ELLOMAY
Pumped Storage (2014) Ltd.

PSPP MANARA

ISRAEL

BASIC DESIGN

GENERAL

Civil Works Outline Design Report

Written:	Kronberger, Lang	15.05.2018 & Nowotny	14.12.2020
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ABBREVIATIONS

AFRD … asphalt faced rock fill dam
DN … nominal diameter
EM … electro-mechanical
f'_c … concrete compressive strength
FSL … full supply level = maximum operation water level
GSRD … geomembrane sealed rock fill dam
HM … hydro-mechanical
kW … kilowatt
MOL … minimum operation water level
MW … megawatt
RCC … roller compacted concrete
OBE … operating basic earthquake
MCE … maximum credible earthquake

REVISION NUMBER

Rev.	date	written	description
1.0			first edition
2.0			Approval by Owner
3.0	14.12.2020	Nowotny	New design of upper and lower reservoirs
3.1	14.01.2021	Nowotny	updates of written, checked, approved dates on cover-page
3.2	03.02.2021	Nowotny	storage-volume curve of LR & distance PH – surge tank corrected

1 INTRUDUCTION

The Israeli Public Utilities Authority (PUA) has decided to increase the instantaneous power available on the grid by adding Pumped Storage Power Plants (PSPP) to the existing generation capacity.

Therefore Ellomay Pumped Storage (2014) Limited intends to build a pump storage scheme in the vicinity of Manara, in the north of Israel. The plant has an installed capacity of 220 MW, whereas the reservoirs are designed for 8.8 hours operation in turbine mode (operating with maximal power) and some 10.9 hours for operating in pump mode (operating with maximal power).

Figure 1-1: Look from lower reservoir region towards upper reservoir region

Figure 1-2: Look towards Lower Reservoir region

2 PROJECT LOCATION

The project area is located in the west rim of Jordan Valley northwest of the Sea of Galilee in the vicinity of Kiryat Shmona.

The pump storage scheme uses the height difference of the mountain hills to the Jordan Valley with nearly 700 m difference.

The lower reservoir is situated on the west side of Jordan valley on the toe of hill flank. The upper reservoir is on top of the mountain. Access of the project parts is via existing access roads in the project area. Most mentionable advantage of the location is the short distance to the existing 2 x 161 kV lines crossing the portal area of the access tunnel and an existing switchyard in the vicinity - approximately 6 km from the access tunnel portal.

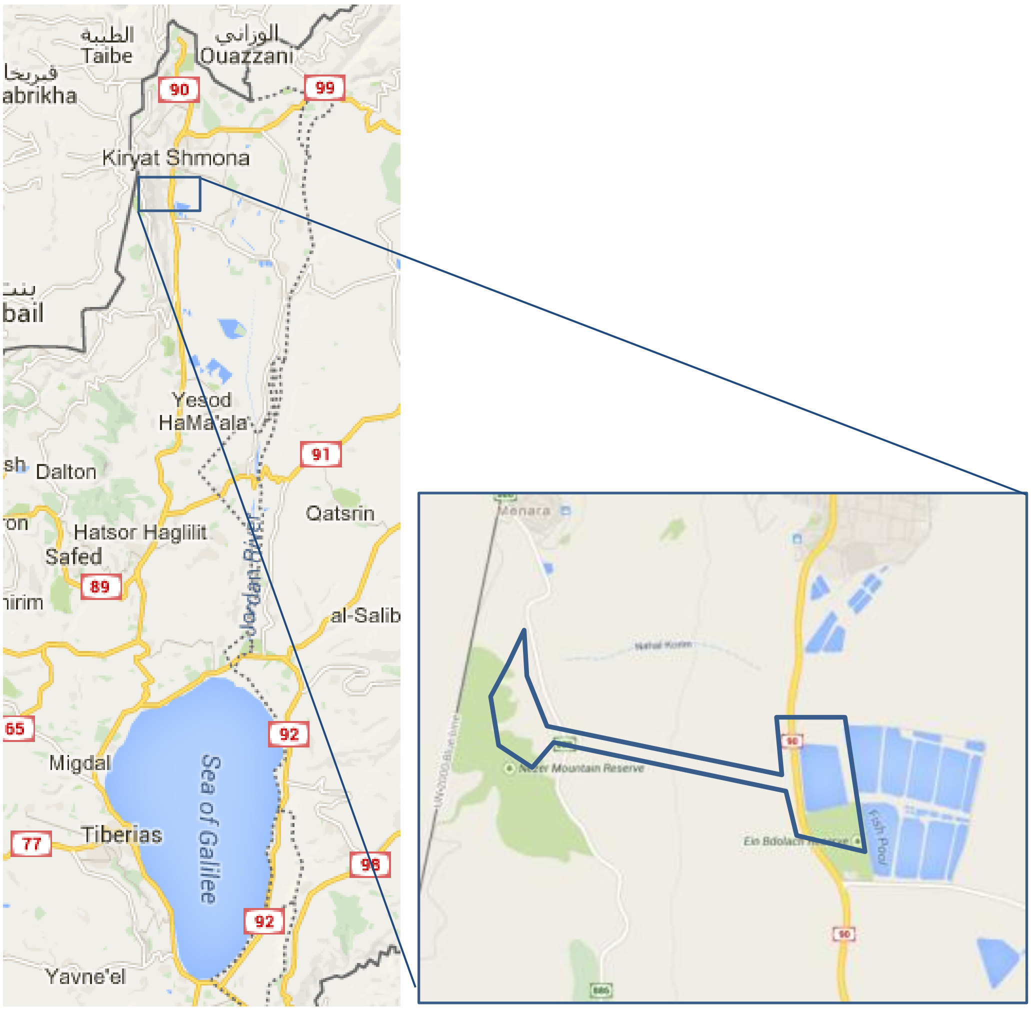

Figure 2-1: Overview Map Sea of Galilee and Jordan Valley with PSPP Manara (Google maps)

3 MAIN PROJECT PARTS

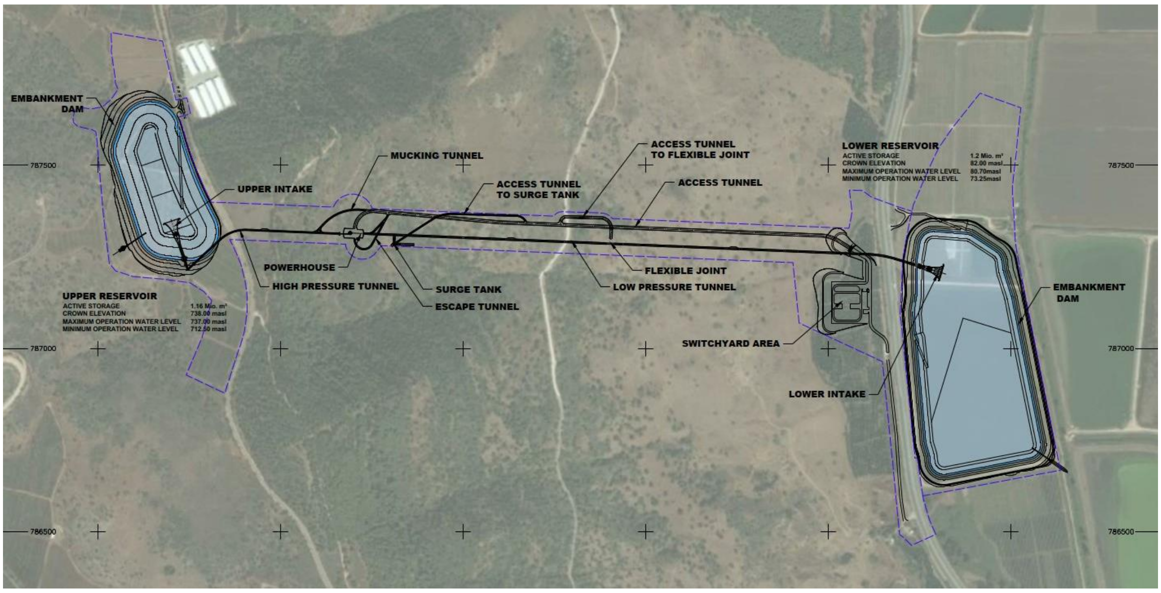

Figure 3-1: Overview of PSPP Manara

The proposed layout for PSPP Manara is shown in Figure 2 above, consisting of

- Upper reservoir and bottom outlet
- Intake/outlet structure in upper reservoir with gates
- High pressure power water way with vertical shaft and pressure tunnel
- High pressure manifold
- Powerhouse – situated in a cavern
- Surge tank
- Low Pressure Tunnel
- Outlet/intake structure with gates
- Lower reservoir and bottom outlet
- Combined access tunnel with transmission and emergency exit tunnel
- Switchyard area including operation building and 161 kV switchyard and portal

4 MAIN PROJECT DATA

In the following table the main parts of the project are summarized.

Project parts	
Upper reservoir	
Reservoir type	In cut with rock fill dam
Sealing system	PVC foil (alternative: Asphalt)
Maximum dam height	18.0 m
Crest length of dam section	770 m
Crest / sill elevation	738.00 / 738.80 masl
Full supply level (FSL)	737.00 masl
Minimal operation level (MOL)	712.50 masl
Freeboard	1.80 m
Reservoir management	Daily operation
Reservoir volume	1.18 mio m³
Active storage	1.16 mio m³
Dead storage	0.03 mio. m³
Reservoir area at FSL	0.079 km²
Duration turbine mode / pump mode (rated discharge)	~8.0 / ~10.7 h
Bottom outlet – design discharge	1.1 m³/s
High pressure shaft and tunnel	
Total length P1 – P5	1064 m
Inner diameter	3.00 m
Lining	Steel lined
Powerhouse	
Type	Cavern
Dimensions L / W / H	55 / 26 / 46 m
Unit type	TBC Single stage Francis runner (vertical)
Number of units	1
Capacity T / P (rated value electrical)	156 / 220 MW
Synchronous speed	750 rpm
Rated discharge turbine mode (156/220) MW	26.8/37.1 m³/s
Rated discharge pump mode (156/220) MW	21.2/29.8 m³/s
Maximal gross head	663.75 m
Generator type	Synchronous
Access tunnel length	1399 m

Low pressure tunnel	
Total length D6 – D1	1524 m
Inner diameter	3.80 m
Lining	Concrete lining
Surge tank type	Throttled with shaft and upper chamber
Lower reservoir	
Reservoir type	Circle rock fill dam
Sealing system	PVC foil (alternative: Asphalt)
Maximum dam height	11.4 m
Crest length	1776 m
Crest / sill elevation	82.00 / 82.40 masl
Full supply level (FSL)	80.7 masl
Minimal operation level (MOL)	73.25 masl
Freeboard	1.70 m
Reservoir management	Daily operation
Reservoir volume	1.36 mio m³
Active storage	1.20 mio m³
Dead storage	0.16 mio. m³
Reservoir area at FSL	0.176 km²
Duration turbine mode / pump mode (rated discharge)	~8.0 / ~10.7 h
Bottom outlet – design discharge	1.2 m³/s
Grid connection	
Switchyard	161 kV

Table 4-1: Main project data

5 DESIGN BASICS

5.1 Geological conditions

5.1.1 Reference drawings

In the table below the geological reference drawings are listed.

NUMBER	TITLE
MANARA/BD/GG/04/001	POWER WATER WAY - VERTICAL SHAFT
MANARA/BD/GG/04/002	POWER WATER WAY - HIGH AND LOW PRESSURE TUNNEL
MANARA/BD/GG/05/001	POWER WATER WAY - ACCESS TUNNEL

Geological and geotechnical basis for the design is the geological report and investigations performed by EDF during 2010-2011. At this section only the most important statements crucial are presented.

The PSPP Manara project is generally situated in sandstones and limestone of the Kurnub formation and limestone, dolomite, chalk, clay, and marl of the Judea formation. The geological description is taken from the report "Manara pump storage project - Geotechnical baseline report" [P2].

The excavation depth of the upper reservoir will reach up to 28 m below the original ground surface in the eastern part of the reservoir. The geological conditions in this zone have been documented precisely by drill hole MNHPS-1, which is located inside the reservoir, close to the intake. The geological conditions of the uppermost 60 m of the rock mass have been evaluated in that bore hole. Based upon this information the upper reservoir is located in slightly to moderately weathered limestone (see figure Figure 5-1 below) which is taken from [P2]).

The lower reservoir will be excavated in the clay soil of the Hula valley. This unit is described as dark brown fat clay, silt and sand with organic matter, carbonate concretions, shell fragments and pebbles (see figure Figure 5-1 below which is taken from [P2]).

The "fat brown clay" material type was investigated with Auger type drilling till a depth of 25 m below dam crest of existing fish pond (MNLR - 1). The drill hole MNLR – 2 was drilled West of the road with 30 m depth and it was noted, that in the last 3 m grain supported conglomerate was documented. The comparison of hole MNLR 2 and 1 reveals a decrease in pebble content towards East as controlled by proximity of the mountain.

The power water way crosses all kinds of geological formations of the project area. A geological longitudinal section of the project can be seen in the Figure 5-1.

The powerhouse cavern is situated in the geological formation "Klei- Limestone and dolomite" and in "Khn- Sandstone black mudstone layers".

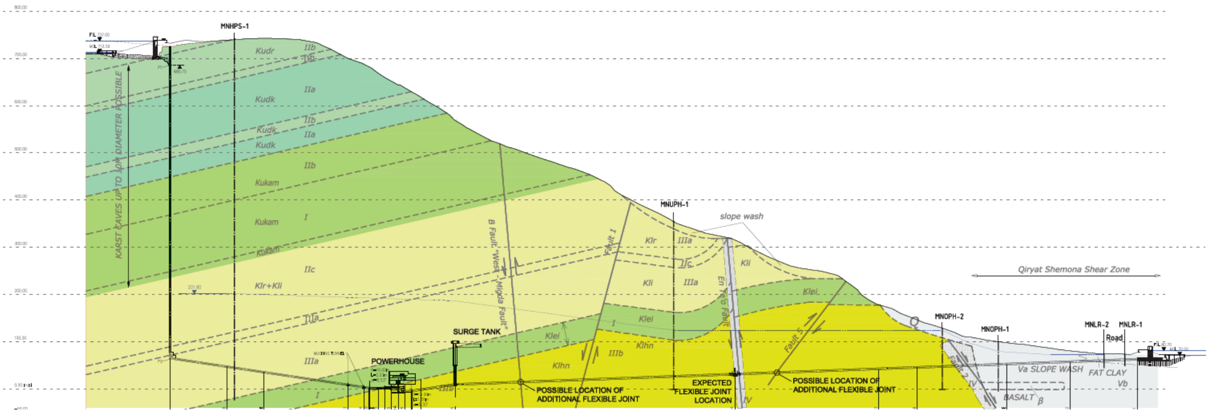

Figure 5-1: Geological cross section of waterway

At the upper reservoir the groundwater elevation is several 100 m below ground surface. At the lower reservoir the groundwater is just few meters (1-2) below natural ground surface.

Special attention has to be given to the possible active faults crossing the access tunnel to the powerhouse and the low pressure tunnel of the power water way. At least one of this faults – En Te´o Fault – is suspected to be active.

5.2 Seismicity and seismic design parameters

The Manara site is influenced by active Dead Sea Transform system and can be characterised as a high seismicity area with maximal earthquakes of Zone VI by Richter. The zone can be characterized by active main and secondary faults with a movement in all 3 directions with average amount of 1 mm/year.

These active faults could have influence on the underground structures, especially on the tailrace tunnel that will intersect most of these zones and the relative movements have to be considered in the design solution.

For both reservoirs, in the “Geotechnical Baseline Report” [P2], the OBE (operational basic earthquake) and MCE (maximal credible earthquake) peak accelerations have been defined and summarised in Table 5-1 below.

For pseudo-static analyses effective horizontal and vertical acceleration were used. The corresponding “effective horizontal acceleration” correlates to 2/3*PGA. The corresponding “effective vertical acceleration” correlates to 0.2*PGA.

Type of Earthquake	PGA	Effective Acceleration		Probability	Return Period
		horizontal	vertical		
OBE	0.24 g	0.160 g	0.048 g	10%	475 years
MCE	0.43 g	0.287 g	0.086 g	2%	2475 years

Table 5-1: Earthquake parameters for upper reservoir

5.3 Wind forces

The "Israeli Standard" No. 414 for wind forces and velocities to be considered during structure design defines as "average velocity in a period of 10 minutes, for a repetition period of 50 years, 10 m above ground level, in an open area".

In the table below, the wind velocities to be used for PSPP Manara project are shown.

Reservoir	Manara
Upper	33m/s
Lower	27m/s

Table 5-2: Wind velocities

6 DESCRIPTION OF THE PROJECT PARTS

6.1 General

6.1.1 Reference drawings

NUMBER	TITLE
MANARA/BD/CD/01/001	LAYOUT
MANARA/BD/CD/01/002	LAYOUT (PHOTO)

6.2 Upper Reservoir

6.2.1 Reference drawings

NUMBER	TITLE
MANARA/BD/CD/02/001	LAYOUT
MANARA/BD/CD/02/002	SECTIONS
MANARA/TD/CD/02/003	DETAILS
MANARA/TD/CD/02/004	SEALING AND DRAINAGE SYSTEM
MANARA/BD/CD/02/005	INTAKE
MANARA/BD/CD/02/006	BOTTOM OUTLET
MANARA/TD/CD/02/007	INSTRUMENTATION

6.2.2 Reference reports

NUMBER	TITLE
MANARA/BD/CR/03/001	CALCULATION REPORT

6.2.3 Design description

The upper reservoir is located on a high point of the Manara mountain at a natural ground elevation ranging from 725 masl to 747 masl. The reservoir is situated in the geological formation *Kudr*, which is slightly weathered limestone.

The main parameters of the reservoir can be summarized as follows:

- Active water volume 1 164 000 m^3
- Dead water volume (below MOL) 16 000 m^3
- Crest (sill) elevation: 738.00 (738.80) masl
- Full supply level (FSL): 737.00 masl
- Minimum operating level (MOL): 712.50 masl
- Pool floor elevation (maximum): 711.07 masl
- Freeboard 1.80 m

In the following figures the layout and the main section of the reservoir is shown.

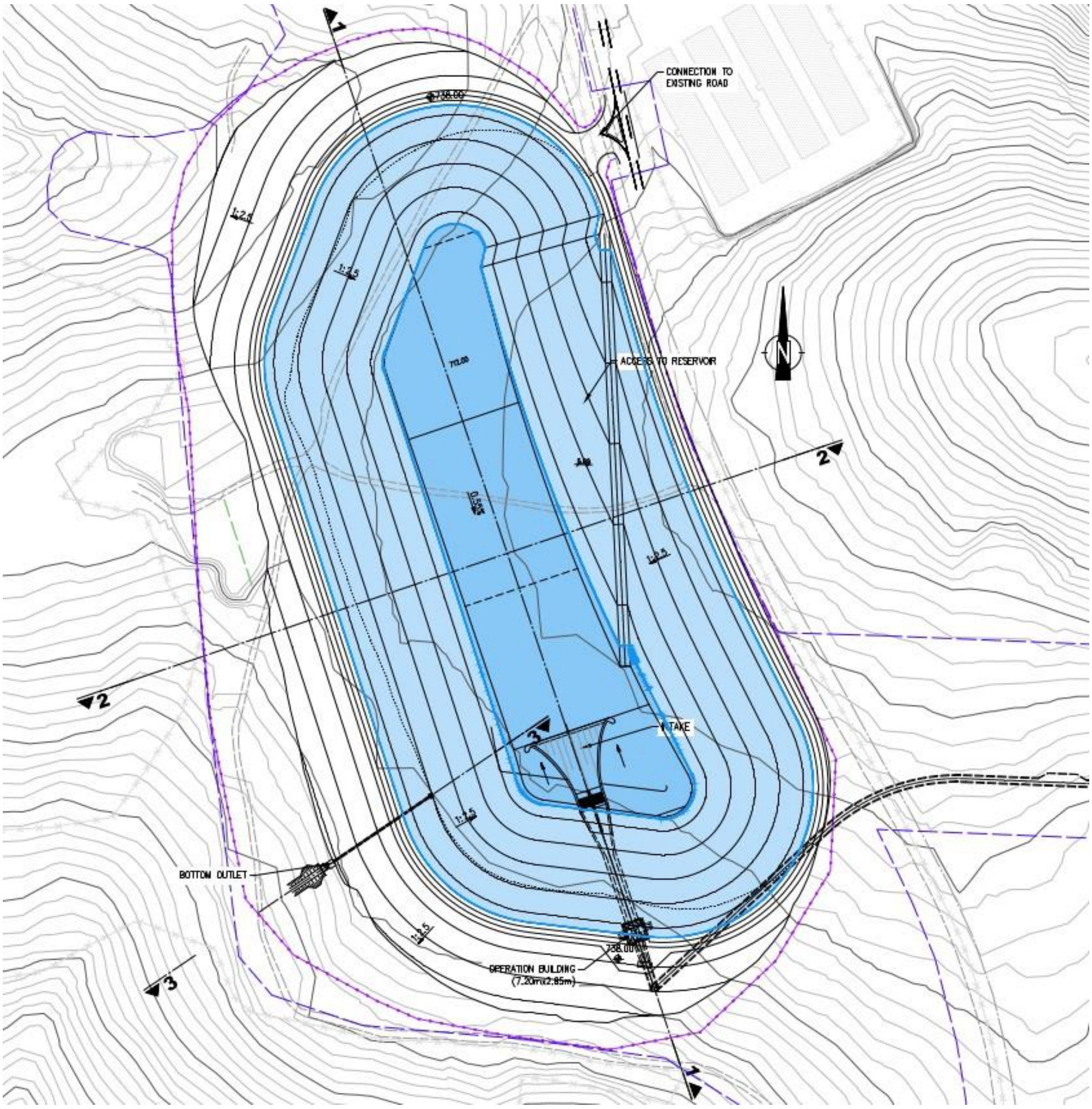

Figure 6-1: Upper reservoir layout

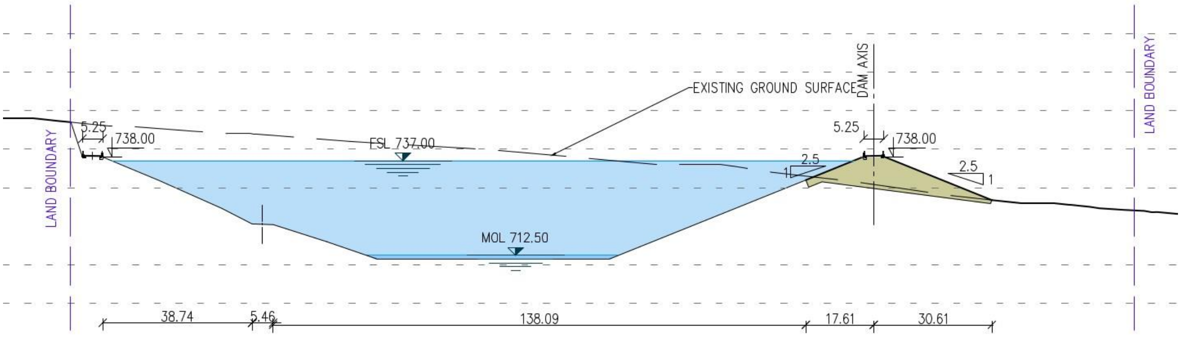

Figure 6-2: Upper reservoir section

The upper reservoir volume is obtained mainly by excavation (mostly in rock material) and partially by a Rock Fill dam. The inner slope inclination of the reservoir is set at 1V/2.5H. This applies to the dam slope and rock excavation slope. The external slope of the dam is designed at 1V/2.5H. The crest elevation is set at 738.00 masl. In combination with a concrete sill with a top at elevation 738.80 masl an adequate freeboard (1.80 m) is provided regarding high wind effect on this particular exposed area.

Due to very low ground water elevation, the risk of uplift pressure is limited. However, a drainage system is designed and shall be installed on the whole reservoir bottom surface as show in drawing MANARABDCD02004 (Sealing and drainage system.

The storage volume curve of the upper reservoir is shown in figure below.

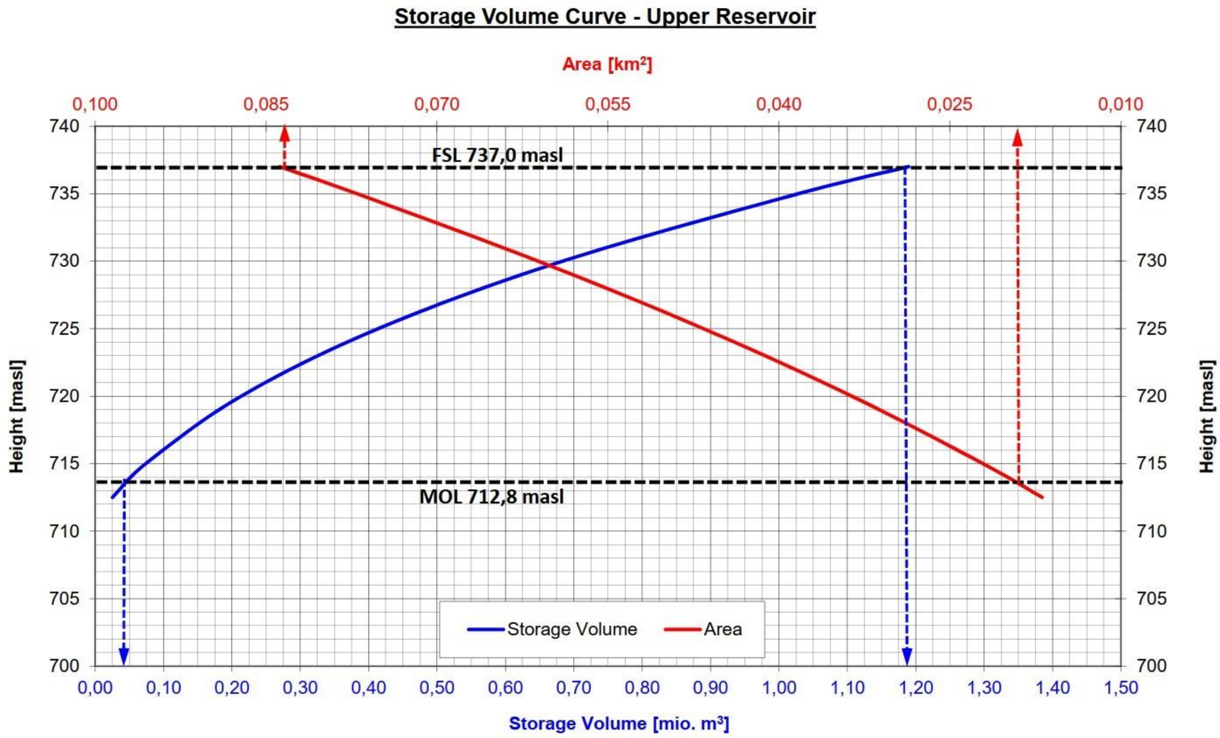

Figure 6-3: Storage volume curve - upper reservoir

6.2.4 Material for dam construction

The main quantities for upper reservoir construction are shown in table below.

ITEM	QUANTITY
Excavation of overburden	40 000 m^3
Excavation of rock	1 208 000 m^3
Fill dam	225 000 m^3
Sealing of reservoir floor	20 200 m^2
Sealing of reservoir slopes	66 000 m^2

Table 6-1: Material quantities for upper reservoir

The rock excavation of the upper reservoir is intended to be used

- for the fill dam construction at the upper reservoir
- for the fill dam construction of the lower reservoir

The above shown table indicates that the quantity of rock excavation at the upper reservoir is adequate to be used as fill material for the entire rock fill dams.

6.2.5 Water proofing and drainage system

The sealing of the Upper Reservoir is proposed by a geomembrane sealing system (GSS). As the GSS for the Lower Reservoir, a PVC geocomposite is foreseen. The geocomposite is made of a 2.5 mm thick thermoplastic geomembrane coupled during fabrication to a 500 g/m2 geotextile. In order to increase the durability in the demanding climate of Israel, the geomembrane may include a special "solar shield", made of a surface lacquered treatment. As anchoring system for the GSS, anchor trenches have to be designed (design for wind / storm, groundwater). Further detailing shall be carried out in the detailed design phase. A concept for driving on the GSS to the intake area with maintenance vehicle has to be provided.

Due to very low ground water elevation, the risk of uplift pressure is limited. However, a drainage system is designed and shall be installed on the whole reservoir bottom surface as show in drawing MAN156TDCD02004 (Sealing and drainage system). A minimum of 1 m of water depth will be maintained below MOL.

The drainage system in the bottom is embedded in a gravel layer with a thickness of 30 cm. The drainage system consists of pipes DN 150 collecting the seepage water at the reservoir floor below the asphalt layers. The collected seepage water flows by gravity to the central drainage shaft, which is part of the intake structure.

As an alternative an asphaltic layer consisting of:

- mastix
- 8 cm asphaltic dense layer
- 8 cm asphaltic binder layer

are overlaying the gravel layer and form a sealing plane in the reservoir bottom and slopes.

6.2.6 Dam stability calculations

A proof of the dam and reservoir slope stability was executed and is shown in below listed report. It can be concluded, that the stability of the slopes and the dam is ensured with the chosen design.

NUMBER	TITLE
MANARA/BD/CR/02/001	CALCULATION REPORT

6.2.7 Freeboard evaluation

The freeboard evaluation for the upper reservoir has been performed in accordance with [G2] and [L1]. The freeboard calculation is shown in

NUMBER	TITLE
MANARA/BD/CR/02/001	CALCULATION REPORT

For the upper reservoir a required freeboard of 1.80 m was calculated. With a FSL level of 737.00 masl plus freeboard results in a minimum crest elevation of 738.80 masl.

The crest elevation is set at 738.00 masl. In combination with a concrete sill with a top at elevation 738.80 masl an adequate freeboard of 1.80 m is provided regarding high wind effect on this particular exposed area.

6.2.8 Bottom outlet

A bottom outlet on the south-western part of the upper reservoir is installed to lower the reservoir water level in case of the following extreme event situation:

- a seismic event damages the membrane and puts the embankment at risk
- the powerhouse is out of order
- possible military events

For this situation it must be possible to lower the reservoir water level. The criterion to be applied, is to lower the reservoir water level from FSL to an elevation where the total thrust on the upstream dam body is halved in less than 8 days.

The calculations are shown in detail in following report:

NUMBER	TITLE
MANARA/BD/CR/02/001	CALCULATION REPORT

Bottom outlet details are shown in following drawing:

NUMBER	TITLE
MANARA/BD/CD/02/006	BOTTOM OUTLET

The bottom outlet starts at the upstream dam toe with a trumpet-shaped inlet structure. That inlet is equipped with a trash rack. The design of the trash rack shall be carried out during detail design stage. The inlet structure is followed by an approximately 91 m long steel pipe DN 400. The steel pipe is completely embedded in concrete. The steel pipe ends at the bottom outlet building, which is placed at the downstream toe of the dam.

A revision valve (DN 400) and a regulating valve (DN 400) are situated in the bottom outlet building. The bottom outlet building is followed by an impact basin with a length of 4.1 m serving as energy dissipater.

The impact basin is followed by a rip rap reinforced open channel.

6.2.9 Upper intake

The calculations are shown in detail in following report:

NUMBER	TITLE
MANARA/BD/CR/02/001	CALCULATION REPORT
MANARA/BD/CR/02002	CFD

Details are shown in following drawings:

NUMBER	TITLE
MANARA/BD/CD/02/001	LAYOUT
MANARA/BD/CD/02/005	INTAKE

The design of the upper intake is based on already executed intake structures (similar geometry and discharge) with good hydraulic behaviour, concerning inflow behaviour and “no swirl formation” in operation. Nevertheless a check of the intake submerge according Gordon/Knauss [L3] has been executed. The design load case is turbine mode at MOL. Additionally the flow pattern have been checked using a CFD simulation. The results are shown in a separate report and show a good flow pattern behaviour.

The upper intake is located on the southern side of the upper reservoir. The intake structure is composed of:

- an inclined concrete slab connecting the reservoir bottom and the intake invert
- an inlet with bottom elevation of 704.55 masl and equipped with a trash rack, a stoplog and a roller gate.

The stoplog is stored on a platform created at crest elevation adjacent to the operation building. The platform is created by means of a concrete retaining wall.

The rated discharges are:

- 38 m^3/s in turbine mode
- 30 m^3/s in pumping mode

The inner structural faces are designed to satisfy adequate flow patterns in both turbine and pumping mode.

The continuity of the reservoir water tightness is provided along the external sides of the intake structure and retaining walls.

A small operation building is situated south of the intake structure at crest elevation, which provides the necessary equipment's for the gate operation. Therefore it will be referenced to report MANARABDGR01002. The building is 7.0 m long, has a width of 3.2 m and a height of about four meters.

6.2.10 Instrumentation

The reservoir is equipped with the following instruments (see also MANARABDCD02007):

- Geodetic measurement points: 18 points are places around the dam of the reservoir on the dam crest.
- Levelling points: For verification of the geodetic measurement points, additionally 18 levelling points are placed on the crest (same position as the geodetic measurement points).
- Drainage measurement weir: The drainage water is measured at the drainage shaft placed at the intake building.
- Staff gauge: One staff gauge is placed at the intake building
- Water level meter: A water level measurement is placed at the intake building with a two out of three measurement (redundant).
- Strong motion accelerograph: On the highest dam section one SMA is placed on the dam crest and one on the dam toe for verification of the accelerations during seismic events.
- Open standpipe piezometer: 5 open standpipe piezometers are placed at the upper reservoir.
- Vibrating wire piezometer (VWP): 2 vibrating wire piezometer are placed in dam axis.
- Vertical extensometer (VEX): Two dam sections are equipped with vertical extensometers, whereat each section has install three extensometers for the verification of the settlements during construction and during operation.

6.3 Lower Reservoir

6.3.1 Reference drawings

NUMBER	TITLE
MANARA/BD/CD/03/001	LAYOUT
MANARA/BD/CD/03/002	SECTIONS
MANARA/TD/CD/03/003	DETAILS
MANARA/TD/CD/03/004	SEALING AND DRAINAGE SYSTEM
MANARA/BD/CD/03/005	INTAKE
MANARA/BD/CD/03/006	INTAKE AND OPERATION BUILDING
MANARA/TD/CD/03/007	BOTTOM OUTLET
MANARA/TD/CD/03/008	INSTRUMENTATION

6.3.2 Reference reports

NUMBER	TITLE
MANARA/BD/CR/03/001	CALCULATION REPORT

6.3.3 Design description

The Lower Reservoir is located south of the town Kiryat Shmona at a natural ground elevation of approximately 72 masl. The lower reservoir will be situated on fat clay, silt and sand, carbonate concretions, shell fragments and pebbles.

The main parameters are:

- Active water volume 1 196 000 m^3
- Dead water volume (below MOL) 160 400 m^3
- Crest (sill) elevation: 82.00 (82.40) masl
- Full supply level (FSL): 80.70 masl
- Minimum operating level (MOL): 73.25 masl
- Pool floor elevation (maximum): 72.00 masl
- Freeboard 1.70 m

The lower reservoir is located on an existing “grey” water storage reservoir. Accordingly, its demolition is to be completed prior to proceeding with reservoir construction. Due to poor quality of water currently stored material (organic material, polluted water, etc.) a necessary foundation treatment might become necessary. After pumping out the existing reservoir and excavating the foundation for the new dam, additional ground survey tests have to be executed.

As the dam is placed manly on fat clay, silt and sand it may become necessary to use a geotextile at the dam foundation before the dam fill material is placed, to separate the clay from the dam fill material. Type and necessity has to be proven during detail design phase. As alternative a graded transition layer could be implemented.

The lower reservoir volume is obtained by excavation (dark brown fat clay, silt and sand with organic matter) and mainly construction of an PVC faced rock fill dam. The inner slope of the dam is set at 1V/2.5H. The external slope of the dam is designed at 1V/2.5H. The crest elevation is set at 82.00 masl. In combination with a concrete balustrade with a top at elevation 82.40 masl an adequate freeboard (1.70 m) is provided regarding high wind effect on this particular exposed area.

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In the following figures the layout and the main section of the reservoir are shown.

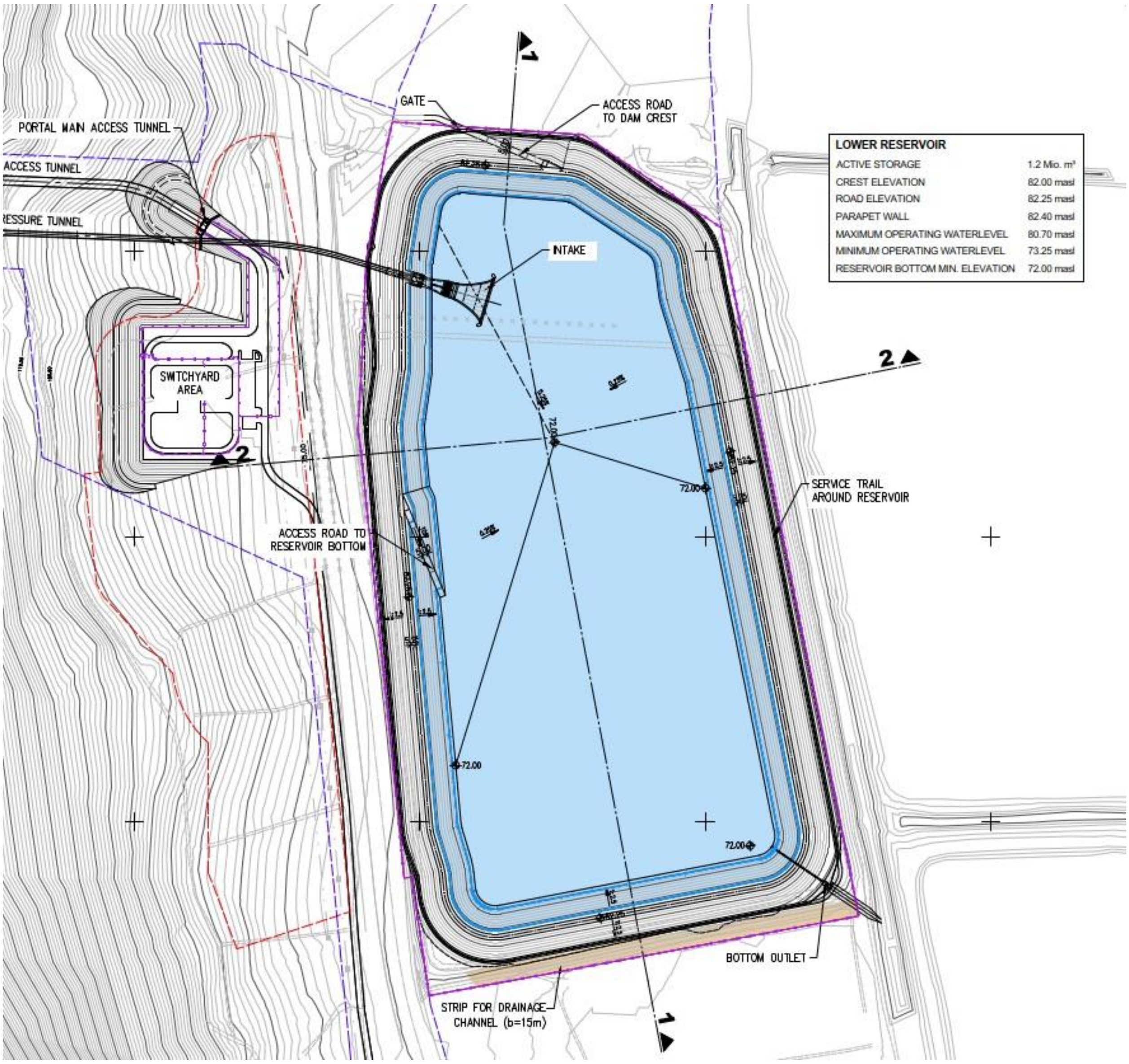

Figure 6-4: Lower reservoir layout

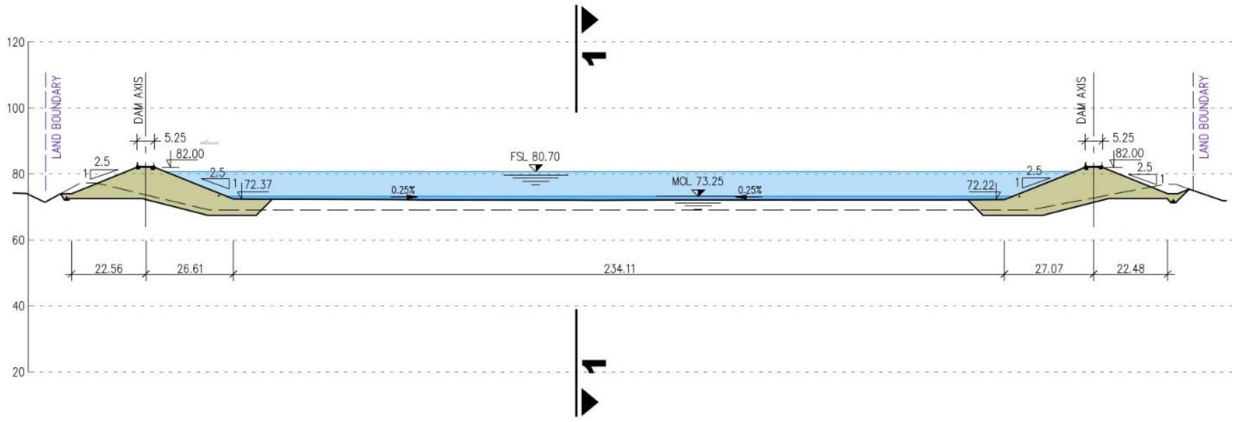

Figure 6-5: Lower reservoir section

The storage volume curve of the Lower Reservoir is shown in figure below:

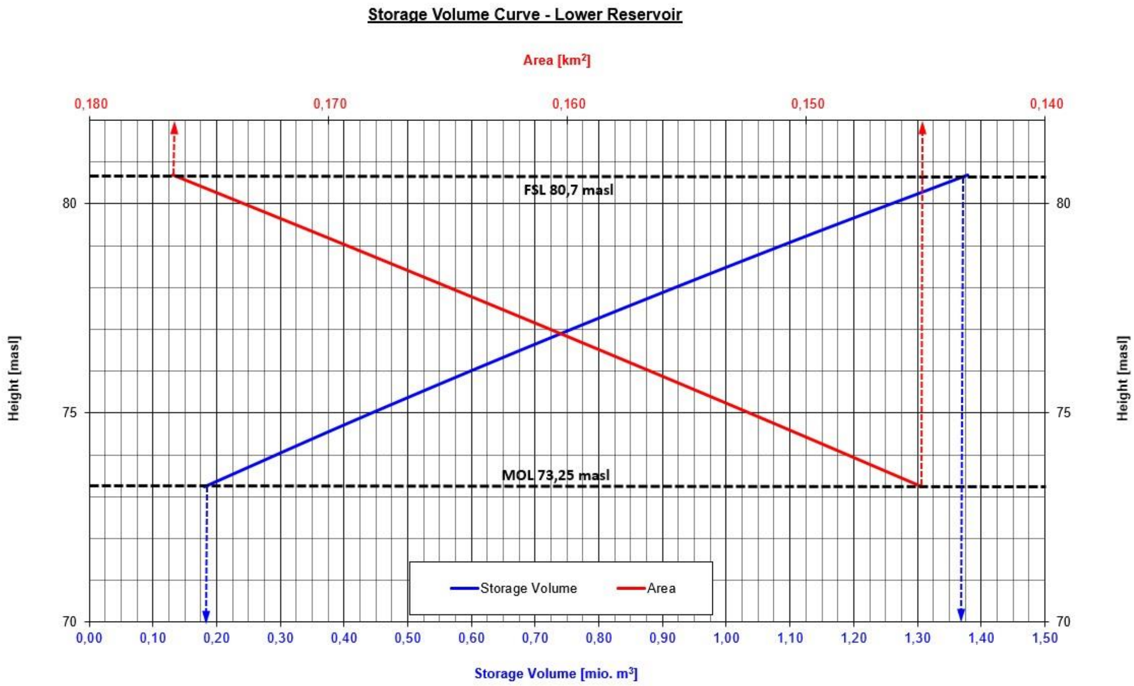

Figure 6-6: Storage volume curve - Lower Reservoir

6.3.4 Material for dam construction

The main quantities for lower reservoir construction are shown in table below:

ITEM	QUANTITY
Excavation	266 500 m³
Rock fill dam	658 800 m³
PVC sealing of reservoir slopes and floor	186 300 m²

Table 6-2: Material quantities for lower reservoir

It shall be assumed that the area of the existing fish pond is polluted and strength parameters of excavated soil, including material from existing fish pond, show very low values. Therefore, rock excavation material from tunnels and cavern as well as the excavation from the upper reservoir is intended to be used for the fill dam construction of the lower reservoir.

6.3.5 Ground water treatment of construction area

As the ground water level at the lower reservoir is intended to be in the range of, or slightly below surface elevation, a dewatering system for the construction area of the lower reservoir has to be installed, just before the construction work starts.

As the subsoil is mainly consisting of dark brown fat clay, silt and sand with organic matter, a system of drainage pits, collection ditches and pit pumping is intended to be a suitable system. Before starting with the construction work, a detailed drainage system has to be designed to ensure a dry construction base for placing the dams and the surface sealing of the reservoir.

Special attention has to be paid to the area where the intake is situated. In this area the construction pit for building the intake and the lower pressure tunnel is some 10 m below the estimated ground water level. A temporary retaining wall system using for example sheet piling will be necessary for achieving a dry and safe construction pit. A system using additional drainage piles in the vicinity of the intake area may also be necessary, for achieving a dry construction pit. This has to be defined in detail design phase.

6.3.6 Water proofing and drainage system

The sealing of the Lower Reservoir is proposed by a geomembrane sealing system (GSS). As the GSS for the Lower Reservoir, a PVC geocomposite is foreseen. The geocomposite is made of a 2.5 mm thick thermoplastic geomembrane coupled during fabrication to a 500 g/m2 geotextile. In order to increase the durability in the demanding climate of Israel, the geomembrane may include a special “solar shield”, made of a surface lacquered treatment. As anchoring system for the GSS, anchor trenches have to be designed. Further detailing shall be carried out in the detailed design phase. A concept for driving on the GSS to the intake area with maintenance vehicle has to be provided.

The ground water level at the lower reservoir is expected just a few meters below ground surfaces (1-2) with seasonal fluctuation. An existing drainage ditch leading on the east side of the reservoir, from north towards south, with a bottom elevation of ~70.5 masl (north) and ~68.0 masl (south) is controlling the level.

The bottom of the reservoir with an elevation of 72.00 masl is a little bit above the expected ground water elevation. The MOL is with 73.25 masl is above the GWL. To secure the maximum height of the groundwater level and to prevent uplift problems on the reservoir sealing, small adoptions of the existing ditch might become necessary. This subject has to be checked once again in detail during execution design considering detail survey data of the ditch.

Due to the high ground water elevation only a drainage system for the dam slopes is foreseen. The drainage system consists of

a. a drainage pipe DN 150 (each per drainage field) within a gravel layer below the PVC layer parallel to the dam crown
b. a pipe DN 150 between “a.” and “c.”
c. a perimeter drainage channel on the outside toe of the dam

The collected drainage water is conveyed by gravity to the south of the reservoir and thereafter via a riprap lined channel to the existing ditch.

The drainage system is show in drawing MAN156BDCD03004 (Sealing and drainage system).

As an alternative an asphaltic layer consisting of:

- mastix
- 8 cm asphaltic dense layer
- 8 cm asphaltic binder layer

are overlaying the gravel layer and form a sealing plane in the reservoir bottom and slopes.

6.3.7 Dam stability calculations

A proof of the dam stability was executed and is shown in detail in below listed report. It can be concluded, that the dam stability is secured for all analysed load cases.

A preliminary calculations for estimating the settlements has been executed. The settlement calculations are based on material parameters derived from core drillings. The derived material parameters are deemed to be conservative. Therefore, subsoil parameters shall be verified preferably after excavation of the lower reservoir and prior to start of fill dam construction for example by means of plate baring tests. Detailed recalculations shall be performed during detail design stage.

NUMBER	TITLE
MANARA/BD/CR/03/001	CALCULATION REPORT

6.3.8 Freeboard evaluation

The freeboard evaluation for the Lower Reservoir has been performed in accordance with [G2] and [L1]. The freeboard calculation is shown in:

NUMBER	TITLE
MANARA/BD/CR/03/001	CALCULATION REPORT

For the lower reservoir a required freeboard of 1.70 m is calculated. With a FSL level of 80.70 masl plus freeboard results in a minimum crest elevation of 82.40 masl.

The crest elevation is set at 82.00 masl. In combination with a concrete sill with a top at elevation 82.40 masl an adequate freeboard (1.70 m) is provided regarding high wind effect on this particular exposed area.

6.3.9 Bottom outlet

A bottom outlet on the south-eastern part of the lower reservoir is designed to lower the reservoir in case of the following extreme event situation:

- a seismic event damages the membrane and puts the embankment at risk
- the powerhouse is out of order
- possible military events

For this situation it must be possible to lower the reservoir elevation. The criterion to be applied is to lower the reservoir elevation from FSL to an elevation where the total thrust on the upstream dam body is halved in less than 8 days.

The calculations are shown in detail in following report:

NUMBER	TITLE
MANARA/BD/CR/03/001	CALCULATION REPORT

Details are shown in following drawing:

NUMBER	TITLE
MANARA/BD/CD/03/007	BOTTOM OUTLET

The bottom outlet starts at the upstream dam toe of the AFRD with a trumpet-shaped inlet structure. That inlet is equipped with a trash rack. The design of the trash rack shall be carried out during Detail Design. The inlet structure is followed by an approximately 77 m long steel pipe DN 500. The steel pipe is completely embedded in concrete. The steel pipe ends at the bottom outlet building, which is placed at the downstream toe of the AFRD dam.

A revision valve (DN 500) and a regulating valve (DN 500) are situated in the bottom outlet building. The bottom outlet building is followed by an impact basin with a length of 4.1 m serving as energy dissipater.

The impact basin is followed by a rip rap reinforced open channel, which leads into an existing ditch.

6.3.10 Lower intake

The calculations are shown in detail in following report:

NUMBER	TITLE
MANARA/BD/CR/03/001	CALCULATION REPORT
MANARA/BD/CR/03/002	CFD

Details are shown in following drawings:

NUMBER	TITLE
MANARA/BD/CD/03/001	LAYOUT
MANARA/BD/CD/03/005	INTAKE
MANARA/BD/CD/03/006	INTAKE AND OPERATION BUILDING

The design of the lower intake is based on already executed intake structures (similar geometry and discharge) with good hydraulic behaviour, concerning inflow behaviour and “no swirl formation” in operation. Nevertheless a check of the intake submerge according Gordon/Knauss [L3] has been executed. The design load case is pump mode at MOL. Additionally the flow pattern have been checked using a CFD simulation. The results are shown in a separate report and show a good flow pattern behaviour.

The lower intake is located on the north-west part of the lower reservoir.

The intake structure is composed of:

- an inclined concrete slab connecting the reservoir bottom and the intake invert
- an inlet with bottom elevation of 64.68 masl and equipped with a trash rack, a stoplog and a roller gate.

The stoplog is stored on a platform created at crest elevation adjacent to the operation building. The platform is created by means of a concrete retaining wall.

The rated discharges are:

- 38 m^3/s in turbine mode
- 30 m^3/s in pumping mode

The inner structural faces are designed to satisfy adequate flow patterns in both turbine and pumping mode.

The continuity of the reservoir water tightness is provided along the external sides of the intake structure and retaining walls. In order to avoid differential settlements that could damage the membrane, piles shall be provided. Their design shall be part of the detailed design stage.

A small operation building is situated some thirty meters south-west of the intake structure at crest elevation, which provides the necessary equipment’s for the gate operation. Therefore it will be referenced to report MANARABDGR01002. The building is 7.0 m long, has a width of 3.2 m and a height of about four meters.

6.3.11 Instrumentation

The reservoir is equipped with the following instruments (see also MANARABDCD03008):

- Geodetic measurement points: 37 points are places around the dam of the reservoir on the dam crest and the dam bottom.
- Levelling points: For verification of the geodetic measurement points, additionally 37 levelling points are placed on the crest (same position as the geodetic measurement points).

- Drainage measurement weir: The drainage water is measured at the southern area of the reservoir. One weir belongs to the east part and one to the west part of the reservoir.
- Seepage inspection point: On every location where the drainage pipe of the dam is connected to the drainage channel, an inspection opening in the channel cover is foreseen (quantity 10).
- Staff gauge: One staff gauge is placed at the intake building
- Water level meter: A water level measurement is placed at the intake building with a two out of three measurement (redundant).
- Open standpipe piezometer: 15 open standpipe piezometers are placed at the lower reservoir.
- Vibrating wire piezometer (VWP): 5 vibrating wire piezometer are placed in dam axis.
- Vertical extensometer (VEX): Two dam sections are equipped with vertical extensometers, whereat each section has install three extensometers for the verification of the settlements during construction and during operation.

6.4 Power Waterway

The power water way can be divided into the following parts:

- High pressure shaft
- High pressure tunnel
- High pressure manifold
- Low pressure tunnel including surge tank and flexible joint

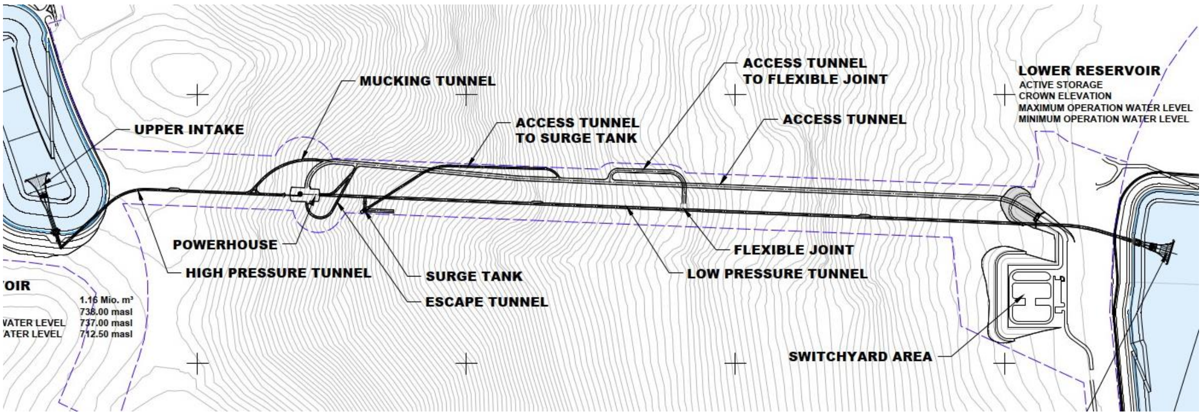

Figure 6-7: Layout of the power water way

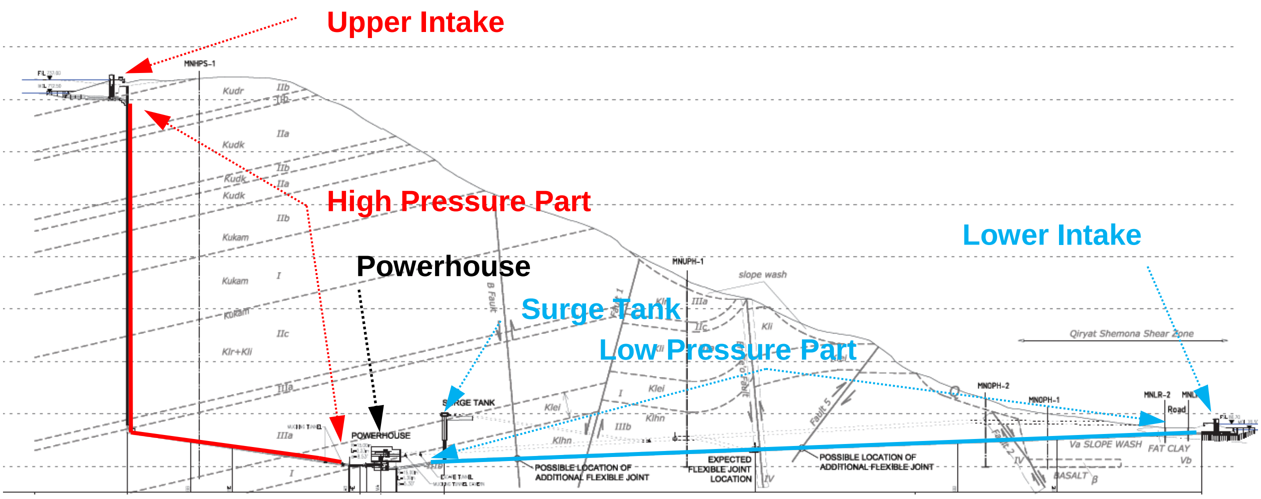

Figure 6-8: Longitudinal section of the power water way

In the following chapter the individual parts are described in detail.

6.4.1 High pressure shaft

6.4.1.1 Reference drawings

NUMBER	TITLE
MANARA/BD/CD/04/001	POWER WATER WAY - LAYOUT
MANARA/BD/CD/04/002	POWER WATER WAY - LONGITUDINAL SECTION
MANARA/BD/CD/04/007	POWER WATER WAY - SUPPORT CLASSES
MANARA/BD/CD/04/008	POWER WATER WAY - SUPPORT CLASSES
MANARA/BD/GG/04/001	POWER WATER WAY - VERTICAL SHAFT GEOLOGY
MANARA/BD/GG/04/002	POWER WATER WAY - HP TUNNEL GEOLOGY

6.4.1.2 Reference reports

NUMBER	TITLE
MANARA/BD/CR/04/001	POWER WATERWAY - HYDRAULIC CALCULATIONS
MANARA/BD/CR/04/002	POWER WATERWAY - EXCAVATION, SUPPORT AND LINING

6.4.1.3 Design description

The high pressure shaft is located below the upper reservoir intake structure. The top of the pressure shaft is set at elevation 686.50 masl (bottom of intake structure) and its bottom elevation is 65.37 masl, leading to a total height of 621.13 m. The internal diameter of the high pressure shaft is 3.00 m.

The shaft is steel lined along the whole length, whereat the lining thickness is in a range from 10 mm to 25 mm, using a steel quality of S460M / S550M. The pre-calculation of the lining is shown in the referenced report in detail. The lining is embedded in concrete, with a thickness of 0.50 m.

At the bottom of the shaft an elbow is serving as a transition to the adjacent high pressure tunnel.

The excavation diameter is in a range from 4.15 up to 4.40 m depending on the rock quality and the according support class.

For the construction of the vertical pressure shaft the following construction stages are recommended:

- Execution of target drill hole from the bottom of the intake structure to the toe of the vertical pressure shaft
- Widening of the borehole by means of raise boring with a diameter of approximately 1.5 m. The mucking of the excavation material is done via subjacent high pressure tunnel, mucking tunnel and access tunnel.
- Widening of the borehole by drill and blast method and excavation mucking via shaft borehole and subjacent high pressure tunnel, mucking tunnel and access tunnel.
- Application of steel lining segments and backfilling concrete

The excavation support classes for the third construction stage (drill and blast) can be seen in the related drawings. Four support classes are designed, whereat an estimation of the percentage for each class can be seen in the related geology drawing of the power water way.

6.4.2 High Pressure Tunnel

6.4.2.1 Reference drawings

NUMBER	TITLE
MANARA/BD/CD/04/001	POWER WATER WAY - LAYOUT
MANARA/BD/CD/04/002	POWER WATER WAY - LONGITUDINAL SECTION
MANARA/BD/CD/04/009	POWER WATER WAY - SUPPORT CLASSES
MANARA/BD/CD/04/010	POWER WATER WAY - SUPPORT CLASSES
MANARA/BD/GG/04/001	POWER WATER WAY - VERTICAL SHAFT GEOLOGY
MANARA/BD/GG/04/002	POWER WATER WAY - HP TUNNEL GEOLOGY

6.4.2.2 Reference reports

NUMBER	TITLE
MANARA/BD/CR/04/001	POWER WATERWAY - HYDRAULIC CALCULATIONS
MANARA/BD/CR/04/002	POWER WATERWAY - EXCAVATION, SUPPORT AND LINING

6.4.2.3 Design description

The high pressure tunnel connects the toe of the high pressure shaft with the high pressure manifold upstream of the underground powerhouse. The high pressure tunnel axis elevation varies from 65.37 masl to 2.00 masl with a constant slope inclination of 15% and a length of 415 m.

The high pressure tunnel is steel lined, whereat the lining thickness is in a range from 25 mm to 30 mm, using a steel quality of S550M / S700M. The pre-calculation of the lining is shown in the referenced report in detail. The lining is embedded in concrete, with a thickness of 0.50 m.

The excavation of the tunnel is recommended with drill and blast method. The high pressure tunnel shows a horse shaped excavation cross section with a minimum excavation diameter of 4.2 m. The inner diameter after installation of the final lining is 3.0 m.

The excavation support classes can be seen in the related drawings. Four support classes are designed, whereat a estimation of the percentage for each class can be seen in the related geology drawing of the power water way.

6.4.3 High pressure manifold

6.4.3.1 Reference drawings

NUMBER	TITLE
MANARA/BD/CD/04/001	POWER WATER WAY - LAYOUT
MANARA/BD/CD/04/002	POWER WATER WAY - LONGITUDINAL SECTION

6.4.3.2 Reference reports

NUMBER	TITLE
MANARA/BD/CR/04/001	POWER WATERWAY - HYDRAULIC CALCULATIONS
MANARA/BD/CR/04/002	POWER WATERWAY - EXCAVATION, SUPPORT AND LINING

6.4.3.3 Design description

The high pressure manifold connects the high pressure tunnel with the underground powerhouse. The high pressure manifold axis elevation is at elevation 2.00 masl (inclination of 0%) and has a length of approximately 25 m.

The high pressure manifold excavation shows an inverted U-shaped excavation cross section with a minimum excavation diameter of 3.5/4.0 m. The manifold is a free penstock with a diameter of 1.50 m and is connected with the main inlet valve.

Pulling forces from a closed main inlet valve are transferred from the steel lining via backfill concrete into the rock mass at fix points upstream of the underground powerhouse.

During construction the access to the high pressure manifold takes place via access tunnel and mucking tunnel. The mucking tunnel will be plugged at the intersection to the high pressure tunnel, after the construction of the high pressure part is finished.

<u>Mucking tunnel:</u>

This tunnel is the link between the access tunnel to the underground powerhouse and the high pressure tunnel. This tunnel is used to dissolve the work on the high pressure manifold, the high pressure tunnel from the work on the powerhouse. The mucking tunnel has an inverted U-shaped cross section with:

- width = 3.50 m
- height = 3.50 m

The mucking tunnel is lined with shotcrete.

The designed radius and junction to the MAT allows the transport of up to 12 m long steel lining parts by the mucking tunnel.

6.4.4 Low Pressure Tunnel

6.4.4.1 Reference drawings

NUMBER	TITLE
MANARA/BD/CD/04/001	POWER WATER WAY - LAYOUT
MANARA/BD/CD/04/002	POWER WATER WAY - LONGITUDINAL SECTION
MANARA/BD/CD/04/011	POWER WATER WAY - SUPPORT CLASSES
MANARA/BD/CD/04/012	POWER WATER WAY - SUPPORT CLASSES
MANARA/BD/CD/04/013	POWER WATER WAY - SUPPORT CLASSES
MANARA/BD/CD/04/014	POWER WATER WAY - SUPPORT CLASSES

6.4.4.2 Reference reports

NUMBER	TITLE
MANARA/BD/CR/04/001	POWER WATERWAY - HYDRAULIC CALCULATIONS
MANARA/BD/CR/04/002	POWER WATERWAY - EXCAVATION, SUPPORT AND LINING

6.4.4.3 Design description

The first portion of the low pressure tunnel is ranging from chainage 0+512 to chainage 0+604. This 92 m long low pressure tunnel part is the link between the low pressure manifold (downstream of the underground powerhouse) and the surge tank. Its axis elevation varies from elevation -5.22 masl to 9.62 masl, with a slope inclination of 15%.

The second portion of the low pressure tunnel is ranging from chainage 0+604 to chainage 2+018. This 1414 m long low pressure tunnel is the link between the surge tank and the lower intake. Its axis elevation varies from elevation 9.62 masl to 66.58 masl, with a slope inclination of 4.03 %.

The excavation of the tunnel is recommended with drill and blast method. The low pressure tunnel shows a horse shaped excavation cross section with a minimum excavation diameter of 5 m. The inner diameter after installation of the final lining is 3.8 m. As final lining a concrete lining with a thickness of 0.40 m is designed. The lining can be divided into three main sections:

1) Chainage 0+512 to 0+680:
   This part of the lining is reinforced
2) Chainage 0+680 to 1+501:
   In this section the lining is grouted with high pressure. This should achieve contact between the lining and the rock surface and avoids overload of the lining by internal water pressure. The high pressure grouting acts in the gap between the final lining and the shotcrete lining.
3) Chainage 1+501 to 2+018:
   In this section high pressure grouting is not possible. Therefore this part is reinforced.

As an <u>alternative solution</u> steel lining of the LPT can be chosen. The inner diameter can be reduced, so that the hydraulic losses are the same as in the concrete alternative, whereas the excavation cross section remains as described above as the excavation is deemed to be at a lower limit for construction works. Thickness of steel lining is 11 mm. If possible the minimum diameter of the excavation cross section can be reduced but at least a minimum of 40 cm concrete lining is requested. This solution might have advantages in the higher progress of work and lower risk for surface defects (of the concrete lining surface).

Based on the geological baseline report the groundwater level along the low pressure tunnel in the rock is above the maximal internal pressure in the tunnel. Under these conditions additional sealing by foil installed between concrete and shotcrete lining could be omitted. As the MAT acts as a drain, the ground water level along the LP tunnel will be lowered. The lowering under the maximal internal pressure will require additional sealing (PVC foil) to reduce losses in the power water during the operation. This item has to be monitored during construction and the decision must be defined based on the observed conditions and in consultation with the Owner.

For the preliminary calculations of the lining it is referenced to the related report in detail. For the high pressure grouting section the following can be stated. The high pressure grouting should achieve contact between the lining and rock surface for all loading conditions and avoids overload of the lining by internal water pressure. The pressure is applied by high pressure consolidation grouting through radial boreholes. The pressure acts in the rock mass filling the existing fissures and joints in the rock mass and especially in the gap between the final lining and the shotcrete lining. The grouting pressure will be reduced by creep and shrinkage of the rock and concrete and by cooling of the lining caused by the first filling. The grouting pressure must be high enough that after reduction of the pressure the whole lining is for all loading conditions during operation (maximal internal water pressure) under compression. Therefore no tensile forces are acting on the lining. The injections have to be executed using radial placed bore holes. In this boreholes a simultaneously injection process has to be executed, to achieve a continuous pressure around the lining. Number and spacing of the injection holes have to be designed in the detail design phase. The injection pressure will be up to 25 bar.

The excavation support classes can be seen in the related drawings. Five support classes are designed, whereat a estimation of the percentage for each class can be seen in the related geology drawing of the power water way. In the area where the pressure tunnel crosses below the highway support class V is foreseen, which includes a pipe roof along the whole section. The calculations are shown in detail in the referenced report. For the excavation phase of the low pressure tunnel beneath the Highway 90, the whole section will be supported using pipe roof for the crown and walls, steel aches and at least 25 cm shotcrete lining for the whole section. It is referenced to the related drawings showing the support class V of the low pressure tunnel. The maximal excavation length can be estimated with 1 m. The section is estimated to be 106 m long. A pre-design of the support is shown in the referenced report. The final lining for this section is reinforced.

As already mention in chapter 5.1, along the alignment the low pressure tunnel will cross three possible faults. At least one of this fault faults – En Te´o Fault – is suspected as active with expected movements of ~1 mm/year in all directions. Therefore for the most likely fault location (chainage 1+196) a flexible joint is designed, which can deal with this movements. A description of the flexible joint is shown in chapter 0 below, whereas a description of the hydro mechanical equipment is given in the report MANARABDGR01002.

During the excavation of the low pressure tunnel and the main access tunnel the faults have to be mapped and monitored. An evidence has to be made by a geologists regarding fault activity. A flexible joints has to be installed in dedicated chambers, where the fault is active in order to absorb the predicted movements.

Substantial length of the low pressure tunnel and the main access tunnel will be excavated inside ground water.

6.4.5 Flexible joint chamber

6.4.5.1 Reference drawings

NUMBER	TITLE
MANARA/BD/CD/04/001	POWER WATER WAY - LAYOUT
MANARA/BD/CD/04/002	POWER WATER WAY - LONGITUDINAL SECTION
MANARA/BD/CD/04/004	POWER WATER WAY - FLEXIBLE JOINT
MANARA/BD/CD/04/006	ACCESS TUNNEL FLEXIBLE JOINT
MANARA/BD/CD/04/016	ACCESS TUNNEL FLEXIBLE JOINT - SUPPORT
MANARA/BD/CD/04/017	POWER WATER WAY - FLEXIBLE JOINT SUPPORT

6.4.5.2 Reference reports

NUMBER	TITLE
MANARA/BD/GR/01/002	EM AND HM EQUIPMENT - DESIGN REPORT

6.4.5.3 Design description

Flexible joints have to be installed if needed on each active (or potentially active) fault along the low pressure tunnel. For the most likely active fault – En Te´o Fault chainage 1+196 – a flexible joint is designed in this stage. Average annual displacement of 1 mm/year in the 3 directions is expected.

The flexible joint is located in a chamber (clearance length / width / height = 11.50 m / 15.60 m / 10.90 m). The chamber is connected with an access tunnel to the main access tunnel. The cross section of the tunnel allows the transport of the flexible joint elements into the chamber. In the chamber the elements will be manipulated using a monorail installed on the crown of the chamber. The main elements of the flexible joint can be given with:

- a steel lining pipe on both ends of the flexible joint element, partially embedded in the concrete lining of the low pressure tunnel including steel ribs and water stops.
- the flexible joint element itself, which is described in detail in the report MANARABDGR01002

Monitoring of these structures will be required in order to anticipate their maintenance program. The maintenance of the devices (access via access tunnel to flexible joint branching from access tunnel to powerhouse) is also described in the report MANARABDGR01002.

The excavation support of the chamber is shown in the referenced drawings and consists of systematic rock bolting and a shotcrete lining.

Access tunnel to flexible joint chamber:

The access tunnel to the flexible joint chamber is designed for two purposes, namely the function as intermediate access for the construction of the low pressure tunnel and the access to the flexible joint chamber for maintenance during operation.

The access tunnel has an inverted U-shaped cross section with:

- width = 6.50 m
- height = 6.75 m

The excavation support is shown in the referenced drawings, whereas three support classes are designed. The final lining is shotcrete.

6.4.6 Surge Tank

6.4.6.1 Reference drawings

NUMBER	TITLE
MANARA/BD/CD/04/001	POWER WATER WAY - LAYOUT
MANARA/BD/CD/04/002	POWER WATER WAY - LONGITUDINAL SECTION
MANARA/BD/CD/04/005	POWER WATER WAY - SURGE TANK

6.4.6.2 Reference reports

NUMBER	TITLE
MANARA/BD/CR/04/001	POWER WATERWAY - HYDRAULIC CALCULATIONS

6.4.6.3 Design description

A surge tank is situated approximately 120 m downstream of the cavern. The surge tank consists of:

- a 87 m high vertical shaft with an excavation diameter of 4.60 / 8.70 m and an inner diameter of 3.80 / 7.50 m. The concrete lining has a thickness of 0.40 / 0.60 m. The excavation support is assumed to be in a range of 0.10 / 0.15 m.
- a 50 m long horizontal upper surge chamber with invert elevation ranging from 88.50 masl to 89.25 masl. The surge chamber has an inverted U-shaped cross section with:
  - width = 5.0 m
  - height = ranging from 7.57 m to 6.00 m

- radius at top = 2.5 m

and is concrete lined with a minimum lining thickness of 0.40 m.

The upper chamber has a dead storage from elevation 88.50 masl to 90.00 masl, which is used as storage volume for the service water of the cavern. This tank is connected to the cavern equipment with steel pipe DN 250. The steel pipe itself is embedded in the concrete lining of the surge tank / low pressure tunnel.

The connection of the surge tank to the low pressure tunnel is equipped with a steel armoured jet throttle, with a diameter reduction from 3.80 m to 2.00 m. This throttle improves the damping behaviour of the water oscillation in the tank.

Access tunnel to surge tank top:

An access tunnel to the surge tank (branching from the main access tunnel to the powerhouse) is joining the top of the tank and is designed for three purposes, namely the construction works, the maintenance and the aeration of the surge tank during operation.

This tunnel is 420 m long with invert elevation ranging from 47.00 masl to 96.50 masl. The tunnel has an inverted U-shaped excavation cross section with:

- width = 3.00 m
- height = 3.00 m

The excavation support is shown in the referenced drawings, whereas three support classes are designed. The final lining is shotcrete.

6.5 Underground Powerhouse

6.5.1 Reference drawings

NUMBER	TITLE
MANARA/BD/CD/05/001	POWERHOUSE - PLAN VIEW - LEVEL 1
MANARA/BD/CD/05/002	POWERHOUSE - PLAN VIEW - LEVEL 2
MANARA/BD/CD/05/003	POWERHOUSE - PLAN VIEW - LEVEL 3
MANARA/BD/CD/05/004	POWERHOUSE - PLAN VIEW - LEVEL 4
MANARA/BD/CD/05/005	POWERHOUSE - PLAN VIEW - LEVEL 5
MANARA/BD/CD/05/006	POWERHOUSE - PLAN VIEW - LEVEL 6
MANARA/BD/CD/05/007	POWERHOUSE - SECTIONS A-A
MANARA/BD/CD/05/008	POWERHOUSE - SECTIONS B-B
MANARA/BD/CD/05/009	POWERHOUSE - SECTIONS C-C
MANARA/BD/CD/05/010	POWERHOUSE - SECTIONS D-D E-E
MANARA/BD/CD/05/011	POWERHOUSE - FIREFIGHTING
MANARA/BD/CD/05/012	POWERHOUSE - EVACUATION LAYOUTS
MANARA/BD/CD/05/018	POWERHOUSE - EXCAVATION SUPPORT
MANARA/BD/CD/05/019	POWERHOUSE - EXCAVATION SUPPORT
MANARA/BD/CD/05/020	POWERHOUSE - EXCAVATION SUPPORT

6.5.2 Reference reports

NUMBER	TITLE
MANARA/BD/GR/01/002	EM AND HM EQUIPMENT - DESIGN REPORT
MANARA/BD/CR/05/001	POWERHOUSE - EXCAVATION AND SUPPORT

6.5.3 Design description

The underground powerhouse is located approximately 1.5 km inside the mountain, with a rock cover of approximately 600 m. The underground powerhouse is designed as single cavern solution, with a combined power and transformer cavern. This solution ensures that the whole cavern can be situated in the geological formation “Klei- Limestone and dolomite” and in “Klhn- Sandstone black mudstone layers” with enough cover of fair rock mass.

In this lithology the geological conditions are fair for building a large cavern. The location of this formation is interpreted from the drillings MNHPS-1 and MNUPH-1. During excavation of the main access tunnel, the location of this formation has to be verified. If the location of the formation deviates clearly from the expected interpretation of the boreholes, the location of the cavern has to be readjusted to guarantee the placing of the cavern in fair rock conditions.

The cavern has the following main dimensions (excavation related):

- length 55.0 m
- width: 26.0 m
- height: 46.0 m

A layout and a the main cross section are shown in the following figures.

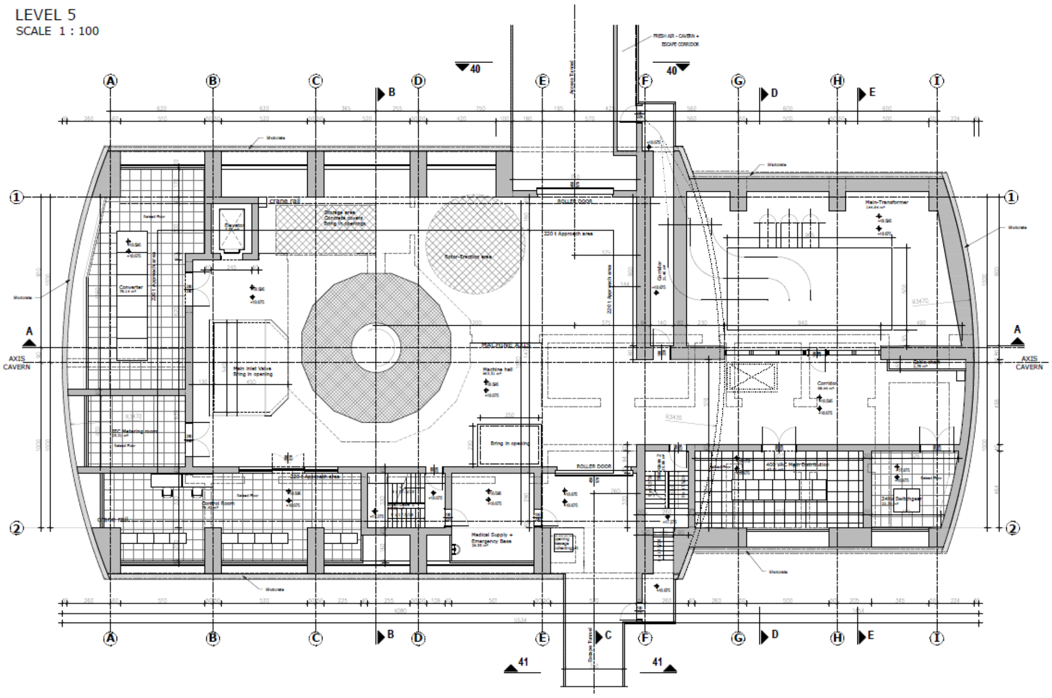

Figure 6-9: Powerhouse plan view - level 5

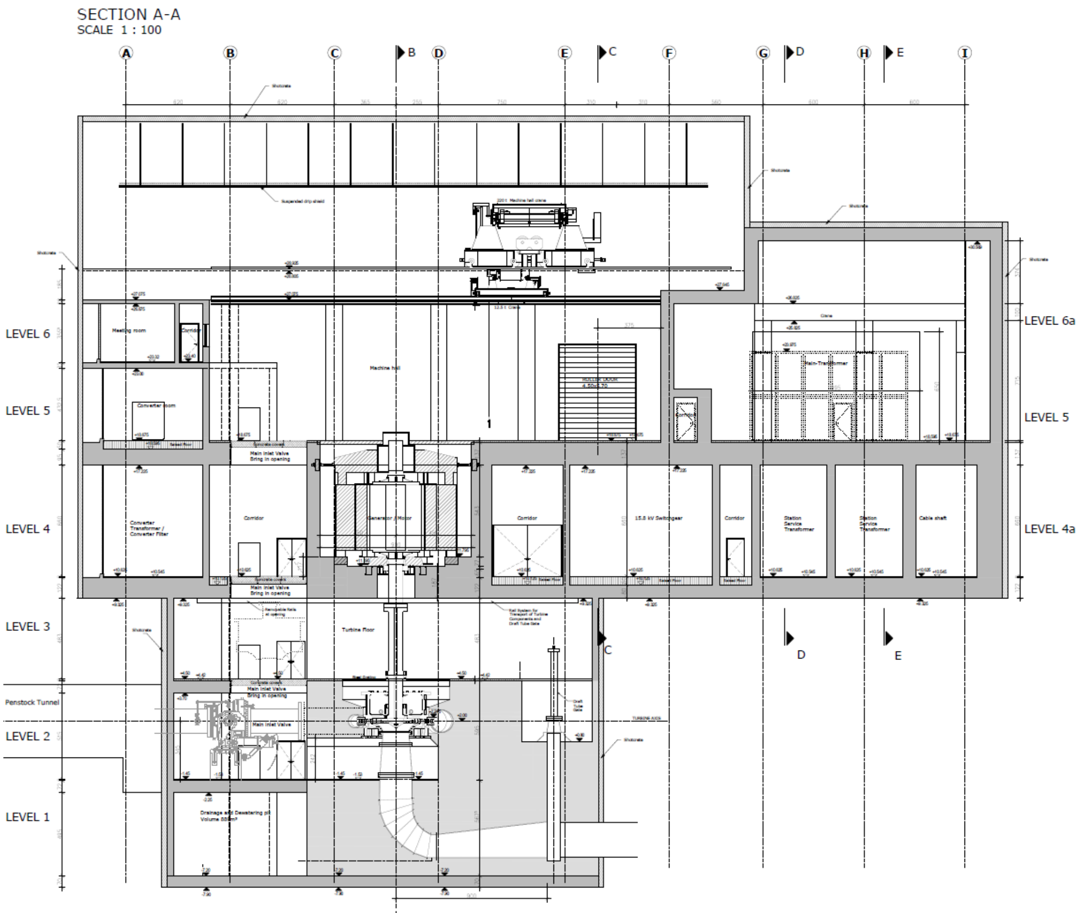

Figure 6-10: Powerhouse main cross section unit

The hydroelectric generating set consists of a Francis type single stage reversible pump turbine with a maximum power output of 209.1 MW in turbine mode operation. In total 1 unit with a vertical axis is installed.

A detailed description of the powerhouse E&M equipment is shown in the related report MANARABDGR01002. A list of the plant components is shown in the following pages.

At Level 1 (floor elevation -7.2 masl) following plant components are located:

- Draft tube including gate
- Drainage and dewatering sump pit
- Oil separator
- Sludge sump pit
- Sump pit in case of emergency

At Level 2 (floor elevation -1.5 masl) following plant components are located:

- Main inlet valve (spherical valve)
- Pumpturbine
- Gate control unit
- Bearing oil tank
- Transport corridor and storages

At Level 3 (floor elevation 4.5 masl) following plant components are located:

- Cooling water and filter station
- High and low pressure compressed air + storage
- Main cooling water + central oil coolers
- Mechanical workshop

At Level 4 (floor elevation 10.6 masl) following plant components are located:

- Generator
- Generator bus duct / Switchgear
- Converter Transformer / Converter Filter
- 15,6kV Switchgear
- Cable corridors
- Station service transformers
- Electrical workshop and storage
- HVAC
- Transformer oil collecting pit

At Level 5 (floor elevation 18.6 masl) following plant components are located:

- Machine hall
- Unit transformer
- 400 VAC Main Distribution
- 24 kV Switchgear
- Converter
- Control room
- Emergency base

- IEC Metering room

At Level 6 and 6a (floor elevation 23.3 masl and 24.7 masl) following plant components are located:

- Storage
- Locker room, Toilets
- Meeting room, Office, Archive
- Battery room
- DC and UPS distribution
- Water spray system at crown region

The different levels are connected with two stair cases and one elevator. For the equipment manipulation during erection and in maintenance case on the different levels several openings are designed. A description for the main elements manipulation (spiral case, MIV,…) is shown in the report MANARABDGR01002.

Excavation and support

The excavation of the cavern is performed using the drill and blast method. Preliminary calculations for the excavation and support have been executed. The results are shown in the report MANARABDCR05001 in detail.

The excavation support is also used as final lining of the cavern. The lining consists of shotcrete including wire mesh, lattice girder and anchors. The detailed anchor types and pattern are shown in the referenced drawings. The lining is not intended to serve as a waterproofing membrane. Systematic drainage boreholes are designed in the cavern walls and the crown. The accumulated mountain water is collected and conveyed to the drainage and dewatering sump pit. The arrangement of the drain holes shall be design in detail during detailed design stage, respectively construction according to prior agreement with the geologist. In order to reduce mountain water inflow deep-seated rock mass injections can be executed.

The crane beam of the cavern is fixed with 30-40 m pre-stressed anchors, to give an adequate safety of the beam, before the powerhouse structural concrete including the columns is constructed.

Structural system

The interior concrete work of the cavern consists in the lower section (up to elevation 4.5 masl) mainly of mass concrete, in which the unit is embedded. Above this elevation, a reinforced concrete sections are situated (up to elevation 18.60 masl), allowing the transfer of pump turbine and generator loads into the surrounding rock mass.

From elevation 18.60 masl further upwards, reinforced concrete columns and machine hall crane beams follow. The loads from the machine hall crane are transferred via crane beam, which are supported on columns, to the mass concrete below. Following the notification of the exact loads by the E&M group a detailed structural analysis of the whole structure must be carried out during detailed design stage.

In the crown of the machine hall (above the crane), a suspended ceiling is planned, to deal with dripping water from the cavern crown.

Drinking water supply

The drinking water supply for the power plant is planned via a connection to the drinking water network of the municipality of Kiryat Shmona. The water is piped from the drinking water network to the portal of the access tunnel and from there via the “Fresh air and escape corridor” (which is part of the access tunnel) to the underground powerhouse.

Treatment of water

For the water treatment in the cavern a separate system is designed.

Mountain water and non-contaminated process water (spiral case, dewatering of mechanical seal) are collected into the drainage and dewatering sump pit and thereafter pumped into the low pressure tunnel.

Contaminated water will be collected via floor drains in the individual rooms. Thereafter, the water is conveyed to the sludge sump pit and further to the oil separator and further to the drainage and dewatering sump pit. An alert will be sent to the operation room (in the control building at the switchyard area) in case of an oil spill causing a functional impairment of the oil separator due to overload.

Wastewater from toilets, wash basins and showers will be collected in a “mobile sewage tank” (at level 11.80 masl). This mobile sewage tank will be orderly emptied, cleaned and disposed.

6.6 Main access tunnels

6.6.1 Reference drawings

NUMBER	TITLE
MANARA/BD/CD/05/013	POWERHOUSE - ACCESS TUNNEL - LAYOUT
MANARA/BD/CD/05/014	POWERHOUSE - ACCESS TUNNEL - SECTION
MANARA/BD/CD/05/015	POWERHOUSE - ACCESS TUNNEL - SUPPORT
MANARA/BD/CD/05/016	POWERHOUSE - ACCESS TUNNEL - SUPPORT
MANARA/BD/CD/05/017	POWERHOUSE - ACCESS TUNNEL - SUPPORT
MANARA/BD/GG/05/001	POWERHOUSE - ACCESS TUNNEL - GEOLOGY

6.6.2 Reference reports

NUMBER	TITLE
MANARA/BD/GR/01/002	EM AND HM EQUIPMENT - DESIGN REPORT

6.6.3 Design description

The access tunnel is the main connection of the underground powerhouse to the surface. This main access tunnel (MAT) has branches to the flexible joint chamber access tunnel, the access tunnel to the surge tank and the mucking tunnel to the high pressure manifold.

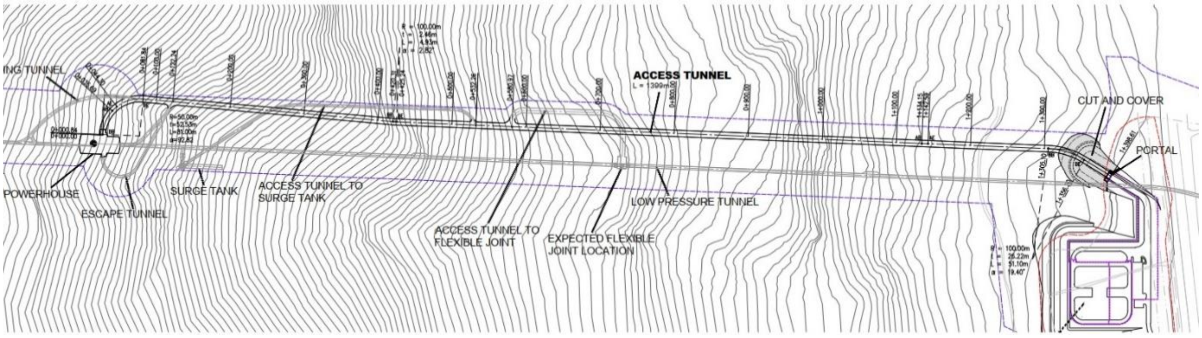

Figure 6-11: Layout of access tunnel

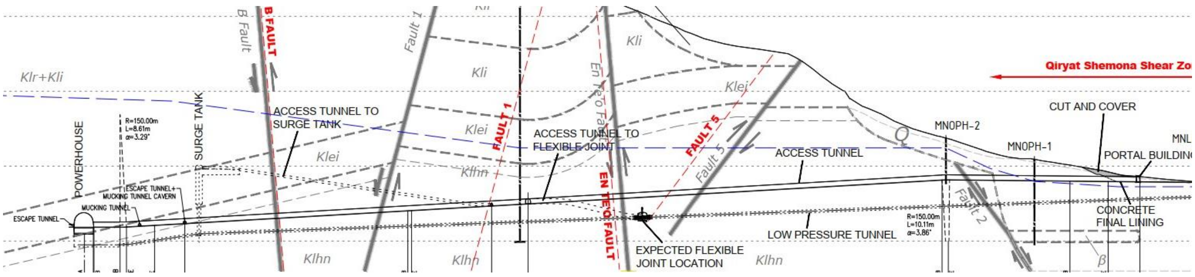

Figure 6-12: Longitudinal section of access tunnel

During the plant operation and maintenance, the main access tunnel will be dedicated to the following functions:

- Energy transmission towards and from the switchyard, with the installation of the power cables
- Fire exhaust smoke extraction in relation with the fire protection system
- Ventilation system of the underground powerhouse with fresh air income (via escape corridor) and exhaust air extraction
- Emergency exit (via escape corridor) if the access tunnel is no more longer available (e.g. a vehicle on fire)
- Dewatering pipe from powerhouse DN600

The cross section of the access tunnel is shown in the figure below.

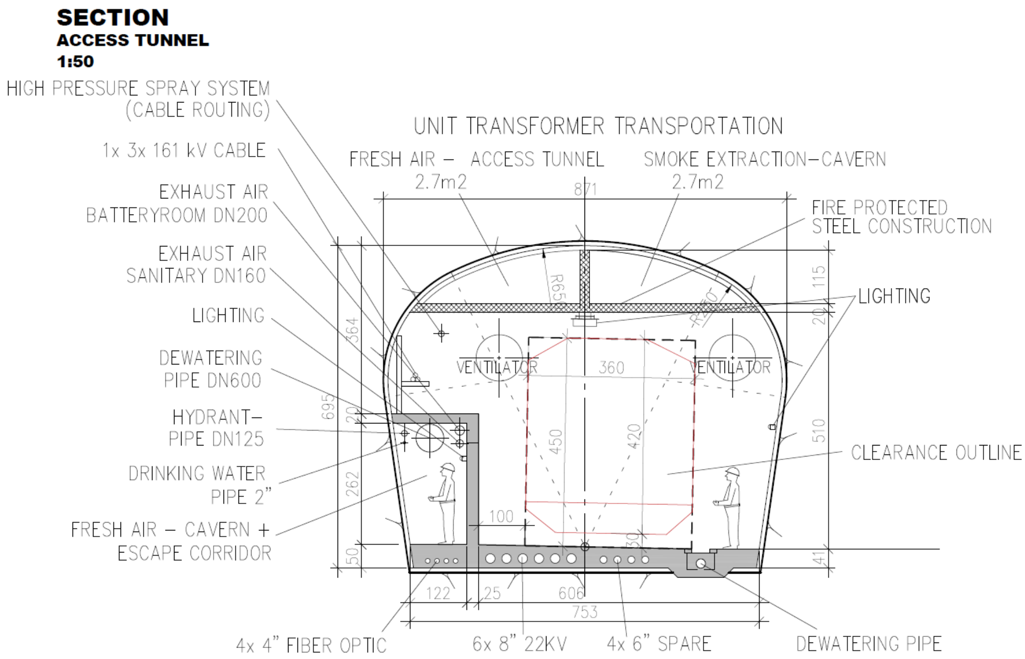

Figure 6-13: Cross section of the main access tunnel

The access tunnel is 1,398 m long with a slope inclination range between 0.0% and 5.75%. The access tunnel invert elevation is varying from elevation 18.60 masl to maximum 81.90 masl. The invert elevation at the tunnel portal is 79.3 masl.

The excavation cross section with the dimensions 8.50 m width and 6.85 m height complies with the delivery of all the equipment during the underground powerhouse construction and equipment erection (including rotor, transformers, spiral case,...).

The final clearance outline of the access tunnel complies with the delivery of all the equipment during the operation and maintenance of the underground powerhouse equipment (transformers,...).

The access tunnel is constructed using the drill and blast method and supported with systematic rock bolting, wire mesh and shotcrete. This support is also used as final lining. The excavation support classes can be seen in the related drawings. Four support classes are designed, whereat a estimation of the percentage for each class can be seen in the related geology drawing of the access tunnel.

The bottom slab (pavement) of the main access tunnel is constructed with concrete if wet conditions occur during excavation and in dry conditions a asphaltic pavement alternative can be constructed.

As the tunnel is situated below ground water level, water can enter into the tunnel via faults zones. The water coming to this faults has to be collected and diverted to the dewatering pipe DN 200 in the base of the access tunnel. The surface water of the access tunnel will be collected via channels / gully’s and also diverted into this dewatering pipe. The pipe leads into the drainage system of the cavern (level 1), where it is collected in pumped from the cavern to the lower reservoir via the dewatering pipe DN600.

A pre-cut section with an embankment inclination of 2:1 (horizontal: vertical) is designed to ensure the construction of the tunnel portal in the construction phase. The slope inclination of the pre-cut section and the first 255 m of the access tunnel are inclined upwards with 1 %, to prevent entering of surface water and water in the slope wash into the access tunnel.

The final portal building is situated some 49 m south east of the construction portal. It is required to stay within the “red” land boundary. The section between the final portal building and tunnel portal during construction is built with a concrete structure and covered afterwards with fill including a landscape rehabilitation between the end of the existing avocado orchard and the end of the cut and cover section.

To ensure a smooth construction sequence of the of the cavern and the power waterway the construction company can install/construct transportation bays in the main access tunnel if needed.

At the portal area of the MAT a portal building is situated. In this portal building the ventilation structure for the fresh air system of the cavern and the tunnel itself (two systems) is situated. The location of the inlet for the fresh air systems has to be placed in that way, that short circuit with the exhaust air and the smoke extraction is avoided.

6.7 Switchyard area

6.7.1 Reference drawings

NUMBER	TITLE
MANARA/BD/CD/07/001	SWITCHYARD AREA - LAYOUT
MANARA/BD/CD/07/002	GRID CONNECTION - CONTROL BUILDING
MANARA/BD/CD/07/003	GRID CONNECTION - CONTROL BUILDING
MANARA/BD/CD/07/004	GRID CONNECTION - CONTROL BUILDING
MANARA/BD/CD/07/005	GRID CONNECTION - CONTROL BUILDING

6.7.2 Reference reports

NUMBER	TITLE
MANARA/BD/GR/01/002	EM AND HM EQUIPMENT - DESIGN REPORT

6.7.3 Design description

The switchyard and the control building are situated south of the portal area of the main access tunnel on elevation 78.5 masl. At this area the following parts are situated:

- control building for power house
- 161 kV switchyard with operation building
- cable channel from the main access tunnel portal to the switchyard
- dewatering pipe DN600 from the main access tunnel portal to the lower reservoir
- permanent access road to the switchyard area and powerhouse
- gate and guard house for main access to the power plant
- parking

After completion of the works comprehensive landscaping will be done at the switchyard and portal to access tunnel in order to restore the initial condition of the lower part of the natural park reserve and minimize the construction impacts on this area.

6.7.4 Control building

The control building is a concrete superstructure and includes the main devices for the control of the power plant. The control building has three main levels as follows:

Level 1 (ground floor):

- Two control rooms
- Communication room
- Battery room
- Electrical room
- Electrical and Mechanical spare part storage
- Shelter
- First aid

- Showers and toilets
- Entry / Lobby
- Workshop

Level 2 (first floor)

- Archive
- Offices
- Shelter
- Meeting room
- Toilets
- Kitchen
- Storage

Level 3 (second floor)

- Common room
- Toilet

The access of the levels is via staircase and elevator.

6.8 Access roads

6.8.1 Reference drawings

NUMBER	TITLE
MANARA/BD/CD/06/001	ACCESS ROADS

6.8.2 Design description

The access to the project areas is ensured via existing roads.

Lower reservoir switchyard and access tunnel portal area:

The access to this area is via the highway 90.

Upper reservoir area:

The access to the upper reservoir is starting from highway 90 and the continuing either

- with road 899 and 886 (south access)
- or with road 9977 and 886 (north access)

6.9 Construction areas

6.9.1 Reference drawings

NUMBER	TITLE
MANARA/BD/CD/08/001	MOBILIZATION AREA

6.9.2 Design description

For the contractor two areas at the upper reservoir and two areas at the switchyard / main access tunnel portal area are available for the installation of the site equipment.

Upper reservoir:

At this area 41,950 m² are available for installing the site equipment. The space is separated in two parts, whereat one is placed in the north of the reservoir and one in the south.

Switchyard / Lower reservoir area:

At the switchyard area 8,270 m² are available for the site equipment and north of the tunnel portal additional 10,710 m² are available.

7 CONSTRUCTION AND ASSEMBLY WORK SCHEDULE

7.1 Reference drawings

NUMBER	TITLE
MANARA/TD/GD/01/003	CONSTRUCTION AND ASSEMBLY WORK SCHEDULE

7.2 Design description

Present it is planned to start the construction work after NTP to EPC contractor.

The first work parts will be the detail hydraulic design of the pump turbine and the according model test. To same in parallel, the site installation works of the construction company starts. After the site installation is placed, the construction works on the upper, the lower and the main access tunnel start parallel.

The commissioning and test procedure is the end of the work and is intended to start 48 months after NTP to EPC contractor. Therefore the construction time from starting the construction until the end of the commissioning is some four years.

The detailed schedule is shown in the referenced drawing MANARATDGD01003.

8 REFERENCES

8.1 Planning Fundamentals

[P1] Manara Pumpted Storage Site, Seismic Site Response Analysis, Geotechnical Consultants Ltd, August 2010

[P2] Geotechnical Baseline Report, Manara Pumped Storage Project, Geoprospect, G.G.S and EDF, September 2011

[P3] EDF Civil Design, Hydro Generation and Engineering, 2011

[P4] Survey Data, Autocad drawing “survey_ttl41.dwg”, 2015

[P5] Manara Pumped Storage Project – Basic Design Report, EDF, 2015

[P6] Pump Storage Powerplant Manara, Conceptual Design, Pöyry Energy Gmbh, 20.02.2015

8.2 Literature

[L1] DVWK 246/1997, 1997. Freibordbemessung an Stauanlagen, Merkblatt 246 (*Freeboard calculations for dams, Leaflet 246*).

[L2] ICOLD Bulletin 135, "Geo-membrane Sealing Systems for Dams-Design principles and review of experience"

[L3] Knauss, J. (1983). Wirbelbildung an Einlaufbauwerken (Luft- und Dralleintrag). DVWK Schriften. Bonn, Germany, Deutscher Verband für Wasserwirtschaft und Kulturbau e. V. (DVWK). Heft 63 [in German].

8.3 Standards and Guidelines

[G1] EUROCODES

[G2] Israeli Standard No. 414