THE USE OF ROCKFILL – SOME DESIGN CONSIDERATIONS

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ABSTRACT
Rockfill has been in use as a construction material for a very long time. Some of the more common uses are as a stabilizing material for steeply inclined slopes, and as rock riprap in water projects. Although the uses of rockfill vary considerably, designers generally call for high quality materials, irrespective of its intended useage. It appears that some of the current requirements for rockfill have been adopted from specifications for rockfill dams and concrete aggregates, where durability and high strengths are esssential. However, the more common usages such as in rock riprap, are mostly in low stress environments. Current practice has no doubt resulted in the construction of robust durable structures, but has also made rock riprap a scarce commodity commanding a high price. This paper examines the current design requirements for rock riprap and suggests changes that would result in a more economical and efficient use of rockfill

RÉSUMÉ
Depuis très longtemps les enrochements sont utilisés comme matériau de construction. Fréquemment, leur emploi sert de revêtement protecteur et stabilisateur de talus, de digues, de rives, etc. Bien que l'utilisation des enrochements soit très diverse, le choix de l'ingénieur se porte, majoritairement, sur des matériaux durables et résistants, indépendamment de leur besoins spécifiques. Les roches sont couramment sélectionnées en fonction des normes prescrites pour les barrages en enrochement ou pour la production du béton. Ce type d'enrochement, très dense et très résistant n'est souvent pas nécessaires et ce choix discriminatoire en fait un matériau de plus en plus rare et coûteux. Cet exposé examine les normes courantes des enrochements et propose des changements quant à leur mode d'utilisation afin d'en réduire leurs coûts.

1. BACKGROUND AND ISSUES

Rockfill is widely used as a construction material in all parts of the world. Some of the engineered uses include the construction of dams, hydraulic structures, revetments, and the stabilization of steeply inclined or unstable slopes. One of the most popular uses of this material is as rock riprap (Simons, 1993) - an energy dissipating erosion resistant layer of rockfill protecting earth slopes on dams and waterways.

Table 1 lists some of the more common uses of rockfill. Although the loading conditions and the performance requirements for the many uses of rockfill differ considerably, the general practice among designers appears to be to call for very high quality rockfill materials. The standard specifications for rockfill that are currently used in construction contract documents (CNMCS, 1996; USBR, 1987; FHWA, 1989) appear to have been adopted from requirements generally stipulated for rockfill dams and concrete aggregates, where strength and durability are prerequisites for the selection of the material. It is seen from Table 1, however, that a majority of the more common uses of rockfill are in low stress environments, such as in rock riprap, where the strength of the rockfill material is unimportant.

Nevertheless due to the very stringent quality assurance requirements that are in force the cost of rockfill, including that of riprap, has risen significantly, largely due to the scarcity of acceptable materials. For example, in Alberta the price of rock riprap is often twenty to thirty times the cost of earthfill, the material that is being protected by the riprap.

Table 1 – Some Common Uses of Rockfill

Engineered - Low stress	Engineered - High stress	Other
Riprap on dams	Dams, dykes;	Fish habitat,
Erosion protection –in streams, channels; structures for grade control; bridge pier scour protection.	Masonry dams Rock retaining walls.	Landscaping Traffic control Ballast.
Hydraulic structures- stilling basins, plunge pools, drains, weirs, coffer dams, river training, sediment traps,	Breakwaters; groynes; weirs; coffer dams.	
Low retaining walls- gabions, and stone masonry; ballast, revetments	Stabilization berms, revetments	

Although, the difficulties of procuring 'acceptable quality' rock riprap were known for the past several decades, very little has changed in the design methodology or in the specifications for its acceptance. In fact some of the requirements for quality appear to have become more stringent and has made 'acceptable quality' rock riprap, an expensive construction material.

The unit price of rock riprap appears to rise rapidly for larger sizes of the material due to scarcity of supply, difficulties in transportation, handling, as well as placement. Anecdotal evidence and published reports (Yaremko, 1971) indicate that this somewhat high and unbalanced cost of the riprap has been a problem in Alberta for more than 30 years.

In the Prairie Provinces, according to Peters and Towle (1979), the cost of riprap slope protection has varied from 5 to 15% of the cost of the dam. In some cases, where long low dykes were associated with large reservoirs, the cost of riprap approached 50% of the total cost of the dam. Questions on the high price of riprap have become increasingly important, due to the competition for capital from a very limited pool of funds available for infrastructure projects.

This paper briefly reviews the current design methodology for rock riprap and argues that more consideration should be given in the design process to probabilistic loading, ease of repair, and the consequence of failure of the structure. The paper points out that in addition to traditional design techniques, some economy in the design is possible if the owner is willing to assume a slightly higher risk of damage and future repair, during the lifetime of the structure.

The ideas expressed in this paper are not new, but have been restated, in order to frame the question of what constitutes acceptable quality riprap as well as to stimulate further debate and discussion.

2. DESIGN CONSIDERATIONS

A large number of methods and formulae are available for the design and sizing of rock riprap, as an energy dissipating erosion resistant layer overlying an earth or rockfill interface. These formulae have evolved from empirical methods, laboratory testing, and analyses.

Some of the more well known methods used in North America include design manuals produced by the U.S. Army Corps of Engineers (USACE, 1995), California Division of Highways (CDH, 1970), United States Soil Conservation Service (USSCS, 1983), the US Bureau of Reclamation (USBR, 1987) and the US Federal Highway Administration (FHWA, 1989).

Canadian design practice tends to follow the US Army Corps methodology, with some emphasis on local conditions such as colder climate, wind speeds, and freeze thaw considerations (Peters & Towle, 1979; Matheson, 1988).

2.1 Some Aspects of Riprap Design

The various formulae and the methods commonly used to design rock riprap are well known and can be found in the literature. However, some aspects of the riprap design, which are within the control of the designer that could lead to reductions in the median size of the riprap and in procurement costs, are discussed herein.

Conventional design methods for riprap can be broadly classified under two categories, based on the proposed usage of the material. The two categories are designs, that provide erosion protection from flowing water (FHWA, 1989; Maynord, 1993), and those that resist wave action (Hudson, 1959; Peters and Towle, 1979; USSCS, 1983; USACE, 1995; Matheson, 1988).

2.2 Channel Protection

The first category is mainly concerned with problems related to channel and stream bank erosion. The formula proposed by the US Federal Highway Administration (FHWA, 1989) for this purpose is listed below to facilitate the subsequent discussion.

$$C\ D_{50} = 0.001\ V_a^{\ 3} / (d_{avg}^{\ 0.5}\ K_1^{\ 1.5}) \qquad [1]$$

where, C = a correction factor
D_{50} = the median riprap particle size,
V = the average velocity in the main channel,
d_{avg} = the average flow depth,
$K_1 = [1- (\text{sine}^2\ \theta/\text{sine}^2\ \phi)]^{0.5}$ – a stability factor,
θ = the inclination of the bank to the horizontal,
ϕ = the angle of repose of the riprap material.

The D_{50} computed from Equation 1 is based on rock riprap materials of Specific Gravity 2.65, and a Safety Factor of 1.2 against movement. However, this D_{50} can be corrected for other values of Specific Gravity and Safety Factors by applying the Correction factor C as given in FHWA (1989).

It is evident from Equation 1, that the D_{50} size of riprap is dependant on the velocity of flow in the channel, the flow depth, and the inclination of the channel banks. Of these factors, the velocity of flow is dependent on other variables such as the volume of discharge, channel gradient, and the roughness coefficient. It is clear that the designer has some control over only two of the factors that influence the riprap design, the channel gradient and the inclination of the channel banks.

2.2.1 Channel Velocity

In Equation 1 (FHWA, 1989), the size of riprap varies directly in proportion to V^3. Thus even a small increase in channel velocity contributes to a rapid increase in the required median size of the riprap particle. For example, for a channel with relatively flat side slopes, a change in the velocity of flow from 1 to 2 m/s requires the armouring to be changed from gravel to rocks with a D_{50} of approximately 0.15m. Therefore, it appears that significant reductions in the size of armour required as well as cost can be achieved by designed reductions of the velocity of flow in the channel.

Traditionally, designers have resorted to the use of grade control structures, as the primary method of controlling the velocity, with little change in other design parameters or standards. However, it should be noted that the most important factor influencing the velocity of flow in a given channel is the volume of flow. It is clear in the case of natural streams, the designer has little or no influence on flow volumes.

In current practice, design flow frequencies ranging from 10 to 50 year recurrence intervals are used in the design of channel stabilization and bank protection. In channel protection projects in the rural Prairies, where the consequence of failure would result in very little damage to property, the adoption of more frequently occurring floods should be considered, especially if repairs can be carried out easily. Such a practice would limit the vertical extent and the volume of riprap protection required.

In any case, if the velocities in the channel can be kept under control or reduced, the designer can use smaller, easily procurable riprap sizes. The current high prices for riprap materials, warrants the investigation of possible measures such as grade control structures, and or tolerating the risk of some future damage, by the adoption of more frequently occurring design floods.

2.2.2 Inclination of Channel Banks

The influence of the channel cross-section and the inclination of the stream banks can also be seen from Equation 1. The equation indicates that steeply inclined banks would require larger rock sizes, while smaller rocks can be used as armour on flatter banks.

For example, an earth channel with approximately 3H: 1V bank slopes, with a flow depth of 1.5m and a velocity of flow of 1.5 m/s, would require protection with gravel armour. However, for the same flow conditions a channel with side slopes at 2H: 1V would require armouring with rocks with a D_{50} size of approximately 0.15m.

This reduction is largely due to the smaller component of the gravitational force tending to displace the rocks on flatter slopes. However, as D_{50} is dependant on the value of $K_{1,}$ some of the benign influence of bank slope flattening is reduced when smaller rocks are used, due to the lower angle of repose of smaller rocks.

From the foregoing it is clear, that at locations where right of way is readily available, some reduction in the median size of the riprap armouring required could be easily achieved by flattening the bank slope. Moreover, if the bank slopes are flattened, the resulting increase in the area of cross section of the channel leads to a slight reduction in the flow velocity. As evident from Equation 1, even small reductions in the channel flow velocities have a very benign effect on the median size of the riprap required, as D_{50} varies in proportion to the third power of the velocity of flow in the channel.

Although, flattening of the banks results in a slight reduction of the vertical extent of channel armouring, some increase in the volume of riprap occurs due to the lengthening of the slope. Thus it is necessary to balance the cost reductions achieved by the decrease in the median size of riprap with the additional cost incurred due to the increased volume of riprap required.

2.3 Protection from Wave Action

One of the most common uses of riprap in western Canada is to protect the side slopes of earth embankments and shorelines in reservoirs from wave attack. Two of the more common causes that generate waves in reservoirs are wind and boat traffic. However as pointed out in Section 2.1, the design basis and the methodology used for slope protection in dams is different from the design methodology for riprap used in channel protection.

Recent Canadian design practice for riprap on dam slopes follows the guidelines established by Matheson (1988) in a report to the Canadian Electrical Association. Peters and Towle (1979) give the methodology used by the PFRA, to design upstream riprap slope protection for dams in the Prairie Provinces.

Both these methods as well as other commonly used formulae for the design of riprap to resist wave attack are based on the Hudson's formula (1959). Although subsequent research has produced more sophisticated design relationships, for the discussion in this paper Hudson's formula as used by the US Soil Conservation Service (USSCS, 1983) will be used.

$$W_{50} = \Upsilon \ H^3 \ / \ K_d (G_s - 1)^3 \ \mathrm{Cot}\ \theta \qquad [2]$$

where, W_{50} = weight of median riprap particle,
Υ = unit weight of riprap,
H = significant wave height; (average of highest 1/3)
Gs = specific gravity of riprap,
K_d = stability coefficient, varies with type of riprap, etc
θ = angle of inclination of the embankment slope.

The calculation of the wave height, H, according to the methodology proposed by Saville et al (1962), for use in Eq.2, requires the speed of wind over water, duration of the wind, mean depth of water in the reservoir, and the computation of the effective fetch.

From Equation 2, it is clearly seen that two factors - wave height and the inclination of the embankment slope, have a significant influence on the riprap size. However, in this case the designer can exercise some influence only on one factor – the inclination of the embankment side slope.

Table 2 shows the influence of the embankment slope on the W_{50} size. The riprap sizes shown in Table 2 were calculated using the USSCS (1983) design charts, for a significant wave height of 1m, specific gravity of rock of 2.65. From Table 2, it is clear that flattening the embankment slope has a benign effect on riprap size and therefore on unit costs.

Lefebvre et al (1993) investigated the performance of riprap at14 dams and dykes in Northern Quebec. Their investigation indicated that the performance was generally poor for riprap placed on slopes steeper than 1.5H: 1V inclination. However, such steep inclinations are rarely found in the Prairies, where the side slopes of dams are generally at inclinations of 3H: 1 V or flatter, due to earthfill construction as well as weak bedrock foundation conditions.

Table 2 Influence of Embankment Slope on Riprap Size

Embankment Slope	W_{50} Size - kg
2.0H: 1V	96
2.5H: 1V	75
3.0H: 1V	64
3.5H: 1V	50
4.0H: 1V	45

2.3.1 Wind Load Conditions

The two most important loading conditions that influence the design of riprap slope protection for dams are the velocity and duration of the wind causing the waves in the reservoir. The mechanics of wave generation as well as the dissipation of the energy carried by the wave via the riprap layer are somewhat complex. However, all common design procedures consider the height of the waves generated by the wind, as the prime indicator of the wave energy to be dissipated. It is clear from Equation 2, that the size of the riprap particle required to resist displacement by wave energy is very sensitive to the significant wave height.

The significant wave height is commonly established by methods described by Saville et al (1962) or using design charts, based on the same method, available from the US Army Corps of Engineers (USACE). Although these procedures are well established in design practice, there is considerable disparity among designers on the frequency of occurrence of the wind speed that is used to generate the significant wave height.

PFRA design practice, as stated by Peters and Towle (1979), uses wind speeds with a recurrence interval of 30 years for dams with an effective fetch length of up to 10km. However, they also recommended the use of a lower wind speed for dams in the eastern part of the Prairies sheltered from the prevailing winds, and higher speeds for dams in southern Alberta where the exposure is considered to be more severe. The exposure condition is considered more severe as the upstream slopes in a majority of the Alberta dams face west - the direction of the prevailing winds.

The USACE (1994) in their Engineer Design Manual use a 1:100 year return period for riprap with maximum exposure during normal operation, with less intense winds for areas with infrequent exposure. The US Soil Conservation Service (USSC, 1983) uses a 1: 50 year return period for high exposure conditions. The Dam Safety Guidelines (CDA, 1999) published by the Canadian Dam Association, does not provide any guidance on the wind load conditions for riprap design. However, in the guideline for freeboard calculations, use of design wind speeds with an annual exceedance probability of 1 in 1000 has been recommended for the maximum normal operating pool. It should be noted, however, that the currently available data base for wind speeds for most locations would not allow the accurate prediction of wind speeds, beyond the 1:100 year recurrence interval using statistical methods.

2.3.2 Thickness of the Riprap Layer

Next to the W_{50} size, perhaps the most important design consideration is the thickness of the riprap layer. Here again the design practice appears to vary. The thickness ranges from 1.5 times the D_{50}, as recommended by the early USACE practice (USACE, 1949; PFRA, 1979; FHWA, 1989) to 2 to 3 times the D_{50} as recommended by Matheson (1988). The layer thicknesses of 3 times the D_{50} was recommended for thinly bedded materials in harsh winter service conditions. The benefit of thickness above 2 times D_{50} is considered marginal (Stephenson, 1979).

The 'disparity' In the layer thickness appears to be largely due to the Safety Factor used by the various designers. The Safety Factor is unity for incipient motion flow conditions over riprap, and is greater than unity for more stable riprap particles (Stevens et al, ASCE, 1976). Some displacement of the riprap material is expected to occur, for design wave height conditions, if the Factor of Safety is less than 1.

2.4 Selection of Materials

Good quality riprap materials are described as rocks with good laboratory index parameters (high specific gravity, unit weight, and low absorption), fine grain size, no micro fractures, no well developed cleavage, hard stable minerals in an isotropic arrangement with good crystal interlock (Matheson, 1988).

Table 3 lists some riprap acceptance criteria used by several Canadian and US agencies. These include, the Canadian Electrical Association (CEA) as recommended by Matheson (1988), Canadian National Master Construction Specifications (CNMCS), U. S. Bureau of Reclamation (USBR) and the U.S. Army Corps of Engineers (USACE).

Generally the selection process for 'acceptable' riprap includes the testing of materials to determine compliance with criteria listed in Table 3. The laboratory testis are based on standard quality assurance tests for coarse aggregate in concrete, as well as Petrographic Analyses.

Table 3 Some Acceptance Test Criteria

Test	CEA 1988	CNMCS 1996	USBR 1987	USACE 1990
Specific gravity	>2.6	≥2.65	>2.6	>2.57
Absorption	<2.0%	≤2.0	a)	<1%
Soundness	<10%	a)	<10	<5%
Abrasion	a)	≤45%	<40%	<20%
Freeze –thaw	a)	b)	a)	<10%
Wet –dry	a)	b)	a)	c)

a) – not specified; b) – as required by engineer; c) no major progressive cracking;

Acceptance test criteria listed in Table 3, indicate that the minimum requirements for riprap have become more stringent over the years in Canada and in some of U.S. agencies. However, some relaxation in the required specific gravity has occurred. California Division of Highways (CDH, 1970) accepts a minimum value of 2.5 in its Bank and Shore Protection Manual, which states that "…excellent results have been reported for lava with a specific gravity of 1.5…" However, most engineers generally regard specific gravity to be a key acceptance criterion, as the minimum specific gravity is one of the basic assumptions made in Eq. 2, used in the design of riprap.

2.4.1 Relevance and Reliability of Tests

The use of the concrete aggregate test methodology for the selection of rock riprap (Table 3) has been questioned due to the differences in the operating environments of the two materials. Unlike, concrete aggregate, riprap performs in a very low stress environment. Moreover, a damaged layer of riprap unlike concrete is often 'self healing' and can be repaired easily.

There is also some debate on the reliability of some of the tests used for riprap selection (USACE, 1981; Laan, 1993). One of the most difficult aspects of riprap testing is the procurement of a representative sample of the material, and has lead to significant variations in the test results even within the same sample.

Due to laboratory testing constraints the riprap sample is usually broken into smaller particles, which rarely represent the attributes of riprap in the field environment. The preparation of this small sample tends to eliminate weak layers/laminations or cracks in the rock that lead to the degradation of riprap in the field. The test results, especially for sulphate soundness, tend to be too favourable due to this test procedure.

2.5 Durability of Riprap

In addition to performing satisfactorily under design loading conditions, riprap is expected to be durable, and not undergo any significant breakdown due to physical weathering. The durability of riprap is particularly important in the harsh winter climate found in the Canadian Prairies. The present practice is to select durable materials by the generic rock type, results of laboratory testing, and past performance if available.

2.5.1 Rock Type

Standard specifications have specifically excluded the use of some rock types such as shale, breccia, and conglomerate due to their poor performance. Other rock types that have been found unsatisfactory are slate, laminated schist, siltstone, porous or chalky limestone, and some sandstone. However, due to the variability of natural rocks, selection of riprap materials by rock type alone is not possible, without testing or data on past performance.

2.5.2 Testing for Durability

Tests that are often used to determine the durability of riprap include the Los Angeles Abrasion, Sulphate Soundness, Freeze Thaw, and the Durability Absorption Ratio. Extensive reviews of the performance of riprap by the CDH (1967), and the USBR resulted in some modifications to the acceptance test procedures. Both the CDH and the USBR found that the Los Angeles Abrasion Test was not a good indicator of durability. This led to the deletion of the Los Angeles Abrasion Test, and the addition of the Durability Absorption Ratio into the specifications.

A review of index tests by the USBR found that the freeze thaw tests were not conclusive. Franklin et al (2003) also found the freeze thaw test wanting and suggested that the Iowa Pore Index Test is a better indicator of riprap deterioration. USACE (1981) found that test results from large blocks of riprap armour material provided a more realistic indication of the rocks in a freeze thaw environment than in the generally accepted testing procedure.

Table 4 shows the results of some durability test results on six samples of sandstone conducted by Alberta Environment (Yaremko, 1971). Although the test results appear to indicate unsatisfactory materials for riprap, rocks from three of these sources (Tests 3, 5, & 6) have performed satisfactorily as riprap for long years. Material listed in Test 3, is in use as riverbank protection since 1971, and that in Test 5 was in use at the inlet of a diversion structure from the early 1950s. Material in Test 6 is in use as riprap at a dam in Southern Alberta for over 50 years. The three samples (1, 2, & 4) were from rock outcrops subject to freeze thaw conditions at two riverbanks and a gravel pit in Southern Alberta.

Table 4 Some Durability Test Results on Sandstone

Test	1	2	3	4	5	6
Sp. Gravity	2.58	2.44	2.16	2.37	2.18	2.24
Absorption	2.1	3.65	6.58	4.95	4.97	6.50
Abrasion	2.6	7.3	23.2	8.5	18.2	4.1
Wet- Dry	3.06	8.13	3.20	8.10	2.80	0.95
Freeze-Thaw	0.25	0.67	4.03	5.55	3.66	3.28
Soundness	0.06	100	60.9	44.6	35.3	2.70

The satisfactory field performance history of the riprap in Samples 3, 5, and 6, despite poor laboratory test results suggests that a considerable amount of usable riprap material such as sandstone is being excluded due to strict applications of the acceptance criteria.

Matheson (1988) investigated the correlation of laboratory test results with the field performance of riprap and found that none of the index tests correlated completely with durability. Lienhart et al (1993) said that some correlation is possible if the test data were based on rock types, rather than as a single correlation for all rock types and recommended against the use of specification limits based on index test results, as properties vary by the rock type.

Authors are of the view that laboratory index tests are a useful tool to identify potentially unsuitable rock sources. However, prior to rejection of the material based entirely on acceptance criteria, the results of the laboratory testing should be viewed in combination with information on the rock type, and performance history of riprap from the same rock source, if available.

3. RIPRAP - PERFORMANCE

The riprap design for a majority of the dams in southern Alberta, constructed in the early 1950s by the PFRA followed the USACE (1949) design practice (Peters & Towle, 1979). The exposure conditions at some of these dams are considered severe, due to their west facing upstream slopes However, the wind loads assumed by the PFRA in the design of the riprap at these dams, though considered adequate at the time, are somewhat less intense than those recommended in current design standards.

Given the discrepancies in the design wind load conditions, and the age of the dams in Southern Alberta, it would be reasonable to expect, some displacement and or degradation of the riprap particles. However, inspections of these dams indicate that for over 50 years, the riprap has performed very well with little or no maintenance, and to date remains in very good condition.

The upstream slope protection at a number of these dams is a mix of sandstones, siltstones, and dolomitic limestone. Except for limestone, some of the other rock types do not satisfy current acceptance criteria for riprap. However, inspections indicate very little deterioration of the riprap, in spite of more than 50 years of exposure to the harsh Prairie winter climate.

4. DISCUSSION OF DESIGN & PERFORMANCE

In Section 2, some design aspects of riprap and the options for reducing the D_{50} size without changes to current design standards were discussed. However, given the highly satisfactory performance of the riprap for more than 50 years at some of the PFRA dams, it needs to be asked whether the current design standards are overly conservative and the acceptance criteria unnecessarily stringent. Moreover, the current high price of riprap appears to be driven to some extent by these stringent specifications, which tends to exclude some of the more abundantly available rock types such as sandstone.

4.1 Risk of Riprap Failure

Analysis of dam failures dating as far back as 1799, collected from all over the world including North America, by the International Commission of Large Dams (ICOLD, 1995), show only two cases of dam failure that were attributed to a failure or a deficiency in the riprap. Both dams were constructed in the early part of the last century, when dam design was still an empirical art.

Again investigations of dam incidents since 1890, by the US Committee for Large Dams (ASCE, 1976; ASCE, 1988), has reported only 22 incidents of riprap failure, none of which led to a failure of the dam. Only sixteen of these incidents were considered major repairs. Given the population of over 77,000 dams in USA, and the nearly 100 years of record examined, it appears that the risk of riprap failure is extremely small. Although the engineered design of riprap generally commenced in about 1950, a majority of the existing dams, including those constructed prior to 1950, have operated with near zero probability of dam failure due to deficiencies in riprap.

4.2 Factors of Safety

Factors of Safety of 1.1 to 1.5 are usually accepted in the stability analyses of dams for various loading conditions. It is well known, that failures or instabilities of the upstream slope, due to a drawdown of the reservoir rarely cause a loss of the reservoir. Recognising this lower risk geotechnical engineers have required a lower Factor of Safety for the upstream slope ranging from 1.1 to 1.3 for the rapid draw down loading condition. In comparison the stability factor assigned to riprap particles to prevent displacement from wave attack ranges from 2.2 to 3.6, and is often near 2.65. A Stability Factor of 2.2 is considered to be equivalent to a zero damage condition.

Lower Stability Factors give a larger W_{50} in the Hudson's formula due to the stability factor being placed in the denominator.

As the two safety factors are not equivalents, a direct comparison of the riprap stability factor with slope stability Safety Factors may not be valid. However, it can be said that the Factors of Safety assigned to individual components of the upstream slope should be compatible with the safety factor assigned to the overall structure. Moreover, the stability factors used in riprap design appear to ignore the self -healing capability of rock riprap.

Furthermore, the upstream slope riprap at some of the older dams, considered to be undersized by current design standards, appear to have functioned reliably for more than 50 years, with little or no upgrading. Therefore designing for zero damage with Stability Factors greater than 2.2 appear to be somewhat conservative.

Generally in the design of structures, if the serviceability and reliability of any design component over an extended period of time is very small, changes are no doubt required in the design methodology. However, in the case of upstream slope protection, both the serviceability and the reliability conditions appear to be very high. However, the standards for riprap design have become more conservative during the last 50 years, both in the loading as well as in acceptance quality criteria.

4.3 Consequence of Failure

As far back as in 1979, long before risk assessment became fashionable, Peters and Towle of the PFRA suggested the consideration of Consequence of Failure of the dams in the design of upstream slope protection and recommended the use of clay, gravel, or riprap as slope protection depending on the Consequence of Failure of the structure, high consequence structures requiring more reliable protection such as rock riprap. This concept is even more relevant today given the current price of rock riprap.

4.4 Probabilistic Design

The design loading condition for riprap, for channel protection as well as for slope protection, is normally associated with a frequency of recurrence. Thus the design methodology for riprap is easily amenable to 'probabilistic design' methods. Such methods for the design of riprap revetments and breakwater structures have been advocated for sometime (Van de Meer, 1988). The method proposed by Van de Meer is more promising as it allows the owner to choose a level of risk and the associated levels damage that could probably occur during the lifetime of the structure.

4.5 Ease of Repair

Most instances of 'failure' reported in the literature are due to wave attack and ice action and rarely cover the whole of the upstream slope. Most dam owners are able to carry out the needed repairs easily, as a part of routine maintenance. Peters and Towle (1979) stated that the volume of annual riprap repair at the PFRA designed dams was less than 1%, and even dams with 'riprap in poor condition' were in no imminent danger of failure.

Authors are of the opinion, given the very low probability of failure associated with riprap, and the ease with which repairs can be carried out a condition somewhat riskier than zero damage to riprap would be acceptable to most owners. Therefore, the use of the 'zero damage condition' often used in riprap design should be reviewed.

5. CONCLUSIONS

5.1. The design methodology for the use of rockfill materials to protect earth slopes in channels and upstream slopes of dams is well established. However, a scarcity of materials caused by stringent acceptance specifications has driven up the cost of riprap in Alberta.

5.2. The acceptance tests for riprap are the same as those used to assess the quality of concrete aggregates. Unlike, concrete aggregate, riprap is not subject to high stresses and functions in a very low stress environment. Moreover, a damaged layer of riprap unlike concrete is often 'self healing' and can also be easily repaired.

5.3. It is difficult to procure representative samples of riprap for laboratory testing. Sample preparation methods also tend to eliminate defects such as weak layers, cracks and laminations that affect field performance.

5.4. Extensive reviews of riprap performance have shown that there is no correlation between acceptance tests and field performance. The acceptance limits listed in specification documents are somewhat arbitrary, as the limits tend to vary by rock type. Strict application of the acceptance criteria tends to exclude some possibly usable rock types, such as sandstone.

5.5. According to published data very few dams have failed due to the poor condition of their riprap. Based on failure statistics the probability of failure of a dam due to deficiencies in slope protection is considered to be very near zero.

5.6. The riprap in a majority of the dams constructed prior to the 1950s was designed using empirical methods, and would be deficient by current design standards. However, these dams have performed satisfactorily, with very little maintenance or upgrading for more than 50 years.

5.7. Some of the stability factors that are used in the design of riprap, based on a zero damage concept appear conservative, in comparison with other safety factors used in slope stability. Given the self -healing properties of riprap and the ease of repair of any damage. the zero damage concept appears to be conservative.

5.8. Probabilistic design methods for the design of riprap have been available for some time. The use of such methods would allow the designer to consider varying degrees of risk and damage during the lifetime of the structure.

5.9. The choice of the slope protection material should to some extent be governed by the Consequence of Failure of the structure. The use of lower quality riprap could be considered for Low Consequence structures.

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