}$$

> #***Total Energy Absorbed***

> $E_tot := E_pl + E_el;$

$$E_tot := 7.229591915\ kJ \quad \textbf{(11)}$$

>

B.2. A400 break energy calculation

```
> restart :
> #A400 Parameters
> h := 400.·mm :
> c := 20.·mm :
> phi_l := 12.5·mm :
> l := 6210.·mm :
> E := 36000.·N/mm^2 :
> I_c := 4421 ·10^6·mm^4 :
> f_y := 1602.15·N/mm^2 :
> ep := 0.035 :
> d := h − c − phi_l/2;
```

$$d := 373.7500000\ mm \tag{1}$$

```
> z := 0.9·d;
```

$$z := 336.3750000\ mm \tag{2}$$

```
> A_p := 1/4·3.1415·(12.5·mm)^2·8;
```

$$A_p := 981.7187500\ mm^2 \tag{3}$$

```
> mm := m/1000 : N := kN/1000 :
> M_Rd := A_p·f_y·z;
```

$$M_Rd := 529.0710163\ m\ kN \tag{4}$$

```
> unassign('mm') : x_u := A_p·f_y/(0.85·45·N/mm^2·381·mm); mm := m/1000 :
```

$$x_u := 107.9279292\ mm \tag{5}$$

```
> #Elastic Strain Energy
> q := 8·M_Rd/l^2;
```

$$q := \frac{109.7541011\ kN}{m} \tag{6}$$

```
> kN := kJ/m :
> E_el := q^2·l^5/(240·E·I_c);
```

$$E_el := 2.912495466\ kJ \tag{7}$$

```
> unassign('kN')
> #Plastic Strain Energy
> L_pl := 2·c + 10·12.5·mm;
```

$$L_pl := 0.1650000000\ m \quad \textbf{(8)}$$

> $theta_pl := \frac{L_pl}{d} \cdot ep;$

$$theta_pl := 0.01545150502 \quad \textbf{(9)}$$

> $kN := \frac{kJ}{m} :$

> $E_pl := M_Rd \cdot theta_pl;$

$$E_pl := 8.174943464\ kJ \quad \textbf{(10)}$$

> #***Total Energy Absorbed***

> $E_tot := E_pl + E_el;$

$$E_tot := 11.08743893\ kJ \quad \textbf{(11)}$$

>

B.3. A500 break energy calculation

```
> restart :
> #A500 Parameters
> h := 500.·mm :
> c := 20.·mm :
> phi_l := 12.5·mm :
> l := 6210.·mm :
> E := 36000.·N/mm^2 :
> I_c := 8229 ·10^6·mm^4 :
> f_y := 1602.15·N/mm^2 :
> ep := 0.035 :
> d := h − c − phi_l/2;
```

$$d := 473.7500000\ mm \tag{1}$$

```
> z := 0.9·d;
```

$$z := 426.3750000\ mm \tag{2}$$

```
> A_p := 1/4·3.1415·(12.5·mm)^2·8;
```

$$A_p := 981.7187500\ mm^2 \tag{3}$$

```
> mm := m/1000 : N := kN/1000 :
> M_Rd := A_p·f_y·z;
```

$$M_Rd := 670.6284788\ m\ kN \tag{4}$$

```
> unassign('mm') : x_u := A_p·f_y/(0.85·45·N/mm^2·381·mm); mm := m/1000 :
```

$$x_u := 107.9279292\ mm \tag{5}$$

```
> #Elastic Strain Energy
> q := 8·M_Rd/l^2;
```

$$q := \frac{139.1197469\ kN}{m} \tag{6}$$

```
> kN := kJ/m :
> E_el := q^2·l^5/(240·E·I_c);
```

$$E_el := 2.514054881\ kJ \tag{7}$$

```
> unassign('kN')
> #Plastic Strain Energy
> L_pl := 2·c + 10·12.5·mm;
```

$$L_pl := 0.1650000000\ m \quad \textbf{(8)}$$

> $theta_pl := \frac{L_pl}{d} \cdot ep;$

$$theta_pl := 0.01218997361 \quad \textbf{(9)}$$

> $kN := \frac{kJ}{m} :$

> $E_pl := M_Rd \cdot theta_pl;$

$$E_pl := 8.174943459\ kJ \quad \textbf{(10)}$$

> #***Total Energy Absorbed***

> $E_tot := E_pl + E_el;$

$$E_tot := 10.68899834\ kJ \quad \textbf{(11)}$$

>

B.4. Calculation Drone impact

```
restart :
#A500 Parameters
h := 500.·mm :
c := 20.·mm :
phi_l := 12.5·mm :
l := 6210.·mm :
E := 36000.·N/mm^2 :
I_c := 8229 ·10^6·mm^4 :
f_y := 1602.15·N/mm^2 :
ep := 0.035 :
b := 1200·mm :
d := h − c − phi_l/2;
```

$$d := 473.7500000\ mm \tag{1}$$

```
z := 0.9·d;
```

$$z := 426.3750000\ mm \tag{2}$$

```
A_p := 1/4 ·3.1415·(12.5·mm)^2·8;
```

$$A_p := 981.7187500\ mm^2 \tag{3}$$

```
mm := m/1000 : N := kN/1000 :
M_Rd := A_p·f_y·z;
```

$$M_Rd := 670.6284788\ m\,kN \tag{4}$$

```
unassign('mm') : x_u := A_p·f_y / (0.85·45·N/mm^2 ·381·mm); mm := m/1000 :
```

$$x_u := 107.9279292\ mm \tag{5}$$

```
#Elastic Strain Energy
q := 8·M_Rd/l^2;
```

$$q := \frac{139.1197469\ kN}{m} \tag{6}$$

```
kN := kJ/m :
E_el := q^2·l^5/(240·E·I_c);
```

$$E_el := 2.514054881\ kJ \tag{7}$$

```
unassign('kN')
#Plastic Strain Energy
```

> $L_pl := 2 \cdot c + 10 \cdot 12.5 \cdot mm;$

$$L_pl := 0.1650000000\ m \tag{8}$$

> $theta_pl := \frac{L_pl}{d} \cdot ep;$

$$theta_pl := 0.01218997361 \tag{9}$$

> $kN := \frac{kJ}{m}$:

> $E_pl := M_Rd \cdot theta_pl;$

$$E_pl := 8.174943459\ kJ \tag{10}$$

> #***Total Energy Absorbed***

> $E_tot := E_pl + E_el;$

$$E_tot := 10.68899834\ kJ \tag{11}$$

> #***Explosion Energy***

> rho $:= 1.225 \cdot \frac{kg}{m^3}$: # *dichtheid van lucht*

> $c := 340.294 \cdot \frac{m}{s}$: # *snelheid van een golf door lucht*

> $W := 74. \cdot kg$: # *gewicht TNT*

> $R := 20. \cdot m$: # *afstand van de explosie*

> $Z := \frac{R}{W^{\frac{1}{3}}};$ # *geschaalde afstand*

$$Z := \frac{4.763791616\ m}{kg^{1/3}} \tag{12}$$

> $tc := solve\left(3.2 \cdot \frac{ms}{kg^{\frac{1}{3}}} = \frac{tc}{W^{\frac{1}{3}}}, tc\right);$

$$tc := 13.43467665\ ms \tag{13}$$

> $pso := 35 \cdot kPa$:

> $kg := \frac{N}{\frac{m}{s^2}}$: $N := \frac{kN}{1000}$: $ms := \frac{s}{1000}$: $kPa := \frac{kN}{m^2}$: kN $:= \frac{\text{kJ}}{m}$:

> $e_expl := \frac{1}{2} \cdot \frac{pso^2}{\text{rho} \cdot c} \cdot tc$;

$$e_expl := \frac{19.73980830\ kJ}{m^2} \tag{14}$$

> $E_expl := e_expl \cdot l \cdot b;$

$$E_expl := 147.1010514\ kJ \tag{15}$$

> $Unity_Check := \frac{E_expl}{E_tot};$

$$Unity_Check := 13.76191171 \tag{16}$$

B.5. Air Burst Calculation

```
> restart :
> Rg := 50·m :                                  # Explosion distance
> W := 450.·kg :                                # Weight of the explosive payload
> Z := Rg / W^(1/3);
```

$$Z := \frac{6.524779400\ m}{kg^{1/3}} \tag{1}$$

```
> pso := 20000.·Pa :                            # Peak reflected pressure, determined from the graph
> tc := solve(3.5·ms/kg^(1/3) = tc/W^(1/3), tc);   #Negative phase duration, determined from the graph
```

$$tc := 26.82083014\ ms \tag{2}$$

```
> E := 22000.·N/mm^2 :                          # E-modulus concrete
> h := 8625·mm :                                # Length of the shear wall
> b := 200·mm :                                 # Width of the shear wall
> l := 5000·mm :                                # Height of the shear wall
> c_nom := 20·mm :                              #Concrete cover
> M := 2679803.·kg :    # Mass of the surface structure absorbed by the shear wall
> d := h − c_nom − 1/2·8·mm; #Effective height of the cross-section
```

$$d := 8601\ mm \tag{3}$$

```
> A := 12800·mm· 53000.·mm;                     # Area of the facade
```

$$A := 6.78400000 \times 10^{8}\ mm^2 \tag{4}$$

```
> Ic := 1/12·b·h^3;
```

$$Ic := 10693652343750\ mm^4 \tag{5}$$

```
> Pa := N/m^2 : kN := 1000·N : m := 1000·mm : Pa := N/m^2 : ms := s/1000 : kg := N/(m/s^2) :
    #Defining units for automatic conversion
> assume(kN > 0) : assume(s > 0) : assume(mm > 0) :
> F := A·pso;                                   # Force of the explosion on the facade
```

$$F := 1.356800000 \times 10^{7}\ N\sim \tag{6}$$

```
> P := F·tc;                                    # Impulse of the explosion on the facade
```

$$P := 363905.0233\ N\sim\ s\sim \tag{7}$$

```
> k := 12·E·Ic/l^3;                             # Wall stiffness
```

$$k := \frac{2.258499374 \times 10^{7}\ N\sim}{mm\sim} \tag{8}$$

> $Fmax := P \cdot \mathrm{sqrt}\left(\frac{k}{M}\right);$ # Resulting force on the shear wall

$$Fmax := 3.340770202 \times 10^{7}\ N\sim \quad \textbf{(9)}$$

> $v := 0.6 \cdot \left(1 - \frac{20}{250}\right);$

#Reduction factor accounting for tension stresses in the stirrups

$$v := 0.5520000000 \quad \textbf{(10)}$$

> $Vrd_max := b \cdot (0.9 \cdot d) \cdot 20 \cdot \frac{N}{mm^2} \cdot v \cdot \left(\frac{1}{\mathrm{sqrt}(2.)}\right)^2;$

#Maximum shear resistance of the shear wall

$$Vrd_max := 8.545953604 \times 10^{6}\ N\sim \quad \textbf{(11)}$$

>

B.6. Ground shock calculation

```
> restart;
> m := ft/0.3048 : kg := lbs/0.45359237 : inch := 25.4·mm :
> Rg := 70.3·m;                    #Distance of explosion
```

$$Rg := 230.6430446\ ft \tag{1}$$

```
> W := 450.·kg;                    #TNT equivalent explosive mass
```

$$W := 992.0801799\ lbs \tag{2}$$

```
>
> Z := Rg/W^(1/3);                 #Scaled distance of explosion
```

$$Z := \frac{23.12551632\ ft}{lbs^{1/3}} \tag{3}$$

```
> u_H := 0.17·(Rg/ft)^(1/3)·(W/lbs)^(1/3) / (Z/(ft/lbs^(1/3)))^2.3 ·inch;
    #Horizontal ground displacement (This is an empirical formula,  the division by lbs. and ft. is
    done to get the right unit in the output)
```

$$u_H := 0.1924722143\ mm \tag{4}$$

```
> unassign('m', 'kg');
    #This is done to prevent the other calculations being displayed in ft. and lbs.
> b := 200.·mm :                   #Width of wall
> h := 8625.·mm :                  #Height of wall
> c_nom := 20.·mm :                #Concrete cover
> d := h - c_nom - 1/2·8.·mm;      #Effective height of wall cross-section
```

$$d := 8601.000000\ mm \tag{5}$$

```
> E := 22000.·N/mm^2 :             #E-modulus of the concrete
> L := 5000.·mm :                  #Span length of wall
> m := 1000·mm : N := kN/1000 :
> I_c := 1/12·b·h^3;
```

$$I_c := 1.069365234 \times 10^{13}\ mm^4 \tag{6}$$

```
> k := 12·E·I_c/L^3 ;              #Wall stifness
```

$$k := \frac{22584.99374\ kN}{mm} \tag{7}$$

> $F := k \cdot u_H;$ #Maximum Force in the wall

$$F := 4346.983755\ kN \quad \textbf{(8)}$$

> $v := 0.6 \cdot \left(1 - \frac{20}{250}\right);$ #Reduction factor

$$v := 0.5520000000 \quad \textbf{(9)}$$

> $Vrd_max := b \cdot (0.9 \cdot d) \cdot 20 \cdot \frac{N}{mm^2} \cdot v \cdot \left(\frac{1}{\mathrm{sqrt}(2.)}\right)^2;$ #Maximum shear resistance of the shear wall

$$Vrd_max := 8545.953604\ kN \quad \textbf{(10)}$$

>

B.7. Direct hit from a ballistic missile

```
> restart :
> #GROUND FLOOR
> l := 6.21·m :
> b := 1.2·m :
> fy := 1597.65·N/mm^2 :
> h := 173.75·mm :
> z := 0.9·h :
> c := 20·mm :
> ep := 0.035 :
> bw := 381·mm :
> A_c := 143823·mm^2 :
> f_ck := 45·N/mm^2 :
> k_1 := 0.5 :
> A_c := 143823·mm^2 :
> sigma_cp := 1100·N/mm^2 :
> D_missile := 0.68·m :
> mm := m/1000 : N := kN/1000 :
> #-------------------------------------------------------------------------------------------------------------
    ----
> A_p := 1/4·3.1415·(9.3·mm)^2·6 + 2·1/4·3.1415·(12.5·mm)^2;
```

$$A_p := 0.0006529921900\ m^2 \quad (1)$$

```
> rho_l := A_p/A_c;
```

$$rho_l := 0.004540248708 \quad (2)$$

```
>
> k := min(2.0, 1 + (200/(h/mm))^(1/2));
```

$$k := 2.0 \quad (3)$$

```
> vrd := 0.12·k·(100·rho_l·f_ck)^(1/3)·(kN/m^2)^(2/3) + k_1·sigma_cp;
```

$$vrd := \frac{550006.5611\ kN}{m^2} \quad (4)$$

> $u_1 := 2 \cdot 3.1415 \cdot \left(\frac{1}{2} \cdot D_missile + 2 \cdot h \right);$

$$u_1 := 4.319562500\ m \quad \textbf{(5)}$$

> $Vrd := vrd \cdot bw \cdot u_1;$

$$Vrd := 905175.1199\ kN \quad \textbf{(6)}$$

> $M := A_p \cdot fy \cdot z;$

$$M := 163.1386835\ m\ kN \quad \textbf{(7)}$$

> $F := solve\left(M = \frac{1}{4} \cdot F \cdot l, F \right);$

$$F := 105.0812776\ kN \quad \textbf{(8)}$$

> $kN := \frac{kJ}{m}$:

> $E1 := 7.23 \cdot kJ$:

> **save**$(l, b, c, ep, bw, f_ck, k_1, sigma_cp, D_missile, E1,$ "variables.m");

> *restart* :

> **read**("variables.m");

> #***FIRST FLOOR***

> $h := 373.75 \cdot mm$:

> $fy := 1602.15 \cdot \frac{N}{mm^2}$:

> $z := 0.9 \cdot h$

$$z := 336.375\ mm \quad \textbf{(9)}$$

> $A_c := 225911 \cdot mm^2$:

> $A_p := 8 \cdot \frac{1}{4} \cdot 3.1415 \cdot (12.5 \cdot mm)^2;$

$$A_p := 981.718750\ mm^2 \quad \textbf{(10)}$$

> $mm := \frac{\mathrm{m}}{1000} : N := \frac{kN}{1000}$:

> $rho_l := \frac{A_p}{A_c};$

$$rho_l := 0.004345599595 \quad \textbf{(11)}$$

> $k := \min\left(2.0, 1 + \left(\frac{200}{\left(\frac{h}{mm} \right)} \right)^{\frac{1}{2}} \right);$

$$k := 1.731516956 \quad \textbf{(12)}$$

> $vrd := 0.12 \cdot k \cdot (100 \cdot rho_l \cdot f_ck)^{\frac{1}{3}} \cdot \left(\frac{kN}{m^2} \right)^{\frac{2}{3}} + k_1 \cdot sigma_cp;$

$$vrd := \frac{550005.5979\ kN}{m^2} \quad \textbf{(13)}$$

```
> u_1 := 2·3.1415·(1/2·D_missile + 2·h);
```

$$u_1 := 6.832762500\ m \tag{14}$$

```
> Vrd := vrd·bw·u_1;
```

$$Vrd := 1.431819955 \times 10^{6}\ kN \tag{15}$$

```
> M := A_p·fy·z;                    # momentcapaciteit
```

$$M := 529.0710163\ m\ kN \tag{16}$$

```
> F := solve(M = 1/4·F·l, F); # plastische lijnbelasting [kN/m];
```

$$F := 340.7864839\ kN \tag{17}$$

```
> kN := kJ/m :
> E2 := 11.09·kJ :
> save(l, b, fy, c, ep, bw, f_ck, k_1, sigma_cp, D_missile, E1, E2, "variables.m");
> restart :
> read("variables.m");
> #SECOND FLOOR
> h := 473.75·mm :
> z := 0.9·h :
> A_c := 278359·mm^2 :
> A_p := 8·1/4·3.1415·(12.5·mm)^2;
```

$$A_p := 981.718750\ mm^2 \tag{18}$$

```
> mm := m/1000 : N := kN/1000 :
> rho_l := A_p/A_c;
```

$$rho_l := 0.003526808007 \tag{19}$$

```
> k := min(2.0, 1 + (200/(h/mm))^(1/2));
```

$$k := 1.649741170 \tag{20}$$

```
> vrd := 0.12·k·(100·rho_l·f_ck)^(1/3)·(kN/m^2)^(2/3) + k_1·sigma_cp;
```

$$vrd := \frac{550004.9750\ kN}{m^2} \tag{21}$$

```
> u_1 := 2·3.1415·(1/2·D_missile + 2·h);
```

$$u_1 := 8.089362500\ m \tag{22}$$

> $Vrd := vrd \cdot bw \cdot u_1;$

$$Vrd := 1.695141245 \times 10^{6}\ kN \tag{23}$$

> $M := A_p \cdot fy \cdot z;$

$$M := 670.6284788\ m\ kN \tag{24}$$

> $F := solve\left(M = \frac{1}{4} \cdot F \cdot l, F\right);$

$$F := 431.9668140\ kN \tag{25}$$

> $kN := \frac{kJ}{m} :$

> $E3 := 10.69 \cdot kJ$

$$E3 := 10.69\ kJ \tag{26}$$

>

> $E_total := E1 + E2 + E3;$

$$E_total := 29.01\ kJ \tag{27}$$

> #***Kinetic Energy of the missile***

> $M_missile := 4615. \cdot kg :$

> $v_missile := 3.3 \cdot 340.294 \cdot \frac{m}{s} :$

> $kg := \frac{N}{\frac{m}{s^2}} :$

> $E_k := \frac{1}{2} \cdot M_missile \cdot v_missile^2;$

$$E_k := 2.909900726 \times 10^{6}\ kJ \tag{28}$$

> $E_net := E_k - E_total;$

$$E_net := 2.909871716 \times 10^{6}\ kJ \tag{29}$$

>

B.8. Required reinforcement Calculations

```
> restart :
> assume(N > 0) : assumption := assume(mm > 0) :
> E2 := 60000. · N/mm^2 :
> E1 := 36000. · N/mm^2 :
> h1 := 200. · mm :
> z1 := 99.43 · mm :
> b := 1200. · mm :
> l := 6210./2 · mm :
> A1 := 143823 · mm^2 :
> I1 := 677.4 · 10^6 · mm^4 :
> f_ck1 := 45. · N/mm^2 :
> f_ck2 := 150/1.2 · N/mm^2 :
> f_carbon := 2400. · N/mm^2 :
> U := 597.235 · kJ :
> h2 := 2600 · mm :
> kJ := 1000 · J : J := N · m : m := 1000 · mm :
> read("variables.m");
> z := z_out; y_old := y_out;
```

$$z := 2457.660259\ mm\sim$$

$$y_old := 978.1135450\ mm\sim \quad \textbf{(1)}$$

```
> #-------------------------------------------
> h_tot := h1 + h2;
```

$$h_tot := 2800.\ mm\sim \quad \textbf{(2)}$$

```
> z0 := 0.9 · h_tot;
```

$$z0 := 2520.0\ mm\sim \quad \textbf{(3)}$$

```
> I2 := 1/12 · b · (h2)^3;
```

$$I2 := 1.757600000 \times 10^{12}\ mm\sim^4 \quad \textbf{(4)}$$

```
> A2 := b · (h2);
```

$$A2 := 3.120000 \times 10^{6}\ mm\sim^2 \quad \textbf{(5)}$$

```
> n := E2/E1;
```

$$n := 1.666666667 \quad \textbf{(6)}$$

> $z2 := h1 + \frac{h2}{2};$

$$z2 := 1500.\ mm\sim \tag{7}$$

> $ze := \frac{A1 \cdot z1 + n \cdot A2 \cdot z2}{A1 + n \cdot A2};$

$$ze := 1462.305230\ mm\sim \tag{8}$$

> $I_trans := \left(I1 + A1 \cdot (ze - z1)^2\right) + n \cdot \left(I2 + A2 \cdot (ze - z2)^2\right);$

$$I_trans := 3.204540388 \times 10^{12}\ mm\sim^4 \tag{9}$$

> $E1 \cdot I_trans$

$$1.153634540 \times 10^{17}\ N\sim\ mm\sim^2 \tag{10}$$

> $q := \mathrm{sqrt}\left(\frac{240. \cdot I_trans \cdot E1 \cdot U}{l^5}\right);$

$$q := \frac{239363.7390\ N\sim}{mm\sim} \tag{11}$$

> $M_Rd := \frac{1}{8} \cdot q \cdot l^2;$

$$M_Rd := 2.884639740 \times 10^{11}\ N\sim\ mm\sim \tag{12}$$

> $N_c := \frac{M_Rd}{z};$

$$N_c := 1.173734136 \times 10^{8}\ N\sim \tag{13}$$

> $y := \frac{1.25 \cdot N_c}{b \cdot f_ck2};$

$$y := 978.1117800\ mm\sim \tag{14}$$

> $z := h_tot - 0.35 \cdot y;$

$$z := 2457.660877\ mm\sim \tag{15}$$

> $NEN := \frac{y}{h_tot};$

$$NEN := 0.3493256357 \tag{16}$$

> $A_carbon := \frac{M_Rd}{z \cdot f_carbon};$

$$A_carbon := 48905.57671\ mm\sim^2 \tag{17}$$

> **if** $\frac{\mathbf{y}}{mm} < \frac{\mathbf{h2}}{mm}$ **and NEN < 0.35 then**
>
> **print**("Drukzone voldoet met een concstructieve druklaag van h2=", $h2$);
>
> **end if;**

"Drukzone voldoet met een concstructieve druklaag van h2=", 2600 $mm\sim$ (18)

> **if** abs$\left(\frac{y}{mm} - \frac{y_old}{mm}\right) < 0.1$ **then**
>
> $print$("----------Iteratie Voldoende----------
> ----------");
>
> **end if**;

"----------Iteratie Voldoende---------- **(19)**

----"

> $z_out := subs([mm = 'mm', N = 'N'], z); y_out := subs([mm = 'mm', N = 'N'], y);$

$$z_out := 2457.660877\ mm$$

$$y_out := 978.1117800\ mm \quad \textbf{(20)}$$

> **save** z_out, y_out , "variables.m" :

>