STRUCTURAL DESIGN OF AN UNDERGROUND CYLINDRICAL SHELL

by

ARTEMIS IOANNI THEOPHILOU

BEng, Civil Engineering (1998)
University of London (University College London)
MEng, Structural Engineering (1999)
Princeton University

Submitted to the Department of Civil and Environmental Engineering
In Partial Fulfillment of the Requirements for the Degree of

MASTER OF ENGINEERING IN CIVIL AND ENVIRONMENTAL ENGINEERING

at the

Massachusetts Institute of Technology

June 2001

Signature of Author..
Department of Civil and Environmental Engineering
May 29, 2001

Certified by ... .
Jerome J. Connor
Professor of Civil and Environmental Engineering
Thesis Supervisor

Accepted by....... ....
Oral Buyukozturk
Chairman, Departmental Committee on Graduate Studies

Structural Design of an Underground Cylindrical Shell

by

ARTEMIS IOANNI THEOPHILOU

Submitted to the Department of Civil and Environmental Engineering
in Partial Fulfillment of the Requirements for the Degree of

MASTER OF ENGINEERING IN CIVIL AND ENVIRONMENTAL ENGINEERING

Abstract

This thesis describes the design of an underground cylindrical shell used to support the load of the excavation. The wall of the shell is comprised of slurry wall panels made of concrete. The cylindrical shape was selected for its efficiency in resisting externally applied pressure. Consequently, the workman labor and period of construction are significantly reduced because of the capability of the shell to resist the excavation without any interior struts. The shell, which has a diameter of 64 meters and a depth of 37.5 meters, is the exterior wall of an underground library which extends 11 stories underground.

Constructibility issues such as the excavation of the slurry trenches and proper concrete mixing and placing techniques, are discussed. Because of the high depth, monitoring the construction so as to ensure the safety of the excavation and to avoid excessive settlements which may cause damage to the nearby buildings, is necessary. Although deflections can be calculated using the best methods and soil data available, it is prudent to measure the actual deflections in field. In case of excessive deflections, a remedial action involving the use of additional stiffening rings to control the deflections, is proposed.

Thesis Supervisor: Jerome J. Connor

Title: Professor of Civil and Environmental Engineering

BIOGRAPHICAL NOTE

Artemis I. Theophilou earned his Bachelor of Engineering degree in Civil Engineering at University of London (University College London) in June 1998. In his first degree he studied all fields of the profession, namely, Structural, Geotechnical and Environmental.

Between 1998-99 he studied at Princeton University where he was awarded the Master of Engineering degree in Structural Engineering. Apart from the courses in Structural Analysis and Design he paid particular focus to the field of Structural Dynamics, Advanced Finite Element Methods and Computational Mechanics.

In September 2000 he joined the program in Massachusetts Institute of Technology, leading to the Master of Engineering degree specializing in Geotechnology, from which he is expected to graduate in June 2001.

By the time written, the author has completed a period of work experience totaling to about two years at his family company J.A. Theophilou Consulting Engineers Ltd., based in Cyprus.

ACKNOWLEDGEMENT

I would like to take this opportunity to thank all those who contributed to helping me bring this thesis to completion.

Staring from my family who apart from their financial contribution to my tuition, they helped me set my targets in life and supported me throughout. My Professors, who conveyed to me all the knowledge and mentored me. My friends, who taught me to believe in me and for all the courage they provided me.

I would like to believe that my work will contribute to the advancement of the profession. I have done all my best in this respect.

CONTENTS

LIST OF FIGURES

LIST OF TABLES

NOTATION

ν_{all}	:	Allowable shear stress of concrete
ρ	:	Reinforcement ratio of concrete section
σ_θ	:	Hoop stress on cylinder / ring beams
b	:	Width of concrete section used in the design
d	:	Effective depth of concrete section
f_c'	:	Design concrete strength
f_{ca}'	:	Adjusted design concrete strength
h	:	Cylinder thickness
q	:	Design externally applied pressure on cylinder / ring beams
q_{cr}	:	Critical externally applied pressure on cylinder / ring beams
r	:	Average radius of cylinder
A_s	:	Area of steel of concrete section
Mu	:	Ultimate moment capacity of concrete section
N_θ	:	Hoop force on cylinder / ring beams

1 INTRODUCTION

This thesis describes the analysis and design of an underground cylindrical shell. The shell is composed of reinforced concrete slurry wall panels, has a diameter of 64 m, a depth of 37.5 m and is buried just below the surface. The structure functions as the exterior diaphragm wall of an underground library. It resists the earth pressure and provides a barrier for the interior chamber which contains the library materials.

The library extends 11 levels underground and was proposed as a solution to the space shortage problem that currently exists at Massachusetts Institute of Technology. The project was designed by a team of MIT graduate students, in the Master of Engineering program (2000-01), of which the author was the acting structural engineer.

The thesis starts with an outline of the various concepts considered in the design. The architect approached the structural engineer with the desire for a structural shape other than a box. The cylindrical shape was proposed, which is aesthetically elegant to satisfy the architect and structurally efficient in resisting lateral loads. Significant cost savings result due to the material efficiency and the easier workman labor. Of significant importance are the load paths, which change after a hole is drilled at the bottom of the shell. The shell action of the diaphragm wall is then disrupted and the earth pressure is transmitted to the interior stiffening rings.

A discussion regarding the several issues involved in the construction of such walls follows. The sequence of construction of single panels and of the entire structure are explained. Proper concrete mixing and concrete placing techniques are recommended, as they greatly influence concrete strength.

In chapter 4 the structural analysis and design are presented. The design was carried out using British codes for concrete design. Example calculations of the wall design are included.

The last chapter presents a proposed monitoring scheme. Due to the significance of the structure, which can accommodate a large number of people, it is necessary to ensure its structural integrity at all times. The horizontal soil deformations are measured in the field using inclinometers, which are installed adjacent to the cylindrical shell and adjacent to the nearby buildings.

2 CONCEPTUAL DESIGN

2.1 Description of Project

The purpose of the project was to prepare a proposal for a new library within the MIT campus, as part of the campus future expansion plan. The new library is expected to relieve the problems which are associated with the space shortage within the libraries, which currently (year 2001) fail to meet the recommended standards.

Examples of these problems are the seats and study areas which are very limited. In addition, old books, once removed from the library are inconveniently sent to a repository facility far away from the campus.

In order to reach their decision for the location of the library the team investigated all possibilities for locating a structure of such a size within the campus. After a thorough investigation the possibility for locating the structure overground was abandoned, since there were future plans for all available areas. The decision for locating the structure underground came after finding an area in a convenient location at the center of the campus, for which there are no future plans for development in that area as it is simply an open yard.

2.2 Architectural Features

An architectural drawing of the library is shown in Figure 1.

The following demands were set by the architect:

- The architect requested a structure of a shape other than that of a rectangular box.

- According to the calculations of the architect the service area within the structure must be of about 35,000 m^2. For the cylindrical shape and radius selected, this corresponds to extending 11 floors underground.

- Fire provisions required the placement of an egress tunnel at the bottom of the library, which would be connected to an egress shaft. This meant that a hole had to be drilled at the bottom level of the shell. In other words, the shell action by which the load was resisted would be disrupted.

- There would be no connection or load transfer of the cylinder wall to the interior of the structure. Due to the expected water seepage through the wall, a gap was allowed so that water drains down to the bottom, where is collected in a sump and pumped up to the surface. Figure 1 clearly shows the gap allowance. The gap is formed between the slurry wall and an interior false wall.

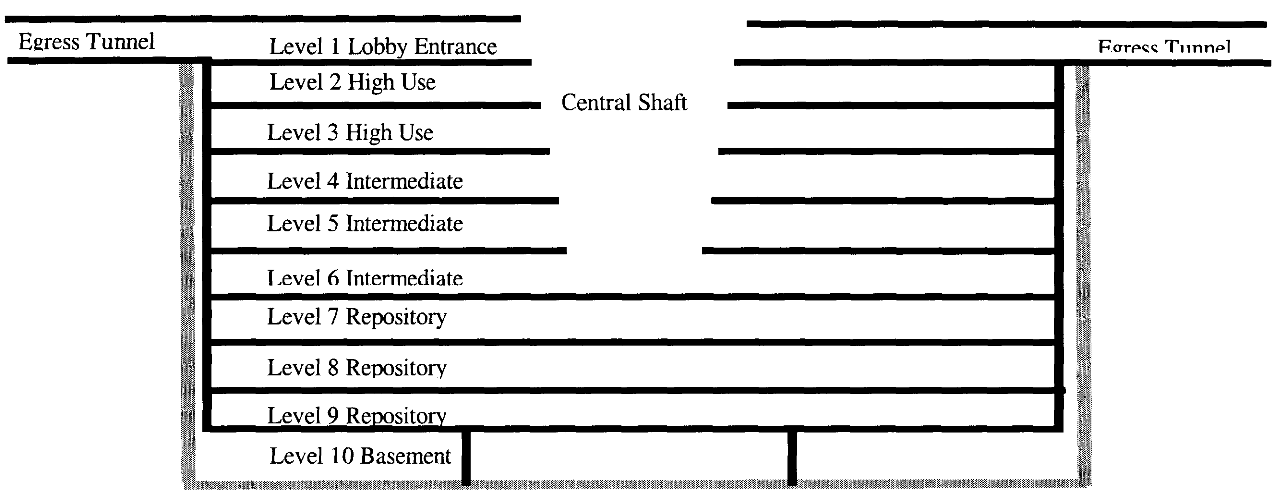

Figure 1. Architectural Cross Section of the Library.

2.3 Structural Configuration

A first step in the design process is the determination of the exact structural type and configuration. The loads to be resisted by the structure are:

- Lateral earth pressure

  Varies triangularly with the depth.

- Seismic loading

  To account for earthquake loading the static lateral earth pressure was increased by 10%. The rationale for selecting this value is that the structure is small enough compared to the wavelength of the seismic wave so the additional earthquake loading will be quite small. In addition the earthquake factor required by the code for the area of Boston is small, with an effective peak acceleration equal to 0.15g so that the increase in the earthquake loading is small enough.

- Selfweight

  The walls of the structure have to be designed for its selfweight.

Since there is no connection of the diaphragm wall with the interior structure there is no load transfer between the two at all. A gap was provided so as to allow for the expected penetration of the water through the slurry wall to drain down to the bottom floor where it is collected and pumped on to the surface.

The shape that was selected for the structure is that of a cylinder, that is, a cylindrical shell. This shape is more efficient in resisting the external pressure because the shell is in a membrane state of stress, that is, there are only in-plane stresses. The stresses perpendicular to the plane of the stress and thus the bending moments are negligible.

In addition to the effectiveness of the structure in resisting the externally applied load, the cylindrical shell has the advantage of supporting the excavation by itself without the need for any internal struts. For this reason the removal of the soil from the interior of the structure becomes much easier, faster and cheaper.

In reality the structure will not be exactly cylindrical; it will consist of 20 plane slurry wall panels. The ends of two adjacent panels will be connected at an angle, with a relatively small bending moment existing there. Due to the construction joints there will be a discontinuity between two adjacent panels at which there will be no moment transfer at all. This is clearly shown in Figure 2.

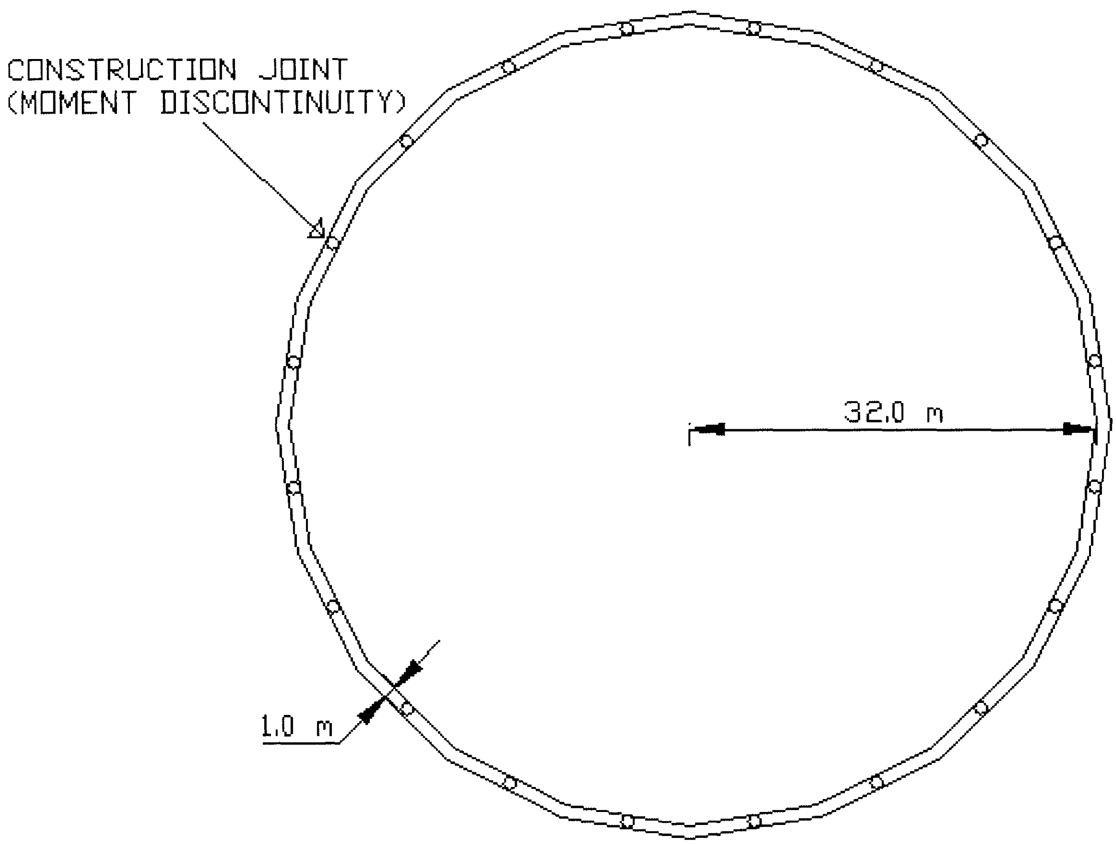

Figure 2. Schematic Plan of Cylindrical Shell.

During the design phase a problem that appeared was that a hole had to be drilled at the bottom of the cylinder to accommodate an egress tunnel. This hole will disrupt the shell action of the cylinder and therefore a second load path had to be provided. For this reason a number of internal ring beams were provided which have the capacity of supporting the external load, even in the event that the shell action of the entire cylinder is completely disrupted. These rings are shown in Figure 7.

These ring beams also function as a second line of security against total collapse, a provision that is required by many building codes.

In addition, these ring beams also provide torsional stability to the slurry wall panels. For this reason it is of immense importance to place ring beams at the top and bottom of the cylinder.

However, it must be stated that the ring beams will be constructed during the process of excavation, that is, after the cylindrical shell has experienced considerable loading and deformation. In this respect, the cylindrical shell will have to resist the design forces by itself. The ring beams will take the entire load of the excavation only in the case of disruption of the shell action.

At the bottom of the cylindrical structure a mat foundation will be constructed. However it will be in place only after the excavation is complete and the structure has already deformed, almost to its final magnitude. Therefore no forces from the cylindrical shell will be transmitted to the mat foundation.

3 CONSTRUCTIBILITY

3.1 Slurry Wall Construction

Slurry walls are structures formed and cast in slurry trenches. The process of construction, shown in Figure 3, is as follows.

1. First the trench is excavated. The mechanical digger used has a linear shape in order to achieve the longitudinal shape required.

2. During the excavation process the trench is filled with bentonite slurry which has a density high enough to apply lateral pressure on the excavation walls and prevent the trench from collapsing.

3. After the completion of the excavation, round tubes are inserted at the two ends of the wall to form the panel joint with the adjacent panel.

4. Then the reinforcement cage is inserted with the aid of a crane. The cage is usually assembled in the nearby construction site.

5. Fresh concrete is then poured into the trench through tremie pipes. The concrete, which is denser than the bentonite, settles to the bottom and displaces the slurry upwards. The removed slurry is then reconditioned and stored for later use.

Slurry trenches are constructed in an alternative sequence. Once the two alternative panels are in place, the intermediate panel is then constructed.

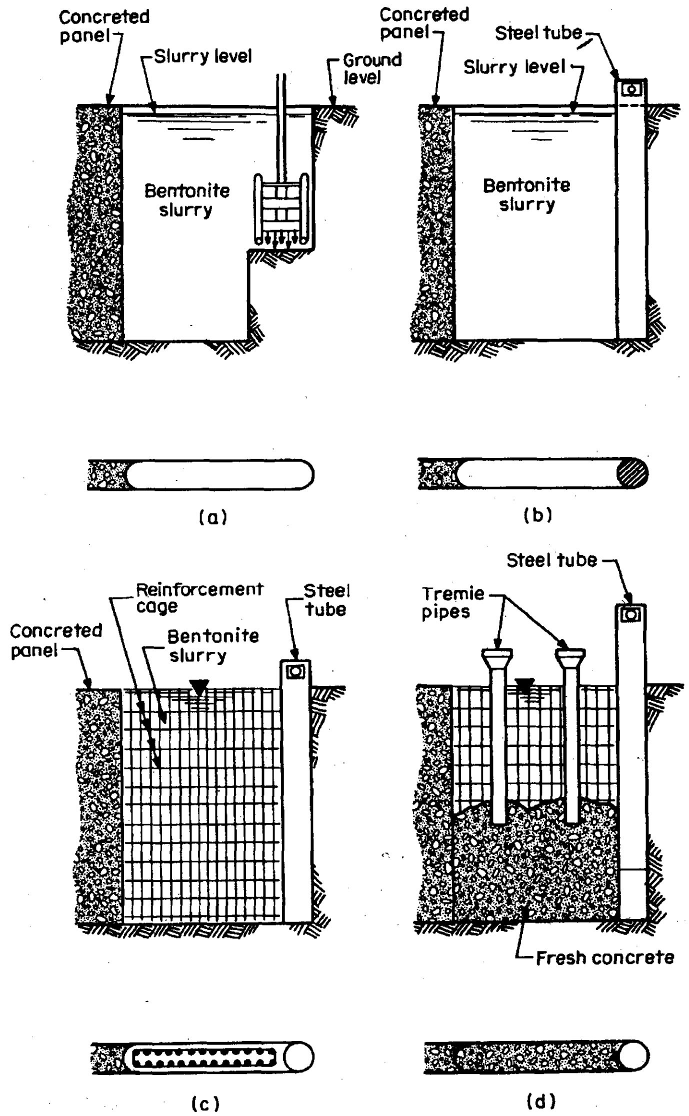

Figure 3. Typical construction sequence of a diaphragm wall, executed in four stages: (a) Excavation. (b) Insertion of steel tubing. (c) Placement of reinforcement cage. (d) Concrete placement. (extracted from Xanthakos (1994)).

3.2 Construction Accuracy and Tolerance

The finished wall section may show some deviation form the design. There are three basic factors that contribute to this deviation. The first is related to the true vertical alignment; the second involves the true wall alignment on plan; and the third involves irregularities and protrusions form the average wall face.

Many factors contribute to these deviations but the predominant is the accuracy of the excavating equipment used and the skills of the operator. Another factor is the type of soil encountered; for example, there is bad control on soil with boulders or sloping layers.

Some ways typically used to minimize these deviations is the use of guide-walls at the top of the trench and the early detection of any deviations in the field.

The design should allow for the possibility of deviations. In the present design an accepted tolerance on verticality of 1/200 (0.5%) is allowed. Based on this tolerance the thickness of the wall required to resist the hoop stress is determined.

The deviation of a single wall panel at a depth of 37.5 m is:

$$37.5\ \text{m} \cdot 0.005 = 0.1875\ \text{m} = 187.5\ \text{mm}$$

The reduction of the contact thickness due to deviation of two adjacent panels by the same amount (187.5 mm) in opposite directions is:

$$187.5\ \text{mm} \cdot 2 = 375\ \text{mm}$$

Therefore, to account for vertical misalignment of the wall, the calculated thickness must be increased by 375 mm (Section 4.2). A schematic diagram of a misaligned panel is shown in Figure 4.

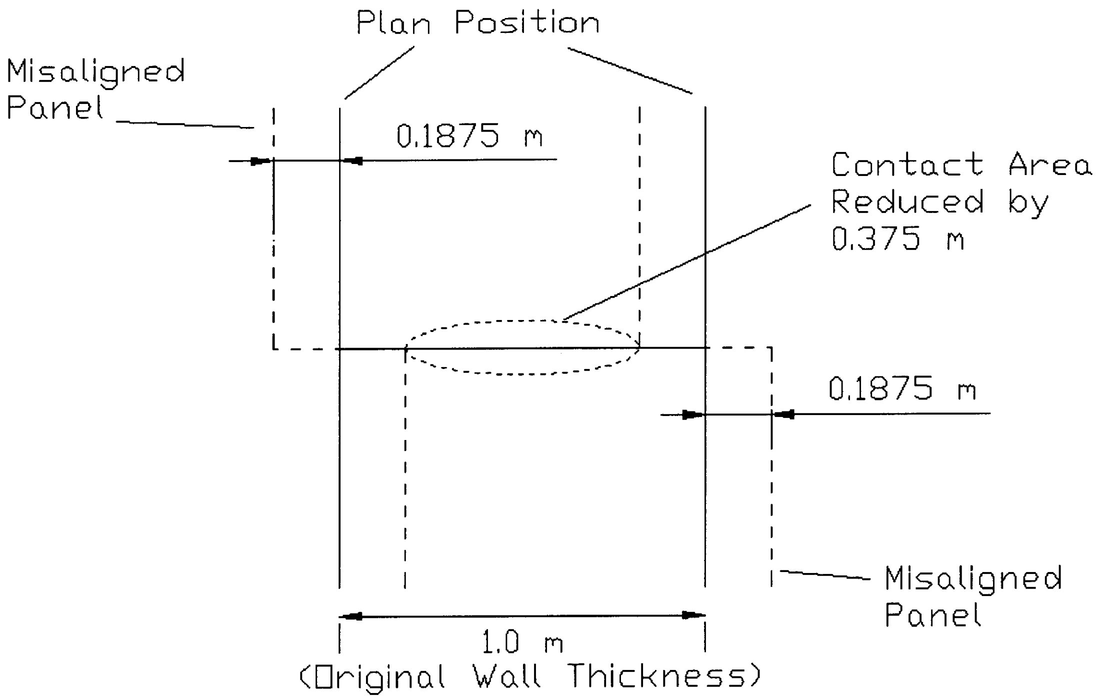

Figure 4. Misaligned Panel.

3.3 Concrete Design and Casting

Slurry wall concrete casting is in many ways different than the casting of building structures. Since the concrete is placed deep in the soil it is impossible to use mechanical vibrators, which play a significant role in achieving the expected strength. The method of placement of the concrete through the tremie pipes requires special attention to ensure that the concrete does not entrap any bentonite. Furthermore, the environment at which concrete is cured is much different.

3.3.1 Water to Cement Ratio

The most important characteristic that concrete must meet is that of strength. There are a number of factors that affect strength, which must be given careful consideration. The most important parameter is that of the water to cement ratio; the lower it is the stronger the concrete. When placing the concrete in the trench, attention must be taken to prevent fresh concrete from mixing with the slurry, event that would increase the water ratio and hence result to a lower strength.

The final strength will be lower if cavities are formed due to intermixing between the fresh concrete and bentonite or slime. Mixing with other nonconcrete materials may also take place, such as soil from the excavation.

3.3.2 Curing

The environment at which concrete matures has a predominant effect on its final strength, as it influences the hydration of cement. In the slurry trench the moisture content is favorable to allow the ultimate strength develop. In addition, temperature also has a significant effect on concrete strength. Deep in the trench the temperature is high enough to allow maximum strength develop. Due to the surrounding soil, temperature differentials in the body of concrete are minimized. In such conditions creep and shrinkage effects will be minimal.

3.3.3 Concrete Strength

It seems that in general the concrete strength of slurry walls is much higher than the theoretical strength. Measurements on concrete cores that were extracted from exposed walls several weeks after pouring showed an increase in strength by as much as 7 N/mm^2. This can well be attributed to the favourable curing conditions. Nonetheless, interpretation of these data should take into account the actual influence of the average construction conditions, the observed variation in strength, the scattering of the data and the fact that quality controls may not be always implemented to the extend desired.

To account for these reasons a factor of safety should be used to the design strength, in the case of slurry wall construction. In the present design, concrete of strength $f_c' = 35$ N/mm^2 was used. This was adjusted according to recommendations by Xanthakos (1994), using a factor of 0.90. The adjusted concrete strength is:

$$f_{ca}' = 0.90 \cdot f_c' = 0.90 \cdot 35 = 31.5 \text{ N/mm}^2$$

3.3.4 Workability

Concrete must be flowable and workable enough in order to be placed through the tremie pipes. Experience has shown that faults, reversed "hanging up" and "whirls" in which contaminated concrete is trapped can be avoided by a mix that is flowable and yet not subject to segregation. To make the concrete mix more workable plasticizers are used. Their use also allows the water content to be reduced which results to an increase in strength. Workability is usually measured using slump tests, which should be between 18 to 22 cm.

3.3.5 Concrete Placement

It is recommended that concrete pouring starts as soon as the reinforcement cage is in place. Once it starts it must continue until the end, to avoid problems such as the formation of cold joints on concrete or blocking of the tremie pipes. The most common way of pouring concrete is through tremie pipes. These pipes extend to the bottom of the slurry. Once concrete is poured it settles down because its density is much higher than that of slurry. The tremie pipe is lifted slightly as concrete is poured so as to ensure that its end is always just immersed in the concrete layer.

During the pouring procedure some intermixing between concrete and bentonite may take place. One way to reduce its degree is to push the concrete from the bottom up, that is, in a "plug motion". Concrete is pumped in batches and the first batch is always at the top, shown in Figure 5. In this way the mud tappings and inclusions are minimized. In addition the concrete has a sweeping action that cleans and removes bentonite from around the reinforcing bars and other vertical surfaces.

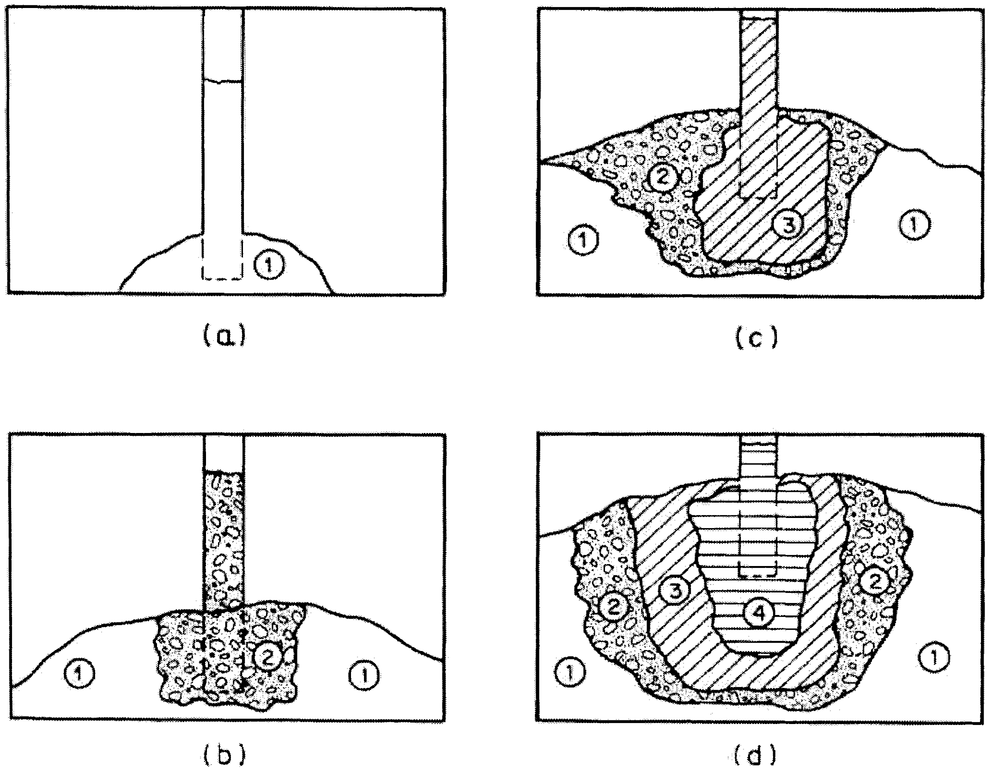

Figure 5. Flow motion of concrete in a long panel through a single tremie pipe: (a) Batch 1 being placed. (b) Batch 2 being placed. (c) Batch 3 being placed. (d) Batch 4 being placed. (extracted from Xanthakos (1994)

3.4 Sequence of Construction

1. Excavation of slurry trench and construction of slurry wall
2. Excavation of soil on the interior side of the cylindrical structure, until the level of the next ring beam.
3. Pause excavation. Construction of ring beam.
4. Continue with item 2 until the excavation reaches the bottom.

5. Construction of mat slab.

During the entire construction period a significant volume of groundwater must be pumped continuously, to keep the water table below the level of the excavation.

3.5 Construction Joints

The joints between two adjacent wall panels are formed using steel tubes. During the construction of the first panel, the steel tube is placed at the edge of the trench so that a hollow cavity is preserved. When the concrete gains enough strength the steel tube is extracted. In the excavation of the adjacent panel, the hollow cavity is used to guide the mechanical excavator. In this way, any vertical misalignments in the first panel are transferred to the second panel, hence minimizing the differential misalignment. The concrete of the second panel completely fills the cavity.

Due to the construction method no reinforcement is continued through the joint. Thus the joint is a point of bending strength discontinuity. However, since it is under constant compressive stress it maintains its shear stress capacity.

3.6 Assembly and Details of Reinforcement

Reinforcement cages are assembled in the nearby site area, therefore in planning the construction ample space must be provided. Essentially, they are assembled in the horizontal direction and placed into the trench using a crane. It is important that the designer considers that the points from which the assemblage will be lifted are strong and stiff enough to carry the load of the entire cage.

The vertical length of the wall is 37.5 m. Since no reinforcing bars exist at such a length, two or more lengths will be required. These lengths have to be connected using U-bolt splices, such as the one shown in Figure 6.

Each individual cage length is moved by the crane into the trench, and supported so that its top is projecting above the ground level. The adjacent cage length is then held by the crane just above, so that the splices between the two are installed. The crane then lowers the two connected cages.

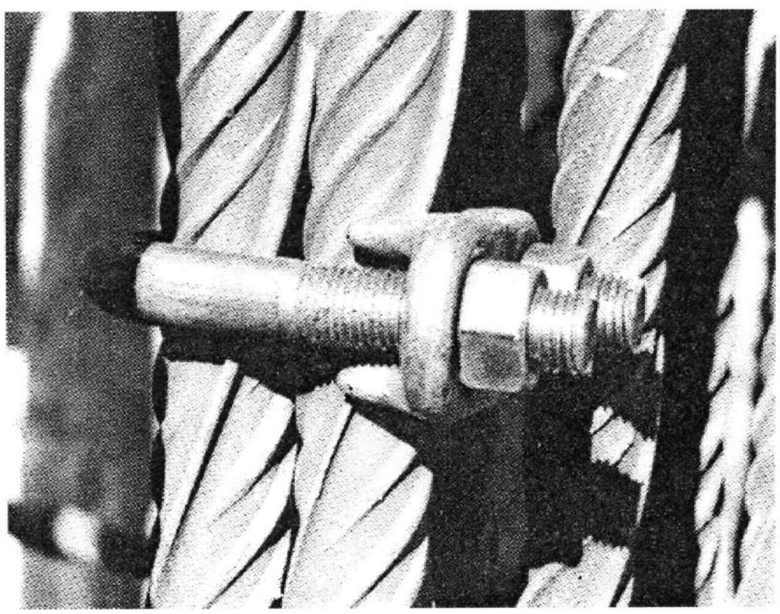

Figure 6. U-bolt connection used to splice the upper and lower part of the reinforcing cage. (extracted from Xanthakos (1994))

4 STRUCTURAL ANALYSIS AND DESIGN

Structural design was carried out according to British Standard 8110. The following parameters were used throughout.

Concrete adjusted design strength:

$f_{ca}' = 31.5\ N/mm^2$

Reinforcing steel yield stress:

$f_y' = 460\ N/mm^2$

Wall thickness

$h = 1000$ mm

4.1 Design Loads

The design load q, is varying proportionally with the depth and is acting on the cylinder walls in the form of externally applied pressure.

At surface level $q = 0\ KN/m^2$

At depth of 37.5 m $q = 605\ KN/m^2$

These values consist of the static lateral earth pressure and hydrostatic pressure, increased by 10% to account for seismic loading.

4.2 Thickness of Cylinder

The thickness of the cylinder will be determined by the hoop stress that exists at the lowest level of the cylinder. This will have to be less than the adjusted compressive stress for concrete f_{ca}'.

Design pressure q, at depth of 37.5 m:

$$q = 605 \text{ KN/m}^2$$

Hoop force, N_θ (per m width)

$$\begin{aligned} N_\theta &= q \cdot r \\ &= 605 \,\text{KN/m}^2 \cdot 32.5 \,\text{m} \\ &= 19{,}962 \,\text{KN} \end{aligned}$$

Required thickness, h:

The following calculations are for a section of unit width (1 m)

Hoop stress < Adjusted design stress

$$\begin{aligned} &\sigma_\theta \le f_{ca}^{'} \\ &\frac{N_\theta}{h \cdot 1\,\text{m}} \le f_{ca}^{'} \\ &\frac{N_\theta}{f_{ca}^{'} \cdot 1\,\text{m}} \le h \\ &h \ge \frac{19{,}962E3\,\text{N}}{31.5\,\text{N/mm}^2 \cdot 1000\,\text{mm}} \\ &h \ge 630\,\text{mm} \end{aligned}$$

In these calculations the only factor of safety used is that for the material. No factor of safety has been used to allow for construction inaccuracies. To allow for possible deviations of the position of the wall in the vertical direction, the calculated thickness was increased by 375 mm, as calculated in section 3.2.

Design thickness of the cylinder wall:

$$h = 630 \text{ mm} + 375 \text{ mm}$$
$$h = 1005 \text{ mm}$$
$$h \approx 1.0 \text{ m}$$

4.3 Design for Bending in the Vertical Direction

There are two cases for which the bending moments in the vertical direction have to be checked:

- The cylinder acts as a shell.

  In this case the bending moment on the walls is (relatively) very small, and results from the non-uniform pressure distribution. It was found that the maximum bending moment is about 3 KNm.

- Shell action is disrupted.

  When a hole is drilled, or any other event occurs that would cause significant damage to the walls of the cylinder and disruption of the shell action, the earth pressure will have to be transmitted through the wall to the ring beams. In this case the cylinder walls act as beams which span between the ring beams. It is obvious that the beam action will induce a much greater bending moment on the wall, and hence the design is based on this case.

As a first step, the location of those ring beams has to be determined, and is based to the following rationale. It is essential that one ring beam has to be on the top and one on the bottom of the cylinder to provide torsional stiffness to the wall panels. It is also important that another ring beam has to be located just above the drilled hole.

It is desirable for constructibility reasons to have a uniform distribution of the main reinforcement from the top to the bottom. To do so the bending moment to be resisted must be of about the same magnitude between all spans and all supports. Such a bending moment distribution between the spans will also have significant economic benefits on the cost of the structure.

So, by trial and error the location of the ring beams was determined based on the above criteria. Five ring beams were used, placed in the positions shown in Table 1.

Ring #	Distance from bottom (m)
1	37.5
2	24
3	14
4	6
5	0

Table 1. Position of Ring Beams.

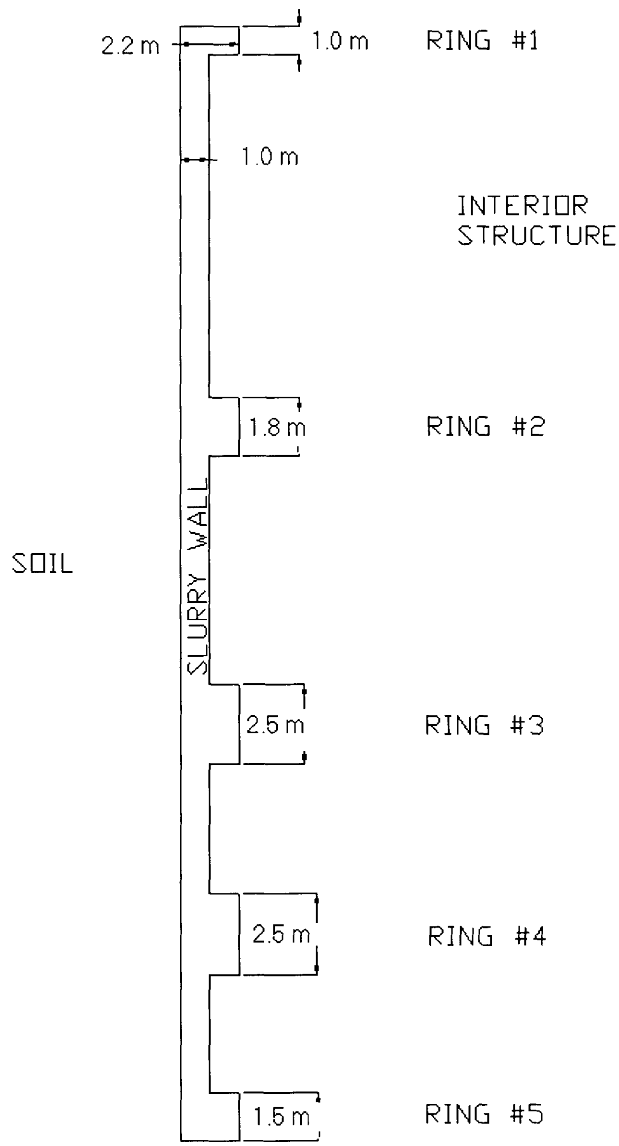

Figure 7. Location and Dimensions of Ring Beams

Structural analysis was performed using the computer code SAP2000. Figure 8 shows the bending moment diagram, with a factor of safety for strength equal to 1.5, and 20% moment redistribution.

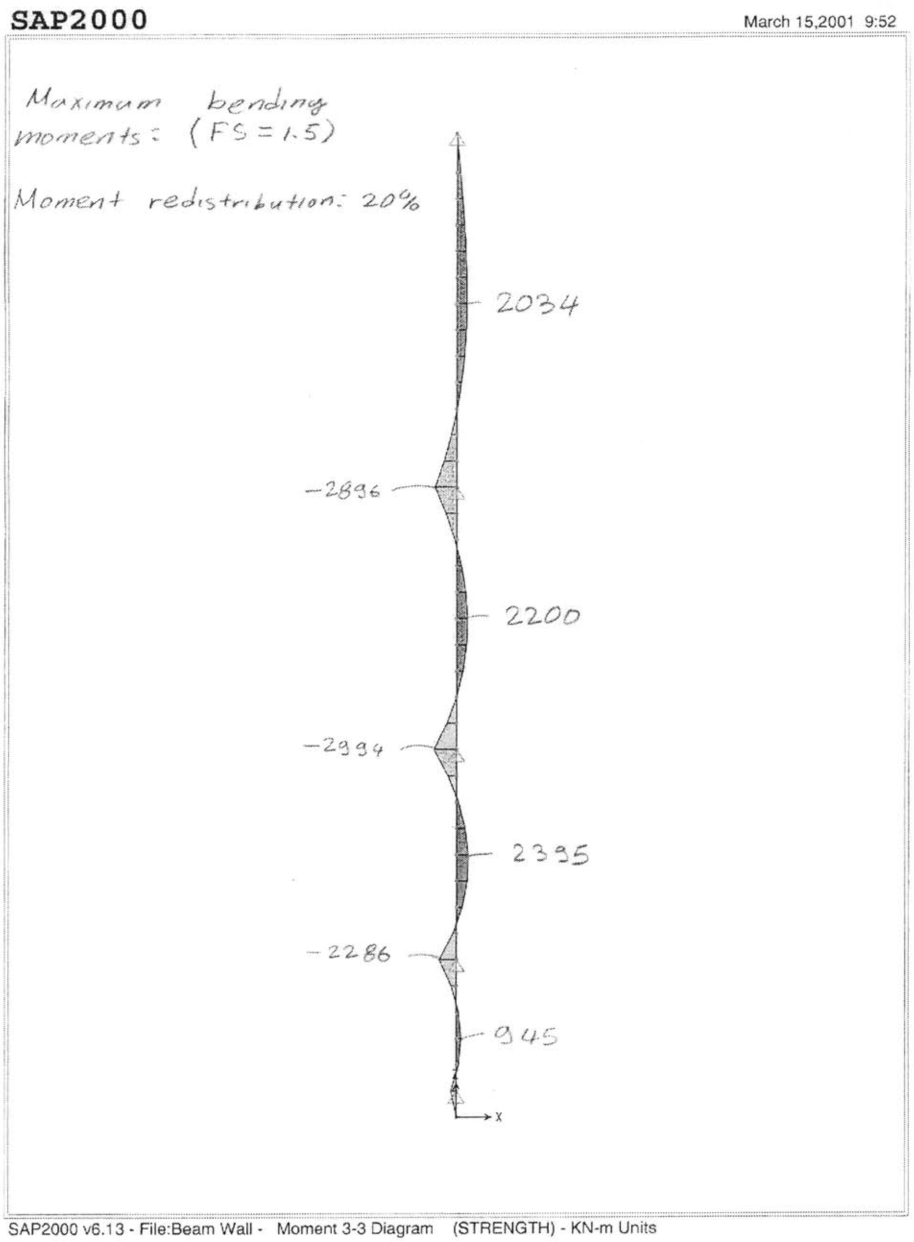

Figure 8. Vertical Bending Moment Diagram of the Slurry Wall.

The maximum bending moments from the analysis are shown in Table 2.

Bending Moments (KNm)	Elastic (FS = 1.5)	Redistributed (20%)
Max +ve	1,834	2,400
Max –ve	-3,742	-3,000

Table 2. Maximum Bending Moments in the Vertical Direction.

4.3.1 Example calculation for the design of the exterior vertical reinforcement, to resist the maximum negative moment.

Moment to be resisted:

Mu = 3,000 KNm

Wall thickness = 1,000 mm

Cover = 50 mm

Try reinforcement of ∅ 40 mm.

Then d = 930 mm

The design is for unit width, so b = 1,000 mm

$$\frac{Mu}{bd^2} = \frac{3{,}000E6 \text{ Nmm}}{1{,}000 \text{ mm} \cdot 930^2 \text{ mm}^2} = 3.4$$

From BS8110 design charts:

$$\rho = 0.010$$

Reinforcement area required:

$$A_s = \rho bd = 0.010 \cdot 1000 \text{ mm} \cdot 930 \text{ mm} = 9{,}300 \text{ mm}^2$$

Use ∅ 40 mm @ 130 mm

Similarly, the design for the interior vertical reinforcement was performed. The reinforcement to be used is ∅ 40 mm @ 160 mm.

4.4 Design for Bending in the Horizontal Direction

As discussed earlier, at mid-span between two adjacent corners there will be bending strength discontinuities. At their corners, two adjacent panels will be monolithically connected and essentially continuity will be provided. The schematic plan of the cylinder is shown in Figure 2. The bending moment diagram is shown in Figure 9.

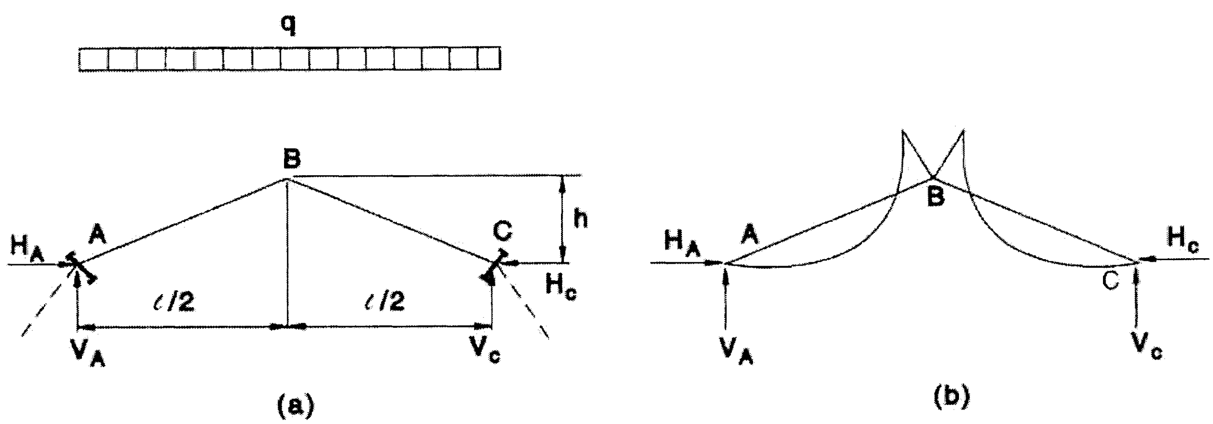

Figure 9. Typical two-sided panel. (a) Geometry and load. (b) Moment diagram. (extracted from Xanthakos (1994)).

Xanthakos (1994) gives the following approximate equations for the analysis.

For the maximum positive bending moment:

$$M_{+ve} = \frac{ql^2}{512}$$

For the maximum negative bending moment:

$$M_{-ve} = -\frac{ql^2}{32}$$

where l is as shown in Figure 9.

In order to achieve some cost savings, the design was conducted for three different levels. Table 3 shows the levels, bending moments and the design.

Segment Depth (m)	Actual Pressure (KN/m^2)	Design Pressure (KN/m^2)	M_{+ve} (KNm)	M_{-ve} (KNm)	Exterior Reinforcement (M_{-ve})	Interior Reinforcement (M_{+ve})
0 – 10	155	233	46	728	∅ 250 mm @ 200 mm	∅ 25 mm @ 200 mm
10 – 20	320	480	94	1,500	∅ 32 mm @ 160 mm	∅ 25 mm @ 200 mm
20 – 37.5	605	908	178	2,838	∅ 40 mm @ 130 mm	∅ 25 mm @ 200 mm

Table 3. Design of Horizontal Reinforcement.

4.5 Check for Shear

The maximum shear stress occurs in the horizontal direction and has a magnitude of

$$v_{\max} = 4.5 \text{ N/mm}^2$$

The allowable shear stress on concrete is

$$\begin{aligned} v_{all} &= 0.95\sqrt{f_c^{'}} \\ &= 0.95\sqrt{31.5} \\ &= 5.3 \text{ N/mm}^2 \end{aligned}$$

$v_{max} < v_{all} \quad \Rightarrow \quad$ OK.

4.6 Design of Ring Beams

The analysis for bending in the vertical direction, shown in Figure 8, has given the reaction forces that are acted on the ring beams. These reaction forces are the actual pressures that are applied on the ring beams. The design pressures are the actual pressures multiplied by a factor of safety equal to 1.2.

The ring beams are made thick enough so that the working stress is less than the adjusted design stress which is $f_{ca}' = 31.5 \text{ N/mm}^2$.

Ring #	Design Pressure (KN/m^2)	Hoop Force (KN)	Ring Thickness (m)	Ring Height (m)	Working Stress (N/mm^2)
1	393	12773	2.2	1.0	6
2	3666	119145	2.2	1.8	30
3	5052	164190	2.2	2.5	30
4	5062	164515	2.2	2.5	30
5	3069	99743	2.2	1.5	30

Table 4. Working Stresses on Ring Beams.

5 STRUCTURE MONITORING

5.1 Purpose of Test

The excavation of the soil from the interior of the cylinder will cause the shell to deform. These deformations of the wall will cause subsequent deformations of the soil medium. It is required to ensure that these soil deformations are small enough that they will not cause problems to the buildings which are located in the proximity of the excavation. A plan showing the excavation location and the nearby buildings is shown in Figure 10. Since the deformations are expected to continue for a period after the completion of the construction, the tests have to be prolonged.

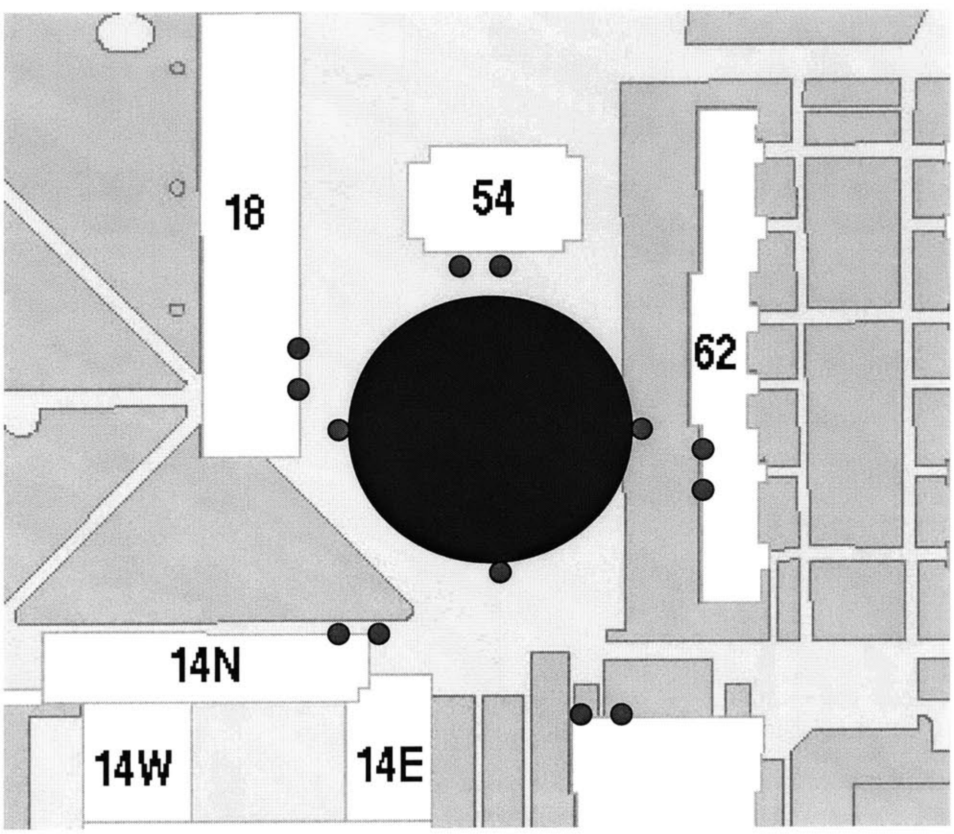

Figure 10. Location of library (large circle), nearby buildings (numbered) and test holes (small circles).

Although the magnitude of settlements has been estimated using the best analysis methods and soil data available, there is still some uncertainty. Taking into account the unfavorable and costly effects that such settlements would have on the structures it is considered appropriate to monitor the actual deformations in the field.

The expected adverse effect of the soil deformation on the nearby structures is extensive cracking due to differential settlement of their foundations. This extensive cracking will have unfavorable aesthetic effects and minor structural effects. It is anticipated however, that such small settlements will not endanger the stability of the buildings.

The cylindrical shell has been designed to fully resist the earth pressure therefore there is not much concern regarding its stability. Even excessive deflections of the wall between the stiffening ring beams will not have any visible impact from the inside of the library, since they will be covered by a false wall.

However, in view of the many uncertainties, the inaccuracies that are involved and the fact that the continuous structure does not have much redundancy (i.e. possible failure of part of the cylindrical wall will cause collapse of the entire structure), it is considered necessary to monitor the deflections. This is in order to ensure the safety of the excavation during the construction period and the safety of the building during its function. Possible failure of the cylindrical wall will have tremendous consequences, both monetary and casualties.

The monitoring of the deflections during the function of the structure can also reveal possible damage that may be caused after earthquake events. Decisions can then be made regarding any rehabilitation action that has to be taken.

5.2 Mechanism of Behavior

When the interior soil of the cylinder is removed, the earth pressure is acting only on the exterior side. This pressure induces hoop stresses on the cylinders of the wall and consequent shortening. There are two components of this shortening; the elastic component, which occurs instantaneously upon loading and the creep component, which takes a long period to come to an end.

As soon as the shell deforms inwards the adjacent soil expands elastically in volume. Its void ratio is increased and flow of water is initiated to fill the expanded voids, namely, consolidation is taking place. During the consolidation process the soil deformations are increased. Although consolidation takes many years to come to an end, the magnitude of the pressure applied on the cylinder does not increase with time.

The deformation of the soil extends to a distance away from the excavation. If the deformation field is large enough it may influence the stability of the foundations of the nearby buildings. If this is the case, the foundations may experience differential settlement which will cause cracking of the walls.

5.3 Observational Method

To avoid any undesirable effects on the nearby buildings the observational method is adopted in conjunction with the calculated settlements. The observational method is based on measurements taken on field to in order to control and modify the construction. Peck (1969) lists the following ingredients for complete application of the observational method:

1. "Exploration sufficient to establish at least the general nature, pattern and properties of the deposits, but not necessarily in detail."

2. "Assessment of the most probable conditions and the most unfavorable conceivable deviations from these conditions. In this assessment geology often plays a major role."

3. "Establishment of the design based on a working hypothesis of behavior anticipated under the most probable conditions."

4. "Selection of quantities to be observed as construction proceeds and calculation of their anticipated values on the basis of the working hypothesis."

5. "Calculation of values of the same quantities under the most unfavorable conditions compatible with the available data concerning the subsurface conditions."

6. "Selection in advance of a course of action or modification of design for every foreseeable significant deviation of the observational findings from those predicted on the basis of the working hypothesis."

7. "Measurement of quantities to be observed and evaluation of actual conditions."

8. "Modification of design to suit actual conditions"

5.4 Anticipated Deformations

Based on the best geological data available, a soil deformation analysis was conducted. The analysis was performed using PLAXIS, a computer code with capabilities of modeling geotechnical problems using geomaterials.

The model which was created had simulated the several stages of the excavation. At each stage the deformations and wall displacements were calculated. After the completion of the excavation the consolidation process was simulated. Figure 11 shows the model that was used, at the stage when the excavation depth was at about 20 m. The figure clearly shows the deformed wall.

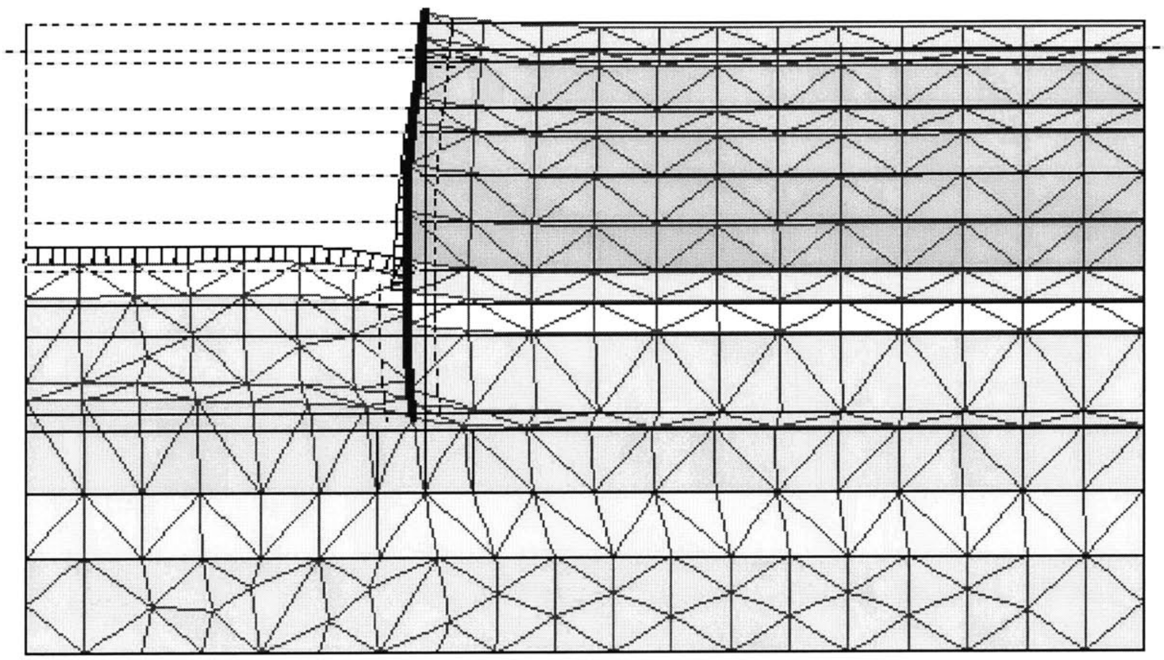

Figure 11. PLAXIS model.

In the PLAXIS model the interior stiffening rings of the cylinder were ignored, therefore the displacement results were exaggerated. It was found that for such a magnitude of displacements the additional settlements on the nearby buildings are negligible.

5.5 Parameters to be Monitored

The soil deformations that will result from the excavation will be mainly in the horizontal direction due to the horizontal deformation of the shell. There will be some heave at the bottom, which however will be limited because the lowest level of the wall almost touches the bedrock. Also, the mat slab that will be built will prevent any long-term heave.

Therefore the most suitable parameter to be monitored is the horizontal soil displacement.

5.6 Location of Test Holes

Thirteen test locations are selected. Three in the close proximity of the cylindrical shell and ten adjacent to the surrounding buildings. The test locations are shown in Figure 10.

The ten test holes that are located adjacent to the buildings serve the purpose of checking whether the effects of the soil deformation on the buildings are alarming. Two test holes are used for every building as a way to establish the correctness of the readings by cross-checking them.

The three test holes around the cylindrical shell serve the purpose of checking the soil deformations during the construction phase. They are expected to yield same deformations, therefore the results can be used to ensure the correctness of the readings.

5.7 Devise Remedial Action

In the event of observing alarming displacements on the nearby buildings, remedial action is required to reduce them. The determination of these alarming displacements is outside the purpose of this project; in actuality they will have to be agreed by the consultant engineer and the contractor.

In the original design five interior ring beams are used, shown in Figure 7. To control soil deformation additional ring beams are required. The selection of the position of the new ring beams will be determined by the contractor and the consultant engineer, based on the deformation profile and the phase of the construction.

Several warning levels can be set, with different criteria and actions to be taken. Table 5 shows a sample of such warning levels, the values and actions of which do not relate in any way to the current project.

Warning Level	Criterion	Action
1	Movement greater than 10 mm an any one survey station	Report to construction management
2	Movement greater than 15 mm at two adjacent stations	Written report and site meeting
3	Movement greater than 20 mm at any one station	Immediate site inspection by consulting engineer and site meeting

Table 5. Example of warning levels (extracted by Dunnicliff (1988)).

5.8 Description of Inclinometer

The type of test selected for the measurement of the soil deformation is the inclinometer. It is defined as a device for monitoring the deformation normal to the axis of a pipe by means of a probe passing along the pipe. Deviations from the normal axis of the pipe are measured by means of a gravity-sensing transducer designed to measure inclination with respect to the vertical. In the present case, the pipe is installed in a borehole and in a vertical alignment, so that the inclinometer will provide data for defining subsurface horizontal deformation.

The inclinometer system has four components shown in Figure 12.

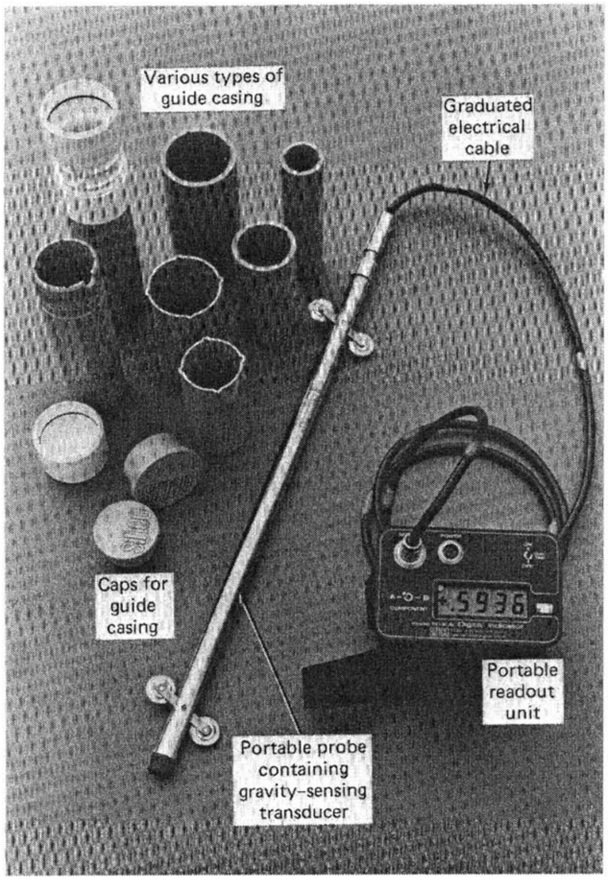

Figure 12. Inclinometer system: Slope Indicator Company Digilit®system.

1. A permanently installed guide casing, made of plastic. This casing is to be installed in a vertical alignment. There are tracking grooves on the guide casing for controlling the orientation of the probe.
2. A portable probe containing a gravity-sensing transducer.
3. A portable readout unit for power supply and indication of probe inclination.
4. A graduated electrical cable linking the probe to the readout unit.

The principle of operation of the inclinometer can be seen in Figure 13, as discussed by Dunnicliff (1988). The bottom of the guide casing is installed on the bedrock so as to measure absolute displacements. The probe is lowered to the bottom and an inclination reading is made. Additional readings are made as the probe is raised incrementally to the top of the casing, providing data for determination of initial casing alignment. The differences between these initial readings and a subsequent set define any change in alignment. Since the one end of the casing is fixed from translation these differences allow calculation of absolute horizontal deformation at any point along the casing.

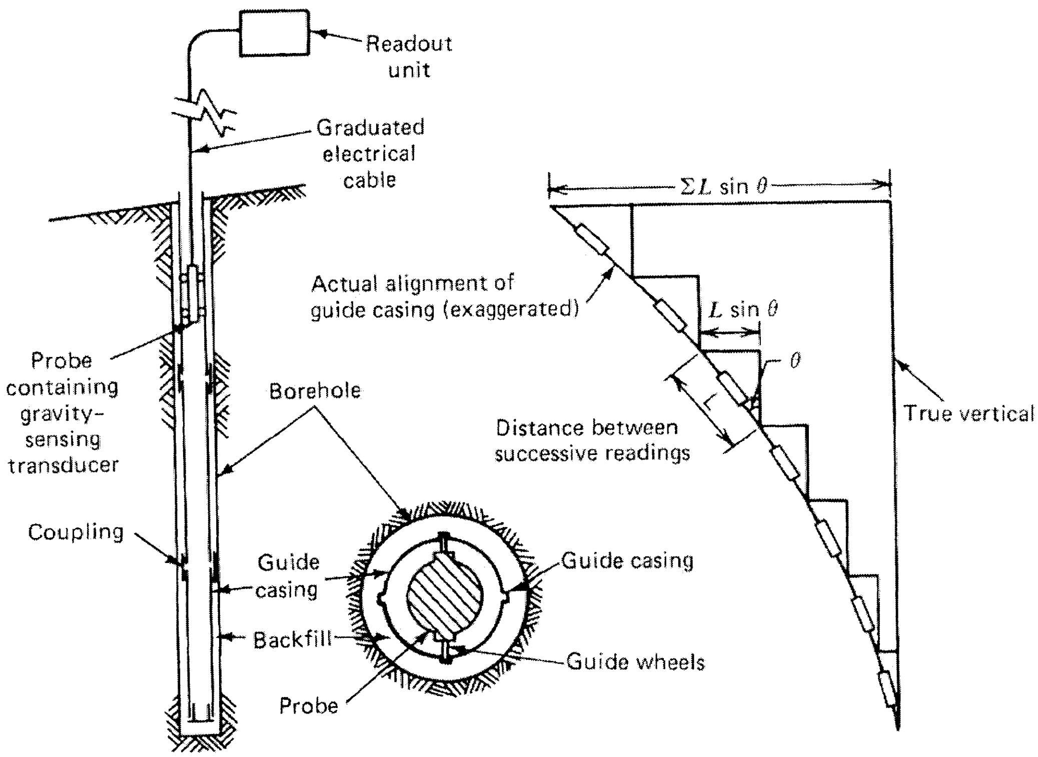

Figure 13. Principle of inclinometer operation.

6 APPENDIX 1 – STABILITY CALCULATIONS

Calculations of elastic stability are based on Timoshenko and Gere (1961).

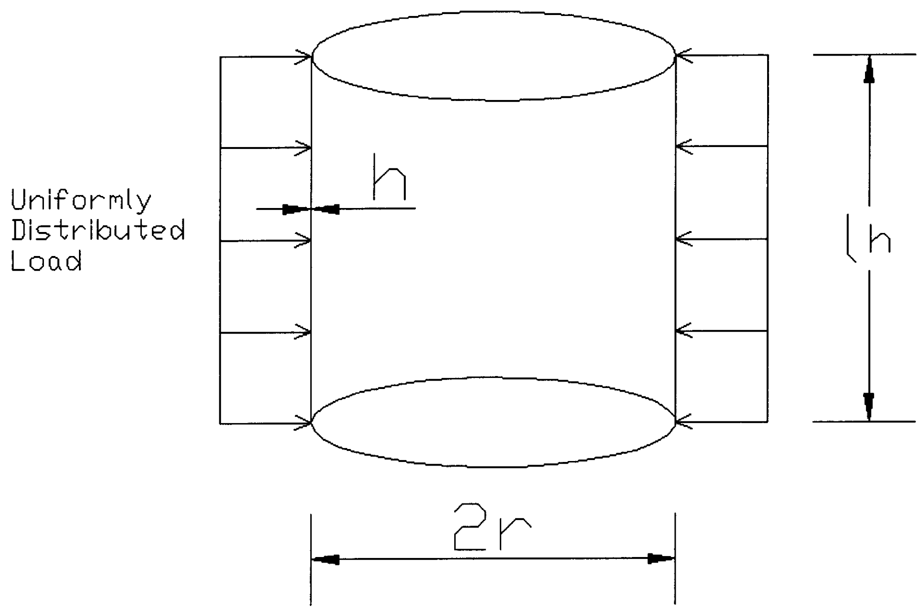

Figure 14. Notation for Cylinder Stability Analysis.

Notation used in the following calculations:

l_h :	Height of cylinder = 37.5 m
h:	Thickness of cylinder = 1.0 m
t:	Width of section considered = 1.0 (for unit width)
E:	Elastic modulus of concrete = 20E+6 KN/m^2
ν:	Poisson's ratio of concrete = 0.2
r:	Radius of cylinder = 32.0 m
q:	Lateral earth pressure
γ_c:	Unit weight of concrete = 24 KN/m^3

Approximation:

The triangularly distributed lateral earth pressure is approximated by a uniformly distributed earth pressure, a conservative approximation.

In these calculations the thickness of the cylinder used, h, is that of the uncracked section. Since the cylinder will constantly be under compressive hoop stress the concrete section is not expected to crack.

Nx: Maximum compressive force at the base, due to selfweight only.

$$N_x = t \cdot \gamma_c \cdot l_h = 1\,\mathrm{m}^2 \cdot 24\,\mathrm{KN/m}^3 \cdot 37.5\,\mathrm{m} = 900\,\mathrm{KN}$$

$$\varphi_2 = -\frac{N_x\left(1-\nu^2\right)}{Eh} = -\frac{900\left(1-0.2^2\right)}{20E6 \cdot 1.0} = 4.32E-5$$

$$\alpha = \frac{h^2}{12r^2} = \frac{1.0}{12 \cdot 32.0^2} = 8.14E-5$$

$$\lambda = \frac{m\pi r}{l_h} = \frac{2 \cdot 3.14 \cdot 32}{37.5} = 5.36$$

(for buckling n,m $\geq$2)

$$C_1 = \left(1-\nu^4\right)\lambda^4 = \left(1-0.2^2\right) \cdot 5.36^4 = 792$$

$$\begin{aligned} C_2 &= \left(\lambda^2+n^2\right)^4 - 2\left[\nu\lambda^6 + 3\lambda^4 n^2 + (4-\nu)\lambda^2 n^4 + n^6\right] + 2(2-\nu)\lambda^2\nu^2 + n^4 \\ &= \left(5.36^2+2^2\right)^4 - 2\left[0.2 \cdot 5.36^6 + 3 \cdot 5.36^4 \cdot 2^2 + (4-0.2) \cdot 5.36^2 \cdot 2^4 + 2^6\right] \\ &\quad + 2(2-0.2) \cdot 5.36^2 \cdot 0.2^2 + 2^4 \\ &= 1.147E6 - 3.291E4 + 4.137 + 16 \\ &= 1.114E4 \end{aligned}$$

$$
\begin{aligned}
C_3 &= n^2\left(\lambda^2+n^2\right)^2-\left(3\lambda^2 n^2+n^4\right)\\
&= 2^2\left(5.36^2+2^2\right)^2-\left(3\cdot 5.36^2\cdot 2^2+2^4\right)\\
&= 3924
\end{aligned}
$$

$$
\begin{aligned}
C_4 &= \lambda^2\left(\lambda^2+n^2\right)^2+\lambda^2 n^2\\
&= 5.36^2\left(5.36^2+2^2\right)^2+5.36^2\cdot 2^2\\
&= 30891
\end{aligned}
$$

For stability:

$$
\begin{aligned}
C_1+C_2\alpha &= C_3\varphi_1+C_4\varphi_2\\
\varphi_1 &= \frac{\left(C_1+C_2\alpha-C_4\varphi_2\right)}{C_3}\\
&= \frac{\left(792+1.114E6\cdot 8.14E-5-30891\cdot 4.32E-5\right)}{3924}\\
&= 0.225
\end{aligned}
$$

To find critical q (q_{cr}):

$$
\frac{q_{cr}r\left(1-\nu^2\right)}{Eh}=\varphi_1
$$

$$
\begin{aligned}
q_{cr} &= \frac{\varphi_1 Eh}{r\left(1-\nu^2\right)}\\
&= \frac{0.225\cdot 20E6\cdot 1.0}{32.0\left(1-0.2^2\right)}\\
&= \frac{4.5E6}{30.72}\\
&= 146{,}484\ \mathrm{KN/m^2}
\end{aligned}
$$

Critical pressure $q_{cr} = 146{,}484\ \mathrm{KN/m^2}$ is much higher than the maximum lateral earth pressure $q = 605\ \mathrm{KN/m^5}$.

Therefore there is no problem of stability.

REFERENCES

1. Timoshenko, S., Gere, (1961), "Theory of Elastic Stability", McGraw Hill.

2. Xanthakos, P.P., (1994), Slurry Walls as Structural Systems, McGraw Hill.

3. British Standards, 8110, Specifications for Concrete Design.

4. Peck, R.B., (1969), "Advantages and Limitations of the Observational Method in Applied Soil Mechanics," Geotechnique, Vol 19. No. 2, pp. 171-187.

5. Dunnicliff, J., (1988), Geotechnical Instrumentation for Monitoring Field Performance, John Wiley & Sons.