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MISE A JOUR DES ETUDES ET ASSISTANCE TECHNIQUE POUR LA CONSTRUCTION DU BARRAGE DE BISRI

BARRAGE BISRI

AVANT PROJET DETAILLE

PIECE 1: DAM PRESENTATION AND JUSTIFICATION REPORT

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Table of Material

List of Tables

List of Figures

List of Appendices

1 INTRODUCTION

The present note was prepared within the framework of the dossier of Bisri dam preliminary detailed design (Avant Projet Détaillé “APD”). It is intended to put forward and to justify the concept of all project components excepting the power plants that are subject to separate studies.

This note is also considered as a synthesis that grasps the whole set of designs that were performed to date, and revisits the main conclusions of the notes of the present study, particularly the ones related to geological and geotechnical aspects, and to hydraulics and stability calculations.

The complete list of contents pertaining to the present dossier is given in appendix A. However, for a better understanding, the present note should be scrutinized in parallel with the base drawings defining the works. These are custom drawings intended for the definition of the design. They are also used in the tender documents of the dam construction. The detailed and the execution drawings are to be prepared at the commencement of construction works.

2 HISTORY

The site of Bisri has raised an interest for the realization of a water storage dam to feed the city of Beirut with potable water since the beginning of the fifties. The Bureau Of Reclamation has carried out in fact a primary study between the years 1951 and 1954, that was followed up by the Office National du Litani. Two axes named “A” and “B” were envisaged, the first, more to the downstream, is close to the Anan sliding (see later), and the second is situated approximately 450m to the upstream of the Bisri village.

It appeared right at the beginning that the alluvial deposit in valley would reach a thickness in the range of 100 m and this is directly related to the Anan sliding. It is in fact established that a major sliding took place near the site of Anan, the sliding has barred the river bed therefore creating a reservoir that has been totally filled later on with sediments. This important fact is largely detailed in the geological report included in the present dossier.

The joint venture ECI/DAHNT has been later on assigned to resume the design between 1982-1984, then between 1994-1995, and during 1997 without being able to achieve the preliminary detailed design (APD). In particular, the results pertaining to the site investigations that were done during 1996 has not been completely integrated in this design phase. It has been considered that the most appropriate setting (location) for the dam implementation corresponds to the axis named “C” situated more to the upstream, where the main investigations took place. It has been considered that the documents produced during the latest design phase are practically corresponding to the expected tendering documents necessary to launch the tender procedures of the execution works. Accordingly the contract that was ratified on 2012 with the JV Dar Taleb/NOVEC was mainly aiming to update previous studies on one hand, and to prepare the tender documents for the selection of the Contractor, on the other hand.

However, it has been quickly revealed that the selected options framework and the interpretations of the geological data should be reviewed, especially that the results of the investigation campaign that was lead in 1996-1997 were partially taken into account.

Moreover, an additional investigation campaign was launched regarding the ancillary structures as well as the dam foundation. The campaign included boreholes with water tests, some were equipped with piezometers along with in-situ and laboratory tests. The whole acquired data on this instance has been integrated in the geological and geotechnical reports included in the present dossier.

It is however outlined that the site hydrological parameters, notably the floods estimation, have been subject to modifications in light of the new available hydrological data till year 2010. Particularly the project’s flood and volume were decreased compared to what has been considered earlier.

Within this context, the present dossier represents a complete recapitulation of the whole set of earlier designs and studies, with mainly the new design of the dam as well as of the ancillary structures.

The Council for development and reconstruction (CDR) as Employer supported by a comity of international experts assured the follow-up and the approval of the design since the start in 2012 through the organization of periodic coordination meetings. In addition, periodic site visits which aimed to follow up and to be aware of the investigations outcomes have been regularly made.

The list of the documents of this dossier is given in appendix A. It was not possible to come across any document related to the design of the fifties.

3 GENERAL DATA

3.1 Location and Accessibility

The Bisri site is situated at approximately 17km from the city of Saida and 30km to the south-east of Beirut.

The dam axis is limited by the 2 points A and B with the following coordinates:

A : X= -334 420	Y= -62 500
B : X= -334 830	Y= -61 710

Starting from Beirut, the Bisri Dam spot can be accessed following the itinerary below:

- From Beirut drive the south highway in the direction of the city of Saida (43 km).
- Before entering Saida, turn left to the east (one of the main collector road) and go up in the direction of Alman, Joun, Deir El Mkhaless and Khirbet Bisri (17 km).
- Before reaching the bridge of Bisri crossing over the river Awali, turn left to the north (dam site access road to right abutment) to reach Bisri Dam crest (1.5 km).
- After the bridge of Bisri crossing over the river Awali, turn left to the north (dam site access road to left abutment) to reach Bisri Dam crest (1.3 km).

3.2 Climatology

The climate in the project area is moderately cold, windy and wet in the winter and warm and dry in the summer and fall.

The mean of the monthly relative humidity at the project site varies from a minimum of 55.6 percent in the month of May to a maximum of 72.1 percent in the month of January.

The average temperatures are shown in the table below:

YEAR	SEPT	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUNE	MAY	AUG
AVERAGE	20.3	17.5	13.6	9.5	7.7	7.9	9.9	13.3	17.4	21.0	22.3	22.7

The wind records are available at KSARA observatory, A.U.B., and at a station located in the project watershed at the College of Machmouche near Jezzine. Maximum wind recorded at the College of Machmouche was a gust with a velocity of 47 m/s.

3.3 Hydrology

3.3.1 Streamflow

The elevation in the drainage basin ranges from 395 m at the dam site to over 1,900 m above mean sea level with a total area of 215 Km^2.

Average annual flow available to the project at the Bisri dam site is estimated as ~130 Mm^3 (4.3 m^3/s), with annual values ranging from a high of 434 Mm^3 (13.8 m^3/s) to a low of 55 Mm3 (1.7 m^3/s).

3.3.2 Diversion Inflow Flood

A flood with a return period of 100 years was considered suitable for diversion purposes at the Bisri dam site. Diversion inflow flood was derived based on flood frequency of annual peak discharges and the shape of the hydrograph was patterned after the April 1971 flood. The 100-year peak value of 600 m^3/s was adopted as the diversion flood for the Bisri Project.

3.3.3 Spillway Design Flood

The presently accepted method of defining spillway design floods for major dams is based on hydro meteorological analysis. The spillway design flood for the Bisri dam was derived from 24-hour basin PMP of 441 mm depth. An exponential hydrograph based on historical flood hydrograph was developed and used to convert the PMP to the Spillway Design Flood.

The spillway design flood for Bisri Dam has a peak discharge of 2,300 m^3/s and a 24 hour volume of 43Mm^3 it correspond the maximum probable flood event.

3.3.4 Reservoir siltation

Siltation of the reservoir is estimated to reach a total volume of around 3 and 9 Mm^3 at respectively 20 and 50 years.

4 SEISMOLOGY

The geology and seismotectonic framework that served as the basis for the recommended seismic loading criteria are described in the following documents, available in appendix B:

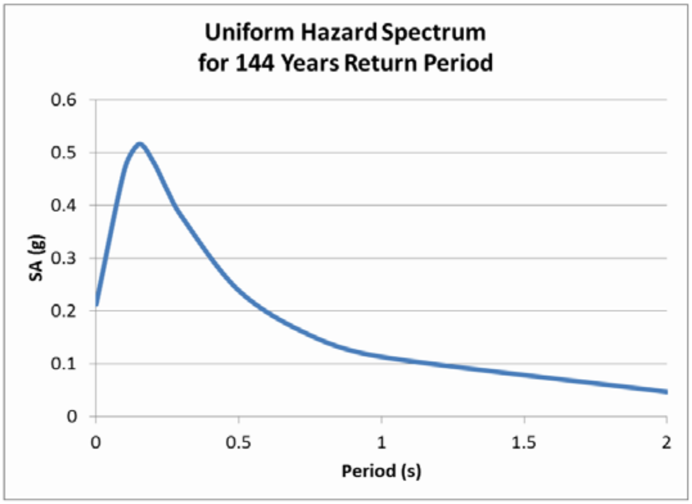
- «REPORT ON THE NEO-TECTONIC SETTINGAND SEISMIC SOURCES FORTHE SEISMIC HAZARD ASSESSMENTOF THEBISRIDAM SITE», AtaRichardELIAS PhD, June2014
- «ASSESSMENT OF SITE-SPECIFIC EARTHQUAKE HAZARD FOR BISRI DAM, LEBANON», M. Erdik, K. Şeşetyan, M.B. Demircioğlu, E. Harmandar, 0ctobre 2014

The first report concluded that different geological evidence and observations of various nature (structural, seismotectonic and geomorphic) from the area are hard to reconcile with the assumption that an active fault runs in the subsurface of the Marj Bisri as assumed in the previous design. In particular, the Roum Fault does not appear to continue in the Marj. There is no evidence for 3m[1] of fault offset associated with any known fault under the dam site. Some minor and small faults with very low tectonic deformation rate accommodating syn or post depositional deformation in the thick sedimentary infill or stress within the Jezzine anticline or at the tip of the Roum Fault may exist in the subsurface of the Marj. Their surface expression are leveled by the much more active geomorphic surface processes. If so these faults are very likely of small extent and should not represent a serious tectonic hazard to the region.

The second report, based on a thorough analysis of the earthquake hazard at the Bisri Dam site, comes to the following earthquake resistant design basis spectra:

The Operating Basis Earthquake (OBE) is determined as the probabilistically assessed earthquake ground motion for an average return period of 144 years. The horizontal and vertical design basis spectra for the operating basis earthquake (OBE) for the random horizontal and vertical component are provided respectively in Figure 1 and Figure 2. The response spectra is provided for 5% damping and for the free-field engineering bedrock outcrop. Under the action of this level of ground motion, the dam, appurtenant structures and equipment should remain functional and, if any, the minor damage should be easily repairable.

Figure 1: Uniform Hazard Spectrum corresponding to 144 years return period for the Bisri Dam site



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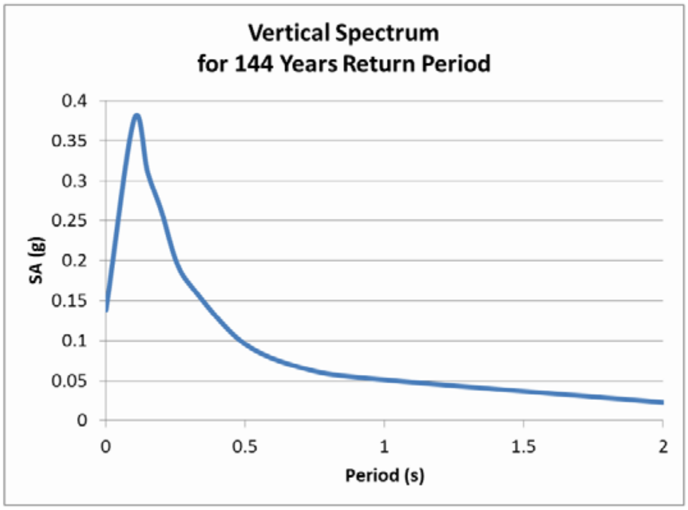
[1] In the previous design it has been assumed that Roum Fault corresponds to the fault identified in the valley bottom (see transversal geological cross sections), and that this fault may experience an offset of 3 meters.

Figure 2: Vertical response spectrum corresponding to 144 years return period, obtained using coefficients after Bommer et.al, 2011)



The Safety Evaluation Earthquake**(SEE)** is determined to correspond to the 84-percentile deterministic MCE. The horizontal and vertical design basis spectra for the safety evaluation earthquake (SEE) for the random horizontal are provided in Figure 3. Since the time domain analysis will be used for the SEE level design 7 sets of spectrum compatible scaled ground motion acceleration data are provided to enable the time-domain analysis. The vertical spectrum should be taken as the average of the vertical spectra of these sets of ground motion indicated in Figure 4. The 5% damped response spectra and the 7 sets of spectrum compatible scaled ground motion provided are applicable for the free-field engineering bedrock outcrop. Under the SEE the stability of the dam and life safety must be ensured with no uncontrolled release of water from the reservoir. SEE is the maximum level of ground motion for which the dam should be designed.

Actually, only 3 sets scaled ground motion acceleration data have been used, because the numerical analysis that is carried out consumes very large time for each set. However the most powerful earthquake sets were selected and the final deformation (displacement) considered corresponds to the maximum obtained from the analyzed sets.

Figure 3: The average (Red, actually Figure 24) and the directivity modified design basis response spectra (Blue). Note that the directivity modification applies only to first periods greater than 0.5s.

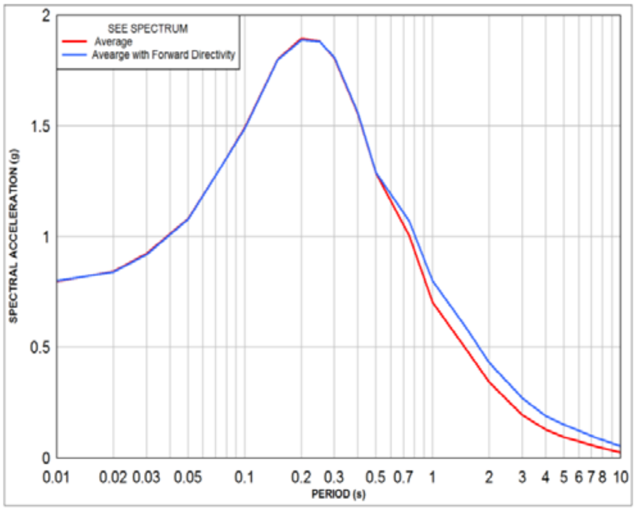
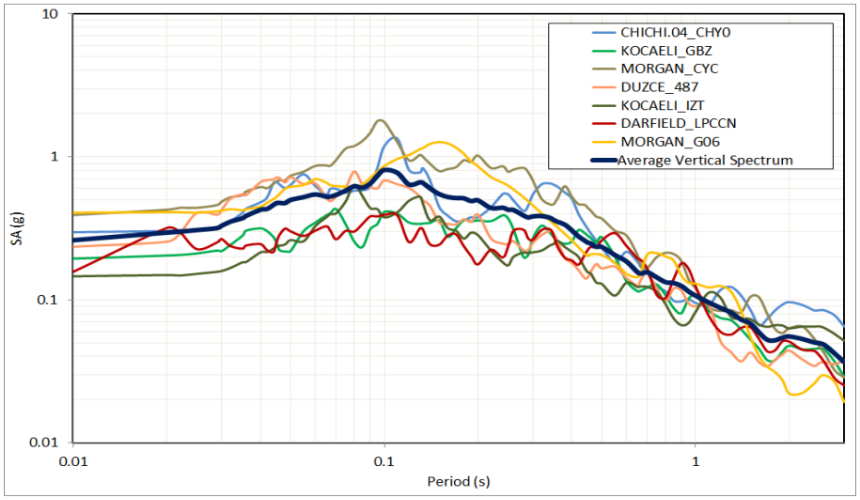


Figure 4: Average vertical spectrum of the 7 scaled vertical ground motion records



The following table summarizes the peak ground accelerations (PGA) of the design SEE and OBE seismic records that were adopted for the numerical dynamic simulations.

Table 1: PGA of design SEE and OBE earthquakes

SEE EARTHQUAKE				
		PGA [g]		
Name		**Vertical**	**Horizontal 1**	**Horizontal 2**
Darfield LPCC		+ 0.5 / - 0.58	+ 0.96 / - 1.33	+ 0.79 / - 0.89
Kocaeli IZT		+ 0.51 / - 0.43	+ 0.82 / - 0.63	+ 0.47 / - 0.59
Morgan CYC		+ 0.44 / - 0.43	+ 0.81 / - 0.62	+ 0.55 / - 1.49

OBE EARTHQUAKE				
		PGA [g]		
Name		**Vertical**	**Horizontal 1**	**Horizontal 2**
Chalfant A		+ 0.19 / - 0.24	+ 0.31 / - 0.20	+ 0.23 / - 0.30
Chalfant B		+ 0.20 / - 0.18	+ 0.13 / - 0.16	+ 0.30 / - 0.28
Darfield LPCC		+ 0.11 / - 0.13	+ 0.21 / - 0.29	+ 0.17 / - 0.20

5 SUMMARY OF THE MAJOR GEOLOGICAL AND GEOTECHNICAL FINDINGS

The dam site is geologically complex, characterized by the following elements:

- The stratigraphy within the abutments mainly consists of karstified and pervious Limestones, Marly Limestones and Sandstones in Monocline forms, ranging in age from middle-upper Jurassic to late Cretaceous. Layers dipping is different from one side to the other of the valley;
- A lacustrine deposit of almost 120m maximum depth which have been formed as a consequence of the large landslide which had blocked the river at Anane village (about 1.5 km downstream from the dam site). The deposit correspond to the siltation of the reservoir during the recent quaternary period. These soft soil stratums which cannot be removed due to their thickness, must be taken into account in designing the dam.

The geological complexity of the site required the realization of over 5 100 meters of exploratory drilling and multiple in-situ and laboratory tests. In 2015 additional investigations by CPTu (static cone penetration test with pore pressure measurement, Piezocone) was carried out as it was considered the most adapted to determine the most relevant geotechnical parameters of the soft deposit. A total of 30 CPTu representing 1590m soundings reaching a depth of 70m, were carried out.

Geological and geotechnical reports of this dossier, detail all the aspects of the nature and quality of the subsurface stratums, from which the most important ones are summarized hereunder:

1. The dam site is situated in an active tectonic zone in vicinity of the two major Roum and Yammouneh active faults[2]. The last two destructive earthquakes which have been reported with an epicenter within the Lebanese territory (M~7, 1837 and M~5.7, 1956) have been attributed to the Roum fault which has an estimated annual rate of movement of ~1mm. It is to be noted that the fault that have been identified and located at the left bank of the river during the previous study, is found inactive and not a branch of Roum fault.
2. Quaternary soil deposits occupying the valley floor with a maximum depth of 120 meters have been formed upon following the occurrence of the large landslide called "Anane" which is located downstream Bisri village. It created an artificial dam on the river bed. Sediments accumulated behind this landslide, are mainly lacustrine, heterogeneous and surely not totally consolidated. It is necessary to mention that these soil deposits cannot be removed from the dam site. The main properties of these soil deposits can be summarized as follows:
    - In principle, these soil deposits can be subdivided into two types, (1) gravely sand or sandy gravel particularly occupying the river stream to a maximum depth of 30 meters and (2) the silty clay mainly dominating the rest of the deposits. Old colluviums overlying the bedrock consist of gravels, cobbles, boulders and core stones of limestone more or less wrapped in a silty clay matrix, were particularly encountered within the right side of river valley.
    - The gravely sand or sandy gravel with maximum grain size of 100mm and with liquefiable passages (in accordance with the results of the SPT and CPTu performed) are relatively clean with fines (passing sieve number 200) up to 10% (4% in average) and are containing materials finer than 5mm of about 38% in average.

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[2] «ASSESSMENT OF SITE-SPECIFIC EARTHQUAKE HAZARD FOR BISRI DAM, LEBANON », M. Erdik, K. Şeşetyan, M.B. Demircioğlu, E. Harmandar, 0ctobre 2014.

- Silty clays interbedded with thin layers of fine sand and/or silt, are the core of the lacustrine soil deposit. The silty clays have a Plasticity Index and a Liquid Limit of 10 to 40% and 30 to 70% respectively. Interbeddings of fine sand and silt which are quite characteristic for the lacustrine soil deposits, will somehow accelerate the rate of consolidation of the lacustrine silty clays during the construction of the dam (the coefficient of consolidation in horizontal direction is considered equal to 10 times the vertical direction).
- CPTu, the most reliable in-situ testing carried out, provided valuable information for the estimation of undrained shear strength (Cu = 0.3xσ'v), horizontal coefficient of consolidation $6x10^{-7}$ m/s and compression index (Cc=0.42). Signs of under-consolidation derived from recent laboratory test results are finally non-confirmed by CPTu. Indeed these signs were associated to the artesianism observed in boreholes BDC15, BDC24, BDC35 and BDC39, however this should be very local with no real incidence on whole behavior of the deposit.
- The quaternary deposit is so thick and enough soft that expected settlement should be important reaching a maximum of around 5.5 to 8 meters at the highest part of the embankment (the vertical of the crest)[3]. This magnitude of settlement is so important that a vertical clay core embankment may be subjected to deformations that may disturb the transitions zones particularly the critical filter and drain. Furthermore, the cutoff wall placed in the continuity if the core may also be damaged although it is made of plastic concrete. The solutions considered to accommodate such deformation are mentioned later in this dossier such as shifting the cutoff wall to the upstream direction with inclined clay-core.
- The SPT and CPTu indicated potentially liquefiable layers in gravely sand and sandy gravel soil deposits. However, this problem can be considered as a minor compared to consolidation and poor foundation soil (lacustrine silty clays) problems. It has been however checked that liquefaction will impact only the embankment toes and couldn't jeopardize the dam integrity.

3. The right abutment mainly consists of Cretaceous Marly Limestone formations in a Monocline form. The water-table seems to be sloping toward the interior of this abutment which leads to think about significant permeability of this abutment without being judged as prohibitive to construct the dam. The general impression given by the right abutment investigation adit is favorable. However, in-situ borehole water permeability (Lugeon tests) indicated high permeabilities in Marly Limestone formations, irrespective of the depth.
4. The left abutment which is also in a Monocline form, consists of lower Cretaceous "Grès de Base" (C1) underlain by Jurassic Limestone formations at the base of the abutment.
5. The Jurassic limestone formations at the base of the left abutment are characterized by:
    - Significant decrease in permeability with depth and Interbeddings of low permeability Chocolate Marlstones
    - Moderately karstified as seen from the taken cores and low permeability (Lugeon values are significantly smaller than those of the Marly Limestone formations at the right abutment).

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[3] Settlement calculations carried out on transversal and longitudinal sections resulted in smaller settlement (5.5m) on the longitudinal corresponding to dam axis section. The large difference is surprising but may be related to the smaller confinement of the foundation on the transversal sections (upstream/downstream).

  - A mylonitized upper part, to a depth of 10 to 20m which is highly fractured, occasionally shattered or reduced to powder in places. The depth of the clay-core excavations at those areas is deepened.

6. The sandstone is variably cemented and disintegrated to sand in places. It is erodible and has a permeability coming mainly from porosity more than that from discontinuities. This Sandstone is actually exploited to produce sand directly through excavations by earthmoving machines without the need of blasting or crushing.

7. It is considered in those circumstances that a large storage dam is feasible for this site. However, the following precautions shall be taken into consideration:

   - The lacustrine soil deposits rich in silty clay should be cut off by a positive diaphragm wall anchored into the bedrock due to the presence of coarse alluviums particularly occupying the left side of the river valley to a maximum depth of ~30 meters and semi-pervious to pervious (depending on their clay content) old colluviums underlying the lacustrine soil deposits (slope wash present on the natural valley slopes before the occurrence of the Anan sliding).

   - The concept of the dam should take into consideration the major deformation of the lacustrine deposits, particularly the cutoff wall to be constructed within these deposits and the transition material in contact with the clay-core.

   - The treatment of the foundation outside the deposits (the bedrock) should lead to limit the leakages to acceptable values. Measures should be taken to allow complimentary treatments after filling the dam in the case of unacceptable water losses.

   - The erodibility of the Sandstone (Grès de Base) must also be taken into account in designing the foundation treatment system.

8. The available geological and geotechnical findings are considered sufficient and enough reliable for the elaboration of the present dossier (APD). Minor adjustments might be introduced at the beginning of the works, on the basis of the additional investigations to carry out. It may involve a trial embankment loading to confirm the lacustrine deposit behavior and deep drilling to check the permeability of the bedrock, along the grout curtain particularly on the right abutment.

6 CONSTRUCTION MATERIALS

Construction materials for the embankment of the dam will be extracted from the valley bottom quaternary deposit of the reservoir area and onsite/offsite rock material sources (outcrops) of strong and durable Limestone. These potential sources were investigated during this study and the previous ones. Boreholes were drilled (coring) within the nearby rock material sources (see Plan G23-16) and recovered samples were tested in laboratories. In the case of being unable to quarry the above indicated areas, active quarries currently under operation (offsite), can be used as rock material sources after testing.

The construction materials needed according to the chosen Dam design are given in Table 2.

Table 2: construction materials needed

Material	Estimated Quantities needed (m^3)	Quantities to be searched from extraction sites and quarries (m^3)
clay-core	920 000	1 300 000
rockfill	2 800 000	4 000 000
Alluviums	4 000 000	6 000 000
Transition material (upstream),	310 000	-
Filter	450 000	-
Drain	370 000	-
Riprap	100 000	-

The volume of rock to be searched from the quarry/quarries will be used also in the riprap production. Active River deposits (gravely sand and sandy gravel) will be used in the production of filter, drain and transition materials after screening and washing processes. Aggregates for concrete can be produced either from crushing the strong and durable limestone or from the active river deposits.

According to the investigations, around 1 million m^3 of clay-core material is available within the upstream terraces (hanging valleys). In the case of inadequacy, additional terraces 500 meters downstream from the axis, which are already under expropriation, can also be used as clay-core material sources after testing. It should be noted that the clay-core material sources are relatively heterogeneous due to the Interbeddings of gravel and sand/silt.

Few samples taken from the silty clays indicated the sign of dispersivity which doesn’t cause a major problem by particularly taking into account the use of well-designed filter and mixing the silty clays during extraction.

Active river deposits (sandy gravel and gravely sand) are available in sufficient quantities (more than 10 million m^3) and of good quality. These deposits have the following properties:

- Continuous gradation
- maximum grain size of 100mm
- 4 to 10% (4% in average) finer than 0.08mm
- 24 to 50% (38% in average) finer than 5mm
- Los Angeles abrasion, less than 30%

According to the properties indicated above, these materials after screening and washing processes can be used in production of filter, drain and transition and as well as in production of concrete aggregates after testing them for Alkali Aggregate Reactions.

Cretaceous Limestone outcrops in satisfactory quality were found along the upper part of the downstream right abutment. If it is impossible to quarry this area, the rock needed for the Dam construction can always be extracted from the nearby quarries under operation.

7 OTHER DOSSIER COMPONENTS

7.1 Yield Study

The data used for the operation studies included reservoir evaporation rates, monthly inflow, monthly water demands and reservoir volume and surface area versus elevation data.

Seepage loss was considered equal to 0.5% of the reservoir volume in determining yield of the Bisri Reservoir. Transmission loss between the reservoir and the user was considered insignificant in the reservoir operation study.

The reservoir operation studies were based on monthly project stream flows at Bisri Dam for 61 years, including 36 years of actual record and 25 years of extended data.

Based on the Master Plan of the Awali Water Project, the following basic monthly water demand scenarios were evaluated in the water supply yield analyses:

- 6-month delivery period between June and November at constant releases of 5.1 m^3/s. No release in other months.
- 6-month delivery period between June and November at constant releases of 5.8 m^3/s. No release in other months.

Evaporation data from Station Kfar Nabrakh were assumed to represent evaporation from the Bisri Reservoir since this station is the closest evaporation station to the Bisri Reservoir.

The area-storage capacity curves for Bisri Reservoir were prepared from a topographic map at a scale of 1:2,000 with a contour interval of 1 meter. For the purposes of reservoir operation studies, the 50-year sediment deposition of 9 Mm^3 was used to modify the area-capacity curves and to take the minimum reservoir elevation of 420 m into consideration.

Three series of simulation analyses were performed for the reservoir with total storage volume of 125 Mm^3 for a 6 month constant demand of 5.1 or 5.8 m^3/sec in addition to a constant release rate of 0.45 m^3/s during the dry months and 0.3 m^3/s during the wet months into the river for environmental purposes.

Water shortage was calculated (as a percentage) by dividing the total water shortage (in Mm^3) by total demand for water during the study period.

The results of the yield-capacity analyses performed for the Bisri Reservoir for three series of inflow data (old data: (1952 to 1981), New data: (1991 to 2012), All data (1952 to 2012)) are summarized in the table below:

Data	01- Old Data	02- Old Data	03- New Data	04- New Data	05- All Data	06- All Data
Number of Years of Data	30	30	22	22	61	61
Demand (m^3/s)	5.1	5.8	5.1	5.8	5.1	5.8
Reservoir Storage (Mm^3)	125	125	125	125	125	125
Annual Potable Water shortage (%)						
Average	0.3%	1.5%	7.8%	11.2%	3.7%	6.2%
Minimum	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
Maximum	6.3%	21.9%	24.8%	24.9%	24.8%	24.9%

Percentage of years of % shortage						
0	93%	87%	50%	32%	69%	61%
0% < 5%	3%	0%	0%	9%	7%	3%
5% < 10%	3%	10%	9%	5%	7%	10%
10% < 20%	0%	0%	27%	27%	11%	11%
20% < 30%	0%	3%	14%	27%	7%	15%
Annual Potable Water Volume (Mm^3)						
Average	92.3	102.0	85.3	91.9	89.0	97.2
Maximum	92.5	103.6	92.5	103.6	92.5	103.6
Minimum	86.7	80.9	69.5	77.8	69.5	77.8
Annual overflow (Mm^3)						
Average	44.4	36.2	12.0	11.2	35.8	30.7
Minimum	0.0	0.0	0.0	0.0	0.0	0.0
Maximum	161.1	150.1	288.3	288.3	288.3	288.3

7.2 Dam Breach Simulation and Emergency Action Plan

The modeling of a potential dam break from the Bisri Dam in the Bisri River Basin was conducted using an unsteady HEC-RAS model. Topographic data for the Bisri River Basin was obtained from actual land survey and converted to a 5m x 5m cell-size Digital Elevation Model (DEM) representing the Bisri River Basin. The Bisri Dam is located at an elevation of 400m above sea level. The reservoir has a maximum capacity of 125 million m^3.

The Bisri River Basin downstream of the Bisri Dam is very steep, dropping 400m in approximately 22 km distance to the mouth of the river at the Mediterranean Sea. The river flows through several deep and winding canyon sections interspersed with wide, flat reaches over this interval making the dam break modeling problem challenging as many cross-sections must be used in the model (10 m spacing between sections), along with a short time step (5 seconds).

A Potential Failure Mode Analysis (PFMA) of the clay core embankment of Bisri Dam identified the main potential failure modes as due to seismic loading and flood overtopping. Overtopping and breaching after seismic activity is the most probable and worst mode of potential failure of the Bisri Dam (consideration of a river flood, even exceptional, would not add much to the peak flood generated by the dam breaching and reservoir emptying). It is the failure mode described and modeled in this report. The unsteady hydraulics of the dam breach due to this failure mode was modeled using U.S. Army Corps of Engineers HEC-RAS software. The breach formation time used in the model is 1.5 hours, as calculated using the U.S. Bureau of Reclamation recommended procedure.

The model results show a peak flow of 43,000 m^3/s at the dam and 41,000 m^3/s at the sea outlet. The flood wave generated by the breach is initially 28 m high and does not abate significantly until reaching the coastal valley where it decreases to 10 m. The lag time between the peak flow at the dam and at the mouth of the river is 30 minutes; the flood wave travels 21.6 km in 30 minutes or 43.2 km/hr. Sensitivity analysis was performed at the time of the dam breach and the width of the formed breach, shows that the model results are sensitive to the breach formation time and not to the width of the breach formation.

Table 3: sensitivity analysis

Cross-section (distance in km from sea outlet)	**Initial Rise Time ***	**Peak Time***	**End Time***	**Peak Flow**	**Flood Height**
	hr:min	hr:min	hr:min	m^3/sec	m
21.6 km(dam)	0:00	1:10	2:40	43000	28
19 km	0:20	1:10	3:00	42800	27
10.9 km	0:40	1:20	3:40	41700	25
5.4 km	0:50	1:30	4:10	41200	20
Sea outlet	1:00	1:40	4:50	41000	10

* *Time from time of breach initiation*

GIS mapping of the areas inundated by the dam break flood were undertaken using the HEC-GeoRAS software.

Validation of the HEC-RAS model was performed by using a three dimensional CFD mathematical model of commercial use "Flow-3D". Based on the comparison between one dimensional modeling (HEC-RAS) and three dimensional CFD modeling (Flow-3D) no significant differences were found.

The inundation mapping downstream the dam considers the envelope of both models. The resulting inundated areas were also mapped in Google Earth for easy interpretation by local communities and responsible officials.

Estimations of loss of human life in consequence to a breach of Bisri dam were conducted based on the new guidance of the USBR's empirically-based method for estimating consequences from dam failure in terms of loss of human life. For the case of little to no warning, the estimation of life loss will range between 5,000 and 13,000. For the case of adequate warning, the estimation of life loss will range between 10 and 340.

The dam break model results were also used in the preparation of an Emergency Action Plan developed for protecting downstream communities in the event of a dam break and subsequent flood. The emergency action plan detailed step-by-step procedures to follow in case of emergency. The emergency action plan also included information such as whom to notify, who should do what, and location of emergency stocks.

7.3 ESIA (Done by another Consultant)

An environmental and social assessment process was put in place to avoid, mitigate and/or compensate those identified potential negative environmental and social impacts.

7.3.1 Legal, regulatory and institutional framework

7.3.1.1 Existing Lebanese Legislation

Following Lebanon's reconstruction and development drive after fifteen years of civil unrest and invasion, Lebanon had no alternative but to rely upon external funds granted by international donors such as the European Commission, World Bank and unilateral donors for whom projects had to be environmentally assessed as a prerequisite for funding.

Subsequently, Draft Decree No. 444 of 2002 defined the binding principles to which all public and private projects are subject in evaluating the impacts projects have on the environment. In accordance with Article 23, all projects are required to undergo an Environmental Assessment, for which the regulatory authority is the Ministry of Environment (MoE). The Draft Decree was eventually passed in August 2012, during the currency of the present project, becoming Decree No 8633, Fundamentals of Environmental Impact Assessment.

7.3.1.2 *Triggered World Bank Safeguard Policies*

In accordance with CDR policy, the Assessment complies with the structure and guidelines of World Bank Operating Policy 4.01 Environmental Impact Assessment for a Category A Project, as well as with the requirements of the Lebanese Ministry of Environment, as recently formalized in Decree No. 8633 of August 2012. Five of the WB Safeguard Policies are triggered by GBWSAP, these are: Environmental Assessment, Natural Habitats, Physical Cultural Resources, Involuntary Resettlement and Safety of Dams.

7.3.2 Analysis of Alternatives

A comprehensive comparative analysis of the economic, social, technical and environmental aspects of potential solutions to the augmentation of Greater Beirut's long-term water supply has been carried out. The GBWSAP ESIA has investigated a range of alternatives; non-dam alternatives, dam alternatives, in addition to the 'Do Nothing' or 'Without Project' alternative. Non-dam alternatives that have been considered are desalination, ground water, rainwater harvesting, wastewater reuse and reduction in 'Unaccounted for Water'.

Based on the above, desalination, albeit it technically, economically and politically the 'Source of Last Resort', is the only non-dam alternative capable of sustaining long term water supplies to Greater Beirut, but at the highest cost. The significant limitations in the Lebanese energy sector currently also impede the development of desalination as an economically feasible alternative.

The ESIA also considered three dam sites other than Bisri dam, all of which are included in the Ministry of Energy and Water's National Surface Storage Strategy; these are dam sites at Damour on Nahr Damour (two sites) and at Janneh on Nahr Ibrahim. Based on the comparative analysis, CDR has opted for Bisri dam being the priority scheme.

The following conclusions can be drawn:

- Given its size, cost effectiveness, and all combined favorable geological settings, Bisri Dam is considered the priority option.
- Janneh Dam could be constructed in phases, catering on short term for Jbeil and Kesrwane needs.
- The first years of construction of Bisri and Janneh Dams will allow for a more in depth study about the feasibility of Damour West Dam, the outcome of which should indicate the way forward either to proceed with Damour West Dam or to advance with the Damour East from a feasibility study into a detailed design. In all cases Damour proposed Dams with their reduced volumes could be compensated by possible conjunctive use with ground water from underlying aquifers.

7.3.3 Environmental and Social Baseline Conditions

- **Climate**: The highest evaporative demands occur during the six dry months from April to August, with a peak in July, when the reservoir is expected to reach its full storage capacity and start delivering water to GBA.
- **Landscape**: The landscape consists of the plant cover which is important for controlling erosion and landslip, promotes aquifer recharge and boosts carbon sequestration.
- **Landuse**: The area of land to be expropriated and inundated on the completion of Bisri Dam is equal to 570 ha..
- **Geology**: The Bisri Dam catchment area encompasses a geological sequence extending from the Jurassic Kesrouane Limestone (J4) in the higher mountainous areas through the

intervening formations to the Cretaceous Sannine Limestone (C4) and the recent Quaternary alluvial and fluvial deposits exposed along the course of the Bisri river and continuing downstream of the dam site.

- **Cultural Heritage**: From the available records of the 2004 and 2005 field seasons carried out by a Polish-Lebanese mission, a total of 78 sites were identified, of which 27 fall within the area of expropriation for the Bisri project and a further 10 sites are within 100 m of the expropriation boundary. The sites identified at Bisri represent almost the full span of human history, from Paleolithic times prior to 8,300 years BCE through to the present day.
- Close to the confluence between Nahr Barouk and 'Aariye', now more commonly known as Wadi Bhannine, lies the temple of Marj Bisri believed to be connected with the Temple of Ashmoun, also known as Bustan El Sheikh, in the Lower Awali Valley, dating back to the 7th Century BC.
- Today, the visible remains of Marj Bisri are limited to four black granite columns, perhaps the entrance to the main temple, and several large dressed stone blocks exposed in the nearby river bank, believed to be the wall of the Temenos, the sacred area surrounding the temple. Pottery sherds of both Roman and Persian origin have been found in the vicinity and it is assumed the buried remains of other buildings and at least a small village will also be present. No comprehensive archaeological surveys of Marj Bisri, neither of another suspected temple site downstream, have been completed, although very preliminary investigations without excavation have been undertaken by the Polish Centre for Mediterranean Archaeology at the University of Warsaw working in conjunction with the University of Balamand.
- Of particular significance as witnesses to the relatively recent cultural heritage of the area are the sites of mar Moussa El Habchi Church and the remains of St. Sophia's Monastery, located very close to each other a short distance upstream of the proposed dam axis. The future of the church is an emotive issue for many Mazraat El Dahr residents. Because access is limited to an unmetalled track that is rough and untended, services are no longer held other than on Mar Moussa Day, 28th August, each year.
- **Surface Water Quality**: Water quality analyses from Nahr Bisri and its tributaries show that the level of treatment required to bring water into compliance with Lebanese and international drinking water standards is afforded by a conventional treatment stream. However, of the organ phosphorous pesticides, minute quantities of Lindane and Dieldrin in concentrations marginally above the limit of detection were present in two samples.

7.3.4 Environmental and Social Impacts and Mitigation Measures

GBWSAP area of influence is defined at two levels: the immediate surroundings of the project's infrastructure works for direct, indirect and induced impacts on the one hand and a substantial area, that extends beyond the direct vicinity of the Project itself. The critical area of influence is the reservoir area and the lower catchment whereby it is impacted by the construction activities as well as the changes that will occur resulting from dam operation be it positive or negative, direct or indirect impacts upon which affected communities' livelihoods are dependent. The upper catchment will impact the environment mainly by what it discharges into the reservoir basin. The critical GBWSAP area of influence extends from the sources of Barouk and Aariye Rivers till the outlet of the Awali River on the coast, covering the agricultural plains downstream of the dam and the villages residing in this area.

GBSWAP area of influence also follows the life cycle of the dam construction material which will be sourced from quarries within the reservoir area. The final suitability of all borrow areas will be determined by the appointed contractor. Wastes will be disposed of at licensed sites. The location of construction camps for workers is more likely to be within the valley subject to areas to be protected such as Marj Bisri. GBWSAP area of influence also encompasses Mar Moussa

Church relocation, migration routes for wildlife, and induced development, to finally reaching water supply for GBA users.

7.3.4.1 Main Environmental Impacts

7.3.4.1.1 Erosion and Sedimentation

A major significance of erosion and sedimentation is that it imparts a progressive decrease in reservoir storage, albeit this reduction is primarily in dead storage rather than operational storage. The reservoir has been designed to accommodate 9 million m^3 of sediment within 50 years operation. This will be provided for by 'dead storage' capacity, the volume that can fill with sediment without impacting the normal operation of the dam.

To minimize sedimentation and the loss of capacity and sediment build-up at the dam, it is important to promote reforestation and soil conservation in the upper catchment and around the periphery of the reservoir, and also to monitor reservoir depth to assess sedimentation. The development of wetland on the main contributing watercourses as well as a reforestation scheme in the upper catchment will reduce sediment load.

7.3.4.1.2 Biodiversity and Habitats

Dam construction will always result in the direct loss of riparian habitats and natural vegetation within recognised fragile and vulnerable ecological zones. This however, must be balanced against the new shoreline habitats that favour the colonization of tree species on the banks of the reservoir.

For native fish fauna, artificial barriers across rivers constitute one of the major factors threatening their population in the Mediterranean region, blocking or delaying upstream fish migration. Impacts on fish are considered to be moderate to minor at Bisri dam site, but some mitigation measures should be taken to maintain fish populations downstream of the dam and to allow the passage for migratory fish so to protect spawning grounds. The construction of Bisri dam will significantly reduce water flow downstream, which will definitely affect the freshwater blenny population surviving in the lower course of the river.

Bisri dam will have direct impacts on reptile and amphibian habitats, both upstream and downstream of the dam, which will include disruption to habitats and/or breeding sites, reducing sources of food, and increasing vulnerability to predators.

Species with poor swimming ability may become stranded and prevented from interacting with mainland populations, particularly for breeding, and make them more vulnerable to illegal hunting. Other species may be positively affected by newly created habitats.

The upper level of the reservoir approaches the lower reaches of the Moukhtara River where there are populations of rare Bufo cf bufo, whose habitat appears to consist mostly of rocky terrain and riparian trees, some of which will be inundated.

The presence of a large body of standing water may disrupt the flyways of migratory soaring raptor species, as they will be deprived of thermal air currents necessary for soaring and saving energy during migration.

Mammals will adapt and adjust their behavior, despite any permanent obstructions to their previous dispersal routes, after dam construction is completed. The reservoir may attract species such as bats and otters. Smaller mammals such as shrews and squirrels will tend to have smaller home ranges, and will therefore be susceptible to both habitat loss and fragmentation. Larger or more mobile species are less likely to experience significant habitat loss, albeit habitat fragmentation.

A preliminary Biodiversity Management Plan has been proposed and describes the mitigating measures, costs and responsibilities of the impacts described above. The biodiversity baseline, conservation management actions and mitigation have been generally identified and reflected in the Biodiversity Management Plan. The biodiversity specialist team described in the Biodiversity Management Plan section will develop a biodiversity monitoring plan to monitor biodiversity and habitat management, the results of which will inform the project on the level of degradation to the sensitive habitats and the presence of any direct or indirect activities/actions potentially degrading these habitats especially as it relates to the identified endangered species of fish (namely the blenny freshwater fish). To supplement the management/mitigation measures, the biodiversity monitoring plan will include surveys that will take place during pre-construction, construction and operational phases of the project. These surveys will measure indicators that include but are not limited to: water quality, environmental flow volume and quality, number of target species as well as numbers of indicator species, and cumulative impacts within the upstream watershed. Supplemental details to the biodiversity management plan will be included in a revised version of the ESIA.

7.3.4.1.3 Consumption of Materials

The consumption of materials for construction will be significant and is estimated at approximately 8 million m3. However all granular materials including clay and gravel are expected to be sourced from within the reservoir site. The exception may be the rockfill and riprap, which because of block size and rock quality specifications may need to be sourced externally. All water consumed on site is likely to be taken from the river and given appropriate treatment prior to use.

7.3.4.1.4 Water and Power Supplies

The proposed Bisri dam water releases will be allocated securing 5.1 m^3/s or 5.8 m^3/s for the domestic needs to Greater Beirut and 0.3 m^3/s and 0.45 m^3/s for the environmental flow to be maintained downstream the dam, in summer and winter respectively.

The production of approximately 11.2 MW hydroelectric power, is considered a "by-product" of the dam releases and as such will not be considered as consumptive usage like the previous ones.

7.3.4.1.5 Reservoir Stratification

The anticipated conditions at Bisri - cold high-volume inflows from spring snow melt and warm low-volume inflows throughout the summer and autumn - are likely to result in the stratification of the reservoir. Failure to identify and control it frequently poses major problems for water service companies and may compromise the effectiveness of water treatment streams, the meeting of regulatory water quality standards and consumer expectation, and the adequacy of environmental flow releases.

Typically, and to be expected at Bisri, stratification becomes more severe during the summer months when the intensity and duration of sunlight increases and mixing due to reservoir inflow decreases; thus coinciding with the main period of Bisri operations. Hence a greater proportion of the reservoir turns anaerobic and in consequence minerals such as manganese, iron, sulphides and arsenic are released from bottom sediments, phosphorous and ammonia may be released. The downstream water treatment plant at Ouardaniyeh (currently under implementation under the parallel and independent Greater Beirut Water Supply Project), has been designed to take these issues into consideration.

7.3.4.1.6 Dam Safety

Dam safety relies first on sound design taking into account the main constraints of the site, i.e. more than 100m depth soft clayey deposit under the dam, high seismicity of the dam site, large floods. The validation of the design by a high level international board of experts offers the necessary warranty on the design relevance. Next step for the dam safety is a good quality construction.

Besides, Dam Safety Plans based on Dam Breach modelling and inundation analysis, as presented hereinabove has been undertaken.

7.3.4.2 *Advisory Panel*

The Advisory Panel is composed of two panels: the Dam Safety Panel and the Environmental and Social Panel.

The role of the Dam Safety Panel is to advise on all critical aspects of the dam; its appurtenant structures, its catchment areas, the surrounding and downstream areas. It is also usually in charge with oversight of project formulation, technical design, construction procedures, and associated works such as power facilities, river diversion during construction, fish ladders, etc. The Dam Safety Panel was appointed in early October 2013 and will remain under contract to CDR until the first fill of the reservoir.

The Environmental and Social Panel will provide independent review of, and guidance on the environmental and social issues associated with the planning, design, construction and operation of Bisri Dam and its appurtenant structures. The Panel will assess the extent to which the Bisri project complies with World Bank safeguards procedures.

7.3.4.3 *Main Social Impacts*

Construction will result in the loss of productive land estimated to extend to some 150 ha, about 25% of the area to be taken. The braided river bed and natural bankside vegetation occupies 105 ha, with built-up areas; farm buildings, housing and heritage, making up less than 1%. The remaining area is primarily uncultivated natural vegetation on the bottomlands away from the river and generally open land and scrub on the lower valley slopes. The number of built-up structures to be inundated is estimated at 135 over a total number of 88 plots with a total area of around 1.0 ha. The majority are already abandoned (some derelict) or only provide seasonal accommodation for agricultural labourers.

Land take will also occur for other project activities and associated infrastructure like the distribution lines and access roads leading to the conveyor. These have been incorporated into the final plans for expropriation.

7.3.4.3.1 Benefit Sharing Program

To ensure an equitable distribution of Project benefits, the project will establish a Benefit Sharing Program to provide the means to improve community services on the surrounding hills and throughout the dam catchment and the local environment. This shall be carried out initially through the capital funds available for the project (as reflected in the RAP budget), later through continued revenue from primary beneficiaries which are the GBA consumers.

Capacity building will be ongoing to mitigate the project's environmental and social risks and to ensure inclusive communication with all project stakeholders.

7.3.4.3.2 Induced Development

Given the relative uniqueness of the Bisri scheme and its proximity to urban centers such as Beirut and Saida, visitor attraction may be expected will commence soon after the start of construction. The precursor to induced development may therefore be coffee vans and refreshment trucks, with existing cafes, petrol stations and other services in Bisri and villages en-route from the highway catering for the influx.

On the overlooking hillsides the demand for land on which to construct villas, apartment blocks, hotels, hill resorts and restaurants, all with access roads and public infrastructure will be extensive. While these may also occupy shoreline plots, waterside land is more likely to induce smaller water sport focused accommodation, camping and picnic sites, bathing areas, shoreline walkways and cycle tracks, boat rental and repair yards, yacht and canoe clubs. In addition to visitor and recreational activities, the reservoir will also afford the opportunity to expand local irrigated agriculture and develop water-based commercial enterprises.

Induced development will only impart positive environmental and social impacts if it complies with a well formulated and agreed Master Plan. If development is not planned and piecemeal, or certain political and/or commercial interests are allowed to violate the Plan, the results may be entirely negative.

7.3.4.3.3 GBSWAP Cumulative Impacts

The cumulative impacts assessment focuses on the interaction of the GBWSAP Project and developments that are realistically defined at the time the environmental assessment is undertaken, where such projects and developments could directly impact on the project area of influence. The Table below is a matrix showing those incremental impacts with some existing developments and others proposed.

The Table below summarizes the impacts that might accrue from Bisri dam and the mitigation measures proposed for each impact, while the table that follows summarizes the estimated costs.

Table 4: Cumulative Impacts on Selected VECs

Existing and Proposed Projects					
VECs	**Parameters**	**GBWSP**	**HEPs (Joun, Awali, and Anan)**	**Sewerage Treatment Schemes**	**Reforestation Scheme**
Water	Water Abstraction	0	0	0	0
	Water Quality	+	0	+	+
	Flow Rate	+	0	0	+
	Domestic Water Supply	+	0	0	0
Air	Greenhouse Gases	0	-	+	+
Power	Power Supply	0	+	0	0
Land Use	Land Cover	0	0	0	+
	Reservoir Sedimentation	0	0	0	+
Habitats and Wildlife	Species Diversity	0	0	0	+
	Species Population	0	0	0	+
Public Health	Health Costs	+	+	+	+

+ Positive Cumulative Impact *- Negative Cumulative Impact* *0 No Cumulative Impact*

7.3.5 Environmental and Social Management Plan

The proposed program of environmental and social monitoring is summarized in the Table below.

Table 5: Summary of Environmental Monitoring Reporting and Costs

Activity	Reports	Implementation Structure	Estimated Costs	Total Estimated Costs	Budget Assignment
Site Inspections	Individual Visit Reports Summary Reports every 6 months	CM reporting to CDR	150,000$	150,000$	CM budget
Environmental Quality Monitoring	Quarterly Reports	CM reporting to CDR, and MOE	Pre-Construction 50,000$/year Construction 50,000$/year Post-Construction 30,000$/year	Pre-Construction 100,000$ Construction 250,000$ Post-Construction 150,000$	CM EMP Budget
Monitoring by Construction Manager	Included in Monthly Construction Progress Reports	CM to CDR	Included in contract supervision	-	CM Budget
Bi-Annual Environmental Reporting	Bi-Annual Reports during construction	CM reporting to CDR and MoE	30,000$/year	300,000$	To CM EMP budget
Land Acquisition Monitoring	In accordance with RAP implementation requirements	CDR and Independent Monitor reporting to GOL and FA	Included in RAP implementatio n costs		To CDR RAP budget
Operational Reporting	Internal BMLWE Reports	BMLWE reporting to MEW	20,000$/ year	100,000$	To BMLWE Budget
Environmental Auditing	Annual Audit of operational EMP implementation	MOE reporting to MEW and BMLWE	20,000$/year	100,000$	To MOE budget in agreement with CDR
Total Monitoring Reporting Costs				**1,150,000$**	

Note: Total costs are calculated for 5 years of operation

7.3.6 Institutional Structure and Responsibilities

The prime institutional stakeholders in respect of expected management structure and responsibilities are shown in the Figure and Table below, respectively.

Table 6: Prime Institutional Stakeholders for Bisri Dam Management

Institution	Prime Responsibilities
CDR	In its planning role, commissions specialist studies and dam design, secures funding, pre-qualifies contractors and manages the tender process through to award, executes land acquisition, and on behalf of GoL acts as the contract administrator.
MEW	The effective dam owner; establishes operational policy including determining available yields and environmental releases. Ensures formal Dam Safety Panel inspections are undertaken according to preagreed schedules, in coordination with CDR.
BMLWE	Day-to-day operational management of the dam and its appurtenances, implements MEW policy, ensures environmental yields are delivered to riparian owners. Maintains the dam, the reservoir shoreline and operational monitoring. Facilitates dam safety and E&S panel inspection visits. Responsible for public safety including the maintenance of warning signage. Manages structures and water resources downstream of the offtake upstream of the Joun power plant to the Awali Conveyor, the treatment plant, posttreatment distribution, leakage reduction, cost billing, etc.
MoE	Setting and monitoring the adequacy of environmental flow releases to cater for non-abstraction requirements. A statutory consultee for the Dam Safety Panel. As existing laws, shoreline development environmental permitting.
EDL	Purchase the hydropower output and sell it on customers at a rate that at least ensures cost recovery.
MPWT	Implements the Bisri Reservoir Shoreline Development Master Plan.
MoA	Puts in place agricultural extension services to maximise the efficiency of downstream irrigation practices for minimum water use. Advises MEW on the adequacy of releases to maintain legal abstractions. Advises the dam operators on the permitting of commercial fish farming within the reservoir.
LRA	Manages the two hydropower plants ancticipated through the Bisri project to offset lost hydropower at the Charles Helou power plant.
DGA	Collection of pottery shards, glass and other artifacts from surface soils and shallow excavations at previously identified sites; Trial pitting and/or geophysical surveying at selected sites where buried structures may be present; Major excavation and the removal of material at Marg Bisri Roman temple; Excavations in the vicinity of Mar Moussa Church and the remains of St. Sophia's monastery. Archaeological finds unearthed and documented during construction
Diocese of Saida	Deconstruction, removal and reconstruction of Mar Moussa Church and of St. Sophia's Monastery; and, Scavenging old building materials from the ruins of 19-20th century houses to provide for new construction adjoining the relocated Mar Moussa Church.
Concerned Municipalities	Implementation of Land Expropriation Procedure
MoSA	Implementation of the RAP especially regarding refugees registered at the UNHCR
UNHCR	Assist the 79 registered UN refugees (as per the date of the project cut off date on March 20, 2014 – see RAP for details) with resettlement to UNHCR designated refugees camps if they are willing to. Facilitate the other 23 non-registered refugees to get registered with the UNHCR and eventually assist them with their resettlement to refugees' camps.

8 NEW PROJECT DESIGN MAIN FEATURES

The new design of the Bisri dam project is based on the following main considerations:

1. The most appropriate location for the dam centerline is site “C” or Axis “C” according to previous recommendations. Any upstream location, aiming at escaping to the permeable limestone on the right bank, could not be considered because of the largest width of the valley and of the presence on the whole left bank of the C1 sandstone, erodible and moderately permeable. The sensitivity of this weak sandstone to erosion is regarded as a threat to the dam safety. On the top left bank abutment of the selected dam axis, made of this sandstone, a positive cutoff is anticipated to stop any seepage although hydraulic gradients are small in this area.
2. Normal water level (NWL) in the reservoir is fixed to 461 m ASL, leading to consider a dam of 70m to 75m height on the deepest point of the natural ground. At NWL, the total stored volume is 125 Mm^3, according to the latest topographical survey. Average annual flow available at the dam site is estimated to ~130 Mm3.
3. Geological, geotechnical and construction materials available on the site or in its vicinity lead quite naturally to select an embankment dam type with a clay core and shoulders made of gravel and rockfill.
4. As mentioned earlier the constitution of the quaternary deposit and the presence of ancient slope wash under this deposit, leads to consider a deep cutoff wall made of plastic concrete, anchored in the bedrock. It should reach almost a maximum depth of 125m.
5. The maximum settlement of the foundation under the dam embankment load is estimated to almost 7 m (5.5 to 8 m depending on the orientation of the 2D section), i.e. ~6% of the maximum thickness of the quaternary deposit. This magnitude of settlement is the result of several reliable investigations methods along with laboratory testing which led to comparable results. The settlement is not distributed equitably on the whole thickness of the lacustrine deposit, it is of course much higher in the more deformable horizons, exceeding locally 8 to 10% and creating tensile stresses that would damage a cutoff wall. Furthermore, the dam zoning on both sides of the core may be so disturbed by such important settlement, that the safety of the embankment may be jeopardized.
6. Foundation treatment either by rigid intrusions or by column stones can reduce the settlement. However it is known that to get a settlement reduction by 2 to 2.5, the incorporation rate should be around 1/5 to 1/6 of the total surface to be treated. Furthermore the treatment depth couldn’t reasonably exceed 50m corresponding to the limit of available equipment possibilities[4].For the cost it represents and expected results, given that the maximum depth of the deposit reaches 120m, it was considered that any treatment based on soil improvement[5] is unable to reduce significantly the magnitude of settlement, and even more since the top 30m of the deposit is sandy, affected by very low settlement.

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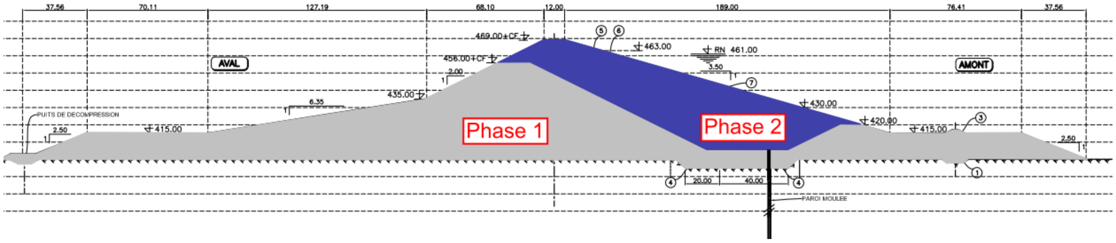
4 Specialized contractors may advertise that they can reach greater depths, but the problem is that beyond 50m, verticality becomes so difficult to control that the distance between the columns became also uncontrolled.

5 Deep mixing or vibro-replacement treatment were also envisaged but they are very expensive for the first one and limited in depth for the second.

7. A vertical clay core along with a cutoff wall in its continuity are therefore not adapted to the site's geotechnical conditions, particularly the large settlement anticipated. To cope with these conditions, the clay core was inclined upstream allowing for much more favorable position of the cutoff and geometry of the core and transitions. Indeed, and as shown in the drawings, this configuration lead to a dramatic reduction of the embankment load over the cutoff. Furthermore, it allows for the construction of the embankment in two main stages as shown in the following sketch. Thus, the risk of the core and transitions disturbance due to the large settlement is mitigated.

Figure 5: embankment construction stages



The cutoff wall is constructed after the completion of stage 1 (phase 1). At this location, expected final settlement is approximately 3.5 m, which represents around 3% of the total thickness of the quaternary deposit and half of the settlement expected at the vertical of the dam axis.

The question to discuss is whether this range of axial (vertical) deformation is acceptable by a cutoff wall, considering that it could reach even 4% locally due to the difference in the stiffness of the deposit horizons. According to S. Hinchberger & al.[6], as far as the plastic concrete is under intermediate (>400 KPa) to high confining pressures (>900 KPa) the hydraulic conductivity (permeability) should remain within acceptable order of magnitude up to 6% axial strain. Furthermore, and as far as the most important deformations will occur within the clay horizons, the most watertight, the efficiency of the diaphragm wall should be more preserved in the more permeable horizons where it is the most needed.

The plastic concrete specified should be the softer possible (Unconfined Compressive Strength smaller than 1 MPa and the Young's Modulus smaller than 200 MPa), in order to follow the foundation deformation when subjected to negative friction. It would not therefore punch the clay core.

8. In order to reduce the settlement before beginning the construction of stage two, i.e. accelerating consolidation, the following was envisaged:

    - Place a 20m height preloading fill during stage one, centered on the wall position,
    - Install wick drains, spaced on 1.5 to 2m triangular intervals to a depth of 30 m to 50m, on the right side of the deposit, corresponding to the area rich in silty clay. At the present time the maximum depth possible is 50m, provided ground is enough soft and free of blocs at intermediate depth. It should also be kept in mind that these drains may deviate during driving leading to very irregular in depth.

9. The following Table 7 summarizes expected settlement of the foundation at the vertical of the cutoff wall, derived from calculation presented in CPTu analysis note. As already mentioned there is a difference (not as big as in the dam axis) between the settlement obtained from longitudinal section (in the plane of the cutoff wall) and transversal sections,

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[6] S. Hinchberger & al. "Mechanical and hydraulic characterization of plastic concrete for seepage cut-off walls", Canadian Geotechnical Journal, Vol. 47, 2010, PP 461-471.

given by the 2D analysis. Both are then presented in Table 7. The actual settlement should be in between both according to the geotechnical parameters considered.

Table 7: maximum settlement along the cutoff wall

adopted arrangements	Settlement (m)			
	Beginning of stage 2		Total (90% consolidation)	
	Longitudinal	transversal	longitudinal	transversal
No preload no drain	-0.11	0.22	3.26	3.91
No preload 30 drain	-0.11	-0.22	3.2	3.92
No preload 50 drain	-0.05	0.01	3.21	3.77
Preload no drain	0.13	0.29	3.24	4.32
preload 30m drain	0.17	0.26	3.19	4.32
preload 50 drain	0.86	0.85	3.22	4.31

Table 7 show that the effect on the reduction of the settlement is observed only when associating deep vertical drains with the preloading. However, according to the transversal sections, the settlement reduction is around 0.4 to 0.5 m when combining both actions, which is not relatively significant. The 30m depth drains are of almost no effect, because they concern the top horizon rich in sand and considered permeable. As the reality is not as perfect as the model, it is recommended to install wick drains on the right side of the valley driven as deep as a possible with a maximum of 50m corresponding to the present technological limit. Furthermore, wick drains have a positive effect on the embankment stability during construction. Decision to place preloading fill will depend on the results of the full scale embankment trial requested in the specifications.

10. Sandy gravel deposit present mainly on the left side of the deposit, is recognized to be sensitive to liquefaction under seismic loading. However liquefaction development could only affect the toes of the embankment where confinement is not sufficient to prevent this phenomenon to occur. It is however considered that the proposed design may accommodate this risk without jeopardizing the dam safety for the following reasons:

    - Regardless of liquefaction, embankment toes will experience large deformations under seismic loading as demonstrated by the dynamic numerical analysis7. This analysis concluded that under the safety evaluation earthquake, the dam will experience acceptable deformations and will remain able to retain the water stored in the reservoir;

    - The toe berms are so wide that any instability that develops at their toe will remain within a limited zone inside the berms;

    - It is anticipated to place part of the unused excavated material in the continuity of the berms providing additional safety. Attention shall be paid to maintaining free exit to the main drain blanket at the downstream toe.

    After all, the specification and the BOQ include the possibility of carrying out stone columns of 10 to 15m depth in order to prevent liquefaction, should the owner consider it necessary.

11. Beside stability calculations and numerical modeling to justify the acceptability of the dam profile under either static of dynamic loading, dam design should be sound by itself and enough conservative to cope with the exceptional unfavorable site conditions. Thus, the crest is 12m width and transitions layers on both sides of the core (filter, drain and transition) are of 4m thickness measured normal to the slope. A large capacity drainage system is provided

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7 ITASCA report presents numerical model of the dam subjected to seismic loading.

at the bottom of the downstream shell, able to safely release large seepage that may occur after a strong earthquake.

12. The topography of the left bank abutment is relatively favorable to the implementation of a surface spillway. It should be a free weir totally founded on the bedrock, with a limited height of the entrance walls, in order to cope with the strong earthquake loading. Bended layout of the chute is introduced to found the structure everywhere of the bedrock, according to the findings of the specific geological investigations carried out recently. The spillway design has been adjusted and checked using a physical hydraulic model.

13. The weir length has been fixed taking into account a freeboard of 8m, coping with the maximum accepted settlement of the crest under the safety evaluation earthquake, fixed to 4m (the calculations conclude to a maximum settlement in the range of 2 m).

14. Bottom outlet is designed as an underground tunnel, along with an underground control chamber and aeration and access shaft, in order to remain operational after any strong earthquake. As a safety practice, the bottom outlet should be able to release water securely in case emergency emptying of the reservoir is needed, particularly after seismic events. Its capacity was determined aiming at emptying the reservoir in less than 15 days considering a mean annual inflow.

    Furthermore, the bottom outlet is used for river diversion during construction, in association with a cofferdam which crest is fixed to el. 417m. It is designed to protect the works for the 100 years return period flood.

15. The following seepage control measures will be used:

    - As mentioned earlier and explained in the synthetic geological report, two positive cutoff walls are used, the most important across the lacustrine deposit and the second in the weak sandstone of the left abutment.
    - Within the limestone bedrock, grout curtain located in the continuity of the clay core footprint. It is extended far in the right abutment and deep in the continuity of the lacustrine deposit's cutoff wall.
    - Galleries allowing for conducting drilling and grouting works regardless of embankment placement, on one hand, and for any additional treatment needed after reservoir impoundment, on the other hand.
    - Consolidation grouting within the limestone bedrock in the footprint of the clay core.

16. Finally a comprehensive monitoring system aiming at controlling the pore pressure and settlement in the lacustrine deposit, during construction and on the long term. The system includes also standpipe piezometers to follow the water table level in the bedrock and inclinometers to check the behavior of the main cutoff wall.

The following chapters presents in detail each of the components of the project.

9 DAM COMPONENTS PRESENTATIONS

9.1 The dam

9.1.1 General

The dam alignment and location is set at the Axis C position, selected in the previous studies and confirmed as explained hereinabove (§5) as a result of surface and subsurface exploration studies that were initiated in 1952 and finalized during the 2014 and 2015 geotechnical investigations. This location has a geology and a topography that are the more advantageous for a dam construction on Bisri river.

It is reminded that the only type of dam adapted to the site conditions (quality of the foundation, available construction materials) is the embankment made with a clay core and gravel/rockfill shoulders.

With a normal water level of the reservoir set at 461 (dam operation simulations), and a freeboard fixed to 8 m (for safety reasons under SEE loading), the crest of the dam is at el. 469 m. Thus the maximum height of the dam on the natural ground is 74m. The dam section and zoning are selected considering established earthquake engineering criteria for dams located in regions of high seismic loading, on one hand and on the other, the high deformability and loose strength of the silty clay composing the main proportion of the lacustrine deposit. Dam crest length and width are respectively ~720m and 12m. Footprint of the dam is near 620m length from upstream to downstream.

The following measures provide a protection against the potentially harmful effects of a large settlement and/or major earthquakes:

- Ample freeboard and 12m width crest in view of large expected settlement under extreme seismic loading;
- Inclined clay core enabling the cutoff wall to be shifted upstream where embankment height is enough small;
- Two stages construction of the embankment, allowing for going forward with the embankment placement concurrently with the cutoff wall installation. This may also help in acceleration the settlement before the placement of the core and transitions;
- Wide transition zones of material which will not support or maintain a transverse crack;
- Ample drainage system to allow for free seepage release through cracks (drawing 20-07);
- Upstream shell made of free draining rockfill;
- Crest details to mitigate erosion in the event of overtopping;
- Ample camber to compensate long term settlement;
- Bedrock protection with a reinforced concrete slab to prevent any erosion of the core trough any crack;
- Weight berms on the upstream heel and downstream toe of the dam.

All materials used for the embankment construction will be subjected to a large-scale test fill to validate material preparation (extraction, water content adjustment and homogenization), the equipment used and placement techniques.

The total volume estimated for each type of material to be used in the dam is given in Table 8.

Table 8: Estimated volume of each material

Material	Estimated volume (m^3)
Clay core	920 000
Rockfill	2 800 000
Gravel	4 000 000
Transition	310 000
Filter	450 000
Drain	370 000
Rip rap	100 000

It is important to emphasize that the actual volume to be placed will be greater because of the gradual compensation of the important lacustrine foundation settlement during construction. This volume is estimated to 550 000 m³.

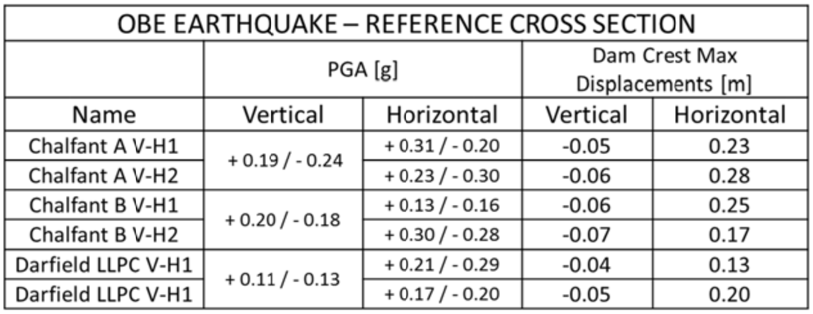
9.1.2 Stability calculations

Typical cross section of the dam is given in drawing 20-07. It is justified in the stability analysis note, either under static or pseudo-static conditions. Safety factor corresponding to the “end of construction” or “rapid draw down” load cases are satisfactory. The geometry derived from this analysis was checked:

1. Using PLAXIS software with undrained parameters of the foundation, derived from CPTu, corresponding to the end of construction situation;
2. Under dynamic loading by numerical modeling using FLAC 3D software. Corresponding results, exposed in the report in PIECE 4, lead to the following conclusions:

- Under OBE (Operating Basis Earthquake) loading, the maximum observed positive and negative displacements for the typical cross section are summarized in Table 9. They are very small leading to consider that the dam has an excellent behavior under this seismic event.

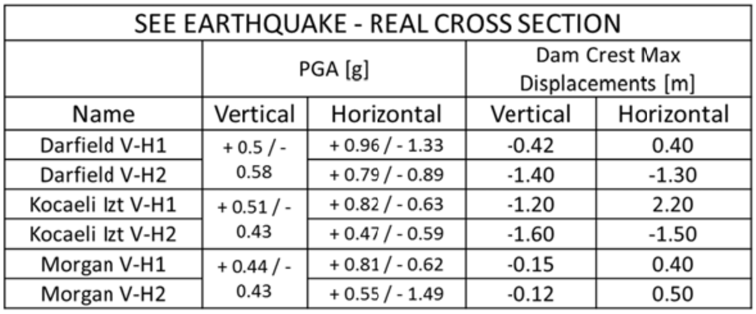
Table 9: OBE earthquakes – Nonlinear analysis – Maximum displacements



OBE EARTHQUAKE – REFERENCE CROSS SECTION				
	PGA [g]		Dam Crest Max Displacements [m]	
Name	Vertical	Horizontal	Vertical	Horizontal
Chalfant A V-H1	+ 0.19 / - 0.24	+ 0.31 / - 0.20	-0.05	0.23
Chalfant A V-H2		+ 0.23 / - 0.30	-0.06	0.28
Chalfant B V-H1	+ 0.20 / - 0.18	+ 0.13 / - 0.16	-0.06	0.25
Chalfant B V-H2		+ 0.30 / - 0.28	-0.07	0.17
Darfield LLPC V-H1	+ 0.11 / - 0.13	+ 0.21 / - 0.29	-0.04	0.13
Darfield LLPC V-H1		+ 0.17 / - 0.20	-0.05	0.20

- Under SEE (Safety Evaluation Earthquake) loading, the maximum observed positive and negative displacements for the typical cross section are summarized in Table 10. They are somehow important, but remain well below the limit of 4m fixed for the maximum acceptable settlement of the crest.

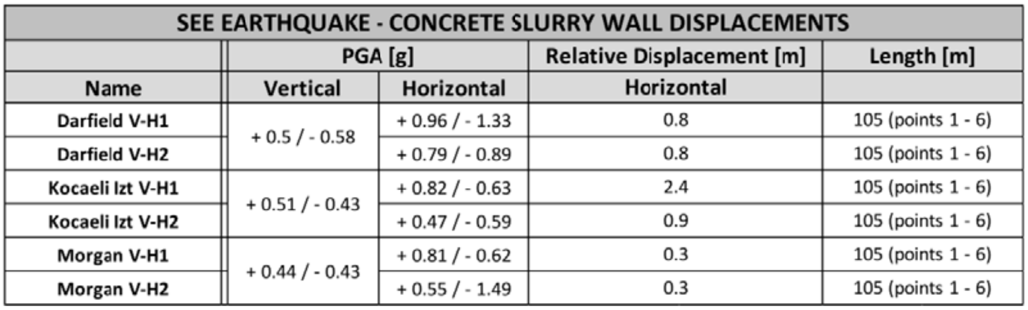
Table 10: SEE earthquakes – Nonlinear analysis – Maximum displacements



SEE EARTHQUAKE - REAL CROSS SECTION				
	PGA [g]		Dam Crest Max Displacements [m]	
Name	Vertical	Horizontal	Vertical	Horizontal
Darfield V-H1	+ 0.5 / - 0.58	+ 0.96 / - 1.33	-0.42	0.40
Darfield V-H2		+ 0.79 / - 0.89	-1.40	-1.30
Kocaeli Izt V-H1	+ 0.51 / - 0.43	+ 0.82 / - 0.63	-1.20	2.20
Kocaeli Izt V-H2		+ 0.47 / - 0.59	-1.60	-1.50
Morgan V-H1	+ 0.44 / - 0.43	+ 0.81 / - 0.62	-0.15	0.40
Morgan V-H2		+ 0.55 / - 1.49	-0.12	0.50

- Under SEE loading, the maximum observed horizontal relative displacements for cutoff wall (slurry wall) on a height of 105m, are given in Table 11. The wall with no harm could support this range of displacement as it is made of plastic concrete. More details on the wall deformations are given in the report.

Table 11: SEE earthquakes, nonlinear analysis - Concrete wall relative horizontal displacements



SEE EARTHQUAKE - CONCRETE SLURRY WALL DISPLACEMENTS				
	PGA [g]		Relative Displacement [m]	Length [m]
Name	Vertical	Horizontal	Horizontal	
Darfield V-H1	+ 0.5 / - 0.58	+ 0.96 / - 1.33	0.8	105 (points 1 - 6)
Darfield V-H2		+ 0.79 / - 0.89	0.8	105 (points 1 - 6)
Kocaeli Izt V-H1	+ 0.51 / - 0.43	+ 0.82 / - 0.63	2.4	105 (points 1 - 6)
Kocaeli Izt V-H2		+ 0.47 / - 0.59	0.9	105 (points 1 - 6)
Morgan V-H1	+ 0.44 / - 0.43	+ 0.81 / - 0.62	0.3	105 (points 1 - 6)
Morgan V-H2		+ 0.55 / - 1.49	0.3	105 (points 1 - 6)

It should be mentioned that further calculations where conducted. They concern the typical section founded on the bedrock and the most unfavorable real cross section but with an upstream face slopped to 4H/1V. This case has been introduced when very large displacements were obtained using Newmark simplified method, also presented in ITASCA report.

Under SEE loading, for almost all tested earthquakes, a sliding occurs at the interface between the core and the upstream shoulder. This remain acceptable as the crest of the core remains well over the normal water level and repair works may be conducted easily because of the gentle slope of the core, and the possibility to completely empty the reservoir. This operation should not be a problem as most of other facilities conveying water will be more or less damaged under so strong earthquake.

In conclusion, dynamic analysis showed that the dam behavior under OBE and SEE is within the requirements of ICOLD, as minor damages occurred under OBE and no uncontrolled water release can take place under SEE. Furthermore, as ancillary works are almost independent of the embankment and are founded on the bedrock or underground, they may be considered safe during earthquakes.

9.1.3 The core

The impervious core will be constructed with clay materials placed wet of optimum to provide a plastic behavior. The core is inclined upstream by 2H/1V and 2.5H/1V respectively for its lower and upper limits. Its normal thickness offers a minimum hydraulic gradient of 3 considered as the limit that shouldn't be exceeded. The width at the foundation contact is larger as shown in drawing 20-08.

Its total volume is estimated to 920 000 m³, to be compared to the total available material identified in the reservoir terraces deposit, estimated to 1 million m³. This seems critical taking into account the difficulties that may arise due to the heterogeneity of the borrow areas. Fortunately, clay deposit available downstream the dam (sites A and B) may provide any necessary supplement, provided that this zone has been already expropriated.

The core will be placed, along with the downstream filter and drain during the second stage of embankment construction (see item 7, § **Error! Reference source not found.**) and after completion of the cutoff wall and all main outdoor grouting works. Thus, a part of the settlement should have already taken place, decreasing the deformations that will affect the core and the transitions.

9.1.4 The filter and drain

Based on past experience and on UACE [EM-1110-2-2300] recommendation, a material conforming to the gradation given in Table 12 should fulfil the requirements for a critical filter. The D15 of the filter should be smaller than 0.5 mm so as the cope with any dispersive clay core material that might be available.

Table 12: Filter gradation (after compaction)

Sieve size (mm)	Percent passing by weight
20	100
10	85 à 100
5	60 à 100
2	40 à 76
1	27 à 58
0,5	16 à 40
0,2	0 à 20
0,075	Max 5 %

The drain gradation compatible with the filter is fixed as shown in Table 13, hereunder.

Table 13: Gradation of the drain

Sieve size (mm)	Percent passing by weight
100	100
63	90 à 100
40	70 à 100
20	45 à 75
10	23 à 54
5	6 à 33
4	0 à 26
2	0 à 12
0,075	Max 3 %

Fines content (minus 200) should be non plastic and should not exceed 5% and 3% after compaction, respectively for the filter and the drain. This requirement is very important as this percentage governs the permeability, but also the apparent cohesion of the filter, which should remain powdery and therefore free of any open crack.

As mentioned earlier, the normal thickness of the filter and drain is set to 4m to accommodate with the large deformations anticipated, either during construction and foundation's long-term consolidation, or during strong earthquake. At the contact with the foundation or concrete structures (spillway), zones considered as the most critical, the filter is widened. Below el. 435m, corresponding to the base of the downstream rockfill zone, the drain became very wide in order to be able to convey any seepage occurring below this elevation, to the main rockfill drain placed at the valley bottom. The free draining horizontal rockfill blanket is 10m height and 60m width, offering a very large draining capacity beneath the semi-pervious gravel zone.

The drainage system of the dam includes also placement on at least 3m thickness of the most clean sandy gravel[8], in the contact of the silty-clay outcropping in the footprint of the dam, aiming at collecting and releasing the water to be brought by wick drains (see drawing 20-09).

The coarse alluvium (sandy-gravel) are proving to have adequate soundness and durability characteristics for processing of filter and drain materials and to sustain required handling, placement and compaction to achieve the required quality, without significant breakdown.

9.1.5 Transition

Transition material (zone 4 drawing 20-07) is placed upstream the core below el. 450 m. It must perform several specialized defensive design functions including:

- Prevent migration of core fines in the upstream direction during reservoir drawdown.
- Transition between the core and the coarse alluvium deposit, upstream the cutoff wall.

To achieve these several criteria, it must consist of well-graded sand and gravel with not more than 8 percent non-plastic fines. It is critical for fines to be non-plastic and maximum particle size can be up to 100mm. The required gradation is given in Table 14.

As shown in the note dedicated to construction materials, the coarse alluvium sand, gravel and cobbles have adequate gradation soundness and durability characteristics for the production of the transition. Natural sandy gravel can also sustain required handling, placement and compaction to achieve a well dense and tight material in accordance with its allotted role. Some scalping and homogenization will be required to produce transition material from natural deposits. It may also partially consist of crushed and processed quarried limestone subject to meeting the above criteria.

Table 14: gradation of the transition

Seive size (mm)	Percent passing by weight
100	100
63	75 à 100
20	52 à 78
5	30 à 58
2	18 à 42
0.5	0 à 22
0,075	Max 8 %

9.1.6 Shell Zone

The purpose of this zone is to provide the primary strength/support for the interior zones and to allow relatively free flow of waters that may come into it. This zone, consisting of both an upstream and downstream portion, comprises the outermost flanks of the embankment.

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[8] Gravel fines (minus 200) content varies between 0 and 10% with an average of 4%

Suitable sources of material for this zone include the river sandy-gravel and quarried sound limestone. Material produced by excavations in limestone and dolomite formations, in the left abutment for the spillway chute, may also be used for this purpose.

Upstream shell and the top part of the downstream one (above el. 435) are made of rockfill. The material would have to be sound (Los Angeles Test smaller than 30%) free draining within each lift once placed and compacted. The rockfill with a maximum particle size equivalent to a placement lift thickness of about 1 m would be limited to a maximum of 40 percent smaller than the 25 mm particle size and minus 200 content limited to 6%. Fines should be non-plastic.

Except on the 3m thick in the contact with the clayey part of the foundation, where only clean and homogenized gravel will be used, elsewhere, in the berms or in the lower part of the downstream shell, natural pit run coarse alluvium is acceptable.

The rationale behind the introduction of natural or homogenized gravel is that this material is available in large quantities in the reservoir (more than 10 Mm^3) and that it is of good quality and therefor much cheaper than the rockfill. It allows also high rate of placement.

9.1.7 Riprap

Riprap will be provided over the upstream slope of the embankment, above el. 430 m, to a minimum normal thickness of 1 m. below this elevation, rock fill should support with no damage waves that may develop. Riprap gradation will conform to ICOLD criteria (Bulletin 91) for a reservoir with a fetch greater than 4 km.

9.1.8 Dam excavations

Dam excavations presented in drawing 20-05 have been defined in compliance with the following principles:

- In the footprint of the shells, a stripping of almost 1m depth aiming at removing trop soil;
- In the footprint of the clay core and downstream filter (see drawings 20-06 and 20-08):
  - On the right bank the top 5m of the bedrock is removed to eliminate the highly weathered and most open marly-limestone.
  - On the left bank, excavations go deeper to a minimum of 10m as the Jurassic limestone is so weathered that it is reduced in places to almost powder near the surface.
  - In the quaternary deposit, excavation is around 5m, ensuring horizontal platforms at different levels to cope with the requirements of the cutoff wall equipment.
- To prevent any arching development, excavations in both abutment are made with very gentle slopes. Therefore, not temporary supports are needed.

Within these principles, core foundation may be locally deeper depending on the real quality of the rock at the bottom of the excavation. On both abutments, the core foundation should be smoothed and any open crack or karst hole sealed with mortar, before protection with a reinforced concrete slab in order to prevent core fines to be eroded through any crack that may open after reservoir impoundment. Next operation will be consolidation grouting, carried out from the concrete slab. On the left bank part of the grouting is conducted from a gallery disposed in a deep trench aiming at providing better barrier against erosion risk of any powdery pocket of limestone.

9.2 Foundation treatment

Foundation treatment is presented and justified in the synthetic geological report. The main components of the foundation treatment are the following:

- Positive cutoff wall crossing the lacustrine deposit and anchored in the bedrock
- Positive cutoff wall crossing the weak sandstone of the left abutment.
- Grout curtain extending on the entire footprint of the core and continuing far in the right bank.
- Consolidation grouting within the limestone bedrock in the footprint of the clay core.

Drawings 20-21 and 20-22 present the principles considered at this stage for the estimation of the foundation treatment. Either the geometry of the cutoff walls or the spacing and depth of the drilling for the grout curtain, will be set out on the basis of additional investigations to be carried out at the beginning of the works. Grouting methods will be clarified on the basis of grouting trial sections concerning each type of foundation.

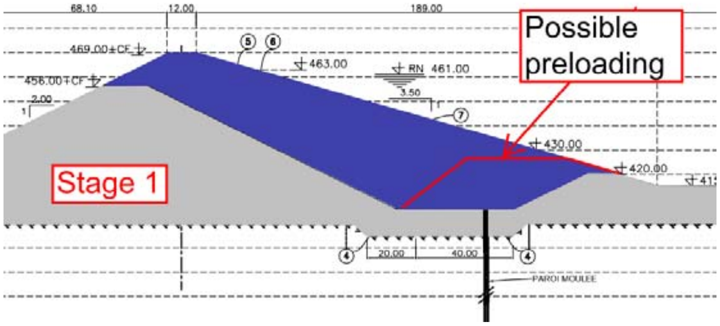
It is reminded that foundation treatment is a key issue in the project. It represents more than 25% of the total cost of the project, and is of great importance for the dam efficiency as it govern the seepage trough the foundation.

9.2.1 Cutoff wall in the lacustrine deposit

The cutoff wall trough the lacustrine deposit (main cutoff wall or "MCW") is almost 120m maximum depth and 1.20m thickness. It is made of plastic concrete, which modulus of deformation should be enough low to accommodate the deformation of the deposit.

As explained hereinabove, the MCW has been shifted upstream to reduce as much as possible the final deformation to a sustainable level. Furthermore, a preloading by almost 20m height gravel as shown in Figure 6 may be considered during stage one, limited to the central part of the deposit, the most rich in silty clay. Preload should be removed before the execution of the MCW.

Figure 6: possible preloading



To cope with the requirement of the equipment intended to realize the MCW, 3 platforms will be managed, respectively at el. 405, 414 and 419m, from the left to the right bank. Each platform is raised to the same level as the adjacent right side one after completion of the MCW and grouting works, allowing for the execution of the connection between platforms.

Works related to the MCW is conducted according to the following sequence (see drawing 20-07, detail 2); beginning by the lowest platform:

1. Excavation of the lacustrine deposit according to drawing 20-05, leveling and if required moistening and compacting;

2. Placement of the transition and filter on respectively 2m and 3m thickness downstream of the MCW and only transition on 5m upstream;
3. Placement of the clay core and adjacent filter, drain and transitions on 5m on the right platform and on 6m on the 2 others. These 3 first operations are conducted within the first stage construction of the dam, they may be followed immediately by the placement of the eventual preloading fill;
4. Carry out the MCW (after the completion of the first stage and the removal of the eventual preloading fill);
5. Carry out in the continuity of the MCW, bedrock grouting using hole reservations managed in the centerline of the wall;
6. Stripping of the top one meter of the clay and the MCW, aiming at purging the surface material disturbed by the circulation of construction equipment and probably also desiccated;
7. Raising the stage 2 embankment.

Steel frame may be used to secure the required position of the reservation holes within the MCW. Reservations should be made of steel pipes to prevent any damage to the wall as synthetic pipes could not withstand drilling rods impact. Drilling near the MCW is not recommended as holes deviations (around 3% admitted) may lead to uncontrolled spacing for depths exceeding 60 to 80m provided that final spacing may be as small as 1m. By placing reservation in the MCW, verticality is well controlled on the full depth of the wall, which is very valuable.

9.2.2 Cutoff wall in the left bank sandstone

As the sandstone (grès de base C1) outcropping in the left abutment is non-groutable, and is semi-pervious and erodible, the only way to control seepage through is a positive cutoff wall. It is called “secondary cutoff wall” or “SCW”.

The SCW is made of regular concrete reinforced on the top 5 meters, to be connected to other concrete structures, i.e. spillway and the grouting gallery of the left bank. It is 1m thick and more than 60m depth, requiring a hydromill for its execution. A temporary fill will be introduced to get horizontal platforms necessary for the circulation of the cranes on the right side of the spillway. On the left side, where hydraulic gradients are small, only grouting may be required aiming at plugging any deep open crack. Decision on this treatment will be taken after review of additional investigations outcomes.

Reservations will also be installed in the SCW for additional grouting if this becomes necessary.

9.2.3 Grout curtain

The grout curtain is one of the main components of this project due to relatively high permeability of the foundation and the absence of a watertight horizon in depth and on the right abutment. The provisions contained in this chapter are widely discussed and justified in the geologic report of this dossier.

In principle, the grout curtain will include:

- The primary grout holes with spacing center to center 8 meters,
- The secondary grout holes with “split spacing” center to center 8 meters,
- The tertiary grout holes with “split spacing” center to center 4 meters,
- Eventually, the quaternary grout holes with “split spacing” center to center 2 meters,

The final spacing of the holes shall reach 1m, where the quaternaries are carried out. Depths of investigation drillings shall exceed the final depth of the grout curtain. At the tertiary stage holes' depth might be reduced.

This grout curtain will follow the alignment of the plastic concrete diaphragm wall at valley floor and will be in continuity of the clay core of the dam within the right and left abutments. It will be executed from:

- Valley floor: Reservations made in the MCW to reduce the depth of drilling and to avoid deviations.
- Left abutment: A cut-and-cover grouting gallery buried in a trench beneath the clay-core. In this abutment the watertightness system (diaphragm wall and/or grout curtain) may be deepened and/or extended into the abutment behind the spillway, depending on the results of the further geotechnical investigations to be carried out prior to start execution works.
- Right abutment: The grouting galleries are situated at elevations 415 and 469 above the sea level. The grouting gallery at elevation 415 will follow the axis of the clay-core until reaching the portal of the grouting gallery at elevation 469 (crest of the dam) and then, it will follow the same route to upstream direction. The upper grouting gallery may be extended to the upstream in search of a substratum of lower permeability. However, this extension should not be carried out, in principal, before the first filling depending on observed seepage.

The total area of the curtain is around 65,000 m². In the cost estimate, the area has been increased by 10% in order to take into consideration any eventual deepening of this curtain, since holes should not stop at a high absorption stage. The consumption is estimated to be 300 kg of cement per linear meter, knowing that the consumption is expected to decrease from primary to quaternary holes.

The gallery on the left abutment is totally anchored in the foundation and has a circular section of 3m diameter and a flat floor. The thickness of the lining is well dimensioned in order to resist to the water and core pressures. Systematic consolidation grouting should be applied to insure its good connection to the foundation ground together with a foundation treatment affected by the trench excavation. The gallery on the left abutment is accessible using the shaft of the bottom outlet. It contains also an aeration system necessary during operation.

On the right abutment, the lower gallery is subject to the hydrostatic pressure and thus should have a circular section. The upper and the access galleries, located outside the influence zone of the reservoir, have a section with vertical walls and circular roof. The access gallery is descending in order to reduce its length but also, and most importantly, to avoid the location of its entrance beneath el. 415 where the ground is covered with thick quaternary deposit. Therefore, the lower point will be drained by a buried pipe passing through the foundation to the downstream drain. The aeration of the gallery will be done through a Ø 300 mm hole connecting it to the upper gallery.

The section of the grouting galleries is very common and requires using compact machines and short drilling tubes.

The construction of the galleries should not pose any problem as far as excavations' stability or seepage are concerned. Limited temporary support system will be needed.

9.3 The ancillary structures

9.3.1 Water Diversion During the execution phase

It has been initially envisaged to divert the water course into a canal that should be implemented on the left bank of the valley, with the execution of the dam fill in two stages: first stage on the right bank over the whole section, then the second stage on the left bank. The bottom outlet gallery will be used to divert the water during the second stage. However the stability conditions of the provisional slopes separating both stages along with the recommended construction sequencing to cope with the quality of the foundation (by preloading), has led to the elimination of the canal option.

The adopted solution consists to utilize the bottom outlet tunnel for the river diversion during works. The implementation of the tunnel on the left abutment is practically imposed by the more appropriate topographical conditions of this abutment. Moreover, the economical saving due to the grouping of the structures of the washout and overflow within the same area, far from the dam downstream footing, is considered as well as determinant factor.

The option of a shaft and an underground valve chamber are always preferred on sites where the seismic loadings are expected to be high such in the case of Bisri. The adopted arrangements are based on the investigations that contributed to locate the upper limit of the rock layers all along the proposed alignment.

The proposed tunnel of a total length of almost 700 m is constituted of (see drawings 20-14 and 20-15):

- An upstream segment having a horse shoe cross section of 5 m clear interior diameter. The justification of such geometry is related to the fact that in the final phase, the lining is subject to an external load, however feeble but non negligible, equal to the kinetic energy of the flow;
- A downstream segment having straight vertical walls with circular upper section of 5 m width and 5 m height. The flow in the final phase is considered as a free surface flow with no significant external hydrostatic pressure, and also due to the fact that practically the lining is not exposed to ground pressure as it is excavated in the Jurassic Limestone.
- The limit between both segments corresponds to the position of the valve chamber that is detailed later on.
- A 0.5 m thick lining on peak points should be sufficient taking into account the nature of the traversed ground, which is considered suitable to the underground excavation works. A provisional light temporary supporting system made of Swellex anchors and shotcrete is to be systematically applied on the top half section, during the excavation phase.

Steel lining and gates should be installed prior to the water diversion. This could be done easily during the two years that are on hand before the start of the first stage of dam embankment works (see the annexed planning of the works). The hydraulic calculation note shows clearly that with the gates installed, the 100 year return period flood with a peak flow of 630 m^3/s, is reduced to a flow of 95 m^3/s, with a water level on the upstream of 416 m.

The upper level of the upstream cofferdam, integrated in the upstream blanket of the dam, has been fixed at 417.00m, offering a freeboard of 1m on the maximum water level during the temporary diversion. The right part of the cofferdam can be accomplished in advance, the closer on the left bank shall be carried out during a dry season so that the waters could be easily diverted through the tunnel.

The cofferdam is composed of clay core surrounded by natural alluviums that are considered largely sufficient for its protection since the loads are feeble and temporary. The core is surrounded as well with alluviums in its higher part beyond the level 410, yet over a restrained width (not represented on drawings). At the end of the works, the core and its wrapping alluvium shall be removed until the el. 410m, over one hundred meter length, and then replaced with rockfill to assure the total drainage of the dam shell until this elevation towards the bottom outlet tunnel.

No waterproofing provision has been made under the cofferdam core since:

- The lowest elevation of the core' excavation is 395, being 2m higher than the lowest river point downstream the dam. It is however possible to manage a small canal at el. 393 to collect the water and to dry the bottom of the core's excavation.
- The gallery entrance is at el. 400, representing 5m of head at a distance of more than 100m from the core and more than 160m from the downstream toe of the cofferdam.
- The first embankment to be placed in the deepest excavation of the clay core are the transition and filter up to elevation 400.

9.3.2 Bottom Outlet

The tunnel alignment was selected so as to minimize the entire length and also to manage its exit on the axis of the spillway flip bucket (see drawings 20-14 à 20-18). Upstream and downstream bends are inevitable, however they do not pose a hydraulic problem. Velocity within the downstream bend, in the range of 15m/s, is perfectly acceptable and does not influence the performance of the downstream exit dispersion block. On the downstream section of the gallery, the maximum filling ratio is 70% and is perfectly compatible with the rise of the water along the final bend.

The gates chamber and the aeration and access shaft are located at the immediate downstream of the grout curtain, corresponding to the dam axis as shown on drawing 20-14. For a location more to the upstream, both structures shall be subject to the full hydrostatic pressure. On the other hand, a downstream location shall expose the unlined upstream segment to an interior hydrostatic pressure that exceeds exterior one.

The designer has almost no flexibility in the transversal direction, because on the left side, there is the spillway, and on the right side, the topography of the abutment is less favorable. The selected location of the shaft beside the spillway provides a good support to the clay core ensuring a satisfactory watertightness in this confined zone.

The bottom outlet comprises:

- To the upstream of the guard gate, a transition segment allowing the passage from the horse shoe section to the rectangular section (4.04x2.60m) over a 10m length, followed by a convergence in roof allowing the transition from the 4.04 height to 3.73m, over a 3.2m length.
- Guard gate valve roller type (3.73mx2.60m), maneuverable from the crest.
- A convergent of 5.35m length allowing the transition of the opening height from 3.73m to 3.00m, corresponding to the obstructed section by the a radial control gate. This one is maneuverable by a swinging single acting servo-motor. The closing is prevailing and is achieved by the own weight of the gate.
- A 19 linear meter auto-resistant steel lining section comprising both valves.
- A shaft located in the vertical extension of the underground gates' chamber. The shaft has a clear interior diameter of 11.50m that enable it to house 3 compartments isolated one from another: one for aeration, the second for access, and the third for the maneuvering of the guard gate. A shaft of such diameter does not pose any construction problem with regard to the ground quality. The temporary supporting that is considered at this stage is made of #2x2m Ø25 dowels, 4m long, sealed using cement grout and coupled to a steel mesh with shotcrete. The thickness of the final tunnel lining is 1m.
- A by-pass at the right bank side between both valves is intended for the restitution of small flows. It is equipped with a butterfly guard valve Ø 500mm and a hollow jet regulating valve Ø 400mm.

- An intake situated on the roof of the steel lining between both valves equipped with a butterfly valve Ø 800mm and followed by steel penstock Ø 800mm, fixed to the downstream roof section of the gallery. The penstock continues till the mini hydropower plant chamber situated underneath the spillway exit beside the downstream exit of the bottom outlet.

The designer considers that it is preferable for a bottom outlet of modest dimensions to be designed with only one opening that is less sensible to clogging than two openings of a two times smaller section. A system of stop logs is provided at the tunnel entrance in order to allow the inspection of fixed components of the guard valve, as well as the inspection of the entire upstream section during the dam operation phase, once there is partial emptying. The inspection may also take place without stop logs if the reservoir is totally empty during dry season.

The bottom outlet is dimensioned to empty the reservoir in less than two weeks. The hydraulic calculation note of this dossier provides the necessary justifications. Table 14 gives the principal hydraulic characteristics of the bottom outlet:

Table 15 Summary of bottom outlet main characteristics

Max flow under NWL (m^3/s)	204
Maximum head on the control valve (m)	60.78 m under NWL/66.48 m under MWL
Time of emptying without inflows (day)	9 day
Time of emptying with inflow of 20 m^3/s (day)	11day
Percentage of filling of the downstream section under the maximum flow	70% at the downstream exit
Velocity of flow downstream the control valve (m/s)	21.3
Velocity at the exit (m/s)	13.2

The entrance of the bottom outlet is constructed in an open cut, having very gentle slopes shaped at 3H/1V guaranteeing their required stability, by a protection of 1m of alluvial material and 1m of rockfill. The ground is sandy-silt in this area, thus requiring such a protection so as to eliminate any instability risk, particularly in the case of seismic shaking. In the occurrence of such events, the washout entrance must remain absolutely free to be able to empty the reservoir, should this be considered necessary.

The tunnel entrance has a hydraulic shape that minimizes the head losses offering at the same time the possibility to install the elements of the stop logs.

The downstream exit is equipped with a dispersion block capable to disperse the jet far from the structure eliminating any risk of dangerous scouring.

9.3.3 Water Intake

A water intake is situated on the right bank to supply the main hydro power plant located at around 4km downstream at the proximity of the existing Joun power plant.

The intake is fixed at el. 422.50m. It is dimensioned for a nominal flow of 6 m3/s. It has the shape of a morning glory protected with a concrete grill and having a full concrete roof. It supplies a steel pipe of Ø2 000mm, implanted where the bedrock is almost outcropping in order to eliminating any risk of settlement due to the dam embankment weight. The pipe is totally wrapped in the bedrock trench with concrete and does not present any hinder for the execution of dam fills or for the core construction.

On the downstream toe of the embankment, the pipe goes through a control chamber before continuing its path till the plant almost 4 km far. At the chamber entrance, the pipe diameter is reduced to Ø1 800 mm. It is equipped with a butterfly guard valve followed by a flowmeter Ø1 400 mm. It recovers its initial diameter at the chamber's exit.

In order to be founded on solid rock, without having to be so far from the dam, the chamber is also anchored in the ground. The handling of the equipment is made with an overhead crane. Their removal to the outside is done through an opening at the roof that can be reached by an access ramp. The personnel access to the chamber is possible through a door at the left side.

9.3.4 Spillway

The dam type and the site natural conditions practically impose the choice of a lateral spillway with a free weir. The left bank topography is more suitable than the right bank as a location of such a structure. The approach channel could be implemented over there within satisfactory flow conditions and the spillway will pass through the abutment and discharge well far from the dam footprint. Moreover, the excavated materials taken from the Jurassic limestone might be used for the dam embankment.

The free weir is practically imposed by the high seismicity of the site. It has a Craeger profile, a height of 5.6m and a circular layout, providing a developed length of 45m. A gradual convergent then allows to convey the water toward a spillway chute of 30 m width before it is slightly diverging 55m in front of the flip bucket, where the width reaches 43.50. This enlargement aims to reduce the unit flow at the exit and accordingly the depth of scouring.

The spillway chute has a uniform slope of 9H/1V and it is marked by a bend of 115m radius introduced in order to provide a permanent bedrock foundation. The bend correspond to a thalweg where the bedrock is missing at the right side limit of the chute. The elimination of the bend would have necessitated the execution of considerable excavations in a forested area in addition to the spillway chute length extension by dozens of meters. The outcropping limestone boundary in this area was obtained by a surveyor in order to ensure a satisfactory foundation along the adopted alignment (drawing 20-12).

A hydraulic model has been built to set out necessary arrangement providing a good flow distribution along the whole width of the flip bucket. Several solutions were examined. The choice was finally made on the two intermediate walls as defined in drawing 20-13, which allow maintaining the split of the flow in 3 almost equal parts at the entrance of the bend and then to restitute all to the downstream. The hydraulic shapes of the walls and their lengths and positions have been fine tuned in the model.

For an economical concern, the downstream exit of the bottom outlet is centered on the axis of the spillway, thus benefiting from the same excavation. The mini hydropower plant is as well advantageously located below the flip bucket.

The necessary excavations for the construction of the spillway are quite important. They practically attain 90m upstream at the approach channel and 50 m at the exit. Excavation slope is 0.5H/1V with berms of 5m width every 10m height, leading to an average excavation slope of 1H/1V which can ensure a satisfactory general stability. But since the nature of the soil is different between the upstream and downstream, therefore the supporting system is not the same:

- On the upstream, the sandstones are friable and erodible. They require a systematic protection made of #3x2m mesh, 3 to 4m long Ø25 dowels sealed using cement grout. They are coupled to a shotcrete applied in two layers, the first one has a 3cm thickness and the second has 5cm, with a steel wire mesh in-between. It is also possible to apply a shotcrete with fibers. At the berms a reinforced concrete slab of 10cm thickness is poured. It should be well connected to the shotcrete of the top and down slopes. In the top area, the shotcrete reverts over the cleaned natural ground.

- On the downstream in the Jurassic limestone, where the rock is expected to be of relatively good resistance, the protection with anchors and shotcrete is carried out only on demand based on the actual condition of the excavation.

The protection and the supporting of the excavations are made progressively as the excavation goes down, because the accesses for the future eventual interventions are considered difficult.

The final design of the spillway as presented on drawings 20-11 à 20-13 has been made with the help of a hydraulic model, for which a synthesis report is exposed in PIECE 7.2 of this dossier.

The following table summarizes the principal hydraulic characteristics of the spillway.

Spillway capacity under MWL (project flood)	~1,260 m^3/s
Maximum flood flow	2,300 m^3/s
Flow velocity in the flip bucket	24.3 m/s
Water depth on the downstream section of the convergent	~2.90 m
Water depth on the immediate upstream of the flip bucket	1.20 m
Distance between impact paint and the weir of the flip bucket	66 m
Theoretical depth of the scour pit	~20m

The spillway shall require a volume of excavation and concrete in the range of respectively 800,000 m^3 and 35,600 m^3.

10 DAM MONITORING

10.1 Objectives

The dam monitoring system is conceived so that the following parameters can be monitored:

- The deformations and pore pressures in the quaternary deposit of the foundation during works and during operation;
- The deformations of the diaphragm wall, during works and during operation;
- The deformations of the dam fills during works and during operation;
- The hydrostatic pressures at both sides of the diaphragm wall at the contact of the embankment with the foundation;
- The hydrostatic pressures in the foundation;
- The earthquake shaking in the substratum and the dam corresponding response;
- The released flow in the river bed.

The proposed monitoring system is indicated on drawings 20-23 to 20-28. It is shown in details in the following paragraphs. Table 15 gives the relative quantities for each device or each type of measurement.

Table 16 monitoring system synthetic table

TYPE OF DEVICE	Quantity
PORE PRESSURE CELL IN THE QUATERNARY DEPOSIT	155
STAND PIPE PIEZOMTER IN QUATERNARY DEPOSIT	5
SETTLEMENT TUBE IN THE QUATERNARY DEPOSIT	19
SETTLEMENT TUBE IN THE DIAPHRAGM WALL	6
GEODESIC BENCHMARKS MARKER EVERY 60 m (PLANIMETRY AND ALTIMETRY)	49
INVAR WIRE MARKS EVERY 20m ON DOWNSTREAM SIDE (LONGITUDINAL MEASURING) AND EVERY 60 m ON UPSTREAM SIDE (TRANSVERSAL MEASURING)	50
STAND PIPE PIEZOMTER IN ROCKFILL DRAIN	2
PORE PRESSURE CELL IN BEDROCK	8
STAND PIPE PIEZOMETER IN BEDROCK	33

The monitored sections can be well identified on drawing 20-23. They are 60m distant and each one is equipped so as to monitor the pressures and deformations in the dam and in its foundation.

10.2 Deformations and pore pressures in the Quaternary Deposit

The follow-up (monitoring) of the deformations and the development of pore pressures, and their gradual evolution during the dam construction works and during dam operation, is fundamental taking into account the incidence of both parameters on the dam behavior.

The proposed system is relatively dense and located in vertical sections upstream/downstream 60m distant between PM280 and 700, making 8 profiles in total. Each profile is made of lines from bank to bank whereas one of the lines is in the axis, corresponding to the most loaded section and two or three under the upstream and downstream shells. The intersections of these two sets of sections represent vertical lines on which are installed pore pressure cells (PPC) almost each 20m height. Along the bank to bank most loaded sections, settlement pipes should complete the vertical line of pore pressure cells.

The PPCs shall be of a vibrating wire type and shall have variable measurement ranges, compatible with the pressures that are subjected to.

The settlement pipes should not protrude in the embankment as they are numerous and will disturb fill placement. Settlement pipes are not accessible and corresponding reading can only be made via a wiring system. At the present time, manufacturers can propose devices able cover the range of settlement anticipated[9]. These devices may consist of:

- a telescopic pipe anchored in the bedrock and equipped with several sensors providing the total and intermediate settlements;
- a sensor placed on the top of the foundation hydraulically connected to a fix point where the settlement may be monitored.

Along with settlement pipes, a topographic survey shall be implemented to follow-up the settlement of each embankment placed layer, considering that compacted alluvium and rockfill settlement is marginal compared to the quaternary deposit.

Where wick drains are installed, the PPCs' position must be centered to the grid of drains to better reflect the efficiency of the latter. In fact the more a PPC is close to a drain, the more the speed of pore pressure dissipation shall be rapid. It shall be needed at the same time to assure the verticality of the drains where a PPC is installed.

The second problem, to which a great attention is required, is the wiring routes and the protection of the wires of the PPCs and the settlement pipes, in order to allow readings out of the embankment placement area. In fact with the expected settlements, it is possible that the wires, laid at the interface between the foundation and the embankment will be subject to extensions that might lead to their rupture. The procedures intended to avoid such ruptures shall be examined on site with the contractor, in association with the manufacturer. Everything should be done to ensure that all the installed equipment in the quaternary deposit will operate satisfactorily during the works and in the following dozens of years.

In order to cope with the principle of redundancy regarding the measurements of settlement and pressures in the quaternary deposit, the following was provided:

- Standpipe piezometers with measuring chamber installed in the quaternary deposit, placed along the downstream berm;
- One standpipe piezometer situated in the rockfill, where any increase of water level in the draining strip will be a sign of increase of leakage.
- A systematic topographical survey of the fill settlements progressively with the dam fill construction. The final geodesic survey is treated later.

10.3 Diaphragm Wall

The deformations of the diaphragm wall are monitored with the help of settlement pipes that are installed on the crossing of its axis with the monitored PM spaced 60m.

The diaphragm wall efficiency shall be nevertheless checked through the follow up of the pressures' monitoring upstream and downstream. Because should the wall shows disorders jeopardizing its waterproofing functions, then upstream pressures would tend to decrease (in a semi-permeable environment), accordingly downstream pressures would increase. For this reason, the monitored profiles have:

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[9] GEOKON manufacturer proposes the 4600 and 4650 settlement systems allowing for settlement monitoring exceeding 10m.

- at the upstream of the wall a series of PPCs placed at the top, at shallow depth, in addition to a PPC at the triple contact between the wall, the core, and the foundation
- at the downstream, two PPCs, one at the downstream triple contact and the other in the filter. This last one is particularly important as it informs on any eventual water head increase.

10.4 Dam deformations during operation

The monitoring of the dam deformations during exploitation is assured by:

- The geodesic survey follow-up of planimetry and altimetry of the installed benchmarks along the profiles which are monitored, on the crest and on the two dam faces (spacing of 60m between profiles).
- The survey by distance-meter with Invar wire of the distance separating the benchmarks of the crest placed on the downstream edge and spaced of 20m, and on the upstream edge spaced of 60m. The first ones serve to monitor the longitudinal deformations and the other ones serve to monitor the transversal deformations of the crest in combination with the first ones.

The installed topographical benchmarks along the face shall not be surveyed unless the upstream water level allows it.

10.4.1 The hydrostatic pressures in foundation

The hydrostatic pressures of the foundation shall be monitored with the help of standpipe piezometers, except when they might be under pressure, in such cases they are called « blind » yet they are equipped with a PPC. The last ones are carried out from the grouting galleries.

They are all of the punctual type with a measuring chamber limited in depth in such way that only one water table level is measured.

All external piezometers are vertical. They are disposed along the right bank according to a nearly squared grid in the order of 75m spacing. They are installed along the left bank on both sides of the spillway. Inside of the grouting galleries of both banks, the measuring chambers are placed downstream the grout curtain. This is what justifies the inclination toward the downstream, of the boreholes for blind or standpipe piezometers.

10.5 Reservoir Water Level Measurement

The water level of the reservoir is monitored by level survey rods that are placed on one of the banks and fixed to stands founded on solid rock each one meter height. A stairway is intended to assure the needed access to undertake the manual measurement and also to maintain the scales.

This device is completed with an air bulb water level recorder that is installed on top of a borehole inclined to the upstream coming from the upper gallery. It shall be checked that the borehole communicates well with the reservoir and provides exactly the same measurements of the level rods.

It is as well necessary to install in the queue of the reservoir an automatic gauging station to monitor inflows. Based on a hydraulic balance taking into account the releases and the inflows, it will be possible to estimate the ranges of seepages, since it is the only way of water losses estimation in the geologic context of the dam.

10.6 Seismic Monitoring

The Bisri dam shall be equipped with three 3D accelerographs:

- Two on the dam, one at the crest and the second at the downstream toe, both in the valley floor (highest alluvial filling depth)
- The third at the bottom of the right bank grouting gallery used to record the signal coming from the substratum and to trigger the two others.

10.7 Flow at the downstream toe and inside galleries

The flow that may appear at the downstream toe of the dam or inside the grouting and drainage galleries shall be measured using a weir witch design is to be adapted to the observed flow, after reservoir impoundment.

In case the flow is very small, then it will be gathered in pipe and corresponding flow measured using a bucket and stopwatch.

11 ACCESS ROADS

Two lane/two way access roads of 6.6m wide each (carriageway) with paved shoulders of 60cm wide adjacent to the traffic lanes, from Bisri to the left and right side abutment dam crest (+469.00) will be constructed along with the downstream valley walls. These aggregate surfaced roads will be paved with asphalt concrete at the completion of the dam to leave permanent flexible pavements for the operations staff.

These roads, around 2.1Km long in total will mainly be constructed by cutting the steep and irregular rocky valley walls into benches with rockfall catchment areas and filling the downhill slopes. The access roads reaching the left and right side abutment dam crest will be connected to the existing road of the village and the service road of the water conveying line respectively.

The slope protection against erosion and rock-mass loosening will consist of applying one layer of steel fiber reinforced shotcrete wall with geocomposite strip drains. Slope stabilization (where needed) else than remediation and in addition to slope protection, will consist of applying two layers of wire-mesh and/or steel fiber reinforced shotcrete and installing grouted in-place rock-bolts with face-plate system.

The vertical alignment of the access roads reaching the left and right side abutment dam crest increases in elevation northeastward. Its lowest point is about 413mabove the sea level and reaches a maximum elevation of 469m.

The straights or grades, joined by vertical curves for a smooth passage from one grade to another, and having a safe sight distance over the full length of the curve, will be used in vertical alignment of the access roads.

The horizontal road alignment will consist of tangents, circular and transition curves. The transition curves will be used to join the straight (tangent) sections smoothly into circular curves having sufficient lengths to avoid the appearance of kink in the road.

Inverted ‘V’ and super elevation crossfalls will be used as cross section elements to facilitate pavement drainage and to help counteract the centrifugal forces on vehicles travelling around the circular curves.

The pavement drainage system will consist of roadside reinforced concrete water canals, pipe culverts with manholes and slope outlet structures with parapet wall extensions.

12 WORK PROGRAMME

The work programme is given in annex C. It is based on the following developed hypotheses, considering 25 working days per month, based on two shifts a day of 10 hours each. The total duration of the works extends over 60 months from the notice to commence the works on the month of January of year 1 and completion at the end of the month of December of year 5. It will be however possible to envisage an anticipated start of water filling at the beginning of the month of October of year 6 in such as to take advantage of the precipitations during fall to engage in the filling of the reservoir.

The planning should be adjusted according to the provisional real date of starting the works making that the diversion of the river towards the washout tunnel is done during a dry season. It might also be adjusted in case the expected consolidation rate of the quaternary deposit is not reached.

The critical path goes through the following activities:

- The bottom outlet including the shafts and the installation of the hydromechanical equipment,
- The upstream cofferdam which closure is to be performed during a dry season,
- The foundation treatment (especially wick drains)
- First phase of dam fills,
- The diaphragm wall in the valley and the related grouting,
- Second phase of dam fills,
- Crest walls and finishing of the dam fills.

The remaining works (activities) may be executed in hidden time, notably the excavations at the abutments, the grouting galleries, diaphragm wall on left bank, the spillway, the intake on the right bank, the grout curtain and the consolidation grouting at the banks.

The necessary additional investigations for the completion of the design concept, concerning especially the grout curtain, are to be carried out right at the beginning of the works and must be completed and taken into account in the project adaptation concept during the first year.

12.1 Mobilization

It starts with the order to commence the works, given on the 1st of January of year 1 and must take over one year, with a slight extension for few months for the procurement of especially necessary equipment for the grouting works

The necessary equipment for concrete production, whether vibrated or shotcreted, including aggregates' preparation, shall require to have an operational batching plant by the end of month 10, in order to initiate the concrete conformity tests.

It is however necessary that earthworks equipment is available by month 7 in order to start the open excavations of the bottom outlet gallery.

12.2 Bottom outlet and water diversion

The excavations of the upstream and downstream entrance of the bottom outlet and water diversion gallery represent an important volume and must start by month 7 on the upstream entrance, the progress should be high seeing the soft nature of the ground. But on the downstream entrance, the excavations require the use of explosives as these excavations are to be made in Jurassic limestone (at least for the deep layers as the upper parts are mostly fractured).

At the downstream, it will not be needed to reach the final limits of the excavations as required for the spillway, since the objective is to attain as quickly as possible the downstream tunnel portal and to begin the excavation over there.

A total of 6 months are estimated to be necessary to reach the underground portals, and carrying out the supporting by dowels and shotcrete. And so, the tunnel excavation may start by the month 13 (January of year 2).

With an average progress of 5m/day, corresponding to 2 spans of 2.5m, the drilling can be achieved within 6 months. This rate can only be achieved from upstream since the progression in the sandstone must be quick, hence offering more time for the execution of downstream excavations, should this be considered useful.

In parallel with the progress of tunnel works, the excavation of the access shaft and valves chamber must be started by month 17, and achieved by month 20, in such as the blasts remain far from the concrete tunnel lining. There is 1 month overlap between excavation of the shaft and the concreting of the gallery. The first must be achieved at month 19 and the second at month 21 or 22.

The gallery concreting rate of 10m/day is realistic taking into account the clear section of the gallery and the possibility to remove the top and walls formwork 8 hours after concreting.

The installation of the steel linings and the hydromechanical equipment in the chamber and in the shaft are assumed to take 6 months, starting from month 23. The Contractor thus has 22 months for the design, the fabrication and procurement of the washout equipment, which is considered a sufficient time.

In these conditions, it will be possible to proceed with the river diversion into the bottom outlet at the beginning of month 29, corresponding to 1st of June of year 3.

12.3 Dam

Upon the completion of the open excavations of the bottom outlet, the core excavations and the clearing of the dam footprint may start, at the beginning of year 2.

Along the first months of year 2, it will be possible to proceed with the execution of wick drains, in addition to the installation of the monitoring equipment in the quaternary deposits. This is in fact about pore pressure cells and settlement pipes, in addition to the cabling (wiring) and centralization points of measurements. This system must in fact be put into operation before the start of dam fill works (excluding the first meters to be carried out at the right bank in preparation of the platforms intended for the wick drains).

Starting from the month 23, the 1st phase of fill works could be initiated at the right bank, waiting for the river diversion that is planned for month 29 (May of year 3) then shall be extended over the whole footprint from bank to bank. The natures of such fills that is essentially constituted of alluviums and cobbles make them easy to work during all weathers, considering an average productivity of 20 000 m³/day. This shall require the mobilization of equipment allowing to achieve a productivity of 30 000 m³/day during the peak period, notably after diversion, because the work space will become very wide and one single material is to be placed.

This phase might include, without any problem, the execution of the provisional alluvial fill of 20m height that is going to be placed in the location of the diaphragm wall where the quaternary deposit is the deepest. Such fill is to be carried out once the excavations and the first phase fill relating to both superior platforms (at 414 and 419m) intended for the execution of the diaphragm wall are achieved.

On month 35 (October of year 3) the construction works of the diaphragm wall in the valley have to start, with an average productivity of 120 m²/day. This productivity takes into consideration a permanent mobilization of two excavation plants needed in light of the estimated total volume of work to achieve of 31 000 m², and of the position of such structure on the critical path of the Programme of Work. The mix design of the plastic concrete as well as the trial section (outside the final alignment) for the execution of this wall has to be put forward. The cutoff wall must be executed starting from the lower compartment on the left bank side toward the higher compartment on right bank, in order to enable the execution of leveling fills necessary to undertake the connection between the compartments.

With the progress of the diaphragm wall works, the grouting of the substratum must follow. The planning anticipates a 2 month lag to achieve the minimum required concrete strength on one hand, and on the other hand, to provide sufficient work space between the grouting activities and the diaphragm wall. This 2 month lag will be maintained until the completion of grouting behind the diaphragm wall.

Phase two of fills will follow starting the month 48 (December of year 4), but with an average productivity of 10 000 m³/day, taking into consideration a simultaneous execution of a large number of materials in the available congested work space that is expected to get smaller with height.

Therefore the fills will be achieved at the month 60, including the crest finishing with the positioning of the benchmarks for monitoring that are fundamental to follow-up the behavior of the dam.

It is worth reminding that the considered fill volumes do not account for additional quantities that are needed in order to compensate for the settlements that will be taking place gradually with the advancement of the works (between 400 000 and 500 000 m³), and also to achieve the camber which height is expected to attain 2.50m.

12.4 Spillway

This is about a structure that is practically independent from the dam, except for the right bank wall that must be ready on time to begin the fill works that leans on. However this is not going to raise any problem since the fill will not reach this region before the middle of the very last year of works.

The programme previews the spillway works between month 13 and 50, like 38 months for the execution of 800 000 m³[10] of excavations essentially in solid rock in addition to 35 000 m³ of concrete, something that could be achieved without difficulty.

It is essential that the excavations are quickly realized, notably at the upstream to be able to start with the diaphragm wall construction within the sandstone as foreseen along year 3.

12.5 Reservoir filling and reinstatement

As indicated above, the reservoir impounding may take place starting from the month of October of year 6 in such as to take advantage of the fall precipitations. In light of the capacity of the bottom outlet and the chosen dam freeboard, there are no risks to be afraid of even if few meters remain till the completion of the dam.

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[10] A part of the excavations should be previously realized with the downstream entrance of the bottom outlet.

The filling of the reservoir presumes the completion of grouting works, which might not be the case for areas with high absorptions particularly the right bank, where more time than expected might be necessary.

Reinstatement works will take place during the very last three months of works. Access roads would have been completed within this period.

The mini hydropower plant equipment could be installed during or after the works, once the steel pipe and the primary civil works of the chamber and accesses are considered ready.

13 PRELIMINARY COST ESTIMATION

This dossier includes a note intended for the cost estimation of the works. It is based on rough quantities computed based on APD drawings, increased to account for the level of accuracy pertaining to the actual design stage. This estimation does not include neither the power plants, nor the penstock connecting the dam to the hydropower plant downstream.

A 15% margin of additional quantities is included in the total because new elements might appear during construction, especially the undertaking of more important foundation treatment. It can also cover the stoppage of the embankment works in case consolidation takes more time.

Table 16 gives a summary of costs for each of the principal activities of the project. Embankment works represent almost the half of project cost, and the treatment of the foundation (diaphragm walls and grouting), constitute ~26%.

Table 17 cost estimate of the project

Activity	Amount (U$)	% per activity
Mobilization/Demobilization	20 000 000	8%
Dam embankment	111 377 100	44%
Bottom Outlet	19 191 000	8%
Spillway	22 167 500	9%
Small galleries	2 774 500	1%
Intake	1 909 050	1%
Hydromechanical equipment	5 328 000	2%
Diaphragm Wall	43 480 000	17%
Boreholes and grouting	21 755 000	9%
Access routes	2 359 705	1%
Miscellaneous	10 000 000	4%
Provisional 15%	39 051 278	15%
Grand total	299 393 134	
Estimation APD	**300 000 000**	

The estimation gives an amount of 300 million American dollars.

Appendix A Content of the dossier of the preliminary detailed design (APD)

Appendix A - Dossier APD

1. Pièce 1 : Dam Presentation and Justification Report
2. Pièce 2 : Rapport Géologique de synthèse (+ Annexes 12 reports)
3. Pièce 3: Rapport Géotechnique
   - 3.1: Note Géotechnique Concernant Les Argiles Silteuse De La Fondation
   - 3.2: Evaluation Des Tassements Du Remplissage Lacustre (+ Annexes 3 reports)
   - 3.3: stabilité
4. Pièce 4 : Numerical Stability Analysis
5. Pièce 5 : Updated Hydrology Report
6. Pièce 6 : Dam Breach Model
7. Pièce 7: Calculs Hydrauliques
   - 7.1: Note de calculs hydrauliques
   - 7.2: Etude Sur Modèle Réduit Hydraulique
8. Pièce 8 : Etude d'impact sur l'Environnement (CD)
9. Pièce 9 : Plans des ouvrages
10. Pièce 10 : Micro-centrale Hydroélectrique
    - 10.1: Note Descriptive Et Justificative
    - 10.2: Note de Calcul

Appendix B The Geology and Seismotectonic Framework

- «REPORT ON THE NEO-TECTONIC SETTINGAND SEISMIC SOURCES FORTHE SEISMIC HAZARD ASSESSMENTOF THEBISRIDAM SITE», Ata Richard ELIAS PhD, June2014
- «ASSESSMENT OF SITE-SPECIFIC EARTHQUAKE HAZARD FOR BISRI DAM, LEBANON», M. Erdik, K. Şeşetyan, M.B. Demircioğlu, E. Harmandar, 0ctobre 2014

Report on the Neo-Tectonic Setting and Seismic Sources for the Seismic Hazard Assessment of the Bisri Dam Site.

Ata Richard ELIAS, PhD.

August 2014

Index:

Preamble:

The below is the final report on the assessment of the neo-tectonic setting of the Bisri Dam site and the nearby seismic sources for use in Seismic Hazard assessment of the dam.

Summary of findings:

1- ***Some of the parameters used for characterizing the major faults were reviewed based on modern and latest accepted models of the geology and tectonics of the region. The Length of the Yammouneh Fault considered in previous geological reports equal to the length of the entire transform plate boundary between the Red Sea and Anatolia is revised. In our opinion and given the widely accepted standards and rules in Seismotectonics, this is an exaggeration of the possible rupture length. Adopting a smaller and more realistic value of 200km of length, resulted in a reduction of the MCE for the Yammouneh Fault to 7.9 in compliance with the definitions set by ICOLD 2010 guidelines.***

2- ***We identified the MLT ramp as a new seismic source that can affect the Dam site and characterized its hazard. This is a blind sub-surface thrust ramp that lies below the northern part of the Dam site and is capable of generating MCE of 7.8. Special care should be given when considering the GMPEs of this source because of its special geometry (a thrust).***

3- ***No proof of major active faulting related to the inferred fault below the Dam site was found in the geomorphology or geology of the site. In particular there is no evidence supporting the 3m fault offset under the Dam site.***

4- ***No convincing evidence was found in support of the continuation of the Roum Fault under the sedimentary cover of the Bisri valley.***

5- ***Instead, the gathered data suggest that no active major fault runs under the Marj Bisri valley.***

6- ***Given the important erosion/deposition rates within the valley, the possibility that surface expression of other, less active, deep-seated faults may have been smoothed and covered cannot be totally ruled out. These faults – if they exist – have very limited extent with little to no long-term effects on the Marj-Bisri geology and morphology.***

A- Review of the tectonic structures and seismic sources of the Bisri Dam site:

The Bisri Dam site is located in the Southern Central Mount-Lebanon area, ~15km east of Saida and at midway to the Yammouneh plate boundary located another ~12km east of the Dam-site.

The main tectonic elements in the geology of the area are:

- **The Yammouneh Fault:** is the local segment of the large Arabia/Nubia, dominantly left-lateral, plate boundary in the Eastern Mediterranean area that stretches from the Red Sea south to the East Anatolian Fault in the north. This plate boundary, also known as the Levant Fault System or Dead-Sea Transform is ~1100km long. The Yammouneh Fault is the main local segment of the system and extends in a straight and almost continuous line over ~200km from Huleh basin in the South to Boukaia basin in the north. Because of its N25-30E strike the fault represents a major irregularity (a right bend) in the geometry of the plate boundary. The fault is seismically active and has produced major earthquakes the latest and strongest is M~7.6, 1202 event, with a coseismic slip of 5-6m (Daeron et al 2007). The fault has a slip rate of 4-6mm/yrs (Daeron et al 2004, Gomez et al 2007). Because of the irregularity in the fault geometry within the LFS, the fault ruptures associated with seismic events are not likely to extend far beyond the limits of this segment (Daeron et al 2007). The Yammouneh Fault is at ~12km from the Dam site.

- **The Mount-Lebanon Thrust (MLT)** is a thrust system that accommodates shortening associated with the Lebanese Restraining Bend of the Levant Fault System (Tapponnier et al. 2001a, Daeron et al 2004, Elias et al 2007). This shortening perpendicular to the Yammouneh Fault is obvious in the surface geology and structuring of the ranges (Elias et al 2003) and was also demonstrated using geodetic measurements as well (Gomez et al 2007). The main MLT fault is an east-dipping thrust ramp that offsets the seafloor offshore Lebanon and is connected at depth with the Yammouneh Fault. The surface expression of this deep ramp is the Lebanese Flexure, a relatively well localized zone of important westward increase of the structural dips of all geological layers that can be mapped over the entire western Lebanon between the coastal Chouf area, north of the Bisri valley in the south and the extreme north of Lebanon in Akkar where the flexure re-connects with the Yammouneh Fault (Fig1a). This Flexure corresponds also with important increase in elevation of the topography to the east, relatively to the western compartment (Fig1b). It represents the western boundary of the high relief of Mount-Lebanon range and thus can be associated with its build-up. The Mount-Lebanon Thrust ramp is mostly a blind fault onshore, the ramp cuts the surface in the offshore. Evidence of rupture of the Quaternary and recent marine sediments at the tip of the MLT faults were observed during the SHALIMAR marine geophysical campaign (Elias et al. 2007, Carton et al 2009). The location of the offshore ruptures and faults corresponds with the observed uplift of the shoreline as indicated by stairs of uplifted marine terraces (Elias et al 2007). A complex system of offshore thrust faults is also associated with the MLT (Fig1a). The MLT and the Flexure run alongside each other over the entire Lebanese coast between Saida and Tripoli. This 40-45°, east–dipping thrust ramp mostly located in the upper, seismogenic, ~16km of the crust (Fig1b) has a vertical slip-rate ~1.5 mm/yr (Elias et al 2007). Although at depth the ramp merges with the Yammouneh Fault, the transfer of slip between the offshore ramp and the onshore compressive structures such as the Roum and Akkar Faults happens over lateral fault ramps such as the Aabdeh fault in the north and Roum Fault in the south. The junction between this Thrust System offshore and its onshore counterpart in the Roum Fault area is complex and not very well known (see below). The Bisri Dam site is located at a distance south of the MLT ramp in the transfer area between the MLT and the Roum Fault (Fig1a).

A geological cross-section of the area at the south end of the ramp helps constrain the geometry of the ramp at depth (Fig1b). The key elements of the cross-section are the location of the fault on the seafloor and within its sediments as revealed by geophysical data from the offshore, and the position and attitude of the Lebanese Flexure and associated structures onland as well as known stratigraphic constrains of the local geology such as thickness of the geological units. The resulting cross-section gives a first approximate estimation of the depth to the base of the ramp located between 16 and 18km depth. The deeper geometry connecting with the Yammouneh Fault is poorly constrained but is of lesser importance for seismic hazard as it is considered to lie entirely outside the seismogenic layer. The M~7.5, AD 551 earthquake is the best-known event associated with this fault system. It ruptured the entire ramp where possible co-seismic rupture between 2-3 m was also inferred based on measured uplift indicators along the shoreline between Tripoli and Saida (Elias et al 2007). The surface rupture of this event was located in the offshore at the tip of the main ramp and further away on some of the smaller thrusts located in front of it (to the west). Based on the mapped fault trace the expected MLT co-seismic rupture can therefore be at least 40km away from the Dam site.

- **The Roum Fault** is a secondary branch of the Plate Boundary that splays into the Lebanon at Huleh south. It has a N-S strike over most of its 35km length. The fault is considered as mostly left-lateral strike-slip with a compressive component increasing towards the north. All known geological mapping literature concur that the Roum Fault doesn't reach beyond the Awali river (the Dam site) as it merges into the Jezzine Anticline south of the Awali. The Mazraa or Chouf Monocline to the north would correspond to its northern structural equivalent (Dubertret 1955, Nemer & Meghraoui 2006, Elias et al 2006). The generally accepted mapping of the Roum Fault is the one based on the geological mapping of the pioneering work done by Louis Dubertret in the 1960s. More recently, within the discovery of the Mount-Lebanon Thrust system located mostly offshore and responsible for the growth of the range, the Roum Fault was reinterpreted as a lateral ramp of this system (Daeron 2005, Elias 2006). This requires the presence of slip transfer between Roum and the offshore thrusts in southern Lebanon. The structures accommodating this transfer are still poorly known. The slip transfer may be accommodated by diffuse deformation on small scale structures. The mapping of Roum fault as done by Daeron (2005) simply reflects this theoretical model and not a ground based structural mapping of the fault trace in the area. No major and continuous structure accommodating the transfer is known from the geology that could represent an important seismic threat for the Bisri area. The geological fault as mapped by Dubertret should serve best for the seismic design of the Dam site.

The Roum Fault is seismically active. It has produced the two most recent destructive earthquakes onshore Lebanon the M~7, 1837 and the M~5.7, 1956 events. Its horizontal slip rate is ~1mm/yr. The mapped geological fault trace stops ~2km south of the Dam-site. Continuation of the fault further north into the Bisri valley was suggested by previous site investigations conducted within the frame of this project. Our own re-assessment of the geology of the area found no evidence of possible active faulting associated with the faults inferred by these studies (to be detailed later). In particular our results question the validity of the suggested 3m of possible offset under the dam.

- Other smaller faults are also present in the vicinity of the dam-site but are of much less significance for the seismic hazard of the dam area. In particular it is worthy to note the existence of a set of E-W to NW-SE faults that appear to offset the Lower and Middle Cretaceous and die out in the upper (Paleozoic) layers. These faults are associated with growth evidence in the Mesozoic times and are considered to be inactive at present. This type of faults is very dominant in the geology of the Chouf area in particular and western Lebanon in general (Dubertret 1955, Elias 2006, Hajj-Chehadeh & Elias 2014).

B- The seismic design criteria of the corresponding faults:

The instrumental seismicity of Lebanon and the area is poorly constrained. Large uncertainties reside in the available earthquake catalogues and no reliable information can be extracted for statistically analyzing the frequency-magnitude relationships for earthquakes in the area. However the historical record of around 2 millennia covers the entire seismic cycle span of most of these faults as shown by paleoseismic studies. Therefore, the maximum observed magnitudes are the best possible approximations of the

Mmax values. Systematically a value of 0.3 units of magnitude can be added to account for uncertainties and variability of earthquake events. Scaling laws as in Wells and Coppersmith (1994) can be used as guidelines or for sanity check in estimating plausible maximum earthquake magnitudes on the different faults.

From the above we can set the following seismic criteria for the area:

	Maximum Observed (Mw)	MCE	Type of faults
Yammouneh Fault	7.6	7.9	Strike-slip
Roum Fault	7.0	7.3	Strike-slip
MLT ramp	7.5	7.8	Thrust

As for the Operating Basis Earthquake, the best estimation for a magnitude with a return period of 144 yrs is exactly the latest strongest observed in this area, given the quality of instrumental data available compared with the long recurrence intervals of earthquakes on the present faults. Therefore the OBE should be the M= 5.7+0.3 event of 1956 on the Roum Fault.

C- Investigating the extension of the Roum Fault under the Bisri Dam site:

- Previous studies:

Correlation of logs from wells drilled inside the Marj Bisri revealed differences in the age of the underlying substratum that suggest the presence of faulting in the bedrock under the mid valley floor under the Dam site. The inferred mapped fault trace strikes NE-SW and then bends to the right in a more E-W direction. Interpretation of the geology shown on all the sections (A-A, B-B, C-C, D-D and E-E in DAHNT/NOVEC 2013) suggests that the fault accommodates vertical slip with eastern/southern block moving up. The geological reports suggest this to be the continuation of Roum Fault in the area.

In the Section 3 (Seismotectonics and Seismic Criteria) of Appendix-A of the Feasibility Report, Paragraph 3.4.3 refers to field investigations done

Moreover, previous geological reports from field surveys in 1983 mention “possible evidence of fault movement” uncovered in two test pits, TP-17 and TP-21, located towards the center of the valley[1]. They report the presence of abnormal dips in clay seams interpreted as possible evidence of fault movement. During field investigations undertaken in 1994 two trenches T1 and T2 were excavated on the northern side of the valley at the edge of the alluvial infill on the highest of the riverbed terraces[2]. In Trench T2 oriented NE-SW and located closest to the talus of the northern flank of the valley, a 20-30cm vertical offset of the infill layers were uncovered and interpreted as evidence of fault ruptures. The suspected fault was assumed to be a subsidiary of Roum and this overall setting was considered as an evidence for the presence of an active Roum fault under the Dam site. These conclusions raise serious questions: What justifies the choice of the two trenching sites, especially that they are located away from the location of the TP-17 and TP-21 where the possible evidence of fault movement was first observed? How is this suggested fault connected with Roum fault? What is the structural relationship between these two hypothetical faults?

Very little information about this particular aspect of the original fieldwork in 1983 and 1994 was available to us. Given the location of the trenches T1 and T2 at the base of the talus of the northern valley flank, the observed offsets are most probably due to slope failure or slope erosion and deposition processes, and does not necessitate the presence of a tectonic fault underneath. Moreover that no lateral extension of this assumed fault was presented on any of the geological documents from previous surveys reveals the difficulty to have an active fault in that area.

-Work method:

We conducted a geological study of the Marj Bisri area in order to constrain the presence of the Roum Fault or any other fault that cut through the site and to characterize its activity.

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[1] Bisri Dam Feasibility Report, Appendix-A, Section 3 - Seismotectonics and Seismic Criteria, Paragraph 3.4.3.

[2] Figure A4-4-Geologic Section and Boring Location Plan, *in* Appendix-A of the Feasibility Report

The study included review of available geological documents related to the area, and inspection of topographic maps, satellite images and other remote sensing products. In particular the analysis of aerial photos numbers 223, 224 and 225 of the region taken in 1962, revealed to be very useful because of the good exposures of the geological outcrops and morphology at a time before the development of heavy vegetation cover. The desk study was also combined with field visits to the area in May 2014.

-The Roum Fault zone within the accepted structural model of South Lebanon:

The Roum Fault was first considered a branch of the active Dead Sea plate boundary in Lebanon. Its North-South strike was considered by some authors to be the only active branch of the plate boundary (Girdler 1990) extending northward under Beirut area, and continuing offshore over the seafloor towards Cyprus (Butler & Spencer 1998).

Recent tectonic research shows that the Yammouneh Fault is the main active segment of the Arabia/Sinai plate boundary in Lebanon accommodating most of the strike-slip plate motion (Daeron et al 2004). Geophysical data acquired offshore Lebanon show undeniably that the Roum Fault does not cut the sea floor. Geodetic and paleoseismic investigations in Lebanon and on the Roum Fault in particular show that the latter is accommodating a minor component of strike-slip as well as a compressive component.

The Roum Fault with an oblique reverse/left-lateral component of slip is the main topographic boundary in the region of south Lebanon separating the high relief area of south Barouk and Mt-Lebanon range to the east, from the relatively flat, low lying and mostly tabular Nabatiyeh plateau to the west (Fig 1).

The fault trace is clearly mapped in a ~N-S direction in its southern segment between Marjeyoun and Jbaa where it has mostly a strike-slip character. North of Jbaa the fault changes direction into a N10-16E and has a less clear geological surface trace where it becomes dominantly compressive and merges with the broad and asymmetric Jezzine anticline. The steep western limb of the anticline seems to be cut by the fault. Secondary folding probably related to frontal splays associated with the Roum fault are also observed to the west. However the intensity of the folding and compression seems to decrease northward and the steep structural dips of the Upper-Jurassic to Lower-Cretaceous units forming the western limb of the Jezzine anticline between Sniye and Aazour decrease to the north near Bisri. The structural elevations also decrease northward: the top of the Upper-Jurassic (J7) layers located at a maximum of 900 masl west of Homsiyeh in the center of the anticline, is around 750 masl southeast of Taaid and decreases fast near Bisri. The periclinal closure of the Jezzine anticline starts at Bisri and is clearly observed in the structure of the northern flank of Ouadi Bisri between Bsaba and Jabal Baiqoun. The structural units of the anticline can be traced astride the valley continuously with no sign for interruption or offset. The hinge line of the Jezzine anticline extending around 9km in a ~N13E direction plunges steeply to the north.

Structural investigations in the Chouf area – north of Bisri Valley – found no continuation of the Roum Fault beyond the Bisri Valley. A smaller equivalent of a deep-seated thrust ramp under the Mazraa Flexure between Gharife and Beit-Eddine, may be the equivalent of the Roum fault under the Jezzine anticline but at a smaller scale and with no trace of surface faulting. Moreover no active fault from the area east or north of Marj Bisri can be a possible candidate for a possible continuation of the fault inferred to exist within the Marj.

The geology of the Chouf region in general and the Jezzine Bisri area in particular reveals the existence of an important number of small, mostly E-W to ESE-WNW striking, normal faults offsetting the Mesozoic geology. Where these faults are well developed they appear to end within the Upper Cretaceous layers. Offsets along these faults can be significant reaching sometimes around 300m of apparent vertical offset (Dubertret 1955, Hajj Chehadeh & Elias 2014). Structural studies (Dubertret 1955, Elias 2006, Homberg 2010) suggest that these are old Mesozoic normal faults inherited from the earliest phases of extension of the western Arabian continental margin. No evidence of recent tectonic activity can be associated with them. An extensive array of similar faults was mapped in the area in the previous geological studies done within the frame of the Bisri Dam Project[3]. Faulting of this type is suitably oriented to be responsible for the observed changes in the lithology of the substratum under the Dam site. In fact the inferred fault has a similar ~E-W trend as these old Mesozoic faults. Moreover not only the inferred fault trace is based on discrete observations done in wells with no direct continuity of observations, but it also results in a very unusual fault trace with almost a 90 degrees bend making its existence very doubtful given the structural difficulty to explain the occurrence of such a fault trace.

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[3] FigA2-1 Geological Map of Reservoir Area, in Bisri Dam feasibility report

-The alluvial infill:

The northern pericline of the Jezzine anticline is breached by the deep incision of the Bisri river. This incision is mostly the resultant of the waters from the Barouk-river as they reach the friable and weak lower Cretaceous sandstone layers downstream from Ouadi Bater. After flowing in an E-W direction over this Chouf Sandstone formation, the river leaves the Jezzine anticline through a breach in the steep western limb west of Bisri village. Downstream the Awali-river has a much narrower valley incised in the monotonous Mid-Cretaceous Cenomanian limestone layers. A number of slope failures – some of them of very large size – has contributed to the damming of the river flow at many times and places. In particular a clear large landslide east of Anane resulted in the flooding of the upstream Bisri valley and the accumulation of alluvials and lacustrine material in the deep valley. The geological and geotechnical studies in the Marj Bisri area presented in the Geological report of the Bisri Dam project suggest the existence of a maximum 135.16m of Quaternary deposits in the valley (DAHNT/NOVEC 2013 p14). This value also roughly corresponds to the difference in elevation between the mean altitude in Marj Bisri (~400m) and the altitude of the deep riverbed immediately downstream from the Anane landslide (~270m) suggesting there is no place for vertical offset of the deep substratum forming the floor of the valley under the Marj, which rules out the possibility of any activity on this inferred fault at least since the incision of the deep substratum and the filling of the valley.

- The geomorphology of Marj Bisri:

Three sets of river terraces were nested in the floodplain of the area since the river breached the damming landslide volume (Fig4). These terraces can be followed over much of the Bisri Valley floodplain but they are the most clear in the western half of Marj Bisri. The elevation of these terraces increases gradually from west to east along the river profile, as expected. However when the same terrace is considered over a segment of the river profile on both sides of the valley, they are always found to be at similar elevation[4]. Although different type of alluvials have been identified within the fill of the Marj, the surface of the terraces has leveled all of them independently of their type[5] which indicates that the terraced morphology of the Marj results from a principal morphogenetic process and not a secondary result of the depositional processes.

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[4] Figure A4-2, Geological section Axis A, from the Bisri Dam Feasibility Report.

[5] Section B-B, DAHNT/NOVEC 2013.

The water of the Bisri River is flowing within the limits of the lowest present terrace or active floodplain. It resulted in many small oxbow lakes and meander scars typical of meandering rivers. Comparison between present situation and aerial photos of the area taken in 1962, shows some important shifts in riverbed position suggesting important lateral erosion of the Bisri river within its lower, active floodplain.

Ouadi Houarit is a smaller branch of the Bisri Valley that opens south of Bisri village in N-S direction. It is caught between Taaid from the east and Arid Qdoum from the west. The valley is filled and covered by the same alluvials as Marj, where nested river terraces have developed. A large upper terrace – the Houarit terrace – is located over most of the surface of this valley, at around 415m asl. The Ouadi Houarit and Ouadi er Rejme are two seasonal streams that incise their streams inside this surface before converging with the Bisri-river.

The exact age of these alluvials and the terraces is not known. They are estimated to be Quaternary in age. In fact given the abundance of landslide scars on the valley flanks, and in particular on the southern flank were the nature and the important northward dip of the rocks is more prone to sliding, the filling of the valley may have happened in different stages as a result of many landslides. Part of this may have happened relatively recently as suggested by the presence of a buried archeological site of a Roman temple in the upper Marj Bisri (Aliquot 2009). However the steep incision of the river near El-Kherbe, exactly where it passes over the slid volume of Anane suggests that this landslide was the latest to dam the valley and to result in the flooding of the upper stream. When did this happen is unknown. Given the important thickness of the sediments the filling should have happened at least during the late Pleistocene.

Marj Bisri and Ouadi Houarit should have been simultaneously filled by alluvials during the same flooding process. But unlike the Marj, the erosion and incision power of the two small streams that cut through the Ouadi didn’t affect much its geomorphology. The two streams are well entrenched in the Houarit surface and do not meander. The elevation and extent of the upper terraces are therefore well preserved and could be used as reliable indicators of any subsequent deformation in this area.

As mapped in previous geological reports for the Dam project, Roum Fault enters the Bisri Valley from Aazour in the south, in the Ouadi Houarit area. According to the geological surveys presented in the previous reports of the Bisri Dam project, the fault continues in the infill of the valley, cuts through the subsurface of the entire Ouadi Houarit and Ouadi er Rejme in a NE-SW direction and continues west of the Bisri village into the wide Marj Bisri plain (Fig 4).

Mapping of the terraces in the Houarit area reveals no deformation effect that can be attributed to the Roum Fault inferred to be present in the infill of its subsurface. The risers of these terrace as well as the channels of the streams are continuous and show no deflection or offset in any horizontal or vertical direction. In particular, the completely abandoned upper Houarit terrace is perfectly flat and connects easily at the same elevation with the other upper terraces of the Marj with no vertical offset suggestive of a fault scarp associated with the deeply seated Roum Fault inferred from previous geological surveys to have a vertical, south block up, slip. The well-incised Ouadi er Rejme channel lies above the inferred Roum Fault trace, but shows no horizontal deflection or offset of its walls, neither does any of the channels incised in the abandoned terraces nor any of the risers (Fig 6). The absence of vertical or horizontal offset in these old and preserved geomorphic features rules out the possibility of the existence of any active fault in the subsurface infill or bedrock of this area. In fact such an active fault if present should have resulted throughout the successive surface seismic ruptures in the formation of fault scarps.

The same observations can also be extended to the wider Marj Bisri. Even though the erosion and deposition rates in this valley are important, the abandoned surface and edges of at least the upper terrace do not show any vertical or horizontal offset. The terraces stand at almost the same elevation on the opposite sides of the valley, and their lateral extension is evident with no interruption astride the mapped fault (Fig 4, 5). This proves that the fault mapped in the substratum had little to no effect on the surface geomorphic features and thus, if present, this fault can be safely considered seismically inactive.

Conclusion,

Different geological evidence and observations of various nature (structural, seismotectonic and geomorphic) from the area are hard to reconcile with the assumption that an active fault runs in the subsurface of the Marj Bisri. In particular *the Roum Fault does not appear to* *continue in the Marj.* There is no evidence for 3m of fault offset associated with any known fault under the dam site. Some minor and small faults with very low tectonic deformation rate, accommodating syn- or post depositional deformation in the thick sedimentary infill or stress within the Jezzine antlicline or at the tip of the Roum Fault, may exist in the subsurface of the Marj. Their surface expression leveled by the much more active geomorphic surface processes. If so these faults are very likely of small extent and should not represent a serious tectonic hazard to the region.

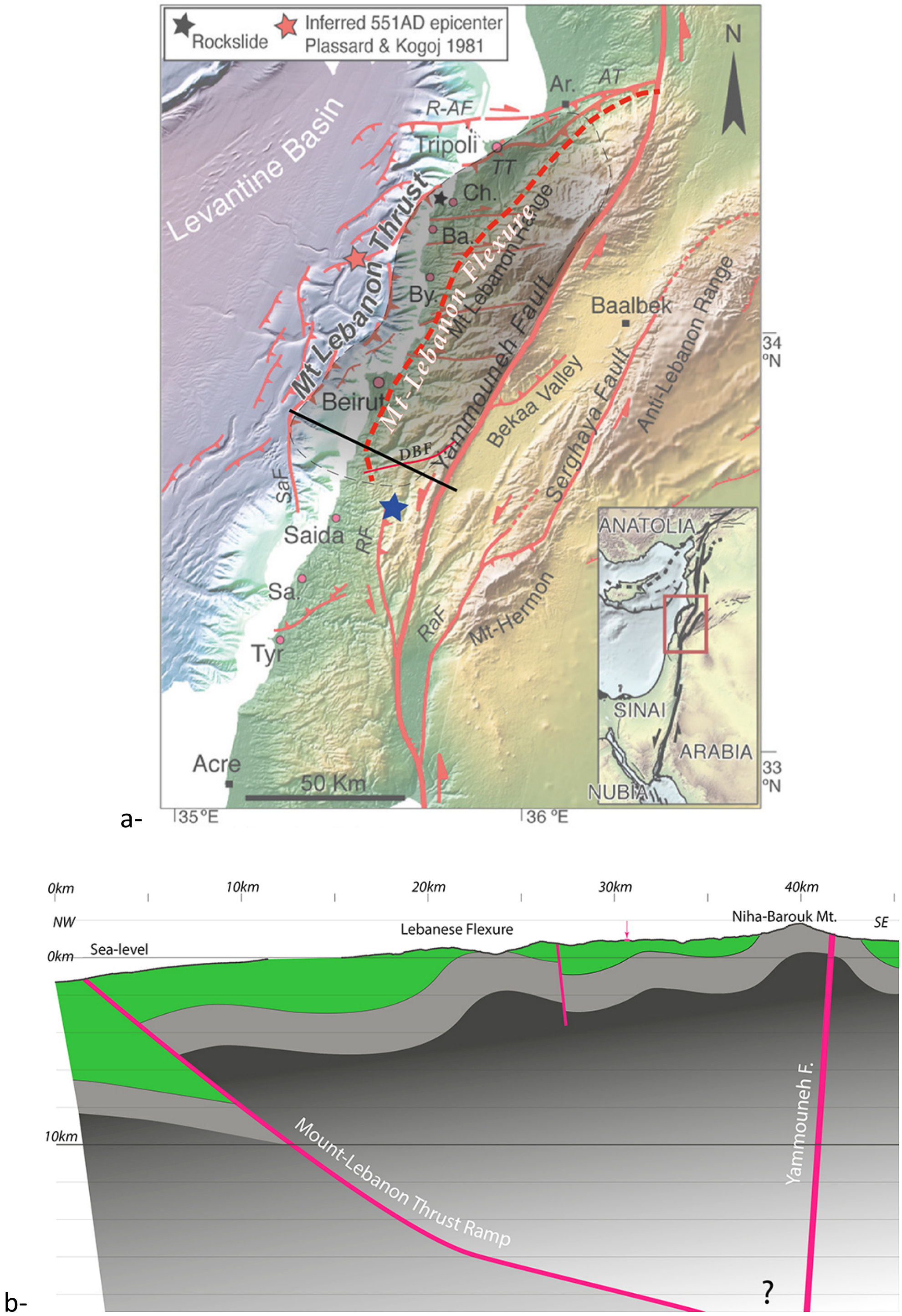

Figure 1 a- Modified from Elias et al 2007, The Bisri Dam site (blue star) within the tectonic map of Lebanon. Red lines are for active faults. Dashed red line is for the Mt-Lebanon Flexure mapped on the surface geology. Gray shadow represents the ramp surface of the MLT in the sub-surface. (R.F. = Roum Fault; DBF= Damour Beiteddine Fault) b- Interpretatitve NW-SE cross-section of Mt-Lebanon showing the deep structure of the range as estimated from surface geology. Depth of the faults are constrained from surface geology and stratigraphy only. (For location see black line on the map above)

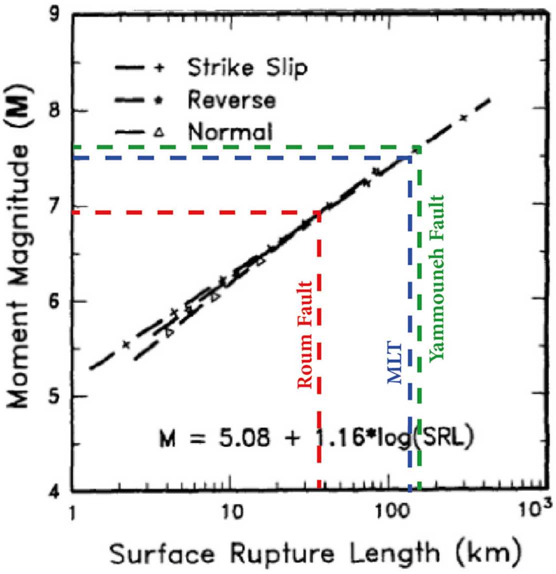

Figure 2 - Estimated moment magnitudes for the three major faults in the area using regression relationship of Surface Rupture Length on Magnitude for different types of faulting (strike-slip, reverse and normal) (Wells and Coppersmith 1994).

Figure 3 Interpreted aerial photo of Marj Bisri area. Only the upper abandoned terrace is mapped. (Photo number 225, mission 1962, scale 1:25000).

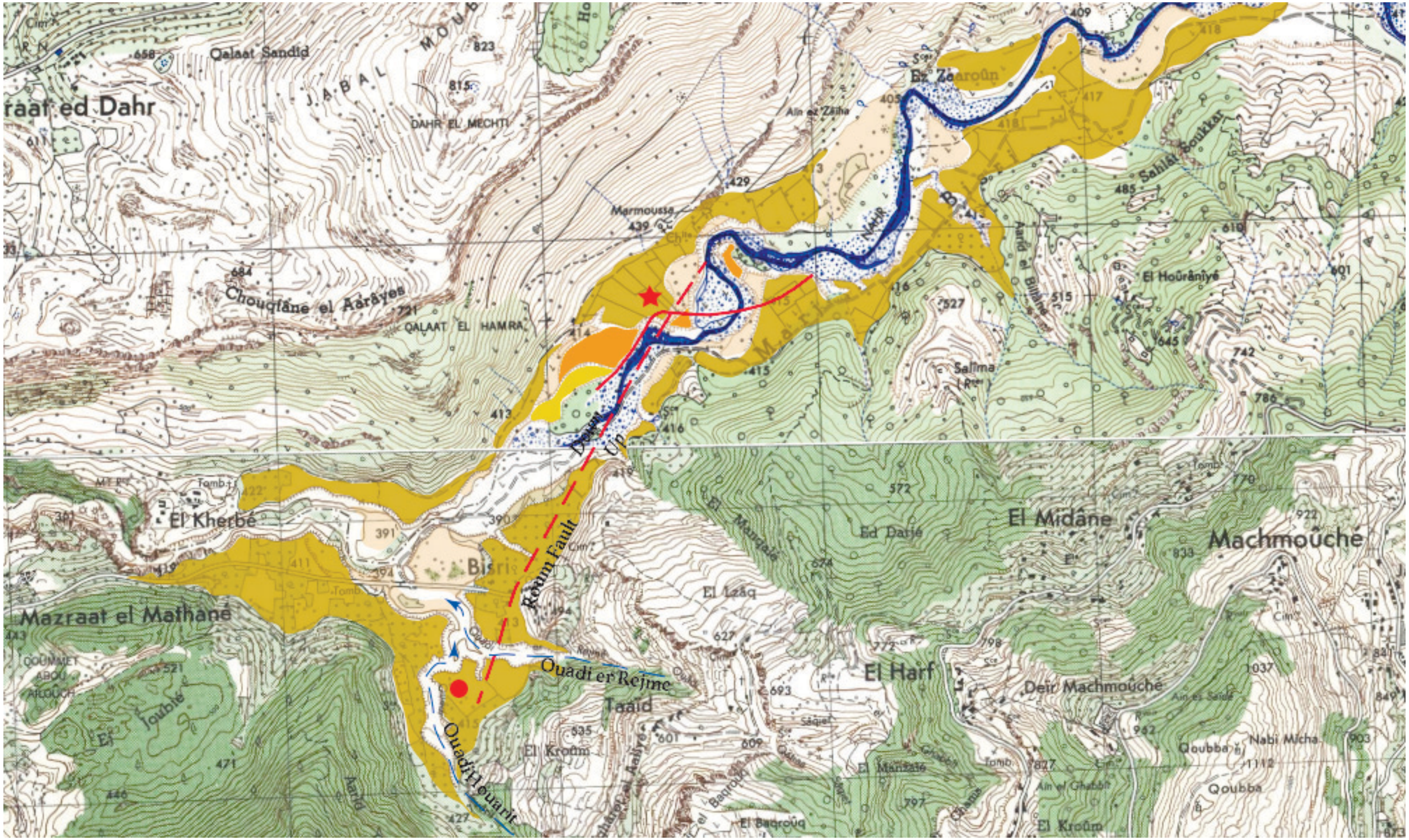

Figure 4 Topographic map of Marj Bisri. Four different levels of abandoned river terraces are mapped with different colours. Red lines are for the inferred Roum Fault mapping from previous geological studies of the Dam project.

Figure 5 Upper abandoned terrace from the northern river bank, located by red star on Fig4. Red line is for inferred Roum Fault from previous studies.

Figure 6 Picture of the Houarit surface showing the absence of any vertical offset of the surface. For location reffer to red circle on Fig4.

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REPUBLIC OF LEBANON
COUNCIL FOR DEVELOPMENT AND RECONSTRUCTION

ASSESSMENT OF SITE-SPECIFIC EARTHQUAKE HAZARD FOR BISRI DAM, LEBANON

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LIST OF CONTENTS

1. INTRODUCTION

This report covers the assessment of the site specific earthquake hazard for Bisri Dam in Lebanon, the location of which is given in Figure 1.

The main physical ingredients of seismic hazard assessment are the tectonic setting of the region, the earthquake occurrences and the local site conditions. These regional physical features, the applicable ground motion prediction models and the appropriate stochastic model for probabilistic hazard analysis will be discussed in the following sections.

Earthquake hazard will be quantified using both probabilistic and deterministic approaches (PSHA and DSHA respectively) in accordance with the selected design criteria. The design basis earthquake ground motion will be quantified using the uniform hazard.

Figure 1. Location of Bisri Dam.

2. EARTHQUAKE RESISTANT DESIGN CRITERIA: DESIGN BASIS GROUND MOTION

The earthquake resistant design criteria prepared for the Bisri Dam (ECIDAH, 1997) essentially refers to the Operating Basis Earthquake (OBE), probabilistically associated with a return period of 144 years (USCOLD Criteria), and the Maximum Credible Earthquake (MCE), 84-percentile ground motion deterministically obtained from the MCE scenarios.

The deterministic OBE and MCE scenarios provided by ECIDAH (1997) are respectively M7.3 and M5.7 on Roum Fault both at 2km distance from the Bisri Dam. The Peak Ground Acceleration (PGA) for the MCE and OBE at the dam site were assessed to be respectively to be 0.7g and 0.52g, presumably for engineering bed-rock. The vertical ground motions were simply taken as 2/3 of the horizontal component.

Figure 2, taken from ECIDAH (1997) displays the 84-percentile spectrum specified for the MCE level design. This was specified to be the response spectra to be used for dynamic analysis of the embankment dam and for rigid structures which require the 5% damping ratio.

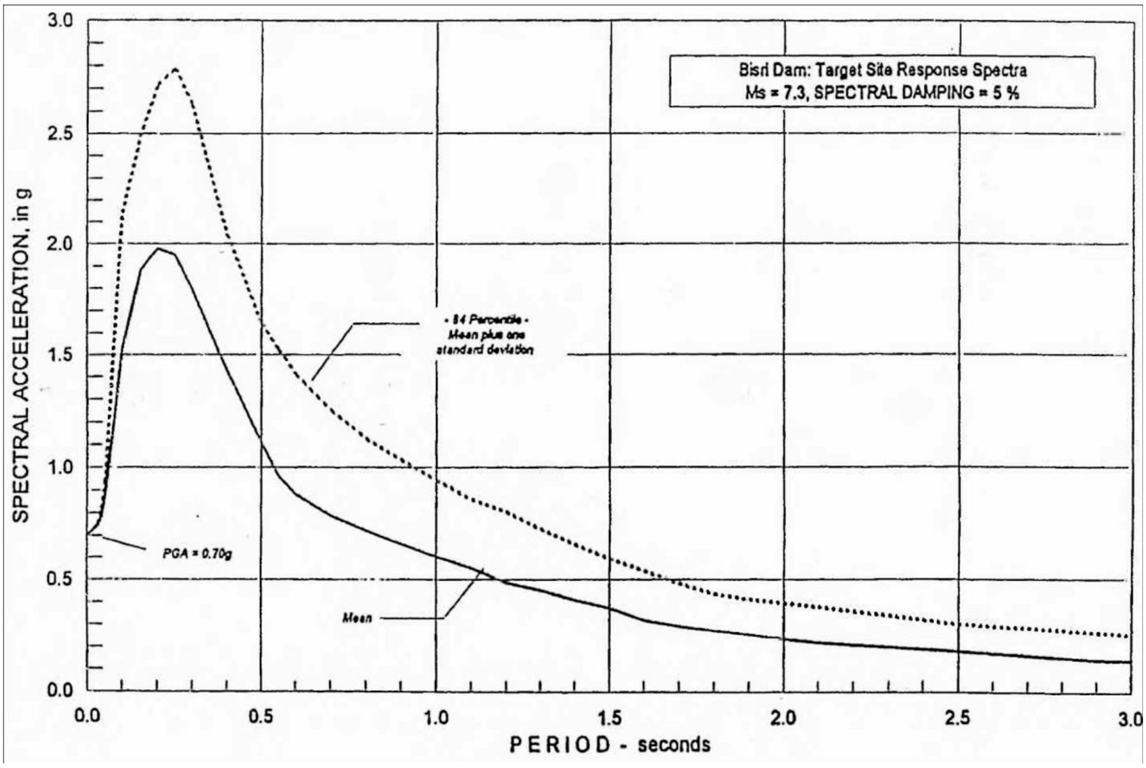

Figure 2. MCE target spectrum (ECIDAH, 1997).

Although, at the time of its computation it represented the state-of-the-art, today, the validity and rationality of this MCE response spectra, developed using the knowledge that dates back to 1976 (e.g. Seed at al., 1976), is highly questionable, since over the almost 40 year period, now we have both quantitatively and qualitatively much better earthquake strong motion data sets and consequently, much more reliable ground motion prediction expressions (GMPEs). Similarly, the frequency-independent, 2/3 ratio between vertical and horizontal motion considered in ECIDAH (1997) is now outdated, especially for near fault conditions.

Bard (2014), who has provided three sets of ground motion compatible with the MCE response spectra originally presented in ECIDAH (1997), has also stated that "*those which have been used in the reference documents are now quite old (1982) and look obsolete, compared to the huge wealth of high quality data recorded by the new networks deployed throughout the world*".

Under the Subsection 5.1 of the Dam Review Board (DRB) Report No.1 (November 2013), it has been requested to "Perform a seismic hazard study to define the characteristics of the earthquakes that may be encountered at the site (design basis ground motion levels)". Although there was no objection to the methodology used in the development of the three sets of spectrum compatible ground motion in Bard (2014), the Board has questioned the target spectra used in this process.

As such, the earthquake resistant design criteria need to be updated and the design basis response spectra associated with the SEE level ground motions need to be recomputed based on the current data, knowledge and state-of-the-art methodologies.

Current approach for the earthquake resistant design of the dams relies on the "performance based design" as documented in the guidelines of the Committee on Seismic Aspects of Dam Design of the International Commission on Large Dams (ICOLD, 2010). ICOLD guidelines call for a two level design based on the Operating Basis Earthquake (OBE) and the Safety Evaluation Earthquake (SEE) together with the associated performance objectives.

Operating Basis Earthquake (OBE)

ICOLD Guidelines (1989). "*The Operating Basis Earthquake (OBE) represents the level of ground motion at the dam site at which only minor damage is acceptable, the dam, appurtenant structures and equipment should remain functional and damage easily repairable from the recurrence of the earthquake not exceeding the OBE*".

FEMA (2005). "*The OBE is an earthquake that produces ground motions at the site that can reasonably be expected to occur within the service life of the project. The associated performance requirement is that the project functions with little or no damage, and without interruption of function. The purpose of the OBE is to protect against economic losses from damage or loss of service. Therefore, the return period may be based on economic considerations*".

USACE (1995, 2007): "*The OBE is an earthquake that can reasonably be expected to occur within the service life of the project, that is, with a 50% probability of exceedance during the service life (corresponding to a return period of 144 years for a project with a service life of 100 years). .... The OBE is determined by a PSHA.*"

Maximum Credible Earthquake (MCE)

ICOLD (1989, 2010). "*The MCE (Maximum Credible Earthquake) is the largest conceivable earthquake that appears possible along a recognized fault or within a geographically designated tectonic province, under the presently known or presumed tectonic framework*". ICOLD (2010), dropping the definition of MDE, further states that "*The most severe ground motion affecting a dam site due to an MCE scenario is referred to as the MCE ground motion".*

USACE (1995, 2007): "*This earthquake is defined as the greatest earthquake that can reasonably be expected to be generated by a specific source on the basis of seismological and geological evidence. Since a project site may be affected by earthquakes generated by various sources, each with its own fault mechanism, maximum earthquake magnitude, and distance from the site, multiple MCE's may be defined for the site, each with characteristics ground motion parameters and spectral shape. The MCE is determined through a DSHA*".

FEMA (2005). "*The MDE or Safety Evaluation Earthquake (SEE) is the earthquake that produces the maximum level of ground motions for which a structure is to be designed or evaluated. The MDE or SEE may be set equal to the MCE or to a design earthquake less than the MCE, depending on the circumstances*".

ASCE 7-10 (2010): Uses the "*Maximum Considered Earthquake (MCE)*" to define the 2%/50 (2475 year average return period) earthquake ground motion, based on probabilistic methods. Deterministically it represents the ground motion that would result from the Maximum Credible Earthquake.

IAEA Safety Guide SSG-9 (2010) recommends both a PSHA and a DSHA be used for the assessment of the SL-2 (often denoted as a safe shutdown earthquake, SSE) level design basis ground motion, generally associated with a 10,000 year average return period. For DSHA implicitly a median-plus-one sigma hazard quantification is suggested to commensurate with the SL-2 level ground motion.

On the basis of these evaluations and considerations the following design basis ground motion levels can be decided:

The Operating Basis Earthquake will be determined as the probabilistically assessed earthquake ground motion for an average return period of 144 years. Under the action of this level of ground motion, the dam, appurtenant structures and equipment should remain functional and, if any, the minor damage should be easily repairable.

The Safety Evaluation Earthquake is the maximum level of ground motion for which the dam should be designed or analyzed. For the earthquake resistant design of the Bisri Dam the SEE level ground motion will be determined to correspond to the 84-percentile deterministic MCE (i.e. median plus one standard deviation). Under the SEE the stability of the dam and life safety must be ensured with no uncontrolled release of water from the reservoir.

3. NEO-TECTONICS

The Bisri Dam site is located in a region of complex tectonic plate interaction along the Dead Sea Transform Fault (DSTF), which is a 1000-kilometer-long left-lateral transform fault system that stretches between the Gulf of Aqaba, at the northern edge of the Red Sea, to the Taurus Mountains in southern Turkey. (Inset in Figure 3)

Daeron (2005), Gomez et.al (2007), Nemer and Meghraoui (2006) and Elias et al. (2007) provide studies of the tectonic features within Lebanon and vicinity. Essentially, DSTF can be divided into two main sections joined by a restraining bend along Lebanon. Within the Lebanese restraining bend, the DSTF splits into five main fault branches: the Roum, Yammouneh, Seghaya, Rachaiya and Hasbaya faults (Figure 3 through Figure 6).

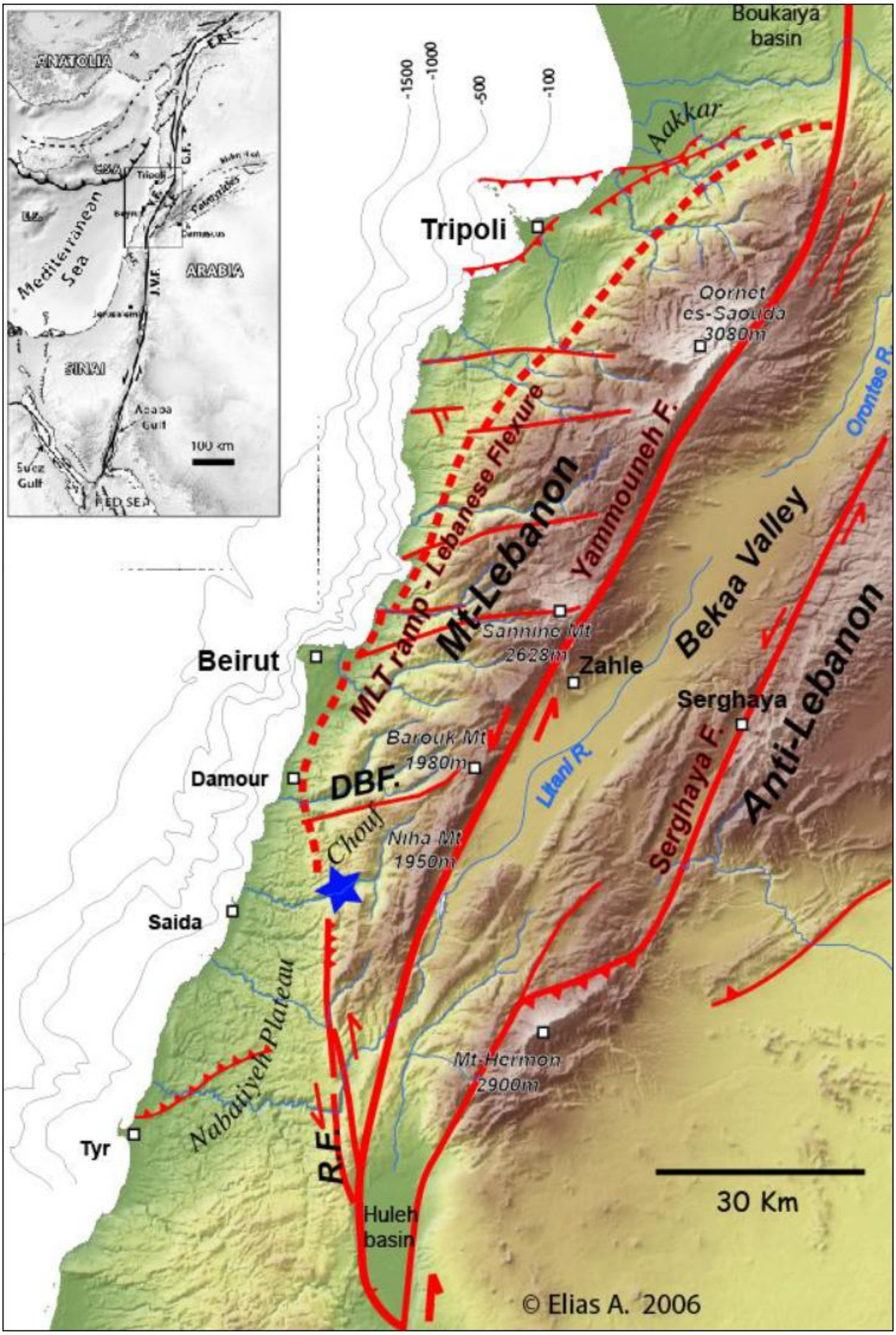

Figure 3. The Bisri Dam site (blue star) within the tectonic map of Lebanon. Red lines are for active faults. Dashed red line is for the blind Mount Lebanon Thrust ramp below the Lebanese Flexure as mapped on the surface. (R.F. = Roum Fault; DBF.= Damour Beiteddine Fault) (after Elias, 2014).

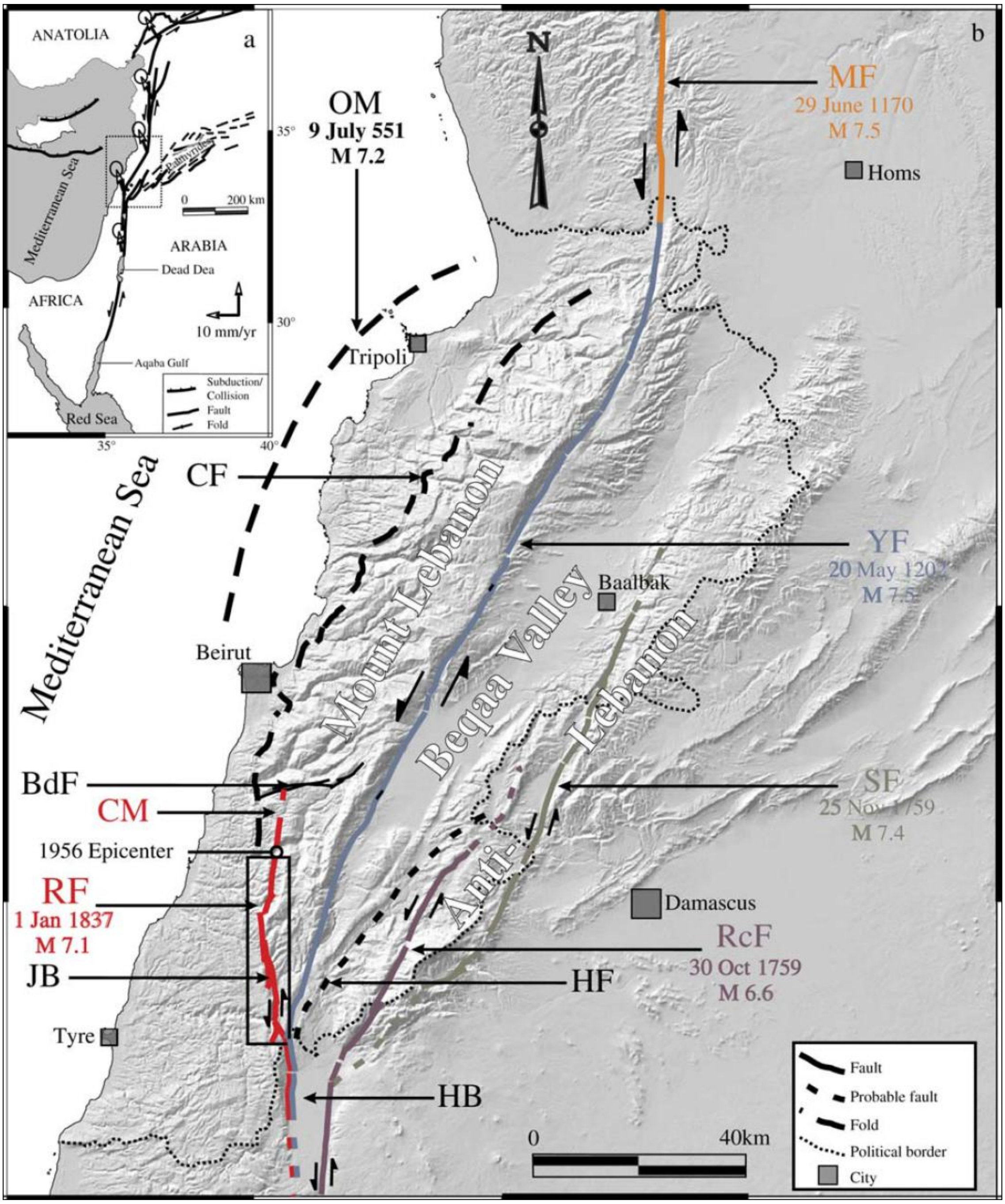

Figure 4. (a) The Dead Sea Transform Fault extending from the Gulf of Aqaba to southeast Turkey. It is a left-lateral transform fault with a general NeS trend except in Lebanon where it bends rightward to form a restraining bend. White arrows indicate the GPS velocities of the Arabian plate relative to adjacent plates. (b) Lebanese restraining bend showing the main units and structures: BdF, Beit-ed-Dine fault; CF, Coastal Flexure; CM, Chouf Monocline; HB, Hula basin; HF, Hasbaya fault; JB, Jarmaq basin; MF, Missyaf fault; OM, Offshore monocline; RcF, Rachaya fault; RF, Roum fault; SF, Serghaya fault; YF, Yammouneh fault. Note the epicentre location of the double-shock of the 1956 earthquake at the intersection between Roum fault and Chouf Monocline. Coloured faults are associated with large historical events (indicated by date and magnitude) that took place within the Lebanese restraining bend (after Nemer and Meghraoui, 2006).

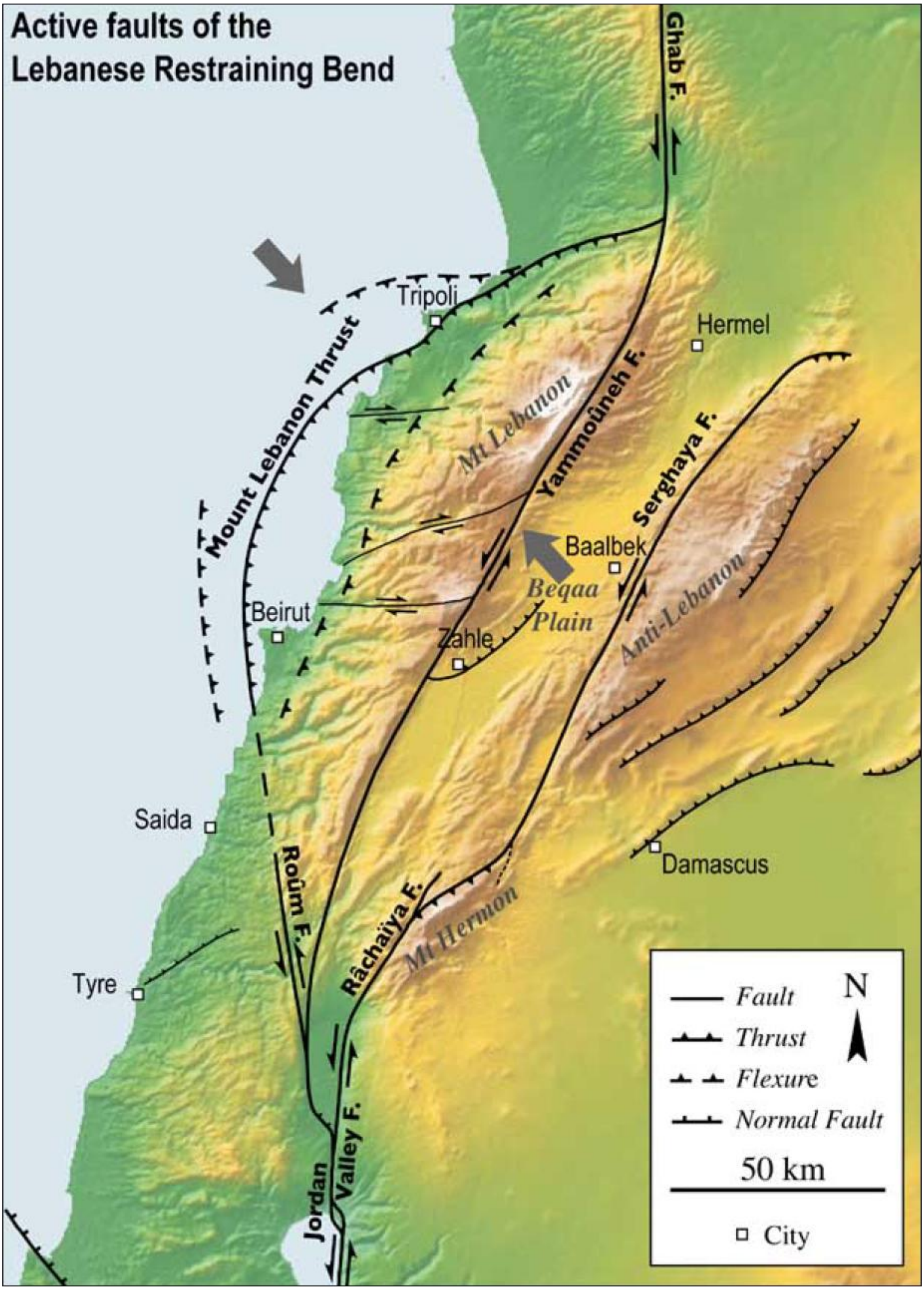

Figure 5. Map of the active faults in the Lebanese Restraining Bend (Daeron, 2005).

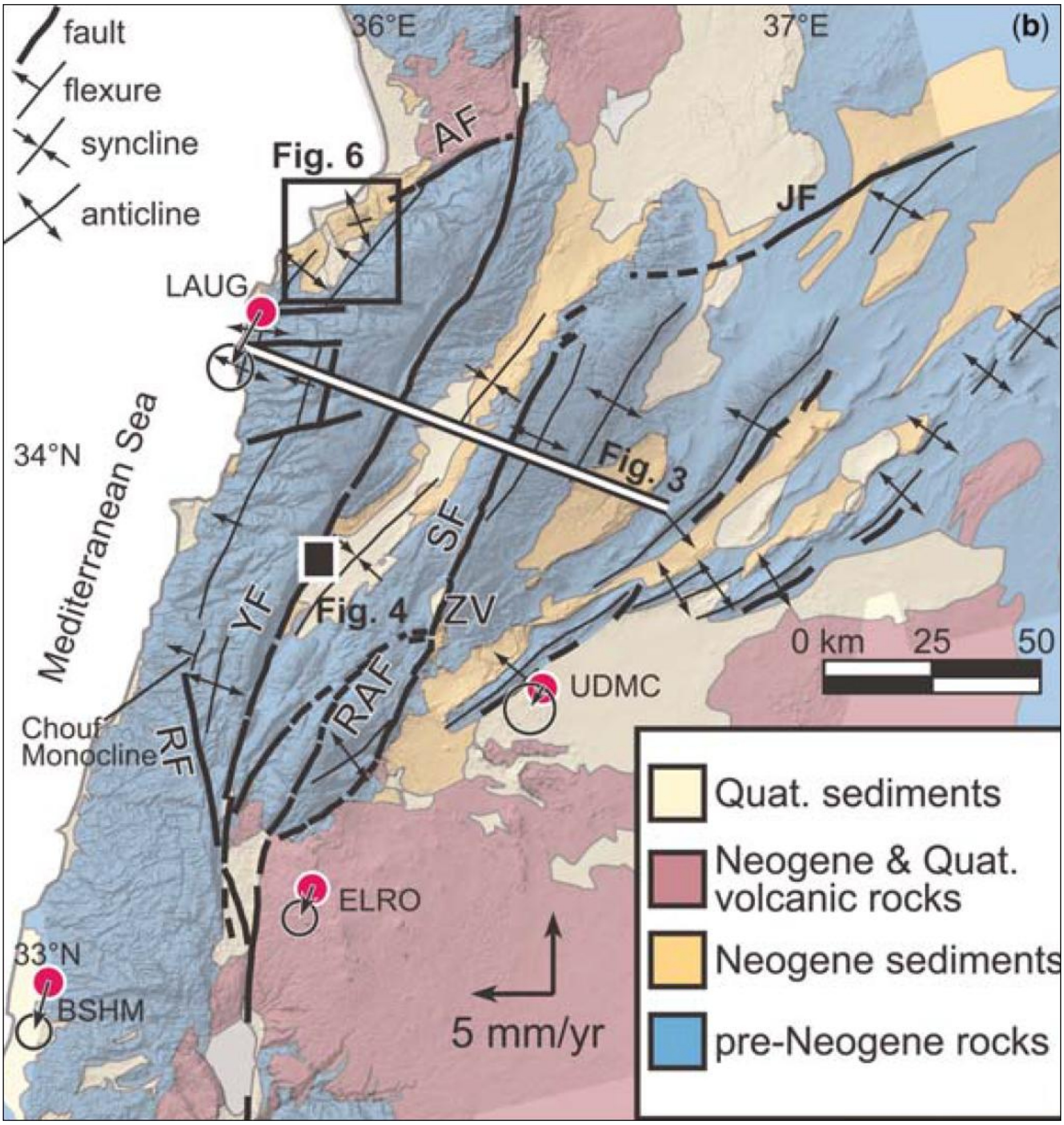

Figure 6. Map showing the general geology and the structure of the restraining bend. Arrows indicate velocities of continuous GPS sites (red circles) in an Arabia-fixed reference frame (Reilinger et al. 2006). Abbreviations: YF, Yammouneh Fault, SF, Serghaya Fault; RF, Roum Fault, RAF, Rachaya Fault; JF, Jhar Fault; HB, Hula Basin; MH ¼ Mount Hermon; TNP, Tyre–Nabatieh Plateau; and ZV, Zebadani Valley (After Gomez et al, 2007).

Elias (2014) provides a through description of the tectonic entities that would control the earthquake hazard at the Bisri Dam site.

The Yammouneh Fault is an approximately 297-km-long, vertical strike-slip fault within the Lebanese restraining bend. The fault is active and has produced major earthquakes, the latest and strongest being M~7.6, 1202 event, with a coseismic slip of 5-6m (Daeron et al. 2007). The fault has a slip rate of 4-6mm/yr (Daeron et al. 2004, Gomez et al. 2007). The Yammouneh Fault is at ~12km from the Dam site (Elias, 2014).

The Roum Fault is a secondary branch of the Plate Boundary that splays into the Lebanon at Huleh south. It has a N-S strike over most of its 35km length. The fault is considered as mostly left-lateral strike-slip with a compressive component increasing towards the north. All known geological mapping literature concur that the Roum Fault doesn't reach beyond the Awali river (the Dam site) as it merges into the Jezzine Anticline to the south. The Mazraa or Chouf Monocline to the north would correspond to its northern structural counterpart or equivalent (Dubertret 1955, Nemer & Meghraoui 2006, Elias et al. 2006). The Roum Fault has produced the two most recent destructive earthquakes onshore Lebanon the M~7, 1837 and the M~5.7, 1956 events. Its horizontal slip rate is ~1mm/yr.

Detailed mapping done by Nemer & Meghraoui (2006) indicated that the lineament of the Roum Fault is limited in extent to about 35 km from north of the Hula basin to the Awali river and that it disappears at ~ 33.67°N, where the fault bends northward and merges with the Chouf monocline. Nemer & Meghraoui (2006) further documented a northward decrease in the amount of deflection of major drainages across the Roum Fault. This was inferred to represent a northward decrease in strike-slip offset along the fault.

Similarly Gomez et al. (2007) states that the Roum fault lineament can be traced until approximately 33.84 N latitude. At the northern extent, the trace of the Roum Fault disappears as it bends northward and merges with the hinge of the Chouf monocline.

On the basis of the investigations done by Elias (2014) the mapped geological fault trace stops ~2km south of the Dam-site. Although the continuation of the fault further north into the Bisri valley was suggested by previous site investigations the site-specific studies and reassessment by Elias (2014) reveals no evidence of such continuation (Figure 7).

The Mount-Lebanon Thrust (MLT) ramp of the Lebanese Flexure is a crustal ramp that accommodates shortening associated with the Lebanese Restraining Bend of the Dead Sea Fault System. It is a shallow, east-dipping fault that offsets the seafloor offshore Lebanon and is connected at depth with the Yammouneh Fault. The surface expression of this blind ramp in the geology of the area is the Lebanese Flexure and associated folds. The Chouf or Mazraa flexure is part of this system. The MLT and the Flexure run along the entire Lebanese coast between Saida and Tripoli. A complex system of offshore thrust faults is also associated with the MLT (Figure 8a). The MLT and the This 40-45°, east-dipping thrust ramp mostly located in the upper, seismogenic, ~16km of the crust (Figure 8b) has a vertical slip-rate ~1.5 mm/yr (Elias et al. 2007). Evidence of rupture of the Quaternary and recent marine sediments at the tip of the MLT faults were observed during the SHALIMAR marine geophysical campaign (Elias et al. 2007, Carton et al. 2009). The location of the offshore ruptures and faults corresponds with the observed uplift of the shoreline as indicated by stairs of uplifted marine terraces (Elias et al. 2007).

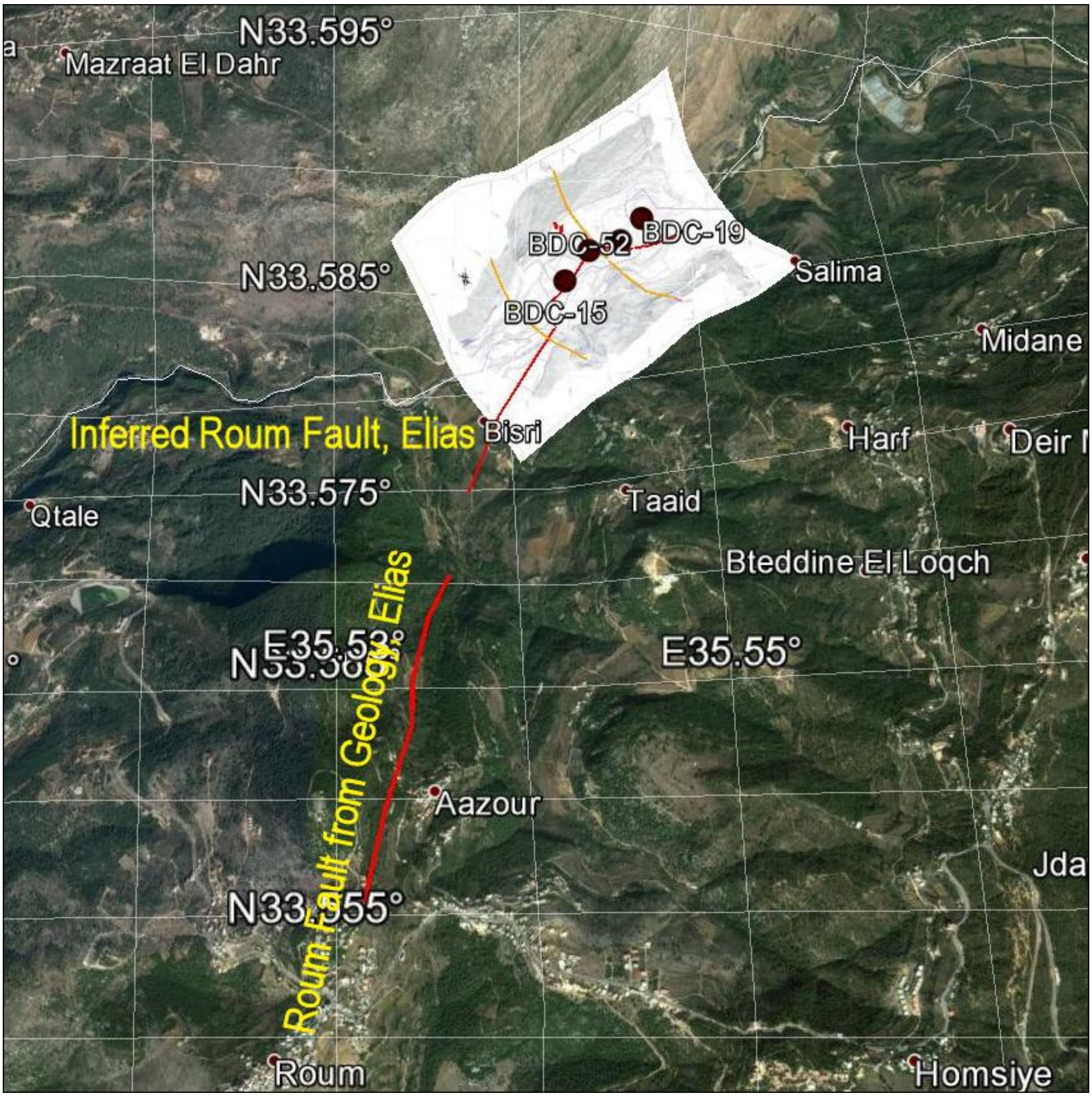

Figure 7. Location of the Roum Fault to the south of the Bisri Dam site (After Elias, 2014).

The Bisri Dam site is located in the south of MLT ramp in the transfer area between the MLT and the Roum Fault (Figure 8a). A geological cross-section of the area at the south end of the ramp helps constrain the geometry of the ramp at depth (Figure 8b).

The M~7.5, AD 551 earthquake is the best-known event associated with this fault system. It ruptured the entire ramp where possible co-seismic rupture between 2-3m was also inferred based on measured uplift indicators along the shoreline between Tripoli and Saida (Elias et. al, 2007). The surface rupture of this event was located in the offshore at the tip of the main ramp and further away on some of the smaller thrusts located in front of it (to the west). Based on the mapped fault trace, the expected MLT co-seismic rupture can therefore be at least 40km away from the Bisri Dam site (Elias, 2014).

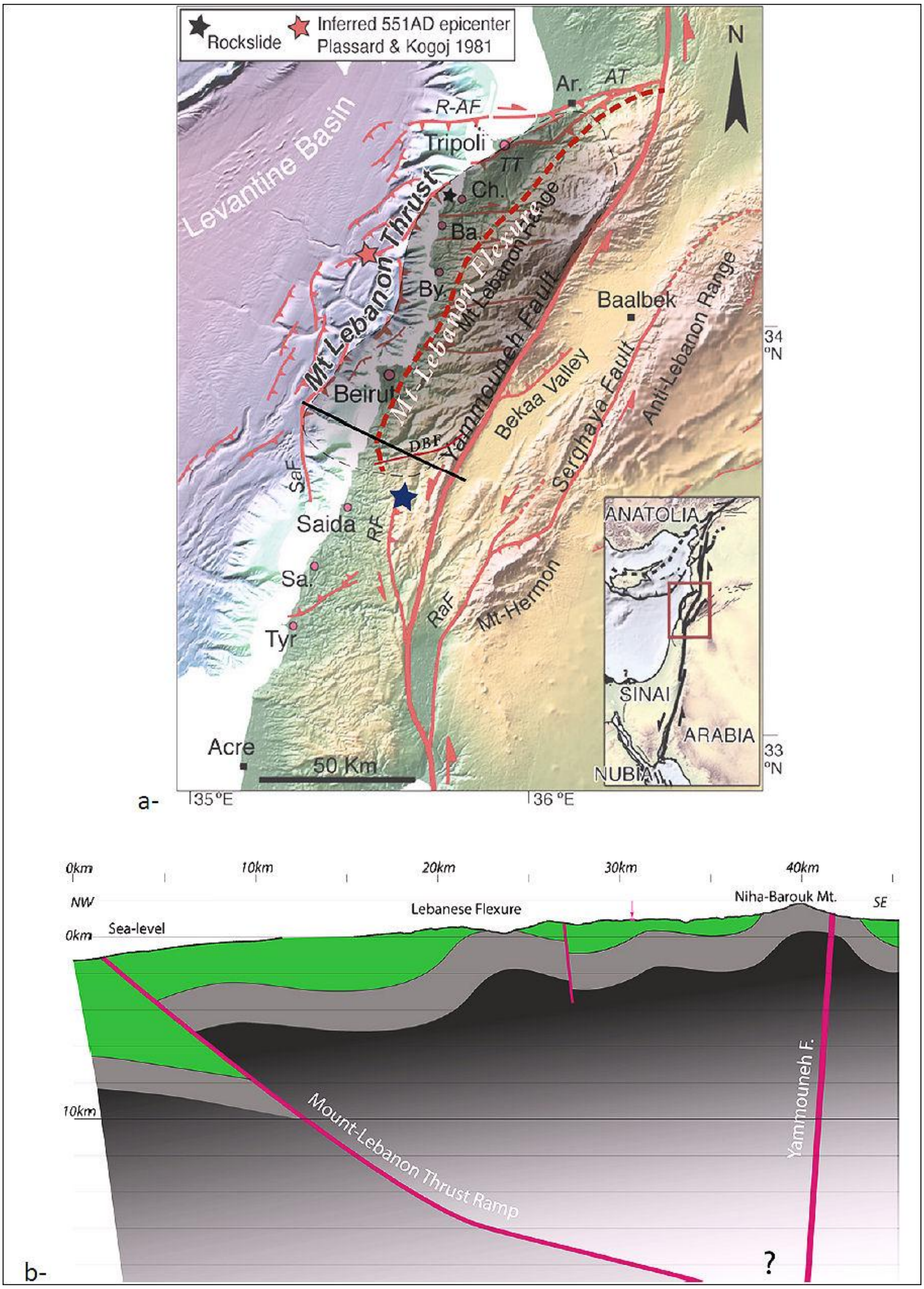

Figure 8. (a) Regional neo-tectonic map. Red lines are for active faults. Dashed red line is for the Mt-Lebanon Flexure. Gray shadow represents the ramp surface of the MLT in the subsurface. Bisri Dam site (blue star) within the tectonic map of Lebanon. (RF= Roum Fault; DBF= Damour Beiteddine Fault), (b) Interpretatitve NW-SE cross-section of Mt-Lebanon showing the deep structure of the range (Elias, 2014).

4. SEISMICITY

The Dead Sea region has been affected by numerous moderate to large magnitude earthquakes over the course of history. Despite the lack of large earthquakes during the instrumental period, historical records suggest a substantial seismic activity (e.g. Ambraseys et al., 1994; Ambraseys and Jackson, 1998).

Three earthquakes stand out in the history of seismic activity in Lebanon: the earthquakes of 551 A.D., 1202 A.D and 1759 A.D. The magnitudes of these earthquakes were estimated based on historical accounts, to be in excess of 7.0, and caused devastating destruction in most coastal cities including Beirut, as well as the ancient city of Baalbeck inland (Sadek and Harajli, 2007). According to Ambraseys and Barazangi (1989), during the last millennium alone, more than 8 strong earthquakes with magnitude greater than 6.5 have struck along the northern continuation of the Dead Sea fault in Lebanon and Syria.

A schematic map of main active faults and the major historical earthquakes of Lebanese restraining bend is provided in Figure 9 after Daeron et al. (2005). Figure 10 (after Ferry et al., 2011) illustrates the seismicity of the historical and instrumental periods along the Dead Sea Transfrom fault, whereas Figure 11 presents the catalogue of the post-1900 period compiled for the EMME project (Zare et al., 2014), together with the location of the Bisri Dam.

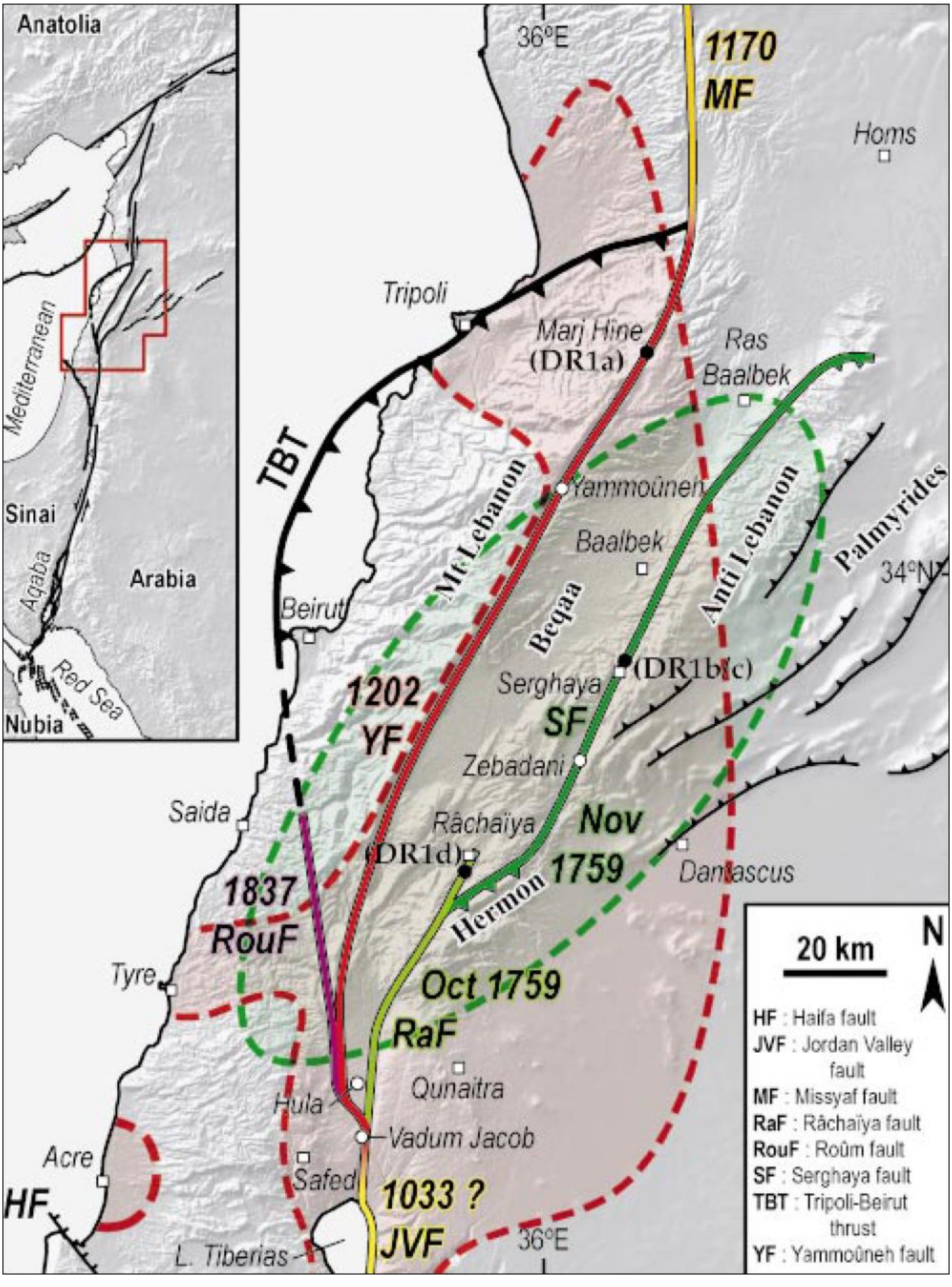

Figure 9. Main active faults and the major historical earthquakes. Bold colored lines show maximum rupture lengths of large historical earthquakes in past 1000 years. Bold dashed lines enclose areas where intensities VIII were reported in A.D. 1202 (red) and November 1759 (green) according to Ambraseys and Melville (1988) and Ambraseys and Barazangi (1989) (after Daeron et al., 2005).

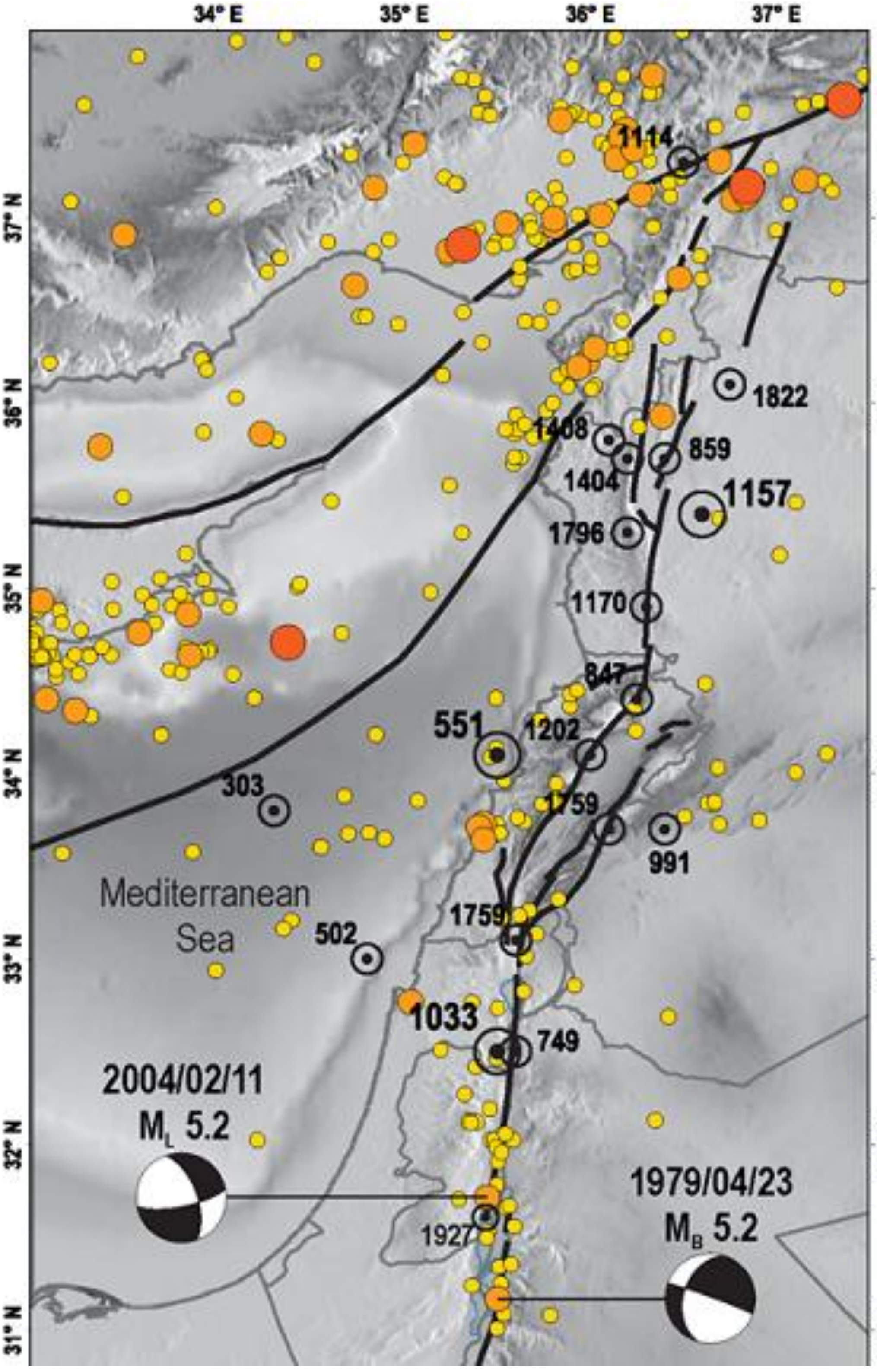

Figure 10. Seismicity of the Dead Sea Transform system (after Ferry et al., 2011). Instrumental events with M ≥4 from 1964 to 2006 (IRIS Data Management Center); Historical events with I ≥ VII (Ambraseys and Jackson, 1998; Sbeinati et al., 2005) in open circles.

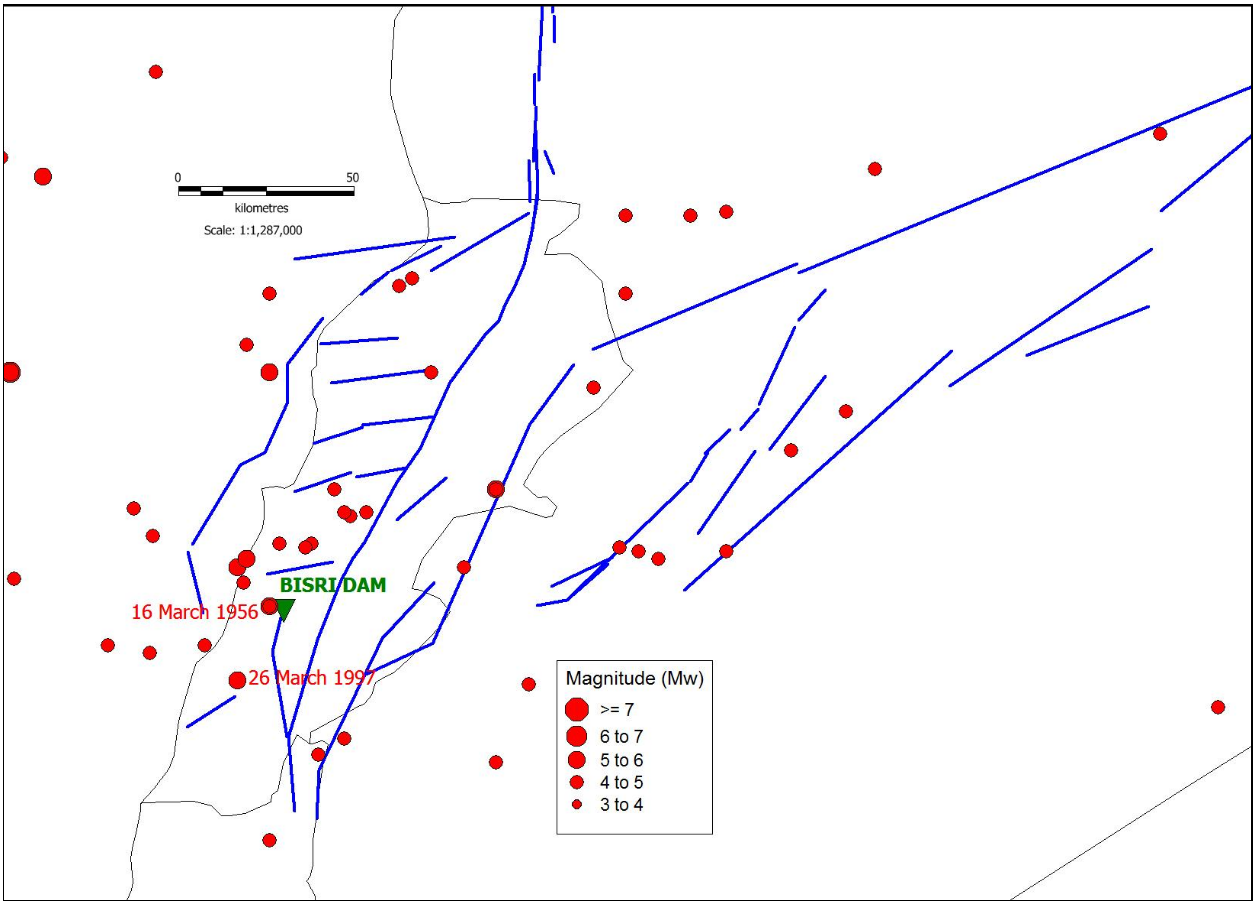

Figure 11. The earthquake catalogue of the region for the period 1900 – 2006 (Zare et al., 2014). Blue lines represent the active fault database compiled for the EMME project.

The Roum Fault which is a secondary branch of the Plate Boundary that splays into the Lebanon at Huleh south has exhibited considerable seismic activity (e.g. Girdler, 1990; Butler, 1997; Darawcheh et al., 2000; Khair, 2001).

The instrumental seismicity (ISC, NEIC, EMSC bulletins) shows a scatter of moderate earthquake epicentres around the Roum Fault with one relatively important event, namely the double shock of 16 March 1956 (Ms 4.8, 5.1). This earthquake left 136 dead, 6000 homes destroyed and about 17000 heavily damaged. More recently, an earthquake of magnitude 5.3 struck Lebanon on March 21, 1997 (Figure 11). The earthquake reached Beirut with an intensity of about VI on the Modified Mercalli (MM) intensity scale and resulted in high levels of shaking for a short duration of about 6 seconds (Sadek and Harajli, 2007)

A magnitude 6.4 or larger earthquake on January 1, 1837 was felt in towns near the site, and aftershocks were felt for at least three months following the main shock (Ambraseys, 1997). From the damage distribution (Figure 12) the source of these events associated with the Roum Fault. However, even though this event seems to have been large enough to produce surface ruptures, no field observations have yet indicated that such ruptures exist (Ambraseys, 1997).

This event as reviewed by Ambraseys (1997):

- caused damage in a narrow zone which extended from Saida to Marjayun, Bshara and Lake Tiberias,
- claimed 3,000 victims
- is associated with three large aftershocks (Jan. 16, 22 and May 20, 1837)
- is likely associated with rupture on the Roum fault and its S continuation W of the Hule

- no conclusive evidence of surface faulting
- has and estimated magnitude of 7.0.

The palaeoseismic study conducted by Nemer and Meghraoui (2006) shows the occurrence of at least 4-5 large seismic events with surface ruptures associated with the Roum Fault during the last 10510 years, the last event being post 84-239 AD (Nemer and Meghraoui, 2006). It is believed that the 1 January 1837 earthquake is the most likely candidate for being the most recent large seismic event along the Roum Fault. A slip-rate of 0.86-1.05 mm/year was derived. Nemer and Meghraoui (2006) further states that the potential of the Roum Fault for producing large earthquakes similar in magnitude to that of 1837 must be taken into consideration within any seismic hazard study of the region.

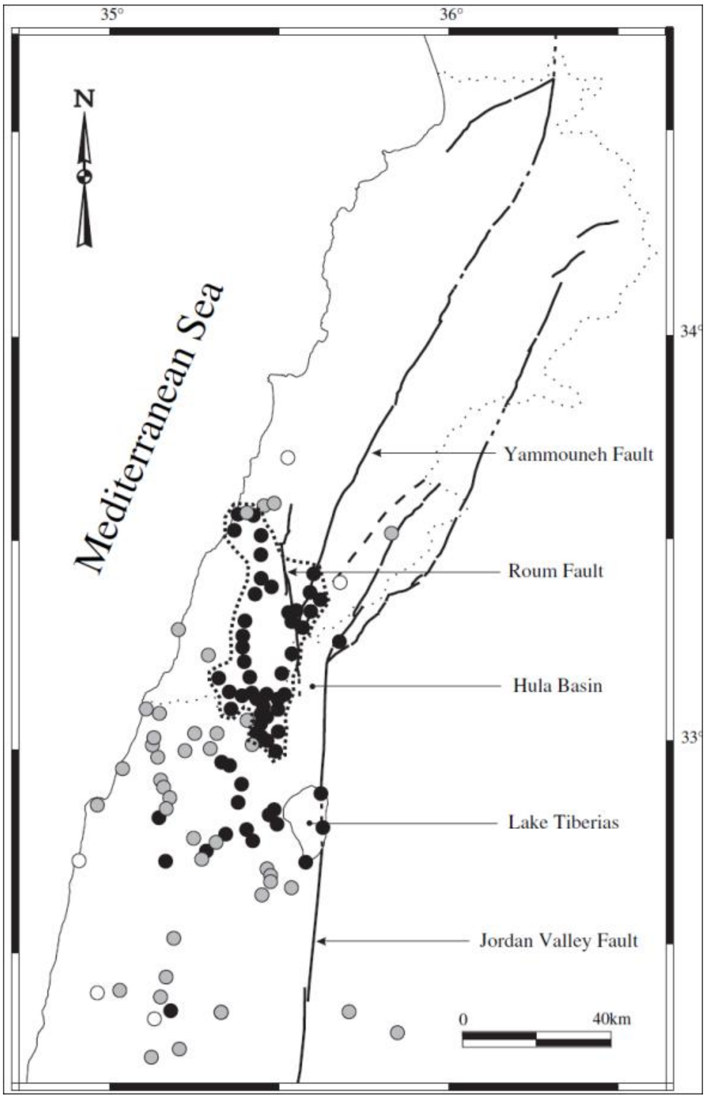

Figure 12. The epicentral region of the 1837 earthquake (after Nemer and Meghraoui, 2006; modified from Ambraseys, 1997). Circles indicate near-field locations of affected sites. Black, grey, and white circles correspond to MSK intensities of VIII, VII, and VI, respectively. Dotted contour bounds the area of concentrated maximum damage.

Although lacking seismic activity in the 20th century, recent paleoseismic investigations have indicated that the Yammouneh Fault, which is the main strand of the Dead sea Transform

Fault in Lebanon, is tectonically active and has ruptured in infrequent but large earthquakes (M>7.0) (e.g., Daeron et al., 2005, 2007; Nemer et al. 2008), including the Ms 7.6 event in 1202 (Figure 13). Another major event is the Ms 7.4 November 25, 1759 earthquake, which occurred on the Serghaya Fault, to the East of the Yammouneh Fault (Figure 14). Despite the apparent lack of present day seismicity, all recent studies reviewed suggested that the Yammouneh Fault is more likely to accommodate most of the plate motion within the Lebanese restraining bend (e.g., Gomez et al., 2003; Daeron et al., 2004; Gomez et al., 2007).

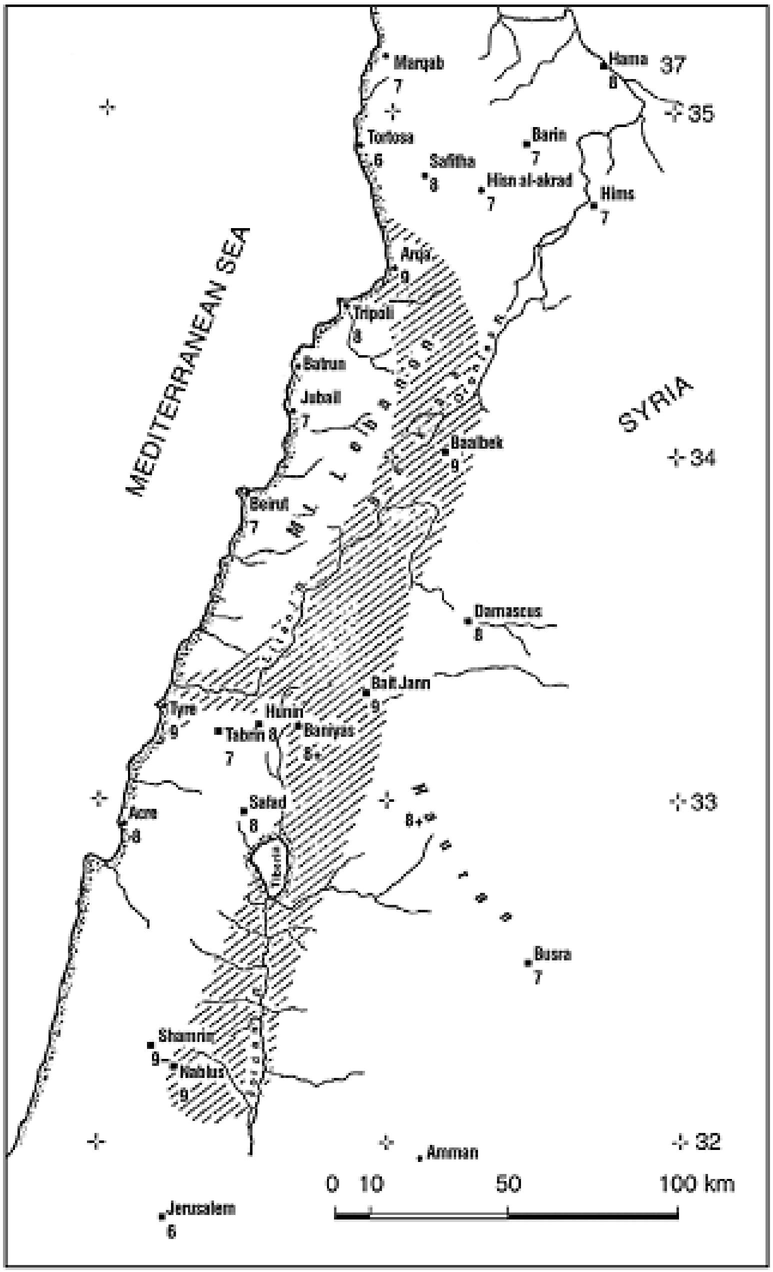

Figure 13. Map of intensity distribution for May 20, 1202 earthquake (Ambraseys and Melville, 1988, figure from Sbeinati et al., 2005). Shaded zone is the most affected region.

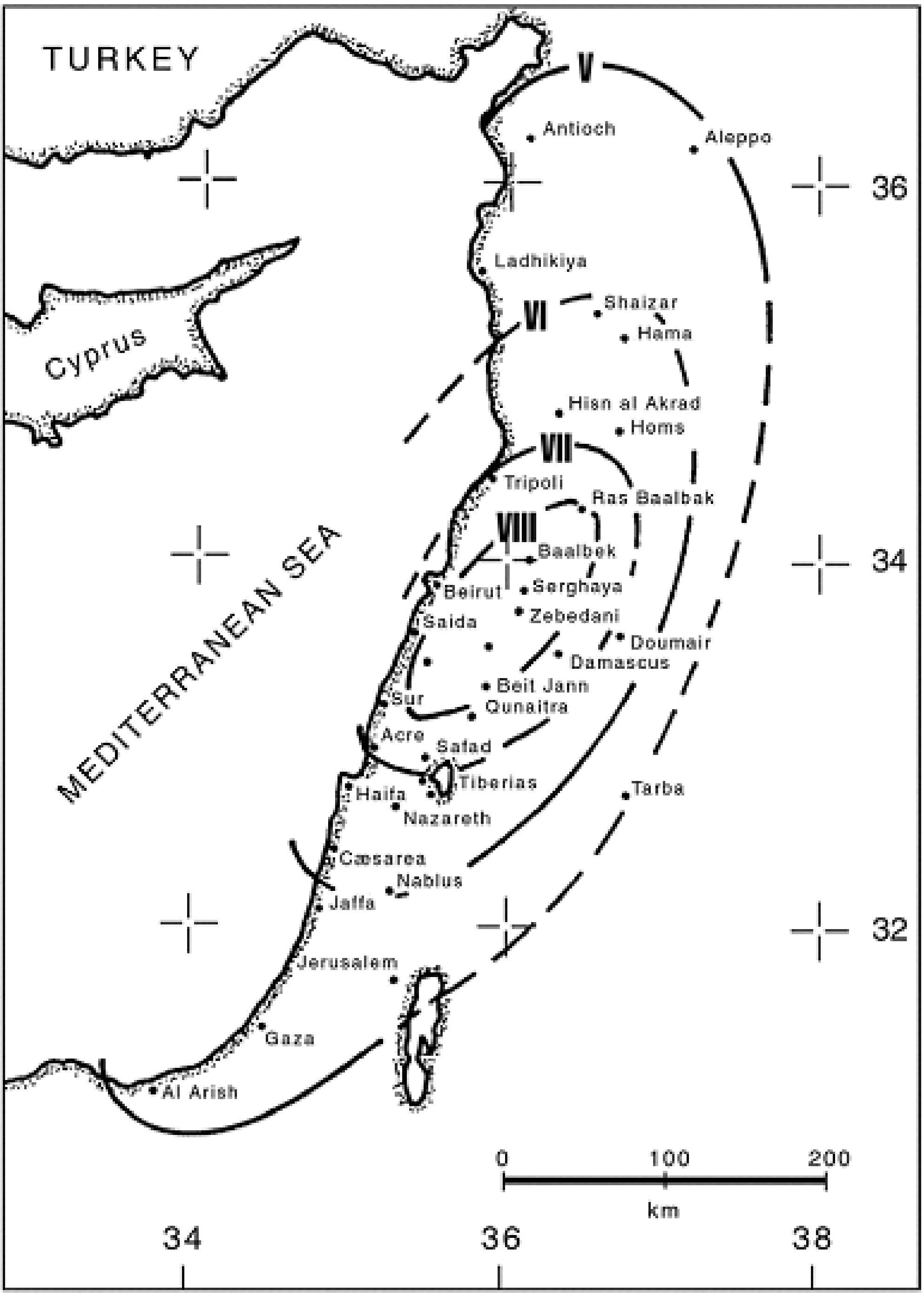

Figure 14. Map of intensity distribution for November 25, 1759 earthquake (Ambraseys and Barazangi, 1989; figure from Sbeinati et al., 2005).

The Mount Lebanon thrust (MLT) has been proposed as the source of the 551 earthquake (M7.2) offshore of Lebanon (Elias et al., 2007) The 9 July 551 earthquake was a destructive event that affected the entire Lebanese coastal area and generated a Tsunami. Darawcheh et al. (2000), based on Byzantine documents, assessed a magnitude Ms of 7.1 to 7.3, proposed an epicentre location offshore Beirut, and correlated it with the nearby strike-slip Roum Fault. Such a correlation, however, lacks evidence of corresponding faulting along the coast (Figure 15).

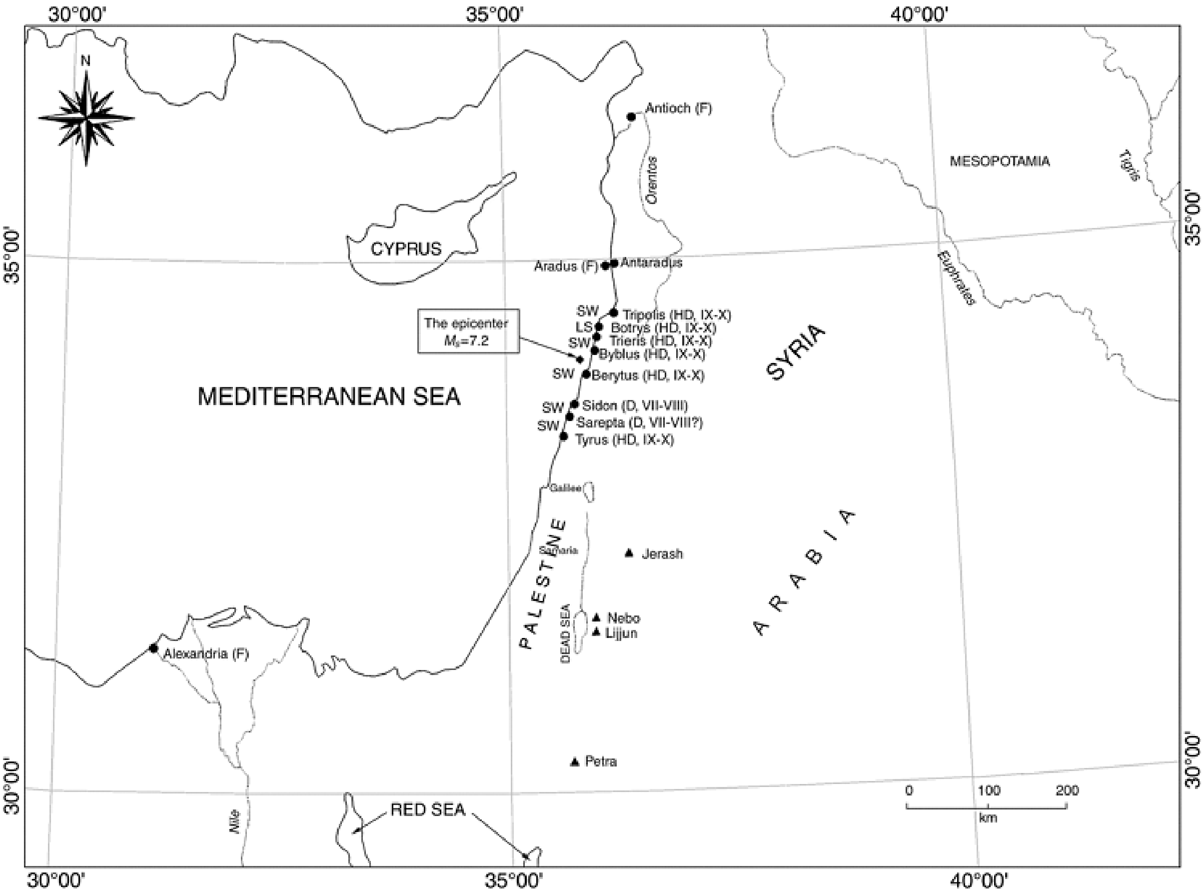

Figure 15. Map of intensity distribution for July 9, 551 A.D. earthquake. F – felt; D – damage; LS – landslide, and SW – Sea-Wave. Triangles represent possible damaged archaeological sites (Darawcheh et al., 2000; figure from Sbeinati et al., 2005).

5. GROUND MOTION PREDICTION EQUATIONS

Ground-motion prediction equations (GMPEs) relate a ground-motion parameter to a set of explanatory variables describing the earthquake source, wave propagation path and local site conditions. These independent variables include magnitude, source-to-site distance and some parameterization of local site conditions, and often style-of-faulting and hanging wall effects. The seismic hazard analysis is faced with the difficult task of deciding which GMPEs to use for a given project, since the resulting predicted spectra are strongly dependent on the GMPEs chosen.

In today's understanding the main separation of the GMPE's are based on the three major tectonic regimes: active crustal regions (ACRs); subduction zones (SZs); and stable continental regions, (SCRs) (Douglas, 2011 and Douglas et al., 2012). In this connection, Bommer et al. (2010) state that the first basis for exclusion of a GMPE model is that it is from a tectonic region that is not relevant to the location of the site for which the hazard assessment is being conducted.

The Next Generation Attenuation (NGA) project developed a series of GMPEs intended for application to geographically diverse regions (substantially from Taiwan, California, and Europe/Turkey); the only constraint is that the region be tectonically active with earthquakes occurring in the shallow crust. The NGA GMPEs are presented by Abrahamson and Silva

(2008), Boore and Atkinson (2008), Campbell and Bozorgnia (2008), Chiou and Youngs (2008), and Idriss (2008). An important issue for many practical applications is whether ground motions or GMPEs for one region can be applied to another. Rigorous comparative studies conducted by Campbell and Bozorgnia (2006), Stafford et al., (2008) and Scasserra, et al. (2009) and the GMPE studies of Akkar and Bommer (2010) indicate that the NGA-GMPEs are applicable and can be utilized for earthquake hazard in assessment Europe.

Due to the lack of recorded strong motion data in and around Lebanon, no comparisons could be made between the existing GMPEs and actual recordings near the site. The region where the Bisri Dam is located essentially qualifies for the ACR type tectonic regime. As such, we will first review the pre-selected GMPEs that would be appropriate to use for the earthquake hazard assessment for the Bisri Dam.

In connection with the Global Earthquake Model (GEM, www.globalquakemodel.org) the following three models were selected (Stewart et.al, 2014) for Active Crustal Regions: Akkar and Bommer (2010), Chiou and Youngs, (2008), and Zhao et al. (2006). These models provide a good geographical spread (respectively, one for Europe and the Middle East, one global, and one predominantly for Japan). Although, the Boore and Atkinson (2008) model was seriously considered for selection as an alternative or supplement to the Chiou and Youngs (2008), the Chiou and Youngs (2008) was preferred over the other pre-selected NGA models because its magnitude-scaling and its anelastic attenuation term were considered to be more appropriate than the other NGA models.

The GMPEs that are chosen in the context of a regional GEM project (SHARE; Seismic Hazard Harmonization in Europe, www.share-eu.org) for Shallow Crustal Active Regions were: Akkar and Bommer (2010) Cauzzi and Faccioli (2008) Zhao et al. (2006) Chiou and Youngs (2008) with the respective weights of 0.35, 0.35, 0.1 and 0.2, to be employed in the logic-tree analysis (Delavaud et.al, 2012a,b).

Under the framework of the Earthquake Model of the Middle East (EMME, www.emme-gem.org) project (a regional project of GEM, that provided a uniform assessment of the seismic hazard and risk in the Middle East and Caucasus region) the ground motion prediction equations (GMPEs) applicable in the region were studied (Erdik et al, 2012). The overall seismotectonic features of the EMME region suggest the consideration of shallow active crustal (SACR) and subduction regions (SR) for seismic hazard calculations. For establishing the GMPE logic-tree for SACRs in EMME, a total of 14 ground-motion predictive models, that fulfill the pre-selection criteria set by Cotton et al. (2006), were selected as candidate GMPEs and were subjected to two analytical testing and ranking methods using a subset of EMME strong-motion (SM) databank, almost exclusively consisting of ground-motions compiled from the SACRs in EMME territory (Kale and Akkar, 2012). The overall performances of Akkar et al. (2014), Chiou and Youngs (2008), Akkar and Cagnan (2010) and Zhao et al. (2006) were found relatively better than the rest of the candidate GMPEs. For the GMPE logic-tree in EMME hazard assessments these GMPEs were used with weights of 0.35, 0.35 0.20 and 0.1 respectively.

On the basis of this review it would be appropriate to use the GMPEs of Akkar and Bommer (2010), Chiou and Youngs (2008) and Boore and Atkinson (2008) for the deterministic assessment of the earthquake hazard at the Bisri dam site. In addition to being identified by GEM, SHARE and EMME projects Akkar and Bommer (2010) and Akkar et al. (2014) are the only GMPEs that are essentially based on the Middle East data. Among these two ground motion prediction models, we suggest Akkar and Bommer (2010) to be more appropriate to be used for the DSHA study of Bisri Dam, as this model also includes the magnitude dependent standard deviation term. As such it will be given a weight of 0.5 in the logic tree combination of DSHA results from the selected GMPEs. On the other hand both Chiou and Youngs (2008) and Boore and Atkinson (2008) are based on an international database that includes more data of large magnitude near fault strike slip earthquakes, similar the deterministic earthquake scenarios considered for the Bisri Dam. In the logic tree combination both of the NGA GMPEs Chiou and Youngs (2008) and Boore and Atkinson (2008) will be assigned a relative weight of 0.25.

6. PROBABILISTIC SEISMIC HAZARD ANALYSIS

6.1. PREVIOUS STUDIES

Previous studies on the seismic hazard of the region surrounding Bisri Dam by different agencies and independent researchers provide a comparison with the results of this study. Commonly, available reports and maps focus on peak ground acceleration (PGA) at return periods of 475 years and 2,475 years. Unless noted, the site conditions utilized by these previous reports are unknown.

The Global Seismic Hazard Assessment Program (GSHAP) estimates peak ground acceleration (PGA) value at Bisri Dam of approximately 0.2g-0.3g for a 475-year return period (Grünthal et al., 1999).

A study of earthquake hazard of Jordan and the surrounding areas by Jimenez et al. (2008) indicates that Bisri Dam is located in a zone associated with PGA values of 0.2g to 0.25g for a 475-year return period.

Al-Tarazi and Sandoval (2007) estimate a PGA value of 0.2g for a 475-year return period in the Bisri Dam site for firm rock.

Seismic hazard maps produced for the U.S. Agency for International Development Assessment for Building Codes Project (Shapira and Hofstetter, 2007) show the site located in an area corresponding to PGA values of 0.2g to 0.25g for a 475-year return period at generic bedrock.

A study of seismic hazard for Lebanon by Hujier et al. (2011) shows a PGA value of approximately 0.25g for a 475-year return period and 0.35g for a return period of 950 years at the Bisri Dam site.

Another study of seismic hazard for Lebanon by Elnashai and El Khoury (2004) shows a PGA value of 0.18g for a 475-year return period and 0.25g for a return period of 2,500 years.

6.2. THE PSHA MODEL USED IN THIS STUDY

EMME (Earthquake Model of Middle East) Project (www.emme-gem.org) is a comprehensive undertaking that aimed to develop a homogenous hazard model for the Middle East Region combining both local, regional and international expertize. American University of Beirut (AUB) was the local partner of the project in Lebanon, together with Rafiq El-Khoury and Partners. The active fault database, the earthquake catalogue and the fault and areal source models of the region have been compiled and developed with the contribution of Dr. Ata Elias from AUB.

The source zonation model of EMME has two distinct branches, one being the areal source (AS) model and the second being the fault source + background seismicity (FS) model.

Poissonian, (i.e. earthquake occurrence without memory) approach has been adopted in the hazard modelling. The AS model assumes that the seismicity is homogenously distributed over each delineated source zone and the magnitude –frequency distribution for each source zone is obtained based on the Gutenberg-Richter earthquake recurrence model, using the homogenized and declustered catalogue of the region (Zare et al., 2014). The FS model has two components: the fault sources and the background seismicity. The active fault database of the region compiled within the project has been used to develop a fault source model, in which segments of the active fault database are converted to linear sources, each defined by top and bottom depths, dip and rake angles, Mmax and slip rates. Earthquakes with magnitude 6 and higher are assumed to occur on fault sources and the activity rate for the fault sources is determined from the slip rate. To complete the fault source model in the regions where faulting information is not available (either by lack of faults or by lack of information) the background seismicity is used. The earthquake catalogue of the post-1900 period is used to obtain smoothed seismicity model with a grid spacing of 0.1 degrees and a smoothing distance of 25 km using the method proposed by Frankel et al. (1996). Within the buffer zones of the fault sources only earthquake rates for magnitude less than 6 are obtained from the smoothed seismicity as the higher magnitude contributions are assumed to come from the fault sources. For regions outside the buffer zones of the fault sources, all activity is obtained from smoothed seismicity.

As defined in Section 5 of this report, the GMPE logic tree for the PSHA study is defined as Akkar et al. (2014), Chiou and Youngs (2008), Akkar and Cagnan (2010) and Zhao et al. (2006) were found relatively better than the rest of the candidate GMPEs. For the GMPE logic-tree in EMME hazard assessments these GMPEs were used with weights of 0.35, 0.35 0.20 and 0.1 respectively.

Figure 16 presents the AS and FS models of the EMME project for the Bisri Dam region. The 0.1 degree grid sources representing the background seismicity of the FS model are not shown on the figure.

To compute the hazard at the Bisri Dam site, the AS and FS models have been assigned, as in the EMME project, weights of 0.6 and 0.4 respectively. The final results are obtained using the logic tree structure composed of the source model and GMPE levels. Figure 17 presents the hazard curves obtained for PGA, and spectral accelerations for T=0.2 sec and T=1.0 sec at the Bisri Dam site from the EMME model. The values obtained for return periods of 72, 144, 475 and 2475 years are also summarized in Table 1. The uniform hazard spectrum corresponding to 144 years return period is provided in Figure 18.

Table 1. PSHA results for the Bisri Dam site

	AS (%60) + FS (%40)		
Return Period (years)	PGA (g)	SA (T=0.2 sec) (g)	SA (T=1.0 sec) (g)
72	0.142	0.315	0.072
144	0.213	0.483	0.113
475	0.419	0.983	0.251
2475	0.781	1.890	0.528

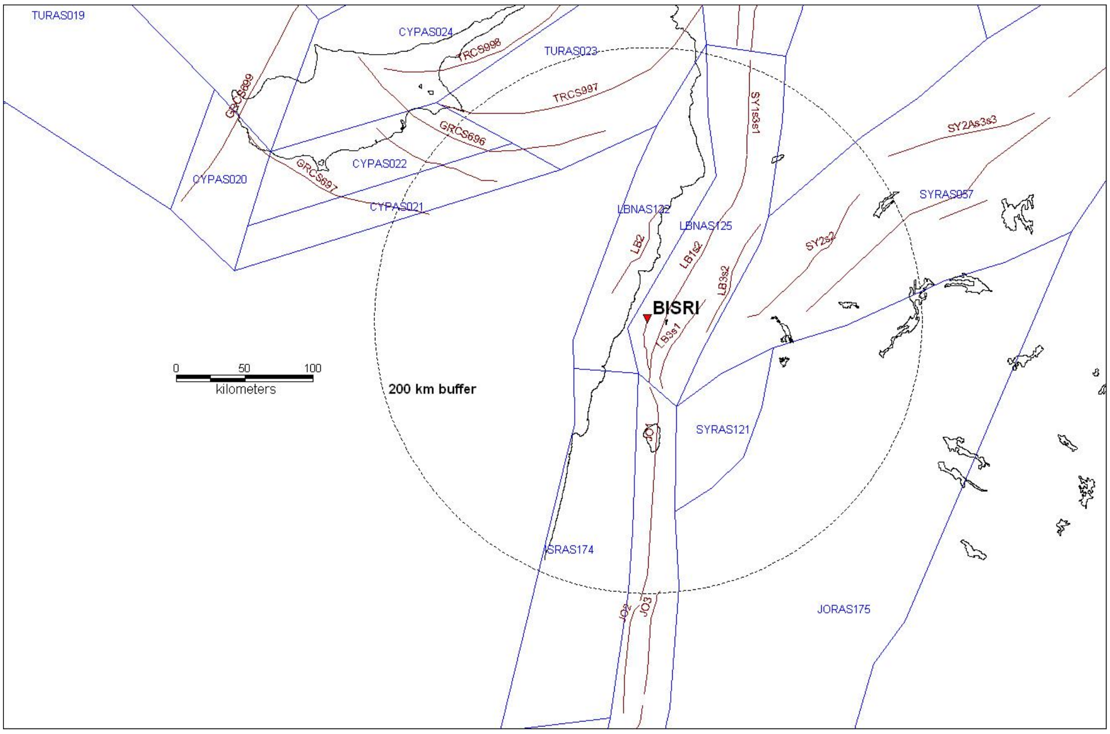

Figure 16. EMME Source zonation model. Both areal and fault source models are shown on the figure.

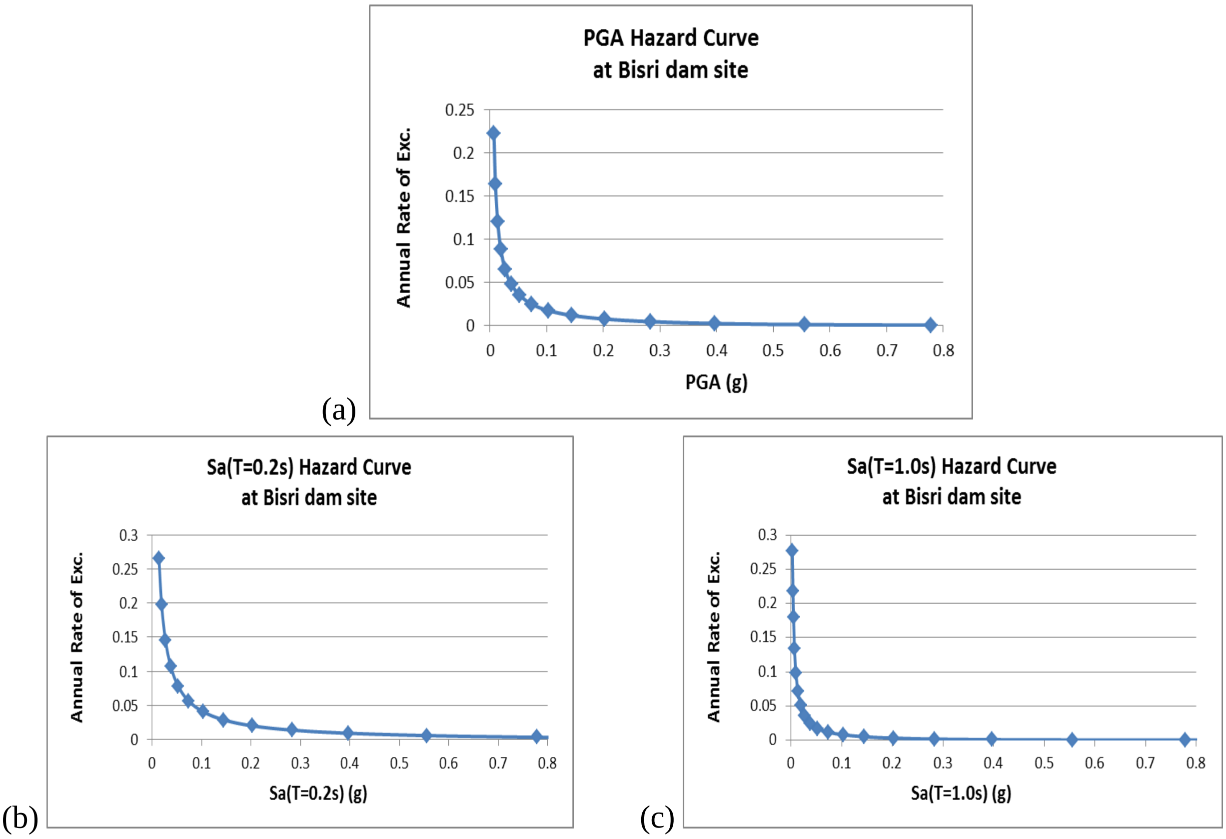

Figure 17. Hazard curves for the Bisri Dam site. (a) PGA, (b) SA (T=0.2 sec) and (c) SA (T=1.0 sec).

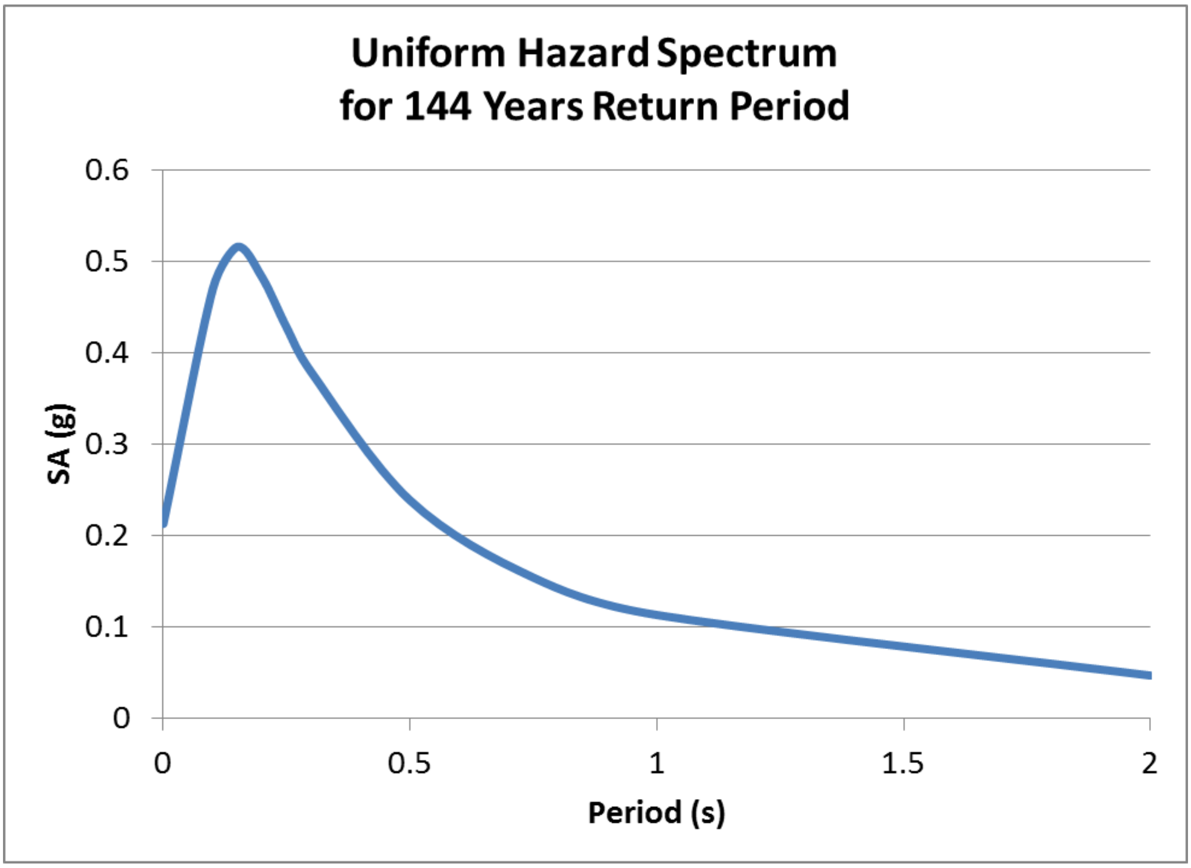

Figure 18. Uniform Hazard Spectrum corresponding to 144 years return period for the Bisri Dam site.

Differences between previous ground motion estimates and values from this PSHA likely result from a combination of the use of different ground motion prediction models, and how crustal faults are dealt with by each study. We note that a majority of the available previous studies do not account for the potential contribution of discrete fault sources to the seismic hazard, but use only historical seismicity to define the regional hazard. We also note that the available previous studies were completed prior to the recent investigations of slip rate for the Yammouneh fault (eg., Gomez et al., 2007; Daëron et al., 2004) and therefore do not incorporate the slip rates and maximum magnitude estimates that are greater than previously available.

7. VERTICAL RESPONSE SPECTRA FOR 144 YEAR RETURN PERIOD

Today the preferred approach for obtaining the response spectrum of the vertical component of motion is to scale the horizontal spectrum by vertical-to-horizontal (V/H) spectral ratios. Bommer et.al (2011) has developed a model for the prediction of V/H ratios for peak ground acceleration and spectral accelerations from 0.02 to 3.0 s is developed from the database of strong-motion accelerograms from Europe and the Middle East (**Error! Reference source not found.**). The predicted ratios are found to be in broad agreement with recent models derived from predominantly western North America data (such as, Gülerce and Abrahamson, 2011).

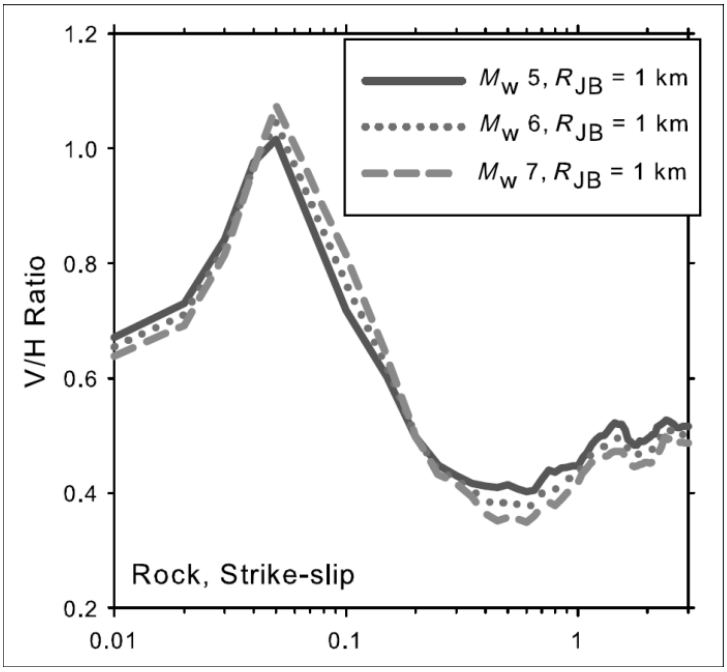

Figure 19. Median V/H spectra for different magnitude ranges and distance = 1km (after Bommer et.al, 2011).

The vertical response spectra corresponding to 144 years return period for the Bisri Dam site was developed by applying the appropriate vertical/horizontal (V/H) ratios, from the Bommer et.al (2011) relationships, to the horizontal design basis spectra is presented in Figure 20.

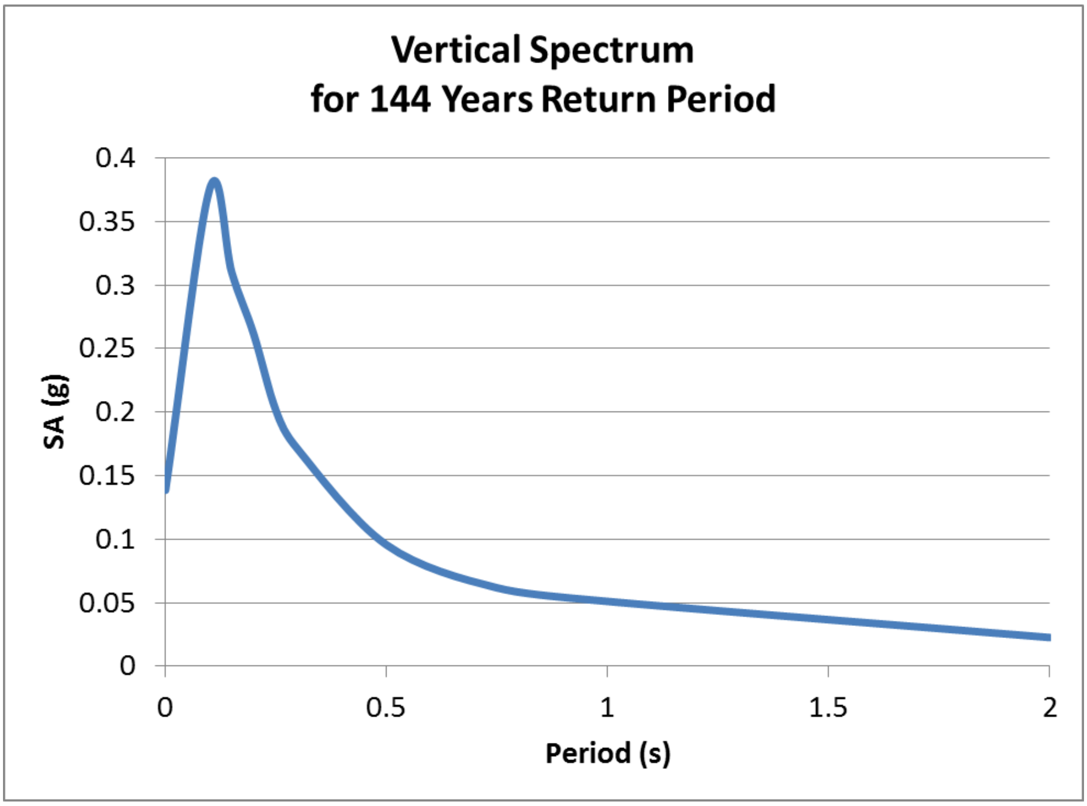

Figure 20. Vertical response spectrum corresponding to 144 years return period, obtained using coefficients after Bommer et.al, 2011).

8. DETERMINISTIC SEISMIC HAZARD

MMAX defines the upper limit of the earthquake recurrence relationship for the source as well as the magnitude levels that would be used in the deterministic earthquake hazard assessment (DSHA). The appropriate and careful selecting of the MMAX is much more important in the DSHA since together with the site-to-source distance it is the most important parameter that would control the earthquake hazard at the site. An evaluation of MMAX for all fault sources was made using empirical relationships that relate fault rupture length and rupture area to maximum magnitude (Wells and Coppersmith, 1994; Ellsworth, 1999, 2003; Hanks and Bakun 2002,2008; Shaw (2008) and; Strasser et.al (2010). Wells and Coppersmith (1994) provide the following formula for strike slip faults, with a standard deviation of 0.28 magnitude units:

$$\text{Mean Mmax}=5.16 + 1.12 \log L \qquad (1)$$

Where L (in km) is the length of the fault rupture. In practice the mean value of the Mmax is used and L is taken as a certain percentage (generally, 50% to 75%) of the total fault length. A plot of MMAX versus the fault rupture area (A, in km2) based on the results of widely considered researches are provided in Figure 21 for strike slip faults.

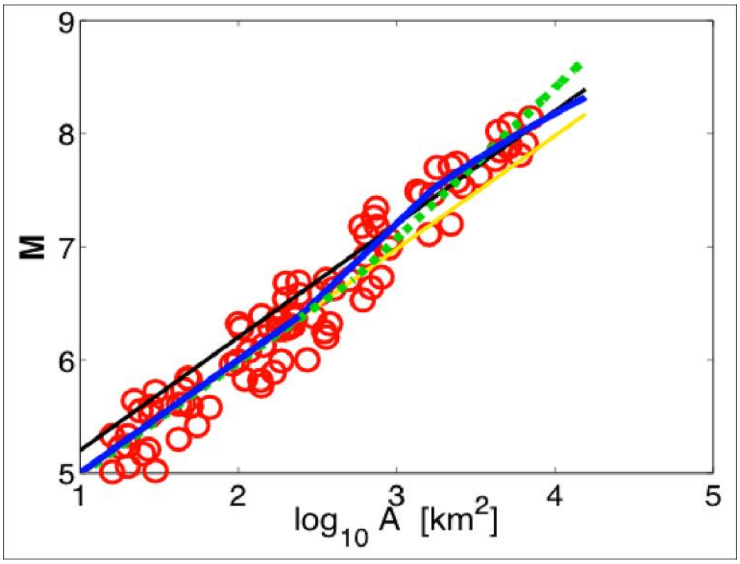

Figure 21. M (magnitude)–log A (area) Data and Relations: Wells and Coppersmith (1994). (Yellow) Ellsworth (1999, 2003) (black). Hanks and Bakun (2002,2008) (Green). Shaw (2008) (Blue). (After Shaw, 2008).

The maximum magnitude earthquake associated with the Roum fault, at a distance of only 2km from the Bisri Dam, will certainly control the earthquake hazard at the dam site. It appears that the previous assessments of the Mmax on the Roum Fault simply (and very conservatively) were obtained as 7.3 by adding 0.3 units to the historical maximum magnitude (M7, 1837 earthquake). It should be noted that the M7 is only an estimate and 0.3 is an ad-hoc value.

For the Roum Fault, even assuming the whole length of the fault will rupture at the maximum magnitude earthquake (this is a very conservative assumption), Equation 1 provides a mean Mmax of Mw6.9. Taking L=35km and the Fault Width as W=20km the Fault Rupture Area A on the Roum Fault can be can be computed as 700km^2. As the average of expressions provided in Figure 21 the mean of the maximum magnitude can be computed as Mw=7, which is essentially identical to the estimated magnitude of the 1837 earthquake. The deterministic earthquakes considered in this analysis are:

- Mw7.9 strike-slip earthquake on the Yammouneh Fault at 12km from the site.
- Mw7.8 thrust (reverse) earthquake on the MLT Ramp at 35km from the site.
- Mw7 strike-slip earthquake on the Roum Fault at 2km from the site.

As it can be seen with the exception of MMAX on the Roum Fault, which was re-assessed as Mw7, all the three scenarios are as same as that provided by Elias (2014).

Results of the deterministic analysis results for these three scenarios are presented in Figure 22 for the 84-percentile (i.e. median+1 standard deviation) response spectra on NEHRP-B type soil (Vs30=760m/s) using the GMPE model of Akkar and Bommer (2010). As it can be clearly assessed the Roum Fault scenario governs the earthquake hazard at the Bisri Dam site.

The deterministic median+1SD response spectra obtained for the Roum Fault scenario using selected AB2010 (Akkar and Bommer, 2010), CY2008 (Chiou and Youngs, 2008) and BA2008 (Boore and Atkinson, 2008) GMPE's for NEHRP B type site class are provided in Figure 23. The average median+1SD response spectra, obtained using logic tree combination of these results with relative weights of 0.50, 0.25 and 0.25 respectively for the GMPEs of AB2010, CY2008 and BA2008 is provided in Figure 24.

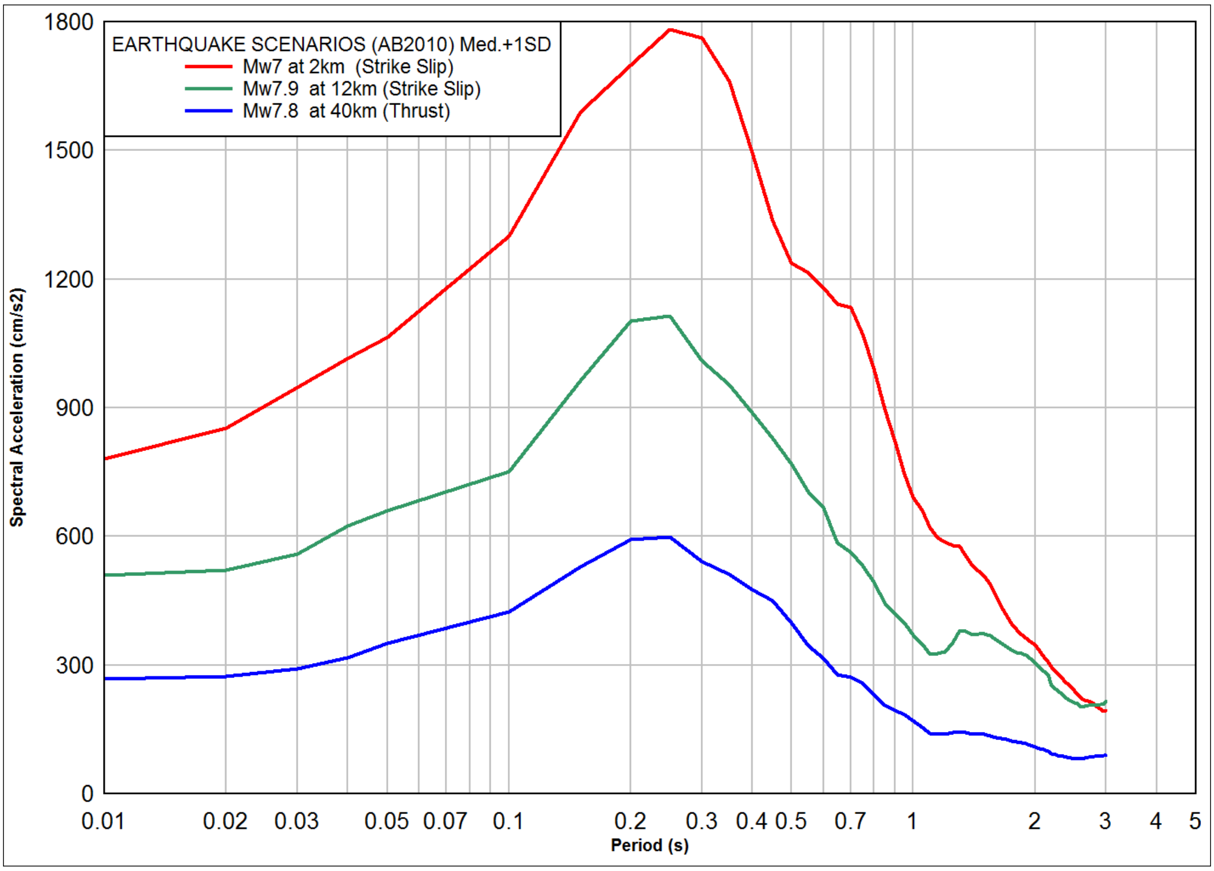

Figure 22. Deterministic Median+1SD (84-percantile) response spectra using AB2010 GMPE for NEHRP B type site class. Red: Mw7 strike-slip earthquake on the Roum Fault at a distance of 2km from the site; Green Mw7.9 strike-slip earthquake on the Yammouneh Fault at a distance of 12km from the site; Blue Mw7.8 thrust (reverse) earthquake on the MLT Ramp at a distance of 40km from the site.

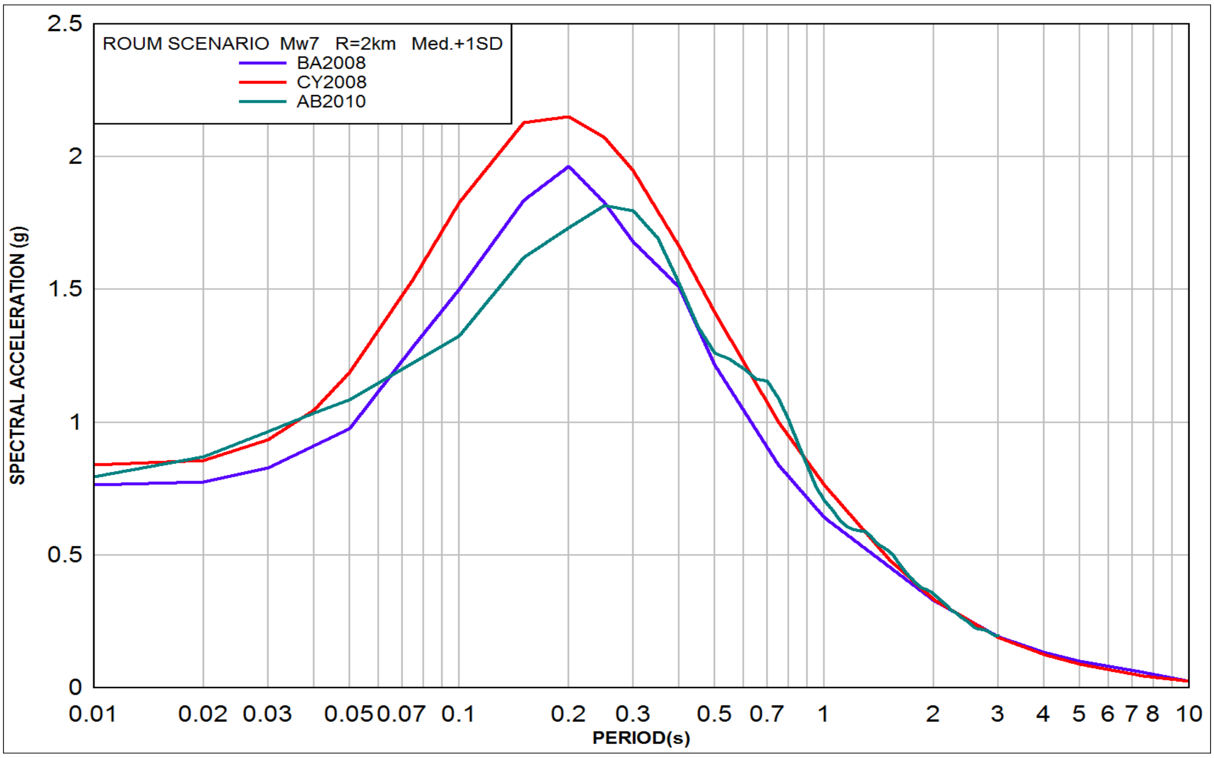

Figure 23. Median+1SD response spectra for Mw7 strike-slip earthquake on the Roum Fault at a distance of 2km from the site (NEHRP B site class) associated with BA2010, CY2008 and BA 2008 GMPE.

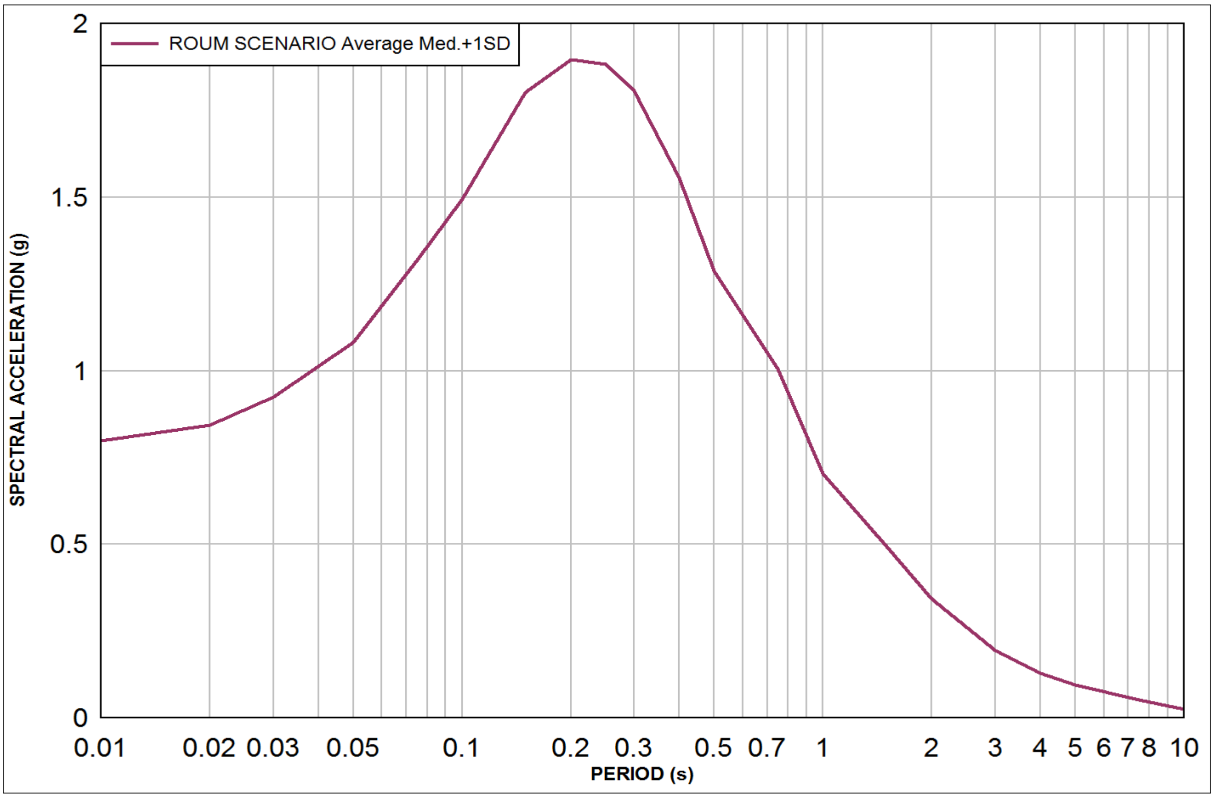

Figure 24. The average median+1SD response spectra for the Mw7 strike-slip earthquake on the Roum Fault at a distance of 2km (NEHRP B site class).

9. DIRECTIONALITY EFFECTS

Common methods to quantify spectral acceleration two-component horizontal shaking are to take the geometric mean of the spectral accelerations of the two as-recorded ground motion components (SaGM), to take the maximum spectral acceleration observed when looking over all horizontal orientations (SaRotD100) , or to take the median spectral acceleration observed when looking over all horizontal orientations (SaGMRotI50).

Most of the recent GMPEs use either SaGM or SaGMRotI50 , which are approximately equal to each other. The Shahi and Baker (2012) study have shown that SaGMRotI50 predictions should be multiplied by approximately 1.2 at short periods (T<0.1s) and approximately 1.3 at longer periods (T>1s) to yield SaRotD100.

Among the current earthquake resistant design codes, including the ICOLD design criteria, the specifics of whether to use SaGMRotI50 or for the design basis response spectra is not provided with the exception of ASCE 7-10 (2010), where SaRotD100 is specified.

Noting that the earthquake resistant design process for dam bodies already incorporates adequate levels of conservatism, the use of either SaGM or SaGMRotI50 will be considered for the assessment of the design basis response spectra of the Bisri Dam.

10. NEAR FAULT EFFECTS

The GMPEs used in the analysis do not explicitly differentiate near-fault effects; thus, it is necessary to modify the results to account for near-field fault rupture effects. These effects (termed Directivity Effects and Polarization) are important at periods of vibration longer than 0.5s are associated with large magnitude (≥ MW 6.5) earthquakes occurring on nearby faults, which essentially represent the case with the Roum Fault scenario.

Ground Motion Directivity

During an earthquake event, fault rupture propagation and earthquake source radiation pattern cause spatial variation of horizontal ground motions in amplitude and duration around the fault. Forward directivity occurs when rupture propagates toward the site, and the slip direction of the fault is aligned with the site (Somerville et al. 1997). In this situation, seismic energy radiated from the fault arrives at the site in a short time interval, resulting in a large pulse of motion at the beginning of the record. The velocity pulse is oriented in the fault-strike-normal direction due to polarization. Directivity effects increase spectral accelerations at locations where the rupture has propagated towards the site of interest "forward-directivity", for periods longer than approximately 0.5 seconds (Baker et. al, 2012). As the current GMPEs do not explicitly include directivity effects several directivity models have been developed to account for the near source ground motion characteristics (Somerville et al., 1997; Abrahamson, 2000; Rowshandel, 2006; Spudich et al., 2004; Spudich and Chiou, 2008; Rowshandel, 2010; Shahi and Baker, 2011).

Somerville et al. (1997) provide a simple empirical model (which was later modified by Abrahamson, 2000) to predict ground motion amplification, duration and polarization due to directivity effects. Spudich and Chiou (2008) developed a physically-based directivity model using isochrone theory, with an improved characterization of directivity that can be used to include the directivity modification to the NGA GMPEs. The average horizontal spectral acceleration from an empirical attenuation relation without directivity effect (Sa) can be modified to obtain the spectral acceleration with directivity effects (Sadir) by the following equation:

$$\ln Sa_{dir} = \ln Sa + f_D, \quad \text{or} \quad Sa_{dir}/Sa = \exp(f_D) \qquad (2)$$

where the factor *fD* is used to quantify the directivity effects. The directivity modification factors that predict directivity-induced variations of spectral acceleration provided by Spudich and Chiou (2008) for strike-slip faults are similar to those of Abrahamson (2000) and are roughly half of the factors predicted by Somerville et al. (1997).

Assuming full forward directivity (the Mw7 earthquake starts at the far end of the Roum Fault and ruptures the whole 35km length of the fault towards the Bisri Dam, located at 2km away from the end of fault) the *Sadir* /Sa values (modification factors) are computed using the Spudich and Chiou (2008) model as shown in Table 2. The factors are taken as the average values associated with AB2006 and CY2006 GMPEs.

Table 2. Directivity modification factors for the SEE level design basis response spectrum

Periyod (s)	0.5	0.75	1.0	1.5	2.0	3.0	4.0	5.0	7.5	10.0
Sa_{dir}/Sa	1.00	1.07	1.13	1.20	1.27	1.38	1.49	1.57	1.73	2.12

The response spectra modified for the directivity effects is provided in Figure 25.

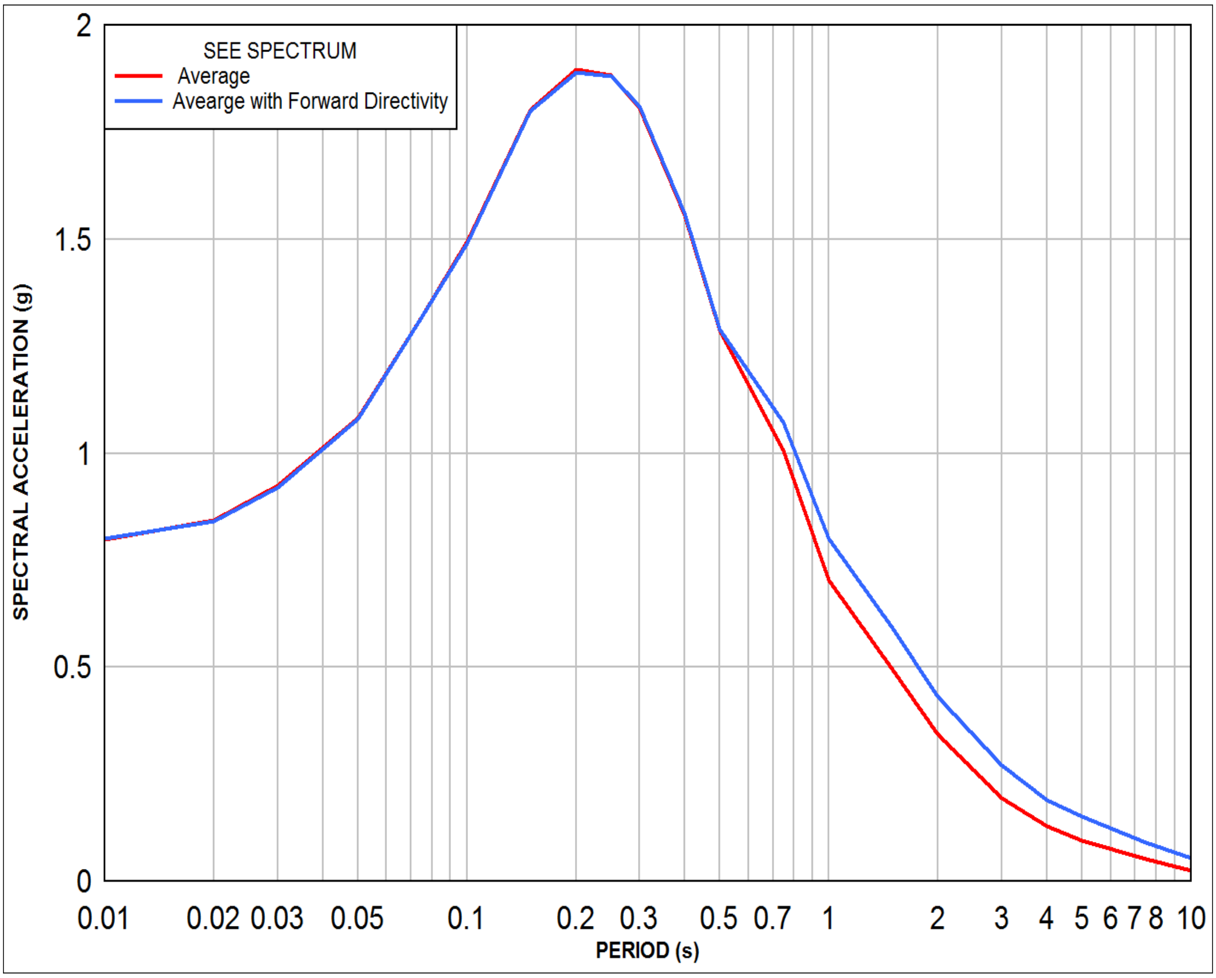

Figure 25. The average (Red, actually Figure 24) and the directivity modified design basis response spectra (Blue). Note that the directivity modification applies only first periods greater than 0.5s.

There also exist systematic differences (called "polarization") of horizontal motions in the directions perpendicular (normal) to the fault strike (termed fault-strike-normal or fault-normal (FN) direction) and in the direction parallel to the fault strike (termed fault-strike-parallel or fault-parallel (FP) direction). The polarization of ground motion in the fault-strike-normal (FN) direction and the fault-strike-parallel (FP) direction caused by directivity effects depends primarily on earthquake magnitude and rupture distance. The ratio of fault-strike-normal to average horizontal motions is period-dependent, and may become significant at a period greater than 0.6 second, indicating a transition from incoherent source radiation and wave propagation conditions at short periods to coherent source radiation and wave propagation conditions at long periods (Somerville et al., 1997). For near-fault structures with very strong direction dependence in response the separation of FN and FP components is foreseen in ASCE 7-10 (2010). The effect of polarization of the ground motion is not considered in other current earthquake resistant design codes and in the ICOLD earthquake resistant design criteria.

11. DESIGN BASIS RESPONSE SPECTRA

On the basis of these evaluations and considerations the following design basis ground motion spectra can be provided:

The Operating Basis Earthquake is determined as the probabilistically assessed earthquake ground motion for an average return period of 144 years. The horizontal and vertical design basis spectra for the operating basis earthquake (OBE) for the random horizontal and vertical component are provided respectively in Figures 18 and 20. The response spectra is provided for 5% damping and for the free-field engineering bedrock outcrop.

The Safety Evaluation Earthquake is the maximum level of ground motion for which the dam should be designed or analyzed. For the earthquake resistant design of the Bisri Dam the SEE level ground motion will be determined to correspond to the 84-percentile deterministic MCE (i.e. median plus one standard deviation). The horizontal and vertical design basis spectra for the safety evaluation earthquake (SEE) for the random horizontal and vertical component are provided respectively in Figures 25 and 30. The response spectra is provided for 5% damping and for the free-field engineering bedrock outcrop.

12. SPECTRUM COMPATIBLE HORIZONTAL GROUND MOTION

When acceleration time histories (also referred to as accelerograms) of ground motions are required for the dynamic analysis of a structure, they should be developed to be consistent with the design response spectrum, as well as have appropriate strong motion duration for the particular design earthquake. In addition, whenever possible, the acceleration time histories should be representative of the design or safety evaluation earthquake in all the following aspects: earthquake magnitude, distance from source-to-site, fault rupture mechanisms (fault type, focal depth), transmission path properties, and regional and geological conditions. Since it is not always possible to find empirical records that satisfy all of the above criteria, it is often necessary to modify existing records or develop synthetic records that meet most of these requirements (NIST, 2011). Therefore, ground motion time histories are selected and scaled to enable response-history analysis that supports either design or performance assessment. The goals of analysis must be clearly understood must have a clear understanding of before choosing procedures to select and scale ground motions.

Since traditionally the seismic hazard at a site for design purposes has been represented by design spectra, virtually all modern seismic design codes and guidelines require scaling of selected ground motion time histories so that they match or exceed the controlling design spectrum within a period range of interest. As performance-based considerations become pre-requisite in the seismic design and evaluation of building structures, the use of nonlinear response history analysis has gained importance. For this method, suites of appropriately selected and scaled (modified) ground motion records compatible with the target spectra are needed.

There are two general approaches to developing acceleration time histories: selecting a suite of recorded motions and synthetically developing or modifying one or more motions. The disadvantage is that the synthetic ground motions are not “real” motions. Real motions generally do not exhibit smooth spectra. Although a good fit to a design spectrum can be attained with a single accelerogram, it may be desirable to fit the spectrum using more than one accelerogram. Such motions have the character of recorded motions since the modeling procedures are intended to simulate the earthquake rupture and wave propagation process. For selecting recorded motions, it is necessary to select a suite of time histories (typically 3 or more) such that, in aggregate, valleys of individual spectra that fall below the design response spectrum are compensated by peaks of other spectra and the exceedance of the design response spectrum by individual spectral peaks is not excessive (preferably at least within the bandwidth of interest for structure specific analysis). For nonlinear analyses, it is desirable to have additional time histories because of the importance of (NIST, 2011).

The recently developed NGA GMPE relationships output an average horizontal spectral demand and the dispersion in that demand, where this average is the rotated geomean denoted as GMRotI50 (Boore et al., 2006). GM denotes the geometric mean of two horizontal components, Rot denotes that rotations over all non-redundant angles are considered, I denotes that period-independent rotations are used, and 50 identifies the prediction of median values. The geometric mean of two horizontal components of ground motions is calculated as the square root of the product of the two horizontal response spectral accelerations at each period of interest. The current USGS probabilistic ground motion maps (http://earthquake.usgs.gov/hazards/designmaps/) are based on the GMRotI50=geometric mean representation of horizontal ground motion

On the basis of these developments, ASCE 7-10 (2010) has changed how site specific ground motions will be developed. The relevant stipulations include:

- Where three-dimensional analyses are performed, ground motions shall consist of pairs of appropriate horizontal ground motion acceleration components that shall be selected and scaled from individual recorded events. Appropriate ground motions shall be selected from events having magnitudes, fault distance, and source mechanisms that are consistent with those that control the maximum considered earthquake.

- For each pair of horizontal ground motion components, a square root of the sum of the squares (SRSS) spectrum shall be constructed by taking the SRSS of the 5 percent-damped response spectra for the scaled components (where an identical scale factor is applied to both components of a pair).

- Each pair of motions shall be scaled such that in the period range from 0.2T to 1.5T, the average of the SRSS spectra from all horizontal component pairs does not fall below the corresponding ordinate of the response spectrum used in the design

- If seven or more pairs of ground motions are used for the response-history analysis, the average value of the response parameter of interest is permitted to be used for design.

Methods for the scaling of ground motion records vary from scaling the ground motion to a single period on the target spectrum, to average matching over a period range to operations involving frequency modification (Abrahamson, 1998) of the ground motion records to match the target spectrum. Since the frequency characteristics and the correlation between the components of the records remain unchanged the use of scaling procedures has gained increasing acceptance especially for long period structures exposed to near field and fault directivity effects.

Work by the PEER Ground Motion Selection and Modification Working Group (GMSM Working Group, 2009) has comprehensively studied the importance of the ground motion selection and spectrum compatible ground motion generation methodologies. The software tool developed by GMSM has opted for the scaling of the selected ground motion to fit to the target spectrum (Baker, 2011). This allows selecting recordings for which the geometric mean of the two horizontal components provides a good match to the target spectrum. The basic criterion is that the spectrum of the time series provides a “good match” to the user’s target spectrum over the spectral period range of interest. The quantitative measure used to evaluate how well a time series conforms to the target spectrum is the mean squared error (MSE) of the difference between the spectral accelerations of the record and the target spectrum. This approach produces scaled recordings that provide the best match to the spectral shape of the target spectrum over the user-specified period range of interest, but whose spectra will oscillate about the target. Minimization of the MSE is achieved by a scale factor given by the mean weighted residual in natural logarithm space between the target and the record spectra. Since MSE is computed over both components with the same value of/applied to both components, the relative amplitude of the two horizontal components as well as the vertical component is maintained.

Earthquake records in this database are sorted out on the basis of their geo-tectonic characteristics and combined with other worldwide data with similar geo-tectonic characteristics. Any earthquake (regardless of the political boundary in which it is located) is eligible, as long as the earthquake is related to the so called “shallow active crustal regions” regions of the world, and conform with the fault mechanisms, magnitude range and source distance range particular to the project. Within the database, ground motion records have been identified as having strong velocity pulses that may be associated with fault rupture directivity effects.

The seven sets of bi-axial earthquake ground motion records selected for scaling to the SEE level target spectrum are provided in Table 3. The selected records are listed according to the best fit in Table 3. The geometric mean of these two horizontal spectra of these 7 sets of scaled ground motion is plotted with the target spectrum in Figure 26 for SEE level target spectrum.

Table 3. Records selected for scaling to the SEE level target spectrum

Event	Year	Station	Magnitude	Mechanism	Rjb (km)	Rrup (km)	Vs30 (m/s)	Scale factor	Mean Square Error
Kocaeli, Turkey	1999	Gebze	7.51	strike slip	8	11	792	4.255	0.032
Chi-Chi, Taiwan-04	1999	CHY074	6.20	strike slip	6	6	553	1.978	0.044
Kocaeli, Turkey	1999	Izmit	7.51	strike slip	4	7	811	3.552	0.058
Morgan Hill	1984	Coyote Lake Dam -	6.19	strike slip	0	1	561	1.142	0.103
Morgan Hill	1984	Gilroy Array #6	6.19	strike slip	10	10	663	3.162	0.134
Duzce, Turkey	1999	IRIGM 487	7.14	strike slip	3	3	690	2.870	0.151
Darfield, New Zealand	2010	LPCC	7.00	strike slip	25	26	650	3.731	0.163

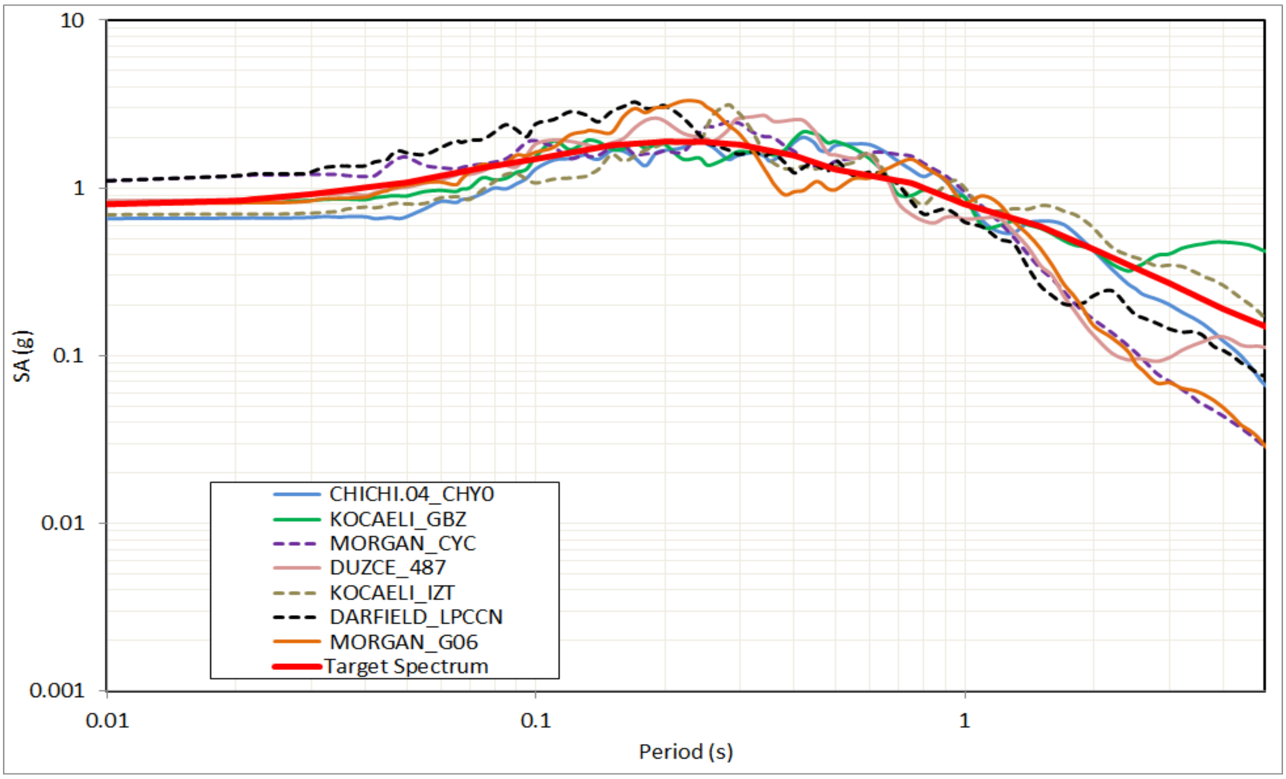

Figure 26. Geometric mean of horizontal spectra of the 7 sets (Table 3) of scaled ground motion and the SEE level target spectrum (Dashed lines correspond to first three best fit of scaled records at 0.5 s).

The geometric mean of two horizontal spectra of first three scaled ground motion records, that best fits to 0.5s at target spectrum, are plotted with this SEE level target spectrum in Figure 27.

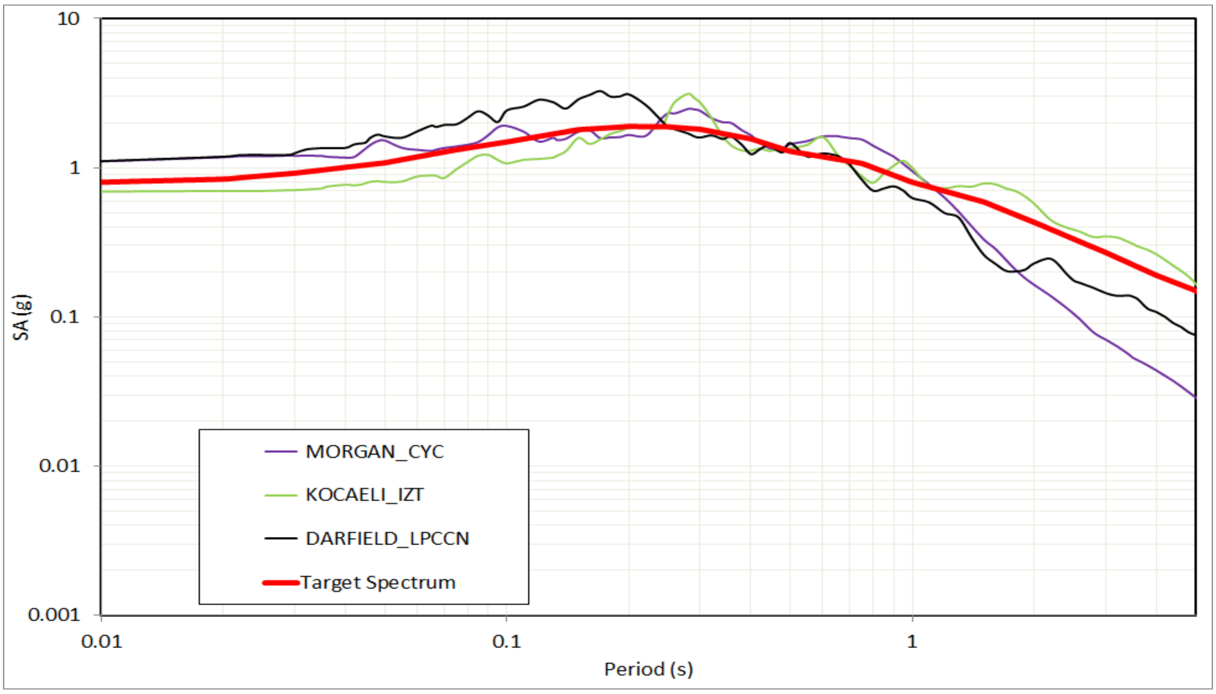

Figure 27. Geometric mean of horizontal spectra of first three best fit of scaled ground motion records at 0.5 s and the SEE level target spectrum.

The comparison of the geometric and arithmetic mean of the scaled spectra plotted on Figure 28 with the SEE level target spectrum is provided.

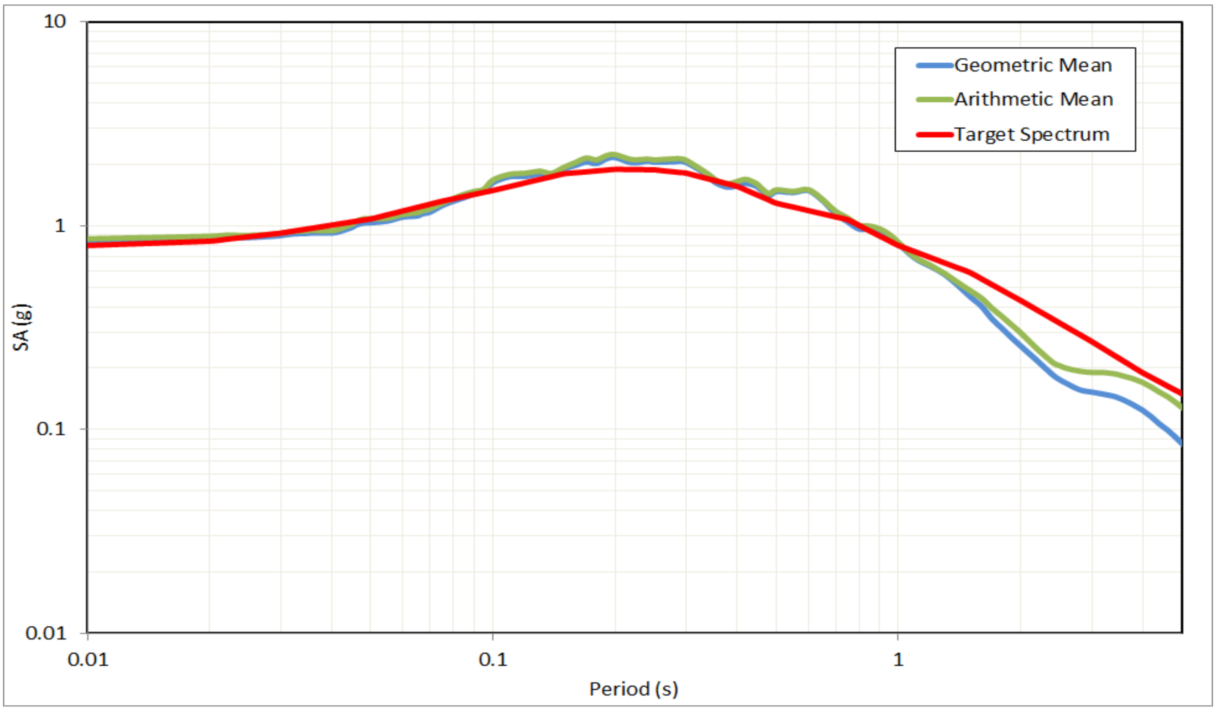

Figure 28. Comparison of the SEE level target spectrum with the geometric and arithmetic mean of the seven scaled horizontal spectra.

Seven vertical ground motion records of the selected earthquakes are scaled by the factors listed in Table 3. Average vertical spectrum of these seven vertical ground motion is shown in Figure 29.

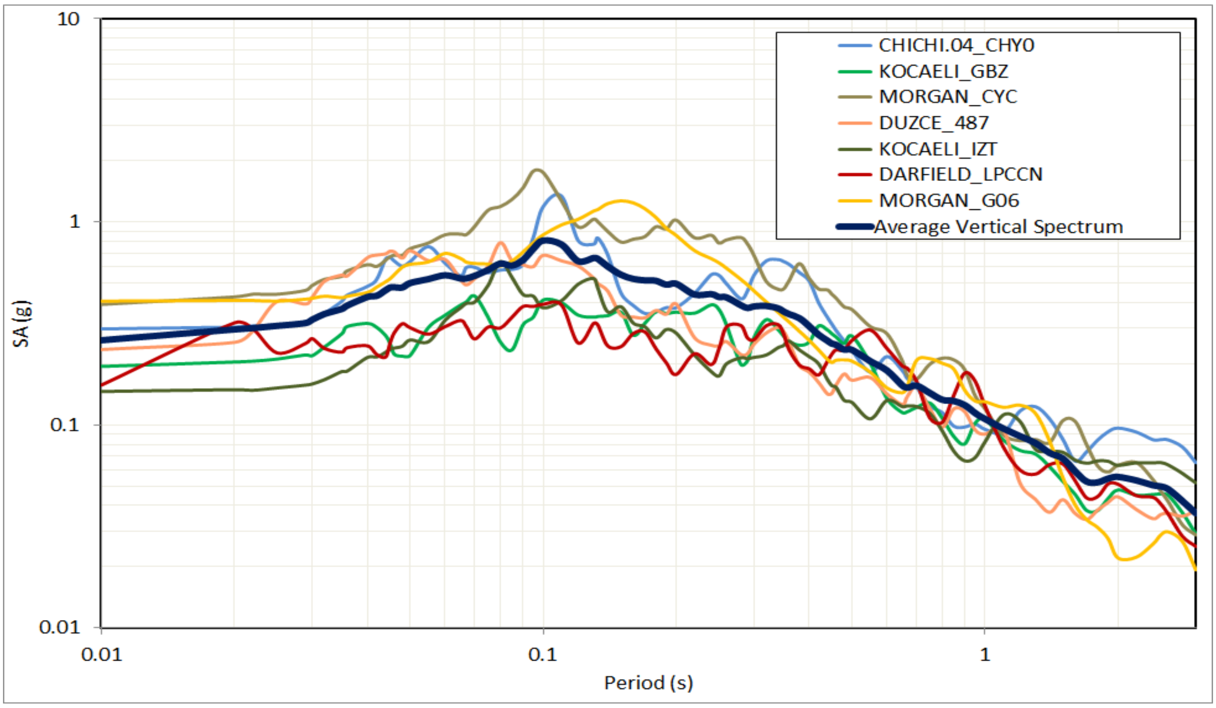

Figure 29. Average vertical spectrum of the 7 scaled vertical ground motion records.

The SEE level vertical response spectra for the Bisri Dam site can also be developed by applying the appropriate vertical/horizontal (V/H) ratios, from the Bommer et.al (2011) relationships, to the SEE level horizontal design basis spectrum, as illustrated in Figure 30. However, this spectrum will not necessarily be in conformity with the vertical components of the 7 sets of horizontal spectrum compatible scaled ground motion. Since the time domain analysis will be used for the SEE level design the vertical spectrum should preferably be taken as the average spectrum indicated in Figure 29.

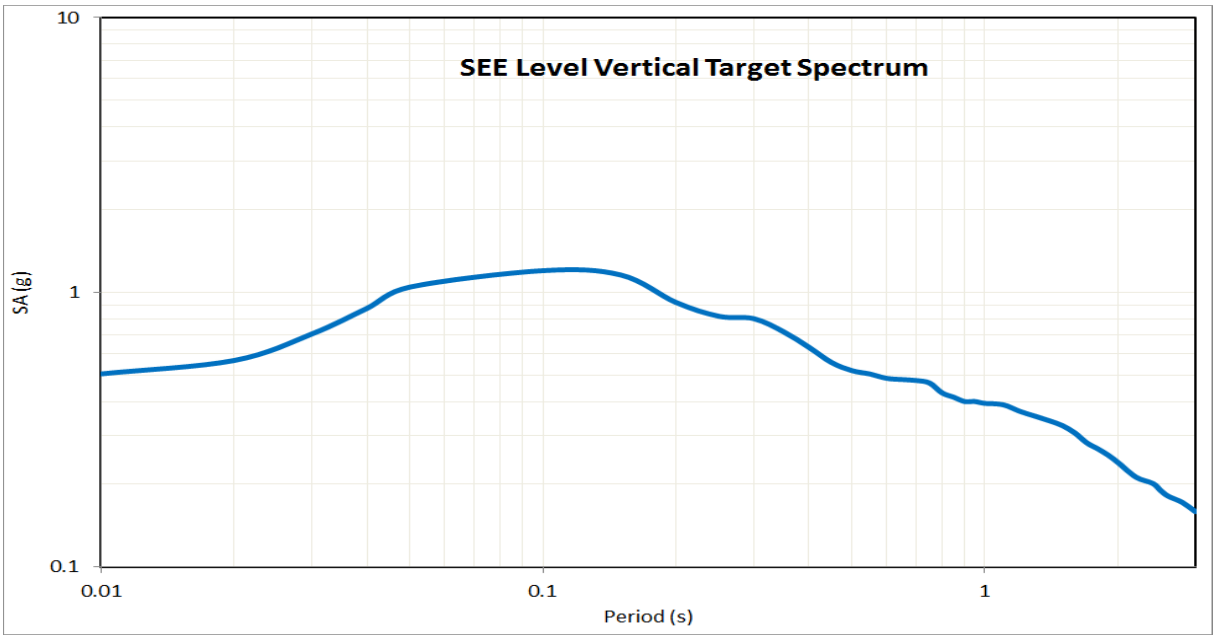

Figure 30. SEE level vertical response spectrum, obtained using coefficients after Bommer et.al, 2011).

13. CONCLUSIONS

A thorough analysis of the earthquake hazard at the Bisri Dam site is conducted to assess the following earthquake resistant design basis spectra:

The Operating Basis Earthquake is determined as the probabilistically assessed earthquake ground motion for an average return period of 144 years. The horizontal and vertical design basis spectra for the operating basis earthquake (OBE) for the random horizontal and vertical component are provided respectively in Figures 18 and 20. The response spectra is provided for 5% damping and for the free-field engineering bedrock outcrop. Under the action of this level of ground motion, the dam, appurtenant structures and equipment should remain functional and, if any, the minor damage should be easily repairable.

The Safety Evaluation Earthquake is determined to correspond to the 84-percentile deterministic MCE. The horizontal and vertical design basis spectra for the safety evaluation earthquake (SEE) for the random horizontal is are provided in Figure 25. Since the time domain analysis will be used for the SEE level design 7 sets of spectrum compatible scaled ground motion acceleration data are provided to enable the time-domain analysis. The vertical spectrum should be taken as the average of the vertical spectra of these sets of ground motion indicated in Figure 29. The 5% damped response spectra and the 7 sets of spectrum compatible scaled ground motion is provided is applicable at for the free-field engineering bedrock outcrop. Under the SEE the stability of the dam and life safety must be ensured with no uncontrolled release of water from the reservoir. SEE is the maximum level of ground motion for which the dam should be designed.

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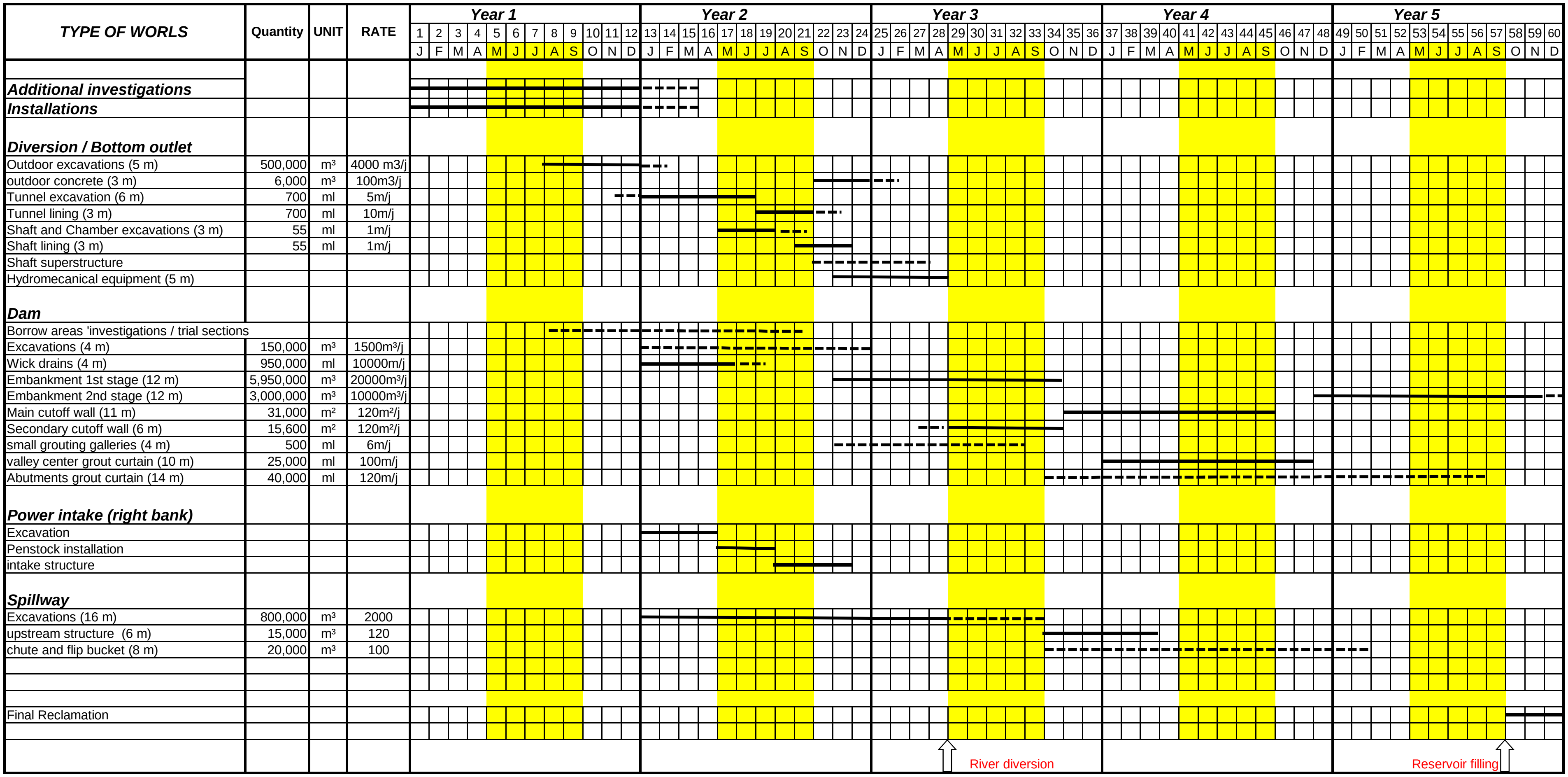
Appendix C Works anticipated planning

Appendix C - Works anticipated planning



TYPE OF WORLS	Quantity	UNIT	RATE
Additional investigations			
Installations			
Diversion / Bottom outlet			
Outdoor excavations (5 m)	500,000	m³	4000 m3/j
outdoor concrete (3 m)	6,000	m³	100m3/j
Tunnel excavation (6 m)	700	ml	5m/j
Tunnel lining (3 m)	700	ml	10m/j
Shaft and Chamber excavations (3 m)	55	ml	1m/j
Shaft lining (3 m)	55	ml	1m/j
Shaft superstructure			
Hydromecanical equipment (5 m)			
Dam			
Borrow areas 'investigations / trial sections			
Excavations (4 m)	150,000	m³	1500m³/j
Wick drains (4 m)	950,000	ml	10000m/j
Embankment 1st stage (12 m)	5,950,000	m³	20000m³/j
Embankment 2nd stage (12 m)	3,000,000	m³	10000m³/j
Main cutoff wall (11 m)	31,000	m²	120m²/j
Secondary cutoff wall (6 m)	15,600	m²	120m²/j
small grouting galleries (4 m)	500	ml	6m/j
valley center grout curtain (10 m)	25,000	ml	100m/j
Abutments grout curtain (14 m)	40,000	ml	120m/j
Power intake (right bank)			
Excavation			
Penstock installation			
intake structure			
Spillway			
Excavations (16 m)	800,000	m³	2000
upstream structure (6 m)	15,000	m³	120
chute and flip bucket (8 m)	20,000	m³	100
Final Reclamation			

This planning is based on 1st January date for notice to proceed
25 working days a month
two 10 hours shifts per day
Critical path : installations, diversion and bottom outlet tunnel, 1st stage embankment, main cutoff wall and grouting in the valley center, 2nd stage embankment.