Chapter 6

The Engineering Geology of the Asmari Formation and Implications on the Five Dam Sites

6.1. Introduction

The geomorphology is basically controlled by lithology, tectonic history, and meteorological conditions. Therefore, the relatively high strength and rigidity of the Asmari limestones on the one hand and the active tectonism of the region on the other hand explain the steep gradients and occurrence in high peaks of the Asmari Formation limestones in the Zagros region.

Carbonate rocks show elastic behaviour under normal stress. These rocks are stretched and created anticline and syncline structures during the folding process. Likewise, the continuity of these processes will result in reverse faulting, thrust faulting and imbricated structures as well as subduction blocks over geologic time. Based on geological investigations in south-western Iran, it is clear that the Zagros basin has undergone intensive folding, faulting and subduction during its geological history (Nogole Sadat, 1985). It is believed that the stretching process has resulted in break up of the outer layers in the southern flanks during the extension stress/strain. Therefore, the highly curved southern flanks contain more *fractures* than the low curved northern flanks and as such different engineering geological characteristics can be expected on the two opposing flanks of the anticlines (Figure 6.1 and 6.2).

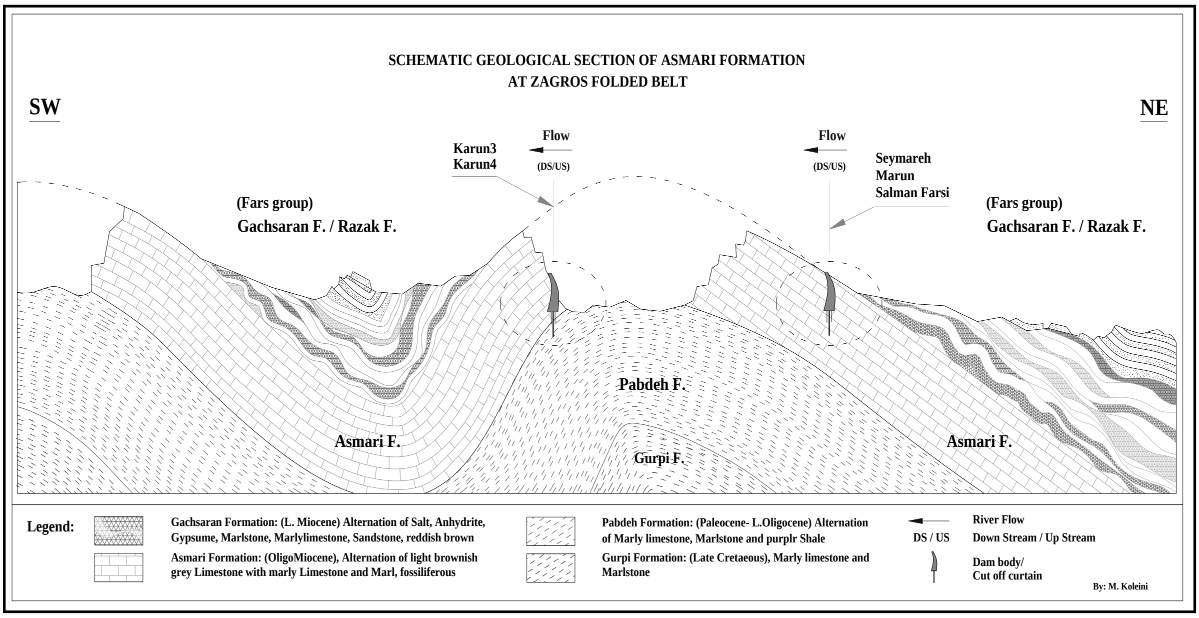


Figure 6.1. Schematic geological cross section of Asmari Formation limestone at the Zagros folded belt. The situation at the dam sites on each flank can be observed as well.

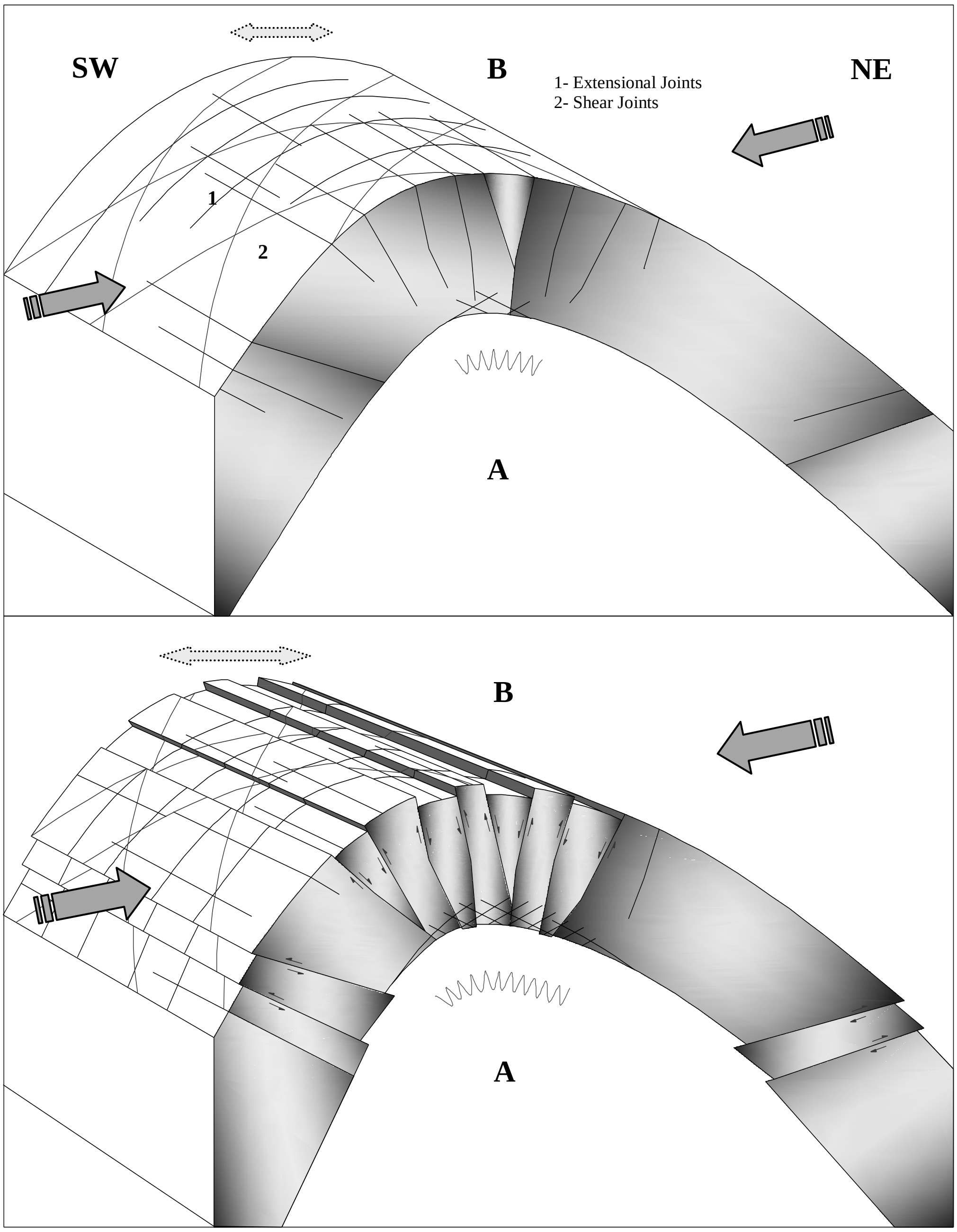


Figure 6.2. A simple block diagram of Asmari formation limestone at the Zagros folded belt. Southern flank clearly indicate much more gradient of strata between 70°- 90°, that the northern flank which has dipping between 20° to 50° toward the northeast. Therefore, due to less curvature of strata, fewer tectonic features can be expected.
A- Reverse and thrust faults due to compression in inner core and in two flanks of anticline.
B- Normal faulting due to extension in outer core of anticline.
1and 2 are extensional and shear joints respectively.

6.2. Permeability and Watertightness

Table 6.1 shows the average rock mass permeability of the Asmari Formation units, measured in exploratory boreholes with lugeon tests. As can be seen the variation of this parameter is caused by the existence of discontinuities with different lithological characteristics. The permeability values indicate large variation in hydraulic conductivity with depth and location. In addition to the hydrogeological anisotropy, this is one of the important characteristics of the dam sites. Generally, in the Asmari Formation, to a depth of 100 m below surface, the hydraulic conductivity varies from very low to very high with the majority of the results in the high range. Below this depth, the measurements indicate very low to medium hydraulic conductivities except at Karun-4 and Salman Farsi which indicate high permeability due to low values of RQD.

Table 6.1. The permeability conditions of various unit of the Asmari Formation at the dam localities.

Asmari unit	Karun-3	Karun-4	Seymareh	Marun	Salman Farsi
U.Asmari	M to VH (20- 100) lugeon	Non to V.H (0- 100) lugeon	H to V.H (30- 74) lugeon	M to H (10- 30) lugeon	Non to L (1- 10) lugeon
M.Asmari	M to VH (10- 100) lugeon	M to V.H (20- 100) lugeon	L to H (4- 45) lugeon	L to M (2- 10) lugeon	L to V.H (10- 100) lugeon
L.Asmari		H to V.H (30- 100) lugeon	Non to M (1- 20) lugeon	L to M (3- 25) lugeon	L to V.H (3- locally 100) lugeon

There is possibly a direct relationship between high coefficients of permeability values and fractured zones. Some zones of high conductivity are probably caused by solution enlargement of the fractures.

Generally, the upper Asmari Formation has relatively higher values of permeability than the middle and lower parts, except at Salman Farsi where impermeable marlstones and marly limestones are widespread. In general, impermeable layers such as marlstone, shale and marly limestone play the main role to reduce the permeability values in each unit. The upper Asmari unit (medium to thinly bedded) show higher influence of shear and extensional joints due to tectonic compressional movements than the other units and therefore lower RQD values can be expected (Figure 6.3).

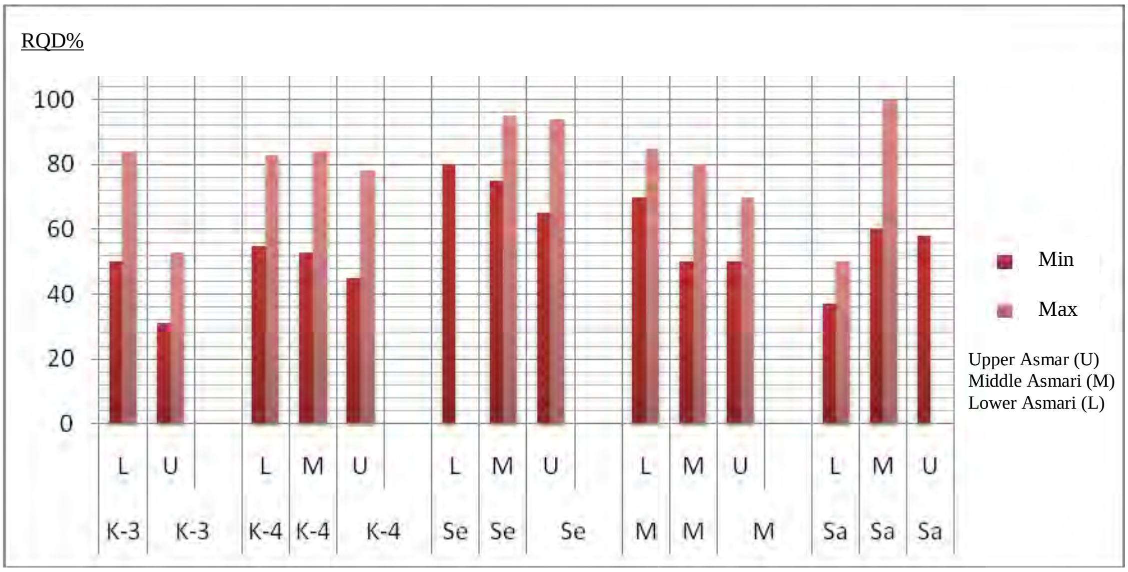


Figure 6.3. Histogram of RQD values for the Asmari formation limestone, calculated for Karun-3 (K-3), Karun-4 (K-4), Seymareh (Se), Marun (M), and Salman Farsi (Sa) dam sites.

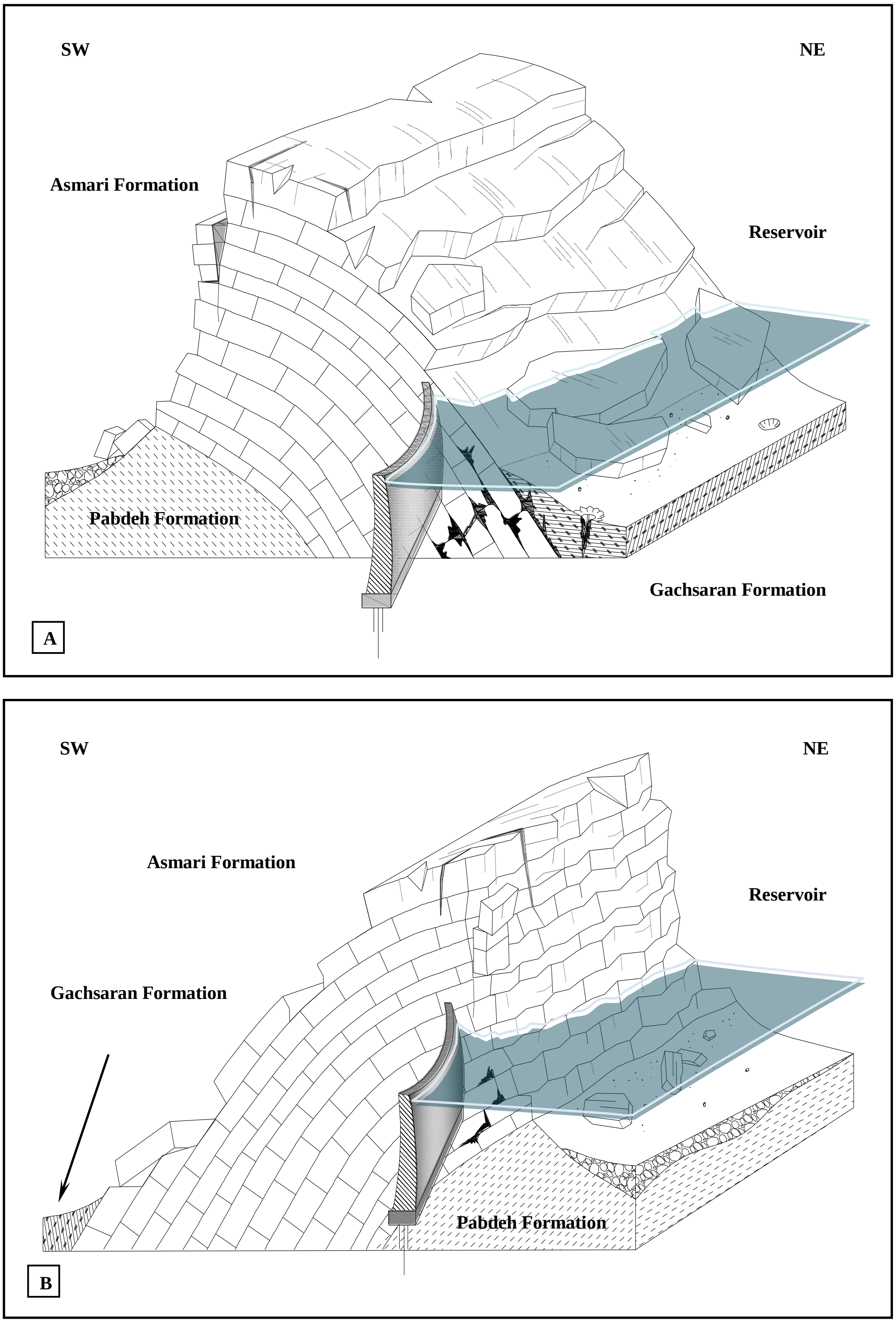


Figure 6.4. The different geological condition of dam localities, (A) Seymareh, Marun, Salman Farsi in the northern flank and (B) Karun-3, Karun-4 in the southern flank of the anticlines. The situation of the dam body/cut-off curtain and reservoir on one hand and distribution of the Pabdeh, Asmari and Gachsaran formations with various permeabilities on the other hand is of considerable matter in the point of view of permeability and watertightness.

The permeability conditions of the Asmari Formation limestone clearly indicate a direct relationship with RQD. It should be considered that the kind and thickness of fracture fillings is also a very important factor leading to reduction in permeability values.

For example, joints with clay mineral infill are almost impermeable but are considered to be discontinuities when calculating RQD.

The Asmari limestone ridges on either side of the dams are dissected by the subparallel system of vertical and subvertical discontinuities (shear / extensional joints and faults), running across the axis of the anticlines. Karstic features commonly developed along these discontinuities and along steeply dipping planes can possibly provide a direct hydraulic connection between the reservoir and the gorge downstream of the dams, i.e. there is the possibility of substantial seepage from the reservoir bypassing the dam. Thus, the *grout curtains* have to be placed in such a way to intersect these discontinuities.

The geological formations which form the reservoir rock bed from old to young generally comprise the Khami Group, Ilam, Gurpi, Pabdeh, Asmari, Gachsaran, Aghajari and Bakhtiary Formations respectively. In addition to the mentioned formations, the surficial materials such as residual soil, slope wash, rock fall and river alluvium are also present in the reservoir areas.

The distributions of geological formations in reservoir areas are not uniform, and they constitute various areas of reservoir bedrock. Generally, the Gurpi and Pabdeh Formations generally outcroup at the southern flank of the reservoir valley with the Gachsaran and Aghajari Formations outcroupping on the northern flanks.

Watertightness of the reservoir areas depends on the occurrence of the two formations with relatively high permeability mainly the Asmari and Gachsaran Formations.

The Asmari Formation is discussed in detail for each of the dam sites and has variable permeability (non/low to very high).

The Gachsaran Formation comprises gypsum, anhydrite, salt and marl with occasional thin beds of limestone. The rocks are generally of low strength with medium strength in the limestone beds. The different lithological units vary in thickness. The marly parts usually have low permeability or are impermeable while the other rock types are permeable with several karst features present in the reservoir areas.

The karst development is believed to be shallow and laterally developed, but evaporites are highly susceptible to karst development in depth (Xuepu, 1988).

This phenomenon can cause deep-seated reservoir leakage along karstic channels in gypsum, although the gypsum layers are confined by impervious marl beds in some localities. This situation needs careful re-examination in the next stage of investigation or construction phases.

From the above results, it is concluded that seepage losses from the dam abutments and reservoirs are expected and further studies should be carried out. In this regard, Figure 6.4 clearly indicates the various geological scenarios of foundation rocks relative to dam body/cut-off curtains and the reservoirs, karstification, and watertightness. In case A (northern flank sites) the dam foundations are on the Lower and Middle Asmari and the cut-off curtain is suspended in relatively permeable limestones of the Lower Asmari unit, whereas in case B (southern flank sites) the Lower to Middle Asmari form the dam foundation rocks and the cut-off curtain is locked in the impervious marls of the Pabdeh Formation.

Case B is obviously more favourable than with regards to watertightness and possible deep-seated leakage through the dam foundation rocks.

6.3. Slope Stability Analysis

6.3.1. Slope Mass Rating (SMR)

Based on the stereographic projection of the major joint sets (Figure 6.5) the critical planes for planar and toppling failures as well as the joint critical intersections for wedge failures were identified (Table 3.7, 3.8, 3.9). The F_1, F_2, F_3 factors were determined in each case according to the slope face dip direction and dip, and the F_4 for natural slopes was used. The SMR values (Table 6.2) in addition to the stereographic projection of the joint sets clearly indicate a potential for *planar failure* (sliding direction) into the reservoir at Seymareh, Marun and Salman Farsi, whereas at Karun-3 and Karun-4, the planar failures will mainly occur towards the downstream area. The slope instability due to wedge and toppling failures can be expected everywhere toward the reservoir and inside the dam valley.

Table 6.2. The SMR values for various units of the Asmari Formation rocks in the study area.

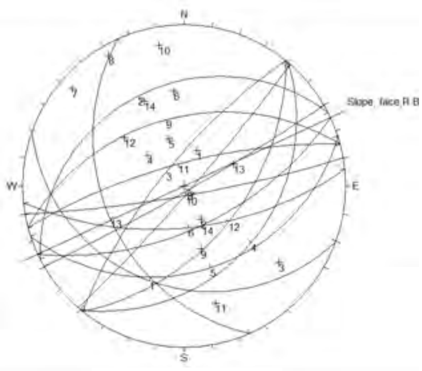
Dam site	Unit	RMR	*RMR_B*	*Slope Mass Rating (SMR)*		
				Planar	*Wedge*	*Toppling*
Karun-3	U.Asmari	44-67	*49-72*	*20- 51*	*12- 36*	*39- 62*
	L.Asmari	64-76	*69-81*	*49- 60*	*24- 45*	*50- 71*
Karun-4	U.Asmari	32-41	*57-66*	*29- 38*	*43- 52*	-
	M.Asmari	32-49	*57-74*	*29- 42*	*43- 60*	-
	L.Asmari	61-71	*61-71*	*33- 43*	*47- 57*	-
Seymareh	U.Asmari	52-69	*57-74*	*55- 71*	*RB; 24- 48 LB; 65- 80*	*RB; 62- 79 LB; 40- 57*
	M.Asmari	56-74	*61-81*	-		
	L.Asmari	56-61	*61-66*	-		
Marun	U.Asmari	51-67	*51-67*	*49- 64*	*RB; 30- 55 LB; 10- 30*	-
	M.Asmari	56-71	*56-71*	-		
	L.Asmari	59-76	*59-76*	-		
Salman Farsi	U.Asmari	25-42	*50-57*	*40- 47*	*RB; 45-74 LB; 61- 90*	*40- 47*
	M.Asmari	50-67	*62-79*	-		
	L.Asmari	25-40	*50-55*	-		

SMR values generally indicate that the Karun-3 and Karun-4 dam sites are unstable to completely unstable, and the large planar and wedge failures can be expected towards the gorge. It means that these two sites are subjected to more structural disturbances and dipping strata.

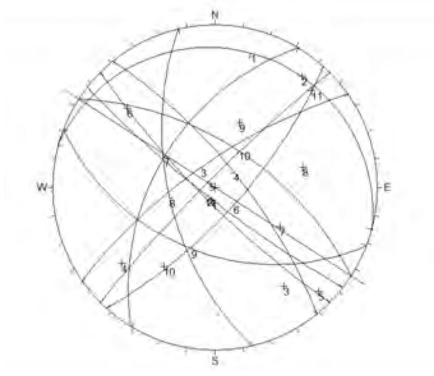
Slope stability analysis (SMR) at the Seymareh, Marun, Salman Farsi dam sites indicates unstable to partially stable rock slopes, especially in the reservoirs where big planar failures can be expected. This is supposed by the historical rock slope failure (planar failure) at the Seymareh dam where the Seymareh river bed was displaced about 1 000 m towards the northeast (Koleini et al., 2010). Displacement and sliding of thousands of cubic metres of rock and soil are the result of a large failure (Figures 4.5.1, 6.5 and 6.6).

A decrease in the shear strength of discontinuities after impoundment in addition to some slope excavations during construction, such as dam abutments, tunnel headwalls and road cuts can also reduce the SMR values according to the method of excavation (Table 3.8).

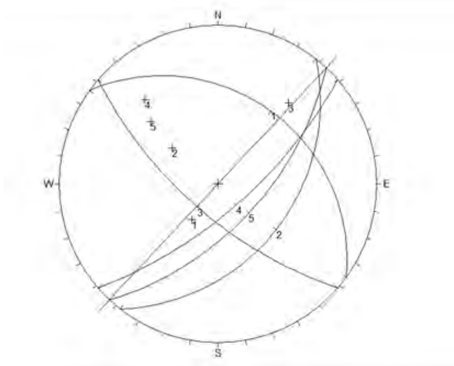
As a general rule, slopes should be designed to be no steeper than any steeply inclined sets of discontinuities along which sliding may occur. Where a slope is undercut by steeply inclined discontinuities or wedges formed by two or more sets of discontinuities, support must be provided to prevent sliding. The overall inclination of any large cut will be no steeper than the steepest natural slope of the same height in similar geological conditions. For example, for slopes of 100 m or more in height, the overall gradient should be no steeper than 65° to 70° in the Asmari Formation or 45° in the Pabdeh Formation (Acres, 1982).



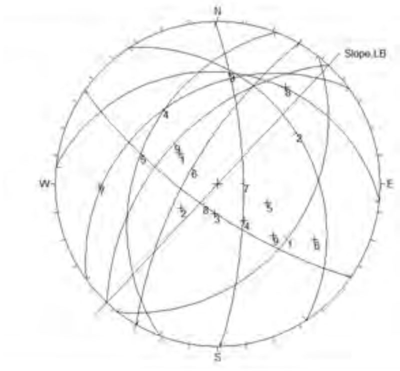
Karun-3, RB



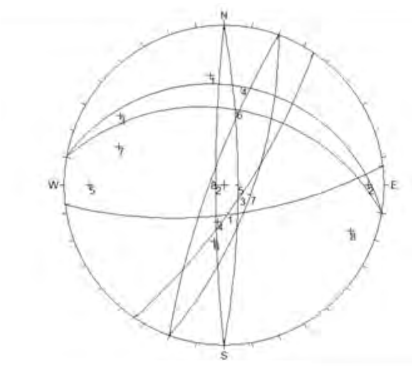
Karun-3, LB



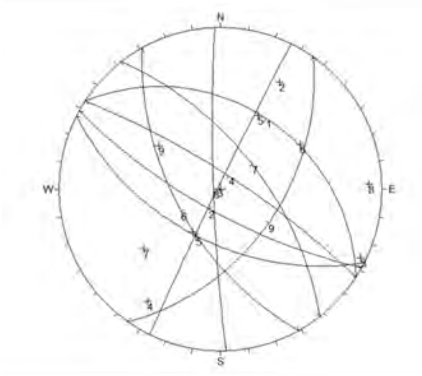
Karun-4, RB



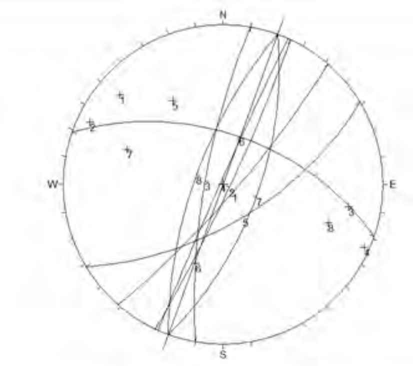
Karun-4, LB



Seymareh



Marun



Salman Farsi

Figure 6.5. The stereographic projection of major discontinuity sets in the Asmari formation limestones at the various dam locations. The slope stability based on intersections of major joint sets and rock slope faces indicates various kinds of rock failures such as planar, wedge and toppling in the area.

It should be considered that the SMR values are calculated only for natural slopes (Romana, 1985). The active tectonics of the area, excavation of natural slopes and lowering of discontinuity shear strength, resulting from reservoir impoundment can extensively increase slope instabilities at all the dam sites. Knowledge of these factors enables a preliminary assessment of the potential for continued or accelerated movement (or reactivation) of the SMR and consequent damage to the projects over the extended time.

6.3.2. Falling Rock Hazard Index (FRHI)

Rockfall is another kind of rock slope instability at the area of research, especially at Marun dam site (Koleini and Van Rooy, 2010). Among many natural hazards, rockfalls are very frequent in mountainous areas. The term rockfall is usually used to describe small phenomena from block falls of a few dm^3 to thousands of m^3. At the Marun dam site, rockfall phenomena takes place almost every day in the downstream area. In the left flank power plant and access roadways and in the right flank roadways, rockfalls are a hazard with falls from cliffs of over 100 m high and dipping at 70°-90°. Observations show that the potential of a large mass falling at this site, especially at the left flank where the power plant situated, is high because of the dip direction of the bedding planes, joint system and the active tectonism of the region. Both dam flanks were assessed according to the Falling Rock Hazard Index (FRHI) classification method (Tables 3.10 and 3.11). Rockfall phenomena can also occur in reservoirs where steep slopes are formed. These slopes are mostly on the Asmari Formation (Figure 6.6).

The FRHI was developed based on work done earlier at the Oregon and Washington Department of Transportation of United States (Singh, 2004). Many factors influence the activation of fractured rock and weathered material to fall from a rock slope face (Table 3.10). Fractures, fissures, cracks, site vibrations and other external forces are related to rock falling. Therefore, before undertaking an FRHI analysis, a stability analysis and survey of rock structures need to be undertaken. In this research, an attempt was made to use this method to determine the seriousness of falling rock hazard at the Marun dam site.
The FRHI at the Marun dam site assessed for minimum size of rock block of 10 - 20 kg. Based on the relevant input data that are listed in Tables 3.10 and 3.11, the FRHI for the left flank is moderate to high and for the right flank is a moderate risk (Table 6.3, 6.4 and Figure 6.8).

Table 6.3. Rock Fall Hazard Index score assessment at left flank.

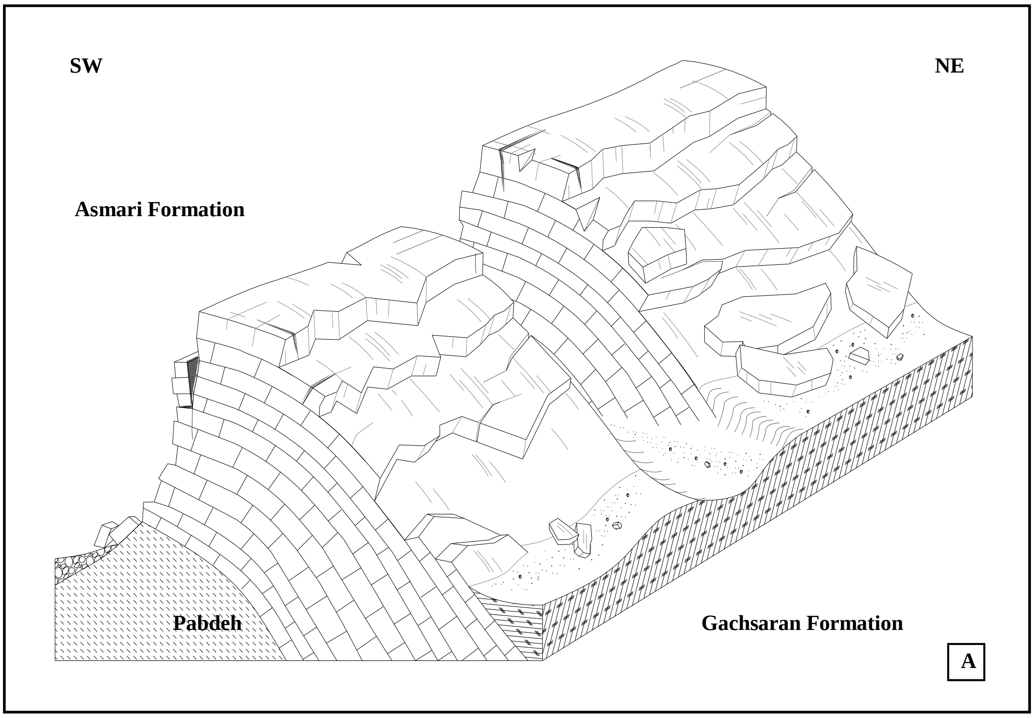
Falling Rock Hazard Index									
Face height	Face inclin.	Face irreg.	Rock condition	Spacing discon.	Block size	Volume of RF	Exc. method	Time factor	Rockfall freq.
12	3	8	3-7	1-4	6- 7	7- 11	4	8	8

Total score: 60 – 72
FRHI class: III - IV (Moderate to High Risk)

Table 6.4. Rock Fall Hazard Index score assessment at right flank.

Falling Rock Hazard Index									
Face height	Face inclin.	Face irreg.	Rock condition	Spacing discon.	Block size	Volume of RF	Exc. method	Time factor	Rockfall freq.
12	9	3	3-7	1- 4	6-7	7-11	1	8	8

Total score: 58 - 70
FRHI class: III (Moderate Risk)



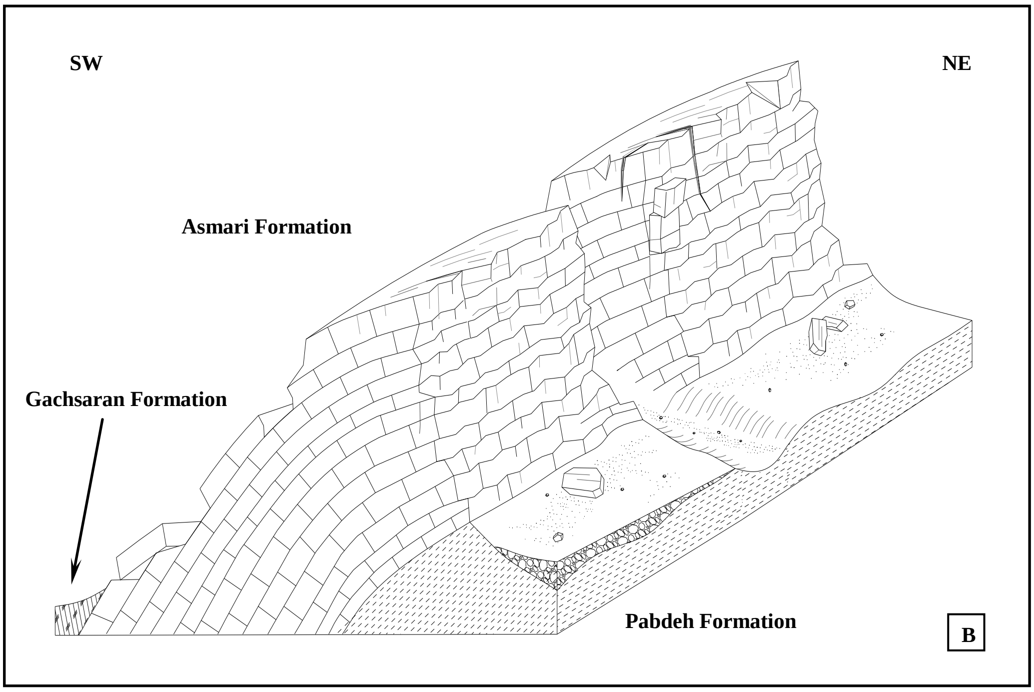


Figure 6.6. The schematic block diagrams showing geological conditions of the Asmari formation limestones as the main dam foundation rocks and dam localities in northern flank sites (A- Seymareh, Marun, Salman Farsi) and in southern flank sites (B- Karun-3, Karun-4). They are typically indicating various types of unstable slopes. In case A, planar and wedge failures toward the reservoir, and wedged, toppling failures toward the gorge. In case B, wedge and toppling failures toward the reservoir and gorge and planar failure toward the gorge will be expected.

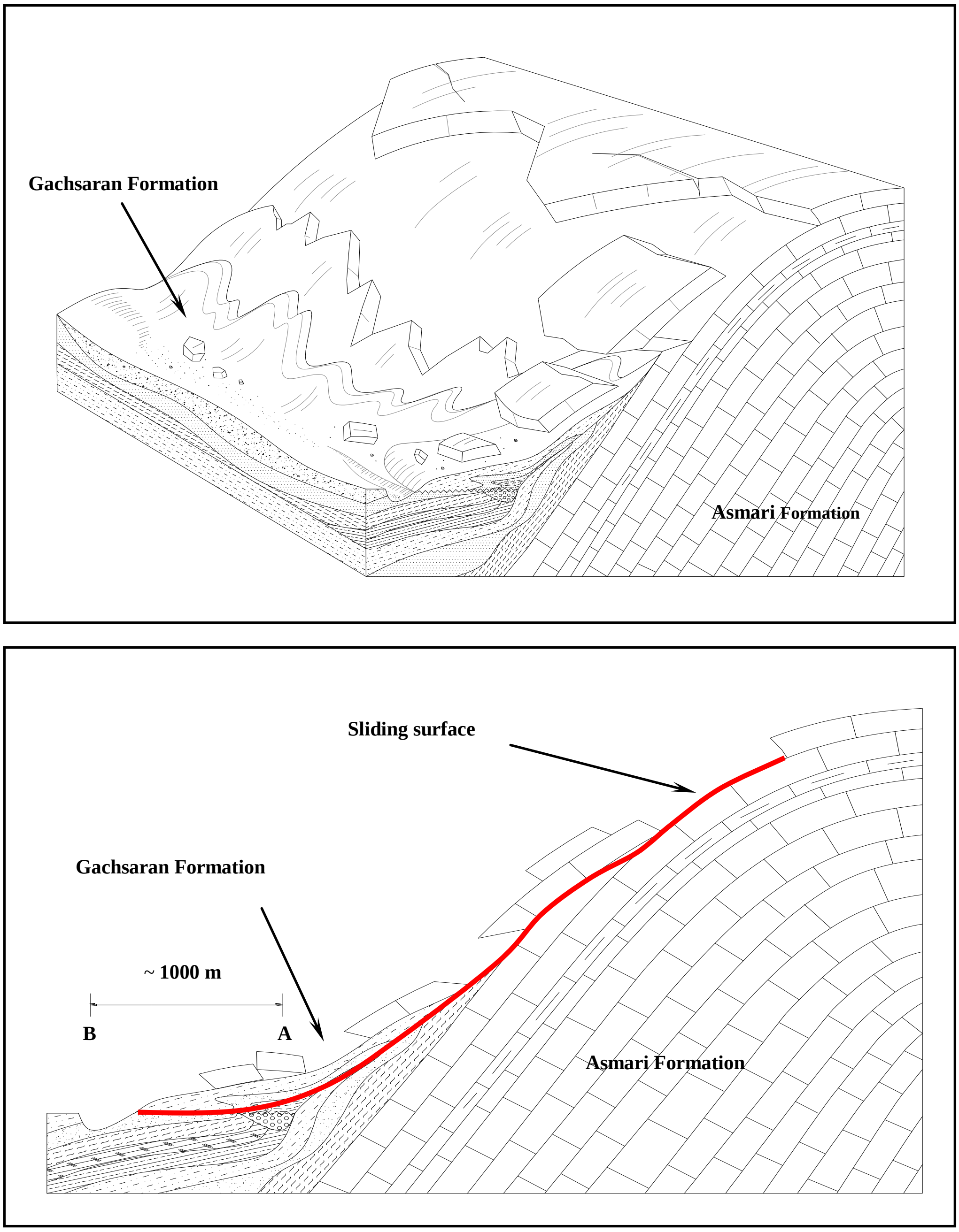


Figure 6.7. The typical block diagram and geological section of the Asmari and Gachsaran formations in the Zagros folded belt and the possibility of land slide hazard after impoundment of the reservoir. In general rock sliding adjacent to the dam locations toward the reservoir, will mainly be planar (in Asmari limestones) and rotational to planar (in Gachsaran evaporites). As a result of rock failure the Seymareh river bed was displaced about 1000 m toward the northeast during historic times.

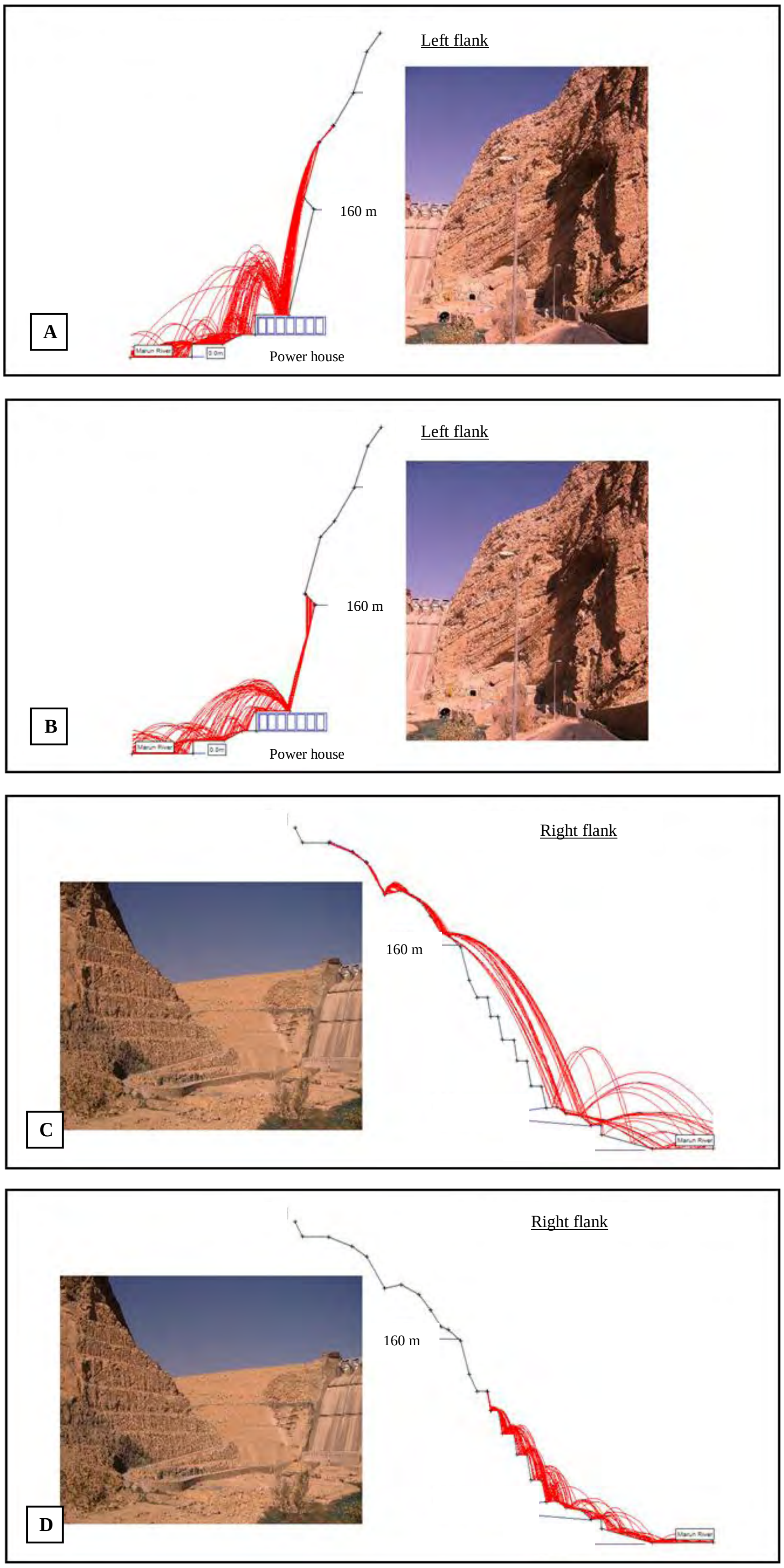


Figure 6.8. Rockfall hazard at Marun dam site in successive stages on the left flank (A, B) and right flank (C, D). The power plant and access roadways are subjected to rock fall hazard every day.

6.3.3. Rock Slope Stabilization

Before the introduction of a suitable plan for rock slope protection, the short-term instability of loose rock materials that can be removed and easily fall from the slope face must be determined. Removal of loose rock materials can be done by scaling and there are different ways to conduct rock slope scaling based on the specific project conditions. Hand scaling with bars or rakes may be an adequate method for most short-term excavations where the face height is less than three metres and only small fractured rocks are likely to fall (Singh, 2004).

Long-term excavations, high slopes, and rock faces having large rocks and overhangs may require heavier equipment, such as, hydraulic splitters, drag scaling, and light explosives (trimming).

The identification of unstable 'keyblocks' will be required simultaneously with scaling of the slope face at this stage. Release of keyblocks can sometimes precipitate rock falls of significant size or in extreme cases large-scale slope failures.

There are different ways to protect a slope face from rock failure and rock fall events and many companies manufacture such systems.

The general procedures to restraining rockfalls are listed below (Hoek, 2000):

- *Berms* are very effective means of catching rockfalls and are frequently used on permanent slopes. However, berms can only be excavated from the top downwards and they are of limited use in minimising the risk of rockfalls during construction.
- *Rocksheds or Avalanche shelters* are widely used on steep slopes above narrow railways or roadways. An effective shelter requires a steeply sloping roof covering a relatively narrow span. In the case of a wide multi-lane highway, it may not be possible to design a rockshed structure with sufficient strength to withstand large rockfalls. It is generally advisable to place a fill of gravel or soil on top of the rockshed in order to act as both a retarder and a deflector for rockfalls.
- *Rock traps* work well in catching rockfalls provided that there is sufficient room at the toe of the slope to accommodate these rock traps. In the case of very narrow roadways at the toe of steep slopes, there may not be sufficient room to accommodate rock traps. This restriction also applies to earth or rock fills and to gabion walls or massive concrete walls.
- *Catchment fences* or *Barrier fences* are commonly used to absorb energy and are designed for various capacities (Figure 6.9 and 6.10).
- *Mesh draped*, is commonly used for permanent slopes and is illustrated in Figure 6.11. The mesh is draped over the rock face and attached at several locations along the slope. The purpose of the mesh is not to stop rockfalls but to trap the falling rock between the mesh and the rock face and so to reduce the horizontal velocity component which causes the rock to bounce out onto the roadway below.

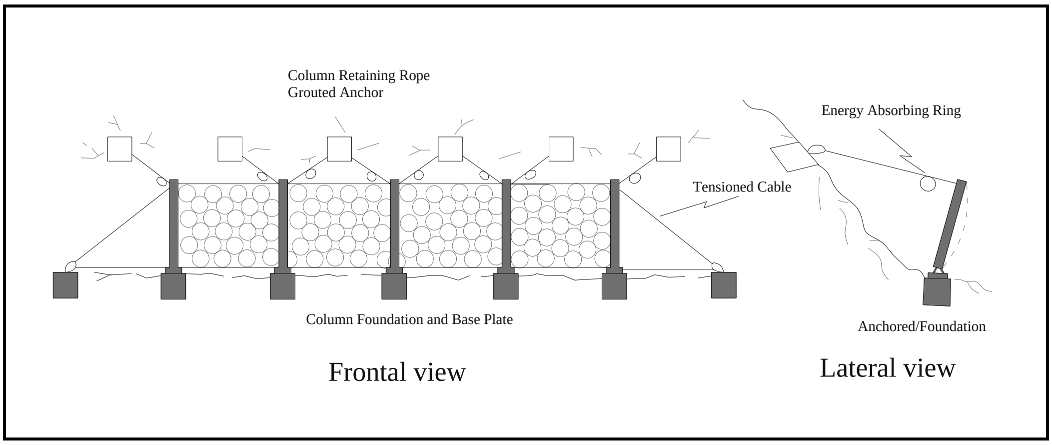


Figure 6.9. Catchment fence or Barrier fence specifications and installation procedure (after Geobrugg AG protection system, Switzerland, 2010).

Figure 6.10. Energy absorbing ring (A), when subjected to impact loading the ring deforms plastically (B) and absorbs the energy of the boulder. (C) Impact sentinel sensors check the status of rockfall protection systems and set off an alarm (Geobrugg AG protection system- Switzerland, 2010).

The most common elements for stabilization of rock slopes are as follow (Hoek, 2000):

- Rock bolt; spot and systematic bolting (Figure 6.12)
- Wire mesh and chain link mesh
- Shotcrete

Fibre steel shotcrete can be used easily and effectively, where the slope face is not accessible and dangerous for operation workers.

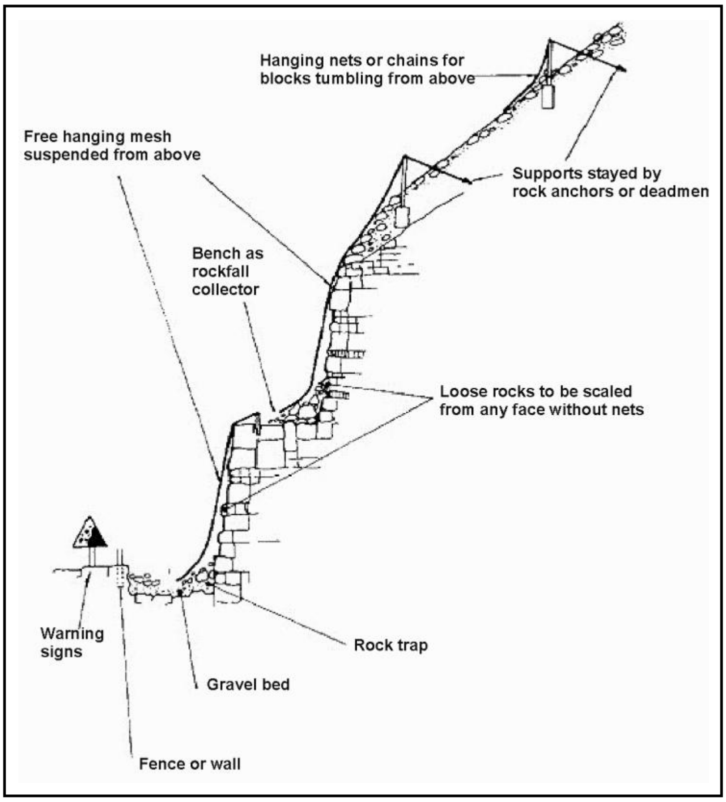


Figure 6.11. Rockfall control by free hanging mesh drape and its installation. It is commonly used for permanent slopes. It can be used effectively at the right flank of the Marun dam (after Fookes and Sweeney, 1976).

Shotcrete is the generic name for cement, sand and fine aggregate concretes which are applied pneumatically and compacted dynamically under high velocity (Figure 6.13). Of the many developments in shotcrete technology in recent years, two of the most significant were the introduction of silica fume, used as a cementitious admixture, and steel fibre reinforcement (Hoek, 2000).

Silica fume or micro silica is a by-product of the ferro silicon metal industry and is an extremely fine pozzolan. Pozzolans are cementitious materials which react with the calcium hydroxide produced during cement hydration. Silica fume, added in quantities of 8 to 13% by weight of cement, can allow shotcrete to achieve compressive strengths which are double or triple the value of plain shotcrete mixes. The result is an extremely strong, impermeable and durable shotcrete (Hoek, 2000). Other benefits include reduced rebound, improved flexural strength, improved bond with the rock mass and the ability to place layers of up to 200 mm thick in a single pass because of the shotcrete's 'stickiness'. However, when using wet mix shotcrete, this stickiness decreases the workability of the material and superplaticizers are required to restore this workability (Hoek, 2000).

Steel fibre reinforced shotcrete was introduced in the 1970s and has since gained world-wide acceptance as a replacement for traditional wire mesh reinforced plain shotcrete (Hoek, 2000) (Figure 6.14). The main role that reinforcement plays in shotcrete is to impart ductility to an otherwise brittle material. Steel fibres used in slab bending tests by Kompen (1989). The fibres are glued together in bundles with a water soluble glue to facilitate handling and homogeneous distribution of the fibres in the shotcrete (Hoek, 2000).

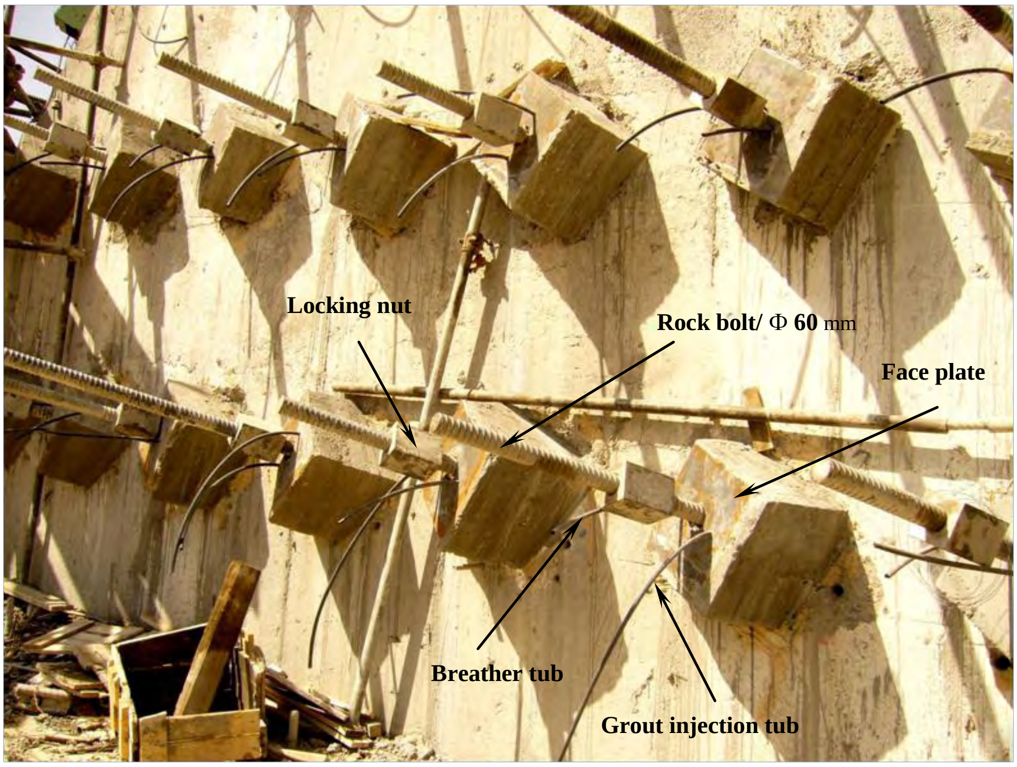


Figure 6.12. Systematic rock bolting (60 mm in diametre) of rock slope face at spillway- right flank of Karun-4 dam (2007).

Figure 6.13. Rock slope failure after application of unreinforced shotcrete on marl units of the Asmari Formation. The marls or such rocks need to be stablized by reinforced shotcrete due to ductility and deformability of the rock mass. The vertical extensional joints and fractures due to gravity movement of the rock mass can clearly be observed (Karun-3 dam site, entrance gate, 2007).

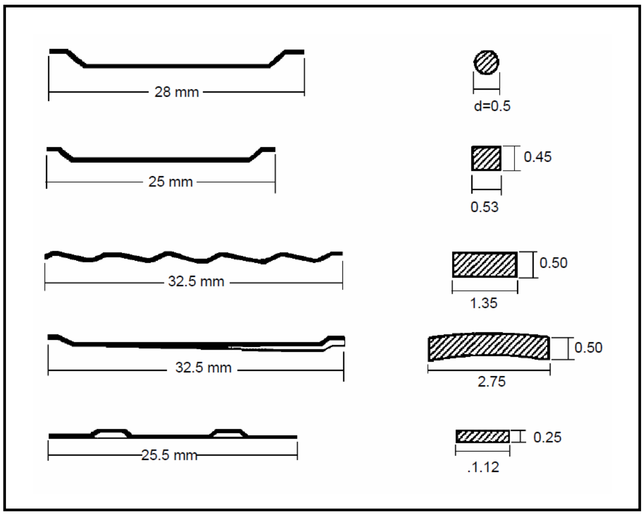


Figure 6.14. Steel fibre types available on the North American market (after Wood et al., 1993). (Note: all dimensions are in mm).

6.4. Effect of Reservoir Impounding

The impounding of reservoirs may on occasion, destabilized the rock/soil masses forming the dam and reservoir flanks and consequently cause landslides. Such landslides may vary greatly in size and they may move very rapidly or very slowly. Landslides may move in response to increased driving forces such as increased depth of saturation, or decreased resisting forces such as loss of shear strength due to saturation along potential slip surfaces. The stability of the reservoir rim depends on some parameters, such as reservoir water level, the nature of formations which have most contact with reservoir water and dip into the reservoir (Singh and Goel, 1999).

Landslides into the reservoir can cause severe damage such as partial or complete blockage of the reservoir or by causing very large waves. Hypothetically at dam sites, the critical case would be the occurrence of extremely large, rapid landslides of the rockfall-debris flow type. Such a rockfall-debris flow would travel at a high velocity and depending on the volume of material involved, could create an enormous wave in the reservoir. If generated close to the dam, they might destroy some structures and installations and in addition, overtop the dams. It is however most unlikely that the dams would be seriously damaged, but at Marun rockfill dam may be can cause serious impact and damage. In 1963 a rapid landslide into the reservoir of the Vaijont arch dam in Italy caused a huge wave overtop the dam. The dam suffered little damage but there was considerable damage downstream (Hoek, 2000).

In view point of the above, it is necessary to assess the probability of landslide activity due to impounding and the probability of damage to the project, should such landslides occur. The instrumentation and monitoring of areas with high landslide potential during the design, construction and operation phases will be helpful to recognise and forecast such phenomena as well.

6.5. Engineering Classification of Rock Mass

The most common methods for evaluating the rock mass are engineering rock mass classifications such as RQD, RMR, Q and GSI methods. The geomechanical parameters of a

rock mass are determined by using the results of rock mechanics lab tests (UCS) in addition to Schmidt hammer field test and results of engineering rock mass classifications.
For this purpose the rock mass classification at each dam site was considered. The values of RMR were calculated for each different rock unit, based on the data obtained from exploratory boreholes, tunnels and results of laboratory tests, field tests and joints surveys. Figures 6.15, 6.16 and 6.17 show frequency distribution of the RMR, UCS and GSI values calculated for each dam site.

In this research the program RocLab© was used to estimate rock mass properties with input data UCS, GSI, mi and D values, and the results introduced in Table 6.5.
The GSI values for the Asmari formation units were plotted on the basic GSI chart (Figure 6.18). They constitute various zones on the GSI chart according to their geological characteristics. However the GSI zones fall close and relatively cover each other, but two distinctive areas can be identified. The first zone is related to the Karun-3, Karun-4 Dams and the other related to the Seymareh, Marun and Salman Farsi rock masses.

In the first case the rock mass is;

- Blocky- Very Well Interlocked and Good (B/G) to
- Blocky Disturbed and Fair (BD/F)

The GSI values are between 35 to 65.

In the second case the rock mass is:

- Blocky- Very Well Interlocked and Good (BG) to
- Very Blocky- Interlocked and Fair (VB/F)

The GSI values are between 45 to 70.

In case of Salman Farsi Dam it is however relatively well matched to the second case but due to extensive development of marlstone, marly limestone with thin interbedded limestone in the lower and upper units on one hand and extensive development of dissolution and karstic features on the other hand a wide range of GSI values are present;

- Blocky- Very Well Interlocked and Good (BG) to
- Blocky Disturbed and Poor (BD/P)

The situation at each dam can be observed on the GSI chart in Figure 5.6.

In general four distinct areas can be distinguished on the GSI chart regarding the behaviour of rock mass in Tunnelling Operations (Marinos and Hoek, 2005):

I. Stable conditions; only at great depth possibillity of rock burst failures.
In very hard massive rock masses at great depths, spalling, slabbing and rockbursting are the modes of failure that may develop, controlled by brittle fracture propagation in the intact rock with only minor influence of the discontinuities.

II. Stability mainly controlled by structural failures.
Attention has to be concentrated on avoiding structural instabilities from wedges. This makes structurally dependant instability more critical and generally demands heavier rock bolting patterns and /or thicker shotcrete.

II/III. Stability controlled by structural failures or mild overstressing.
In the case of a more fractured limestone and marly limestone (eg. GSI values of 25-40) the behaviour is controlled by sliding and rotation on discontinuity surfaces with relatively little failure of the intact rock pieces (zone II/III). In this range of GSI values the RQD values can be very low. This is normal, given the structure of the rock

mass, but some of the frictional behaviour of the unaltered pieces of the mass is retained. Thus the control of stability can be effectively improved during excavation of the tunnel by keeping the rock mass confined.

III. Stability controlled by stress dependent rock mass failure with significant squeezing at depth.

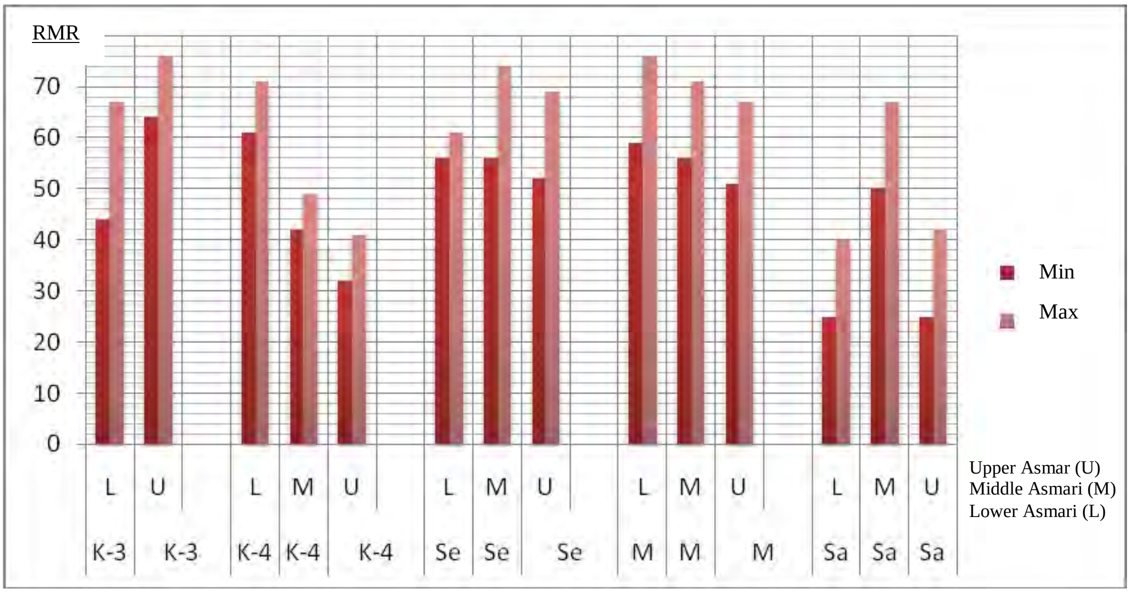


Figure 6.15. Histogram of RMR values for the Asmari formation limestone, calculated for Karun-3 (K-3), Karun-4 (K-4), Seymareh (Se), Marun (M), and Salman Farsi (Sa) dam sites.

Ravelling from the face may occur in masses corresponding to the low areas of zone II/III and in zone III.
In this case (poor quality rock mass such as marlstone and shale), due to either weathering or shearing, blockiness may be almost completely lost and clayey sections with swelling materials may be present.

Hoek and Karzulovic (2000) used the GSI and strength of rock masses and suggested a range of GSI for different Excavation Methods. They proposed that the rock mass can be:

a) *Dug* up to GSI values of about 40 and a rock mass strength value of about 1 MPa.
b) *Ripped* up to GSI values of about 60 and a rock mass strength value of about 10 MPa.
c) *Blasted* when the GSI values are greater than 60 and rock mass strength value more than 15 MPa.

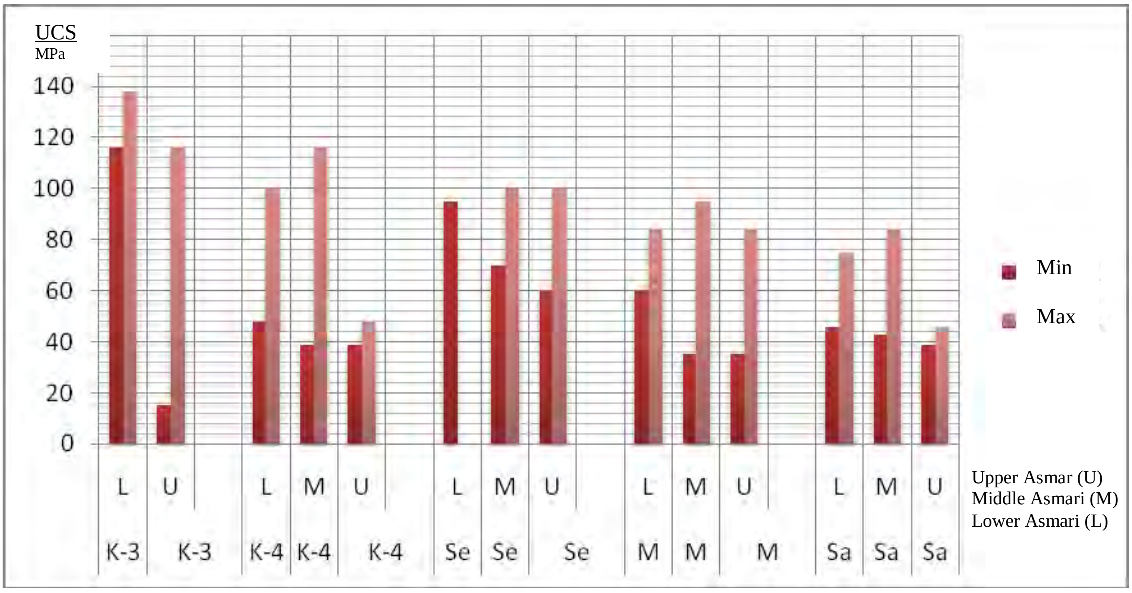


Figure 6.16. Histogram of UCS values for the Asmari formation limestone, calculated for Karun-3 (K-3), Karun-4 (K-4), Seymareh (Se), Marun (M), and Salman Farsi (Sa) dam sites.

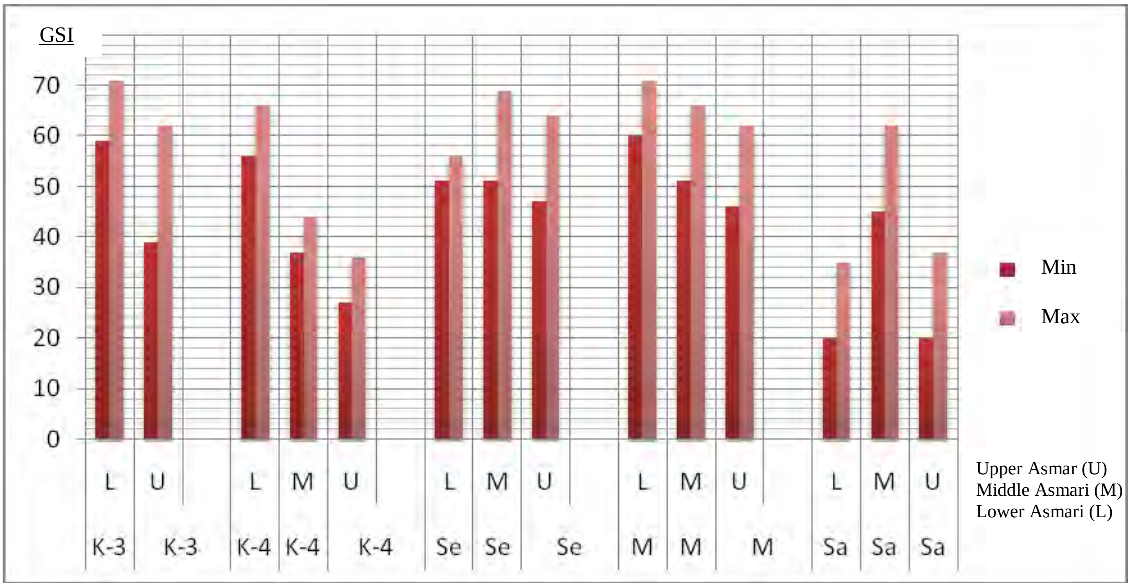


Figure 6.17. Histogram of GSI values for the Asmari formation limestone, calculated for Karun-3 (K-3), Karun-4 (K-4), Seymareh (Se), Marun (M), and Salman Farsi (Sa) dam sites.

Table 6.5. A summary of the engineering rock mass properties of the Asmari Formation at the different dam sites.

Dam site	Unit	Porosity%	Permeability	UCS/MPa	RQD%	RMR	Q	GSI	Mohr coulomb fit		Rock mass parameters		
									C, MPa	Phi (°)	Sigt, MPa	Sigc, MPa	Em, MPa
Karun-3	U.Asmari	0.75-13.8	M- VH	15- 116	31-53	44-67	1.2-58.4	39-62	0.55-6.7	24.4-32.1	-0.02 to -0.8	0.47- 13.9	2056.1-19952.6
	L.Asmari	1- 15.7	M- VH	116- 138	50-84	64-76	35.1-268.6	59-71	1.6-3.97	57-57.4	-0.6 to -1.7	11.7-27.4	16788-33496.5
Karun-4	U.Asmari	0.5- 5	Non to V.H	39-48	45-78	32-41	0.15-0.7	27-36	1.2-1.8	21.9-24.6	-0.02 to -0.05	0.54-1.23	1661.6-3094.7
	M.Asmari	1- 7	M to V.H	39-116	53-84	42-49	0.84-2.8	37-44	1.45-4.9	24.9-26.9	-0.04 to -0.2	1.07-4.9	2954.8-7079.5
	L.Asmari	0.75- 15.2	H to V.H	48-100	55-83	61-71	21-151	56-66	0.8-2.3	47.8-53.1	-0.2 to -0.96	4.08- 15	9786.4-25118.9
Seymareh	U.Asmari	0.75- 4.4	H to V.H	60-100	65-94	52-69	4.6- 82	47-64	0.7-2.1	47.3-53	-0.14 to -0.83	3.03-13.4	6517.4-22387.2
	M.Asmari	0.6- 7.5	L to H	70-100	75-95	56-74	9.1-191.4	51-69	0.84-2.8	49.2-53.2	-0.22 to -1.2	4.5- 17.8	8862.4-29853.8
	L.Asmari	1.4- 5.2	Non to M	95	80	56-61	9.1- 21.1	51-56	4.5-4.9	28.9-30.4	-0.3 to -0.43	6.1-8.1	10324.3-13767.7
Marun	U.Asmari	2.1- 5.4	M to H	35-84	50-70	51-67	3.9-58.4	46-62	0.48-1.6	43.2-52.1	-0.07 to -0.6	1.7-10.1	4699.3-18286.9
	M.Asmari	1.4- 11	L to M	35-95	50-80	56-71	9.1-115.1	51-66	0.6-2.22	44.5-52.9	-0.1 to -0.9	2.23-14.3	6266.6-24482.8
	L.Asmari	1.3- 14.9	L to M	60-84	70-85	59-76	15.1-268.6	60-71	0.85-2.6	48.8-52.5	-0.23 to -1.2	4.6-16.7	9751.6-30700.1
Salman-Farsi	U.Asmari	1.5- 19.4	Non to L	39-46	58	25-42	0.047-0.84	20- 37	0.9- 1.6	18.8-23.8	-0.13 to -0.06	0.3- 1.3	1110.54-3209.1
	M.Asmari	0.3- 8	L to V.H	43-84	60-100	50-67	6.4- 58.4	45- 62	0.64-1.5	50.3-55.9	-0.8 to - 0.4	2.7-10.1	5808.66-18286.9
	L.Asmari	1- 5.6	L- VH (local)	46-75	37- 50	25-40	0.047-0.6	20- 35	1.1-2.2	18.8-23.3	-0.02 to- 0.07	0.37- 1.6	1206.1-3399.8

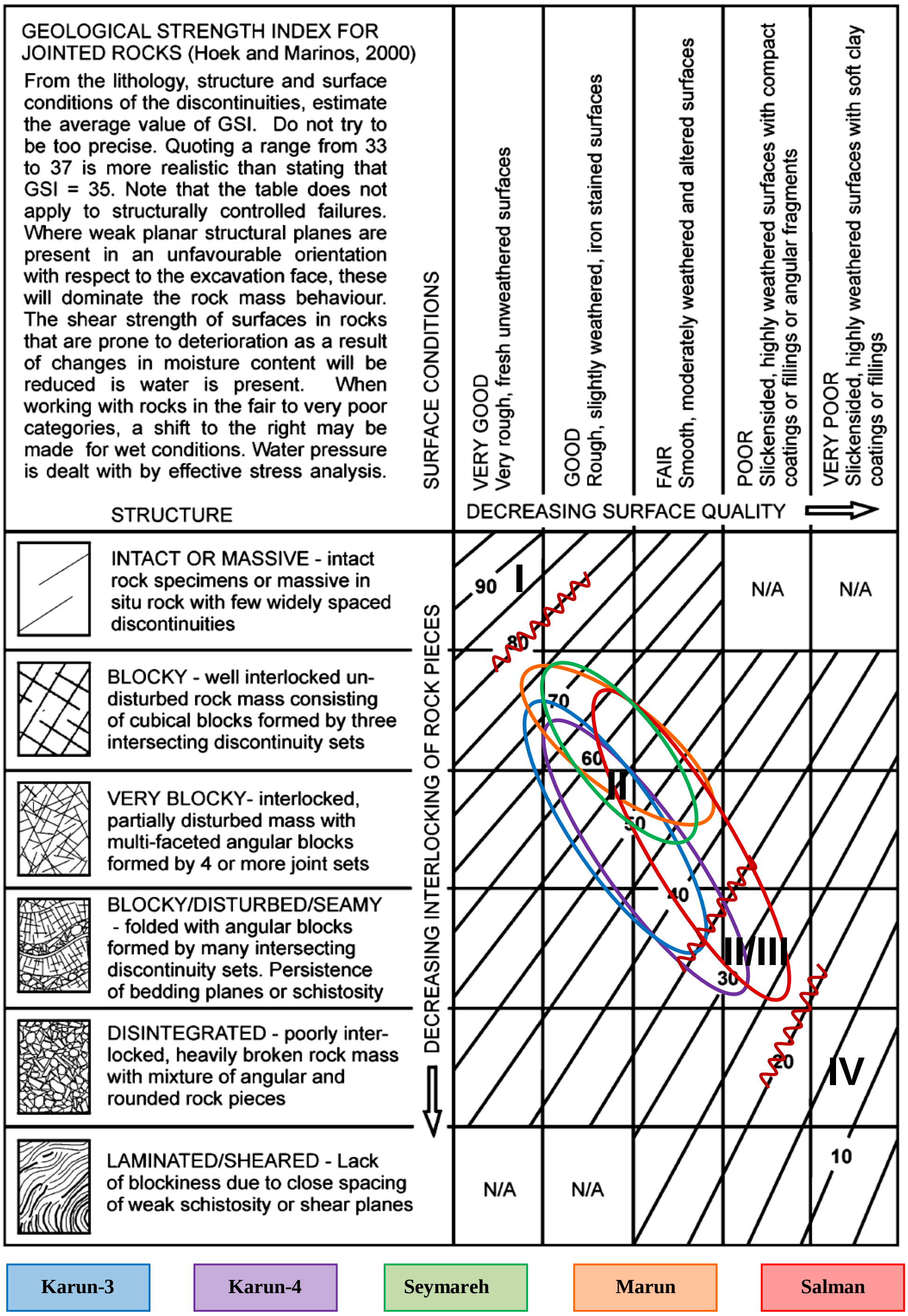


Figure 6.18. Geological Strength Index (GSI) chart, for jointed rock mass (Hoek and Brown 1997, Hoek and Karzulovic, 2001, Marinos and Hoek, 2005). The shaded areas indicate the distribution of geological strength index values of the various rock mass units of the Asmari Formation.

6.6. Stability of Dams against Horizontal Sliding

6.6.1. DMR (Dam Mass Rating)

Bieniawski and Orr (1976) proposed the following adjustment factors for the effect of joint orientation in horizontal stability (Table 6.6) based on experience and on consideration of stress distributions in the foundation rock mass as well as on the assumption that in a dam structure, both the arch and the gravity effects are present.

Table 6.6. Adjusting factor for dam stability after joints orientation (after Bieniawski and Orr, 1976)

Dam	VF	F	Fa	U	VU
Gravity	Very favourable	Favourable	Fair	Unfavourable	Very unfavourable
Dip (°)	0- 10	30- 60	10- 30 DS	10- 30 US	60- 90
Rating	0	-2	-7	-15	-25

Snell and Knigth (1991) approached the problem of dam stability systematically taking account of all the forces and stresses acting on the dam. Based on their study, it appears that a different set of adjusting factors must be applied. Table 6.7 shows these new tentative adjusting factors according to the main discontinuity orientations. The numerical rating values proposed originally by Bieniawski have been retained.

Table 6.7. Adjusting factors (R_{STA}) for the stability according to the joint orientation (after Romana, 2003)

Type of Dam	VF Very favourable	F Favourable	FA Fair	U Un favourable	VU Very unfavourable
Fill	Others	10- 30 DS	0- 10 A	-	-
Gravity	10- 60 DS	30- 60 US 60- 90 A	10- 30 US	0- 10 A	-
Arch	30- 60 DS	10- 30 DS	30- 60 US 60- 90 A	10- 30 US	0- 10 A
R_{STA}	0	-2	-7	-15	-25

DS. dip downstream, US. dip upstream, A. any dip
Gravity dams include CVC and RCC concrete dams

When the dip direction of the significant joint is not almost parallel to the downstream-upstream axis of the dam, the danger of sliding diminishes due to the geometrical difficulties to slide. It is possible to take account of this effect by multiplying the rating of the adjusting factor for dam stability R_{STA}, by a geometric correction factor (CF).

$$CF = (1- \sin (\alpha_d - \alpha_j))^2 \qquad (\alpha_d > \alpha_j) \qquad (5.1)$$

$$CF = (1- \sin (\alpha_j - \alpha_d))^2 \qquad (\alpha_j > \alpha_d) \qquad (5.2)$$

Where α_d is the upstream-downstream direction of the dam axis and α_j is the dip direction of the significant joint.

$$DMR_{STA} = RMR_{BD} + CF \times R_{STA} \qquad (5.3)$$

Where RMR_{BD} (basic dry RMR) is the addition of the RMR five parameters and R_{STA} is the adjusting factor for dam stability (Table 5.9).
Actually there are no data allowing to establish a correlation between the value of DMR_{STA} and the degree of safety of the dam against sliding. As a rule of thumb, it is suggested (Romana, 2003) that if:

- ***DMR_{STA} > 60*** No primary concern

- ***60 > DMR$_{STA}$ > 30*** Concern
- ***30 > DMR$_{STA}$*** Serious concern

The above values can not be taken as numerical statements but only as danger signals for the designer. Dam stability must always and in any case be checked by the designer taking into account the distribution of pore water pressure across the dam foundation and of the mobilized shear strength of the significant joints (Romana, 2003).
According to the DMR$_{STA}$ values (Table 6.8) from the above calculations the stability of the various dams against horizontal sliding can generally be classified into; *No primary concern*.

Table 6.8. DMR evaluation of dam foundation rocks at the five dam sites.

Dam site	Unit	RMR	RMR$_{BD}$	Mean. Weighted RMR$_{BD}$	α_d	α_j	R$_{STA}$	DMR$_{STA}$
Karun-3	U.Asmari	44-67	60- 80	78	270	230	-7	77
	L.Asmari	64-76	70-89					
Karun-4	U.Asmari	32-41	62- 71	73	225	218	-7	67.4
	M.Asmari	42-49	62- 79					
	L.Asmari	61-71	72- 79					
Seymareh	U.Asmari	52-69	65- 82	77	197	30	-7	73
	M.Asmari	56-74	69- 87					
	L.Asmari	56-61	69- 74					
Marun	U.Asmari	51-67	62- 75	77	212	33	0	77
	M.Asmari	56-71	67- 79					
	L.Asmari	59-76	70- 84					
Salman Farsi	U.Asmari	25-42	60- 62	71	199	19	-2	69
	M.Asmari	50-67	70- 87					
	L.Asmari	25-40	55- 60					

6.7. Underground Rock Support

The underground structures requiring rock support are the diversion tunnels, hydropower tunnels and power chambers. The aim of the rock support is to ensure that the strength of the rock surrounding the excavation is mobilized to the extent that the rock mass is self supporting. In other words, this will be the primary form of support with no allowance or contribution from linings placed for hydraulic purposes. The performance of any rock support system during the lifetime of the excavation will be a function of the load/deformation characteristics of the ground and lining. Further field and laboratory investigations will be performed to accurately define these characteristics for detailed design.
The evaluation of tunnel support requirements has been done according to empirical approaches by Bieniawski (1978) in which support requirements are determined by means of a classification system.

In general rock bolting and shotcreting form the basis of the support system for the tunnels. The level of application will depend on the quality of the rock mass. This is reflected in three classes of support which are proposed for the tunnels. The differences in level of support relate to variations in thickness of shotcrete, density and length of rock bolts, application of steel wire mesh.
In this regard three rock mass ratings from the geomechanical classification of Bieniawski (1984) were recognized.

1. ***Good quality rock mass (II)***
   Comprise massive to thickly bedded limestone, crystalline limestone and dolomitic limestone
2. ***Fair quality rock mass (III)***
   Comprised marly limestone with thin interbedded limestone

3. Weak quality rock mass (IV)

Mainly comprise marlstone with thin interbedded limestone and shale.

These categories have support elements which have been discussed in detail in Chapter 3. The three types of support elements can be summarized as follows:

II. Light Support

4 to 5 m rockbolt with 2.5 m to 2.5 m by 2.5 m to 2.5 m grids of bolts above the spring line, fully grouted with locally wire mesh, 20 to 30 mm shotcrete in crown and in sides if required.

III. Medium Support

5 to 6 m rockbolt with 2 m to 2 m by 2 m to 2 m grid of bolts in crown and sides, fully grouted with wire mesh, 50 - 100 mm shotcrete in crown and 30 mm in sides.

IV. Heavy Support

5 to 6 m rock bolt with 1.5 m to 1.5 m by 1.5 m to 1.5 m grid of bolts in crown and sides, fully grouted with wire mesh, 100- 150 mm shotcrete in crown and 100 mm in sides.

It is implicit in the above approach that the performance of the tunnel support systems should be carefully checked and monitored during construction. This will allow a readjustment of support levels, depending on the results.

6.8. Cuttability of Asmari Formation Limestone

The cuttability of rock is particularly important when using roadheader-boom type tunnelling machines. According to Fowell and Johnson (1982), interpretation of borehole information at the site-investigation stage for predicting roadheader cutting rates is facilitated by the use of rock mass classifications.
Based on 20 field results, Fowell and Johnson (1982) derived a relationship between the RMR values and the cutting rate (m^3/h) for the heavyweight class of *boom tunnelling machines*.
The results are given in Figure 6.19. The authors reported that the only modification they made in the use of the Geomechanics Classification was in the rating for orientation, since, for excavation in general, an inverse relationship exists between support requirements and ease of excavation. It can be concluded that the RMR system provide a remarkable consistent relationship with the *roadheader* cutting rate.
The Cuttability rate of the Asmari formation limestones according to roadheader cutting rate (boom tunnelling machine) of Fowell and Johnson (1982) experimental method is shown in Table 6.9.
The cuttability rate for various rock mass type of the Asmari Formation can be categorized as follows:

I.	Massive to thickly bedded Limestone and Dolomitic limestone;	15- 40 m^3/h
II.	Medium to thinly bedded limestone, Marly limestone;	60-100 m^3/h
III.	Marlstone and Shale;	80- >160 m^3/h

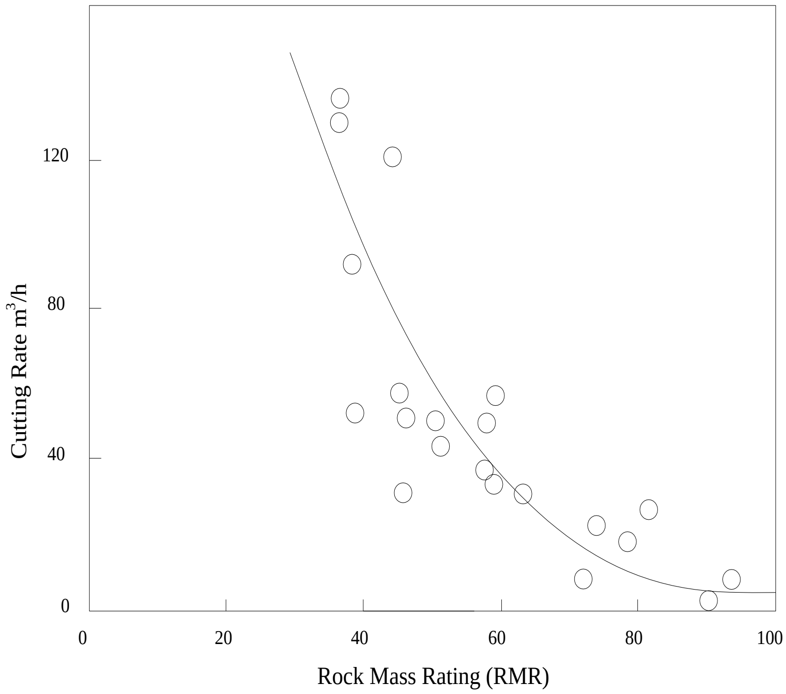


Figure 6.19. Relationship between RMR and rock cutting rate. (after Fowell and Johnson, 1982).

Table 6.9. The cuttability rates of the Asmari formation limestone based on Fowell and Johnson (1982) experimental method.

Dam site	Unit	UCS/MPa	RQD%	RMR	Cuttability m^3/h
Karun-3	U. Asmari	15- 116	31-53	44-67	79- 23
	L. Asmari	116- 138	50-84	64-76	28- 16
Karun-4	U. Asmari	39-48	45-78	32-41	135- 93
	M. Asmari	39-116	53-84	42-49	89- 64
	L. Asmari	48-100	55-83	61-71	34- 18
Seymareh	U. Asmari	60-100	65-94	52-69	54- 20
	M. Asmari	70-100	75-95	56-74	44- 14
	L. Asmari	95	80	56-61	44- 34
Marun	U. Asmari	35-84	50-70	51-67	58- 16
	M. Asmari	35-95	50-80	56-71	44- 18
	L. Asmari	60-84	70-85	59-76	37- 16
Salman Farsi	U. Asmari	39-46	58	25-42	>160- 89
	M. Asmari	43-84	60-100	50-67	60- 23
	L. Asmari	46-75	37- 50	25-40	>160- 97

6.9. Net Allowable Bearing Pressure Classification

The rock mass rating (RMR) of Bieniawski (1973) can be used to determine the net allowable bearing pressure of a rock mass based on Table 6.10 (Singh, 1991 and Mehrotra, 1993). The information in Table 6.10 results from plate load tests at 60 construction sites for spread foundations 6 m wide, and with a 12 mm settlement. Figure 6.20 indicates the precise relationship between RMR values and net allowable bearing pressure carried out at the Indian Institute of Technology Roorkee- India (Mehrotra, 1993).

Table 6.10. Net allowable bearing capacity according to RMR values (after Mehrotra, 1993).

Rock Class	I	II	III	IV	V
	Very Good	Good	Fair	Weak	Very Weak
RMR	81- 100	61- 80	41- 60	21- 40	0- 20
qa (t/m^2)	440- 600	280- 440	135- 280	45- 135	30- 45

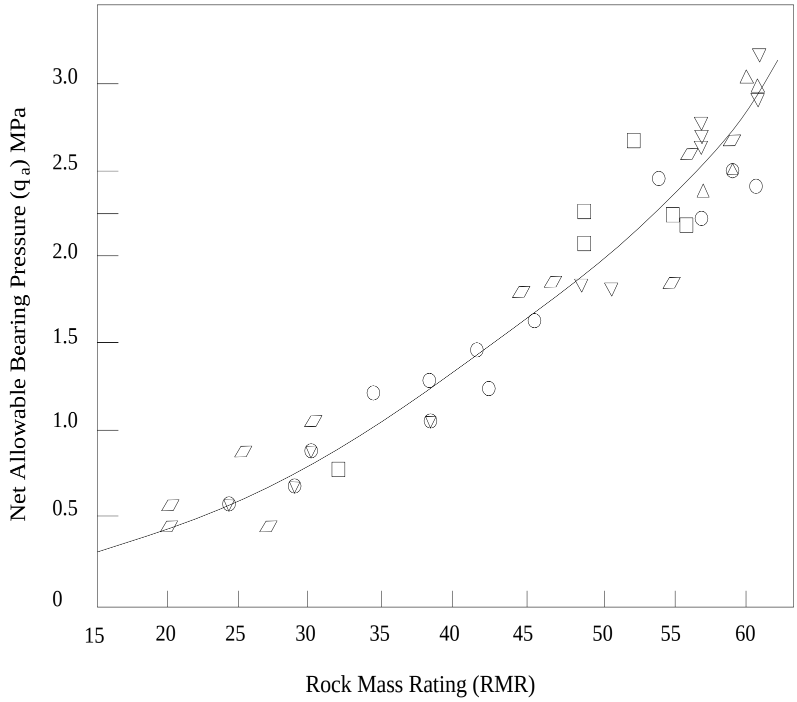


Figure 6.20. Relationship between net allowable bearing capacity and Rock Mass Rating in natural moisture content (after Mehrotra, 1993).

In this case, the RMR values must be estimated for rock mass foundations to a depth equal to the foundation width. If the upper parts of the foundation are in low quality rock mass, the RMR values related to this part should be considered for foundation design, or this part should be completely removed and filled by a suitable concrete design.

The net allowable bearing pressure values during seismic loading should be increased up to about 50% according to the rock mass rheological behaviour (Mehrotra, 1993). The net alloable bearing capacity of he Asmari Formation rock masses according to RMR values, based on Mehrotra (1993) are classified in Table 6.11.

Table 6.11. The RMR values of the Asmari Formation limestone

Dam site	Unit	RMR	Classification
Karun-3	U.Asmari	44-67	Fair- Good
	L.Asmari	64-76	Good
Karun-4	U.Asmari	32-41	Weak- Fair
	M.Asmari	32-49	Weak- Fair
	L.Asmari	61-71	Good
Marun	U.Asmari	51-67	Fair- Good
	M.Asmari	56-71	Fair- Good
	L.Asmari	59-76	Fair- Good
Seymareh	U.Asmari	52-69	Fair- Good
	M.Asmari	56-74	Fair- Good
	L.Asmari	56-61	Fair- Good
Salman Farsi	U.Asmari	25-42	Weak- Fair
	M.Asmari	50-67	Fair- Good
	L.Asmari	25-40	Weak

6.10. Foundation Consideration

The entire area under the dams should be excavated to sound, fresh rock. The depth of the planned foundation excavation has been determined by topographical considerations and is well below the average depth of the weathered zone. As the dams will rest on the Asmari Formation and considering this formation is weathered and crushed at surface, it is estimated the depth of the materials to be removed will range from 1m to 10 m and on average 3-5 m. The thickness of the alluvium to be removed from the river bed commonly ranges from 25 to 50 m. After the removal of weathered rock, the foundation will rest on sound rock. In addition to the above mentioned, attention should also be paid to the dam abutments. In accordance with exploratory adits and also the drilling investigation borehole data, there are some low strength beds which have been eroded previously and then filled by secondary materials. These materials should be removed from these locations. Filling materials along the faults (brecciated zones and gouge materials) at foundation rocks should be replaced by suitable concrete aggregate.

Dental excavation (so rough) of shear zones and weathered rocks should also be performed. Such areas must be backfilled with concrete as necessary. Detached block of rocks should be removed or rock bolted and or grouted. Rock overhangs must be trimmed and a regular surface formed. The side walls of the foundation excavation should be cut taking slope stability analysis into consideration preferably no more than *natural gradient* of the rock slopes. In some areas, due to the presence of unstable rock blocks and wedges, pattern bolting will be required, although local blocks or zones may require support in the form of rock bolts, shotcrete or steel wire mesh. The occasional use of high tensile strength rock anchors in some of the higher cut slopes should be considered.

6.10.1. Grouting

6.10.1.1. Consolidation Grouting

Consolidation grouting should be performed over the whole area of the dam foundation. The consolidation holes should be drilled on a 3 m by 3 m grid pattern and should extend to a depth of 10 m below the foundation. The orientation of the holes will be such that they intersect the majority of discontinuities.

6.10.1.2. Curtain Grouting

Grout curtains are critical components of the dams constructed on slightly karstic bedrock foundations. In this geological environment such as the Asmari formation limestone, grout curtains are more extensive and require much higher volumes of cement than is normally the case in other rock types (Acres, 1982).

The grout curtain should be extended to over 100 to 150 m below the base of the dam foundation and over 200 m into each abutment (depending on karst development). A multiline curtain, comprising 2 to 3 rows of holes, should be installed in the medium to high permeability limestone in each abutment and beneath the dams. High grout takes can be anticipated in this part of the grout curtain. The grouting will be performed mostly from tunnel galleries and designed according to the size of drilling equipment. Grout holes will be approximately 50 mm in diameter and have average spacings of 3 m, although spacings as low as 1.5 m can be expected at some localities. The holes will be oriented such that they intersect the maximum number of bed rock discontinuities.

6.10.2. Treatment of arge Caverns

All large cavities along the grout curtain alignment or close to the grout curtain will have to be plugged with concrete, and the plug structures have to be connected to the curtain. An acceptable uniform model for treatment of large karst voids does not exist.
Total filling of huge caverns with concrete should be avoided due to economical reasons. Narrow parts in the rock mass (karst channels) have to be explored by dental investigations for determining the best way for plugging (JV. Stucky/Electrowatt, 1992).
The general guidelines for the plugging technology above the water table are:

1. Provision of access adits and shaft excavations from the main grouting galleries up to the karst channels and cavities.
2. 3D geological and speleological mapping for the purpose of treatment decisions.
3. Drilling into inaccessible (small diameter) karst voids, including geophysical investigations.
4. Preparation of rock materials such as pebbles and boulders for partly filling of accessible caves
5. Preparation of concrete pumps for concrete injection.
6. Preparation for rock bolt installation.
7. Provision of contact grouting after plugging operations.

The technology for large karst cavity treatment below the water table, however, without using any back pressure (mostly from the lowermost galleries) needs large diameter drill rigs, with preventers, operating from the grouting galleries. This is supposed to be a pure filling process, without the use of packers.
Previous exact mapping with the provision of contours of every large cavity has to be provided using small diameter rotary drilling and selected geophysical investigation methods (JV. Stucky/Electrowatt, 1992).

6.11. Construction Materials

6.11.1. Granular Materials

Test pits have proven large amounts of very weakly cemented granular material in some areas around the dam sites such as main and seasonal river bed alluvium. The test pits show mostly stratified gravel and sandy gravel with minor amounts of silt and occasional cobbles and boulders. Particles are rounded to subrounded and mostly composed of limestone. Most particles have surface discoloration and in some cases alteration extends up to 1 mm into the rock surface.
Sodium sulphate soundness tests indicate less than 10% losses after five cycles for materials in some of the areas investigated and between 10% and 40% in some other areas. The specific gravity ranges from 2.6 to 2.7. Fine material passing the No. 200 mesh (0.074 mm) varies from 1% to 85% and gravel size from 0% to 95%.

6.11.2. Excavated Rocks

The use of excavated rocks or rocks from quarries is assumed to be the main source for concrete aggregate. Assessment of suitability was based only on visual examination of rock cores and excavated rock from adits, service record of similar rock at other structures, petrographic examination and laboratory analysis such as alkali aggregate reaction (AAR).

According to the lithological columns shown in Figure 5.4, each unit which contained higher percentages of interbedded marlstone, shale, marly limestones and dolomitic limestone should be rejected for the concrete aggregates. These rock particles are often elongated with sharp edges in addition to a platy shape and have a high potential for deterioration due to low strength. In this regard it refers to the upper Asmari in Karun-3, lower Asmari in Salman Farsi and Seymareh dam sites due to a high concentration of argillaceous mineral content.

The dolomitization of limestones can be observed in all dam foundation rocks. This phenomenon varies from slightly to intensive dolomitization. For example the middle Asmari unit at Seymareh and Salman Farsi dam sites are relatively influenced by high dolomitization but at the other sites dolomitization is only observed locally.

It is generally believed that alkali-carbonate reaction occurs between certain argillaceous dolomitic limestones and the alkaline pore solution in the concrete.

Alkali-aggregate reaction is a *chemical reaction* between certain types of aggregates and hydroxyl ions (OH-) associated with alkalis in the cement. Usually, the alkali comes from the Portland cement but it may also come from other ingredients in the concrete or from the environment. Under some conditions, the reaction may result in expansion and cracking of the concrete. Concrete deterioration caused by alkali-aggregate reaction is generally slow, but progressive (Shrimer, 2005).

Cracking due to alkali-aggregate reaction generally becomes visible when concrete is 5 to 10 years old. The cracks facilitate the entry of de-icing salt solutions that may cause corrosion of the reinforcing steel, thereby accelerating deterioration and weakening a structure (Shrimer, 2005).

Finally, the suitability of the Asmari limestone for use as concrete aggregate must be confirmed by further laboratory testing. Samples should be tested for gradation, absorption, specific gravity, sulphate soundness and Los Angles abrasion characteristics. Petrographic analysis should be carried out and it will be necessary to check concrete aggregate for the presence of deleterious constituents. The program was initiated but testing for alkali-carbonate reaction is a long term process.

6.11.3. Impervious Fill

Adequate quantities of impervious fill, suitable for cofferdam construction have been located in the area of the dam sites. Test pitting showed these materials to be stratified, stiff, moderately plastic silty clay with some silty sand bands. Adequate impervious materials are available at each dam site. Laboratory testing carried out on this material considered the moisture content, Atterberg limits, proctor density, specific gravity and mechanical gradation.

Sand content varies from 10% to 35%, the specific gravity is 2.6 and the average proctor maximum dry density is 1790 kg/m^3. The average optimum moisture content of 15% is higher than the natural moisture content which varies from 4.5% to 13%.

6.12. Reservoir-Induced Earthquakes

In recent years, there have been many examples of small and medium sized earthquakes occurring beneath or adjacent to recently filled reservoirs. Classic cases are the Koyna dam in India, Kariba in Africa, and Krenasta in Europe (Campbell, 1981, Acres, 1982). Lesser cases have occurred at *Bajina Basta* in Yugoslavia, Sringagarind in Thailand and elsewhere. There is often a statistical correlation between the depth of water in the reservoir and the rate of occurrence of foreshocks which generally precede these earthquakes, and this correlation is assumed to signify a relationship between the filling of the reservoir and the occurrence of seismic events (Campbell, 1981, Acres, 1982).

The mechanism by which filling of a reservoir might induce the occurrence of earthquakes is not fully agreed by all authorities on the subject, but there is a general consensus that such events can only happen in regions such as the Zagros belt which is subjected to significant tectonic stress at the time the reservoir is filled. It is assumed that these stresses are released or partially revealed by fault movement at depth during earthquakes, and that the filling of the reservoir serves as a trigger to permit such movement on the fault (Acres, 1982).
The most likely explanation is that the raising of the water level causes an appreciable increase in pore pressure in the rock beneath the reservoir, and that this increase in pore pressure causes a decrease in the effective stress within a pre-existing plane of weakness in the rock, such as a fault.

The resistance to shear stress is decreased and movement can occur in response to the forces which are acting on such planes of weakness. Assuming such an explanation is valid, then reservoir-induced seismicity could only occur in areas where pre-existing tectonic stresses are of appreciable magnitude and where the new reservoir is of considerable depth. Most cases of reservoir-induced seismicity occur in reservoirs at depths greater than 100 m (Campbell, 1981, Acres, 1982). It is not yet possible to predict with any degree of reliability whether a particular proposed reservoir will induce the release of tectonic stress in the form of earthquakes. However it is possible to say that the more than 160 m depth of the reservoirs in the area of research (especially at Karun-3 and Karun-4 with reservoir depths over 180 m) will have many of the characteristics common to such cases.
In most cases of reservoir-induced seismicity, the shocks are of relatively small magnitude. However, there have been a few events, notably at *Koyna* and *Kariba*, of magnitude 6.5 or possibly even 6.8. No reservoir-induced earthquake larger than this has ever been recorded (Campbell, 1981; Acres, 1982).

6.13. Conclusion and Recommendations

In general the following can be concluded based on the geological investigations in the area:

I. Since Pliocene time the tectonic history of the investigated area produced intense folding and thrusting of the outcropping sedimentary rocks. The continuous convergence of the Arabian and Central Iran on Plate causes an *uplifting* of the belts estimated to be about 1 mm/year (deep river valleys, higher alluvial terraces and fossil beaches and uplift of historical channels) and results in intensive seismic activity caused by basement high angle reverse faults. With regard to the magnitude of the destructive earthquake a reactivation of folding-related discontinuities is likely to occur. It was concluded that events larger than magnitude 7 Richter are not expected in the Zagros seismotectonic province. Publications since the original 1978- 1979 Acers/ Appolonia study have confirmed this. The data published by Ambraseys and Melville (1982) indicate that all events in the Zagros seismotectonic province are of magnitude 6.8 or less.

II. The Asmari Formation is of Oligo-Miocene age and comprise lithologically massive/thick to medium bedded grey to light grey limestone, dolomitic limestone, marly limestone, marlstone with thin interbeds of limestone and shale. Petrographical analyses indicate *Intrabiomicrite* to *Biodolomicrite*, *Wackestone* to *Packstone* except, in Salman Farsi which indicates locally *Biointrasparite*, *Grainstone*. Moreover, dolomitization (*Dolosparite*) is locally well developed in the middle part at Salman Farsi and Seymareh dam projects. This succession mainly constitutes the dam foundation rocks and based on engineering geological aspects is commonly divided into three rock units. Each of these has different strength and rock mass properties.

III. The minimum/ maximum porosity values of the Asmari formation limestone, according to microscopic quantitative method/point-counting analysis, are between 0.3% to 15.7%. In general the porosity values based on the Cherenyshev/Dearman Classification indicate Medium *to* Extremely High porosity in the Asmari formation limestones. Total porosity values of 35% and 13% are related to the Fracture/Channel porosity types on the southern and northern flank sites respectively. This clearly explains much more tilting (70°- 90°) or more curvature of strata in the south-western flank of anticlines resulting from tectonic movements as well.
Asmari Formation limestone has been affected by karstification process and the caves, stalagmites, stalactites, karstic channels and enlarged fissures are influenced by aggressive water dissolution. Karst chimneys with apertures from a few centimetres to a few metres have most frequently been detected. In addition, large caves with volumes of thousands of cubic metres are present in some areas. The chemical compositions of spring waters are *calcium sulphate, sodium chloride* and *carbonate*. These compositions obviously indicate that, there are *hydraulic connections* between the *Asmari* and *Gachsaran* formations with high karstification as well although the field tests by MG. Co. (1984, 2003) do not confirm this connection! The closed depression caves, sinkholes and collapse sinks in the Gachsaran Formation due to chemical and physical dissolution of evaporite rocks are well developed. The Asmari Formation limestones may be influenced by active mineral solutions originating in the Gachsaran Formation and this is the main factor for the development of karst features in the Asmari limestones especially in upper and middle parts. The interaction between the future reservoir and the karst at the dam sites can be quantitatively appreciated by extensive injection and tracing tests. The acquired results will help in the design, to appreciate the costs and test the efficacy of the *grout curtain*. The water tightness of the dam sites and reservoirs should be investigated in more detail to recognize karstified zones in the Asmari Formation and Fars Group, especially the Gachsaran Formation, in addition to contact zones between the two formations.

IV. Tectonic conditions in the area have caused the creation of different discontinuities in the rock mass. These discontinuities include fissures, joint sets, major joints, fractures and faults in addition to the bedding planes. Fracture systems in the Asmari Formation were investigated from exploratory boreholes and surface fracture studies. The similarity between the fracture density and curvature rate of strata at construction sites coincide well with the asymmetrical fold structures with different curvature rates in the *Zagros* region. It can also be expected that there is a direct relationship between *fracture intensity* and *curvature rate* in fold structures in the region. This is obviously indicated by the RQD, RMR and GSI values. Due to compressional stress, more reverse faults and tectonic disturbances can be observed in the southern flanks of fold structures such as at Karun-3 and Karun-4 dam sites.

V. The dip direction of the Asmari limestones at two dam sites at the northern and southern flanks are almost perpendicular to the dam valleys and will produce two different conditions of adjusting factors according to the main discontinuity orientation, if the bedding planes are considered to be the main discontinuity set. Adjusting factors for the stability (R_{STA}), according to joint orientation (Romana,

2003) for the northern flanks are fair to very favourable but in the southern flanks, where the *Karun-3* and *Karun-4* dams are located, this factor is a fair condition.

VI. Generally, the stability of a reservoir rim depends on parameters, such as reservoir water level, the nature of formations which have most contact with the reservoir water and their dip with respect to the reservoir. Planar and rotational sliding of rocks normally after impounding of reservoirs can be expected. These instabilities in the rock mass commonly occur around the reservoir walls, but only deep seated sliding surfaces can produce destructive hazards at dam projects. Therefore identification of such cases, in addition to the provision of a landslide hazard zonation (LHZ) at reservoirs will be necessary (Anbalagan and Gopta, 1995). It should be stated that the potential of sliding on the contact between the Asmari and Pabdeh formations after reservoir impounding, will increase, where the Asmari limestone commonly constitutes high angle cliffs around the reservoir due to its rigidity. The highly permeable Gachsaran/Razak formations (mainly evaporites and marl) will also be highly prone to instability due to tectonic disturbances, solubility and high flexibility of the rock mass. The sliding failures in this case will commonly be *circular or rotational failures*. Water absorption by interbedded marls after reservoir impounding will influence rock sliding towards the reservoir. In this regard, the active tectonism of the region can easily activate and trigger such sliding. Furthermore, all types of rock mass failure such as wedge, toppling; planar failures and rockfalls, adjacent to the dam locations are expected. Slope stability analysis indicate *Unstable to Partially stable* of rock slopes on both flanks of the dams.

Forcasting of rock slope instability by instrumentation and monitoring/remote monitoring will be needed especially at the dam walls. Obtaining accurate measurements of the rock face can be a major challenge when assessing risk on very large rock slopes, which are often difficult to access and potentially dangerous. However traditional discontinuity measurements such as scanline, cell mapping and geologic structure mapping have several major disadvantages (Priest and Hudson, 1981, Priest, 1993, Hack, 1998). Conventional techniques, such as vertical aerial photography and extrapolation from topographic maps, provide very poor data sets due to the small footprint of a steep slope. Some success have been achieved with oblique photography, but this approach requires considerable post-processing based upon large numbers of tie points.

Impact sentinel sensors check the status of rockfall protection systems and set off an alarm if limit values are exceeded. Hence, potential accidents involving personal injuries or economic damage can be effectively prevented. This system is specifically designed for difficult access places where wiring or power supply is not available and where it could only be implemented at great expense. Impact sentinel can be used permanently, for instance, in remote areas or temporarily, to help secure construction sites. (Geobrugg AG. Protection system, Impact Sentinel- Remote Monitoring of Rockfall Barriers. 2009).

More recently Terrestrial Laser Scanning (TLS) has provided a method of rapidly capturing morphological data. TLS instruments are designed to record surfaces under a wide range of environmental conditions and can operate at ranges of up to about 2,000 m (Nagihara et al. 2004). Aoki et al. (1997) report using TLS to monitor volcanic cone deformation; Nagihara et al. (2004) for the morphometric analysis of sand dunes; Fardin et al. (2004) for rock surface roughness and Rowlands et al. (2003) for landslide analysis.

VII. As the dams will be founded on the Asmari Formation and considering this formation is weathered and crushed at surface, it is estimated that the depth of the materials to be removed ranges from 1m to 10 m with an average of 3-5 m. The thickness of the alluvium to be removed from the river beds commonly ranges from 25 to 50 m. After the removal of weathered rock, the foundation will rest on sound rock. In addition to the above mentioned, attention should also be paid to dam abutments. Exploratory adits and drilling investigation borehole data, indicate some low strength beds which have been eroded and then filled in by secondary materials. These materials should be removed and the fill materials along faults (brecciated zones and gouge materials) in the foundation rocks should be replaced by suitable concrete aggregate.

VIII. In general, the geological and geotechnical investigations of the Asmari Formation limestone showed the rock to be fairly suitable foundation material for dam construction in the Zagros region. According to geological assessments (Table 6.5 and Figure 6.18) it can be concluded that the engineering rock mass conditions at the Karun-3 and Karun-4 dams are;

- *Blocky- Very Well Interlocked* and *Good (B/G)* to
- *Blocky Disturbed/Seamy* and *Fair (BD/F)*

The GSI values are between 35 to 65.

The engineering rock mass conditions at the Marun and Seymareh dams are;

- *Blocky- Very Well Interlocked* and *Good (BG)* to
- *Very Blocky- Interlocked* and *Fair (VB/F)*

The GSI values are between 45 to 70.

In the case of Salman Farsi however it is relatively variable due to extensive development of marlstone, marly limestone with thin interbedded limestone in the lower and upper units on the one hand and extensive development of dissolution and karstic features on the other hand. This results in a wide range of GSI values from;

- *Blocky- Very Well Interlocked* and *Good (BG)* to
- *Blocky Disturbed/Seamy* and *Poor (BD/P)*

The GSI values are between 25 to 65.