STATE OF THE ART 3D STRESS AND STABILITY ANALYSIS OF A GRAVITY DAM

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ABSTRACT

The objective of this paper is to present a 3D stress and stability analysis of Powell Lake Dam that was initially analyzed using traditional two-dimensional stability analysis. The 2D stability analysis found that numerous blocks in the dam were unstable under the flood and seismic load conditions. After further review of the initial 2D stability assessment, it was concluded that consideration of 3D behavior could reduce anchor requirements. A non-linear three-dimensional finite element model of the entire concrete dam was developed. The static analysis was performed using the ANSYS FEA program. The model used surface contact elements at vertical contraction joints in the concrete dam. These contact elements allow relative opening/closing and sliding along these joints. In addition, similar contact elements were used at the concrete/rock interface. The model allows non-linear redistribution of forces due to opening/closing and sliding at these interface elements. The stability of the dam was evaluated by gradually lowering the shear strength at the concrete/rock interface. An automated procedure was developed to iterate the uplift pressure as the dam/rock interface joint opened. The seismic analysis was performed using the LS-DYNA FEA program. The foundation mass was included in the analysis. The mass foundation allows for the dam-foundation interaction (i.e., radiation damping, etc.) to be properly accounted for in the analysis. However, the seismic input in mass foundation requires special analysis procedures. The earthquake input was applied using the Lokke and Chopra (2019) method in domain reduction framework (Bielak and Christiano, 1984). The damping in the FEA model was estimated using the half bandwidth method.

1. INTRODUCTION

The Powell Lake Hydroelectric facility consists of a 55 ft. high, 600 ft. long concrete gravity dam with 19 radial gated spillway bays across its crest and a forebay/intake structure on the left abutment (looking downstream). The intake structure provides water to a conduit system consisting of four steel penstocks, surge tanks and a manifold system which distributes flow to three powerhouses at tidewater containing five generating units.

The 2D stability analysis found that numerous blocks in the dam were unstable under the flood and seismic load conditions. After further review of the initial 2D stability assessment, it was concluded that consideration of 3D behavior could reduce anchor requirements. Note that the traditional rigid body 2D stability analysis for gravity dams does not consider the interaction between adjacent monoliths, and therefore it may underestimate the factor of safety against sliding and overturning. Also, a linear 3D analysis does not properly re-distribute loads between various monoliths due to the fact that the vertical contraction joints are not allowed to open and close, i.e., the dam behaves like a fixed-end beam spanning the valley because unrealistic tensile bending stresses can develop across contraction joints. The actual overall dam stability of a concrete gravity dam can be increase when the arching of loads between the adjacent monoliths are considered in the analysis. If the adjacent monoliths have greater sliding resistance available compared to the block under consideration, the friction forces which develop at the vertical contraction joints can be relatively large. However, the re-distribution of forces due to frictional behavior is highly nonlinear. Under extreme cases, i.e., earthquake or PMF loading, some monoliths may undergo a relative displacement or slide a small amount, as horizontal loads are re-distributed. The horizontal driving forces in the flow direction are re-distributed between adjacent blocks as some blocks offer more resistance to sliding than other blocks. The additional sliding resistance may come from embedment, a rough concrete/rock interface, and having a supporting structure downstream of the dam/intake i.e., powerhouse. Thus, using the traditional methods it is not possible to estimate the sliding factor of safety if the non-linear force re-distribution is ignored.

2. FINITE ELEMENT MODEL

A three-dimensional finite element model of Powell Lake Dam was created for nonlinear static and dynamic analysis with ANSYS & LS-DYNA programs, respectively. Figure 1 presents the Finite Element Model of Powell Dam.

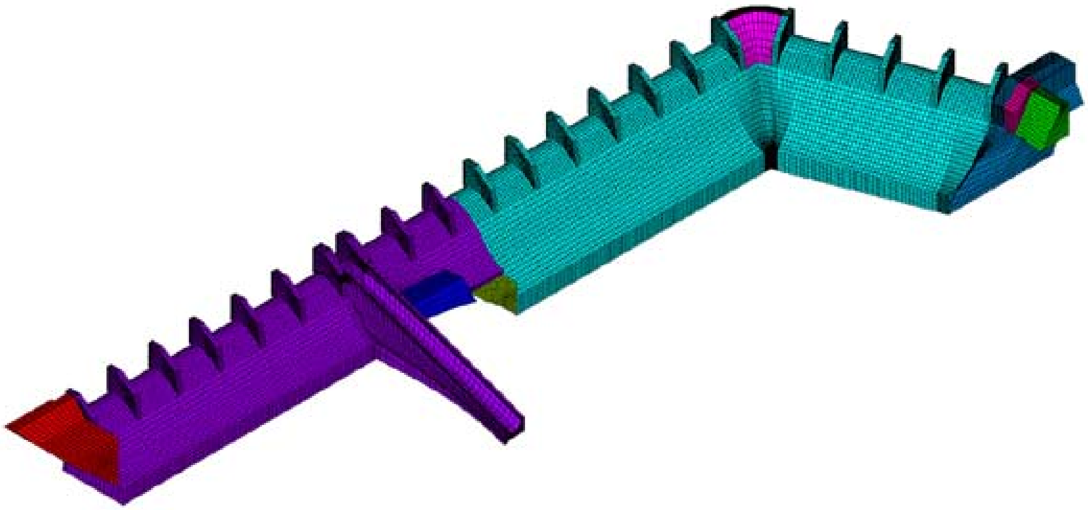


Figure 1 : Finite Element Model of Powell Dam

During a review of historical drawings and our field inspection, it was not possible to ascertain the location of vertical contraction joints i.e., repaired surfaces hindered these observations. However, it should be noted that during the 1910's the concrete strength gain was very slow and batch plant capacities were low during this time period. Therefore, high concrete temperatures were not anticipated during the construction. Construction was done in small blocks in an interlocking layout. Therefore, in the initial analysis only three vertical contraction joints were assumed. Figure 2 presents the selected location of the vertical contraction joints. However, an additional sensitivity analysis was also performed with additional vertical contraction joints as shown in Figure 3.

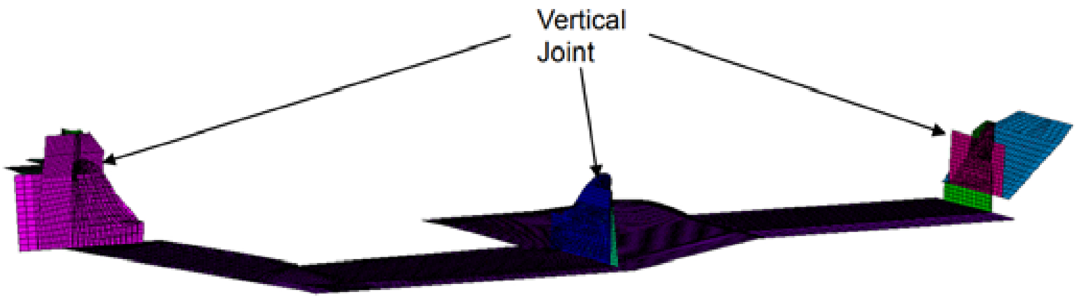


Figure 2 : Selected Vertical Contraction joints at Powell Dam

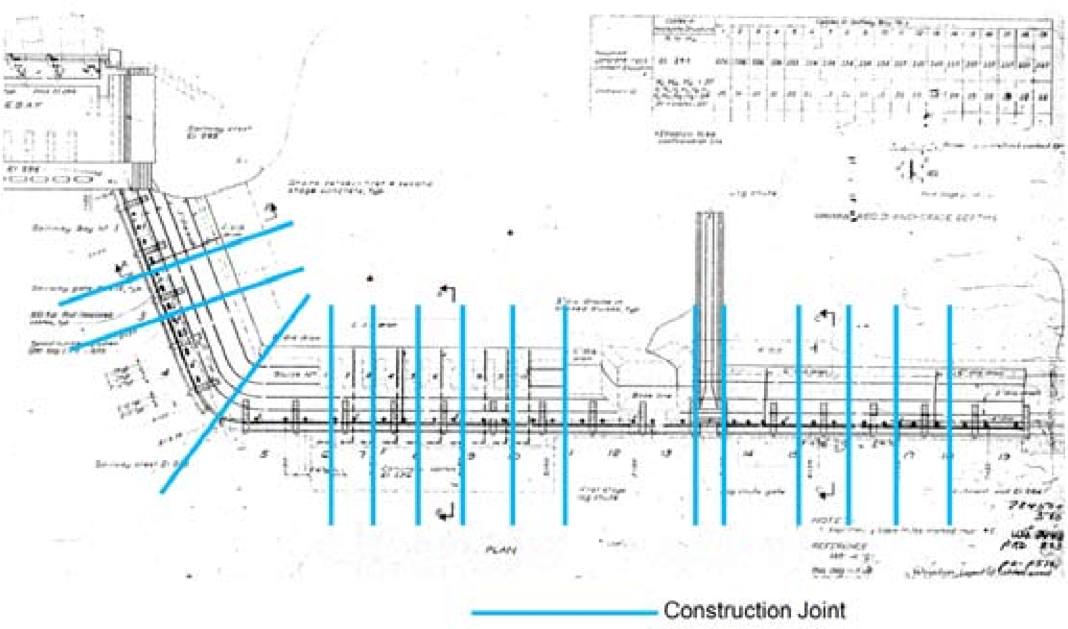


Figure 3 : Selected Location of Potential Construction Joints

The vertical construction joints were modeled with a 45º friction angle. The dam-foundation CRI contact was modeled with a 57º friction angle. The analysis was performed with 5% damping. However, it is almost impossible to impose 5% damping on the modal frequencies in a nonlinear seismic analysis. Therefore, it is important to estimate damping in the model associated with the natural frequency of the dam. The half power bandwidth was used to determine the damping in the model. Figure 4 presents the estimated damping, i.e., 3.83% in the system. The estimated damping of 3.83% is close to the target damping of 5%.

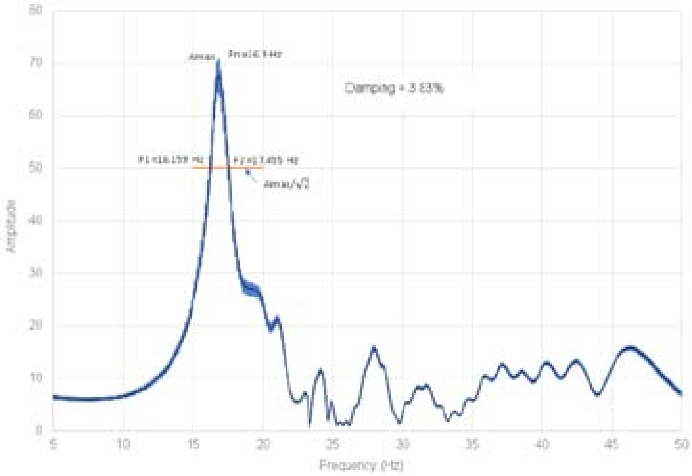
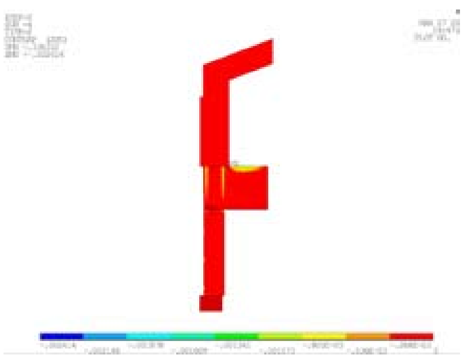


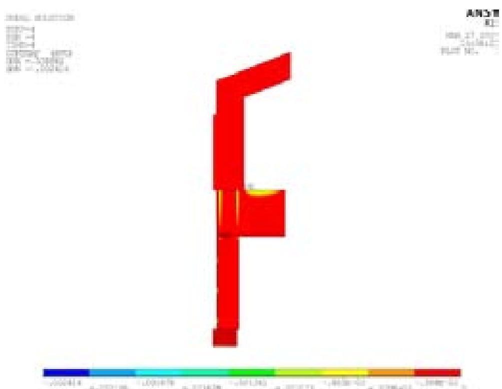
Figure 4 : Estimated Damping for Powell Dam Model

3. STATIC ANALYSIS

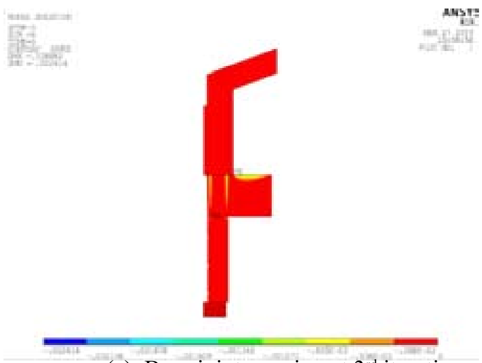
Uplift pressure was included for static and dynamic analysis load cases. Linearly varying uplift pressure from headwater to tailwater was applied at the dam-foundation interface. A base cracking was also performed for normal and flood load cases. The base cracking analysis was performed in an iterative manner. Similar to a conventional gravity analysis, whenever the base cracking is indicated by the presence of tensile stress at dam-foundation in the interface elements, the uplift distribution was modified as per CDA guidelines. However, since contact elements are allowed to open and close with no tensile strength assigned at the dam foundation joint, the uplift was adjusted based on the opening of the dam foundation joint. The uplift at a node at the dam foundation joint was modified if the opening is more than 0.035 mm. Uplift was iterated until the dam foundation joint opening had stabilized. Macros were developed within ANSYS program to automate the iteration process. Figure 5 shows the opening joint of dam base during the iterations.



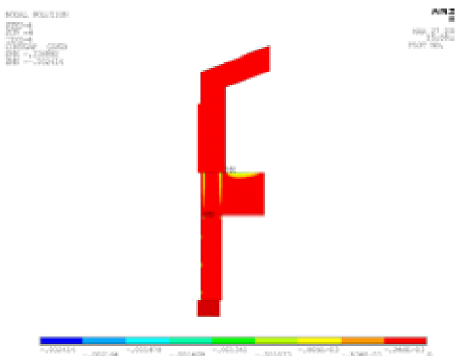
(a) Base joint opening at 1st iteration



(b) Base joint opening at 2nd iteration



(c) Base joint opening at 3rd iteration



(d) Base joint opening at 4th iteration

Figure 5 : Open Joint Development at Dam/Foundation interface in 4 iterations

The dam safety criteria were checked on three main aspects:

- The dam blocks were safe against sliding along one or more planes..
- The dam is safe against overturning
- The stresses in the body of the dam or in the foundation should not exceed the allowable values.

For a concrete gravity dam the blocks hold themselves in place by gravity and frictional forces. Therefore, the inclusion of frictional surface contact elements is well suited for this analysis. The sliding and overturning stability is easily checked, i.e., if the analysis converges and dam blocks do not overturn, then the overall resultant force must be within the dam base. Similarly, the dam safety against sliding is also easy to check if the computed downstream sliding movements are small. If the sliding displacements exceed the allowable value, the dam is considered to be unsafe. The dam-foundation contact friction angle was reduced to evaluate the stability for given factor of safety values.

In traditional 2D analysis, the Factor of safety of each block is calculated individually without taking into account the friction force in vertical joints provided by adjacent blocks. The equation can be expressed as:

$$FS = R/S \quad (1)$$

Where R is the resisting force provided by the friction at the dam base, S is the driving force, which causes sliding. The resisting capacity of each block can be evaluated at the dam base, since both the normal force (self-weight and vertical forces acting on the block) and the friction coefficient are known.

In a 3D non-linear model, the external load S is distributed to both the dam base and the vertical contraction joints, i.e., adjacent monoliths. If the load S increases as a result of higher water levels or seismic loading, then arching of loads to stronger monoliths would increase the resistance R. The increased portion of load S is redistributed to the monolith base and the adjacent monoliths. The sliding resistance at the dam base cannot be estimated directly. In the present analysis, this problem is solved by evaluating if the dam is stable for the reduced friction angle (θ_{red}) for a given FS value (Curtis et al, 2003). Thus

$$\theta_{red} = \text{tan-1} (\tan \theta_{base} / FS) \quad (2)$$

The analysis is performed for the reduced friction angle to check if the dam is stable for the given load case. For NOC and PMF load cases the dam-foundation contact friction angle was reduced from 57º to 45.7º and 49.8º, respectively, i.e., for given factor of safety values of 1.5 and 1.3, respectively. The observed base displacements for reduced dam-foundation contact friction angle were small and acceptable. Thus, the dam was stable under NOC and PMF load cases. Figure 6 presents the displacements (mm) in stream direction (x-direction) for PMF load case. From Figure 6, the observed displacements for PMF load case at the end of uplift iterations were small and acceptable.

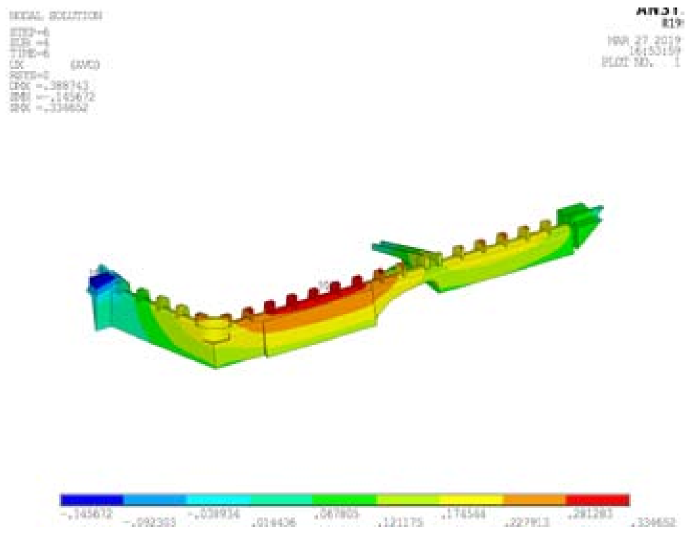


Figure 6 : Displacements (mm) in stream direction (x-direction) for PMF load case.

Figure 7 presents the longitudinal stresses at Block 6. From Figure 7, a maximum -0.2 MPa longitudinal stress can be observed. Note this small longitudinal stress shown in Figure 7 significantly affects the stability of the dam, as small stresses over a large area result in a considerably large stabilization force.

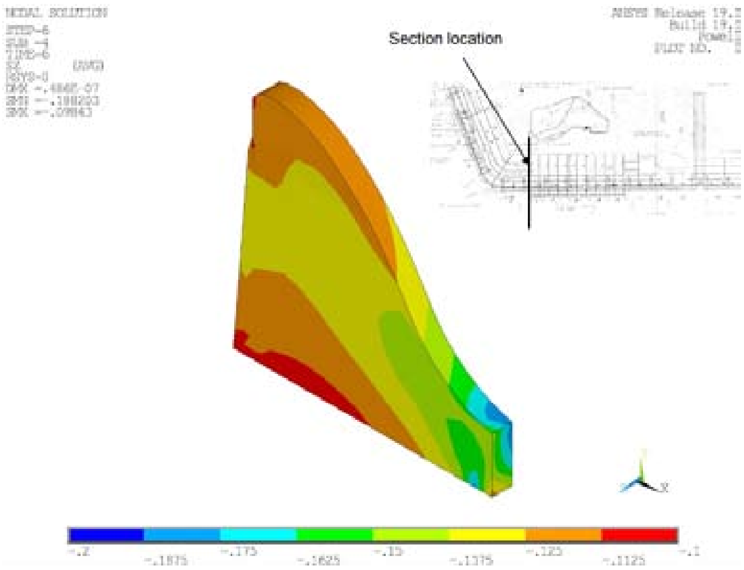


Figure 7 : Longitudinal Stress (Mpa) at Block 6

4. DYNAMIC ANALYSIS

The dynamic analysis was performed for ½ between the 2,500 and 10,000 year return earthquake period. The earthquake input was applied using the sub-structuring approach (Bielak and Christiano, 1984). Figure 8 shows the systematic diagram showing the process used for the Powell Dam analysis. First, a large foundation model (shown in Blue color) was prepared which included the smaller foundation (shown in Red color) to be used for separate seismic analysis in which the dam was included. The PML elements (shown in green) were applied to boundary. PML elements are similar to standard non-reflecting Lysmer elements. However, the PML elements are much more effective compared to Lysmer elements. Note, no deconvolution was applied during this analysis, as the abutment walls at Powell Dam are far from each other. The deconvolution becomes necessary for a dam analysis where canyon walls are higher in depth and closely situated along the dam, which is not the case here. Therefore, in the large foundation model the earthquake was applied 2 elements below the dam-foundation interface. For the horizontal component earthquake time histories, the nodes in the vertical direction were constrained in the outer foundation part (shown in blue color). Similarly, for the vertical earthquake time history the nodes were constrained in horizonal direction. Analysis was performed for each horizontal and vertical earthquake component separately. The reactions and velocities at the interface of the large foundation (shown in blue) and smaller foundation (shown in red) were recorded for each run. A total of about 19,000 files were generated which contained recorded reactions and velocities. The recorded velocities and reactions were reapplied to a smaller foundation model shown at right in Figure 8. This smaller model included the dam and was used for the seismic analysis.

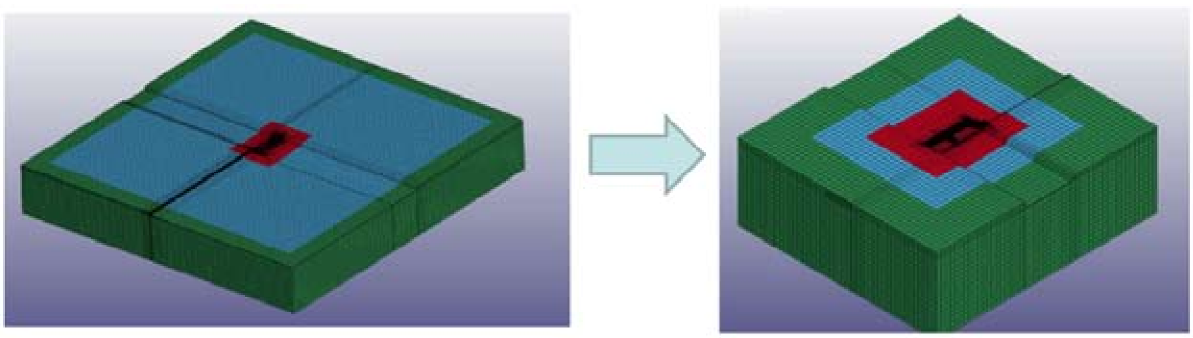


Figure 8 : Earthquake Input Application Approach

Figure 9 shows the comparison of the target and output response spectrum for EQ1 at the dam base. From Figure 9, a very good match has been achieved for target and output response spectrum for EQ1.

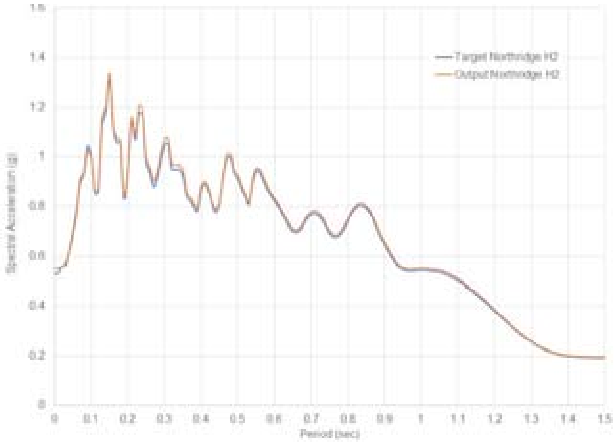


Figure 9 : Comparison of target and output response spectrum for EQ1 at the base

Figure 10 present the time history of dam base sliding displacements at Block 1 for EQ1. From Figure 10, the observed dam base sliding displacements are small and acceptable. Similarly, Figure 11 presents the maximum principal stress (MPa) at T=13.2 seconds. From Figure 11, the maximum tension on the downstream face was 0.35 MPa. A stress concentration at trunnion point was also observed.

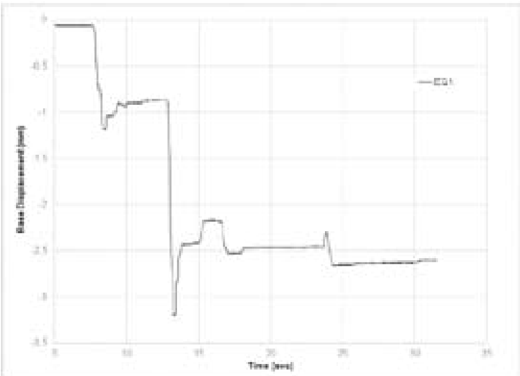


Figure 10 : Time history of dam base sliding displacement at Block for EQ1

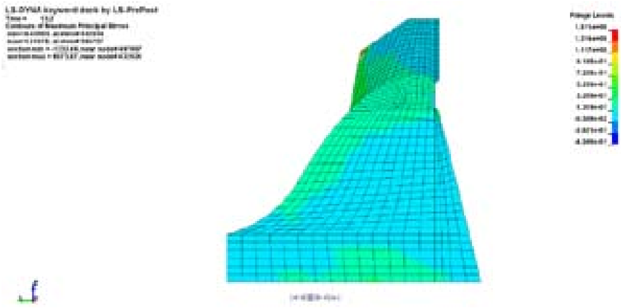


Figure 11 : EQ1: Maximum Principal Stress (MPa) at T=13.2 seconds

5. CONCLUSION

The following conclusions can be drawn from the analysis results:

The dam was found to be stable under static loading including hydrostatic normal operating water level and PMF water levels.

The dam was also found to be stable under the seismic loads which were analyzed with a non-linear earthquake time history analysis.

A sensitivity analysis was also performed assuming additional vertical construction joints. The dam was also found to be stable under the seismic loads with additional vertical construction joints included in the model.

REFERENCES

Løkke, Arnkjell; Chopra, Anil K. (2019),"Direct finite element method for nonlinear earthquake analysis of concrete dams including dam-water-foundation rock interaction", PEER Report 2019/02.

Bielak and Christiano,"On the effective seismic input for non-linear soil-structure interaction systems", Earthquake Engineering and Structural Dynamics, 1984.