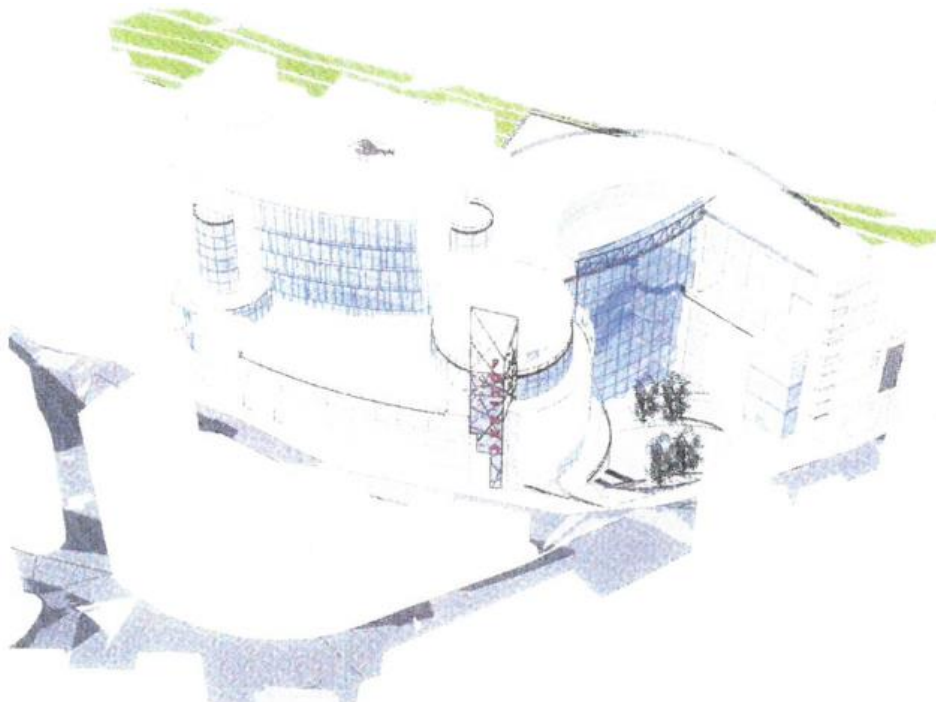
Multipurpose Centre, underground garage
and spaces with complementary functions
6 Haşdeu str., Cluj-Napoca, Romania

STRENGTH
Technical report
DBP phase

September 24, 2007



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TECHNICAL REPORT
STRENGTH

1. INTRODUCTION

1.1 General aspects

The present project deals, at a building permit draft level, with aspects regarding the load bearing structure of the objective entitled "'Arkadia' Multipurpose Centre" in Cluj Napoca, at 6 Haşdeu str.

This document describes the solution selected for the load bearing structure. The following matters are expanded upon:

• Description of the structure and its interface with the architecture
• Performance criteria and client requirements
• Description of the superstructure and the infrastructure
• Design loads and actions
• Proposed execution technology
• Delivery schedule

1.2 Standards and legislation:

The present draft is in compliance with the standards and regulations listed below:

1. Occupational health and safety general rules NGPM-2002
2. Fire prevention and suppression general rules, approved as per Minister of the Interior's Order no. 775 from 22.07.1998
3. STAS 2561/1-83: Foundation soil. Piles. Classification and terminology
4. STAS2561/2-81: Foundation soil. Piles. Pile testing
5. C 160-75: Technical instructions for executing piles
6. GE 029-97: Practical guide for executing piles beneath foundations
7. NE 012-99: Practical guide for executing concrete and reinforced concrete works.
8. CR 0-2005: Design code. Basic notions on the design of structures in constructions.

9. CR 1-1-3/2005: Design code. Assessing the effects of snow on constructions
10. P 100 — 1 / 2006: Seismic design code
11. NP — 082-04: Design code. Basic notion on design and actions upon constructions. Wind action.
12. CR 2.1-1-1/2004: Design code for structures with reinforced concrete walls.
13. STAS 10107/0-90: Design of concrete, reinforced concrete and pre-stressed concrete elements.
14. SP 009/2005: Technical specification on performance requirements and criteria for concrete reinforcements.
15. Law 10/1995 on quality in constructions.

1.3 Inclusion into classes and categories

According to "Seismic design code – Part I: Design provisions for buildings P100/1 – 2006", the site, namely Cluj-Napoca city, is characterised by $a_g = 0.08g$ and $T_c = 0.70s$. The building falls into importance and earthquake exposure class II, for which y = 1.2.

In accordance with GD 766/97, Annex 3 and GO 261/1994 "Regulation on determining the importance category of constructions", the present construction falls into importance category C (a building of normal importance). For this type of building, a technical inspection of the project is necessary as per requirement A (strength and stability), B (operational safety), C (fire safety), D (hygiene and environmental protection), E (thermal and waterproof protection of the building) and F (noise protection), in compliance with the technical design inspection rules, approved as per M.L.P.A.T (Minister of Public Works and Spatial Planning) Order no. 77N/28.10.1996.

1.4 Current site description

The land that the buildings are to be located across has a nearly triangular shape and lies on a sloped area north of Cluj city. The area of the land is approx. 6,980 sq m. The ground elevation varies from around 378 m above the sea level to around 369 m. The land is bordered by Victor Babeş str. to the east and a nameless street to the south. The western limit is marked by an internal road to the neighbouring hospital, whereas the northern side by a steep slope and two low-height buildings towards Victor Babeş street.

1.5 Proposed building

The new building is a premises with complex functions, including commercial and leisure areas, restaurants, office spaces, a medical clinic and a parking lot. The total built area is approx. 42.000 sq m.

The proposed building has 10 levels. Two of these levels (basements -2 and -1) are exclusively intended for the underground parking lot, access ramps and technical areas. Levels SB2 and SB1 are partially buried due to the land slope, but do reach the ground level along the eastern side of the site in order to form the entrance into the building. These levels mainly comprise commercial areas, traffic areas and entrance areas.

The upper levels host commercial areas, offices, restaurants and leisure areas. Above the ground level, the building footprint becomes much smaller and limited within its actual perimeter. On the roof it is possible to fit a heliport. The decision is to be confirmed by the client.

The building footprint is polygonal and follows closely the land limit. Above the ground level, the building footprint withdraws on all sides to make room for a small plaza and the main entrance. Inside the building, the entrance area consist in an atrium designed over the entire construction height, up to the roof, boasting a peripheral braced girder supporting a roof and a frontage, both made of glass.

Fig. 1. Aerial view of the proposed building

2. GEOTECHNICAL AND ENVIRONMENTAL CONSIDERATIONS

2.1 Geotechnical investigations

A preliminary geotechnical report was drawn up in July 2006 by prof. dr. eng. Augustin Popa. The report contains the result of the contamination tests and a basic land profile.
However, the information contained in this report is not sufficient for the detailed design of foundations, which prompted a request for a series of additional drillings and investigations on the land. At the time of drawing up this report, the results of the new series of tests were not yet available.

2.2 Foundation-laying conditions

The general conditions that are likely to be encountered on site, relying in that respect on the preliminary geotechnical report, require laying the foundation within a layer of shingle and sand on top of a layer of clay, which becomes harder with depth. It is well known that the preliminary investigations revealed numerous freestone boulders of various sizes. In certain cases such boulders could be drilled through, while in other cases the drilling had to be ended because of boulder sizes and densities that made it impossible to continue. This aspect has to be taken into account by the designer of the piles during the DE (Detailed Execution Plan) phase.

2.3 Underground water

The water level occurs during drillings at 4-5 m below ground level. It is not yet clear whether there is a significant hydraulic gradient along the land, which would indicate a major water flow. Still, the possibility that this construction would interrupt the water flow across the land must be considered as a potential risk. One possible solution would be installing a perimeter draining system, at depth, around the building, intended to collect water at the upper end of the land and releasing it into a sewage system at the lower end of the land.

2.4 Environment

The report does not clearly state whether there is any contamination. However, there may be contamination around the chlorination tanks on the north-eastern side of the land.

2.5 Underground networks

A plan of the site underground networks is known to exist, but it has not been made available yet.

3. STRUCTURAL CONCEPT

3.1 Transmission of vertical loads

For most of the structure, vertical loads are discharged straight into the foundation, as the poles are vertically continuous from the roof to the foundation. However, the uppermost level requires transfer beams in order to change the position of certain poles within the building. Given the seismic design conditions, it is preferable to discard all transfer structures inside the building as they are particularly vulnerable to seismic section, with the exception of the uppermost level poles, for which 2nd degree bearings are allowed.

3.2 Lateral stability

The stability of the building against the action of wind- and earthquake-generated horizontal forces is secured with the provision of reinforced concrete load-bearing walls, in the areas of elevators, staircases or other adequate locations. The horizontal forces within the superstructure are transferred through floors to these load-bearing walls and the infrastructure levels.

Further on, these horizontal forces are transmitted, through the infrastructure floors, to the enclosure walls, where they are taken over by the passive pushing of the soil. However, due to the land slope, this effect shall only be fully deployed below the 367.5-m depth, where the entire construction is in line with or below the ground level.

The load-bearing walls may sometimes be interrupted below ground level, however, in this case it is recommended to have them continue up to foundations level so as to be in line with the seismic design regulations. Due to the building separation joint, these stability systems need to be duplicated so as to exist on both sides of the joint, thus becoming two separate structures. A well-distributed system of load-bearing walls also ensures a much safer building process. These concepts are visually described in the diagrams below.

Due to the less settled nature of the soil at ground level, in comparison with the deeper soil, the earth over the height of one level below ground level has been neglected in terms of passive resistance that should balance the horizontal forces within the load-bearing walls. Nevertheless, below this level one should reasonably rely on the passive resistance of the compacted soil in order to generate a horizontal block.

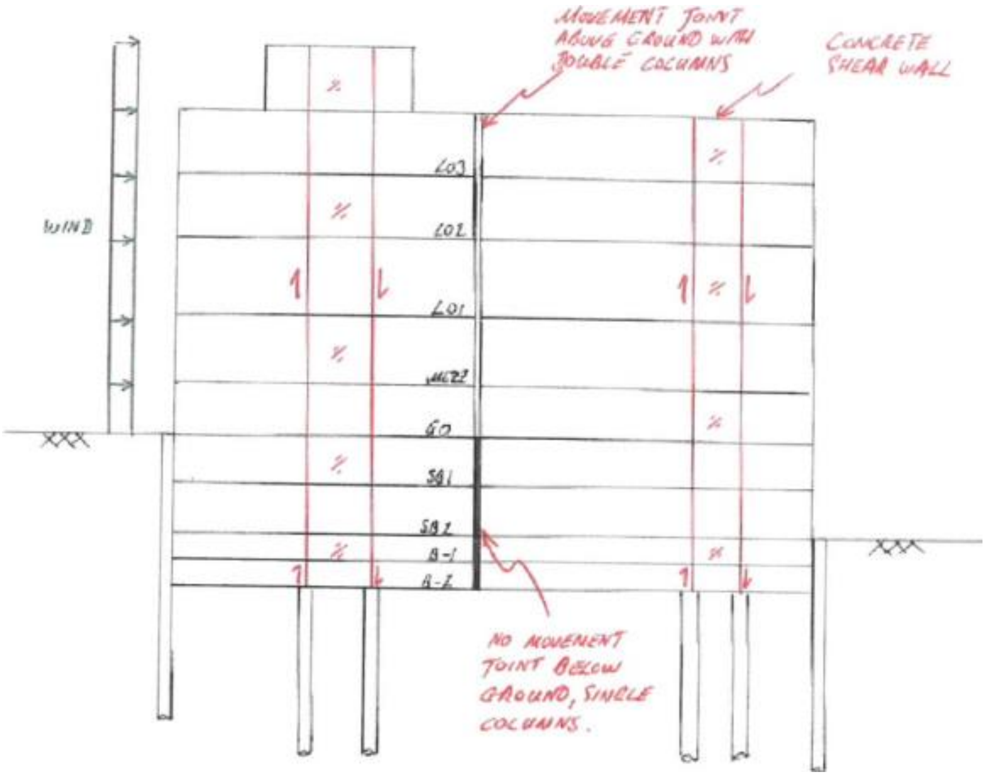

Figure 2. Stability concept for the structure.

3.3 Displacements

The main causes of displacements that have to be taken into account are temperature variations, wind action, differentiated settlements and concrete contraction effects. In this case, one must also take into account displacements induced by seismic actions. Land shifts caused by the execution of the piles and the laying of the foundation are discussed in the infrastructure section of this report.

In regard to displacements triggered by temperature variations, the total in plane dimensions of the building are approximately 138 m x 78 m. With a typical design value of ±20 °C for the temperature variation and under the assumption that the building has a central separation joint, the total dilation/contraction movement of the building generated by temperature variations of this magnitude may be around ±20 mm.

The horizontal displacements of the building under the wind action shall be limited to less than the building height/1000 and less than the storey height/300 for each storey. Given the building total height, from ground level to the roof, of 40 metres, the maximum wind-generated displacement shall be limited to 40 mm.

A displacement joint is sometimes used when o taller part of the building is adjacent to a lower part. This may lead to differentiated land settlements between the taller part and the lower one. They can also occur when the foundation-laying conditions across the built-up area vary. As far as our building is concerned, this effect can be avoided by designing the infrastructure so that it incurs even settlements across its entire area.

The effects of contraction within the concrete slabs may be significant for a building of this size. Long-term contraction for a building with a central joint will be around -12 mm on each side of the joint. Half of this deformation may take place over the execution period. This deformation needs to be taken into account with regard to in-joint displacement and the stresses within the slab.

Displacements due to seismic action may be significant and will determine the final width of the joint. A 3D calculation of the structure will be required in order to determine the length of these displacements and set forth the final width of the joint.

The considerable pushes within and underneath the plates, exerted by the enclosure walls, create a situation where, at these lower levels, the floors will no longer be able to relatively move and a joint is no longer needed below the 372.5-m depth. Moreover, it is beneficial to have a continuous floor in order to transfer the forces across the entire structure and have the forces within the enclosure walls on opposing sides of the building balance among themselves. Where the building is partially buried, it is, nevertheless, presumed that these forces are transferred to the perimeter enclosure wall of the building.

The known displacements (including a preliminary analysis of the seismic displacements) shall be combined, the result being a displacement range of +40mm to -32mm. The joint width required to take over dilations from both sides shall therefore be around 100mm. However, this width shall have to be revised during the project DE phase.

These +40mm to -32mm total displacements (excluding the seismic displacement for the frontage design) have to be considered when designing closing systems, particularly between the floor at the ground floor and the first level, where such displacements are more significant. In the worst possible case, these displacements shall take the form of horizontal translations of +10mm or -16mm at each extremity of the building, with gradually shorter displacements towards the centre of the building. In reality, however, the displacements may be shorter due to the hindrance posed by the load-bearing walls distributed throughout the building. These aspects may be further defined during the DE phase.

This separation joint must be integrated with the closing finishes and the roof. A difficult task when dealing with a 100-mm joint. A typical joint detailed view, at the roof level, is presented below.

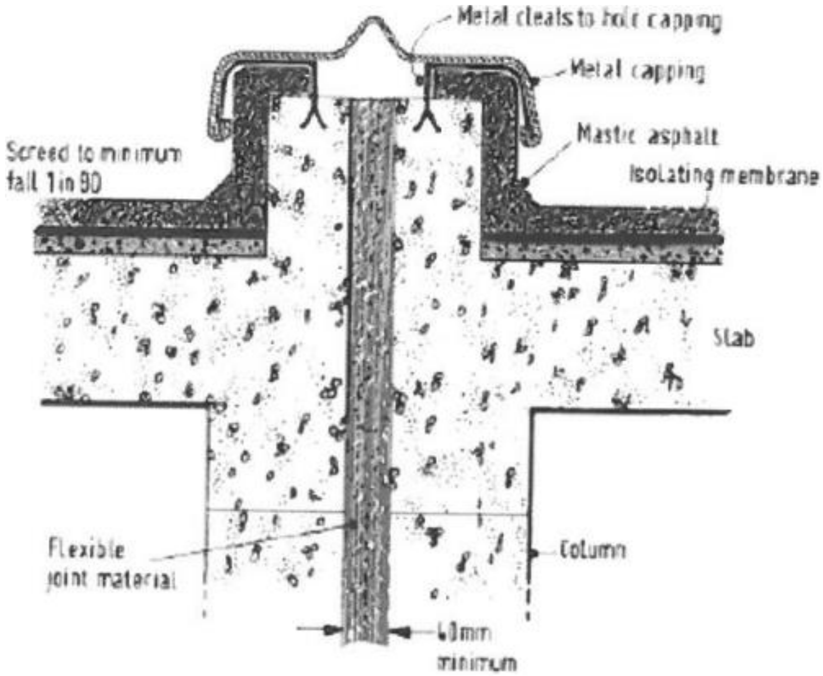

Figure 3. Typical joint detail view at the roof level.

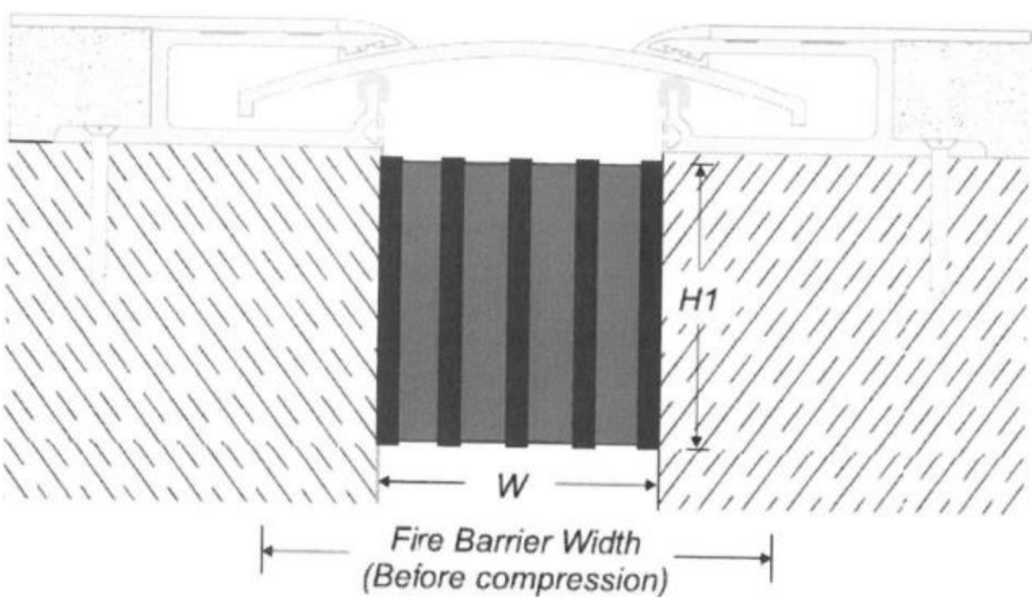

Figure 4. Typical joint closure detail view

3.4 Structural sturdiness and integrity

In order to avoid progressive failure, the building has to be fastened at the level of each floor, meaning that any pole has to be held in position by metal girders or a two-way concrete plate. In this case, the poles are almost entirely continuous from the roof to the foundation, making sturdiness inherent in the project.

3.5 Vibrations

The plate shall be designed in accordance with the latest provisions on vibrations. The client, the designers and the owners shall specify if there are items of equipment particularly sensitive to vibrations. It is expected to have certain vibration-generating equipment, e.g. coolers, other technical equipment and the heliport on the roof. Where possible, the equipment shall be installed on vibration-dampening frames. Should the heliport have to be completely isolated from the structure by means of vibration-dampening frames, this has to be reflected in the cost plan.

The acceptability criterion for vibrations is generally considered the multiple of the lowest humanly perceptible vibration acceleration. This is, in terms of “Response factor”, the lowest perceptible vibration (0.005 m/s_2). The proposed design is for a response factor equal to 8, corresponding to an Office environment (0.04 m/s2). There were no other requirements from the client or the users in regard to vibrations.

Type of office	Response factor, *R*
General office	8
Special office	4
Busy office	12

Table 3.1. Recommended criterion for vibrations as a multiple of base acceleration

3.6 Service life

The main structure shall be designed in accordance with the Romanian Execution Codes and the National Standards in order to reach a service life suitable for the building intended purpose and the client’s requirements. It is presumed that the architect shall also take into account the service life of other building elements, such as finishes and insulations, known to have a shorter service life.

The concrete structure inside the building has low exposure conditions. The concrete that comes into contact with the soil shall be selected taking into account the chemical properties of the soil, which is moderately sulphurous. For the concrete ramps there shall be no special requirements in terms of durability in relation to abrasions and antifreeze salts brought along by motor vehicles.

The technical rooms or the stack-type structures at the roof level shall be fitted with accesses for future maintenance works and must additionally meet special durability requirements. All the metallic elements used within the structure shall be galvanised in order to minimise maintenance.

The elements used to fix exterior components into place, such as curtain walls, etc.., shall meet general corrosion requirements, as well as requirements on electrolytic corrosion induced by various metals.

3.7 Fire protection

An engineer in charge with fire safety and design has been designated by the client to advise on general requirements. A fireproofing assessment is necessary both for the superstructure and the infrastructure, the locations of service staircases or service elevators, and to meet ventilation requirements regarding basement smoke clearance, etc.

If there are rooms hosting transformer stations at the basement level, these shall meet additional fireproofing requirements.

3.8 Tolerances

The execution tolerances require particular attention during the DE design phase. The main issues that have to be considered are the tolerances of buried profiles and of piles, the marginal areas between the capping and the plate. The typical tolerances that are likely to be reached on the construction site must be mentioned from the very beginning by the contractor and taken into account in the DE phase calculations by the architect and the structural engineers.

3.9. Earthquake-related considerations

The seismic activity in Romania is mainly generated in the SE part of the Carpathian Mountains, in Vrancea region. An unusual seismic intensity (an average frequency of 3 earthquakes with a magnitude in excess of 7 on the Richter scale per century) originates here, caused by a source at an intermediate depth (within the 60-200Km approximate range), across an area of nearly vertical subduction of the tectonic plates. This source releases more than 95% of the country's seismic energy, affecting around more than half the territory of Romania and generating high-intensity earthquakes.

Cluj Napoca, as seen on the historical seismic map below, is subject to surface earthquakes (depth lower than 60Km), taking the form of lower magnitude and more localised earthquakes, with calculation land accelerations below 0.08g and magnitudes rarely exceeding 6 on the Richter scale. A preliminary calculation of the seismic forces has been performed and it is considered that seismic loads do not require a significant increase of the pole cross-sections. Still, a 3D analysis must be conducted during the DE phase, to be used in order to formulate the final stability requirements for the building.

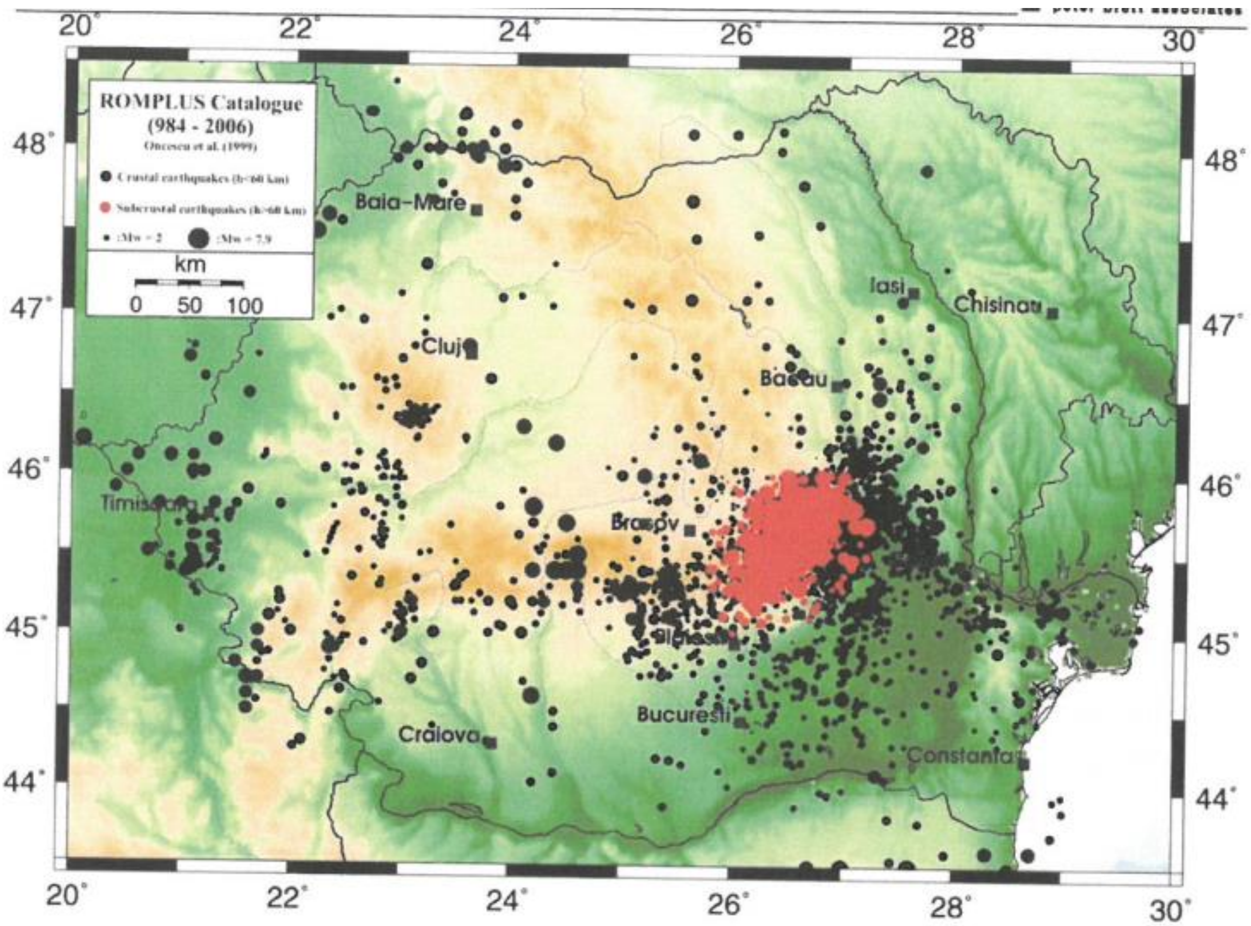

Figure 5. Historical map of earthquakes in Romania

4. SUPERSTRUCTURE

4.1 Design criteria

It is considered that, for a multipurpose building of this type and for this location, the main design criteria are as follows:

- Standard design criteria – the economic criterion, structural performance, durability, vibration resistance, etc.

- Integration of installations – a building with complex installations needs to have a structure that integrates efficiently with cu electrical, plumbing, heating and ventilation installations, leading to savings both for the said structure and the said installations, reducing the space required for those installations.

- Ease of construction – given the crowded nature of the site and the surroundings, and the large amounts of construction materials required by the construction site, one shall take into account the transportation of such materials, as well as the easiness with which - and speed at which -

they will be carried around by crane, the planning of the works and the construction site impact upon the neighbouring constructions and inhabitants.

• Architecture quality – the topic requires a "brand" image able to unify the highest levels of architecture and innovation.

• Basement construction – one shall consider the waterproofing degree, the environment, the construction method, the impact of soil movements upon adjacent buildings, etc.

• Seismic considerations – the loads and displacements caused by the seismic activity can generate restrictive design conditions. A 3D analysis is necessary in order to estimate the impact upon structural sizing.

• Planning of works – the schedule is dense, with 17 months of construction works, followed by 3 months of tests. This is a speedy schedule for a building the basement of which is so large that one needs to consider all the possible ways to expedite the works.

4.2 Estimation of design loads

In order to size up and verify the ultimate limit state and the limit state of normal operation, the following groups of loads were taken into account:

The combinations of loads were formulated as per Normative CR 0-2005, "Design code. Basic notions on the design of structures in constructions".

The fundamental group:

$$1{,}35\sum_{j=1}^{n} G_{k,j} + 1{,}5\, Q_{k,1} + \sum_{i=2}^{m} 1{,}5\, \psi_{0,i}\, Q_{k,i}$$

- $G_{k,i}$ is the effect upon the structure exerted by the permanent action *i*, considered with its characteristic value
- $Q_{k,i}$ – the effect upon the structure exerted by the variable action *i*, considered with its characteristic value
- $Q_{k,1}$ – the effect upon the structure exerted by the variable action, which has the predominant weight among the variable actions, considered with its characteristic value
- $\Psi_{0,i}$ is a factor of simultaneity of the effects upon the structure exerted by the variable actions *i* (*i*=2,3...m), considered with their characteristic values, having the value:

The special group:

$$\sum_{j=1}^{n} G_{k,j} + \gamma_I A_{Ek} + \sum_{i=1}^{m} \psi_{2,i}\, Q_{k,i}$$

- A_{Ek} is the characteristic value of the seismic action which corresponds to the average recurrence interval, *ARI*, adopted by the code (*ARI* = 100 years in P100-2006)
- $\Psi_{2,i}$ - coefficient used to determine the quasi-permanent value of the variable action Q_i, having the recommended values in the following table:

Action type	$\Psi_{2,i}$
Actions caused by wind and temperature variations	0
Actions caused by snow and due to operation	0.4
Loads within deposits	0.8

- γ_1 – importance coefficient of the construction/structure (=1,2)

Effective load

The national standards provide strictly indicative values in order to estimate loads. However, if the clients so wish, they can increase the long-term flexibility of the building by means of using higher design loads. They only apply to loads that do not originate from the weight of load-bearing elements and finishes.

There are areas where loads can only be determined by experts or designers – such as the heliport. PBAI shall assess in this document the design loads, to be subsequently acknowledged by the contractors and experts.

PBAI recommends the following design values for effective loads:

Table 4.1. Design loads recommended by PBAI

Intended purpose	Recommended by PBAI
Commercial areas	4.0 kN/m^2
Hospital	2.0 kN/m^2
Green areas	3.0 kN/m^2
Office spaces	2.0 kN/m^2
Lorry traffic areas	15 kN/m^2
Balconies, corridors	4.0 kN/m^2
WC	2.0 kN/m^2
Coffee shops	2.0 kN/m^2
Ground floor entrance	4.0 kN/m^2
Water tank	Depending on the tank dimensions

Wind load

According to normative NP-082-04 – “Wind loads” – the site is located in an area with a wind reference pressure gv = 0.4 kPa/m2.

Snow load (as per the normative)

According to design code CR 1-1-3-2005 – “Design code. Assessing the snow action upon constructions” – the site is located in an area with a characteristic value of the snow load on the ground s0k = 1.5 kPa/m2.

Earthquake loads

The seismic action value is taken into account as per P100-01/2006 – Seismic design code - Part I: Design provisions for buildings.

According to the zoning map, for the given site, the maximum land acceleration is ag =0.08g and the corner period TC = 0.7 s.

The structure type is “structure with reinforced concrete walls”; the dissipative effect of the coupling beams is negligible. Consequently, the base behaviour factor is considered q = 4.

Given that the structure does not display any monotonous lines along the horizontal or the vertical axis, the structural calculation is performed taking into account a behaviour factor q decreased by 30%.

Since the construction will host more than 400 persons, it falls into the 2_{nd} importance class and has γl = 1,2.

4.3 Load-bearing structure

Considering the high envelope costs, obtaining a floor with the lowest possible thickness is a significant aspect to remember for this building. Another major factor is the Works Plan. Due to the contracts concluded with the owners, it is unlikely to find flexibility in the execution of the building and, in order to meet the deadlines, a quick construction schedule has to be developed.

Steel prices in Romania are high, which would make a slab-type floor the preferable option. As such, as part of the preliminary structural calculation, a slab-type floor was considered. On earthquake-related grounds, it is beneficial to build the lightest possible structure, which will require one to examine a number of alternatives the result of which is an easier construction effort at a lower cost. The main alternatives are as follows:

- Option 1 – slab-type monolithic floor – easy to build, with the disadvantage of adding to the building weight;

- Option 2 – post-stressed slab-type monolithic floor, bringing the advantage of a slab, but being low-weight; unfortunately, this technology, as well as the required skilled manpower, are difficult to source in Romania;

- Option 3 – coffered monolithic floor – the result is a concrete floor that is considerably lighter, but requiring a slightly increased execution time. From a seismic standpoint, the weight reduction will have an impact on the structural dimensions of the pole walls.

Considering the above-listed options, preference leans towards the coffered floor.

The general diagram of the floor is provided below. The advantages of this alternative are:

- Good long-term flexibility of the installations area;

- Good characteristics provided by the continuous floor, in regard to deformation and vibrations;

- Easy execution in areas having gaps within the floor or cantilevered areas;

- Able to bear significant pushing forces at the basement level;

- It requires less coordination of the structure with the installations, as the areas are completely separate;

- Local availability of materials.

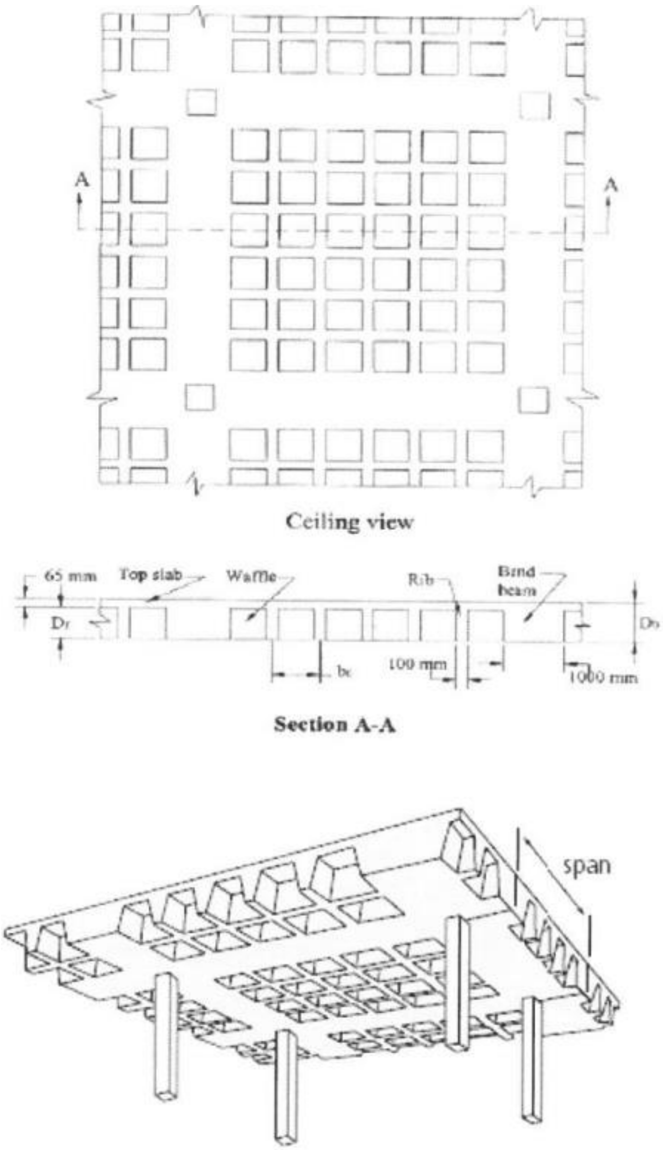

Figure 8. Plan and cross-section views for the proposed coffered floor

4.4 Floor and frame beams

The proposal is to use a coffered floor, without greater height beams, for most of the structure. However, in certain areas, greater height beams are necessary, for example, around the gaps, in areas with significant punctual loads or in order to transfer poles (this is only allowed at the highest storey level).

The ceiling and the service zone shall be located beneath the floor, over a height agreed upon with the client and the designers. It is recommended to have a not too restrictive service zone in order to give users flexibility for future changes. The general set-up is illustrated below (with the coffers omitted for greater drawing clarity).

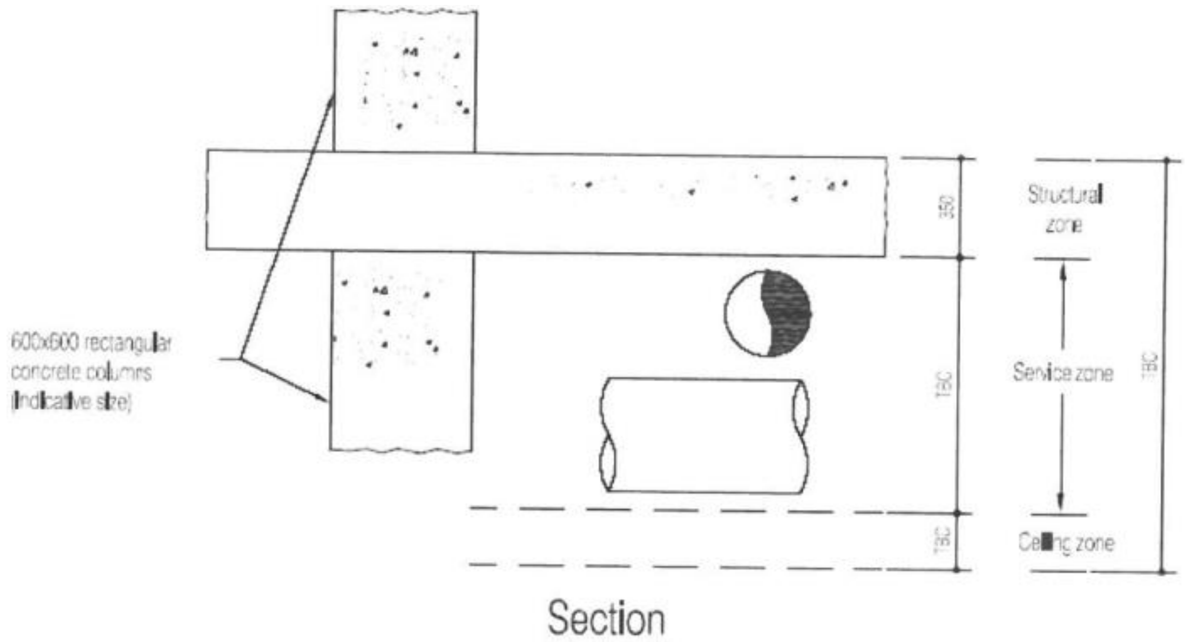

Figure 7. Floor cross-section

Based on a nominal 8-meter grid, the resulting floor measures 350mm in most areas. The larger spans require a 450-mm floor. The use of a coffered floor has a beneficial impact upon the foundation dimensions and upon stability due to the lighter structure.

Due to the significant pushing forces exerted by the premises walls, the floors at ground level and at the lower levels shall withstand additional compressive stresses. For these areas it is recommended to build a full floor, since a coffered floor would not be able to take over these compressive stresses.

There are certain areas that require vertical beams (in order to change the position of a pole or in areas with greater concentrated loads). Due to the height of these beams in those particular areas, we shall create local barriers for the distribution of installations, which can be partially avoided by drilling holes through these beams, though the number and size of these gaps should be limited. It is recommended that such holes be drilled midway along the beam height (measured from the upper surface of the plate), as well as avoid the areas near support frames so as to prevent any unfavourable effects upon structural behaviour. Another aspect to remember is that beams with spans larger than 10 m may require an initial counter-flexure so as to counteract flexures occurring after shuttering removal, which may reflect itself in cost estimates for transfer beams.

The connections of metal poles with concrete plate represents a key detail that could potentially affect the execution rate if a complex detail is selected. This matter is presented in more detail in the following section.

4.5 Poles

One should keep in mind that the top-down construction technique proposed for the basement can only be achieved with metal poles embedded in the foundation piles. This entails that the poles below ground level shall be made of steel and encased or not in concrete. Therefore, it would be effective to consider the use of metal poles across the entire structure, together with the concrete plate. This would result in smaller overall dimensions of the poles.

Considering the implementation methodology of the top-down construction technique, the encased poles, spread from the pile-casting level to the foundation plate, must be continuous, which is why it might be necessary to fit the encased poles, for instance, between level B02 at 361.5m and level SB01 at 372.5m, usually with an additional length for encasement into piles, making it necessary that the tallest poles cover a length of around 14-16 m. These poles can be either continuous or weld-jointed during execution. See figure 8 below.

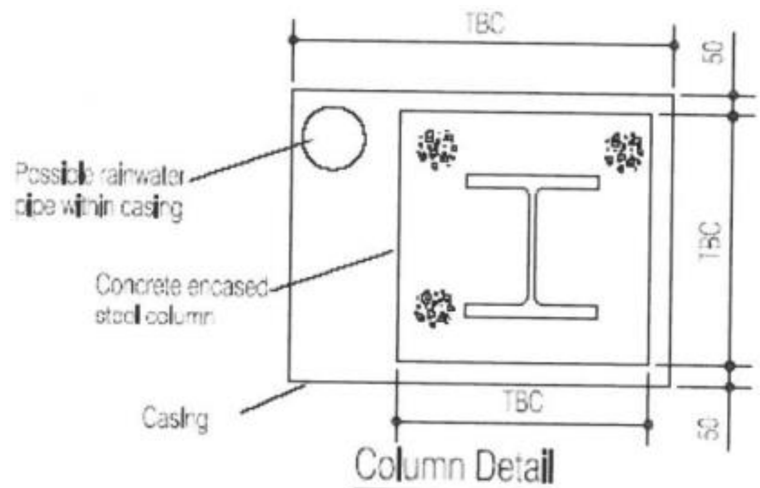

Figure 8. Metal pole encased in concrete

The poles encased below ground level usually have a position tolerance of ±75mm, which is outside the admissible limits for metallic structures. The specified tolerances, requested by the pile contractor , shall comply with the tolerances typical to the metallic constructions of this project.

4.6 Finishes

In general, it was presumed that a 90-mm thickness for finishes is present in all floor areas, with the exception of the roof and the parking lot levels. The roof shall benefit from a thicker screed, 100mm on average, to allow fitting drainage slopes on the roof. All the screeds are presumed to be reinforced with a minimal steel mesh in order to avoid fissuring.

4.7. The roof

There are various roof areas and finishes, including the flat outer terrace with a garden roof, the heliport and elevator shaft, as well as roof areas of nuclei. In that respect, compensatory loads were taken into account during the current phase so as to allow flexibility over the subsequent phases.

5. INFRASTRUCTURE

5.1 Design criteria

This construction has been provided with two completely underground levels and two semi-basements. As a result of building a 4-tier infrastructure, considerable structural limitations can appear. These include significant hydrostatic forces, a limitation of land shifts, powerful pushing forces due to the horizontal soil pressure, etc.

Other general design matters include the execution method and the delivery schedule, the drain underneath the construction, the lift forces, etc.

Considerable hydrostatic forces are applied to the basement walls and the outer waterproofing plate. The foundation plate execution level is around 360.5m rMN (relative to the Black Sea level). Given the water design level of 368m, the water head height in line with the basement is around 7.5 m. This generates a 75kPa hydrostatic pressure across the foundation plate underside and the enclosure walls. The upward forces and displacements caused by the excavation of 17m of soil are significant and shall be carefully taken into account during the design, upon receiving the final geotechnical report. The lateral movements of the enclosure wall triggered by horizontal loads are strongly influenced by the adopted nature and order of execution of the construction works. These matters are dealt with in more detail below.

The schedule is also influenced by the selected construction method. For this project, the basement construction schedule is critical to complying with the provided overall delivery schedule.

5.2 Specific design conditions

There is a significant number of aspects to be taken into consideration for the construction of the four basement levels, among which the following:

- The water level on site is high in relation to the foundation-laying depth, being around 4.5 m below ground level and generating a 75kPa hydrostatic pressure

- The granular nature of the shingle requires the casing of the piles over a few metres below ground, if not over their entire depth

- The buildings adjacent to the site are generally 5-10 m away from the basement perimeter, which calls for a minimisation of soil displacements caused by the basement excavations so as to avoid the excessive damaging of said buildings. It is known that certain buildings in the area are designated as historic monuments and shall therefore require special assessment or monitoring during the execution of the work. The loads from the foundations of adjacent buildings will have to be taken in into account, as well, when designing the enclosure wall.

• The streets around the basement perimeter may be densely populated with underground networks. One must pay attention to such networks that have to be diverted and coordinate efforts with the local authorities and the utility providers in question.

• The land conditions in the site area are deemed sufficiently good to bear the loads generated by the foundation. However, the lift forces due to the excavation of up to 17 m of soil exert strong lift forces on the foundation piles, forces that have to be taken over by means of reinforcements spread across the piles. The estimated tensile force to be taken over within the piles is around 2500kN per pilot.

5.3 Execution method

Various structural options and construction methods for the basement have been assessed, including slurry walls, secant walls, top-down construction and conventional open excavation with reinforcements, as well as the bottom-to-top construction of the building, etc. All of the above have been considered in relation to formulating an economic structural solution and lowering client risk by minimizing land shifts around the excavation and thus preventing any damages to the adjacent premises.

These considerations favour the top-down construction as it has the advantage of a short execution time and that of a low risk of damages caused by soil movements. The construction and finishing of the upper levels is possible while the excavation and execution of the basement are still ongoing. This general method reduces the client's risk of damaging adjacent buildings, reduces execution times, as well as allows for reasonable savings through the elimination of most temporary reinforcements within the basement due to the use of the permanent floor as a reinforcement during the digging works.

According to the proposed top-down construction method, the peripheral walls and the internal piles are executed, in the early stages, from ground level. The internal piles within the construction footprint contain metal poles on which the structure sits as the digging works advance. The following step is to excavate the first infrastructure level and partially expose these poles, followed by casting the plate at this level. This floor shall later on operate as reinforcement for the enclosure wall while the excavation works continue to descend. It is followed by excavation and the construction of the successive floors downward, up to the lowest basement level, and the foundation plate casting.

The top-down construction does away with most temporary reinforcements within the execution process as the structure itself provides reinforcement as the digging works advance when the construction works are definitive, as well. For that reason, the soil movements outside the basement are minimised and the risk of damaging adjacent buildings is low. Another advantage is that the construction of the building above ground level may occur concurrently with that of the basement, providing substantial time savings.

If one opts for the top-down construction technique, the internal, large-diameter piles shall be placed from ground level onto the positions of the poles. These piles may slightly vary in length due to the different load, but will probably be 25-30 m long from the lower part of the basement, and accompanied by an encased metal pole from ground level to the foundation plate level.

1000 mm is probably be the smallest pile size that will allow the encasement of the buried poles due to construction tolerances and size limitations. Testing the piles under load will not be possible due to the metal structure; for that matter, a design factor of safety equal to 3 will be required, unlike the regular 2.5 accepted value for the piles tested under load. Still, in order to consider the fact that conducting tests under operational loads is not possible, additional piles were provided for testing purposes, in the tender book, for the execution of the piles.

The goal is to have, at ground level, positioning tolerances for the metal poles of +/- 10 mm in plane. As previously stated, mush better tolerances can be obtained on the construction site provided that the placement of the poles is carefully controlled. One must take into account, in the design, cases of additional loads due to possible cases of exceeded execution tolerances. Additionally, the piles will have to be cased through the shingle layer.

5.4 Foundation plate

The foundation plate will consist in a concrete plate suspended by the concrete piles. The piles are currently designed to take over the permanent load and the effective loads from the 4 tiers of the building. Any additional loads shall be taken over by the basement plate operating as a foundation plate. The thickness is, therefore, determined by this additional weight of the structure. The foundation plate slab is deigned to take over a 100kPa force. This provision is effective as the lift forces are estimated to reach this approximate value.

These lift forces are generated by the elimination of the excavated soil weight and will require several years to be completely mobilised. Due to the potential soil movements triggered by the lift force, it is recommended that all the drains or installations crossing the plate be able to take over a vertical movement of 30-40mm.

The hydrostatic loads across the lower side of the foundation plate are generally estimated at 75 kPa, therefore lower that the predominant lift force.

A possible economic alternative would be installing a drainage network underneath the foundation plate so as to eliminate this hydrostatic pressure. If it is combined with a compressible layer that eliminates lift forces, the foundation plate thickness can be significantly reduced; however, to achieve this, the piles need to have their size increased so as to take over the total weight of the building. The water retained by the drainage network shall be collected and then pumped together with the waste coming through the building sewage system to the external sewage network. This method depends to a great extent on the water flow rates, which are to be investigated at a later date.

The connection of the foundation plate to the enclosure wall is critical both in regard to its load-bearing capacity and the time required to execute it. A typical foundation plate cross-section is illustrated below.

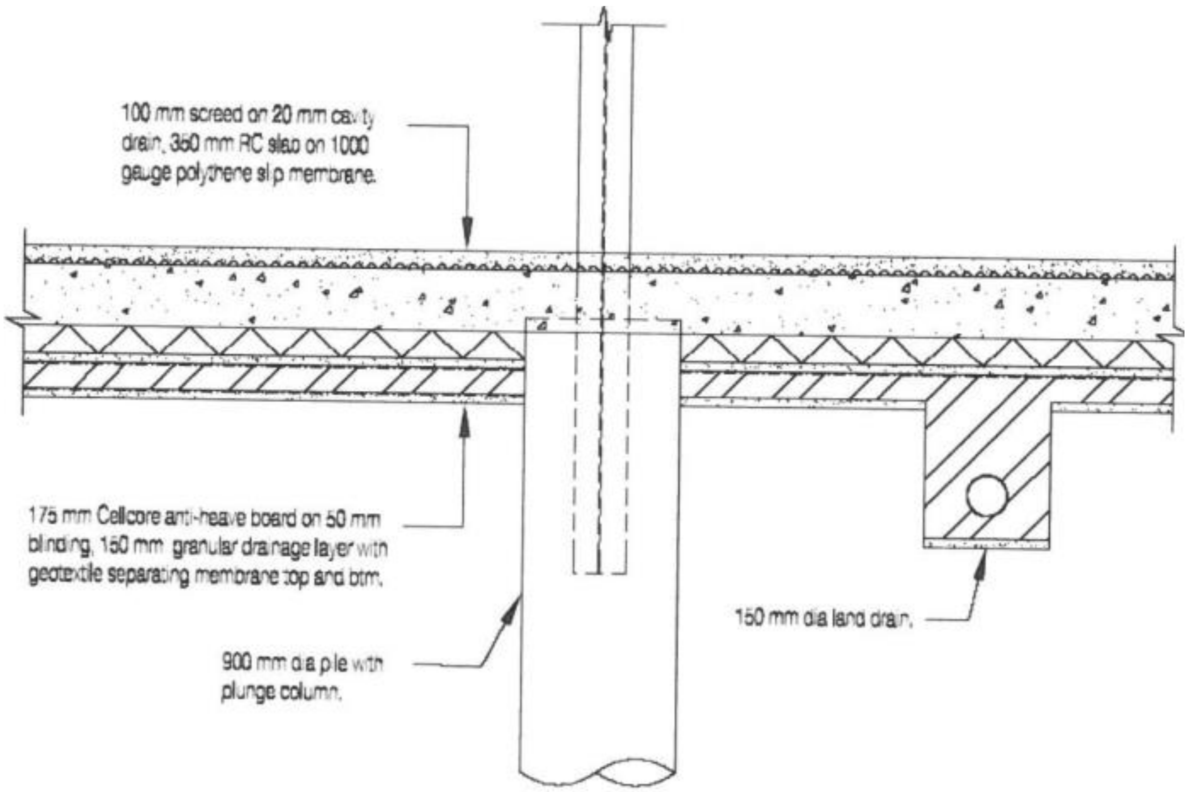

Figure 10. Basic foundation plate cross-section

5.5 Enclosure wall construction

The basement enclosure wall shall consist in secant piles, generally STAR piles with a 600mm – 800mm interaxis diameter, intersecting soft piles of the same diameter, placed at the same interaxis diameter. The wall made of secant piles must be continued downward up to the hard clay layer, below the foundation plate level, in order to form a water protection wall, the absence of which would allow a considerable water flow inside the premises during construction.

The secant wall has to be reinforced at the level of each foundation floor.

In order to take over the pile misalignment tolerance one shall cast a 250-mm thick concrete wall onto the internal facet of the piles, strengthened with a reinforcement mesh that requires around 400 mm^2/m in both directions.

In certain areas without any level permanent floors, as there are, for instance, at staircases, etc., the secant wall will require jointing beams on top of these areas for purposes of temporary reinforcing during execution.

In order to take into account the execution tolerances and the interior finishes, the on-site distance around the site limit shall be around 1500 mm from the inner facet of the interior finishes. It relies on piles of no more than 900 mm in diameter and shall increase proportional to the increasing diameter of the piles.

An alternative to the secant wall would be the slurry wall. However, it is much more expensive than the secant piles. Moreover, it takes a large construction site area to store the materials for this method. It is, nevertheless, believed that, given the price criteria, the secant wall option will likely be deemed as favourite. This ensures a good load-bearing capacity and proper temporary waterproofing, however, for approximately half the cost of the slurry wall option.

5.6 Land shifts

Due to the site location and the proximity of existing buildings and networks during the construction, the execution of the enclosure wall and its possible shift during execution are critical design issues. To mitigate this risk, it is deemed essential to only consider reinforced enclosure walls and minimise the cantilevered piles. The horizontal deformation of the piles in the temporary setting must be controlled by the general contractor in charge with the piles, in terms of controlling the potential damage to the existing paved surfaces, the underground networks and the adjacent buildings.

This reinforcement not only reduces deformations within the enclosure walls and, therefore, risk, but also leads to a lower overall thickness of the enclosure wall and increases the basement usable area. The land shifts in the case of a reinforced enclosure wall generally follow the system illustrated below and originate from the following sources:

- **The pile execution procedure**
    - The procedure of drilling the hole, removing the earth and concreting in order to build the pile may involve vibrations and the loss of integrity within the soil surrounding the pile.

- **Excavation of the earth near the inner facet of the enclosure wall**
    - Removing the weight of the earth inside the excavation triggers vertical and horizontal shifts of the surrounding land.

- **The basement execution procedure**
    - The movement generated through digging is strongly influenced by the manner and sequence of execution stages. The top-down construction type described above was selected, among others, in order to minimise land shifts.

- **Water flow inside and around the digging site**
    - A change in the water pattern in the area may lead to land shifts. This change may be caused by infiltrations into the digging site, dewatering works in the surrounding area using a well system, etc.

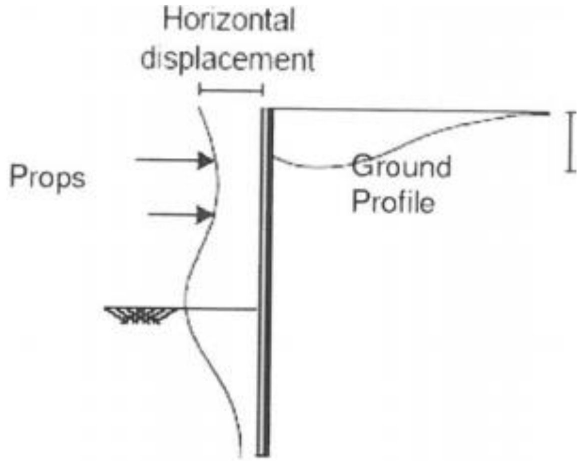

Figure 11. Deformed profile of a reinforced enclosure wall

Considering the execution method that we have proposed using secant piles, the top-down construction and the existing foundation-laying conditions, the land shifts during the construction works have to be calculated by the contractor in charge with the piles. The PBAI specifications for assessing and classifying land shifts shall provide general information regarding the admissible land deformation levels in order to avoid the excessive damaging of adjacent structures and infrastructures. The limits of damages are provided in terms of horizontal specific deformation and distortion angle, and not in absolute dimensions.

These are maximum shifts and shall depend upon the final diameter of the piles and the distances between reinforcements employed. At the corners of the digging site, these shifts shall be considerable shorter, probably by around 50%. A decrease in land shifts can be achieved by means of good quality labour and a careful monitoring of the works.

5.7 Management of risk entailed by damages following land shifts

Considering the type and sensitivity of the buildings in the vicinity of the site, the land shift extent and the potential damages these buildings can incur have to be carefully controlled. By comparing the recorded shifts of the land with values provided by geotechnical calculations, one may take steps to control subsequent shifts if such shifts are outside admissible limits.

A tender book for monitoring land shifts has been drawn up by PBAI. The categories of acceptable damages resulted from land shifts and are acceptable to adjacent buildings must be established, together with the client and the neighbouring users, checking the categories defined in this report. They are listed below.

Category 1. For buildings categorised as historic monuments, defined as having "Fine fissures that can be easily repaired using normal decorations". Damages are generally limited to finishes of inner walls, and fissures are rarely visible across the outer masonry. Typical fissure widths of up to 1 mm".

Category 2. For other neighbouring buildings. It is defined as having „Fissures that are easy to fill. Recurrent fissures, not necessarily visible on the outside; certain exterior repairs may be necessary in order to ensure protection against unfavourable weather conditions. Doors and windows may get slightly jammed and require adjustments. Typical fissure widths of up to 5 mm.

These are reasonable damage levels that can be expected from a significant construction activity of this kind and shall be deemed the upper limit with which the Contractor has to comply. The Contractor shall take full responsibility for and be bound to remedy any greater damage level than this.

These classifications shall be discussed with and approved by the client and shall determine the land shift upper limits for this site. The General Contractor must make sure that the basement execution is conducted in a manner that falls within the said limits.

5.8 Cranes

It is estimated that at least 2 tower cranes will be required for the construction of the building located within the digging area. The Contractor shall take into account the foundations required for tower cranes.

5.9 Basement waterproofing

Table 5.1 explains basement classification depending on the provided usage and also identifies the generic waterproofing system for each basement type. The basement waterproofing degree may be subdivided in order to correspond to the usage of a given area.

Commercial areas require at least one grade 3 (habitable). These entail a dry (non-permeable to water and vapours) environment, fit for finishes.

Technical areas and parking lot areas can be grade 1 (basic utility) or grade (better utility), depending on the aesthetic requirements. Grade 1 allows limited water penetration, with tolerable humidity vapours, whereas grade 2 prevents water penetration, but allows vapour penetration. Service yards must also be at least grade 2.

Nevertheless, during this design stage, it is deemed cautious to assume the highest classification grade (grade 3) for the entire basement area to make it easier to define details and assess costs. This approach may be later refined during the design process to include savings.

One must keep in mind that the qualified authorities usually request to see in tender books type C drained premises for areas such as transformer stations, etc. It is particularly important to identify any basement area or item of equipment that requires a grade 4 environment. It is presumed that the design team and the client will take into account these classification types and confirm the above-mentioned assumption regarding grade 3 as suitable for their purposes.

PBAI shall provide a detailed description of waterproofing system for a drained area, for the basement walls and the plate, except for the parking lot levels, consisting in a profiled polyethylene sheet. The water collected by this system shall be directed, through narrow drains within the plate, to the sewage system, from where it shall be pumped to the surface sewage system.

Grade of basement	Basement usage	Performance level	Form of protection*	Commentary Table 1 of BS8102: 1990
Grade 1 (basic utility)	Car parking; plant rooms (excluding electrical equipment); workshops	Some seepage and damp patches tolerable	Type B. Reinforced concrete design in accordance with BS8110	Unless there is good ventilation, or local drainage, visible water may not be acceptable even for the suggested uses. BS8110: Part 1 contains only limited guidance on crack control and lacks consideration of early thermal movement. Using Part 1 may result in the formation of cracks with widths unacceptable in permeable ground. Additional guidance on the importance of cracks is given in Section 3.4.2. Groundwater should be checked for chemicals, which may have a deleterious effect on the structure or internal finishes. The performance level defined in BS8102 for workshops is unlikely to meet the requirements of the Building Regulations, Approved Document C for workshops, which are more likely to require a Grade 3 (habitable) environment.
Grade 2 (better utility)	Workshops and plant rooms requiring drier environment; retail storage areas	No water penetration but moisture vapour tolerable	Type A Type B. Reinforced concrete design in accordance with BS8007	Membranes may be applied in multiple layers with well-lapped joints. The performance level assumes no serious defects in workmanship, although these may be masked in dry conditions or impermeable ground. Groundwater should be checked as for Grade 1. A high level of supervision of all stages of construction is necessary.
Grade 3 (habitable)	Ventilated residential and working areas including offices. restaurants etc., leisure centres	Dry environment	Type A. Type B. With reinforced concrete design to BS8007 Type C. With wall and floor cavity and DPM	As Grade 2 In highly permeable ground multi-element systems (possibly including active precautions) will probably be necessary.
Grade 4 (special)	Archives and stores requiring controlled environment	Totally dry environment	Type A. Type B. With reinforced concrete design to BS8007 plus a vapour-proof membrane Type C. With ventilated wall cavity and vapour harrier to inner skin and floor cavity with DPM	As Grade 3
* **The "form of protection" suitable for each grade is rigorously examined In this report and detailed guidance for concrete basements is given in Section 3.3.**				

Table 5.1 Guide for basement waterproofing levels

The basement types can be executed by resorting to the protection forms identified in the table below.

Type A	**Tank protection** Protection is provided by a continuous membrane system fitted either on the outside or on the inside. This method can ensure imperviousness to water and water vapours.
Type B	**Integrated structural protection** Protection is provided strictly by the structure, using reinforced concrete designed as per BS8110 or BS8007 (high crack control). This method can ensure imperviousness to water, but not to water vapours, as well, without additional measures.
Type C	**Protection with drained cavity** Additional protection to the one provided by the structural envelope, ensures by means of an internal ventilated and drained cavity. This method can provide imperviousness to water and water vapours.

Table 5.2 Necessary forms of protection

6. UNDERGROUND DRAINAGE

6.1 Existing infrastructure

A drainage plan for the existing site is pending. It would also be prudent to perform an inspection of all existing drains, prior to execution, to avoid any possible issues.

It is estimated that several separate networks will be located around the site, consisting in sewers, surface sewage or all systems combined. Additionally, many of these systems may serve several buildings around the site, which makes existing flow rates difficult to gauge.

6.2 Proposed drainage concept

PBAI's drainage design objective includes drainage underneath the foundation plate, inside the building, and exterior drainage up to the existing sewage network. Drainage above the foundation plate, inside the building, falls into the scope of delivery of PBAI's plumbing facility designer.

It would be beneficial if all the water drains from the roof areas were to be vertically collected up to below the ground floor level, where they could be horizontally distributed and exit the building at ground level.

The sewage system for the portion above ground level could follow a similar path. The sewage system for the portion below ground level could be extended up to the foundation plate level and the water brought along could be pumped upwards in order to be sent to the exterior sewage network. This concept minimises the number and size of basement pumps.

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