THE STUDY OF THREE-DIMENSIONAL STABILITY FOR A COMPLEX ARCH DAM ABUTMENT

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ABSTRACT

Abutment stability is obviously fundamental to the feasibility of an arch dam and an arch type structure will impose higher structural loads than any other dam type on its abutments. Consequently, an arch dam design must always include an evaluation of the stability of both abutments under the various important applicable loads. For a complex abutment rock mass, with various dominant joint sets, the importance of the abutment analyses increases and the abutment must consequently be defined as an appropriate series of rock wedges, the stability of each of which must independently and collectively be established.

In this paper, the authors describe a particular analysis relating to a complex abutment rock mass for a 275 m high double curvature arch dam. Subsequent to the analysis, rock mass relaxation related to the joint sets analysed resulted in a change in the abutment design and configuration. While the related issues are consequently only of academic interest, it is, however, considered of value to present the evaluation process followed as an example of an appropriate 3-dimensional discontinuum analysis of a jointed arch dam abutment rock mass of a super-high arch dam using a finite element analysis model.

1. BACKGROUND

The dam under study is a 275 m high, double-curvature, conventional concrete arch dam, with a crest length of 500 m. The dam arch structure will contain approximately 2.5 million m$^3$ of concrete.

A comprehensive geotechnical model of the dam foundation rock mass was established through extensive investigations, primarily undertaken during the early stages of the construction contract. The rock mass model revealed a series of unfavourably orientated joint sets on the upper left abutment, for which it was considered that a discontinuum analysis was required, reviewing the stability of a series of associated rock wedges specifically under the thrusts to be imposed by the dam structure during operation.

2. INTRODUCTION

The structural integrity of an arch dam is determined by its ability to transfer the load from the reservoir it impounds into the foundation upon which it is constructed. When loaded, the arch structure develops structural compressions that are transferred into thrusts on the supporting abutments.

Whereas a gravity dam will transfer load directly downwards into its foundation, an arch structure transfers a substantial part of the load forces laterally, consequently resulting in higher levels of concentrated thrusts on the abutments. Furthermore, the thinner section of an arch implies a smaller dam/foundation contact area over which the thrusts are applied and correspondingly the stability of the arch abutments under load become a critical factor in the overall integrity and safety of the structure-foundation system.

With extensive jointing and certain joint sets daylighting at various points on the left abutment slope, a detailed analysis of the stability of the abutment and the associated rock mass joint wedges under load was consequently considered of particular importance. Although subsequent developments resulted in a substantial design change for the left abutment, and consequently the associated issues are only of
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academic interest, the authors present in this paper the basic methodologies and analyses applied to quantitatively review the associated dam abutment stability. In addition, the initial measures considered for increasing the rock mass shear resistance are discussed.

Figure 1 illustrates the basic extent of jointing in the foundation rock mass of the dam.

![Figure 1. Joints and fault lines within rock mass foundation](image)

### 3. ANALYSIS METHODOLOGY

#### 3.1 2-Dimensional Stability Analysis

The 2-Dimensional stability analysis of a rock wedge, on a sliding slope, under gravity conditions is a relatively simple exercise. A sliding FoS is determined as the ratio of the resisting force capacity along a selected slope to the applicable driving force along the same slope. The indicated sliding FoS is compared to the associated sliding stability criteria requirements, as applicable under the particular loading condition.

A simple illustration of the forces acting on a rock wedge analysed in 2-D is provided in Figure 2. The FoS is calculated as indicated in the following equation:

\[
FOS = \frac{cl + W \cos \psi \tan \phi}{W \sin \psi}
\]

Where:
- \( c \) = cohesion (kPa)
- \( l \) = length of joint (m)
- \( W \) = weight of wedge (kN)
- \( \psi \) = angle of slope (°)
- \( \phi \) = internal friction angle at joint (°)
3.2 3-Dimensional Stability Analysis

To calculate the FoS in 3-Dimensions all three vector components of force on the relevant planes in contact with the rock wedge need to be considered. For each contact plane one of the vector components will act normal to it and the other 2 tangential to it. The 2 components tangential to the plane can be further simplified to a resultant acting on the plane.

The planes in contact with the rock wedge can provide vertical or lateral support to the rock wedge. Planes that experience tensile stress become pull-off planes and should be modelled as such. Planes in compression are sliding planes and are considered to provide stability.

For a rock wedge, the most critical failure mode needs to be postulated and analysed. The most critical failure mode is considered that with the lowest FoS. This mode is defined on the sliding plane in the direction of the resultant tangential force acting on it. The sliding plane needs to dip out of the canyon slope and daylight to make sliding failure physically possible.

For a rock wedge to fail it would need to disconnect from pull-off planes and move along the sliding plane. The various vector component forces acting on the contact planes are either driving or resisting movement of it.

The equation used to determine the sliding FoS is provided

\[
FoS = \frac{\text{Resisting Force at Joint}}{\text{Driving Force a Joint}}
\]

\[
FoS = \frac{F_z \cdot \tan \phi + c \cdot A}{\sqrt{F_x^2 + F_y^2}}
\]

Where

- \(F_z\) = Force normal to sliding plane (MN) (resisting force)
- \(\Phi\) = Angle of internal friction (°)
- \(c\) = Cohesion strength of joint interface (MPa)
- \(A\) = Area of sliding plane (m²)
- \(F_x\) = Force tangential to sliding plane in parallel to dip direction (MN)
- \(F_y\) = Force tangential to sliding plane perpendicular to dip direction (MN)
A localised coordinate system is defined on the sliding plane. Trigonometry can be used to determine the direction of the failure mode in the sliding plane relative to the dip direction (local x-direction).

\[ \theta = \tan^{-1}\left(\frac{F_y}{F_x}\right) \]

To determine the forces exerted on the foundation rock wedges by the dam under load, the Finite Element Method (FEM) can be used. Results of these complex analyses are post-processed, allowing the thrust forces exerted by the dam on each specific wedge to be extracted for comparison with the shear resistance capacity of each particular wedge in the applicable direction of movement.

### 3.2.1 Modelling and Analysis of Jointed Rock Mass

A detailed and comprehensive geotechnical rock mass model allowed an accurate representation of the affected foundation rock mass to be developed for analysis. The dam and foundation were modelled with 3-Dimensional solid quadratic tetrahedral elements, using Midas FEA software. Distinct joint planes were defined in the FE model creating an accurate discontinuum model of the actual series of rock wedges that comprise the left abutment of the dam site. Failure modes were postulated for each of the relevant rock wedge sets.

The complete formation of the numerous possible rock wedges created by the identified joints on the left bank of the dam site is illustrated in Figure 3 and Figure 4. Each set and combination of wedges were evaluated in respect of sliding and release planes and in relation to the direction of the thrust forces imposed by the arch structure. For a rock wedge to fail in sliding along a sloped plane, the wedge obviously needs to be kinematically free, meaning the joint creating the sliding plane would need to “daylight” on the abutment slope. Considering the potential failure mechanisms and modes, scenarios were formulated of the various critical wedge combinations and the associated surfaces on which these might be displaced under load. While numerous wedge combinations were analysed, for purposes of this publication only the critical rock wedge system and the associated failure mode are used to illustrate the analysis procedure.

![Figure 3. Aerial view of various possible rock wedge formations due to joints on the left abutment](image)
The rock wedge formation illustrated in Figure 5 indicates a failure mode incorporating wedges W01 and W02. In this case, joint planes dipping out of the canyon slope act as the sliding planes, whilst sub-vertical joint sets dipping steeply upstream act as the associated release, or pull-off planes. As mentioned earlier, for practical purposes pull-off planes were given zero tensile strength.

Figure 6 presents an illustration of the potential failure plane beneath wedges W01 and W02, as well as the associated release planes and the critical potential movement direction under normal dam and gravitational loads.
4. SHEAR STABILITY EVALUATION

4.1 Analysis Parameters

The dam and foundation FE model was given linear elastic material properties, as indicated in Table 1. In the basic analysis completed, the dam was loaded under static normal loading conditions including hydrostatic, gravity, silt and drained uplift. Should the associated dam arrangement have been taken forward, the subsequent stage of analysis would have reviewed additional loadings, including the critical seismic loadings.

The rock mass and joint properties, as determined through the geotechnical investigations, are also indicated in Table 1.

<table>
<thead>
<tr>
<th>Parameter (symbol)</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dam concrete unit weight (γ)</td>
<td>24 kN/m³</td>
</tr>
<tr>
<td>Dam concrete elastic modulus (E)</td>
<td>23 GPa</td>
</tr>
<tr>
<td>Dam concrete poisson value (ν)</td>
<td>0.2</td>
</tr>
<tr>
<td>Foundation unit weight (γ)</td>
<td>27 kN/m³</td>
</tr>
<tr>
<td>Foundation elastic modulus (E)</td>
<td>6.5 GPa</td>
</tr>
<tr>
<td>Foundation poisson value (ν)</td>
<td>0.29</td>
</tr>
<tr>
<td>Foundation discontinuity internal friction angle (ϕ)</td>
<td>45°</td>
</tr>
<tr>
<td>Foundation discontinuity cohesion value (c)</td>
<td>100 kPa</td>
</tr>
</tbody>
</table>

4.2 Implementation of Analysis

The finite element model was used to extract the resultant thrust forces on the applicable wedges and the normal forces on the applicable shear planes, post-processing the associated shear over the full contact area. Subsequently, the associated shear resistance capacity was derived at the same locations and in the resultant direction of the destabilising force. The destabilising forces and the resistance
capacities were subsequently summed to allow a simple sliding stability calculation for the full applicable wedge system.

5. SHEAR STABILITY ANALYSIS RESULTS

The analysis results and associated FoS’s for the critical rock wedges W01 and W02 are provided in Table 2.

Table 2. Stability analysis results for rock wedges

<table>
<thead>
<tr>
<th>Wedge No.</th>
<th>Failure Mode No.</th>
<th>Resisting force (N)</th>
<th>Driving force (N)</th>
<th>Angle of driving force to dip direction (°)</th>
<th>FoS</th>
</tr>
</thead>
<tbody>
<tr>
<td>W01</td>
<td>1</td>
<td>1.23E+10</td>
<td>7.20E+09</td>
<td>25.10</td>
<td>1.71</td>
</tr>
<tr>
<td>W02</td>
<td>2</td>
<td>1.31E+10</td>
<td>7.36E+09</td>
<td>35.34</td>
<td>1.78</td>
</tr>
</tbody>
</table>

When reviewing the FoS values for the wedges (W01 and W02) formed, it is important to consider that the load case applied for analysis was a normal, static, operational loading condition for which a minimum sliding factor of safety of 2 would be the design criterion, assuming accurately defined joint shear strength.

As is apparent, the analysis indicated that the target FoS values were not achieved for the critical wedge combination, implying that measures would be required to enhance the stability of the upper left abutment rock mass.

6. EVALUATION OF TYPICAL STABILISATION MEASURES

6.1 Stabilisation Requirements

To ensure stability of the identified rock wedges on the left abutment, the analyses indicated that measures were required to increase the critical FoS against sliding from approximately 1.7 to 2.0.

Due to the steepness of the upper left flank at the dam site and the general local topography, the only available and practical opportunities to increase FoS values were through active cable anchoring and/or the construction of shear keys across the problem shear plane.

6.2 Stabilisation of Wedges with Pre-stressed anchors

The anchorage system applied on site for rock cut stabilisation used post-tensioned cable anchors with a maximum individual anchor force of 1.8 MN. To stabilise the applicable wedges using such anchorage tendons, proved impractical. This is due to the required unrealistically close anchor spacing, as indicated in

6.3 Stabilisation of Wedges with Shear Keys

A more appropriate stabilisation solution would be to construct shear keys across the joint, to increase the total shear resistance. Such a solution would involve concrete-filling tunnels excavated through the joint on an alignment perpendicular to the sliding plane dip direction. Subsequent movement on the problem sliding plane would necessitate shear failure through the concrete mass, which indicates substantially higher shear strength than the joint plane.

The most practical approach for construction is to follow the strike of the fault line, allowing horizontal tunnel excavation.

Table 3. Furthermore, the anchor lengths required to intercept the problem planes are excessive, while the practicality of arranging necessary anchors perpendicular to the problem plane would also be unreasonable.
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Table 3. Required anchors for Rock Wedge Stabilisation

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Anchorage Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wedge No.</td>
<td>W01</td>
<td>Required Normal Force from Anchors</td>
<td>2100 MN</td>
</tr>
<tr>
<td>Interface Area</td>
<td>18718 m²</td>
<td>No of Anchors Required</td>
<td>1167</td>
</tr>
<tr>
<td>Driving Force</td>
<td>7200 MN</td>
<td>Area per Anchor</td>
<td>16.04 m²</td>
</tr>
<tr>
<td>Resisting Force</td>
<td>12300 MN</td>
<td>Applied Pressure</td>
<td>0.11 MPa</td>
</tr>
<tr>
<td>Required FoS</td>
<td>2</td>
<td></td>
<td>2</td>
</tr>
<tr>
<td>Current FoS</td>
<td>1.71</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

For the preliminary analyses, the size, shape and arrangement applied for a similar fault treatment at a similar large dam were assumed, as illustrated in Figure 7. Applying a conservative approach, a shear surface width of 3 m was assumed for this configuration, together with a maximum total concrete shear strength of 2.5 MPa.

A typical consolidation grouting arrangement for the specified fault treatment shear key is indicated in Figure 8. Consolidation grouting of the rock mass immediately around the shear tunnels would prevent localised rock failure due to fissured or fractured rock in vicinity of shear key. Annulus grouting will also be necessary to accommodate post-hydration thermal shrinkage of the concrete.
The absolute shape and dimensions of the sliding plane of the wedge would finally govern the setting out of the fault treatment shear key, while the full required length of shear key would not be achieved using a single row. Multiple rows of shear keys, running approximately parallel to each other, would accordingly be applied, with each constructed and aligned to intercept the fault surface as centrally as possible and thereby to develop the maximum possible shear resistance.

Figure 9 provides an illustration of the typical envisaged arrangement of the shear key system. The shear key system is joined by a central spine gallery, running perpendicular to the alignment of rows to allow for drainage and to create additional tunnelling faces to expedite construction. This gallery would also assist in tracing the shear surface alignment. For purposes of illustration, this gallery is aligned perpendicular to the shear key galleries.

It was envisaged that each tunnel would daylight at both ends, but it is not certain how much bench excavation and excavation of weathered surface material would be necessary and accordingly, a conservative approach was taken by limiting the lateral extent of each tunnel row.
7. CONCLUSION

The structural stability of an arch dam foundation rock mass is imperative to ensure the overall stability of the dam structure. The analysis and stabilisation design of complex problematic rock mass formations in 3-Dimensions is not possible using hand calculations or simple 2-Dimensional methods.

The analysis of such complex 3-Dimensional solid structures is only possible using the FEM. In the study discussed in this paper, a 3-dimensional FE system was used to produce force and stress information on specifically identified discontinuities, subsequently allowing a post-processing summation of total destabilising forces and applicable shear resisting capacity. Consequently, an overall factor of safety could be derived through a simple calculation. With this technique, the level of stability of a complex network of rock wedges could be quantitatively determined and compared with the project design criteria.

A FE model with comprehensive interface joint element modelling capabilities can greatly assist in the analysis of complex foundation rock masses and in the design of any necessary stabilisation measures.

8. REFERENCES