Journal of Rock Mechanics & Tunnelling Technology (JRMTT)
24 (2) 2018 pp 95-108

Available online at www.isrmtt.com

Numerical Modelling to Evaluate Support System of Large Underground Openings in Bhutan Himalaya – A Case Study

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ABSTRACT

Numerical modelling simulates the behaviour of the rock mass during and after the excavation of the underground structures based on the prevailing geological conditions, engineering properties of the rock mass and methodology and sequence of excavation based on which the optimum support system for ensuring the stability of the structure can be designed. Numerical modelling not only validates the stability of the underground structures but also predicts the stress conditions along with the anticipated deformation along the caverns. With the aid of the numerical analysis realistic estimates of the engineering problems may be determined leading to realistic cost and schedule estimations. In this context when a shear zone of 1 to 1.5m thickness with associated fracture zone of 5 to 8m thickness was encountered along the underground power house caverns three dimensional numerical model analysis was undertaken to arrive at a proper engineering and cost effective solution for the problem which initially appeared detrimental for the stability of the caverns.

Keywords: Numerical modelling; Large underground structures; Shear zone treatment; Geotechnical monitoring; Support evaluation

1. INTRODUCTION

The 720 MW Mangdechhu Hydro-Electric Project is in advance stage of construction in Trongsa Dzongkha of Bhutan. The project envisages construction of 112m high concrete gravity dam, a 6.5 m diameter and 13.521km long horse-shoe-shaped head race tunnel, an underground power house complex comprising of 155m long, 23m wide and 41m high machine hall and a 135.5m long, 18m wide and 23m high transformer hall cavern. Water from machine hall would be released back into the river at an elevation of 1000m via, 1.333 km long, 8m diameter tail race tunnel. About 82% of the works have been completed till date and the project is expected to be commissioned by 2018 as per schedule. During the excavation of the centre gullets of the machine hall and transformer hall caverns a shear zone of 1 to 1.5m thickness with associated fracture zone of about 5 to 8m thickness was encountered. The disposition of the shear zone was cutting along the walls of the caverns and also the rock pillar between both the caverns, which necessitated re-evaluation of excavation methodology and sequence and support system to ensure stability of the structure, which was achieved by systematic three dimensional numerical model analysis of the caverns.

2. GEOTECHNICAL APPRAISAL OF THE UNDERGROUND POWER HOUSE CAVERNS

The rock mass encountered during the excavation of both the caverns is represented by grey colored, slightly to moderately jointed, strong quartzite with intercalated schist bands of 1 to 5cm thickness and is traversed by randomly oriented pegmatite and amphibolite intrusions. Quartz veins with thickness upto 40cm were encountered between RD 0 m to 40m of the power house cavern. Foliation parallel to 5 - 8m thick band of biotite schist was encountered along the upstream wall of power house cavern between RD 130m to 155m. This weak band was characterised by presence of multiple interfolial shear seams. Both the caverns were traversed by a shear/fracture zone of thickness of 5 to 8m thickness, with attitude of N30°E – S30°W dipping 10 to 24° due N60°W. This shear/fractured zones intercepted both the upstream and downstream walls of the caverns along the long axis. The mean attitude of foliation recorded along the caverns is N 28° W – S 28° E dipping 33° due S 62° W. Apart from foliation the rock mass was traversed by three other prominent sets of discontinuities along with some randomly oriented joints. The stereoplot of discontinuity data along with attitude of shear zone, in lower hemisphere of equal area stereonet is shown in Fig. 1.

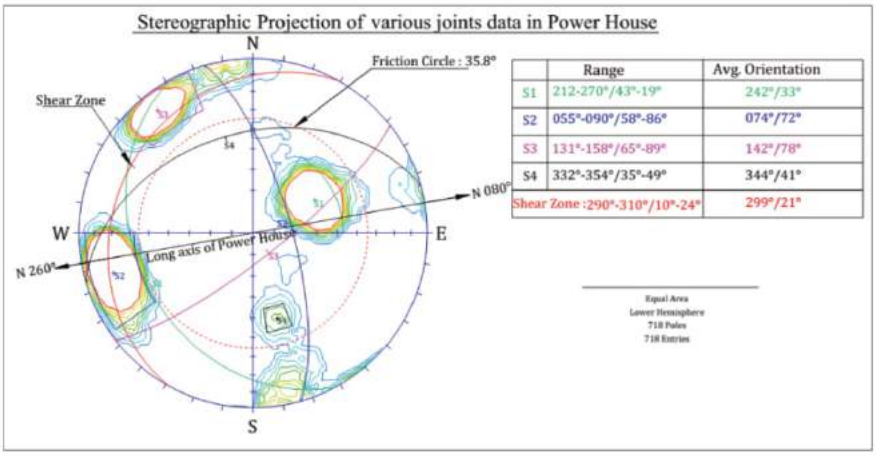


Fig. 1 - Stereographic projection of discontinuity data recorded along power house caverns

Presence of a shear/fracture zone, cutting across the upstream and downstream wall of both the caverns along the long axis of caverns and large size of the caverns, necessitated re-evaluation of the support system and excavation methodology and sequence along with monitoring and analysis of the behaviour of the rock mass during the course of excavation. As such, progressive 3-D stress analysis of the power house caverns along with monitoring of the caverns by aid of geotechnical instrumentation was undertaken. The predicted behaviour of rock mass as inferred from 3-D stress analysis was regularly compared with the observed behaviour as recorded by geotechnical instrumentation to optimize excavation methodology and sequence along with optimization and validation of support system.

2.1 Treatment of the Caverns

Presence of previously un-envisaged shear/ fracture zone of 5 to 8 m thickness appeared detrimental for the stability of the caverns; as such reassessment of the stability of both caverns, while also revalidating the rock support and excavation methodology, was undertaken. The consultant suggested shifting the caverns about 40m laterally toward valley side whereby the crown of the caverns could be relatively free from the affect of the shear zone. However this would have resulted in the shear/fractured zone intersecting the turbine foundation and its continuity along the walls and rock column between the caverns would have still not been avoided, further this would necessitate re-alignment of pressure shafts and tail race tunnel, which were excavated for a considerable length, along with realignment of main access tunnel, involving considerable time and financial losses. Therefore MHPA dared to opt for the in-situ treatment of the shear/fracture zone.

Considering the presence of weak rock mass associated with the shear zone and to effectively deal with frequent loose fall from these zones steel ribs support comprising of ISMB 350 spaced 0.5m c/c between RD 0 to 35 and RD 100 to 155m at the power house cavern and between RD 0 to 20m and RD 60 to 120m in the transformer hall cavern were provided; followed by extensive contact and consolidation grouting to the depth of 6m into the rock mass. Longer rock bolts of 12m length at spacing of 3m c/c were also provided along the entire crown of both the caverns in addition to 7.5 and 9m long rock bolts as proposed in construction drawings.

2.2 Treatment of the shear zone

While the crown affected by shear zone of both the caverns was strengthened by steel ribs support and grouting, the shear/fractured zone along the walls was treated progressively during course of excavation of both the caverns. The shear/fractured rock mass was scooped out upto a depth of 2.5 to 3m, followed by cleaning of the void and filling it with reinforced concrete of M30 grade. The reinforced back filled concrete was stitched to bed rock by the aid of 18m long rock bolts spaced 1.5m c/c, followed by consolidation grouting to a depth of 20m using micro fine/ultrafine cement in stages of 3m each with grout consistency ranging from 5:1 to 0.8:1 (ratio by weight of water and cement) under a pressure of 2 to12kg/cm^2 (0.2 to 1.2MPa) and hole spacing of 1.5m.

3. 3-D NUMERICAL MODELLING OF THE CAVERNS

Three dimensional stress analyses of the caverns was undertaken using 3DEC software which is a three-dimensional numerical program based on the distinct element method for discontinuum modelling. The basis for this program is the extensively tested numerical formulation used by the two-dimensional version, UDEC (Anon, 2013). Numerical modelling using 3DEC software simulates the response of discontinuous media (such as a jointed rock mass) subjected to either static or dynamic loading. The discontinuous medium is represented as an assemblage of discrete blocks. The discontinuities are treated as boundary conditions between blocks; large displacements along discontinuities and rotations of blocks are allowed. Individual blocks behave as either rigid or deformable material. Deformable blocks are subdivided into a mesh of finite difference elements, and each element responds according to a prescribed linear or nonlinear stress-strain law. The relative motion of the discontinuities is also governed by linear or nonlinear force-displacement relations for movement in both the normal and shear directions. 3DEC has several built-in material behaviour models, for both the intact blocks and the discontinuities that permit the simulation of response representative of discontinuous geologic or similar, materials. 3DEC is based on a Lagrangian calculation scheme that is well-suited to model the large movements and deformations of a blocky system.

Based on the various geological condition, a three dimensional numerical model was prepared comprising of power house cavern, transformer hall cavern, bus ducts, drainage gallery, penstocks, draft tubes and other excavations.

3.1 In-situ Geotechnical Parameters and Engineering Properties of Rocks

To simulate a realistic behaviour of the rock mass in 3DEC model the prevailing in-situ stress along with the in-situ geotechnical parameters of the rock mass and engineering properties of the rocks were determined by NIRM and used as input parameters. In-situ stress conditions were measured by hydro fracturing method in accordance to method suggested by Cornet (1986), while the geotechnical parameters viz., in-situ deformability and elasticity were determined by plate load test as per ISRM (1979) guidelines, in-situ shear strength parameters were evaluated by direct shear test method as per ISRM (1974) in class - III type of rock mass and in shear zone. The physico-mechanical properties of the rocks were determined by testing various types of rock samples collected from exploratory boreholes in accordance to ISRM (1981). The determined parameters are tabulated in Tables 1, 2, 3 and 4.

Table 1 - Summary of in-situ stress measurement results

Principal Stresses	
Vertical stress (σ_v) in MPA (Calculated with an overburden of 180m and density of rock 2.7gm/cc	4.76 MPa
Maximum horizontal principal stress (σ_H)	8.38±0.6496 MPa
Minimum horizontal principal stress (σ_h)	5.59±0.4331 MPa
Maximum horizontal principal stress direction	N 70^0
$K = (\sigma_{H+} \sigma_h)/2/ \sigma_v$	1.47
$K = (\sigma_{H/} \sigma_v)$	1.76

Table 2 - Summary of plate load test

In-situ deformability parameters	Class – III	Shear Zone
Modulus of deformability of rock mass	2.01 to 3.99	0.46 to
Modulus of elasticity of rock mass (E_{em}) in GPa	2.31 to 5.24	0.49 to 1.36

Table 3 - Summary of in-situ direct shear test results

Loading Condition	Rock mass category	Cohesion (c) MPa	Angle of internal Friction (ϕ) Degrees
Peak shear parameters	Class – III Rock	0.375	35.8
	Shear zone	0.354	25.6
Residual shear parameters	Class – III Rock	0.274	34.2
	Shear Zone	0.231	25.3

Table 4 - Physico-mechanical properties of different type of rocks of underground caverns

Properties	Rock Type		
	Pegmatite	Quartzite	Pegmatite from Shear
Density (kg/m^3)	2590 –	2642 – 2646	2586
Specific gravity	2.61	2.66	-
P-wave velocities (km/s)	3.24	4.13 – 4.35	2.59
Porosity (%)	0.87	0.57	-
Water absorption	0.33	0.21	-
UCS (MPa)	114 – 143	237	40
Young's modulus (GPa)	38.5 – 45.5	67	12
Poisson's ratio	0.32	0.17	0.49
Tensile strength (MPa)	6.08	10.64	-

3.2 3DEC Analysis

Based on the geological conditions encountered during excavation of crown of both the caverns and projection of information as obtained from subsurface investigations geological sections along and across the caverns were prepared (Figs. 2, 3, 4, 5, 6a & b), which were used for preparing a three dimensional numerical model in 3DEC software comprising power house cavern, transformer hall cavern, bus ducts and other excavations (Fig. 7). The different sections of simulated model are shown in Fig. 8.

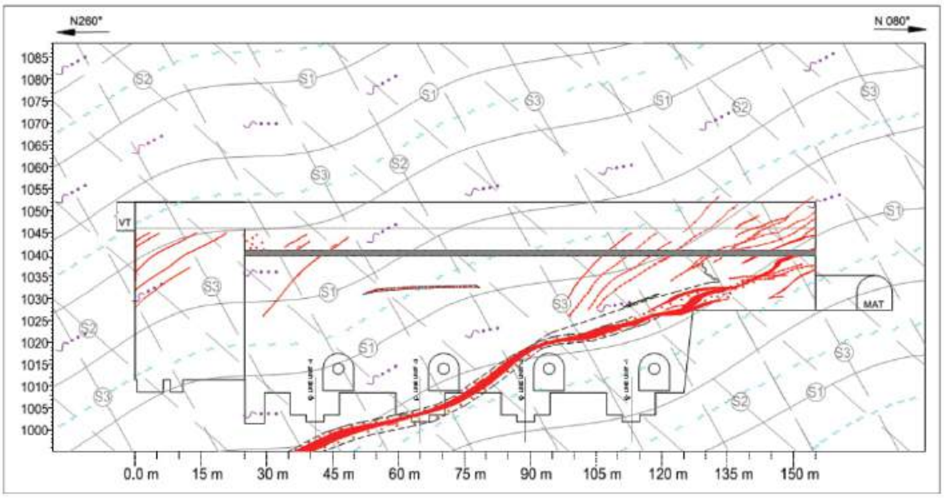


Fig. 2 - Geological section along upstream wall of power house cavern showing disposition of shear zone

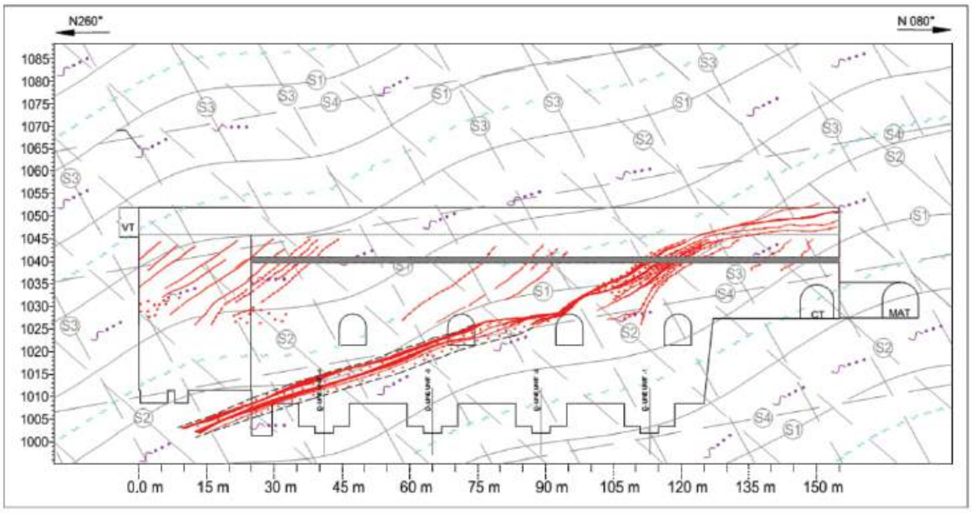


Fig. 3 - Geological section along downstream wall of power house cavern showing disposition of shear zone

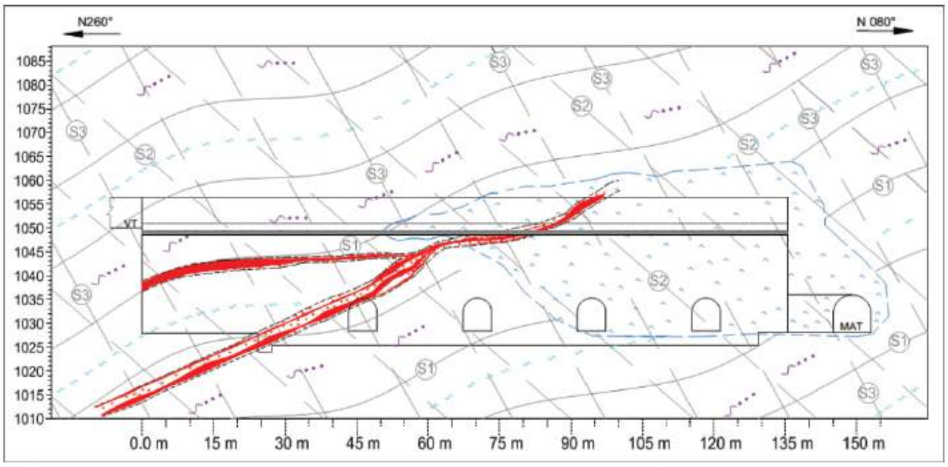


Fig. 4 - Geological section along upstream wall of transformer hall cavern showing disposition of shear zone

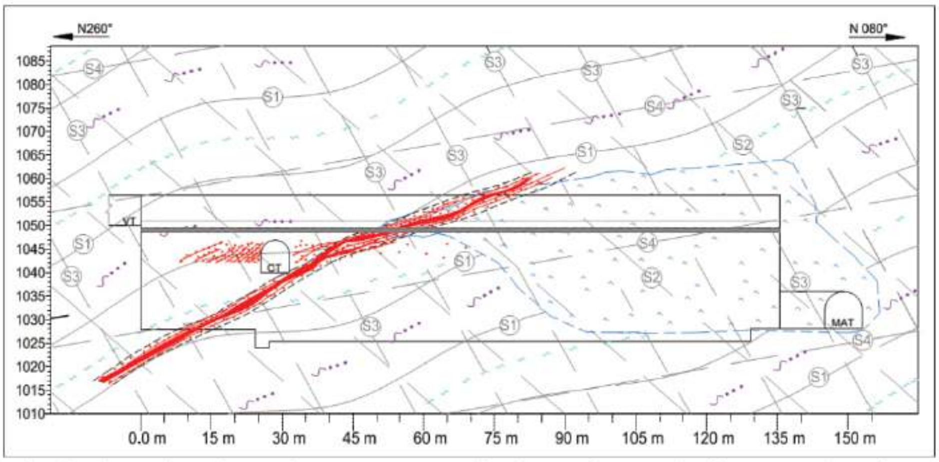
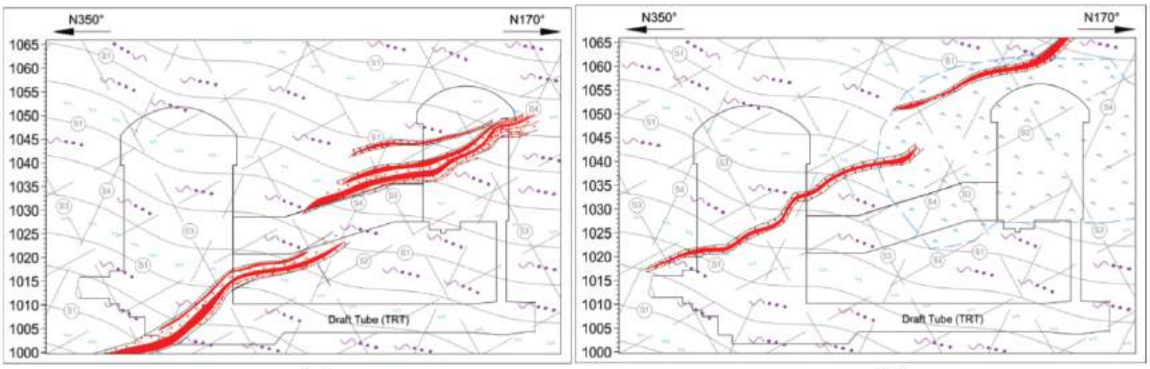


Fig. 5 - Geological section along downstream wall of transformer hall cavern showing disposition of shear zone



(a) (b)

Fig. 6 a & b - Geological section across power house and transformer hall showing disposition of shear/fractured zone

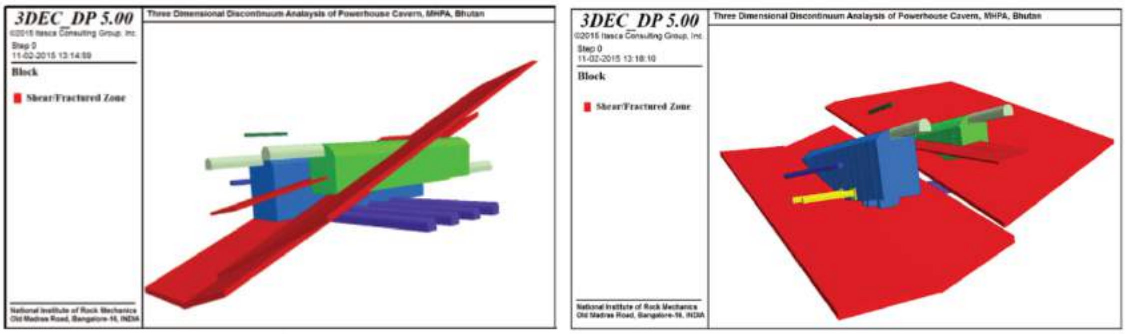


Fig. 7 - Simulated 3D model showing all the components and shear/fractured zone

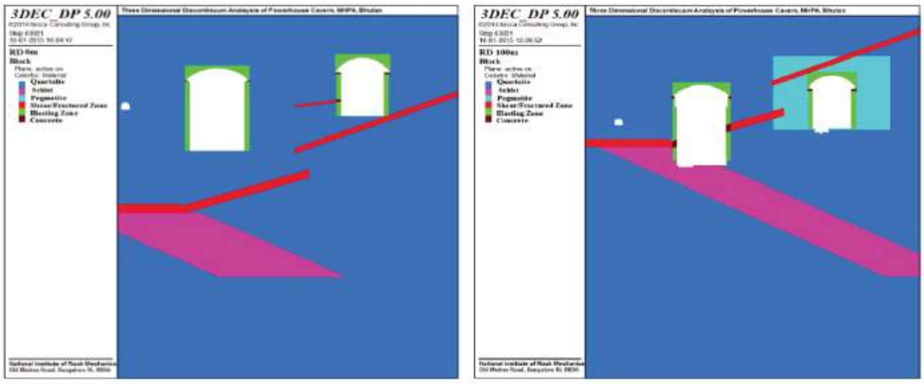


Fig. 8 - Simulated geological sections at RD 0 and 100

In the model, excavation was simultaneously carried out in the powerhouse and transformer hall caverns. The bench heights were optimized, and the following sequence of excavation was adopted (Table 5).

Table 5 - Benching sequence adopted for simulation of 3- DEC numerical modelling

Stage- I	Excavation of crown (upto EL. 1045.5 in PHC and EL. 1051m in THC)
Stage- II	Bench-I (EL. 1045.5 - EL. 1039.8 in PHC and EL. 1051 – 1043.5 in THC)
Stage - III	Bench–II (EL. 1039.8–EL. 1033.5 in PHC and EL. 1043.5–1038.5 in THC)
Stage - IV	Bench – III (EL. 1033.5 – 1027.4 in PHC and EL. 1038.5 – 1033.5 in THC)
Stage - V	Bench – IV (EL. 1027.4 – 1021 in PHC and EL. 1033.5 – 1028.5 in THC)
Stage - VI	Excavation of bus ducts.
Stage-VII	Bench – V (EL. 1021 – EL. 1015.5 in PHC)
Stage -VIII	Bench – IV (EL. 1015.5 – EL. 1011.5 in PHC)
Stage - IX	Bench – VII (other excavation till EL. 1001.5 in PHC)
Stage - X	Excavation of penstocks, TRT manifolds and drainage gallery

The rock mass properties for the model were estimated from the Geological Strength Index (GSI) determined from the excavated part of the caverns as suggested by Hoek (1994) and modified by Hoek and Brown (1997), Hoek et al. (1998), Sonmez and Ulusay (1999) and Marinos and Hoek (2001) and the data obtained from laboratory and in-situ testing of the rock mass in accordance with the method suggested by Hoek et al. (2002) and Hoek and Diederiches (2006) and are tabulated below (Table 6).

Table 6 - Rock mass properties used for preparing model for 3-DEC analysis

Rock Mass Type	UCS (MPa)	GSI	E_i (MPa)	m_b	s	a	c (MPa)	ϕ	E_{rm} (MPa)
Quartzite	237	45	67000	2.805	0.002	0.508	1.911	58.52	14984.5
Pegmatite	128	45	42000	4.067	0.002	0.508	1.549	57.38	9393.3
Schist	50	30	33750	0.821	0.0004	0.522	0.597	37.39	2746.68
Shear/Fracture	40	-	-	-	-	-	0.354	25.6	850

Based on the laboratory normal and shear stiffness data and the prevailing stress levels around the caverns, following joint stiffness and joint shear strength parameters were calculated and used in model.It should be realized that jointed rock has low shear modulus than the modulus of deformation because joint shear stiffness is lower than its normal stiffness.

- Joint normal stiffness: 50 GPa/m
- Joint shear stiffness: 0.6 GPa/m
- Joint cohesion: 0 MPa
- Joint friction: 40□

The support system for power house and transformer hall caverns incorporated in the model is tabulated in Table 7. The rock pillar between the bus ducts were cross stitched with 36mm diameter rock bolts spaced 1.5m c/c.

Table 7 - Support system used for numerical analysis

Power House Cavern		
Support	Crown	Walls
Shotcrete	250mm thick SFRS	250mm thick SFRS
Rock Bolts	36mm dia.7.5m long @ 3m c/c, 36mm dia. 9m long @ 3m c/c, 36mm dia. 12 m long @ 3m c/c,	36mm dia. 12 m long @ 1.5 m c/c, along u/s and d/s wall and 36mm dia.7.5m long @ 1.5m c/c on gable
Ribs	ISMB 350 @0.5 c/c from RD 0 to 35 & 100 to 155m	
Transformer Hall Cavern		
Shotcrete	250mm thick SFRS	250mm thick SFRS
Rock Bolts	36mm dia.7.5m long @ 1.5 m c/c	36mm dia. 9 m long @ 1.5 m c/c, along u/s and d/s wall and 36mm dia.7.5m long @ 1.5m c/c on gable
Ribs	ISMB 350 @ 0.5 c/c from RD 0 to 20 and 60 to 120m	

The behaviour of the rock mass was evaluated based on the analysis by computing deformations at every 10m RD and at different elevations as a function of excavation sequence, and corresponding factor of safety, using three dimensional Hoek and Brown yield criteria as suggested by Melkoumian et al. (2008).

3.3 Results of 3-DEC Analysis

The three dimensional numerical model was simulated with the input parameters and excavation sequence as discussed above. Output from the simulation was obtained in the form of displacements and factor of Safety (FOS), using three dimensional Hoek and Brown yield criteria. The results are discussed below.

3.3.1 Displacements

The displacements at the crown of power house and transformer hall are shown in Figs. 9 a & b, while for the walls and floor are shown in Figs.10 a & b. The displacement at the centre of the crown of the powerhouse cavern varied from 17.40mm (at RD 150m) to 27.80mm (at RD 40m). Along upstream side of crown the displacements varied from 16.03mm (at RD 10m) to 21.12mm (at RD 140m). Along transformer hall cavern the displacements varied from 7.01mm (at RD 80m) to 39.76mm (at RD 50m) at centre crown while along upstream and downstream side of crown it varied from 15.10mm (at RD 10m) to 34.65mm (at RD 110m) and 16.16mm (at RD 10m) to 37.48mm (at RD 90m) respectively.

Along the upstream and downstream walls of power house cavern the magnitude of displacement varied from 30.9mm (at RD 10m, El. 1029.36m) to 78.69mm (at RD 120m, El. 1020.99m) and 28.12mm (at RD 28.12mm (at RD 150m, El. 1036.27m) to 61.49mm (at RD 120m, El. 1039.8m) respectively. Displacement along upstream wall of transformer hall cavern varied from 31.02mm (at RD 10m El. 1038.3m) to 73.31mm (at RD 60m, El. 1047.2m), while that along downstream wall varied from 35.24mm (at RD 10m, El. 1033.5m) to 60.72mm (at RD 50m, El. 1048.5m).

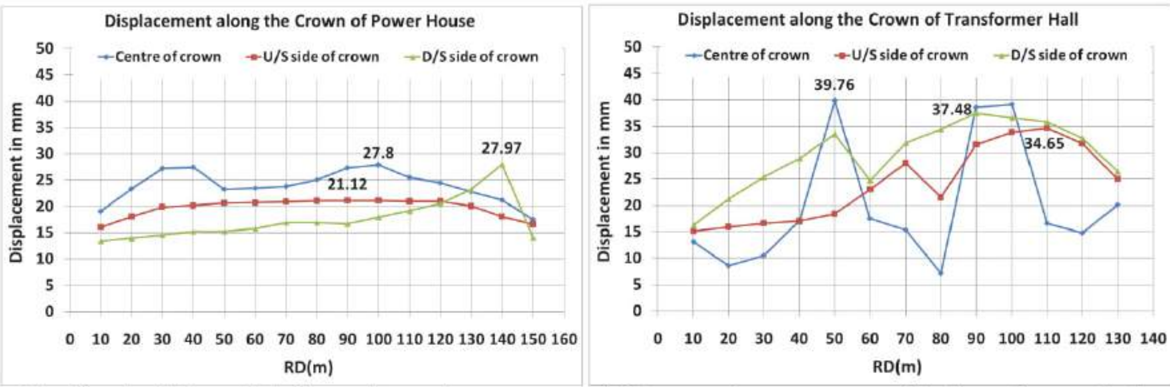


Fig. 9 - Anticipated deformations along crown of (a) powerhouse cavern (b) Transformer hall cavern

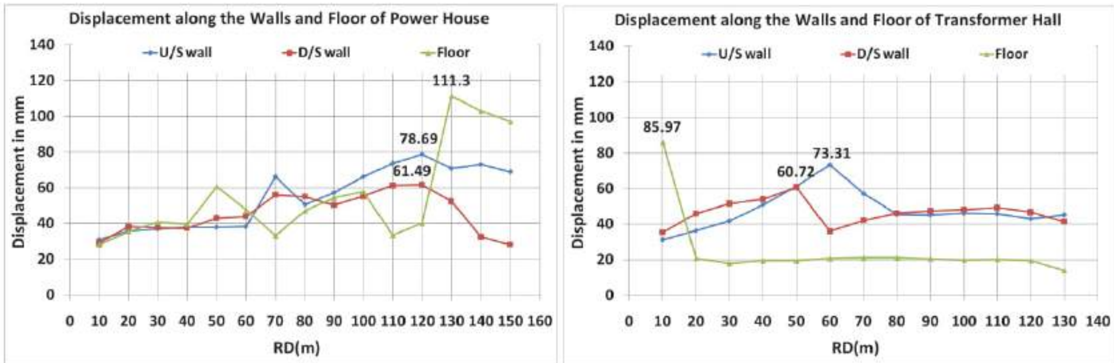


Fig. 10 - Anticipated deformations along walls and floor of (a) powerhouse cavern (b) transformer hall cavern

3.3.2 Factor of safety (FOS)

The model was considered to be elasto-plastic with yielding of rock mass at failure. Using the three dimensional Hoek and Brown yield criterion as proposed by Melkoumian et al. (2008) for analysis of failure state, factor of safety contours were obtained at every 10m RD for the model by using

FISH programming in 3DEC code. Three dimensional principal stresses were obtained from the model after final stage of excavation. These stresses along with strength parameters (m_b, s, a) were incorporated in into 3DEC to obtain factor of safety contours (Fig. 10).

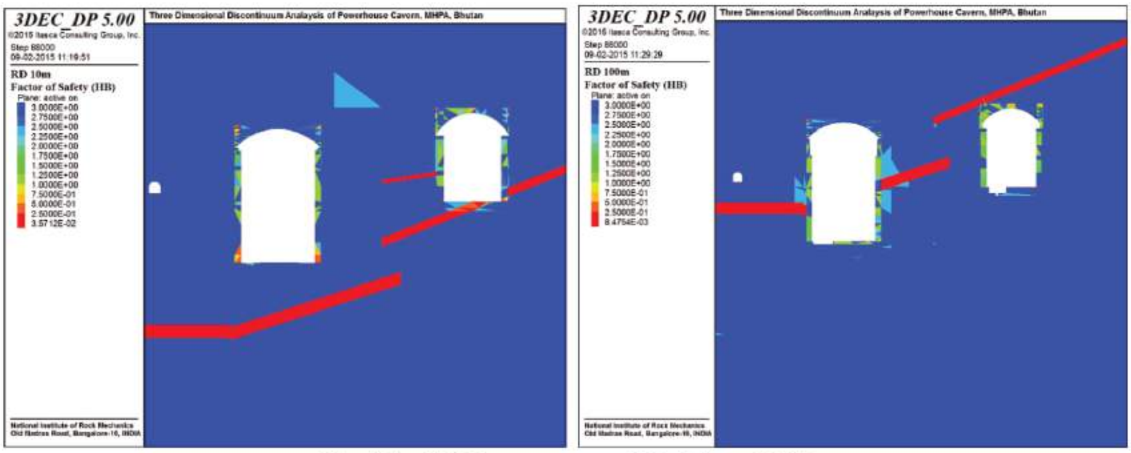


Fig. 10 - FOS contours at RD 10 and 100

The FOS plots revealed that the shear/fracture zone had low values (less than 1) indicating failure in that zone. The pillar between the powerhouse and transformer hall cavern showed increased FOS due to stitching of the bus duct in comparison to model without cross stitching them, though it did not have any significant impact on the FOS of shear zone. However, it did not influence the overall stability of the caverns. The rock mass in the blast zone shows FOS in range of 1 to 2. The rock mass beyond the blasting zone was found to have factor of safety of more than 2. In view of the low FOS obtained for the shear zone, longer rock bolts of 18m length were used in shear zone along with extensive grouting by aid of 20m deep holes.

3.3.3 Validation of support system

The performance of the installed/suggested support system was evaluated by obtaining the failure state of the rock bolts as shown in Fig. 11. It was observed that at some places, the bolts have reached the yield load of the rock bolts. The cables installed along the crane beam and cross bolts installed to stitch the rock pillar between bus ducts have positive effect on the behaviour of the rock mass (stress distribution). The axial force distribution in cable anchors at crane beam and cross stitching rock bolts along bus ducts is shown in Fig. 12.

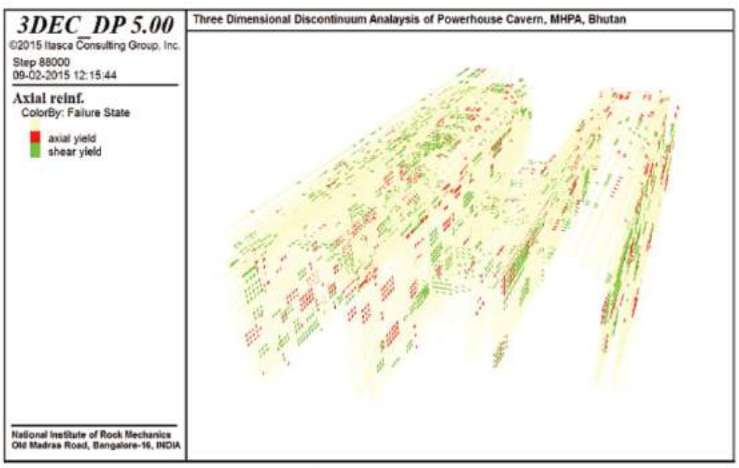


Fig. 11 - State of rock bolts in the caverns

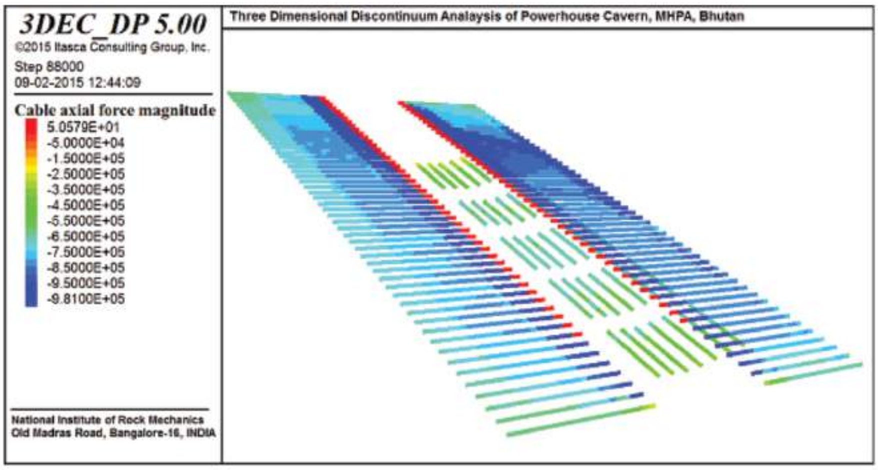


Fig. 12 - Axial stress distribution in the cable anchors at crane beam and cross bolts along the bus ducts

3.3.4 Excavation sequence of bus duct, penstock and TRT manifolds

To evaluate the sequence of the bus duct, penstock and TRT manifold excavation two models were considered. In the first model (Case A), Bus ducts were proposed to be excavated after excavation of fifth bench i.e. after excavation upto El. 1021 in PH and El. 1028.5m in TH, followed by stitching of the rock pillar between bus ducts by cross bolts. Excavation of penstock and TRT manifolds were proposed after reaching El. 1001.5m. In the second model (Case 2) after excavating fifth bench, excavation of penstock and TRT manifold was proposed. Excavation of bus ducts was proposed after reaching El. 1011.5 in the machine hall (PH). Results from both the models were compared at the sections passing through centre line of bus ducts and TRT manifolds. Displacement contours, stress distribution (both minimum and maximum principal stress) and factor of safety were computed and results for both the cases were compared (Table 8). From the comparison it was evident that Case A was better option. In Case B maximum principal stresses were higher along most of the sections and minimum FOS particularly at TRT sections were lower. It was also inferred that higher stresses development in Case B may further deteriorate the shear/fracture zone. In Case A, the cross bolts along the bus ducts were found to strengthen the rock mass whereas in the Case B, bus ducts were to be strengthened after considerable excavation in the machine hall. In view of the above results the excavation of the bus ducts, TRT manifolds and penstocks was executed as per Case A.

Table 8 - Comparison of results of 3DEC analysis for Case A and Case B

RD	Displacement (mm)			Max. Principal Stress			Min. Principal Stress			Min. FOS		
	Case A	Case B	Difference	Case A	Case B	Difference	Case A	Case B	Difference	Case A	Case B	Difference
	Section along centre line of Bus Ducts											
47	64.08	59.67	**-4.41**	52.21	99.38	**47.17**	16.32	15.9	**-0.42**	2.67E-03	8.39E-04	**-0.001831**
71	65.72	83.13	**17.41**	32.29	33.2	**0.91**	8.05	10.28	**2.23**	2.90E-03	5.44E-03	**0.00254**
95	68.44	66.54	**-1.9**	66.76	77.96	**11.2**	14.54	16.11	**1.57**	2.28E-03	3.50E-03	**0.00122**
119	76.47	78.92	**2.45**	35.99	34.82	**-1.17**	20.39	13.44	**-6.95**	1.45E-02	1.82E-02	**0.0037**
	Sections along centre line of TRT manifolds											
41.6	55.73	55.62	**-0.11**	97.5	95.12	**-2.38**	11.55	11.55	**0**	2.68E-03	8.75E-04	**-0.001805**
65.6	66.86	67.09	**0.23**	81.68	83.31	**1.63**	50.26	55.74	**5.48**	3.14E-03	2.94E-03	**-0.0002**
89.6	63.95	59.08	**-4.87**	60.64	63.06	**2.42**	16.05	9.18	**-6.87**	1.63E-03	8.97E-04	**-0.000733**
113.6	77.36	79.57	**2.21**	25.7	34.82	**9.12**	7.98	12.12	**4.14**	1.09E-02	7.89E-03	**-0.00301**

3.4 Comparison of Numerical Modelling Results with Instrumentation Observations

The behaviour of rock mass during excavation was closely monitored by aid of elaborated geotechnical instrumentation program. *The deformations recorded at site during excavation were much less than those anticipated from 3DEC numerical modelling analysis. The maximum deformation anticipated at centre, upstream and downstream side of crown in machine hall were 27.8, 21.12 and 27.97mm respectively, while actually observed maximum deformations along centre, upstream and downstream side of crown of Machine Hall were 13, 12.8, 9.8mm respectively. Similarly maximum anticipated deformations along upstream and downstream walls of machine hall were 78.69 and 61.49mm respectively against which only 27.4 and 42.8mm of deformation was recorded along upstream and downstream walls respectively (Fig. 13).*

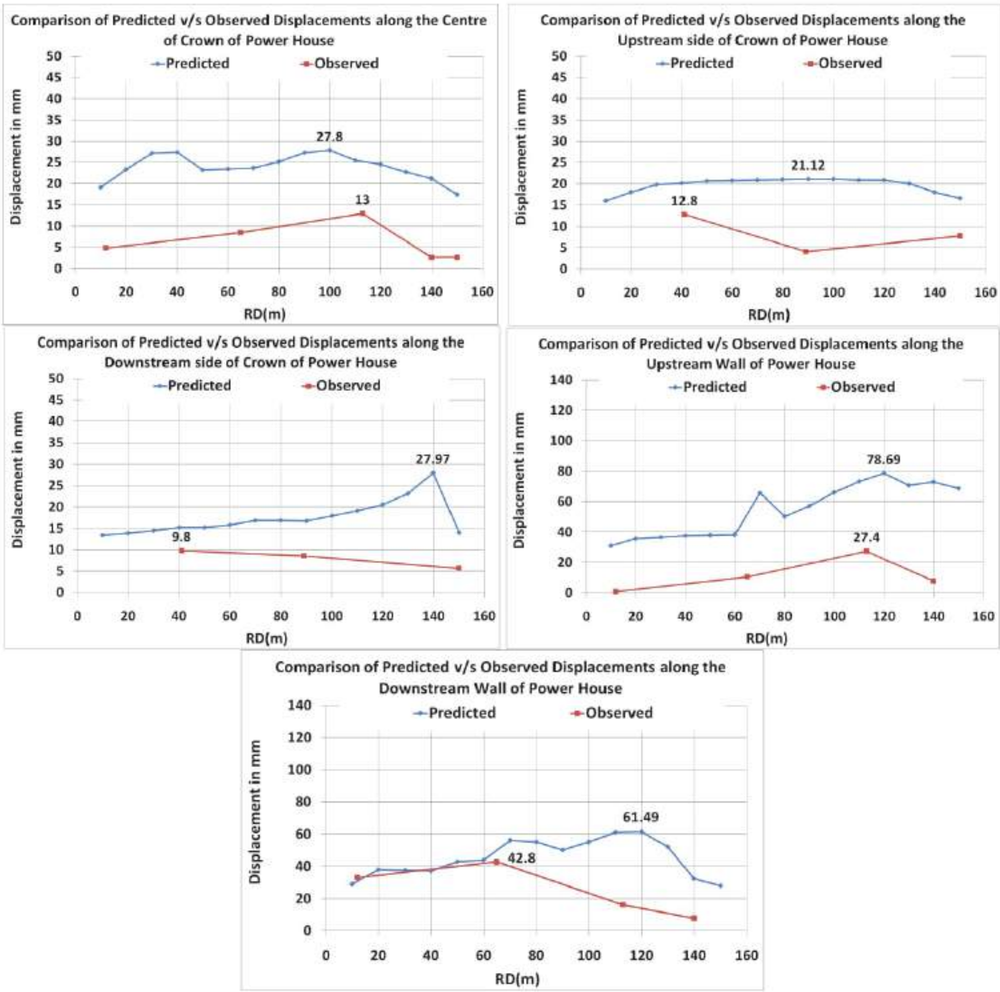


Fig. 13 - Graphical comparison of anticipated v/s observed deformations along powerhouse cavern

Along centre, upstream and downstream side of crown in transformer hall maximum deformations of 39.76, 34.65 and 37.48mm were anticipated of which only 18.7, 8.5, 6.0mm of deformation was observed during excavation of cavern. Along upstream and downstream walls of transformer hall cavern a maximum deformation recorded was 6.01 and 5.7mm respectively against anticipated deformation of 73.31 and 60.72mm respectively (Fig. 14).

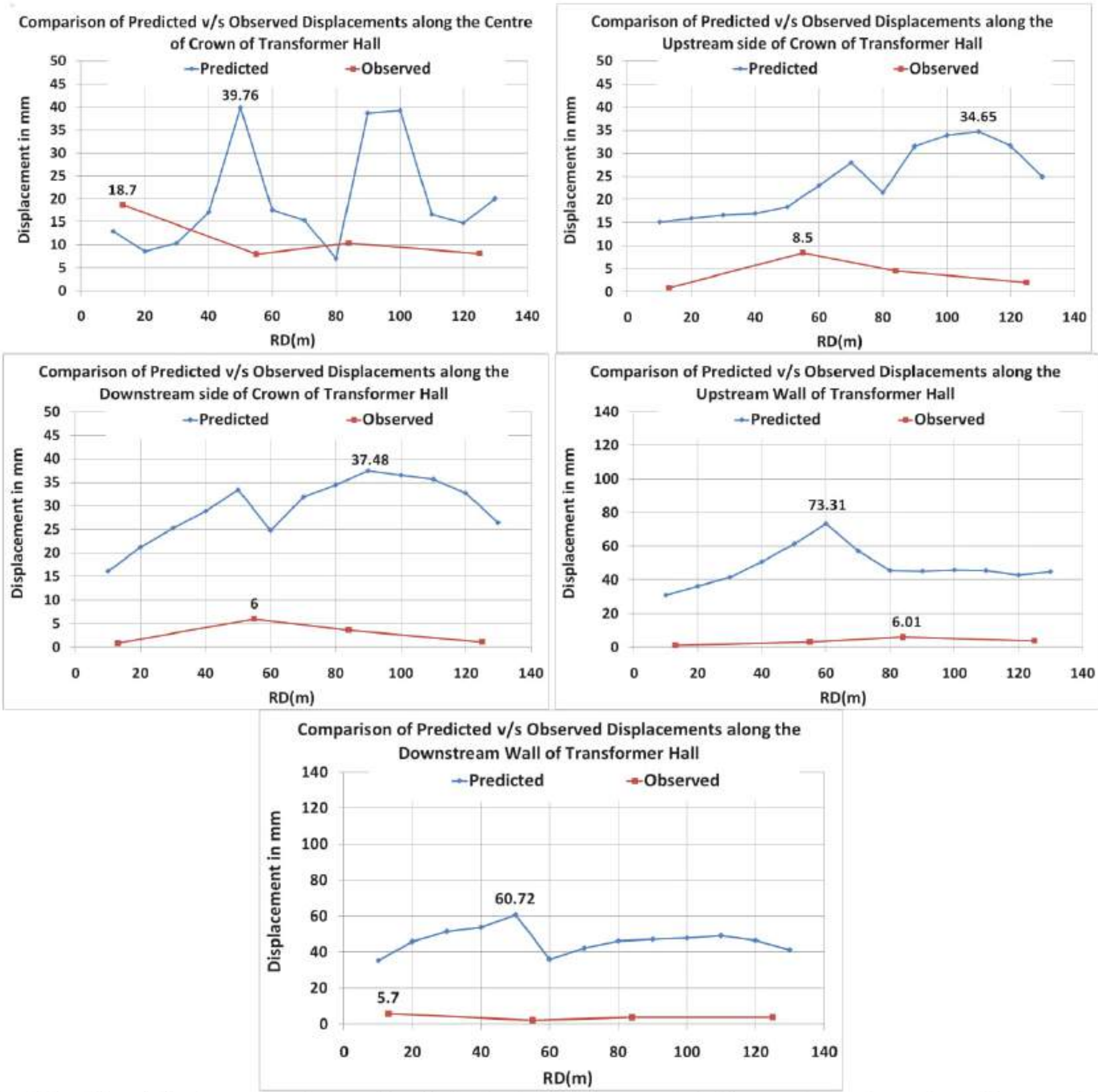


Fig. 14 - Graphical comparison of anticipated v/s observed deformations along transformer hall cavern

The low deformation observed during excavation were due to the fact that in view of higher deformations simulated in 3-DEC analysis, the bench height which in analysis were 5.5m to 6m were restricted to 3m only, and after benching down to every three meter depth, complete support system along with extensive consolidation grouting was undertaken. All these efforts led to restriction of anticipated deformations.

4. CONCLUSIONS

Presence of un-envisaged shear zone/ fracture zone along the underground power house caverns of Mangdechhu Hydro-Electric Project, posed a serious threat to the stability of the caverns, while consultant suggested shifting the caverns laterally outwards towards the valley, MHPA opted for in-situ treatment of the weak feature to avoid loss of time and increase in cost. To ensure the stability of the structure a systematic and advance methodology was adopted whereby the realistic three dimensional behavior of the rock mass during excavation was simulated by numerical modelling and based on the simulated results an optimum excavation methodology and sequence of excavation along with optimum support system was designed. The behavior of the rock mass was carefully monitored by aid of extensive and systematic geotechnical instrumentation program. The model was progressively updated based on encountered geology and instrumentation observations and the installed support system was validated after excavation of each bench. Based on the progressive simulation of rock mass behavior, excavation methodology and support system was regularly modified and optimized. With the aid of progressive three dimensional numerical modelling analysis, excavation of the underground power house caverns was successfully completed, within the scheduled time. The deformations observed along the caverns were much less than the maximum anticipated deformation as obtained by numerical model studies, validating the adopted excavation methodology and sequence and installed support system. We must understand that the both normal and shear stiffnesses increase drastically when stresses are reduced (distressed) across the joints.

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