RAPID DRAWDOWN IN DRAINAGE CHANNELS WITH EARTHEN SIDE SLOPES [1]

David J. Kerkes, Ph.D., P.E. [2] and Jeffrey B. Fassett, P.E. [3]

ABSTRACT

The rapid drawdown case is one of the most severe loading conditions that an earthen slope can experience and is quite common in storm water drainage channels along the Texas Gulf Coast. Flooding in adjacent rivers can leave water levels high in these drainage channels, which can drop relatively rapidly once floodwaters recede. While the development of deep seated failure surfaces is possible, the effect on earthen side slopes is most commonly seen in the form of relatively shallow slope failures, which if left unattended lead to the gradual deterioration of the channel slopes. Though the information presented in this paper is generally applicable to any earthen side slopes, of particular interest are slopes consisting of overconsolidated clays where it is not uncommon for slope failures to occur long after a channel has been constructed. This paper discusses a number of the factors associated with this loading condition, as well as some of the design issues, and presents the results of a series of finite element seepage analyses that were performed to investigate the effect of some of the variables on the advance of the zone of saturation into the channel side slope.

METHODS OF ANALYSIS & SOIL STRENGTH CONSIDERATIONS

The rapid drawdown case has long been recognized as one of the most severe loadings conditions that a slope can be subjected to and it is well documented in the literature. The condition is perhaps most commonly associated with the upstream slope of embankment dams; however, failures are also very common in natural and man-made slopes along rivers and man-made drainage channels as a result of flooding. Flood events can leave water levels high in rivers and drainage channels for significant periods of time and then drop relatively rapidly once the floodwaters recede. The effect of this inundation on the soil in the slope, both prior to and subsequent to drawdown, is the essence of the rapid drawdown loading condition. Therefore, to understand rapid drawdown one must consider what is happening to the soil in the slope, both in terms of soil strength and pore pressure development.

The rapid drawdown loading condition has been analyzed using two different methods: (1) the total stress method, and (2) the effective stress method. There appears to be a misconception that

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[2] Consulting Geotechnical Engineer, 1311 Mustang Trail, Houston, TX 77339 tel. 281-358-5155

[3] Associate, Golder Associates Inc., 500 Century Plaza Dr, Suite 190 Houston, TX 77073 tel. 281-821-6868

the effective stress method is only appropriate for long term (drained) loading conditions, and since rapid drawdown is a short term (undrained) loading condition the case should be analyzed using undrained shear strengths. In fact, the effective stress method can be applied to both long term and short term loading. The difficulty in applying the effective stress method to the short term case is in defining the pore pressures that exist for the particular loading condition. It is often assumed that this difficulty can be avoided by using an undrained shear strength and performing a total stress analysis; however, this approach is not as simple as it may first appear.

The mistake that is most frequently made is that an incorrect value for the undrained shear strength is used. It has long been recognized that the clay soils in the Beaumont Formation generally tend to be highly overconsolidated, often cemented and typically highly expansive. The dessication process that led to the high degree of overconsolidation also left the clay soils extensively slickensided, although the orientation of these slickensides generally tends to be random. The combination of overconsolidation, cementation, and soil suction in unsaturated samples often yields very high values of undrained shear strength as determined from the conventional unconfined compression test, which is perhaps the most common test used to determine undrained shear strength. These high undrained shear strength values often give a false sense of confidence in the long term strength and performance of excavations in the clay soils of the Beaumont Formation. The determination of an appropriate value for undrained shear strength to use in a rapid drawdown analysis is much more complicated than many realize.

In order to use an undrained shear strength the soil sample must first be brought to, and allowed to come to equilibrium under the stress state that exists in the slope prior to rapid drawdown. The stress state of the soil will change throughout the slope in response to the unloading that will occur as a result of excavation for a drainage channel. In addition, consideration must be given to the extent to which the soil's degree of saturation has changed under the elevated water level in the channel, as well as any softening due to swell in clays of high plasticity. Then the laboratory test must replicate in the test sample the same change in stresses that occurs in the slope during rapid drawdown loading. The soil's strength is still governed by the effective stress state of the sample, and if the loading conditions are properly replicated, then in theory the same change in pore pressures will occur in the test sample as in the field and all of this will be reflected in the undrained shear strength obtained from the test. Unfortunately, another fact that complicates the analysis is that prior to and during rapid drawdown both the initial stress state and change in pore pressure vary throughout the slope in the field. Consequently, a single value of undrained shear strength is not appropriate for the entire slope.

In considering the pore pressure changes in the soil it may not be sufficient to simply examine the distribution of pore pressure in the slope immediately after the exterior water level drops. Consideration may also have to be given to the manner in which the pore pressure changes in the soil as a result of the shear stresses applied during loading. In a saturated soil that tends to experience a volume increase or dilate as it shears (dense or overconsolidated soils) there will be a reduction in pore pressure resulting in an increase in effective stresses, whereas the pore pressures will tend to increase in saturated soils that undergo a volume reduction during shear (loose or normally consolidated soils). However, the decrease in pore pressures and attendant increase in effective stresses in dilative soils cannot be relied upon if cavitation or drainage occurs during loading. The reader is referred to the paper by Duncan, Wright, and Wong (1990) for a good discussion on the selection of strength parameters for a total stress analysis of the rapid drawdown case.

To perform a total stress analysis that properly considers the factors governing undrained shear strength is not a simple matter, and one may find that it is basically no more complicated to perform an effective stress analysis. One of the advantages of an effective stress analysis is that the key components of the analysis can be considered individually. Just as with a total stress analysis, selection of appropriate strength parameters is critical to an effective stress analysis. There is significant evidence to indicate that the shear strength mobilized in the long term stability of stiff clays is actually less than the values measured in the laboratory, and an excellent treatment of this subject is given by Mesri and Shahien (2003). While their work focuses on values of friction angle, the conclusions reached by Mesri and Shahien are not inconsistent with the behavior observed in slopes constructed in the clay soils of the Beaumont Formation. They observe that swelling, softening, and creep contribute to the development of the residual condition, even in slopes that have never previously experienced any slope movement. Similar observations were made by Stark and Duncan (1991) in their review of a slide that occurred in the upstream slope of San Luis Dam subsequent to drawdown. Stark and Duncan concluded that the failure was attributable to strength reductions that occurred when a stiff clay of high plasticity in the foundation was first softened upon wetting, then subjected to cyclic loading associated with multiple occurrences of reservoir drawdown over a period of some 14 years. Consequently, just as with a total stress analysis it is incumbent on the designer to consider the manner in which the effective stress strength parameters are likely to change over the life of the project.

Perhaps the most difficult aspect of an effective stress analysis is the definition of the pore water pressures that exist at the time of rapid drawdown, which may be the major reason that some designers prefer a total stress approach. While there are a number of factors that complicate this aspect of the analysis, the fundamental elements of seepage and pore pressures are not difficult to understand. In fact, an understanding of these basic concepts enables the designer to make some simplifying assumptions that can expedite the analyses. The following section addresses this aspect of the rapid drawdown loading condition.

FLOW NET BASICS AND SEEPAGE ANALYSES

One reason that a total stress analysis may seem appealing to some is the idea of avoiding the subject of pore pressure distribution associated with drainage of the slope, independent of the pore pressures that may arise as a result of shear strain. With the advent of computerized numerical methods this problem need not be as intimidating as it once was, when the solution necessitated the development of a drawdown flow net. Nevertheless, the principles that apply to a flow net solution apply equally to a computerized numerical solution. Consequently, an understanding of these principles is still necessary for a complete understanding of the problem. Figure 1 illustrates the rapid drawdown flow net for the case of a completely saturated earthen slope over an impervious foundation. The flow net illustrates the seepage pattern after instantaneous drawdown, just as the slope begins to drain. Note that the uppermost equipotential line is still at the same elevation as the water level prior to drawdown. Naturally, the degree to which the slope drains during drawdown is a function of the permeability of the soil in the slope and the rate at which the water level drops, but the flow net in Figure 1 illustrates the worst case, where no drainage has actually taken place. The reader will recall that the total head (H_t) along each equipotential line is constant and equal to the sum of the elevation head (h_e) and pressure head (h_p) at each respective point along the equipotential line, which is expressed in Equation 1.

$$H_t = h_e + h_p \qquad \text{Eq. 1}$$

The reader will also recall that the change in total head (ΔH_t) between equipotential lines is a constant in any given flow net. Consequently, the distance between points where equipotential lines intersect the face of the slope must also be constant and equal to the change in elevation head (Δh_e) between the respective points. The reason for this is that after drawdown the pressure head (h_p) at all points along the face of the slope is zero gauge (atmospheric pressure); therefore, from Equation 1, the magnitude of the total head (H_t) must be equal to the elevation head (h_e) at the point where the equipotential line intersects the face of the slope.

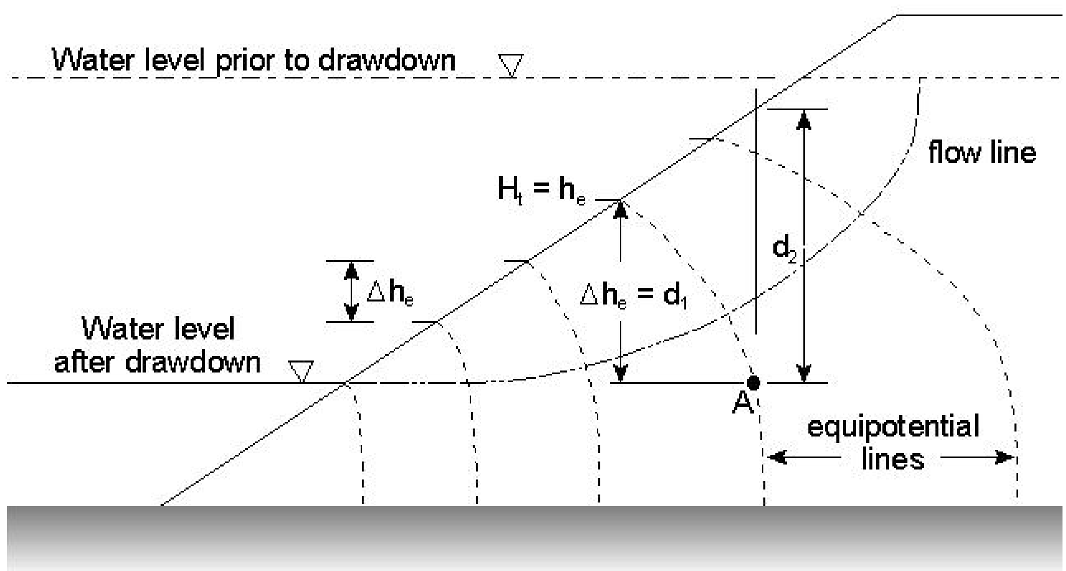


Figure 1: Flownet for a saturated slope after instantaneous drawdown.

Referring again to Figure 1, consider Point A, which is located a distance d_1 below the intersection of the slope face and the equipotential line on which Point A lies. From the previous discussion it has been established that the value of the total head (H_t) along that equipotential line is equal to the elevation head (h_e) where the equipotential line intersects the face of the slope, and the magnitude of the elevation head at Point A ($h_{e\,@\,A}$) is simply h_e - d_1; therefore, the pressure head at Point A ($h_{p\,@\,A}$) can be computed from Equation 1 simply as:

$$h_{p\,@\,A} = H_{t\,@\,A} - h_{e\,@\,A} = h_e - (h_e - d_1) = d_1 \qquad \text{Eq. 2}$$

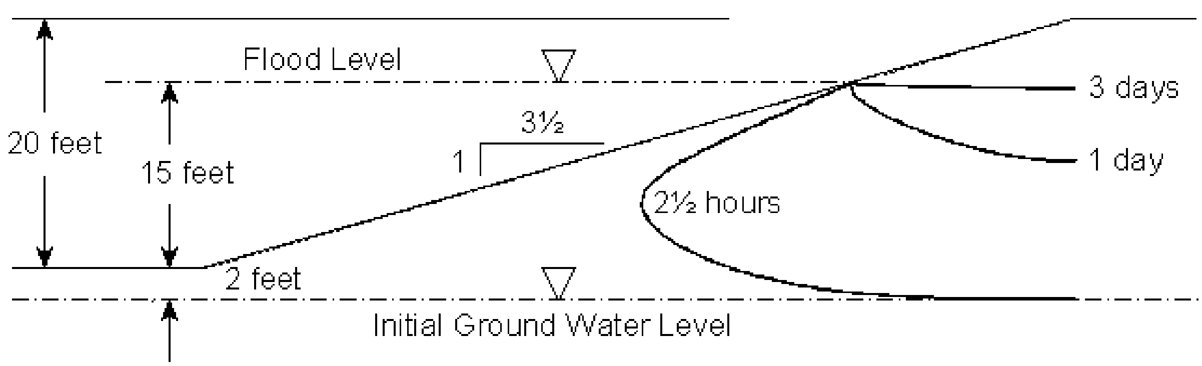
The reader will recall that pore pressure is simply the product of pressure head (h_p) and the unit weight of water. Thus, while the exercise of drawing a flow net may be a difficult one, it is a straightforward process to compute the pore pressure from a flow net for any point in a slope as that slope drains. However, it is not a straightforward matter to provide this information as input data for a computerized slope stability analysis. Consequently, a simplified approach is sometimes used in computerized slope stability analyses to estimate pore pressures due to rapid drawdown, which consists of defining a piezometric surface that runs up along the face of the slope and then into the slope at the elevation of the water level prior to drawdown. In this manner the pore pressure at Point A in Figure 1 would be computed simply, and conservatively, as the vertical distance (d_2) between Point A and the face of the slope directly above the point. From an examination of the flownet in Figure 1 the reader can see that this approximation is not too bad at points near the face of the slope and near the toe of the slope, but deep within the slope, such as Point A, the approximation can be quite conservative. In this particular example the approximate value is about fifty percent greater than the actual pore pressure. The approximation is all the more conservative when applied to drainage channels with earthen side slopes, as opposed to embankment dams, because the entire channel slope may not become completely saturated over the period of time during which the water level in the channel is at flood stage. Consequently, if it is assumed simply that the slope if fully saturated, then overestimates of pore pressures deep in the slope can result in overly conservative safety factors for deep seated failure surfaces.

It is relevant to note that an analysis of rapid drawdown will often yield critical potential failure surfaces that are very shallow, approaching that of the infinite slope case with seepage parallel to the slope. While an argument might be made to consider a lower factor of safety for such a surface it is important for the designer to also investigate deep seated failure surfaces in the slope to ensure that the minimum safety factors are within the limits normally accepted for surfaces where the consequences of failure are more significant. For this reason it is important that the estimated pore pressures associated with deep seated potential failure surfaces not be unduly conservative, since a decision to either reinforce a slope or flatten the overall slope can have significant cost consequences. Therefore, the designer should consider the degree to which the zone of saturation is likely to advance over the period during which the channel slope is flooded. However, defining the pore pressure distribution deep within an earthen slope as a result of periodic flooding is by no means a simple task.

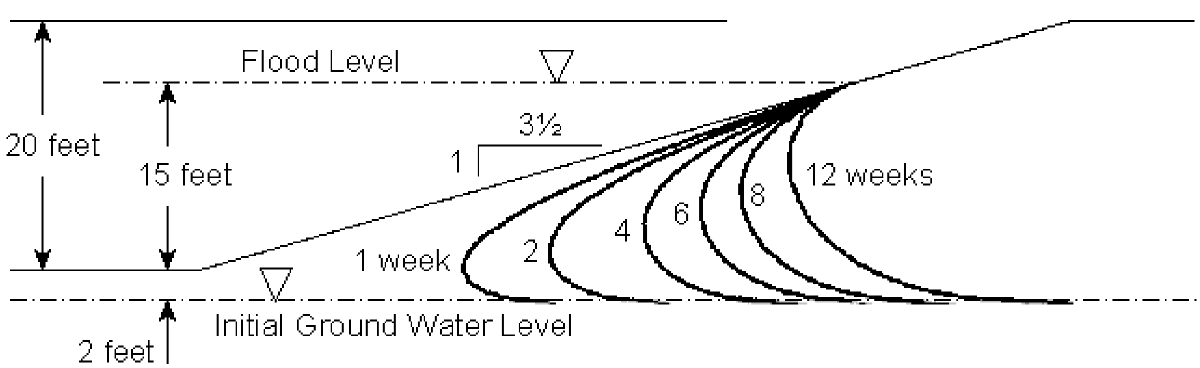
A number of factors will influence the location of the zone of saturation in a channel side slope as a result of flooding, but the primary factors will be: (1) permeability of the material in the slope, (2) location of the groundwater level in the slope prior to inundation, (3) initial degree of saturation in the unsaturated areas, and (4) the length of time that the water level in the channel remains at flood stage. Defining any one of those factors is difficult enough, but in combination it quickly becomes apparent that the problem is intractable (if not impossible).

Perhaps the best a designer can do is to perform a parametric study to determine the sensitivity of the analyses to the variables involved. With that in mind, to provide some insight into the extent of the advance of a zone of saturation into the earthen side slopes of a drainage channel, a series of finite element analyses were performed. The analyses were performed using the finite element program SEEP/W, which is a software product produced by GEO-SLOPE International

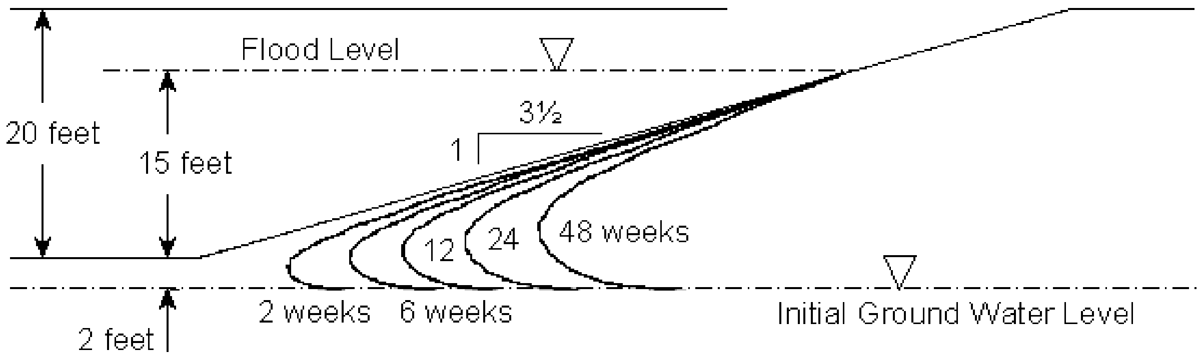
Ltd. (www.geo-slope.com). In addition to traditional steady state saturated flow analysis, the saturated/unsaturated formulation of SEEP/W makes it possible to analyze the migration of a wetting front. The analyses considered a slope height of 20 feet at an inclination of 3½ horizontal to 1 vertical. The assumed maximum water level in the channel at flood stage was taken as 15 feet above the channel invert, with the initial ground water level 2 feet below the bottom of the channel. A minimum initial degree of saturation of 80 percent was assumed for the slope and three values for soil permeability were considered: 10^{-3}, 10^{-6}, and 10^{-8} feet per minute (5×10^{-4}, 5×10^{-7}, and 5×10^{-9} centimeters per second). The advance of the zone of saturation was then determined as a function of time. The results of the analyses are presented in Figures 2, 3, and 4. It is important to note that the analyses assume a uniform permeability throughout the slope. In addition to the obvious fact that permeability is not expected to be perfectly uniform in an actual slope, a factor perhaps more important is that cracks are likely to develop in the face of the slope, particularly in clays of high plasticity, which will significantly impact the advance of the zone of saturation. These cracks can develop to depths of several feet, and one approach to dealing with this factor is to simply shift the zone of saturation into the slope an amount proportionate to the estimated depth of cracking.



Figures 2: Position of the zone of saturation as a function of time, with the water level in the channel at flood stage, for a soil permeability of 10^{-3} feet per minute.



Figures 3: Position of the zone of saturation as a function of time, with the water level in the channel at flood stage, for a soil permeability of 10^{-6} feet per minute.



Figures 4: Position of the zone of saturation as a function of time, with the water level in the channel at flood stage, for a soil permeability of 10^{-8} feet per minute.

It is hoped that the information presented in Figures 2, 3, and 4 will be of some assistance to designers in arriving at reasonable assumptions for the position of the phreatic surface within a slope after a period of sustained flooding in a channel or river. The information can then be used to perform an effective stress analysis of the stability of the slope using the simplified method for estimating pore pressures previously described, wherein the piezometric surface is extended up the face of the slope and then follows the position of the phreatic surface within the slope. Unfortunately, most computerized slope stability programs will only allow the piezometric surface to be defined in terms of increasing x-coordinates. That is, a piezometric surface cannot curve back upon itself because this results in x-coordinates along the piezometric surface having more than one value for the corresponding y-coordinates. The problem, however, is easily solved by using one or more values of pore pressure ratio in combination with a piezometric surface, where the pore pressure ratio (r_u) is simply the ratio of the pore pressure (u) to the total vertical stress (F_v) at any given point:

$$r_u = \frac{u}{\sigma_v} \qquad \text{Eq. 3}$$

For the purpose of the approximate method, the pore pressure is simply the product of the unit weight of water and the depth of the point below the face of the slope. The total vertical stress at that same point is computed as the product of the saturated unit weight of soil and the depth of the point below the face of the slope. For example, referring again to Figure 1, the pore pressure at Point A would simply be the product of the unit weight of water and the vertical distance d_2, and the total vertical stress at Point A would be the product of the saturated unit weight of soil and the vertical distance d_2; therefore, the pore pressure ratio for the rapid drawdown approximation becomes simply the ratio of the unit weight of water to the saturated unit weight of soil. Consequently, the pore pressure ratio for the rapid drawdown approximation generally results in a value around 0.5, since the saturated unit weight of soil is typically about twice the value of the unit weight of water. Thus a soil with a saturated unit weight of 130 pounds per cubic foot would yield a value of 0.48 for the pore pressure ratio. Figure 5 illustrates how a channel slope geometry might be defined for a rapid drawdown analysis. A pore pressure ratio would be defined for the soil within the small zone below the face of the slope between the piezometric surface (line) and the edge of the zone of saturation. At all other locations the pore pressures would be defined by the piezometric surface.

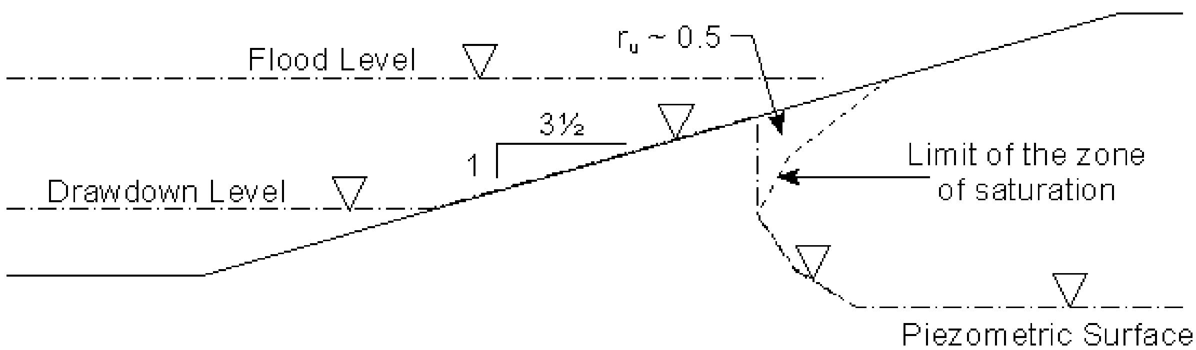


Figure 5: Illustration of the use of a pore pressure ratio and piezometric surface for a rapid drawdown slope stability analysis.

ERRATA (17 April 2006)

Figure 5 as it appears in the conference proceedings contains an error. That error has been corrected in this copy of the paper.

CONCLUSIONS

When designing channel side slopes to withstand rapid drawdown loading the engineer must give careful consideration to the soil strength parameters to use in the analysis, particularly for stiff clays in light of observations made on the long term performance of slopes in such soils. There is also evidence that indicates that repeated occurrences of rapid drawdown constitute a cyclic loading condition that can potentially reduce soil strength in the long term. It is perhaps easiest for the designer to examine variations in soil strength parameters through an effective stress analysis, which permits the pore pressures to be treated separately. While the development and distribution of pore pressures in a slope subjected to sustained flooding and subsequent rapid drawdown is extremely complicated, a simplified approach can be applied that tends to yield conservative results. Even with the simplified approach, however, the designer should give consideration to the probable advance of the saturation front in light of the permeability of the soil within the slope or the analyses may produce unreasonably conservative results.

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