7. CONCLUSIONS AND RECOMMENDATIONS

7.1 CONCLUSIONS

This thesis is divided into two sections. The first section of this thesis (Chapter 2) describes the creation and analysis of a database on concrete and masonry dam incidents known as *CONGDATA*. The second and main section of this thesis (Chapters 3-6) had its origins in the results of Chapter 2 and the general interests of the author. It was found that failure through the foundation was common in the list of dams analysed and that information on how to assess the strength of the foundations of dams on rock masses was limited. This section applies to all applications of rock mass strength such as the stability of rock slopes.

7.1.1 The Analysis of Concrete and Masonry Dams

The author has collected a far more extensive database on concrete and masonry dam incidents (*CONGDATA*) than has previously been attempted

Generally, unlike some failure modes for embankment dams, e.g. internal erosion and piping, the stability of concrete and masonry dams is analysable and hence can readily be checked. The major unknowns for these dams lie in the foundation where sliding and piping failures can occur. It is for this reason that foundation problems (sliding, leakage and piping) are the main causes of failure to concrete dams. Overtopping tends to play a bigger part in the failure of masonry dams, possibly reflecting limited understanding of floods when they were built. It should be noted that the actual failure mode for ‘overtopping’ failures was often unknown.

Foundation shear strength is the main cause of failure for dams with rock or unknown foundations. Shale (interbedded with other sedimentary units) has a greater tendency to be involved with sliding failure because of the likely presence of weaknesses in the bedding such as bedding surface shears. Shale and limestone (often interbedded) have a high incidence of failure. The limestone has a high proportion of accidents generally due to excessive leakage through dissolution.

An analysis of the water levels at failure show most dams failed at their highest recorded water level (regardless of the age of the dam). Several of these were only slightly higher than that recorded previously. The database showed that, where information was available, most dam failures had some warning that could have resulted in the warning and evacuation of residents downstream. Often the warning was a sudden increase in the amount and rate of leakage. The actual volume of leakage was not as significant a guide. This indicates that although a dam may have performed satisfactorily in the past, increases in water level (above historical maxima) should be treated with caution and the dam sufficiently monitored as the water level rises.

Piping is the main cause of failure for concrete and masonry dams with soil foundations. The alluvial soils have a tendency to pipe under the high gradients imposed. No gravity dam has been reported to have failed by sliding on alluvial soils. Piping tends to occur early in a dam's life (<5 years, with one exception).

From the data collected it appears that the failed dams suffered from a lack of 'good engineering'. Very few dams were found with galleries (1 dam); drainage (1 dam); grout curtains (4 dams); and shear keys (1 dam). The downstream slopes appeared to be too steep. Six gravity failures had downstream slopes of 0.6:1 (H:V) or less. Failed dams, particularly gravity dams, were usually located in relatively wide valleys or were composite sections with earthfill dams. Three-dimensional effects are unlikely to have contributed any strength in these cases. h_{wf}/W ratios ranged from 0.6 to 2.1 with an average of 1.35.

The author has used the analysis of CONGDATA and a 'population' database to develop a method for assessing the first order probability of failure of masonry or concrete gravity dams. The method accounts for dam age, year commissioned and type; failure mode; foundation geology; height to width ratio; and monitoring and surveillance. General probabilities of failure for arch, buttress and multi-arch dams, based on failure and population statistics, are included.

The author cautions that this approach should only be used as a first order approximation of the annual probabilities of failure. It is clearly very approximate, and suffers from being based on small numbers of failures, and limited quality data. Where significant decisions on dam safety are being made, detailed deterministic and/or probabilistic methods should be used.

Whilst all care has been taken in compiling the data in *CONGDATA*, it should be remembered that the information in *CONGDATA* has come from numerous sources, not all of which could be validated. The analysis of dams in CONGDATA does not take into account such things as: surveillance; quality of construction; and quality of geological description. It is therefore recommended that this work be used in a qualitative sense only.

7.1.2 The Shear Strength of Intact Rock

An overview of the strength of intact rock has been presented. It was demonstrated that the method of fitting the criterion to the test data has a major effect on the estimates obtained of the material properties. The results of a recent analysis of a large database of test results demonstrated that there are inadequacies in the Hoek-Brown empirical failure criterion as currently proposed for intact rock and, by inference, as extended to rock mass strength. The parameters m_i and s_c are not material properties if the exponent is fixed at 0.5. Published values of m_i can be misleading as m_i is not related to rock type. The Hoek-Brown criterion can be generalised by allowing the exponent to vary. As expected, this change resulted in a better model of the experimental data. The most accurate method of estimating m_i and α is through using triaxial tests on intact rock. The recommended method for regression of the data is modified least squares, Equation 7.1, combined with the extended formulation of the generalised Hoek-Brown criterion, Equation 7.2. The equations are repeated below.

$$\left.\begin{array}{ll} \left(\text{measured } s_1' - \text{predicted } s_1'\right) & \text{for } s_1' > -3s_3' \\ \left(\text{measured } s_3' - \text{predicted } s_3'\right)\times m_i & \text{for } s_1' \le -3s_3' \end{array}\right\} \qquad (7.1)$$

$$\left.\begin{array}{ll} s_1' = s_3' + s_c\left(\dfrac{m_i s_3'}{s_c} + 1\right)^a & \text{for } s_3' > -s_c/m_i \\ s_1' = s_3' & \text{for } s_3' \le -s_c/m_i \end{array}\right\} \qquad (7.2)$$

Analysis of individual data sets indicated that the exponent, a, is a function of m_i which is, in turn, closely related to the ratio of s_c/s_t. A regression analysis of the entire database provided a model to allow the triaxial strength of an intact rock to be estimated from a reliable measurement of its uniaxial tensile and compressive strengths. The method proposed is the most accurate of those methods that do not require triaxial testing and is adequate for preliminary analysis. An analysis was presented that showed applying the Hoek-Brown criterion to most rocks results in systematic errors. Simple relationships for triaxial strength that are adequate for slope design were presented.

7.1.3 The Shear Strength of Rockfill

A general overview of the shear strength of rockfill is presented. An analysis of a large database of test results was used to develop two new shear strength equations, one relating the secant friction angle and normal stress (Equation 7.3) and the other the principal stresses (Equation 7.4). The parameters can be found using equations 4.21-4.23 and 4.25-4.30 in Chapter 4 respectively. The equation for principal stresses provided a much better fit to the data and is recommended. The equations presented effectively give the mean strengths of the data. Graphs are provided showing the range of strengths and the affect of various parameters on the shear strength of rockfill. Of the parameters statistically investigated, the unconfined compressive strength, particle angularity, fines content, maximum particle size and void ratio were found to have the most significant effect on the shear strength of rockfill.

$$f' = a + b s_n'^{c} \tag{7.3}$$

$$s_1' = RFI\, s_3'^{a} \tag{7.4}$$

7.1.4 Empirical Slope Design

A review of current empirical methods of slope design using rock mass characterisation has also been presented. The findings highlighted the lack of well tested methods. Current slope design methods were based on limited databases with no failures and slopes of limited height. Many of the methods were incorrectly advocated for structurally controlled slopes. The author has presented new slope design curves, based on slopes that have had rock mass failure components, that can be used for preliminary slope design.

7.1.5 The Shear Strength of Rock Masses

The Hoek-Brown criterion is the most commonly used strength criterion for rock mass and has thus been the main subject of this section of the thesis. The author has examined the appropriateness of the equation for predicting strengths at the two limits of rock mass (intact rock and rockfill). A study of the strength of intact rock using a large database of triaxial tests shows that the exponent, a, in the Hoek-Brown criterion should vary from about 0.2 to 0.9. The study of a database of 988 triaxial tests on rockfill shows that, if it is assumed a rockfill is representative of a very poor quality rock mass, the exponent, a, should approach 0.9 to 0.95 (with m_i approximately 2.4-2.7) as GSI approaches zero. The current Hoek-Brown criterion assumes for most rock masses a is 0.5 and limits a to approximately 0.62 for a very poor quality rock mass. A problem with simply modifying a is that a and m_i (or m_b) are interrelated.

The author has developed a new method of determining the parameters in the Hoek-Brown criterion to overcome these problems. It is strongly suggested that the intact rock parameters m_i & a_i should be obtained using triaxial testing and statistical methods discussed in Chapter 3. The author has provided approximate methods of determining m_i and a_i where no triaxial test results are available. Equations have been derived for rock mass to address the limit ($a_b \approx 0.95$, $m_b \approx 2.5$, $s_b = 0$) of very poor quality rock masses. The equations developed allow for a reduction in m_b from m_i (and associated increase from a_b from a_i) and s_b from s_i to this limit. A summary of the method is presented below.

The basic form of the shear strength equation remains unchanged from the Hoek-Brown criterion.

$$\sigma'_1 = \sigma'_3 + \sigma_{ci}\left(\frac{m\sigma'_3}{\sigma_{ci}} + s\right)^a \qquad (7.5)$$

For intact rock $m = m_i$ and $\alpha = \alpha_i$. These should preferably be measured from triaxial tests on intact rock samples. Alternatively an approximation can be made using the uniaxial compressive strength, σ_{ci}, and tensile strength, σ_{ti}, of the intact rock and the equations below.

$$m_i = \left|\frac{\sigma_{ci}}{\sigma_{ti}}\right| \tag{7.6}$$

$$\alpha_i = 0.4 + \frac{1.2}{1 + \exp\left(\frac{m_i}{7}\right)} \tag{7.7}$$

The estimation of m_b, α_b and s_b can be made using the following equations:

$$m_b = \min\begin{cases} m_i \dfrac{GSI}{100} \\ 2.5 \end{cases} \tag{7.8}$$

$$\alpha_b = \alpha_i + (0.9 - \alpha_i)\exp\left(\frac{75 - 30m_b}{m_i}\right) \tag{7.9}$$

$$s_b = \min\begin{cases} \exp\left(\dfrac{(GSI - 85)}{15}\right) \\ 1 \end{cases} \tag{7.10}$$

The equations presented by Hoek et al (2002) can be used to estimate the cohesion, *c*, and friction angle, ϕ, of the rock mass, as the form of the Hoek-Brown equation has not been changed.

7.2 RECOMMENDATIONS FOR FURTHER RESEARCH

7.2.1 The Analysis of Concrete and Masonry Dams

The analysis of CONGDATA and the method for predicting probabilities of failure of concrete and masonry dams is based on field data of varying quality. Further detailed analysis of new incidents would improve the confidence in the conclusions presented in this thesis. Further detailed information on the geology of the foundations of dams would allow a better prediction of the likelihood of failure. Research into the effectiveness of monitoring and warning systems using the outcomes from this thesis would be of value.

The author believes that it is better to do a probabilistic analysis of stability modelling uncertainty in the geology, shear strengths of the foundation and foundation pore pressures (uplift) as modified by grouting and drainage.

Where large defects exist below a dam the shear strength in the foundation will be governed by these defects. As discussed in Section 7.2.2 the shear strength of field scale defects is still poorly understood. Further work in this area is required. Studies could examine large-scale failures, preferably with insitu shear tests and laboratory scale shear tests for comparison. However, these would be limited in number and quality. Alternatively studies could use numerical modelling to look at the effect of increasing the scale of defects. The use of the program *PFC* which models the movement and interaction of circular particles by the distinct element method has shown some promise in this area.

7.2.2 The Shear Strength of Rock Masses

The methods presented for estimating the shear strength of intact rock and rockfill are based on substantial databases. These provide good bounds on the shear strength of rock masses. The development of the equations for estimating the strength of the transitional rock masses is based on a limited amount of field and laboratory data. Further analysis and reporting of well-documented failures and lab testing of rock masses of varying quality would improve the confidence in the results presented in this thesis and would also provide a better understanding of the degree of uncertainty in the results obtained by the equations presented in this thesis. The publishing of more data on failures would also assist in improving the slope design curves presented in this thesis.

The equations for rock masses provided in this paper are principally for cohesionless rock masses. Further work could be carried out to assess cohesive rock masses. Modifications to the parameter s based on cohesive properties of the rock mass would allow the Hoek-Brown criterion to better model these types of rock masses at low confining stresses. This would be of value for predicting strengths for pit slope benches.

The effect of the intermediate principal stress, σ'_2, could be incorporated into the equations for intact rock and ultimately rock masses. The use of Lade's (1993) work would be of benefit here.