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ABSTRACT

During planning for dam projects, good practice usually calls for appropriate investigations to help assure the development is not located on or immediately adjacent to active faults. There are examples of projects, however, where an active fault has been discovered in or close to a dam foundation and engineering solutions have been incorporated into design to enable satisfactory performance of the dam should fault displacement occur during project operation. In some cases, the existence of such hazards only becomes evident late in the design cycle when the dam site is considered fixed and hazard avoidance is not an easy option. The 960-MW Neelum-Jhelum Hydroelectric Project in Pakistan is being constructed within a geologically complex and seismo-tectonically active setting. During feasibility studies, a major thrust fault at the dam site was deemed inactive. However, during detailed design and after commitment to start construction with international contractors, the potentially active nature of the fault came to be understood.

The dam is re-designed as a composite structure, with a zoned fill section overlying the fault and the remainder of the dam consisting of a concrete gravity feature with integral gated spillway. The fill section designed to accommodate the maximum amount of offset that could occur on the fault below. This concept has been adopted on other projects elsewhere, such as the approach dams leading to the new Pacific Locks Complex, which are intersected by segments of the active Pedro Miguel fault for the new Third Set of Locks of the Panama Canal. During construction of the Neelum-Jhelum dam, the actual fault trace was found unfortunately to be not entirely beneath the fill section and that some of the concrete super-structure would overlie the fault. Innovative subsurface foundation treatments have been developed to help direct any potential future fault movement into the fill and away from the concrete part of the dam. This paper describes these measures and the analytical methods used in design development.

INTRODUCTION

The Neelum-Jhelum Hydroelectric Project, located near Muzaffarabad in the state of Azad Jammu Kashmir in eastern Pakistan, is being developed by the Water & Power Development Authority (WAPDA) of Pakistan. The project utilizes a gross hydraulic head of about 430 m by diverting water from the Neelum River (known upstream in India as the Kishaganga River) with a dam and intake works at Nauseri to the lower branch of the Jhelum River through a 32.1-km-long tunnel system and underground powerhouse complex - see Figure 1. The installed generating capacity of the project is 963 MW. Various planning and feasibility studies were completed by about 1995 that were followed by detailed design activities completed by late 1997, each supported by geological investigations.

The project was put on hold for ten years until late 2007 when WAPDA entered into a contract with a consortium of two Chinese companies for construction and supply and installation of equipment. In May 2008, WAPDA appointed a joint venture of consulting companies to serve as the Consultant for design review and the Engineer for the supervision of the Works.

It was soon recognized that the earlier designs had various short-comings. Among these, it was apparent that the seismic design parameters adopted by the earlier designer were too low and a devastating M 7.6 earthquake that afflicted the region on 8 October 2005 confirmed this. The Consultant had to adopt higher, more realistic seismic design parameters resulting in the redesign of many of the main structures. In addition, a major regional fault, known as the Main Boundary Thrust (MBT, sometimes locally named the "Murree Thrust"), was found to pass through the right side of the dam foundation. Although this was known to the earlier consultants, it had been assumed to be inactive and that there was no potential for surface rupture.

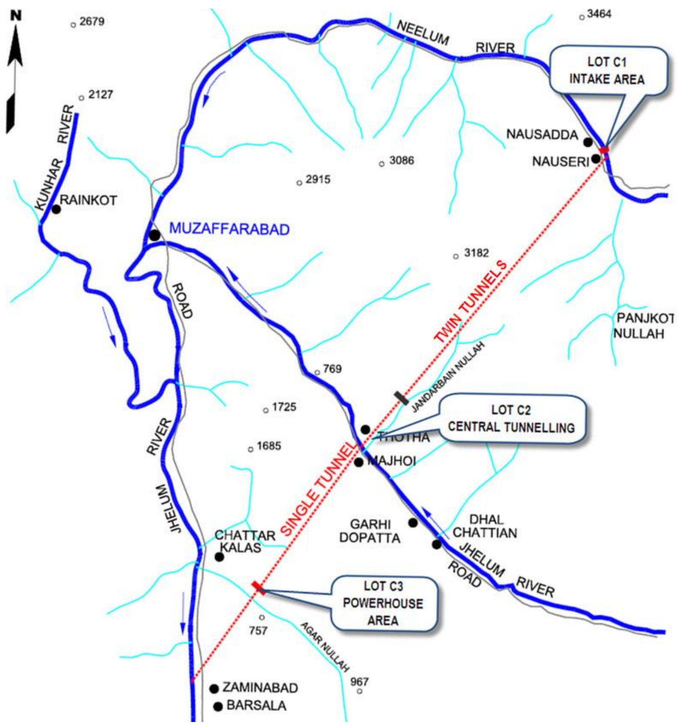

Fig. 1. Location Map of Project Area

MAIN BOUNDARY THRUST (MBT) FAULT

The MBT is a major thrust feature and continental suture of the Himalayan region, extending some 2500 km from Assam in the east to beyond the Indus in the west. In the Dam Site area, it is observed to cut obliquely across the Neelum River on the right abutment of the dam axis. It separates the greenstone of the overlying Panjal Formation from the underlying shales and sandstones of the Murree Formation.

A comprehensive seismic hazard evaluation was conducted by the Consultant (NJC 2010), which included field investigations and desk-top analyses focused on developing seismic parameters for design of the project features and as input for risk analyses. Field studies included review of satellite imagery and aerial photographs, field verification (ground-truthing) of photo-geological interpretations, geological mapping at various scales at the Diversion Dam site (Lot C1) and in C2 and C3 areas, examination of fault traces and ground rupture related to the 2005 earthquake, and mapping along the trace of the MBT. In accordance with international professional practices, deterministic and probabilistic hazard analyses were conducted to develop estimates of earthquake ground motions. Independent technical reviews and peer reviews were carried out by internationally recognized industry experts.

The actual position of the MBT and its physical characteristics were not yet properly defined at the start of construction in 2008. A single exploration hole had supposedly penetrated the fault and had encountered a 6- to 8-m-wide shear zone with brecciated material. Further investigations were carried out in 2008-2011 given the importance of the MBT and the need to characterize it properly. The program included detailed geological mapping and drilling of several cored drill holes in the river valley and in the abutments on both sides of the river.

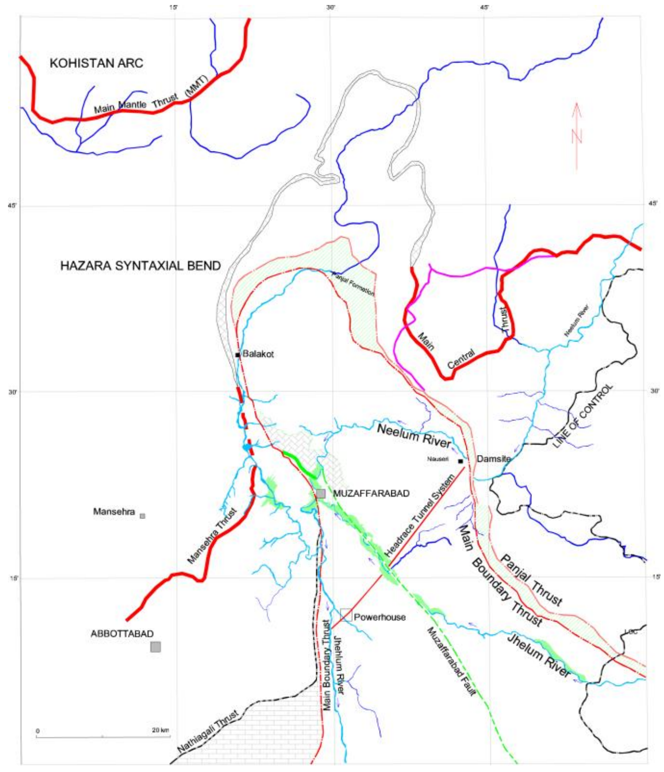

Fig. 2. Regional Fault Map (source: Geol Survey Pakistan)

Determination of whether a fault is active or not cannot be readily or normally ascertained from drill hole data. Rather, it was considered more appropriate to follow current best practices used in neotectonics and paleoseismic investigations elsewhere, including attempting to establish evidence of most recent movement and estimate slip-rate.

In spite of extensive studies, it has not yet been possible to categorically rule out the potential for future movement on the MBT at the dam site, particularly since there has been demonstrable recent displacement on the structure elsewhere in the region. For this reason, it is not only prudent but also in keeping with industry practice to consider this fault as capable of displacement during the lifetime of the project and to design critical project features accordingly. Further paleoseismic investigations might still be able to establish that the fault is no longer active or capable of movement.

The following fault parameters were derived from the investigations (NJC, 2010):

- **Slip rate**: 2.5 mm/yr to 5 mm/yr.

- **Fault Dip**: Regionally known to vary from 50° (from horizontal) to nearly vertical. At the dam site, dip is about 80°-85° to the northeast.

- **Sense of movement**: thrust/reverse with the hanging-wall (up-thrown side) on the northwest or right side of the river valley. The mechanism involves compression and there should be little to no opening or dilation of the fault in the foundation.

- **Maximum displacement**: maximum amount of in-plane vertical movement due to a maximum rupture event is assumed to be about 3 m. There is a slight possibility of some oblique slip, but how much and in what direction is unknown. There is no evidence of coseismic movement on this structure during the 2005 earthquake event, which occurred on the Muzaffarabad Fault (Fig 2).

- **Width**: based on the results of drilling, the width of the rupture zone was thought to be about 3 m, a highly disturbed zone about 10 to 30-m-wide, and the total width of the affected zone perhaps tens of meters. Detailed mapping during foundation excavation has provided better definition of these dimensions (described later in this paper).

CONCEPTUAL DESIGN SOLUTIONS

Various design concepts were evaluated to satisfactorily accommodate displacements of up to the 3 m maximum vertical movement and a minor amount of oblique movement. Structures are also designed for very high ground accelerations: for the MCE event, PGA= 1.16g. The principal solution categories included:

1. Relocation of the dam to another stretch of the river where the dam would not be overlying a fault capable of movement. It may seem difficult to understand why or how a project can continue with an axis intersected by a major fault which might well be potentially active. Information available now would point out that it would be unwise to situate the dam axis at this location and that it should be shifted downstream a short distance such that the MBT could be positioned somewhere high up on the right abutment – where, if it moved, any deformations would not impact the foundation of the dam structure itself. However, major decisions had already been made, including land acquisition, and a serious delay was not tolerable. This is one of those cases where the existence of a major hazard only becomes evident late in the design cycle when the dam site is considered fixed and hazard avoidance is not an option.

2. Provision of an engineered joint in the concrete dam to accommodate movement in the foundation. Design and construction of a concrete gravity dam equipped with a slip joint has been done before, as seen at Clyde Dam in New Zealand and Kárahnjúkar Dam in Iceland. This alternative was dropped due to technical reasons including introduction of an untested slip joint, uncertainties in location of foundation rupture and amount/direction of movement, and need for more space/wider river section to accommodate a longer dam.

3. Design of a composite dam with a zoned embankment section (rockfill with clay core) spanning the fault zone on the right side of the valley and a concrete structure to accommodate the gated spillway and other features. The concept has been adopted for the Third Set of Locks of the Panama Canal where the approach dams leading to the new Pacific Locks Complex are intersected by segments of the active Pedro Miguel fault with the same order of potential rupture as estimated for the MBT on the Neelum-Jhelum project. This option was selected because it provided more flexibility in the face of foundation uncertainties and the fact that there are examples of embankment dams that have performed satisfactorily in spite of considerable foundation movement during earthquake.

DESIGN DEVELOPMENT

The fill section was subsequently designed to accommodate the maximum amount of offset that could occur on the underlying fault. Design details followed previously accepted practices in such situations, including widening the core and flaring the filter zones. Unexpectedly, however, during excavation for the Neelum-Jhelum dam, the actual fault trace was found to be not entirely beneath the fill section and that some of the concrete super-structure would overlie the fault.

Various innovative subsurface foundation treatments were then examined that could help direct any potential future fault movement into the fill and away from the concrete part of the dam. These included: 1). construction of a release plane or preferred plane of slip, such as a slurry wall or row(s) of bentonite drill holes, 2). deeper excavation on the hanging wall side and backfill with low shear strength materials, 3). deeper excavation on the footwall side and replacement with concrete, and 4). combinations of the above. The option to shift the entire dam and appurtenant works further to the right and off the fault was not considered feasible. Of these options, the first was considered the most feasible and realistic given the existing contract provisions without the potential for unacceptable delays and excessive claims.

The objective was to minimize the potential for damage to the concrete structure caused by movement along the fault zone during a seismic event. Therefore treatment was needed along, or near to the fault zone to help direct fault movement away from the concrete structure. The proposed method involved drilling two rows of closely spaced vertical holes beneath the rockfill section of the dam to intersect the fault at a depth of about 25-40 m below the foundation surface. The 10- to 15-cm-diameter holes would be backfilled with bentonite (or bentonite slurry), and the rows of holes would act as a release plane, or preferred slip plane, during a rupture (earthquake) along the MBT. Because of the low permeability backfill, the release plane would not become a line of preferred seepage. The intent is for deformations to be focused along the release plane and into the rockfill section of the dam.

GEOTECHNICAL EVALUATION OF FAULT ZONE

Excavations in the dam area progressed to the foundation level exposing the trace of the MBT along the right abutment of the dam and associated tectonized materials. The entire area was mapped in detail and a geological characterization of the rock units was performed. At the foundation level, the MBT is characterized as an approximately 25-m-wide zone of rock in a varying degree of tectonic disturbance. The fault surface brings into contact the Panjal and Murree Formations and is located about 8 m west of the toe of the right abutment and about 3 m west of the transition between the rockfill and the concrete structure. The MBT strikes approximately north–south and dips steeply 85° to the east. A photograph of the fault zone is depicted on Fig 3.

Fig. 3. Main Boundary Thrust, faulted contact Panjal Fm (left) and Murree Fm (right)

The foundation materials were grouped into geomechanical classes based on geological mapping and other field observations (such as rippability and groutability). A brief description of all the foundation materials near the fault is provided below. The geomechanical classes are shown in the cross section of the 2D *Phase 2* (Rocscience Inc, 2011) model presented in Fig. 4 and are described below.

<u>Disturbed Panjal Formation; Greenstone (PF-D)</u>: Underlies most of the right abutment of the dam (rockfill section); consists of greenstone (altered basic volcanics) and is generally a strong, partially disturbed rock mass with fair to good discontinuity conditions. Locally there is a 2-m-wide zone at the contact with the Panjal Cataclasite material where the rock mass is highly fractured and foliated with poor quality discontinuities. Overall, the Geological Strength Index (GSI) (Marinos and Hoek, 2005), is estimated to range from 25 to 45.

<u>Panjal Formation Cataclasite - Graphitic Schist (PF-C):</u> Located in the hanging wall and is about 9-m-wide. The material is a highly sheared and poorly interlocked rock mass with small blocks of heavily fractured calcareous schist and marble in a matrix of weak friable graphitic schist. GSI estimated to range from 10 to 20.

<u>Murree Cataclasite (MF-C)</u>: Located in the footwall and is about 2-m-wide. It consists of a red-orange siltstone, shale, and occasional sandstone layers of the Murree Formation and is highly sheared with poorly interlocked rock fragments. GSI estimated to range from 15 to 25.

<u>Disturbed Murree Rock Mass - Siltstone – Sandstone Interbeds (MF-D):</u> 12-m-wide zone of interbedded siltstone and sandstone with persistent bedding planes; rock mass is tectonically deformed, moderately to highly fractured, folded, and locally sheared. Condition of the discontinuities is fair to poor. GSI estimated to range from 25 to 45.

<u>Murree Formation – Sandstone (MF-SS):</u> Underlies most of the concrete structure and left dam abutment; consists of thinly bedded calcareous sandstone and is characterized as moderately fractured, interlocked, partially disturbed and locally disturbed rock mass with fair to good discontinuity conditions. GSI estimated to range from 40 to 60.

MATERIAL PROPERTIES

The geomechanical classifications were used as a basis for developing foundation material properties for the finite element model. The foundation materials are modeled using the Generalized Hoek-Brown criterion (Hoek E. et al., 2002). The equivalent Mohr-Coulomb strength parameters calculated for a maximum σ_3 value of 2 MPa are presented only for reference.

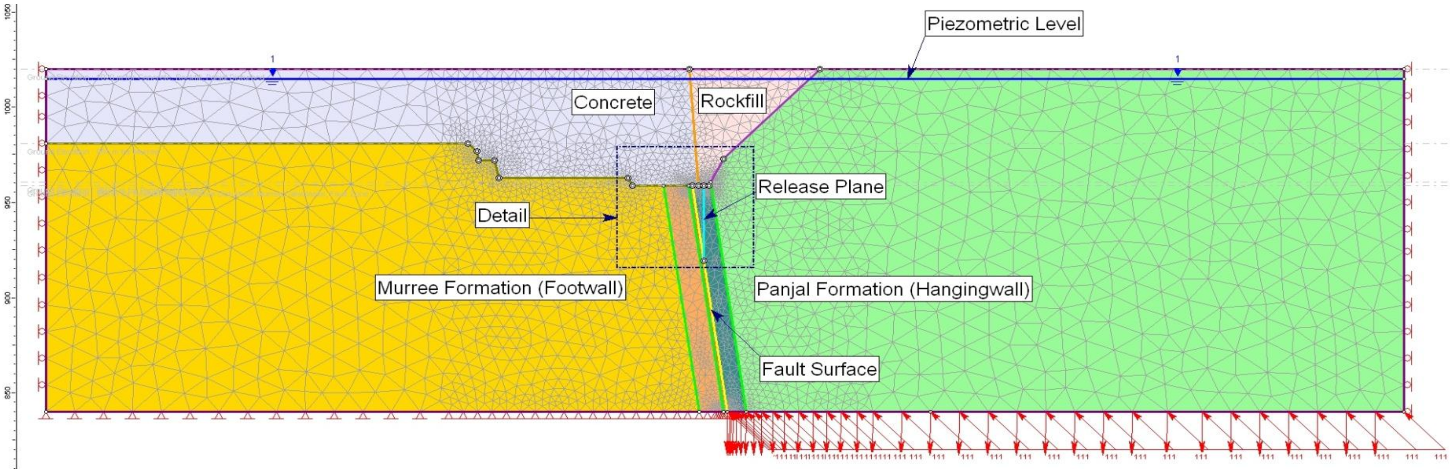

Fig. 4. Finite Element Model

The intact rock strengths and deformation moduli were obtained from laboratory testing programs, the *RocLab* software program (Rocscience Inc., 2011), the observed GSI classifications, and the estimated material constants and rock mass modulus ratios. The foundation material parameters are summarized in Table 1. The Hoek-Brown parameters and deformation modulus for each geomechanical class were determined and imported directly into the Phase2 model.

Table 1. Foundation Material Parameters

Parameter	MF-C	MF-D	MF-SS	PF-C	PF-D
Unit weight (kN/m^3)	26	26	26	25	25
Poisson's Ratio	0.35	0.30	0.30	0.35	0.30
UCS (MPa)	45	45	100	45	60
Avg. GSI	20	35	50	15	35
Equiv. Coh. (MPa) (σ_n = 2 MPa)	0.4	0.5	1.2	0.3	0.8
Equiv. Fric. (σ_n =2 MPa)	33	39	55	31	51
Def. Mod (MPa)	720	1,800	8,400	570	2,800

The dam materials listed in Table 2 were estimated from comparable existing structures.

Table 2. Dam Material Parameters

Parameter	Concrete	Clay Core	Rockfill
Unit weight (kN/m^3)	23.6	18.0	21.0
Poisson's Ratio	0.15	0.30	0.35
Elastic Mod. (MPa)	25,000	35	100
Parameter	Concrete	Clay Core	Rockfill
Peak/Residual Cohesion (MPa)	10.5/NA	0/0	0/0
Peak/Residual Friction Angle	35/NA	32/32	45/40

The interfaces between clay core, rockfill, foundation rock, and concrete structure, as well as the fault surface and the proposed release plane, were modeled as joint elements. Joints were assigned strength (Mohr-Coulomb) and stiffness parameters and allowed to slip inelastically. The cohesion and friction angle of each interface were defined by the strength of the weaker of the two adjacent materials. Similarly, the normal and shear stiffness were estimated based on the weaker material properties of the two adjacent materials and the estimated thickness of each interface. The material properties of the modeled joints in the FEA are presented in Table 3**.**

Table 3. Material Interface Parameters

Interface	Cohesion (MPa)	Friction Angle (deg)	Normal Stiffness (MPa/m)	Shear Stiffness (MPa/m)
Rock – Concrete	1.2	55	16,900	6,500
Concrete – Clay Core	0.0	32	3,500	1,350
Rockfill – Rock	0.0	45	200	74
Fault Surface	0.3	31	1,910	710
Release Plane	0.0	15	117	40

ANALYSIS

A two-dimensional (2D) elastoplastic finite element analysis (FEA) was employed to model the foundation-structure interaction and the release plane during a seismic event originating along the MBT. The *Phase2* software program (Rocscience, Inc., 2011) was used to model the structure and underlying geology in the vicinity of the fault. The FEA model comprises two stages. Stage 1 represents post-construction conditions at which equilibrium is established. Stage 2 simulates a seismic event by applying a distributed load along the base of the model.

Deformations beneath the dam were evaluated with and without the proposed release plane.

The section along the dam axis, as shown in Fig. 4, was developed. The figure shows the concrete and clay core rockfill dam with the fault impinging the edge of the concrete section.

Assumptions: Several simplifying assumptions were incorporated into the 2D model and include the following:

- The analysis is performed using plane stain conditions.
- The piezometric level in the model coincides with the normal reservoir level.
- The analysis does not consider any additional measures to protect the concrete structure such as construction joints between concrete monoliths, additional reinforcement, or thicker concrete sections at the foundation level, which may also be considered as the design process continues.
- The dam abutments are modeled as the same elevation as the dam crest (EL. 1020 m). The model does not take into account effects due to the mountain topography above the dam crest, particularly on the right abutment.
- The concrete structure is modeled as linear elastic; all other materials exhibit elastic – perfectly plastic behavior. The use of perfectly plastic behavior is meaningful because the fault surface and surrounding faulted rock units are heavily sheared and thus considered to exhibit residual (post-peak) shear strength parameters. Thus, no different residual strengths or any dilation are considered for the fault materials.
- An earthquake originating at depth beneath the dam was simulated as a distributed load applied diagonally at the bottom model boundary of the hanging wall. A certain number of iterations per load step are permitted to generate a total displacement along the fault of approximately 3 meters in the model without the release plane.

Model Geometry: The complete geometry, mesh, and boundary conditions of the FEA model are shown in Fig. 5. The top boundary of the FEA model is horizontal and equivalent to the dam crest elevation of El. 1020 m, amsl. The model extends 119 meters beneath the dam foundation (about twice the height of the dam). The lateral extent of the model at each side of the fault is about 330 meters, or three times the length of the MBT fault trace to avoid boundary effects. The mesh element density is increased around the fault surface to include the concrete structure, the rockfill portion, and the fault rock units.

Fig. 5. Detail of Fault Zone Geology and Treatment

is a detail of the model showing the various foundation materials along the fault zone. All geologic units are modeled as dipping parallel to the MBT. The release plane is assumed vertical and extends to a depth of approximately 40 meters to intersect the MBT fault surface.

The top boundary elements represent the free ground surface and are unrestrained. The side boundaries of the model are restrained from movement in the horizontal direction. The footwall portion of the model's bottom boundary is restrained from moving horizontally or vertically. The bottom boundary of the hanging wall is modeled as infinite elements to allow for the application of the external load in Stage 2. According to *Phase2*, infinite elements extend to infinity and allow displacements to decay gradually from the external boundary toward the infinite domain (Rocscience, Inc., 2011).

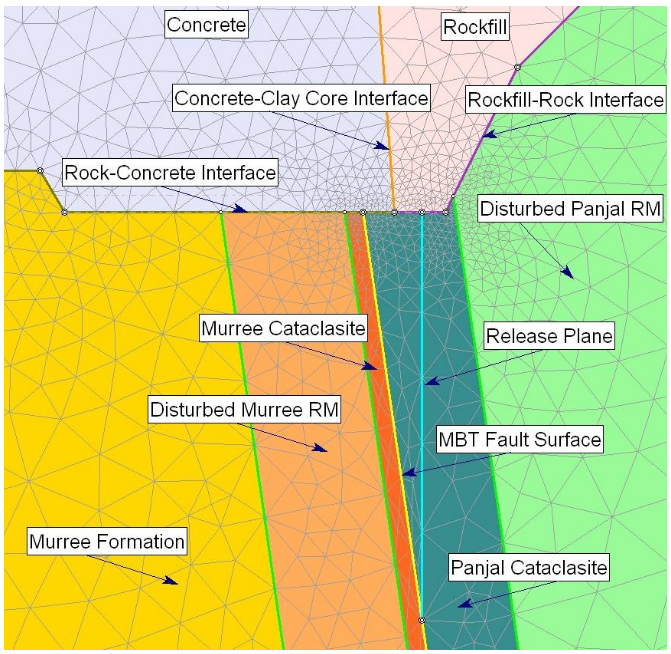

Fig. 5. Detail of Fault Zone Geology and Treatment

2D Finite Element Analysis: The applied loads in the FEA model comprise the material self-weights, the in-situ stress field, including the hydrostatic pressure, and load to simulate a seismic event. Material self-weights were calculated using the unit weights and ground surface elevations of each material. Due to lack of information regarding the *in situ* stress conditions in the dam area, the field stress ratio (i.e., horizontal to vertical stress ratio) is assumed to be K=1 for both the in-plane and out-of-plane directions. The hydrostatic pressures are considered by applying a piezometric surface equivalent to the reservoir normal maximum service level. The above loads were applied in Stage 1 of the model to establish equilibrium for the dam operating condition. Minor deformations that occur in Stage 1 were reset prior to Stage 2. In Stage 2, the distributed load was applied along the base of the hanging wall to simulate the earthquake rupture.

The model was performed using two configurations. The first configuration was constructed without the release plane to estimate the load required to achieve a displacement of approximately 3 meters at the foundation-fault surface, which is considered representative of a large seismic event in the region (NJC, 2010). The modeling indicates that the loading required to achieve 3 meters offset is equivalent to about 111 MN/meter length of the footwall (i.e., out-of plane direction), and which is dependent on the model geometry. In addition, the number of model iterations to achieve a 3 meter offset was recorded.

For the second configuration, which includes the release plane, a load of 111 MN/meter was applied along the base of the hanging wall. The second configuration was run using the same number of model iterations as the first configuration. The output from the two models was then compared.

RESULTS

The results of the modeling are best represented in terms of displacements. The contours of total displacement in the vicinity of the fault on the models, without and with the release plane, are shown in Fig. 6 and Fig. 7, respectively.

Comparison of the figures demonstrates that displacement is directed along the release plane and through the rockfill. Thus, the total displacements beneath the concrete section of the dam are reduced significantly.

The results of the analysis were further evaluated with respect to displacements at reference points in the foundation interface and the dam crest. Fig. 8 indicates the locations of the several reference points. Table 4 summarizes the displacements at each reference point, and the percent reduction in total displacement realized with the release plane included.

With the proposed release plane included, the total displacements at the foundation and crest of the concrete structure (points A, B, C, and G) decrease about 75 to 80 percent compared to the displacements of the model without the release plane. Similarly, total displacements within the clay core rockfill dam section (points E and F) decrease approximately 55 percent with the release plane included. At the location of the release plane (point D) on the foundation surface, total displacements decrease approximately 70 percent compared to the model configuration without the release plane. These results demonstrate that foundation treatment that incorporates a release plane consisting of drill holes backfilled with bentonite could significantly reduce the effect of a seismic event on the performance of the concrete structure by directing fault movement through or into the more deformable portion of the dam.

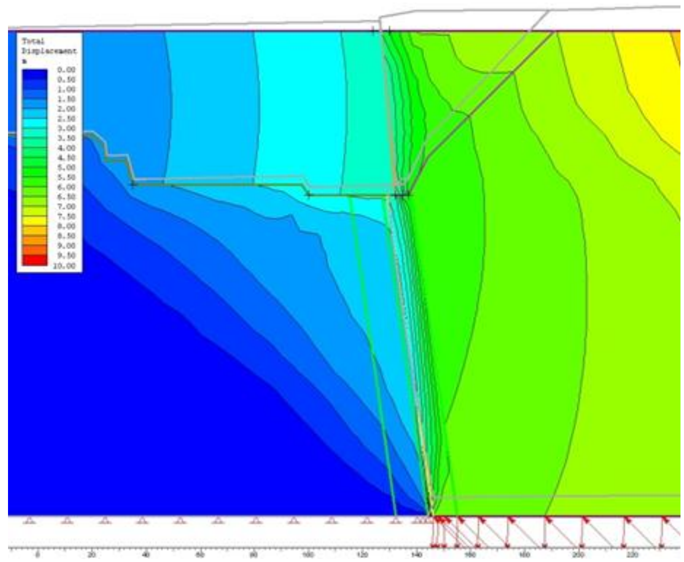

Fig. 6. Total Displacements without Release Plane

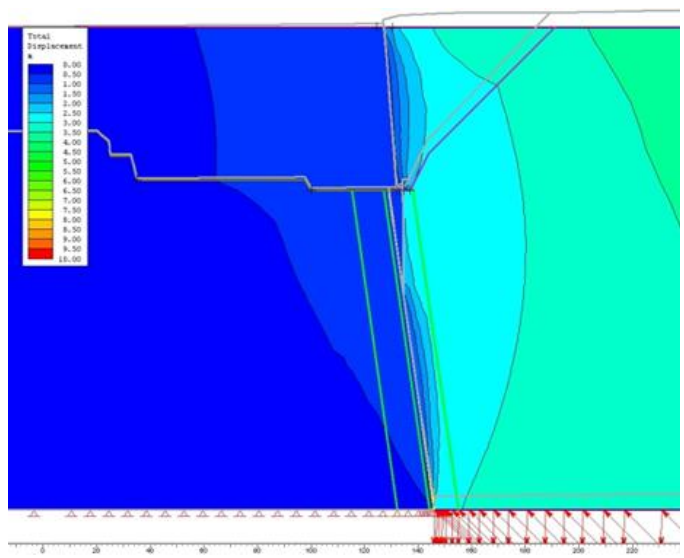

Fig. 7. Total Displacements with Release Plane

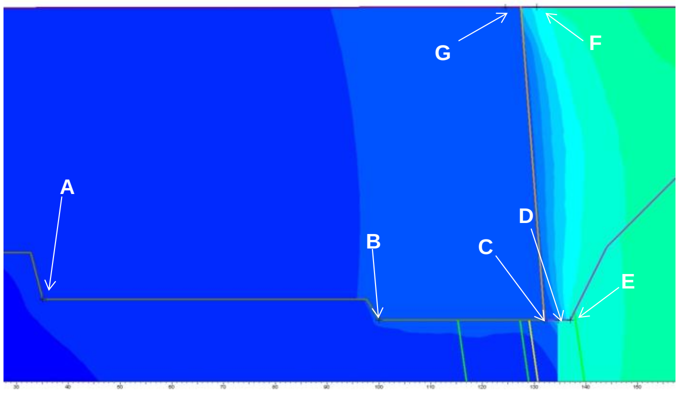

Fig. 8. Locations of Reference Points for Deformation Analysis

Table 4. Summary of Displacements

ID	Location	Displacement		% Redux.
		w/o Plane	w/ Plane	
A	Foundation beneath concrete structure	1.8	0.4	79
B	Foundation beneath Gate No. 3	2.8	0.7	77
C	East end of concrete structure	3.3	0.8	77
D	Trace of release plane at foundation	3.7	1.1	71
E	East foundation of rockfill dam	5.3	2.5	52
F	Rockfill crest, 3m east of interface	5.0	2.1	57
G	Concrete crest, 3m west of interface	3.2	0.8	76

CONCLUSIONS AND RECOMMENDATIONS

Based on the results from the finite element analysis, the following conclusions are noted:

- The analysis shows that the mitigation measure proposed to direct the MBT rupture away from the concrete section of the dam and into the more deformable embankment section is potentially effective. Preliminary analyses demonstrate that total displacements on the dam and foundation could be reduced by up to 80%. Therefore, the potential damage to the concrete structure is likely to be reduced if the release plane were constructed at the proposed location due to a reduction of the earthquake-induced stresses in the concrete.

- The potential benefits of the proposed foundation treatment are likely to increase if additional strengthening measures are incorporated into the concrete structures, such as construction joints between concrete monoliths, additional reinforcement, or stiffer concrete sections at the foundation level

- Additional analyses are being performed to further optimize the foundation treatment by examination of the concrete and clay core rockfill dam sections. Optimizing includes adjusting the orientation of the release plane, modifying the width and properties of the fill section to reduce the average dam friction angle, and modifying the concrete structure near the fault zone.

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