Hydraulic Fracture Testing for the Xe Pian - Xe Namnoy HPP

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Abstract

The Xe Pian - Xe Namnoy Hydroelectric Power Project is a high head hydropower scheme under construction. The project straddles the Champasak and Attapaeau Provinces of southern Lao P.D.R. It is located on the Bolaven Plateau and its flanking slopes to the south. The project includes two trans-basin water transfer systems, from Houay Makchan to Xe Pian and from Xe Pian to Xe Namnoy catchments, two dams on the Xe Pian and Xe Namnoy Rivers, power waterways (low pressure headrace tunnel, high pressure shaft, high pressure tunnel and a long external penstock) leading to an outdoor powerhouse with 430 MW total capacity under a rated head of 655 m, and terminal tailrace channel discharging to the Xe Kong River.

For economic and safe design and construction of any underground structures one of the most important pieces of information to obtain is knowledge of the in-situ stress conditions. For the design of the liner requirements of the pressure shaft and high pressure headrace tunnel and in particular the requirement for steel liner within the high pressure headrace tunnel, the in-situ stress regime needed to be evaluated. To do this first hydraulic-fracturing testing was carried out during the feasibility study in 1997/98, but the results were questionable. In 2011, further hydro-fracture testing at 20 intervals within two deep boreholes PSDH-1 and PSDH-2 was conducted.

This paper presents a brief theory of hydraulic-fracturing testing, the execution of the tests, a discussion of the results and their use for safe and economical design of the underground works for the Xe Pian - Xe Namnoy HPP.

Key words: In-situ stress, underground structures, hydraulic-fracturing testing

1 Introduction

Feasibility studies for the Xe Pian - Xe Namnoy Hydroelectric Power Project were first prepared in 1995 by Electrowatt Engineering Services, Ltd. for the Korean Company Dong Ah Construction Industrial Co., Ltd. It consisted of a series of studies undertaken over several years, which lead to the proposal of a project layout. Following the accomplishment of the 1995 feasibility study, Harza Engineering Company International, L.P. carried out the Preliminary Design, which was completed in 1997. In the same year, construction work by Dong Ah commenced but was halted by the Asian financial crisis soon after.

SK E&C decided to resume the project and concluded a memorandum of understanding with the Committee for Planning and Investigation of Lao People's Democratic Republic (Lao P.D.R.), signed in August 2006, to build, own, operate, and transfer the Hydroelectric Power Scheme. In 2007 AF-Consult Switzerland Ltd. (formerly Colenco Power Engineering) provided technical services regarding the Project's feasibility, focusing in particular on Project optimization, environmental and social impact studies. The Xe-Pian Xe-Namnoy Power Company Limited (PNPC) was founded in March 2012 by the four shareholders namely SK Engineering and Construction (SK E&C), Korea Western Power (KOWEPO), Ratchaburi Electricity Generating Holding PCL. (RATCH), and Lao Holding State Enterprise (LHSE). The Basic Design, the preparation of the EPC Contract Documents and the assistance to the EPC Contract negotiations..

1.1 Project description

The Project is located in southern Lao P.D.R. on the Bolaven Plateau, approximately 550 km to the southeast of the capital Vientiane, 80 km to the east of Pakse on the Mekong River, and 35 km to the west of Attapeu, a town located on the Xe Kong River south of the plateau (Fig.1).

Fig.1. Project location

The main components of the Project are a seasonal storage reservoir impounded by a dam on the Xe Namnoy River, long underground waterways to develop a head of some 650 m, exploited in an open air powerhouse at the foot of the Bolaven Plateau, and a tailrace channel that connects the powerhouse with the Xe Kong River.

The water resources available for the Project are enhanced by diverting the run-off of two adjacent watersheds, Houay Makchan and Xe Pian to the Xe Namnoy reservoir. The diversion is made in cascade, from the Houay Makchan catchment to the Xe Pian reservoir, and from there to the Xe Namnoy reservoir.

The main elements of the underground works of the Xe Pian - Xe Namnoy HPP are; a power intake structure at the Xe Namanoy reservoir, a horizontal 13.7 km long low pressure headrace tunnel (HRT) with 5.0 m internal diameter, a vertical pressure shaft (485 m) of internal diameter 4.4 m and a horizontal 1.620 m long high pressure HRT of internal diameter 4.4 m and 3.6 m, which leads to the surface penstock (Fig.2). At the upper elbow between the low pressure headrace tunnel and pressure shaft a valve chamber (17.2x16.2x14 m) is constructed which houses a butterfly valve. A surge chamber is constructed directly upstream of the valve chamber.

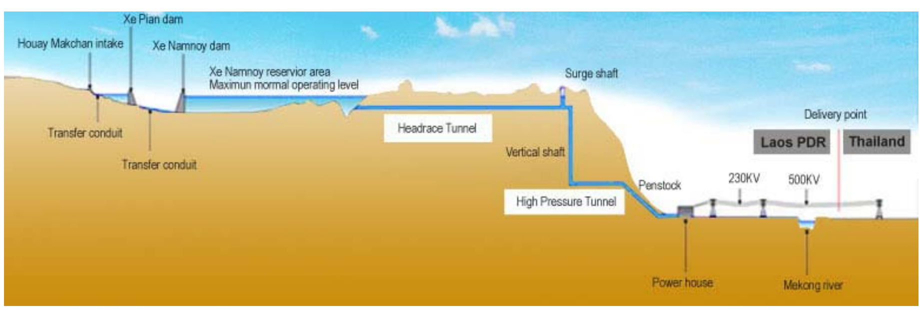

Fig.2.Schematic layout of the Xe Pian - Xe Namnoy HPP

1.2 Geology/Topography

The Bolaven Plateau lies between elevations 800 m a.s.l. and 1300 m a.s.l. with the highest peaks at the southern edge. The Xe Kong valley to the south lies between elevations 80 to 100 m a.s.l. (Fig. 2). A high escarpment exists at the southern margin, which enables a high head to be developed.

The geology of the Bolaven Plateau comprises essentially a sequence of lower Jurassic to Cretaceous terrestrial sandstones, siltstones and mudstones deposited within the Khorat Basin of the eastern South East Asia Peninsula, these formations dip gently to the North (Fig. 3). Pliocene to quaternary basalts cap this sequence due to late basin extension.

Legend:

β II-IV	Quaternary	Basalt
β N_2 - Q_1:	Pliocene-Pleistocene	Basalt and Laterite
J_3 - K:	Upper Jurassic – Cretaceous	'Champa formation': Sandstone and Siltstone, Conglomerates
J $_{1-2}$:	Lower – Middle Jurassic	'Tholam formation': Siltstone, Clayshale and Sandstone
T $_{1-2}$:	Trias	Volcanites: Rhyolites, Dacite, Basalt
	Dam / Power House Sites	

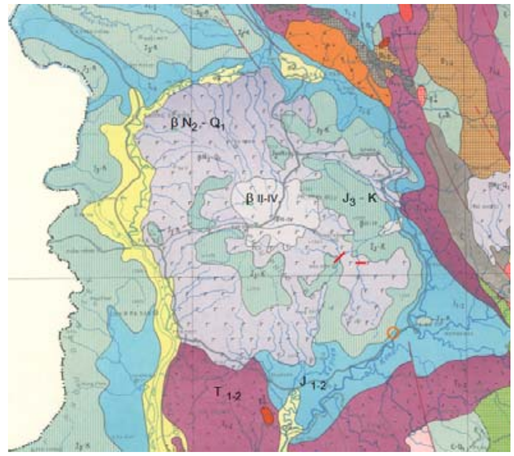

Fig.3. Regional Geology of the Bolaven Plateau

The sedimentary sequences, relevant to the underground works, are divided into the 'Tholam' Formation (Lower – Middle Jurassic) and the unconformably overlying 'Champa' Formation (Upper Jurassic – Cretaceous). The 'Tholam' Formation comprises essentially siltstones and mudstones with subordinate sandstone, whereas the 'Champa' Formation is dominated by Sandstone with capping strong coarse sandstones and conglomerate.

1.3 Stress measurement programme

For the design of the underground structures and in particular the requirement for steel liner within the high pressure headrace tunnel, the in-situ stress needed to be evaluated. During the feasibility study in 1997/98 hydraulic-fracturing tests had already been carried out, but the results were not considered reliable. Therefore, for the basic design, hydraulic fracturing testing was proposed to be carried out at 20 intervals within two deep boreholes PSDH-1 of 750 m depth and PSDH-2 of 440 m depth. PSDH-1 was located at the top of the slope at elevation 946 m a.s.l. close to the planned alignment of the vertical pressure shaft. PDSH–2 was located further down the slope towards the powerhouse at elevation 635 m a.s.l.

2. Hydraulic-fracturing testing background/theory

Since the first hydraulic-fracturing (hydrofrac) stress measurements by H. von Schoenfeld in an underground mine in northern Minnesota in 1968, the technique has been applied in thousands of shallow to (ultra-)deep boreholes all over the world and has gained the interest of engineers for the planning and design of underground excavations. The method is simple and straight forward: a part of a borehole (usually less than 1 m) is sealed-off with a straddle packer and is subsequently pressurized by fluid injection to induce and propagate a tensile fracture in the borehole wall rock.

The classical treatment for hydraulic fracturing is based on the Kirsch's solution for the stress distribution around a circular hole in a homogenous, isotropic, elastic material subjected to far-field compressive stresses. It was used by Hubbert and Willis (1957) who stated that a fracture will be induced if the acting fluid pressure in a hole exceeds the minimum tangential stress and the tensile strength of the rock. For fracturing in vertical boreholes drilled from the surface this is generally expressed by the relation:

$$P_c = 3 \cdot S_h - S_H + T - P_p \qquad (1)$$

Where the critical pressure at fracture initiation, P_c, is denoted as breakdown pressure, S_h and S_H are the minimum and maximum horizontal principle stresses, T is the rock tensile strength and P_p is the pore pressure. Since one also assumes that the fracture propagates in the direction of the least resistance, the pressure to merely keep an induced vertical fracture open is equal to the minimum principle horizontal stress:

$$P_{si} = S_h \tag{2}$$

In practice, P_{si} is called the shut-in pressure. Neglecting the pore pressure, the principle stresses can be than expressed by:

$$S_h = P_{si} \tag{3}$$

$$S_H = 3 \cdot P_{si} - P_r \tag{4}$$

$$S_v = \rho \cdot g \cdot z \tag{5}$$

Where $P_r = P_c$ - T is the pressure to re-open an induced fracture during subsequent pressurization cycles. The assumption that for a flat-lying region, the vertical principle stress S_v is equal to the weight of the overburden rock with given density ρ is a matter of pure static's and force equilibrium (Jaeger and Cook, 1969) and is confirmed by numerous measurements.

The classical approach neglects the fact that rocks contain pre-existing fractures with different orientations with respect to the orientation of the principal stresses. By fluid injection into a sealed-off borehole interval containing such a fracture, it will open as soon as the fluid pressure exceeds the normal stress S_n acting across the (arbitrarily oriented) fracture plane. In this case the shut-in pressure P_{si} to keep the fracture open after the pressurizing system is shut-in is equal to the normal stress S_n. Solutions to compute the in-situ stress-field from the values of observed normal stresses by inversion techniques were presented by Cornet (1986) or Baumgärtner and Rummel (1989). This requires that at least 5 values of S_n at various depths on fractures with different orientation are available. The procedures are known as HTPF-method (hydraulic testing of pre-existing fractures) or as P_{si}-method in the literature. The inversion solutions are attractive since no assumptions on the pore pressure are necessary for deriving the principal stresses.

3. Test equipment and test procedure

The hydraulic-fracturing tests in boreholes PSDH-1 and PSDH-2 were carried out by using a wireline hydrofrac system, described in detail by Rummel (2002). The technique allows fast 'stress-logging' in the absence of an on-site drill-rig. Furthermore, compared to conventional systems with drill-rods, the wireline approach has the advantage of a high hydraulic system stiffness, which enables detailed fracture growth control and the possibility of online downhole high-resolution pressure recording. A schematic view of the system with all its main components is shown in Fig.4.

In the case of the hydraulic fracturing experiments in the 96 mm diameter boreholes, a straddle packer assembly with Kevlar-reinforced packer elements of 40 MPa pressure capacity (OD : 91 mm) was used. The tool is moved within a borehole on a seven-conductor logging cable with a mobile winch. The packers and the injection interval are pressurized via a stainless steel coil tubing which is clamped to the wireline cable. A mechanically operated push-pull valve on top of the packer assembly permits to switch from packer pressurization to injection into the 0.7 m long test interval. A mechanically operated push-pull valve on top of the packer assembly permits to switch from packer pressurization to injection into the 0.7 m long test interval.

Hydraulic pressure is generated by a frequency controlled electric-driven three plunger pump with a maximum working pressure of 40 MPa and a maximum injection rate of 10 l/min. The measuring and recording system consists of appropriate pressure transducers downhole and on surface, a surface flow-meter, and a digital data acquisition system (16 bit resolution, 5 Hz sampling rate). The orientation of induced or stimulated fractures was determined by an impression packer tool. The test conduction closely followed the recommendations of both, the ISRM standard (2003) and the ASTM standard D4645 (1997). A typical test record to illustrate the test procedure is shown in Fig. 5 for the test at 736.0 m depth in borehole PSDH-1.

Prior to fracturing the permeability of the rock mass is measured by rapidly pressurizing the test interval to about 2-3 MPa and observing the subsequent pressure decline in the hydraulically closed system. After complete venting of the test interval, the pressure is increased again until either a sudden pressure drop related to the initiation of a fracture in the borehole wall (frac cycle) or to the stimulation of an existing fracture. In both cases, the initiated or stimulated fractures are extended during several subsequent pressure cycles (refrac cycles) and a final step-rate injection cycle. During each test, a total volume of 20-50 liters of water was injected which was partly recovered during depressurizations depending on whether the induced or stimulated fracture remain isolated or intersect an open joint system. Such tests were repeated at different depth sections to derive a stress-depth profile.

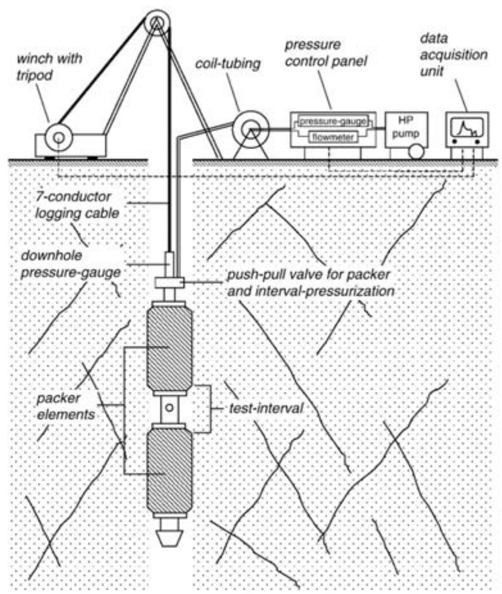

Fig.4. Schematic view of the wireline hydrofrac system

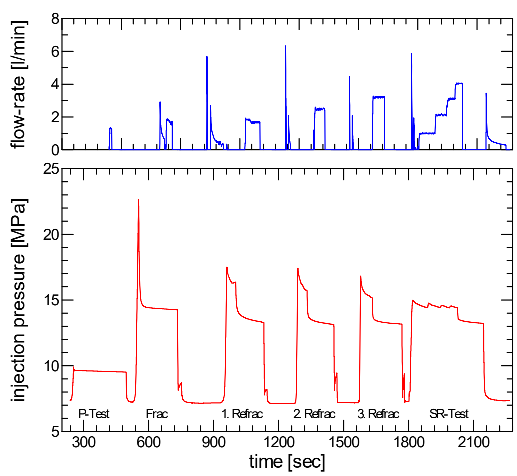

Fig.5. Downhole injection pressure and surface flow-rate record of the hydrofrac-test at 736.0 m depth in borehole PSDH-1

4. Results

Because the accuracy of hydraulic fracturing stress measurements strongly depends on the correct interpretation of the pressure-time records obtained during the tests, the characteristic hydrofrac pressure values breakdown pressure P_c, refrac- or re-opening pressure P_r, and shut-in pressure P_{si} were identified using various graphical procedures discussed by Baumgärtner and Zoback (1989). The results of the analysis together with the orientation of induced or stimulated fractures are summarized in Table 1.

Most of the impression packer test results show that NE-SW striking axial (dip $\alpha = 90°$) or steeply inclined fractures (dip $\alpha \geq 70°$) were initiated or stimulated. Therefore the stress estimation was carried out on the basis of the Hubbert and Willis (1957) concept by neglecting the ambient pore pressure. The results are also presented in Table 1. Since the derived in-situ horizontal principal stress data do not indicate a systematic depth dependency, the results were summarized by mean stress magnitudes at the corresponding mean depth ranges:

borehole	mean depth [m]	S_v [MPa]	S_h [MPa]	S_H [MPa]	θ_{SH} [deg]
PSDH-1	396.5 ± 92.5	10.5 ± 2.5	6.2 ± 0.7	11.6 ± 2.6	N 43 ± 9
	664.0 ± 72.0	17.6 ± 1.9	14.5 ± 0.4	24.3 ± 2.1	
PSDH-2	370.5 ± 61.0	9.8 ± 1.6	5.9 ± 0.6	10.7 ± 1.0	N 37 ± 12

where S_h and S_H are the minor and major horizontal principal stresses, and θ_{SH} the orientation of S_H with respect to magnetic North. The vertical principal stress S_v was calculated assuming a mean overburden rock mass density of 2.7 g/cm^3:

$$Sv\ [MPa] = 0.0265 \cdot z\ [m] \quad (6)$$

As shown in Fig.6, the results of the hydrofrac tests demonstrate that in borehole PSDH-1 and PSDH-2 between approx. 300 m and 490 m (in PSDH-1) / 430 m (in PSDH-2) the stress situation is characterized by $S_h < S_v \leq S_H$, where the minimum horizontal principle stress S_h is the least principle stress with rather consistent estimations of the horizontal stress magnitudes. The deeper tests in borehole PSDH-1 between approx. 590 m - 740 m suggest higher horizontal stresses with $S_h < S_v < S_H$.

The stress measurements in boreholes PSDH-1 and PSDH-2 yield consistently an approximatelyNE-SW orientation of the major horizontal principal stress S_H (N 43° ± 9° in borehole PSDH-1, N 37° ± 12° in borehole PSDH-2).

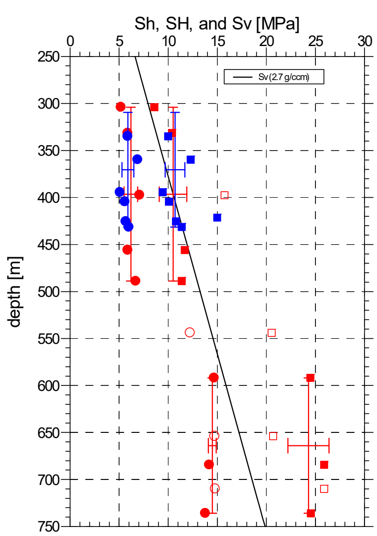

Fig. 6. Principal stresses in borehole PSDH-1 (red symbols) and PSDH-2 (blue symbols), circles: S_h, squares: S_H)

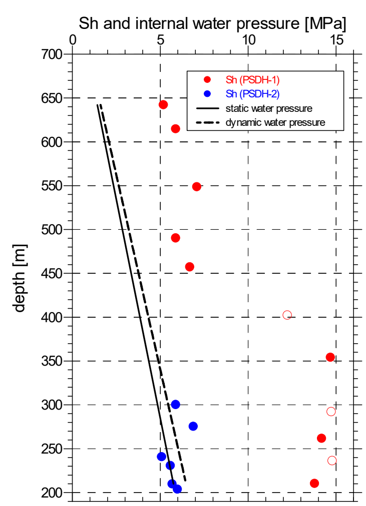

Fig. 7. Minimum principle stresses and internal water pressures vs. elevation

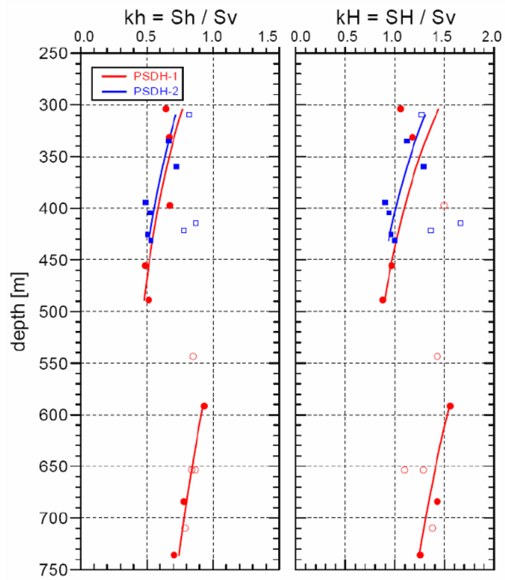

Fig. 8. Normalized stress plots for boreholes PSDH-1 and PSDH-2

5. Steel liner design

a. Introduction

At Feasibility design stage (2007) based on very low derived minimum principle stresses from hydraulic-fracturing testing, and low modulus of deformation of the rock mass a steel liner was proposed as the most adequate solution for the entire length of the vertical pressure shaft and the high pressure headrace tunnel. Additional hydraulic-fracturing testing was carried out in an attempt to optimize the design.

b. Hydraulic conductivity

The data from the pulse tests carried out in both PSDH-1 and PSDH-2 indicated rock mass hydraulic conductivities of the order 10^{-9} - 10^{-11} m/s. Hydraulic conductivities were slightly raised in PSDH-1 between 400 m and 500 m (elevation 550 – 450 m a.s.l.) to 10^{-9} - 10^{-8} m/s. In PSDH-2 the hydraulic conductivities between 400 m and 450 m (230 – 200 m a.s.l.) were raised slightly to10^{-10} - 10^{-9} m/s. The step rate tests indicated generally higher rock-mass hydraulic conductivities in the range10^{-8} - 10^{-7} m/s. In PSDH-2 the hydraulic conductivities between 400 m and 450 m (230 - 200 m a.s.l.) were raised to range 10^{-7} - 10^{-6} m/s. Generally it was considered that the rock mass was is characterized by very low hydraulic conductivity with low seepage.

c. Rock mass jointing

The impression packer data from both PSDH-1 and PSDH-2 indicated a dominant opening of sub-vertical joints with strike NW-SE to E-W. It was assumed that any potential seepage losses would be by hydraulic jacking of these joints

d. Principle stresses

A plot of the minimum principle stresses with the anticipated internal water pressure in the pressure shaft vs. elevation (Fig. 7) shows that at the location of the vertical pressure shaft (PSDH-1) the internal water pressure cannot induce hydraulic fractures. However, at the intercept position of PSDH-2 with the high pressure headrace tunnel 695 m downstream of the bottom elbow of the pressure shaft the minimum principle stresses is approximately equal to the internal water pressure of the power waterways.

e. Final design

Based on the data derived from the 2011 campaign it was proposed that no steel liner was required in the vertical pressure shaft. A liner was still required due to the low deformation modulus and slaking potential of the mudstone- horizons present and a steel reinforced concrete lining was proposed in the basic design. It was further proposed by the designer that the length of steel liner in the horizontal section could, by linear interpolation of the hydraulic fracture data between the intercept points of PSDH-1 and PSDH-2 with the tunnel alignment, be further optimized. At a chainage 370 m downstream of the bottom elbow of the pressure shaft the minimum principle stress is greater than the internal water pressure with a factor of safety of 1.5 and thus a total length of 1,250 m steel lined section was proposed. The first 370 m after the bottom elbow was to be steel reinforced concrete lined. Higher specification reinforcement was proposed at the elbow. Prior to final detail design it was specified that further hydraulic fracturing tests have to be carried out from within the excavated high pressure headrace tunnel

6. Vertical pressure shaft: Excavation methodology

An interesting result came from the execution of the hydraulic-fracturing test programme that can be seen in Fig. 8. The S_H/S_v ratio in PSDH-1, which record the entire geological section, reduces with depth from 1.5 down to unity, which is a normal phenomenon due to residual horizontal stresses remaining after erosion and exhumation of a rock mass. However, below 500 m, the S_H/S_v ratio increases again to 1.5 and thereafter reduces to the base of the borehole. This phenomena corresponds with the unconformity between Tholam' Formation and 'Champa' Formation. It was considered that this stress anomaly present within the mudstone siltstone sections may result in engineering problems during excavation. To reduce rock mass disturbance and enable control monitoring, excavation by raise boring was proposed, which is also much safer and easier to muck out.

7. Closing comments

The execution of high quality hydraulic-fracturing tests for the design of underground structures, not only for the present study, but also for cavern design generally, is of paramount importance. Although the initial investigation investment cost may appear high, much greater savings are possible in the design and greater safety is possible during execution.

Acknowledgments

The Authors wish to acknowledge gratefully the owners Xe Pian Xe Namnoy Power Company Limited comprising the four shareholders, SK Engineering and Construction (SK E&C), KoreaWestern Power (KOWEPO), Ratchaburi Electricity Generating Holding PCL. (RATCH), and Lao Holding State Enterprise (LHSE).

The in-situ hydrofracture tests were conductes by MeSy-Solexperts GmbH (Bochum, Germany) in close cooperation with GDP Corp. Ltd (Bangkok, Thailand). We are particularly grateful to Mr. B. Mangkongkam (GDP Corp.) for his continuous support.

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Table 1: Results of hydraulic fracturing tests in boreholes PSDH-1 and PSDH-2 (P_c: breakdown pressure, P_r: refrac pressure, P_{si}: shut-in pressure, θ: fracture strike direction counted N over E, α: dip of fracture with respect to horizontal, S_v: vertical stress, S_h: minimum horizontal stress, S_H: maximum horizontal stress, θ_{SH}: direction of maximum horizontal stress).

depth (m)	P_c (MPa)	P_r (MPa)	P_{si} (MPa)	θ (deg.)	α (deg.)	S_v (ρ=2.7 g/cm^3) (MPa)	S_h (MPa)	S_H (MPa)	θ_{SH} (deg.)
borehole PSDH-1									
304.0	9.6	7.0	5.2	56	75	8.1	5.2	8.6	N 56
331.5	9.6	7.3	5.9	53	74	8.8	5.9	10.4	N 53
397.5	-*	(5.55)	7.1	34	90	10.5	7.1	(15.75)	N 34
456.0	-*	6.0	5.9	41	90	12.1	5.9	11.7	N 41
489.0	-*	8.7	6.7	39	90	13.0	6.7	11.4	N 39
544.0	20.9	16.2	12.25	13	63	14.4	(12.25)	(20.55)	(N 13)
592.0	22.7	19.6	14.7	33	90	15.7	14.7	24.5	N 33
654.0	25.7	22.7-24.4	14.5-15.0	45	46	17.3	(14.5-15.0)	(19.1-22.3)	(N 45)
684.4	28.0	16.7	14.2	34	90	18.1	14.2	25.9	N 34
710.0	21.5	17.0	14.8	44	63	18.8	(14.8)	(25.9)	(N 44)
736.0	22.6	16.85	13.8	50	76	19.5	13.8	24.55	N 50
borehole PSDH-2									
309.5	15.0	9.7	6.7	54	61	8.2	(6.7)	(10.4)	(N 54)
335.0	12.5	7.7	5.9	38	75	8.9	5.9	10.0	N 38
359.8	16.3	8.4	6.9	44	72	9.5	6.9	12.3	N 44
394.5	15.9	5.85	5.1	48	72	10.45	5.1	9.45	N 48
404.5	-*	6.7	5.6	49	90	10.7	5.6	10.1	N 49
414.5	24.8	10.5	9.6	2	70	11.0	(9.6)	(18.3)	(N 2)
421.5	17.6-19.4	10.75-11.1	8.7	0	61	11.2	(8.7)	(15.0-15.35)	(N 0)
425.5	13.9	6.3	5.7	24	77	11.3	5.7	10.8	N 24
431.5	16.3	6.6	6.0	21	79	11.4	6.0	11.4	N 21

* stimulation of pre-existing fractures   () neglected since fracture dip is < 70° or strike direction not NE-SW