Landslides and Avalanches: ICFL 2005 Norway – Senneset, Flaate & Larsen (eds)
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Displacement analysis of a landslide

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ABSTRACT: An analysis procedure for calculating displacement of a landslide is presented. The proposed procedure is based on continuum-mechanics principles and can be used to calculate: (a) factor-of-safety against the onset of failure, and (b) potential displacements of the slide mass after the onset of failure. The proposed procedure is used to calculate displacements of a landslide in the hills surrounding a reservoir. The calculated displacements compare favorably with the field measurements.

1 INTRODUCTION

In geotechnical engineering practice, an item of primary interest with regard to a slope failure is the onset of failure. Accordingly, numerical analyses of slope stability problems are generally performed using limit-equilibrium-based solution procedures that satisfy equations of static equilibrium (force and/or moment) to calculate factor-of-safety (FoS) against the onset of failure; and an FoS = 1 is generally taken to imply slope failure. An item of similar interest in geotechnical engineering practice is the magnitude of displacement that a slide mass undergoes before coming to rest in a stable configuration. However, limit-equilibrium-based solution procedures are not meant for calculating displacements of the slide mass, just the onset of failure, i.e., the FoS. Solution procedures based on continuum-mechanics principles can be used to calculate FoS and displacements of the slide mass before and after the onset of a slope failure. The objectives of this paper are to present: (a) an effective and efficient continuum-mechanics-based procedure to estimate the post-failure maximum displacement of a slide mass; and (b) an application of the procedure to the Quaternary landslide (Qls)-18 shown in Figure 1. The proposed procedure can be implemented in essentially any continuum-mechanics-based solution procedure; however, for analysis of the field problem presented herein, a commercially available computer program FLAC (Itasca Consulting Group 2000) was used. FLAC is an acronym for Fast Lagrangian Analysis of Continua; it is a two-dimensional explicit finite

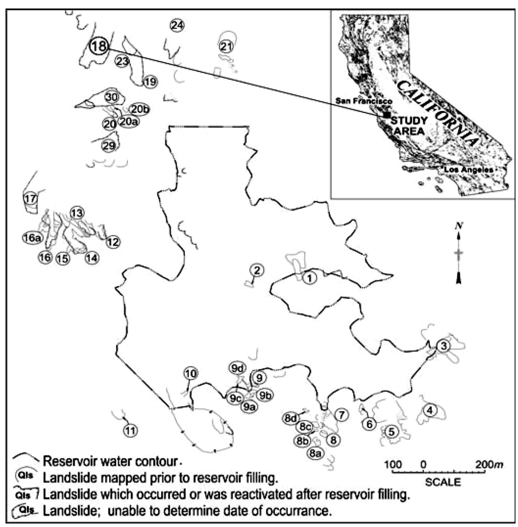

Figure 1. Qls-18 landslide vicinity map.

difference program; and its adoption was for convenience. Because the landslide is being studied after the occurrence of the failure, it provides an ideal situation to gain confidence in the ability and usefulness of numerical analysis procedure(s) to not only match the occurrence of the failure but also the magnitude of displacement of the slide mass using site specific data. It is suggested that even for new slope designs (such as embankments), displacement calculations should be made on a routine basis to understand how far a potential slide mass can displace in the local environs before acquiring a stable configuration. It is anticipated that the procedure and information presented in this paper will be of interest to geotechnical engineers who have interest in landslides and slope stability problems.

2 PROPOSED PROCEDURE

2.1 *FoS and failure surface calculation*

The proposed procedure for calculating FoS and locating the critical failure surface using the continuum-mechanics approach has the following steps: (a) Create a discretized model of the continuum encompassing the slope; (b) Define the appropriate pore water pressure (ground water) conditions; (c) Assign appropriate material properties to the continuum materials; (d) Apply the appropriate surface loads/forces and boundary conditions to the numerical model; and (e) Perform a FoS analysis. At the end of this analysis, the FoS is known; and the failure surface associated with this FoS is obtained by plotting the contours of maximum shear strain rate and joining the apexes of the closed contour loops. Unlike in limit-equilibrium-based procedure where a failure surface has to be predefined for which the procedure calculates the FoS, in a continuum-mechanics-based procedure the FoS and the associated failure surface are determined as a part of the solution and the two results are the lowest FoS and the critical failure surface. Alternatively, one can use a limit-equilibrium-based procedure and calculate a FoS for a pre-defined failure surface or use an automated search procedure that guides the determination of a shear surface with the lowest FoS (this later solution, however, is not unique as different search procedures lead to different shear surfaces with different associated lowest FoS). Either way, one needs to know the failure surface along which the displacements of the slide mass (real or potential) need to be calculated. If the slope has already failed, one can use the field estimated failure surface. A reasonable match between the computed solution (FoS and shear surface) and the field performance of the slope give confidence in the appropriateness of the numerical model and the field data used. If the match between the computed results and the field performance is less than satisfactory, one may need to re-examine/adjust the input data used in the numerical model.

2.2 *Displacement calculations*

The proposed procedure for calculating displacement of a landslide using the continuum-mechanics approach has the following steps: (a) Create a discretized model of the continuum encompassing the slope and identify the locations where displacement results are of interest; (b) Include in step 2.2(a), the failure surface as an interface between the soil mass above the failure surface and the intact soil mass below the failure surface; (c) Assign appropriate material properties to the continuum materials and the interface; (d) Apply the appropriate surface loads and boundary conditions to the numerical model; and (e) Perform a deformation analysis for a finite number of steps and record the magnitude of cumulative displacement(s) at the locations identified in step 2.2(a) and the displaced configuration of the slide mass above the interface. If desired, the analysis can be paused and the shear strength values along the interface adjusted (lowered) for the next set of calculation steps to reflect a post-peak strength loss. This pausing of the analysis and adjusting of shear strength values along the interface should simulate the progression of shear strength loss with increasing displacement as observed in a repeated direct shear or a rotational shear test. Instead of pausing the analysis and adjusting the strength values manually, one can write a sub-procedure (to run concurrently with the continuum-mechanics-based procedure) to make the desired adjustments in the strength values automatically. As the slide mass begins to move out and down from the initial position, the shear strength along the interface begins to decrease from the peak strength of the material to the residual value. The geometry of the potential future interface (new riding surface) plays an important role in dissipating the energy of the moving mass and causing it to stop. All things considered, a slide mass will come to rest in a stable configuration which is consistent with the newly acquired geometry and the prevailing shear strength along the interface (new and/or existing). It is inferred from studies of several landslides (with large displacements) in clayey soils that there is a minimum shear strength of about 6° to 7° friction angle common to most observed failures regardless of the strength values at the instant of initial failure (Mesri & Shahien 2003). Thus, for maximum displacement of a slide mass, one can incrementally (in a step-wise manner) decrease the shear strength along the interface from the initial value (FoS = 1) to about 6° friction angle (zero cohesion) gradually and in accord with the progression of displacements at the locations of interest. Other factors, such as changes in pore water pressure along the interface (due to drainage and/or pore water turning into steam due to frictional heat), if known, can be affected at any stage of the analysis and become the defined data for the next set(s) of calculations.

3 SAMPLE PROBLEM

3.1 *Problem description*

During the design phase of an embankment dam, 32 different historic landslides were mapped and analyzed in the field within the reservoir area and adjacent hills (Fig. 1). Table 1 lists the observed field data on these landslides. In addition to these 32 numbered landslides, there are 17 landslides that are identified but not numbered or described. It was anticipated in the design phase (1980s) that areas within the reservoir containing fat-clay topsoils, overlying predominantly fine-grained soil units with north to northwest facing slopes of 2H:1V (2 horizontal to 1 vertical) to 5.5H:1V that had failed in the past would be susceptible to additional sliding due to likely seepage of reservoir water into the pervious sand units. To the northwest, permeable sand units extend through a hill on the reservoir rim and the Qls-18 landslide is located on the hillside facing away

Table 1. Field data on landslides in the reservoir area and adjacent hills.

Landslide ID Number	Original ground slope H:V	Measured head scarp Height (m) /Slope	Elevation at top of head scarp m	Distance moved at head scarp m	Distance from head scarp to toe m	Estimated toe elevation m	Estimated slide thickness m	Toe bulge m
Qls-1	4:1	1.75/vertical	160.4	0.2	37	151.0	5.0	0.5
Qls-2	3.5:1	0.7/vertical	141.5	0.0	21	135.5	3.0	0.5
Qls-3	~4:1	1.85–2.1/ vertical to ¼:1	181.0	0.25	60	165.0	8.0	0.8
Qls-4	~4.5:1	4.2/¼:1	182.3	1.1	55	159.5	8.0	0.5
Qls-5	~5.5:1	3.75/¼:1	182.0	0.9	85	166.0	10.0	1.0
Qls-6	2:1	1.2/½1	160.2	0.5	7.5	156.5	~1.0	0.4
Qls-7	5.5:1	2.1/1:1	169.0	1.3	67	156.5	8.5	0.3
Qls-8	~2.5:1	0.6/vertical	170.1	0.0	NA	163.5	2.5	0.6
Qls-8a	~2:1	0.6/vertical	173.5	0.0	NA	NA	~1.0	0.5
Qls-8b	~2:1	0.6/vertical	165.0	0.0	NA	NA	~1.0	0.0
Qls-8c	~2.5:1	0.5/vertical	165.0	0.0	NA	NA	~1.0	0.0
Qls-8d	~3.5:1	0.8/1:1	168.5	0.6	15	164.0	~2.5	0.4
Qls-9	~4:1	2.4/1:1	160.0	1.7	28	153.0	~3.5	0.3
Qls-9a	~2.5:1	0.3/vertical	162.2	0.0	NA	NA	0.5–1.0	0.3
Qls-9b	~2.5:1	0.5/vertical	163.3	0.0	NA	NA	~1.0	0.3
Qls-9c	~2:1	0.4/vertical	147.3	0.0	NA	NA	0.5–1.0	0.3
Qls-9d	~2:1	1.5/1:1	145.5	1.1	9	141.0	1.5–2.0	0.3
Qls-10	~2:1	3.0/¼–½:1	150.6	0.9	22	140.0	3.5	0.7
Qls-11	~4:1	2.4/½:1	123.8	1.1	41	114.0	4.0–4.5	1.25–2.1
Qls-12	~4.5:1	4.0/¾1	161.0	2.4	44	151.0	4.5–5.0	0.5
Qls-13	~3.5:1	4.0/½:1	141.2	1.8	27	133.0	5.0	1.0
Qls-14	~3:1	1.3/vertical	147.7	0.2	26	138.0	3.5	0.7
Qls-15	~3:1	1.8/1:1	161.1	1.3	54	141.0	4.0	0.8
Qls-16	3:1	3.0/1.5:1 to 3.5/½:1	166.5	4.0	82	139.0	9.0	0.8–1.0
Qls-16a	~3:1	3.2/2:1 to 2.1/¾:1	149.6	4.1	34	137.0	5.0	0.5
Qls-17	~4.5:1	2.8/¼:1	136.4	0.7	80	118.0	7.0	0.4–0.9
Qls-18	~4.5:1	2.6/½:1	116.0	1.2	81	98.5	10.0	0.6–0.9
Qls-19	~3.5:1	~15/2:1	138.2	13.4	62	120.0	4.5	0.3–0.5
Qls-20	4:1	1.0/vertical	132.6	0.1	26	126.0	~2.0	0.4
Qls-20a	~2:1	0.3–0.5/vertical	126.3	0.0	6	124.5	1.0	0.0
Qls-20b	2:1	0.5/vertical	137.6	0.0	5	136.0	0.75	0.3
Qls-21	~2:1	3.1/¾:1	112.4	1.8	15	104.0	3.5–4.0	0.6–0.8

NA Not available; ~ Approximate.

from the reservoir. With the filling of the reservoir, this prediction came true. Table 1 conveys the complexity of the geologic environment and the enormity of the landslide activity in this relatively small area.

3.2 *Site geology*

The geologic units at the site under study are composed of unindurated, compacted sediments deposited in both marine and lacustrine environments. The geologic units are estimated to be over 2500 m thick consisting of interbedded lean and fat clays and silty and clayey sands. These deposits are overlain by an average of 1 m thick fat-clay.

3.3 *Qls-18 landslide activity*

Table 2 shows the 2003 and 2004 movement data for the Qls-18 landslide. The Qls-18 landslide occurred or was reactivated subsequent to the first filling of the reservoir. Seepage from the reservoir into the pervious sand units has been attributed to be the cause of the formation of this (Qls-18) and other landslides west and northwest of the reservoir (U.S. Bureau of Reclamation 2004). The Qls-18 landslide is about 10 m thick and occurred in a 4.5H:1V ground slope (Table 1); and the slide materials include fat-clay topsoil and underlying Quaternary Pliocene-Pleistocene sediments (QPs) of the marine and lacustrine environments. Figure 2 is a March 2004 photographic view of the landslide.

Figure 3 shows the May 2003 and March 2004 boundaries of the Qls-18 landslide. In May 2003, new landslide movement began to occur in the area immediately adjacent to, and to the east of, the head scarp of the historic limits of the Qls-18 landslide. Water levels in an observational well (located in the lower portion of the slope and just to the east of the historic limits of the Qls-18 landslide) had risen by about 9 m above the historic levels. Seepage exiting the Qls-18 landslide area had increased from an average of 10–15 liters per minute (L/min) to about 400 L/min. There was very limited downslope movement and only minor ground distortion immediately downslope of the new (1 to 2.5 m high and about 60 m across at the top) head scarp.

By March 2004, the 2003 head scarp had developed into a new lobe of sliding, about 60 m wide, that along with portions of the 2003 slide had moved 23 to 30 m downslope in the toe area; and the head scarp in the new lobe of sliding was about 7 to 9 m high. Also, the western side-scarp in the old portion of Qls-18 had additional movement during May 2003 and March 2004. Seepage exiting the Qls-18 landslide area had decreased to 10–15 L/min. Water flowing from the head scarp was ponding on a newly formed bench just downslope from the head-scarp – the pond covered an area of about 8000 m^2 and the water depth was about 1 to 1.5 m. The observation well (that formerly was just east of the limits of the Qls-18) was in a downslope portion of the new Qls-18 landslide area and was destroyed.

Figure 4 shows the geologic cross section with different soil units in the hillside. The location and geometry of the failure surface shown on Figure 4 were estimated in the field from observations of the scarp and toe of the slide and data from drill logs.

Table 2. Field data on Qls-18 landslide displacements.

May 2003		March 2004	
Downslope movement at new head scarp m	Vertical drop at new head scarp m	Downslope movement at toe of Qls-18 m	Vertical drop at new head scarp m
Very limited	1 to 2.5	23 to 30	7.5 to 9

Figure 2. March 2004 photographic view.

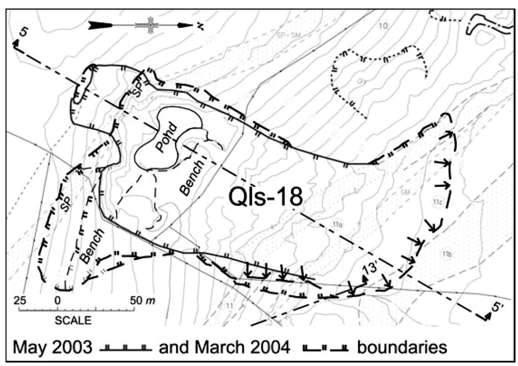

Figure 3. Plan view of the May 2003 and March 2004 bondaries.

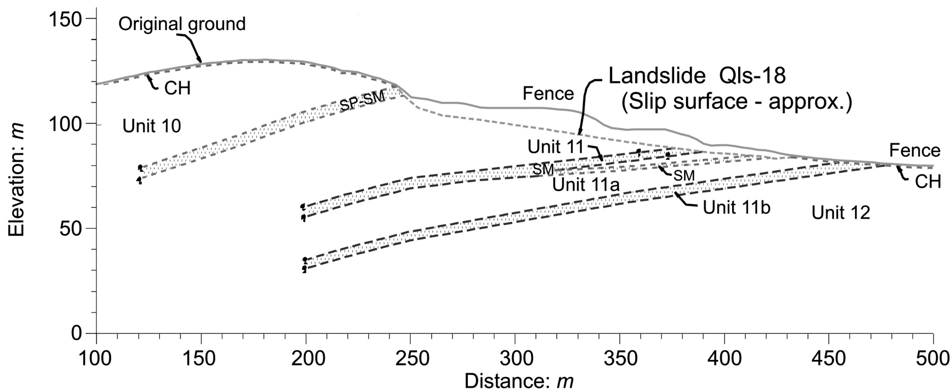

Figure 4. Geologic cross section 5–5′.

Table 3. Material properties for stability and deformation assessments of Qls-18 landslide.

	Density	Material strength		Elastic constants	
	ρ	c′	φ′	Bulk modulus	Shear modulus
Material Identifier	kg/m³	kPa	°	kPa	kPa
CH	1970	14	16	1.5e7	1.0e7
Unit 10	1970	14*	12*	1.5e7	1.0e7
SP-SM	2000	0	35	1.5e7	1.0e7
Unit 11	2000	0	35	1.5e7	1.0e7
Unit 11a	1800	14	12	1.5e7	1.0e7
Unit 11b	1800	0	27	1.5e7	1.0e7
SM	1940	0	27	1.5e7	1.0e7
Unit 12	1970	14*	12*	1.5e7	1.0e7

* Residual shear strength values: c = 0, φ′ = 6°;
Materials are assigned tensile strength = c/tan φ.

3.4 *Groundwater condition*

The water table in the Qls-18 landslide was taken to be at the top of the poorly-graded-sand to silty-sand (SP-SM) layer (elevation 120 m) and then followed the field estimated slip surface shown in Figure 4.

3.5 *Material properties*

Because of the interbedded nature of the QPs, the foundation soils are grouped into the major geologic units shown in Figure 4. Soils from different geologic units were tested to determine the fully-softened and residual shear strengths using triaxial compression, repeated direct-shear, and torsional ring shear tests. All laboratory tests were performed in the 1980s for the design of the dam and appurtenant structures. Shear and bulk modulus values were assumed for the continuum analysis. The interface strength properties correspond to those of the slide mass, and the stiffness values are assumed. Table 3 shows the material properties and Table 4 shows the interface (slip surface) properties used in the deformation analysis of the Qls-18 landslide.

Table 4. Interface properties for FLAC model analyses of Qls-18 landslide.

	Interface properties			
	c	φ	Normal stiffness	Shear stiffness
Condition	kPa	°	kPa/m	kPa/m
May 2003	14	16	2e5	6e3
March 2004	0	6	2e5	6e3

4 NUMERICAL MODEL

Figure 5 shows the numerical model of the geologic cross section shown in Figure 4. The estimated slip surface is modeled as a frictional interface between the slide mass and the parent material (base). Along this interface, the slide mass is allowed to: (a) move relative to the base; and (b) separate from the base. The constitutive model for the geologic materials is Mohr-Coulomb. The displacements of the slide mass were sampled at three locations shown in Figure 5.

4.1 *Numerical analysis*

For numerical analyses of the landslide conditions of May 2003 (assuming first time occurrence due to high groundwater conditions and limited movements), fully-softened shear strengths and other material properties

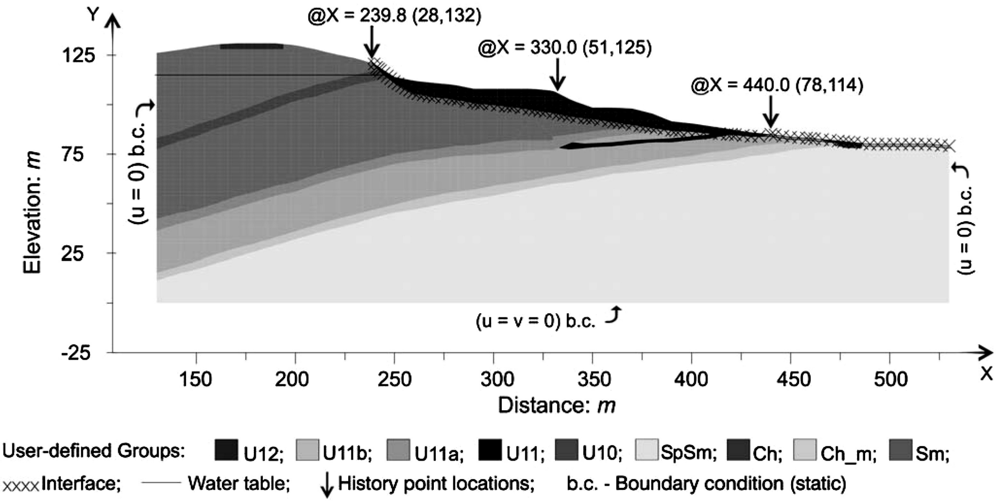

Figure 5. FLAC model of the geologic cross section (Fig. 4)

shown in Table 3, interface properties corresponding to the fully softened shear strengths shown in Table 4, and the water table at the top of the SP-SM layer (elevation 120 m) and flowing along the field estimated slip surface were used to represent the first time occurrence of sliding.

For numerical analyses of the landslide conditions of March 2004 (substantial movements, observed water pond, normal seepage flow conditions), fully-softened shear strengths and other material properties shown in Table 3, interface properties corresponding to the residual shear strengths shown in Table 4 and the water table at the top of the SP-SM layer (elevation 120 m) and flowing along the field estimated slip surface were used to represent prior sliding along the interface.

All analyses were performed using the commercially available computer program FLAC.

4.1.1 *Displacement results*

Deformation analysis of the model in Figure 5 was started with the May 2003 conditions set-up and brought to a successful completion (equilibrium solution reached). Interface properties were changed to the March 2004 conditions and the analysis was continued to a successful completion (equilibrium solution reached). Thus, results for the March 2004 conditions include the results of the May 2003 conditions. Results of these analyses are shown in Figures 6 and 7. Figure 6 is for the May 2003 field conditions and shows: (a) a progression of horizontal displacements at the three selected locations shown in Figure 5; (b) the spread of elastic, plastic-yield and tension failure zones; and (c) the relative position of the slide mass. Figure 7 is the counterpart of Figure 6 and is for the March 2004 field conditions. Table 5 shows the computed and field-observed displacements for the May 2003 and March 2004 conditions.

In brief, the computed and observed displacements at the corresponding locations for the May 2003 and March 2004 conditions are as follows: for the May 2003 field conditions – the computed horizontal and vertical displacement at monitoring location 1 (head scarp) are 0.5 and 0.3 m, respectively while displacements observed in the field are: very limited horizontal movement and 1 to 2.5 m drop at the head-scarp; and for the March 2004 field conditions – the computed horizontal displacement at monitoring location 3 (toe) is 36.1 m while field measurement is 23 to 30 m and the computed vertical displacement at monitoring location 1 (head scarp) is 12.2 m while the field measurement is 7.5 to 9 m. Thus, comparisons between the computed and field observed displacements are good.

5 LOCKE ISLAND LANDSLIDE (APPENDIX A)

The proposed procedure has also been used successfully in studying stability and deformation of the Locke Island landslide in South-central Washington State (Chugh & Schuster 2003). The Locke island landslide (a group of interconnected individual landslides) with an estimated volume of 20 million cubic meter of displaced material is the largest of a group of landslides in the 30-mile-long White Bluffs area along the northeast bank of the Columbia River in South-central Washington State. Early 1980s estimate

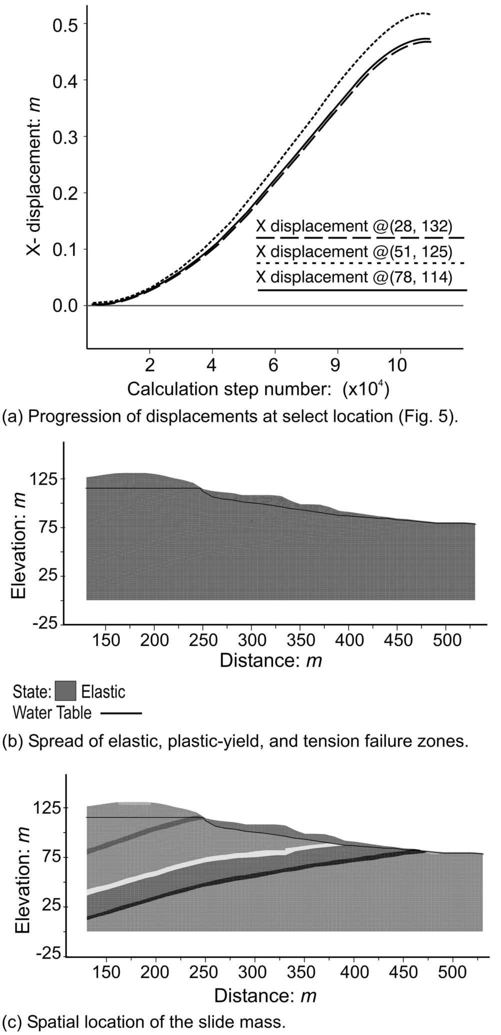

Figure 6. Static analysis results for the May 2003 conditions.

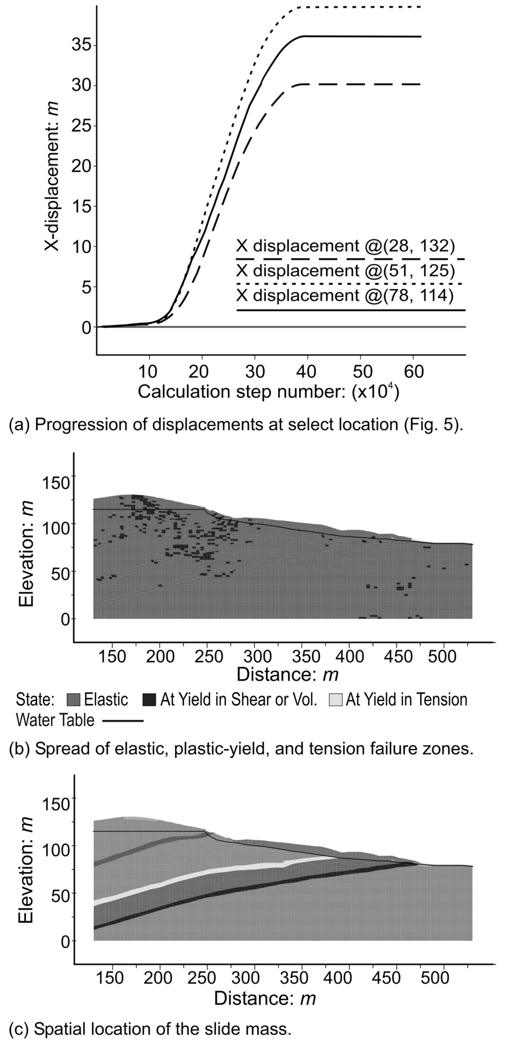

Figure 7. Static analysis results for the March 2004 conditions.

Table 5. Comparison of displacement results with the field data (Table 2).

	May 2003		March 2004	
Condition	Downslope movement at head scarp m	Vertical drop at head scarp m	Downslope movement at toe m	Vertical drop at head scarp m
Field measurements	very limited	1 to 2.5	23 to 30	7.5 to 9
FLAC results	x-displacement m	y-displacement m	x-displacement m	y-displacement m
	0.5	−0.3	36.1	−12.2

Negative y direction in the FLAC model corresponds to drop in elevation.

on magnitude of movement at the toe of individual landslides was in terms of one-third of the way ($\approx$150 m) across the northeast channel of the river; by 2002, this distance had increased to about 200 m. Using the procedures presented in the paper, Chugh & Schuster (2003) were able to confirm instability where failures had occurred in the field and stability where failures had not occurred in the field; and the computed displacement results compared favorably with the field observations. It was an interesting/challenging application of the proposed procedure to a complex landslide.

6 CONCLUSIONS

The computed displacements of Qls-18 landslide using the proposed procedure in a continuum-mechanics-based program FLAC are in good agreement with the May 2003 and March 2004 field data which pertain to the onset of slope failure and stable configuration acquired by the slide mass after failure, respectively.

Similar favorable experiences with the use of the proposed procedure on other landslides give credence to the viability of the procedure and information presented in the paper.

The favorable agreement between calculated and observed displacements suggest that analysis of landslide movements can be accomplished by: (a) treating the landslide mass and the foundation materials as two separate elastic continuums separated by an interface along the slip, and (b) incrementally decreasing the shear strength along the interface from the initial value (FoS = 1) to the residual value.

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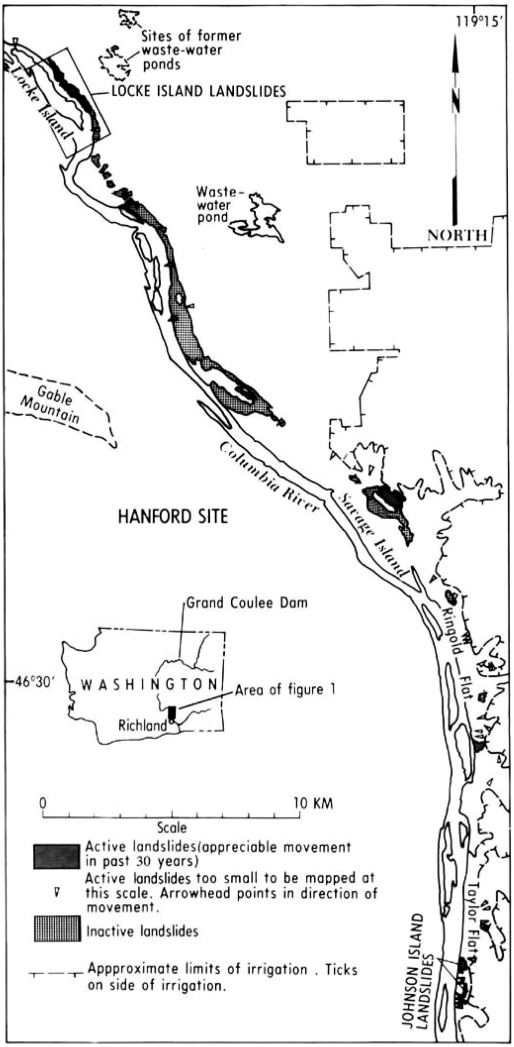
APPENDIX A

Locke Island landslide figures



Map of irrigation-induced landslides area, south-central Washington.

Locke Island landslide, 1985 view, looking northwest. (Locke Island is at far left).

Locke Island landslide, 2001 view, looking northwest. (Locke Island is at left-center).