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LHC CIVIL ENGINEERING CONSULTANCY SERVICES

PACKAGE 02

DESIGN METHODOLOGY REPORT

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JULY 1996

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LHC CIVIL ENGINEERING CONSULTANCY SERVICES

PACKAGE 02
DESIGN METHODOLOGY REPORT

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1 INTRODUCTION

1.1 Project Description

The Large Hadron Collider (LHC) project involves the construction of a new machine within existing underground and surface facilities which currently house the Large Electron Position accelerator (LEP). Additional new structures are required at several locations around the CERN sites. Civil Package 02 encompasses the civil works at Point 5 on the LEP ring. A new detector, the Compact Muon Solenoid (CMS), will be installed. This will require two new caverns, two new shafts, several tunnels and galleries and a number of surface buildings in steel and concrete construction.

1.2 Design Methodology Report Objective

The Consultancy Services Contract requires in the production of a Design Methodology Report in Phase I, describing in principle the methods to be used for the design of the Underground Structures, Surface Structures and Other Works. This report fulfils this requirement.

The objective of the Report is to set out the design philosophy and design criteria for the works, establishing agreed principles of design before the commencement of Phase 2.

1.3 Design Methodology Report Scope

LHC Civil Package 02 is divided into the Advance Works and five phases for the main works. Phases 1, 2 and 3 cover the Preliminary, Tender and Construction Design respectively. Phases 4 and 5 cover the Site Supervision and Defects Liability Period.

This Design Methodology Report is produced as part of Phase 1. It gives general information for all phases, but is specifically addressed at phases 2 and 3 of the works, plus the Advance Works. The methodology covers the design of the Civil Engineering works at Point 5 on the LHC project.

1.4 Design Objectives

The general objective of the Design is to provide buildings and structures which meet the requirements of CERN. These overall requirements are developed, with input from CERN, through the design process into the final design.

Key requirements for underground structures are:

- Robust design to accommodate the anticipated range of ground conditions with flexibility, as necessary, to allow for alteration of type and quantity of rock support depending on the actual ground conditions encountered
- All structures to be concrete lined, with no visible signs of water ingress and with a sealant on the exposed surface to reduce airborne dust where required by CERN
- Inner faces of concrete lining to remain outside CERN spatial envelope with appropriate tolerances provided to nominal dimensions.
- No structural maintenance throughout the lifetime of the facility (50 years)
- No water pressure to build up behind the inner concrete linings (except shafts)
- The external periphery of the shafts should not form a path for migration of water from the moraine to the underground structures
- Account is taken of the interaction between the various components of the Underground Structures, the Surface Structures, the Other Works and the existing structures

Additionally, it is required that ground movements in the molasse are to be controlled in order to limit groundwater flow into the drainage system and additional loading on the structure.

It is CERNs objective to construct the civil works in a manner consistent with these requirements to least reasonable cost within the overall CMS programme and with a low risk of cost escalation.

1.5 Design Development

The design of the buildings and structures is developed through a series of phases. In Phase I, Preliminary Design, sufficient design is carried out to develop and confirm the feasibility of the layouts and permit preliminary costing and programming of the construction works. A 'frozen layout' is established as the basis for the further phases. These are reported in the Phase 1 Summary Report.

For the purpose of the Phase I Preliminary Design the `frozen layouts' indicate the key positions and internal dimensions of the works, which should require no further amendment during the remainder of the project. These form the agreed basis on which the Tender Design can proceed. Dimensions which do not impinge on the internal space, such as concrete lining thicknesses, will be defined as the design is developed.

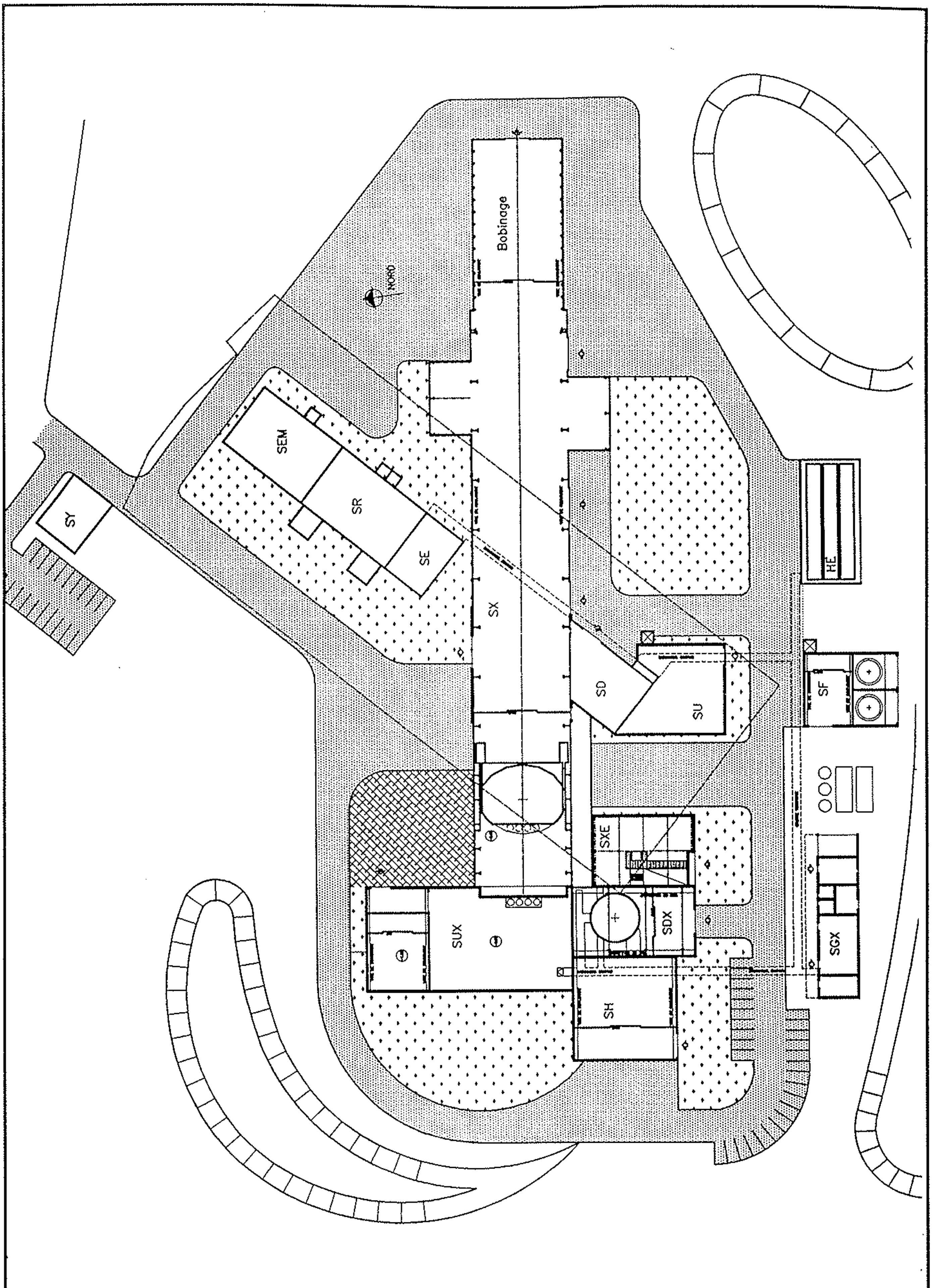
During Phase 2 the design is developed to form the basis of the Tender Documents. During Phase III the Construction Design and drawings are produced.

The CERN layouts for the underground structures are provided on drawings LHC CE1 3500 0001 to 0011. The CERN preliminary layouts for the surface buildings are given in drawings `Plan Remis Par L'architecte', sheets 1 to 5 and `Plan Niveau', No. LHC PAU1 3599 0003.

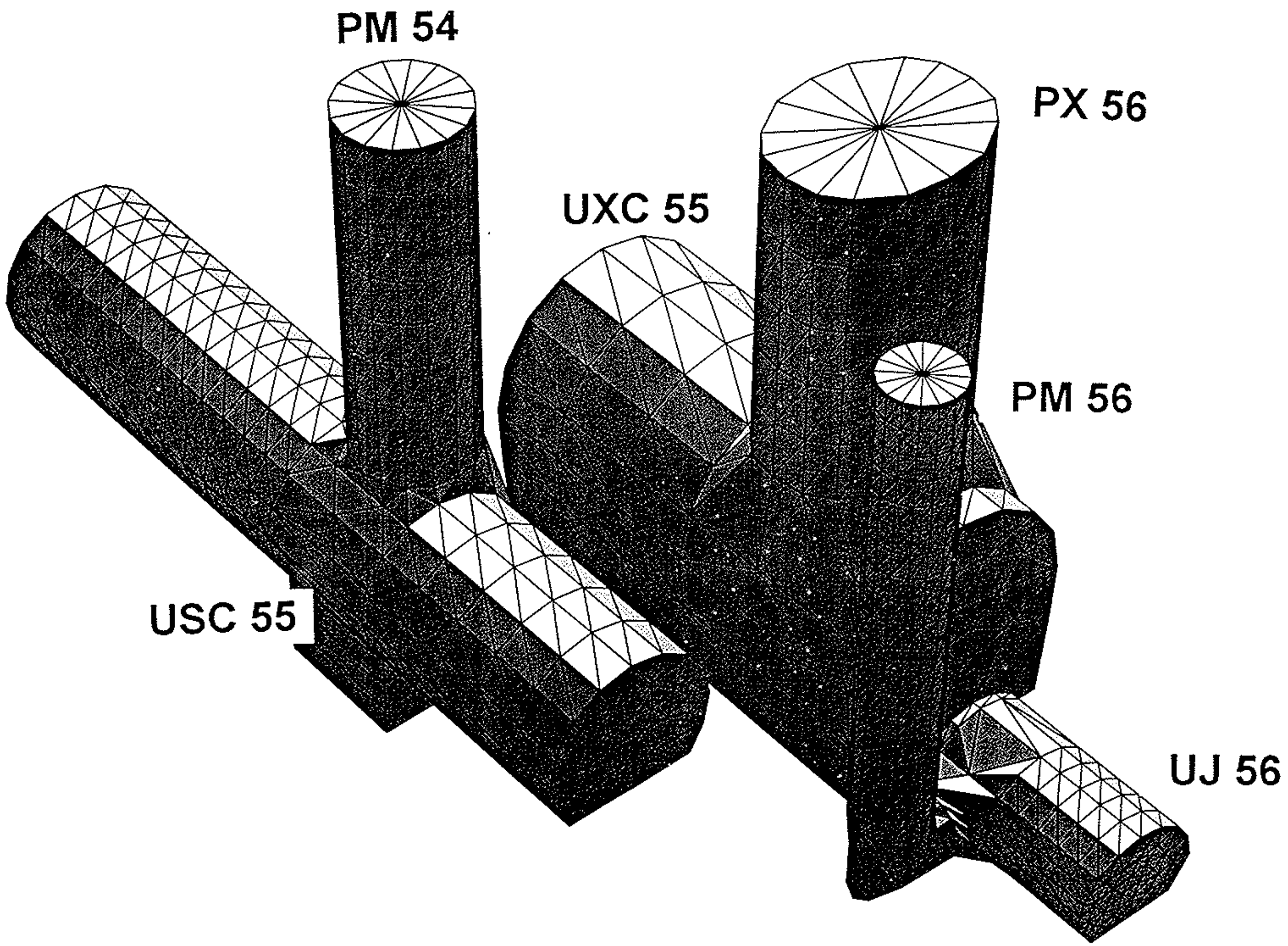
The required layouts are undergoing a process of development and refinement as the design proceeds. The detailed dimensions of the surface buildings are therefore subject to change. The dimensions given in the text and on the accompanying drawings are based on the layouts originally provided by CERN and should be regarded as approximate. The details in this report are sufficient for the purposes of defining the methodology. The accurate dimensions will be given in the Frozen Design, in the Summary Report.

Layout sketches of the Surface Buildings and the Underground Structures are given on figures 1.1 and 1.2 overleaf.

The geotechnical data for the design is taken from the Geotechnical Interpretative Report, GIBB-SGI-GeoConsult Document 02.



GIBB / GEOCONSULT / SGI INGENIERIE	CERN - LHC PROJECT- POINT 5	J96107A	
	LAYOUT OF SURFACE STRUCTURES	Date JULY 1996	Figure 1.1



GIBB / GEOCONSULT / SGI INGENIERIE	CERN - LHC PROJECT- POINT 5	J96107A	
	LAYOUT OF UNDERGROUND EXCAVATIONS	Date JULY 1996	Figure 1.2

2 DESIGN OF SURFACE BUILDINGS AND OTHER WORKS

2.1 Design Criteria

2.1.1 Design standards

- *Structures :*

  - Swiss standards SIA :

160 :	Actions on structures
161 :	Steel structures
162 :	Concrete structures

- *Roads and Parks :*

  - Swiss standards VSS (Union des professionnels suisses de la route)

- *The construction details shall be designed according to the French legislation*

2.1.2 Material properties

• *Structural Concrete:*	*According to SIA*	*According to Eurocode*
In general:	B 35/25 35 = average strength 25 = minimum strength on cube	C 25/30 25 = strength on cylinder 30 = strength on cube
For precast concrete and elements with very high stresses:	B 45/35 or higher	C 35/45 or higher

• *Reinforcement:*	Yield strength at 0.2% strain	$f_y = 460\ N/mm^2$
	Characteristic yield strength	$f_{yk} = 500\ N/mm^2$
	Rupture strength = ftk	$f_{tk} / f_{yk} > 1.08$
	(SIA standards: S 500)	

• *Structural Steel:*	Standards:	EU 25-72	Fe 360 or	Fe 510
		EN 10025	S 235 or	S 355
		SIA	Fe E 235 or	Fe E 355
	Yield strength		$235\ N/mm^2$	$355\ N/mm^2$

2.1.3 Loads

2.1.3.1 General loads

(a) Permanent loads

- *Density of reinforced concrete* — *25.0 kN/m^3*
- *Density of structural steel* — *78.5 kN/m^3*
- *Density for other materials* — *according to SIA 160*

Roofs

- thermal insulation and waterproofing	*0.15*
- gravel 5 cm	*1.0*
- service installation	*0.5*
	1.65 kN/m²

- *Metallic external walls*

- internal bended sheet metal	*0.12*
- external corrugated metal	*0.08*
- thermal insulation	*0.10*
	0.30 kN/m²

(b) Service loads

The general service loads are :

• *Stairs*	q_r	*4.0 kN/m²*
• *Pavements : if nothing else mentioned*	q_r	*200.0 kN/m²*
• *Snow load (H = 510 m)*	q_r	*1.0 kN/m²*
• *Wind load in rural environment*	q_r	*0.9 kN/m²*
• *Earthquake Work Class Co II*	*Seismic Zone Z1*	

[ie. Low risk zone, medium building importance class]

2.1.3.2 Particular Loads

(a) SX

(a1) Floor Loading

The CMS detector is to be assembled and tested in the SX Building. Subsequently, it will be dismounted and lowered into the UXC 55 cavern. It is therefore appropriate that the various loads must be taken into consideration at each phase of the assembling operation.

The load information provided by CERN is as follows:-

The CMS detector comprises a central barrel weighing 2500 tonnes. During the first test a 600 tonnes load is added. The weight of the central barrel is then increased to 3100 tonnes. This is supported on two 1 x 2.5 m supports during movement. The load per support point is therefore 1550 t at this time. More typically the load on each support is 1255 tonnes.

The outer barrels (one on each side of the central barrel) weigh 1800 t each, and are each supported on two 1 x 2.5 m supports. The two ends are comprised of 3 members having the following weights :

endcaps RF1 and - 1	:	1300 t each
endcaps RF2 and - 2	:	740 t each
endcaps RFS and - 3	:	380 t each.

Thus, the total weight of the CMS detector is :

$$2500\,t + 2\left[2 \times 1800\,t + 1300\,t + 740\,t + 380\,t\right] = \underline{14540\,t}$$

This load is spread over a length of 20 meters. Its position is central with respect to axes 9 and 10 of the SX Building.

Each member is mounted separately and then moved by means of air pads to its testing position along a pre-defined sliding area. The first member to be positioned is obviously the central barrel. Each outer barrel, after it has been mounted, is moved until it abuts against the preceding member.

In each wing there is a load of 700 tonnes spread over 1 x 4 m.

Furthermore a load of 30 t/m^2 can act anywhere else, over an areas estimated as 10 x 5 m, i.e. a total load of 1500 t.

See appendix A.1.

(a2) Travelling bridge crane

- *From axis 1 to axis 13* *- See Appendices A.2.1 and A.2.2* — *Load Capacity* *80 t*
- *From axis 10 to axis 18* *- See Appendix A.3* — *Load Capacity* *20 t*

(a3) Ventilation equipment

- *Room between Axis 1 and 2 :* *to be determined*
- *Equipment on the roof - Extraction Turrets* *to be determined*

(b) **SXE**

- *False floor self weight* — *1.0 kN/m²*
- *Screed 5 cm* — *1.2 kN/m²*
- *Service load on slabs/false floors* — *5.0 kN/m²*

(c) **SUX**

- *Travelling bridge crane :* *Load Capacity* *10 t*
  - See Appendix A.4

(d) **SH**

- *Travelling bridge crane :* *Load Capacity* *20 t*
  - See Appendix A.5

(e) **SDX**

- *Travelling bridge crane :* *Load Capacity* *80 t*
  - See Appendix A.6

(f) **SEM**

- *False floor self weight* — *1.0 kN/m²*
- *Screed 5 cm* — *1.2 kN/m²*
- *Service load on slabs/false floors* — *5.0 kN/m²*

(g) **SGX**

- *pressure produced by an explosion*
  on the walls — *1.5 kN/m²*
  In accordance with design review meeting No. 4 of 18.4.96

- *the roof shall be frangible at this pressure*

(h) **SY**

- *Service load* — *5.0 kN/m²*

(i) **HE**

- *Service load* — *to be determined*

(j) **SF**

- *Travelling bridge crane* *Load Capacity* *3.2 t*

- See Appendix A.7

(k) SD

- *Existing building - no additional loading requirements*

(l) Services galleries

The services galleries support either the service loads from the buildings or from the parks and roads according to their situation.

For the technical equipment, a permanent load of 5 kN/m^2 is taken along each wall.

(m) Roads and parks

- *The outdoor areas are defined by appendix A.8.1*

Loads assumed :

- Area 1 :	*distributed loads :*	*5 kN/m^2*
	local heavy loads :	*Worst case of CERN data: trailor with 2 tractors. See appendix A.8.2., and French vehicle loading standard*, wheel loading to system B and patch load type D and E at 40 kN/m^2*
	These loads act on the galleries and circulation areas.	
- Area 2 :	*distributed loads :*	*100 kN/m^2*
	concentrated loads :	*75 kN on 400 x 400 mm* = 46 tons/m^2
- Area 3 :	*no loads (landscaped area).*	

Area1 will be designed with Swiss roads standards (Normes VSS). Area 2 will be designed with the same standards as structure elements.

* Ministére de L'equipment et du logement, Fascicule no.61, titre II, Programmes de charges et épreuves des ponts-routes.

2.1.4 Resistance to environmental conditions

2.1.4.1 Limitation of crack widths

- *In accordance with SIA 162 for usual requirements*

2.1.4.2 Protection against aggressive soil conditions

- *Based on the available ground investigation information and as noted in the Geotechnical Interpretative Report, Document 02, the soil has no aggressive characteristics*

2.1.4.3 Cover to reinforcement

- *In accordance with SIA 162, item 4.32.2 :*

- for surfaces against formwork	*:*	*30mm*
- for surfaces without formwork	*:*	*35 mm*

2.1.4.4 Impact of Subsurface Excavation

It is recognised that subsurface excavation will induce surface settlement which may potentially impact on both existing and proposed surface buildings. The magnitude of ground surface movements will be confirmed during Phase II and surface structures assessed for their sensitivity to anticipated strains.

2.1.5 Ground bearing parameters

The surface buildings are situated in an area of compact Till subsoil. The nature of the subsoil conditions and preliminary recommendations for foundations are presented in the Geotechnical Interpretative Report. These recommendations are summarised here for completeness as follows:

- *Shallow foundations*

  Low rise and lightweight buildings will generally be founded on shallow foundations. Typically, with a foundation level at 1.5 meters below ground level, the allowable bearing pressure will be about 300 kPa.

  Depending on the subsoil under a particular building, the geometry and nature of the applied loading and the tolerable settlements, it may be appropriate to amend the *allowable bearing in specific circumstances. Where such circumstances arise the allowable bearing pressure will be confirmed during Phase II.*

- *Deep foundations*

  Heavily loaded structures may need to be founded on deep foundations in the sandy gravel Fluvio-glacial deposits (Lithostraphic Unit N° 6a), or in the Wurmienne Gravel Moraine (Lithostraphic Unit N° 7).

These layers provide a competent bearing stratum with the length of the deep foundations being designed with two criteria :

- *concrete strength*
- *ground bearing parameters.*

2.2 Structural types

2.2.1 SX and Coil Winding Building

2.2.1.1 General characteristics

See appendices B.1.1, B.1.2, B.1.3, B.1.4, B.1.8 for layout drawings.

The SX building is a large hall which is internally 182.90 m long. The hall is divided into two distinct sections :

- a main hall for assembly operations, in which is installed a 80 tonnes travelling bridge crane, having the following approximate external dimensions :

• *length*	:	*140.7 m*
• *width*	:	*23.9 m*
• *height*	:	*23.0 m*

- a hall designed for coil winding operations, equipped with a smaller 20 tonnes travelling bridge crane, having the following approximate dimensions :

• *length*	:	*42.4 m*
• *width*	:	*22.1 m*
• *height*	:	*14.0 m*

The SX building is to be constructed in two stages. Upon termination of the underground construction work the building above the shaft is completed. After installation of the CMS detector, the easterly part of the building including the coil winding hall will be demolished. This is discussed more fully in Section 2.3.

The SX building comprises a steel frame covered with a lightweight cladding :

- the walls are constructed from self-supporting metal insulating boxes/casings (8 cm thick) placed horizontally, fastened to vertical posts and covered with a thermally insulating layer 3 cm thick, which in turn has a cladding of vertical galvanised metal sheets having an enamel coating on the outer face.

- the roof is constructed from a self-supporting ribbed metal sheet made of galvanised stainless steel fastened to purlins and covered with a thermally insulating layer 8 cm thick and a composite waterproofing layer. It includes two sections sloping transversally with a 3% slope to facilitate the drainage of rain water.

The coil winding building and SX building extensions, which are to be removed at a later date, will have an economical roofing system appropriate for a 5 year design life.

2.2.1.2 Superstructure

The two sections of the building are each designed differently.

a) Assembly building

The main hall must be designed in such a manner as to satisfy the two following requirements :

- limit as much as possible the volume of the building : the proposed solution limits the building height to about 23 m.

- allow the lowering of the roof by 8 m in the final phase of use of the hall : the resulting requirements placed on the steel frame and on the walls are examined in Section 2.2.1.6.

The steel frame of this hall comprises fixed base portal frames at 12 m centres consisting of:

- two deep, fabricated column sections (depth 1600 mm generally, 1400 mm on axes 1, 2 and 3) for the 80 tonnes travelling bridge crane and the double upper frame. Each of these steel columns includes 5 circular holes through the web 80 cm in diameter, distributed along their height.

- a double frame seated on the head plates of the large lower columns, consisting of profiled HEB columns and of transverse profiled HEB supporting beams.

- profiled IPE purlins placed between the transverse beams, spaced 2.89 m apart and supporting the various roofing layers.

- crossbracing members placed horizontally in the roof and vertically in the plane of the walls to ensure lateral stability.

A small portion of the roof over the PX56 shaft shall be removable in order that the temporary 2500 tonnes crane cables can be lowered into the building and down the shaft.

b) Coil winding building

This hall will be demolished in the final phase of operations as noted previously. The steel frame and the cladding are easily dismounted and could be assembled again on another site should the need arise.

The load-bearing structure of this hall consists of fixed base portal frames spaced approximately 8.4 m apart and comprising:

- profiled HEB columns, to which are attached supports for the 20 tonnes travelling bridge crane

- profiled HEB rafters

- continuous profiled IPE purlins

- crossbracings members placed horizontally in the roof and vertically in the plane of the walls to ensure lateral stability.

2.2.1.3 Foundations

Due to the static system adopted, the foundations of the steel framework must accommodate, in addition to a vertical load, a fixed end moment. Furthermore differences in subsidence must be limited.

Deep foundations are most appropriate for meeting these requirements, both from the technical and from the economical point of view.

In order to accommodate the fixed end moment, a pile will be positioned beneath each column and load will be shared between pile and foundation slab of the hall.

2.2.1.4 Foundation slab

The foundation slab of the SX hall must be capable of withstanding very high loads, as noted in Section 2.1.3.2. These loads cannot be satisfactorily supported by shallow foundations.

A combined raft and piled foundation system will be used. This system is economical since load will be shared between pile and foundation slab.

Piled foundations will be placed at the locations where high loads are anticipated, such as beneath the sliding and installation areas, and in the detector test area (axes 7 and 8).

2.2.1.5 2500 t crane foundations

A temporary crane with a 2500 t load capacity will be used to lower the CMS detector components underground.

Following discussions with CERN, two heavy lifts specialist organisations have put forward proposals for undertaking the work, including outline details of the necessary foundations for the crane.

The large loads are such that piled foundations will be adopted in a design which could be used for either of the two proposed systems.

The adoption of pile foundations for these temporary structures will minimise impact of the large crane loads on shaft lining.

2.2.1.6 Roof lowering

In order to lower the roof of the Assembly hall of the SX building without having to resort to sophisticated and expensive equipment, we propose to segment by spans the upper frame situated above the large steel columns and use transverse double frames to form self-supporting rigid stool-shaped elements which are disposed contiguously.

The method proposed for lowering the roof is as follows

1) dismantle the walls about 8 m, using removable panels

2) use four lattice towers with four motor winches (the total weight to be lowered amounts to the total weight of one bay element : 40 tonnes).

3) dismount one bay at a time by unbolting the sections of the columns and of the vertical posts of the walls

4) lower the rigid bay elements

5) bolt the columns and the vertical posts of the walls at level + 9.20 m.

6) place joints on the walls and on the roof.

2.2.1.7 Movable concrete shield

A movable concrete shield which is two meters thick is required to cover the PX 56 shaft when the LHC is in operation and to slide back to the west end of the SX building during shutdown. Furthermore, four ventilation ducts running up the PX56 shaft wall must pass beneath this movable shield. In addition to acting as a shield, this movable component must also be capable of supporting the detector component parts before they are lowered underground, taking into account that the heaviest component part weights approximately 2500 tonnes.

This concrete shield will be designed by CERN, design of the adjacent floor and foundations will take account of the shield loading.

Above the ventilation ducts, the concrete layer will be at least 1 meter thick.

2.2.1.8 Services gallery

The SX building floor slab must pass over the existing service gallery. The thickness of the slab over the gallery in this location will control the SX building floor slab. It is therefore proposed to provide a heavily reinforced slab in this area which spans to piled supports arranged parallel to and alongside the gallery. Under the heavily loaded area the reinforcement will be HEB. This will avoid the need to lower the roof of the gallery and ensure lateral loads acting on the gallery are not increased. A separation layer will be provided between the roof of the gallery and the floor slab to avoid load transfer to the gallery. A one metre wide access will be provided perpendicular to the gallery on the centreline of the building.

2.2.2 SXE

2.2.2.1 General characteristics

See appendix B.2 for layout drawings.

This building is constructed from reinforced concrete. It has two floors above the basement and its external ground dimensions amount to 17.6 m by 25.5 m. The building has a staircase and a lift. The slabs of the first and of the second floors, as well as a portion of those of the basement receive a false floor.

The roof slab comprised of two sections having a 3% slope receives the following layers : 10 cm of insulating material, a composite waterproofing layer, and a 5 cm protective layer of gravel.

2.2.2.2 Superstructure

The load carrying structure made of reinforced concrete consists of :

- 30 cm thick peripheral concrete walls cast in situ
- flat slabs 36 cm thick supported by the peripheral walls and the lift shaft and by two columns along the central longitudinal axis of the building.

2.2.2.3 Foundations

- continuous footings for the peripheral walls
- spread footings for the columns and for the lift shaft.

2.2.3 SUX

2.2.3.1 General characteristics

See appendices B.3.1, B.3.2 for layout drawings.

This building is constructed wholly from reinforced concrete, with plan dimensions of 35.6 m by 50.4 m. The building is divided into two sections :

- A northern section of 15.0 m by 25.0 m equipped with a 10 tonnes travelling bridge crane, with a final elevation of the floor at - 0.50 m
- A southern section of 34.8 m by 25.0 m, with a final floor elevation at - 0.50 m

The elevation of the parapet is +16.10 m.

The roof slab with two sections having a 3% slope receives the following layers : a 4 cm layer of a thermally insulating material, a composite waterproofing layer, and a 5 cm protective layer of gravel.

2.2.3.2 Superstructure

The load bearing structure made of reinforced concrete consists of :

- 30 cm thick peripheral walls, cast in situ

- a roof comprised of prefabricated transverse supporting beams spaced 5 m apart and prefabricated slab elements with in situ concrete infill, the final slab thickness amounting to 20 cm.

2.2.3.3 Foundations

Continuous footings for the peripheral walls.

2.2.4 SH

2.2.4.1 General characteristics

See appendices B.4.1, B.4.2 for layout drawings.

The SH building is a reinforced concrete structure with plan dimensions of 25.0 m by 25.60 m, equipped with a 20 tonnes travelling bridge crane. The elevation of the finished floor is 0.00, the elevation of the parapet is + 9.75 m. The slab forming the roof has two sections having a 3% slope and it carries a 4 cm layer of a thermally insulating material, a composite waterproofing layer and a 5 cm protective layer of gravel.

2.2.4.2 Superstructure

The load carrying structure of the main concrete building is the same as that of the SUX building (see paragraph 2.2.3.1)

2.2.4.3 Foundations

Continuous footings for the walls and the peripheral walls

2.2.5 SDX

2.2.5.1 General characteristics

See appendix B.5 for layout drawings.

The SDX building is a hall 17.4 m wide and 30.2 m long. The elevation of the parapet is +16.10 cm. The building is equipped with a travelling bridge crane.

This building is located between the SXE and SH concrete buildings and consists of a steel frame carrying a lightweight shell :

- the walls are constructed from self-supporting insulating boxes/casings (80 cm thick) placed horizontally and fastened to vertical posts, covered with a thermally insulating layer 3 cm thick which in turn is covered by a cladding of vertical metal sheets galvanised on the outer surface.

- the roof is comprised of ribbed self supporting sheets made of galvanised steel, fastened to the purlins and carrying a thermal insulation layer 8 cm thick and a composite waterproofing layer. It is in two sections with 3% slope to facilitate the drainage of rain water.

2.2.5.2 Superstructure

The steel frame of this hall comprises five fixed base portal frames at 7.30 m centres consisting of:

- 2 deep, fabricated columns sections (depth 1400 mm) for the tracks of the 80 tonnes travelling bridge crane and the upper frame; these steel columns each include five circular holes of 1100 mm diameter through the web

- an upper frame fastened to the head plates of the large lower columns, consisting of profiled HEB columns and transverse profiled HEA beams

- profiled IPE purlins placed 3 m apart supporting the various roofing layers

- crossbracings members placed horizontally in the roof and vertically in the plane of the walls to ensure lateral stability

2.2.5.3 Foundations

The magnitude of vertical load and fixed end moment permit shallow pad foundations to be adopted. These will be positioned eccentrically to the columns.

2.2.6 SEM

2.2.6.1 General characteristics

See appendix B.6 for layout drawings.

This building is contiguous to the SE and SR buildings and its dimensions are 17.0 m by 24.3 m . The elevation of the parapet is that of the SR building : + 7.30 m.

We have assumed that the building SEM is of the same nature as the building SR and is therefore of the same design ie:-

- a steel framework formed by self-supporting frames in the transverse direction and crossbracing members in the longitudinal direction

- a cladding consisting of insulating plates of the Durisol type or similar

- a roof of the same type as those of all the other buildings with a steel frame, including a galvanised ribbed sheet covered with a layer of thermal insulation material 8 cm thick and a composite waterproofing layer.

2.2.6.2 Superstructure

The steel frame consists of

- portal frames with HEB profiled columns spaced 6.0 m apart and profiled HEA support beams

- continuous profiled IPE purlins, spaced 2.7 m apart

- crossbracing members which are horizontal in the roof and vertical in the plane of the walls for lateral stability.

2.2.6.3 Foundations

Continuous footings along the entire periphery of the building

2.2.7 SGX

2.2.7.1 General characteristics

See appendix B.7 for the layout drawing.

The SGX building is 40 m long and 13 m wide. It is divided into three sections :

- a main building of 34.5 m by 10.0 m, including 6 closed rooms
- a covered passage 34.5 long and 3.0 m wide
- an outdoor storage area 5.375 m long and 12.75 m wide

Elevation of the parapet : 14.84 m.

2.2.7.2 Superstructure

All the walls are 25 cm thick, and as well as the two columns of the covered passage, are made of concrete, cast in situ.

The roof comprises profiled HEA rafters at 3.74 m centres with continuous profiled IPE purlins at 1.30 m centres.

The steel framework of beams and purlins is covered by aluminium sheeting. The roof cladding of the closed rooms must be capable of separation from the remainder of the building in an explosion occurring inside with a force exceeding 1.5 kN/m^2.

The frame of the covered passage consists of an edge truss beam spanning 4 columns spaced apart by 11.25 m and 12.00 m and receiving profiled IPE supporting beams spaced 3 m apart and three continuous profiled UPN purlins.

2.2.7.3 Foundations

The foundation slab under the load bearing walls is made of reinforced concrete 20 cm thick, with an edge beam for protection against ground freezing.

2.2.8 SY

2.2.8.1 General characteristics

See appendix B.8 for the layout drawing.

The SY building at Point 5 is identical to that of Point 6 with a plan area of 9.18 m by 10.68 m projecting outwards beyond the perimeter wall by 1.20 cm on all four sides.

The lightweight walls consist of profiled tubes of 50/30 « Foster » type or similar with enamel coated metal sheets and 5 cm of a thermally insulating layer.

The roofing is comprised of a ribbed galvanised metal sheet carrying an insulating layer 8 cm thick and a protected waterproofing layer. The elevation of the parapet is + 3.25

2.2.8.2 Superstructure

The steel frame of the roofing is comprised of bi-directional truss beams having a depth of about 60 cm spaced by 3.03 m in one direction and 3.03 m, 4.46 m and 3.03 m in the other direction.

This bi-directional framework is supported by sixteen intersections of beams on sixteen tubular RHS columns.

2.2.8.3 Foundations

The foundation slab is made of reinforced concrete 20 cm thick, with an edge beam for protection against ground freezing.

2.2.9 HE

The foundations necessary for receiving the helium tanks are presently being investigated by a firm commissioned by the CERN. We are waiting for their conclusions.

2.2.10 SF

2.2.10.1 General characteristics

See appendix B.9 for the layout drawing.

This building has a length of 23.0 m and a width of 18.0 m and houses two cooling towers. This corresponds to one half of the cooling tower arrangement at Point 6.

This building, made entirely of reinforced concrete, is divided into clearly distinct sections :

- one for housing a pump unit having a length of 17.5 m and a width of 11.55 m and which is equipped with a 3.5 tonnes travelling bridge crane

- two identical compartments having a length of 10.65 m and a width of 8.60 m, each one housing a cooling tower and exhibiting a wide opening on their back wall.

2.2.10.2 Superstructure

With the exception of the secondary framework of beams (continuous profiled HEA and IPE purlins, spaced apart by 1.50 m), the roofing of ribbed galvanised steel sheet of the « pumping » section (transversally oriented 3% slope), all the load carrying structures are made of reinforced concrete.

The walls, columns and the paving is cast in situ, whereas the support beams are prefabricated :

- main framework of beams of the pumping section, span 10.85 m, spacing 4.375.

- main framework of beams and secondary framework of beams in the compartments for housing the cooling towers.

- the different constituent components of the diffusers of the cooling towers.

2.2.10.3 Foundation

A large foundation slab for the entire building.

2.2.11 SD

2.2.11.1 General characteristics

This is an existing building with a steel frame and lightweight cladding.

The construction of the SX building requires :

- the demolition of the existing SU building and its reconstruction south of the building. This will require that several openings be made in the south facing wall for the passage of the ventilation equipment.

- the partial demolition of the north facing sections of the SD building and its connection to the facing wall of the new building SX.

2.2.11.2 Superstructure

The demolition at an angle of the north facing section will require that the existing load carrying structure be completed by columns and edge steel support beams before rebuilding the north facing wall.

2.2.11.3 Foundations

A continuous footing will be laid at the north wall.

2.2.12 Service galleries

The service galleries will be built from reinforced concrete. A sealant of the « Inertol » type or similar will be applied over the entire outer periphery of the gallery to improve watertightness.

2.2.13 Roads and parks

The outdoor areas are defined by appendix A.8.1.

There are 3 types of area :

- Area 1 : circulation areas for vehicles
- Area 2 : storage area
- Area 3 : landscaped areas.

Area 1 :

These areas are for vehicles, circulation and parking.

Roads are formed with a bituminous wearing course and base course, a gravel sub-base and a geotextile filter layer.

Roads have cross slopes and are equipped with a stormwater drainage system linked to the public network.

Kerb stones of either natural stone or precast concrete will be provided at the edge of the pavements.

Area 2 :

This is storage area. The slab will be of reinforced concrete with a special treatment to resist against frost and loads. Alternatively a precast concrete paving block system will be considered.

The slab will include movement joints.

Underneath the slab, there is a foundation layer and a geotextile similar to the roads.

Area 3 :

These areas are landscaped with material from the shaft excavation and covered with topsoil.

2.3 Construction Sequencing of Surface Buildings

See appendices C.1.1, C.1.2, C.1.3, C.1.4, C.1.5.

Several constraints must be taken into account when planning the construction, namely :

- *Dismantling of the Lep machine will begin in October 2000.*
- *Special areas must be set aside for allowing work on the shafts and on the caverns*
- *The CMS detector must be assembled and tested on the surface before being dismounted and taken down into its cavern*
- *One section of the SX hall must be dismounted at the end of the construction work and on the remaining section of this same hall, the roof must be lowered.*

Furthermore, the date at which each building is to come into operation depends on its function.

Stage 1 : Preparatory works such as modification of the mains drainage and advance works on the SU building

Stage 2 : The construction of the SX hall. This is on the critical path, since it fixes the beginning of the assembling of the CMS detector.

- *Demolition of the old SU building*
- *Transformation of the SD building*
- *Construction of the SY building*
- *Construction of the SX building axis 5 to 13 and of the coil winding hall.*

Stage 3 : Construction of buildings without any relation with the underground works

SF	Cooling towers
SGX	Gas Building
SEM	Electricity Building
HE	Foundations for helium tanks

As soon as the shaft PM 54 is finished, construction of the SH cryogenics equipment may commence.

Stage 4 :	As soon as the underground work and the outer concrete covering of the inner lining of the shafts are finished :	

Completion of the SX	:	construction of the second part of the SX building and concreting of the shield
Construction of the SUX	:	Ventilation building
Construction of the SDX	:	Building with PH 54
Construction of the SXE	:	Experiment control
Landscaping and road works		

Stage 5 : Transformation of the SX Building

- *Dismounting of the coil winding hall*
- *Partial dismounting of the SX hall (axes 9 to 13)*
- *Lowering of the roof.*

It is necessary that the services gallery construction takes place on a number of distinct stages to accommodate other works. The work must therefore be programmed around activities in stages 1 to 4.

2.4 Design methods

2.4.1 Calculation of stresses

Member stresses will be calculated based on linear elastic analysis of the structures considered.

For structures forming any plane or spatial system of members, the STATIK2 program, will be used based on the deformations method. It enables the calculation of the second order effects, as well as the eigenvalues in the case of a dynamic analysis. This program was initially developed by the Swiss Federal Institute of Technology of Zurich and is now available commercially from the firm Cubus AG in Zurich.

For the structures consisting of flat plates or of shell elements, the MAPS program will be used. This is a finite element analysis program which takes into account the deformations generated by shear stresses. This program may also be used for the analysis of the foundation slab on a elastic ground. This program, developed in the Swiss Federal Institute of Technology of Lausanne by Dr. A. Bouberguig is now sold commercially by the firm MAPS Diffusion SA.

2.4.2 Design

The design of the sections is carried out in accordance with the standards set out in section 2.1.1.

3 DESIGN OF UNDERGROUND STRUCTURES - PRIMARY SUPPORT

3.1 Design Philosophy

The engineering behaviour of the molasse formation, within which the principal underground structures are located at Point 5, will be closely related to the intrinsic nature of the rock mass and of the nature and extent of construction induced overstress and disturbance. A fundamental understanding of the rock mass geomechanical properties and their sensitivity and interaction with the proposed works is therefore essential for the safe and economic design of excavations and associated support elements. It therefore follows that excavation stability should be evaluated on the basis of representative characterisation and modelling of the rock mass to ensure reliable estimation of mass behaviour, and that a robust design should reflect the key aspects of the engineering behaviour of the molasse:

- non-homogeneity and variability
- stratified and bedded system of alternating rock mass properties
- near horizontally bedded, anistropic rock mass with low horizontal shear strength
- time, moisture and deformation dependant behaviour

The design of rock support will be based on a fundamentally based, analytical approach which recognises appropriate aspects of rock mass behaviour, both during construction and in the longer term over the service life, together with the construction sequence and processes. The rock support measures shall:

- provide sufficient rock support to achieve safe and stable excavation conditions in the roofs, side walls and floors in caverns and tunnels as well as in the shafts.

- ensure rock mass support adjacent to particular underground openings so as to maintain the global stability of the underground works

- permanently support local weak layers, such as layers of Marl Grumeleuse, to achieve permanently stable rock mass conditions for long term load cases.

- control deformation in the molasse rock mass to limit water seepage from the basal moraine aquifer down into the molasse rock mass.

All excavated openings will be supported by the systematic provision of rock support measures designed to accommodate loadings. It is recognised that unfavourable geomechanical or geometrical conditions will require particular consideration and that localised special support measures may be necessary in some areas. The rock support system may comprise the following measures which will be installed according to design requirements:

- Staged multiple drifts (SMD)
- Shotcrete
- Rock bolts
- Rock anchors
- Concrete
- Strengthening ribs of steel or concrete
- Rock mass grouting

The stability of caverns and tunnels will be ensured by achieving sufficient bearing capacity of the rock mass strengthened by rock support measures. During the detailed design phase, rock mechanical design calculations will be carried out to prove the adequacy of rock support measures proposed.

The support philosophy of all underground openings at Point 5 is based on staged excavation and incremental installation of rock support measures.

The benefits of progressive support enhancement will be explicitly considered in developing a cost effective and practical construction sequence.

Clay rich molasse units, and particularly the Marl Grumeleuse, are considered highly sensitive to moisture content changes. The sensitivity of these materials to moisture changes with associated deterioration in rock mass strength and the potential for long term swelling, require that measures are taken to inhibit moisture ingress. To ensure robust behaviour of the structure, accommodation will be made in the design for inevitable consequences of moisture changes.

Primary rock support will be designed to accommodate design loads from the surrounding rock mass. Hence, design loading for the secondary lining, which is discussed more fully in Chapter 4, will be limited to gravity, shrinkage, temperature and other local loads such as crane loads, with allowance for load attracted to the secondary lining by deformation of the rock mass as a consequence long term creep and swelling.

Within the Moraine, the primary support of the shafts will be achieved by an engineered initial support lining formed before bulk excavation of the particular shaft by either ground freezing or diaphragm wall techniques or similar. Following excavation, a final shaft lining will be formed. This will be designed to withstand the full hydrostatic loading. In the molasse, primary and secondary shaft linings may be treated in a similar manner to cavern excavations as primary rock support in the form of shotcrete and rock bolts will be designed to meet geomechanical and geometric constraints.

3.2 Design Methodology

3.2.1 Fundamental Considerations

The design of the excavation - support system for the shafts, caverns and tunnels is based on the following fundamental considerations :

- The rock mass is the main bearing element of the structures.
- The rock mass and the excavation - support system form a composite structure.
- The optimum support system is an integrated function of the excavation method and vice versa.
- The support system will be optimised during the design but will comprise, in general, both reinforcement of the rock mass and the provision of integrated primary lining systems.
- The support system is determined from both analytical design predictions and by experience from previous construction at CERN and elsewhere in similar rock types.
- The design process is continued throughout the construction period to verify the adequacy of the support and make adjustments as may be necessary for the actual conditions encountered.

A flow chart for the excavation - support design is presented in Figure 3.2.1.

3.2.2 Scope of Studies

The underground structures will be studied at a global level, in detail and also with specialised analyses to consider particular effects. The modelling and numerical codes will be selected to be appropriate for the particular level of study. The broad scope of the studies is as follows:

i) Base Model (Global Analyses)

- Stress/redistribution/3-D effects (for detailed 2-D model inputs)
- Potential overstress - ubiquitous bedding structures
- Effects of K (stress ratio)
- Effects of $\sigma_{h/}$ σ_n orientation wrt excavations
- Prediction of likely overstress zone
- Identification of critical zones wrt deviatoric stresses

ii) Design Model (Detailed Analyses)

- Intrinsic response of rock mass (unsupported runs)
- Comparisons of alternative design options
- Construction Stage Assessment and Options
- Suitability (type)/adequacy (loading capacity) of support system(s).
- Engineering assessment of stability (in terms of overstress ratios)
- Induced surface effects
- Preliminary analysis of strength - deformation sensitivity due to ongoing (inferred) state change effects.

iii) Long-term state model (Specialised Analyses)

- Time - dependant state analysis (progressive material state change) with overstress - deformation outputs
- Swelling (crown zone, invert zone, sidewall zone/localised) with water interaction potential (fissure permeability)
- Water (forces, secondary (fissure) permeability development)
- Special features - major geological structures modelled in space (if present)
- Stability assessment of shaft - cavern intersection zone

3.2.3 Design Phases

As previously noted, at the completion of Phase I preliminary studies will have been completed on the principal elements of the underground works, in order that the internal geometry may be frozen, and a rock mass behavioural model may be developed. These will form the two primary inputs to Phase II.

Outline methodologies for design development in Phases II and III are as follows:

Phase Two - Tender Design

More detailed studies will be performed on the potential behaviour of the rock mass during excavation of the underground structures. These studies will include numerical simulations of the soil and rock mass considering the main features such as bedding planes and joints to determine potential typical deformation and failure scenarios. Interaction of the different excavations and sequence options will be studied to determine the extent of potential overstressing and hence an optimum excavation - support arrangement.

These detailed studies would be principally performed using 2-D rock mass simulation models such as UDEC. This is discussed more fully later in this chapter.

The calculations will be performed first using models of the unsupported excavations to determine the intrinsic stability limits of the rock mass. These will then be re-analysed to include support elements. The support will then be optimised using an iterative procedure until the design criteria are met.

Pre-excavation support measures will be considered, including cast-in-place pile systems, incremental excavation, shotcrete and rock reinforcement measures. The excavation sequence will need to address both optimum excavation methodology as well as efficient and effective shaft and cavern support measures.

The potential for the marls to swell will be fully accounted for in the study and incorporated as appropriate into the design procedures. It is proposed that a number of parallel alternative approaches will be used in the first instance to determine sensitivity of the lining design to both direct and indirect swelling pressures and the redistribution of load as a consequence of stiffness degradation.

The potential for strain induced permeability changes in the molasse, particularly between the large cavern roofs and the moraine/molasse interface will be investigated at this stage. Inferred seepage gradients and pore water pressures will be modelled as appropriate, with associated effective stress and stress-strain characteristics to determine long term (time dependant) rock mass - support interaction.

Experience from other cavern precedence in related rock mass conditions will be reviewed at this stage and a focused assessment of precedent carried out, including consideration of information from the previous construction of tunnels and caverns for the LEP. Proposed details will be reviewed in the light of this precedent. Particular factors which will also be considered in the context of the LEP data will be the treatment of shafts within the molasse to avoid transmission of water to cavern linings and the treatment of marl layers.

Based on the above studies and following an Expert Review process, the tender excavation - support system will be derived. It is intended to provide flexibility in the excavation sequence in the context of rational support solutions and incremental stabilisation requirements.

Following on from the Expert Review, consideration will be given to the identification of all existing and ongoing critical areas for detailed consideration during Phases Three and Four. A monitoring system will be designed which will be capable of verifying the planned behaviour of the structures or any deviations thereof.

Phase Three - Detailed Design

Phase Three studies are intended to produce the detailed designs based on a more critical appraisal of particular design elements with the assistance of additional test data.

The scope and execution of the detailed design will basically follow that of Phase Two but with greater emphasis on key design issues.

Following a further review of the detailed designs, a final risk assessment will be carried out and appropriate contingency (variation) measures will be derived to cover all possible event scenarios. Such a procedure is considered to be particularly important in areas where the proposed construction may be demonstrably beyond precedent.

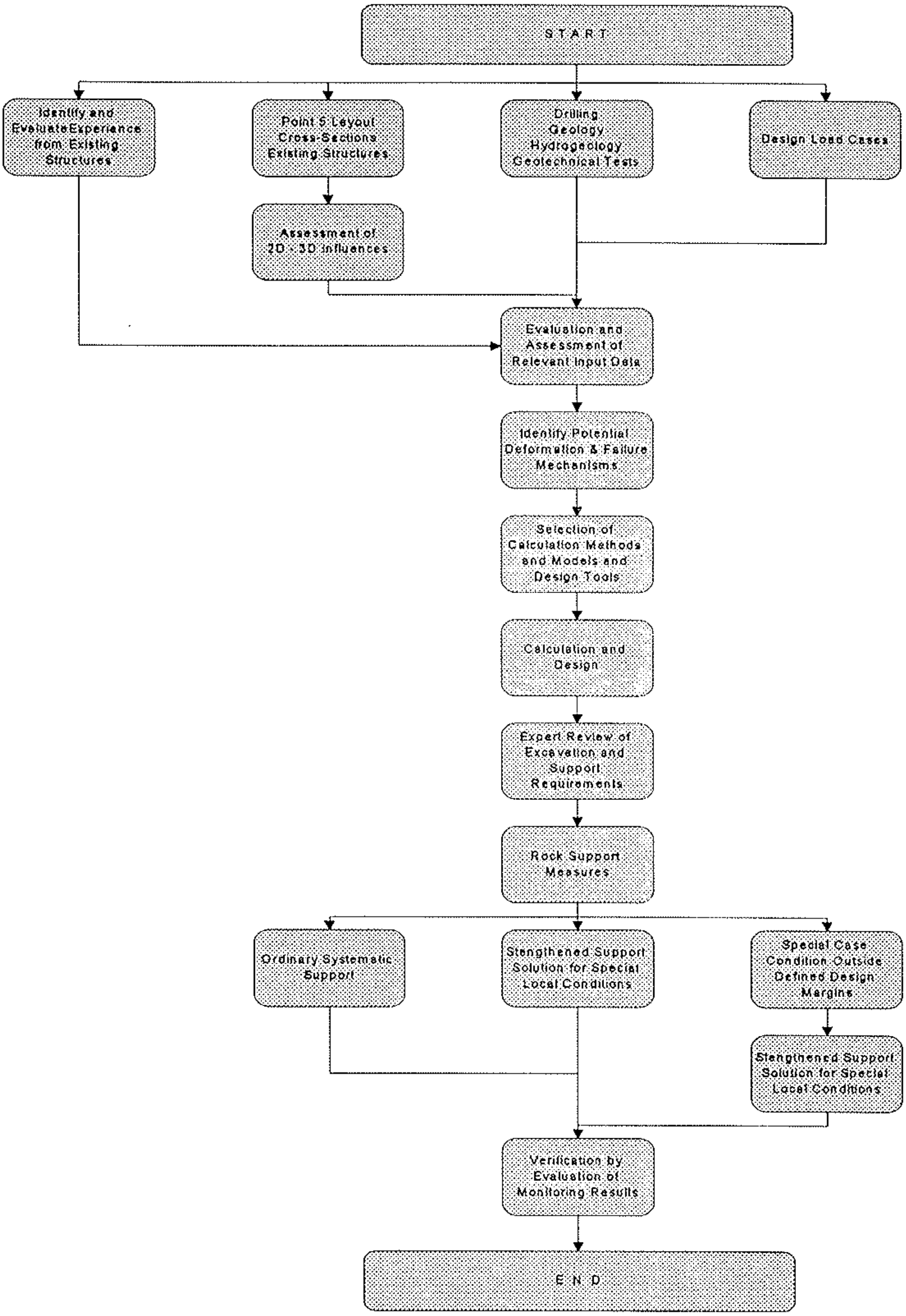
The monitoring control system will be further reviewed and refined as necessary.

3.3 Geotechnical Model

At Point 5, ground conditions consist of about 45 m of dense, permeable and predominantly granular moraine over an alternating sequence of very weak to weak marls and weak to strong sandstones, with transitional sandy marls and marly sandstones, which are collectively termed molasse. The molasse contains tectonised and pre-sheared layers and discontinuities of low shear strength.

Based primarily upon information derived from the 1995 investigation and within the context of the Geneva basin, a geological model has been constructed for Point 5 in order to be able to predict the geological materials which are likely to be encountered in the shafts, caverns and other underground excavations. This model is described in the Geotechnical Interpretative Report.

Figure 3.2.1. - Principle Steps of Design Methodology



It is fundamental to the design process as it forms the framework for geotechnical characterisation and subsequent analytical work.

The geotechnical characterisation has been derived from a study of all pertinent geotechnical data available for the moraine and the molasse. This includes data previously acquired and evaluated for the LEP project, together with that of the 1995 ground investigation. Within the molasse, the geomechanical properties of the rock mass are stratigraphically controlled. The weaker lithologies, such as the marl grumeleuse and marl tectonisée, as well as the poorly cemented sandstones, therefore exert primary control on mass behaviour. Particular attention has therefore been paid to the determination of moderately conservative design parameters for these visits.

Discontinuities within the rock mass predominantly comprise bedding partings of very low or zero tensile strength between alternate units. These have been characterised as pre-existing bedding planes.

Rock mass units have therefore been defined with explicit characterisation of low strength discontinuities, pre-sheared zones and interfaces in order that all geological features identified in boreholes may be specifically incorporated into numerical models. The following units are therefore defined and modelled:-

1.	Marl Grumeleuse	MG
2.	Marl Grumeleuse interface	mgi
3.	Sandstone weak	SW
4.	Sandstone strong	SS
5.	Sandy Marl	SM
6.	Marl (laminated)	ML
7.	Marl boundary	mb
8.	Moraine	G

It should be noted that the engineering behaviour of the molasse is both a function of its intrinsic properties and the nature and extent of construction-induced overstress and disturbance. The geotechnical characteristics presented in the Geotechnical Interpretative Report reflect the intrinsic and time independent aspects of the rock mass, as inferred from available data, and preliminary evaluation of primary stability mechanisms. Further refinement of the geotechnical model will therefore be required during Phase II as part of the design development to take full account of interactive and progressive effects leading to strength and/or stiffness degradation. This is discussed more fully in subsequent sections.

3.4 Load Cases

3.4.1 Static Load (Excavation)

Prior to excavation for LHC works the rock mass is in a primary stress condition, influenced by the existing structures of LEP at Point 5. The primary stress condition is governed by the overburden weight and the horizontal initial stresses.

The excavation of the shafts and cavern complex for the LHC causes stress re-distributions, destressing and deformation of the rock mass. This load case will govern the design of the primary supporting systems of the underground openings.

3.4.2 Swelling

After excavation and application of support, moisture changes in the environment of Marl grumeleuse layers in the proximity of excavation surfaces may activate swelling effects.

Swelling pressures will be considered in the design of primary support systems and secondary linings. The primary support will be designed to withstand the potential swelling pressures and/or to allow for the deformation necessary to release pressure. Where swelling occurs after installation of the secondary lining, deformation will be induced in the complete system rock mass / primary support / secondary lining. The compatibility of the secondary lining with these deformations will be checked in the design.

3.4.3 Creep

It is expected that the deformation of the rock mass caused by the excavation of the shafts and caverns will comprise instantaneous and time-dependent (creep) components. Both components are considered for the design of the primary support by choice of appropriate deformatbility parameters. After completion of the excavation, some small creep deformation will be observed that may induce deformation in the secondary lining.

In addition the behaviour of the marl is sensitive to time, moisture, deformation and stress state. The effect of strength reduction due to change in state will also be assessed. This will influence both the primary support and the secondary lining in a similar way to normal creep.

3.4.4 Water Pressure

Moraine Gravels:

Design of primary support and secondary lining in the moraine gravels will consider full height of water loads up to a level of approximately 502.5 maSL.

Molasse:

The Molasse in undisturbed conditions is practically watertight. The design of the excavation and primary support is focused to maintain this property. Water pressure within the molasse caused by seepage from the Moraine shall be minimised by measures to control the rock mass deformation and, where appropriate, by preventive drainage measures.

During the excavation and primary support phase water pressure adjacent to the excavation will be relieved. Drainage measures may also comprise drilling of drainage holes at specified patterns to prevent the build up of water pressure prior to installation of the final lining. As the long term effectiveness of drainage of the rock mass cannot be guaranteed, a hydraulic gradient from the moraine to the final lining will be considered for the final, long term design condition. The effect of water pressure within the rock mass is primarily to create `effective stress' conditions, rather than to impose an external load on the system.

Since drainage systems will be installed at the interface between secondary lining and rock, no water pressures will be considered for the normal load cases on the secondary lining. This is discussed more fully in Chapter 6.

Where grouting of the molasse is considered, the effect on the rock mass and the lining will also be assessed.

3.5 Numerical Analysis

3.5.1 Modelling Strategy

It is recognised that the complex layout of large caverns, shafts and other underground structures will result in a complex stress field within the rock mass dependant on the precise nature and sequence of construction works undertaken. The design of rock mass support must reflect the true nature of these interactive and progressive conditions through the numerical modelling undertaken.

This section describes the general modelling strategy to derive relevant design values:

Global Analysis

A combination of two-dimensional (2D) and three dimensional (3D) analyses will be undertaken to determine primary stability mechanisms for various structures within the framework of the overall stress state. The 3D analyses will focus only on major cavern and shaft structures as these control the global stress field.

Detailed Analyses

2D modelling of structures will be undertaken to interactively determine appropriate excavation - rock support strategies. Confirmation of the adequacy of proposed support systems will be made in the context of the global items analyses.

Sensitivity Analysis

2D modelling will investigate the sensitivity of particular and appropriate sections to variations in key input variables. This will include variations in the geological model.

Analysis of Long Term Behaviour

Analysis of potentially sensitive structures will be undertaken using a psuedo non-linear procedure within a 2D model. Iteration will enable an appropriate stress-strain compatible equilibrium to be derived to reflect to progressive strain (creep) and swelling. Re-distribution of load will be explicitly accounted for in excavation-support analysis.

3.5.2 Programme Codes

(a) Examine - 3D

For the assessment of three dimensional behaviour of the underground structures at Point 5 the programme Examine-3D is used.

Examine-3D is a three-dimensional analysis program for the design of underground structures. It enables simplified modelling of the rock mass using an elastic stress analysis based upon the boundary element method. The programme allows stress and displacement distributions to be investigated around complex underground excavation geometries, which could not otherwise realistically be modelled using conventional two-dimensional numerical analysis techniques.

The programme provides a means of investigating the effect of key parameters on the induced three-dimensional stress distribution. Such sensitivity analyses may be carried out rapidly, and provide reliable indications of critical parameters which may be further investigated using more rigorous (but 2D) analytical procedures and programs (eg. UDEC).

For the CERN LHC Project, EXAMINE-3D has been used for the following tasks:

- investigation of the induced three-dimensional stress field
- effect of different k-ratios on the induced stress field
- investigation of critical cavern/shaft intersections
- investigation of the induced stress distribution in key areas of the underground layout, including effects on existing structures

These areas will be investigated in greater detail using UDEC.

(b) UDEC - 2D

For two dimensional calculations of the stability of tunnels and caverns, the distinct element code UDEC is used to analyse the rock mass behaviour due to progressive excavation and rock support installation and to check that the proposed rock support is adequate and sufficient.

UDEC is a two-dimensional discontinuum modelling approach for simulating the behaviour of discontinuous rock mass. The rock mass is divided into blocks with different mechanical properties. In the model, the blocks are bound by discontinuities which have different mechanical properties assigned. The rock mass at Point 5 is considered a weak rock mass but having still significant bearing capacity. Therefore, full deformatbility is given to the blocks modelled. Full deformation of blocks is achieved by discretisation into finite difference zones. The deformability of interfaces between the blocks is represented by spring-slider systems.

The rock mass will be given properties obeying a linear-elastic/perfectly plastic constitutive law where Mohr-Coloumb yield conditions are considered.

Different rock support measures such as rock bolts and shotcrete can be considered explicitly in the UDEC-code.

<u>Simulation of Rock Mass Features:</u>
The rock mass is simulated by a fully deformable isotropic constitutive model. Due to the iterative step calculation sequence non-linear material behaviour is achieved even when using linear constitutive laws. The Mohr-Coulomb criterion is used as failure criterion.

Simulation of Discontinuities (Interfaces):

Discontinuities are simulated by spring-slider connections of blocks. Due to the iterative step calculation sequence non-linear joint material behaviour is achieved even when using constant spring stiffnesses. The Mohr-Coulomb failure criterion is used as yield surface.

In-situ Stress Field:

Arbitrary in-situ stress field conditions can be modelled. The weight of rock cover is used to simulate vertical in-situ stresses. To cater for the variation of horizontal in-situ stresses, different K-ratios are used to define both initial in-situ stress conditions as well as boundary conditions during the calculation.

Excavation Sequence:

The excavation of the caverns and the tunnels is simulated by subdivision into an appropriate number of excavation steps. Excavation is simulated by deletion of excavated blocks from the model. However, the central pillar will be backfilled by concrete before bulk excavation in the caverns is commenced. After excavation, these pillar blocks can be re-activated having concrete material properties assigned.

Rock Support Measures:

Rock bolts are considered to be the main rock support measure at Point 5. As isotropic rock mass is modelled, the "Cable Command" is used for simulation of rock bolts. This approach considers an interaction of the whole system bolt-grout-rock along the entire grouted bolt length.

Loads:

Loads are applied on the surface of the calculation model or considered to be inherent such as gravity. Water pressure or swelling loads can be applied by definition of domain pressures acting in specific interfaces.

Simulation of 3D-effects:

3D effects may be considered by application of additional loads to the 2D model. These additional loads are derived by comparison of results from the overall 3D calculation model to the results of relevant 2D calculations.

(c) FLAC - 2D

FLAC is used primarily for the design of the final linings, see Section 4. The FLAC model is calibrated against the UDEC model to achieve the optimum modelling of the rock mass, before analysing the lining. For two dimensional calculations of the stability of shaft and cavern linings, the two-dimensional finite difference code FLAC-2D will be used. Materials are represented by elements, which form a grid that is adjusted by the user to fit the shape of the object to be modelled. Each element behaves according to prescribed linear or non-linear stress/strain law in response to the applied forces or boundary restraints. If the stresses or stress gradients are high enough to cause the material to yield , the grid can actually deform in large strain mode. Structures such as tunnel linings which interact with the surrounding rock mass may be modelled with structural element logic.

Simulation of Rock Mass:

The rock mass is simulated by a fully deformable anisotropic constitutive model. Due to the iterative step calculation sequence non-linear material behaviour is achieved even when using linear constitutive laws. The material properties of Sandy Marl are considered to represent average coupling stiffness. Therefore, properties of Sandy Marl are assigned to the rock mass simulated in the FLAC models. No rock joints are simulated.

Simulation of Discontinuities (Interfaces):

Coupling interfaces between tunnel lining and surrounding rock mass are simulated by spring and spring-slider connections. Due to the iterative step calculation sequence non-linear interface material behaviour is achieved even when using constant spring stiffnesses. Mohr-Coulomb is used as yield surface along the plane of interface between rock mass and the lining structure.

In-situ Stress Field:

Arbitrary in-situ stress field conditions can be modelled. The weight of rock cover is used to simulate vertical in-situ stresses. To cater for the variation of horizontal in-situ stresses, different K-ratios are used to define both initial in-situ stress conditions as well as boundary conditions during the calculation.

Excavation:

Excavation is simulated by deletion of excavated area from the model.

Rock Support Measures:

Rock bolts are modelled in a similar manner as described for the UDEC code above.

Loads:

Loads may be applied on arbitrary surfaces within the calculation model or considered to be inherent such as gravity. Swelling pressures will be applied by hearing of model zones subjected to swelling processes. Water pressures can be applied by definition of pressures acting in specific interfaces.

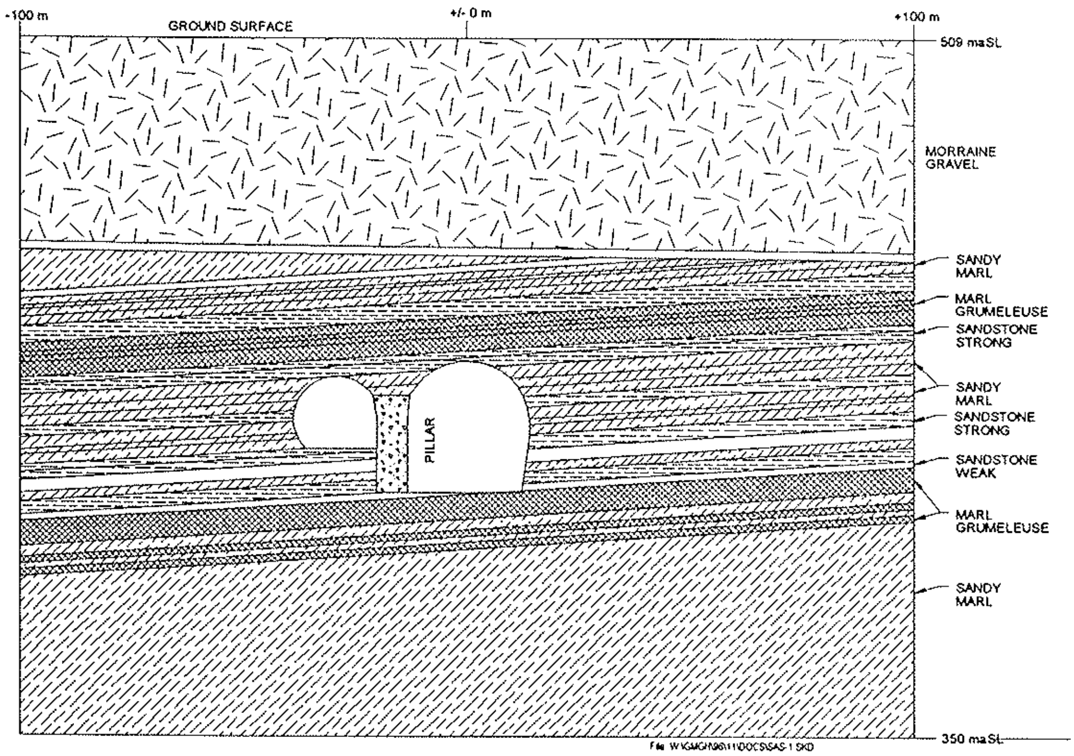
3.5.3 2D Numerical Calculation Model for Caverns

The numerical calculation model developed to represent the geological and geometrical conditions at Point 5 is presented in Figure 3.5.3 below. In accordance with the Geotechnical Interpretative Report all geotechnical features are modelled.

All geotechnical zones identified in the Geotechnical Interpretative Report were simulated in the form of detailed block discretisation. In addition, all relevant interfaces and block boundaries such as thin layers of Marl Grumeleuse are represented in the model.

The geotechnical calculation model for rock mechanical analysis is based on a block of rock mass, 200 m wide and 160 m high. All relevant geotechnical zones and interfaces as well as excavation geometries are shown in the figure below.

Figure 3.5.3 - Global Stability Model SAS-1, UDEC Model Geometry



Geotechnical analysis will be performed to give the information on the:

- Stress / strain situation and distribution in the rock mass.

- State and extension of possible yielding zones.
- Large scale rock mass behaviour such as induced slips, etc.
- Pillar stresses.
- Development of rock arching effects.
- Response of rock support proposed and interaction between rock mass and support.
- Rock support utilisation.
- Need for specific rock support measures.

When applicable, this information will be used as input to local stability analyses. In some cases it is possible to use part of the global model as a local stability model. See Section 3.5.4. below.

Although, immediate installation of rock support is considered inevitable during excavation, calculations will be performed with and without simulation of rock support. A comparison of both calculations will enable an assessment of the relevance of rock support to achieve global stability in the numerical model.

For the purposes of the global stability analysis, rock support for a given stage is taken to be that installed on completion of the previous stage of excavations, with no benefit taken from support installed during the subsequent excavation process.

3.5.4 Local Analytical Calculation Model

Local stability analyses will be performed in situations where the global analysis or analytical calculations require enhancement to ensure appropriate confidence.

Local analyses will be performed for a particular local geometry or geological condition such as the occurrence of heavily jointed rock or discrete, unfavourably oriented rock blocks or wedges.

These local models are created using results of the global stability analysis as an input. Therefore, the local model may be a continuation of the global model calculation putting emphasis on local geometry, local loads, and local deformation behaviour. Only the most critical geometrical arrangement will be investigated. Results of local models will give the information on the:

- Stress/strain behaviour and overstress ratio situation in rock mass.
- Response to applied load.
- Interaction between rock mass and support with induced load on support system
- Rock displacements, internal stresses and forces.

- Residual forces and stresses in wedges.
- Effects of load redistribution due to rock mass creep.
- Basis for the decision on the necessity of special rock support measures.
- Rock support utilisation.

3.5.5 Calculation Model for Tunnels

In this context, numerical calculations to be performed for excavations other than USC 55 or UXC 55 are considered tunnel calculations.

Therefore, numerical analyses performed to design primary rock support for the tunnels will be similar to local stability calculations as described above. Results from other geotechnical calculations may be used as input to the design calculations for primary rock support in tunnels.

3.5.6 Calculation Model for Shafts

Moraine:

It is assumed that special construction methods such as ground freezing, diaphragm walling or similar will be employed to sink the shafts. These construction methods provide primary support in the Moraine gravel section. Therefore, in the moraine gravel section the primary shaft linings will be designed based on the construction method adopted. Generally, in the Moraine gravels support measures additional to those provided by the specific construction method will be installed only to act as a seal or to adapt to local conditions. As the support system is dependent on the construction method adopted, the detailed construction design of initial support will be the responsibility of the Contractor. The primary support tender design will make use of advice provided by appropriate specialist contractors, and will be carried forward to the level required for the tender. The secondary lining will be designed to carry all the loading. For the circular PM54 simple hand calculations are anticipated to be sufficient for the tender design of the primary support. For the elliptical PX56 horizontal sections will be considered with FLAC-2D or similar software. It is not anticipated that sophisticated numerical modelling will be required for the design of the shaft primary support system and a linear elastic model will be adopted.

Molasse:

In the lower shaft sections, in the Molasse rock mass, the shafts will be treated in the same way as the caverns and tunnels, as described above.

Primary support in the molasse will be designed based on plan section 2D numerical modelling and evaluation of stress enhancement from 3D analyses, load redistribution due to creep and swelling will be controlled by the primary support system, as for other structures in the molasse.

3.6 Rock Support System

As discussed in Section 3.1, Rock Support Philosophy, a range of rock support measures will be considered. These include:-

- systematic rock bolting and shotcrete
- use of rock anchors
- the structural use of concrete in the pillar between caverns UXC55 and USC55 and in the invert of the caverns.
- a primary structural concrete lining for UXC55 achieved by means of staged multiple drifts (SMD).

Particular requirements for specific properties of rock support currently assumed for design purposes are given below. Properties specified in this section shall, at a later stage, serve as a basis for the detailed technical specification of construction materials required. If required, data and values given below will be updated to be in accordance with the particular technical specification.

Rock Bolts:

Systematic rock bolt support measures will consist of fully grouted steel rebar rock bolts. Alternatively tensioned Double Corrosion Protection (DCP) systems could be used where required for the large cavern structures. In particular applications during excavation, swellex bolts may also be appropriate.

Water is not expected to be mobile in the Molasse rock mass and corrosion of fully grouted rock bolts will be very limited. Properly installed fully grouted rock bolts with appropriate allowance for corrosion are therefore assumed to have permanent effect.

For side walls in areas of strong and competent sandstone, spot bolting alone may be sufficient. However, in cases of locally poor rock conditions or unfavourable geometrical configurations stronger rock bolts and dense patterns may be required.

The following parameters for the properties of rock bolts are assumed:

Yield stress, steel	460 MPa
Ultimate stress, steel	500 MPa
E-Modulus, steel	206000 MPa
Steel elongation at failure	12%
G-Modulus, grout	9000 MPa
Shear strength, grout	5.0 MPa

Shotcrete:

At Point 5, shotcrete will be applied as a rock support measure in all underground structures as both a sealing system and as a structural element in the composite support system.

The following parameters for the properties of shotcrete are assumed:

Minimum in-situ compressive strength	25 MPa after 28 days	
Tensile bending strength	6 MPa	
Young's modulus	20000 MPa	hard shotcrete
	5000 MPa	green shotcrete
Density	2400 kg/m^3	

If steel reinforcement is necessary, the reinforcement shall conform to relevant design standards with dimensions as shown in the design calculations.

In the event that fibre reinforced shotcrete is used it shall fulfil the following requirements regarding compressive and flexural strength:

Minimum in-situ compressive strength	:	25 MPa after 28 days
Flexural strength	:	6 MPa average value

Rock Anchors:

Rock anchors will be considered to provide additional rock support for special cases of large loads and/or extended poor rock mass conditions. As for rock bolts rock anchors are considered permanent rock support.

The following parameters for the properties of rock anchors are assumed:

Single threaded anchors (bars)

Yield stress, steel	1080 MPa

Ultimate stress, steel	1230 MPa
E-Modulus, steel	206000 MPa
Steel elongation at failure	6 %
G-Modulus, grout	9000 MPa
Shear strength, grout	5 MPa

Bundled strand anchors (cables)

Yield stress, steel	1570 MPa
Ultimate stress, steel	1770 MPa
E-Modulus, steel	206000 MPa
Steel elongation at failure	6 %
G-Modulus, grout	9000 MPa
Shear strength, grout	5 MPa

Mass Concrete:

The use of mass concrete as a systematic rock support measure is considered for the following cases:

- The pillar between caverns USC 55 and UXC 55. The following parameters for the properties of pillar concrete are assumed. Due to staggered cavern excavation sequence, specified compressive strength is required only after 56 days:

Minimum in-situ compressive strength	40 MPa after 56 days
Tensile strength	6 MPa
Young's modulus	25000 Mpa after 56 days
Density	2450 kg/m^3

- The concrete arch in the invert of the larger caverns. This may be in mass or reinforced concrete, depending of the structural demand on the arch..

  The following parameters for the properties of invert arch concrete are assumed:

Minimum in-situ compressive strength	30 MPa after 28 days
Tensile strength	4 MPa
Young's modulus	25000 MPa after 28 days
Density	2450 kg/m^3

- Staged Multiple Drifts (SMD) around the perimeter of the large UXC 55 cavern which are backfilled with mass concrete to act as a closed stiff protection ring during cavern bulk excavation.

  The following parameters for the properties of SMD concrete are assumed:

Minimum in-situ compressive strength	22.5 MPa after 28 days
Tensile strength	2 MPa

Young's modulus	20000 MPa after 28 days
Density	2300 kg/m^3

Reinforced Concrete:

The use of reinforced concrete structures as a systematic rock support measure is not anticipated. However where required the required grade will be determined from the structural design.

The following parameters for the properties of reinforced concrete are assumed:

Minimum in-situ compressive strength	30 MPa after 28 days
Tensile strength	6 MPa
Young's modulus	25000 MPa after 28 days
Density	2450 kg/m3
Yield stress, reinforcement steel	460 MPa
Ultimate stress, reinforcement steel	500 MPa
E-Modulus, reinforcement steel	206000 MPa
Steel elongation at failure	10%

Rock Mass Grouting:

Rock mass grouting is not considered a regular rock support measure in the Molasse rock mass. However, it may have relevance during shaft construction in the Moraine gravels. It may also be used as a means for sealing against water inflow at the transition of Moraine gravels to Molasse rock mass where the shaft enters the Molasse.

Based on the actual conditions, cement based as well as chemical grouting materials may be used.

3.7 Rock Support Design

3.7.1 Design Sections

Rock support design will be performed in sections of the following facilities of the underground scheme:

USC 55, UXC 55: Numerical analyses (2D and 3D), local modelling, analytical calculations for local situations or verification.

PM 56, PX 56: Numerical analyses (2D and 3D), local modelling, analytical calculations for local situations or verification.

UJ 53, UJ 57, Cryo Feed Box: If necessary numerical analysis, local modelling, analytical calculations for local situations or verification.

UL 54 and UL 56: Local modelling, analytical calculations for local situations or verification.

UP 56, UPX 56, TU 56, UP 53: If necessary local modelling, analytical calculations for local situations or verification.

Other structures: Analytical calculations for local situations or verification.

3.7.2 Design Criteria

The design of structural elements shall be in accordance with ultimate limit state principles. For ultimate limit state design using the analytical calculation methods presented above partial safety factors for strength of materials and load factors are used.

Factors are used to take into account the following unknowns:

- Relevance of limit state considered.
- Possible increase in loading
- Local weaknesses.
- Differences between actual and laboratory strength values.
- Inaccurate measurements of the effects of the loading
- Unforeseen stress distributions in the analysis model

Widely recommended values are 1.4 for the partial load factor, 1.5 for the partial material factor of shotcrete and concrete, and 1.15 for the partial material factor of steel. These values will be used if not prescribed otherwise by relevant codes or standards. However, if sufficient experience exists of the actual ground conditions for the actual loading to be predicted with confidence, other load factors may be defined at a later stage.

Design criteria applied for primary lining design shall be in accordance with the codes and standards used and meet the requirements regarding both the load assumptions for the secondary lining structure and the performance of the structures defined by CERN.

3.8 Construction Sequence

3.8.1 Overall Construction Sequence

The overall excavation sequence is considered an essential means in the control of the stability of the cavern complex as well as rock mass deformation, in particularly the development sequence of the two main caverns and the pillar between them. At the same time programming constraints have to be taken into account. These programming constraints include CERN operational requirements, construction access requirements and integration with building erection and machine assembly. The overall construction sequence will be developed based on the geomechanical principles presented in this report, considering these programming restraints. It will be presented in the Phase I Summary Report, together with the preliminary construction programme.

3.8.2 Excavation Sequence

In a particular underground opening, excavation will be staged, if necessary, to allow early incremental application of rock support. Where appropriate excavation of larger openings will be subdivided into smaller steps.

Excavation of the large caverns will be performed adopting a top heading-bench sequence. Depending on the size of the particular opening, excavation of each topheading and each bench may further be split into a number of sub steps.

Tunnels will also be excavated in topheading-bench sequence. For connection tunnels, pillar drifts and other small openings, full face excavation may be considered and applied.

Shaft excavation will be performed in full face excavation of one round. Then rock support and or surface protection will be applied to the full round.

3.9 Design Standards

In the design of underground structures of the form proposed at Point 5, the definition of stability and the determination of support measures to achieve stability including excavation sequences is not codified in internationally acknowledged standards.

However, the design of underground structures at Point 5 will follow the general guidelines presented in:

- Deutsche Gesellschaft für Erd- und Grundbau: Empfehlungen für den Felsbau unter Tage.
  ("Recommendations for the design and construction of underground structures".)

These recommendations deal with investigation, design and safety calculation methods, design of support and lining, monitoring, classifications, tender and construction.

In addition, where economic benefits are possible and considered appropriate:

- BS 8081: Rock Reinforcement

- BS 8110: Structural Use of Concrete
- BS 5628: Structural Use of Masonry (used for mass concrete)

Codes or standards, other than listed in this section may be applied if found appropriate or feasible.

4 DESIGN OF UNDERGROUND STRUCTURES - SECONDARY LINING

4.1 Design Philosophy

The design of the secondary lining structure for the caverns and tunnels will be based on the philosophy that the primary support system will take all external loads. However, due to the deformation of the rock mass which may be associated with swelling or creep, the secondary lining may undergo induced deformation in the long term. The secondary lining, therefore, will be designed for loads from long-term rock mass and primary support deformations, gravity, shrinkage, temperature, local crane loads, other suspension loads and the loads of the detector. Secondary lining will also be designed to ensure it may satisfactorily accommodate a nominal water pressure in the event of partial drainage failure.

The secondary lining structures will be designed as structural concrete elements. In large caverns and shafts it is envisaged that secondary linings will include reinforcement to resist structural actions and to control cracking. Thick concrete sections for the main underground structures will be designed to minimise the possibility of cracking by appropriate mix design. Reinforcement will however be included to distribute surface cracking and to ensure that early age cracking will self heal. This will be based on normal structural requirements, not a water retaining level of steel. Secondary lining structures in tunnels and other small excavations are intended to be designed in plain concrete only. However, in areas of intersections with shafts or other structures, reinforcement will be provided according to the results and requirements of particular design calculations.

The shaft linings within the Molasse will be considered together with the cavern lining. For the shafts within the Moraine, the secondary lining will be designed to withstand the total imposed load, taking full water head on the impermeable membrane between the initial and the secondary lining. Particular attention will be paid to preventing the passage of water down the shaft lining surface from the moraine to the molasse.

4.2 Design Methodology

The design is expected to proceed in the following steps:

1. Establish the structural system for the lining. Define the interface details between primary linings and final linings including the effect of waterproofing systems and the need or otherwise for composite connections.

2. The type of structural calculation to be made will depend on the structural system identified. It is anticipated that joints will be provided at many intersections. In this case most of the calculations can be performed using 2-D frame analyses. At some of the structures, like the roof of the UXC 55 cavern, 3-D shell analysis may be necessary because of the inability to assess the situation sufficiently with 2-D models. This shell analysis will be performed using FE element codes.

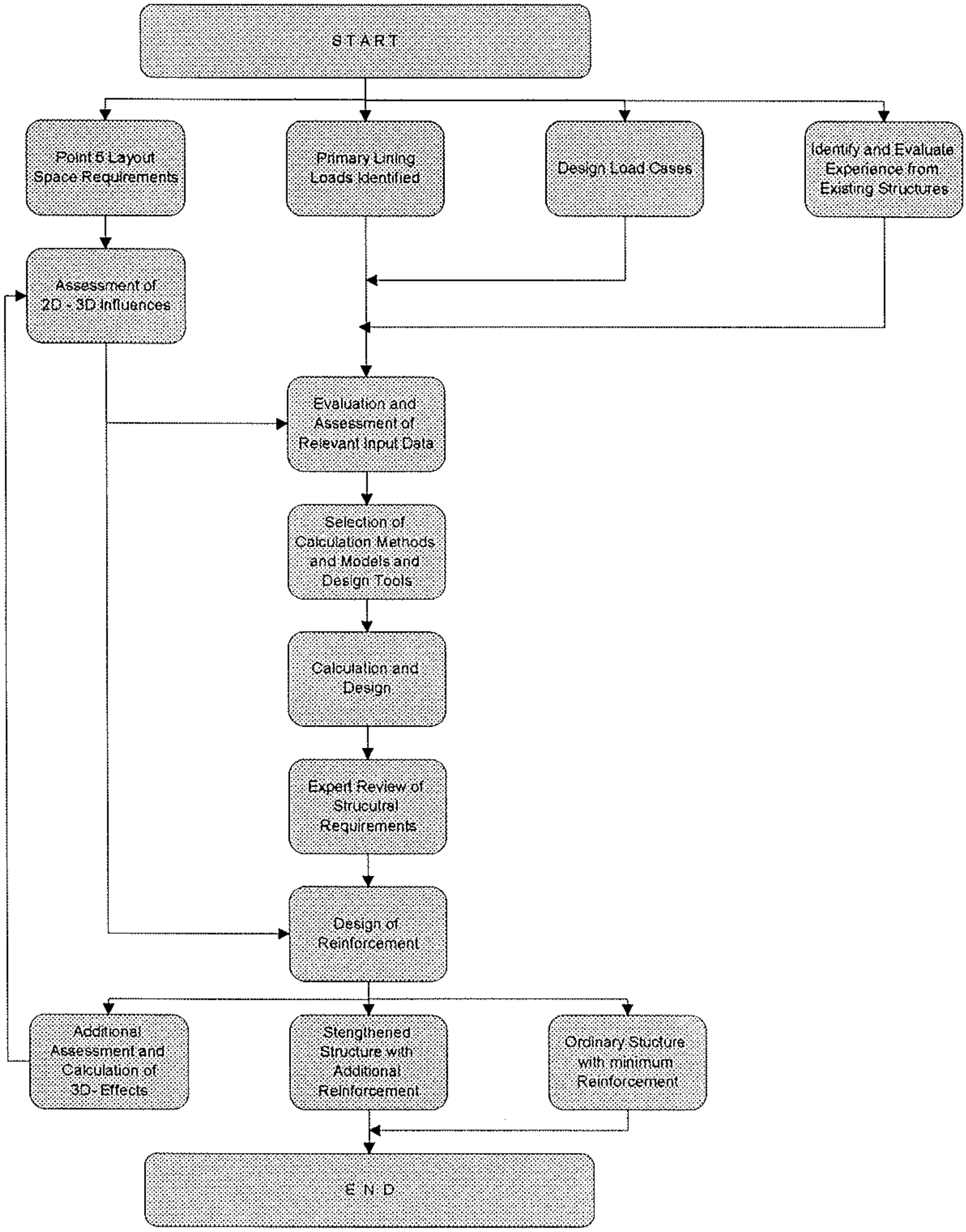
3. The structural calculations provide section actions which will be used to design concrete and reinforcement requirements according to the structural design codes. It is assumed that the reinforcement will typically be governed by minimum reinforcement requirements based on the envisaged space usage and finish requirements, with some local enhancement at critical locations.

The design will determine the capacity of the secondary lining to allow for induced deformation. This will be compared with the predictions of the rock mass modelling to confirm the design.

Should a freeze wall system be employed for temporarily cutting off flows to the shaft excavation, then the potential impacts of the freeze wall on the lining during construction and thawing and the possibility of non-uniform water pressure on the lining during thaw will be considered.

A flow chart for the secondary lining structural design is presented in Figure 4.2.1. below.

Figure 4.2.1 - Main Design Steps for Secondary Lining



4.3 Load Cases

4.3.1 Gravity

After closure of the lining and removal of shutters weight of the lining structure will introduce section forces into the concrete. The concrete weight as well as additional shutter loads will be considered in the design calculations.

4.3.2 Shrinkage

During the curing process of the concrete, shrinkage will result in local debonding and change in the degree of local confinement yielding section forces. Shrinkage will be considered in accordance with the standards applied.

4.3.3 Temperature

During placing of concrete and the following curing process temperatures are different to those anticipated during operation. This will result in additional lining deformations yielding changes in the degree of local confinement and resulting in additional section forces in the lining structure.

During operation the air temperature is normally controlled at 20°C. In the event of equipment failure a lower bound of 10°C is adopted for design.

4.3.4 Swelling

As mentioned in Section 4.1 above, due to deformation of the rock mass related to swelling of Marl layers, the secondary lining may receive induced deformation in the long term. Swelling effects will be simulated in numerical calculations causing deformation of the supported rock mass. The corresponding deformation of the interface primary support / secondary lining will be imposed on the secondary lining.

4.3.5 Rock Mass Creep

The capacity of the secondary lining to allow for additional imposed deformations will be determined by numerical calculations and permissible creep rates derived to determine the timing for erection of the secondary lining.

4.3.6 Additional External Loads

The secondary lining structure will be loaded by equipment installed at a later stage. Loads such as the weight of the detector, crane loads, loads from gangways and other installations fixed to the lining structures will be considered.

4.3.7 Water Pressures

The cavern and tunnel linings are drained to prevent build up of water pressures. Water pressure will therefore not be considered as a normal load case for these structures. The lining will be checked for the effect of local failure of the drainage system, leading to local water pressures. For the shafts in the moraine full hydrostatic head will be taken on the external face of the final lining.

4.4 Numerical Analysis

4.4.1 Program Codes

4.4.2 2D Calculations

For the assessment of the two dimensional behaviour, calculations using the FLAC code will be performed in relevant cross sections. For a detailed description of the FLAC code and further details of modelling see Section 3.5. above.

Generally, the same principles given for the FLAC analyses in connection with the primary lining design apply also to analyses in connection with the design of the secondary lining.

4.4.3 3D Calculations

For the assessment of three dimensional behaviour of complex and complicated intersection geometries, three dimensional shell calculations will be performed, if necessary. Appropriate programme codes will be employed.

4.4.4 Lining Model

For the structural design of the secondary lining structure, section forces will be calculated using numerical models. In these models (2D as well as 3D), the rock mass will be simulated as homogeneous material with an average stiffness in accordance with the geomechanical model of the rock mass. The lining will be modelled with structural elements. If necessary, the whole geometrical extent of the particular cross section and relevant loads will be simulated. Section forces and deformations will be calculated. In 2D calculations, shafts will be modelled in horizontal sections.

These numerical models are capable of simulating all load cases. Consequently, all relevant loads will be applied, effects studied and sectional forces derived for lining design.

Details of how loads will be simulated in the 2D model are given below:

- Gravity and suspension load can be directly simulated.
- Shrinkage loads will be simulated by cooling of the concrete structure.
- Temperature load cases will be simulated by introduction of design temperatures in the lining structure as required.
- Swelling loads will be introduced in the model by either heating of relevant rock parts or application of internal pressures at relevant locations in the rock mass.
- Effects due to additional rock mass deformation such as creep will be studied by imposing additional deformations onto the lining model.

For secondary lining structures in complex geometries such as the connection of the shafts to the caverns, three dimensional numerical models will also be developed capable of simulating relevant load cases.

Details of how loads may be simulated in the 3D model are given below:

- Gravity and suspension load can be directly simulated.
- Shrinkage loads will be simulated by cooling of the concrete structure.
- Temperature load cases will be simulated by introduction of design temperatures in the lining structure as required.
- Swelling loads will be introduced in the model by either heating of relevant rock parts, application of internal pressures at relevant locations in the rock mass, application of pressures directly onto the lining concrete.
- Effects due to additional rock mass deformation such as creep will be studied by comparison to 2D calculation results..

4.5 Secondary Lining System

In this section, particular requirements for specific material properties currently assumed for design purposes are given below. Properties specified in this section shall, at a later stage, serve as a basis for detailed technical specification of construction materials required. If required, data and values given below will be updated to be in accordance with the particular technical specification.

The secondary lining structure will be designed as a reinforced concrete structure for the large caverns and shafts. For the remaining underground openings, it will be designed as plain concrete structure.

The quality of concrete used shall be chosen according to calculation results and structural requirements. Concrete shall be in accordance with relevant design codes and standards. If reinforcement is necessary, it shall comply with relevant design codes and standards.

As a particular requirement, the following parameters for the properties of reinforced concrete are assumed:

Minimum in-situ compressive strength	30 MPa after 28 days
Tensile strength	6 MPa
Young's modulus	25000 MPa after 28 days
Density	2450 kg/m3
Yield stress, reinforcement steel	460 MPa
Ultimate stress, reinforcement steel	500 MPa
E-Modulus, reinforcement steel	206000 MPa
Steel elongation at failure	10%

4.6 Secondary Lining Design

4.6.1 Design Sections

Design of secondary lining will be performed in sections of following facilities of the underground scheme:

USC 55, UXC 55: Numerical analyses, 2D and 3D.

PM 56, PX 56: Numerical analyses, 2D and 3D.

UJ 53, UJ 57, Cryo Feed Box: Numerical analyses, 2D.

UL 54 and UL 56: Numerical analyses, 2D.

UP 56, UPX 56, TU 56, UP 53: If necessary numerical analyses, 2D.

Other structures: If necessary numerical analyses, 2D.

4.6.2 Design Criteria

For the design of the secondary concrete lining structures load and material factors will be used in accordance with the codes and standards adopted.

These factors shall take into account the following unknowns:

- Possible increase in loading
- Differences between actual strength of materials and laboratory values.
- Inaccurate measurements of the effects of the loading
- Unforeseen stress distributions in the analysis model
- Relevance of the limit state being considered

Design criteria applied for secondary lining design shall be in accordance with the codes and standards used and meet the requirements regarding the performance of the structures defined by CERN in particular the requirements for robust structures with no structural maintenance and no visible signs of water ingress.

4.7 Design Standards

The design of concrete elements in the underground complex will generally be performed in conformance with the following standards:

- DIN 1045: Beton- und Stahlbetonbau, Bemessung und Ausführung

In addition, where economic benefits are possible and considered appropriate:

- BS 8110: Structural Use of Concrete
- BS 8007: Water Retaining Structures*
- BS 5628: Structural Use of Masonry (for mass concrete)

Codes or standards, other than listed in this section may be applied if found appropriate or feasible.

*BS 8007 includes methods of reinforced concrete design for crack control. It is not intended to design the lining to a full `water retaining' standard.

5 WATERPROOFING OF UNDERGROUND STRUCTURES

5.1 Requirements

The molasse rock mass is practically tight in the virgin state. The moraine overlying the molasse is, by comparison, a recognised aquifer.

The primary goal of the waterproofing for the cavern complex is the prevention of leakage from the aquifer along the newly built shafts into the cavern complex. This aspect is potentially critical both in relation to the space and operational requirements and the possible adverse effects of water on the marl rock mass long term, particularly where it has been disturbed (deformed) by excavation.

To limit downward flow of water from the moraine through the excavation disturbed zone and/or shaft concrete to ground interface it will be necessary to incorporate an engineered barrier around the section of shaft in the molasse.

All underground structures will be waterproofed to ensure that no water seepage occurs through the secondary tunnel lining.

To achieve this aim, methods for active waterproofing (sealing) and drainage as well as special methods for cutting off routes of potential water seepage (along shaft linings) will be applied.

Drainage systems shall be designed so that they do not act as a distribution system for water.

5.2 Waterproofing Systems

In tunnels and caverns, the following waterproofing system will be applied:

- In the case of local water inflow, special drainage measures will be taken to catch and route the water directly into the drainage system.
- Protection & drainage fleece on the shotcrete intrados.
- Waterproofing membrane fixed to the surface. At construction joints, the membranes will be welded together.

- Joint strips (rubber water stops) in the construction joints of the secondary concrete lining.

In the shafts the following water proofing systems will be applied:

Shaft Linings (Moraine and Molasse):

- Protection & drainage fleece on the shotcrete intrados.
- Waterproofing membrane fixed to the surface. At construction joints, the membranes will be welded together.
- Joint strips (rubber waterstops) in the construction joints of the secondary concrete lining.

Cut-off Systems (Molasse only):

- In the Molasse rock mass, ensure multiple cut-off of potential water routes in the form of specially shaped, horizontally slightly enlaged cross sections (keys), grouting measures, chemical hydrophilic sealants or similar.
- In the case of local water inflow, special drainage measures will be taken to catch and route the water directly into the drainage system.
- Drainage rings at regular distances within the molasse especially at the locations of the cut-off keys.

5.3 Design Standards

The design of waterproofing structures and elements will be performed in accordance with the principles of the following recommendations:

- DS 853: Vorschrift für Eisenbahntunnel (VTU), mit Ergänzungsbestimmungen Ez VTU 10 (Abdichtung) und Ez VTU 11 (Entwässerunganlagen), Deutsche Bundesbahn, 1984

  Regulation for railway tunnels, additional regulations for waterproofing and drainage systems, German National Railways.

Codes or standards, other than listed in this section may be applied if found appropriate or feasible.