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1 PURPOSE AND SCOPE

The purpose of the present document is to outline the design criteria to be applied in the basic design of the Luang Prabang Hydroelectric Power Project.

It is understood that the design criteria outlined in the present documents are based on the data and general requirements known in November 2019, and may be subject to adjustments as new information and updated requirements will become available. Such adjustments will be documented in future revisions of the present document.
2 GENERAL DESIGN CRITERIA

2.1 Introduction

This document contains the design concepts, assumptions, parameters, criteria and methods to be employed for the Basic Design for the civil structures of the Luang Prabang Hydroelectric Power Project (LP HPP).

All designs shall be performed according to agreed codes and standards, and sound engineering practice in a consistent way. The design guidelines presented in this document have been prepared to be in accordance with the American Standards and Codes of Practice, but have been supplemented to account for the experience of Pöyry in the design and construction of power plants and appurtenant structures. In addition, the requirements stipulated in the Lao Electric Power Technical Standard (LEPTS) are incorporated.

The present Design Criteria Report summarizes the input for the stability analysis and the structural calculations which are presented in separate reports. It is valid for the following structures:

- Navigation Lock (NL)
- Spillway (SP)
- Powerhouse (PH)
- Left Pier (LP)
- Right Pier (RP)
- RCC Closure Structure (RC)

Figure 2.1-1: Main Structures of Luang Prabang HPP
2.2 Description of the Project

The Luang Prabang HPP is located approx. 25 km upstream of Luang Prabang at kilometre 2036. It is a barrage type hydroelectric run-of-river scheme which comprises:

- Powerhouse equipped with 7 Kaplan turbine/generator sets (200 MW and 765 m³/s each). The total installed capacity for the main units is 1,400 MW with a discharge of 5,355 m³/s. The rated is about 29.56 m and the maximum gross head is 36.80 m
- Auxiliary Powerhouse equipped with 3 Kaplan turbine/generator sets (20 MW each), using water from fish attraction flow for the upstream and downstream migration facilities (approx. 180 m³/s), totaling to a maximum of 60 MW capacity, located in the Right Pier.
- Spillway structure with six (6) radial surface gates (19 m x 25 m, sill level 288.0 m). Three (3) low level outlets (12 m x 16 m, sill level 275.0 m asl)
- Two-step Navigation Lock system for 2x500 DWT vessels with a usable lock chamber length of 120.0 m and a width of 12.0 m
- Fish pass system for up- and downstream migration
- RCC Closing Structure formed by an approx. 50 m high RCC concrete gravity dam, in total 281 m long.
- 500 kV transmission line with intermediate substation to Vietnam with an approximate length of 400 km to the Vietnamese border and 200 km to the next suitable substation. Alternatively to Thailand with an approximate length 250 to 300 km.

2.3 Reservoir Water Level

The Luang Prabang HPP is a Run-of-River type hydropower plant, i.e. the discharge through the powerplant (Powerhouse, Spillway) equals the inflow. The Full Supply Level (FSL) at elevation 312.00 m asl will be maintained most of the time during operation (increase or decrease of the FSL might be required during Spillway operation or other exceptional operating cases). For practical purposes (control system) an “operating range” for the FSL of around 0.50 m will be required. The maximum operating water level at elevation 312.50 m asl will be used for the design of the civil structures and is called Design Full Supply Level (FSL_D).

Based on these rating curves for the Luang Prabang Spillway the resulting upstream water levels are 314.20 m asl for PMF and 313.60 m asl for the 10,000 year design flood (one Surface Spillway Gate not in operation).

It is not foreseen to lower the reservoir water level with the aim to reduce the reservoir water levels and the backwater effects in the reservoir during floods.

2.4 Tailrace Water Levels

2.4.1 Tailwater Rating Curve

The tailwater level rating curve is given in Figure 2.4-1. The corresponding numerical data are given in Table 2.4-1.
**Figure 2.4-1:** Tailwater Rating Curve for Luang Prabang HPP

**Table 2.4-1:** Tailwater Rating Curve for Luang Prabang HPP

<table>
<thead>
<tr>
<th>Discharge [m³/s]</th>
<th>Tailwater level [m asl]</th>
<th>Discharge [m³/s]</th>
<th>Tailwater level [m asl]</th>
</tr>
</thead>
<tbody>
<tr>
<td>516</td>
<td>274.77</td>
<td>17,000</td>
<td>291.95</td>
</tr>
<tr>
<td>1,000</td>
<td>276.31</td>
<td>18,800</td>
<td>292.98</td>
</tr>
<tr>
<td>2,000</td>
<td>278.59</td>
<td>20,600</td>
<td>293.89</td>
</tr>
<tr>
<td>3,000</td>
<td>280.18</td>
<td>21,700</td>
<td>294.42</td>
</tr>
<tr>
<td>3,213</td>
<td>280.43</td>
<td>22,900</td>
<td>294.97</td>
</tr>
<tr>
<td>4,000</td>
<td>281.34</td>
<td>24,400</td>
<td>295.59</td>
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<td>5,000</td>
<td>282.44</td>
<td>25,800</td>
<td>296.04</td>
</tr>
<tr>
<td>7,500</td>
<td>284.87</td>
<td>27,800</td>
<td>296.67</td>
</tr>
<tr>
<td>10,000</td>
<td>286.93</td>
<td>29,300</td>
<td>297.12</td>
</tr>
<tr>
<td>12,500</td>
<td>288.85</td>
<td>32,700</td>
<td>298.13</td>
</tr>
<tr>
<td>13,700</td>
<td>289.71</td>
<td>34,100</td>
<td>298.53</td>
</tr>
<tr>
<td>15,000</td>
<td>290.62</td>
<td>39,500</td>
<td>300.01</td>
</tr>
</tbody>
</table>
2.4.2 Probable Maximum Flood PMF

The Probable Maximum Flood (PMF) is estimated by applying a Probable Maximum Precipitation (PMP) storm as input for the hydrological model of the Mekong basin. The PMF discharge peak is 41,400 m³/s, resulting in a tailwater level of 300.50 m asl.

2.5 Hydrology and Meteorology

2.5.1 Flood Discharges

<table>
<thead>
<tr>
<th>Return interval</th>
<th>Q LP HPP [m³/s]</th>
<th>Return interval</th>
<th>Q LP HPP [m³/s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>12,800</td>
<td>200</td>
<td>25,300</td>
</tr>
<tr>
<td>5</td>
<td>16,200</td>
<td>500</td>
<td>27,300</td>
</tr>
<tr>
<td>10</td>
<td>18,200</td>
<td>1,000</td>
<td>28,800</td>
</tr>
<tr>
<td>20</td>
<td>20,000</td>
<td>5,000</td>
<td>32,100</td>
</tr>
<tr>
<td>50</td>
<td>22,200</td>
<td>10,000</td>
<td>33,500</td>
</tr>
<tr>
<td>100</td>
<td>23,800</td>
<td>PMF</td>
<td>41,400</td>
</tr>
</tbody>
</table>

2.5.2 Flow Duration Curve

![Flow Duration Curve for Luang Prabang HPP Site](image)

2.5.3 Lancang Cascade

To estimate the future inflow to Luang Prabang HPP the operation of the Lancang Cascade has to be taken into account. Besides the impact on the energy production, the flood peaks are slightly lower, while the dry season flows are significant higher than the natural inflow and well above 1,100 m³/s.
2.6 Construction Stages

The construction of the Luang Prabang HPP is done in two construction stages:

- In the first construction stage, the Mekong flows in its natural river bed. To limit the width of the river and to protect the construction pit area, an upstream and a downstream cofferdam as well as a concrete diversion wall at the Left Pier will be built.

  During the first construction stage, the Powerhouse, Spillway, Navigation Lock, Left Pier, Right Pier and the Fish Passing Facilities on the right bank will be built.

  The construction pit – during the main construction works – shall be safe for a 100-years return period flood HQ100.

- For the second construction stage, the Mekong river shall be diverted through the Low Level Outlets and the Surface Spillways that must be operational. To avoid excessive interruption of the navigation, the Navigation Lock should also be operational shortly after the river diversion.

  During the second construction stage, the RCC Closure Structure will be built. The requirements for flood safety may be reduced compared to the first construction stage.

2.7 Service Life

The service life for the Luang Prabang HPP main structures is 100 years.

2.8 Operational Aspects

2.8.1 Normal Operation

The Luang Prabang HPP is a Run-of-River type hydropower plant, i.e. the discharge through the powerplant (Powerhouse, Spillway) equals the inflow. For practical purposes (control system) an “operating range” for the Full Supply Level (FSL) of around 0.50 m will be required, i.e. the FSL will vary between 312.00 and 312.50 m asl. Reservoir lowering as preparation for floods is not foreseen.

2.8.2 Powerhouse

The Powerhouse is operated to keep the reservoir water level as explained in Chapter 2.8.1. The design discharge - also referred to as normal operation – of the Powerhouse with all 7 units in operation is 5,355 m$^3$/s and the median discharge that is exceeded during 50 percent of the year is approximately 2,100 m$^3$/s.

2.8.3 Spillway

The operation of the Spillway will be such that the first bays in operation will be the Low Level Outlets in order to route “turbidity currents” through the Spillway and to minimise sedimentation in the reservoir area. When the capacity of the Low Level Outlets is reached the Surface Spillway will start operation. All gates of the Surface Spillway will be equipped with flap gates to allow spill of floating debris in front of the Spillway into the tailwater area.
2.8.4 Navigation Lock

According to the MRC Guidance the Navigation Lock has to be operated between a 30 years flood and 95% flow duration of the river in natural conditions, leading to an operating range for flows in the Mekong River between 1,100 m$^3$/s and 21,700 m$^3$/s. The maximum head difference between the upstream “operating range” FSL of around 312.50 m asl and the minimum downstream water level 276.50 m asl is about 36.0 m.

During impounding the Navigation Lock needs to be operational even when the FSL is not reached; the minimum upstream water level for the operation of the Navigation Lock is 294.25 m asl (Lowest Operating Level, LOL). The minimum depth under the lowest Navigation Lock Level is 5.0 m.

The lockage time for a two-step ship lock is expected to be shorter than the required 50 minutes.

2.8.5 Fish Migration Facilities

According to the MRC Design Guidance fish passage facilities have to operate from minimum flows to a 1-year flood, leading to an operating range for flows in the Mekong River between 1’170 m$^3$/s and ~10,650 m$^3$/s, or tailwater levels between 276.7 m asl to 287.4 m asl.

Facilities for upstream and downstream migration are provided at the following structures:

- Upstream migration at the d/s face of the Powerhouse and the fish locks at the Left Pier
- Downstream migration at the u/s face of the Powerhouse and through the exit chute in the Right Pier
- Downstream migration at the Powerhouse through the fish friendly turbines
- Downstream migration through the Spillway (only wet season)
- Upstream migration at the right bank with the fish lock at the Navigation Lock and the upper channel (only wet season)

2.9 Project Classification

According to the Lao Electric Power Technical Standards, the water retaining structures of the project shall be classified as “Extreme”, because of the large storage capacity and the large potential risk with the city of Luang Prabang being relatively close to the Luang Prabang HPP site.

According to the definition of ICOLD, the Luang Prabang HPP is considered a large dam.

2.10 General Design Codes

The Lao Electric Power Technical Standards (LEPTS) shall be the primary general design code to be met.

In North American and many Western European countries, there are no codes or standards strictly regulating the design of hydropower projects. There are various sets of guidelines developed by different entities for their own use and adopted by others,
commonly with modifications to suit the Owner’s preferences or specific project requirements.

The most commonly referenced guidelines include those published by the following United States agencies:

- International Committee on Large Dams (ICOLD): Guidelines and recommendations.

In addition, other general design codes, design manuals and specifications to be considered will include as applicable:

- American Society of Civil Engineers (ASCE): Minimum design Loads for Buildings and Other Structures

2.11 Other References

- Luang Prabang HPP, Feasibility Study Report (Revision 0 dated on 10.05.2019)
- Luang Prabang HPP, Seismic Hazard Assessment Report (Revision 2 dated on 02.09.2019)
- Luang Prabang HPP, Geotechnical Report – Material parameters (revision 0 dated on 11 November 2019)

2.12 Units

All measures, dimensions, quantities and calculations shall be presented in the International System of Units (SI units). The coordinate grid system to be used in the Project shall be defined in accordance with the UTM WGS 84 coordinate system. Elevations shall be referred to the UTM WGS 84 datum.
3 HYDRAULIC DESIGN CRITERIA

3.1 Reservoir

3.1.1 Maximum Levels in Flood Conditions
The design of the Spillway shall assure that the following levels are not exceeded:

- For floods below the design flood of 33,500 m$^3$/s (10,000 years flood), the Spillway gates shall be operated in such a way that the water level does not exceed the Design Full Supply Level ($FSL_D$).
- For Mekong discharge exceeding the 10,000 year flood all the Spillway gates will be operated in fully opened position, even though the $FSL_D$ will be exceeded for PMF conditions (41,400 m$^3$/s). The resulting water level in the reservoir at the dam site shall not exceed the deck level of 317.0 m asl (no freeboard required, but no overtopping allowed). According to calculations carried out at FS stage, the resulting reservoir level will be around 314.20 m asl.

3.1.2 Minimum Operation Water Level

3.1.2.1 For Power Generation Purposes
The minimum normal reservoir water level for energy generation purposes is established at elevation 312.00 m asl.

3.1.2.2 For Navigation Purposes
In order to fulfil the availability requirements according to the MRC Design Guidance (availability of at least 98% of its scheduled operating time), the reservoir water levels as shown in Table 3.1-1 have been defined for the design of the Navigation Lock.

The minimum upstream water level, the Lowest Operating Level, is the minimum water level in the upper lock chamber, and defines the minimum water level that needs to be reached and maintained during impounding and construction of the closing structure in order to enable navigation.

Table 3.1-1: Reservoir Water Levels Defined for the Navigation Lock Design

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal Operating Level (NOL)</td>
<td>312.00 m asl</td>
</tr>
<tr>
<td>Highest Operating Level (HOL)</td>
<td>312.50 m asl</td>
</tr>
<tr>
<td>Lowest Operating Level (LOL)</td>
<td>294.25 m asl</td>
</tr>
</tbody>
</table>

3.1.2.3 For the Operation of the Fish Passing Facilities
The minimum reservoir water level for the operation of the fish passing facilities is the same as considered for the energy generation.
3.1.3 Freeboard on Water Retaining Structures

The freeboard shall be established according to the Lao Electric Power Technical Standards, Chapter 2 – Hydropower Civil Engineering Facilities, Article 19. The waves’ height shall be computed on the basis of the sustained wind velocity with 100 years return period.

3.2 River Handling Stages

3.2.1 Design Floods for the Cofferdams Operation

The design floods for the cofferdams during the two construction stages have been established balancing (i) estimated cofferdams cost, plus (ii) estimated monetary value of the damages that could occur if cofferdams are overtopped. Accordingly, the following design floods are defined:

- For the first stage construction works (main construction phase, including Powerhouse, Spillway and Navigation Lock): 100 years return period (about 23,800 m$^3$/s).
- For the second stage of the river diversion (construction of the RCC closing structure): 10 years flood (about 18,200 m$^3$/s). It is assumed that the RCC dam can be overtopped during the rainy season.

The Contractor shall not lower the design flood of the cofferdams below the values indicated above.

3.2.2 Limits for Navigation during Construction

In principle, navigation shall be maintained along the entire construction period, except for the month(s) when the second stage cofferdams across the river will be built.

For the first stage of the river diversion, velocities in the main river channel will not be significantly higher than in natural conditions. For more information refer to the hydraulic investigations. If required, tugs to assist boats in their crossing will be used in the construction area.

3.2.3 Freeboard on Cofferdams

Freeboard of cofferdams over the water levels corresponding to the design flood will be established as follow (reference to the Lao Electric Power Technical Standards, Chapter 2 – Hydropower Civil Engineering Facilities, Article 19):

- First stage embankment cofferdams: $F_r = h_w + 1.00$ m, with water level not higher than the crest of the impervious material.
- Second stage embankment cofferdams $F_r = h_w + 1.50$ m, with water level not higher than the crest of the impervious material.

With $h_w =$ waves height caused by a sustained wind velocity with 10 years return period (for the first stage cofferdams, $h_w$ will be most likely negligible).
3.3 Navigation Locks

3.3.1 Main Design Parameters

The main design parameters for the Navigation Lock are as outlined in the following table.

**Table 3.3-1: Main Design Parameters for the Navigation Lock**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of lock chambers</td>
<td>2</td>
</tr>
<tr>
<td>Design vessel</td>
<td>2 x 500 DWT (109 x 10.8 x 2.0 m)</td>
</tr>
<tr>
<td>Width of lock chambers</td>
<td>12 m</td>
</tr>
<tr>
<td>Lengths of chambers</td>
<td>2 x 120 m</td>
</tr>
<tr>
<td>Depth under lowest WL</td>
<td>4.0 m</td>
</tr>
<tr>
<td>Safety margin to sill-bottom</td>
<td>1.0 m</td>
</tr>
<tr>
<td>Max. passage time of a vessel</td>
<td>50 min</td>
</tr>
</tbody>
</table>

The Navigation Lock comprises two consecutive locks with three miter gates, an upstream, middle and downstream miter gate. The filling of the chambers of the Navigation Lock shall be done via a gravity based water feeding system from the headwater of the plant controlled by bonneted wheel gates.

3.3.2 Range of Operation

The navigation system shall be designed to operate within the water level limits as shown in Table 3.3-2.

**Table 3.3-2: Water Levels Defined for the Navigation Lock**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal Operating Level (NOL)</td>
<td>312.00 m asl</td>
</tr>
<tr>
<td>Highest Operating Level (HOL)</td>
<td>312.50 m asl</td>
</tr>
<tr>
<td>Lowest Operating Level (LOL)</td>
<td>294.25 m asl</td>
</tr>
<tr>
<td>Lowest Navigable Level (LNL)</td>
<td>276.50 m asl</td>
</tr>
<tr>
<td>Highest Navigable Level (HNL)</td>
<td>294.25 m asl</td>
</tr>
</tbody>
</table>

The water levels are defined for the upstream stretch (influenced by the hydropower plant) and downstream, free-flowing stretch. The minimum upstream water level, the Lowest Operating Level, is the minimum water level in the upper lock chamber, and defines the minimum water level that needs to be reached and maintained during impounding and construction of the closing structure in order to enable navigation.
The two locks shall be designed to divide in two equal parts the maximum head to be passed: maximum upstream water level 312.00 m asl, minimum downstream water level 276.50 m asl. Accordingly, the maximum lift per lock amounts to 17.75 m.

3.3.3 Minimum Water Depth
Maintaining 4 m minimum depth under the lower water levels including 1 m safety margin, the sill level of the navigation facilities shall be:

- Upstream approach channel: 298.25 m asl
- Upstream lock: 298.25 m asl
- Downstream lock: 271.50 m asl
- Downstream approach channel: 271.50 m asl

3.3.4 Freeboard
The upper lock is considered a water retaining structure. Thus the same freeboard requirements apply as outlined in Chapter 3.1.3.

The freeboard on the downstream part of the Navigation Lock shall be sufficient to avoid flooding of the structures during PMF conditions.

3.3.5 Locks Filling / Emptying Speed
The foreseen water feeding system is a filling and emptying system with a longitudinal culvert in the left side pier of the Navigation Lock, and extends from the upstream channel to the downstream approach channel. Along each lock chamber a total of seven (7) diffusers distribute the water transversely from the main water conduit along the bottom of the Navigation Lock, and the water enters via five (5) openings at each diffuser into the lock chamber.

A total of three (3) gates along the main conduit can regulate the water flow, the upstream service gate the flow from the intake to the upper chamber, the central service gate the flow between the upper and the lower chamber, and the downstream service gate the flow between the lower chamber and the outlet.

The lockage time for a two-step ship lock is expected to be in the range of 50 minutes corresponding to a benchmark outlined in the preliminary version of the MRC Guidance (2009). A breakdown of available moving times and raising/lowering times is listed:

- 10 minutes for moving from the approach channel to the first lock and securing the vessels.
- 10 minutes for balancing the water level of the two locks.
- 10 minutes for moving from the first lock to the second and securing the vessels
- 10 minutes for raising / lowering the second lock.
- 10 minutes for moving from the second lock to the approach channel

According to this breakdown about 10 minutes are allowed for the raising or lowering of the vessels in a chamber, which defines further the design and layout of the water feeding system. In order to achieve this filling and emptying times a maximum
discharge of the feeding system of around 105 m3/s is required for the given size of the lock chambers.

The design requirements for the main components of the water feeding system for the Navigation Lock are as follows:

- **Intake:** The intake shall be designed to allow for a uniform flow at all three intake bays, and the maximum velocity at the intake is limited to 2.0 m/s.

- **Main Water conduits:** The velocities within the water conduits shall be below 8.0 m/s in order to minimise losses and cavitation.

- **Diffusers and openings** shall be designed for a “uniform” flow distribution within the lock chamber for ensuring the vessel safety during lockage.

- **Gates** to regulate and control the filling and emptying of the lock chambers.

- **Outlet:** The outlet into the d/s approach channel shall be designed for a uniform flow at all bays.

### 3.3.6 Navigation Lock Equipment

Both chambers of the Navigation Lock shall be equipped with a ship arrester system and floating mooring bitts. The ship arrester cable system will be implemented to protect the miter gates against impacts and damages due to a wrong maneuver of a vessel entering the lock. Four cable systems shall be provided, downstream of the upstream gate, on both side of the middle gate, and upstream of the downstream gate. Additionally, regularly spaced floating mooring bitts shall be installed on both side walls on the Navigation Lock to secure the boats during the lock filling and emptying process. Ladders for evacuation purposes shall be foreseen as well.

### 3.3.7 Approach Channel

The lock approach channel is the transition of the navigable river and the lock (upstream and downstream of the hydropower plant). The approach channels needs to be separated from the main river course by a guide wall ensuring smooth hydraulic conditions. The approach shall provide a straight alignment of at least 250 m. The following criteria shall govern the design of the approach channel:

- Minimum curve radius of 300 m,
- Width allowance \( \Delta B \) in curve of 6.25 m,
- A straight stretch of at least 75 m before and after the curve,
- For a clear unobstructed view in a curve, over a length of 250 m the line of view has to be kept free from structures, bushes and other obstacles.

According to the MRC Guidance three different berthing areas should be provided, each able to berth one full-sized design vessel/convoy, and including:

- A lay-by area, where ships can be ready to immediately enter the Navigation Lock as soon as the preceding vessel has left it
- A waiting area, where ships can wait if there are several others ahead of them
- The overnight area, where ships and vessels can spend the night. The overnight area does not necessarily have to be connected to the lay-by or waiting areas, but all three areas should have access to land via a catwalk or floating pontoons.
• One of the above 3 areas may provide facilities such as potable water, clean water, solid and liquid waste facilities, emergency aid, etc.
• Mooring posts for both down- and upstream moving vessels.

3.3.8 Small Boats Transfer System
For the transfer of small vessels trailers for overland transfer shall be provided. The small boats shall be loaded onto a trailer at ramps located at the right bank, one downstream of the lower Navigation Lock chamber, and the other upstream of the upper Navigation Lock chamber.

3.4 Spillway

3.4.1 Design Floods
The Spillway design flood is the flood with a 10,000 years return period, and the safety check flood shall be a PMF flood event (see Table 2.5-1).

3.4.2 Design Floods for Crest Structure
The crest structure shall be designed to be safe against overtopping during floods including the freeboard requirements as defined in Chapter 3.1.1 and Chapter 3.1.3.

3.4.3 Design Floods for Stillling Basin
The stilling basin shall be designed to pass without damages at least the 50 years flood. The stilling basin shall be divided in two parts, prolonging the pier between the central gates, in order to allow inspection and maintenance works by closing half of the stilling basin with a low downstream embankment cofferdam in dry season.

3.5 Power Facilities

3.5.1 Velocity and Submergence at the Power Intakes
The gross velocity at the racks section shall not exceed 1.0 m/s. Bars spacing, centre to centre, shall not exceed 180 mm unless different requirements are established by the turbines manufacturer.

The submergence of the power intake shall be sufficient to avoid vortices under the minimum head water level established in the previous Chapter 3.1.2.1 (elevation 312.00).

There required submergence in order to avoid vortexes shall be checked by the hydraulic physical model tests.

3.5.2 Minimum Tailwater Level for Turbines Operation
The minimum tailwater level to be considered for turbine operation (setting of turbine submergence) shall be according to the flow duration curve as shown in Figure 2.5-1 and the tailwater rating curve shown in Figure 2.4-1.
3.5.3 Head Losses Calculation

Head losses calculations for the power waterways shall consider the head losses at the trash racks, assuming 10% obstruction, plus the head losses at the upstream stoplogs slots, at the upstream gate slots, and at the draft tube stoplogs slots.

3.5.4 Downstream Freeboard for Powerhouse

The freeboard of the upper deck of the Powerhouse shall not be less than computed according to the Lao Electric Power Technical Standards, Chapter 2 – Hydropower Civil Engineering Facilities, Article 19, considering the tailwater level under PMF conditions (about elevation 300.50 m asl without waves). The waves' height shall be computed on the basis of the sustained wind velocity with 100 years return period.

3.6 Fish Passing Facilities

3.6.1 General Considerations

3.6.1.1 Guidelines and Regulatory Framework

On the basis of the cross-national Agreement on the Cooperation for the Sustainable Development of the Mekong River (MRC, 1995), the MRC has developed preliminary guidelines for fish passing at dams on the Mekong Mainstream. Design criteria for project purpose shall be developed on the basis and in compliance with the above mentioned guidelines. Due to the fact that these guidelines are still rather general, the design criteria outlined in the following Sections will be subject to further clarification and verification. Because in our understanding the Developer is deemed responsible for demonstrating adequate mitigation measures, it is considered necessary that the Developer takes the initiative and search a continuous dialog with the GOL, the MRC and their expert groups.

3.6.1.2 Targets of the Facilities

The target of the fish passing facilities shall be to minimise as far as possible the impact of the project on the migrating habits of the different fish species present in the river stretch affected by the project. It shall be clearly stated that the intention is to provide facilities that will work, not just a minimum “pro-forma” structure, nominally satisfying the requirements, but which in reality will prove useless. As a guidance for the design of the fish passing facilities developed for the Luang Prabang HPP, the fish passing facilities at the Xayaburi has been used as a basis, which was successfully commissioned in May 2019, and which was further developed and optimized.

3.6.2 Design Criteria

3.6.2.1 General

The most relevant general criteria to be used for the design of the fish migration facilities are as follows:

- Fish passage facilities for both, upstream and downstream migration should be incorporated.
• The Navigation Lock could also be used as additional fish passage during migration periods.
• The fish migration facilities shall operate the whole year, optimally between the lowest flow and the 1-year flood.
• Design of the upstream and downstream fish passage for fish between 5 cm and 300 cm, as well as downstream drifting eggs and larvae during wet season.
• Fish migration system shall use a minimum of 10% of low flow ($Q_{95}$) and 1% of the 1-year flood; two aspects should be incorporated into the design, attraction (locating the entrances for fish approaching the entrances) and passage (through the fishways).
• Maximum water velocity for short distances (< 0.2 m) is 1.4 m/s, maximum velocity in channels is 0.5 m/s.
• Minimum depth is 3.0 m under all flows
• Use of fish friendly turbines.

3.6.2.2 Range of River Flows and Water Levels

Table 3.6-1 provides the relevant discharges in the Mekong River and tailwater levels which are relevant for the design of the fish migration facilities:

<table>
<thead>
<tr>
<th>Description</th>
<th>Discharge</th>
<th>Tailwater Level</th>
</tr>
</thead>
<tbody>
<tr>
<td>Minimum Flow</td>
<td>$1,170 \text{ m}^3/\text{s}$</td>
<td>276.70 m asl</td>
</tr>
<tr>
<td>1-year flood</td>
<td>$10,650 \text{ m}^3/\text{s}$</td>
<td>287.43 m asl</td>
</tr>
</tbody>
</table>

3.6.3 Fish Migration Concept

The fish migration concept provides the following upstream and downstream migration possibilities as shown in Figure 3.6-1.
During dry season (Spillway not in operation), upstream migrating fish will be attracted by the Powerhouse discharge and will enter into the upstream fish passing facilities developed over the entire length of the Powerhouse, with entrances at the Right Pier (towards the Spillway), multiple entrances along the downstream face of the Powerhouse, and at the Left Pier. The fish in the collection galleries are guided by attraction currents to two fish locks, where the fish are lifted up into the headwater of the reservoir.

When both the Powerhouse and the Spillway are in operation during wet season, fish migrating upstream will be also attracted into the Spillway discharge channel. Spill patterns inside the stilling basin will create a hydraulic barrier forcing the fishes to the side edges. A supplementary fish entrance on the left side of the Spillway at the Right Pier is providing a pathway for upstream migratory fishes guiding them to the fish passing facilities at the downstream side of the Powerhouse. Fish passing through an additional upstream fish migration system at the right bank shall be considered as a supportive means for attracting and passing fish at the right bank of the project during operation of the Spillway for upstream migration.

The main downstream fish migration system provides entrances along the entire upstream face of the Powerhouse and a collecting gallery, where the fish are guided to the Right Pier and released down to the tailwater through the terminal chute. Downstream migration is also possible through the Spillway (when in operation) and for smaller fish (smaller than the trash rack clear spacing) through the turbines of the Powerhouse. The Powerhouse is equipped with fish friendly turbines in order to reduce fish mortality during turbine passage.

### 3.6.3.1 Upstream Fish Passing Facilities

The main upstream fish passage facilities is incorporated in the Powerhouse and comprise the following components:
- Entrances above the turbine draft tubes and on the left and right side of the Powerhouse.
- A fish collecting gallery across the downstream side of the Powerhouse connecting all entrances.
- Two large fish locks including fish crowder and movable screen floor on the left bank pier.
- A feeding pond at the Right Pier inclusive feeding system, with feeders along the collecting gallery and at the back of the fish locks, to create fish attraction currents throughout the collecting gallery and through the various entrances of the facility.
- A fish monitoring station

**Entrances and fish collecting galleries**

Fish migrating upstream attracted by the flow through the Powerhouse can enter the upstream fish migration facility via one of the 2 x 7 entrances (2 m wide and 3 m high, sill at 273.5 m asl and 278.5 m asl), located at different levels above the turbine draft tubes along the entire length of the Powerhouse, and additionally via the two openings of the right side entrance (b: 2 m x h: 7 m ; sill at 272.5 m asl and 279.5 m asl) or via the three openings of the left side entrance (b: 6 m x h: 7 m (2x) and b: 6 m x h: 3 m (1x); sill at 272.5 m asl and 279.5 m asl). During the wet season, fish can also use the two openings of the Spillway entrance (b: 2 m x h: 7 m; sill 279.5 m asl), as well as on the Spillway tailrace side.

After entry into this system, fish heading up to the upstream area are successively passing through the collecting gallery and one of the two fish locks and are released into the headwater. The general arrangement of the upstream fish passage facilities are shown in Figure 3.6-2.

![Figure 3.6-2: General Arrangement of the Upstream Fish Migration Facilities](image-url)
Fish Locks
Two parallel fish locks connect the fish pass to the headwater. The operation cycle of the locks includes an attraction phase and a transfer phase. Both locks will operate alternately to maintain a continuous fish migration pathway.

During the attraction phase, flow is provided from the auxiliary Powerhouse at the back of one of the fish lock chamber, to attract fish into approach channel and into the lock chamber itself. Each lock is provided with a fish crowder in its approach channel. The fish crowder will act as a trap during the fish attraction phase, forming an inscale with its two leaves. The two leaves of the inscale will close at the end of the attraction period and the fish crowder will force the fish to enter into the lock chamber. The transfer phase will continue by closing the fish lock entrance gate and slowly filling the lock. A movable screen floor in the lock chamber will force the fish to move up with the raising water level. Once the water level in the lock chamber is equalized with the reservoir level, an exit gate will be open to release the fish into the headwater of the reservoir.

Feeding System
The feeding system provides fish attraction currents throughout the collecting gallery and through the various entrances of the facility, to guide the fish from the river to the fish locks.

The water for the feeding system operation is provided by the auxiliary units during the dry season (TWL < 283 m asl). Additional water is supplemented by a gravity water supply gallery during the wet season (TWL > 283 m asl). This water is collected in the feeding pond, in the Right Pier, and is sourced from the following:

- The water from the downstream migration, provided via two auxiliary units, between 80 and 120 m³/s,
- Water from the third auxiliary unit taken from the headwater, 60 m³/s,
- Via the gravity water supply (bypass) system from the headwater, up to 45 m³/s.

From the Right Pier feeding pond a total of four feeding galleries provide the water for the attraction flow at the collection galleries and the various entrances:

- The feeding galleries 1 and 2 feed the right and the left part of the Powerhouse fish collection galleries.
- Feeding gallery 3 feeds the entrances at the Left Pier.
- Feeding gallery 4 supplies the feeders located at the back of the two fish locks.

Hydraulic Design
The flow required for the operation of the upstream fish passage facility (attraction flow) varies with the tailwater level and ranges approx. from 80 to 180 m³/s. The required flow depending on the TWL and corresponding water level in the feeding pond (due to the head losses in the feeding galleries) are given in the following Figure 3.6-3.
The flow velocities through upstream collection gallery entrances ranges from 1.0 m/s on the left side of the Powerhouse to 0.85 m/s on the right side of the Powerhouse. The velocity in the fish collecting gallery ranges from 0.2 m/s to 0.5 m/s, and depend on the tailwater level and the location in the gallery.

**Fish monitoring station**

For the upstream fish migration system a fish monitoring station is foreseen at the fish lock exit. The purpose of this fish monitoring station is as follows:

- **For continuous fish monitoring** a hydro-acoustic fish survey system is foreseen. However, the exact type and technology for the fish counting will be decided at a later stage in order to install a modern and state-of-the-art technology.

- **Fish viewing and counting facilities** should be arranged at a suitable point at the outfall structure and may include a viewing facility with windows, video cameras and a light panel. Fish viewing and counting is accomplished by guiding the fish into an area where they are sufficiently visible to be identified and counted.

- **Fish trapping and viewing facilities** can be arranged laterally to the upstream outfall structure and may include holding boxes and traps equipped with slide gates to either retain fish or return them to the ladder. Similar to the fish counting and viewing system, there are many different systems which may be used for fish trapping and sorting, depending on the monitoring requirements. Fish trapping and sorting facilities may be used for brood stock collection, fish tagging or other research activities.

### 3.6.3.2 Right Bank Fish Passing Facility (Upstream Migration)

Fishes migrating upstream generally are guided by the current, preferentially keeping near the banks of the river. During the wet season when the Spillway is in operation, fish will get attracted by the Spillway discharge. For fishes at the right river bank, a possibility for upstream migration will provided near the Navigation Lock.

Thus, facilities for upstream migration are foreseen at the Navigation Lock comprising the following components:
• A Fish Lock downstream of the stilling basin next to the Navigation Lock.
• A culvert crossing the downstream approach channel, providing sufficient clearance for the moving ships and vessels at the downstream approach channel.
• An open channel at the right bank leading to the upstream approach channel.

The fish entering the fish lock will be lifted up to the headwater level, and will be released into the u/s culvert and channel leading to the upstream approach channel of the Navigation Lock. The flow in the approach channel of about 5 m$^3$/s will be used as attraction flow at the entrance of the fish lock.

An additional entrance inside the approach channel of the Navigation Lock connected to the Fish Lock which provides an additional upstream migration possibility. This entrance will be in operation during the main fish migration period (April, May).

### 3.6.4 Downstream Fish Passing Facilities

The downstream fish passage facility of the Powerhouse comprises a collecting gallery above the Powerhouse intakes and a downstream stepped chute (exit chute) to the tailrace channel.

Additionally, fish-friendly turbine technologies are implemented to mitigate lethal injuries during turbine passage for smaller fish that will pass through the trash rack of the Powerhouse intakes.

Finally, downstream migration through the Spillway may occur as well when Spillway is opened for spillage flows.

Figure 3.6-4 shows the general arrangement of the downstream migration system at the Powerhouse.

![Figure 3.6-4: Downstream Migration System](image)
Main downstream migration facilities

Downstream migrating fishes are to be collected at a fish collecting gallery at the upstream side of the Powerhouse leading to the Right Pier. The 14 entrances to this gallery (2.5 m wide and 3.0 m high, sill at 302.0 m asl) are located above the inlet structures, and are provided over the entire length of the Powerhouse. At the Right Pier a downstream chute will guide the fish into the tailwater area.

The total flow through the downstream migration facility ranges from 100 to 140 m$^3$/s and the corresponding velocity through the entrances of the collecting gallery ranges from 1.0 m/s to 1.4 m/s.

A constant discharge of about 20 m$^3$/s is used for the continuous operation of the downstream chute. The exceeding water (80 to 120 m$^3$/s) will be diverted and used for attraction flow for the upstream migration system (released through two auxiliary units into the Right Pier feeding pond). The downstream migration facilities need to operate during the entire year.

The flow pattern in this collecting gallery and at the flow splitting structure (to the two auxiliary units) will be checked during the Detailed Design stage with a 3D-CFD modelling. If required, the geometry of the intakes of the auxiliary units as well as the geometry of the fish exclusion screen (angle bar rack) to be installed at the intake are will be adapted.

The exit chute has been designed to minimize energy levels in the pools (< ~500 W/m$^3$) while ensuring a sufficient hydraulic profile at each weir between the pools (about 2 m x 2 m), and for maximum head drop of 3 m from pool to pool. The flow in the exit chute is controlled by a combination of a flap and a sluice gate. A physical model test will be carried out to confirm the proposed geometry.

Fish Friendly Turbines

The main turbines for the Luang Prabang HPP will be provided with several fish-friendly features, which are directly related to mitigate lethal injuries at fishes passing the turbines downstream, such as:

- Reduction of the runner blades by turbine and reduction of rotational speed of the turbine in order to mitigate turbine induced mortality by blade strikes, one of the most relevant causes of mortality during turbine passage.
- A fish friendly guide vane design can further minimize turbulence and shear stress regions, thus improving the fish survival rate.
- High precision in manufacturing the periphery turbine components allows to reduce gaps and thus reduce water flowing through the gaps leading the subsequent turbulences.
- Fish friendly Kaplan turbines have oil free hubs compared. The servomotor is located inside the runner hub with the surrounding space filled with water and additives for corrosion protection. Such a design is a major positive impact on the water quality and thus on the environment compared to the conventional design.
3.6.5 Daily Hours of Operation

It has to be assumed that fish in the Mekong Mainstream migrate at different times of the day. Therefore, the fish passing facilities should operate on a continuous basis.

3.6.6 Criteria for Operation Control

The control system shall permit the fish passing facilities to be monitored and operated in accordance with the respective protocol to be established. It shall be compatible with the other systems, and designed for reliability, fault tolerance, and stability. Furthermore, it shall be serviceable.

Regarding the hardware equipment and software incorporated into the control system, it must be compatible with the main system, and shall have a proven track record for reliability and dependability.

The control system shall consist of the electro-mechanical, electrical and electronic components used to monitor a structure, and to initiate gate and valve movements. The control system should not be designed in isolation from the structural/mechanical components of the project. Typical components of the control system may include:

- Actuators for controlling the movements of gates and valves.
- Sensors, indicators and displays for monitoring water levels, gate and valve positions, flow rates, electrical power status, alarm conditions, and other structure related parameters.
- Programmable logic controllers (PLCs) connected to the actuators, sensors, indicators, and displays. The PLCs implement and enforce the operating rules for the gates and valves. The PLCs also collect and process information from sensors and indicators, and supply data to the display units.
- Communication hardware to connect the PLCs and display terminals, and for relaying information to the central monitoring facilities.
- Human-Machine Interface (HMI) software to display information collected by the PLCs. The HMI is a graphical presentation of real time data, and allows an operator to control and monitor the structure remotely.
- Uninterruptible Power Supply (UPS) for providing backup power for the PLC.

As part of the initial detailed design process, it is important that a process and identification diagram (P&ID) is prepared prior to proceeding with the detailed design of the control system. The design must also include appropriate lightning and surge protection measures.
4 GEOTECHNICAL DESIGN CRITERIA

4.1 Basis
The project area as well as the foundation of Luang Prabang HPP civil structures is governed by two main bedrock types in the sub-stratum. Volcanic bedrock types (Volcanic-Basaltic Andesite) alternate with thinly to medium banked calcareous sediments (Sediment – Siltstone & Shale). The strength of the main bedrock types varies in general between acceptable/fair to favourably high.

The design criteria and the corresponding design for the slope excavation as well as for dam embankments are not part of this report.

4.2 Symbols
The following symbols are used to describe the foundation properties:

- \( c \) Cohesion
- \( D \) Disturbance factor
- \( k_f \) Coefficient of permeability
- \( k_s \) Modulus of subgrade
- \( n \) Porosity
- \( \sigma_1 \) Major principal stress
- \( \phi_d \) Residual friction angle (dynamic friction angle)
- \( \phi_s \) Peak friction angle (static friction angle)

4.3 Codes and Standards
The design of all geotechnical works shall be in accordance with recognized standards, and with codes and standards referenced herein:

- United State Army Corps of Engineers (USACE)
  - EM 1110-1-2908: Rock Foundations
  - EM 1110-2-3506: Grouting Technology
  - EM 1110-2-1902: Slope Stability
  - EM 1110-2-2006: Roller-Compacted Concrete
  - EM 1110-2-2200: Gravity Dam Design
  - EM 1110-2-2300: General Design and Construction Considerations for Earth and Rock-Fill Dams
  - EM 1110-2-2502: Retaining and Flood Walls
  - EM 1110-2-2503: Design of Sheet Pile Cellular Structures Coffer Dams and Retaining Structures
- Federal Energy Regulatory Commission (FERC):
  - Engineering Guidelines for Evaluation of Hydroelectric Projects
• Federal Emergency Management Agency (FEMA):
  – Federal Guidelines for Dam Safety: Earthquake Analyses and Design of Dams
• International Commission on Large Dams (ICOLD):
  – The Specification and Quality Control of Concrete for Dams
  – A Review of Earthquake Resistant Design of Dams
  – Finite Element Methods in Analysis and Design of Dams
  – Seismicity and Dam Design
  – Earthquake Analysis Procedures for Dams
  – Static Analysis of Embankment Dams
  – Selecting Seismic Parameters for Large Dams

4.4 Geology

The strength of the main bedrock types varies in general between acceptable/fair to favorably high. For simplification, the various geological rock masses have been condensed into two main groups.

  - Volcanic – Basaltic Andesite (green in Figure 4.4-1): part of PH, RP, SP, NL
  - Sediment – Siltstone & Shale (brown in Figure 4.4-1): RC, LP, part of PH

![Figure 4.4-1: Geological section of Luang Prabang HPP](image)

4.5 Shear Strength Parameters of Rock Foundation

For the two main geological members, material parameters (Hoek-Brown and Mohr-Coulomb) have been determined. Mohr-Coulomb parameters in a stress range of 250 kPa to 2000 kPa major principal stress (vertical stresses or load of structure) have been derived from Hoek-Brown parameter sets. Two different disturbance factors (D) are assumed for the excavated rock surfaces. The D factor for Volcanic-Basaltic Andesite was set to 0.8 whereas a D factor of 0.5 for Sediment – Siltstone & Shale was applied. The variation in D respects the higher effort for breaking and ripping the hard rock (Volcanic-Basaltic Andesite). On the basis of project experience and academic literature, the maximum peak friction angle for calculations (\(\phi_s\)) was set to 50°, which is comparable to the friction angle between concrete and clean rock surface.
4.5.1 Peak Shear Strength and Sliding Friction Shear Strength

Table 4.5-2 to Table 4.5-5 show the Mohr-Coulomb parameters for the main structures in respect to the assumed stress level in the rock/concrete interface. The values consider failures through the rock and through the concrete-rock interface.

For this project the required factors of safety, as a result of a stability analysis, are defined in the USACE and in the LEPTS. For the calculation of safety factors, for usual and unusual load cases against sliding, two different assumptions are taken into account. The resistance against sliding, respectively reached F.o.S. are determined for the LEPTS load cases considering the friction angle and the cohesion, whereas for the USACE the cohesion has been neglected.

For the definition of the static friction angle for the RCC Closure Structure, it has been assumed that the last excavation layer will be excavated with special care (smooth blasting/protective layer).

Table 4.5-2: Shear strength parameters for sliding stability (Powerhouse)

<table>
<thead>
<tr>
<th>Rock type in contact zone</th>
<th>c [MPa]</th>
<th>$\Phi_s$ [$^\circ$]</th>
<th>$\Phi_d$ [$^\circ$]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Volcanic - Basaltic Andesite (D = 0.8)</td>
<td>0.14</td>
<td>63.6</td>
<td>30*</td>
</tr>
<tr>
<td>Sediment - Siltstone &amp; Shale (D = 0.5)</td>
<td>0.04</td>
<td>50.2</td>
<td>25*</td>
</tr>
</tbody>
</table>

* assumption c=0 for $\tau(\phi_d)$ residual, $^1\sigma_1$=750 kPa; $^2$ average value

Table 4.5-3: Shear strength parameters for sliding stability (Spillway)

<table>
<thead>
<tr>
<th>Rock type in contact zone</th>
<th>c [MPa]</th>
<th>$\Phi_s$ [$^\circ$]</th>
<th>$\Phi_d$ [$^\circ$]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Volcanic - Basaltic Andesite</td>
<td>0.17</td>
<td>50</td>
<td>30*</td>
</tr>
</tbody>
</table>

* assumption c=0 for $\tau(\phi_d)$ residual, $^1\sigma_1$=550 kPa; $^2$ average value

* maximum peak friction angle for calculations ($\phi_s$) was set to 50°
Table 4.5-4: Shear strength parameters for sliding stability (Navigation Lock)

<table>
<thead>
<tr>
<th>Rock type in contact zone</th>
<th>c [MPa]</th>
<th>$\phi_s$ [°]</th>
<th>$\phi_a$ [°]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Volcanic – Basaltic Andesite</td>
<td>0.19</td>
<td>50</td>
<td>30*</td>
</tr>
</tbody>
</table>

* assumption $c=0$ for $\tau(\phi_d)$ residual, $^1\sigma_1=700$ kPa; $^2$ average value

Table 4.5-5: Shear strength parameters for sliding stability (RCC Closure Structure)

<table>
<thead>
<tr>
<th>Rock type in contact zone</th>
<th>c [MPa]</th>
<th>$\phi_s$ [°]</th>
<th>$\phi_a$ [°]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sediment – Siltstone &amp; Shale</td>
<td>0.08</td>
<td>46**</td>
<td>25*</td>
</tr>
</tbody>
</table>

* assumption $c=0$ for $\tau(\phi_d)$ residual, $^1\sigma_1=830$ kPa; $^2$ average value

** special care for the last excavation layer

Table 4.5-6: Shear strength parameters for sliding stability (Right Pier)

<table>
<thead>
<tr>
<th>Rock type in contact zone</th>
<th>c [MPa]</th>
<th>$\phi_s$ [°]</th>
<th>$\phi_a$ [°]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Volcanic – Basaltic Andesite</td>
<td>0.19</td>
<td>50</td>
<td>30*</td>
</tr>
</tbody>
</table>

* assumption $c=0$ for $\tau(\phi_d)$ residual, $^1\sigma_1=750$ kPa; $^2$ average value

Table 4.5-7: Shear strength parameters for sliding stability (Left Pier)

<table>
<thead>
<tr>
<th>Rock type in contact zone</th>
<th>c [MPa]</th>
<th>$\phi_s$ [°]</th>
<th>$\phi_a$ [°]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sediment – Siltstone &amp; Shale</td>
<td>0.08</td>
<td>42</td>
<td>25*</td>
</tr>
</tbody>
</table>

* assumption $c=0$ for $\tau(\phi_d)$ residual, $^1\sigma_1=750$ kPa; $^2$ average value

4.6 Bearing Capacity

The bearing capacity for the main structures (Powerhouse, Spillway, Navigation Lock and RCC Closure Structure) in respect to the geological conditions as well as estimated stress range in the foundation have been calculated according to EM 1110-1-2908.

The below indicated values for usual and extreme loading conditions already consider the safety factors as indicated in Table 6.4-3.
### Table 4.6-1: Bearing capacities for Powerhouse (σ1=750 kPa)

<table>
<thead>
<tr>
<th>Powerhouse Load condition</th>
<th>Allowable Bearing capacity</th>
<th>Allowable Bearing capacity</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Sediment Siltstone &amp; Shale [MPa]</td>
<td>Volcanic-Basaltic Andesite [MPa]</td>
</tr>
<tr>
<td>usual</td>
<td>9.0</td>
<td>28.3</td>
</tr>
<tr>
<td>unusual</td>
<td>10.4</td>
<td>32.5</td>
</tr>
<tr>
<td>extreme</td>
<td>13.5</td>
<td>42.4</td>
</tr>
</tbody>
</table>

### Table 4.6-2: Bearing capacities for Spillway (σ1=550 kPa)

<table>
<thead>
<tr>
<th>Spillway Load condition</th>
<th>Allowable Bearing capacity Volcanic-Basaltic Andesite [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>usual</td>
<td>30.8</td>
</tr>
<tr>
<td>unusual</td>
<td>35.4</td>
</tr>
<tr>
<td>extreme</td>
<td>46.1</td>
</tr>
</tbody>
</table>

### Table 4.6-3: Bearing capacities for Navigation Lock (σ1=700 kPa)

<table>
<thead>
<tr>
<th>Navigation Lock Load condition</th>
<th>Allowable Bearing capacity Volcanic-Basaltic Andesite [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>usual</td>
<td>17.8</td>
</tr>
<tr>
<td>unusual</td>
<td>20.5</td>
</tr>
<tr>
<td>extreme</td>
<td>26.8</td>
</tr>
</tbody>
</table>

### Table 4.6-4: Bearing capacities for RCC Closure Structure (σ1≈830 kPa)

<table>
<thead>
<tr>
<th>RCC Closure Structure Load condition</th>
<th>Allowable Bearing capacity Sediment Siltstone &amp; Shale [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>usual</td>
<td>5.2</td>
</tr>
<tr>
<td>unusual</td>
<td>6.0</td>
</tr>
<tr>
<td>extreme</td>
<td>7.9</td>
</tr>
</tbody>
</table>
Table 4.6-5: Bearing capacities for Right Pier ($\sigma_1 \approx 750$ kPa)

<table>
<thead>
<tr>
<th>Right Pier Load condition</th>
<th>Allowable Bearing capacity Volcanic-Basaltic Andesite [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>usual</td>
<td>30.5</td>
</tr>
<tr>
<td>unusual</td>
<td>35.1</td>
</tr>
<tr>
<td>extreme</td>
<td>45.7</td>
</tr>
</tbody>
</table>

Table 4.6-6: Bearing capacities for Left Pier ($\sigma_1 \approx 750$ kPa)

<table>
<thead>
<tr>
<th>Left Pier Load condition</th>
<th>Allowable Bearing capacity Sediment Siltstone &amp; Shale [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>usual</td>
<td>7.2</td>
</tr>
<tr>
<td>unusual</td>
<td>8.3</td>
</tr>
<tr>
<td>extreme</td>
<td>10.8</td>
</tr>
</tbody>
</table>

As the bearing capacities of the rock at the location of Luang Prabang HPP are generally very high, the levelling concrete bearing pressure becomes governing. Therefore, the maximum allowable bearing pressure for the civil structures is set as follows:

$$ f'_c = F \cdot f_c = 0.65 \cdot 25 \text{ MPa} = 16.25 \text{ MPa} $$

$f'_c$: Compressive Strength of Levelling Concrete

$F$: Strength reduction factor as defined in ACI 318 table 21.2.1

4.7 Modulus of Subgrade

Table 4.7-1 shows the determined modulus of subgrade in respect to various structures and related rock members. For calculation purpose, the modulus of subgrade, in the outer 10% of the foundation-structure interface can be increased to $2 \times k_s$.

Table 4.7-1: Modulus of subgrade

<table>
<thead>
<tr>
<th>Foundation</th>
<th>Structure</th>
<th>$k_s$ [MN/m$^3$]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Andesite</td>
<td>Powerhouse, Spillway</td>
<td>800</td>
</tr>
<tr>
<td>Andesite</td>
<td>Navigation Lock</td>
<td>1,000</td>
</tr>
<tr>
<td>Sediment</td>
<td>Powerhouse, RCC Closure Structure</td>
<td>400</td>
</tr>
</tbody>
</table>
4.8 Hydraulic Parameters of Rock Masses

<table>
<thead>
<tr>
<th>Material</th>
<th>$k_f \text{ [m/s]}^1$</th>
<th>n [%]</th>
<th>$K_y/K_x$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Volcanic-Basaltic Andesite</td>
<td>$10^{-6}$</td>
<td>1.5</td>
<td>1</td>
</tr>
<tr>
<td>Sediment – Siltstone &amp; Shale</td>
<td>$10^{-5}$</td>
<td>1.6</td>
<td>1</td>
</tr>
</tbody>
</table>

1 saturated

4.9 Criteria for Seepage Control

There are no established criteria for the allowable seepage amount for a run-of-river project. In particular, criteria possibly applicable to storage projects, establishing the maximum seepage during dry season as a given percentage of the reservoir capacity, would have no meaning for a run of river project on a large river like the Mekong. In general the only important aspect to be considered is to avoid internal erosion (e.g. piping) though the weakest zones of the foundation. Till date no critical weak foundation zones have been detected.

4.9.1 Seepage Gradient

Hydraulic gradients in the foundation shall be controlled to avoid erosion phenomenon of possible weak foundation zones (e.g. faults filled with fines etc.). In further project stages and based on improved geotechnical knowledge, critical seepage gradients shall be determined in respect to material characteristics and effective stress conditions.

Given the characteristics of the structures, all rather long in the upstream – downstream direction, and the low head, there should be no special difficulties in meeting the in further design stages defined requirements (e.g. hydraulic gradients etc.) with provided foundation treatments.

4.9.2 Maximum Allowable Seepage

In terms of total annual energy production loss, the seepage into the drainage gallery is negligible. The allowable maximum seepage however impacts the drainage pump capacity and the energy consumption of the drainage pumps. The allowable seepage rate shall be kept low with a reasonable effort, but will be specified in a further design phase.

4.10 Foundation Treatment

The treatment of the foundation generally consists of excavation, consolidation grouting, curtain grouting and drainage.

4.10.1 Excavation

A separate report deals with the stability of the excavation and the stabilization of the excavation.

The main structures require excavation to fresh rock levels. The foundation properties relevant for the design of the civil structures are summarized above. Further information is given in the geotechnical design report.
4.10.2 Consolidation Grouting

Requirements for consolidation grouting of the foundation shall be defined by geologists during the detailed design and/or during the execution. Till date no or only minimal consolidation grouting is expected, considering adequate foundation preparation before placing concrete. The probability that consolidation grouting is required for the RCC Closure Structure is higher than for the other structures given that the structure is founded on the weaker sediment- siltstone & shale formation and that a higher friction angle due to smooth blasting / protective layer at the foundation has been assumed.

4.10.3 Curtain Grouting and Foundation Drainage

Foundation drains are normally provided to reduce uplift, while the purpose of curtain grouting is to form a deeper and narrower zone of low permeability, to lower the seepage gradients and thus the seepage flow. The grout curtain shall also reduce the erosion risk of weak zones.

Depending on the seepage gradient, the requirements for the curtain grouting shall be defined by the geologist during the detailed design and/or during the execution. Till date the grout curtain depth is assumed equal to 2/3 of the water head.

Depending on the stability analysis requirements, i.e. the application of uplift pressure reduction, the requirements for the drainage holes shall be defined by the geologist during the detailed design and/or during the execution. Till date the drainage holes are assumed to be not less than half of the grout curtain depth.

The assumed alignment of grout curtain and drainage holes is given in Figure 6.8-1.
5 SEISMIC DESIGN CRITERIA

5.1 Basis
The basis for the seismic design is the Luang Prabang Seismic Hazard Assessment revision 2, a site specific seismic hazard analysis, dated on September 2019. The study includes a probabilistic (PSHA) and a deterministic (DSHA) seismic hazard analysis. The results i.e. the peak ground accelerations (PGA) are given in Chapter 6.8.6.8.

Luang Prabang is located in a region with moderate to high seismicity, in an area that lacks of information on historical and recorded earthquakes.

5.2 Symbols
Additionally to the loading conditions defined in Table 5.4-1, the following symbols are used in the seismic design:

- $C_s$: Seismic response coefficient (dimensionless)
- $E, E_h, E_v$: Effects of horizontal and vertical earthquake induced forces
- $F_a, F_v, F_{PGA}$: Short-period, long-period and PGA site coefficient (depending on site class / foundation)
- $I, I_c$: Seismic importance factor (depending on the risk category)
- $k_h, k_v$: Seismic coefficient for PSA, representing the acceleration
- PSA: Pseudo-static analysis
- $R$: Response modification factor, representing the ductility (energy dissipation in the inelastic range)
- RSA: Response spectrum analysis
- $S_a$: Design spectral response acceleration
- $S_s, S_1$: 5% damped short term and 1 second spectral response acceleration parameter
- $T$: Fundamental period of a structure
- THA: Time history analysis
- $V$: Total design lateral force or shear at the base

5.3 Codes and Standards
- ICOLD: International Commission on Large Dams
- USACE: United States Army Corps of Engineers:
  - EM1110-2-2100 (2005): Stability Analysis of Concrete Structures
  - EM 1110-2-6050 (1999): Response Spectra and Seismic Analysis for Concrete Hydraulic Structures

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5.4 Seismic Load Conditions and Designations

The following table gives an overview of the commonly used seismic loading conditions.

Table 5.4-1: Seismic Load Conditions in ICOLD, USACE and ASCE

<table>
<thead>
<tr>
<th>Name</th>
<th>Reference, return period</th>
<th>Comment and performance criteria</th>
</tr>
</thead>
<tbody>
<tr>
<td>MCE</td>
<td>USACE, ICOLD Not specified (DSHA)</td>
<td>The largest earthquake than can reasonably be expected. Site specific.</td>
</tr>
<tr>
<td>SEE</td>
<td>USACE</td>
<td>Term not existing. The MDE (for critical structures is the same as the MCE) has the same objective.</td>
</tr>
<tr>
<td></td>
<td>ICOLD</td>
<td>Earthquake a dam must be able to resist without uncontrolled release of reservoir.</td>
</tr>
<tr>
<td>MDE</td>
<td>USACE</td>
<td>Maximum level of ground motion for which a structure is designed. No loss of life or catastrophic failure (i.e. no uncontrolled release of reservoir).</td>
</tr>
<tr>
<td></td>
<td>ICOLD</td>
<td>For large dams replaced by SEE</td>
</tr>
<tr>
<td>MCE</td>
<td>ASCE 7 2'475 years</td>
<td>For structural design. Other factors such as a global reduction factor 2/3 and the risk category / importance factor ( I ) are applicable. These factors impact the value of the ground motions and can also be interpreted as factors that change the return period.</td>
</tr>
<tr>
<td>DBE</td>
<td>USACE</td>
<td>Term not existing. The MDE (for non-critical structures with a return period of 950 years) has a similar objective.</td>
</tr>
<tr>
<td></td>
<td>ICOLD</td>
<td>Design Earthquake for appurtenant structures.</td>
</tr>
<tr>
<td>Name</td>
<td>Reference, return period</td>
<td>Comment and performance criteria</td>
</tr>
<tr>
<td>--------------------</td>
<td>--------------------------</td>
<td>----------------------------------</td>
</tr>
<tr>
<td>OBE Operating Basis Earthquake</td>
<td><strong>USACE, ICOLD</strong> 145 years (50% in 100 years)</td>
<td>Expected to occur during the lifetime. No damage or loss of service must happen for the main structures and appurtenant structures and equipment should remain operational.</td>
</tr>
<tr>
<td>CE Construction Earthquake</td>
<td><strong>USACE, ICOLD</strong> Not specified</td>
<td>A reasonable PGA value for a seismic event that is in the range of a 10-years return period is selected.</td>
</tr>
</tbody>
</table>

Note: In order to avoid confusion with the maximum credible earthquake, the seismic actions and the associated ground motion with a return period of 2,475 years (2% in 50 years) will be named MDE,<sub>structural</sub>.
## 5.5 Seismic Design Approach

### Table 5.5-1: Earthquake designation for LP HPP

<table>
<thead>
<tr>
<th>Designation</th>
<th>Return Period</th>
<th>Applicability and Performance Criteria</th>
<th>Method</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>SEE</td>
<td>10'000 years (1% in 100 y) or -- (DSHA)</td>
<td>Stability analysis evaluation for dam safety relevant structures. Permanent displacements must remain within acceptable range. In addition the post-SEE stability shall be verified.</td>
<td>BD, THA or RSA</td>
<td>THA: Damping 7% but no R-factor RSA: Damping 5% with R-factor THA &amp; RSA: with permanent sliding</td>
</tr>
<tr>
<td>MDE, stability</td>
<td>950 years (10% in 100 y)</td>
<td>Stability analysis for non-dam safety relevant main structures, extreme LC. For dam safety relevant structures, the LC is only informative.</td>
<td>THA, RSA or PSA</td>
<td>PSA: 2/3-factor applicable but no R RSA: Damping 5% (scaled from DBE) with R-factor but no 2/3-factor</td>
</tr>
<tr>
<td>MDE, structural</td>
<td>2'475 years (2% in 50 y)</td>
<td>Structural design for all structures according to ASCE. 2/3 factor and importance factor can be interpreted as modification of return period (see below).</td>
<td>wN in BD THA or RSA</td>
<td>THA: Damping 5% but no R-factor RSA: Damping 5% with R-factor THA: with DCR, but I, and 2/3-factor (ASCE) not applicable RSA: DCR, I, and 2/3 (ASCE) not applicable</td>
</tr>
<tr>
<td>DBE</td>
<td>475 years (10% in 50 y)</td>
<td>Stability analysis for appurtenant structures, extreme LC.</td>
<td>PSA</td>
<td>PSA: 2/3-factor applicable but no R-factor</td>
</tr>
<tr>
<td>OBE</td>
<td>145 years (50% in 100 y)</td>
<td>Stability analysis for all structures, unusual LC.</td>
<td>BD, PSA</td>
<td>PSA: 2/3-factor applicable but no R-factor</td>
</tr>
<tr>
<td>Designation</td>
<td>Return Period</td>
<td>Applicability and Performance Criteria</td>
<td>Method</td>
<td>Comment</td>
</tr>
<tr>
<td>-------------</td>
<td>---------------</td>
<td>----------------------------------------</td>
<td>--------</td>
<td>---------</td>
</tr>
<tr>
<td>CE</td>
<td>Not specified</td>
<td>Structural design for all structures with un-factored spectral accelerations.</td>
<td>wN in BD RSA</td>
<td>Damping 5% with R-factor L and 2/3-factor not applicable</td>
</tr>
</tbody>
</table>

**BD**: Basic Design  
**wN**: where deemed necessary  
2/3 factor for stability (USACE) and structural (ASCE): see further explanations below
For the pseudo-static approach, the seismic coefficient $k_h$ can be assumed as follows:

- $k_h = \frac{2}{3} \cdot PGA$, where
  - PGA: Peak ground acceleration according to Table 6.8-10
  - 2/3-factor according to USACE 1110-2-6053, Chapter 7.3

**Note:** USACE 2/3-factor only to be used for PSA. The factor (only used in the stability analysis) accounts for ductility and the period of the structure and is therefore different from the ASCE 2/3-factor.

For the ASCE 7-10 approach of the MDE$_{\text{structural}}$-earthquake load case, the following can be considered:

- $S_a = \frac{2}{3} \cdot S_{aM} \cdot I_c$ (ASCE 7-10 formula 21.3-1 and $I_c$ from Chapter 11.5)
  - $S_a$: Spectral accelerations
  - $S_{aM}$: MDE$_R$ (MCE$_R$ in ASCE) spectral response accelerations as given in Table 6.8-8
  - MDE$_R$ (or MCE$_R$): Risk-based maximum design earthquake (maximum credible earthquake in ASCE).

**Note:** ASCE 2/3-factor only to be used for MDE$_{\text{structural}}$ (or MCE$_{\text{ASCE}}$) but not for SEE or OBE. The factor (only used in the structural design) aims to reduce the seismic force to a DBE level and is therefore different from the USACE 2/3-factor.

- $F_a$ and $F_v$ factors are not applicable.
- Risk category IV is considered for all dam safety relevant structures because their failure could pose a substantial hazard to the community.
  - This results in a seismic importance factor $I_c = 1.50$.
- Risk category III is considered for a series of non-dam safety relevant structures wherever the economic impact that a potential failure would cause is substantial.
  - This results in a seismic importance factor $I_c = 1.25$.
- Risk category II is considered for other appurtenant structures
  - This results in a seismic importance factor $I_c = 1.00$.

**Note:** $I_c$ factor only to be used for MDE$_{\text{structural}}$ (or MCE$_{\text{ASCE}}$) structural design but neither for other structural seismic levels nor for any level of stability analysis.

- Response modification factors (R-factors) depend on the structural system and will be defined in the relevant structural reports. R-factors are listed in the ASCE 07 Table 12.2-1.

**Note:** R-factors only to be used for response spectrum analysis (RSA) for structural design and stability analysis. The factor accounts for the ductility, energy dissipation and the force reduction due to ductility.

The MDE$_{\text{stability}}$ spectral response accelerations for the THA with a return period of 950 years can be scaled from the DBE spectral response accelerations as follows:

- $S_{a,950y} = S_{a,475y} \cdot \frac{0.29}{0.23}$ for horizontal accelerations
- $S_{a,950y} = S_{a,475y} \cdot \frac{0.23}{0.18}$ for vertical accelerations
5.6 Dam Safety

The structures of the Luang Prabang HPP will be separated into dam safety relevant and non-dam safety relevant structures. As also indicated in Table 5.5-1, the dam safety relevant structures will be designed for SEE and must not result in catastrophic failure (i.e. uncontrolled release of reservoir). Damages during an SEE event however are acceptable. Figure 5.6-1 and Figure 5.6-2 show the separation between those two categories. Table 5.6-1 defines the seismic design level for the main civil structures and gives an overview over the intended seismic design levels for the hydro mechanical equipment.

Table 5.6-1: Dam Safety Concept

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<th>Structure</th>
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<td>I_c = 1.25 for MDE_\text{structural}</td>
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<td>Lower Lock</td>
<td>MDE</td>
<td>OBE</td>
<td>I_c = 1.25 for MDE_\text{structural}</td>
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<tr>
<td>Upstream and downstream guide wall</td>
<td>DBE</td>
<td>MDE_\text{structural}</td>
<td>I_c = 1.0 for MDE_\text{structural}</td>
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<td>Middle mitre gate</td>
<td>SEE</td>
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<tr>
<td>Upper and lower mitre gate</td>
<td>DBE</td>
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<tr>
<td>Stoplogs</td>
<td>DBE (u/s)</td>
<td>OBE (d/s)</td>
<td>Closing under balanced conditions. Elements from LLO</td>
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<td>OBE</td>
<td></td>
</tr>
<tr>
<td>RCC Closure Structure</td>
<td>SEE</td>
<td>MDE_\text{structural}</td>
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</tbody>
</table>

The MDE\_\text{stability} for the stability analysis of dam-safety relevant structures is informative.

**Figure 5.6-1: Overview of the Earthquake designations**

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5.7 Seismic Load Combinations

The seismic force may act in any horizontal direction as well as vertically. It is assumed that the seismic forces are acting in the worst horizontal direction (generally in flow direction for the stability analysis and often perpendicular to the flow direction for structural calculations).

For the stability analysis, the following seismic load combination is used:

- Pseudo static
  \[ E = \frac{1}{3} E_{\text{hor}} + E_{\text{vert}} \]
  or
  \[ E = \frac{1}{3} E_{\text{hor}} + E_{\text{vert}} \]
- Response spectrum analysis  same as pseudo static
- Time History
  \[ E = E_{\text{hor}} + E_{\text{vert}} \]

For the structural design, the following load combination will be used:

- Response Spectra Analysis  SRSS or CQC method
- Time History Analysis
  \[ E = E_{\text{hor},x} + E_{\text{hor},y} + E_{\text{vert}} \]

5.8 Coincident Pool

According to the USACE EM-1110-2-2100 manual, the coincident pool represents temporal average pool conditions, which are used for load combinations that include seismic loads. The tailwater level for seismic load cases is taken as the median or 50% of time flow exceeded value of the flow duration curve (which is lower than the mean value in general more critical).

5.9 Permanent Displacement

The maximum permanent displacement in the SEE load case shall be limited to a value that safety relevant pipes and conductors are not harmed and waterstop in contraction joints are expected to withstand. A target value for LP HPP is 50 cm.
5.10 Demand Capacity Ratio DCR

A demand to capacity comparison, utilizing a demand to capacity ratio (DCR) as a performance indicator, is the basis for performance evaluation of reinforced concrete structures subject to earthquake ground motions. The DCR is defined as the ratio of force or moment demand (ultimate; from the structural calculations) to force or moment capacity (nominal strength or resistance; without strength reduction factor $F$). The DCR approach shall follow the description in the USACE EM-1110-2-6053 manual, assuming that:

- The structures are lightly reinforced
- Beams, slabs, walls and other load carrying members are controlled by flexure
- The seismic detailing requirements of ACI 318 are not met
- Wall and other vertical load carrying members have axial load ratios $\frac{P}{A_g \cdot f_c'}$ less than 0.1

5.11 Minimum Flexural Steel

The USACE EM-1110-2-6053 suggests a minimum flexural steel content for structures in high seismic regions to ensure larger ductile performance than ordinary concrete hydraulic structures. This criterion will be checked but not be thoroughly applied for the LP HPP.

$\frac{M_N}{M_{cr}} \geq 1.2$

$M_N$: Nominal moment strength (without strength reduction factor $F$)

$M_{cr}$: Cracking moment

$M_{cr} = \left( f'_{c} + \frac{P}{A_g} \right) y_i = \frac{0.62 \cdot \sqrt{f'_{c} + \frac{P}{b t'}}}{\frac{t}{2}} \frac{b t^3}{12}$ for rectangular structures

- $b$: width of structure
- $t$: thickness of structure
- $P$: axial load

5.12 Deformations

In general, large elastic deformations resulting in some damages are accepted for the maximum design earthquake. Only for the operating basis earthquake OBE, the deformations are controlled. For the safety evaluation earthquake SEE, even inelastic and therefore significantly larger deformations are accepted.

An exception to the above is the deformations at the Spillway piers. The deformations shall remain elastic and should follow the limits of the radial gate supplier. This means that the DCR concept shall not be applied to the Spillway piers (or where the functionality of the radial gates may be affected).
6 CIVIL DESIGN CRITERIA

6.1 Basis

This chapter establishes the basic civil / structural design criteria that will be used for the outline-design of the civil engineering structures of Luang Prabang Hydroelectric Power Project.

A stability analysis will be performed for the main structures:

- Navigation Lock (NL)
- Spillway (SP)
- Powerhouse (PH)
- RCC Closure Structure (RC)

The stability of the Right Pier (RP) and the Left Pier (LP) will be proven by analogy conclusion to the Powerhouse.

A structural verification shall confirm that the main concrete member dimensions are adequate. Structural verification of members will be done where deemed necessary by the engineer.

6.2 Codes and Standards

The design of all civil structures is in accordance with the codes and standards referenced herein:

- **LEPTS** Lao Electric Power Technical Standard:
  - Lao Electric Power Technical Standards, December 2018
- **USACE**: United States Army Corps of Engineers:
  - EM 1110-2-2100 (2005): Stability Analysis of Concrete Structures
  - EM 1110-2-2106 (2000): Roller Compacted Concrete
  - EM 1110-2-6050 (1999): Response Spectra and Seismic Analysis for Concrete Hydraulic Structures
- **ACI** American Concrete Institute:
  - ACI 318 (2014): Building Code Requirements for Structural Concrete
  - ACI 207.5R (1999): Roller-Compacted Mass Concrete
The USACE EM 1110-2-2104 (with the changes made in 2003) – Strength Design for Reinforced-Concrete Hydraulic Structures – is not considered due to the conservative load combinations and the additional hydraulic factor that is used in lieu of performing serviceability checks.

### 6.3 Design Method

The stability analysis for seismic load cases is done according to chapter 5.5:

- For the OBE and DBE, a pseudo static approach (PSA) was pursued
- For the MDE and SEE load case, the dynamic forces for the SEE load case are calculated using a finite element (FE) model with response spectra analysis (RSA) or time history analysis (THA).
- Permanent sliding for the SEE load case is calculated with a separate computer program to evaluate the transient response for concrete dams subject to seismic loads.

The structural design will be carried out using FE models. The calculation will be done with RSA or THA.

### 6.4 Stability Analysis

The stability analysis described in this Chapter is based on the recommendations of the USACE manual EM 1110-2-2100 and other USACE manuals as per Chapter 6.2.

The stability analysis as stipulated in the LEPTS follows the same principles as the USACE, although different load factors are defined.

#### 6.4.1 Load Condition Categories

The USACE manual defines the following load condition categories and return periods to be considered for the stability analysis.

<table>
<thead>
<tr>
<th>Load Condition category</th>
<th>Return period</th>
</tr>
</thead>
<tbody>
<tr>
<td>Usual</td>
<td>Less than or equal to 10 years. Primary function of the structures</td>
</tr>
<tr>
<td>Unusual</td>
<td>Greater than 10 years but less than or equal to 300 years</td>
</tr>
<tr>
<td>Extreme</td>
<td>Greater than 300 years, emergency conditions or the combination of unusual loading events. Major accidents</td>
</tr>
</tbody>
</table>

#### 6.4.2 Site information and designation

The site information can be classified as well-defined site information, ordinary site information or limited site information. This classification has an impact on the stability
analysis safety factors (see Chapter 6.4.7). For ordinary site information, the following requirements must be satisfied:

- Foundation strengths have been established with reasonable exploration and testing procedures.
- Foundation strengths can be established with a high level of confidence (or is sufficiently conservative).
- The governing load condition can be established with a high level of confidence (or is sufficiently conservative).

For the stability analysis, the structures are designated as either critical or normal. According to the USACE manual EM 1110-2-2100 (clause 3.5 and Appendix H), structures designated as critical are those structures on high hazard projects whose failure will result in loss of life (directly or from secondary effects).

**Designation for Luang Prabang HPP:**

- **Ordinary** site information
- All main structures within the Mekong river bed, whether dam safety relevant or not dam safety relevant (see Chapter 5.6) are **critical** structures
- A series of appurtenant structures, all not dam safety relevant, can be considered **normal** structures

### 6.4.3 Flotation

The safety factor is defined as the ratio between the forces than can cause instability and the weight of materials that resist flotation.

\[
FS_f = \frac{W_S + W_C + S}{U - W_G}
\]

- \(W_S\) [kN]: weight of the structure and soil
- \(W_C\) [kN]: weight of the water contained within the structure
- \(S\) [kN]: surcharge loads
- \(U\) [kN]: uplift forces acting on the base of the structure
- \(W_G\) [kN]: weight of water above top surface of the structure

As it is common practise, the weight of the various E&M equipment (turbine, generator, MIV, overhead cane, BOP) will not be taken into account.

### 6.4.4 Sliding

The factor of safety is calculated according to the below formula. As mentioned in Chapter 6.4.2, the required safety factors (see Chapter 6.4.7) depend on the type of the structure which must be classified as normal or critical and on the available geotechnical site information. Depending on the project stage, the safety factors may vary as well.

\[
FS_s = \frac{N \tan \phi + cL}{T}
\]

- \(N\) [kN]: force acting normal to the sliding failure plane
- \(T\) [kN]: shear force acting parallel to the base
- \(L\) [m²]: length (or area) of the base
- \(c\) [kPa]: cohesive strength of the foundation material
- \(\phi\) [°]: angle of internal friction of the foundation material
6.4.5 Rotation and Overturning

The rotational behaviour (overturning) is evaluated by determining the location of the resultant of all applied forces with respect to the potential failure plane.

For usual loading conditions, the entire base must be in compression, so there is no chance for higher uplift pressures to develop in a crack. This is in line with the general assumption of linear uplift pressures. For the unusual and extreme loading conditions, higher uplift pressures may develop.

\[ e = \frac{L}{2} - \frac{\sum M_z}{V} \]

- \( e \) [m]: eccentricity of the resultant force
- \( L \) [m]: length (or area) of the base
- \( M_z \) [kNm]: driving moments that cause rotation of the structure
- \( V \) [kN]: vertical component of the resultant force

**Table 6.4-2: Rotation requirements**

<table>
<thead>
<tr>
<th>USACE criterion</th>
<th>Maximum eccentricity</th>
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<tbody>
<tr>
<td>100% of the base in compression</td>
<td>L/6</td>
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<tr>
<td>75% of the base in compression</td>
<td>L/4</td>
</tr>
<tr>
<td>resultant within base</td>
<td>L/2</td>
</tr>
</tbody>
</table>

Additionally, rotation will be checked comparing the resisting moments with the overturning moments. However, no safety factor limitations are applied.

\[ FS_r = \frac{\sum M_{res}}{\sum M_{driv}} \]

- \( M_{res} \) [kNm]: Resisting moments
- \( M_{ driv} \) [kNm]: Overturning or driving moments

6.4.6 Allowable Bearing Pressures

The bearing pressures of the structures will be checked against the maximum allowable bearing pressure. A rigid foundation is assumed and the below formula for maximum and minimum bearing pressures is applied.

\[ \sigma^+ = \frac{H}{S} + \frac{6 \sum M_z}{B^2L} ; \quad \sigma^- = \frac{H}{S} - \frac{6 \sum M_z}{B^2L} \]

6.4.7 Safety Factors

Based on the recommendations of the USACE manual, the following safety factors shall be met for reinforced concrete structures.
Table 6.4-3: Safety factor requirements (from USACE EM 1110-2-2100)

<table>
<thead>
<tr>
<th>Load Condition</th>
<th>Flotation</th>
<th>Sliding*</th>
<th>Overturning</th>
<th>Foundation bearing pressure</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Critical</td>
<td>Normal</td>
<td>100% of Base in Compression</td>
</tr>
<tr>
<td>Usual</td>
<td>1.3</td>
<td>2.0</td>
<td>1.5</td>
<td></td>
</tr>
<tr>
<td>Unusual</td>
<td>1.2</td>
<td>1.5</td>
<td>1.3</td>
<td>75% of Base in Compression</td>
</tr>
<tr>
<td>Extreme</td>
<td>1.1</td>
<td>1.1</td>
<td>1.1</td>
<td>Resultant within Base</td>
</tr>
</tbody>
</table>

*) Values for ordinary site information

Table 6.4-4: Safety factor requirements (from LEPTS Chapter 2-3-2, Article 26)

<table>
<thead>
<tr>
<th>Load Condition</th>
<th>Sliding with cohesion</th>
<th>Overturning</th>
</tr>
</thead>
<tbody>
<tr>
<td>Usual</td>
<td>3</td>
<td>Middle third (100% of Base in Compression)</td>
</tr>
<tr>
<td>Earthquake or flood (no return period specified)</td>
<td>2</td>
<td>Middle half (75% of Base in Compression)</td>
</tr>
</tbody>
</table>

6.4.8 Basic Loading Conditions

The below basic loading conditions shall be investigated for the basic design. During the detailed design further loading conditions (e.g. construction load cases) are expected to be added.

**Loading Condition 1 – Normal operating + low tailwater – Usual**
- Dead loads (concrete, soil, water and permanent equipment)
- Headwater at High Operating Level
- Minimum Tailwater level
- Uplift with drainage efficiency as specified
- Waterways full
- Static soil pressures if applicable

**Loading Condition 2 – Normal operating + high tailwater – Usual**
- Condition 1 but with high Tailwater level HQ

**Loading Condition 3 – Maintenance + low tailwater – Usual or Unusual**
- Condition 1 but with empty waterways

**Loading Condition 4 – Maintenance + high tailwater – Usual or Unusual**
- Condition 2 but with empty waterways

**Loading Condition 5 – Drainage failure + low tailwater – Unusual**
- Condition 1 but neglecting the drainage

**Loading Condition 6 – Drainage failure + high tailwater – Unusual**
- Condition 2 but neglecting the drainage

**Loading Condition 7 – Normal operating + OBE – Unusual**
- Dead loads (concrete, soil, water and permanent equipment)
6.5 Structural Design

6.5.1 General

The Limit State Design (LSD) or Load and Resistance Factor Design (LRFD) according to the ACI 318 will be used for the structural design of all structural elements. The load factors of the ACI 318-2011 are in line with the ASCE 7-2010 requirements.

The nominal strength (N) according to the ACI 318 multiplied by a strength reduction factor $F$ shall be higher than the required strength $U$ (ultimate limit state) from the structural analysis for all structural elements:

$$\Phi N \geq U$$

The required strength $U$ is the maximum of the load combinations considering the load factors given in Table 6.5-1 below.

6.5.2 Load Combinations

The below basic load combinations according to the ACI 318 chapter 9.2 will be applied for the structural design. More detailed load combinations or explanations may be given in the separate structural design reports for the various structures.
Table 6.5-1: Load combinations and load factors structural design

<table>
<thead>
<tr>
<th>Load case</th>
<th>D</th>
<th>L1*</th>
<th>L2*</th>
<th>L3*</th>
<th>H**</th>
<th>F</th>
<th>E</th>
<th>W</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal 1</td>
<td>1.4</td>
<td>1.4</td>
<td>1.6</td>
<td>1.6</td>
<td>1.4</td>
<td>1.6</td>
<td>1.4</td>
<td>1.4</td>
</tr>
<tr>
<td>Normal 2</td>
<td>1.2</td>
<td>1.6</td>
<td>1.6</td>
<td>1.6</td>
<td>1.4</td>
<td>1.6</td>
<td>1.2</td>
<td></td>
</tr>
<tr>
<td>Construction</td>
<td>1.4</td>
<td>1.4</td>
<td>-</td>
<td>-</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Unusual flood / waterlevel</td>
<td>1.2</td>
<td>1.4</td>
<td>1.4</td>
<td>1.6</td>
<td>1.4</td>
<td>1.6</td>
<td>1.4</td>
<td>1.6</td>
</tr>
<tr>
<td>Extreme flood / waterlevel</td>
<td>1.2</td>
<td>1.2</td>
<td>1.2</td>
<td>1.6</td>
<td>1.2</td>
<td>1.2</td>
<td>1.4</td>
<td></td>
</tr>
<tr>
<td>Earthquake OBE &amp; CE</td>
<td>1.2</td>
<td>1.0</td>
<td>1.2</td>
<td>1.6</td>
<td>1.2</td>
<td>1.2</td>
<td>1.0</td>
<td></td>
</tr>
<tr>
<td>Earthquake MDE (unusual)</td>
<td>1.2</td>
<td>1.0</td>
<td>1.0</td>
<td>1.6</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.6</td>
</tr>
<tr>
<td>Earthquake SEE (extreme)</td>
<td>1.2</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>Wind</td>
<td>1.2</td>
<td>1.0</td>
<td>1.2</td>
<td>1.6</td>
<td>1.2</td>
<td>1.2</td>
<td>1.0</td>
<td>1.6</td>
</tr>
<tr>
<td>Serviceability 1: Normal</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Serviceability 2: OBE</td>
<td>1.0</td>
<td>1.0</td>
<td></td>
<td></td>
<td>1.0</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* Live loads are not considered if acting beneficial.
** The loads H are considered in the load combination with the load factor 0.6 which is more conservative than the lowest strength reduction factor F.

D: Dead load of structure, fluid and soil weight which is not or hardly removable
L1: Live loads including uniform floor loads, equipment laydown loads transport or truck loads and similar items.

1) The permanently installed portion of the live load or 25% of the variable live load shall be considered for horizontal seismic action and the indicated serviceability load combinations.

2) The load of the fresh concrete on the subsequent floor is considered in this load combination.
L2: Live loads including the self-weight of cranes
L3: Equipment operation loads including all dynamic loads caused by the turbine, the generator and other equipment in contact with water.
H: Loads due to weight and pressure of soil and water in soil
F: Fluid loads with controllable maximum heights: All loads due to water pressure in the waterways.
E: Load effects of earthquake, including dynamic soil and water pressures
W: Wind loads

6.5.3 Strength Reduction Factors F

The following values of strength reduction factors F as defined in the ACI 318, section 9.3:

- F = 0.90 for tension controlled sections (bending moment)
- F = 0.75 for axial compression with spiral reinforcement
- F = 0.65 for axial compression of other reinforced members
- F = 0.75 for shear and torsion
- F = 0.75 for strut and tie models

6.5.4 Flexure and Axial Loads

The structural design follows in principle Chapter 10 of ACI 318, satisfying the static equilibrium and the compatibility of strains, neglecting tensile strength of concrete.

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\[ \Phi M_n = \Phi T \cdot \left( d - \frac{a}{2} \right) \geq M_u \]

\[ a = \frac{A_s \cdot f_y}{0.85 \cdot f'c \cdot b} \]

Figure 6.5-1: Flexure design

6.5.5 Shear Strength

The shear strength design follows in principle chapter 11 of the ACI 318, with minimum shear reinforcement where \( V_u \) exceeds 0.5×\( F V_c \) (also for solid slabs).

\[ \Phi V_u = \Phi (V_c + \ell) \geq V_u \]

6.5.6 Allowable Stresses for RCC structures

Table 6.5-2: Allowable Stresses

<table>
<thead>
<tr>
<th>Load Condition</th>
<th>Type</th>
<th>Allowable stresses</th>
<th>USACE 1110-2-2200</th>
<th>USACE 1110-2-2006</th>
<th>LEPTS, chapter 2-3-2, Article 26</th>
</tr>
</thead>
<tbody>
<tr>
<td>Usual</td>
<td>Compression</td>
<td>(&lt; 0.3 \times f'c ))</td>
<td>(&lt; f_c /3 )</td>
<td>No tension at u/s face</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Tension</td>
<td>(&lt; 0.1 \times f'c ) within the concrete mass</td>
<td>(&lt; 0.05 \times f'c ) at lift joints</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Unusual or</td>
<td>Compression</td>
<td>(&lt; 0.5 \times f'c )</td>
<td>(&lt; f_c /2 )</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Earthquake or flood (no return period specified)</td>
<td>Tension (^1)</td>
<td>(&lt; 0.1 \times f'c ) within the concrete mass</td>
<td>(&lt; 0.05 \times f'c ) at lift joints</td>
<td>(&lt; f_c /40 )</td>
<td></td>
</tr>
<tr>
<td>Extreme</td>
<td>Compression</td>
<td>(&lt; 0.9 \times f'c )</td>
<td>-</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Tension (^2)</td>
<td>(&lt; 0.1 \times f'c ) within the concrete mass</td>
<td>(&lt; 0.05 \times f'c ) at lift joints. For cracked sections dynamic stability analysis required</td>
<td>-</td>
<td></td>
</tr>
</tbody>
</table>

\(^1\) Allowable tensile stresses during earthquake loading (OBE) conditions may be increase by factor 1.5 as longs as all other performance criteria are met.

\(^2\) \( f_c \) is 1-year unconfined compressive strength of concrete.
2) In accordance with USACE EM 1110-2-6053 Section 6-4. d. 1.0. The level of nonlinear response or cracking for SEE Conditions is considered acceptable if demand-capacity ratios are less than 2.0 and limited to 15 percent of the dam cross-sectional surface area, and the cumulative duration of stress excursions beyond the tensile strength of the concrete falls below the performance curve given in Figure 6-3 of USACE EM 1110-2-6053.

6.6 Serviceability Requirements

6.6.1 Minimum Reinforcement
The minimum reinforcement of structural element for temperature and shrinkage stresses shall be 0.0028 times the gross cross sectional area, distributed equally on both faces. However, this shall not be less than 420 mm²/m or more than 2,100 mm²/m per face.

Minimum reinforcement for ductility see chapter 5.11.

6.6.2 Crack Width
Crack width calculations will be done in the detailed design stage.

The crack width calculation (for service loads) should follow the recommendations of the ACI 224R.

6.6.3 Deflections
Deflection calculations are done in the detailed design stage, except:
- Spillway pier at the radial gate (perpendicular to the flow direction. The allowable relative deformations are in the range of millimetres to a few centimetres and subject to confirmation with the radial gate supplier.

6.6.4 Stiffness and Eigenfrequency of the Structure
Requirements to the stiffness or rigidity of the structure and the eigenfrequency from the turbine supplier are not yet available.

Calculations will be done in the detailed design stage.

6.7 Material

6.7.1 Concrete
It is envisaged to use C25 concrete for all structures. The high strength concrete may be used in smaller areas where high and concentrated loads are acting or where high abrasion must be expected. The concrete class to be used will be defined on the concrete outline drawings.

The concrete classes as indicated in Table 6.7-1 and according to ASTM C150 shall be used.
Table 6.7-1: Concrete properties

<table>
<thead>
<tr>
<th>ACI class</th>
<th>C17</th>
<th>C25</th>
<th>C35</th>
</tr>
</thead>
<tbody>
<tr>
<td>Description</td>
<td>Backfill concrete</td>
<td>Standard Levelling concrete</td>
<td>If required, Precast</td>
</tr>
<tr>
<td>$f'_c$ (cylinder, 28 days) [MPa]</td>
<td>17</td>
<td>25</td>
<td>35</td>
</tr>
<tr>
<td>$E_c$ (modulus of elasticity [MPa] (=4700\times f'_c^{0.5} ))</td>
<td>18'800</td>
<td>23'500</td>
<td>27'800</td>
</tr>
<tr>
<td>$n$ (Poisson’s ration)</td>
<td>0.20</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Dynamic properties of concrete can be adjusted according to the USACE manual EM 1110-2-6053, section 5.1:
- Dynamic modulus is equal to 1.15 times the static modulus
- The dynamic Poisson’s Ratio is equal to 0.70 times the static ratio
- The dynamic compressive strength is equal to 1.15 times the static compressive strength
- The dynamic tensile strength is equal to 1.50 times the static tensile strength
- The dynamic shear strength is equal to 1.10 times the static shear strength

6.7.2 RCC

Roller Compacted Concrete of class C17 shall be used. The material properties considered for RCC C17 are as follows:
- Compressive strength $f'_c$: 17 MPa
- Unit weight: 23 kN/m$^3$
- Tensile strength of the lift joint $f'_t$: 0.85 MPa
- $E$-modulus (static) $E_c=4\times 700\times f'_c^{0.5}$: 19'500 MPa
- Dynamic Compressive Strength $f'_{c,dyn}=1.5\times f'_{c,static}$: 25.5 MPa
- Dynamic tensile strength of lift joint $f'_{t,dyn}=1.5\times f'_{t,static}$: 1.275 MPa
- Cohesion of the lifting joint $2\times f'_t\times 60\%$: Peak: 1.0 MPa, Residual: 0 MPa
- Friction angle of the lifting joint: Peak: 45°, Residual: 43°

6.7.3 Hardfill Concrete

Hardfill concrete shall be considered to have a compressive strength of 5 MPa. Its purpose is to have sufficient cohesion and tensile strength, not to develop any active pressure against walls.

The hardfill concrete is not required to have low permeability.
- Compressive strength $f_c$: 5 MPa
- Unit weight: 20 kN/m$^3$
6.7.4 Reinforcement
The below indicated reinforcement steel grade is used. The modulus of elasticity is taken as 200'000 MPa. The following rebar diameters are available:

- DB10, DB12, DB16, DB20, DB25
  ASTM A615M grade 420 or TIS 24-2548 grade SD40 $f_y = 400$ MPa
- DB28, DB32
  ASTM A615M grade 520 or TIS 24-2548 grade SD50 $f_y = 500$ MPa

Reinforcement with lower or higher yield strength shall not be used.

6.7.4.1 Rebar spacing
The rebar spacing shall be kept at 200 mm wherever possible.

6.7.4.2 Concrete Cover
According to the ACI 318 chapter 7.7, the minimum cover for the main reinforcement of in-situ concrete shall be as follows:

- In general (exposed to earth and weather): 50 mm
- Concrete elements with thickness $\leq 0.6m$, not exposed to outside air and not in contact with soil: 40 mm
- Surfaces in contact with water (high velocity): 100 mm
- Surfaces in contact with water: 75 mm
- Surfaces in contact with soil, backfill, rock or shotcrete: 75 mm
- Precast elements in general: 30 mm

6.7.5 Structural Steel
Structural steel is foreseen for the roof structures of the Powerhouse. It is envisaged to use the following structural steel:

- ASTM A36: $f_y = 250$ MPa
  E-modulus: 210’000 MPa

6.8 Loads

6.8.1 Dead Load (D)
The dead load consists of the self-weight of the structure including all installations, permanent equipment and materials.

The unit weights of the common construction materials will be taken as follows:

- Concrete (reinforced and plain) $24.0 \, \text{kN/m}^3$ (for the stability analysis)
  $25.0 \, \text{kN/m}^3$ (for the structural design)
- Water $10.0 \, \text{kN/m}^3$
- Steel $78.5 \, \text{kN/m}^3$
- Common brickwork Not considered
- Backfill (soil) See Chapter 6.8.5
For the stability analysis a slightly reduced self-weight of the concrete shall be considered as defined above.

6.8.2 Live Loads (L1)

6.8.2.1 Uniform Loads

The live loads include floor uniform loads, equipment laydown loads, equipment handling wheel loads, trucks and similar items.

The following minimum live loads shall be used:

- Erection bay, generator and turbine floor: 100 kN/m²
- Turbine floor: 50 kN/m²
- Machine Hall floor: 30 kN/m²
- Workshops (in erection bay): 25 kN/m²
- Floor load in general (see comment below): 25 kN/m²
- Backfill, Transformer and Switchyard area: 15 kN/m²
- Stairs, corridors, offices, locker rooms, toilets: 5.0 kN/m²
- Steel roofs (not accessible; depending on the size): 0.6 kN/m² to 1.0 kN/m²

The equipment and live loads are generally not considered for the stability analysis unless for a verification of the maximum base pressures or for lateral earth pressures.

The general floor load covers the construction with 1 m fresh concrete on the subsequent floor during the construction.

6.8.2.2 Impact Loads

Impact loads like barge impact and Hawser pull (USACE EM-2-2062) are not relevant for the current design stage of the concrete structures.

6.8.3 Crane Loads (L2)

Two machine hall cranes will be installed in the powerhouse. The capacity of each crane is 380 ton, resulting in a total load of 760 tons for the two cranes coupled. Further crane loads are not relevant for the current design stage.

6.8.4 Equipment Operation Loads (L3)

The equipment operation loads are not relevant for the current design stage or are considered according to the designer’s experience.

6.8.5 Earth and Rockfill Loads (H)

6.8.5.1 Backfill

The static lateral loads exerted by the backfill on the structural elements shall be determined for the at-rest condition using the following parameters. Where not distinguished between wet and saturated, the average values shall be considered.
Table 6.8-1: Backfill properties

<table>
<thead>
<tr>
<th>Backfill</th>
<th>$\gamma$ [kN/m³]</th>
<th>$c$ [MPa]</th>
<th>$\phi$ [°]</th>
<th>$k_f$ [m/s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>wet</td>
<td>20</td>
<td>0</td>
<td>35</td>
<td>$10^{-3}$</td>
</tr>
<tr>
<td>saturated</td>
<td>23</td>
<td>0</td>
<td>30</td>
<td>$10^{-3}$</td>
</tr>
<tr>
<td>average</td>
<td>22</td>
<td>0</td>
<td>33</td>
<td>$10^{-3}$</td>
</tr>
</tbody>
</table>

$\gamma$: unit weight  
$c$: cohesion  
$\phi$: effective internal friction angle  
$k_f$: coefficient of permeability

Where traffic is possible on the backfill or rockfill, a surcharge load according to Chapter 6.8.2 shall be considered. The soil pressures for dynamic load cases are explained in Chapter 6.8.7.2.

The soil pressures for static loads are calculated with the at rest earth pressure $K_0$. For a vertical wall without friction and a horizontal backfill surface, $K_0$ can be assumed as:

$$K_0 = 1 - \sin\phi$$

Active earth pressures require yielding of backfill which is only possible with wall deformations. The soil pressure reduces to active soil pressure if the deformation at the top of the structure is in the range of 0.4% to 0.5% for rotation around the toe of the structure or 0.2% for parallel movement of the structure. Passive earth pressure require significantly higher deformations than active earth pressure and are therefore not considered.

6.8.5.2 Sedimentation and Silt

The following table indicates the design soil levels upstream of the plant, in case of sedimentation.

Table 6.8-2: Design elevations for sedimentation levels

<table>
<thead>
<tr>
<th>Location</th>
<th>Max sediment elevation</th>
<th>Elevation of rock or foundation</th>
<th>Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>RCC Closing Structure upstream</td>
<td>312.50 m a.s.l.</td>
<td>302.00 m a.s.l.</td>
<td>Unusual</td>
</tr>
<tr>
<td>RCC Closing Structure downstream</td>
<td>Not considered (above backfill)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Powerhouse</td>
<td>270.00 m a.s.l.</td>
<td>254.00 m a.s.l.</td>
<td>Unusual</td>
</tr>
<tr>
<td>Spillway Surface</td>
<td>Not governing for civil</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Spillway LLO</td>
<td>Not governing for civil</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Navigation Lock</td>
<td>Not governing</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
### 6.8.6 Hydrostatic Loads (F)

#### 6.8.6.1 Design Water Levels

For the calculation of all hydrostatic loads, the unit weight of water will be taken as 10 kN/m³.

The following tables indicate the design water levels:

| Table 6.8-4: Design elevations for the Upstream water levels – final stage |
|---------------------------------|---------|---------|
| Upstream water level | Value | Type |
| PMF Level (Q=41'400 m³/s) | 314.20 m a.s.l. | Extreme |
| Maximum considered surcharge | 317.00 m a.s.l. | Extreme |
| 10'000-year flood (n-1 gates operational, Q=33'500 m³/s) | 313.60 m a.s.l. | Informative |
| HQ₁₀₀ (Q=23'800 m³/s) | See FSL | Informative |
| Design Full Supply Level (FSL₁) | 312.50 m a.s.l. | Normal |
| Full Supply Level (FSL) | 312.00 m a.s.l. | Informative |

The maximum considered surcharge is 5 m above the FSL. The water would flow over the Spillway radial gates. Considering all radial gates closed, the discharge over the radial gates with the maximum considered surcharge is less than 3’000 m³/s.

The tailrace rating curves are shown chapter 2.4. Table 6.8-5 shows the water levels that are relevant for the design of the civil structures.

| Table 6.8-5: Tailrace water levels |
|-----------------------------------|---------|---------|
| Tailrace water level | Tailwater Level | Type |
| Minimum water level measured (Q=516 m³/s) | 274.77 m a.s.l. | Informative |
| Minimum Operating Level (MOL, Q=745 m³/s) | 275.50 m a.s.l. | Normal |
| Median Annual Discharge (Q=2’100 m³/s) | 278.80 m a.s.l. | Normal |
| Normal Operating Level (NOL, Q=5’355 m³/s) | 282.50 m a.s.l. | Normal |
| 10-year flood (Q=18’200 m³/s) | 292.60 m a.s.l. | Normal |
| 100-year flood (Q=23’800 m³/s) | 295.30 m a.s.l. | Unusual |
| Maximum Operational Level (HOL, Q=24,180 m³/s) | 295.50 m a.s.l. | Informative |
| 1’000-year flood (Q=28’800 m³/s) | 297.00 m a.s.l. | Informative |
| 10’000-year flood (Q=33’500 m³/s) | 298.40 m a.s.l. | Informative |
| PMF level (Q=41’400 m³/s) | 300.50 m a.s.l. | Extreme |
6.8.6.2 Navigation Lock

According to the MRC Guidance the Navigation Lock has to be operated between a 30 years flood ($Q=21'700 \text{ m}^3/\text{s}$) and 95% flow duration of the river in natural conditions ($Q=1'100 \text{ m}^3/\text{s}$).

The main water retaining mitre gate is the middle mitre gate. For all extreme load combinations such as all seismic load cases except OBE and high water levels including the PMF load case, it shall be assumed that the lock is not in operation and the middle mitre gate is closed.

Table 6.8-6: Design elevations for the Navigation Lock

<table>
<thead>
<tr>
<th>Navigation Lock</th>
<th>Upper Chamber</th>
<th>Lower Chamber</th>
<th>Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>Minimum Operating Level (Q=1'100 m$^3$/s)</td>
<td>312.50 m a.s.l</td>
<td>294.25 m a.s.l</td>
<td></td>
</tr>
<tr>
<td></td>
<td>294.25 m a.s.l</td>
<td>276.50 m a.s.l</td>
<td>Normal</td>
</tr>
</tbody>
</table>

6.8.6.3 Fish Migration Facilities

According to the MRC Design Guidance fish passage facilities have to operate from minimum flows ($Q=1’170 \text{ m}^3/\text{s}$; tailwater level 276.70 m a.s.l.) to a 1-year flood ($Q=10’650 \text{ m}^3/\text{s}$; tailwater level 287.40 m a.s.l.).

Table 6.8-7: Design elevations for the Fish Migration Facilities

<table>
<thead>
<tr>
<th>Fish Migration</th>
<th>Tailwater Level</th>
<th>Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>Minimum Operating Level (Q=1’170 m$^3$/s)</td>
<td>276.70 m a.s.l</td>
<td>Normal</td>
</tr>
<tr>
<td>Maximum Operating Level (HQ, Q=10’650 m$^3$/s)</td>
<td>287.40 m a.s.l</td>
<td>Normal</td>
</tr>
</tbody>
</table>

The water level in the Right Pier feeding pond and the Powerhouse feeding galleries is assumed to be 0.50 m above the tailwater level given in Table 6.8-5. An overflow with sill elevation on 289.38 m asl is located at the feeding pond.

The water level in the left pier fish locks and the upstream fish migration exit channel can reach the PMF water level (see Table 6.8-4).

The water level in the right side upstream fish migration upper channel is designed for the FSL water level because the gates at the intake / outlet structure will automatically be closed at higher flood discharges.

The discharge in the d/s chute is 20 m$^3$/s during the entire operation of the d/s migration. It is likely that the d/s migration operates also for higher discharges than defined in Table 6.8-7.

6.8.6.4 Construction Stage Water Levels

During the construction stage of the main structures, the Mekong flows in its natural river bed, flanked by the cofferdams and concrete structures to protect the construction pit. These care of river structures are designed to withstand a 100 years flood. The load condition categories (type) in the above table is different from the recommendation for permanent concrete hydraulic structures defined in the USACE manual EM 1110-2-2100.
Table 6.8-8: Design elevations for the Upstream and tailrace water levels – construction stage

<table>
<thead>
<tr>
<th>Construction stage</th>
<th>U/S Cofferdam</th>
<th>Diversion Wall</th>
<th>D/S Cofferdam</th>
<th>Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>HQ2 (Q=12’800 m³/s)</td>
<td>290.70 m a.s.l.</td>
<td>289.20 m a.s.l.</td>
<td>289.00 m a.s.l.</td>
<td>Normal</td>
</tr>
<tr>
<td>HQ10 (Q=18’200 m³/s)</td>
<td>294.60 m a.s.l.</td>
<td>292.60 m a.s.l.</td>
<td>292.60 m a.s.l.</td>
<td>Unusual</td>
</tr>
<tr>
<td>HQ100 (Q=23’800 m³/s)</td>
<td>297.80 m a.s.l.</td>
<td>295.30 m a.s.l.</td>
<td>295.30 m a.s.l.</td>
<td>Extreme</td>
</tr>
</tbody>
</table>

In a second construction stage during which the RCC closing structure is being built, the Mekong is diverted through the Spillway. In this construction stage, the minimum navigation level should be kept in the reservoir.

6.8.6.5 Maintenance Water Levels

Table 6.8-9: Design elevations for maintenance load cases

<table>
<thead>
<tr>
<th>Structure</th>
<th>Upstream</th>
<th>Downstream</th>
<th>Type</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Powerhouse</td>
<td>312.00 m a.s.l.</td>
<td>295.30 m a.s.l.</td>
<td>Unusual</td>
<td>HQ100</td>
</tr>
<tr>
<td>Spillway (surface and LLO)</td>
<td>312.00 m a.s.l.</td>
<td>291.40 m a.s.l.</td>
<td>Unusual</td>
<td>HQ5</td>
</tr>
<tr>
<td>Navigation Lock</td>
<td>312.00 m a.s.l.</td>
<td>291.40 m a.s.l.</td>
<td>Unusual</td>
<td>HQ5</td>
</tr>
</tbody>
</table>

6.8.6.6 Water Hammer Loads

Not considered.

6.8.6.7 Uplift Pressure

The water level in the ground (uplift, groundwater level) shall be chosen considering the water levels as indicated chapter 6.8.6.1. It shall be taken into account that these water pressures do fluctuate slower than the water pressures in the river. A water pressure rise due to dynamic actions (e.g. Westergaard) is not considered.

A drainage gallery is foreseen along the RCC Closure Structure, the Powerhouse and the Spillway. A grout curtain can be constructed from this gallery if required. A grout curtain without gallery is foreseen at the upper part of the Navigation Lock towards the left bank.

For all structures an uplift water pressure reduction of 35% can be assumed at the drainage gallery for all load cases except where permanent sliding occurs. In a separate load case, a failure of the uplift water pressure reduction system shall be investigated.

The uplift pressure at the Navigation Lock shall consider the waterpressure from the hillside and possible drainage layers in the Navigation Lock backfill.
6.8.6.8 Water Pressure in Cracks and Pores

Water pressure in cracks or in concrete pores shall be considered for all load cases for walls subject to water pressure on one or both sides. An increased water pressure in the crack due to short-term crack opening for example in seismic load cases can be neglected.

Figure 6.8-2: Water pressure in concrete cracks and concrete pores

6.8.7 Seismic Loads (E)

6.8.7.1 Seismic Design Values

The below indicated seismic design values and acceleration response spectra shall be used for the Luang Prabang HPP. The values originate from the Seismic Hazard Assessment Report. The seismic design approach as described in Chapter 5 and the seismic load combination according to 5.6 shall be taken into account.
### Table 6.8-10: Seismic design values

<table>
<thead>
<tr>
<th>Design Earthquake</th>
<th>Return Period</th>
<th>Exceedance Probability</th>
<th>PGA [g]</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>SEE</strong> Safety Evaluation Earthquake</td>
<td>10,000 y (PSHA) --- (DSHA)</td>
<td>1% in 100 years Not applicable</td>
<td>0.49 0.40</td>
</tr>
<tr>
<td><strong>MDE</strong> Maximum Design Earthquake</td>
<td>2,475 years</td>
<td>2% in 50 years</td>
<td>0.37 0.30</td>
</tr>
<tr>
<td>structural design</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>MDE</strong> Maximum Design Earthquake</td>
<td>950 years</td>
<td>10% in 100 years</td>
<td>0.29 0.23</td>
</tr>
<tr>
<td>stability analysis</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>DBE</strong> Design Basis Earthquake</td>
<td>475 years</td>
<td>10% in 50 years</td>
<td>0.23 0.18</td>
</tr>
<tr>
<td><strong>OBE</strong> Operating Basis Earthquake</td>
<td>145 years</td>
<td>50% in 100 years</td>
<td>0.13 0.10</td>
</tr>
<tr>
<td><strong>CE</strong> Construction Earthquake</td>
<td>50 years</td>
<td>63% in 50 years</td>
<td>0.06 0.04</td>
</tr>
</tbody>
</table>

The below figure and table shows the final site specific acceleration response spectra curves based on PSHA (OBE, DBE and MDE) and DSHA (MCE) methods with 5% damping.
Figure 6.8-3: Final site-specific acceleration response spectra (5% damping, rock surface) from seismic hazard assessment - graph
Three scaled accelerograms (time histories) for OBE, DBE and SEE are available with the seismic hazard assessment.

### 6.8.7.2 Inertia Force due to Mass of Structure

For the structural analysis, the inertial force due to the mass of the structural wedge shall be considered calculating the total design base shear force.

\[
F = k_h \cdot W
\]

- \( k_h \): Seismic coefficient (see Chapter 5.5)
- \( W \): Gross weight of structural wedge (including soil above the heel and toe and water contained within the structure)

### 6.8.7.3 Effects of Water

The hydrodynamic forces are based on the Westergaard method and act additionally to the hydrostatic water forces. As indicated in Figure 6.8-5, the hydrodynamic forces act on all submerged surfaces of a structure but not along foundations or backfilled walls. The resultant force, acting on 0.4\( \cdot h \) above the ground surface, is calculated as follows:

- \( F_w = \frac{7}{12} \cdot k_h \cdot \gamma_w \cdot h^2 \)
For the analysis with the acceleration response spectra and time history, the hydrodynamic forces on the structure can be modelled with resonating water masses according to Westergaard added mass.

\[
m_{wi} = \frac{7}{8} \cdot \rho_w \cdot \gamma_w \cdot h_w \cdot \sqrt{1 - \frac{h_i}{h_w} \cdot \Delta h_i}
\]

For limited reservoir width like for example in the Navigation Lock, the dynamic water pressure according to Westergaard can be reduced. The below table shows the reduction factor \( b \) according to Brahtz and Heilborn.

6.8.7.4 Inertial Effects of Soil

Dynamic backfill pressures are related to the relative movement between the soil and the structure and the stiffness of the backfill. Depending on the behaviour of the backfill, the following forces shall be applied:
a. Non-yielding backfill

For deformations of the structure where the backfill does not yield (see also chapter 6.8.5.1), the dynamic soil pressures and associated forces in the backfill can be assumed according to Wood’s method. \( F_{sr} \) is assumed to act at a distance of 0.63h above the base of the structure.

\[
F_{sr} = \gamma \cdot h^2 \cdot k_h
\]

\( F_{sr} \) Lateral seismic force representing dynamic soil pressure effects

b. Yielding backfill

The earth pressure loads due to the earthquakes where the relative movements of the structure and the backfill are large enough, the Mononobe-Okabe method will be applied. The total active force from the Mononobe-Okabe wedge, based on the below shown earth pressure coefficient formula is divided into a dynamic component \( D_{PAE} \) whose resultant is placed at the upper one-third point and the static component \( P_A \) based on the active earth pressure.

The value of \( K_{AE} \) will be verified for its plausibility.

\[
K_{AE} = \frac{\cos (\phi - \psi - \theta)}{\cos \psi \cos \theta \cos (\psi + \theta + \delta) \left[ 1 + \frac{\sin (\phi + \delta) \sin (\phi - \psi - \beta)}{\cos (\beta - \theta) \cos (\psi + \theta + \delta)} \right]^2}
\]

Figure 6.8-7: Seismic earth pressures (from USACE EM 1110-2-2100)

6.8.8 Wind Loads (W)

Wind loads are not relevant for the current design stage of the concrete structures.

6.8.9 Temperature Loads (T)

Temperature loads are not relevant for the current design stage.

6.8.10 Rainfall Intensity

The rainfall parameters used to design the storm water drainage will be determined in a future design stage.