Restoration of Dam Components Subjected to Dynamic Loads Using Geosynthetics: A Case Study of Ukai Dam, India

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Abstract. India has been known for having a large number of multipurpose dams. Some dam components are required to be designed for taking hydrodynamic loads. Divide bund or guide bund separating flows from the spillway and the hydropower turbines needs to have an armor that can take care of cyclic loads. In majority of old dams, such protective armors used to be in the form of rubble mound or pitching of suitable thickness. Their deterioration under the effect of trains of water waves required a mandatory periodic repair. Divide bund of Ukai dam of India used to require heavy repairs. It was restored using geosynthetics instead of conventional materials. As a flagship task, its design required to be founded on principles and approaches applied to some other kinds of problems. Promising results of this pilot project have prompted documenting experiences so as to encourage application of geosynthetics in designs of many parts of the dams subjected to hydrodynamic loads and hence this paper.

Keywords: Dam, Divide Bund, Dynamic Load, Geosynthetics, Water Waves

1 Ukai Dam and Its Divide Bund

1.1 Salient Features of Dam

India has built more than 5400 dams in last 75 years out of which many are over 50 years old [1]. Ukai dam was built on Tapi river in 1972 in Songadh district of Gujarat State of India. Tapi is the second largest west flowing river of India with 724 km length. Earthen embankment on left side of the dam is 1562.87 m long and on right side is 2495.99 m long and in between is the spillway towards the left. Earthen embankments are zoned rolled sections and the spillway is gravity type composite section with rubble masonry with concrete encasing. Spillway and power dam together span 868 m of length. Ogee spillway has a trajectory bucket at its end. The spillway is equipped with 22 radial gates of 15.444 m x 14.782 m size each. River bed is of basaltic rock and its average level is 50.30 m R.L. and the Highest Flood Level is 106.985 m R.L. with design flood of 59,917.88 m^3/s. Gross storage potential of the dam is 7414.29 million m^3. Discharge capacity of the spillway at Full Reservoir Level is 37, 865 m^3/s. River

Bed Power House has 4 units of 75 MW each which are operable with maximum head of 57 m and minimum head of 34 m.

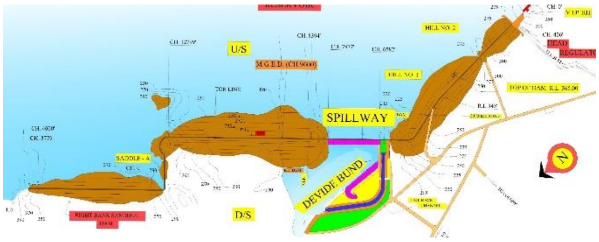

Fig. 1. Layout of Ukai dam

1.2 Divide Bund and Its Distress

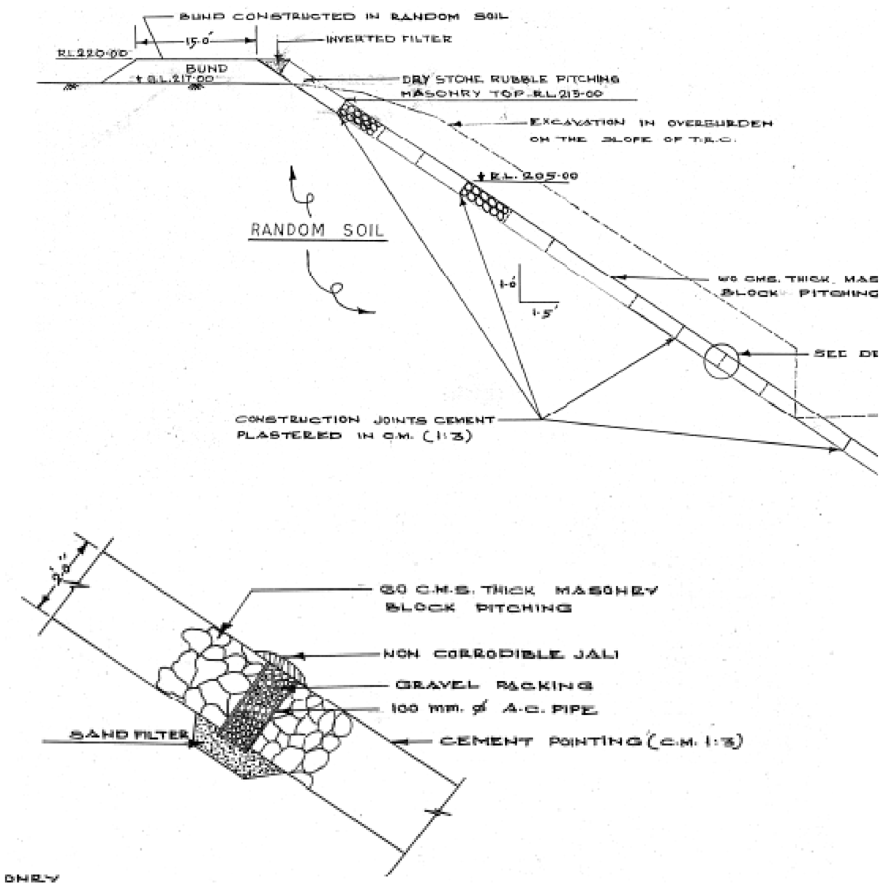

Fig. 2. Profile details of divide bund and armor

Alignment of divide bund of the Ukai dam is having an inclination of 45° and side slope of the bund is 1:1.5 (V:H). It is made of random soil which is predominantly intermediate clay. The armor is 60 cm thick rubble masonry with cement mortar of 1:3 nominal mix. Pore pressure release is facilitated by drainages. Intermediate berm and toe are also properly designed and keyed in to the river bed. The profile was designed with 3D model study as the divide bund was supposed to act as a guide bund during release from the spillway (see Fig. 2).

At various locations the armor manifested distress and necessary repairs were carried out periodically. Once in 2006 the flood release was as much as the design flood value and that caused a major damage to the armor. The repairs whatever was done then could not as good as the original construction. As such heavy releases during monsoon is a routine for the Ukai dam and hence subsequent years witnessed increasing damage. The repairs were taken up but could not serve the purpose. Nosing stretch of about 100 m length was found severely damaged. Not only disintegration of rubble masonry but also pulverization of stones was observed. In 2017, the distress was significant (see Fig. 3). Earthen slope of the bund was found disturbed. The berm and the toe were badly eroded and there was an entrenchment found in the rocky riverbed near toe which was about 5 m deep and 8 to 10 m wide which could endanger the stability of the bund. Earlier observations indicated damages and they were repaired, but, in 2017, the site inspection revealed alarming damage requiring enhancement of hydraulics besides repairs of the damaged portions.

Fig. 3. Divide bund after distress

2 Forensic Aspects

Repetitive need of repair of the armor of the divide bund, particularly in the terminal stretch of about 100 m i.e., near nosing, and, the level of distress posed the requirement of reviewing the repair strategy and exploring the scope for enhancement in hydraulic behavior of the flow. Detailed site observations in various flow conditions and revalidation of design of armor in light of various standards, guidelines and other literature provided some insightful learnings which are summarized as follows:

1. Ogee type spillway with trajectory bucket has been provided in Ukai dam considering basaltic rock as geological formation of the river bed. All bucket types of energy

dissipators require a guide wall or a training wall to ensure conducive hydraulic conditions in the downstream. Site-specific conditions required the divide bund to act as a guide bund as well so that the release from the spillway could be redirected. As a guide bund, it was designed as a non-overtopping section requiring no crown wall. Rubble pitching with grouting was preferred as its armor keeping in view the requirement of smooth outer profile that could deflect and divert the flow rather than to act as a wave breaker. River bed was of basaltic rock and hence bucket type energy dissipation was provided.

2. In trajectory bucket, the flow coming down the spillway is thrown away in air from the toe of the structure to a considerable distance as a free discharging upturned jet which falls on the channel bed downstream. The hard bed can tolerate the spray from the jet and erosion by the plunging jet would not pose any significant problem for the safety of the structure. Thus, although there is very little energy dissipation within the bucket itself, possible channel bed erosion close to the downstream toe of the dam is minimized. In the trajectory bucket, only part of the energy is dissipated through interaction of the jet with the surrounding air. The remaining energy is imparted to the channel bed below. The design of the trajectory bucket presupposes the formation of large craters or scour holes at the zone of impact of the jet during the initial years of operation [2]. Over a period of time, roller formation is facilitated by the scour holes [3]. Scour holes had occurred in this case in the river bed, but, they being at intermittent locations, the bucket formation could not occur uniformly and hence the downstream dynamics remained a challenge.
3. In this case, divide bund cum guide bund has been provided with an inclined alignment (in plan with angle of inclination of 45°) which negotiates the water way facilitating safe passage of turbulent flow. Spillway operation is always done in a systematic way and the release is gradually increased as a strategy for modulation of flood. Therefore, flow trajectory occurs during initial stage (see Fig. 4), and, subsequently, when the release is greater than certain value, roller action takes place in downstream of the bucket (see Fig. 5). Over a period of time, scour hole is developed in the river bed which facilitates better roller occurrence and resembles the hydraulic behavior of a roller bucket type energy dissipation. In both the situations i.e., jet occurrence and roller occurrence, the turbulent flow suddenly gets unbounded once it crosses the nosing of the guide bund. Nosing of the bund is subjected to the combined effect of sudden change in hydraulics and deflection of flow.

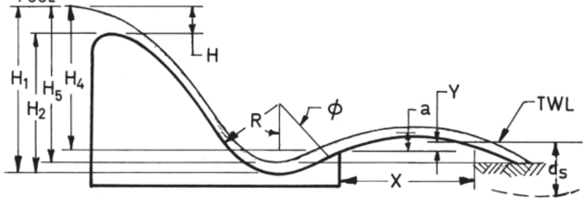

Fig. 4. Trajectory type bucket [2]

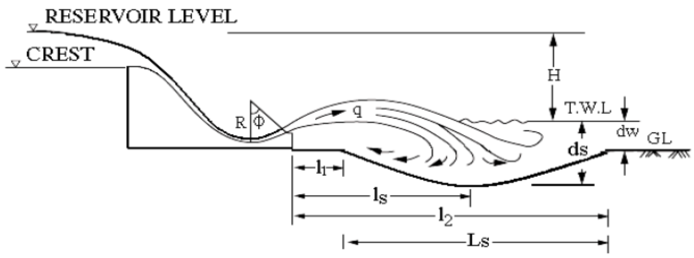

Fig. 5. Scour hole and roller formation in downstream of bucket [3]

4. Important aspects of design of armor are the size of individual stones and the thickness of the armor. Methods for designs of armor for breakwater and river training work are based on the said aspects considering free flow condition. In this case, transition of flow conditions in a short length needs to be separately accounted for.

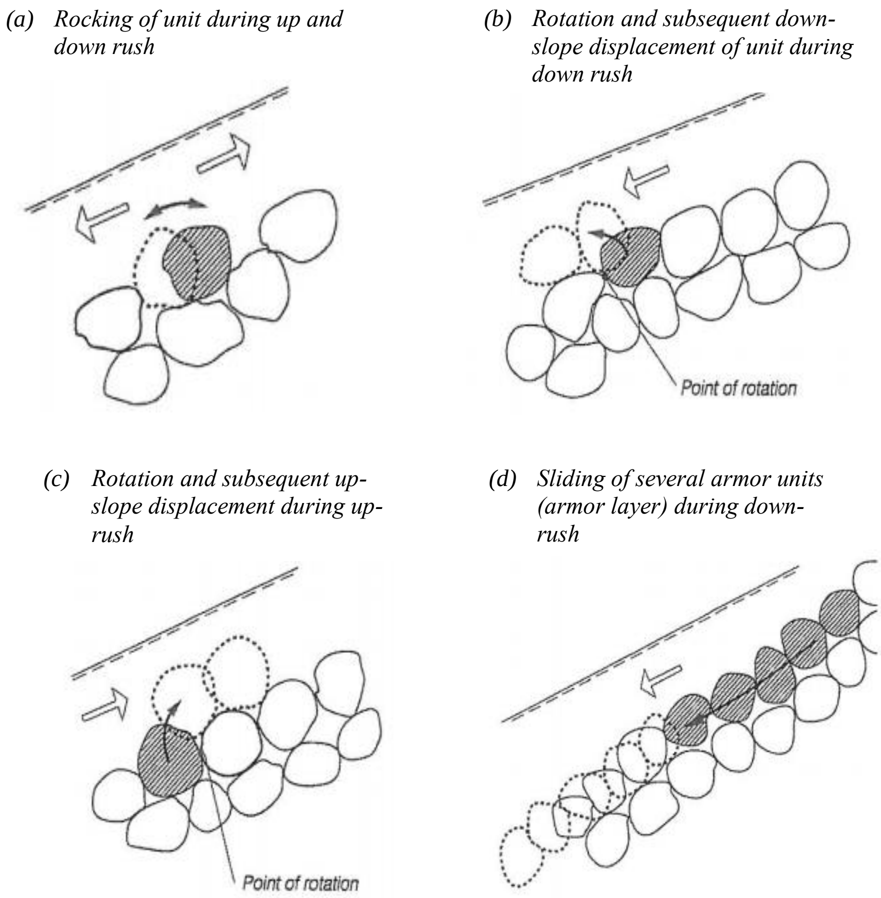

Fig. 6. Typical armor layer failure modes [4]

5. Observation of distressed armor suggested disintegration of masonry followed by pulverization of some of the stones. Generally, strength of cement mortar is much lower than that of the rubble and hence it gets disintegrated due to impact and then individual stones are subjected to the wave effects. As compared to the rubble mound armor, rubble masonry contains smaller stones and therefore the process of breakage and dislodgment of stones becomes faster once the mortar loses its strength. Stones of the armor without mortar bond may be displaced in different modes under hydro-dynamic loads [4] (see Fig. 6).
6. Bureau of Indian Standards and Indian Road Congress [5,6,7,8] have published guidelines for design of armor and slope protection work for guide bunds following international practices. Basic concept suggests stability of stones to be taken as a ratio between drag plus lift force and gravity force [4].

$$\frac{F_D+F_L}{F_G} \approx \frac{\rho_w D_n^2 v^2}{[g(\rho_s-\rho_w)D_n^3]} \approx \frac{v^2}{g\Delta D_n} \quad (1)$$

where $D_n = (armor\ unit\ volume)^{\frac{1}{3}}$, ρ_s and ρ_w are the mass densities of armor units and water, respectively, and v is a characteristic flow velocity. By inserting $v \approx (gH)^{\frac{1}{2}}$ for a breaking wave height of H, the following stability parameter, N_s is obtained.

$$N_s \approx \frac{H}{\Delta D_n} \quad (2)$$

where $\Delta = \rho s /(\rho w - 1)$. Non-exceedance of instability, or a certain degree of damage, can then be expressed in the general form.

$$N_s = \frac{H}{\Delta D} \leq (K_1^a K_2^b K_3^c) \quad (3)$$

where the factors depend on all the other parameters, except H, Δ and D_n, influencing the stability.

Balance of forces on armor is estimated by various researchers.

$$\frac{H}{\Delta D_n} = Kcos\alpha \text{ (due to Svee)} \quad (4)$$

$$\frac{H}{\Delta D_n} = K(cot\alpha)^{\frac{1}{3}} \text{ (due to Hudson)} \quad (5)$$

$$\frac{H}{\Delta D_n} = K(tan\varphi\ cos\alpha - sin\alpha) \text{ (due to Iribarren)} \quad (6)$$

where φ is the angle of repose of the armor. The coefficient K includes some level of damage as well as all other influencing parameters not explicitly included in the formulae.

Indian design practices recommend size of individual stones as following [4]. It is the diameter of a round stone which may be suitably modified for angular quarry stone. Mechanical locking obtained due to angular shape of quarry stones may improve the actual performance.

$$D = Kv^2 \quad (7)$$

Where, D = equivalent diameter of stone in meter

$$K = \frac{\left(\frac{3}{2}\right)C_d\left(\frac{k_v^2}{k_0}\right)}{\{2g(S_s-1)\}} \quad (8)$$

where C_d is coefficient of drag which varies between 0.25 and 0.5; S_s = Specific gravity of stones; k_0 = slope factor = $(cos^2\theta\ tan^2\varphi - sin^2\theta)^{0.5}$; θ = angle of inclination of the side slope with horizontal; φ = angle of internal friction; k_v = velocity factor = $\frac{v_s}{v} = 0.9$ where v_s is the surface velocity above stone surface and v is mean velocity of flow near bank.

Thickness of the armor t is designed as following.

$$t = \frac{v^2}{[2g(S_s-1)]} \quad (9)$$

Standards for Indian design practices provide similar design methodology for slope protection work as following [6,7].

$$W = \left[0.02323\frac{S_s}{K(S_s-1)^3}\right]v^6 \quad (10)$$

where

$$K = \left[1-\left(\frac{sin^2\theta}{sin^2\varphi}\right)\right]^{0.5} \quad (11)$$

$$D_s = 0.124\ \sqrt[3]{\frac{W}{S_s}} \quad (12)$$

Where W = weight of stone in kg; Ss = specific gravity of stones; θ = angle of repose of protection; φ = angle of sloping bank; v = velocity in m/s and D_s = size of stone (minimum dimension of stone).

Thickness of the armor t is computed as per Eq. 9 above.

Table 1. Weight of stone and thickness of armor for different velocities

v (m/s)	D_s (m)	D_1 (Size of Angular Stone) (m)	t (m)	W (kg)
3.5	0.30661	0.368	0.3677	40.73
4	0.40047	0.481	0.4797	90.75
4.5	0.50684	0.608	0.6071	183.98

Original design of the armor (see Fig. 2) was revalidated through the above provisions, which came in to effect much later, for different flow release conditions and was found that the armor thickness was suitable for velocity up to 4.5 m/s.

It is recommended using safer of the outcomes obtained from Isbash and Iribarren formulae. It is also suggested to use stone-crates or masonry lining with computation of thickness as follows [8].

$$d = \left(\frac{\gamma_w}{\gamma_p}\right)\left(\frac{v^2}{2g}\right)\left\{\frac{1+S^2}{S}\right\}^{0.5} \quad (13)$$

where d = thickness of lining i.e. masonry; γ_w = specific gravity of water; γ_p = specific gravity of pitching material; v = velocity of flow and S = slope of bank expressed as S (Vertical) : 1 Horizontal.

An important finding is that the size of individual stones and thickness of the armor are almost same for the outer slope of 1.5:1 (H:V). Practically keeping same dimensions for stone and masonry thickness are not viable. Velocity at bucket invert is 32.84 m/s and the throw distance is 101.9 m at critical discharge. Velocity in the tail channel is more than 8 m/s and uneven scouring of the river bed does not allow uniform roller occurrence. In such a critical situation, the armor is required to be specially designed in the nosing portion. Issue of bed scoring and foundation endangering as a case study of Ukai dam exhibits concern [8] which suggests that this guide bund has become a source of learning for making standards. Slope 2:1 or more gentle is found to be better suited with the present hydraulic condition but its practicability is an issue at this stage.

3 Convergence to Solution

Results of methods recommended by all Indian Standards and Guidelines are almost same as their approaches are same and based on international practices which are mainly depending on velocity. Velocity for design of breakwater is estimated from the fetch length because wind-induced-waves dominate the hydraulic behavior; but, in this case, velocity is due to sudden drop in potential energy. All methods for design of armor are in general for dry revetment whereas the existing design of the bund is with stone masonry. However, cement concrete blocks or crated stones are recommended when the velocity is greater than 3.5 m/s [6]. No exclusive design method for concrete blocks or panels has been provided therein and hence the same size of individual concrete blocks are supposed to be taken if this method is applied. Considering all these limitations, a detailed approach to design of armor was resorted to for restoration of nosing portion of the guide bund.

3.1 Design for Restoration: Preliminary Issues and Outlining Solution

Constructability and execution. Present slope of the divide bund i.e. 1.5:1 (H:V) has been based on a 3D model study which could not be changed. Borrowing good soil and spreading it on the damaged outer surface of the earthen bund and preparing subgrade involved difficulty in compaction. Design of armor with the same type of rubble masonry required same value for size of rubble and thickness of armor. Therefore, either use of smaller stones to attain requisite thickness of masonry armor or to use requisite size of stones and increasing thickness of armor were the options to be explored. Carrying heavy rubble upslope was a difficult and time taking task. River condition permitted workable period of hardly four months a year. Completing restoration in so short a period requires proper planning and activity scheduling but still meeting time line was a challenge. Working in piece meal every year and taking a few years was not advisable considering chance of requirement of controlling heavy flood.

Bottom key for stability. Entrenchment at the toe of the divide bund, its filling up, securing it against future erosion along with keying of sloped armor and ensuring stability of the earthen embankment were major concerns. River bed profile was required to be used such that the bottom key would help make desirable force resolution.

Proposition of multilayer mechanism. Rubble masonry was vulnerable with original dimensions under high velocity flow condition. With reworked dimensions it was very difficult to place heavy stones on a steep slope. Therefore, alternative proposition for solution was required to be explored. A multilayer composite system involving materials with various stiffnesses so as to avail better load dispersal was found worth exploring. The outer layer was conceptualized as a rigid layer of concrete blocks, the intermediate layer as semiflexible using gabions and the bottom layer as flexible using geosynthetics. Estimation of impact loads, deciding design methodology and fairly estimating the material properties were the challenges. Gleaning various concepts from different theories and researches and translating them in to design was not easy as no tailormade procedure could be availed. However, oceanic conditions of water dynamics and riverine ones have some parallels which could be used for firming up the design procedure.

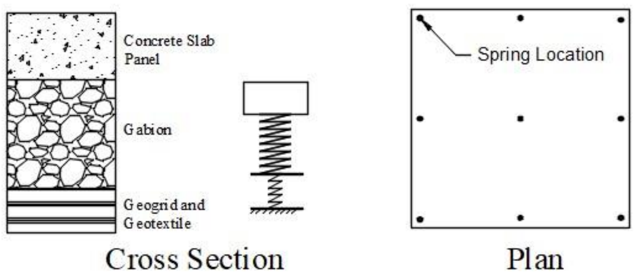

Fig. 7. Armor section and its idealization as Multi Degree Freedom System (MDOF)

Muti-layer solution conceived here was derived from principles of flexible pavement design with geosynthetics in which geosynthetics are used for three functions – lateral restraint, bearing capacity increase and membrane tension support (See Fig. 8). Instead of dynamic wheel load, here it would be water wave impacts. Asphalt surface layer on the top would be replaced by concrete slab panels of 3 m x 3 m x 0.5 m each, granular base course would be replaced by 2m x 2m x 2.5 m gabions and biaxial geogrid (tensile strength = 40 kN/m) reinforced subgrade would be taken as it is (see Fig. 7).

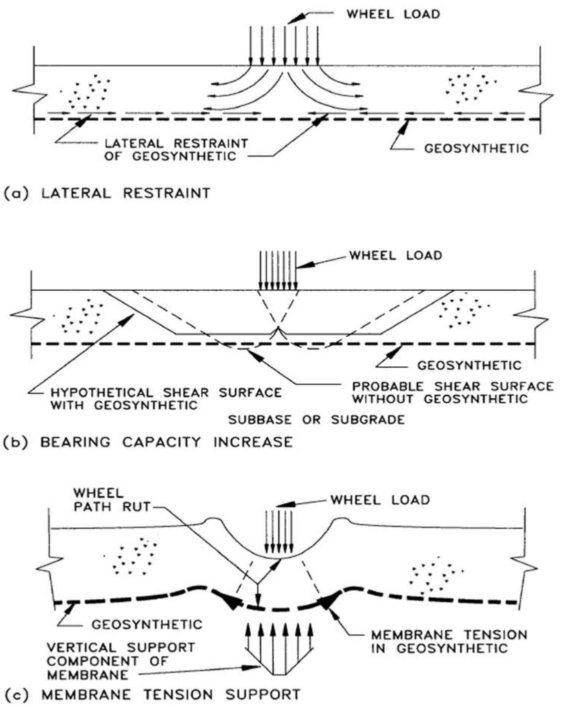

Fig. 8. Functions of geosynthetics in subgrade stabilization [9]

3.2 Design Propositions

In oceanic and riverine conditions, wave striking with breakwater or guide bund is the cause of impact transfer in to the armor. However, wave energy in oceanic condition is due to wind velocity and fetch length whereas in riverine conditions with a dam in the upstream, wave energy is due to sudden drop of water from a significant height. During initial release from a dam, water depth in the downstream of spillway is very low and the flow tends to induce heavy wave impacts on the armor of the guide bund especially down the trajectory which are generally much greater than pulsating loads.

In any investigation of dynamic forces, the prime relationship is Newton's Second Law [10].

$F = m.a = m\left(\frac{dv}{dt}\right)$ (14)

Integrating F with respect to time, the law of impulse-momentum is derived.

$\int_{t1}^{t2} F\,dt = \int_{t1}^{t2} m\,.\,a\,dt$ (15)

$\int_{t1}^{t2} F = mv_2 - mv_1$ (16)

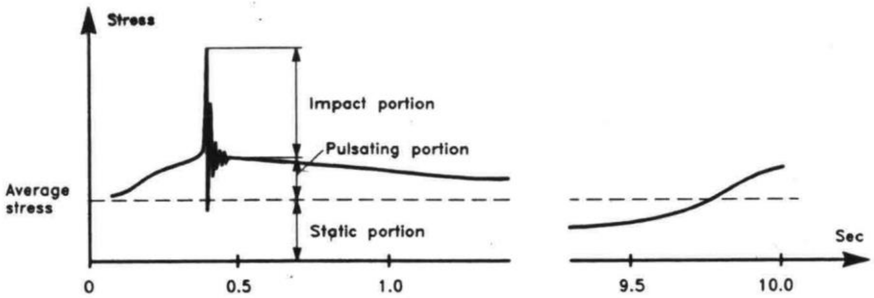

Fig. 9. Stress signal for an armor unit [11]

Impulse-momentum requires time interval and initial and final velocities for a singular wave. However, it is important to recognize that interpretation of the impulse per wave cycle is not possible, since the terminal wave momentum is not equal and opposite to the initial value, but is less by some unknown amount as the result of energy dissipation in the breaking process [9]. Thus, the only change in wave momentum which can be calculated is from the initial state to the situation of zero horizontal component.

Actual problem involves estimation of force for a train of waves which becomes more complex and requires experiment-based approach. Effect of cyclic waves on armor surface could be better understood by plotting stress signal (See Fig. 9). In such a condition, effect of wave impacts may be estimated using studies carried out by Weggel and Maxwell who recommended relationship between maximum pressure (P_{max}) and time of transfer of impact (t) as following [12].

$P_{max} = a.\,t^b$ (17)

where a and b are dimensionless empirical coefficients with values of a and b as 232 and -1 respectively. However, subsequent researchers have suggested different values of a and b from over experiments keeping the basic equation as it is following which large variations in the P_{max} may be obtained. Values suggested by Blackmore and Hewson, by Kirkgoz and by Witte were tried out. Maximum values were found from over research made by Blackmore and Hewson.

Aforesaid works have been for vertical walls whereas inclination of outer surface of the divide wall of Ukai dam is 1.5:1 (H:V), and hence P_{max} may be considered with some reduction in light of outcomes of studies and experiments [13]. Though there is a gentler inclination as compared to the said study, reduction factor applied here was 44% i.e. the same as suggested from the said study ensuring extra safety. Once the roller action gets established, estimation of kinetic energy based on Froude number may be applicable. The former stage would be effective for the lower portion of the divide wall and the latter for the upper portion. In comprehensive consideration of various researches and the model study, design velocity was taken as 8.1 m/s and design P_{max} as varying between 65 kN/m^2 and 868 kN/m^2 for various flow conditions. Considering large variations in inclination of various concrete slab panels on nosing and vortex formation due to sudden enlargement of spillway channel, designing for various values of loads became necessary. These values were taken as uniformly distributed load on either entire unit or a part of outer surface of the concrete slab panel as per basic logic of the empirical formulas adopted. Moreover, shear resistance and structural stability of panels while P_{max} occurring on edge were also required to be adequately ensured. Static analysis was resorted to. As energy was supposedly transmitted in to the armor in a very short time and through a small area, design of armor required somewhat an over safe design. Dynamic analysis might be done of the multilayer complex but that would be rather meaningful with case specific experimentation to specify the design parameters and inputs.

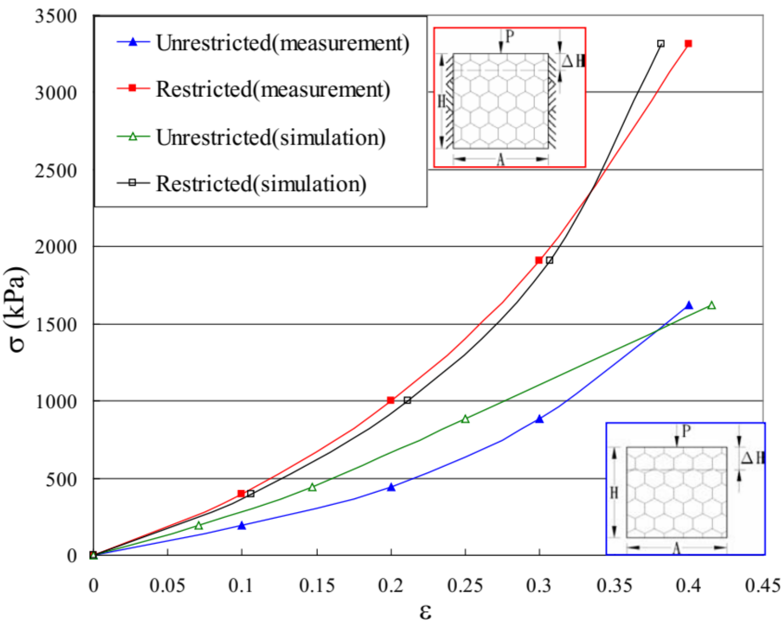

Fig. 10. The stress-strain curves of restricted and unrestricted lateral expansion in uniaxial compression test on single gabion unit [14]

For static equivalent analysis, idealization of multilayer armor system was made as a rigid mass (concrete slab panel of 3 m length, 3 m width and 0.5 m thickness) attached to two springs in series (gabion and reinforced soil subgrade). As gabions were to predominantly take compression and were large in size, they were assumed to remain in almost unconfined or unrestricted state (sideway enlargement permissible), and, accordingly their elasticity co-efficient was taken as E_{gabion} = 2000 kN/m^2 from upon results of experimental work [14]. As E_{gabion} followed stress dependent nonlinear curve (see Fig. 10), value of E_{gabion} was based on stress induced on the concrete slab panels and load dispersal through concrete slab panel was discounted to be on safer side considering very short duration of load transmission and localized impact. Spring stiffness for gabion K_1 was taken as 400 kN/mm.

Elasticity coefficients of geogrid, geotextile and soil subgrade were preferred to be taken as a composite value since their performance would not be as individual entities. Elasticity coefficient of geogrid itself followed nonlinear curve and hence secant modulus was required to be applied to get its stiffness [15]. Considering cyclic loading, secant modulus of geogrid was reduced by 40% [16]. Moreover, tension transferred to geogrid, creep, etc. would be depending on properties of the sand filter above it and the soil subgrade beneath it. All these aspects would make its in-situ behavior complex and hence estimating modulus of reinforced subgrade reaction required experiment-based approach and accordingly equivalent spring stiffness could be idealized. Limiting strain approach was applied in order to restrict shear stresses and accordingly the global and local strains for subgrade complex were derived to compute design spring stiffness for subgrade complex i.e. K_2. Cyclic response of coarse base course materials would be complex due to its highly nonlinear behavior [17]. Relative density, moisture content, properties of geogrid and its interface with soil, its depth from point of application of load, etc. were the determinants to estimate subgrade modulus. Based on different values for the said parameters, different values for resilient modulus could be obtained. Many experiments carried out by different researchers for pavement design and much higher values estimated by them with specific site conditions suggested that the actual site condition here suited 50 kN/mm as K_2. It was also considered that the said studies were conducted using plate load tests whereas here the impact could be applied to a wide range of area – much smaller to much bigger than a standard plate; and, time of transmission of energy could be shorter.

3.3 Finalization of Design and Actual Performance

Concrete slab panel as the outer layer, gabion as the intermediate layer and geogrid reinforced subgrade as the bottom layer constituted the solution. As per alignment of the guide bund, angularity of the three layers was provided. Sand filter of 0.2 m thickness with geogrid and non-woven fabric was put on properly dressed soil base. For design velocity of 8.1 m/s, thickness of slab panel was required to be 1.97 m following Indian standard practices [6]. Instead, a multilayer armor made of concrete slab panel thickness as 0.5 m and was gabion with 2 m thickness with reinforced subgrade was designed. The slab panel was for taking the shear from the impact of the water waves and to provide smooth surface for better hydraulics.

Static equivalent analysis of the armor system suggested soil settlement of 4 mm under extreme load condition without any tension at any corner of any panel which ensured its structural stability. Pressure due to impact was found being shared by gabion and reinforced subgrade. Step by step reduction in stiffness was aimed at better dispersal of impact load and energy absorption which would safeguard the earthen embankment of the divide bund. Entrenchment in the river bed was used to form a bottom key (see Fig. 11). For design of the bottom key, hydraulic behavior of the channel flow in the downstream of the bucket was estimated using tail water depth and Froude number relationship [19].

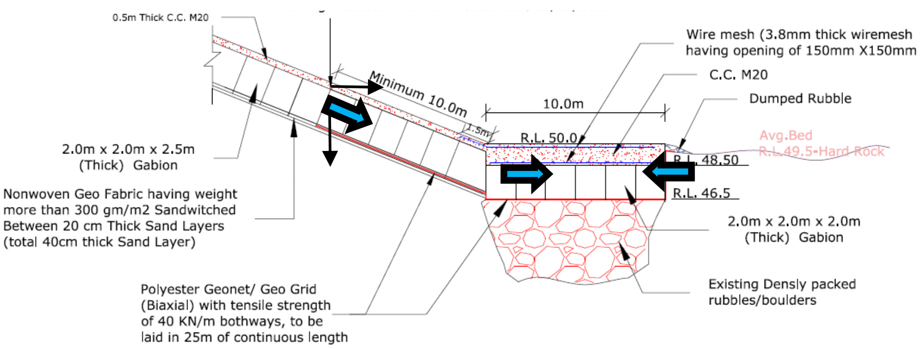

Fig. 11. Multilayer armor design [17]

Analysis showed a decrease of at least 36% in soil base reaction as compared to a single layer armor. In case of pavement design, increase in bearing capacity due to reinforced subgrade in the form of Bearing Capacity Ratio is an accepted factor to quantify the benefit accrued by changing design but in armor design there is no term coined so far to determine net benefit.

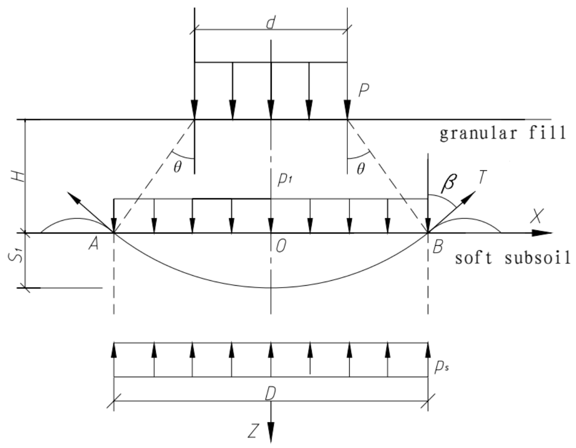

Fig. 12. Model of geogrid reinforced unpaved road [20]

Theoretical approach based on tensioned-membrane function of geosynthetics for pavement design taking in to account the lateral restraint of the subgrade by assuming the subgrade soil vertically below the footing [20] was also applicable here and validation of results was tried out accordingly (see Fig. 12). However, in case of armor design, permissible settlement is much lower than that in case of pavement design and therefore soil base reaction is of concern and hence the base course thickness is greater.

Shear strength of the concrete slab panel was also verified for P_{max} obtained from formulae suggested by Blackmore and Hewson [12] with rationalization considering the fact that spillway operation might not be highly random in any case. Against permissible shear stress of 2.8 N/mm^2 for concrete used for slab panels in general loading condition with 50% of additional allowance for emergency situation, theoretical maximum shear stress was found to be 3.33 N/mm^2 in exceptional situation which was acceptable. As the concept of P_{max} was applied, fatigue allowance was not accounted for considering discounted stress dispersal through concrete slab panels as a compensating factor.

Structural stability after every critical spillway operation was meticulously observed after restoration for four years and was found satisfactory. Due to smooth outer surface of the concrete slab panels and better energy absorption, significant improvement in hydraulic behavior of flow in the downstream of spillway could be achieved (see Fig. 13). Vertex formation near nosing of the divide bund and plunge wave formation could also be overcome. Pitting as a usual problem with concrete elements taking impacts was nowhere found here.

Fig. 13. Performance of divide bund after restoration [17]

4 Lessons and Conclusion

Energy dissipation was not done through rigidity of concrete or shape of the concrete element; rather, it was done by energy absorption through flexibility of geosynthetics which was a paradigm shift in the design philosophy. Impact resistance requires hardness of the surface with conventional design methodology and hence higher grade of concrete is recommended in hydraulic applications; but, pitting remains unavoidable due to the fact that with large increase in grade of concrete, strength could be much enhanced but not the hardness in the same proportion.

Large scale usage of geosynthetics in hydraulic structures has come in to vogue in recent years. Membrane applications have been amply executed. Yet, there is ample scope for exploring reinforcement applications. This case study has opened avenues to explore the field of energy dissipation for reinforcement applications of geosynthetics. Instead of using thick concrete panels as rigid armor for energy dissipation, flexible solutions could perform better is the net takeaway.

This case study is a product of inferences from over various researches applied to similar situations in different types of applications. Researchers may contribute a lot in the form of theoretical and experimental advancements so as to formalize design methodologies for application of geosynthetics for better energy dissipation in hydraulic structures.

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