Deflections of Upstream Membrane of Rockfill Dams During Reservoir Filling

Fernando Saboya Jr., Pedricto Rocha Filho, Arthur D.M. Penmam

Abstract. This paper describes a numerical investigation into the deflection of the upstream membrane of rockfill dams during reservoir impounding. The influence of the angle of the upstream slope on the subsequent deflection suffered by the concrete face slab is investigated. The procedure used for this investigation takes into consideration the gradual change between reloading and primary loading elastic modulus based on stress-state criterion.
Key words: rockfill dam, face deflection, instrumentation.

1. Introduction

De Mello (1982) and others have criticized some of the flatter slopes currently being designed for concrete faced rockfill dams, pointing out that in the past, some rockfill dams were designed with upstream slopes as steep as 60° to the horizontal. Modern compacted rockfill should be able to sustain even steeper slopes, and it was noted that an upstream slope of 65° to the horizontal was used for the Malpasso Dam in Peru. In a potentially highly seismic area of Japan, the Tarumizu Dam was built with an upstream slope of only 15° to the horizontal.

In view of these wide variations, this study was initiated to determine the effect of upstream slope angle on the deflections caused by the pressure from the impounded reservoir water on the upstream membrane. For comparison the actual measured deflections of three CFRD have been included in the study.

In general, it is intended to minimize slope deflections under the imposed hydraulic forces from the impounded reservoir water by good compaction of the rockfill. The stresses imposed on rockfill by the placement and compaction efforts, are usually exceeded by gravity forces by the time the rockfill has reached full height, before the concrete face slab has been placed. The major principal stress is in a generally vertical direction and the minor principal stress in a generally horizontal direction at regions beneath the centerline. The water load imposed normal to the slab by the filling reservoir, increases the minor stress, so reducing the existing stress difference and causing the slab deflections to be minimized. As the reservoir continues to fill, however, the directions of the principal stresses continue to rotate and the direction of the major principal stress may approach that of the direction of the water thrust on the slab. Thus, the existing rockfill stresses are exceeded, causing slab deflections to increase more rapidly. This is particularly the case because the major principal stress is now acting in a general direction along the placed layers, which is a much weaker direction than the vertical direction in which they were compacted.

Research carried out at the Catholic University of Rio de Janeiro, (Saboya Jr *et al.*, 1993), has shown that the angle of the upstream slope plays a very important role in the subsequent deformations suffered by the concrete face slab, and forms an added factor for detailed consideration at the design stage.

2. Stress-Path During Reservoir Filling

Saboya Jr. & Byrne (1993) and Mori & Pinto (1988) have shown that major part of the upstream shell reaches a shear unloading condition at the beginning of the reservoir filling. Despite the fact that the first stress invariant is increasing, the shear stress decreases because the increase in minor principal stress is higher than that of the major principal stress (Fig. 1). When the complete principal stress axis rotation happens, some points will follow the reloading stress path and as the primary loading condition is reached, these points will behave in a much softer manner, as shown by Saboya Jr. (1993).

The interface between reloading and primary loading condition has to be suitably modeled for the understanding of the influence of upstream slope on the slab deflection. Thus, the model proposed by Saboya Jr. & Byrne (op. cit) will be used. This model states that the mechanism that governs the transition between primary loading and unloading is different from that of the reloading and primary loading. The most usual criterion to establish if a localized zone within the body of the dam is under primary loading or unloading-reloading state is called stress level criterion & given by Eq. (1).

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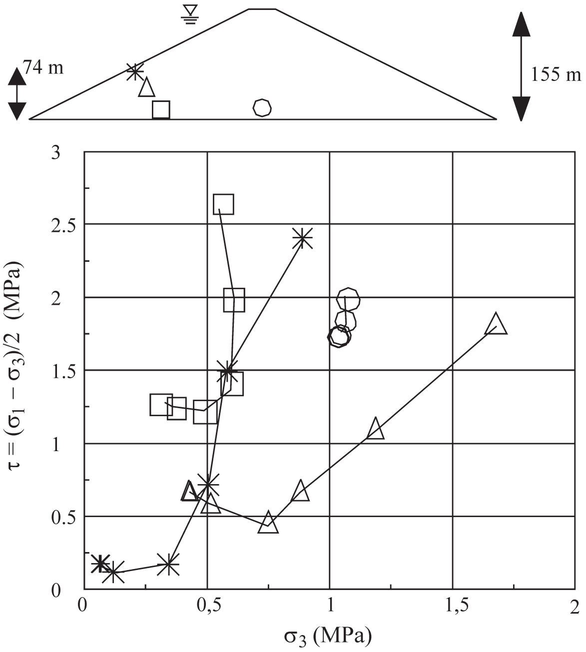

Figura 1 - Stress path during reservoir filling of Foz do Areia Dam (Saboya Jr., 1993).

$$S_L = \frac{(\sigma_1 - \sigma_3)}{(\sigma_1 - \sigma_3)_f} \tag{1}$$

where S_L is the stress level and $(\sigma_1 - \sigma_3)_f$ is the deviator stress at failure.

If the current stress level is equal or higher than the maximum past stress level ever experienced by the element, then it is considered to be in primary loading condition and the loading tangent modulus is used, otherwise the unloading reloading modulus is used.

Duncan *et al.* (1984) show that this criterion should be modified in order to take into account, not only the influence of the stress level, but also the change in confining stress. This criterion, known as the stress-state criterion, is defined as follows:

$$S_S = S_L \left(\frac{\sigma_3}{P_a}\right)^{1/4} \tag{2}$$

where S_S is the stress state and P_a is the atmospheric pressure.

The stress-state criterion considers that the primary loading tangent modulus is used when the current stress state is higher than the maximum past stress state ever experienced by the element. Both criteria are shown in Fig. 2 where the straight and curved lines represent, respectively, the stress level and the stress state criteria.

The main modification proposed by Duncan *et al.* (op. cit) is the gradual transition from primary loading modulus to unloading-reloading modulus. This has been proposed aiming to avoid abrupt change in elastic tangent modulus, which could lead to numerical instability in a finite element program. The gradual change in elastic modulus is based on the critical stress level, above which, the primary loading modulus is used and is given by the following expression:

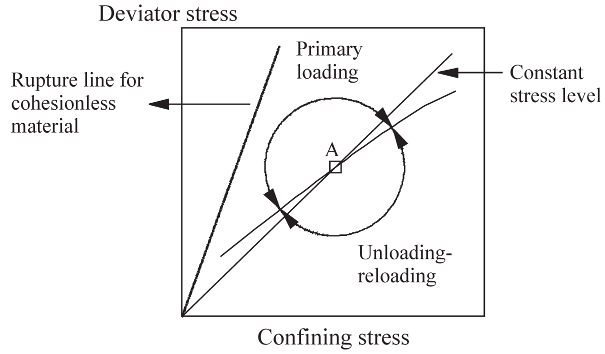

Figura 2 - Stress level and stress state criterion (after Duncan *et al.*, 1984).

$$S_L^{crit} = \frac{S_S^{max.\,past}}{\left(\frac{\sigma_3}{P_a}\right)^{1/4}} \tag{3}$$

The gradual transition between unloading-reloading and primary loading modulus is given when stress level is situated between ¾S_L^{crit} and S_L^{crit}, as shown in Fig. 3a.

However, Saboya Jr. & Byrne (op. cit), figured out that such a gradual change holds true only for the reloading phase and the modification proposed by Duncan *et al.* (op. cit) strongly overestimates the predicted slab deflection. Thus the model indicated in Fig. 3b has been proposed. It is worth noting that the change from primary loading to the unloading phase is considered to occur in an abrupt way and the change from reloading to the primary loading occurs gradually, using S_L^{crit} as a criterion. This seems to be closer to the actual behavior of rockfill materials.

Figure 4 shows the predicted and observed vertical displacements of Foz do Areia Dam using both criteria. It can be noticed that gradual change for the elastic modulus strongly overestimated the predicted displacement. However, the most important feature is the small influence of the

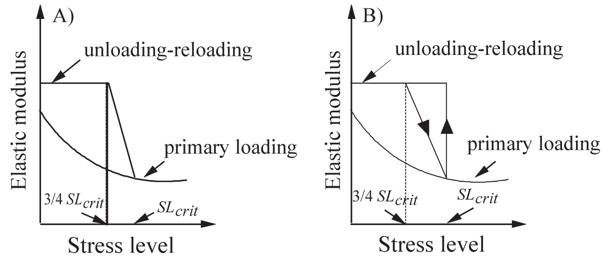

Figura 3 - Loading and unloading criteria: a) Proposed by Duncan *et al.* (1984); b) Proposed by Saboya Jr. & Byrne (1993).

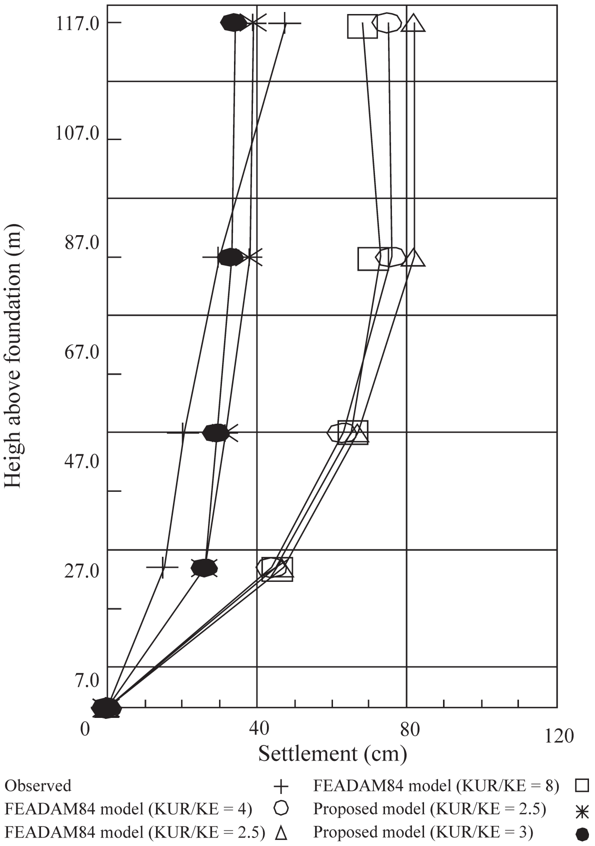

Figura 4 - Predicted and observed vertical displacement under Foz do Areia dam axis (Saboya Jr. & Byrne, 1993).

relationship of unloading-reloading and primary loading hyperbolic parameters, K_{UR} and K_E respectively. This seems to be related to the fact that most parts of the upstream shell might be located at the transition to the unloading zone, during the reservoir filling.

3. Influence of Upstream Slope on the Face Deflection

Hence, it can be said that slab deflection is a function of how the material will respond to different stress paths combined with its initial stress state. As the stress paths, imposed by the reservoir filling, are dependent on the principal stress increment, one can say that the angle of the upstream slope plays a very important role on the final displacements.

To verify such an influence, a hypothetical 100 m high dam with upstream slopes varying from 2V:1H to 1V:3H, was analyzed. These analyses involved both construction and reservoir filling stages. The simulation of the construction stage was necessary, because the determination of the final state of stress is very important. The response of the dam due to water thrust will depend strongly on the initial state of stress that represents the final one obtained from the construction analysis.

A nonlinear elastic hyperbolic model was used and the parameters were derived using the methodology proposed by Saboya Jr. & Byrne for Segredo rockfill material IB. These parameters are presented in Table 1. The analysis were carried out using FEADAM84 computer code (Duncan *et al.*, 1984) and modified by Saboya (1993).

The reservoir filling was simulated in five steps of 20 m each, in order to reveal the stress path followed by the elements during reservoir filling. The most favorable condition will be considered to be that for which the dam shows the highest percentage of elements in the unloading condition at the final load step. In this case, the dam tends to present a stiffer behavior, resulting in smaller slab deflections.

One can say that, for steeper slopes, the increase in minor principal stress is higher than for flatter slopes, leading, therefore, to an earlier unloading situation. However, such a statement might not be true if the face has a certain slope where a most part of the upstream shell comes back earlier to its primary loading conditions. On the other hand, the same reasoning can be applied to flatter slopes, where the percentage of the upstream shell area in the unloading condition, due to smaller increments in minor principal stress, can be very small. Therefore, it can be concluded that there is an "optimum" slope, independent of the height of the dam, in which the combination among the percentage of unloading, reloading and primary loading reaches the most favorable condition.

Figure 5 shows the results obtained for maximum deflection versus face slope. It can be seen that the best slope, considering only a single value of maximum deflection, is about 1V:1H (45 degrees). However, this value alone is not enough to define the most suitable slope. It is very important to emphasize that the best behavior of the face must be related to the smallest deflection gradient. The deflection gradient can be understood as the difference in deflection at different elevations. It can be seen in Fig. 6 that the angle for which the smallest deflection gradient takes place is also 45 degrees.

It is worthwhile also to show the development of the face displacements as a function of principal stress axis rotation. For this reason, two different elevations of the upstream face, were considered: 20% and 40% of the total

Tabela 1 - Hyperbolic parameters used in the analysis (Saboya Jr. 1993).

Material	C_U [1]	K_E	n	R_f	K_B	m	ϕ	$\Delta\phi$
IB	6	350	.37	.60	100	.13	47	8.3

[1]Uniformity coefficient.

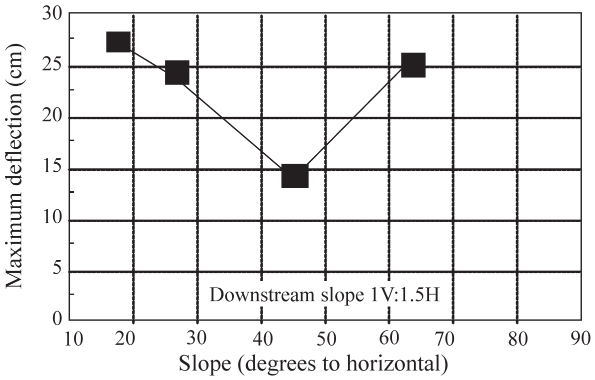

Figura 5 - Maximum deflection *vs.* upstream slope.

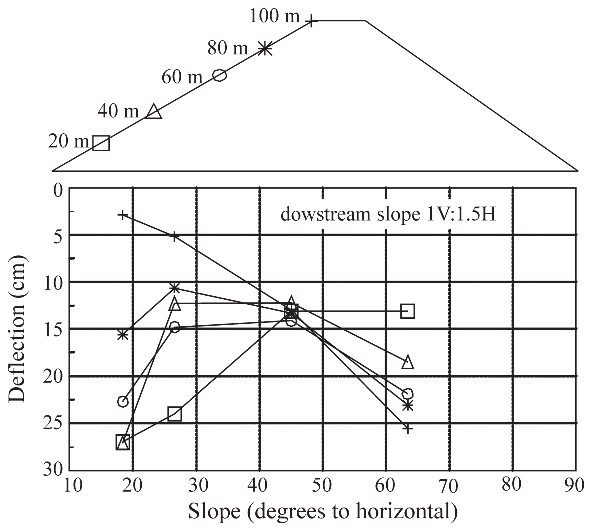

Figura 6 - Differential deflection at different levels.

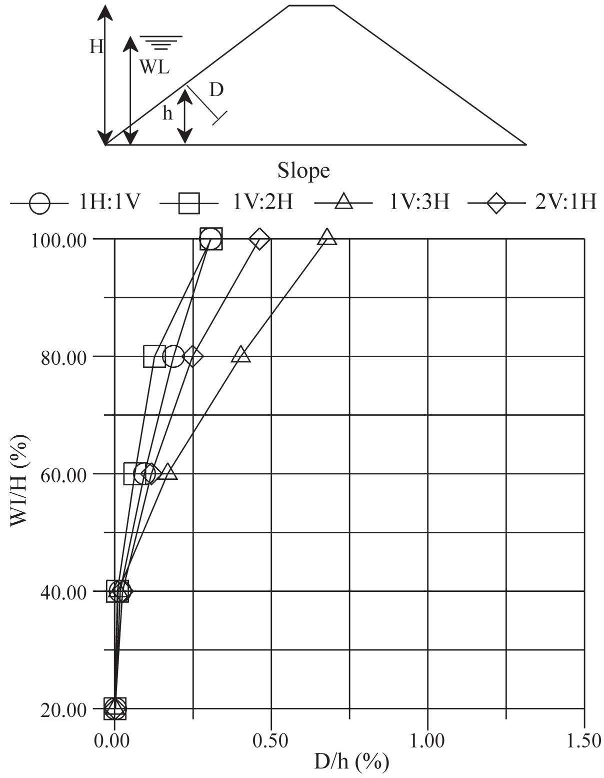

Figura 7 - Non-dimensional deflection at El. 20 m.

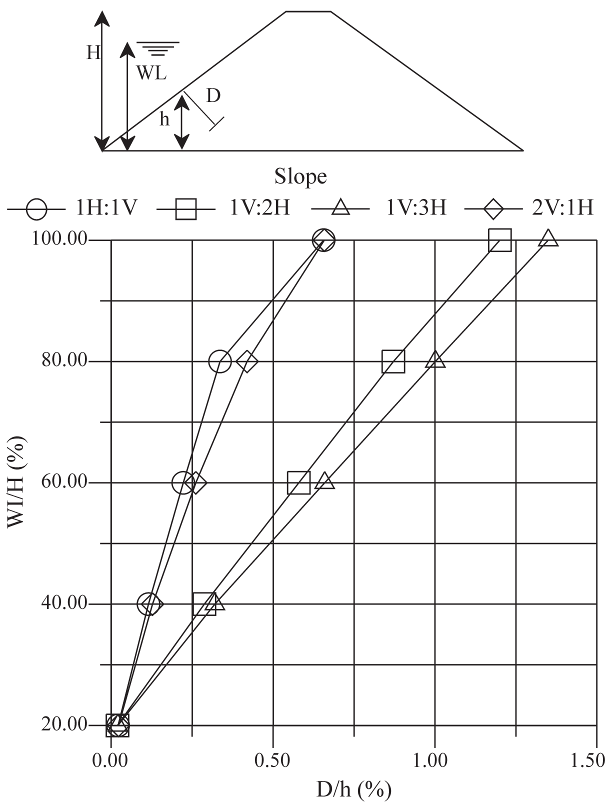

Figura 8 - Non-dimensional deflection at El. 40 m.

height of the dam H. Deflections were plotted in a dimensionless way to make them independent of the height of the dam (Figs. 7 and 8).

3.1. Deflection at El. 20% of the total high of the dam

By analyzing the face deflection of a point on the face located at El 20% of H (Fig. 7), it can be noticed that the slopes representing 1V:1H and 2V:1H indicate similar behavior, showing a pronounced increase in the deflection rate when the reservoir level reached 80% of total height of the dam. This seems to be linked to the change between unloading-reloading and the primary loading phase. For slopes of 1V:2H and 1V:3H, it seems that, at that elevation, points never reached the unloading situation, maybe due to small initial stress level (in terms of shear stress) and high load increments in their vicinity. It can be seen that, after reaching the primary loading state, the lines tend to be parallel, indicating similar elastic modulii. In fact, they are not precisely parallel because the confining stresses are different for each curve.

3.2. Deflection at El. 40% of the total high of the dam

Figure 8 shows deflections obtained to the point located at El. 40% of H and, as can be seen, the observed behaviour is quite different from that at El. 20% of H, mainly due to the stress state reached during the construction phase. This can explain the good response of the 1V:2H slope where at this elevation the stress level is enough to "hold" the unloading-reloading situation until the reservoir level reaches 80% of the full height of the dam. Similar behaviour is shown by the 2V:1H slope where the deflection rate increases after the reservoir level reaches 80% of the full height of the dam. The 1V:3H slope reaches the primary loading condition at 60% of the dam height, indicating that the stress level at the beginning of the reservoir filling was quite small when compared with the load increments. As noticed for elevation 20% of H, the lines which have reached the primary loading condition, are approximately parallel, and the line representing the 1V:1H slope never reached the primary loading condition. The upstream slope deformed shapes of the hypothetical dams are depicted in Fig. 9. As can be seen, the 1V:1H slope shows smoothest deformed shape due to the fact that most part of the upstream shell is under unloading/reloading condition. The same can be verified for the 2V:1H slope.

3.3. Points of maximum deflection and comparison with Foz do Areia, Segredo and Xingo dams

So far, the evolution of the deflection occurring at fixed elevations for different slopes has been shown. Nevertheless, one might find it of interest to consider only the points of maximum deflection for each slope. This is shown in Fig. 10 and it is quite interesting to notice that the 1V:1H and 2V:1H slopes presented better behavior than the others. The less favorable adopted slope is 1V:3H because of the small value of minor principal stress at the end of the construction period. However, the most interesting feature presented by the plot is that the loci of the maximum deflection are different for each slope studied. In some sense, it cannot be said that the point of the maximum deflection is related to the point of the load resultant, which is always located at one third of water load triangle from the base. The same explanation does not hold true for the direction of the resultant deflection. For instance, the 2V:1H slope had its point of maximum deflection at El. 100% of H, *i.e.*, at the crest. Observed membrane deflections of Foz do Areia, Segredo and Xingo dams were inserted in this plot in order to verify its applicability. It is interesting to notice that for Segredo and Xingo dam, which have upstream slope of 1V:1.3H, their non-dimensional deflections are quite similar to those of the hypothetical dam. As for Foz do Areia dam, despite the fact its upstream slope is 1V:1.4H, the behavior of upstream slope was indeed unexpected. However the main reason for this behavior is that horizontal displacements were not recorded for Foz do Areia Dam and deflections were extrapolated by considering the normal projection of the vertical displacement at the face. This, of course, can lead to overestimated displacements.

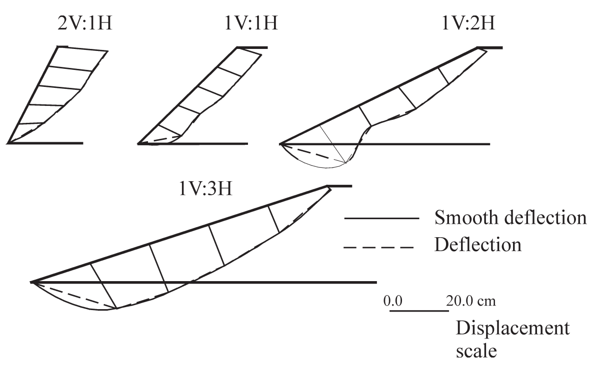

Figura 9 - Deformed shape of the upstream face after the reservoir filling.

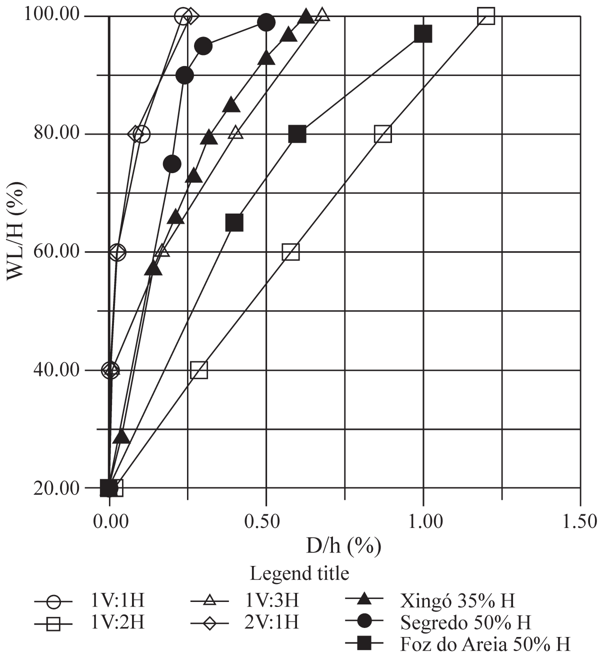

Figura 10 - Comparative evolution of non-dimensional deflections.

The elevation of the points of maximum deflection is supposed to be strongly related to the development of zones under unloading-reloading conditions. To verify such a statement, one can judge it necessary to evaluate the stress state at the end of reservoir filling and try to establish some link between stress-state and deflections. Figure 11 shows the stress state resulting from reservoir filling for these hypothetical dams. As can be seen, embankments with upstream slope varying from 1V:2H to 1V:3H show no elements in the unloading-reloading condition, in the face and, therefore, the non-dimensional deflection curve does not present any point of inflection.

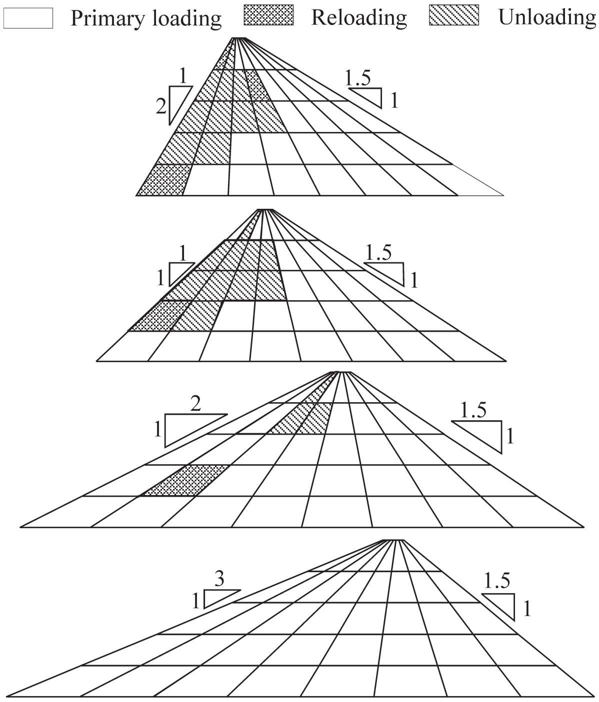

Figura 11 - Stress-state conditions at the end of reservoir filling.

4. Conclusion

The adoption of steeper upstream slopes in concrete face rockfill dams, seeking for the most economic geometry, is a very important task. The importance of this aspect is mainly due to the fact that slope deflections are closely related to the stress-state at the beginning of reservoir filling. Furthermore, the slope angle plays a crucial role on the stress-state reached at the end of construction phase. Thus, the use of elastic analysis for the prediction of face deflection, must incorporate the effects of principal stress axis rotation. Otherwise, the increase on deformation rate, as the reservoir is filling, cannot be modeled, unless more sophisticated elasto-plastic models are used.

Gradual change in elastic modulus during unloading-reloading strongly overestimates such predictions. On the other hand, gradual change in elastic modulus during the reloading curve, seems to be quite suitable in simulating the actual behavior of rockfill dams. These features can be easily incorporated in the hyperbolic model and the predicted responses are quite satisfactory. Results from this research have shown that at slope angles between 1V:1H and 2V:1H, deflections are much smaller than for others angles.

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Notation

C_U = Uniformity coefficient
K_E = Elastic modulus hyperbolic parameter
K_B = Bulk modulus hyperbolic parameter
n, m = Hyperbolic exponent parameters
ϕ = shear strength internal angle
$\Delta\phi$ = Decrease do ϕ for a log cycle of confining stress
R_f = Stress ratio hyperbolic parameter
K_{UR} = Unloading-reloading elastic modulus hyperbolic parameter