

A detailed structural design of a data centre in Frankfurt am Main area,
In accordance with the national construction regulations

Final Report for the Civil Engineering Bachelor thesis

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Arup Deutschland GmbH

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Abstract

This final report for acquiring the bachelor of Civil Engineering focuses on the structural and geotechnical design of the foundation of a data centre, to be built on contaminated soil. In order to determine the most suitable design in accordance to *the Client's* requirements, the research into the structural and geotechnical design specifications of foundations was carried out, applying the acquired information to the project.

Three different foundation techniques (these included a raft foundation, the large diameter mono-piles as well as a conventional pile group design) in two different locations were considered for finding the most suitable solution. The analysis of the six solutions was done using a selective method. After preparing the preliminary designs for all three variants, two of the solutions were ruled out, either for not being structurally viable or for being considered as conservative. The remaining solutions were compared and evaluated based on the two criteria – duration and cost, finally coming to the final decision.

The purpose of investigating the possibility for the relocation of the building was to reduce the amount of polluted soil within the construction site. After the evaluation of the construction site logistics and the volume of soil to be exchanged, the decision was made to change the location of the building and allow the replacement of the required soil prior the construction of the foundation.

The concluding evidence showed the pile group design in the altered location to be the most suitable choice. The advantage of quick installation, as well as lower preliminary direct costs were the evidence for the decision made.

All criteria considered, the final design of the foundation resulted in a pile group foundation of 210 bored friction piles, 1.2 metres in diameter and varying in length between 10 and 20 metres. The piles were grouped by either four or six units, connected via pile cap, which then had a depth of either 1.8 or 2 metres. Additional design considerations were taken for the row of columns on axes A and B, as well as the area for staircases and elevators, located outside of the main building grid (for more details see “Appendix F – Final design”).

The design is based on the German annex of the European construction regulations – Eurocode 2 “Design of concrete structures”, Eurocode 3 “Design of steel structures” and Eurocode 7 “Geotechnical design”, as well as a set of national regulations *DIN-Normen*, where necessary.

With the final foundation design, the competencies in both geotechnical and reinforced concrete designs are shown. In order to prove the application of the steel design regulations, an additional steel structure for a façade was designed in accordance to Eurocode 3 and can be found in “Appendix G – Façade sub-structure design”.

For preparing the design, different software were used: *Microsoft Excel* for optimizing the manual calculations, *Frilo* for the distribution of horizontal loads, design of beams and the shallow foundation. *Sofistik* for the raft foundation analysis and *GSA* for the steel structure design. The drawings were prepared with *AutoCAD 2016*.

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1. Introduction

The placement for the graduation thesis has been provided by *Arup Deutschland GmbH* in Frankfurt am Main, Germany - a widely known independent firm of designers, planners, engineers, consultants and technical specialists, offering a broad range of professional services in different fields of engineering. The office in Frankfurt is mostly involved in the design of industrial buildings and operates all over the world. For the purpose of the final thesis, the possibility to join the structural engineering team in designing this industrial facility has been provided by the company.

Having signed a Non-Disclosure Agreement, any *Arup Deutschland GmbH* employee involved in the project is prohibited from sharing the name of *the Client*, as well as any detailed project related information with any third party.

1.1.Location

The project site is located in the North-Eastern part of Frankfurt am Main in Germany (Figure 1-1) and it is an industrial area with several already existing buildings and a couple yet to be designed.



Figure 1-1 Location of the project site – Frankfurt am Main,

The construction site is a considerably flat area of 17,000 square meters with the topography, varying between 99.50 and 101.10 mNN¹. Since the year 1980, this area was an old industry, until *the Client* purchased the land and took over the facilities, shortly after that establishing a clear vision for its future development.

However, due to the infrastructural pre-utilisation in the past, the remains of the slag substrates increased the heavy metal solids content in the subsoil level up to 6m deep in the building site area, varying in depth, depending on the location.

Unfortunately, there is no detailed information about the exact amount of contaminated soil, however the indication of the polluted layers within the soil profile can be seen in Figure 1-2.

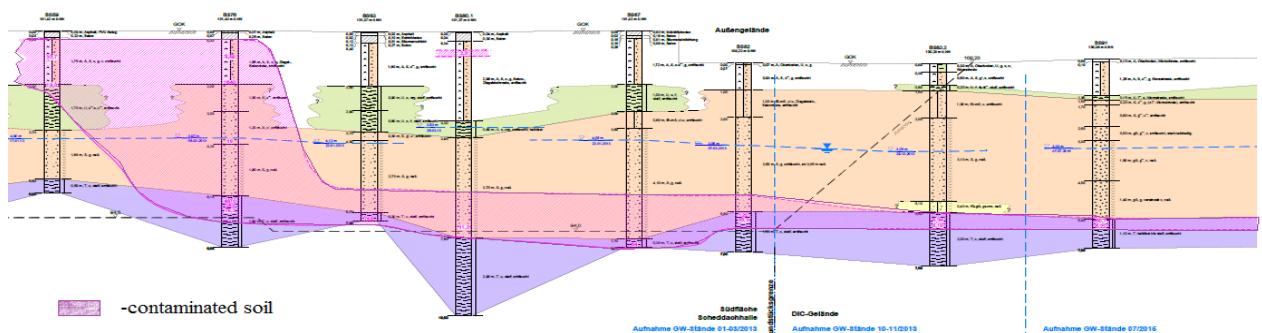


Figure 1-2 the indication of contaminated soil (highlighted in purple) within the soil profile

¹ mNN – “meter über Normal-Null“ is a German vertical datum system. “Normalnull” was defined with a reference to the Amsterdam Ordnance Datum in 1879, and it represents an imaginary point 37 metres below the *Normalhöhenpunkt 1879*

The contaminated soil contains slag-like filling substrate – stony waste matter, separated from the metals. If the soil is not removed, the groundwater may be subject to the toxic elements and therefore may get polluted, causing a public health issue in the area (IBU Hofmann, 2011). Because of this reason, the local government has issued an official command to replace the polluted soil prior any new construction activities in the area.

The Figure 1-2 represents one of the cross-sections, obtained from the geotechnical expert after the performed ground survey. Because of the reason, that the contaminated soil layer is not constant throughout the area, it was not able to estimate the exact amount of it. However, judging from this cross-section, the layer is usually 1-2 metres thick (except some certain locations with the large accumulated amounts, like on the left side of the Figure 1-2. In addition, it is safe to assume that no less than fifty per cent of the total soil volume is unpolluted soil and can be returned to the excavation pit after the removal of the contaminated layers (IBU Hofmann, 2011).

1.2. Influence of the contaminated soil

In the Figure 1-3, the initial planning is presented with the buildings within the site, owned by *the Client*. The layout shows the planned position of the buildings prior taking the necessary soil remediation activities and their influence into consideration.



The two buildings in the South-West FR2.6 and FR2.7 are not built yet. Both of them will have a function of a data centre, and FR2.6 is the one, analysed in this report. Note that there is an old basement still present underneath the planned data centre FR2.6, which needs to be demolished and removed together with the polluted soil.

Figure 1-3 Initial Master Plan, with contaminated soil is indicated in brown colour (Celli, 2017)

The remaining buildings are already existing:

- FR2.5 and FR2.3/4 were recently built by *the Client*, the design was made by *Arup*; FR2.5 is a working data centre and because of this reason, any vibrations from the installation of the retaining structures and the foundation for the new buildings nearby are strictly forbidden;
- Logistics facility is the building taken over from the previous owner, still in use at the moment, however, planned to eventually be reconstructed in the future
- The exact function of the remaining buildings is unknown.

Figure 1-4 shows the visual representation of the polluted soil and the planned ground excavation sequence. The complete area of 7211 m² is divided into six parts – EB1 to EB6 respectively. The soil excavation activities were planned by the geotechnical expert company.

Depending on the depth at which the contaminated soil lays within the soil profile, the amount of soil to exchange is different. The exact amounts of the contaminated soil were not defined in detail, however the thickness of the contaminated layers is usually between one and two metres (Huber, Toker, & Glaser, 2017).

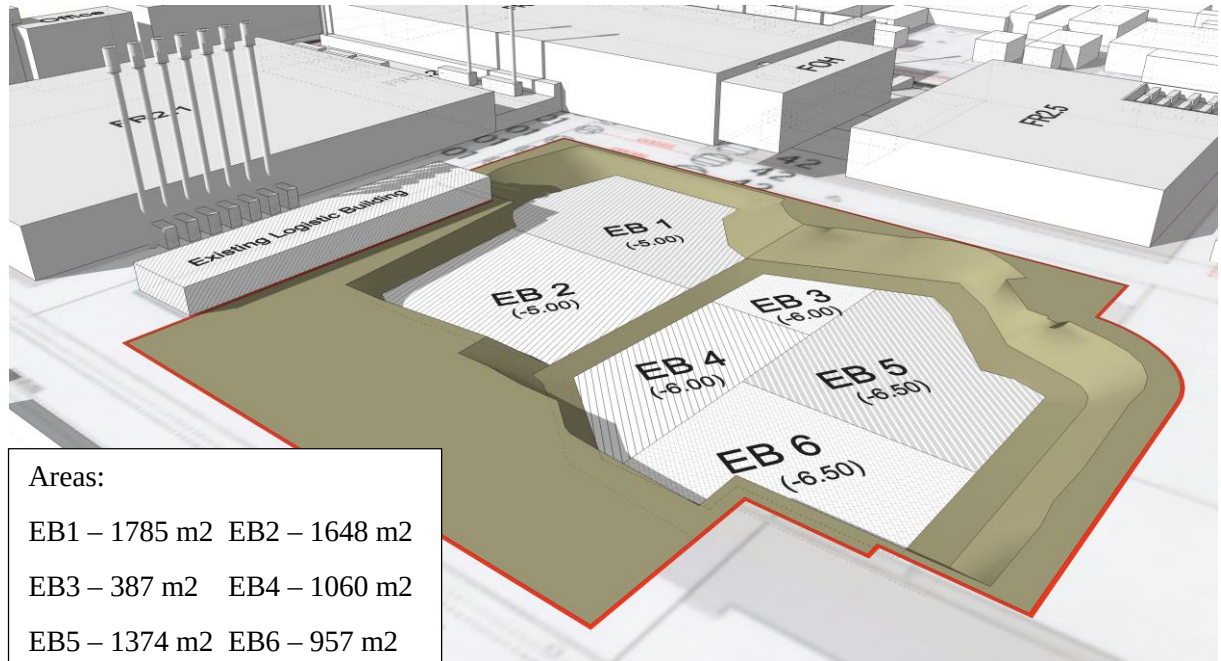


Figure 1-4 Soil remediation areas describing the planned sequence of the ground works, the numbers in brackets indicate the depth of excavation in meters below ground level (Celli, 2017)

The total volume of soil to exchange is 41 000 m³ (Huber, Toker, & Glaser, 2017). It is important to mention however, that a part of the excavated soil, which is not polluted, may be placed aside and brought back to the building pit, together with the clean soil, brought from the outside location.

In addition, after demolishing the previously existing building, an underground basement has been left in the ground (Figure 1-5). During the soil exchange activities, the structure is planned to be dismantled and refilled with clean soil.

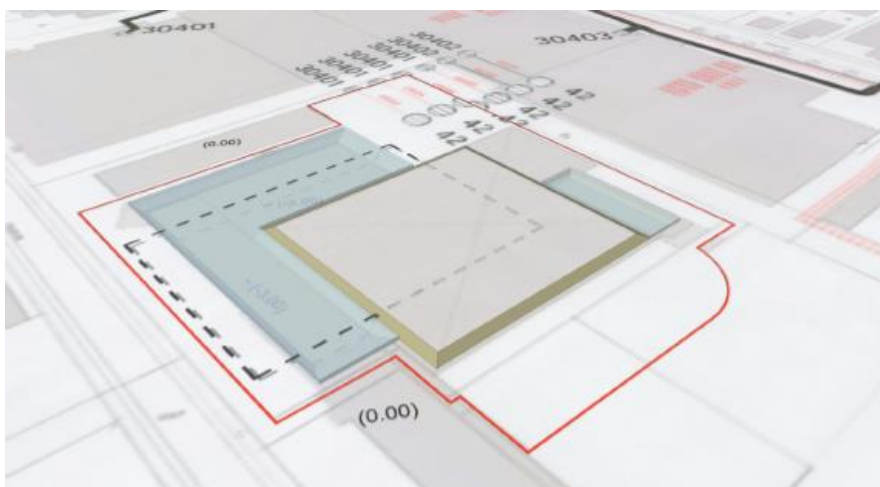
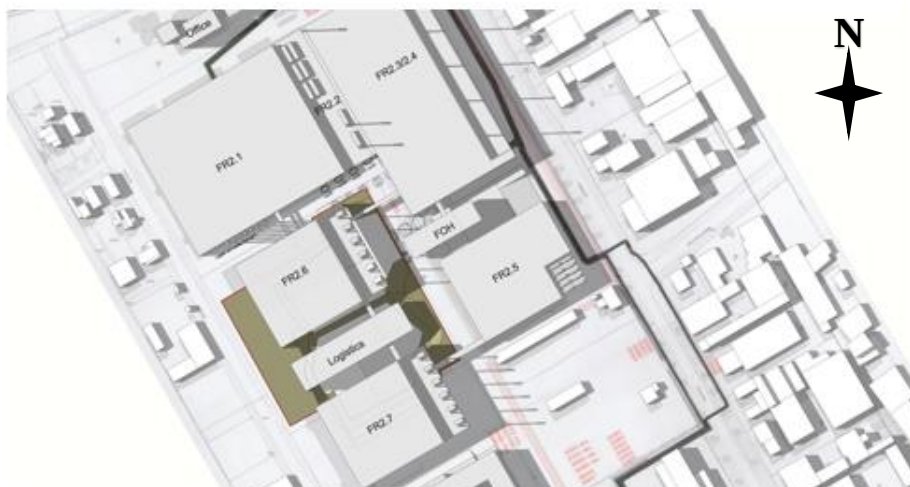


Figure 1-5 Existing basement (in blue) to be dismantled prior any soil remediation activities (Celli, 2017)

Since the new data centre is located right above the area with contaminated soil, the remediation activities must be completed prior the construction. The duration of the construction activities is a very important factor for the Client, due to the competitiveness between the companies within the same field of work.

In order to address the issue of the contaminated soil, three foundation design variants were compared in this report in order to select the most suitable solution. However, aside from analysing the foundation techniques and their installation influence to the project planning, *Arup* proposed the possibility to alter the initial Master Plan and change the location of the new building in order to minimise the amount of polluted soil below it. After the discussion with the Client, it was decided that the location altering alternative should be analysed as well and could potentially be accepted, if proper reasoning is provided.

For the implementation of the building in the newly proposed location, it was argued to first complete the exchange only for the areas EB1 and EB2 and immediately start the construction, allowing the other areas to be finished independently. In this case, the alternative is to demolish the logistics building and move the FR2.6 data centre further towards the North-West (Figure 1-6).



The demolition of the logistics facility would however require the temporary storage of the logistics equipment, until the new building is designed and built. For that purpose *Arup* would provide the design of a steel shed, which is not within the scope of this thesis.

Figure 1-6 Proposed new Master Plan, relocation of the buildings to be confirmed (Celli, 2017)

The logistics building is a simple, single-storey precast concrete structure, therefore its demolition is expected to take from 2 weeks up to a month, together with sorting out the materials and cleaning up the site. This value is based on the experience from the previous projects, also taking into account a confined working space and the construction in the city. In terms of the construction planning, the duration of the excavation works for the initial master plan (all six areas EB1 to EB6) is comparable to the excavation of the two areas (EB1 and EB2), including up to a month for the demolition of the logistics building.

The analysis of the different foundation techniques and, if necessary, the idea for the relocation of the new data centre, is further developed and analysed in the following chapters of the report.

1.3. Agreements with the previous owner

When purchasing the land from the previous owner, a contract was signed between the two parties - them and *the Client*. In that contract, the presence of contaminated soil was recognized, assigning the responsibility of the required remediation activities and their expenses to the previous owner.

The contract was signed in the year 2012, allowing four years for soil exchange and the handover, so that *the Client* would be able to build the data centre in one year and have it running in 2017.

However, due to the lack of collaboration and the failure of the previous owner to recognise the responsibility for taking care of the polluted soil, the ground investigations were delayed as well.

At the moment, the project is developed in a way that *the Client*, the current owner of the land, takes care of exchanging the polluted soil directly affected by the construction of the new building (earlier described as areas EB1 to EB6) and the previous owner covers the necessary costs. Other ground work related activities in the remaining area is fully the responsibility of *the Client*.

1.4. Implementation requirements

Due to the failed collaboration between the two parties and the lack of responsibility from the previous owner of the land, the project is suffering from at least one year of delay. Because of this reason, any design decisions taken at this moment must assure that the implementation time of the ground works and construction of the building takes as little time as possible.

After the long discussions with the Client, stretching out through several weeks of private and design meetings, a new time planning was discussed, analysing the influence of the different foundation design alternatives as well as the location altering possibility.

By following the new planning, a new data centre is expected to be finished and working within 12 months, out of which 7 months are planned for the building superstructure: 3.5 months for building of the superstructure, 3.5 months for “fit out” (the setting up of the electrical and mechanical equipment). The remaining 5 months should cover the groundwork and the foundation construction.

Because of the limited amount of time to finish the project, a quick implementation shall be kept in mind when undertaking any design decisions.

1.5. Overview of the building superstructure

The total height of the building is 32.6 m (Table 1-1), as the latest dimension with the reference to the architectural drawings (Figure 1-7). This value includes a 6.15 m high sound barrier². The barrier in the drawing is however still shown lower than designed in reality. The height has to be equal to the cooling equipment. The total area, which the building covers, is: $89\text{m}^2 * 61\text{m}^2 = 5429\text{m}^2$

Table 1-1 Floor-by-floor building dimensions

Floor levels:	Floor height (m)	Upper edge ceiling slab (m)
(Sound barrier)	(6.15)	(32.6)
3 rd floor	7.25	26.45
2 nd floor	6.4	19.2
1 st floor	6.4	12.8
Ground floor	6.4	6.4

Note that because the barrier is a cantilevered free-standing wall and the presence of the technical equipment cannot be accounted for, the values for the force coefficient for the walls are increased for safety reasons. See chapter “Schedule of requirements” for more details.

The height of the top floor is considered as 7.25m in the calculations, where necessary, and it is an average value, as the height varies because of the sloped roof. The height at the edge of the building is equal to the other floors – 6.4 m. The superstructure consists of the following reinforced concrete elements: Precast beams and columns, half-precast TT-slabs, precast walls, and *in-situ* foundation elements.

² Sound barrier – a sound-proof wall on the roof, designed to minimize the noise produced by the mechanical equipment on the roof. In *Frilo* software output it is referred to as “Attic” or “Attika” in German.

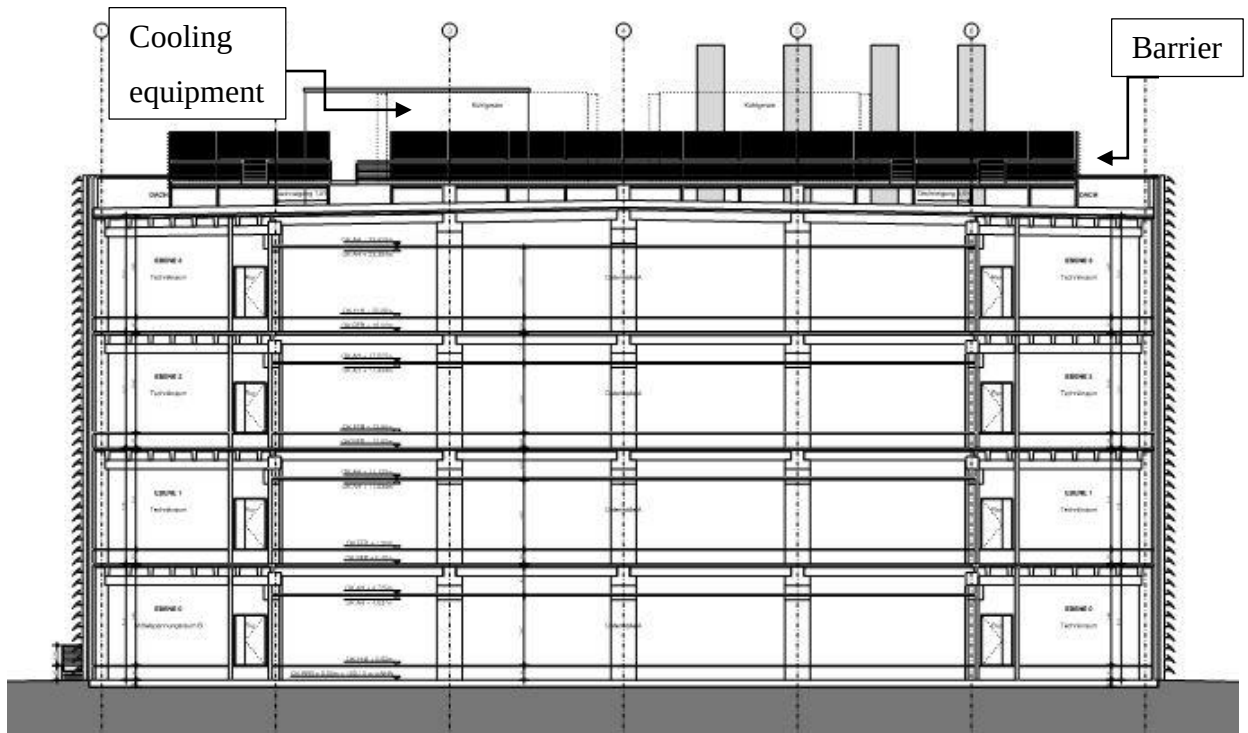


Figure 1-7 Section - building superstructure (architectural drawing) (Celli, 2017)

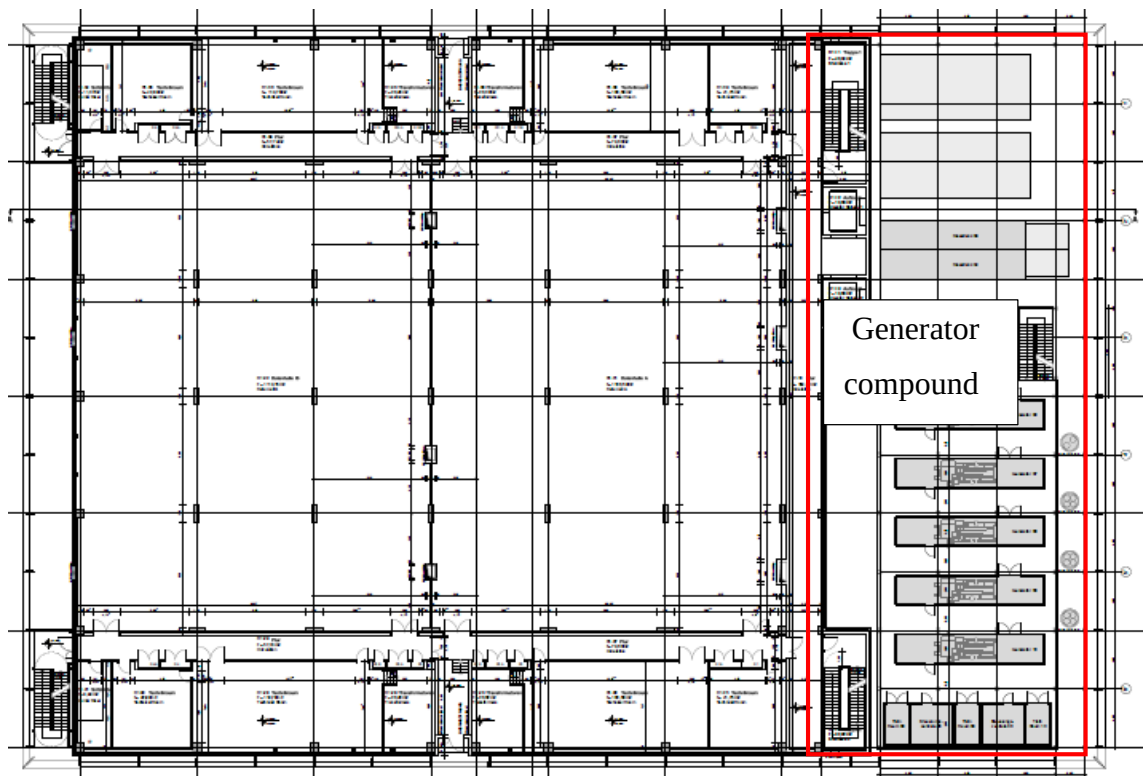


Figure 1-8 Building plan view first floor (architectural drawing); column spans 9.6m; the area under “generator compound” is not within the scope of the thesis (Celli, 2017)

1.6. Specifications of a data centre

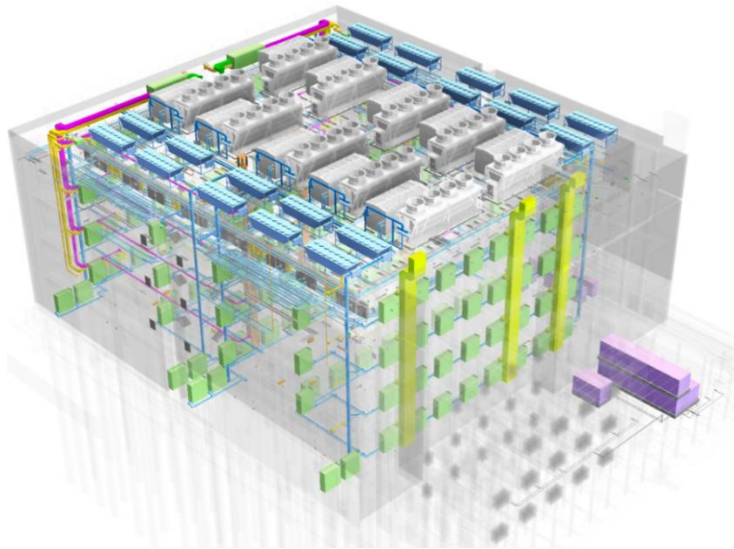


Figure 1-9 A visual representation of the data centre – mechanical design overview (Celli, 2017)

The design of a data centre requires the consideration of a few specific design decisions, applicable to these type of buildings, which are important to be mentioned.

To begin with, the structural design decisions have a major effect not only on the cost, but also on the performance of the data centre (Datacenter Dynamics, 2008). The function of its infrastructure is to support heavy Information Technology equipment. To assure a qualitative functioning, the facility requires great amounts of power supply as well as constant cooling (Figure 1-9).

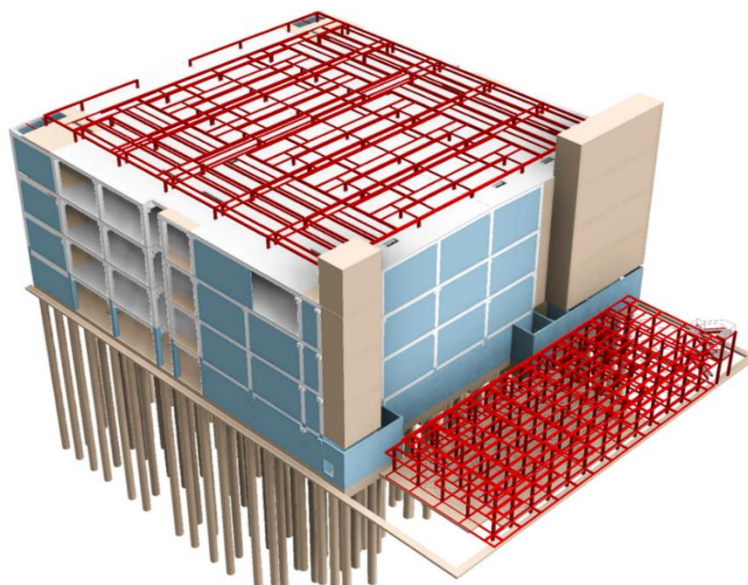


Figure 1-10 A visual representation of the data centre – structural design overview (Celli, 2017)

These circumstances add specific design considerations to the structure of the building (Datacenter Dynamics, 2008). For instance, the heavy cooling system is usually located on the roof of the building or next to it, placed on a steel supporting structure, called a gantry (Figure 1-10). As a consequence, it increases the vertical loads on the building and its foundation.

Since the building is to be built in the city, the sound, produced by the equipment on the roof, has to be reduced, therefore a sound-proof barrier, a free-standing wall, is requested to be built. The barrier consequently increases the total height of the building, having an

effect to the overall stability. Aside from that, because of another functioning data centre close by, no vibrations are allowed when installing the foundation. Therefore, a possibility for bored concrete piles was investigated (Barnes, 2010).

In addition, column spacing can also have a big impact on the production, as it determines, how many racks of equipment can fit within the designed grid. Therefore, large spans between columns are not uncommon. As well as that, designing the building with higher raised floor can have a positive impact on the construction and operating costs, as it creates a possibility for better cooling performance (Datacenter Dynamics, 2008).

1.7.Problem statement

The main issue regarding the design of the data centre, was the delayed preparation for the treatment of the polluted soil within the construction site. Because of this interruption of the initial planning, a prolonged preparation for the construction of the building and consequently a further finish date was anticipated. If no alterations are made to the design or the planning, the building would be finished at end of the year 2019 (Figure 1-11). A building construction time of twelve months is considered here, to assure the implementation of any design chosen for the foundation.

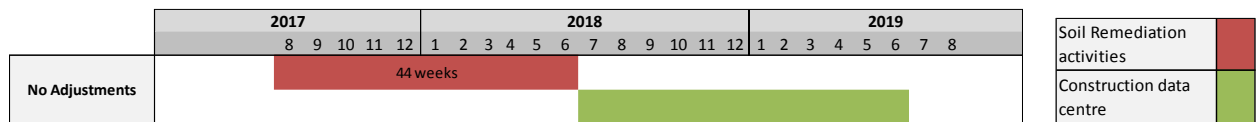


Figure 1-11 Planning initial situation

However, *the Client's* wish is to quicken the implementation process and therefore, *the Client* has requested to come up with a solution which allows the data centre to be built and functioning in 12 months.

Therefore, the goal of the research was to analyse the different foundation techniques, their structural integrity for this design and the implementation details, also considering the possibility for the relocation of the building and the influence of it to the initial planning.

For obtaining qualitative results from the research and preparing a proper design, the main research question and the set of sub-questions were used as a guidance.

Main question:

What is the optimal solution for the foundation design of a data centre, to be built on contaminated soil in Frankfurt am Main area, with respect to the duration and cost of the implementation activities, by following the national construction regulations and common practice?

Research sub-questions:

1. What are the most important issues, regarding the soil type and its contamination, as well as the groundwater level and the surroundings of the construction site?
2. What are the boundary conditions, functional and technical requirements of the project?
3. What are the permanent and temporary loads, acting on the structural elements of the building and the foundation?
4. What are the preliminary direct costs for each foundation variant, including materials and installation?
5. How long does it take to implement each of the foundation techniques and can the process be quickened?
6. Can the total implementation time be shortened, if the location of the new building is changed and by how much?
7. What is the influence on the construction site logistics for the movement of the machinery, considering both building locations?
8. What influence do the demolition activities have on the overall implementation of the project?
9. What is the optimal design and the necessary material properties, regarding the durability of the final structure?

The set-up of the report is as described below:

Chapter 2 Theoretical Framework: The chapter describes the theory behind the research problem and serves as a basis for carrying out further research activities, including the functional and technical requirements, boundary conditions and the scope of thesis. The three selected foundation variants and the two analysed locations are introduced in this chapter as well.

Chapter 3 Method: The chapter describes the sequence of the research and design activities, which were undertaken in order to analyse the main problem and come up with a qualitative solution, for which this report serves as a final product. The chapter also describes the cross-reference of the foundation variants and two locations, resulting in six possible outcomes. Followed by the choice of the evaluation criteria, the methodology chapter shows the strategy, which was followed for reaching the final optimal solution for the research problem.

Chapter 4 Results: The chapter presents the preliminary designs of the foundation techniques. After evaluating those designs, only structurally viable solutions were considered further and then compared according to the remaining two criteria – duration and cost. The final solution is therefore chosen at the end of the chapter. The choice is based on the selective method, explained in the methodology chapter.

Chapter 5 Final foundation design: The chapter contains an overview of the structural foundation design of the selected variant, describing the design steps taken and the most important details – basis for the structural calculations. The full calculations are however provided in the appendix of the Final Design.

Chapter 6 Discussion. In the discussion chapter, the possible variations of results are described, considering what could be improved within the method or the final design, some recommendations are provided in terms of optimising the design and the foundation costs.

Chapter 7 Conclusions of the research, conducted for answering the main research question and preparing the final product – foundation design.

Chapter 8 References – list of references, works cited.

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2. Theoretical framework

In this chapter, the theoretical framework is presented. It describes the theory behind the research problem and serves as a basis for carrying out further research activities. For this purpose, the applicable construction regulations are mentioned, the current soil conditions are presented and the selection of the variants for the foundation techniques is described. In addition, an alternative for the current location is introduced in order to analyse if it is worth changing the initial master plan.

Step 1

- Description of the soil conditions in the area

Step 2

- Schedule of requirements and the project scope

Step 4

- Description and analysis of the three foundation variants

Step 4

- Introduction of the location alternative

Step 5

- Application of the soil parameters for different foundation designs

2.1. Soil conditions in the study area

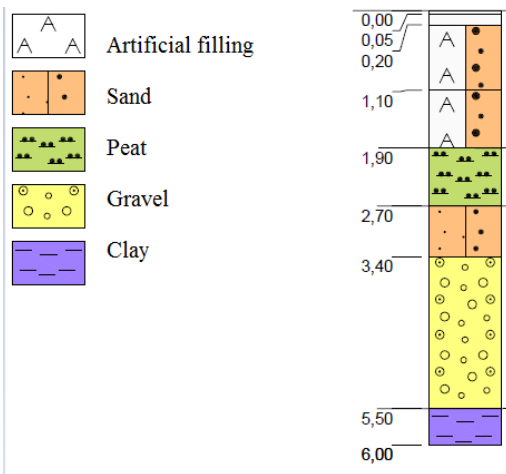


Figure 2-1 Soil profile (Huber, Toker, & Glaser, 2017)

The soil testing in the area of study was carried out by an external company, specializing in geotechnical engineering. The results, which were received, are summarised in this chapter with the reference to the official report, which is not provided as an appendix due to a Non-Disclosure Agreement between *Arup Deutschland GmbH* and the geotechnical engineering company.

Ground level is on average 100.1 mNN. Soil subsurface layers in the area of study are dominated by peat, sand and gravel up to 5-5.5 m depth, holding a moderate to poor bearing capacity. The most homogeneously formed, highly cohesive soil - clay - is below, 5-5.5 to 30 m deep with high uniaxial compression bearing capacity (Huber, Toker, & Glaser, 2017).

The groundwater is found varying in the top 5 metres below the ground surface level with maximum of 98.5 mNN (- 1.6 m below ground level “GL”). The values of soil capacity described in Table 2-1 represent the situation prior to the soil exchange.

Table 2-1 Soil properties in the study area (Huber, Toker, & Glaser, 2017)

Layer		Weight		Shear strength			Stiffness
		γ	γ'	ϕ	c'	c_u	E_s
		[kN/m ³]		[°]	[kN/m ²]	[kN/m ²]	[MN/m ²]
1	Fill	18 ÷ 19	9 ÷ 10	22 ÷ 30	0, 2	--	--
2	Peat	19 ÷ 20	9 ÷ 10	20, 25	2, 5	20 ÷ 50	5, 10
3	Sand	19 ÷ 21	9 ÷ 11	35,0 ÷ 37,5	--	--	40 ÷ 80
4	Clay	20	10	20	15, 25	> 80	10, 30

Once the soil remediation is complete, it is expected to have better bearing soil capacity in the top five to six metres of the soil profile. However, before the submission date of this thesis it was not possible to determine the increase in soil bearing capacity and thus the initial values were used for the preliminary designs as well as the final design.

Furthermore, due to the choice of two possible locations for the building, the lower values were used prior receiving the results from the multi-criteria analysis, as one of the two locations does not require soil excavation within the complete area of the building which would imply present values for soil parameters to stay the same and not improve.

2.2. Schedule of requirements

This chapter describes the full overview of the boundary conditions, functional and technical requirements, either demanded by the Client or ascribed in the national construction regulations.

2.2.1. Functional requirements

- The new building will have a function of a data centre, all necessary considerations, which may have some influence on the structural design, shall be considered, these include large spans between columns, high ceilings, importance of the mechanical installations within the building;
- The implementation time is to be optimised so that the duration does not exceed the requirements of *the Client*.
- The design variants are to be evaluated and compared between each other, in order to successfully select the optimal solution by following the chosen method.
- The preliminary designs of the variants as well as the final foundation design are to be prepared taking into account the national construction regulations for structural and geotechnical design, as well as the common practice, in addition to using the experience from the similar projects.
- The final foundation design should be coherent with the schedule of requirements and boundary conditions. It shall be prepared in a way that it is feasible and realistic.

2.2.2. Technical requirements

In Germany the design of structures is regulated either by the German Annex to the Eurocode or by national regulations *DIN Normen* (*English: DIN regulations*), where necessary. The distinction between the codes used was chosen following the practice of K.J. Schneider in *Bautabellen für Ingenieure*³.

Loads and combinations (determining self-weight of materials, defining permanent and variable area loads on structures):

- DIN EN 1991-1

Foundation design (application of the soil conditions to the foundation design and the installation, selecting foundation variants, specifying pile reinforcement detailing):

- DIN EN 1997-1 (Eurocode 7 for geotechnical design)
- DIN 1054: 2010 (national regulations)

³ English: *Construction tables for engineers*

- DIN EN 1536: 2015 (regulations for special geotechnical design: bored piles)
- DIN SPEC 18140: 2012 (regulations for special geotechnical design: bored piles)

Reinforced concrete design (determining the required concrete classes for different concrete elements, specifying reinforcement design for pile caps, foundation slab, integrated beams and shallow foundation):

- DIN EN 1992-1-1 (Eurocode 2)
- DIN 1045-2

Steel design (selection of applicable steel profiles, checking their strength and stability, designing welded and bolted connections):

- DIN EN 1993-1 (Eurocode 3)

A detailed description of the technical requirements can be found in “Appendix A – Schedule of technical requirements”.

2.3.Boundary conditions

- The building dimensions are approximately 61 m x 58 m, four storeys high (ground floor to second floor – 6.4m, third floor – 7.25m with an inclined roof);
- The column locations are based on a symmetrical 9.6x9.6m grid, except for those on axis A. The length of spans is quite long, making it useful for the processing of the data centre, as more equipment rows can fit between the columns;
- The contaminated soil directly under the new building must be exchanged with clean soil as per requirement from the government, in order to avoid public health issues in the area.
- The soil parameters for the design are to be provided by an external company.
- The duration of the project implementation is the first criterion, communicated by *the Client*, for choosing the foundation design. Once the duration can be optimised, the costs are to be considered accordingly;
- The complete duration of the project (from start of the soil remediation to the working data centre, excluding the documentation acquiring process) is to take- no more than 12 months. The start shall be considered as August 2017 and the building to be finished and working the summer of 2018.
- Regarding the implementation of the foundation, the vibrations are not allowed because of an operating data centre close by:
 - Bored concrete piles to be designed;
 - If retaining structures are necessary, they must be installed using a static load to prevent strong impact to the environment;
- The logistics building is to be demolished (second location to be considered) only after proving that it is worth doing so – the solution is time- and cost-efficient.
- The locations to be compared depending on their accessibility and the amount of soil to exchange.

The further sub-chapters describe the scope of the thesis, describing which aspects were not considered when performing the research and design activities throughout the different stages of the project.

2.3.1. Vertical and horizontal loads

- The self-weight of the ground floor slab is left out from the calculation of vertical loads.
- Any openings in the slabs or walls are ignored for the purpose of self-weight calculation and the distribution of horizontal loads.
- The staircases/elevator shafts are not included in the calculation for vertical loads because of multiple penetrations in or the absence of slabs.

2.3.2. Preliminary design

- The staircases and elevator shafts were not considered for the determination of loads for the basic design. When final design was prepared, the applicable loads were added to the overview and the updated version can be seen in “Appendix F – Final design”.
- Concept design of foundation alternatives based only on the effect of vertical loads.
- For the basis of concept pile design, the pile cap was not designed, therefore the weight of it is also not included in the self-weight of the pile groups.
- The reduction factor for bearing capacity due to the group effect for the pile groups was not considered, assuming the distance between two piles is not less than three times the pile diameter (Schneider, 2010).
- Four pile categories were chosen only for the purpose of cost and installation comparison, without taking into account the positions of the piles. For the detail design the location and number of piles was considered as fixed.
- The manual calculations for the preliminary design of the foundation variants do not include the wind loads, these were only considered in the Sofistik software model for the analysis of the raft foundation.

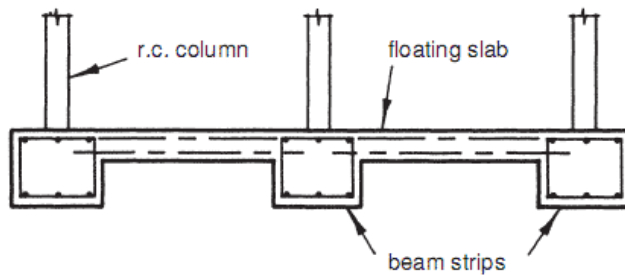
2.3.3. Final design

- The lateral loads due to temperature differences, seasonal moisture variations, frost action induced movements were not considered in the final design of the pile foundation.
- For simplification purposes, all piles subject to column loads were grouped by either four or six, in order to calculate the pile cap as a combination of beams for the reinforcement design. The loads were assumed to be transferred equally between the adjacent piles. In case of two columns designed per one pile cap, the decisive load was considered.
- The connection between the precast column and the pile cap is with special anchors, for which the details are mentioned in the Appendix of final design, but the design is not included in the scope of the report.
- Due to the lack of information in the geotechnical report for the soil parameters about the current or the future (after soil exchange) situation, the following decisions were made:
 - the design was completed in accordance to the current situation and must therefore be revised once the updated geotechnical values are obtained;
 - Assuming that the clay layer is over-consolidated and the increase of the stresses is not high, the effect of down-drag was not taken into consideration for the final design of piles;
 - The settlement for a group of six piles were not calculated.
- For basic design loading schemes refer to Appendices B and C for vertical and horizontal loads. For final design some changes have been made in the layout of the building (as a result of updated request from *the Client*) and therefore updated loading plans and plans used were specified separately in “Appendix F – Final design”. The changes include an increase of loads because of the area for staircases and elevator shafts and altered length of two stiffening walls.

2.4. Foundation design variants

2.4.1. Option 1 – Raft foundation

Raft foundation (also sometimes referred to as “mat foundation”) is a large embedded reinforced concrete slab supporting several columns in two or more rows. The bearing capacity of the foundation is increased by combining all individual footings into one raft – since the bearing capacity is proportional to the width and depth of the foundations. It also spreads out the loads, applied to foundation and helps level out the differential settlements (Barnes, 2010).



Typically, for these type of foundations, down-standing concrete beams with bar reinforcement are designed beneath the structural walls or line of columns, in order to provide sufficient stiffness to the foundation and increase its resistance to bending deflections (Barnes, 2010).

Figure 2-2 Raft foundation with down-standing beams (Builder's Engineer, 2017)

The variant is a quite straightforward solution, without causing a lot of construction risk, as the implementation activities do not include working at high elevations or digging to great depths, there is more margin for installation inaccuracies than, for example, for pile foundations (Barnes, 2010).

However, this solution requires prolonged implementation time, when compared to others. That is because the construction of the foundation can start only when all the contaminated soil in the area of the new building is fully replaced with clean soil and compacted, assuring the necessary load bearing capacity (Barnes, 2010). In addition, the entire area cannot be poured at once, as construction joints must be created, dividing the complete area in smaller compartments. Obviously, in this way, there is very little room for making the process quicker, because both groundwork and foundation construction are subsequent events to each other.

A preliminary estimate for the duration of the implementation can be based on the capacity of the concrete pouring machinery. As an example, a chosen concrete mixing transport truck (Figure 2-3, right image) has a volume of 15m³, a value which would be compared to the total volume of concrete (Stetter, 2017).

The unit price for the raft foundation, including the materials and well as the service cost, is estimated as 250 euros per cubic meter of concrete, as an example from similar projects.



Figure 2-3 Raft foundation implementation (left) (Zenith, 2017); concrete mixing transport truck (right) (Stetter, 2017)

2.4.2. Option 2 – Pile groups

The second option is the conventional method of pile groups' foundation, where several piles are connected via a pile cap for load distribution (Barnes, 2010). The load from the column is divided between the groups of piles, which then transfer the load deep in the ground.

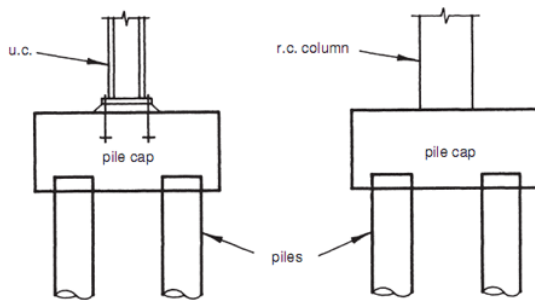


Figure 2-5 Pile cap (Builder's Engineer, 2017)

Standard axial pile diameters for larger structures vary from 900 to 1200 mm (Tomlinson & Woodward, 2015). Due to the requirements of foundation installation without extensive sound or vibrations, bored concrete piles were chosen, also as a common practice for similar projects in Germany.

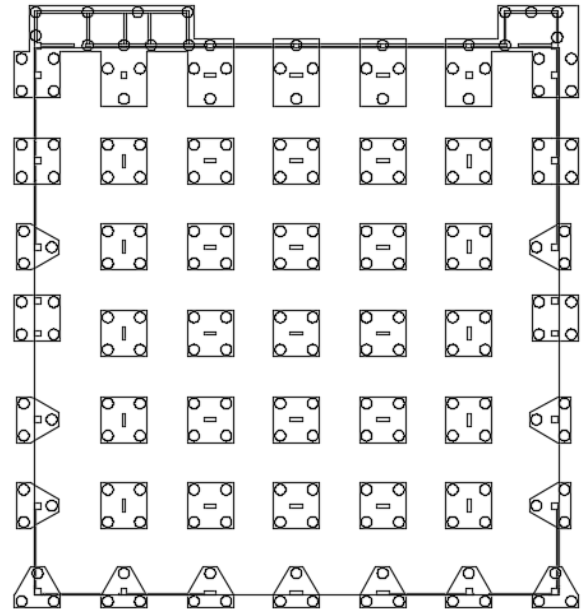


Figure 2-4 Preliminary estimation of the pile group foundation

If the designed diameter does not exceed the 1200mm, a suitable solution for reducing the loosening of surrounding soil and making the installation quicker is to use the continuous flight auger “CFA” method. The soil conditions in the area favour the augered bored piles as well, as the stiff clay provides the necessary stability when boring the holes and pouring the concrete, allowing the absence of the removable steel tube, which is a very important advantage for reducing the foundation costs. However it is important to mention that for the cast-in-situ concrete piles installed without the permanent supporting tubes, the concrete cover should not be less than 75mm (DIN EN 1536:2015-10).

An approximate duration for installing a single pile varies from 20 to 60 minutes, depending on its diameter and length (Zeman, 2014). In soft wet soils, this may be as little as 25 percent of the time it would take to complete an equivalent shaft using conventional drilling methods with casing and slurry (Zeman, 2014). The advantage is therefore faster production and reduced labour, equipment, and fuel costs. An estimated unit cost for a meter of pile, taken as a reference from similar projects, is 260 euros, including materials and installation.

The CFA piles are formed by drilling to the required depth using a hollow stem continuous flight auger. After reaching the designed depth, a high slump concrete is then pumped through the hollow stem. While the concrete is being pumped, the auger is withdrawn at a controlled rate, removing the soil and forming a shaft of fluid concrete extending to ground level. A reinforcing cage is then inserted into the fluid concrete. In addition, the auger operates

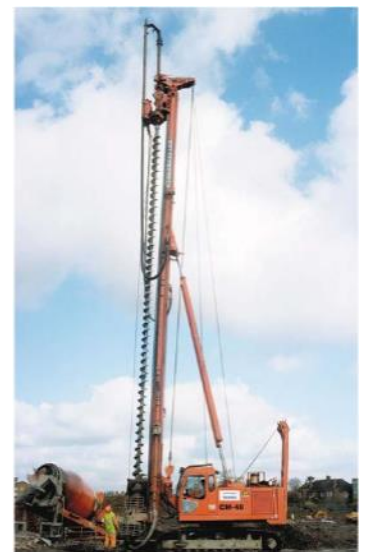


Figure 2-6 Continuous auger (Piling Contractors, 2017)

without causing excessive sound and vibrations. (Tomlinson & Woodward, 2015). The downside of this solution is similar of that for the raft foundation, because, even though the continuous flight auger system provides quick installation, the foundation construction requires the complete area of contaminated soil to be excavated and exchanged.



Figure 2-7 Hollow stem in the continuous auger for pouring the concrete (left); pile installation equipment on site (right) (Foundation Equipment, 2017)

Machinery needed:

1. Continuous auger
2. Concrete mixer
3. Pump (pumping up the concrete to the end of the auger)
4. Excavator for removing excess soil

2.4.3. Option 3 – Mono-piles

The third foundation variant is a foundation of large diameter mono-piles (Barnes, 2010). This variant was introduced as a result of the soil contamination issue.

A single mono-pile is large enough to support the entire load of the column and it is placed directly underneath it. Having one large pile instead of groups of several smaller ones results in larger distances between the adjacent piles. This situation proposes the idea for soil excavation around the already installed piles (Figure 2-8).

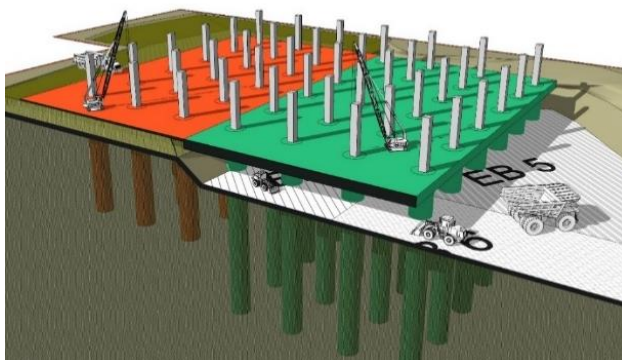


Figure 2-8 parallel construction and excavation (Celli, 2017)

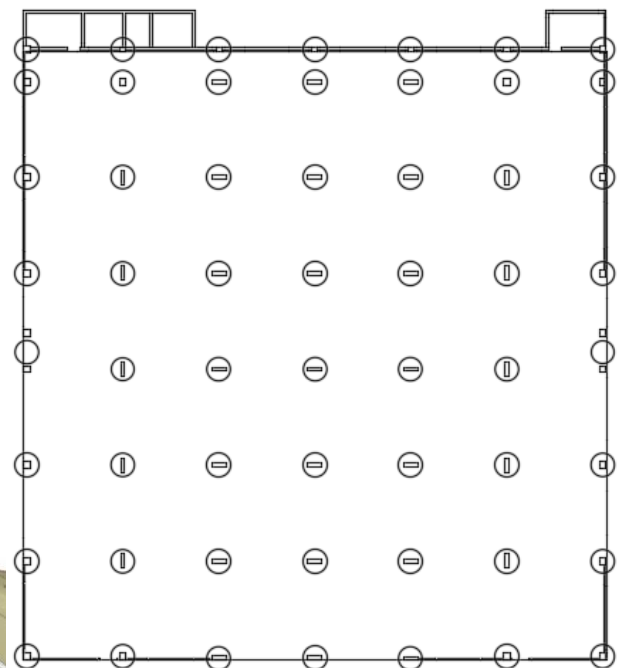


Figure 2-9 Preliminary design of mono-piles foundation

The advantage is that once one half of the piles is installed, a ground slab would be constructed and thus the excavation of soil can be performed around them and in addition to implementing the rest of the foundation, and the erection of the building superstructure may

be done in parallel as well. Therefore the duration of implementation would last as much as the longest activity out of the three: soil exchange, boring the second half of the piles or constructing the building superstructure.

Disadvantage: the borderline between the different excavation areas would be kept a slope instead of a sheet pile and digging works would be implemented around the already installed monopiles in order to shorten the duration of the implementation. This solution raises high risks regarding the stability of the structure - the top five meters of the pile are not supported during the temporary excavation works - pile acts as a cantilevered structure and thus is subject to eccentric lateral loads from the wind which would cause instability due to the created moment. If the piles are relatively short (length/depth ratio < 10), then failure will be governed by rotation of the pile as a rigid body. If the piles are long (l/d ratio > 10) then lateral resistance will be governed by a plastic hinge developing in the pile at a certain depth, called “the point of fixity” (Figure 2-10) (Tomlinson & Woodward, 2015).

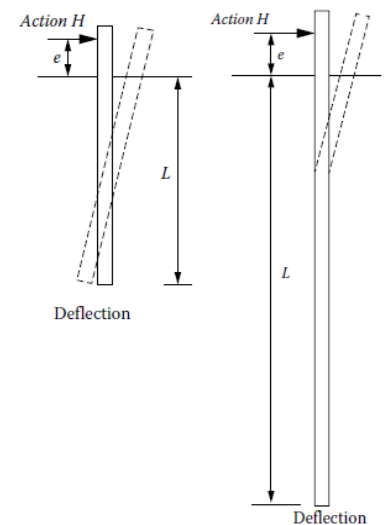


Figure 2-10 The indication of failure mechanisms for short and long piles under lateral loading (Tomlinson & Woodward, 2015)

In case of increased lateral loads and the deformation/collapse of the structure, it would become dangerous for the people working on the construction site. In order to avoid these consequences, the piles would need to be temporarily braced or in any other way supported, or have a steel casing, reducing the likelihood for bending, in order to assure the safety during the excavation.

In addition, the larger than usual diameter of piles is a downside when considering the installation, as large augers are necessary and therefore this option is not preferred by the contractor companies, in addition to the high cost: the unit price, taken as an example from a similar project, suggests 750 to 780 euros per meter of pile, including both the materials and the service cost.

The pile installation is described as conventional bored pile method or drilled raft method. These piles can be drilled in very large diameters and provided with enlarged or grout-injected bases, if additional support is necessary to withstand high applied loads. The installation characteristics closely depend on the speed of the auger, which commonly can drill 6 metres in one hour for diameters up to 2.2-2.4 m and then enlarge them up to 4 m. A reamer blade (Figure 2-11) cuts a larger diameter than the previous pass and the cuttings fall into an open topped bucket for removal.



Figure 2-11 Example of a reamer blade, set up for increasing the pile shaft diameter (Piling Contractors, 2017)

The second pass is implemented a bit faster, usually completing 7 metres in an hour (Piling Contractors, 2017). If reinforcement is required, a light cage is then placed in the hole, followed by the concrete. In loose or water-bearing soils and in broken rocks, casing is needed to support the sides of the borehole, this casing being withdrawn during or after placing the concrete. In stiff to hard clays and in weak rocks, an enlarged base can be formed to increase the end-bearing resistance of the piles (Tomlinson & Woodward, 2015).

Because of the large diameter and great depth, it is not possible to apply the CFA method as for the pile groups. Thus, the duration for implementation becomes at least

three or four time longer, as suggested by practice (Zeman, 2014) and the price increases as well. An estimated unit cost for meter of pile, including materials and installation varies from 750 to 780 euros.

Machinery needed:

1. Truck-mounted rotary auger drill (Figure 2-13)
2. Hopper/tremie + pipe (to direct the concrete)



Figure 2-13 Watson 2100 truck-mounted auger drill (Tomlinson & Woodward, 2015)



Figure 2-12 installation of large diameter piles (Piling Contractors, 2017)

2.5. Comparison of the location alternatives

The issue of contaminated soil within the construction site was earlier addressed by introducing the third foundation variant of large-diameter mono-piles and a possibility to carry out excavation around them, reducing the duration of the project. However, another possibility for solving the problem was to alter the initial Master Plan and change the location of the building.

In this paragraph the two location alternatives are compared depending on the amount of soil, which is required to be excavated and back-filled.

2.5.1. Excavation quantities and machinery capacities

In order to evaluate the volume of soil to exchange as a quantity of time, a simple comparison of the sequence of construction phases was set up for both locations. The possible capacities suggested an approximate duration the project would require, depending on how much soil needs to be dug.

Table 2-2 represents the quantities of soil to excavate per six excavation areas. Depending on the location of the building, the amount is different and thus can be compared.

Table 2-2 The indication of the size and soil volume for all excavation areas EB1 to EB6

Excavation areas and soil quantities			
EB1	1785 m ²	5 m	8925 m ³
EB2	1648 m ²	5 m	8240 m ³
Total EB1 + EB2	3433 m²		17165 m³
EB3	387 m ²	6 m	2322 m ³
EB4	1060 m ²	6 m	6360 m ³
EB5	1373 m ²	6,5 m	8924,5 m ³
EB6	957 m ²	6,5 m	6221 m ³
Total all excavation areas	7210 m²		40992 m³



Figure 2-14 Examples of different sizes of excavators - small, middle and large (XCMG, 2017)

Due to a confined space, because of carrying out of the construction activities in the city area, the accessibility to the building site is rather difficult and therefore, it was decided to limit the choice of machinery to one large excavator (for digging the soil) and several trucks (for removing the polluted soil (Figure 2-15).



Figure 2-15 Soil being loaded onto a truck (Shutterstock, 2017)

In order to meet the duration requirements, set by the Client, the large excavator (Figure 2-14) was considered for calculating the daily productivity (Table 2-3 and Table 2-4). However, should the duration exceed the limit by more than a week or two, an additional excavator should be added.

Table 2-3 Determining the excavation cycle for calculating the daily capacity of each excavator

single excavation cycle	30 s	(value based on experience)	
cycles per hour	100 -	(considering 10min break per working hour)	
cycles per day	800 -		
efficiency	0,75 -	(considering spillage, difference in soil)	
		(value based on experience)	

Table 2-4 Daily capacity of excavators of different sizes (XCMG, 2017)

capacity small excavator	180 m3/day
capacity medium excavator	720 m3/day
capacity large excavator	1200 m3/day

2.5.2. Initial situation

Initially, the time planning for the construction was strictly divided into two parts, as shown in the previously mentioned Figure 1-11:

		2017					2018								2019														
		8	9	10	11	12	1	2	3	4	5	6	7	8	9	10	11	12	1	2	3	4	5	6	7	8			
No Adjustments		44 weeks																										Soil Remediation activities	
																												Construction data centre	

The approach was to dismantle the old basement (indicated in blue - Figure 2-16, left image), excavate the entire area of 7211m² (Figure 2-16, right) to the required depth, back-fill it with clean soil, which is then compacted and prepared for the construction of the foundation and the rest of the building. The complete site preparation was planned to take 11 months and together with the 12 months of construction of the building, the data centre was to be opened in May, 2019 the latest.

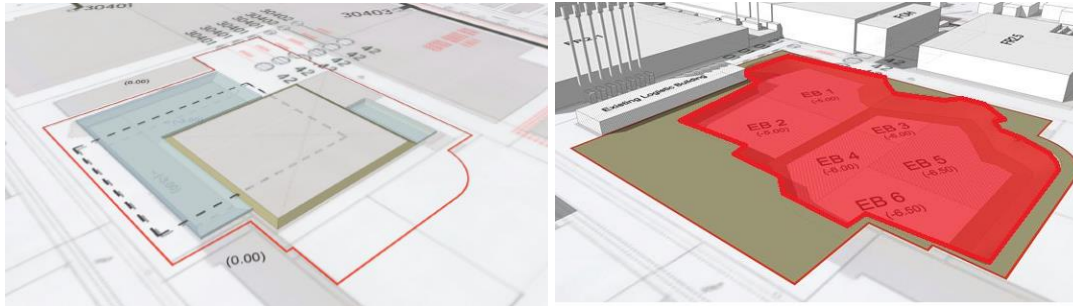


Figure 2-16 Indication of the old basement to be dismantled in blue (left); Excavation area in red (right) (Celli, 2017)

2.5.3. Parallel implementation Location A

Location A has been the initial choice from the beginning of the project. The new building was planned to be built here, in accordance to the earlier designed Master Plan, as it was already submitted for tendering process. The latter discovery of the influence of soil remediation activities lead to reconsideration of the building location. The disadvantage of this location is the amount of polluted soil, as the entire building area requires soil exchange. In terms of the previously mentioned excavation areas, the location requires that all six of them are remediated and prepared prior the construction of the building. Luckily, a part of that area may be implemented in parallel, by isolating the building pit with sheet piles (Figure 2-17).

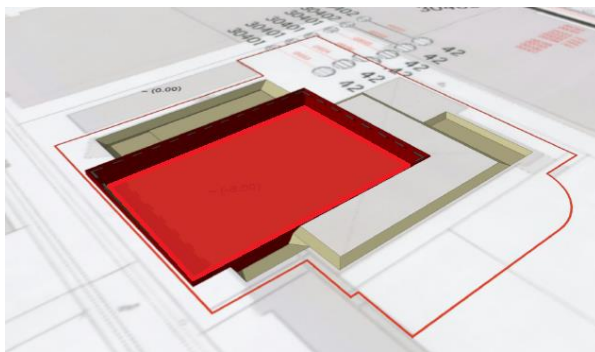


Figure 2-17 Excavation area Stage 1

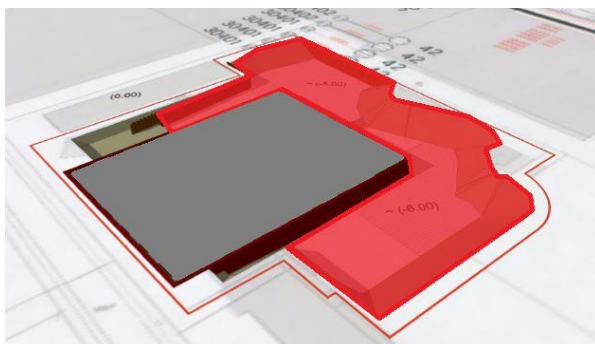


Figure 2-18 Excavation area Stage 2

In order to optimise the implementation, the total area would be excavated in two stages: firstly, the area directly under the new building, surrounding the building pit with sheet piles (Stage 1), and secondly, while the foundation is already being built, the rest of the soil (Stage 2). Approximate soil volume for Stage 1 excavation:

$$89\text{m} \times 61\text{m} = 5429\text{m}^2 \times 6\text{m} = 32\,574\text{ m}^3$$

The remaining soil volume to excavate is the amount for Stage 1 subtracted from the total:

$$40,992\text{ m}^3 - 32,574\text{ m}^3 = 8,418\text{ m}^3$$

Required time for Stage 1, if the large excavator is chosen (Figure 2-14):

$$32,574 / 1,200 = 28\text{ days}$$

Required time for Stage 2:

$$8,418 / 1,200 = 7\text{ days}$$

What is an important advantage, is that the second stage of the soil remediation activities can be implemented in parallel with the starting construction of the building foundation.

2.5.4. Implementation alternative Location B



Figure 2-19 Changing the location of the data centre (Celli, 2017)

Location B is an alternative to the initial location, by moving the building further to the North-West, taking the place of the existing logistics facility (Figure 2-19). By choosing the second location, the demolition of the old building would be required, however *the Client* is planning to expand the logistics building anyway, and therefore designing it in a different place is a possible choice. When considering the second location, the amount of contaminated soil is much less as well, which is a very important advantage, making this alternative worth further consideration.

The required duration is clearly less than for the first location A, as the volume of soil is much smaller. However, the demolition of the existing logistics facility is another factor, which should be taken into consideration when describing the second location.

Because of the fact, that once the sheet pile wall is set up along the boundary, the relocation of the building allows the remaining areas EB3 to EB6 to be finished independently is a very important advantage. It helps reduce the required time for the implementation of the new data centre.

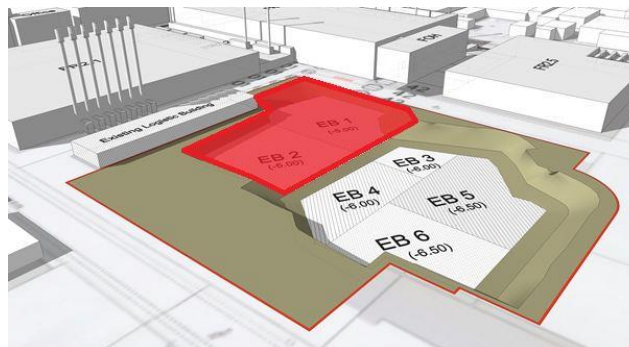


Figure 2-20 The amount of soil to exchange for location B is the volume from the two excavation areas EB1 and EB2

EB1	8925	m3
EB2	8240	m3
Total EB1 + EB2	17165	m3

The required minimum time to excavate this amount of soil, when considering a large excavator, is therefore:

$$17,165 / 1,200 = 15 \text{ days}$$

The required duration is clearly less than for the first location, as the volume of soil is much smaller. However, the demolition of the existing logistics facility is another factor, which should be taken into consideration when describing the second location.

2.5.5. Demolition works

During the course of demolition, the stability of the building under demolition and any remaining parts of it shall be maintained at all times. In high water table areas, assessment shall be made to ensure that the remaining structure will have adequate factor of safety against uplift upon demolition at all stages. If necessary, the uplift pressure acting on the basement structure shall be relieved before demolishing the structure (Buildings Department, 2004).

If a dewatering system is required, the effect of the dewatering on adjacent buildings, structures, land, street and services must be considered in the design. It is also important that the disposal of the ground water shall not affect the quality of the surrounding water resource and/or cause localised flooding.

In order to dismantle a structure, the following machinery is required:

- One or a few multi-processor Caterpillars or Brokk machines with changeable tools
- Excavator, bulldozer and dump trucks (for sorting, collecting and removing debris)

As a rule, hand-held tools are only used when alternative methods have proved themselves unsuitable, or when there is insufficient space on site for rig-mounted equipment or when the coverage area of the latter is inadequate. This is due to high costs (due to labour costs involved) as well as health risk for the people, operating these tools throughout the day.

Nowadays rig-mounted equipment is available that is specially designed, compact and easy to operate and ideal for use in cramped narrow spaces (Figure 2-21) (Brokk, 2000).



Figure 2-21 Rig-mounted crusher (Brokk, 2000)

The use of rig-mounted crushers for demolition has become an extremely interesting alternative when requirements stipulate low noise levels whilst demolition work is being carried out. The crushers grip the demolition material and crush it and the only sound to be heard is that of the material being crushed and of rubble falling. Compared with using a rig-mounted breaker this method often makes it possible to have much longer working shifts per day when conditions and restrictions limit the length of time a breaker can be used.

The crusher also successfully separates materials on site, allowing for easier and faster sorting for recycling process (Brokk, 2000). What is useful, is that the same machine can also be equipped with buckets or grapples (Figure 2-22).

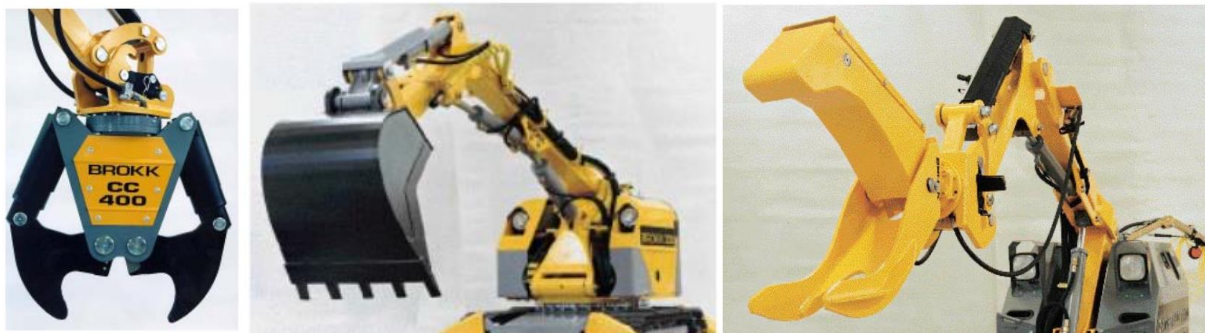


Figure 2-22 Changeable dismantling equipment of a rig-mounted machine. From the left: crusher, buckets and grapples (Brokk, 2000)

The machines are transported either on trailer, by lorry or using a hired vehicle (e.g. a carrier). The machines are loaded onto these by means of ramps and are simply driven on. Other alternatives are to lift the machines using a truck or something similar (Brokk, 2000).

The logistics building demolition is rather simple as it is a single storey building, as well as the basement (also one level only) – no cranes or platforms specifically for the machinery are necessary, estimated duration couple of weeks to a month, unit cost for a Brokk crusher is around 80 euros per working hour (Brokk, 2000).

2.5.6. Construction stages and building site logistics

When comparing the accessibility of the both construction sites, some differences are noticeable. Building location A (Figure 2-23, left image) is positioned more in the middle of the entire

construction site, which makes it more difficult to manoeuvre, takes longer time from the entrance to the construction site until the exit, especially for the dumper trucks, which have to come and go many times per day for removing the contaminated soil off the site. Location B (Figure 2-23, right image), on the other hand, is more to the side of the entire construction area, which makes it closer to the main traffic. This reason is also important for carrying out demolition works, as it is much easier and quicker to remove materials and waste. For location B, two separate roads for entering and exiting the site are planned, which is very useful when more than one vehicle is moving at once, as it eliminates the risk of traffic jams and reduces delays in production.

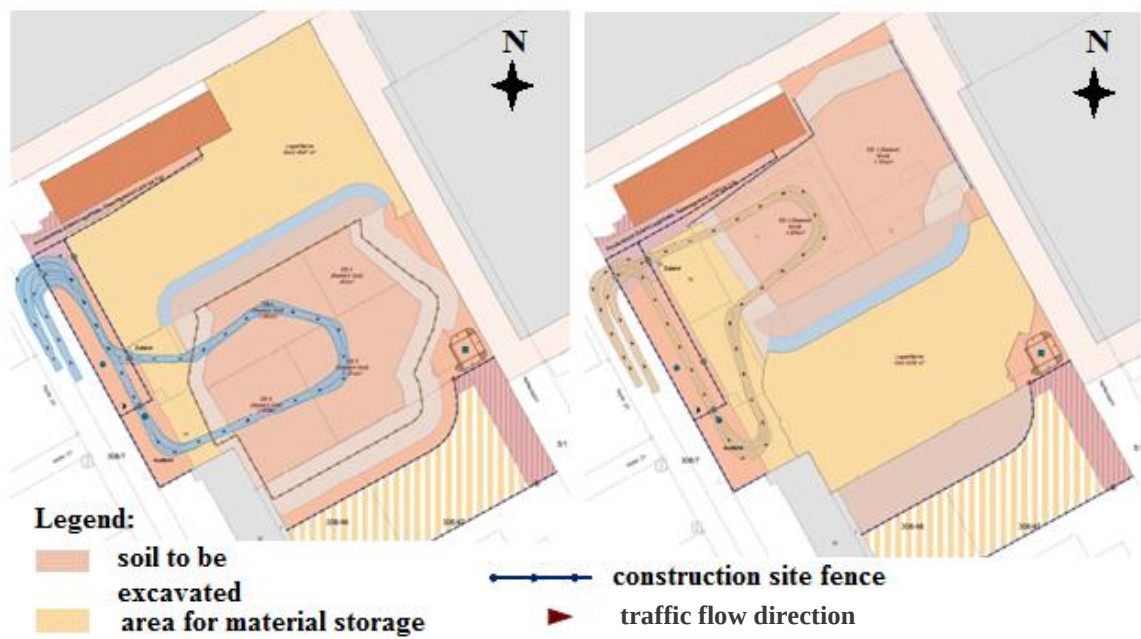
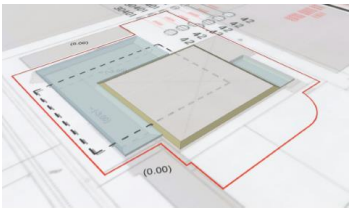
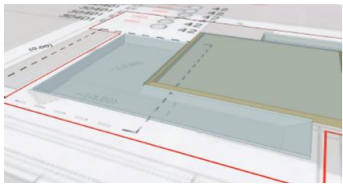
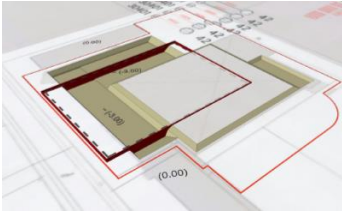
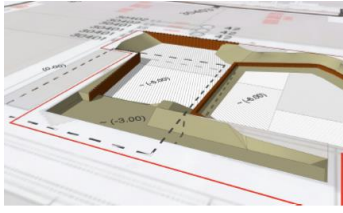
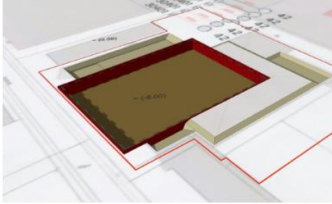
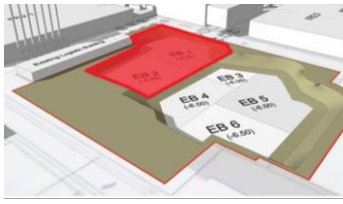
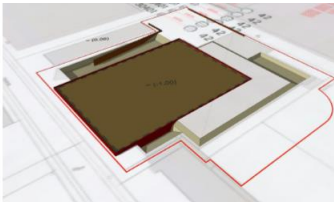
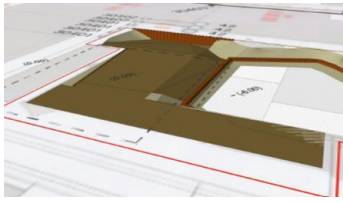
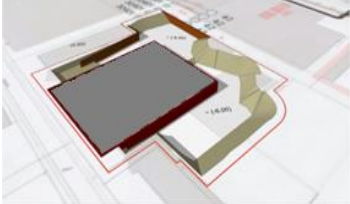
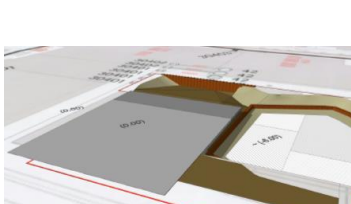


Figure 2-23 The accessibility for the construction machinery location A (left image) and location B (right image) (Celli, 2017)

The preliminary planning was created for both location alternatives, considering a single excavator, in order to see how the size of the area and the volume of soil directly influences the complete duration of the construction activities.

Table 2-5 Construction phases for the two locations and the comparison of the preliminary duration estimations

Construction phase	Location A		Location B	
Phase 1: Fencing the site, setting up of the containers	1 week		1 week	
Phase 2: Basement dismantling old building		2 weeks		2 weeks

Phase 3: Site clean-up and installing of sheet piles		2 weeks		2 weeks
Phase 4: Soil excavation		4 weeks		2 weeks
Phase 5: Backfill		3 weeks		1 week
Phase 6: Soil compaction	4 weeks		2 weeks	
Phase 7: Site clean-up, constr. prep.	4 weeks		2 weeks	
Phase 8: “Stage 2” for location A ⁴	6 weeks (excavation, backfill and compaction – 2 weeks each)		-	
Buffer for delay	2 weeks		1 week	
Construction of foundation (duration considered separately)				
Total preliminary duration	~28 weeks		~13 weeks	

The duration of Location A shows an obvious difference from that of Location B. The entire implementation process was however expected to be slower not only because of the larger amount of polluted soil to exchange, but also because of the reduced accessibility to the construction site. Location B could be implemented even faster, however one week of delay is considered. If

⁴ A separate phase added for the Stage 2 excavation in Location A. In theory, the excavation may be implemented at the same time as building of the foundation, however, due to limited area, there would not be enough free space for movement of the machinery or storing of the materials, therefore the additional time is considered in the comparison.

necessary, the machinery can be doubled to quicken the excavation activities, as the situation with logistics is much better for the second location.

In short, the anticipated duration was put in the same graph as the initial situation (prior any changes in the master plan or implementation activities):

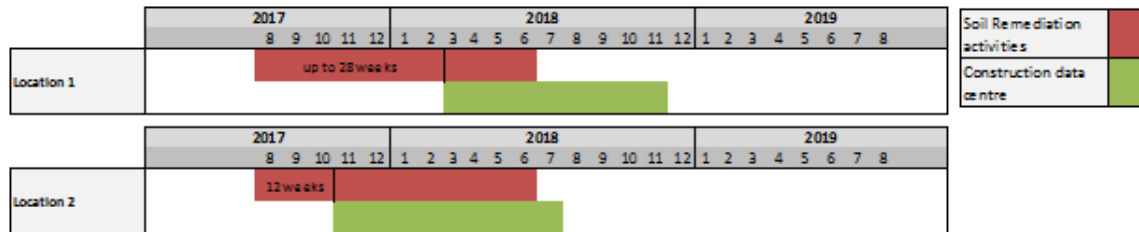


Figure 2-24 The visual representation of the preliminary estimation of construction duration for both locations

As can be seen in Figure 2-24, the second location alternative is possible to carry out within the required time, even with the demolition of the logistics facility, which may be implemented in parallel to the demolition of the old basement. The comparison suggests that if the construction does not take longer than 9 months to finish, out of which the first two 2 months is the foundation installation, the project duration is within the allowable limits.

For location A, the amount of machinery might be increased (in order to decrease the construction time), however considering the higher costs and a difficult accessibility, it is not advised to do so.

2.6. Application of the soil parameters

This sub-chapter summarises the soil parameters, which were used for completing the preliminary designs of the three foundations as well as the final foundation design. Note that there was no information provided about the soil parameters after the remediation activities, therefore the designs are based on the initial situation.

Due to the possibility for the relocation of the building and, in that case, the reduced area of the soil exchange, preparing the designs based on the present values, is a safer approach.

2.6.1. Soil stiffness for the shallow foundation design

In case the raft foundation is designed in the altered location, only a part of the area contains polluted soil. Because of this reason, some of the soil will be replaced and some will not. This issue was addressed by the geotechnical engineering company, in order to allow preparing the preliminary designs, and a set of estimated values were provided for further calculations. Figure 2-25 shows the strategy which was followed for completing the basic design of the raft foundation, including the areas of full soil exchange (this area has a higher anticipated soil stiffness) and the areas of only 1 meter of soil exchange (which includes the removal of artificial fillers).

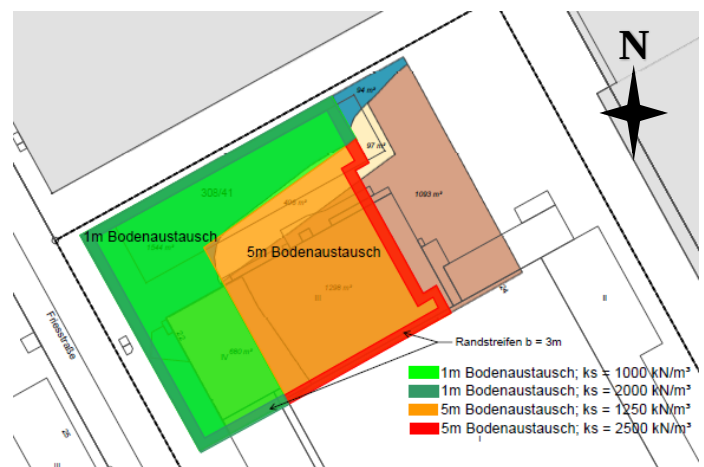


Figure 2-25 Soil exchange areas location B (considered as a safer approach) and the indication of the soil stiffness. 'Bodenaustausch' - soil exchange in English (Huber, Toker, & Glaser, 2017)

The soil parameters are therefore the following:

For areas with 1 m or less of soil exchange the soil stiffness: $k_{s,1} = 1.0 \text{ MN/m}^3$

For areas with 5 m of soil exchange anticipated soil stiffness: $k_{s,5} = 1.25 \text{ MN/m}^3$

On the edge strip of $b = 3 \text{ m}$ the stiffness module shall be doubled (value considered for checking the settlements).

2.6.2. Soil parameters for pile foundation design.

In order to dimension the bored pile foundation, the following values for base resistance and shaft friction were used (Table 2-6)

Table 2-6 Base and shaft resistance values, used for pile design (Huber, Toker, & Glaser, 2017)

Soil layer	Layer position	Pile tip resistance $q_{b,k}$ [MN/m ²]	Pile skin friction $q_{s,k}$ [MN/m ²]
1. Silt	0-2 m	-	0,025
2. Sand	2-5 m	-	0,080
Adhesion [MN/m²]			
3. Clay	>5 m	1,2	0,060

The indicated pile tip resistance refers to the settlement value of $s/D=0.01$, wherein “s” is the occurring settlement, depending on the pile diameter “D” (Schneider, 2010). This value is set as a limiting amount of settlement for the serviceability limit state of pile design.

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3. Method

3.1. Research methodology

The Table 3-1 below describes the activities taken for answering the before set-up research questions and the products, planned to acquire, in order to find the optimal solution for the research problem.

Table 3-1 Research methodology

Research question	Activity	Product
1. What are the most important issues, regarding the soil type and its contamination, as well as the groundwater level and the surroundings of the construction site?	Obtaining the geotechnical report from the external company, analysing the soil conditions and sorting out the information, useful for the report. <i>Reference: geotechnical report (Huber, Toker, & Glaser, 2017)</i>	Chapter with the summary of soil parameters, information on the soil contamination
2. What are the boundary conditions, functional and technical requirements of the project?	Communication with the Client through the in-company supervisor (senior engineer of the project), understanding the important requirements. Analysing the national standards. <i>Reference: meetings and other means of communication, set of national regulations (Schneider, 2010)</i>	Schedule of requirements and thesis scope
3. What are the permanent and temporary loads, acting on the structural elements of the building and the foundation?	Acquiring information about the area loads within different floors, calculating total vertical loads per column and distributing the horizontal loads between the stiffening walls. <i>Reference: Structural design regulations (Schneider, 2010)</i>	Detailed calculation of loads on the foundation (Appendix B – Calculation of vertical loads)
4. What are the preliminary direct costs for each foundation variant, including materials and installation?	Preparing preliminary designs for different foundation variants and estimating direct costs, based on the expenses for similar projects within the company. <i>Reference: meetings with the in-company supervisor for discussion about similar projects</i>	Cost estimations for foundation variants
5. How long does it take to implement each of the foundation techniques and can the process be quickened?	Carrying out research about different construction equipment and machinery, necessary for the dismantling and construction of buildings, ground excavation, etc. estimating approximate duration of implementation activities, taking into account some buffer for possible delays.	Preliminary implementation planning and estimation of construction duration

	<i>Reference: literature and internet research, handbooks of machinery and equipment (Piling Contractors, 2017), (Brokk, 2000), (Tomlinson & Woodward, 2015), (Zeman, 2014) and others.</i>	
6. Can the total implementation time be shortened, if the location of the new building is changed and by how much?	Analysing the difference between the two possible locations in terms of amount of soil to be exchanged, the construction site logistics, accessibility, etc. Considering and comparing with the influence of the foundation techniques if it is worth changing the location and altering the initial Master Plan. <i>Reference: discussions with the in-company supervisor, applying the research about construction equipment and its capacity.</i>	Comparison of the two locations A and B
7. What influence do the demolition activities have on the overall implementation of the project?	Researching about demolition of buildings and the necessary equipment, in order to estimate approximate expenses and duration. <i>References: (Brokk, 2000), (Buildings Department, 2004)</i>	Estimation of the demolition duration and involvement of it in the preliminary planning
8. What is the influence on the construction site logistics for the movement of the machinery, considering both building locations?	Analysing the set-up of the construction site, the distance from the main roads, accessibility by construction machinery, available space for storing the materials, etc. <i>Reference: Site logistics plans, meetings with the in-company supervisor</i>	Comparison of the two locations A and B for their accessibility
9. What is the optimal design and the necessary material properties, regarding the durability of the final structure?	Analysing the national construction standards and structural design calculations of similar projects, in order to obtain the necessary information for the final design, once the selection of the foundation technique is completed. <i>Reference: A set of construction regulations (Schneider, 2010)</i>	Set of technical requirements for the foundation design, final design calculations and drawings

The sequence of activities for obtaining the project results are based on the selective method. In the beginning, the six variants are created, based on the information in the theoretical framework, including all possible foundation variants as well as considering two different locations. After preparing the preliminary designs or based on common understanding, two of those solutions were ruled out. The remaining solutions were compared and evaluated, based on the two criteria – duration and cost, finally coming to the final decision.

The below graph presents the three steps followed:

Step 1

- Firstly, the three designs and two locations, introduced in the previous chapter, were cross-referenced and in total six possible outcomes were considered:
- Location A Option 1 (A1)
- Location A Option 2 (A2)
- Location A Option 3 (A3)
- Location B Option 1 (B1)
- Location B Option 2 (B2)
- Location B Option 3 (B3)

Step 2

- The second step was to prepare preliminary designs for the three foundation techniques and check if all of them are structurally viable for this project, according to the obtained information about the soil conditions, surrounding area and following the national construction regulations.

Step 3

- Lastly, when the variants were sorted, the remaining few were compared for preliminary implementation duration and costs. The selected variant was therefore the solution to the research problem and the final design of the project.

3.2. Methodology flow-chart

The sequence of activities, performed in order to find the most optimal solution for the research question can be best summarized with the following flow-chart:

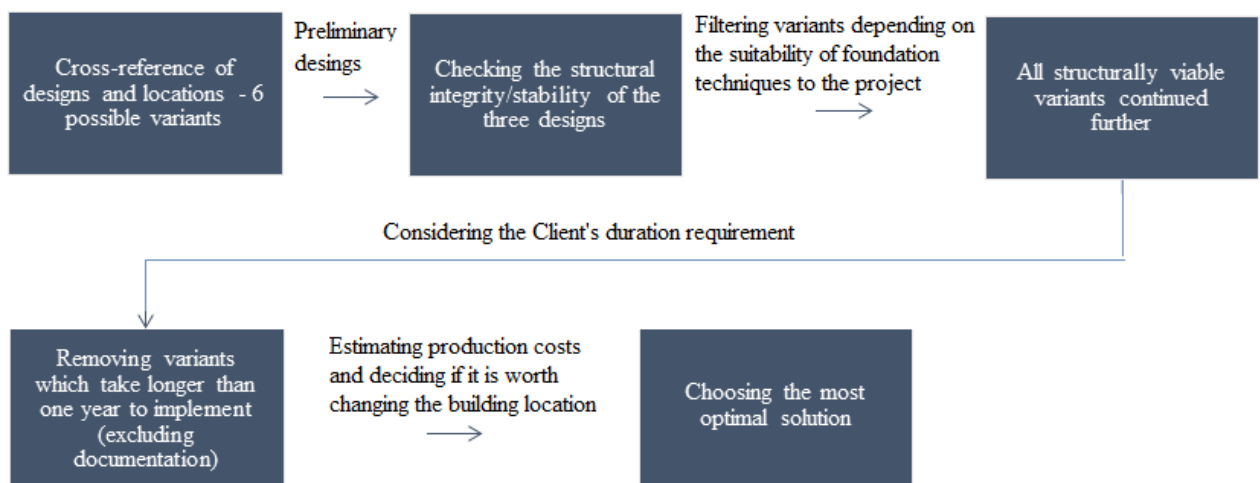


Figure 3-1 Research strategy or choosing the most optimal solution to the research problem

The purpose of creating preliminary designs for each of the foundation variants is to determine the approximate dimensions, estimate direct material costs and understand their implementation. After obtaining the mentioned detail about each alternative, it is then easier to compare them and select the variant which is the most suitable for the desired situation of the project.

3.3.Preliminary design – raft foundation

The preliminary design of the raft foundation included checking the minimum required depth of the foundation as well as the stresses in the foundation as well as occurring settlements. After obtaining these specifications, the structural integrity of the design was judged.

For estimating the amount of expected settlements and material stresses, *Sofistik* software was used. The software is widely used in Germany for finite element design and it has been proven by previous projects to be very useful for estimating the behaviour of foundations and the soil-structure interaction. Sofistik is a useful tool to use for the analysis of the raft foundation due to an increased level of detail, inclusion of the horizontal loads as well as different values for soil stiffness, earlier presented in the theoretical framework.

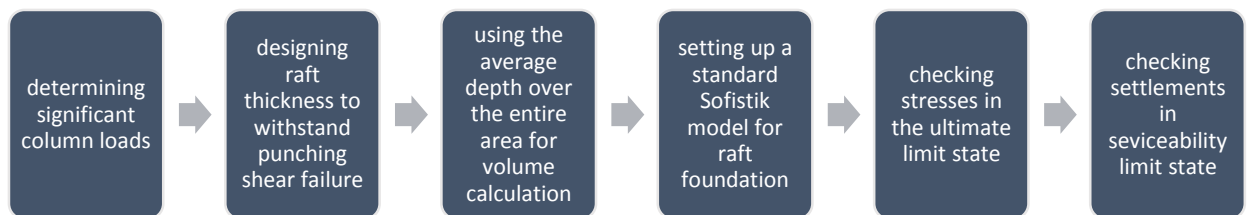
3.3.1. Loads on foundation

The following Table 3-2 shows the vertical loads, considered for the raft foundation design. The calculation for obtaining these loads can be seen in “Appendix B – Calculation of vertical loads”.

Table 3-2 Loads on raft foundation - Sofistik input

Load summary - SOFISTIK input							
* these loads do not include the self-weight of the shear walls as they are modeled separately as line loads							
Type	Position	G, k	Q, k	γ_g	γ_q	G, d	Q, d
		kN	kN	-	-	kN	kN
Type 1	C1, C7	683	358	1,35	1,5	921	537
Type 2	C2, C6	946	578	1,35	1,5	1277	867
Type 3	C3 - C5	973	578	1,35	1,5	1314	867
Type 4	C8, C14	2 169	1 433	1,35	1,5	2928	2150
Type 5	C9, C13	3 975	3 358	1,35	1,5	5366	5037
Type 6	C10 - C12	3 787	2 805	1,35	1,5	5112	4207
Type 7	C15, C21	3 020	2 150	1,35	1,5	4077	3224
Type 8	C16, C20, C23, C27, C32, C36, C39, C43	5 686	5 037	1,35	1,5	7676	7555
Type 9	C17-C19, C24-C26, C33-C35, C40-C42, C47-C49	5 379	4 207	1,35	1,5	7262	6311
Type 10	C22, C28	3 131	2 150	1,35	1,5	4227	3224
Type 11	C29, C30	685	139	1,35	1,5	925	208
Type 12	C31, C37	3 054	2 150	1,35	1,5	4123	3224
Type 13	C38, C44	3 147	2 150	1,35	1,5	4248	3224
Type 14	C45, C51	3 681	2 150	1,35	1,5	4969	3224
Type 15	C46, C50	5 782	5 037	1,35	1,5	7806	7555
Type 16	C52, C58	2 177	1 075	1,35	1,5	2938	1612
Type 17	C53, C57	3 212	2 150	1,35	1,5	4336	3224
Type 18	C54 - C56	3 096	2 150	1,35	1,5	4179	3224

3.3.2. Structural calculations



3.3.2.1. Important notes

For the hand calculation it has been assumed that the foundation is of rectangular shape, without including the areas for the staircases and elevator shafts. More realistic analysis was carried out with Sofistik software for checking the pressure and settlements with a modelled design.

The wind forces from the stiffening walls are transferred into the foundation as "push-pull" effect. The tension component was considered as insignificant due to the high vertical loads from the building onto the foundation. However, to be on the safe side, equal compressive forces were considered on both ends of each wall, taking into account the possibility for wind forces from either of the two directions.

Foundation design (application of the soil conditions to the foundation design and the installation):

- DIN EN 1997-1 (Eurocode 7 for geotechnical design)
- DIN 1054: 2010 (national regulations) (Schneider, 2010)

3.3.2.2. Depth of foundation – punching shear check

For the design of the raft foundation, in order to define required depth of the raft, three differently positioned columns were checked against punching shear failure. An average depth without considering punching shear reinforcement was defined and used for the calculation of the total concrete volume.

Punching shear is an effect of the concentrated load on the slab causing shearing stresses on a section around that load. The critical surface for checking punching shear is a perimeter, located at $2.0d$ from the loaded area, wherein “ d ” is the effective depth of the slab (Mosley, Bungey, & Hulse, 2007). For example, the effective area where punching shear is supposed to be checked for one of the columns can be calculated as follows:

$$u = 2 * 0.74m + 2 * 0.64m = 2.76m$$

$$A (\text{punch. shear}) = u * d = 2.76m * 1.5m = 4.14m^2$$

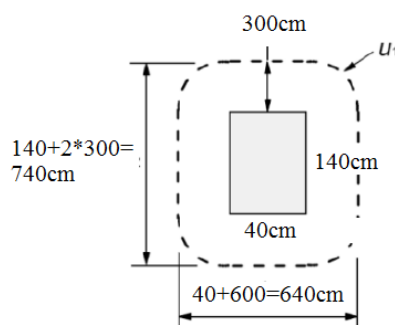


Figure 3-2 Indication of the punching shear perimeter calculation (Mosley, Bungey, & Hulse, 2007)

Ensuring that maximum punching shear stress is not exceeded, i.e. $v_{Ed} < v_{Rd,max}$ at the column perimeter, wherein v_{Ed} is applied shear stress and $v_{Rd,max}$ - design value of the maximum punching shear resistance, expressed as a stress (Mosley, Bungey, & Hulse, 2007).

For the raft foundation design, the punching shear check was performed for three different columns: interior column (column Type 15), side column (Type 7) and corner column (Type 16), as shown

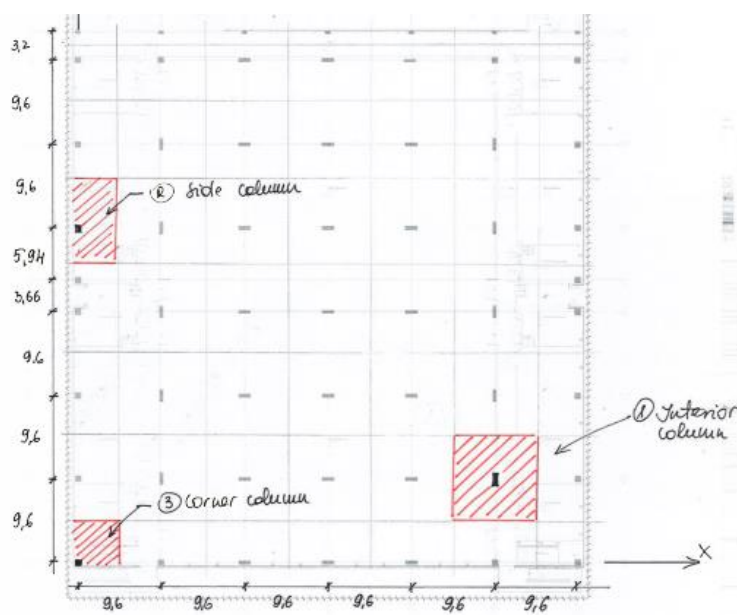


Figure 3-3 Indication of the three types of columns for which the punching shear was checked

in the Figure 3-3. Interior column represents the most heavily loaded column, of which the each face of the column is at least four times the slab thickness away from a slab edge, side column is less than four times the slab thickness away from the slab edge and corner column for when the two adjacent sides of the column are less than four times the slab thickness from slab edges parallel to each other. These column categories are recognised for each structure when calculating punching shear, as, the closer to the edge the column is, the less area it has for taking the shear stress (the punching shear perimeter is incomplete) and the risk of failure is increased.

3.3.2.3. Stresses and settlements

In order to determine the material stresses, the soil stiffness parameters were used, as mentioned earlier in the theoretical framework:

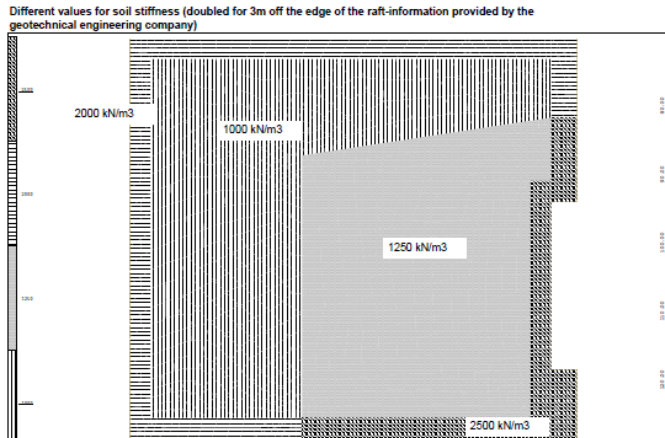


Figure 3-4 Sofistik input soil stiffness

For areas with 1 m or less of soil exchange the soil stiffness:

$$k_{s,1} = 1.0 \text{ MN/m}^3$$

For areas with 5 m of soil exchange anticipated soil stiffness:

$$k_{s,5} = 1.25 \text{ MN/m}^3$$

On the edge strip of $b = 3 \text{ m}$ the stiffness module shall be doubled (value considered for checking the settlements).

If the resultant of all loads acting on the foundation passes through the centre of gravity of the slab, the contact pressure is given by:

$$\sigma = \frac{\sum P}{A}$$

If the resultant has an eccentricity in the x and y direction, the contact pressure becomes:

where

$$I_x = B L^3 / 12 = \text{moment of inertia about the x-axis}$$

$$I_y = L B^3 / 12 = \text{moment of inertia about the y-axis}$$

$$M_x = \sum P e_y = \text{moment of the column loads about the x-axis}$$

$$M_y = \sum P e_x = \text{moment of the column loads about the y-axis}$$

$$q = \frac{\sum P}{A} \pm \frac{M_x y}{I_x} \pm \frac{M_y x}{I_y}$$

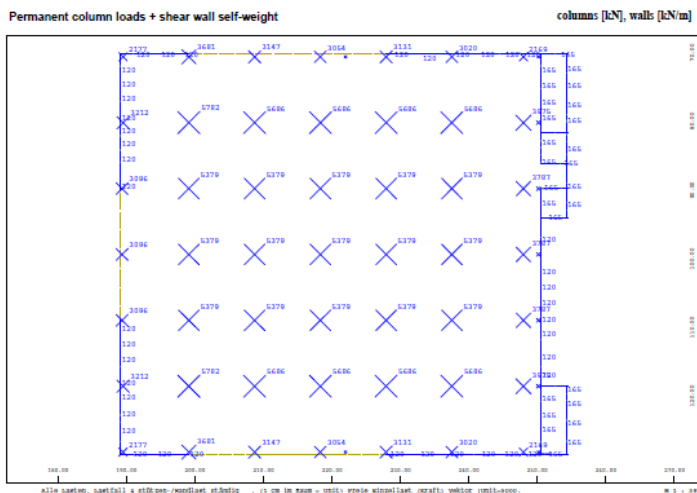


Figure 3-5 Sofistik input loads

The centre of loads was compared to the centre of gravity of foundation and the maximum stresses were determined for serviceability limit state as well as ultimate limit state accordingly. Firstly, the estimation was done manually and then compared with Sofistik software, additionally considering the wind loads, line loads from the stiffening walls and the area loads on the ground slab (Figure 3-5). The model helped verify the hand calculations and provide additional information about the distribution of stresses.

The settlements were also determined with the same *Sofistik* model. The full set of the preliminary design calculations can be found in “Appendix D – Raft foundation (preliminary design)”.

3.4. Preliminary design – pile foundations

Both large diameter mono piles and smaller piles (as part of the pile groups) were calculated using the same design procedure and analysing the same soil characteristics. The difference was that the mono piles resulted in much greater depths because of the high concentrated column loads and their self-weight, which was considered in the preliminary calculations of the pile foundations. The piles were checked for their base and shaft resistance, applying the necessary reduction factors, as described in the regulations of the geotechnical design. Eventually having set the possible dimensions (pile diameter and length) as well as the amount of piles, the direct costs were calculated for the two remaining variants as well. Note that the pile caps were not included in the preliminary design, therefore the volume of concrete, which would be required, was not considered either.

3.4.1. Loads on foundations

58 columns were grouped by loading in 18 types and for the purpose of preliminary pile design, again in 4 design categories (Table 3-3), from least heavily loaded to most heavily loaded columns accordingly. G_k and Q_k are the permanent and variable compression loads accordingly.

Table 3-3 Column design categories for preliminary pile design

Category	Column position	G_k kN	Q_k kN
Design 1	C1-C7, C29, C30	2250	560
Design 2	C8, C14, C31, C37, C38, C44, C52, C58	3185	2150
Design 3	C9-C13, C15, C21, C22, C28, C45, C51, C53- C57	4550	3360
Design 4	C16-C20, C23-C27, C32-C36, C39-C43, C46-C50	5785	5040

3.4.2. Structural calculations

The full set of calculations for both pile groups and mono-piles can be found in “Appendix E – Pile foundation (preliminary design)”.

3.4.2.1. Important notes

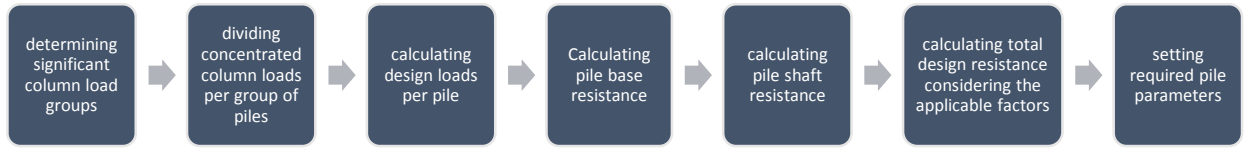
The permanent design load is considered including the self-weight of the pile(s). Weight of the pile caps is not included.

The size of each pile regarding the length and diameter was determined by trial and error method in Excel.

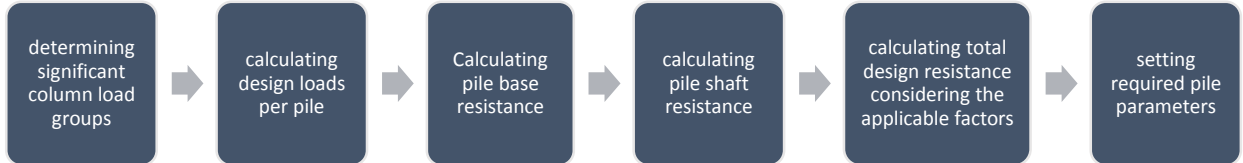
Foundation design (application of the soil conditions to the foundation design and the installation):

- DIN EN 1997-1 (Eurocode 7 for geotechnical design)
- DIN 1054: 2010 (national regulations)
- DIN EN 1536: 2015 (regulations for special geotechnical design: bored piles)
- DIN SPEC 18140: 2012 (regulations for special geotechnical design: bored piles)

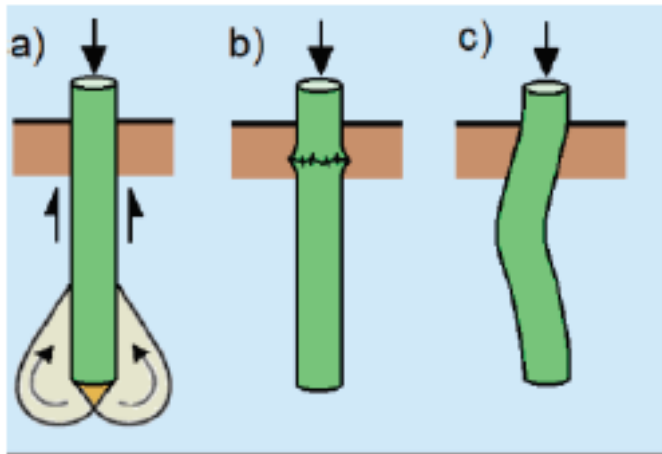
3.4.2.2. Pile groups design



3.4.2.3. Mono-piles design



3.4.2.4. Design regulations – axially loaded piles



Equilibrium equation to be satisfied in the ultimate state design of axially loaded piles in compression:

$$F_{c,d} < R_{c,d}$$

Design axial load:

$$F_{c,d} = \gamma_G G_{rep} + \gamma_Q Q_{rep}$$

Wherein G and Q are the representative permanent and variable loads and γ_G and γ_Q are the corresponding partial action factors (Vrettos, 2007).

Figure 3-6 Piles load capacity in compression (Wrana, 2015)

Load bearing capacity of piles:

$$R_{c,d} = \frac{R_{b,k}}{\gamma_b} + \frac{R_{s,k}}{\gamma_s} = \frac{A_b * q_b}{\gamma_b} + \frac{\sum A_{s,i} * q_{s,i,d}}{\gamma_s}$$

Where $R_{b,k}$ is end-bearing capacity of the pile, calculated by multiplying the cross-sectional area with the unit base resistance and $R_{s,k}$ is the total shaft resistance, calculated by multiplying the surface area of pile with the characteristic shaft resistance (friction for sand and adhesion for clay) per unit area in the i -th layer (Vrettos, 2007).

The pile design is based on the soil conditions without soil improvement (as a safer approach).

The reduction in bearing capacity due to increased stresses was neglected for the purpose of preliminary design of the pile groups – it is assumed the distance between the piles is sufficient for avoiding the negative effect.

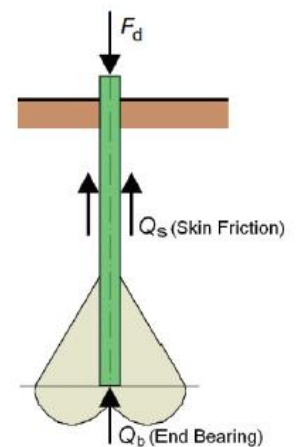


Figure 3-7 Bearing capacity of piles (Wrana, 2015)

3.5. Methodology for sorting out the solutions

As mentioned in the beginning of the chapter, the sequence of activities for obtaining the final project design are based on the selective method.

Six solution variants were created, based on the information in the theoretical framework, including the three possible foundation variants and cross-referencing them with two different locations.

The six solutions and the selection criteria are therefore described below.

The first three solutions are based on the different foundation techniques, constructed in the initial location A (Figure 3-8):

Location A/Option1 – Raft foundation

Location A/Option2 – Pile groups

Location A/Option3 – Mono-piles

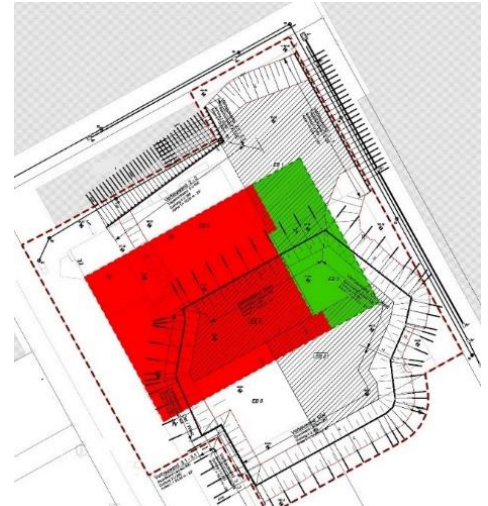


Figure 3-8 Location A

And the rest are the same techniques, but implemented in the new location (Figure 3-9), having less contaminated soil:

Location B/Option1

Location B/Option2

Location B/Option3



Figure 3-9 Location B

3.5.1. Evaluation criteria

3.5.1.1. Criteria 1 - Structural integrity

Firstly, the completion of the preliminary designs helped determine whether the foundation techniques are suitable for this project. The most important issues checked were the material stresses and the settlements for the raft foundation (whether the material properties can carry the applied stresses and if the settlements do not exceed the allowable limit), as well as pile design resistance (combination of base and shaft resistance) for the piled foundations, for determining their parameters.

This criteria was the first step for sorting out the viable variants.

3.5.1.2. Criteria 2 - Duration

After the variants were judged on their structural integrity, the implementation duration (the second criteria) was estimated on a preliminary basis. Based on the Client's requirement, only those variants, which took not more than one year to implement (including both soil remediation activities and the construction of the foundation and superstructure), were considered further. The duration estimations were based on these considerations:

- One year equals 12 months and a month is generalised and calculated as 4 weeks;
- Each week was decided to have 7 working days of 7 hours of continuous work (standard 8-hour working day minus the lunch break and other necessities of the working staff).

3.5.1.3. Criteria 3 - Costs

The final comparison of the variants was based on the cost of the foundation construction only, which includes material costs as well as installation. The expenses for the soil remediation activities were not estimated or taken into consideration in this analysis because these matters are to be covered by the previous owner of the land, as explained in the introduction of the report.

The demolition activities, even though causing additional expenses, were not considered in the evaluation criteria either, because the logistics building was planned to be demolished and reconstructed anyway, only at the later stage of the site development. Because of this reason, the expenses would still eventually have to be covered, even if the location of the data centre is not changed for this project. The demolition of the old basement, however, applies to all of the variants and is therefore non-influential.

All these criteria applied to the six possible solutions in the described sequence determine the final design of the project.

4. Results

4.1.Option 1 – raft foundation

Table 4-1 Raft foundation preliminary design summary

Slab dimensions	57,6 m x 60,8 m +50cm edge distance on all sides against punching shear
Maximum vertical loads to be supported	<u>Interior column:</u> G, d = 7806 kN Q, d = 7555 kN Total factored load used: 15 360 kN <u>Side column:</u> G, d = 6133 kN Q, d = 3224 kN Total factored load used: 9357 kN <u>Corner column:</u> G, d = 3795 kN Q, d = 1612 kN Total factored load used: 5407 kN
Slab thickness	160 cm (interior column) 175 cm (side column with 50cm edge distance) 175 cm (corner column with 50cm edge distance in both directions)
Material details	Reinforced concrete
Volume of concrete	[1] 165cm taken as a uniform thickness [2] width of slab: 57.6m + 0.5 m+0.5 m (edge distance) + 0.7 m (twice half the column) [3] length of slab 60.8m + 0.5 m+0.5 m + 0.7 m [1]*[2]*[3] = 1.65*59.3* 62.5 $\approx$ 6115 m ³
Unit price	250 € / m ³ (based on experience from similar projects)
Total expected cost	6115 m ³ * 250 € / m ³ = 1 528 750 €

4.1.1. Design evaluation

Due to the high loads, applied on the foundation, the thickness of the slab was checked against punching shear failure without additional shear reinforcement and applied as a uniform value across the entire area of the foundation. This resulted in extensive amount of concrete and therefore accordingly high costs. In order to avoid using the largest thickness all across the area, a possible solution for heavily loaded rafts is to use down-standing integrated beams under a row of columns or shear walls (Barnes, 2010). For the purpose of the preliminary design estimation, this solution was not considered.

In addition, the maximum loads for the serviceability limit state design from the superstructure reach up to 11 000 kN, causing maximum settlements of roughly 15 cm (Figure 4-1), which is an unacceptable value for a four-storey building. Because of this reason, the design is not considered as a suitable alternative from a structural perspective for this project.

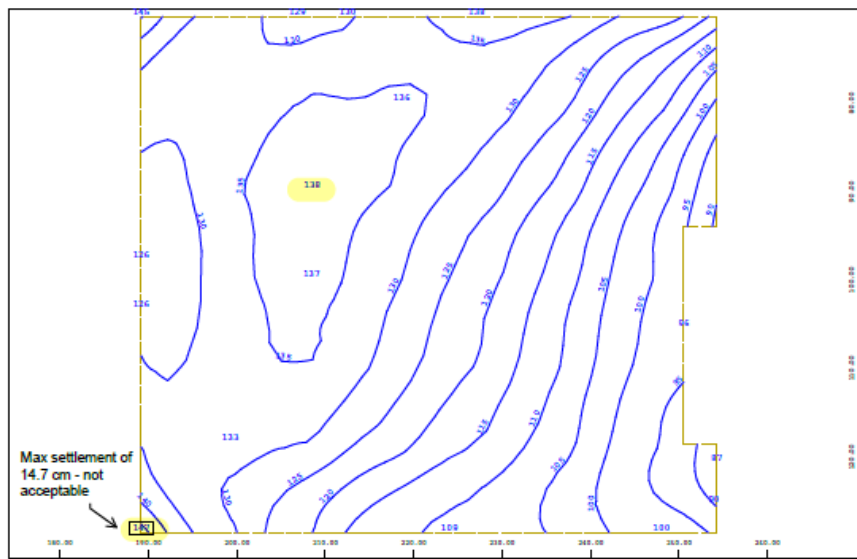


Figure 4-1 Sofistik output – settlements in the serviceability limit state

4.2. Option 2 – Pile groups

Table 4-2 Preliminary design summary - pile groups

Group nr.	$F_{c,d}^*$ [kN per pile]	$R_{c,d}$ [kN per pile]	Group of	Number of piles	Diameter [m]	Length [m]
PILE GROUPS						
Design 1	2 320	2 558	2	18	1.2	10
Design 2	3 081	3 366	3	24	1.2	15
Design 3	3 445	3 689	4	64	1.2	17
Design 4	4 797	4 982	4	100	1.2	25

*The permanent design load is considered including the self-weight of the piles. Weight of the pile caps is not included.

Table 4-3 Cost calculation Option 2

Total number of piles	206
Unit price (based on experience from similar projects)	260 € / m of pile
Total expected cost	$260 \text{ €} * 18 \text{ piles} * 10 \text{ m} = 46\,800 \text{ €}$ $+ 260 \text{ €} * 24 \text{ piles} * 15 \text{ m} = 93\,600 \text{ €}$ $+ 260 \text{ €} * 64 \text{ piles} * 17 \text{ m} = 282\,880 \text{ €}$ $+ 260 \text{ €} * 100 \text{ piles} * 25 \text{ m} = 650\,000 \text{ €}$ $= 1\,073\,280 \text{ €}$

4.2.1. Design evaluation

Advantage: 1.2m pile diameter suggests a standard implementation procedure. Cost-effective solution, when compared to others.

Disadvantage: The amount of piles installed is much more than with mono-piles. Even though it is a standard solution, having the piles densely positioned would not allow the excavation around

them. This means that the construction of the foundation and the superstructure cannot be implemented in parallel.

Regarding the utilization of the piles regarding the vertical loads, the diameter could possibly be reduced for the less heavily loaded piles (Design 1 and 2). However, regarding the design of this option, the variation in diameter for the pile groups was tried to be avoided. Therefore, all piles were considered as the same diameter, just grouped in different ways of either two, three or four piles, which also vary in depth according to the loads applied to them.

4.3. Option 3 – Mono-piles

Table 4-4 Preliminary design summary - mono-piles

Group nr.	F _c , d* [kN per pile]	R _c , d [kN per pile]	Number of piles	Diameter [m]	Length [m]
MONO-PILES					
Design 1	4 938	5 341	7	2.0	10
Design 2	10 642	10 870	8	2.8	15
Design 3	19 055	19 321	16	3.0	33
Design 4	32 605	31 730	25	3.5	40

*The permanent design load is considered including the self-weight of the pile

The variation in the diameter of the mono-piles could be improved/avoided, however for the purpose of the preliminary design, the goal was to minimize the length of the piles as much as possible, by adapting the diameter accordingly.

Table 4-5 Cost calculation Option 3

Total number of piles	58 (for basic design assumed pile per column)
Unit price (an estimate based on experience from similar projects)	700-780 € / m of pile, depending on the diameter chosen
Total expected cost (the price for installation of the largest diameter piles considered for the entire design, as it is impractical to have four different pile diameters within one structure - they require changing of the digging equipment for enlarging the shafts)	$780 \text{ €} * 9 \text{ piles} * 10 \text{ m} = 70\,200 \text{ €}$ $+ 780 \text{ €} * 8 \text{ piles} * 15 \text{ m} = 93\,600 \text{ €}$ $+ 780 \text{ €} * 16 \text{ piles} * 33 \text{ m} = 411\,840 \text{ €}$ $+ 780 \text{ €} * 25 \text{ piles} * 40 \text{ m} = 780\,000 \text{ €}$ $= 1\,355\,640 \text{ €}$ (the price of the temporary steel casings is not included)

4.3.1. Design evaluation

In reality, one diameter would be used for the piles in order to avoid bringing several different boring machines to the building site. As a result, the piles need to reach great depths in order to carry their own weight (which was considered for the preliminary designs), and not only the vertical loads transferred from the building. After checking the most heavily loaded columns (Design 4), the piles resulted in diameter of 3.5 metres and length of 40 metres. Because of this reason, the solution is more pricy as well, when compared to the conventional pile group design

A possible alternative would be to use steel piles (pressed with a static load in order to avoid vibrations during installation), filled with concrete at the several meters at the top in order to increase the bearing capacity. This solution is however not within the scope of the thesis.

4.4. Analysis of the solutions

4.4.1. Analysis step 1 – structural viability

The first step in the methodology followed after the preliminary designs, was checking the structural integrity of the foundation variants.

Location B / Option 1

This option includes having a raft foundation in the new location.

The solution of having the raft foundation in the new location is structurally not viable because of the amount of occurring settlements. Based on the soil parameters, it was estimated to have maximum 15cm of settlements. This value might be accepted for a larger building, however for this data centre, being just four-storeys high, it is not a suitable solution and was therefore denied.

Location B / Option 3

This option includes having a foundation of mono-piles in the new location.

Since the location allow the full exchange of soil prior the construction (the amount of soil is 17 165 m³ compared to 40 992 m³ of location A), the option with mono-piles would result in an unnecessary increase of implementation costs and installation complexity, therefore it is no longer considered after changing the location.

4.4.1. Analysis step 2 – duration of implementation activities

A preliminary estimation of the implementation duration for each of the four solutions was prepared and put in a comparison Table 4-6, providing the impression of the analysis of the variants.

Firstly, the table describes the selected machinery and its capacity (with reference to the theory provided in the theoretical framework), resulting in value in weeks for “Duration 1 – Foundation”. As mentioned before, the planned time settings were:

- One year equals 12 months and a month is generalised and calculated as 4 weeks;
- Each week was decided to have 7 working days of 7 hours of continuous work (standard 8-hour working day minus the lunch break and other necessities of the working staff).

Furthermore, depending on which location applies to the solution, the amount of soil and the required duration is stated (with reference to the theoretical framework as well), based on the machinery used. It results in the amount of time for the “Duration 2 – Ground work”.

Lastly, the building of the superstructure was considered (7 months for all the variants, as the set value, agreed upon between *Arup* and *the Client*) and the total duration in months was calculated.

Due to the requirement of the 12-month implementation, the solutions which exceeded this margin of time were denied.

Table 4-6 Comparison of duration implementation of the four selected solutions

		Solution A1		Solution A2		Solution B2		Solution A3	
EQUIPMENT CAPACITY	Material volume/ Number of piles	6730	m ³	206	piles	206	piles	58	piles
	Machinery characteristics	3 concrete mixing transport trucks (15 m ³), each 5 rounds a day.		Time for full installation of pile with the continuous auger - 1 hour.		Time for full installation of pile with the continuous auger - 1 hour.		Shaft drilling speed: 6m/h; Concrete pour - same as A1.	
	Final capacity	(3*5*15)-15=210	m ³ /day	7	piles/day	7	piles/day	21	m/day
WORKING HOURS		7*7=49	h/week	7*7=49	h/week	7*7=49	h/week	7*7=49	h/week
DURATION - 1: FOUNDATION		6730/(210*7)≈5	weeks	206/(7*7)≈4,2	weeks	206/(7*7)≈4,2	weeks	1+1,5+7+14≈23,5	weeks
LOCATION		A (initial)		A (initial)		B (altered)		A (initial)	
AMOUNT OF SOIL TO EXCHANGE		40 992	m ³	40 992	m ³	17 165	m ³	40 992	m ³
DURATION - 2: GROUND WORK*		28	weeks	28	weeks	13	weeks	28	weeks
DURATION - 3: BUILDING OF SUPERSTRUCTURE		7**	months	7**	months	7**	months	7**	months
TOTAL DURATION		(6+28+28)/4≈16	months	(4,2+28+28)/4≈15	months	(4,2+13+28)/4≈11,5	months	6+7+1= 14 14-2(overlap)=12	months
		16months > 12 months		15 months > 12 months		11,5 months < 12 months		12 months = 12 months	
CONCLUSION		SOLUTION DENIED		SOLUTION DENIED		SOLUTION APPROVED		SOLUTION APPROVED	

* Reference to the estimated duration in the theoretical frame work

** Reference to the implementation requirements described in the introduction of the report

Location A / Option 1

This option includes having a raft foundation in the initial location. The great amount of in-situ concrete volume ($6\,730\text{ m}^3$, including 10% additional volume to cover any material losses) suggested having several concrete mixing transport trucks. The large volume trucks of 15m^3 were considered, each doing 5 rounds a day. This value was an approximation, as no information was provided about the location where the concrete is mixed and it was not possible to calculate the travel time. Also, by following the assumed cycle, the last truck would arrive to the building site on time, however, would not be emptied. Because of this reason, 15 cubic metres are subtracted from the total daily capacity. The total duration therefore includes 6 weeks for laying the foundation, 28 weeks for soil exchange and 28 weeks for the superstructure.

Designing a building in location A also means large area for soil remediation and no or very little flexibility regarding the accessibility of the site for the machinery. Total duration was estimated to be 16 months and therefore the solution is not suitable for the project.

Location A / Option 2

This option includes having a foundation of pile groups in the initial location. The foundation duration included the installation of 206 piles of 1.2m, considering 60mins per pile using CFA method – continuous auger. The daily capacity was estimated to be 7 piles a day, resulting in 4.2 weeks for the foundation (7 days of work, 7 piles per day). Location A groundwork is the same 28 weeks as previously described, as well as 28 weeks for the superstructure.

Similarly to Option 1, location A means prolonged duration of the soil remediation activities and little flexibility regarding the construction site logistics. Total duration was estimated to be 15 months and therefore the solution is not suitable for the project.

Location B / Option 2

This option includes having a foundation of pile groups in the new location. Pile groups is a good solution for this location, as soil exchange is completed prior their installation. CFA installation method is efficient, as mentioned in the evaluation of the previous solution. The strong advantage of this solution is the changed location, which requires less soil exchange - $17\,165\text{ m}^3$ compared to $40\,992\text{ m}^3$ of location A. The estimated time for the groundwork of location B was 13 weeks, therefore the final duration resulted in 11.5 months and is therefore a suitable solution for the project..

Location A / Option 3

This option includes having a foundation of mono-piles in the initial location. The installation of large piles and the excavation of soil around them allow for parallel construction activities – this was the reason for introducing the design option in general. The duration for installing the piles was calculated by using the information about the auger working speed and the estimated concrete pouring speed in the first solution (A1). The auger can drill a shaft up to 2.2m in diameter with speed of 6 metres/hour. Then, the shaft is widened up to the required 3.5 m (it was assumed that all the piles have the same diameter, in order to reduce the difference in capacity) with the speed of 7 metres/hour. However, if considering some additional time for changing the auger head and the manoeuvring, it was decided to consider 6 m/h as a uniform speed. In this manner, the auger can drill 21m per day. For pouring the concrete, the daily capacity is similar, when considering $210\text{m}^3/\text{day}$, for a pile of 3.5m in diameter, 22 m (in depth) can be poured per day.

Therefore, for the piles of 10m in length, 2 piles can be fully installed per day (20m of drilling and 20m of pouring concrete). For piles of 15m in length, one pile can be finished in a day, using the remaining time for removing the soil off site or similar cleaning-up activities. A single pile of 33m in depth was estimated to be installed within 3 days – 1 day drilling up to 21m, second day finishing drilling the 12m and pouring 10m depth of concrete, finishing the third day. In this situation the use of temporary steel cases is therefore required, if the installation cannot be continued overnight. A single pile of 40m therefore would be installed in approximately 4 days.

Number of piles	Depth, m	Piles/day	Total installation time, weeks
9	10	2	1
8	15	1	1,5
16	33	$\frac{1}{3}$	7
25	40	$\frac{1}{4}$	14

The foundation duration was calculated to be 6 months. However the introduction of the parallel construction suggests that the soil can be dug while the superstructure is being built. For that, an additional month is considered for installing the ground floor slab. The duration is therefore $6+1+7=14$ months. Due to a large area of the building, the groundwork may begin on one side of the building, while a few of the last piles are being installed on the other. Therefore, the 2 months out of the 6 were allowed to overlap with the other activities, resulting in total duration of 12 months. Solution is therefore viable.

4.4.2. Analysis step 3 – cost comparison

Mono-piles in the initial location and the pile groups in the new location were the two final solutions. Therefore, a cost comparison was made and the total calculated expenses for the foundation techniques were:

- 1,073,280 € for the foundation of pile groups;
- 1,355,640 € for foundation of mono-piles.

The expenses for the soil remediation activities were not taken into consideration in this analysis because these matters are to be covered by the previous owner of the land, as explained in the introduction of the report.

The demolition activities, even though causing additional expenses, were not considered in the evaluation criteria either, because the logistics building was planned to be demolished and reconstructed anyway, only at the later stage of the site development. Because of this reason, the expenses would still eventually have to be covered, even if the location of the data centre is not changed for this project. The demolition of the old basement, however, applies to all of the variants and is therefore non-influential.

4.4.3. Conclusion

In addition to being the more expensive solution, large diameter mono-piles, even though permitting the solution for the overlapping of works, come with the risks for the stability of the structure and the safety of the people working on site, uncommon design requires special equipment. All reasons considered, the foundation of pile groups in the new location B is the most suitable solution and is therefore designed as the final product.

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5. Final foundation design

After analysing all the possible variants using a selective method and considering the criteria for structural integrity, implementation duration and foundation costs, it was decided that a pile group foundation in the new location B is the most suitable solution for this project.

This chapter therefore summarises the structural calculations for the foundation design. A full set of calculations can be found in “Appendix F – Final design” together with structural drawings in “Appendix H – Drawings”.

5.1. Guidelines for determining applicable loads

In the following chapter the overview of the vertical and horizontal loads from the building superstructure is summarized.

EN 1991-1-1 gives design guidance and actions for the structural design of buildings and civil engineering works, including the following aspects:

- Densities of construction materials and stored materials;
- self-weight of construction elements, and
- imposed loads for buildings.

Permanent loads

Permanent loads include the self-weight of the structural and non-structural elements. In Eurocode these are defined as:

- Structural elements comprise the primary structural frame and supporting structures;
- Non-structural elements are those that include completion and finishing elements, connected with the structure, including road surfacing and non-structural parapets. They also include services and machinery fixed permanently to, or within, the structure.

Reinforced concrete	g_0	=	25.00 kN/m ³
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Area loads:

Steel construction on the roof (Arup assumption, based on the experience on similar projects)	g_1	=	0.50 kN/m ²
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Slab over 3 rd floor (roof slab)			
Sealing and isolation	g_1	=	1.80 kN/m ²
Ceiling construction and technical installation	g_2	=	1.50 kN/m ²
	Σg	=	3.30 kN/m ²

Slab over 2 nd and 1 st floor			
Floor construction and technical installation	g_1	=	1.80 kN/m ²
Ceiling construction and technical installation	g_2	=	1.50 kN/m ²
	Σg	=	3.30 kN/m ²

Ground floor slab			
Floor construction and technical installation	g_1	=	1.8 kN/m ²

Visual and/or sound screen on the roof	g_1	=	1.0 kN/m ²
Facade (incl. steel sub-structure)	g_1	=	1.0 kN/m ²
(Arup assumption, based on the experience on similar projects)			

Imposed loads

Data Hall	q	=	12.00 kN/m ²
Equipment rooms (incl. foundations of the machines)	q	=	15.00 kN/m ²
Roof – maintenance load	q	=	1.0 kN/m ²
Roof – equipment (average loading)	q	=	9.0 kN/m ²

Snow loads

Frankfurt am Main area: Snow zone 1, Terrain height 114.0 mNN. The influence of the barrier on the snow load is neglected.

$$S_k = 0,19 + 0,91 * \left(\frac{h + 140}{760} \right)^2 > 0,65 \frac{kN}{m^2}$$

$$h = 114 \text{ m}$$

$$S_k = 0,29 \frac{kN}{m^2}$$

Thus the characteristic snow load is:

$$S_k = 0,65 \frac{kN}{m^2}$$

Wind loads

Frankfurt am Main area: Wind zone 1, Terrain category III. These parameters define the equivalent wind pressure (Schneider, 2010):

$$q_{b,0} = 0,32 \frac{kN}{m^2}$$

For buildings higher than 25 metres, following the table 3.26b (Schneider, 2010):

$$q_p(z) = 1,6 * q_b * \left(\frac{z}{10} \right)^{0,31}$$

$$q_p(z = 32.6) = 1,6 * 0,32 \frac{kN}{m^2} * \left(\frac{32.6}{10} \right)^{0,31} = 0,73 \frac{kN}{m^2}$$

Wherein 32.6 (m) is the total height of the building, including the barrier on the roof.

Seismic loads

Due to the geographical location of the new building, the seismic loads are not applicable.

5.1.Design specifications

Foundations should be constructed so that the undersides of the bases are below frost level. As the concrete is subjected to more severe exposure conditions a larger nominal cover to reinforcement is required. A concrete class of at least C30/37 is required to meet durability standards (Mosley, Bungey, & Hulse, 2007).

A few changes were introduced to the layout of the building after the completion of preliminary designs and it was decided to incorporate those changes and create the final design in accordance to the last building layout (see “Appendix F – Final design”). Consequently, the design was split into three different parts, described further in this chapter.

5.2. Summary of loads for foundation design

5.2.1. Vertical loads

Vertical loads on the foundation include the self-weight of construction elements, in addition to permanent and imposed area loads.

The complete load on the foundation therefore consists of:

- Permanent and imposed area loads from technical installation, data centre equipment, maintenance walkways and similar;
- Self-weight of slabs (two types were considered: one for the roof slab and one for the remaining floors);
- Self-weight of shear walls, façade and screen on the roof;
- Self-weight of beams (10 different types of beams, depending on their size and shape).

Table 5-1 summarises the vertical loads from the building on the foundation. The load overview is based on the position of columns, which were assigned a number from 1 to 58 (Figure 5-1). Depending on loading characteristics, columns were grouped into 4 design categories for the purpose of standardising the loads and minimising the amount of data, which is then used for the design of the foundation. For the full calculation see Appendix B.

Table 5-1 Summary of vertical characteristic loads on the foundation by category

Category	Column position	G, k [kN]	Q, k [kN]
Design 1	C1-C7, C29, C30	2250	560
Design 2	C8, C14, C31, C37, C38, C44, C52, C58	3185	2150
Design 3	C9-C13, C15, C21, C22, C28, C45, C51, C53-C57	4550	3360
Design 4	C16-C20, C23-C27, C32-C36, C39-C43, C46-C50	5785	5040



Figure 5-1 Column Plan

5.2.2. Horizontal loads

Lateral loads on the foundation of the building include wind pressure, as well as notional inclination of the vertical members, representing imperfections. The distribution of the horizontal loads was determined using *Frilo* software for stability calculation. Note that the so called “staircase area” was not considered for the distribution of the horizontal – the stiffening walls mark the boundary of the building for the calculation (Figure 5-2).

The following parameters have been set for the software input:

- Stiffening walls: 8
- Concrete class: C45/55 (standard used for precast elements)
- Building area: 57.60 m x 60.80 m
- Wind zone: 1, Terrain category: III
- Sound barrier (“Attic”) height: 6.15 m
- Force coefficient (external wind pressure): 1.3

The below shows the output from the software. The loads are shown per each floor of the building and include both wind loads and imperfections.

Table 5-2 Overview of horizontal loads per floor. See calculation appendix for more details

Wind X - direction		Wind Y - direction	
Slab over 3 rd floor	1470.75 kN	Slab over 3 rd floor	1650.64 kN
Slab over 2 nd floor	523.94 kN	Slab over 2 nd floor	576.29 kN
Slab over 1 st floor	499.24 kN	Slab over 1 st floor	548.36 kN
Slab over Ground floor	491.34 kN	Slab over Ground floor	540.44 kN

Table 5-3 Summary of horizontal characteristic loads on the foundation

Stiffening wall position	Load per wall [kN]
W1	1972
W2	112
W3	653
W4	672
W5	107
W6	1395
W7	1372
W8	20

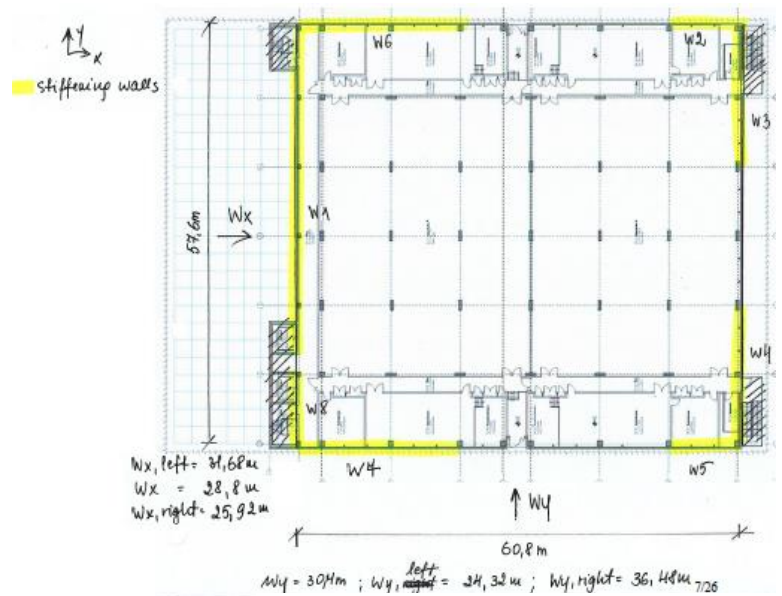


Figure 5-2 Indication of stiffening walls and the wind directions considered

Slab over 3rd floor receives significantly higher load due to the presence of the sound barrier.

The sum for total forces in X and Y direction is more than 100%. This is because for the distribution of lateral loads, the position of the wind force was taken with 10% eccentricity in both directions – W_{left} and W_{right} accordingly.

The loads of each floor is distributed between the eight stiffening walls (Table 5-3):

- Horizontal loads from Y – direction are supported by walls W1, W3, W4 and W8;
- Horizontal loads from X – direction are supported by walls W2, W5, W6 and W7.

For the full calculation of the horizontal loads see Appendix C.

5.3.Foundation design - Part 1

In the first part, the design of the piles and their groups is described. The design is applicable to the foundation of the building, excluding that in axes A-B (for that see design “Part 2”). The design includes a pile plan, calculations for choosing the amount and dimensions of piles and pile caps as well as reinforcement calculations and the reasoning behind the decisions made.

5.3.1. Pile plan

The pile plan was created by analysing all the vertical and horizontal loads, described previously, and designing groups of 4 or 6 piles, which were eventually grouped into 8 cases (Figure 5-3). Cases 1 to 7, representing groups of 4 piles, were calculated in the first part of the design and summarised in Table 5-4. Case 8 of 6 piles is described in the following sub-chapter of “Part 2”.

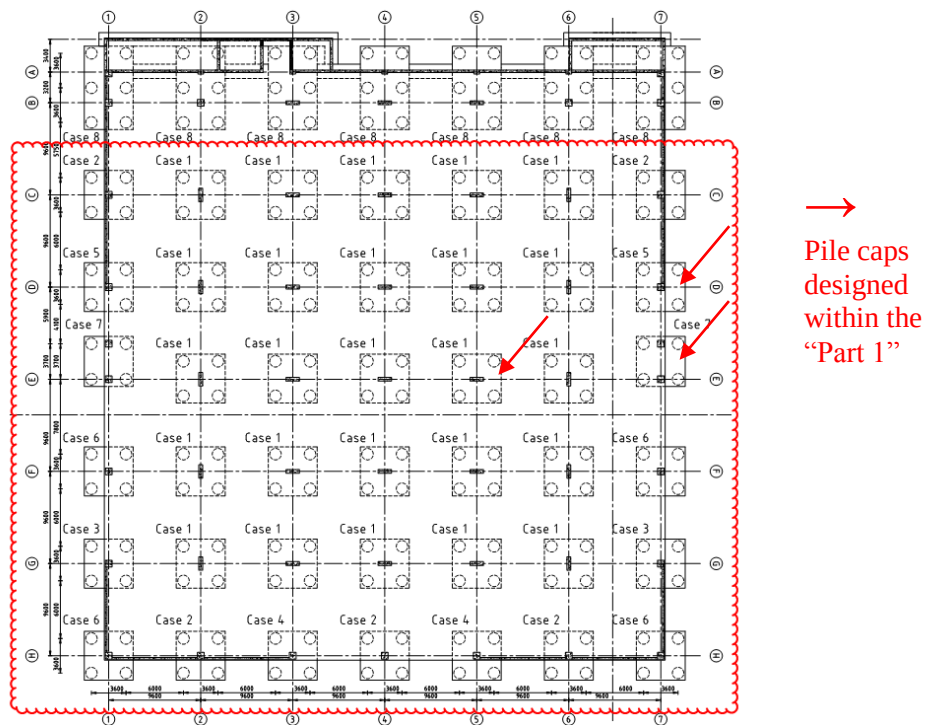


Figure 5-3 Scope of Final Design Part 1

Table 5-4 Pile plan - summary of different cases and applied loads

Case number	Applicable columns	Characteristic permanent load G_k [kN]	Characteristic variable load Q_k [kN]	Wind load (if applicable)
Case 1	C16-20, C23-27, C32-36, C 39-43, C46-50	5785	5040	-
Case 2	C15, 21, 53, 55, 57	4550	3360	-
Case 3	C45, 51	4550	3360	230
Case 4	C54, 56	4550	3360	670
Case 5	C22, 28	4550	3360	1220
Case 6	C38, 44, 52, 58	3185	2150	910
Case 7 *	C29+31, C30+37	5435	2710	-
Case 8 **	C1-14	See “Part 2”	See “Part 2”	See “Part 2”

* Case 7 represents a single pile group carrying the load of two columns considered together. The reason behind the combined load is a near placement of the two columns with considerably low loads, which allowed them to be combined as applied to one pile group with a single cap.

**Columns applicable to Case 8 have updated loads since the concept design, therefore analysed separately.

5.3.2. Pile characteristics

The following calculations define the different pile capacities in order to select the necessary lengths for each of the pile group cases.

Design bearing capacity (resistance) can be defined as:

$$R_{c,d} = \frac{R_{b,k}}{\gamma_b} + \frac{R_{s,k}}{\gamma_s}$$

Wherein $R_{b,k} = A_b * q_b$ and $R_{s,k} = \sum A_{s,i} * q_{s,i,d}$

Where $R_{b,k}$ is end-bearing capacity of the pile, calculated by multiplying the cross-sectional area with the unit base resistance and $R_{s,k}$ is the total shaft resistance, calculated by multiplying the surface area of pile with the characteristic shaft resistance (friction for sand and adhesion for clay) per unit area in the i -th layer (Wrana, 2015).

Partial factors on effect of structural actions ($A1$): 1.35 and 1.5 (Schneider 11.4)

Partial factors for soil parameters γ_b and γ_s as 1.4 (based empirical correlations, Schneider 11.5b)

The calculation was performed with Microsoft Excel. Then each of the seven cases were calculated to determine the design loads per pile and assign the piles with necessary design resistance values.

Following the selection, a check for the significance of the down-drag was checked, in case of replacing the top layers with sand and having an increased loading on the compressible clay layer underneath. The change in pressure was insignificant, however, the influence of compaction was not considered, therefore, the results cannot be verified to be correct. In addition, no information was provided about the degree of consolidation of the clay layer or the history of stresses, thus, without having the necessary information, it was assumed that the clay layer is over-consolidated and that a slight increase in effective stresses would not cause the soil to settle and the down-drag was therefore neglected.

Finally all the piles were decided to be of the same diameter of 1.2m and the remaining parameters resulted in:

Case 1 – Design load = 15,370 kN / 4 piles - 19m in length;
 Case 2 – Design load = 11,183 kN / 4 piles - 12m in length;
 Case 3 – Design load = 11,528 kN / 4 piles - 13m in length;
 Case 4 – Design load = 12,188 kN / 4 piles - 14m in length;
 Case 5 – Design load = 13,013 kN / 4 piles - 15m in length;
 Case 6 – Design load = 8,890 kN / 4 piles - 10m in length;
 Case 7 – Design load = 11,403 kN / 4 piles - 13m in length;

The pile design checks against structural failures were checked only once for the most significant piles:

- Compression check for the most heavily loaded pile (Case 1),
- Tension check for a pile with the decisive wind load (considering the tensile force from the push-pull effect) – Case 5.

Table 5-5 Summary of pile resistances

10	m	R _{c,d}	2500	kN
11	m	R _{c,d}	2660	kN
12	m	R _{c,d}	2820	kN
13	m	R _{c,d}	2980	kN
14	m	R _{c,d}	3140	kN
15	m	R _{c,d}	3300	kN
16	m	R _{c,d}	3460	kN
17	m	R _{c,d}	3620	kN
18	m	R _{c,d}	3790	kN
19	m	R _{c,d}	3945	kN
20	m	R _{c,d}	4110	kN

Concrete class:

C30/37

Compressive strength of concrete

 $f_{ck} = 30 \text{ N/mm}^2$

Tensile strength of concrete

 $f_{ctm} = 2.9 \text{ N/mm}^2$

Reduction factor for bored concrete piles without permanent casing

 $\gamma = 1.1$

Pile diameter:

1.2 m

Compression check for pile of Case 1: $G, k = 5,785 \text{ kN}$ $Q, k = 5,040 \text{ kN}$

Compressive resistance: 28.3 MN

Design load: 15.4 MN

 $28.3 \text{ MN} > 15.4 \text{ MN} \rightarrow$ pile design sufficient for decisive load;

Figure 5-4 Pile in compression sketch (governing check)

Tension check for pile of Case 5: $F_{wind} = 1,220 \text{ kN}$

Tensile resistance: 3,016 kN

 $3,016 \text{ kN} > 1,220 \text{ kN} \rightarrow$ sufficient for tension being decisive force;Buckling check:

For piles completely embedded in ground failure by buckling is unlikely – shall be checked for those piles which are to be built in soil layers with characteristic undrained strength of less than 15 kPa (Tomlinson & Woodward, 2015). In this design only the top layer of the ‘fill’ might have this low strength (the considered soil parameters were described in the theoretical framework), however, being just 1m in depth and exchanged after the soil remediation activities, is neglected. Therefore buckling failure is not applicable.

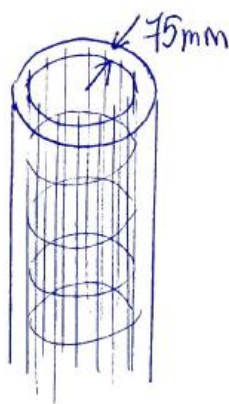
5.3.1. Pile reinforcement design

Figure 5-5 Pile reinforcement cage sketch

Due to the reason that the piles are not subject to any significant lateral loads and do not have a risk of buckling, the minimum amount of reinforcement according to the national regulations for bored piles DIN EN 1536 was designed as the same for all the piles.

Steel grade: B500A

Longitudinal reinforcement: 15Ø16 as required, chosen 16Ø16 for assuring equal spacing between bars as well as avoiding intersection with the reinforcement of the pile cap (see drawings for more details).

Transverse reinforcement: Ø10-250

Reinforcement placed within the top 1/3 of the pile's length (DIN EN 1536)

Concrete cover for uncoated piles in soft ground, if reinforcement is pressed into the fresh concrete (DIN EN 1536) = 75 mm.

5.3.2. Pile settlements check

The settlement calculation was performed by the geotechnical company.

The reason behind this check is to study the effect of having a densely implemented pile group instead of just a single pile. The values were compared against the allowable amount, described in the construction regulations as “Sg” for the serviceability limit state.

$$S_g = 0.1 * D_s, \text{ wherein the } D_s \text{ is the diameter of the pile (Schneider, 2010)}$$

Two types of calculations were done – for a pile group of 4 piles and a pile group of 3. The piles considered in the calculation were taken as 20m long with design loads of 14 055 kN and 9900 kN as permanent and variable loads respectively.

The outcome of the settlement check for both the pile and the pile group is considered acceptable for all the piles within this design. See Final Design appendix for a copy from the report.

5.3.3. Pile cap design

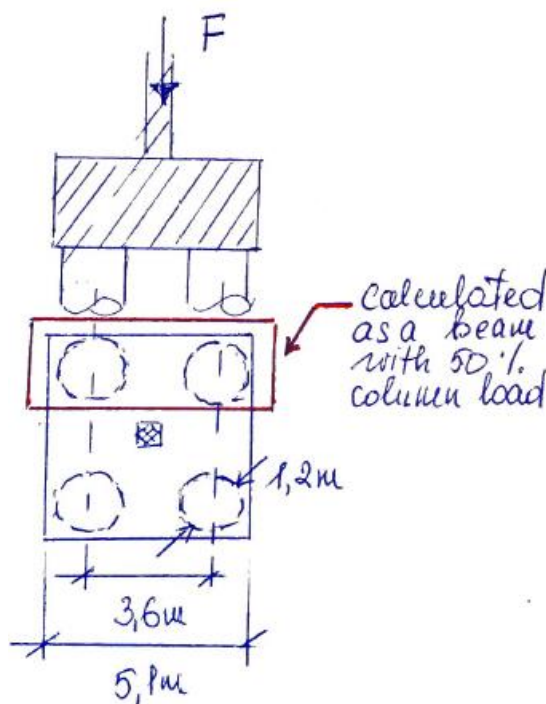


Figure 5-6 Pile cap sketch

For practical implementation of the foundation construction works, the variety between the thickness of the pile caps was limited to two options: larger thickness for the most heavily loaded piles (in the middle of the building, as well as Case 7 where two columns are sharing one pile cap) and lesser for all the remaining. In calculations they are referred to as Case 1 and Case 5 accordingly.

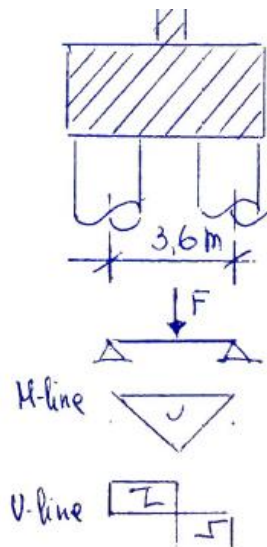
The column load was divided by two, assuming that it spreads equally between the two sides of the cap and into the piles accordingly. The cap reinforcement was designed considering the strips over each two adjacent piles as a beam, subject to half the load (Figure 5-6).

From the beam design, reinforcement amount at the bottom and at the top of the beam was calculated. In the middle of the cap, a single layer at the top and at the bottom is provided against crack control (see drawings for more detail).

5.3.3.1. Pile cap Case 1

Pile cap material summary:

Concrete class	C30/37
Steel grade	B500A
Concrete cover	50mm
Exposure class	XC2



Heaviest column loads (also applicable for Case 7 due to a combination of column loads)

G, k = 5,785 kN

Q, k = 5,040 kN

Combined design load = $15,370 \text{ kN} / 2 = 7,685 \text{ kN}$

$$M = \frac{F * l}{4}$$

$M_{ULS} = 6,917 \text{ kNm}$

$M_{SLS} = 4,872 \text{ kNm}$

$V_{ULS} = 7,685 / 2 = 3,843 \text{ kN}$

Dimensions:

Pile cap thickness 2 m

Area 5.1 x 5.1 m

Figure 5-7 Pile cap design as a beam - structural schemes

Pile cap thickness was decided on after checking the cap against punching shear failure (without additional shear reinforcement) with the *Frilo* software.

The centre-to-centre distance between the two adjacent piles is 3.6m (Figure 5-7), due to a requirement that it has to be no less than three times the diameter in order to avoid the accumulation of stresses in the ground. Additional 15cm were considered on each of the edges. Therefore the pile cap width became:

$$3.6\text{m} + 1.2\text{m} + 0.3 = 5.1\text{m}$$

Reinforcement details:

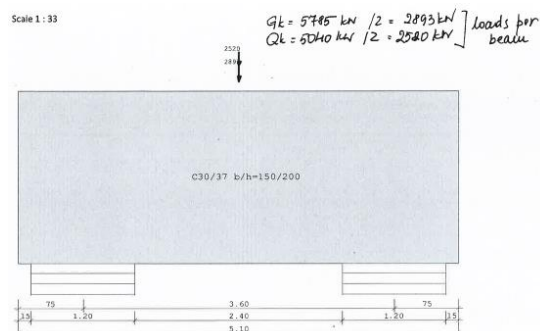


Figure 5-8 Pile cap modelled as a beam for reinforcement calculations Case 1

Bottom reinforcement (placed in two layers)

20Ø25 required, 22 Ø25 chosen to avoid the bars intersecting with the reinforcement in the pile

Top reinforcement 10Ø25 required, 11 Ø25 chosen

Shear reinforcement Ø12-150

Crack control check provided in the Appendix of the Final Design.

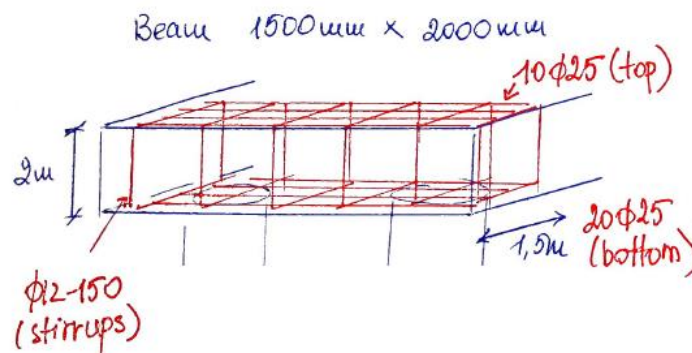


Figure 5-9 Reinforcement sketch pile cap Case 1

5.3.3.2. Pile cap Case 5

Pile cap material summary:

Concrete class	C30/37
Steel grade	B500A
Concrete cover	50mm
Exposure class	XC2

The significantly loaded case from all the remaining:

$G, k =$	4,550 kN
Q, k (wind load included) =	4,580 kN
Combined design load = $13,013 \text{ kN} / 2 =$	6,506.5 kN

$$M_{ULS} = 5,855.85 \text{ kNm}$$

$$M_{SLS} = 4,108.5 \text{ kNm}$$

$$V_{ULS} = 6,506.5 / 2 = 3,253 \text{ kN}$$

Dimensions:

Pile cap thickness	1.80 m
(less than the previous design due to lower vertical load=lower shear stress)	
Area	5.1 x 5.1 m

Reinforcement details:

Bottom reinforcement (placed in two layers) 26 Ø20

Top reinforcement 13Ø20

Shear reinforcement Ø12-150

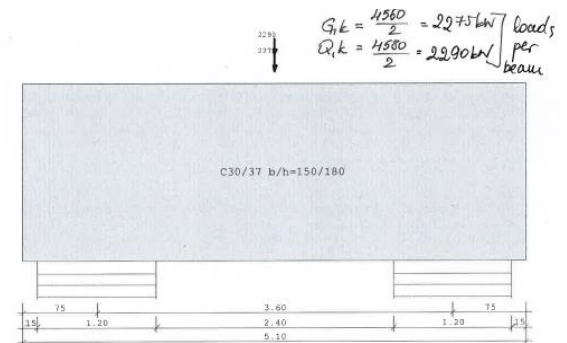


Figure 5-10 Pile cap modelled as a beam for reinforcement calculations Case 5

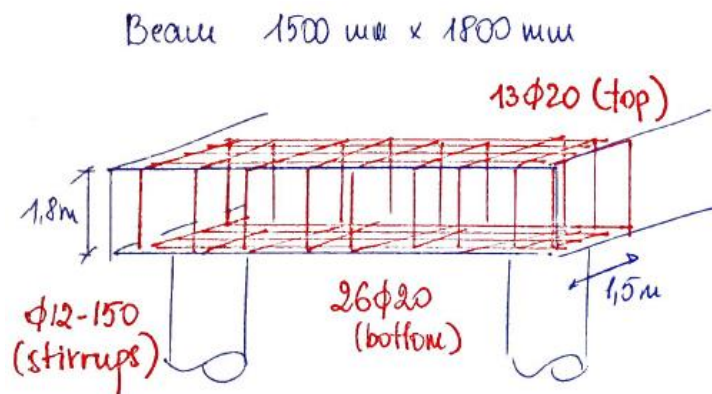


Figure 5-11 Reinforcement sketch pile cap Case 5

5.3.3.3. Pile cap Case 7

Due to a special case of two columns standing on the same pile cap, the loads were higher than those for Case 5. Because of this reason, Case 7 was compared with the loads and the reinforcement chosen for Case 1 (see Final Design appendix).

In conclusion, pile cap design is equal to that for Case 1 except that the cap dimensions were chosen to be 5.2m in length rather than 5.1m in order to keep the centre line of the two adjacent

piles in line with the centre lines of the columns. The change of 10cm was not considered significant to require a recalculation.

Loads:

G, k = 5435 kN

Q, k = 2710 kN

Dimensions:

Pile cap thickness 2 m

Area 5.1 x 5.2 m

Reinforcement details:

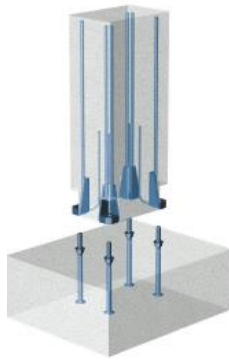
Bottom reinforcement (placed in two layers)

20Ø25 required, 22 Ø25 chosen to avoid the bars intersecting with the reinforcement in the pile

Top reinforcement 10Ø25 required, 11 Ø25 chosen

Shear reinforcement Ø12-150

5.3.4. Column to pile cap connection



The difference between the installations of a precast concrete column and the in-situ pile cap requires a thought-through connection. For the designs of similar projects special anchors were chosen, which are also suitable for this design (Figure 5-12).

The columns are manufactured with the special holes for connecting the anchors during the installation and therefore the process is rather simple (Figure 5-13) (Peikko Group, 2017).



Figure 5-12 Precast column to pile cap connection detail (Peikko Group, 2017)

Figure 5-13 Precast column manufactured for the anchor connection (Peikko Group, 2017)



Figure 5-14 Columns connected to the foundation on site (Peikko Group, 2017)

5.4. Foundation design - Part 2

Part 2 focuses on a specific part of the building (axes A-B) and deals with a new addition to the foundation design which is referred to as the "staircase area" (Figure 5-15).

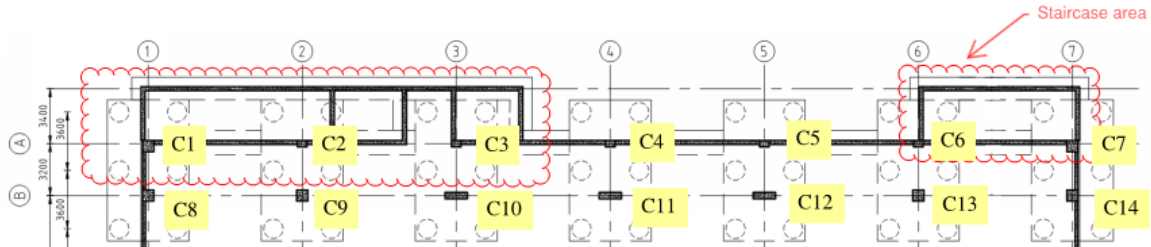


Figure 5-15 Additional area considered in the Part 2 of the Final Design

This part was not included in the preliminary designs, therefore is analysed in the final design. In addition, the length of two stiffening walls (W1 and W8) and a distribution of loads changed as well. The differences are explained in the calculations and plans attached in the appendix of the Final Design.

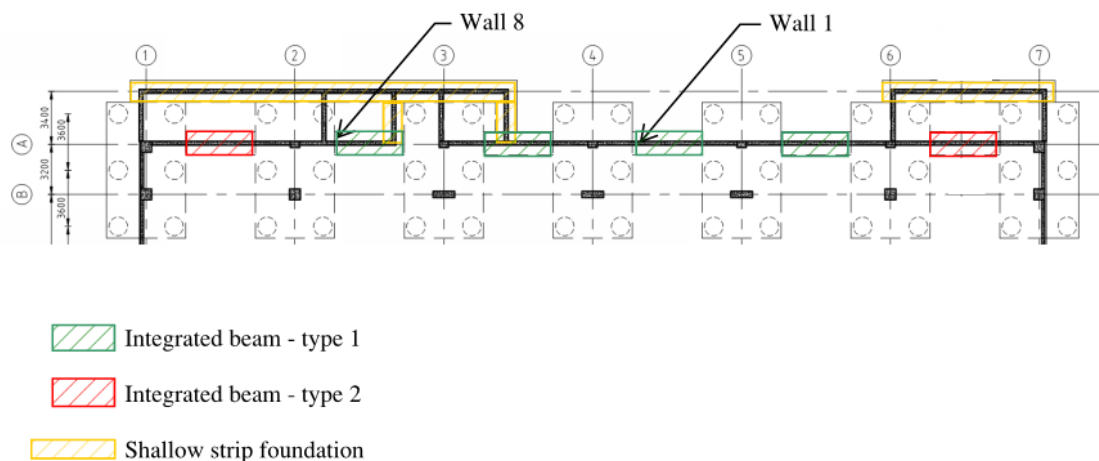


Figure 5-16 Scope of Final Design Part 2

Part 2 therefore presents the design of integrated beams (positioned between pile caps at axis A) for distributing the wind load and area loads carried by the walls. The difference between this part and the rest of the building regarding the lateral loads, is that other walls are laying on beams at each floor, which then transfer the loads to the piles directly. In axis A, there are no beams present and walls are stacked on top of each other and therefore require a structure at the foundation level for the distribution of loads.

The design includes the calculation of the beams and their reinforcement, considering the connection between them and pile caps. As well as that, the design of a shallow foundation under the walls of the staircases and elevator shafts is included.

The detailed calculation of modified loads is carried out in the Final Design appendix.

5.4.1. Update: wind loads Axis A

On axis A the wind load is carried by two walls: W1 and W8. After the completion of the preliminary designs, the layout of the building was slightly changed and therefore the walls became different in length.

When considering the total moment at the foundation level for walls 1 and 8, the linear loads are calculated and considered in the further design of the intermediate beams.

Wall 1:

$$\begin{aligned} M &= 33379 \text{ kNm} \\ L &= 39 \text{ m} \\ q &= 132 \text{ kN/m} \end{aligned}$$

Wall 8:

$$\begin{aligned} M &= 5752 \text{ kNm} \\ L &= 15.8 \text{ m} \\ q &= 138 \text{ kN/m} \end{aligned}$$

Therefore 138 kN/m is the decisive wind load on foundation on axis A.

In reality, the beams further away from the edges of the wall would get less load than that calculated, however, the equal load was used for the design of the beams for safety purposes.

5.4.2. Update: Increased loads on columns Axis A

Because of the reason that the final foundation design considers an additional “staircase area”, the columns on axis A get additional load. The increase includes the self-weight of the slabs, concrete staircases and landing staged, where applicable, in addition to area loads within the building.

The following calculations describe the derivation of loads, which are:

Staircases: 630 kN per staircase (four floors)

Landing stages twice per floor: 25 kN/m²

Area loads considered:

G, k = 3.3 kN/m²

Q, k = 5 kN/m²

The increased column loads from C9 to C13 (Axis B) were too high in order to design the pile group of four piles without increasing their depth to more than 20 metres, which was intended to be avoided. Therefore, also considering the changed loading plans for Axis A, it was decided to combine the two columns into a pile group of 6 piles connected via a single cap. The design of a pile cap with 6 piles provides a possibility for an easy connection with the intermediate beams, in terms of overlapping of the reinforcement layers.

Following the updated forces on the foundation, it was anticipated to have two types (cases) of pile groups for columns C1 to C14 (case 8 and case 9 in the calculation respectively). However, after determining the size of the pile cap considering the required minimum distance between two adjacent piles, it was noticed that most of the walls perpendicular to the gridline of axis A are standing on the pile caps instead of the shallow foundation. Because of this reason, an influence on the group of piles was checked and the piles were designed accordingly. Appendix provides more information about this issue.

In the end, all piles on axes A and B were chosen to be 10m in length, grouped by 6 and all assigned to Case 8.

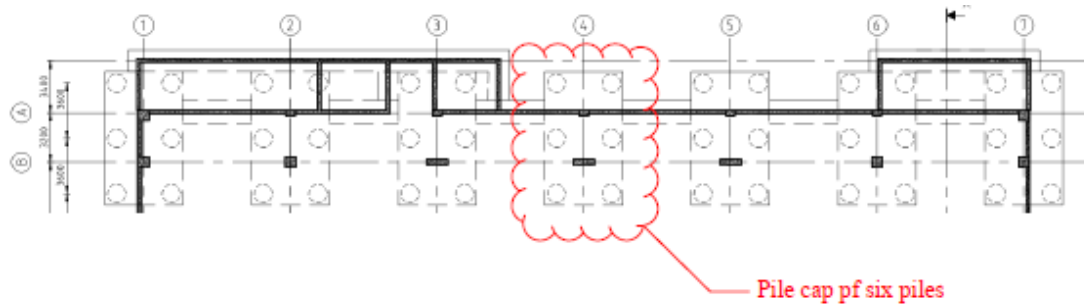


Figure 5-17 Indication of the 6-pile cap on axes A, B

5.4.3. Pile cap design Case 8

Pile cap material summary:

Concrete class:	C30/37
Steel grade:	B500A
Concrete cover:	50 mm
Exposure class:	XC2

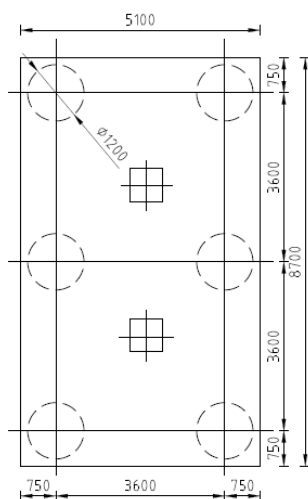


Figure 5-18 6-pile cap dimensions

Dimensions:

Pile cap thickness	=	1.80 m
Area	=	5.1 x 8.7 m

The pile cap for six piles was calculated in the same manner as those for four piles.

In this design however, the two types of beams considered were:

Beam 1 - of single span
(between two piles)
 $G, k = 3,975 / 2 = 1988 \text{ kN}$
 $Q, k = 3,358 / 2 = 1680 \text{ kN}$
Half of the decisive column load (C9/C13)

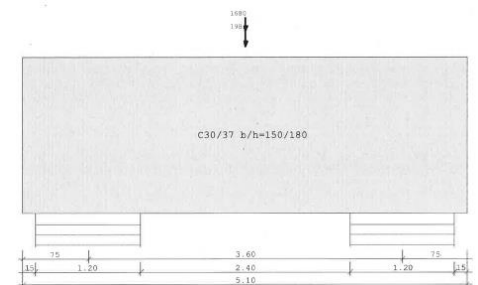


Figure 5-19 Beam 1 of the 6-pile cap

Beam 2 - double span (between three piles)
 $G, k \text{ (span 1)} = 3,975 / 2 = 1988 \text{ kN}$
 $Q, k \text{ (span 1)} = 3,358 / 2 = 1680 \text{ kN}$
 $G, k \text{ (span 2)} = 2,580 / 2 = 1290 \text{ kN}$
 $Q, k \text{ (span 2)} = 1,910 / 2 = 955 \text{ kN}$
Half of the decisive column load (C2/C6)

Reinforcement design:

Side of two piles:
 Bottom – 2 x 12Ø20
 Top – 12Ø20
 Shear – Ø12-150 (4 legs per cap, 2 legs per “beam”)

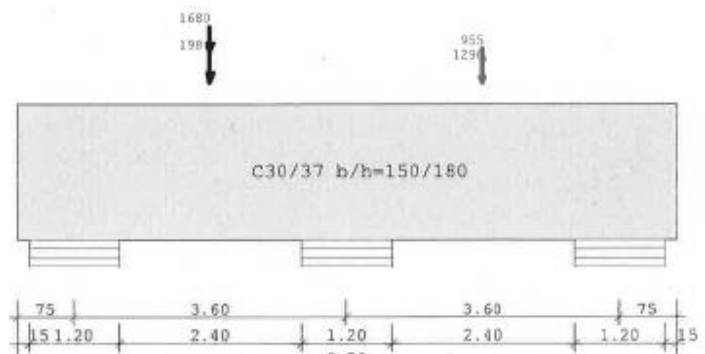


Figure 5-20 Beam 2 of the 6-pile cap

Side of three piles:

Bottom – 2 x 10Ø20 (between piles)

Top – 2 x 10Ø20 (above mid-pile)

Shear – Ø12-150 (6 legs per cap, 2 legs per “beam”)

Higher moment of the two spans was considered for the reinforcement amount, a second layer of bars is planned at the peak of the moments (at the bottom between the piles and at the top over the middle pile).

5.4.4. Integrated beam design

Two types of intermediate beams were distinguished (Figure 5-21):

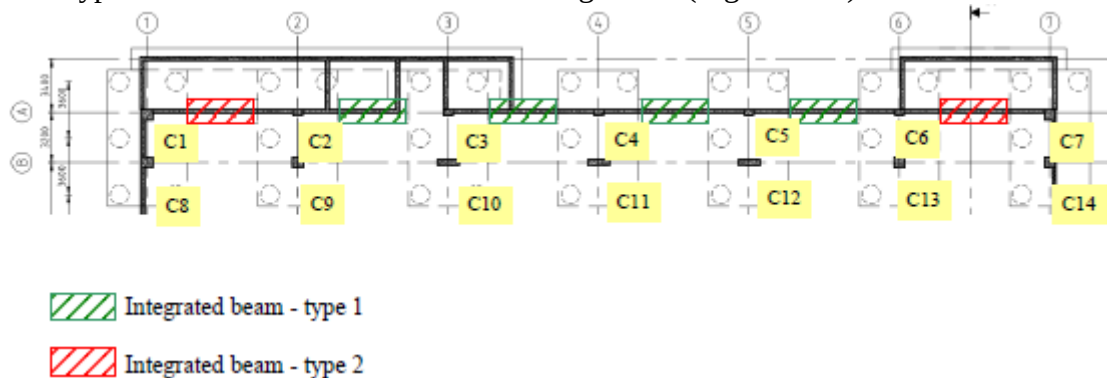


Figure 5-21 Identifying the integrated beam types 1 and 2

The difference between the beams is recognised due to the reason that the walls between columns C1 and C2 as well as between C6 and C7 are load bearing and in addition to the wind forces, carry also the area loads as well as the slab self-weight from within the building as well as the area loads and staircase self-weight from the “staircase area”. These loads are transferred by the intermediate beams (Type 2) to the pile caps.

Whereas beam Type 1 carries only the wind forces and the self-weight of walls. The load is significantly less, therefore it was decided to use less reinforcement for these “inner” beams as well.

Important note: the reinforcement for the beams was designed in a way that the structure can easily be connected with the pile cap. Because in the pile cap Ø20 bars were used, the beam design is based on this size of bars as well. The width of 1.5m for the beam was chosen in order to make sure that the beam fits in between the two piles and the reinforcement can be placed without any problems. The height of 1.8m was kept the same as that for the pile cap.

5.4.4.1. Integrated beam design – Type 1

Material properties:

Concrete class:	C30/37
Steel grade:	B500A
Concrete cover:	50 mm
Exposure class:	XC2

Dimensions:

Height	=	1.8 m
Width	=	1.5 m

Effective length = 6 m

Loads:

G, k (wall weight) = 165,3 kN/m

Q, k (wind) = 138 kN/m

Reinforcement design:

Top: 10Ø20

Bottom: 10Ø20

Stirrups: Ø12-150

Side bars specified in the drawing.

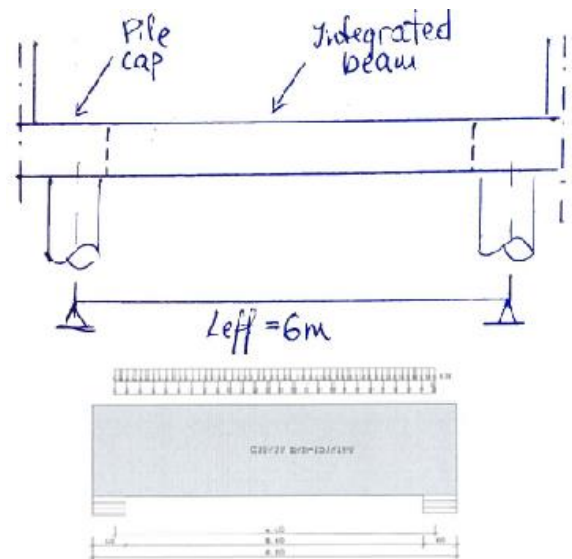


Figure 5-22 Beam Type 1 span characteristics and loading

5.4.4.2. Integrated beam design – Type 2

Material properties:

Concrete class: C30/37

Steel grade: B500A

Concrete cover: 50 mm

Exposure class: XC2

Dimensions:

Beam height = 1.8 m

Beam width = 1.5 m

Effective length = 6 m

Loads (calculation described in the appendix):

G, k total = 378.2 kN/m

Q, k total = 246.5 kN/m

Reinforcement design:

Top: 10Ø20

Bottom: 19Ø20 required, 20Ø20 chosen to have two equal layers

Stirrups: Ø12-150

Side bars specified in the drawing.

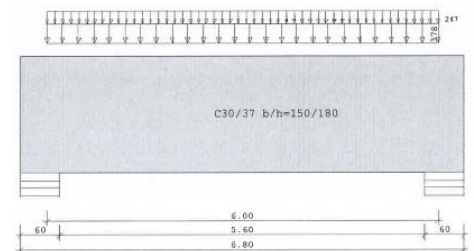


Figure 5-23 Beam Type 2 - span characteristics and loading

5.4.5. Shallow foundation design

The shallow strip foundation was designed under the 0.25m walls from the “staircase area” (Figure 5-24). No pile were necessary there due to the fact that the walls do not carry high loads and shallow foundation is sufficient.

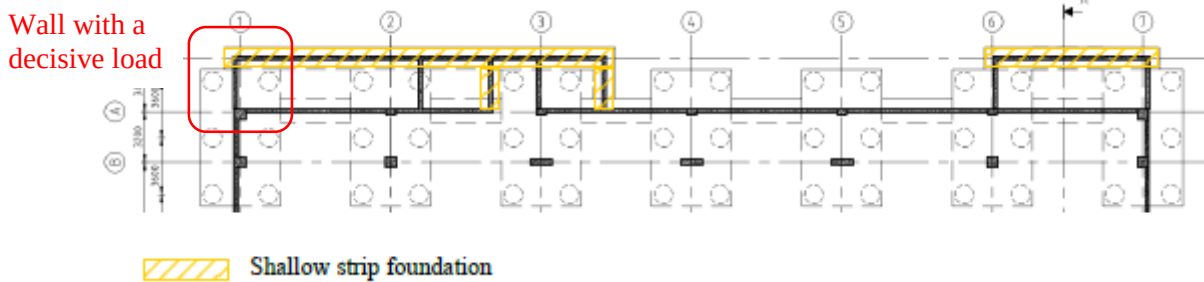


Figure 5-24 Shallow foundation "staircase area"

The necessary design bearing capacity (anticipated after soil exchange) was provided in the geotechnical report. The value is applicable when the bottom of the strip foundation is at -1 metre below the ground level.

$$\sigma_{R,d} \text{ (design bearing capacity)} = 415 \text{ kN/m}^2$$

Decisive load from walls on the foundation (calculations explained in the appendix):

$$G, \text{ k} = 310 \text{ kN/m}$$

$$Q, \text{ k} = 34 \text{ kN/m}$$

Material properties:

Concrete class	C30/37
Steel grade	B500A
Exposure class	XC2

Dimensions:

$$\text{Strip width} = 1.4 \text{ m}$$

$$\text{Strip height} = 0.6 \text{ m}$$

Min. reinforcement at the bottom:

$$\varnothing 10-100$$

Stirrups: $\varnothing 8-100$

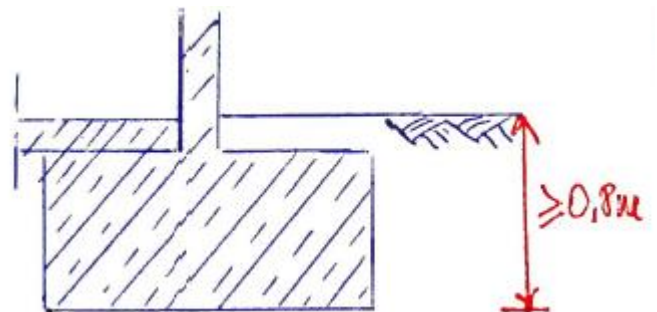


Figure 5-25 Shallow foundation sketch, indication of minimum footing depth for frost protection

5.5. Foundation design - Part 3

The last part includes the design for an elastically bedded foundation slab over the entire building area, subject to area loads from the ground floor of the building. The calculations include the design for minimum reinforcement amount, sliding resistance check, temperature differences expected with regards to a working data centre, as well as floor deflection check.

Material properties:

Concrete class	C30/37
Steel grade	B500A
Crack width bottom of slab	0.2 mm
Crack width top of slab	0.3 mm
<i>Values for crack widths based on a similar project</i>	
Concrete cover (considering a binding concrete layer under the slab)	35mm

Dimensions:

Thickness of slab

0.35 m

At the edge of the building, frost protection provided up to 0.8m (Figure 5-26)

Effective span for deflection check

6 m

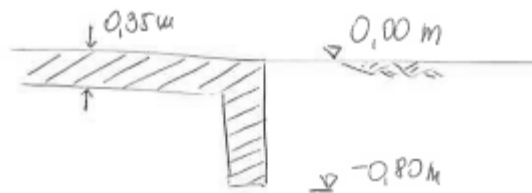


Figure 5-26 Foundation slab sketch - required frost protection depth at the edge of the building

Area loads:

G, k

 $= 1.8 \text{ kN/m}^2$

Q, k

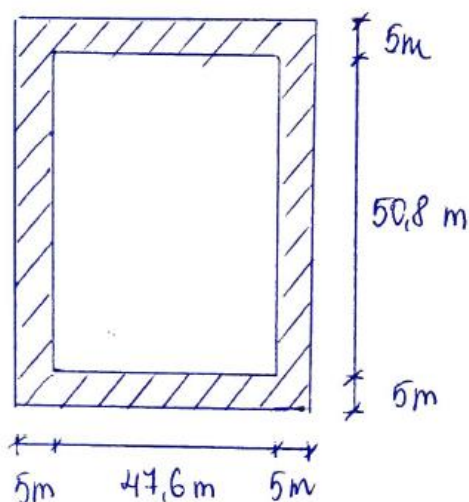
 $= 15 \text{ kN/m}^2$ Reinforcement design:

Reinforcement for crack control at the bottom

Ø12-80 per layers

Reinforcement for crack control at the top

Ø12-100 per layer

5.5.1. Foundation slab - sliding resistance check

5m away from every edge of the slab is the insulation layer under the slab, where no friction is considered. The remaining area has a gravel layer underneath to provide friction against sliding.

The vertical loads on the slab were not considered as a safer approach.

Horizontal load in Y-direction (decisive) = 4,974 kN

Sliding resistance $R_h, d = 6,732.15 \text{ kN}$

$4,974 \text{ kN} < 6,732.15 \text{ kN} \rightarrow$ Sliding resistance sufficient

Figure 5-27 Sliding resistance check foundation slab;

5.5.2. Foundation slab – deflection check

The slab is elastically bedded on soil, however, if the soil does not provide sufficient stiffness, some deflections may still occur. A check was made assuming a single span beam with effective length of 6 metres subject to deflection (Figure 5-28).

$$f_{max} = \frac{5}{384} * \frac{ql^4}{EI}$$

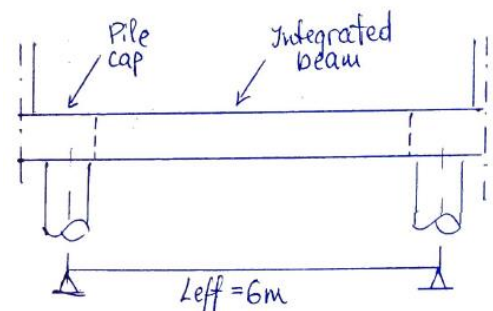


Figure 5-28 Effective span for deflection check

$$E \text{ (cracked concrete)} = 8,000 \text{ N/mm}^2$$

$$I = 1,000 \text{ mm} \times 350 \text{ mm}^3 \times 1/12 = 3.6 \times 10^9 \text{ mm}^4$$

$$f_{max} = \frac{5}{384} \times \frac{16.8 \times 6000^4}{8000 \times 3.6 \times 10^9} \cong 10 \text{ mm}$$

$$\text{Allowed deflection: } L / 250 = 6000 / 250 = 24 \text{ mm}$$

24 mm > 10 mm → sufficient, deflection not exceeded

5.5.3. Reinforced concrete crack control

In the geotechnical report a concern was described about a possible rise of the water level up to the ground surface in an intensive rain event. Due to this reason, it was checked if the reinforcement amount is enough to resist the cracking from the strain, induced by the temperature differences, in order to prevent the creation of cracks and allowing for the water to pass inside the building.

Temperatures for checking the induced strain inside the slab:

Soil temperature (assumed) = 10°C

Maximum temperature inside a working data centre = 38°C

$$f_{ctm} = 2,900 \text{ GPa}$$

$$E_{cm} = 33 \text{ GPa}$$

$$\text{Max concrete strain } \epsilon_{ctu} \text{ (calc. provided in the appendix)} = 108 \text{ } \mu\epsilon$$

$$\text{Induced strain } \Delta\epsilon_r \text{ (restrained)} = 50.23 \text{ } \mu\epsilon$$

If $\Delta\epsilon_r / \epsilon_{ctu} < 1$ → risk of cracking is considered as low.

$$50.23 \text{ } \mu\epsilon / 108 \text{ } \mu\epsilon = 0.47 \rightarrow \text{concrete strength sufficient.}$$

Amount of reinforcement does not have to be increased from that calculated. However, movement joints to be provided for thermal movements at the connections around columns. Appropriate precautions to be taken when pouring the concrete in order to avoid structure cracking – the use of cooling measures or reduced time between castings to keep the temperature differences to minimum.

In the “Appendix F – Final design” a table “Final foundation design summary” is presented. The content of the table is to be considered as the final decisions made regarding the foundation design.

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6. Discussion

The approach for the concept design of the pile foundation in Germany can be considered as rather conservative, as it usually includes the self-weight of the pile. However, because the unit weight of concrete is not much higher than that of soil, in addition to not taking into account the positive effect of soil overburden, the weight of the pile could be easily neglected.

When performing the calculation for the final design, this was indeed the case in addition to checking the piles against compressive and tensile failure. The latter was indecisive as well as the buckling risk, therefore only the minimum amount of reinforcement according to DIN EN 1536:2015-10 was considered.

For the final design a few optimizations could be made as well.

First of all, even though it is a common practice in Germany to design a pile foundation based on empirical correlations (or experience from previous practice), the design could be optimized if *in-situ* testing of piles was performed. Even though the tests are rather expensive, a confident increase in pile resistance (by lowering the partial safety factors γ_b and γ_s from 1.4 to 1.1) would result in shortened length, allowing the reduction of the overall budget. Where the foundation includes a large amount of piles, the *in-situ* testing could definitely become beneficial.

For simplification purposes, all piles subject to column loads were grouped by an even amount (even four or six piles), in order to carry out the calculation for a rectangular pile cap. However, in some cases, where the vertical loads are not too high, a smaller amount of longer piles would be sufficient. If this change was adopted, the duration of the foundation implementation could be reduced, because placing one pile of larger length requires less time than placing two accordingly shorter ones.

Nevertheless, the lack of detail from the geotechnical report about the soil parameters in the study area lead to an incomplete design. Without any information about the properties of the soil after the exchange (except the anticipated soil bearing resistance values for the shallow foundation design), most of the calculations were carried out in accordance to the current situation and must therefore be revised once the updated geotechnical values are obtained.

In the description of the clay layer, no information was provided about either the degree of consolidation or the history of stresses. Regarding the influence of the possible down-drag of the pile, only the difference between the current effective stress and the possible stress in the future after the soil exchange was calculated. It resulted in almost the same value, as there was no information stated about the desired unit weight of the compacted sand. Assuming that the clay layer is over-consolidated and the increase of the stresses is not high, the effect of the down-drag was not considered in the final design.

For the purpose of this thesis, the variant study on the foundation techniques contained several important aspects for selecting the desired solution and implement the final design. Those criteria were: the duration and costs, as a requirement from *the Client*, as well as the overview of the possible risks from the structural perspective, which were analysed by performing the preliminary designs of each variant. Even though the presence of the contaminated soil in the building site is the main problem for the project, an alternative design could be proposed, if the increased duration of implementation was allowed.

Having the necessary time and budget, the soil in the area could be improved by carrying out the Deep Soil Mixing with cement or compound binders, which is widely used for increasing the stiffness of weak soils. As a result, the amount of settlements caused by the raft foundation would

be reduced and the design would become the preferred one, as its implementation is less complex than the construction of pile groups.

Further investigation into the possibility of applying this alternative to the project is however necessary as it was not within the scope of the thesis.

7. Conclusion

The focus of the thesis, for which this report serves as a final product, was the detailed structural design of the foundation of a data centre in Frankfurt am Main, Germany.

The issue of contaminated soil in the area led to the increase of implementation duration, as vast amounts of soil needed to be excavated and exchanged in order to start the construction of the new data centre. Due to the lack of responsibility from the previous owner to take care of the soil remediation activities on time as well as the failed collaboration between them and the current owner (*the Client*), the project was suffering from at least one year of delay. In order to solve this problem, three different foundation techniques (these included a raft foundation, the large diameter mono-piles as well as a conventional pile group design) in two different locations were considered for finding the most suitable solution.

The analysis of the six variants was done using a selective method. After preparing the preliminary designs for all the three variants, which were based on the national construction regulations as well as the common practice, two of the solutions were ruled out. The raft foundation was not structurally viable due to the large settlements (if considering the provided soil parameters) and the mono-piles in the new location were declined, being considered as a conservative solution. The remaining options were compared and evaluated based on the two criteria – duration and cost. A very important requirement included the duration, limited to no more than 12 months (considering the soil remediation activities, as well as the construction of the foundation and the superstructure). The duration was therefore one of the main selection criteria and was analysed for all the feasible variants. It was important to understand the specifications of the foundation design and its installation and apply the known and gained knowledge for the optimisation of the design.

The mono-piles in the initial location as well as the pile groups in the altered location were both suitable regarding the implementation duration, therefore they were compared by direct costs as a governing criteria. Foundation of pile groups was a cheaper choice than mono-piles with 1,073,280 euros versus 1,355,640 euros. In addition to being the more expensive solution, large diameter mono-piles, even though permitting the solution for the overlapping of construction works, come with the high risks for the stability of the structure and the safety of the people working on site, as well as the uncommon design requiring special equipment. Therefore, all criteria considered, the final decision for the project of the data centre design, was to relocate the building, minimizing the amount of polluted soil from 40 992 m³ to 17 165 m³ and thus reducing the time for soil remediation activities in half - from approximately 28 to 13 weeks - as well as improving the accessibility of the construction site.

The foundation design of bored pile groups was chosen as the most time- and cost-efficient solution. The advantage of the quick pile installation with the continuous flight auger technique, as well as lower preliminary direct costs were the evidence for the decision made.

Even though the thesis topic did not provide much room for innovation, the problem analysed in the report can be considered as highly relative to the work field of Civil Engineering. The skill of a civil engineer to have the critical understanding of the contractual agreements, the application of the complex geotechnical design specifications as well as the ability to produce feasible and optimal structural designs, when applied in practice, can have a difference between a failed and a successful project.

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